

INTER INSTITUTIONAL WORKSHOP ON BREAKWATERS



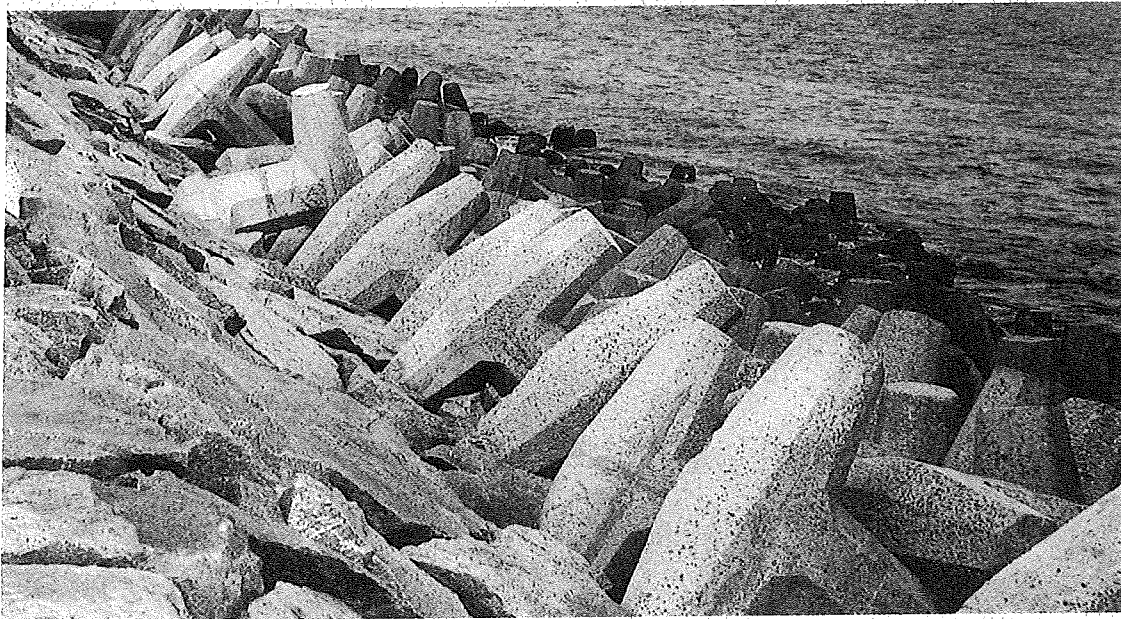
MARCH 9-10, 2000



Jointly organised by
Ocean Engineering Centre
I. I. T. Madras, INDIA
and
Faculty of Civil Engineering
and Geosciences,
T.U. Delft, The Netherlands

Coordinators
Prof. V. Sundar (I.I.T. Madras)
Prof. K.d' Angremond (T.U.Delft)

INTER INSTITUTIONAL WORKSHOP ON BREAKWATERS MARCH 9 - 10, 2000



**Jointly organised by
Ocean Engineering Centre
I. I. T. Madras, INDIA
and
Faculty of Civil Engineering
and Geosciences,
T.U. Delft, The Netherlands**

**Coordinators
Prof. V. Sundar (I.I.T. Madras)
Prof. K.d' Angremond (T.U. Delft)**

P R E F A C E

This Inter Institutional Workshop on **Breakwaters** organised jointly by Ocean Engineering Centre, Indian Institute of Technology Madras, India and Department of Civil Engineering and Geosciences, Technical University Delft, The Netherlands is perhaps the first of its kind. There has been a tremendous progress in the development of ports especially fishing harbours along the coastline of India. Among the various components of a harbour, a considerable portion of the expenditure for the development of harbours is towards construction of breakwaters which is the main component. This has forced Engineers and Scientists to have a critical look into the different concepts of breakwaters, its stability, minimum expenditure towards its maintenance without compromising for the tranquillity of waves on its leeward side. The Ennore Satellite Port which is likely to be commissioned this year is formed by breakwaters with armour layer of accropods which is first of its kind in our country. The Technical University Delft have been involved in this project in a big way. OEC, IIT Madras have contributed significantly towards the experimental investigation on the performance characteristics of new concept of breakwaters and also have been involved on a number of consultancy projects mostly the study on the stability of breakwaters through physical modelling. The studies also covered suggestions for the remedial measures for rehabilitation of a few damaged breakwaters. In addition, a number of projects involving the suggestion for alignment of breakwaters through application of numerical models developed in Ocean Engineering Centre have been carried out. The expertise of TU Delft in the area of Coastal Engineering is known worldwide. With these background, the coordinators decided to organise a workshop exclusively on Breakwaters. It is felt that the workshop may be a forum to project the expertise of both the Institutions and also to bring out the latest trends in design of breakwaters to the delegates of this Workshop. The main topics that will be covered under this workshop are Functional requirements for breakwaters, Case studies on the stability of breakwaters in India and New concepts of breakwaters.

The Coordinators wish to thank the speakers Mr. S. Gopalan, Port Development Advisor, Ministry of Surface Transport, New Delhi for his lecture on "Breakwater in India - An Overview", Mr. L.A.Meyboom, Mr. S.Pearson and Mr.R.Haggie from Haskoning, Chennai for their lectures on "The details of the Ennore Satellite Port Project" and Mr. Om Prakash, Director, CICEF, Bangalore for his lecture on "Development of Fishing Harbours in India". The Coordinators also wish to record their thanks to all the co-sponsors, Research Organisations, State and Central Government Agencies and Consulting Companies for having sponsored their Officers to the Workshop. It is earnestly hoped that the deliberations of this workshop will be extremely useful to the delegates.

Coordinators

Prof.d'Angremond
Prof.V.Sundar



CO-SPONSORS



INTER INSTITUTIONAL COURSE ON COASTAL ENGINEERING AND WORKSHOP ON BREAKWATERS (6-10 MARCH, 2000)

- CHENNAI PORT TRUST, CHENNAI
- CHILIKA DEVELOPMENT AUTHORITY, BHUBANESWAR
- DHARTI DREDGING AND CONSTRUCTION LIMITED, CHENNAI
- DREDGING CORPORATION OF INDIA LIMITED, CHENNAI
- HASKONING CONSULTING ENGINEERS AND ARCHITECTS, CHENNAI
- HOCHTIEF (INDIA) PRIVATE LIMITED, CHENNAI
- IRRIGATION MANAGEMENT TRAINING INSTITUTE, TAMILNADU
- KANDLA PORT TRUST (KUTCH)
- KVAERNER CEMENTATION INDIA LIMITED, CHENNAI
- MECON LIMITED, RANCHI
- MORMUGAO PORT TRUST, GOA
- NAVAYUGA ENGINEERING COMPANY LIMITED, VISAKHAPATNAM
- VAN OORD ACZ INDIA, NEWDELHI

List of Speakers

From IIT Madras

Mani, J.S.

Neelamani, S.

Sundar, V.

From T.U. Delft

d'Angremond, K.

From other Organisations

Gopalan, S. , Port Development Advisor,
Ministry of Surface Transport, New Delhi.

Haggie, R. , Haskoning Consulting Engineers and
Architects, Chennai.

Meyboom, L.A. , Haskoning Consulting Engineers and
Architects, Chennai.

Om Prakash , Director, Central institute of Coastal Engineering
for Fishery, Bangalore.

Pearson, S. , Haskoning Consulting Engineers and Architects,
Chennai.

CONTENTS

Page No.

1. Functional requirements for Breakwaters Prof. K.d' Angremond	1
2. Development of fishery harbors in India Mr. K. Omprakash	14
3. Non-rubble Breakwaters and optimisation Prof. K.d' Angremond	38
4. Wave energy caisson Breakwaters Dr. S. Neelamani	54
5. Partially suspended porous wall Breakwaters Dr. J.S. Mani	73
6. Case studies on stability of Breakwaters Prof. V. Sundar	80
7. Introduction on Ennore coal port project Mr. L.A. Mayboom	101
8. Design of Breakwaters for Ennore port Mr. R. Haggie	107
9. Construction of Breakwaters for Ennore port Mr. S. Pearson	128
10. Profiles of Co-Sponsors	
11. List of participants	



FUNCTIONAL REQUIREMENTS FOR BREAKWATERS

Kees d'Angremond
Delft University of Technology
Email: k.dAngremond@ct.tudelft.nl

1. INTRODUCTION

Breakwaters can fulfil a variety of functions, of which the most important ones are:

- Protection against waves. This can be subdivided in firstly, protection of ports and shipping and secondly, shore protection.
- Guiding of Currents
- Protection against shoaling
- Provision of dock or quay facilities

2. PROTECTION AGAINST WAVES

2.1 Ports and shipping

Vessels at berth

The protection function against wave action must be split into some sub-categories. The best-known protection function relates to navigation. Over the years, breakwaters are connected with ports. It makes a large difference, however, what is the status of the vessels or installations that are to be protected. In other words, one shall have an idea how vulnerable the protected area is to decide what degree of protection shall be provided.

In general, the most vulnerable condition for any vessel is when it is moored alongside a rigid structure as a quaywall or a jetty, or alongside another vessel. The acceptable wave height is related to the size of the vessel on one side and the height, period and direction of the waves on the other side. Thoresen [1988] gives suggestions for ships at berth in head seas. These values are slightly modified in Table 1 according to the experience of the authors. The acceptability of the conditions refers to both, damage to the vessel and damage to the structure.

Type of vessel	Maximum H_g in m At berth (head sea)
Pleasure craft	0.15-0.25
Fishing vessels	0.40
Dredges and dredge barges	0.80-1.00
General cargo (<30,000 dwt)	1.00-1.25
Dry Bulk cargo (<30,000 dwt)	1.00-1.25
Dry Bulk cargo (up to 100,000 dwt)	1.50
Oil tankers (<30,000 dwt)	1.00-1.25
Oil tankers (100,000 to 200,000 dwt)	1.50-2.50
Oil tankers (200,000 to 300,000 dwt)	2.50-3.00
Passenger vessels	0.70

Table 1

Loading and unloading operations may impose extra restrictions. It will be clear that (un) loading liquid bulk cargo via a flexible hose allows larger ship movements than placing containers in a slot. Velsink and Thoresen approach this question from a different angle. Thoresen gives values for acceptable ship movements; Velsink [1987] gives limiting wave heights for different directions. The approach of Velsink relates more directly to the functional requirements of the breakwater. Therefore, his data are given in Table 2, Maximum Wave Heights for Loading and Unloading Operations. A comprehensive review of the problem of ship movements is given in PIANC report II-24 [1995].

Type of vessel	Limiting wave height H_g in m	
	00° (head or stern)	45° — 90° (beam)
General cargo	1.0	0.8
Container, Ro/Ro ship	0.5	
Dry bulk (30,000-100,000); loading	1.5	1.0
Dry bulk (30,000-100,000); unloading	1.0	0.8 — 1.0
Tankers 30,000 dwt	1.5	
Tankers 30,000 — 200,000 dwt	1.5 — 2.5	1.0 — 1.2
Tankers >200,000 dwt	2.5 — 3.0	1.0 — 1.5

Table 2, Maximum Wave Heights for Loading and Unloading Operations

How often exceedance of these limits is accepted is not indicated in the above figures. In other words, it is not indicated during which percentage of time loading and unloading operations may be interrupted, or how often certain berths must be left by the vessels to find a safer place to ride out a storm. This question shall be answered on the basis of a thorough economic analysis, including the risk of negative publicity

for the port. Such studies are beyond the scope of this book. The answer must be known, however when the design of the actual breakwater is started. It is stressed here that these considerations will lead to the definition of service limit states (SLS) that are generally different from the Ultimate Limit State (ULS) that deals with the survival of the structure under extreme conditions.

Fig. 1 shows the layout of a harbour where the breakwater typically protects the harbour basin, including berths for loading and unloading.

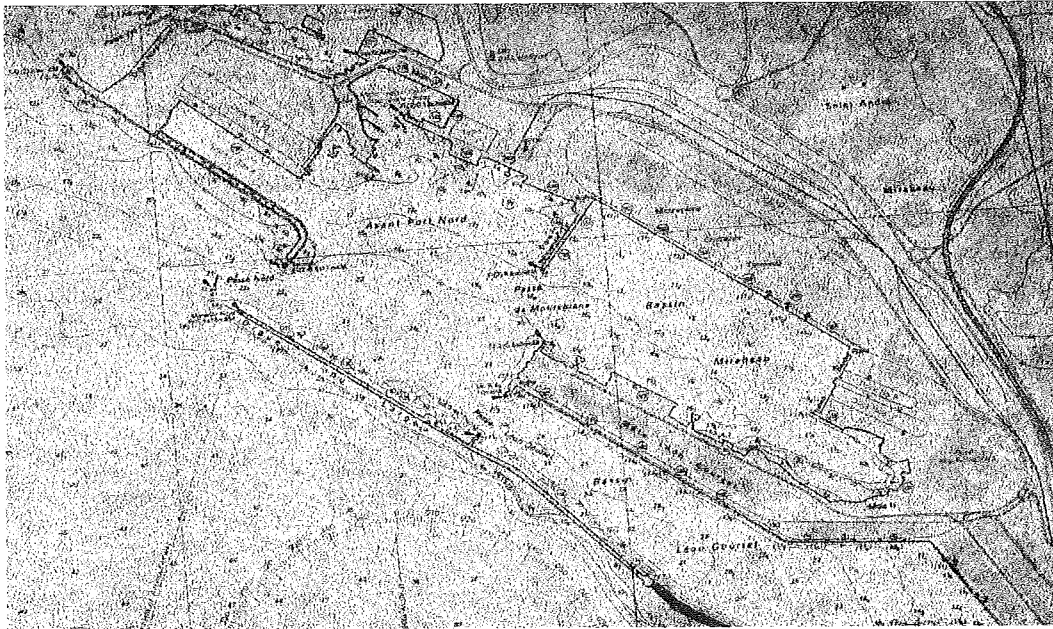


Fig. 1 Harbour of Marseille (France)

Sailing vessels

So far, we considered the protection required by vessels at berth. Free sailing vessels are fortunately a lot less vulnerable.

National regulatory bodies like the (Netherlands) shipping inspectorate strictly control the operation and the design of ocean going vessels. The work of those national organisations is co-ordinated by the international Maritime Organisation, IMO. Apart from those Government related regulatory bodies, there are also private regulatory bodies that check the design of vessels, often on behalf of the insurers. Such private bodies are Bureau Veritas, Det Norske Veritas, Lloyds, etc. These bodies issue certificates of seaworthiness with or without certain restrictions.

In this way, ocean going vessels with an unrestricted certificate are designed to cope with the highest waves. In severe conditions they may adapt course and speed to the prevailing wind and wave direction, but modern vessels with an unrestricted certificate can in principle survive the most severe conditions at sea. The situation changes when a free choice of course and speed becomes impossible, for instance because of the proximity of land, the need to sail in a specific (dredged) fairway, or the wish to come to a halt at a mooring or anchorage. The more confined the conditions, the stricter will be the limits with respect to wind, waves and currents.

What applies for vessels designed to sail non-restricted at the high seas does not apply to all categories of vessels. Some vessels have a certificate that limits their operations to certain areas (coastal waters, sheltered waters, and inland waters) or to certain periods in relation to certain areas (North Atlantic summer). Such restrictions refer not only to the structural aspects of the vessel, but also to skill and number of crew.

What does all this mean for the operation of a port, and for the functional requirements of its breakwater? Can any vessel enter the port under any circumstances? This is certainly not true, but we have concluded already that a sailing vessel is less vulnerable than a moored vessel. The functional requirements for a breakwater that protects an entrance channel only are thus much less than those for a breakwater that protects a harbour basin. Still, the actual situation will change from place to place. In case ships need tugboat assistance during the stopping operation and the subsequent turning or mooring, the waves shall be attenuated to a level that makes tugboat operation feasible. In general, one can assume that a significant wave height of 2 to 2.5 m is acceptable for tugboats and their crews working on deck. If only tugboats with an inland waters certificate are available, working of those tugboats may be restricted to significant wave heights of 1 to 1.5 m. Exceeding the limits imposed by the certificate often means that insurers will not cover the cost of damages.

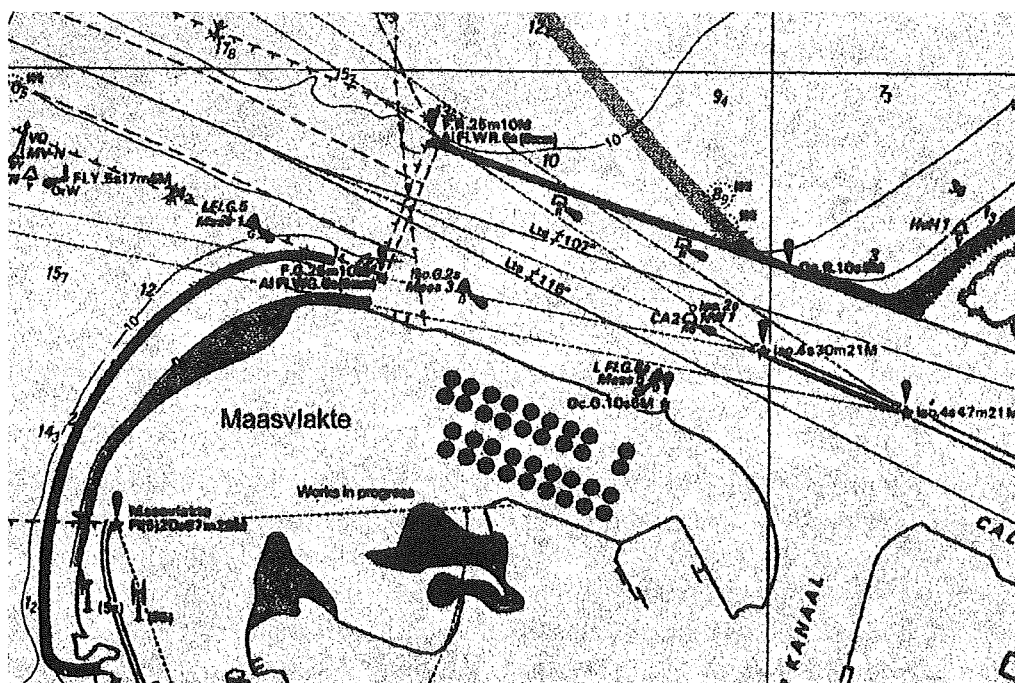


Fig. 2 Breakwater at the Europoort Entrance

Fig. 2 shows an example of a breakwater, which does not protect any berths.

Here again, decisions shall be made as to how frequent interruption of the navigation is accepted due to closure of the port for weather conditions. One must realise that also pilotage becomes a limiting factor under heavy sea and swell conditions.

Port facilities

A third condition that needs attention is the harbour basin itself, with its facilities that may suffer damage if the wave heights in the basin are getting too high. Quays and jetties and the installations that are placed on their deck may be damaged

even in the absence of vessels. Here again, it shall be decided whether such damage is acceptable at all, and if so what chance of occurrence is acceptable. It is evident that in case the harbour installations are damaged, one is not only concerned about the direct cost of repair, but also about the consequential damage due to non availability of the cargo transfer systems. In this respect one may try and imagine what happens if the only power plant or refinery in a region must be closed because no fuel can be supplied.

Shore Protection

From coastal engineering theory, we know that waves cause both, longshore transport and cross-shore transport. Both phenomena can cause unwanted erosion, specifically on sandy shores.

As far as cross-shore transport is concerned, the erosion is often connected with changes of the equilibrium profile. A more gentle profile (after erosion of dunes!) is connected with higher incoming waves, whereas a milder wave climate tends to restore the beach by a landward sediment transport. Similarly, when erosion is due to a gradient in the longshore transport, the effect will be less when the wave heights are lower.

In general terms one can therefore conclude that a reduction of wave heights in the breaker zone will mitigate beach erosion. Such reduction of wave heights can be achieved by constructing offshore breakwaters, parallel to the shore (*FIG. 3*). From literature it is known, however, that one must be careful with this solution. Due to wave set-up, the water level on the lee side of the breakwater rises, which causes a concentrated return current, (comparable with a rip current) between the breakwater sections (Bowder, Dean and Chen [1996]).

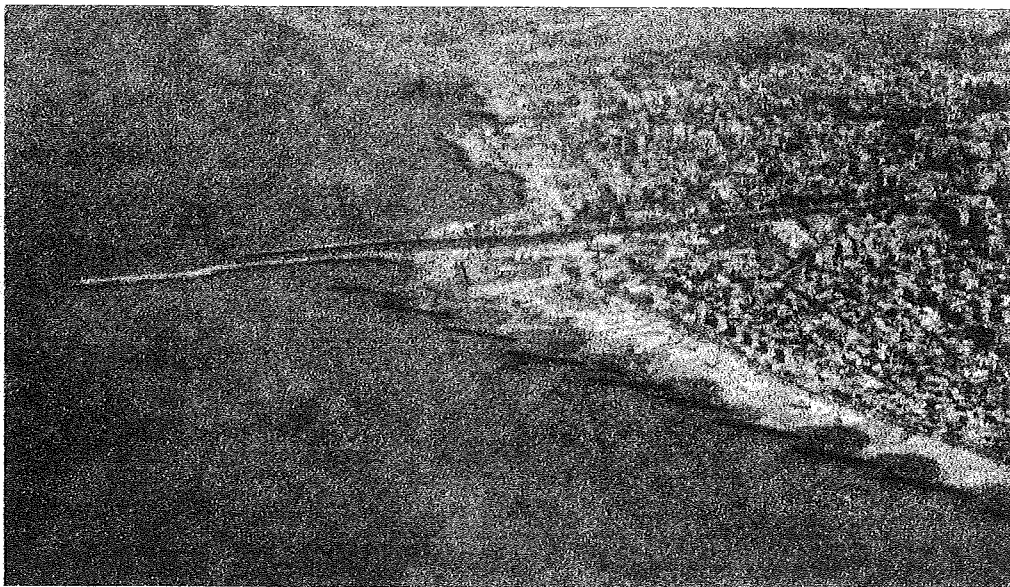


FIG. 3 A SYSTEM OF DETACHED BREAKWATERS AT FIUMICINO, ITALY

3. GUIDING OF CURRENTS

When approaching a harbour entrance, vessels are slowing down by reducing power. This is done because at high speed, the stopping length is rather long, and the

vessels produce a high wave and a strong return current. A slower speed means that the vessel is more affected by cross current (or a crosswind), since the actual direction of propagation is the vectorial sum of the vessels own speed and the current velocity. To sail a straight course into the port along the axis of the approach channel means that the vessel will move more or less crab-wise.

Closer to the shore, one must expect at the same time stronger tidal currents parallel to the shore. If the port entrance is protruding into the sea, there will possibly be a concentration of flow lines near the head of the breakwater.

Combining the slower speed of the vessel and the potentially stronger cross currents at the harbour entrances poses problems for the manoeuvrability. In the lee of the breakwater tugboats can assist the vessel, but it takes some time (about 15 minutes) before the tugboats have made a connection with the vessel, and in the mean time, the vessel continues to sail without external assistance. Assuming a speed of 4 knots, the vessel travels a distance of about 1 nautical mile (1850 m), before the tugboats can control the course of the vessel. Only then, the remaining stopping procedure can be completed. The vessel gives full power astern, and it will stop within 1 to 1.5 times the length of the vessel.

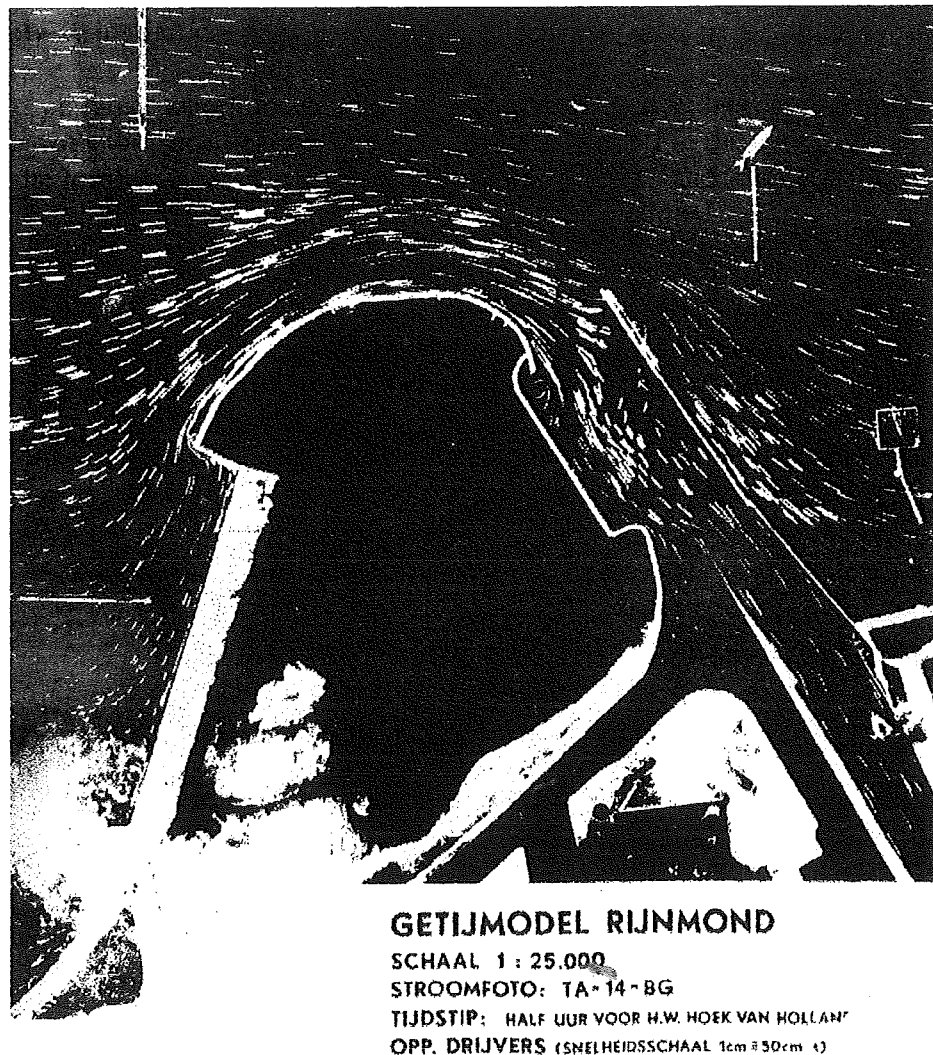


Fig. 4 Flow Pattern at the Europoort Entrance

This means that cross currents are critical over a considerable distance from well outside the harbour entrance to the point where tugboats assume control. It is not only the velocity of the crosscurrent that is important, but also the gradient in the crosscurrent, since this forces the ship out of its course.

The harbour entrance of Rotterdam is a good example of an entrance where the layout (plan) of the breakwater is designed to cope with the current pattern (Figure 4). In this case, the function of the breakwater is twofold: it guides the current and it damps the waves to a level that the tugboats can work.

4. PROTECTION AGAINST SHOALING

Many ports are located at a river entrance or in an estuary. It is known in coastal engineering that the entrance channel has an equilibrium profile that is mainly determined by the tidal prism. (d'Angremond et al. Introduction to Coastal Engineering [1998]). If the natural depth in the entrance channel is insufficient for nautical purposes, one may decide to deepen the channel by dredging. Though this may be a very good solution, disturbance of the equilibrium means that dredging has to be continued forever. In a number of cases it has therefore been decided not to dredge, but to restrict the width of the natural channel and to force the channel to erode its bed. This may also be the functional purpose of a breakwater that is designed to guide currents. An example of such solution is the port of Abidjan (*Fig. 5* and *Fig. 6*).

It is stressed here, that improvement of the efficiency of dredging and the lower cost of dredging operations have caused a shift from building breakwaters to accepting the annual cost of dredging.

Another challenge for entrance channels into a port is the existence of the longshore current along sandy shores. Under the influence of oblique waves, a longshore current develops in the breaker zone. Due to the high turbulence level in the breaker zone, a lot of sand is brought in suspension.

This material is carried away, along with the longshore current. It will be deposited at places where the velocity is less, i.e. where the water depth is larger because of the presence of the shipping channel. Thus a dredged or even a natural channel may be blocked after a storm of short duration and high waves or after a long period of moderate waves from one direction. To avoid this, a breakwater can be constructed. For a proper functioning, the head of the breakwater shall extend beyond the breaker zone. In that case, sand will be deposited on the "upstream" side of the breakwater, whereas erosion will take place at the downstream side. This is the classical example in coastal engineering for erosion problems due to interruption of the longshore transport. A good example is shown in *Fig. 7*, which shows the actual situation in IJmuiden (the Netherlands).

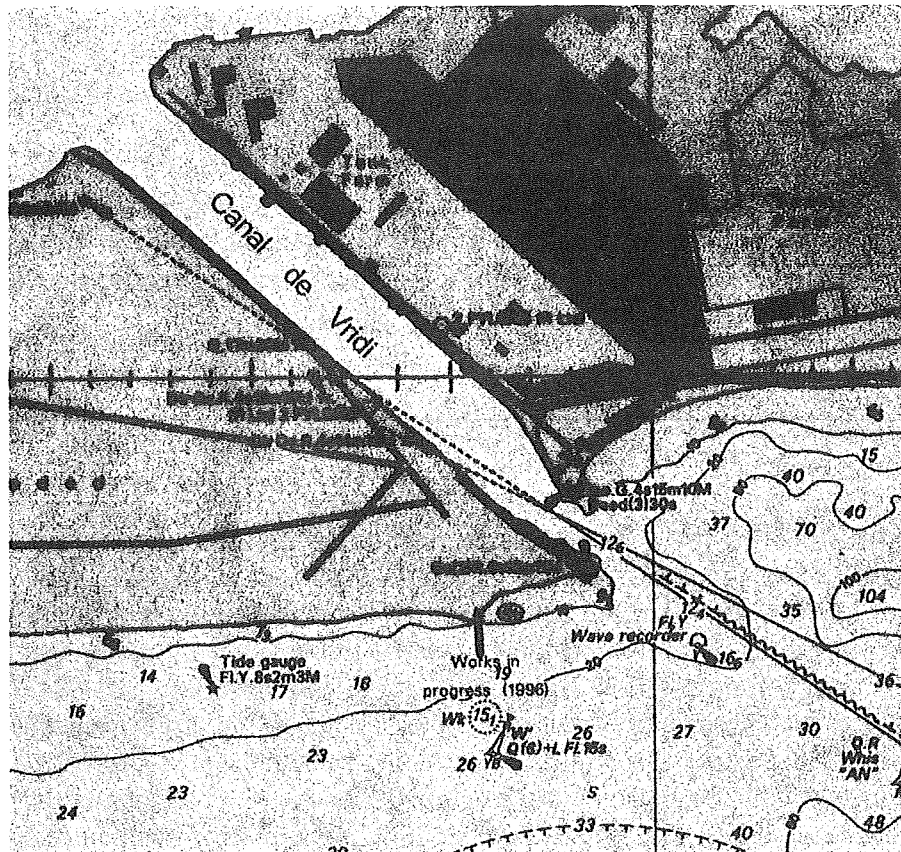


Fig. 5 Entrance to the Port of Abidjan

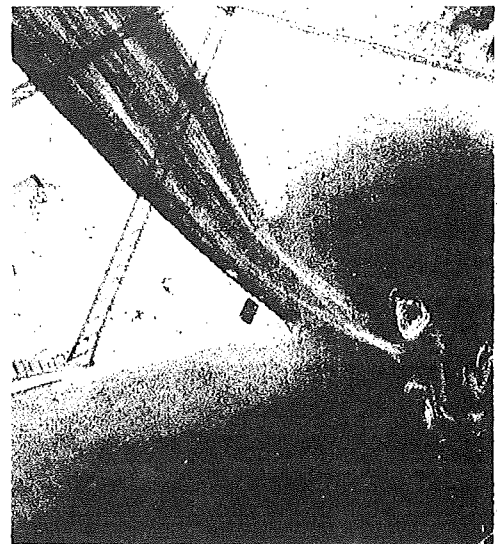
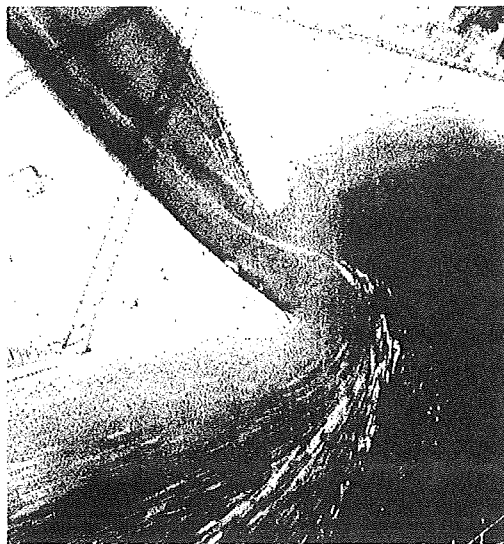


Fig. 6 Flow Pattern at the port of Abidjan

Even if the breakwater is present, sedimentation of the port's entrance channel may occur. This is the case when so much sediment has been deposited on the upstream side of the breakwater that the accumulated material reaches the end of the breakwater

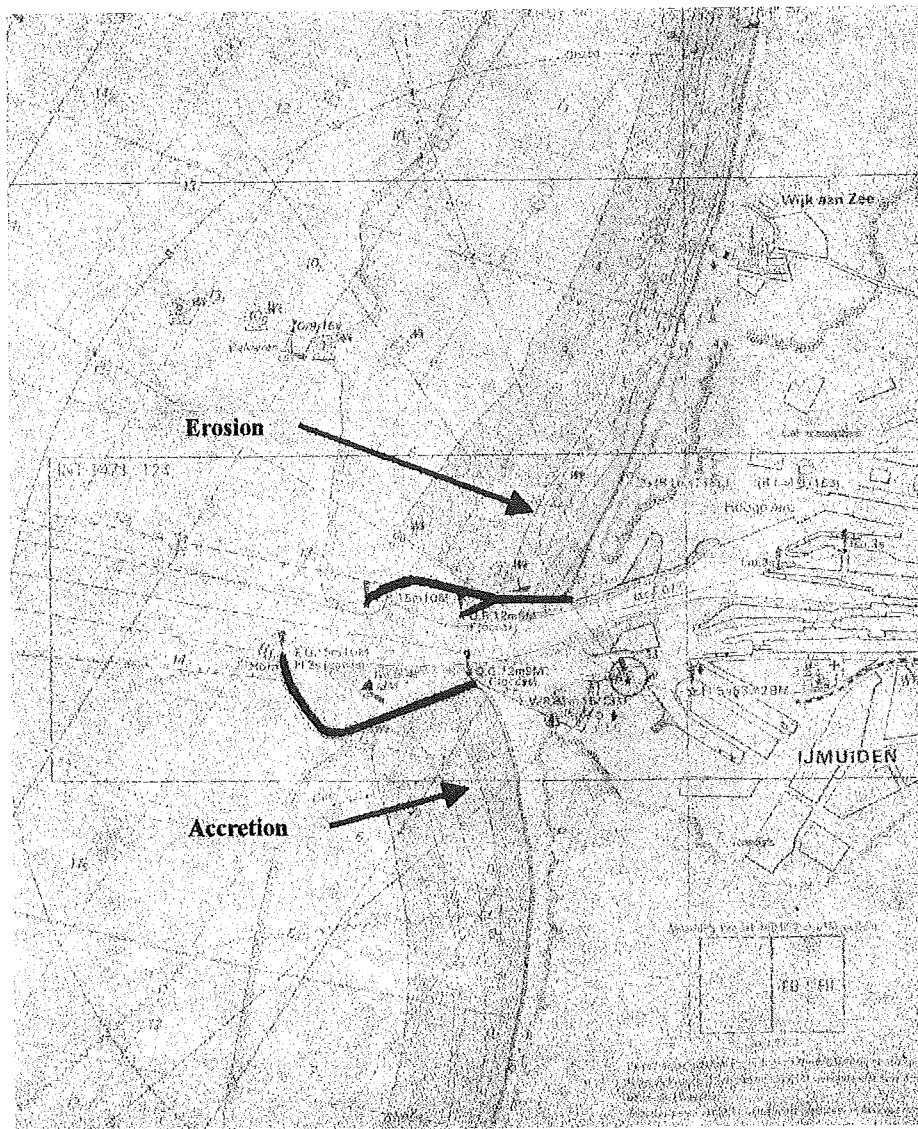


Fig.7 Port and Breakwaters at IJmuiden

and passes around it's head. Dredging is difficult in such case because of the proximity of the breakwater. An example of a breakwater that is too short is the breakwater of Paradip (India), shown in Figure 8.

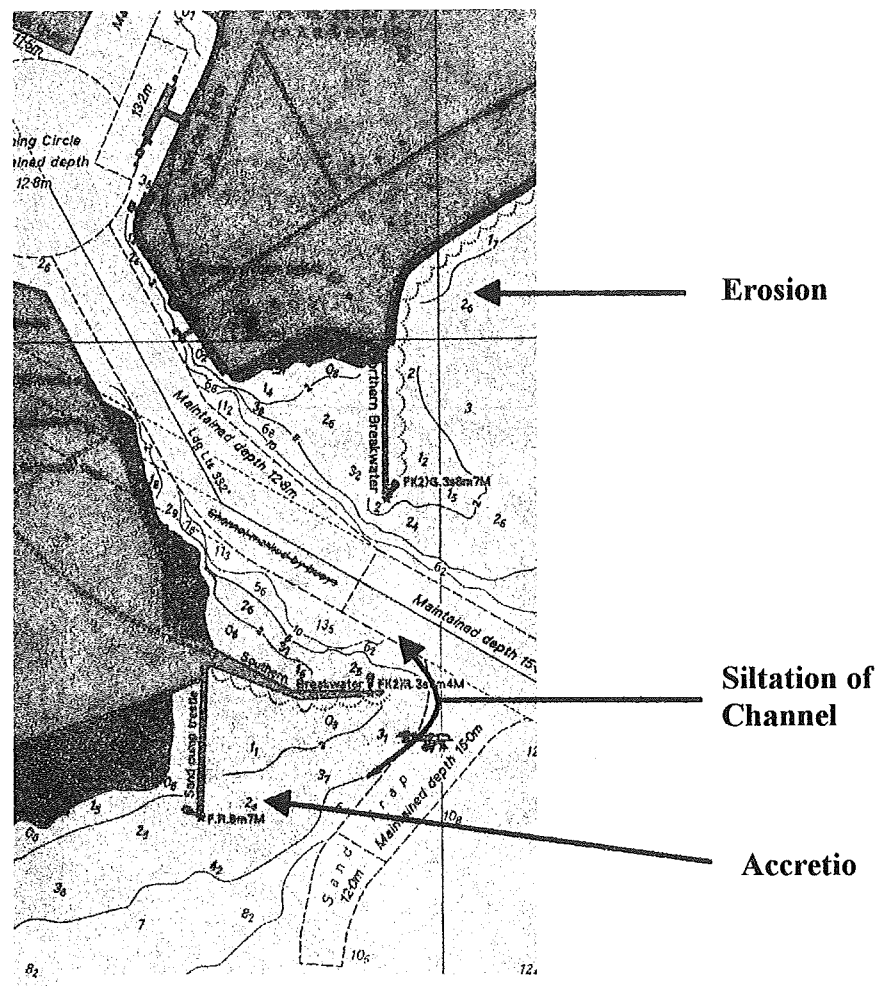


Fig. 8 Entrance to the Port of Paradip

5. PROVISION OF DOCK OR QUAY FACILITIES

Especially when the breakwater is directly protecting a harbour basin (and therefore already quite high), it is attractive to use the crest of the breakwater for transport of cargo and passengers to and from moored vessels. Special facilities shall be provided in that case to enable the vessels to berth alongside the breakwater. These facilities may consist of a vertical wall at the inside, or a (piled) jetty connected to the breakwater.

In this case, it must be ascertained that the conditions on or direct behind the crest of the breakwater are safe. Again a distinction can be made between operational conditions (Service Limit State or SLS) and extreme conditions like survival of the installations (Ultimate Limit State or ULS).

6. STRUCTURAL IMPLICATIONS

The functional requirements have a direct impact on the design of cross sections. They have specifically a large impact on crest level and wave transmission. Therefore, some research has been done in this field.

Wave transmission is the phenomenon that wave energy passing over and through a breakwater creates a (reduced) wave action in the lee of the structure (Figure 9). This will certainly happen when considerable amounts of water are overtopping the structure. Wave transmission is also possible, however, when the core of the structure is very permeable and the wave period is relatively long. It is specifically the influence of these two factors that has for a long time prevented the derivation of an acceptable formula for wave transmission.

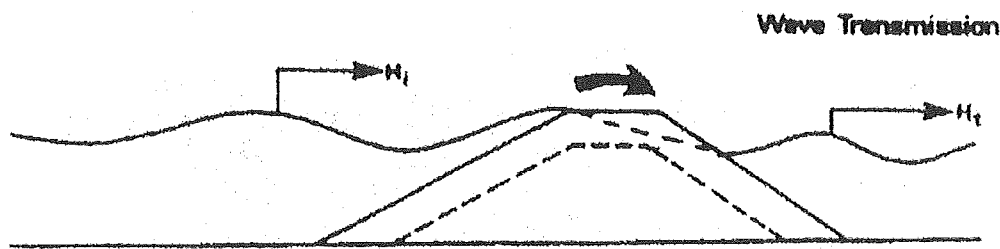


Fig. 9 Typical Wave Transmission

The effects of wave transmission has been investigated by many authors (Seelig [19801], Powell and Allsop [1985], Daemrich and Kahle [1985], van der Meer [1990]).

This resulted in a diagram presented in Figure 10.

It must be noted that the transmission coefficient can never be smaller than 0 or larger than 1. In practice, limits of about 0.1 and 0.9 are found in practice (Fig. 10).

It is remarkable that for $R_c = 0$, which represents a structure with the crest at SWL, the transmission coefficient is in the order of 0.5. This means that a relatively low structure is already rather effective in protecting the harbour area behind the breakwater. In combination with the requirements for tranquillity in the harbour, the designer can decide on the minimum required crest level.

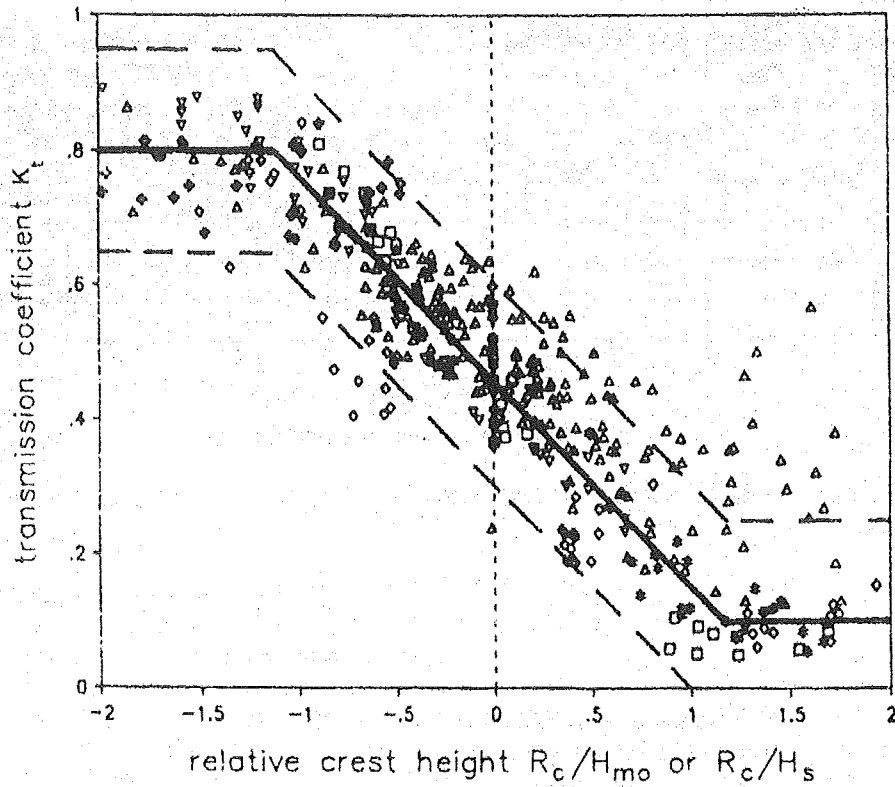


Fig. 10

Eventually, Daemen (1991) (See also Van der Meer and d'Angremond [1991]) in his MSc thesis has been able to produce an acceptable formula that relates the transmission coefficient to a number of structural parameters of the breakwater. To account for the effect of permeability, Daemen has decided to make the freeboard R_c of the breakwater dimensionless dividing it by the armour stone diameter. This eliminates a lot of the scatter that was present in previous approaches. The Daemen formula reads (for traditional low crested breakwaters) as follows:

$$K_t = a \frac{R_c}{D_{n50}} + b$$

with:

$$a = 0.031 \frac{H_{si}}{D_{n50}} - 0.24$$

and

$$b = -5.42s_{op} + 0.0323 \frac{H_{si}}{D_{n50}} - 0.0017 \left(\frac{B}{D_{n50}} \right)^{1.84} + 0.51$$

in which:

K_t	=	H_{si}/H_{st}	=	transmission coefficient
H_{si}	=			incoming significant wave height
H_{st}	=			transmitted significant wave height
R_c	=			crest freeboard relative to SWL
D_{n50}	=			nominal diameter armour stone
B	=			crest width
sop	=			wave steepness

Use of the Daemen formula is complicated in case it is decided to use a solid crown block, or to grout armour stones with asphalt into a solid mass. Therefore, another MSc student, R.J. de Jong (1996), reanalyzed the data and came up with a different expression. He choose to make the freeboard dimensionless in relation to the incoming wave height:

$$K_t = a \frac{R_c}{H_{si}} + b$$

with

$a = 0.4$, and

$$b = 0.64 \left(\frac{B}{H_{si}} \right)^{-0.31} * \left(1 - e^{-0.5\xi} \right)$$

The factor 0.64 is valid for permeable structures; it changes into 0.8 for impermeable structures.

7. REFERENCES

- Angremond, K.d', Meer, J.W. van der, and Jong, R.J, de, "Wave transmission at low-crested structures" Proc. 25th ICCE, Orlando, ASCE, New York, USA.
- Angremond, K. d' et al. (1998) "Introduction to Coastal Engineering", Lecture notes Delft University of Technology, Faculty of Civil Engineering and Geosciences.
- Browder, A.E., Dean, R.G. and Chen, R. (1996) "Performance of a submerged breakwater for shore protection", Proc. 25th ICCE, Florida, ASCE New York, USA.
- Daemen, I.F.R. (1991) "Wave transmission at low crested breakwaters" MSc thesis, Delft University of Technology, The Netherlands.
- Jong, R.J. de (1996) "Wave transmission at low crested structures" MSc thesis, Delft University of Technology, Delft, the Netherlands
- PIANC (1995), "Criteria for movements of moored ships in harbours" Report of working group II-24, PIANC, Brussels, Belgium.
- Thoresen, C.A. (1988) "Port Design, Guidelines and recommendations", Tapir Publishers, Trondheim, Norway.
- Velsink, H. (1987) "Principles of integrated port planning", PIANC Bulletin no. 56, Brussels, Belgium.



DEVELOPMENT OF FISHERY HARBOURS IN INDIA

K. Omprakash
Central Institute of Coastal Engineering for Fishery
Ministry of Agriculture, Bangalore
Email: cicefko@bgl.vsnl.net.in

1. INTRODUCTION

1.1 Importance of fishery sector in economy, employment generation, etc.

Fisheries play an important role in the economy of India in augmenting food supply, generating employment, raising nutritional levels and earning foreign exchange. In order to increase production and productivity in fisheries, the Fisheries Division of the Department of Animal Husbandry and Dairying, Ministry of Agriculture, Government of India have been undertaking various production-oriented schemes, input supply programmes, infrastructure development projects, etc., either directly or through States/Union Territories. A number of Institutions have been established for development of fisheries. According to estimates prepared by the Central Statistical Organisation, the contribution of fisheries sector to the Net Domestic Product has shown more than six and half times increase from Rs.14790 million in 1984-85 (base year for Seventh Plan) to Rs.98260 million in 1994-95 at current prices.

India has a long coastline of 8041 kms covering the east and west coasts of the peninsula as well as the Andamans, Nicobar and Lakshadweep group of islands with a continental shelf area of about 0.5 million sq.km. The coastline of India gives an Exclusive Economic Zone (EEZ) covering an area of 2.02 million sq.km. The exploitable marine fishery resources in the EEZ have been estimated at 3.9 million tonnes. India has a long tradition in fishing with fish constituting the main supply of animal protein as well as an important source of foreign exchange earnings.

Fisheries is a very important sector with tremendous potential for income and employment generation, poverty alleviation and foreign exchange earnings. Continuous efforts have been made to increase fish production, both for domestic consumption and export. The total fish production from both inland and marine sector has increased from 2.8 million tonnes in 1984-85 to 5.4 million tonnes during 1997-98, out of which about 2.95 million tonnes have been exploited from the marine resources. India is now sixth largest producer of fish in the World.

1.2 Fish Production and Exports

There are about 3726 fishing villages all along the Indian Coastline and fish are being landed in 2337 landing centres. The total fishermen population of India has been estimated at 6.0 million which include 2.4 million full-time fishermen, 1.5 million part-time fishermen and 2.1 million occasional fishermen. Out of the total exploitable marine fishery resources of about 3.9 million tonnes, the Country currently produces about 2.95 million tonnes leaving a scope for exploiting an additional one million tonnes of fish. The State-wise marine fish production during 1997-98 is given at Table-1.

There has been tremendous growth in the export of marine products. The export of marine products has increased from 86,187 tonnes valued at Rs.3843 million in 1984-85 to 3,85,818 tonnes valued at Rs.46975 million during 1997-98. There are about 47,000 mechanised fishing vessels (MFVs) and 1,91,200 traditional craft (including about 32,000 motorised craft) in operation in the country. A statement showing the fishing crafts in the Maritime States/UTs as in 1994-95 is enclosed at Table 2.

2 Development of Fishery Harbours and Fish Landing Centres

2.1 Landing and berthing facilities – a historical perspective

The main thrust has been to harvest the available fishery potential through efficient and sustainable exploitation of the EEZ by promoting operation of fishing vessels. To meet this objective, landing and berthing facilities by way of fishery harbours with ice plants, chilled storage, workshop, repair facilities, auction hall, net mending sheds, etc., are the essential infrastructure facilities required by the marine fishing industry.

At the end of First Five Year Plan, there were 863 mechanised fishing vessels operating along the Indian coast. By the end of Sixth Plan, this has increased to about 24,000 mechanised boats and at the end of Seventh Plan, the country had about 34,000 mechanised boats and 26,000 motorised craft. During beginning of the Ninth Plan there are about 47,000 mechanised boats and 32,000 motorised craft operating in the country. There are over 170 deep-sea fishing vessels having length of 23 m. and above.

During the Second Five Year Plan, Government of India began to give technical and financial assistance to State Governments for establishment of fishery harbours and sought assistance from Food and Agriculture Organisation (FAO), for survey and preparation of feasibility reports for establishment of fishery harbours. During the period from 1955 to 1961, FAO experts identified about 40 sites for development of fishery harbours and fish landing centres and prepared feasibility reports. During the Fourth Plan, Government of India with the assistance of FAO/UNDP established the erstwhile Pre-Investment Survey of Fishing Harbours at Bangalore for pre-investment

survey, preparation of techno-economic feasibility reports and related work in the fishery harbour construction and development. During the Second, Third and Fourth Five Year Plans, emphasis was given mainly for construction of minor fishery harbours and fish landing centres. During the Fifth Plan, construction of major fishery harbours at Sassoon Dock, Cochin, Chennai, Visakhapatnam (Vizag) and Roychowk were sanctioned. The development of fishery harbours and landing centres continued under the schemes subsequently in the Sixth, Seventh, Eighth and Ninth Five Year Plans.

2.2 Govt. of India Scheme

The Department of Animal Husbandry and Dairying, Ministry of Agriculture, Government of India have been implementing a Central Sector Scheme (CS) and Centrally Sponsored Scheme (CSS) since 1964 with the objective of providing infrastructure facilities for landing and berthing of mechanised fishing vessels, traditional fishing craft and deep sea fishing vessels. Under CS, the Port Trusts are provided with 100 % grant on the capital cost for the development of major fishery harbours at Major Ports. Besides construction, management and operation of the fishery harbours after its completion are also the responsibility of the respective Port Trusts. Under the CSS, the Maritime State Governments are provided with 50% grant on the capital cost for development of minor fishery harbours and fish landing centres. The construction and subsequent management and maintenance of such facilities created after completion are the responsibility of the respective State Governments. The Union Territories are provided with 100% grant under the Scheme.

During the Seventh Five Year Plan, an allocation of Rs. 170 million under the CS and Rs. 180 million under the CSS totalling to Rs. 350 million was made for the development of fishery harbours and the funds were utilised in full. In view of the increased demand from the State Governments/UTs and the fact that large number of fishery harbours were under construction, besides need for development of more number of fishery harbours the allocation of funds was considerably increased to Rs. 540 million during the Eighth Five Year Plan in respect of CS and Rs. 470 million for CSS. Out of the total allocation of Rs. 1010 million, Rs. 950 million were utilised during the Plan period.

2.3 Total number of facilities sanctioned, completed and under construction under Central Sector Scheme

Since inception of the Scheme in 1964, under the Central Sector Plan Scheme, 100 % financial assistance is provided as grant for the development of major fishery harbour at Major Ports by the Govt. of India. Till date, the Govt. of India have sanctioned six major fishery harbours at Cochin Stage I and II in Kerala, Sassoon Dock in Maharashtra, Chennai Stage I and II in Tamil Nadu, Vizag Stage I, II and III in Andhra Pradesh, Paradip in Orissa and Roychowk in West Bengal. All the five major fishery harbours except Sassoon Dock have been commissioned. The fishery harbour at

Sassoon Dock is almost complete and expected to be put in operation very soon. The fishery harbour at Chennai Stage II is under construction.

2.4 Centrally Sponsored Scheme

The objective of the Centrally Sponsored Scheme is establishment of minor fishery harbours and fish landing centres for landing, berthing, out fitting, repairs and operation of mechanised fishing vessels and traditional craft. Under the Scheme, the Govt. of India have sanctioned 45 minor fishery harbours and 153 fish landing centres. Of these 29 minor fishery harbours and 120 fish landing centres have been completed and the remaining are under various stages of construction. A statement showing the present status of fishery harbours and landing centres commissioned/under construction in both the Schemes is enclosed at Annexure I. The location and name of minor and major fishery harbours commissioned/under construction under the Schemes are shown in the drawing enclosed at Annexures II and III respectively.

The landing and berthing facilities are presently available for 1/4th of the total fishing fleet only. Therefore, there is an imperative need to develop more number of fishery harbours and landing centres to meet the requirements of fishing fleet operating in the Country. Keeping the above in view, the outlay for the Ninth Plan has been increased to Rs. 1400 million against the allocation of Rs. 1010 million during the Eighth Plan period.

2.5 Outlays earmarked and actual expenditure for development of Fishery Harbours

The total Plan outlays and expenditure incurred for the development of major and minor fishery harbours besides fish landing centres upto the end of Eighth Five Year Plan may be seen at Annexure IV. It may be noted that the outlays earmarked and the expenditure incurred for the Schemes have increased from Rs. 1.7 million during the Third Plan to Rs. 950 million during Eighth Plan. During the Ninth Five Year Plan, the two schemes are proposed to be combined as a Centrally Sponsored Scheme with a pattern of assistance of 50:50 share for State Government and 100% for Port Trusts and UTs. An outlay of Rs. 1400 million will be provided in the Ninth Five Year Plan for both the Schemes. The break up of total outlay and year-wise phasing is given at Annexure V.

3.0 Updating of Master Plan for the development of Fishery Harbours in India

The Central Institute of Coastal Engineering for Fishery (CICEF), Bangalore, Govt. of India prepared a Master Plan for the development of fishery harbours in the country during the period 1978 to 1981. A total number of 117 fishery harbour sites comprising 14 major, 7 medium and 96 minor sites had been identified at that time.

This Master Plan was prepared based on knowledge of fishery resources, the size and draft requirement of fishing vessels operating then. The State-wise details of the sites identified during 1978 to 1981 together with the fishery harbour facilities available/under construction are given at Annexure VI. As a step forward in this direction, the CICEF has reviewed and updated the above Master Plan by identifying more number of fishery harbour sites in Maritime States/UTs. The Institute has prepared Master Plan reports for the fishery harbour sites reconnoitred in various Maritime States/UTs. The potential sites identified by CICEF for development of fishery harbours and fish landing centres State-wise are furnished at Annexure VII. The sites found suitable for development of fishery harbours/fish landing centers are furnished at Annexure VIII. Based on the priorities of the Central and State Governments, the potential sites identified by CICEF form the basis for taking up detailed engineering and economic investigations by the Institute during the Ninth Five-Year Plan and beyond.

4.0 Procedure adopted for sanction of Harbour Projects

4.1 Pre-Investment Evaluation Studies

In a developing Country like India where resources available for undertaking developmental works are limited, the resources are tapped and used judiciously so as to ensure better returns for investment and better service facility to the industry concerned. In order to develop a fishery harbour at a particular fishing centre, data regarding the existing status of the fishery industry of that centre is collected at micro and macro levels for detailed analysis. The micro level information covers number of mechanised fishing vessels operating at the centre, annual landings, species composition, vessel economics, infrastructure facilities, etc. At macro level, information regarding the quantum of fishery resources available in the waters off the proposed site, potential markets, disposal of landings, processing facilities available in the region etc., is covered. After analysing the data collected from the field, a detailed techno-economic feasibility report is prepared covering engineering and economic aspects. While the engineering portion provides details regarding survey, sub-soil investigations, designs, layout and cost estimate etc., of the fishery harbour, the economic evaluation portion furnishes the projected fishing fleet, annual landings, vessel economics, disposal of landings, operational costs, Investments, cash inflow and finally the Financial Internal Rate of Return (FIRR). The FIRR decides the feasibility of the project proposal from investment view point.

4.2 Investigations and Preparation of Techno-Economic Feasibility Reports

The Central Institute of Coastal Engineering for Fishery (CICEF), Bangalore, a subordinate office of the Department of Animal Husbandry and Dairying, Ministry of Agriculture, Govt. of India is responsible for undertaking techno-economic feasibility studies for the development of fishery harbours in the country under the Schemes. CICEF formerly known as Pre-Investment Survey of Fishing Harbours was established

in January 1968 by the Government of India, in collaboration with FAO of the United Nations with a primary objective to carry out reconnaissance surveys/pre-feasibility studies to identify potential sites for development of fishery harbours and follow it up by engineering and economic investigations besides preparation of techno-economic feasibility reports. The Institute has been entrusted with the task of monitoring the progress of construction of on going fishery harbours sanctioned under the Schemes and provide technical advice on the engineering and economic aspects to the State Governments/UTs. The Institute till the end of January 2000 had carried out investigations at 64 sites and prepared project reports for 56 sites.

Based on the request from Maritime States/UTs and on approval from the Govt. of India, CICEF carries out pre-feasibility studies, detailed engineering and economic investigations and prepares techno- economic feasibility reports in consultation with the concerned State Govts. The project reports are sent to Govt. of India for according Administrative Approval and Expenditure Sanction. The Ministry of Agriculture, on receipt of the project reports from CICEF/States, scrutinises the reports. An Appraisal note is prepared and sent to all concerned Ministries namely Ministry of Finance, Ministry of Environment and Forest, Ministry of Surface Transport, Planning Commission, and other related Ministries for their comments and approval. Based on the suggestions/comments received from the concerned Ministries, a final note is prepared and discussed in the meeting for approval of the project. Depending on the cost of the project, under the delegated financial powers, the Ministries/Departments expedite investment decisions and approval. Once the project is sanctioned, the Ministry of Agriculture, accords Administrative Approval and releases funds to the State Govts./UTs/Port Trusts depending upon the progress achieved for taking up construction. Further, funds are released to the States on the basis of the physical and financial progress achieved during construction period.

4.3 Environmental Clearance

Environmental clearance is a pre-requisite and an important factor considered for sanction of the project. The proposal is referred to the Ministry of Environment and Forest for assessing the environmental impact. The Pollution Control Board of the concerned State and Ministry of Environment and Forest examine and assess the environmental impact analysis. All project proposals located in the Coastal Regulation Zone area require environmental clearance. The necessary component on environmental protection works have to be involved in the project proposal.

5.0 Fishery Harbour projects developed by CICEF and concerned State Governments with particulars emphasis on Design of Breakwaters

Among the various fishery harbour projects designed and developed by the CICEF and the State Governments with particular reference to the design of breakwaters, a few projects are cited below:-

I. GUJARAT

Mangrol:

The Fishery harbour at Mangrol is situated in Junagadh District on the South west coast of Gujarat State. It is about 35 km northwest of Veraval Port. The Fishery Harbour is located on the open coast. Breakwater designed in 1976. The depth at the structure of the breakwater governs the wave height. The breakwater has been designed the following parameters. Wave height considered in the design of Trunks of the breakwater is based on the bed contour and depth consideration.

Wave height (breaking)	7 m		
Period	10 secs		
Armour Unit	Head	dolos	11.14 t
	Trunk	dolos	5.57 t and 2.23 t

Veraval

Veraval fishery harbour is situated in Junagadh District in the Southwest coast of Gujarat and at 35 km South of Mangrol. Breakwater designed in 1976. The Breakwater has been designed for the following parameters. The trunk position is designed for the wave height arrived based on the bed contour and the available depth at the structure.

Wave height(breaking)	7 m		
Period	10 secs		
Armour Unit	Head	dolos	11.14 t
	Trunk	dolos	5.57 t and 2.23 t

II. MAHARASHTRA

Ratnagiri

Ratnagiri fishery harbour is situated in West coast of India in Ratnagiri District of Maharashtra State. It is about 240 km South of Bombay. The western Breakwater is designed in 1976 for the following parameters:

Wave height	4 m		
Period	9 secs		
Armour Units:	Head	tetrapods	5.5 t
	Trunk	quarry stones	0.8 t to 3.0 t

The northern Breakwater is designed for a defracted wave developed by orienting the structure suitably and the armour units are of quarry stones.

III. KERALA

Puthiappa:

The Fishery harbour is situated in open coast in Calicut District of Kerala State. It is linked to the National Highway No.17. It is at about 4 kms from Calicut Town. The breakwater is designed in 1983 with following parameters.

Wave height (breaking)	3.5 m.
Armour: Head stones	3 to 5 t
Trunk stones	1 to 2 t

Vizhinjam:

The Fishery harbour is situated in the open coast at a distance of 16 kms south of Trivandrum in Kerala on the west coast of India. The extension of sea ward breakwaters have been designed for the following parameters in the year 1976.

Wave height:	4.8 m
Period:	9 secs
Armour Units: Head tetrapods	8 t
The leeward breakwater is designed for the distracted wave.	
Wave height:	2 m
Armour : quarry stones	2 to 3.5 t

IV TAMILNADU

Chinnamuttom

The harbour is situated at 2.5 North-East of Kanyakumari in the open coast in Tamilnadu. It is about 80 kms South-West of Tuticorin. The main breakwater is designed in 1982 for the following parameters.

Wave Height (breaking)	5 m
Armour Units: Head dolos	4.16 t
Trunk dolos	2.84 t

Western breakwater is designed for a defracted non-breaking wave.

Armour Units: stones	0.6 to 1 t
----------------------	------------

Rameswaram:

The project is formulated in 1997. The project site is situated 2 kms South-east of Fisheries Jetty in Rameswaram. The site is also at a distance of 60 kms east of Ramanathapuram, the District headquarters.

Breakwater is designed for the following wave parameters generated under trade wind conditions.

Main breakwater:			
Wave height(breaking)		2.9 m	
Period		5.21 secs	
Armour Units	Head	dolos	1.5 t
	Trunk	dolos	1.5 t
	Stones		2 to 4 t

6.0 Issues in Construction Management and Maintenance

The fishery harbour projects sanctioned by the Govt. of India, are stipulated with a definite time period for completion. In order to complete the projects on time, State Govts. and Port Trusts are required to plan the time schedule of the project by utilisation of Programme Evaluation and Review Technique (PERT) and Critical Path Method (CPM). In certain cases, the projects are delayed due to reasons beyond the control of the Executing Agency. Some of the reasons attributed to these inordinate delays in completion of the project on time are land acquisition problems, delay in getting clearance from the environmental angle, public litigation and multiplicity of agencies involved in the execution of the project. Most of the projects have resulted in time and cost over run due to the above reasons. Availability of the right type of equipment, labour, material, selection of experienced contractor executing marine projects and timely availability of funds are the basic requirements in completing the project on time. In many cases, poor approach road, non-availability of electricity, water supply and natural calamities like cyclones, storms, etc., also contribute to delay in construction of the projects resulting in cost/time over run. The cost escalation arises due to:

- time over run on account of natural calamities such as cyclones, prolonged monsoon, etc.,
- dispute over the contractual work in the Court of Law,
- revision of schedule of rates by the State Govts./UTs/Port Trusts,
- delays in land acquisition, award of contract, delay in proper technical investigations by the construction department,
- delay in conducting model studies and
- delay in timely availability of funds from the State Budget.

Most of the fishery harbours are not properly maintained due to lack of management and revenue collection. On completion of construction and commissioning of fishery harbours, the responsibility of maintenance and management vest with the user agencies. Only in a few fishery harbours, revenue is being collected regularly. The revenue collected at some of the harbours is so meagre which makes it difficult to manage and maintain. In some harbours, revenue has not been collected at all, thereby resulting in poor maintenance. The availability of adequate funds and collection of revenue for maintenance of these facilities in general and dredging in particular is essential. This has significant bearing on the availability of facilities for productive purposes.

Most of the State Governments and Port Trusts are approaching the Govt. of India, for extending financial assistance in the management and maintenance of fishery harbours. No arrangement for extending such financial assistance to the State Governments in respect of minor fishery harbours and landing centres exists. Harbours which are more than a decade old need to be rehabilitated. The approach towards development of new fishery harbours requires specialised engineering designs and need to be reviewed to meet the requirements of the Quality of Systems such as HACCP and ISO 9000.

7.0 Conclusion and Recommendations

Planning and development of a viable fishery harbour has a direct bearing on following few important aspects:-

- Selection of proper site which has a direct access with existing internal communication systems such as roads, railways availability of construction materials in close proximity, free from location of very high tidal range, tidal bore, high current velocity and cyclone prone areas as well as the locations free from places involving huge capital dredging.
- Collection of adequate field information by way of engineering investigations such as topo-hydrographic survey, sub-soil investigations, Meteorological data, Oceanographic data and finally the economic investigations.
- Adequate fishing activities in selected site to justify the investment on the project in order to ensure viable returns.
- Need of the testing proposals on model wherever breakwater/training walls are proposed in order to check the stability, to gather information on hydraulic performance of the structures, siltation pattern and protection of shore morphology in the areas adjacent to the project site.

Planning of the fishery harbour should include some of the functional requirements as follows:-

- Navigational channel
- Harbour basin
- Quays and Jetties
- Breakwaters/training wall wherever required
- Communication facilities and other shore based functional and operational components required for the harbour

Some of the fishery harbours in operation in India, lack certain requisite facilities conforming to international standards. As a result, proper management of harbour is not ensured.

Special design approaches covering layout formation need to be adopted by the engineers and the organisations involved in formulating new harbour projects to meet

the requirements of International Standards laid down by HACCP and ISO 9000 in order to ensure effective maintenance of the harbour after construction.

Some of the important factors needing impetus in the development of fishery harbors are:-

- ◆ Suitable design approaches for construction of modern auction halls and allied facilities for hygienic handling of fish.
- ◆ Adopting effective strategies for rehabilitation of the existing fishery harbours and development of new fishery harbours at economic cost.
- ◆ Maintenance of facilities created in the harbour by periodical maintenance dredging to ensure uninterrupted vessel traffic in and out of the harbours and to enable speedy landing, handling and disposal of fish catch to markets.
- ◆ Collection of appropriate and workable user charges from the user groups using the facilities is necessary for mobilisation of funds required for keeping the establishment as a self-sustainable element, and

Finally, it is needless to mention that if all the points highlighted above are given proper emphasis and brought into practice in site selection, investigation, planning, designing, construction and maintenance of the harbour, the fish quality and quantity can be ensured which in turn contribute to the augmentation of the earnings in the trade.

Table 1

STATE-WISE MARINE FISH PRODUCTION 1997-98

(In Tonnes)

STATES/UTs	
1. Kerala	526342
2. Karnataka	189859
3. Goa	88809
4. Maharashtra	453000
5. Gujarat	745706
6. Tamil Nadu	355100
7. Andhra Pradesh	146545
8. Orissa	156081
9. West Bengal	164000
10. Pondicherry	38420
11. Daman & Diu	18807
12. Andaman & Nicobar	27225
13. Lakshadweep	10550
14. Deep Sea	30000
Total	2950444

TABLE - 2

FISHING CRAFTS IN MARITIME STATES/UTS AS IN 1994-95

States/UTs	Traditional Crafts	Motorised Traditional Crafts Out of Col.(2)	Mechanised Boats	Total
(1)	(2)	(3)	(4)	(5)=(2)+(4)
Gujarat	12653	4283	8365	21018
Maharashtra	9988	286	7930	17918
Karnataka	13141	1189	3655	16796
Kerala	40786	12913	4206	44992
Tamil Nadu	32077	5340	8230	40307
Andhra Pradesh	57269	3269	8911	66180
Orissa	10249	2453	1665	11914
West Bengal	4361	270	1880	6241
Lakshadweep	1078	298	443	1521
A & N Islands	1340	160	230	1570
Pondicherry	6265	365	553	6818
Goa	2000	900	850	2850
Total	191207	31726	46918	238125

Annexure I

Present Status of Fishery Harbours and Landing Centres Commissioned/Under Construction under the Govt. of India Schemes

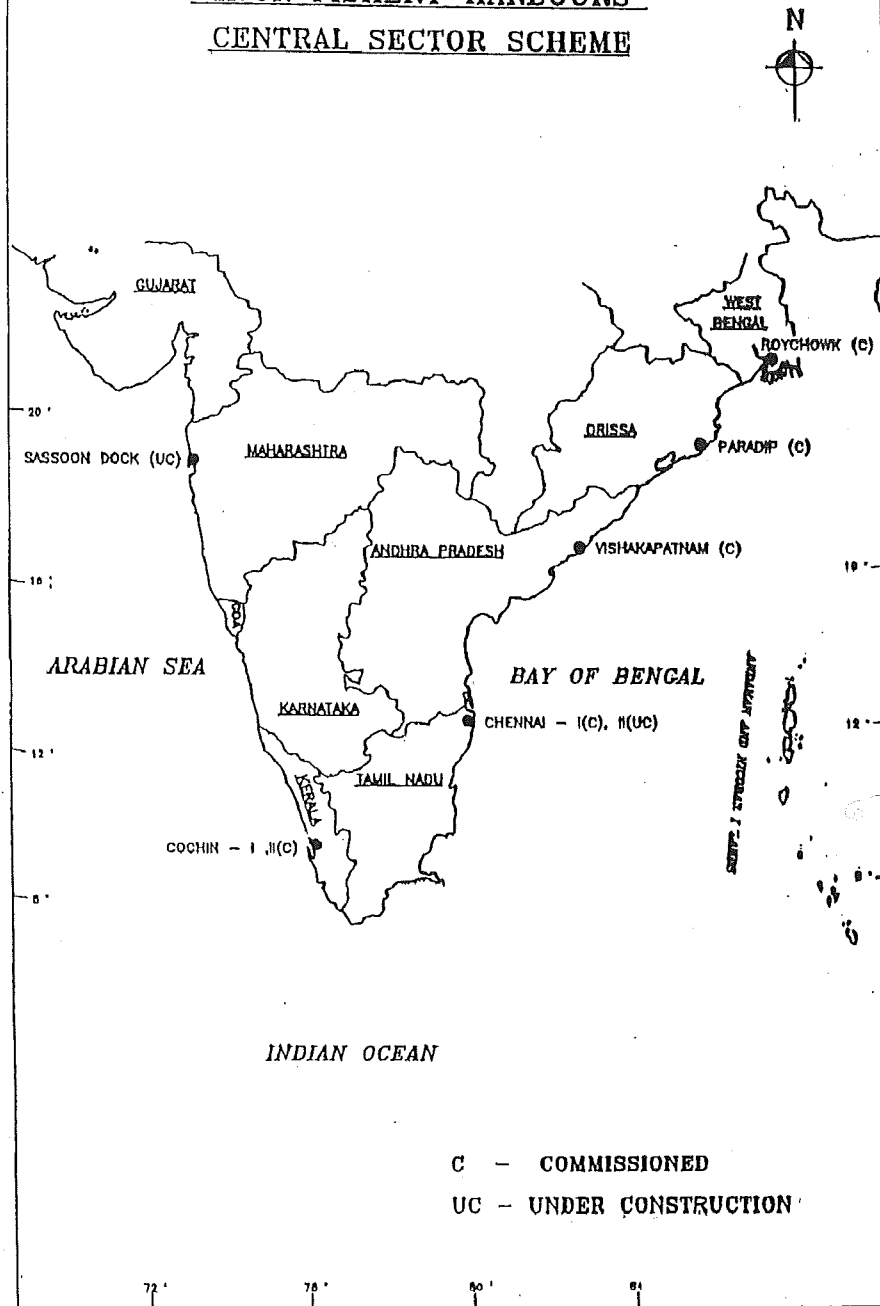
Sl. No.	State	Name of Fishing Harbour	
		Commissioned	Under Construction
A. Major Fishing Harbour			
1	Kerala	Cochin Stage I&II	
2	Maharashtra		
3	Tamil Nadu	Chennai Stage I	
4	Andhra Pradesh	Viszag- Stage I, II & III	Sasson Dock
5	Orissa	Paradip	Chennai Stage -II
6	West Bengal	Roychowk	
B. Minor Fishing Harbour			
1	Kerala	Vizhinjam Stage I	Chombal
		Puthjappa	Mopla Bay
		Munambam	Kayamkulam
		Vizhinjam Stage II	Vizhinjam Stage II
		Neendakara	Thangassery
2	Karnataka	Karwar	Malpe stage II
		Honnavar	Mangalore Stage II
		Tadri	Karwar Stage II
		Mangalore	
		Malpe Stage -I	
3	Gujarat	Veraval	Jakhau.
		Mangrol	Mangrol Stage - II
		Porbandar	
4	Maharashtra	Ratnagiri	Agrao
5	Tamil Nadu	Tuticorin	Chinnamuttom
		Mallipatnam	
		Kodiakarai	
		Vallinokkam	
		Tondi	
		Pazhayar	
6	Andhra Pradesh	Kakinada	Machilipatnam
		Nizamapatnam	
		Bavanapadu	
7	Orissa	Gopalpur	Dhamra Stage II
		Dhamra	
		Naugarh (Astrang)	
8	West Bengal	Fraser Ganj	Digha stage - II
		Digha stage -I	
9	Pondicherry		Pondicherry
10	Andaman & Nicobar	Phoenix Bay	

C Fish Landing Centres				
1	Kerala	Kasaragod	Dharmadon	Punnappra
		Ponnani	New Mahe	Quilandy
		Cannanore	South Paravoor	Moylali
		Beliapatnam	Vellayil Beach	Kaddappuram
		Neeleswaram	Vallikunu	Kattoor Pollathai
		Thottappally	Vizhinjam South	
		Munakkadavu	Vizhinjam North	
		Cheruvathur	Chettuvai	
		Beypore	Arthugai	
		Palacode	Chalil Gopalpettah	
2	Karnataka	Coondapur	Gangolli	Kodibengre
		Bhatkal	Sadasivgad	Alvekodi
		Kagai Heni	Belikeri	Gangolli - II
		Mulki	Belambar	Hejmadikodi
			Keni	Belikeri Stage-II
3	Goa	Cortalim		Malim
4	Maharashtra	Karanja	Dakti-Dahanu	Sarjekote
		Navalgaon	Khardanda	Alibagh Koliwada
		Borli Mandla	Ekdara	Tarkarli
		Nandagaon	Mandavi	Achara Peerwada
		Nurad	Mulgaon	Taramumbri
		Thoorinda	Navapur	Rajpuri Koliwada
		Ajanala	Onne-Bhatti	Ekdara Koliwada
		Ade-Uttambar	Thurnvadi	
		Agrao	Thai	
		Borin	Uttoon	
		Burondi	Vashi	
		Bigmandla	Wadrai	
		Datiware	Rajpuri	
		Dahanu	Jeevne Bundar	
			Mahim Causeway	
5	Gujarat	Navapur	Sachana	Navabandar
		Jalliabad	Salaya	
		Umbergaon	Mandvi	
		Kolak	Madhwad	
		Jakhau	Surajbari	
		Hirakot	Jakhau I	
		Vansi Borsi	Umersadi	
		Chorwad	Dholai	
		Magod Dugari	Rajapara	
		Kosamba	Port Onjal	

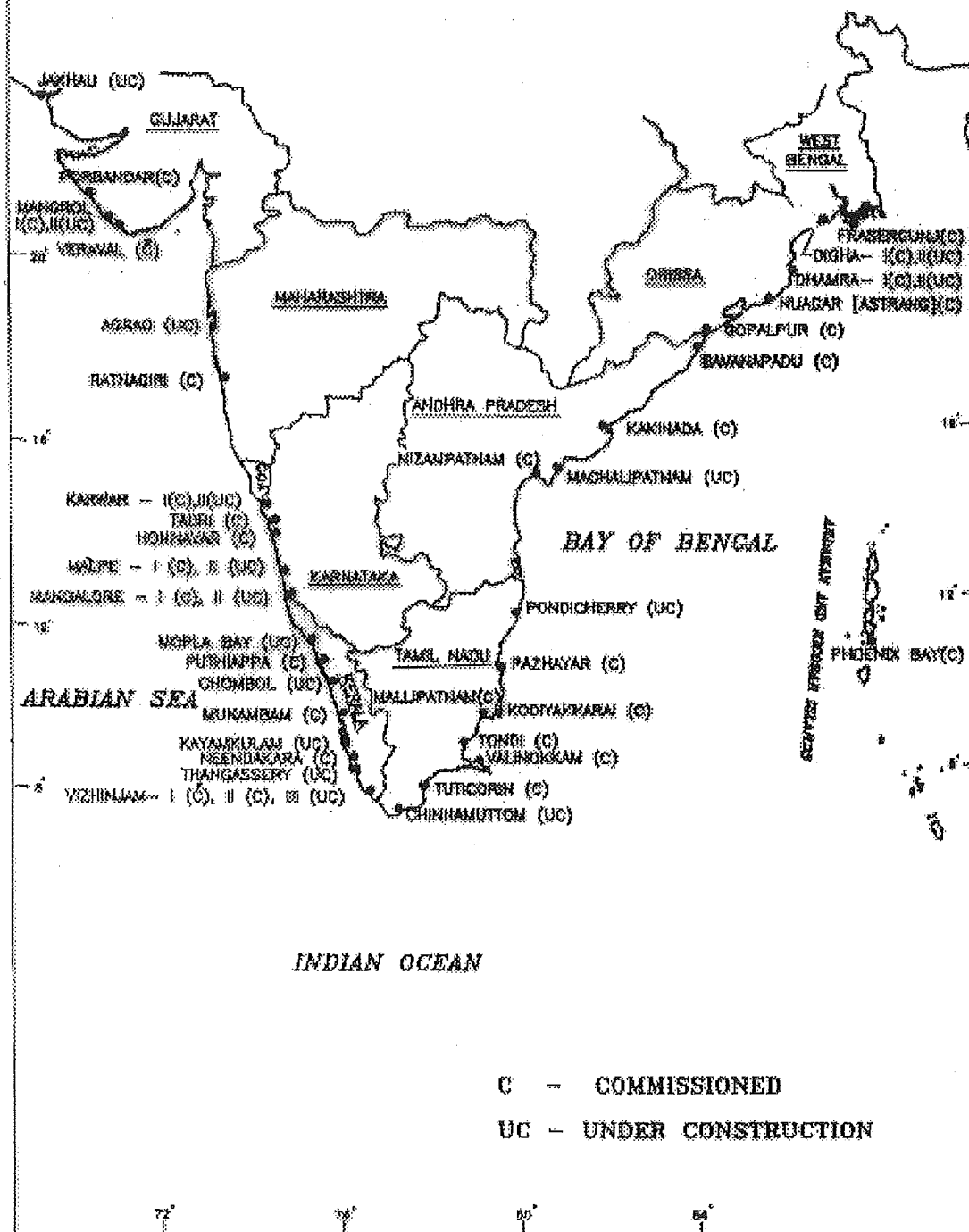
6	Tamil Nadu	Cuddalore	Muttom	
		Nagapattinam	Poompuhar	
		Rameswaram	Vallapallam	
		Palk Bay	Kodimunai	
		Kottaipatanm	Vallavillai	
		Erawadi		
7	Andhra Pradesh	Calingapatnam		Mangipudi
8	Orissa	Chandipur	Talchua	Tantiapal
		Sabelia	Jamboo	Sorala
		Pathara	Kharansi	Bandara
		Chudamani	Palaur	Khandiapatna
		Nairi	Chandrabhaga	Bhusandpur Baliapatpur
		Panchubisa	Kansabans	Kirtania
		Nairi-II	Rushikulya	Talasari
			Soran	Ponthakata
				Gopalpur on sea
				Bahabalpur
9	West Bengal	Namkhana	Bamanagar	Madanganj
		Jalda	Ganeshpur	Brajabalavpur
		New Jalda	Akhoy nagar	
		Kalinagar	Junput	
		Kharpai	Soula	
10	Pondicherry	Mahe		
11	Lakshadweep	Karavatti		
		Minicoy		
		Agatti		
12	Daman & Diu			Ghogla
				Vanakbara

Summary			
Category of Harbours	Commissioned	Under Construction	Total
Major Fishery Harbours	5	1	6
Minor Fishery Harbours	29	16	45
Fish Landing Centres	120	33	153

MAJOR FISHERY HARBOURS
CENTRAL SECTOR SCHEME



MINOR FISHERY HARBOURS CENTRALLY SPONSORED SCHEME



ANNEXURE IV

**OUTLAYS AND EXPENDITURE FOR DEVELOPMENT
OF MAJOR AND MINOR HARBOURS (GOVT. OF INDIA)**

(Rs.in Million)

Plan Period	Major Harbours		Minor Harbours	
	Outlay	Expenditure	Outlay	Expenditure
Third Plan	0.5	1.2	-	-
Three Annual Plans (1966-67 to 1968-69')	25.3	0.3	29.5	15.7
Fourth Plan	135	15.8	60	48.5
Fifth Plan	180	121	120	32.1
Annual Plans				
(i) 1978-79	50	30.8	60	39.4
(ii) 1979-80	55	20.6	-	0.1
Sixth Plan	170	132	190	179.5
Seventh Plan	170	168.4	190	216.9
Annual Plans				
(i) 1990-91	49.2	49.2	43.2	43.2
(ii) 1991-92	50	54.5	60	60
Eighth Plan				
(i) 1992-93	80	127.9	60	60.1
(ii) 1993-94	140	120	70	67.5
(iii) 1994-95	100	100	110	111
(iv) 1995-96	110	75.1	112.5	116.7
(v) 1996-97	91.3	34.5	115	135

ANNEXURE V**Total outlay and year-wise phasing during Ninth Five
Year Plan (Govt. of India)**

Year	Central Government Financial outlay (Rupees in Million)
1997-1998	190.3
1998-1999	200.0
1999-2000	300.0
2000-2001	300.0
2001-2002	409.7
Total	1400.0

ANNEXURE VI

Master Plan for the development of fishery harbours in India (As prepared during 1978 to 1981)

State/UT	Fishery Harbour sites	Facilities available/ Under Construction			Sites recommended for investigation		
		Major	Medium	Minor	Major	Medium	Minor
Gujarat	19	1	1	4	1	-	12
Maharashtra	14	1	1	-	1	-	11
Goa	3	-	-	-	-	1	2
Karnataka	8	1	-	6	-	-	1
Kerala	14	1	-	6	-	-	7
Tamil Nadu	20	1	1	7	1	-	10
Andhra Pradesh	17	1	-	3		2	11
Orissa	12	-	1	1	1	-	9
West Bengal	3	1	-	-	-	-	2
Pondicherry	2	-	-	-	-	-	2
A&N Islands	3	1	-	-	2	-	-
Lakshadweep	2	-	-	-	-	-	2
Total	117	8	4	27	6	3	69

ANNEXURE VII

Potential sites identified by CICEF development of Fishery Harbours and Fish Landing Centres

State/UT	Proposed Harbour Facilities	
	Minor FH	Fish Landing Centre
1. Gujarat	4	3
2. Maharashtra	4	-
3. Goa	2	-
4. Karnataka	7	4
5. Kerala	5	-
6. Tamil Nadu	11	4
7. Andhra Pradesh	1	-
8. Orissa	3	-
9. West Bengal	1	1
10. Daman & Diu	2	1
11. Pondicherry	1	-
12. Andaman & Nicobar Islands	-	12
Total	41	25

ANNEXURE VIII

Sites found suitable for development of Fishery Harbours/ Fish Landing Centres under Master Plan

State/ Union Territories	Proposed	
	Fishery Harbours	Fish Landing Centres
1. Gujarat	Rupen	Madhavpur
	Mangrol Bara	Sutrapada
	Dholai *	Dhamlej
	Umbergaon*	
2. Maharashtra	Deogad *	
	Sakharinate	
	Harnai	
	Agardanda *	
3. Karnataka	Karwar **	Gangavali
	Belambar	Belekeri**
	Alvekodi *	Koderi
	Mangalore Stage II **	Shiroor
	NMPT	
	Gangolli *	
	Amadalli*	
4. Goa	Chicalim	
	Malim **	
5. Kerala	Ponnani	
	Muthalapozhy	
	Kasargode	
	Neeleswaram	
	Chettuvai	
6. Tamil Nadu	Cuddalore Stage II	Portonovo
	Pazhayar Stage II *	Tirumullaivasal
	Mallipatnam Stage II	Periyatalai
	Tuticorin Stage II	Ovari
	Poompuhar *	
	Arcotthurai	
	Rameswaram*	
	Veerapandiyanpattinam *	
	Kulasekharapattinam	
	Colachel*	
	Thengapattinam	

7. Andhra Pradesh	Krishnapatnam *	
8. Orissa	Dhamra Stage II **	
	Bahabalpur	
	Chudamani	
9. West Bengal	Harwood Point	Diamond Harbour
10. U.T. of Daman	Nani Daman	Ghoghla *
& Diu	Vanakabara *	
11. U.T. of Pondicherry	Karaikal *	
12. A & N Islands		Junglighat
		Panighat
		Guptapara
		New wandoor
		Havelock Island
		Neill Island
		Uttara jetty
		Yeratta
		Rangat Bay
		Betapur
		Maya bundar
		Durgapur

* Sites investigated by CICEF

** Projects Sanctioned by the Ministry of Agriculture

Fishery Harbours

Fishery Harbour sites proposed for development : 41

Fishery Harbour sites investigated by CICEF : 15

Fishery Harbour projects sanctioned by the Ministry : 4

Fishery Harbour project reports prepared and awaited sanction : 4

Fish Landing Centres

Fish Landing Centre sites proposed for development : 25

Fish Landing Centre sites investigated by CICEF : 1

Fish Landing Centre projects sanctioned by the Ministry : 1

Fish Landing Centre project reports prepared and awaited : 1



NON-RUBBLE BREAKWATERS AND OPTIMISATION

Kees d'Angremond
Department of Civil Engineering & Geo Sciences
Delft University of Technology
Email: k.dAngremond@ct.tudelft.nl

1. INTRODUCTION

It is complicated to address two completely different subjects in one paper. Still, an attempt will be made to do so.

When discussing non-rubble breakwaters, one opens the mind for a wide variety of solutions: all except the breakwaters composed of quarry stone. This is already an indication for the popularity of rubble mound breakwaters. And such world-wide popularity must reflect the advantages of that type of design, and so it does.

Of course, there are other types of breakwaters. The best known type is the monolithic structure, often consisting of a caisson to form the body of the structure. This type will mainly be discussed in this contribution. Other types of breakwaters that one finds occasionally described in literature are the floating breakwater and the pneumatic/hydraulic breakwater. These two types will not be discussed extensively because of their limited performance. Although it is possible to damp waves by using a floating object, and although it is possible to damp waves by blowing water or air from a submerged perforated pipe, these methods fail to provide adequate protection, in particular when one attempts to damp long-periodic waves. Study of these types of breakwaters is in my opinion rather an academic than a practical exercise.

Last but not least, attention will be paid to the economic optimisation of breakwaters. We know a lot of formulae to assess breakwater stability, but all these formulae introduce the wave height as decisive load parameter. None of the formulae, however, guide us in the selection of the numerical value of the design wave height. This aspect will cover the second part of this contribution.

2. MONOLITHIC BREAKWATERS

The problem of the stability of monolithic breakwaters has not been solved in a satisfactory and generally accepted way yet. Research efforts are under way, but have

not resulted in a generally applicable theory or formula. Nevertheless, monolithic breakwaters are being built, and designers do use practical formulae. In this chapter, we will discuss a theoretical approach and a practical method developed in Japan. As the stability is a joint effect of wave load and subsoil resistance, some soil mechanics will be discussed as well.

Because of the intense interest in many countries, a rapid development of the knowledge of monolithic breakwaters must be expected, compatible with the evolution around rubble mound breakwaters between 1988 and 1993. For the reader it means that always the most recent sources of literature shall be consulted.

3. WAVE FORCES AND THEIR EFFECTS

3.1 Quasi static Forces

In the linear wave theory, there is a formula for the pressure distribution under a wave:

$$p = -\rho g z + \rho g \eta \frac{\cosh[2\pi(z+h)/L]}{\cosh(2\pi h/L)}$$

On the basis of this formula, Sainflou [1928] developed a method to calculate pressures on a vertical wall by non-breaking waves. Rundgren [1958] carried out a series of model experiments and concluded that Sainflou's method overestimates the wave force for steep waves. Rundgren then used and modified the higher order approach as proposed by Miche [1944]. This Miche-Rundgren method gives satisfactory results for steep waves, whereas the original Sainflou-method is best suited for long and less steep waves.

The main and important aspect of the Miche-Rundgren approach is the definition of a parameter h_0 , which is a measure for the asymmetry of the standing wave around SWL. This leads to pressure diagrams as shown schematically in Figure 1.

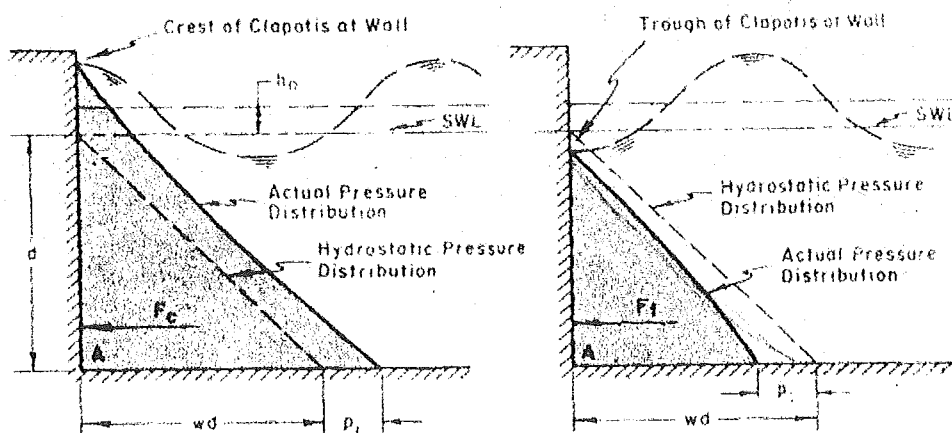


Fig. 1 Schematic Pressure Distribution for non-breaking Waves

In this figure, w_d and p_1 are given by:

$$w_d = \rho g h \text{ and } p_1 = \left(\frac{1+r}{2} \right) \frac{\rho g H_1}{\cosh\left(\frac{2\pi h}{L} \right)}$$

The Shore Protection Manual gives design graphs for the calculation of h_o as a function of wave steepness, relative wave height (H/h) and reflection coefficient. It also gives graphs to calculate integrated pressures and resulting turning moments for crest and trough of the wave.

This leads to a relatively simple load diagram (Figure 2), in which the horizontal hydrostatic forces on the front and rear wall have been omitted because they eliminate each other. For stability, one must consider the resistance against translation and the resistance against rotation. It is stressed here that the resistance against rotation can not be taken simply as the sum of the moments around point A. Long before the structure starts rotating, the pressure under point A has reached a value that leads to failure of the subsoil or failure of the corner of the structure.

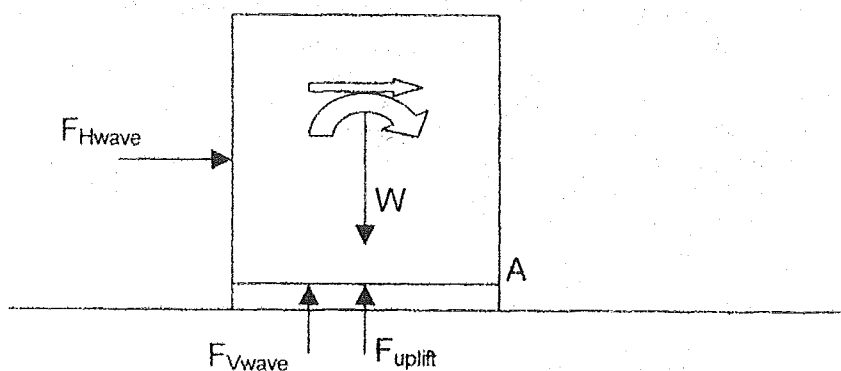


Fig. 2 Load and Equilibrium Diagram

Since these formulae have been derived for regular monochromatic waves, it is necessary to combine them with spectral theory and arrive at a statistical distribution of wave forces and overturning moments. It can then be decided what frequency of exceedance is accepted during the lifetime of the structure. In this way, the design loads can be established.

The loads defined so far are called quasi-static forces, because they fluctuate with the wave period of several seconds and do not cause any (direct) dynamic effects. Inertia effects need not be taken into account.

3.2 Dynamic forces

In 3.1, we restricted ourselves to the forces by non-breaking waves. When waves are breaking, we know, however, that impact or shock pressures occur in the vicinity of

the water surface. The duration of those pressures is very short, but the (local) magnitude is very large. The quasi static pressures are always in the order of $\rho g H$, but the impact pressures can be 5 to 10 times higher. An example of a pressure record is given in Figure 3.

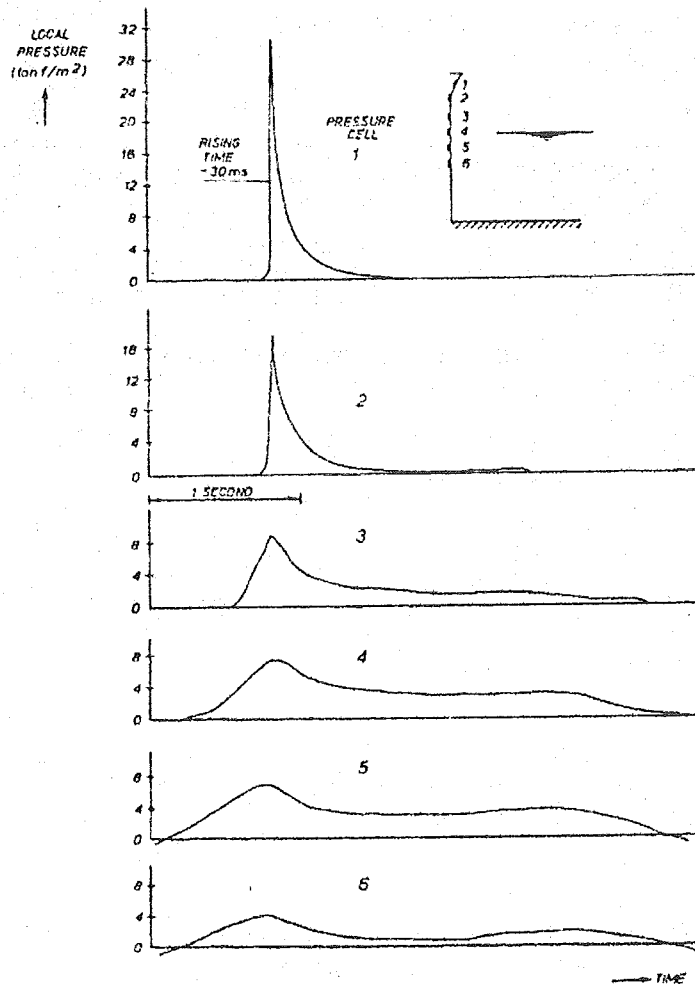


Fig. 3 Example of a Pressure Record

Many researchers have studied the phenomenon in the laboratory, and none have come with a satisfactory explanation that can predict the occurrence and the magnitude of a wave impact as a function of external parameters. Bagnold [1939] was the first of those researchers. He found that the impact pressure occurs at the moment that the vertical front face of the breaking wave hits the wall, and mainly when a plunging wave entraps a cushion of air against the wall.

Apparently, the deceleration of the mass of water in the wave crest, combined with the magnifying effect of the air cushion, causes the high pressures. Two models can be used to describe and calculate this effect-

- The continuous water jet hitting a plane yields a pressure:

$$p = \frac{1}{2} \rho u^2 \text{ (u is the water velocity in the jet)}$$

- The water hammer effect, resulting in:

$$p = \rho u c$$

in which:

u = the water velocity in the conduct

c = the celerity of sound in water (1 543 m/s)

The water velocity in the crest of the breaking wave is equal to the wave celerity (in shallow water: \sqrt{gh})

Substitution of reasonable figures leads to a water velocity in the order of 10 m/s and impact pressures:

Continuous jet: 55 kPa (5.5 mwc)

Water hammer: 16,000 kPa (1600 mwc)

In reality, we know that the impact pressures reach values between 50 and 150 mwc.

Measurement of the impact pressures in a model is complicated because the short duration of the load requires a very stiff measuring system to provide proper data. Moreover, the compressibility of the water (influenced by entrained air) is an important factor because it determines the celerity of the compression wave in water. Uncertainties about model conditions endanger upscaling into prototype figures.

Minikin [1955 and 1963] has given a method to calculate wave impact pressures, but his method overestimates impact pressures and does not lead to satisfactory results.

From all experiments it has become clear, however, that the duration of the wave impact is short, and the area where the impact takes place at the same time is small.

This means that the wave impact forces can not be used for a static equilibrium calculation. The dynamic effects must be taken into account, inclusive mass and acceleration of the breakwater in conjunction with its elastic foundation and the added mass of water and soil around it. Preliminary analysis has shown that it is specifically the momentum connected with the breaking wave that determines the stability or loss of stability of the breakwater. Care must also been taken of potential resonance phenomena, when the loading frequency coincides with the resonance frequency of the structure as a whole or for some individual members of the structure.

It would be a sound method of design to establish a physical relation between the impact pressure, the hydraulic parameters and the structural parameters. On the basis thereof, one should establish the exceedance curves of certain loads during the lifetime. Taking into account the response of the structures one can then determine the probability of failure of the structure during its lifetime. Unfortunately, the physical description of wave impacts is insufficient to start this approach.

The most important lesson that can be learned from this paragraph is the uncertainty that is connected with wave impact forces as such and their effect on the stability of monolithic breakwaters. It is therefore good engineering practice to try and avoid exposure of monolithic breakwaters to breaking waves. In this context it is good to point at the fact that even if no breaking waves are expected at the location of the breakwater, the breakwater cross section may induce them itself, specifically when the monolith is placed on a high mound of stone (Figure 4).

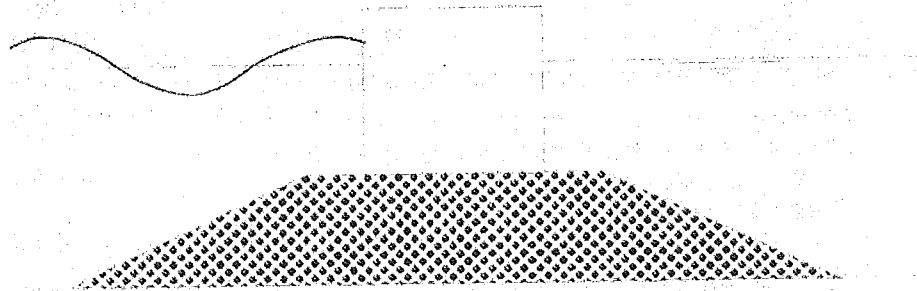


Fig. 4 Changes to incoming wave Front induced by high Mound Breakwater

It can further be concluded that the risk of local impact pressures increases for structural elements that entrap breaking waves. If water can escape sideways from the impact area, the pressures remain low (compare free jet). If water can not escape, the local pressures may become quite high (compare water hammer). In this way, certain details of monolithic breakwaters are relatively vulnerable (Figure 5).

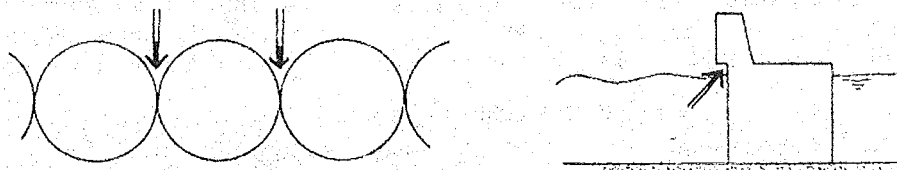


Fig. 5 Risk of local impact Forces

3.3 A working compromise: the Goda formula

Where the uncertainties around the design of vertical breakwaters have reduced the number of such breakwaters in Europe and the USA, in Japan, construction continued with varying satisfaction. Goda analysed many of the successful and unsuccessful structures and came up with a practical formula that can be used to analyse the stability of a monolithic breakwater. From a theoretical point of view, one can object that Goda is not consistent in his definition of design load and risk. In practice, the safety factors he proposes are apparently adequate, as long as one realises that conditions with breaking waves should be avoided as much as possible. If this is not possible, extensive model investigations are to be carried out, followed by a dynamic analysis of structure and foundation. In that case, one must take into account all inertia terms.

Goda [1992] has summarised his work in an article published in 1992 at the short course on design and reliability of coastal structures. Pending further theoretically based developments, the Goda formula can help to establish a first idea about stability of a monolithic breakwater.

3.4 Influencing the forces

It has been shown that the quasi-static forces and the dynamic forces have a trend to translate and rotate the structure, resulting in displacement of the structure and/or damage to the foundation and the bottom corners.

The effect of the external forces can be reduced by changing the direction of the horizontal force, or by spreading the force in space and in time.

The first effect can easily be understood if one realises that the water pressure is always acting along the normal on a plane. When the front wall of the monolith is tilted, it means that the wave force is no longer horizontal, but directed towards the foundation. This reduces the horizontal component and strengthens the vertical component of the force. Altogether, the likelihood of sliding reduces and the overturning moment is also reduced. (figure 6).

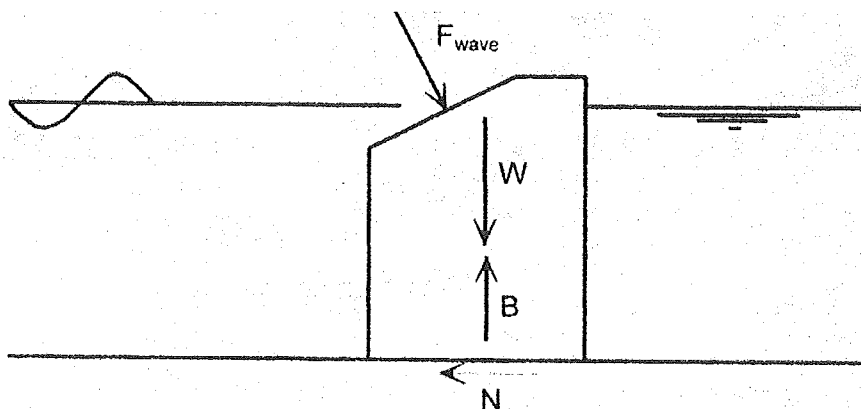


Fig. 6 Hanstholm Caisson

Another method is the creation of a chamber in front or on top of the structure, so that the point of application of the force is spread over two walls, and a time lap is created between the two forces. This reduces the maximum instantaneous force, although the duration is elongated. Jarlan (1961) first applied such idea (Figure 7), partly to reduce forces, partly to reduce the reflection. In Japan, a large number of similar ideas has been developed and brought into practice. In a number of cases, the idea is combined with power generation. Many of these designs have been described by Tanimoto and Takahashi (1994). Some typical design features are given in Figures 8 through 11.

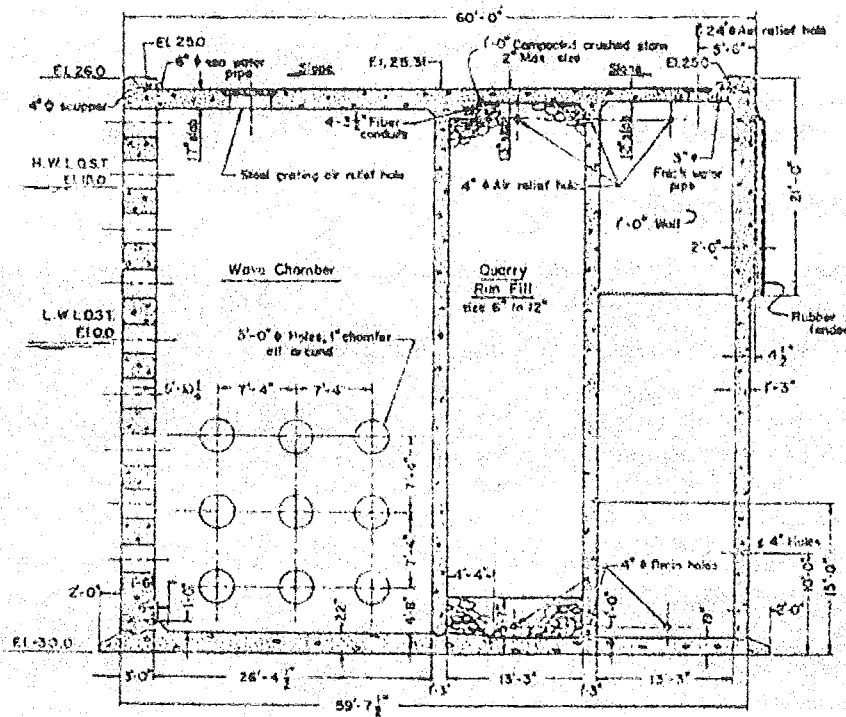


Fig.7 Jarlan Caisson

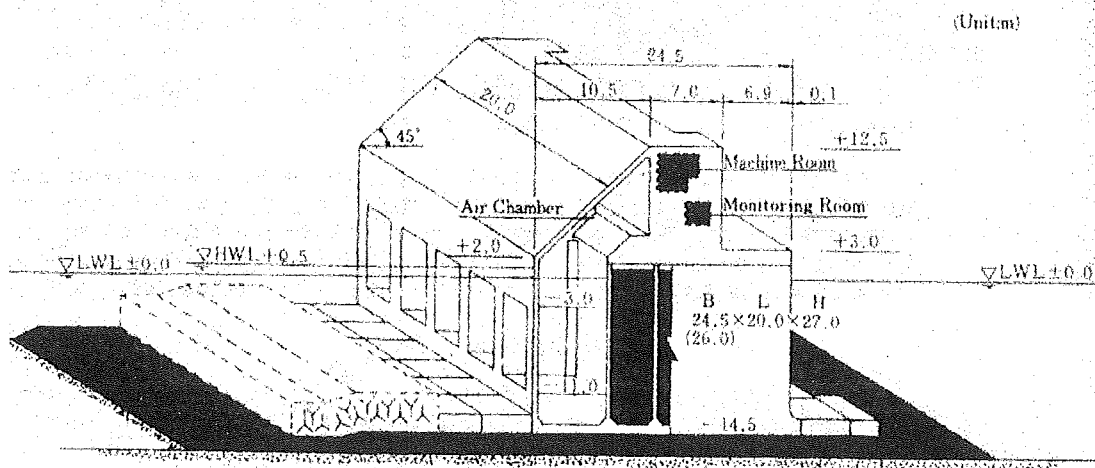


Fig. 8 Breakwater with Power generating unit

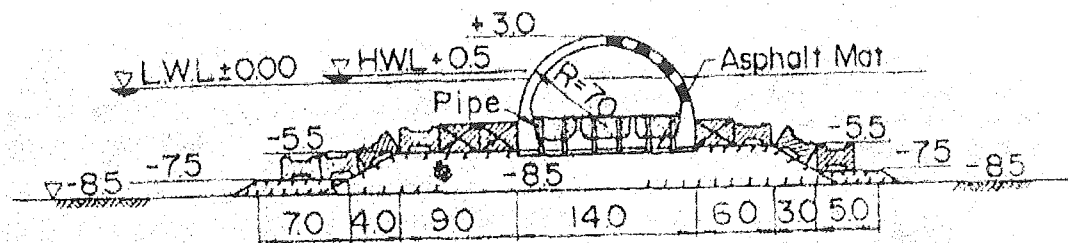


Fig. 9 Possible Cross-section of semi-circular Caisson Breakwater for extremely high Breakers

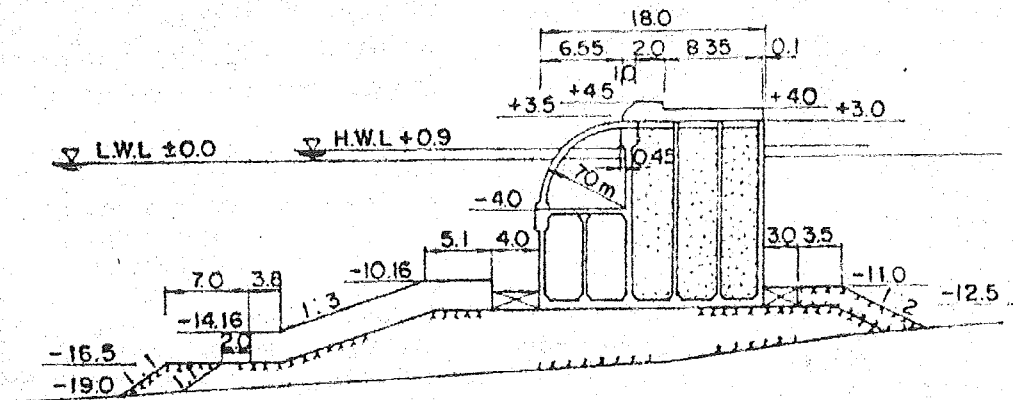


Fig. 10 Cross Section of curved-slit Breakwater at Funakawa Port

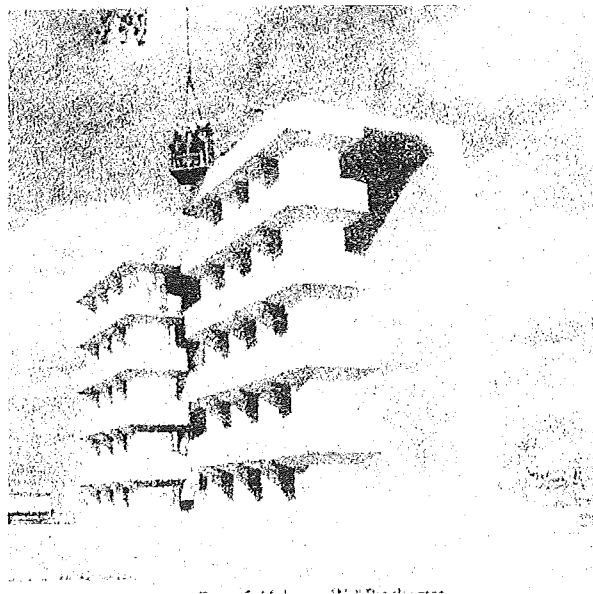


Fig. 11 Honey Wall Breakwater

4. FOUNDATION

The hydraulic forces exerted on the caisson plus the weight determine what will be the local pressures in the interface between the caisson and the foundation. It will be clear that these pressures must not lead to (soil mechanical) failure. Because the foundation is flexible to a certain extent, it must be verified whether the mass-spring system formed by caisson (mass) and foundation (spring) gives rise to resonance phenomena. Depending on the outcome of that investigation, one may decide that a static stability analysis is sufficient (as is often the case). Soil-mechanical failure is nevertheless one of the most likely failure modes.

Even if it is decided after analysis that a quasi-static approach is justified, the cyclic effect of the load may not be overlooked. The load will anyway cause an increase of the total stress level (σ) and initiate a compression of the subsoil. In first instance this will lead to a higher stress level in the ground water (p). Depending on the permeability of the soil, the excess water will drain and gradually, the effective stress (σ') will increase. This all in accordance with one of the basic laws from soil mechanics:

$$\sigma = p + \sigma'$$

Because of the cyclic character of the load, it is possible that drainage of excess water is not complete when the next loading cycle starts. In this way, the water pressure may gradually increase due to rocking of the caisson. Eventually, this will lead to a condition that the effective stress σ' becomes very low or even zero. A low effective stress will greatly reduce the resistance against sliding; an effective stress equal to zero leads to liquefaction or the formation of quick-sand. This is the main reason that care is recommended when designing monolithic breakwaters in areas that are sensitive for liquefaction: soil consisting of fine, loosely packed sand as in the SW part of the Netherlands.

Preventive methods against liquefaction are possible, but expensive. Soil replacement and compaction of the subsoil are the most common methods. Widening the base of the caisson is also an effective measure.

Because of the possibility that high ground water pressures occur under the corners of the monolith, also large vertical gradients are likely. It is therefore necessary to cover a (fine) grained subsoil with an adequate filter. Because of the large gradients, it is recommended that the filter be designed as a geometrically impervious filter. Filter rules have been treated extensively by Terzaghi.

A granular foundation layer may also be required if the structure is placed on an uneven hard seabed. In that case, it is the function of the foundation layer to flatten the seabed and to avoid pressure concentrations and an unpredictable support pattern of the structure. Alternatively, one may create pre-designed contact areas in the bottom of the structure, so that the bending moments in the floor plate can be calculated.

To create a perfect and homogenous contact plane between the foundation and the structure, sometimes a grout mortar is injected. This technique has been developed in the offshore industry for the foundation of gravity platforms, but the use has spread to regular coastal engineering projects as well. To avoid loss of grout, a skirt is provided along the circumference of the bottom of the caisson. This skirt (mostly a steel sheet) penetrates into the foundation and creates a chamber that can be filled with the grout mortar.

5 OPTIMISATION

The optimisation process is equally valid for rubble and monolithic breakwaters. In the present paper examples are given for a rubble breakwater.

5.1 Micro Level

Optimisation at micro level can best be explained in the deterministic design process. It aims at a design that leads to the minimum total cost for a given strength level. To achieve this goal, it is necessary that all material in the structure fulfils its function, and is used in the optimum way.

This can be compared with designing a frame. It is then attempted to select the members such that all are exposed to a stress level close to the maximum admissible stress. In the same way, it can be attempted that all elements in a breakwater are close to (partial) failure when exposed to the design load.

In a probabilistic design process, it means that one should avoid a very large contribution to overall failure by a single partial failure mechanism while other mechanisms do not at all contribute to the probability of failure. It is wise to distribute the contribution to overall failure over a number of failure mechanisms. In fact, one should base this distribution on considerations of marginal cost. If a construction element is relatively cheap it is not so much of a problem if it is over-designed. If it is relatively expensive, over-designing in comparison with other elements leads to too high cost.

It means that the designer shall attempt to make a balanced design. This can easily be explained when considering the cross section of a rubble mound breakwater. If the crest level is designed so high that no overtopping occurs even under severe conditions, it makes no sense to protect the inner slope with heavy armour stone. For a low crested breakwater on the other hand it is essential to carefully protect the inner slope.

5.2 Macro Level

Also optimisation at macro level can best be explained in the deterministic design process, when only one failure mechanism with simple load and strength parameters is considered. When more mechanisms and parameters play a role, the

calculations become rapidly more complicated, and one should be careful not to make mistakes that lead to completely false conclusions.

The method was developed by Paape and Van de Kreeke [1964] for rubble mound breakwaters as early as 1964. The method is discussed in the following, and a sample calculation is given in an Annex. References to Tables and Figures refer to that Annex.

The method starts with the assumption that there is a direct relation between one load parameter (the no damage wave height, H_{nd}) and a strength parameter (the weight of the armour units, W). It is further assumed that the wave climate is known and available in the form of a long-term distribution of wave heights (Table A-1). The interaction between load and strength is determined on the basis of laboratory experiments, which indicate that damage starts when a threshold value (H_{nd}) is exceeded. The damage to the armour layer increases with increasing wave height until the armour layer is severely damaged and the core of the breakwater is exposed. This occurs at an actual wave height $H = 1.45 H_{nd}$. It is assumed that damage is then so far extended that no repair is possible, and that the structure must be rebuilt completely. For intermediate wave heights, a gradual increase of damage is assumed, expressed in a percentage of the number of armour units to be replaced (Table A-2).

The breakwater is then designed for a number of design wave heights, where a higher design wave causes a heavier and more costly armour layer, whereas the core remains unchanged. The cost of construction is I . The cost of rebuilding the breakwater is assumed to be equal to the estimated construction cost, the cost of repairing damage to double the unit price of the armour units. It is then possible to list the construction cost and the anticipated cost of repair, still split over the three categories of damage (4%, 8% and collapse). Adding up the three categories of damage for a particular design wave height yields the average annual risk anticipated for that design if all damage is repaired in the year the damage took place. If it is decided not to repair the breakwater except in case of collapse, the risk is just the risk caused by the category collapse.

Since the risk is still expressed in a value per annum, it must be ascertained what amount of money shall be reserved at the moment of construction to be able to pay the average annual repair cost during the lifetime of the structure. Although money is regularly spent from this repair fund, it still accrues interest at a rate of $\delta\%$ per annum.

If the annual expense is s , the interest rate $\delta\%$, and the lifetime of the structure T , it can easily be derived that the fund to be reserved (S) is:

$$S = \int_0^T e^{-\frac{\delta}{100}t} dt = s \frac{100}{\delta} \left(1 - e^{-\frac{\delta T}{100}} \right)$$

for $T = 100$ years $S = s \cdot 100/\delta$, and
for $T = 10$ years $S = 0.35s \cdot 100/\delta$.

The interest rate is generally set in the order of 3.5%.

By adding the initial construction cost (I) and the capitalised risk (S), one arrives at the total cost of the structure. When this total cost ($I + S$) is plotted as a function of the

design wave height, it appears that there is an optimum design wave height or an optimum strength of the structure.

Similar calculations can be made for monolithic breakwaters. The difference is that the failure behaviour of monolithic breakwaters is more "brittle": the range between start of damage and complete failure is smaller. That means that in general for monolithic breakwaters a lower probability of failure is accepted, merely because of economic considerations.

6. REFERENCES

Bagnold, R.A. (1939) "Interim report on wave impact research" Journal of the Institution of Civil Engineers, Vol 12, London, UK

Goda, Y. (1992) "The design of upright breakwaters" Proc. Short course on design and reliability of coastal structures, Venice, Instituto di Idraulica Università di Bologna, Italy.

Jarlan, G.L.E. (1961) "A perforated wall breakwater" The Dock and Harbour Authority 41 no. 486, London, UK

Miche, R. (1944) "Mouvements ondulatoires de la mer en profondeur constante ou décroissante" Annales des Ponts et Chaussées Vol. 114, Paris France

Minikin, R.R. (1955) "Breaking waves; a comment on the Genoa breakwater" Dock and Harbour Authority, London UK.

Minikin, R.R. (1963) "Winds, waves and maritime structures: Studies in harbor making and the protection of coasts" 2nd rev. ed., Griffin, London, UK.

Kreeke, J. van de and Paape, A. (1964) "On optimum breakwater design" Proc. 9th ICCE, ASCE, New York, USA.

Rundgren L. (1958) "Water wave forces" Bulletin no. 54, Royal Institute of Technology, Division of Hydraulics, Stockholm, Sweden.

Sainflou, M. (1928) "Treatise on vertical breakwaters" Annales des Ponts et Chaussées, Paris, France

Shore Protection Manual (1984) US Army Coastal Engineering Research Center, Vicksburg, Miss. USA

Tanimoto, K. and Takahashi, S. (1994) "Design and construction of caisson breakwaters, the Japanese experience" J. of Coastal Engineering Vol. 22 pp 55-77, Elsevier, Amsterdam, the Netherlands.

ANNEX WITH TABLES AND FIGURES

Wave Height H (m)	Probability of Exceedance (times per annum)
4	1.11
5	$1.58 \cdot 10^{-1}$
5.2	$8.4 \cdot 10^{-2}$
5.5	$7.62 \cdot 10^{-2}$
5.8	$3.8 \cdot 10^{-2}$
6	$2.47 \cdot 10^{-2}$
6.5	$7.35 \cdot 10^{-3}$
7.15	$3.0 \cdot 10^{-3}$
7.25	$2.63 \cdot 10^{-3}$
7.8	$9.0 \cdot 10^{-4}$
7.98	$8.0 \cdot 10^{-4}$
8.7	$1.5 \cdot 10^{-4}$

Table A 1, Long-term wave climate

Actual Wave Height H	Damage in % of armour layer
$H < H_{nd}$	0
$H_{nd} < H < 1.3H_{nd}$	4
$1.3H_{nd} < H < 1.45 H_{nd}$	8
$H > 1.45 H_{nd}$	Collapse

Table A 2, Development of damage

The initial construction cost I of the breakwater is estimated to be:
\$ 8620 for the core and \$ $1320 \cdot H_{nd}$ for the armour layer.

For design wave heights of 4, 5, 5.5 and 6 m this results in initial construction cost as per Table A 3.

Design wave height H_{nd} (m)	Initial cost breakwater "C" (\$ per running meter)	Initial cost Armour Layer "A" (\$ per running meter)
4	13900	5280
5	15220	6600
5.5	15900	7280
6	16540	7920

Table A 3, Initial Construction cost per running meter

H_{nd}	$1 < H < 1.3 H_{nd}$ $n = 4\%$ damage			$1.3 H_{nd} < H < 1.45 H_{nd}$ $n = 8\%$ damage			$H > 1.45 H_{nd}$ Collapse		
	Δp	Δw	$\Delta p \cdot \Delta w$	Δp	Δw	$\Delta p \cdot \Delta w$	Δp	Δw	$\Delta p \cdot \Delta w$
(m)	(1/yea r)	(\$)	(\$/yea r)	(1/yea r)	(\$)	(\$/yea r)	(1/yea r)	(\$)	(\$/yea r)
4	1.02	420	430	$4.6 \cdot 10^{-2}$	860	40	$3.8 \cdot 10^{-2}$	13900	530
5	$1.5 \cdot 10^{-1}$	530	80	$4.7 \cdot 10^{-3}$	1060	5	$2.6 \cdot 10^{-3}$	15220	40
5.5	$7.4 \cdot 10^{-2}$	580	40	$2.2 \cdot 10^{-3}$	1160	-	$8 \cdot 10^{-4}$	15900	10
6	$2.4 \cdot 10^{-2}$	630	15	$7.5 \cdot 10^{-4}$	1260	-	$1.5 \cdot 10^{-4}$	16540	3

Table A- 4, Annual risk for various values of H_{nd} per category of damage level

Note:

Δp = $p_I - p_{I+1}$ probability of occurrence of the wave height in the indicated interval

p_I = probability of exceedance of the wave height at the lower limit of the interval

p_{I+1} = probability of exceedance of the wave height at the upper limit of the interval

Δw = cost of repair of the armour layer ($2 \cdot n \cdot A$) respectively cost of replacement (C)

This leads to the values of average annual risk $s = \Sigma(\Delta p \cdot \Delta w)$ as shown in Table A-5.

H_{nd}	$s = \Sigma(\Delta p \cdot \Delta w)$		
	Full repair of partial damage	Only repair of serious damage(>8%)	No repair of partial damage
(m)	(\$ per year)	(\$ per year)	(\$ per year)
4	1000	570	530
5	125	45	40
5.5	50	10	10
6	18	3	3

Table A- 5, Average annual maintenance cost for various maintenance strategies

For a lifetime of 100 years, which is a reasonable assumption for a breakwater, capitalisation on an interest rate of 3.33% leads to the figures as given in Table A-6.

H_{nd}	Capitalised risk S		
	Full repair of partial damage	Only repair of serious damage(>8%)	No repair of partial damage
(m)	(\\$)	(\\$)	(\\$)
4	30000	17100	15900
5	3750	1350	1200
5.5	1500	300	300
6	540	90	90

Table 6, Capitalised maintenance cost for various maintenance strategies

It is now a simple exercise to add the initial cost I and the capitalised maintenance cost S as in Table A-7.

H_{nd}	Total cost I + S		
	Full repair of partial damage	Only repair of serious damage(>8%)	No repair of partial damage
(m)	(\\$)	(\\$)	(\\$)
4	43900	31000	29800
5	18970	16570	16420
5.5	17400	16200	16200
6	17080	16630	16630
6.5	17300		

Table A- 7, Total cost for various maintenance strategies

The optimum values are printed **bold**.



WAVE ENERGY CAISSON BREAKWATERS

S. Neelamani

Ocean engineering centre

Indian Institute of Technology Madras

Email: sneel@pallava.iitm.ernet.in

ABSTRACT

Rubble mound type is the common breakwater used around the world to provide tranquility condition for the harbours. Arrays of concrete caissons are used as breakwater in countries (ex: Japan, Italy), when it is viable & cost effective. These caissons can be modified to provide a system by which the wave energy acting on this caisson can be converted into usable forms of energy (like electric power). When a part of the incident wave energy on the caisson is converted into electric power, there is ample scope in reducing the wave load on the caisson. There exists many technically challenging problems in the conversion of wave power into electric power. Research and developments worldwide should continue in a fast phase in order to make such non-conventional energy cost competitive. This paper describes some aspects of the wave power research & development in India.

INTRODUCTION

Wave energy is one of the promising forms of renewable source of energy which has received considerable attention. Sponsored by the Department of Ocean Development, Government of India, a pilot plant to generate electricity from Ocean waves has been built off Trivandrum coast by the Ocean Engineering Centre, I I T, Madras. The system consists of (a) Concrete caisson (b) Power module mounted on the top, comprising of a butterfly valve, an air turbine and an induction generator. Fig. 1(a) shows the cross section of the system, 1(b) the plan of caisson, 1(c) the location plan and 1(d) the cross section showing the system and the approach bridge connecting to the breakwater.

PRINCIPLE OF OPERATION

The Oscillating Water Column (OWC) concept, chosen for absorption of energy from waves, consists of a chamber exposed to wave action through an entrance in the front. Under the wave action, air inside the chamber gets compressed and rarified. The

bi – directional air movement is used to drive a special type air turbine known as Wells turbine (Fig. 1(a)). (Also see Fig.1(e) for better description of the principle)

FUNCTIONAL REQUIREMENTS

The OWC chamber dimensions were selected based on extensive laboratory investigations, to make it resonate with the incoming wave. Since the wave parameters vary from time to time and from place to place, it is very important to see that the device absorbs energy equally well over the range of waves predominant at the site. This means that the device should have a very broad frequency bandwidth of absorption. The present device is tuned for an optimum wave period of 10 seconds.

SITE FOR SEA TRIAL

The choice of site was arrived based on the following criteria:

-The system Power availability: From the analysis of wave data collected at several places along the India's coastline, it was found that wave power along Trivandrum coast is promising with an annual average wave power of about 11 kW per meter length of coast.

-Extreme Wave Conditions: The OWC caisson must be designed to withstand the extreme waves likely to occur at the particular site. The maximum wave recorded for Trivandrum coast was 6 m between 1983 and 1987 and the design wave is chosen as 7m. On the other hand, waves upto 9m have been measured on the east coast. It is also known that during the last 100 years no cyclone crossed the west coast near Trivandrum.

-Constructions facilities: The finishing harbour at Vizhinjam near Trivandrum offered the required infrastructural facilities for the construction and installation of the caisson.

-Sea Bed: The sea bed at chosen location consists of dense medium to coarse sand, densely packed, offering a good base for supporting the gravity structure.

WAVE ENERGY CAISSON

The wave energy caisson comprises of a bottom raft 23.2 m x 17.0 m x 3.0 m high, supporting a 12.0 m high chamber with a lip wall in front and guide walls on either sides to facilitate wave entry (Fig. 1a & b). Over the oscillating chamber is a double cubic curve concrete dome 10 m x 7.75 m at bottom, reducing to 2.0 m diameter circle at top and 3.0 m high to support the power module. The caisson top is at +5.00 m with respect to still water level (SWL).

DESIGN WAVE PARAMETERS

Operating condition

The system is expected to deliver a peak power of 150 KW at a significant wave height of 1.52 m and design wave period of 10 seconds.

Design extreme condition

Based on the wave data collected off Trivandrum coast the design wave height was estimated to be 7.0 m with a probable wave period of 10 secs. A maximum of 15° angle of wave attack to the longitudinal side of caisson is considered for analysis of the structure.

ESTIMATION OF WAVE FORCES

The theoretical estimation of wave forces on such non conventional type of structures are in general cumbersome. Unlike circular cylinders, the incident wave direction also has a significant influence on flow pattern around the structure. As the OWC caisson has an opening on one side, the estimation of wave forces becomes uncertain because of the complicated fluid flow and wave oscillations inside it.

Non-breaking wave force

Because of the uncertainty for the estimation of non breaking wave forces, all the known methods have been applied to get the relative magnitudes of the calculated wave forces. The source distribution technique for rectangular caissons, Isaacson (1979), Morison's equation (1950) and the linear diffraction theory proposed by MacCamy & Fuchs (1954) for circular bottom fixed cylinders and Sainflou's method (1928) for vertical wall are used. The structure was idealised as a closed rectangular caisson while using Isaacson method; as an equivalent circular cylinder while using Morison's equation and linear diffraction theory, and as a vertical wall for the Sainflou's method. Even though the reflection is partial, for the purpose of wave force calculation, 100% reflection has been assumed to allow for erring on the safe side. The results obtained using these methods are tabulated below:

Table 1: Non breaking wave force on wave energy caisson

S.No.	Method Used	Design wave force kN
1	Isaacson (1979)	14000
2	Sainflou (1928)	14840
3	Morison's eqn. (1950)	12800
4	MacCamy & Fuchs linear diffraction theory (1954)	12000

The author is of the opinion that none of these idealization are closer to reality. Hence, finally to be safe, the structure has been designed for a non-breaking wave force of 15000 kN.

Breaking wave force

For breaking wave force estimation, the empirical methods proposed by Hiroi (1919), Minikin (1963) and Goda (1974) are used. Hiroi assumed a uniform pressure distribution from sea bed till a height of 1.5 times the incident wave height above SWL. A controversy exists in using Goda's method and Minikin's method. Minikin's method yields more pressure intensity than Goda's method for a higher relative water depth (d/L), whereas, Goda's method yields a higher pressure intensity than Minikin's method for a smaller relative water depth ratio (Yen-his chu, 1989). Goda's method is widely used in Japan and is recommended for the design of semi-rigid type structures. Since the present structure is in the intermediate water condition, a conservative approach given by Minikin is used. The breaking pressure intensity and the total breaking wave forces are given below:

Table 2: Breaking Wave force on caisson

S.No.	Method Used	Pressure intensity at SWL (tons/sq.m)	Total force (kN)
1	Hiroi (1919)	10.8	28700
2	Goda (1974)	6.34	12870
3	Minikin (1963)	118.0	33000

The front lip wall is a critical part of the caisson, since it is expected to be exposed to direct wave attack. The conservative breaking pressure at SWL obtained according to Minikin (1963) was chosen and the lip was designed for this. The total magnitude of breaking wave force is about 33000 kN. Elaborate measurements from the prototype structure and scale model studies in the laboratory are required to estimate the wave forces accurately. The wave pressure due to breaking on the lip wall and on the innerside of the OWC Chamber (back wall) are given in Fig.2a. The idealised wave pressure when the wave is acting with an inclination of 15° to the longitudinal axis of the caisson is given in Fig.2b.

STRUCTURAL ANALYSIS AND DESIGN

The general arrangement of the caisson is shown in Fig. 1(a) and 1(b). This consists of a bottom raft of size 23.2 m x 17.0 m x 3.0 m which supports two walls on either side and one wall in the rear. The outer plan dimension of the chamber is 20.0m x 14.0 m and the height of the walls is 12.3 m. The lip is connected to the side walls. The thickness of the guide walls and lip wall was selected based on hydrodynamic studies for maximum efficiency of wave power absorption. The curvatures at the entry

point on the guide walls and lip wall were provided for smooth entry of waves and to minimise energy losses due to vortex shedding. After considering several possible alternatives, cellular construction was chosen for the walls, lip and bottom raft. The cellular structure was adapted to minimise the weight because of the floating mode of construction and towing and also to minimise the construction cost. The caisson structure was analysed using finite element method for global and local forces. M30 concrete and high yield strength deformed bars of grade 415 were adopted.

Walls and lip

The caisson walls and lip were analysed using thin quadrilateral (flat) shell elements with 30 degrees of freedom. They are assumed to be fixed on the raft and consequently all the degrees of freedom at the bottom are arrested. As the wall is assumed to be thin plate, it is rigid in its own plane and hence all the degrees of freedom (rotational) perpendicular to its own plane are arrested. The wave force distribution on the structure shown in Fig. 2 is converted into equivalent rectangular pressure blocks acting normal to the element. The lip was considered free at top and bottom and connected to the side walls. The maximum bending moments and dimensions of the back walls, side wall and lip are given Table 3.

Table 3: Maximum bending moments and dimensions of the walls

Component	Maximum bending moment		Overall wall thickness (mm)	Thickness of vertical panel (mm)	Thickness of horizontal diaphragm (mm)
	Horizontal direction (kN m / m)	Vertical direction (kN m / m)			
Back wall	+318	+297	2500	200	250
Side wall	-1440	-616	2000	200	250
Lip	+4000	+363	2000	250	400

Raft

The raft is also analysed using thin plate elements and is assumed to be resting on equivalent soil springs. The reaction from the bottom most elements of the wall (due to wave forces) are also taken as part of the load on the raft apart from the submerged weight of walls. The maximum bending moments along the length and width of the bottom raft and its dimensions are shown in Table 4.

The raft was checked for the stresses during the various stages of construction of the caisson floating mode.

Dome

The dome consists of two cubic parabolas meeting at mid height, the height of the dome being 3.0 m. Thickness of the dome is 250 mm. The sectional profile of the dome varies from place to place and it has a quadrantal symmetry. Finite element analysis was carried out using thin plate and shell element

Table 4: Maximum bending moments and the dimensions of the Raft

Max. Bending moment		Overall depths of the box	Thickness of horizontal slab		Thickness of vertical ribs
Along the length kN m/m	along the width kN m/m		Top	Bottom	
		mm	mm	mm	mm
-553	-208	3000	200	250	200

of the SAP IV finite element library. The dome has been designed for the following load cases.

- Internal pressure of 1 bar + self weight of dome+ weight of power module.
- Internal pressure of -0.5 bar + self weight of dome + weight of power module.

The maximum meridinal membrane force and moment in the meridinal direction considered for the design are 500 kN/m and 58 KN-m/m respectively. The percentage of steel is about 1.5% of the cross sectional area of the dome in the meridinal direction.

STABILITY OF CAISSON

The caisson should be stable against overturning and sliding at its final location. Stability is ensured during various stages of construction and towing.

During construction and towing

As the caisson is not symmetrical about its transverse axis, it tilts as the construction progresses in floating mode. To correct this tilt, predetermined quantities of sand were added in the chambers of the bottom box. The metacentric height was always ensured to be greater than 5% of the draught during construction by appropriate ballasting using sand.

On the prepared sea bed

The structure has adequate factor of safety (F.S. > 1.5) both against horizontal sliding and overturning for the design non – breaking wave forces. The author is of the

opinion that the breaking wave force need not be considered for overall structural stability. However, even for this condition the F.S. is greater than 1.

MATERIAL QUANTITIES

Some of the approximate quantities of the materials used for the construction are

Concrete	- 1020 cubic mts.
Reinforcing	- 1450 KN
Structural steel	- 1100 KN
Stones for Sea bed	
Foundation	- 6200 KN
Stones for scour	
Protection around caisson	- 8000 KN

CONSTRUCTION ASPECTS

Caisson

The caisson construction and installation are of major importance, particularly in view of the fact that no slip ways or heavy-lift facilities are available at the site. Keeping in view, the bathymetry, site conditions and availability of the harbour (Fig. 1c) the following methodology was adopted. Fig. 3a-d shows the major sequences of construction.

-The bottom 3 m height concrete box was constructed in a pit 5 m deep, on the beach inside the harbour. The water table was held down below the construction level by well point dewatering system.

-The bottom box was made to float by allowing the water table to rise. Subsequently, the sand bund between the pit and the harbour basin was breached by dredging.

-The box was then towed to deep water area near a jetty inside the harbour, where further construction of walls and other portions was continued in floating mode. Climbing forms were used for the construction of walls to achieve accurate alignment and speed of construction. As the construction proceeds, the draught of this asymmetric structure increases and hence to ensure floating, a temporary steel gate was erected in stages to close the front opening. The gate has overall dimensions of 10 m x 10 m x 1 m and was held in position by a lock channel arrangement. The horizontal level of the structure was kept by ballasting the different chambers of the bottom box with sand/water.

Dome

Wooden joints were cut to lines and levels to form the basic shape of the dome on which plywood shuttering were fixed. The form work for the dome started from the

bottom box slab which is 15 m below. Because of the special shape of the dome, the shuttering work was expensive and labour intensive.

Sea bed preparation

Scour Protection model studies on circular and square cross sections conducted at the Ocean Engineering Centre show that scour is predominant in the front of the structure and the maximum scour occurs at points 45° to the flow direction. Superposition of waves on current results in an increase of scour depth by 20% to 62%. The current velocity was found to be low in magnitude at the location of the caisson. The sea bed foundation was designed and prepared carefully for proper seating of the caisson and for preventing scour around the base of the structure due to wave action (Fig. 4.). An area of about 30 m x 23 m was marked on the sea bed and stones of 20 mm to 40 mm size were neatly packed to lines and levels to form an even horizontal bed. The original plan was to lay geo-fabric material on the sea bed below the stones. But due to non-availability of material to meet project schedule, the geo-fabric layer was left out. Stones were dumped from a pontoon and leveled by divers. The level of the prepared foundation was checked by depth soundings and taking levels using a mast resting on the bed and projecting above the water surface. Underwater photographs provided a fairly good indication of the evenness of the bed.

Towing and seating

The towing and seating operation of the caisson was a very critical one. The scope of the work was to tow the completed caisson from temporary jetty site (see Fig. 1c) to the final location outside the harbour and install it on the prepared sea bed. Inclination test was carried out to assess the metacentric height, GM and controlled sand ballasting was carried out in order to make the GM greater than 5% of the draught of the structure. Subsequently the caisson was towed at high tide (tidal height of 1.25 m) with a draught of 9.9 m and GM of 0.55 m. The caisson was towed out of the harbour to the final location using powerful tugs at the aft, stern and abreast. Finishing trawlers also assisted in the operation. The caisson was brought over the prepared sea bed during ebb tide. Using three transit poles on land, the caisson was aligned and brought in correct position and ballasted continuously to seat it over the sea bed. The exact positioning was done by controlling winches on board the caisson connected to bollards on the shore and anchors on the sea bed.

Subsequently, a steel bridge (45 m long) was erected to span the caisson and breakwater for the transport of the power module and access to caisson top (Fig. 3d).

Power module

The power module mounted on top of the dome consists of an air turbine of 2 m diameter coupled to an induction generator of 150 kW rating. The induction generator system has been selected because it is cheaper and does not require rectification and inversion normally associated with a variable speed alternator. The induction generator

will always be connected to grid, drawing power from mains when the turbine speed is below synchronous speed and pumping power to the grid when the speed of turbine increases above the synchronous speed. When the grid fails, or when the wave heights are higher than the capacity of the turbine, a butterfly valve provided between the turbine and the caisson automatically closes the passage of air flow to the turbine.

The turbine has been designed to match the bi-directional air flow from the OWC, based on detailed model studies carried out by the project group (refer Ravindran et al. (1989).

PERFORMANCE MONITORING

The monitoring of the structural performance of the device to understand the behaviour of the structure and plan for future optimisation has commenced. The caisson is instrumented and provided with a Data Acquisition and control system for this purpose.

Parameters

The structural performance evaluation consists of measuring:

- a) Wave induced pressures on the caisson in various sea states (changing over the various seasons).
- b) Scour around the caisson and tilt, if any, of the caisson at various points of time.
- c) Monitoring the behaviour of the materials used (concrete and steel).

The pressure measurements are being done on the lip and backwalls at different levels by means of transducers, fixed on the caisson. The tilts are being measured by means of an inclinometer. The scour was observed by taking bed levels around the structure by means of a dead weight - chain system.

The power module was mounted on the structure and the system commissioned for trial runs by end October 1991. The performance of the system with regard to the hydrodynamic behaviours, turbine performance and power generation was encouraging. During the period November to February, the wave intensity at location was low, the peak being during May to September. The system will be continuously monitored and evaluated during the next nine months (till end of December 1992).

FUTURE PROSPECTS FOR WAVES ENERGY

Before venturing into commercial level wave power plants, it is warranted to go for a field testing with a pilot wave power plant, mainly to understand the technical feasibility, analyse and solve critical problems and foresee any major problems during

the commercial production of wave power. This had led to the first sea trial with a 150 kW capacity plant. The following are the lessons learned:-

- a. It is positively possible to convert wave power into electric power.
- b. Efficiency of conversion is very low (to the order of 5 to 20%). Hence further research must be done to improve the efficiency.
- c. Since corrosion is the major problem, the mechanical components must be designed using materials which are prone for less corrosion.
- d. The wave power varies with time and hence a flexible power converting system should be designed, etc.

The average annual wave power potential around our country varies from 5 to 20 kW/m (Fig.5). This average is less when compared to countries closer to northern latitude countries. Hence aiming for only wave power conversion may not be cost effective for us. This has led to proposal for Multipurpose wave energy caisson system, which consists of a number of caissons placed one adjacent to others (Fig.6) to form a part of breakwater. The advantages in this system are

- a. The cost of rubble mound breakwater over the stretch of placement of caisson can be saved.
- b. The rear side of the caisson can be used for berthing of vessels.(Fig. 7)
- c. A part of the incident wave power can be converted into electric power, hence the wave loses its vigor and induces less forces on the caisson.

Based on the experience gained in the design, construction, stability analysis, towing and seating of the first prototype caisson, many improvements are proposed for the future caissons for breakwaters (Fig.8).

The following are the major improvements:-

- a. The integral structure itself is modified to provide sufficient Buoyancy chambers, improved stability in floating and sufficient space to fill sand ballast for increased stability against sliding and over turning.(Fig.9)
- b. Sufficient width is provided at the rear side of the caisson for the mobility of power vehicles.
- c. The power module is kept in an enclosed room to reduce the noise level in the atmosphere.(Fig.10)
- d. Horizontal axis turbines of smaller capacities and two number for each caisson is proposed for effective utilisation and conversion of wave power.

This type of Multiple oscillating water column (MOWC) systems can be used in India, whenever a new harbour is proposed to be built as shown in Figure 11, which was proposed for Thangassery Fishery harbour in Kerala during 1995.

FORCE REDUCTION TECHNIQUE ON THE ARRAY OF WAVE ENERGY CAISSON

It is always better if the wave forces on the caisson is reduced by some means. Wave energy caisson offers a chance for implementing this proposal. If the air pressure built into the caisson is released to the atmosphere effectively during severe wave climate, then the wave force can be reduced significantly. The wave force on the wave energy caisson array and the wave force on a vertical wall type caisson breakwaters was studied using physical models. Force reduction to the extend of atleast 20 to 30% was possible if an air pressure relief system is provided with the caisson. It should be designed such that if the air pressure inside the OWC Chamber exceeds a prespecified value (Threshold pressure), it should open up and release the air pressure automatically (Fig.12). This is similar to the pressure relief valve used in the pressure cookers for cooking purposes in our homes.

SUMMARY

The design, construction and installation aspects of the concrete caisson of overall size 23.2 m x 17.0 m x 18.3 m for the wave power plant installed off the South-West coast of India have been presented. The caisson structure consists of a bottom raft, two side walls, a back wall, a lip and a dome. It was built , partly on land and partly in floating mode in a harbour basin and installed in open sea on a prepared sea bed. The estimation of wave forces on the structure was quite uncertain owing to the complex fluid flow and oscillations inside it. Several known methods for the estimation of wave forces have been attempted. However being a pilot plant, conservative approaches have been chosen. The structural analysis was carried out using Finite Element Method. Because of the asymmetric shape of the structure, the construction and towing of the caisson was quite difficult and involved meticulous planning. As of now the caisson has withstood wave actions for the past one decade. Field observations show that the device is performing according to predictions. An extensive performance monitoring was being carried out by NIOT to understand the behaviour of the structure and the power device.

We should go for installation of multipurpose wave power plants of the order of 1 to 2 MW capacity as a part of the breakwater, if new artificial harbours are proposed. Such scheme will be cost competitive.

It is possible to reduce the wave loads on the oscillating water column type wave energy caissons, by providing proper air pressure relief systems.

ACKNOWLEDGEMENT

The materials presented here is a part of the work done under a Project sponsored by the Department of Ocean Development, Govt. of India and implemented by Indian Institute of Technology, Madras. The author was a part of the project team. Many

colleagues, project officers within IIT and many organisations from outside assisted in the project implementation. The Harbour Engineering Department of the Kerala Government was responsible for the local infrastructure and construction. The caisson was built by L & T – ECC Construction Group. The assistance of all the above is gratefully acknowledged.

REFERENCES

Final report on scientific investigations on wave climate, wave regulation and wave power (1987). Report submitted to Department of Ocean Development, Government of India by the Wave Energy Group, Ocean Engineering Centre, Indian Institute of Technology, Madras, February.

Goda, Y. (1974). "New Wave pressure formula for composite breakwaters", Proceedings of the 14th conference on Coastal Engineering.

Hiroi, I (1919). "On a method of estimating the force on waves", Bulletin of Engg. Dept., Tokyo Imperial University, Vol.10, No.1, pp.19

Isaacson, M. de St. Q. (1979). "Wave forces on rectangular caissons", Proc. Of the Civil Engg. in the Oceans, IV, ASCE, San Fransisco, Vol I.

MacCamy, R.C and Fuchs, R.A. (1954). "Wave Forces on piles: A Diffraction theory" U.S. Army Beach Erosion Board, Technical Memorandum No.69, 17 pages

Minikin, R.R. (1963). "Wind, Waves and maritime structures", Studies in the harbour making and protection of coasts, 2nd edition, Griffin, London.

Morison, J.R., O'Brien, M.P., Johnson, J.W. and Schadf, S.A (1950). "The forces Exerted by surface waves on piles" Petroleum transactions, AMIE, Vol.189, pp.149-154

Ravindran, M., Swaminathan, G., and Koola, P.M. (1989). "Model Studies for the Sea Trial of a 150 KW Wave Energy System" Proc. Eighth OMAE (ASME) Conf., The Hague, Mar. pp. 19-23.

Sainflou, G. (1928) "Essai Sur les diques maritimes verticales" Annales des ponts et Chausses, Vol.98, No.1, pp. 5-48

Sea Trial of a 150 KW wave energy device off Trivandrum coast. Project report submitted to DOD by the Wave Energy Group, Ocean Engineering Centre, Indian Institute of Technology, Madras, 1987.

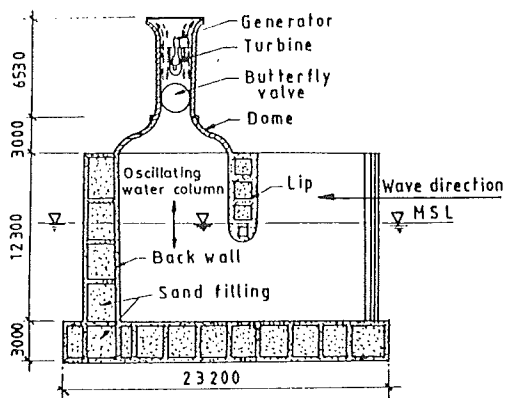


FIG. 1a. CROSS SECTION OF WAVE POWER PLANT

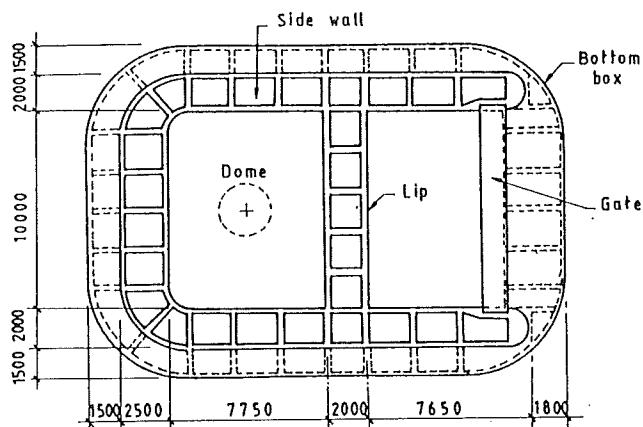


FIG. 1b. CROSS SECTIONAL PLAN OF CAISSON

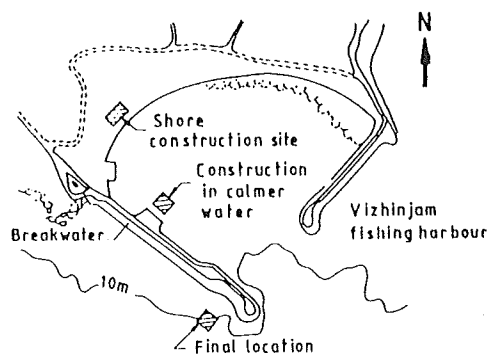


FIG. 1c. LOCATION PLAN

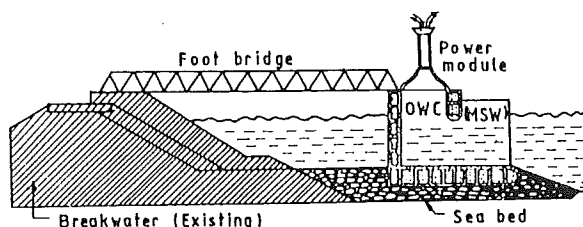


FIG. 1d. CROSS SECTION OF WAVE ENERGY DEVICE AND BREAKWATER (SCHEMATIC)

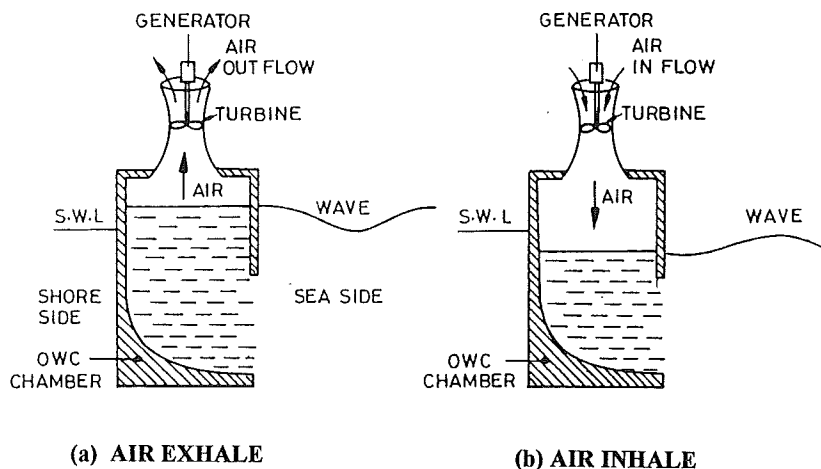


FIG. 1e. PRINCIPLE OF THE OSCILLATING WATER COLUMN WAVE ENERGY DEVICE

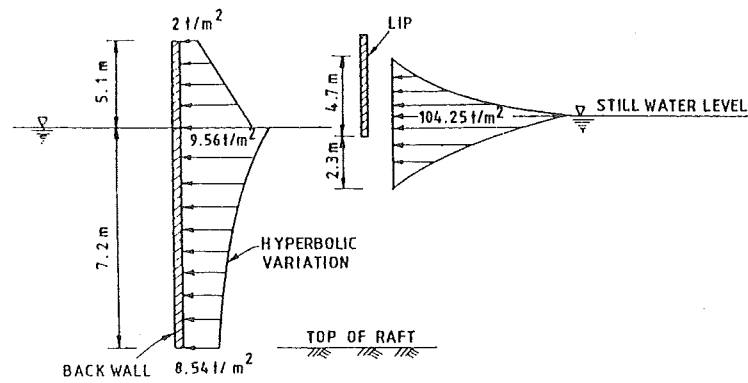
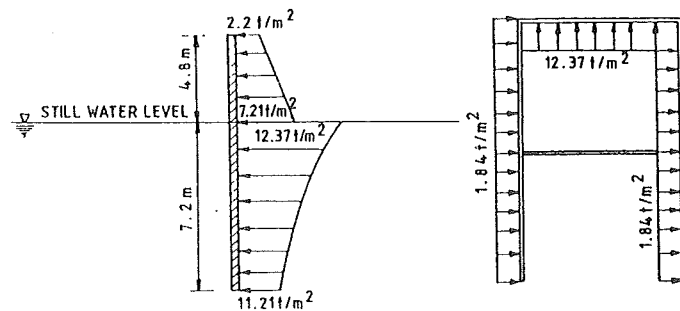


FIG.2a. BREAKING FORCE IN ELEVATION



(i) Pressure distributions along the height of the back wall

(ii) Plan of pressure distributions

FIG.2b. 15° INCLINED NONBREAKING WAVE FORCE

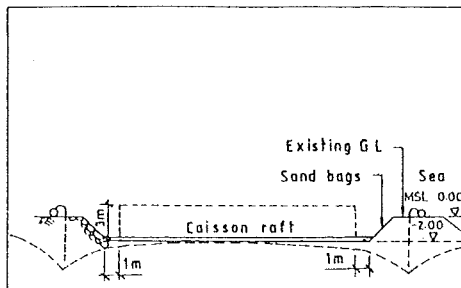


FIG.3a. CONSTRUCTION OF BOTTOM BOX AND BUND

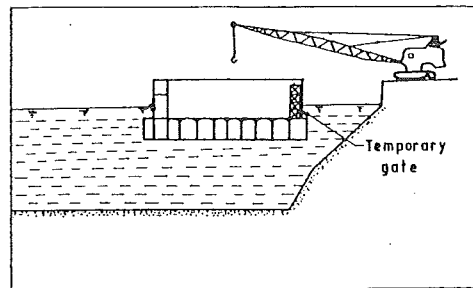


FIG.3b. CONSTRUCTION IN FLOATING MODE

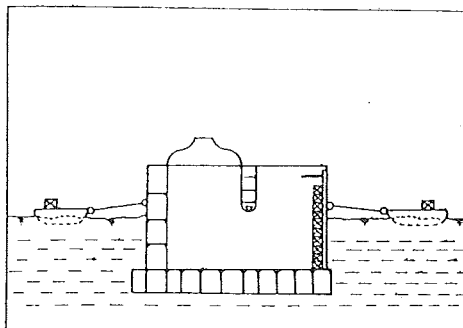


FIG.3c. TOWING

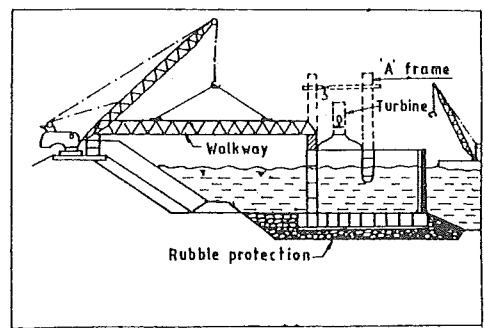


FIG.3d. ERECTION OF BRIDGE

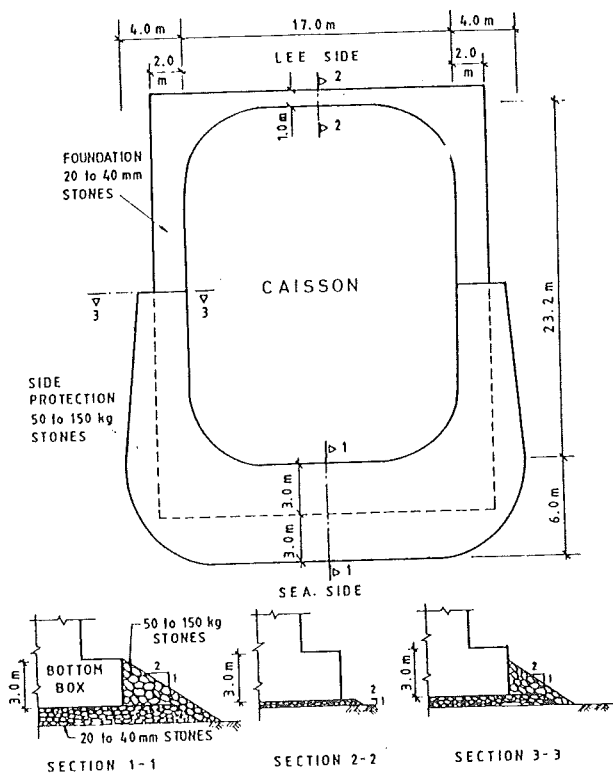


FIG.4. BED PREPARATION FOR WAVE ENERGY CAISSON

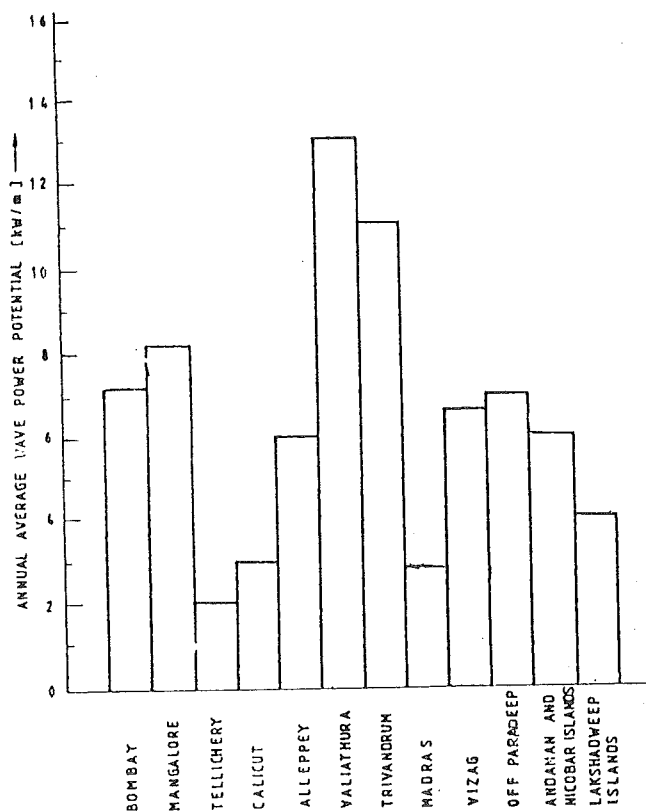
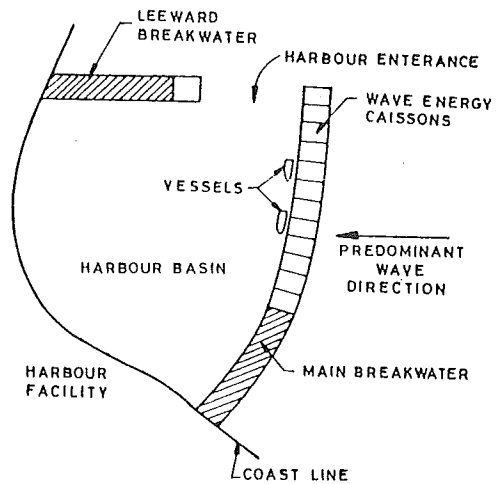
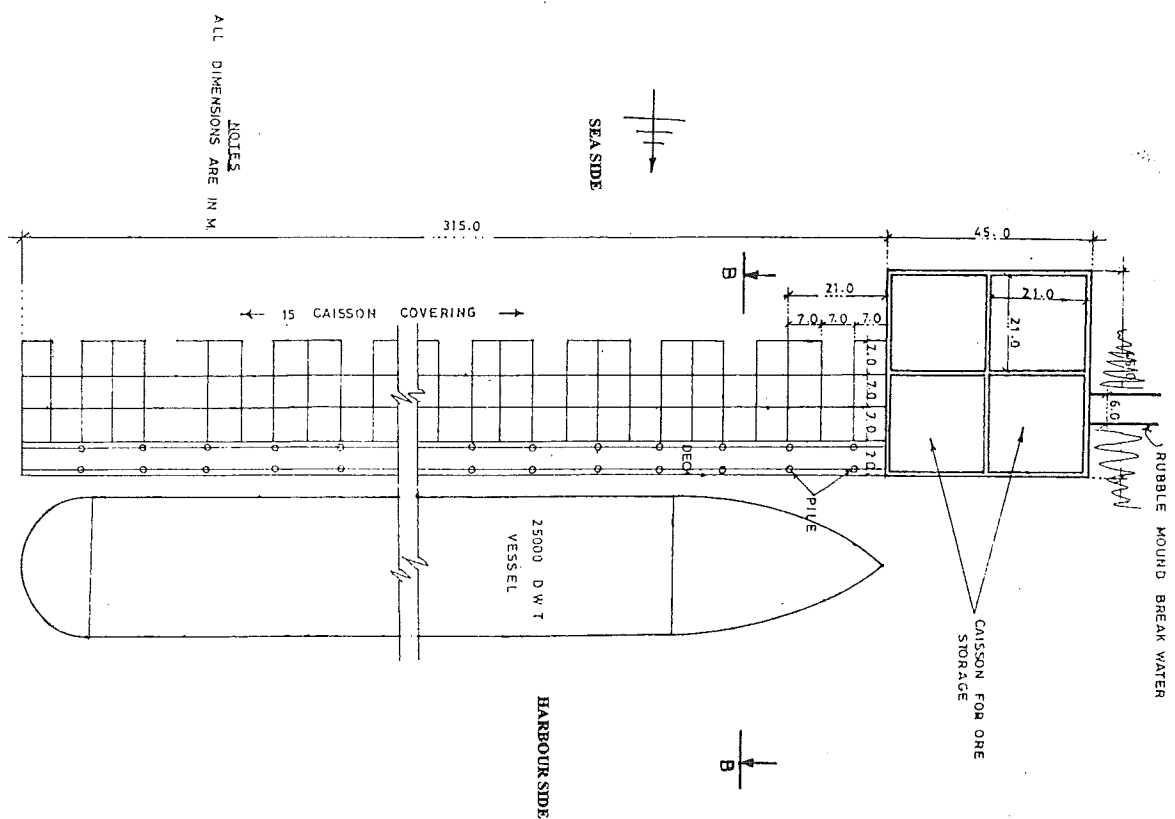


FIG.5. WAVE POWER POTENTIAL ALONG INDIAN COAST



**FIG.6. CONCEPT OF MULTIPURPOSE BARRIER
TYPE WAVE ENERGY SYSTEM**



**FIG.7. MOORING & BERTHING OF VESSELS AT THE
REAR OF WAVE ENERGY CAISSON ARRAYS**

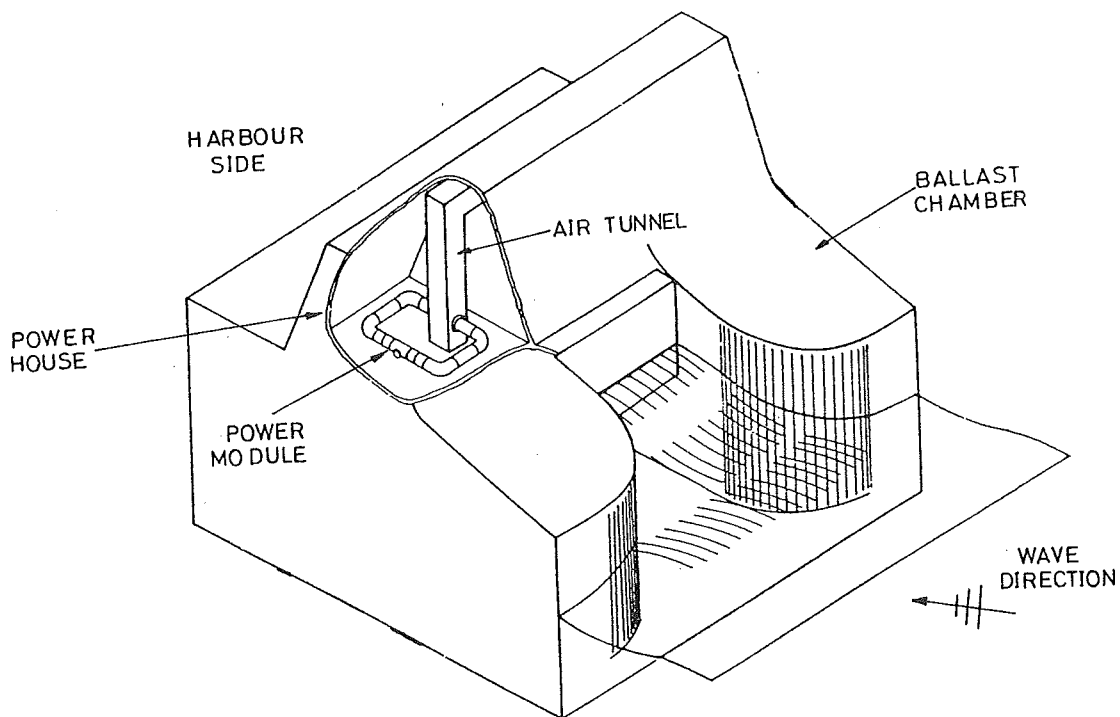


FIG.8. IMPROVED WAVE ENERGY CAISSON MODULE TO BE PLACED IN AN ARRAY TO FORM A PART OF BREAKWATER

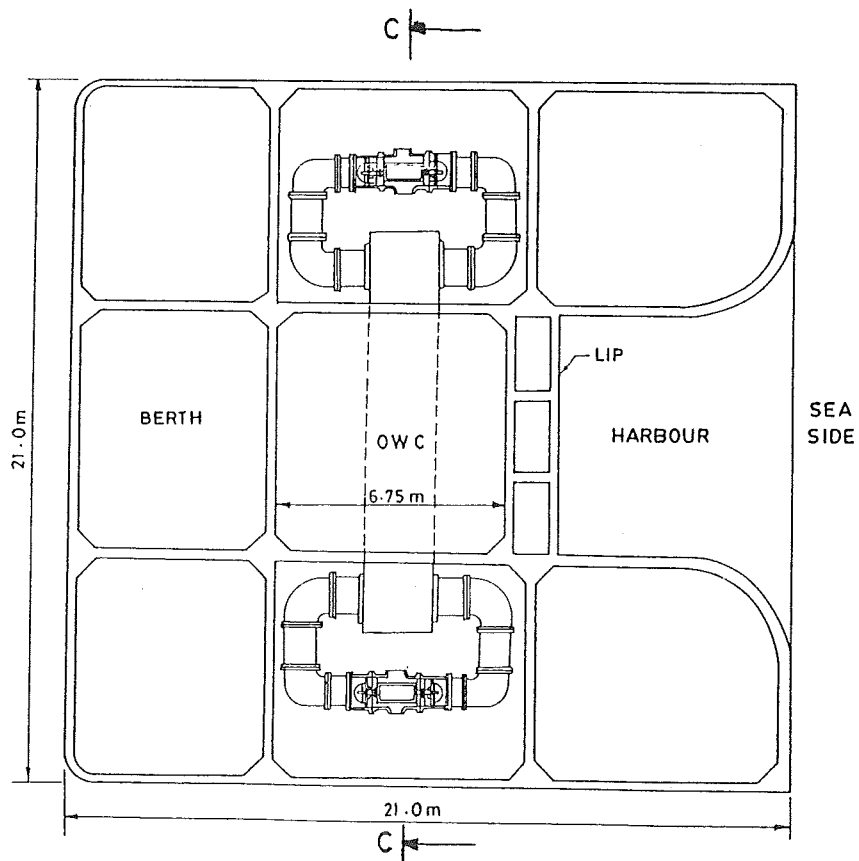


FIG.9. ARRANGEMENT OF POWER MODULE ON CAISSON

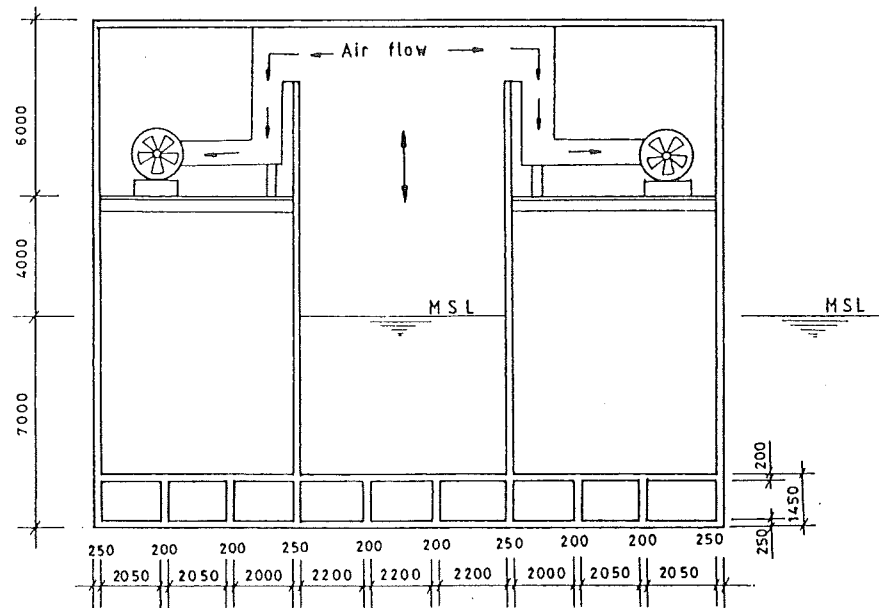


FIG.10 SECTION C - C OF FIG.9

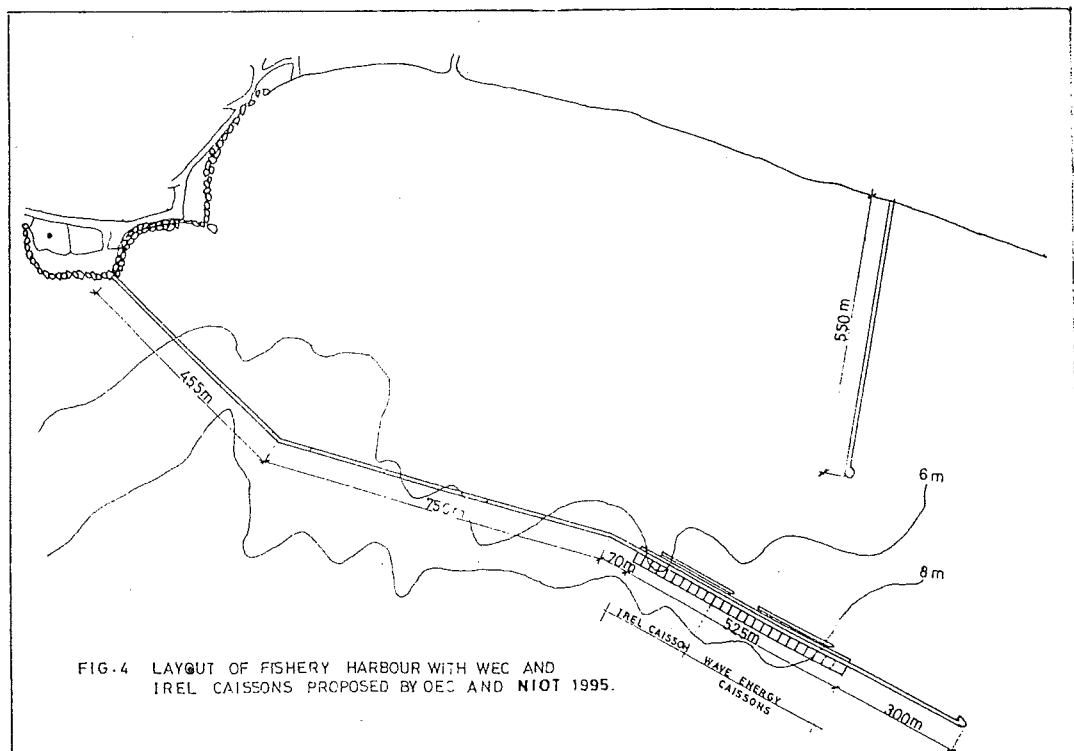
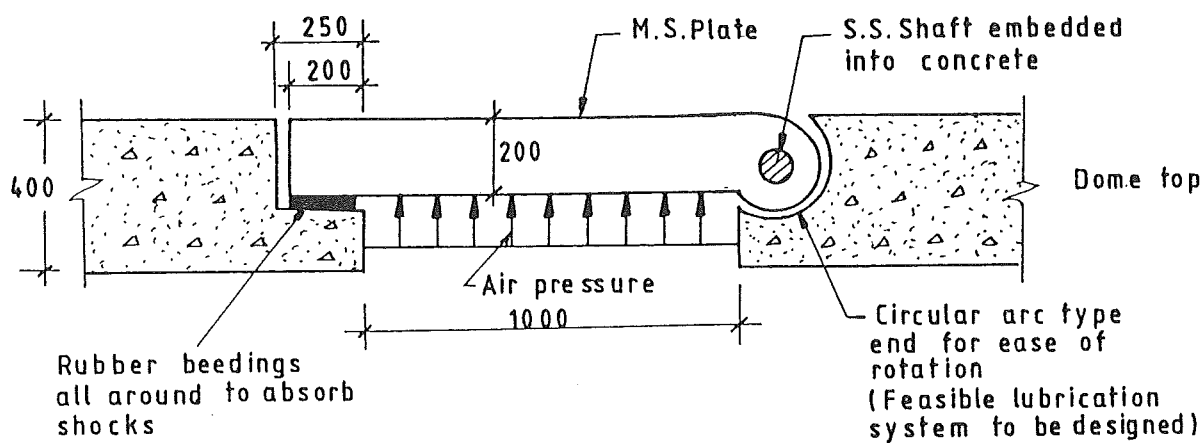


FIG.11. LAYOUT OF FISHERY HARBOUR WITH WEC AND IREL CAISSONS PROPOSED BY OEC AND NIOT, 1995



ALL DIMENSIONS ARE IN mm

OWC AIR CHAMBER

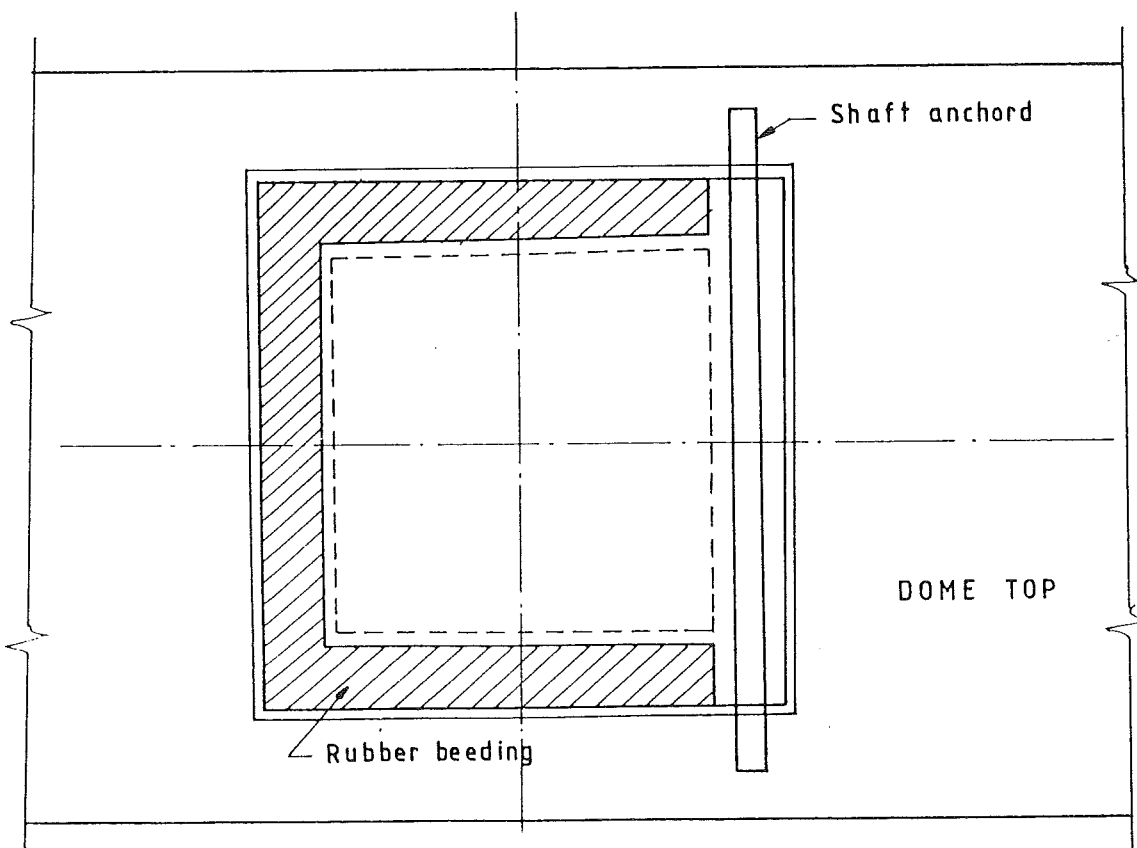


FIG.12. AIR PRESSURE RELEASE SYSTEM FOR OWC TYPE WAVE ENERGY CAISSON



PARTIALLY SUSPENDED POROUS WALL BREAKWATER

J.S. Mani

Ocean Engineering Centre

Indian Institute of Technology, Madras

Email: manijs@hotmail.com

1. INTRODUCTION

Numerous facilities along the coast worldwide such as recreational harbors, marinas, bays and fishing harbors are threatened by massive waves, resulting in the loss of human life and permanent structures. For recreational harbors, coastal swimmers and surfers prefer to have acceptable wave conditions to suit their sporting activities and for fishing harbors, creation of still water conditions is not a necessity. In such cases, expensive rubble-mound breakwaters may not be the right choice, as they are meant for providing very calm waters.

To control the wave disturbance in these partially enclosed water bodies, breakwaters like (i) floating (ii) tethered and (iii) submerged types are considered. These breakwaters can be adopted if the water depths are relatively shallow and the breakwaters can withstand the severity of incident waves. However, for relatively large water depths, pile breakwaters are suitable. In the recent past, steel-pile breakwaters are recommended especially for fishing and recreational harbors, wherein moderate wave disturbances are admissible in the harbour basins.

As the cost of a closely spaced pile breakwater is directly related to the number of piles needed for breakwater construction, an attempt is made to develop a partially suspended porous wall breakwater that is cost-effective, easy to install, and capable of reducing the incident-wave height by 50%. Results of the experimental tests conducted for the partially suspended porous wall breakwater are discussed in this paper.

2. EXPERIMENTAL SETUP

Experiments were conducted in a 30-m-long, 2-m-wide and 1.5-m-deep wave flume. The wave machine installed in the flume is capable of generating monochromatic incident waves of height H_i varying between 6 and 24 cm with period ranging between 0.80 and 2s. The details of partially suspended porous wall breakwater are shown in figure 1. The support pipes, 16 cm in diameter, (representing a pile in nature) were positioned close to the flume walls and a frame containing a row of 4-cm-diameter pipes spaced with a b/d ratio (b = clear gap; d = pipe diameter) was suspended in between the pipes. The frame was made to slide through vertical slot provided in support pipes, so as to facilitate testing the breakwater with different y/h ratios (y = draft of the pipe ; h = water depth). Experiments were conducted for b/d ranging from 0.11 to 1.0 and y/h ranging from 0.26 to 0.56. Resistance-type wave gauges were installed, one each on the wave generator side and on the absorber side of the breakwater. The transmitted wave height was obtained on the absorber side by a wave gauge positioned at a distance of 5m from the breakwater. Transmission coefficient K_t , defined as the ratio of transmitted-wave height (H_t) to incident-wave height (H_i) was determined for studying the performance characteristics of the breakwater.

3. EXPERIMENTAL RESULTS

3.1 Determination of Optimum Value of b/d and y/h

Figure 2 shows the variation of transmission coefficient with b/d and y/h ratios for four typical values of H_i/gT^2 , that is to say, 0.0016, 0.0034, 0.0061 and 0.016. The values of transmission coefficient K_t shown in the figure correspond to an average value obtained after several test repetitions. From the figure it is possible to infer the following :

1. Comparison of K_t values for $b/d = 1.00$ and 0.44 indicates that by restricting the b/d value to 0.44, an appreciable reduction (of the order of 26%) in transmission coefficient can be obtained (especially for steep waves). However, if the b/d ratio is restricted to 0.22 and 0.11, the reduction in K_t would be of the order of 33% and 35%, respectively.
2. For the given range of b/d ratio, increase in y/h from 0.46 to 0.56 has less influence on transmission coefficient compared to the K_t values for 0.36 to 0.46.

These results imply that further reduction in the b/d ratio and increase in the y/h ratio may not significantly influence the transmission coefficients. Thus it is concluded that the optimum values for b/d and y/h should be equal to 0.22 and 0.46 respectively.

3.2 Variation of Transmission Coefficient

Figure 3 shows the variation of transmission coefficients with H_i/gT^2 and b/d for $y/h = 0.46$. The trend curves suggest that for the given range of b/d , the transmission coefficient decreases for an increase in H_i/gT^2 . From the results it is evident that

1. By maintaining a b/d ratio of 0.22, the breakwater is capable of reducing the magnitude of incident-wave height by 50% for a wide range of wave steepness ($0.05 < H_i/L < 0.106$).
2. For $0.22 < b/d < 0.66$, 40% reduction in incident-wave height is predicted; this is also true for a narrow range of wave steepness.
3. For $0.66 < b/d < 1.0$, incident-wave height is reduced by 20% only.

3.3 Analysis

The results imply that the following benefits can be derived by adopting the present type of breakwaters rather than a pile breakwater.

1. As the number of piles needed for construction is drastically reduced, the system assures a net savings in the cost of material, fabrication, and erection. Cost comparison for a 20-m length of suspended pipe breakwater with a similar length of pile breakwater is given here. (Note that cost estimate was made based on the rates quoted by an Indian Company engaged in pile driving operations in open sea.)

* Pipe breakwater :

Cost of support piles (1.6-m dia)

= No. of piles x cost per pile

= $2 \times \$12,000 = \$24,000$

Cost of suspended pipes made of high-density polyethylene pipes (40-cm dia)

= No. of pipes x cost per pipe

= $34 \times \$1,300 = \$44,200$

Total cost of suspended pipe breakwater

= \$68,200

* Pile breakwater

Cost of pile breakwater (1.6-m dia)

= No. of piles x cost per pile

= $10 \times \$12,000 = \$120,000$

* Comparison

Reduction in the total cost by using the partially suspended porous wall breakwater

= 43%

2. Time required for installation is effectively reduced.
3. Easy replacement of pipes in case of damage or loss to a certain stretch of breakwaters (which is not possible in case of pile breakwater).

4. As the partially suspended porous wall breakwater blocks only 48% of the vertical sheet of water, versus 84% by the conventional pile breakwater, the breakwater would help in maintaining adequate flow exchange between the partially enclosed water body and the open sea.
5. As the vertical area blocked by the pipe breakwater is only 48%, the structure would experience slightly less total hydrodynamic force compared to a conventional pile breakwater.

3.4 Further development on Porous breakwater

Studies conducted further on partially submerged porous wall breakwater filled with suitable porous material (instead of pipes) yield promising results. This type of breakwater has been installed to protect a jetty on the west coast of India in the year 1997 and the structure has witnessed two monsoons seasons including a severe cyclone. The performance of the breakwater is shown in figure 4.

4. CONCLUSIONS

- (1) Partially suspended porous wall breakwater (with a row of pipes) is an economical and promising substitute for a pile breakwater and is as efficient as a pile breakwater in attenuating incident waves,
- (2) A gap to diameter ratio b/d of 0.22 and draft to water depth ratio y/h of 0.46 are recommended for suspended pipes to achieve a transmission coefficient of 0.5.
- (3) For $H_i/gT^2 > 0.008$, the present breakwater can attenuate incident waves by 50%. However, for $0.005 < H_i/gT^2 < 0.008$, incident waves are reduced by 40%.
- (4) Partially suspended porous wall breakwater (with suitable light weight porous material) with 30% submergence, can restrict wave transmission between 10 and 67% for $H_i/gT^2 > 0.003$. In practical range of H_i/gT^2 ($0.006 < H_i/gT^2 < 0.020$), maximum value of K_t is 0.45.
- (5) Wave force estimated based on average pressure distribution over partially suspended porous wall breakwater (with suitable light weight porous material) indicates force reduction of the order of 50% compared to an equivalent impermeable breakwater.

5. REFERENCES

- Grune, J., and Kohlase, S. (1974). "Wave transmission through vertical slotted walls", 14th Coast. Engrg. Conf., ASCE, New York, N.Y., Vol.III, 1906 - 1923.
- Hayashi, T., Hattori, M., and Shirai, M. (1968). "Closely spaced pile breakwater as a protection structure against beach erosion, "Coast. Engrg, in Japan, 11, 149 - 156.

Hayashi, T., and Kano, T. (1966). "Hydraulic research on the closely spaced pile breakwater, "10th Coast. Engrg. Conf., ASCE, New York, N.Y., 873-884.

Khader, M.H.A., and Rai, S.P. (1981). "Wave attenuation due to closely spaced circular cylinders, "Int. Assoc. for Hydr. Res., XIX Congrg. New Delhi, India, 95-102.

Mani, J.S. (1989), "Wave damping characteristics of pile breakwaters." Proc. Third Nat. Conf. on Dock and Harb. Engrg., Dept. of Appl. Mech. KREC, India, 181-188.

Wiegel, R.L. (1960). "Transmission of waves past a rigid vertical thin barrier." J.Wtrwy. and Harb. Div., ASCE, 86(1), 1-12.

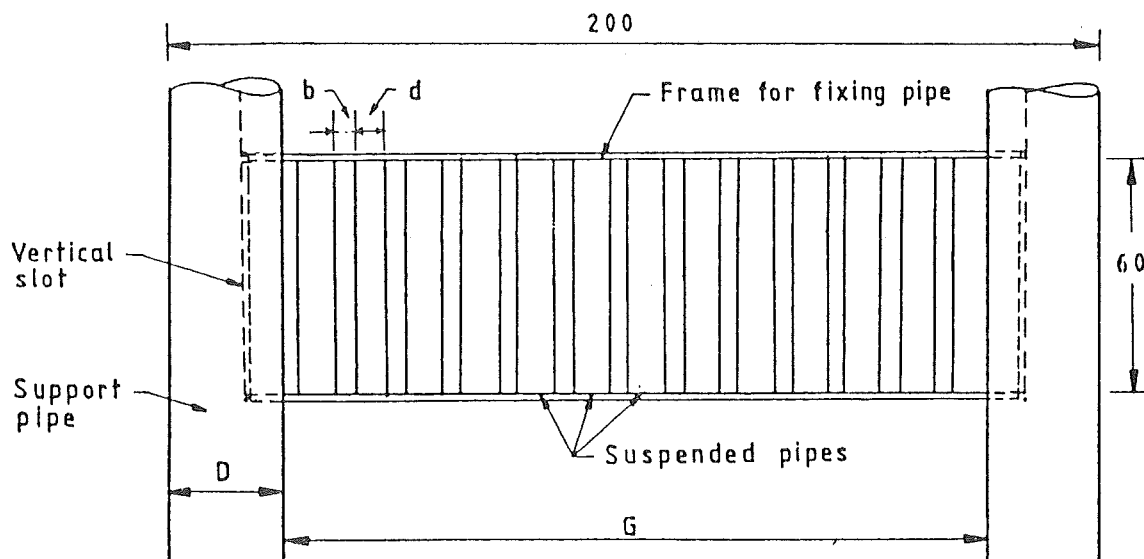


Figure 1 Partially Suspended Porous Wall Breakwater

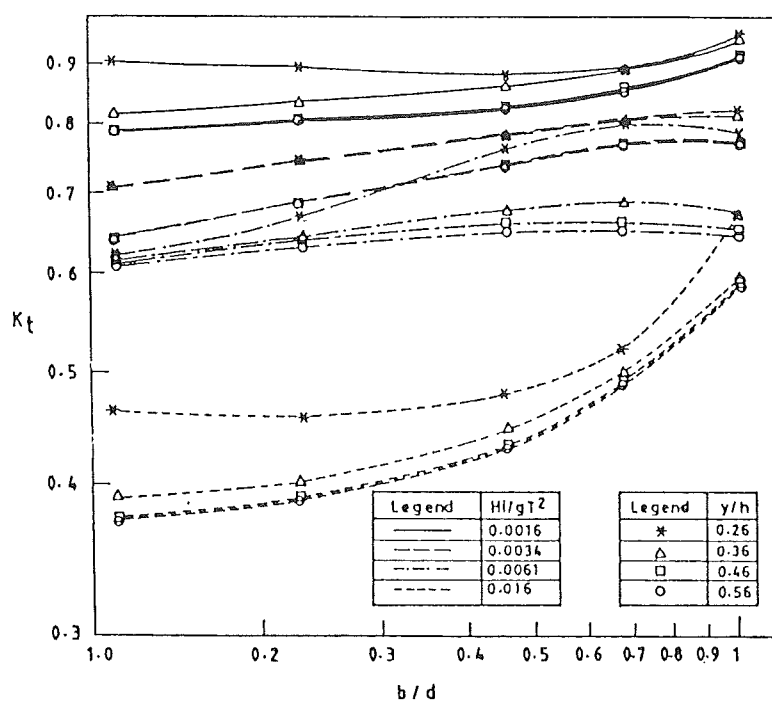


Figure 2 Variation of Transmission Coefficient K_t with b/d , y/h , and H_i/gT^2 Ratio

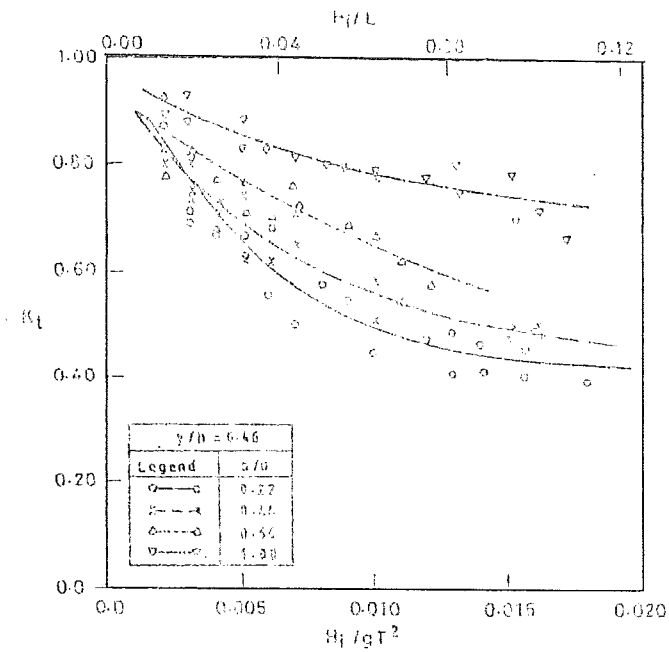


Figure 3 Variation of Transmission Coefficient with Wave Steepness Parameter

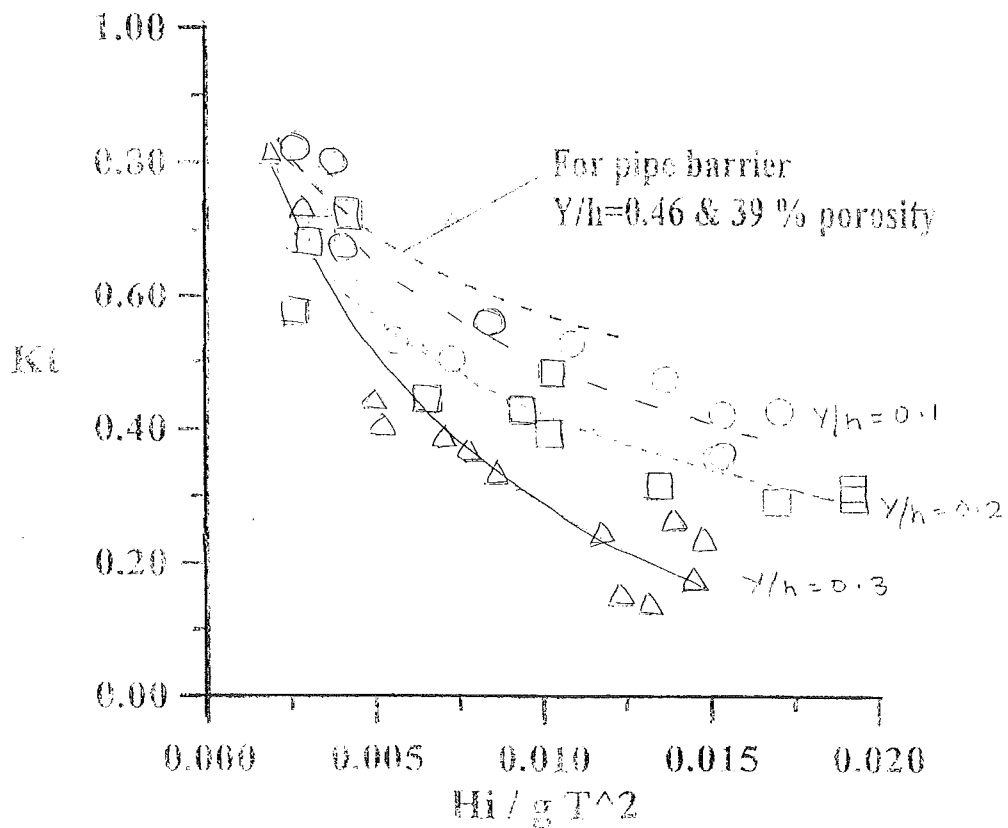


Figure 4 Variation of Transmission Coefficient for Partially Suspended Porous Wall Breakwater (with suitable light weight porous material)



CASE STUDIES ON STABILITY OF BREAKWATERS

V.Sundar, R. Sundaravadivelu, and M.R.Pranesh
Ocean Engineering Centre
Indian Institute of Technology, Madras
Email: vallamsundar@hotmail.com
Email: dr_velu@hotmail.com
Email: mrpranesh@hotmail.com

1.0 INTRODUCTION

The design of cross section of a rubble mound breakwater is quite well known.

$$W = \frac{\gamma H^3}{K_D (S_r - 1)^3 \cot \theta}$$

The size of the individual armour block is arrived using the Hudson's formula.

Where W is Weight of individual armour block in ton, γ is mass density of armour block in ton/m^3 , H is design wave height in m, K_D is Stability coefficient, $S_r = \gamma/\gamma_w$, γ_w is mass density of sea water in ton/m^3 , and θ is slope of breakwater.

Though, the design of a breakwater cross section is straight forward, it is essential to carry out physical model tests to verify the stability of the primary layer as well as to arrive at the crest elevation considering the overtopping of waves. A number of studies for several harbours in India have been carried out in Ocean Engineering Centre (OEC), Indian Institute of Technology Madras (IIT Madras) and details of few of the studies are highlighted. In addition, studies related to recent concepts of breakwaters being carried out in OEC herein have also been considered and salient results of such studies are also reported.

2.0 EXPERIMENTAL FACILITIES

Most of the tests are carried out in a 72.5m long, 2m wide and 2.7m deep wave flume. A wave maker is installed at one end of the flume and other end of flume is provided with an absorber which is a combination of a parabolic perforated sheet and a

rubble mound below it for deep water waves and shallow water waves respectively. In this flume, water depth can be varied from 0.5m to 2.0m. The details of the wave flume is shown in Fig.1. Within the mechanical, geometric and hydrodynamic limitations of the system the wave generating system is capable of generating regular waves or random waves of any pre-defined spectral characteristics. The wave maker can operate in two different modes: (a) in piston mode for generation of shallow water waves, or (b) in hinged mode for generation of deep water waves.

The maximum water depth in the flume must never be exceeded less than or equal to 1.0m and less than or equal to 2.0m for the piston mode and hinged mode operations respectively. Wave generator generates waves through a servo actuator with remote control system. One personal computer to servo actuator is used to give input wave signal to the wave generator and the same computer is used for data acquisition.

3.0 TESTING PROCEDURE

The testing procedure was followed as discussed by Owen and Allsop(1983). The damage to the breakwater will be in terms of the number of armour units, which have been totally dislodged from the armour layer. This number may be expressed as a percentage of the total number of units on the armour face.

The movement of the armour units can assess the damage and four categories of armour unit movements are given below:

P - Unit seen to be rocking, but not permanently displaced

Q - Unit displaced by upto 0.5 D

R - Unit displaced between 0.5 and 1.0 D

S - Unit displaced by more than 1.0 D

Where D is the equivalent diameter of the armour unit.

The damage level is ascertained by adding the percentage of damage in Q, R and S categories. The performance of the armour layer is continuously monitored by an underwater video camera.

4.0 MODELLING OF BREAKWATER AND WAVE CLIMATE

The modelling of present wave interaction problem consists of modelling of wave climate and breakwater cross section. For the reproduction of the ocean waves, wave heights are modelled according to the basic model scale, λ and wave periods as $\lambda^{1/2}$. The weights of individual units are modelled as λ^3 .

5.0 STABILITY FOR BREAKWATER SECTION

5.1 Breakwater section for proposed port at Ganeshgule, India

The stability tests on breakwater (both trunk and round head portion) proposed to be constructed along the coast of Maharashtra at Ganeshgule was carried out. The layout of the harbour is shown in Fig.2. As per the stability analysis, the core layer of both southern and northern breakwaters consists of stones varying from 0 to 80 kg. The secondary layer consists of 1.0 to 2.0 ton stones having a thickness of 1.5m on the sea side, the primary armour layer consist of 12ton tetrapods of thickness 2.5m, whereas, the armour layer of the round head section is formed by placing 18ton tetrapod of thickness 4.0m. The cross section was designed for a design wave height of 4.5m and wave period of 10sec. The cross-sections of the trunk and head portions to be tested for its stability are shown in Fig.3 and Fig.4 respectively.

A model scale of 1:17 and 1:20 was adopted for the trunk and round head section respectively. According to the model scale, the core layer of the trunk section consists of stones varying from 0 to 120gm. The secondary layer consists of 0.192 to 0.383kg stones having a thickness of 90mm on the seaside. The crown plate (L shaped) has been fabricated with mild steel channel sections and plates. The primary armour layer consists of 2.3kg tetrapods of thickness 144mm, whereas, the armour layer of the round head section is proposed by 2.3 Tetrapod of thickness 200mm.

The tests were carried out for three water depths i.e., Highest High Water Level (HHWL, Prototype: +3.05m and model: 0.90m), Mean Higher High Water (MHHW, Prototype: +2.30m and model: 0.85m) and Mean Sea Level (MSL, Prototype: +1.50m and model: 0.81m). The experiments were carried out by increasing the wave heights from 50% to 120% of design wave height in steps of 25%. For each wave height, the test was carried out for 500 waves.

Based on the Hydrodynamic tests on trunk section of breakwater, it was observed that the maximum damage of 2.05% occurred for the water depth of 0.90m(HHWL) when the trunk section is subjected to 120% of the design wave height (i.e., 0.32m). The wave breaking conditions was observed for the design wave height of 0.32m (120% of the design wave height) in a water depth of 0.85m(MHWS) and 0.90m(HHWL). The observation made in the flume is shown in Fig.5. Significant overtopping was observed for a 120% of design wave height for a water depth of 0.90m(HHWL).

In the case of head section, it was observed that the maximum damage to the round head section is of about 1% when it is subjected to design wave height of 0.30m in a water depth of 0.86m(HHWL). The observation made in the flume is shown in Fig.6. Based on the hydrodynamic tests, it was concluded that both the trunk and head section for the proposed breakwater at Ganeshgule are safe, since, the damage to the trunk and head sections are well with in the permissible damage level of 5%. Similar model studies were carried out for other proposed breakwaters along the coast of Maharashtra at Redi, Jaigad and Vijaydurg.

5.2 Stability of horizontal composite breakwater at Mormugao port, India

Mormugao Port situated on the west coast of India in the state of Goa has been serving as a major port for the last 100 years. The layout of the Mormugao port is shown in Fig.7. The harbour works consists of breakwater of length of about 550m. The caisson wall is constructed with 8ton laterite concrete blocks sitting on the rubble mound at low water level. On the sea side of the caisson wall, two types of wave breakers that is, laterite blocks of 12ton and 18ton are placed over on the rubble mound in order to protect the caisson against the severe attack of waves. The bed level is at an elevation of -9.0m to the chart datum. The top of the breakwater is at $+4.5\text{m}$. The wave height and period for the design of breakwater adopted are 5.7m and 10 sec respectively. The elevation of Highest High Water Level (HHWL), Mean Sea Level (MSL) and Low Water Level (LWL) are $+2.3\text{m}$, $+1.3\text{m}$ and $+0.0\text{m}$ respectively. The cross section of the existing breakwater is shown in Fig.8(a). The port authorities reported that, a few wave breaker blocks overtopped and thrown onto the leeside and few of them rolled towards the seaside during the monsoon period. This necessitated examining the means of strengthening the armour layer.

The breakwater is considered as a caisson and has been fabricated with mild steel plates and angle sections, which is equivalent to the weight of the caisson for 2m length. A model scale of $1:16$ was adopted for the physical model studies. The laterite blocks of 8ton adopted for the rubble mound below the caisson and wave breaker blocks of 12ton and 18ton have been fabricated according to the model scale corresponding to 2.04kg , 3.07kg and 4.60kg blocks respectively. The damage to the existing section after being subjected to the waves is shown in Fig.8(b). Since the breakwater could not withstand the wave attack and a damage of about 20% was observed for the existing section, it was necessary to revise the design.

Three modifications were made to the existing horizontal composite breakwater and they were studied. They are

1. Flattening the seaward slope to $1:2$ providing a berm of 3.0m at HHWL ($+2.3\text{m}$) with armour layer formed by 9ton tetrapods of thickness 3.0m . The details of the cross section of the composite breakwater strengthen with tetrapods is shown in Fig.9. A model scale of $1:16$ was adopted for the model study.
2. Since it was felt that the Mormugao port has gained considerable expertise in handling laterite cubes, this modification was suggested. This encompasses usage of 15ton concrete cube blocks. A berm width of about 4m is provided at HHWL ($+2.3\text{m}$) with a slope of $1:2$. The concrete cubes of weight 15ton are used for the primary layer of thickness 3.6m . The details of the cross section of the composite breakwater strengthened with concrete cubes is shown in Fig.10. A model scale of $1:16$ was adopted for the model study.
3. While planning for these tests, representative of SOGREAH, France suggested the usage of accropods for improving the stability of the existing breakwater. The cross section was suggested by SOGREAH, France. The details of the cross section of the composite breakwater strengthen with accropods is shown in Fig.11. For the testing

an accropod alternative, a model scale of 1:29 has been adopted to suit the accropod unit (395gm) which was air lifted from France.

The tests were carried out with wave height of 50% of the design wave height and increased gradually in steps of 25% upto the design wave height. The duration of the test is for 500 waves, that is, twenty one minutes for the model and one hour twenty four minutes for the prototype.

The observations made in the 2m wide flume on the existing section, existing section strengthened with tetrapods, concrete cubes and accropods are shown in Fig. 12. Based on the hydrodynamic model tests carried out on the existing breakwater section and that strengthened with three different armour blocks, that is, tetrapods, concrete cubes and accropods, the following conclusions are drawn:

1. Damage of about 20% was observed for the existing breakwater section.
2. Tetrapods and concrete cubes are found to be stable in protecting the existing breakwater. Damage in both the cases is less than 3%, which is well within the permissible damage level of 5%.
3. Accropods are more stable in protecting the existing breakwater section with 0% damage and a significant overtopping is observed continuously.

5.3 Restoration of the breakwater of fishing harbour in Thangassery, Kerala

The fishing harbour at Thangassery in Kerala west coast of India is formed by a breakwater of length 2.1 km. The layout of the Thangassery harbour is shown in Fig.13. The breakwater is formed with armour layer of 3ton to 5ton quarry stones upto a water depth of 9m, beyond which the armour layer consists of 8ton tetrapods. The breakwater had served effectively for two years after construction. The monsoon in the year 1998 lead to serve damage of the armour layer resulting in total displacement of armour units, wash away of the leeside of the breakwater especially at chainages 1612m to 1622m and 1642m to 1652m. A detailed study had to be taken up immediately to look into the methods for restoring the damaged portion of the breakwater.

The average slope of the existing breakwater after damage by carrying out a survey as on January 31,1999 was found to be 1:1.1 as against a slope of 1:2.5. The details of the cross section is shown in Fig.14. Physical model tests were carried out initially to check the stability of the cross section as per design when it is subjected to a significant wave height of 3.75m occurring during the monsoons as reported by the harbour authorities. The stability tests were carried out for the two most frequently occurring wave periods of 8 and 10 sec. A model scale of 1:9 was adopted for the physical model study. The damage of the section was found to be more than the permissible level to the extent of 6.15% for 8 sec wave and 17.4% for 10 sec wave. The tests were followed up with suggestive measures of strengthening the armour layer. This consisted of provision of a berm at mean high water level of width of 4m by adopting the same size of rubble stones of 3ton to 5ton. The details of the existing cross section strengthened with 3ton to 5ton rubble stones is shown in Fig.15. The tests were repeated

and the damage to the modified cross section was found to be less than 2% which is less than permissible damage level of 5%. The section was found to be stable. The tests were also carried out for the trunk section beyond 9.0m water depth and head section, both the trunk section and head section are formed with 8ton tetrapods as primary layer. The trunk and head section were found to be stable since the damage is less than 2% which is well within the permissible damage level of 5%. The details of the trunk section beyond 1685m chainage (that is beyond 9.0m water depth) is shown in Fig.16 and that of head section is shown in Fig.17. Observations made on the breakwater cross sections after being subjected to waves is shown in Fig.18.

5.4 Stability tests carried out for other ports in India

S.No.	Name of the project	Primary layer
1.	Stability of breakwater models for Colachel port, Tamilnadu	Tetrapods
2.	Stability of rubble mound for approach jetty in Gujarat (L&T)	Tetrapods
3.	Stability of rubble mound breakwater for Muthalapozhy, Kerla.	Quarry stones and Tetrapods
4.	Stability of rubble mound breakwater for Rameswaram, Tamilnadu.	Quarry stones and Dolos
5.	Experimental studies on stability of breakwater and pressures induced on them due to regular and random waves	Tetrapods and Accropods (M.Tech Thesis)
6.	Stability of Frustrum Concrete Blocks for Breakwaters and Coastal Protection Works	A new armour block (Twin Frustum Concrete Block)

6.0 ONGOING RESEARCH WORK

6.1 Semicircular breakwater (SBW)

This is an Indo-German project. Semicircular breakwater is a recent concept evolved to meet the requirements of ever increasing marine traffic. The first ever SBW was constructed in Miyazaki Port, Japan. The submerged breakwaters have been constructed in Coastal zone for shoreline or harbour protection or to prevent beach erosion. On the other hand the submerged breakwaters will enhance the scenery. To achieve the combined advantages of these two types of breakwaters a model of

submerged semicircular breakwater was fabricated and a detailed experimental study was carried out in a wave flume in Ocean Engineering Centre, Indian Institute of Technology, Madras. The details of the SBW model is shown in Fig.19.

The objectives of this project are as follows:

1. Measurement of Hydrodynamic pressures along the surface of the semicircular breakwater model(SBW) model due to regular and random waves.
2. Investigations of Hydrodynamic forces due to both regular and random waves.
3. Investigation of reflection, runup and transmission characteristics due to both regular and random waves.

The semicircular breakwater model was subjected to the action of regular waves of heights ranging from 0.03m to 0.21m at intervals of 0.03m with each of the wave height having wave periods ranging from 0.8sec to 2.2 sec at intervals of 0.2 sec. The horizontal, vertical forces, hydrodynamic pressures and the runup and rundown on the model alongwith the variations of wave elevation in front of the model at the three different locations as stated earlier were acquired simultaneously through the same personal computer that is used to drive the wave maker. The tests have been carried out for the six different water depths based on hw/ht ratios, that is, 0.6, 0.7, 0.8, 1.0, 1.2 and 1.4 where hw is the water depth and ht is the total height of the model, to study the effect of water depth keeping hs/hr ratio constant. The definition of the variables is given in earlier figure. The tests were also carried out for three different hs/hr ratios that is 4.6, 2.5 and 1.67 to study the effect of height of the rubble keeping hw/ht ratio constant, where hr is the height of the rubble and hs is the height of the caisson.

The SBW model subjected to random waves generated from Pierson-Moskowitz spectrum. The significant wave height ranging from 0.03m to 0.21m in interval of 0.03m and peak wave period ranging from 0.8sec to 2.2sec in the interval of 0.2sec were adopted for the above said spectrum. Similar tests were carried out for random waves as discussed for regular waves.

The analysis of regular waves involved the determination of reflection characteristics, shoreward peak pressures, forces for all the frequencies and amplitudes tested. The salient features of the analysed results due to regular waves are given below.

Exposed SBW

- For the SBW protruding above the free surface for $hs/hr = 4.6$ and for $hw/ht = 0.6, 0.7$ and 0.8 , the reflection coefficient varies from about 0.5 to about 0.9 for the scattering parameter ranging from 0.7 to 3.2. As hw/ht increases the reflection coefficient decreases.
- The transmission coefficient is negligible, this is due to fact that for waves generated in lesser water depths, a significant amount of incident wave energy is reflected by the caisson and part of the energy is spent in the wave running over the curved surface facing the seaside.

- The dimensionless pressure, $\{(P_c)_{\max}/\gamma H\}$ [where, $(P_c)_{\max}$ is peak shoreward pressure, γ is mass density of water and H is the incident wave height] decreases with increase in scattering parameter for a constant h_s/h_r of 4.6 for different $h_w/h_t = 0.6, 0.7$ and 0.8 . Similar trend is seen for different $h_s/h_r = 4.6, 2.5$ and 1.67 for constant $h_w/h_t = 0.8$.
- The variation of the horizontal dimensionless force $\{(F_H)_{\max}/(\gamma a^2 H/2)\}$ [Here $(F_H)_{\max}$ is total horizontal force] and dimensionless vertical force $\{(F_V)_{\max}/(\gamma a^2 H/2)\}$ [where, $(F_V)_{\max}$ is total vertical force] decreases with increase in scattering parameter for different $h_w/h_t = 0.6, 0.7$ and 0.8 for constant $h_s/h_r = 4.6$. The results indicates that higher h_s/h_r results in larger vertical forces and lesser horizontal forces. The effect of h_s/h_r is found to be greater for higher values of scattering parameter.

Submerged SBW

For the submerged SBW, for $h_s/h_r = 4.6$ and for $h_w/h_t = 1.0, 1.2$ and 1.4 , the reflection coefficient varies from about 0.15 to about 0.25 for the scattering parameter, $ka (=2\pi a/L)$ ranging from 0.7 to 3.2. As h_w/h_t increases the reflection coefficient decreases.

- The transmission coefficient is found to be ranging from about 0.3 to 0.95. As h_w/h_t increases the transmission coefficient increases.
- The loss coefficient is found to be decrease with increase in scattering parameter.
- The dimensionless pressure, $\{(P_c)_{\max}/\gamma H\}$ decreases with increase in scattering parameter for a constant h_s/h_r of 4.6 for different $h_w/h_t = 1.0, 1.2$ and 1.4 .
- The variation of the horizontal dimensionless force $\{(F_H)_{\max}/(\gamma a^2 H/2)\}$ and dimensionless vertical force $\{(F_V)_{\max}/(\gamma a^2 H/2)\}$ decreases with increase in scattering parameter for different $h_w/h_t = 1.0, 1.2$ and 1.4 for constant $h_s/h_r = 4.6$.

6.2 Pile supported quadrant front face breakwater

A new type of structure pile supported front face breakwater was formulated by Ocean Engineering centre, Indian Institute of Technology, Madras, a detailed experimental study was carried out in a wave flume in Ocean Engineering Centre, Indian Institute of Technology, Madras. The details of the cross section is shown in Fig.20. The quadrant front face of radius of 500mm supported on piles of height 550mm and diameter 59mm were subjected to both regular and random waves in three different water depths of 0.8m, 0.9m and 1.0m. The effect of spacing between the piles was also studied. The wave period was varied from 1 to 2.4sec in intervals of 0.2sec. For each wave period, atleast five wave heights were tested. The wave steepness ranged from about 0.004 to 0.12m and a relative water depths ranged from 0.15 to 0.4m. The reflection, transmission and loss coefficient were computed and reported as a function of wave steepness for different relative water depth values. In addition, the wave elevation on the quadrant front face was measured and the dimensionless runup and rundown were also computed and reported as a function of wave steepness for different relative water depth values. Hydrodynamic pressures and forces were also measured. The salient results based on the hydrodynamic model tests analysed till now are represented below.

For Regular waves

- The reflection coefficient decreases slightly with increases in wave steepness and is found to vary from about 0.2 to 0.5. The reflection coefficient decreases with increase in h/d where h is the height of the pile and d is the water depth.
- The transmission coefficient decreases with increase in wave steepness and varies from about 0.2 to 0.4. The transmission coefficient increases with increase in h/d .
- The loss coefficient increase with increases in h/d and increase with increase in wave steepness and ranges from about 0.8 to 0.9.
- For lesser relative water depths, the dimensionless runup (runup/wave height) is found to be also greater than 1 and varies from about 0.3 to 1.1. The dimensionless rundown (rundown/wave height) varies from 0.1 to 0.6.
- The dimensionless force ($F_{H \text{ or } V} / \gamma r^3$) in the vertical and horizontal direction increases with increase in wave steepness, where $F_{H \text{ or } V}$ is force in horizontal or vertical direction, γ is the unit of water and r is the radius of the quadrant face.

7. REFERENCES

Owen, M.W and Allsop, N.W.H(1983) "Hydraulic Modelling of rubble mound breakwaters", Proceedings of conference of Institution of civil engineers, London. pp 71-78.

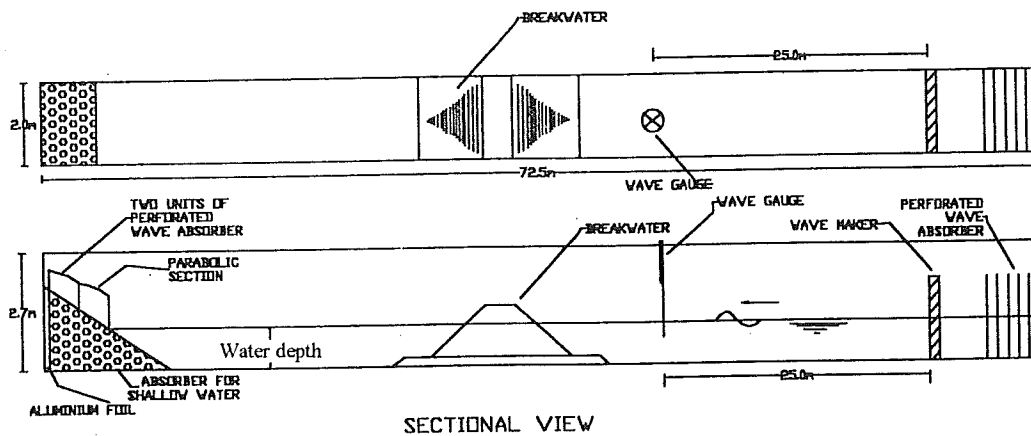


Fig.1 Details of the 2m wave flume

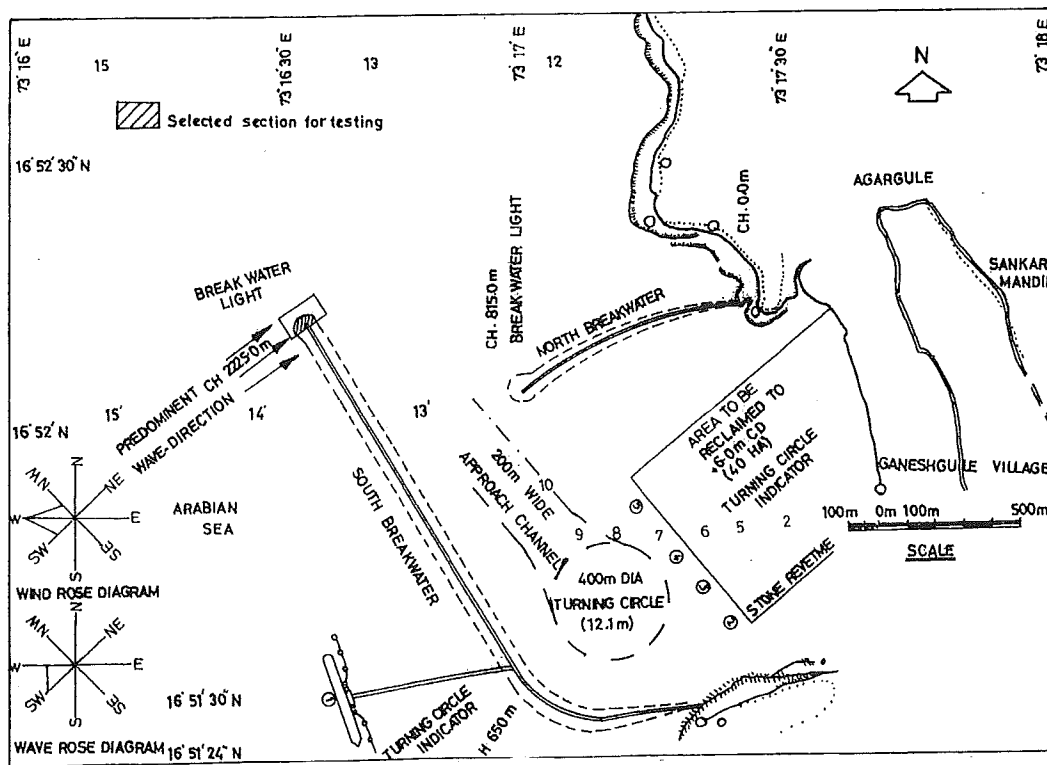


Fig.2 Layout of the Ganeshgule harbour

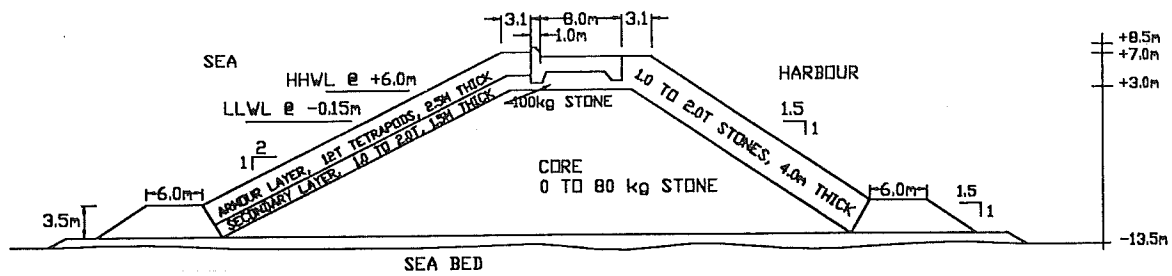


Fig.3 Trunk section (prototype) for Ganeshgule

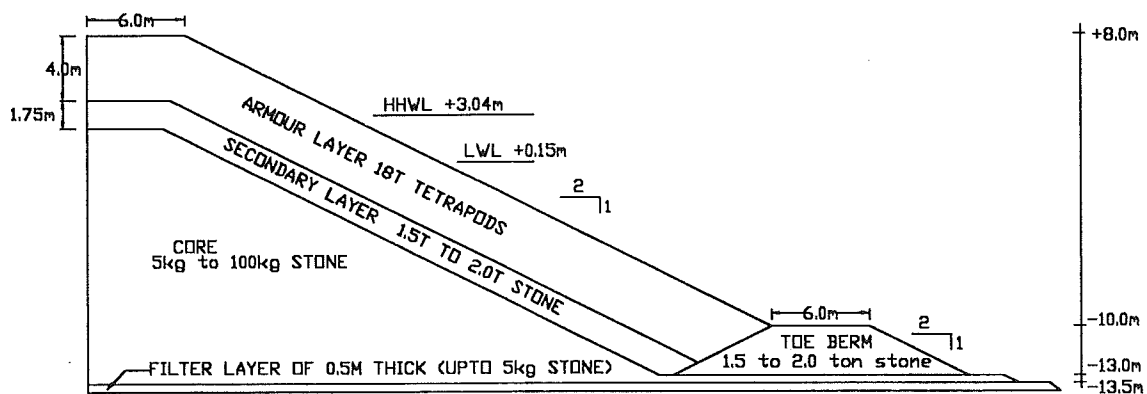
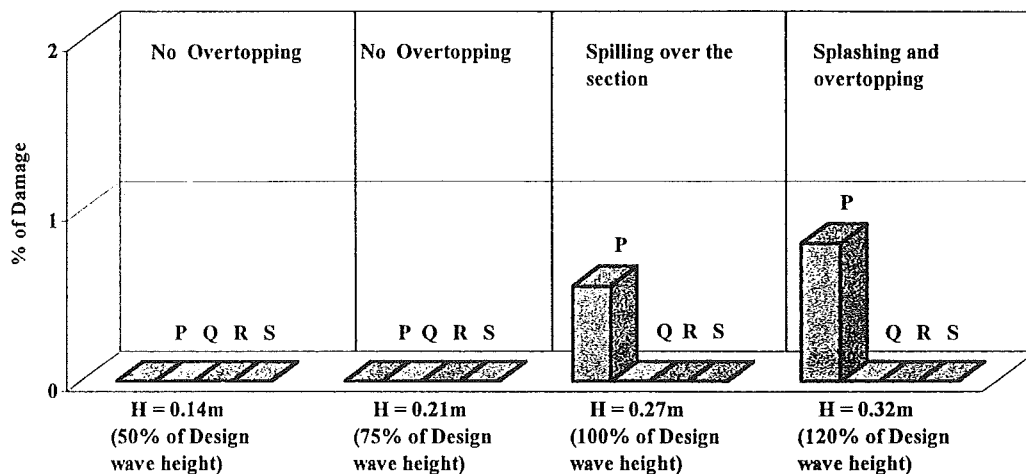
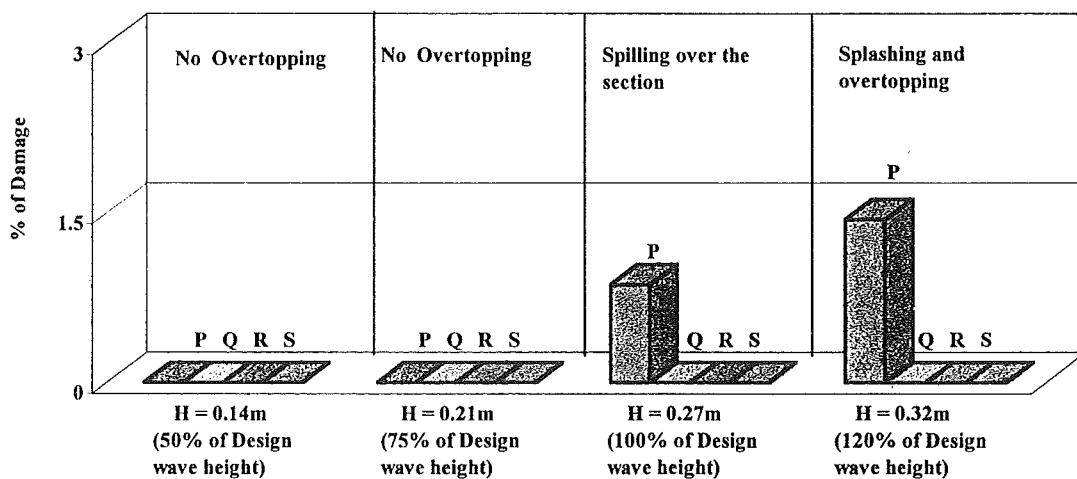


Fig.4 Roundhead section (prototype) for Ganeshgule

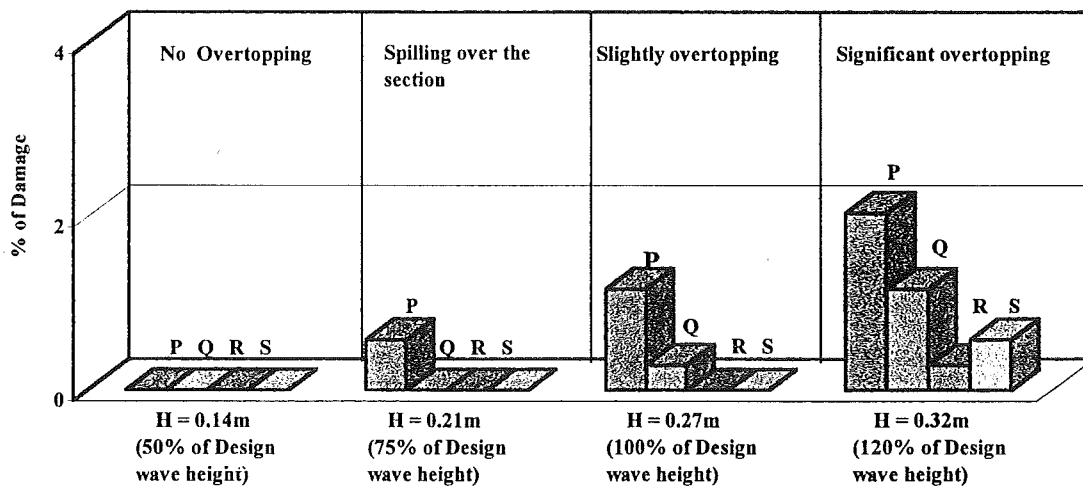
Model scale 1:17



(a) Water depth = 0.81m (MEAN SEA LEVEL)



(b) Water depth = 0.85m (MEAN HIGHER HIGH WATER)



(c) Water depth = 0.90m (HIGHEST HIGH WATER LEVEL)

Fig. 5 Observations made on Trunk section (Ganeshgule)

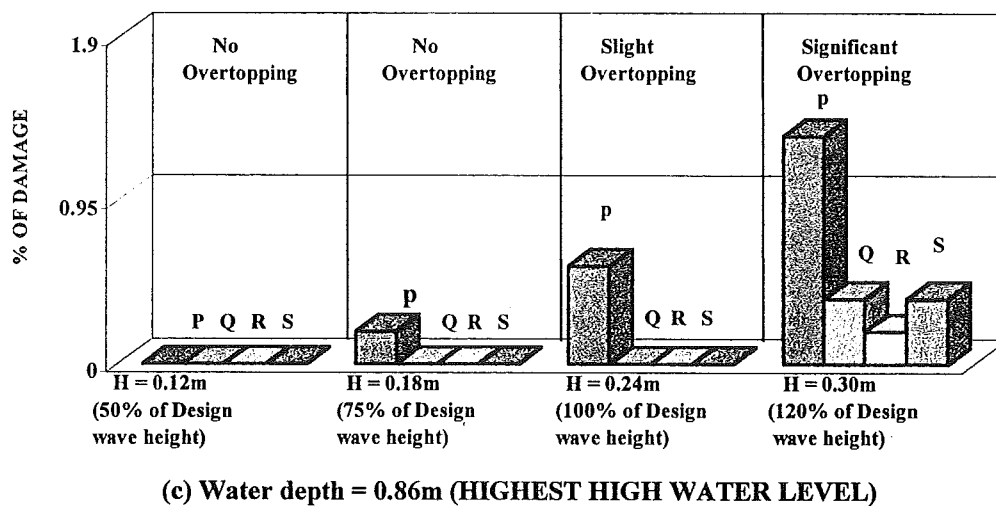
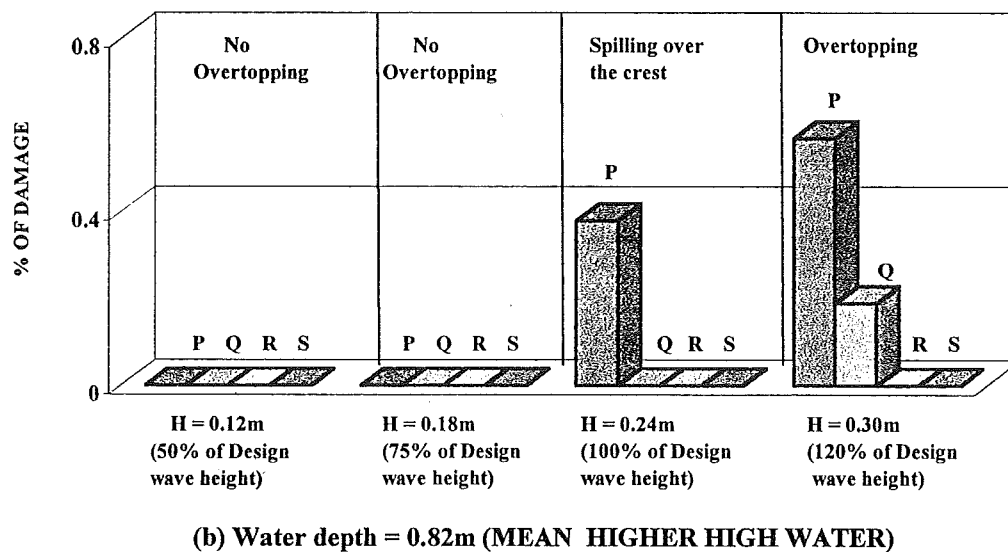
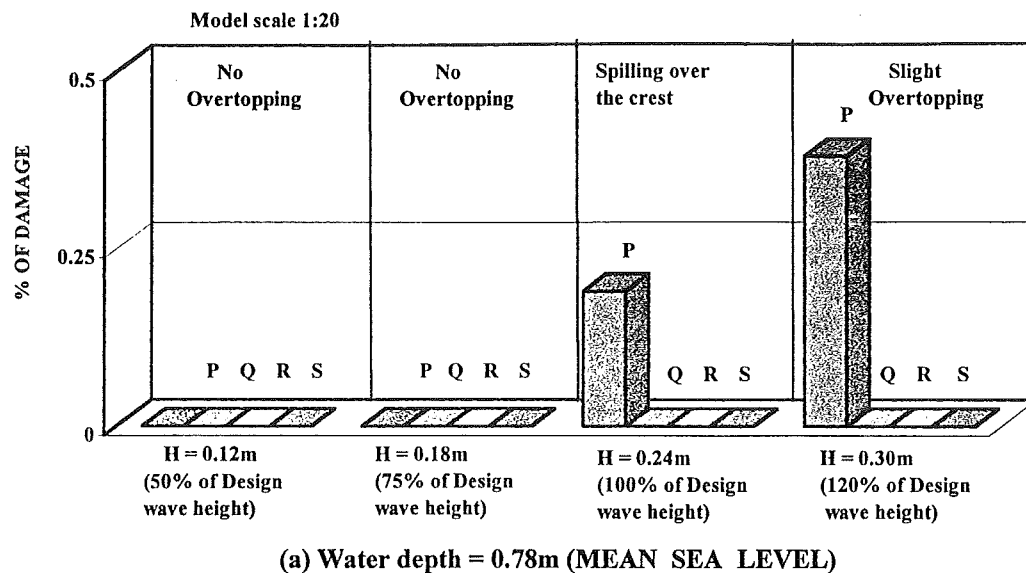


Fig. 6 Observations made in the flume for Head section (Ganeshgule)

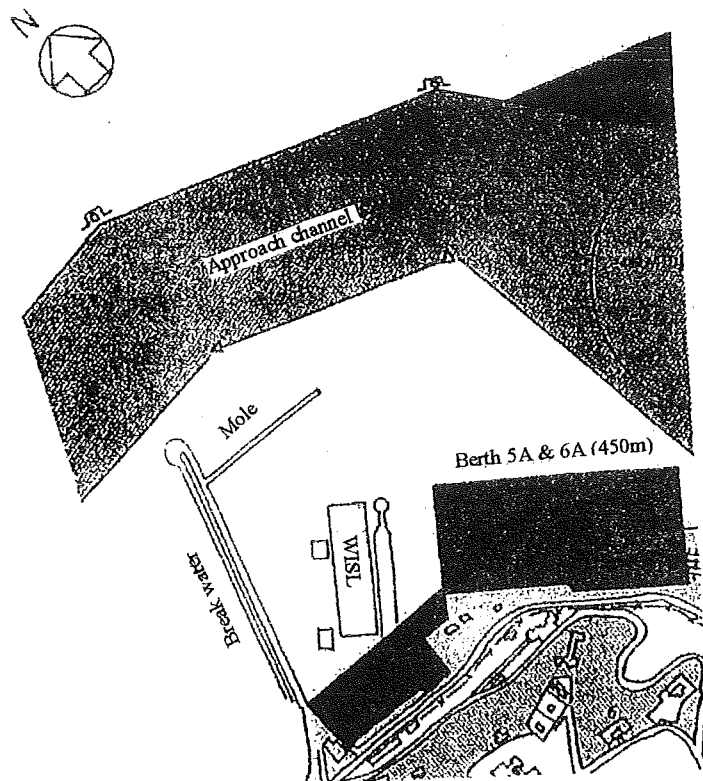


Fig. 7 Layout of the Mormugao port

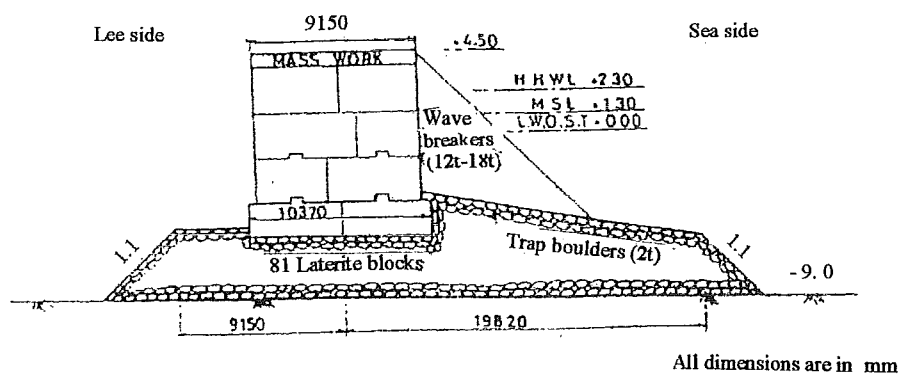


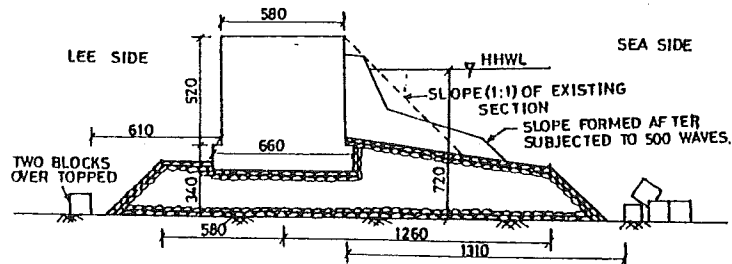
Fig. 8(a) Existing cross section of the breakwater (Mormugao port)

Notes

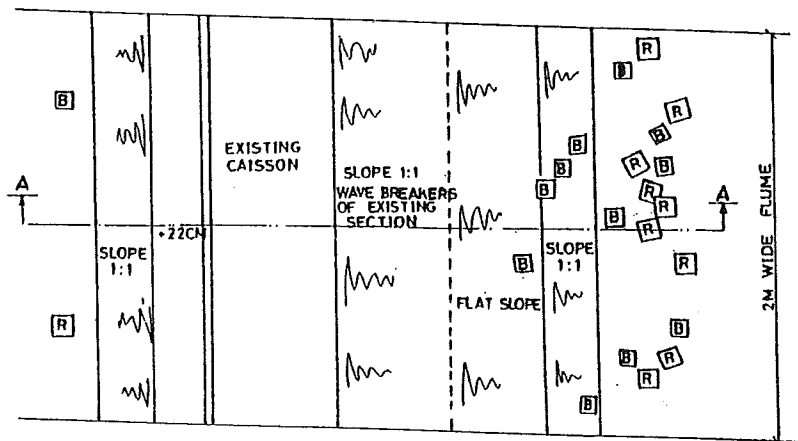
All dimensions are in mm

R: 4.60kg concrete cubes

B: 3.07kg concrete cubes



SECTION AT A-A



PLAN SCALE 1:20

Fig. 8 (b) Damage due to the existing cross section after being subjected to waves

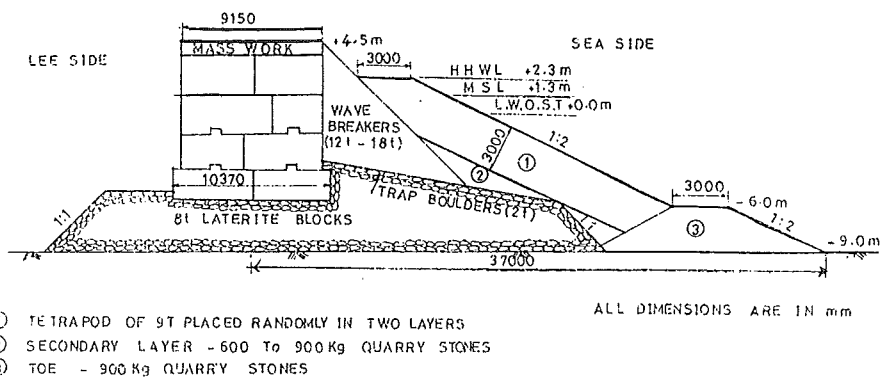


Fig.9 Cross section of the caisson breakwater strengthened with tetrapods

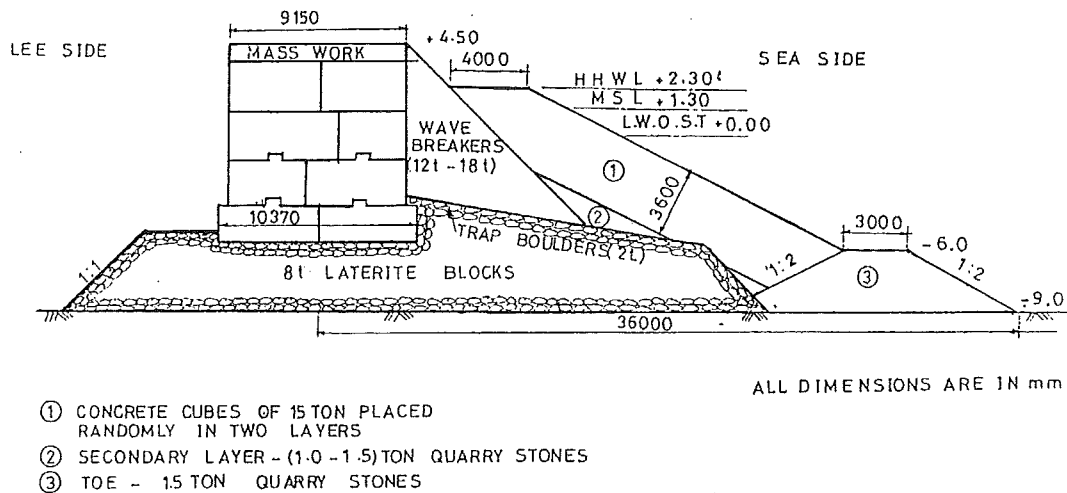


Fig.10 Cross section of the caisson breakwater strengthened with concrete cubes

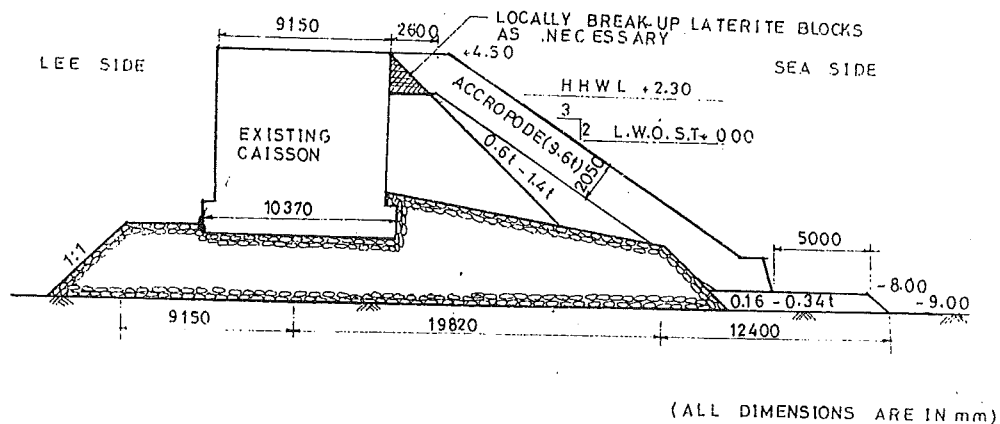
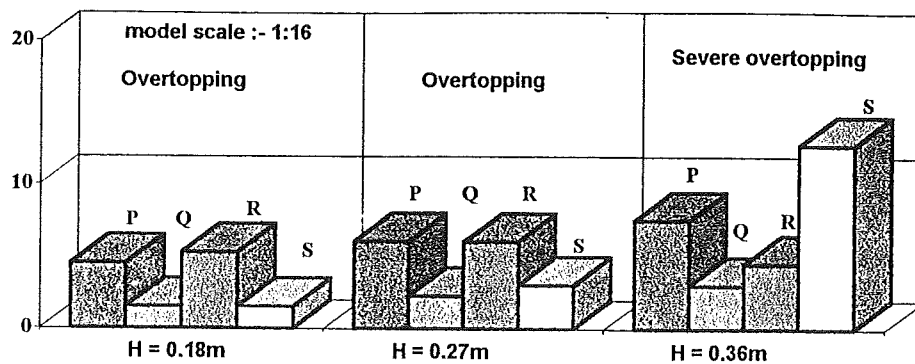
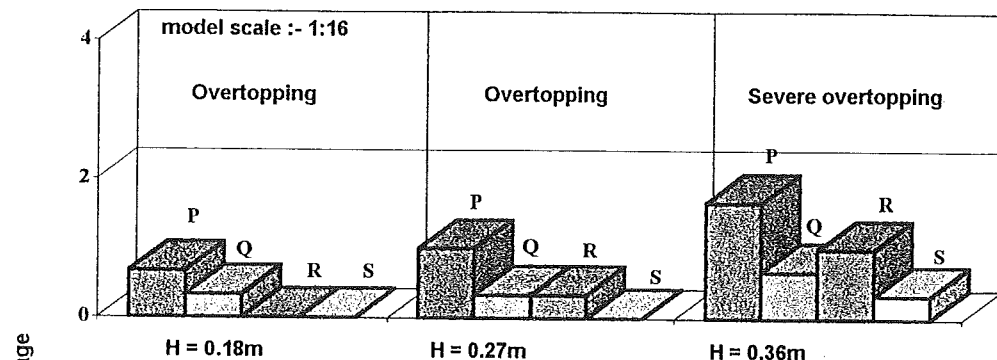


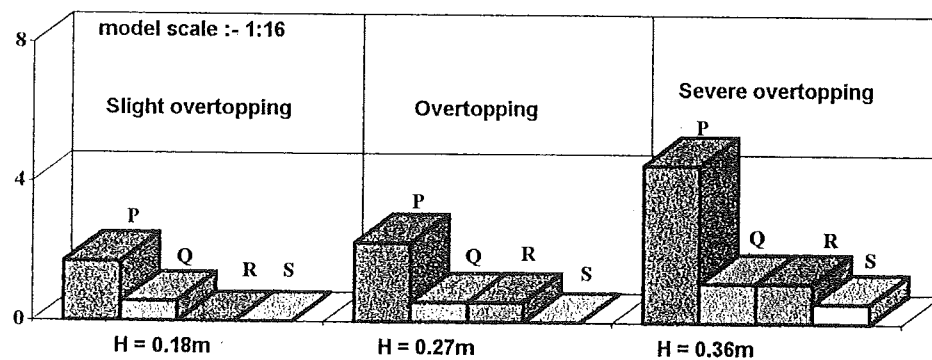
Fig.11 Cross section of the caisson breakwater strengthened with accropodes



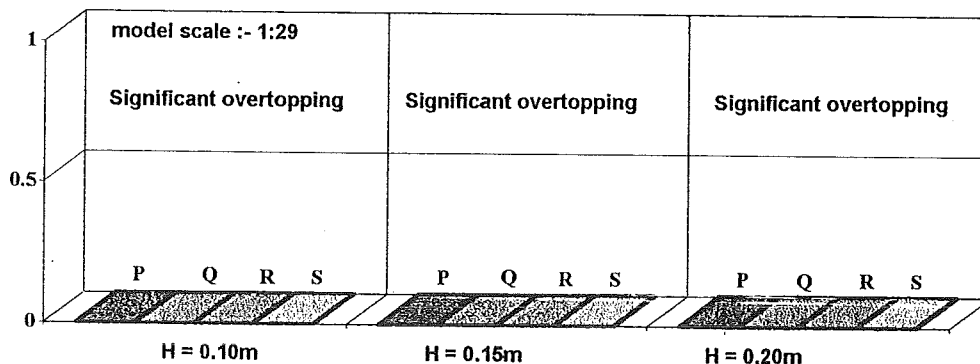
(a) Observations made on existing section



(b) Observations made on existing section strengthened with Tetrapods



(c) Observations made on existing section strengthened with Concrete cubes



(d) Observations made on existing section strengthened with Accropods

Fig. 12 Observations made in the Flume

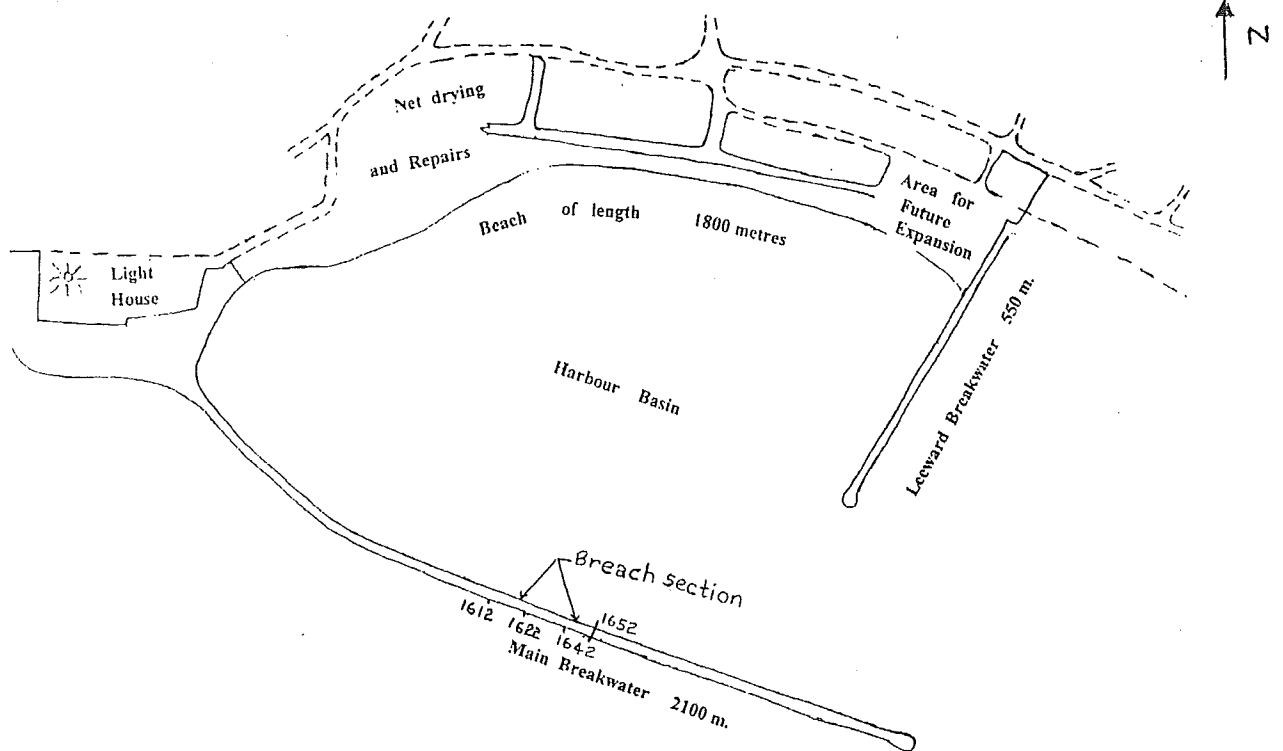


Fig.13 Layout of Thangassery Fishing harbour

- ① Armour layer-3 To 5ton Stones
- ② Secondary layer - 300 Kg To 500 Kg Stones

1. All dimensions are in Metres

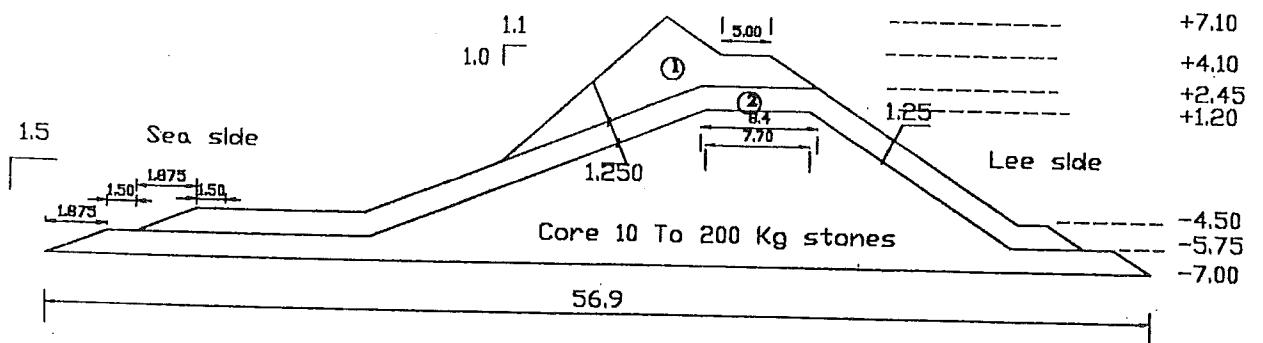


Fig.14 Cross section at the breached section as on 31.01.1999

- ① Armour layer - 3 to 5 ton stones
- ② Secondary layer - 300 to 500 Kg stones
- ③ Proposed remedial measure (3 - 5 ton stones)

1. All dimensions are in Metres

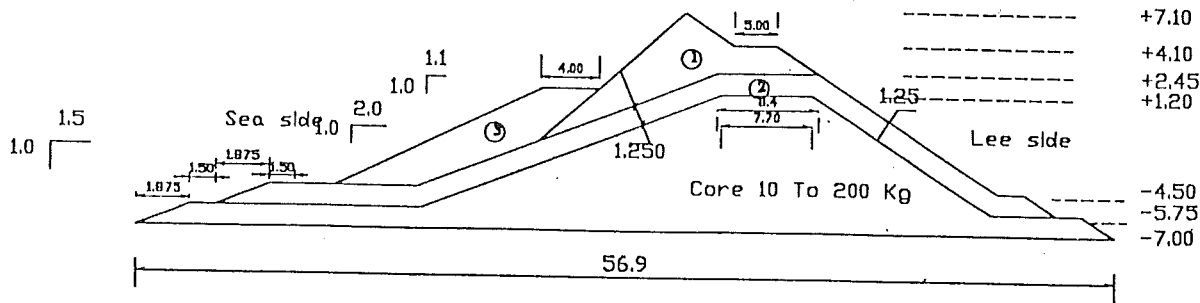


Fig.15 Cross section at the breached section strengthened with quarry stones

- ① Armour layer - 8T Tetrapods
- ② Secondary layer - 300Kg To 1200Kg
- ③ 300Kg To 500Kg stones
- ④ 3T To 5T stones

All dimensions are in metres

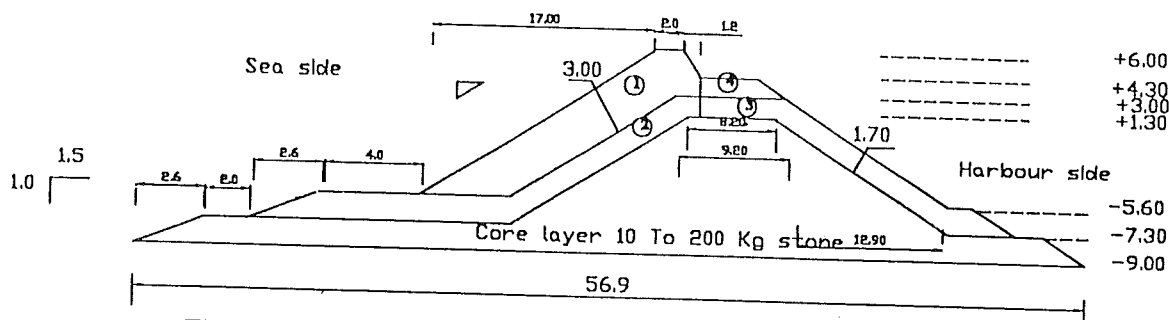


Fig.16 Breakwater cross section beyond 1685m chainage

- ① Armour layer - 8T Tetrapods
- ② Secondary layer - 800 To 1200Kg stones
- ③ 3T To 5T stones
- ④ 300 To 500 Kg stones

All dimensions are in metres

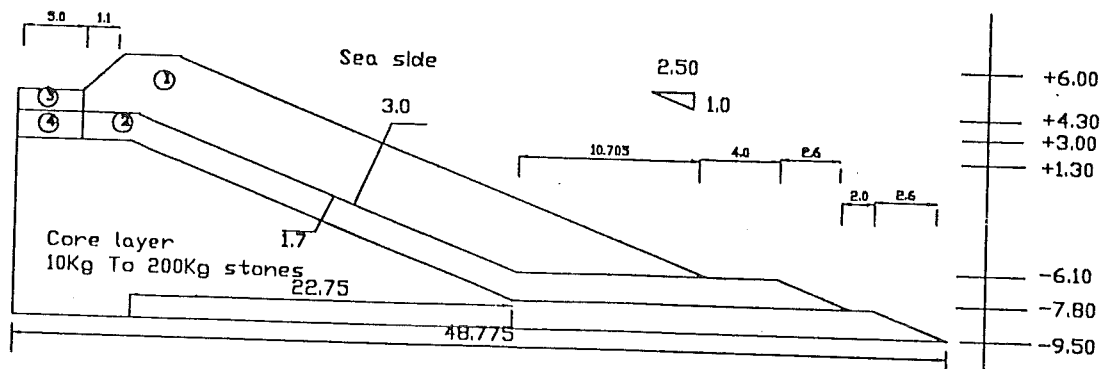


Fig.17 Roundhead section

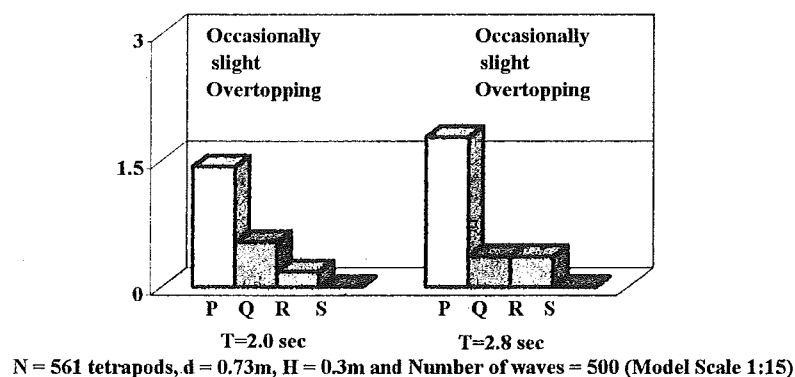
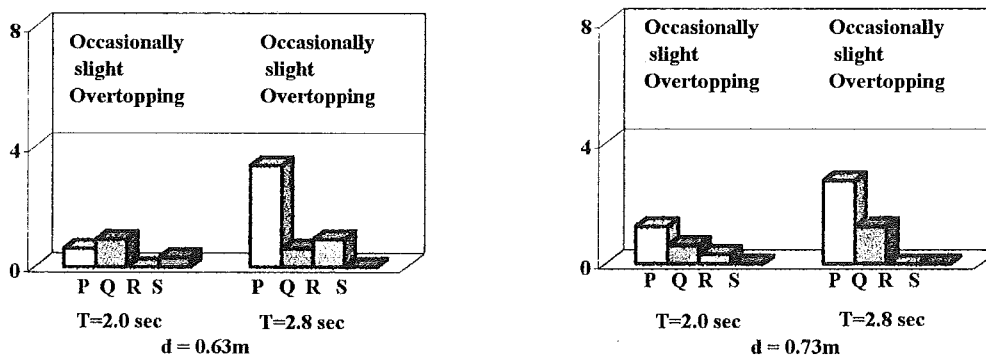
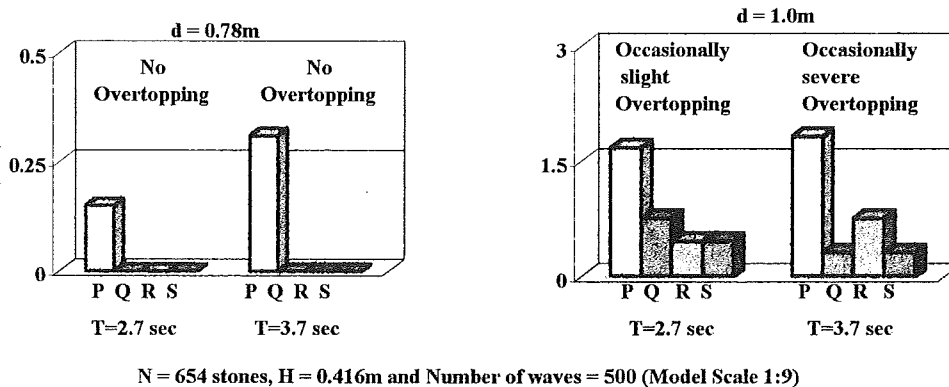
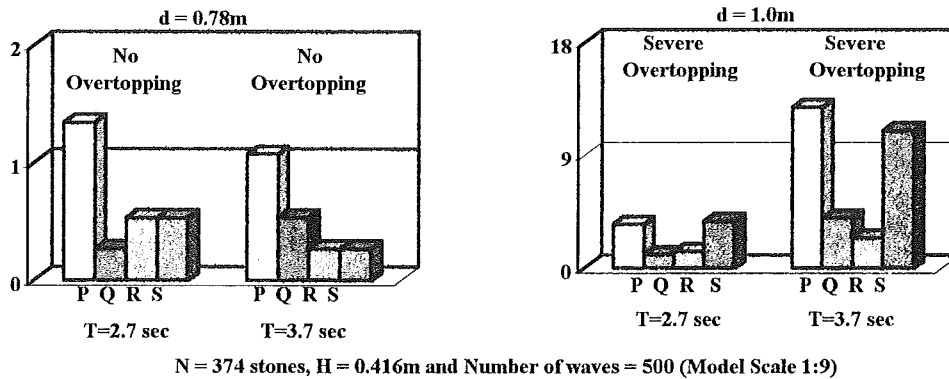
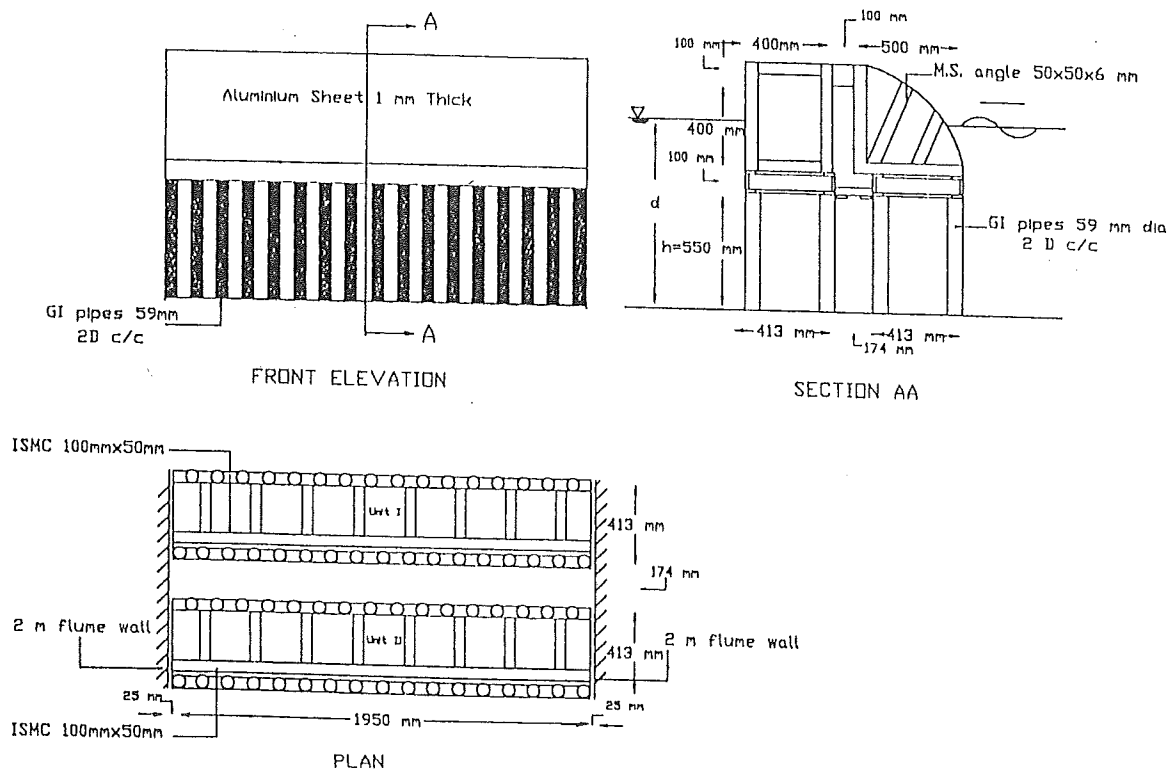
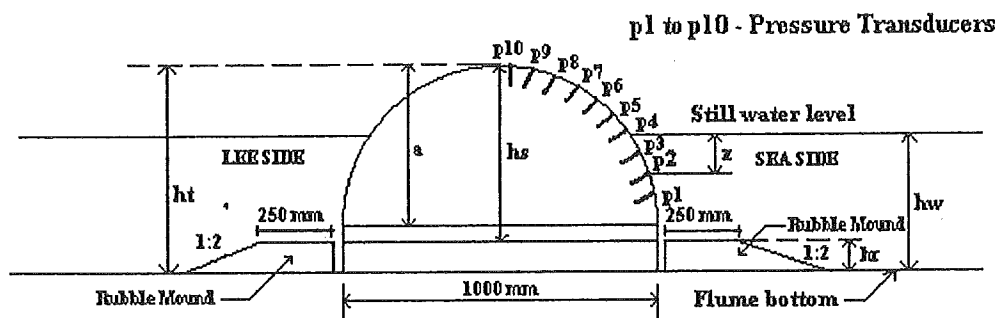


Fig.18 Observations made in the wave flume





INTRODUCTION ON ENNORE COAL PORT PROJECT

L.A. Meyboom
Haskoning
Chennai
Email: hasko_ch@satyam.net.in

INTRODUCTION

The requirements of the new Ennore Port have been formulated in a feasibility study performed by HASKONING - RITES in 1988 - 1990 and subsequent reviews performed by the Government of India, Asian Development Bank and Madras Port Trust.

The New Port will initially serve incoming coal transportation in bulk carriers up to Panamax size (65,000 DWT) for the benefit of the Tamil Nadu Electricity Board (TNEB). The design capacity for incoming coal is 16 million tonnes per year. To meet this capacity requirement two fully utilized berths are foreseen at the Southern end of the new outer port. The movements of ships and the coal unloading of coal from the ships at the berth shall in principle be a round the clock activity during all seasons. The Port and its coal handling facilities shall be operable round the clock and all year, except for some odd hours or days per year, when extreme conditions occur.

During HASKONING's Feasibility Study a port layout has been developed with an entrance from East to West. A Northern and a Southern breakwater of about equal length were designed to give the harbour the necessary protection. A dredged entrance channel outside the breakwaters would provide a straight access by ships heading Westwards.

Drawbacks of this layout were identified within the present Consultancy work and therefore alternative layouts have been formulated, studied and compared.

Apart from the feasibility layout with an Eastern entrance there seems to be only two other options feasible on the basis of relevant criteria.

One alternative layout has an approach route with a compass angle of initially 250° and before entry 225° ; consequently the harbour mouth is towards the Northeast. The sea entry of this port layout is close to the Ennore shoals. This is the so called **layout with North-East entrance** (Fig. 1).

The other alternative layout has a straight approach route with a compass angle of 345° and a harbour mouth between South and SSE. This is the so called **layout with Southern entrance** (Fig. 2).

In order to arrive at a conclusion for the nautical safety of the two alternative layouts, the following observations are made in addition to the fast time simulation results, for the layout with a **NE-entrance**:

- The last stretch of the approach route before entering the mouth is straight allowing for easy entry of the critical passage through the narrow harbour mouth.
- The last stretch of the approach route before entering through the mouth is very close to the Ennore shoals. The danger of grounding by a ship with slight manoeuvring problems is real, but the occurrence of manoeuvring problems is only very small. To reduce this risk the entrance channel has been widened to 250 m. If however one ship is lost on the shoals once in ten years it is at a cost which can not be ignored.
- The last stretch of the approach route is parallel to the depth contour lines, which Mariners dislike and Port planners try to avoid.

The following observations are made in addition to the fast time simulation results, for the layout with a **Southern entrance**:

- A straight entrance is projected from deep sea in to the outer harbour along an initially 180 m wide channel, widening to 250 m near the head of the Eastern breakwater and into the outer port. This will facilitate a safe entry, while the shiphandler can conveniently adjust his course for local changes in current, wind and wave effects.
- The straight approach channel outside the breakwaters makes a convenient angle of 30° with the prevailing depth contour lines. This is a good port planning practice and Mariners will appreciate this when approaching Ennore. The channel is consequently shorter than for the other alternative.
- The stopping manoeuvre of single screw vessels with right hand turning propulsion tend to deviate to starboard. Allowance has been made for this deviation just after the passage of the harbour mouth and consequently the Eastern breakwater alignment has been bowed seawards.

It is concluded that the criteria for safe navigation are met in both the layouts, however the Southern entrance is still preferred because of:

- a straight and shorter entrance channel, which has convenient potential for expansion to allow safe negotiation by larger ships in the future,
- the presence of a shoal adjacent to the last stretch of the NE-entrance channel, which might be a threat to Mariners in very exceptional cases.

Table 1 Comparison of construction costs

Item	Description	S Entrance		NE Entrance	
		Crores Rs	US \$ million	Crores Rs	US \$ million
1	Rock quarrying and transport	192.96	64.32	205.20	68.40
2	Breakwater				
	- construction	150.33	50.11	174.18	58.06
	- allowance for soil improvement	30.00	10.00	30.00	10.00

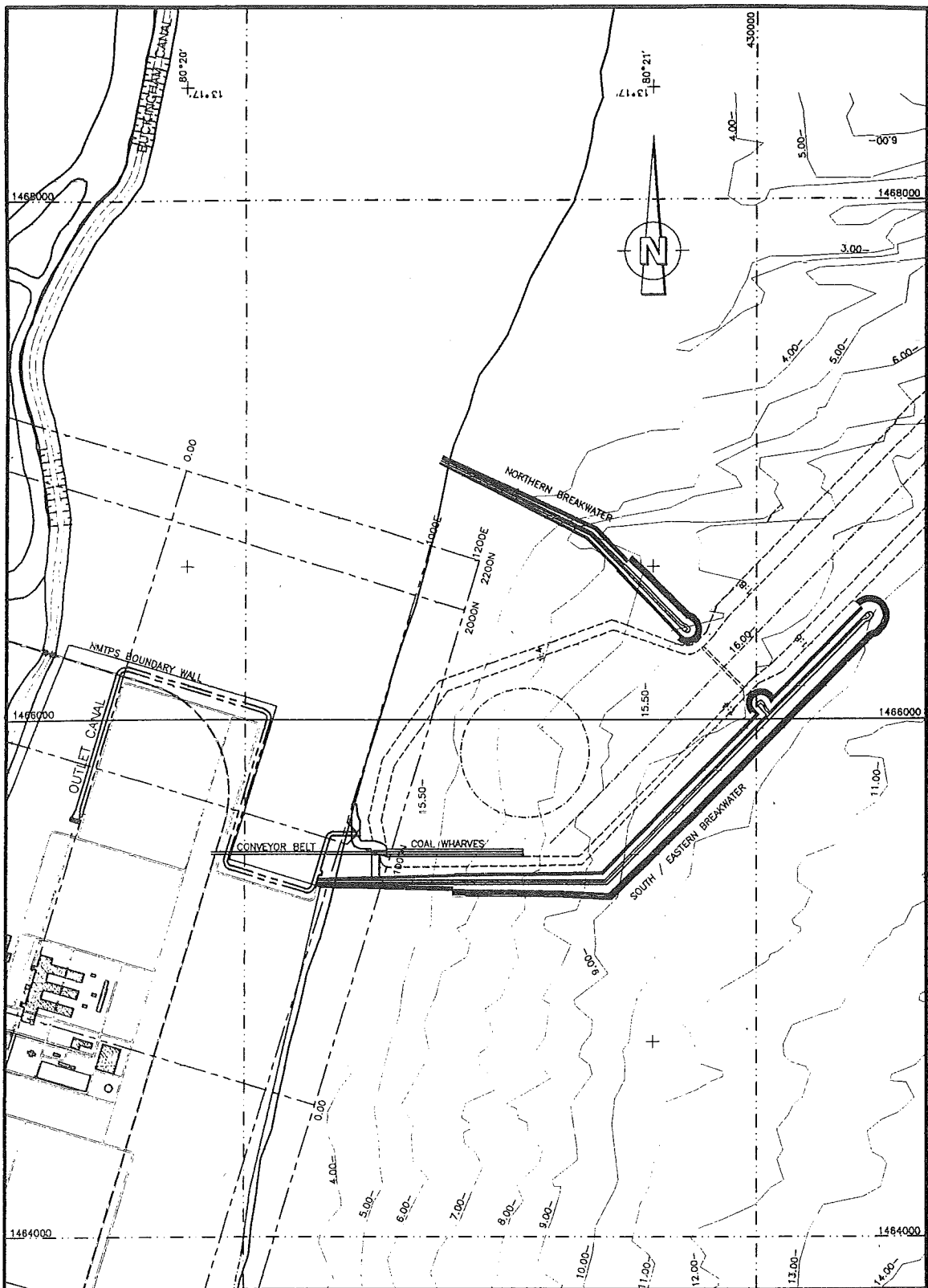
		S Entrance		NE Entrance	
3	Dredging works	120.60	40.20	153.90	51.30
4	Coal berths and small craft Jetties	74.95	24.98	74.95	24.98
5	Groynes and shore protection - groynes - slope protection	10.35 3.75	3.45 1.25	0.00 3.75	0.00 1.25
6	Onshore facilities: - priority works - other works	18.28 17.48	6.09 5.83	18.28 17.48	6.09 5.83
7	Tugs and other craft	79.50	26.50	79.50	26.50
8	Navigation aids	1.77	0.59	2.64	0.88
9	Contingencies 10%	70.00	23.33	75.99	25.33
	TOTAL COST	769.97	256.65	835.87	278.62

Exchange rate : 1 US \$ = 30 Rs

Table 2. Multi-criteria analysis

		Port layout with North East Entrance	Port layout with Southern Entrance
1	Nautical Safety along Entrance Channel	<ul style="list-style-type: none"> ■ Bend in channel (-) ■ Channel parallel to depth contour lines (-) ■ Channel very near to shoal (-) ■ Negative impacts of above, reduced through wider (250 m) channel 	<ul style="list-style-type: none"> ■ Straight entrance channel (+) ■ Channel at 30° with depth contour lines (+)
2	Nautical Safety inside harbour	<ul style="list-style-type: none"> ■ Coal berth at end of entry/stopping course (-) ■ Fast time simulation results good (+) 	<ul style="list-style-type: none"> ■ For stopping ships deviating to starboard, extra manoeuvring space provided (bend in north-eastern breakwater) ■ No berth at end of entry/stopping course (+) ■ Fast time simulation results good (+)

		Port layout with North East Entrance	Port layout with Southern Entrance
3	Tranquillity at berth	<ul style="list-style-type: none"> ■ Largest persistent wave attack from NE and ENE, causing very poor tranquillity conditions at coal berths (--) ■ H_s at berth is 60% of H_s outside. H_s outside = 2.5m → H_s berth = 1.5m ■ Penetrating waves attack at the side of the ship, causing rolling of ship (-) ■ Downtime due to unrest at coal berth - 30 days per year (--) 	<ul style="list-style-type: none"> ■ Tranquillity at berth good (++) ■ H_s at berth is 10% of H_s outside harbour H_s outside = 2.0m → H_s berth = 0.2m
4	Tranquillity at turning basin	<ul style="list-style-type: none"> ■ Tranquillity is poor, (-) H_s turning basin = 50% of H_s outside harbour 	<ul style="list-style-type: none"> ■ Tranquillity is acceptable, (+) H_s turning basin = 25% of H_s outside harbour
5	Accretion Siltation	<ul style="list-style-type: none"> ■ Accretion at south side of harbour will be controlled by requirement to keep Ennore creek open. Ample storage capacity of accreted sandy material south of harbour (+) ■ Siltation in harbour and entrance channel due to fines suspended in water - about 0.45 million m^3/y ■ Maintenance dredging by trailer hopper - 0.9 million m^3 once in 2 years. 	<ul style="list-style-type: none"> ■ Accretion at south side of harbour will be controlled by requirement to keep Ennore creek open. Storage capacity between creek and harbour limited - 3 small groynes required (-) ■ Sand fill in entrance 0.10 million m^3/y ■ Siltation in harbour and entrance channel due to fines suspended in water - about 0.35 million m^3/y ■ Maintenance dredging by trailer hopper 0.9 million m^3 once in 2 years.
6	Coastal Erosion at the Northern side of Port	<ul style="list-style-type: none"> ■ Erosion conditions equal to other layout. Dispose 2.0 million m^3 sand north of harbour during construction, will counteract erosion for about 10-15 years. 	<ul style="list-style-type: none"> ■ Erosion conditions equal to other layout. Dispose 2.0 million m^3 sand north of harbour during construction, will counteract erosion for about 10-15 years.
7	Future expansion potential	<ul style="list-style-type: none"> ■ Major terminals in basin complex and compromise terminals (3) along north breakwater (+) 	<ul style="list-style-type: none"> ■ Major terminals in basin complex and compromise terminals (4) along the present coastline (+)
8	Comparable cost differences	<ul style="list-style-type: none"> ■ Total construction cost is 100%. 	<ul style="list-style-type: none"> ■ Total construction cost is 92% of other alternative.



Madras Port Trust
Ennore Coal Port Project

LAYOUT WITH NE ENTRANCE



HASKONING
 Royal Dutch Consulting
 Engineers and
 Architects

in association with

COLES
 of India

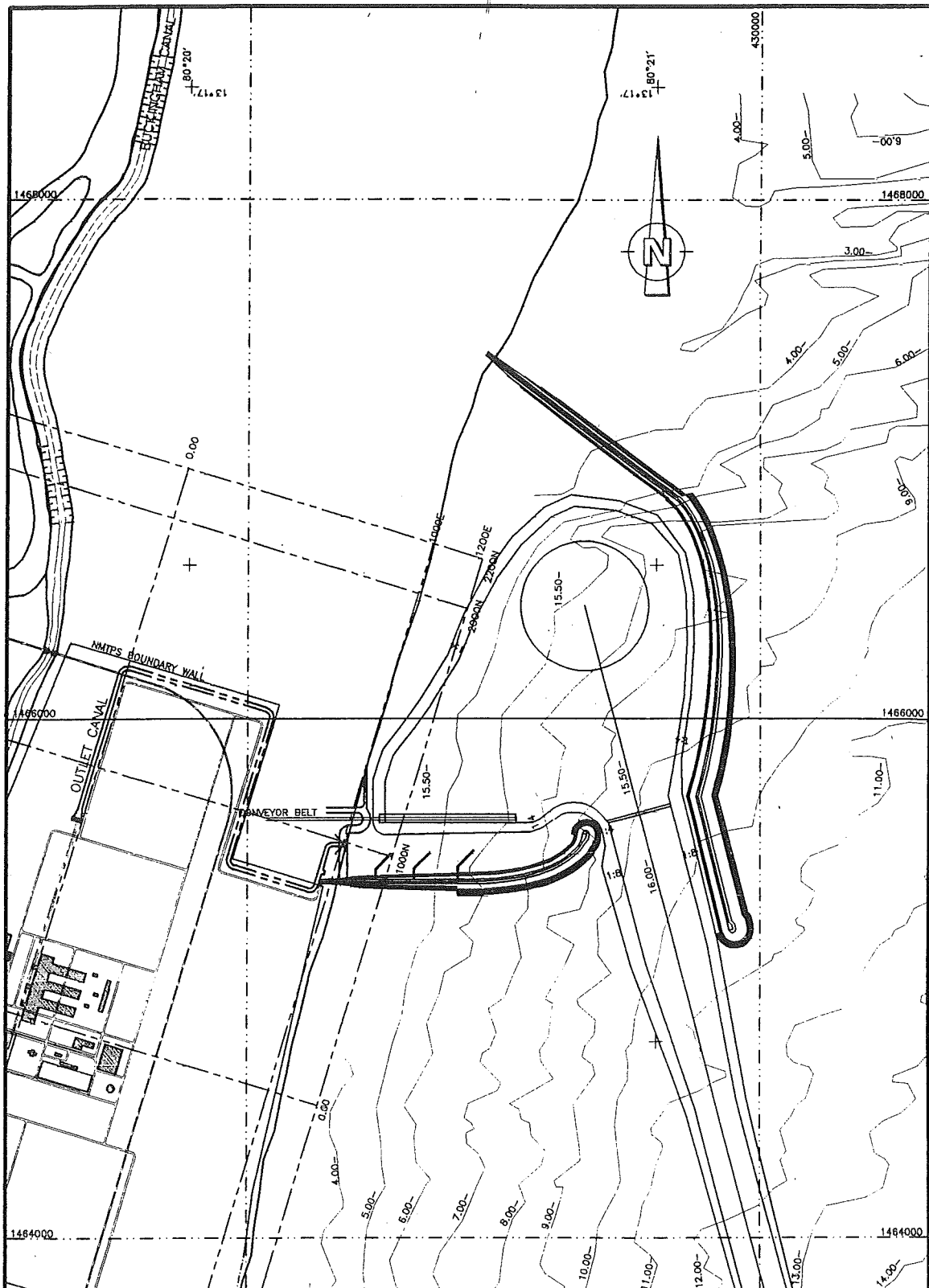
date	drawn	revision	checked
10-03-95	JCA		
17-02-95	JCA		

SCALE 1 : 20000 A4-size

Project No 9019A5528.21

DRWG. No 10-014

Fig. 1



**Madras Port Trust
Ennore Coal Port Project**

LAYOUT WITH S ENTRANCE



HASKONING
Royal Dutch Consulting
Engineers and
Architects

in association with



of India

date	drawn	revision	checked
10-03-95	JCA		
17-02-95	JCA		

SCALE 1 : 20000 A4-size

Project No 9019.A5528.21
DRWG. No 10-013

Fig. 2



DESIGN OF BREAKWATERS FOR ENNORE PORT

R. Haggie
Haskoning
Chennai
Email: hasko_ch@satyam.net.in

1.0 INTRODUCTION

1.1 Project Setting

The site of the New Port of Ennore is approximately 20 km north of Chennai (formerly Madras), on the Coromandel Coast in the State of Tamil Nadu. At the coast some small sand dunes have formed to a height of approximately + 3.5 m Chart Datum (hereafter referred to as CD). Behind the dunes the land drops slightly to a marshy area at approximately + 3.0 m CD and continues west to the Buckingham Canal (approx. 2 km).

The site is bordered to the south by the North Madras Thermal Power Station (NMTPS). A cooling water outlet of the power station marks the Southeast corner of the Port site. On the West side, the Buckingham Canal borders the Port area and the NMTPS. A number of salt pans are located to the west of the Buckingham Canal which vary in size, have an average bottom depth of + 0.0 m CD and at present are not in use.

South of the NMTPS, the Ennore Creek (outlet of the river Kortaliyar) discharges into the Bay of Bengal. To the Northeast of the Port site are the Ennore Shoals. The distance between the Ennore Shoals and the Ennore Creek is approximately 5 km. The coastline between the Ennore shoals and the Ennore Creek is orientated in a North-northeast (NNE) direction. The general Port layout is shown in Figure 1.

1.2 Functional Requirements

A breakwater normally fulfils a number or all of the following functions:

- providing tranquillity in the harbour entrance and Basin;
- blocking littoral drift and thus avoiding sedimentation of the Entrance Channel and the Port Basin;

- protection of the Entrance Channel and Port Basin against waves and current and thus providing safe manoeuvring and berthing conditions for vessels;
- providing safe and/or operational berthing conditions for ships;
- providing land access to berths;
- providing visual guidance (enhanced by navigation beacons) for ship traffic.

The breakwaters of the Ennore Port will have to fulfil all these functions except for the safe berthing conditions during heavy cyclone conditions, since the Port will not be designed as a survival port for these conditions.

The most important function of the breakwaters is to provide a sufficient level of tranquillity in the Entrance Channel and in the Port Basin. The tranquillity in the Port is not only governed by the wave penetration through the entrance but also influenced by the wave energy that is transmitted by overtopping and flow through the breakwaters. The level of transmission depends on the crest level, porosity and width of the breakwater.

The tranquillity required for normal operational conditions should be achieved during a sufficiently high percentage of the time, otherwise too much down time of the port operation will occur. The tranquillity in the Port during heavy cyclone conditions should also be controlled and should have a sufficient low level to prevent unacceptable damage to the waterfront structures. Safe berthing conditions during heavy cyclones is not a design condition because the Port will not be designed as a survival port.

A breakwater should fulfil its functions during a certain pre-set period of time, called the lifetime of the structure. This lifetime is normally set at 50 or 100 years. The longer the design lifetime of a breakwater the higher the probability it will be subject to extreme wave conditions and consequently the stronger the structure has to be. The 1/100-years conditions have been selected as design conditions for the breakwater, being in-line with the practice in India. The difference between the 1/50 and 1/100-years event for the wave conditions is very small due to the fact that the waves are depth limited. The expected future level of maintenance is a functional requirement and also a design parameter.

1.3 Design Approach

The preferred design for the breakwaters has been defined as the design that fulfils the above functional requirements for the lowest cost. During the Preliminary Design a cost comparison was executed for the following four alternative designs of the primary armour of a rubble mound breakwater:

- Natural rock, S-shaped berm;
- Concrete Cubes for the deepest sections of the breakwaters;
- Concrete Tetrapods for the deepest sections of the breakwaters;
- Concrete Accropode units for the deepest sections of the breakwaters.

The comparison showed Accropodes to be the cheapest solution. Based on the argument of lowest cost and on the fact that all four alternatives showed an equal degree of stability and functionality, the alternative with Accropode armouring was believed to

be the most promising solution. This alternative was subsequently developed on a Preliminary Design level.

The Designers recommended carrying out three-dimensional testing on the basis of the Preliminary Design and, based on the test results, to further elaborate on the Preliminary Design during the Final Design. The Client, Chennai Port Trust, confirmed this recommendation and subsequently the three-dimensional model test was executed.

After the model test, the Preliminary Design was altered, as found necessary based on the test results, leading to the Final Design of the breakwaters. The Final Design is and the design process, as presented hereafter, is not only based on breakwater requirements (functional/stability) but also on feasible construction techniques.

2.0 BOUNDARY CONDITIONS

2.1 Site conditions

The southern part of India experiences two monsoon seasons separated by transitional periods of calm weather:

- Southwest monsoon (mid-April to mid-August);
- Transitional (mid-August to mid-October);
- Northeast monsoon (mid-October to mid-January);
- Transitional (mid-January to mid-April).

The impact of the Northeast monsoon is more pronounced than that of the Southwest monsoon. Tropical depressions and cyclones are a recurrent phenomena, usually occurring in the months of October through to November. The location of the future Port of Ennore is affected by cyclonic activity every year. Cyclonic disturbances originating in the Bay of Bengal track westwards towards the Indian coastline. Wave heights associated with these cyclones can be as high as 5 to 8 m.

During the Northeast monsoon (mid-October to mid-January) the current is directed southward and in the Southwest monsoon (mid-April to mid-August) the current is directed northwards. Currents in the coastal zone are approximately 0.15 to 0.25 m/s

The tides experienced at the Port of Chennai are semi-diurnal with mean spring tide ranges of 1.0 m and mean neap tide ranges of 0.4 m (MHWS is + 1.10 m CD and MLWS + 0.10 m CD). The coast is sandy and the coast gradient is gentle; it is exposed to a continuous moderate surf.

2.2 Wave conditions

2.2.1 Normal wave conditions

The normal wave conditions for the location of the Ennore port have been derived from a Global Hindcast Study [14] which used available data showing the probability of occurrence of the resultant sea and swell in given height and period

classes. These results are presented in Figure 2 and show the percentage of time per year that a certain wave height will be exceeded. These results have been used as a basis for assessing feasible construction methods in relation to the design requirements.

From Figure 2 it can be seen that a significant wave height, H_s , of 1.0 m will be exceeded 10% of the time per year. For exceedence percentages of 1% and 0.1% the significant wave height becomes 1.60 m and 2.15 m respectively. The once per year wave height (= 1/1-year return period) can also be determined from Figure 2. If one assumes a storm duration of 6 hours the exceedence percentage becomes $6 \text{ hours} / (365 \text{ days} \times 24 \text{ hours}) \times 100\% = 0.0685\%$ which corresponds with a significant wave height of 2.25 m.

2.2.2 Extreme wave conditions

a) Deep water

Extreme wave conditions for the location of the Ennore Port have been derived from a Cyclone Hindcast Study [15] and involved the statistical evaluation and extrapolation of cyclonic storm records along the east coast of India within the Bay of Bengal. The deep water wave conditions were established at 18 grid points near to Ennore and the average significant wave height and the standard deviation for different return periods were calculated (see Figure 2).

As a check on the results of the extrapolation, the Cyclone Hindcast Study also hindcasted two very severe cyclones along a shifted track approaching the Ennore site. At the peak of these storms a maximum significant wave height of 12 m was calculated.

Comparison of these two very severe events with the hindcasted results, in combination with an evaluation of the return frequency of very severe events occurring at the Ennore site, gives the final estimation of the extreme wave conditions at deep water. The results are slightly higher than those based on hindcast heights only.

The 1/1-year wave height was based on the normal wave conditions and amounts to 2.25 m. Normal wave conditions give a gentle sloping line up to and slightly beyond the 1/1-year wave height. For larger return periods the line steepens sharply as these conditions are generated by cyclones.

b) Shallow water

Before any accurate determination could be made of inshore wave heights, the water levels at various points along the alignment of the breakwater were determined within the Cyclone Hindcast Study. These levels can be summarized as follows:

- + 1.57 m CD for a sea-bed level of - 15 m CD;
- + 1.68 m CD for a sea-bed-level of - 10 m CD;
- + 1.80 m CD for a sea-bed level of - 5 m CD.

From physical scale model testing of breakwaters it is well known that on a very gentle foreshore (1:50 or gentler) the ratio of H_s/h (breaker index) is very close to 0.50 (where H_s is the significant wave height at the local water depth h). This fact is also

proven by the CIRIA/CUR-Manual [1] where graphs are given to establish the maximum significant wave heights on uniform sloping beaches.

The maximum significant wave height at various bottom levels was calculated for the 1/100-year event. The breaker index and corresponding wave height ($H_{s \max}$) varied from 0.64 and 3.1 m at - 3 m CD to 0.50 and 6.6 m at - 11.5 m CD. The wave conditions at -11.5 m CD are shown in Figure 2.

These maximum significant wave heights at various bed levels were used to determine the required primary armour unit weights for the breakwaters at various water depths. However, before these wave heights could be used they needed to be adjusted to allow for modifications in the wave height distribution during breaking.

When waves break on a beach the highest waves break first, thus changing the wave height distribution, which is a so-called Rayleigh distribution at deep water. The stability of structures, and particularly breakwater armour layers, is determined by the highest waves in a sea state. The significant wave height represents the sea state at deep water fairly well however, it does not represent the highest waves. For stability calculations of armour layers the $H_{2\%}$ is a better wave height to use. The ratio of $H_{2\%}/H_s$ is 1.40 for deep water. For severe wave breaking this ratio can decrease to 1.1 to 1.2.

The reduction of the ratio $H_{2\%}/H_s$ has been established for a 1:100 slope by physical model tests and can be described by:

$$\gamma_h = 1 - 0.03 \times \left[4 - \left(\frac{h}{H_s} \right) \right]^2$$

For a water depth of 13 m and a depth limited wave height of 6.6 m the reduction coefficient becomes $\gamma_h = 0.88$. The significant wave height can be reduced by this factor for stability calculations of armour layers. The corresponding $H_{2\%}/H_s$ ratio becomes $0.88 \times 1.40 = 1.23$. For armour weight calculations a minimum ratio of $H_{2\%}/H_s = 1.25$ has been adopted.

2.3 Soil conditions

As part of the Feasibility Study onshore and offshore geotechnical investigations were carried out. The offshore investigation comprised of borings and Cone Penetration Tests (CPT) at seabed levels of - 5 m and - 10 m CD that corresponded with the proposed locations of the berths and breakwaters.

The borings showed a soil profile with an upper layer of coarse dense sand decreasing in depth in a seaward direction. The bottom level of this layer varies between - 6 and -8 m CD which coincides with the seabed level at a distance of 500 to 1000 m from the shoreline. Underlying this sand layer is a loose to very loose silty sand layer with a thickness of approximately 5 m. The loose silty sand layer is exposed on the seabed seaward of the - 6 m to - 8 m CD seabed level.

Soft clay layers of 3 to 5 m thick are present between the shoreline and the - 6 to -8 m CD seabed-level under the loose silty sand layer. Stiff clay and dense cemented

sand layer is encountered under these soft layers, with a top level between – 15 and – 20 m CD.

3.0 PRELIMINARY BREAKWATER DESIGN

3.1 Alternatives

During the feasibility stage of the project the rubble mound breakwater was selected as the most economically and technically sound option for construction of breakwater at Ennore. The main reasons leading to this choice were the wide and relatively cheap availability of rock, the ease of construction and need for the structure to be flexible to account for settlements and consolidation of the poor subsoil conditions.

Quarry investigations [7 & 8] were initiated during the preliminary design phase in order to establish the quarry yield and the maximum size of rock feasible. However, because the results of quarry investigations and the economics of producing large rocks were unknown at the time of the Preliminary Design, it was necessary to consider the use of concrete armour units for the deep water section of the breakwater (seabed level – 9.5 m CD and deeper). The following three concrete armour units were considered:

- Cubes;
- Tetrapods;
- Accropodes.

Cubes are considered to be strong and massive units, which are easy to construct. The stability of the units of the same weight is slightly lower for cubes when compared to Tetrapods. Tetrapods are more slender but have better interlocking, resulting in higher stability compared to Cubes. Both units are placed in a double layer on the outer face of the breakwater. The Accropode is a single layer system developed in the early eighties. The units have good interlocking capabilities and have to be placed on a steep slope of 1:1.33; steeper than that used for Cubes and Tetrapods.

3.2 Armour layer design

3.2.1 Concrete units as primary armour

The program BREAKWAT [2], which was developed on the basis of the results of many physical tests, was used during the Preliminary Design to determine the required mass of the primary armour layer using the design formulae of van der Meer for armour layers with concrete units [12].

The required mass of each type of concrete armour unit was established for the wave conditions occurring at the head of the northern breakwater at a seabed level of CD – 11.5 m (see Figure 3). Based on the 1/100-year design significant wave height (H_s) of 6.6 m (depth limited) associated with a mean wave period of 9.6 seconds, a structure slope of 1:1.5 and a damage level of N_{od} of 0.5, the required mass of each armour unit is:

- 20 tonne for the Cubes;
- 15 tonne for the Tetrapods;
- 15 tonne for the Accropodes.

For shallower water (less than – 11.5 m CD) the unit weights can be reduced. A reduction in the maximum significant wave height gives a similar reduction in the nominal diameter D_n . The weight of 15 tonne can be reduced to 10 tonne at a seabed level of – 9.5 m CD.

Based on a cost comparison of each concrete armour unit, in which two different seabed levels (-11.5 and - 9.5 m CD) were considered, the Accropode alternative required the lowest quantity of rock and proved to be the most economical solution. The Accropode alternative was consequently recommended for further study in next design phase.

It should be appreciated that all the alternatives considered are of an equal degree of stability and from a technical point of view there was no overruling argument to select another alternative.

3.2.2 Rock as primary armour

The program BREAKWAT was also used to design the rock primary armour for the sections of the breakwater with seabed levels shallower than – 9.5 m CD. The input for the calculations consists of maximum wave heights at a certain water depth and the corresponding mean wave period for the 1/100 year return period. The input required to perform the calculations and to represent the results graphically, by means of damage curves for rock armour is very similar to that presented in Figure 3. The formulae of Van der Meer [12] for establishing the rock armour size in shallow water conditions are:

For plunging waves the formula is given as:

$$\frac{H_{2\%}}{\Delta D_{n50}} = 8.7 P^{0.18} \left(\frac{S}{\sqrt{N}} \right)^{0.2} \xi_m^{-0.5}$$

For surging wave the formula is given as:

$$\frac{H_{2\%}}{\Delta D_{n50}} = 1.4 P^{-0.13} \left(\frac{S}{\sqrt{N}} \right) \sqrt{\cotan(\alpha)} \xi_m^p$$

where:	$H_{2\%}$	wave height exceeded by 2% of the waves	[m]
		$H_{2\%} \approx 1.4 H_s$ (Rayleigh distribution)	
	Δ	relative buoyant density ($\frac{\rho_s - \rho_w}{\rho_w}$)	[-]
	D_{n50}	nominal diameter	[m]
	P	permeability	[-]
	S	damage level	[-]
	N	number of waves	[-]

$$\xi_m \quad \text{surf similarity parameter, } \xi_m = \frac{\tan(\alpha)}{\sqrt{\frac{2\pi H_s}{g T_m^2}}} \quad [-]$$

α structure slope [rad]

T_m mean wave period [sec]

The required rock weights were determined at various seabed levels. The Preliminary Design of the breakwaters resulted in the required mass of the primary armour (M_{50}) varying between 3 tonne at – 3.0 m CD to 10 tonne at – 7.5 m CD for structure slopes of 1:1.5.

In addition to convention armour layers (straight slopes) the designers also considered an alternative S-shaped breakwater which comprised a steep upper and lower slope of 1:1.5 and a gentle intermediate slope of 1:4 around the waterline. The Preliminary Design of this alternative gave a required M_{50} of 4 tonne at – 8 m CD and 7 tonne at – 11.5 m CD.

It was finally recommended to adopt the conventional design for armour layers in the next stage of the design because it required the least amount of rock and was more economical when compared to the S-shaped berm.

3.3 Breakwater dimensions

3.3.1 Relation crest height - wave transmission

The required minimum crest height of the breakwater is determined by the allowable wave overtopping and wave transmission. Ennore Port is not considered a survival port, which means that considerable wave transmission could be allowed under severe cyclonic conditions. The acceptable maximum significant wave transmission by overtopping has been established at 2 m and the port facilities have been designed to withstand this wave.

The crest height has been established + 5.0 m CD, resulting in crest freeboards of $R_c = 3.0$ and 3.5 m (with a water level elevation of + 1.5 m). The required crest height was determined using the program BREAKWAT which can calculate the wave transmission for low rock structures with an armour layer of rock on seaside, crest and rear [12]. The wave transmission application of BREAKWAT is based on many physical tests executed at Delft Hydraulics. Calculations were made with 15 tonne rock (the same nominal diameter as Tetrapods and Accropode units) and with a crest width of 12 m.

3.3.2 Crest width

The crest width is normally determined by functional requirements (road/crown wall on top), hydraulic requirements (minimum 3 to 4 stones on the crest) or construction methods used (access on the core by trucks or cranes).

The breakwaters at Ennore were to be constructed with a concrete crown wall/road to facilitate future maintenance and it was envisaged that the breakwater would be constructed by a combination of both marine and land based equipment. The

requirement for the maintenance road on the breakwater at + 4 m CD was 9 m and this resulted in a core width at + 3 m CD in the order of some 6 - 8 m which would facilitate 2-way access on the core for cranes and trucks during construction.

3.3.3 Secondary armour

The secondary armour layers are designed as 1/10 to 1/15 of the mass of the primary armour layer [3]. A further consideration given to the required rock size was the normal wave conditions to be expected during construction because the secondary armour layers would be exposed for some time before being covered by the primary armour. The layer thickness was determined by applying a $2 \times D_{50}$ thickness and a reduction factor of 0.8.

3.3.4 Toe berm

The toe berm is the lower support for the armour rock. The stability of the toe berm has been checked for different design storm water levels because the crest of the toe will be either submerged or above the water level.

For conditions where the toe is above the design water level the rock stability has been determined using BREAKWAT for a statically stable structure with non-overtopping. A reduction factor is then applied for overtopping considering a low crested structure.

For conditions where the toe is submerged in relation to the design water level, rock stability has been assessed using the results of recent research of Delft Hydraulics [12] and the Danish Hydraulic Institute:

$$\frac{H_s}{\Delta \times D_{n50}} = 8.7 \times \left(\frac{h_t}{h} \right)^{1.40}$$

where: H_s	=	significant wave height	[m]
Δ	=	buoyant mass density of rock	[kg/m ³]
D_{n50}	=	nominal rock size	[m]
h_t	=	depth of toe	[m]
h	=	water depth	[m]

The selected rock grading for the toe varies from 0.3 – 1.0 tonne from the shoreline to a bed-levels of – 4 m CD, 0.5 – 2.5 tonne between – 5 and – 7 m CD and 1.0 – 5.0 tonne from the – 7 m CD to the head of the breakwater for the northern breakwater.

3.3.5 Filter

The granular filter under the toe berm of the breakwaters on the seaside and under the primary/secondary armour on the portside of both breakwater was designed as a 1.5 m thick layer of well graded material, e.g. 25 % of 2 - 10 kg and 75 % of 10 - 50 kg.

3.3.6 Head

The design of the head requires special attention as the sphericity of the head leads to a reduced interlocking of the rock/armour units. The detailing of the head of the breakwater was carried out at the beginning of the Final Design.

3.4 Geotechnical aspects

The geotechnical stability of the breakwater was analysed during the Preliminary Design Phase and was based on the limited offshore soil information collected in the framework of the Feasibility Study. The poor subsoil conditions led the Designers to concluding that special attention would be required with respect to the stability, settlements and erosion of the loose sandy soil in front of and beneath the breakwater.

3.4.1 Stability

Slip circle calculations showed that stability of the subsoil against slope failure was less than is normally acceptable for this type of structure. Considering the poor strength of the subsoil it was decided to identify soil improvement measures that could be taken to improve the soil conditions:

- 1) Remove the loose and soft material and replace by better material;
- 2) Vertical drains;
- 3) Apply a very strong geotextile on the seabed;
- 4) Design a breakwater with flatter slopes;
- 5) Raise the breakwater in stages.

Based on an assessment of the above improvement measures for the conditions prevailing at Ennore it was concluded that a phased implementation consisting of pre-loading the subsoil appeared to be the most practical and economical solution. The method would involve placing the lower part of the breakwater by floating equipment about one year prior to placing the upper part.

3.4.2 Settlement

The subsoil conditions at Ennore indicated areas of loose sand and soft clay which, if not removed, would result in settlement of the breakwater during and after construction. Based on the information available at this stage of the project the total settlement was estimated to be between 0.45 and 0.75 m. An additional quantity of rock for compensating for this settlement was estimated to be in the region of 100,000 m³.

3.4.3 Erosion of bed material

Erosion of subsoil material from beneath the breakwater and ultimately settlement of the breakwater might happen if the filter properties of the base material of the breakwater are unable to prevent the loose underlying sand from washing out under wave and current action. In addition, erosion in front of the breakwater (often termed scouring

[17]) can undercut the toe berm, which could result in a slip failure of the toe berm or much worse a flow slide of any loose sand encountered adjacent to this area.

It was finally decided to adopt one granular filter (1 - 50 kg) under the breakwater to provide a filter between the core material of the breakwater (1 – 1000 kg) and the underlying sand. In addition, the length of the filter was extended seawards of the breakwater in order to keep scouring away from the toe berm.

4.0 FINAL BREAKWATER DESIGN

4.1 Detailed Investigations

Four extensive investigations were carried out to assist in the process of detailing and fine-tuning the design of the breakwater:

- 3-dimensional physical modeling;
- near shore wave heights studies;
- wave penetration studies;
- offshore soil investigation.

4.1.1 3-D physical model

The 3-D physical model test [9] was carried out in September and October 1995 in the laboratories of Delft Hydraulics, located in the Voorst, the Netherlands. The head and an adjacent section of the trunk of the breakwater were tested with a model built at a scale of 1:35. The basin is equipment equipped with a wave generator that can produce random distributions of waves following a specified energy-density spectrum. The test was executed in two series, the first (Series A) with the model constructed according to the Preliminary Design and the second (Series B) with some adaptations to the model, established on the basis of results of the Series A (see Photographs 1 and 2).

The objective of the test was to confirm the applied design philosophy and verify the validity of the design formulae used in the Preliminary Design [6] with respect to wave transmission/overtopping and the stability of the primary armour and toe berm. The results of the test confirmed that the Preliminary Design concept with Accropodes was a suitable solution and that the results could be used to finalize the design.

4.1.2 Near shore wave height and wave penetration studies

The near shore wave height study and the wave penetration study were carried out by Delft Hydraulic using their mathematical models HISWA-2D and PHAROS [4] respectively. Both studies provided detailed information of the wave heights to be expected on the sea and port sides of the breakwaters. The results of these tests indicated that the design criteria for determining the primary rock armour on the head and trunk of the southern breakwater could be better defined.

4.1.3 Offshore soil investigation

An extensive offshore soil investigation [5] was executed by Hydro Soil Services from Belgium using a jack-up barge. Work started in January 1995 and was completed in May of the same year. Boreholes and cone penetration tests (CPT) were executed together with extensive disturbed and undisturbed sampling. The results of the laboratory testing on the soil samples collected during the investigation became available in July 1995. The results of these investigations provided the basis upon which a comprehensive geotechnical design comprising aspects of stability and settlement of the subsoil was carried out.

4.2 Modifications to the Preliminary Design

The following modifications to the Preliminary Design were incorporated in the Final Design based on the outcome of the physical model tests. Examples of the final breakwater cross section for the northern breakwater are presented in Figure 4.

4.2.1 General Modifications

- The first filter layer under the seaside toe was reduced from a thickness of 1.5 m to 1.0 m. The second filter layer was too thin for practical construction purposes and as a result its thickness was increased from 0.5 m to 1.0 m. The total thickness of both filter layers remained 2 m. The grading of the first filter layer was changed to between 10 cm and 100 kg. The second layer was changed to 100-500 kg and provides an improved fit to the grading of the overlying toe. The toe itself remained unchanged because it proved to be stable for all tested conditions.
- On the basis of engineering judgement, the layer thickness coefficient of 0.8 that was adopted in the Preliminary Design was increased to 0.9 in order to facilitate construction. As a result all rock layers in the breakwater which were $2 \times D_n$ thick, were increased in thickness.
- The model tests with the toe elevated by 1 m showed that the toe (and the rest of the breakwater) remained stable. Consequently, elevating the toe by 1 m above the original seabed by backfilling with sand was acceptable but had to be limited to a 1 m thick layer at the location of the seaside toe.
- The first filter layer under the head of the breakwater was extended to prevent erosion of the bed material near to the head of the breakwater. On the head of the northern breakwater, the filter layer was extended an additional 10 m outside the profile with a thickness of 0.5 m. On the head of the southern breakwater, the extension was 5 m with a layer thickness of 0.5 m.

4.2.2 Modifications to Accropode primary armour

- Accropode armour units of 15 tonne are acceptable for the head of the northern breakwater provided an increased head radius of 20 m is adopted.
- Accropode armour units with a mass of 12 tonne (layer thickness 2.20 m) will be stable at the deepest part of the breakwater. The mass of the Accropode could also

be reduced to 9.6 tonne (layer thickness 2.05 m) as from the transition of the strait section, almost parallel to the entrance channel, to the curved section.

- Raising of the crest at the head from an elevation of + 5.40 m CD to + 7.40 m CD (as proposed and tested) was not been adopted in the Final Design because it was believed that the increase in the radius has by far the largest positive effect on the stability. Lower breakwaters are in principle not less stable than higher ones, mainly the rear side is more heavily attacked however, Accropodes are placed all around the head. Raising of the head will decrease overtopping and wave transmission, but as only the Entrance Channel, and not the Basin would benefit from a higher crest, this was not incorporated into the Final Design.

4.2.3 Modifications to rock primary armour

- The southern breakwater is protected from easterly wave directions by the northern breakwater. The most critical directions for the design conditions are from a bearing of 110° to 150°. The near shore wave height study [4] showed these directions changed at the structure to 114° to 131° due to refraction. The head of the southern breakwater is more protected from these directions than the section from the head to – 8 m CD seabed level.
- Based on the wave penetration studies [4] it was concluded that the head of the southern breakwater itself could be designed with a wave height, which is 60% of that at the head of the northern breakwater. It was also concluded that a section of the trunk between 200-300 m from the head of the southern breakwater could be designed with 75% of the design wave height at the head of the northern breakwater.
- For the sections of the breakwaters where the direction of the design waves are very oblique to the structure a different approach was adopted. Recent research has shown that rock stability increases when waves become more oblique (> 45- 60 degrees) to the structure. This means that calculations using the formula of Van der Meer for parts of the rock structure subjected to oblique waves are conservative as they consider perpendicular wave attack. Based on research work carried out in the framework of MAST, funded by the European Community [11] the following formulae was adopted for determining the required weight of rock for the sections of the breakwater subjected to oblique waves:

$$D_{n50} = \frac{H_s}{\Delta * 1.20 * 2}$$

where:

H_s	=	significant wave height	[m]
Δ	=	relative buoyant density of material	[-]
1.20	=	ratio $H_{2\%}/H_s$	[-]
2.00	=	stability number	[-]

- In addition to the effects of oblique waves, one should also consider the effects of wave overtopping on the required rock weight. When the crest of the trunk is relatively low a part of the wave energy will pass over the top of the structure. In these situations the required armour mass can be reduced in accordance with the amount of

wave overtopping. The reduction factor [12] is a function of the wave steepness and the relative crest height R_c/H_s . The reduction factor for low crested structures has been calculated with BREAKWAT.

4.2.4 Modifications to the Geotechnical design

Stability

Based on the detailed subsoil data which was obtained during this phase of the Project, the Designers identified two alternative solutions to improve the soil strength and hence the stability of the breakwater:

1. Vertical drains in combination with geotextiles;
2. Removal of soft material, backfilling with sand and compaction.

After detailed geotechnical studies both alternatives proved to be technically feasible however, the second option proved to be more economical. Furthermore, the second alternative gave additional advantages, the main one being that pre-dredging, back-filling and compacting involved less risks in achieving the required accuracy and reliability during construction.

The additional subsoil information which was collected during the major offshore soil investigation [5] in the first half of 1995 led to the recommendation that soil improvement was only required over the deeper part of both breakwaters.

A suitable borrow area for sand backfill was initially identified within the Port Basin which comprised fine sand. Based on this type of sand it was decided that compaction of the backfill material would be required by means of vibro-flotation or dynamic compaction and this would be controlled by static cone penetration tests.

During the latter stages of the Final Design a more suitable borrow area was identified some 10 km south of the Project area in a water depth of 20 m. The sand originating from this new location was relatively coarse ($D_{50} = 850$ microns) and free of fines. As a result of the improved characteristics of the sand backfill the compaction requirements were revised and compaction was not required.

Raising of the seabed

With the aim of economising on the quantity of rock required for the breakwater construction and counteracting soil settlements, the designers studied the possibility of depositing additional sand back-fill in order to raise the seabed level along the alignment of the soil improvement.

During the 3-dimensional physical model test the stability of the toe was checked against the raising of the seabed level. It was concluded that the maximum level by which the seabed could be raised would be 1.0 m. Based on the foregoing, it was decided to place back-fill to a height of 0.5 m above the original seabed level plus a variable additional height based on the settlement expected to occur during construction.

Settlement

Settlement of the sand fill and consolidation of the sub-soil were analysed for various cross-sections of both the northern and southern breakwaters. The results of the analyses provided essential information about the level to which the breakwaters needed to be constructed in order to compensate for short and long term settlements.

The results of these analyses showed that settlements under the crest and toe berm would be in the order of 0.60 – 1.00 m and 0.25 – 0.45 m respectively for the northern breakwater and 0.20 – 0.30 m and 0.45 – 0.70 m for the southern breakwater. Construction and post construction settlements were estimated to be 40% and 60% of the total settlement respectively.

It is interesting to note that due to the soil improvement recommended above the settlements of the breakwaters were reduced to 40 to 50% of the values predicted for the situation without improvement.

5.0 LIST OF REFERENCES

'Manual for the use of rock in coastal and shoreline engineering', CIRIA Special publication 83 / CUR Report 154, 1991.

Breakwat Program, Copyright Delft Hydraulics, 1990

'Shore Protection Manual' Volume I & II, US Army Coastal Engineering Research Centre, 1984

'Ennore Coal Port Project - Report on PHAROS and HISWA-2D calculations', Delft Hydraulics, December 1995

'Ennore Coal Port Project - Offshore Soil investigations - Final Report (Field investigation and laboratory test)', Hydro Soil Services, September 1995

'Ennore Coal Port Project - Preliminary Design Report', HASKONING Royal Dutch Consulting Engineers and Architects, April 1995

'Quarry Investigations Karikkal - Reports on Trial Blasting, Laboratory Tests. Core Drilling', Larsen and Toubro, August 1995

'Ennore Coal Port Project - Rock for Breakwater Construction, Rock Quality and Quarry Yield Analysis - Karikkal Quarry', HASKONING Royal Dutch Consulting Engineers and Architects, October 1995

'Ennore Coal Port Project - Breakwater Model Investigation, Three Dimensional Model Tests', Delft Hydraulics, January 1996

'Environmental study Construction of Ennore Port', MTI - Institute for Environmental Technology, the Netherlands, in association with Mantec Consultants, India, August 1995

'Rubble mound breakwater stability under oblique wave attack', J.C. Galland (EDF, France), MAST G6-S Coastal Structures, Proceedings of final workshop, Lisbon, November 1992

'Conceptual design of rubble mound breakwaters', J. v.d. Meer, Delft Hydraulics, December 1993.

'Ennore Coal Port Project, Risk Analysis and Probabilistic Modelling, Final Report', Delft University of Technology, December 1995.

'Ennore Coal Port Project, Global Hindcast Study', Delft Hydraulics, December 1994.

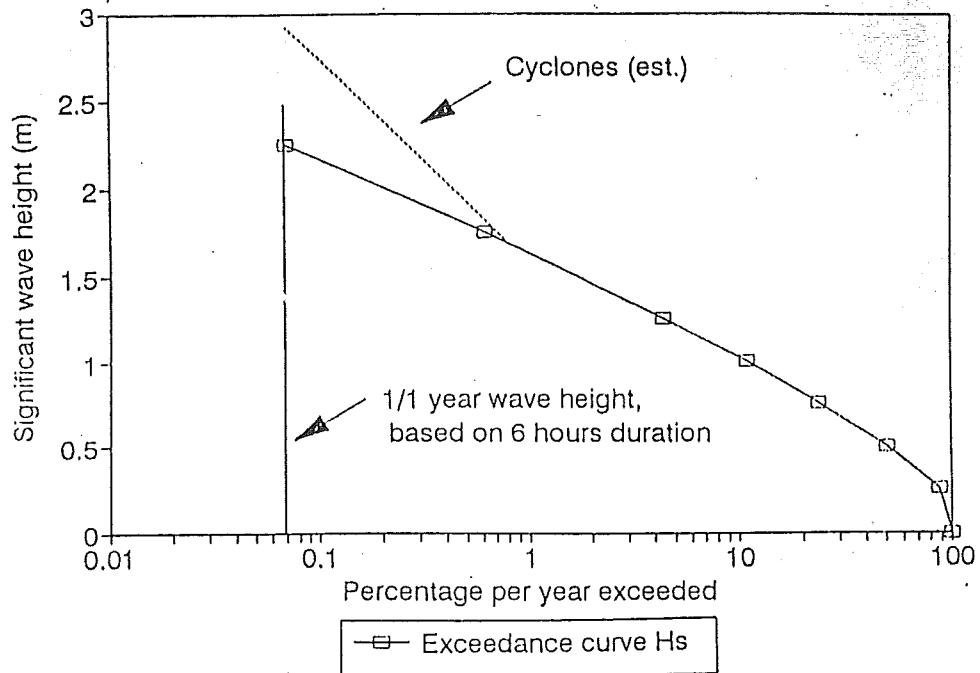
'Ennore Coal Port Project, Cyclone Hindcast Study', Delft Hydraulics, February 1995.

'On the comparison between the threshold of sediment motion under waves and unidirectional currents with a discussion of the practical evaluation of the threshold', P.D. Komar and M.C. Miller, *Journal Sedimentary Petrol*, 1975

'Scour around the head of a vertical wall breakwater', T. Gokce, B.M. Sumer and J. Fredsoe, *HydroPort '94*, International Conference on Hydro-Technical Engineering for Port and Harbor Construction, 1994.

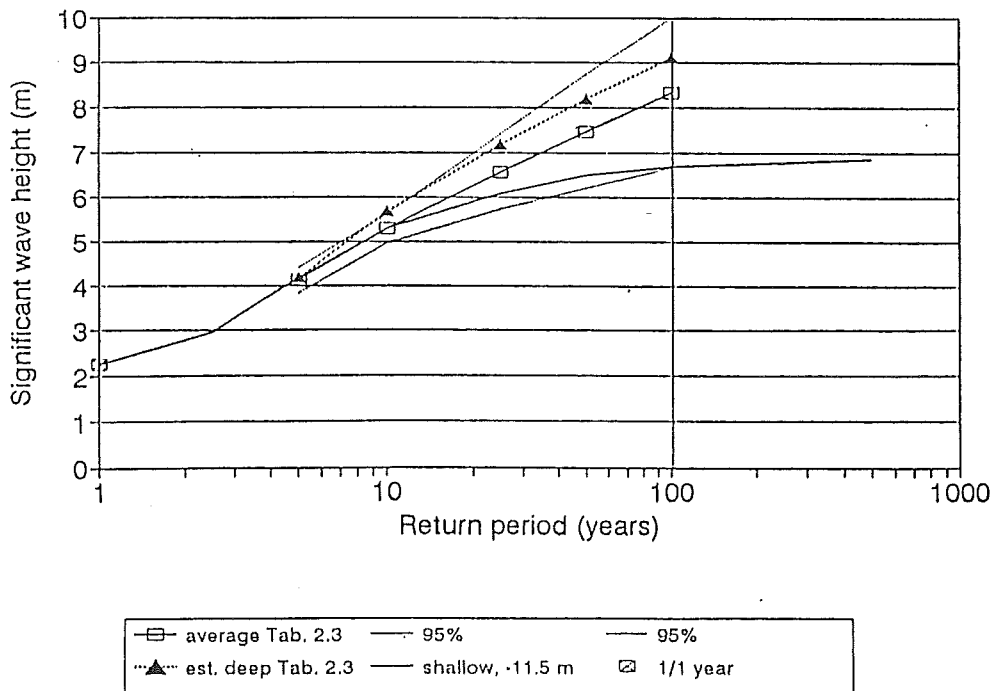
'Wave Runup and Overtopping at Dikes', J.W. van der Meer and J.P.F.M. Janssen, included in ASCE publication 'Wave Forces on Inclined and Vertical Wall Structures', edited by N. Kobayashi and Z. Demirebilek, 1995.

NORMAL WAVE CONDITION



EXTREME WAVE CONDITION

Ref: Delft Hydraulics H1974, Jan. '95



Chennai Port Trust
Ennore Coal Port Project

NORMAL AND EXTREME
WAVE CONDITIONS AT ENNORE



HASKONING
Consulting Engineers
and Architects

in association with

COE
of India

date	drawn	revision	checked

SCALE

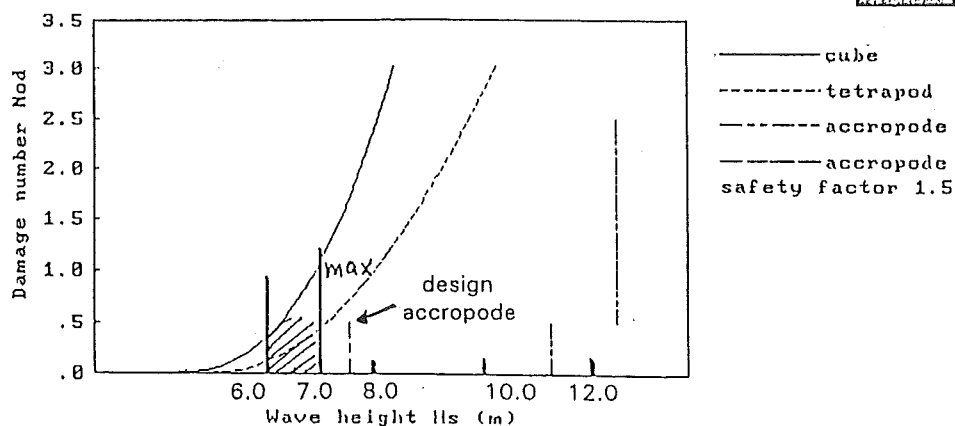
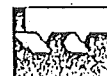
A4-size

Project No 9019.A5528.21
DRWG. No

Fig. 2

06:39 AM Thursday 26 January 1995. program = BREAKWAT Version 2.01
 Licensed user: for internal use only Copyright DELFT HYDRAULICS 1990

Damage curves concrete units



For $H_{2\%}/H_s = 1.25$

Stability curves concrete units:

Combined graph: Cubes, Tetrapods and Accropode

Input :

mass of unit : M = 20000.000 (kg)
 mass density of unit : rho-a = 2400 (kg/m³)
 mass density of water : rho-w = 1025 (kg/m³)
 wave steepness : sm = .040 (-)
 number of waves : N = 2500 (-)

Chennai Port Trust
 Ennore Coal Port Project



HASKONING
 Consulting Engineers
 and Architects

in association with

COUBS
 of India

DAMAGE CURVE FOR 20 TONNE ACCROPODE

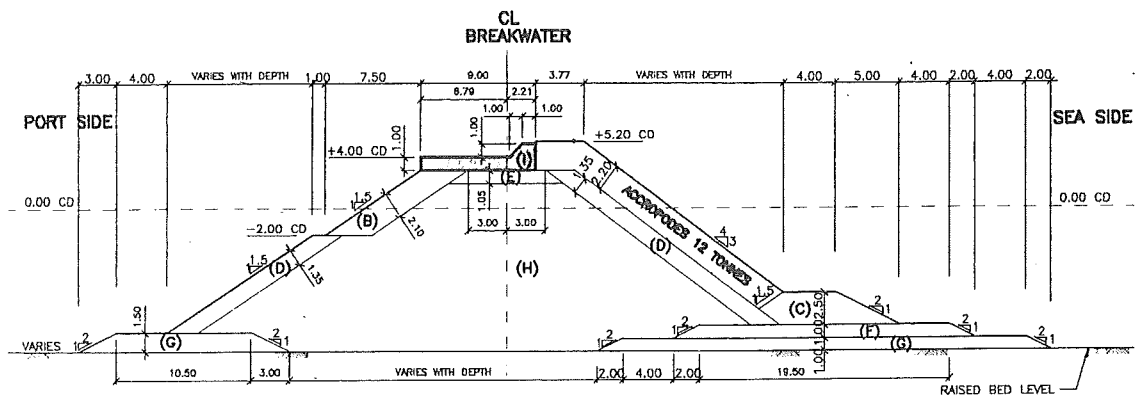
date	drawn	revision	checked

SCALE

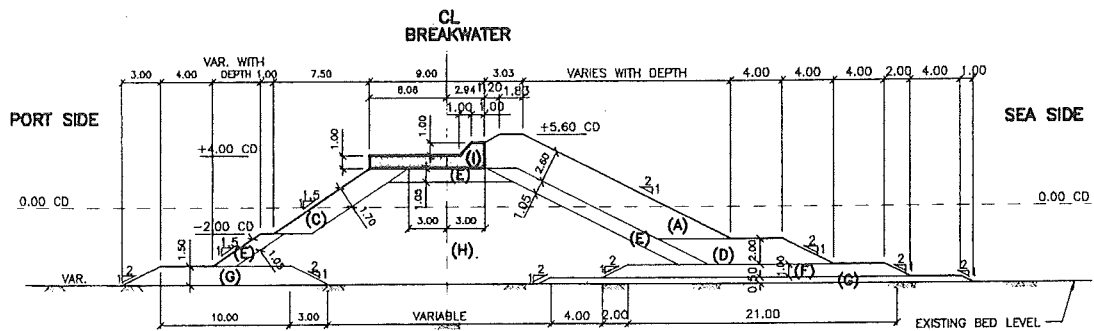
AA-size

Project No 9019.A5528.21
 DRWG. No

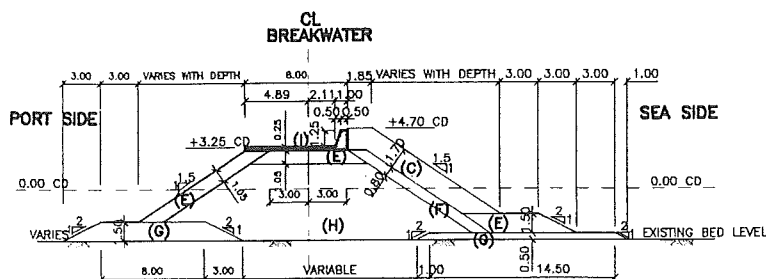
Fig. 3



TYPICAL CROSS SECTION OF NORTH BREAKWATER
FOR TRUNK SECTION FROM HEAD TO -10.40 CD (EXISTING BED LEVEL)
CROSS SECTION A-A



TYPICAL CROSS SECTION OF NORTH BREAKWATER
FOR TRUNK SECTION FROM -7.00 CD TO -5.00 CD (EXISTING BED LEVEL)
CROSS SECTION I-I



TYPICAL CROSS SECTION OF NORTH BREAKWATER
FOR TRUNK SECTION FROM -4.00 CD TO -3.00 CD (EXISTING BED LEVEL)
CROSS SECTION F-F

LEGEND:

- (A) ROCK, 5.0-12.0 TONNE
- (B) ROCK, 2.5-7.0 TONNE
- (C) ROCK, 1.0-5.0 TONNE
- (D) ROCK, 0.5-2.5 TONNE
- (E) ROCK, 0.3-1.0 TONNE
- (F) ROCK, 100-500 Kg
- (G) ROCK, 1-50 Kg
- (H) QUARRY RUN, 1-1000 Kg
- (I) CONCRETE CREST

CL = CENTER LINE

NOTES :

1. ALL DIMENSIONS AND LEVELS ARE IN METRES
2. LEVELS ARE RELATED TO CHART DATUM
3. THIS DRAWING TO BE READ IN CONJUNCTION WITH DWG. No. C4-301 AND C4-302

Chennai Port Trust
Ennore Coal Port Project



HASKONING
Consulting Engineers
and Architects

in association with

ROUTES

of India

**NORTH BREAKWATER
CROSS SECTIONS**

date	drawn	revision	checked

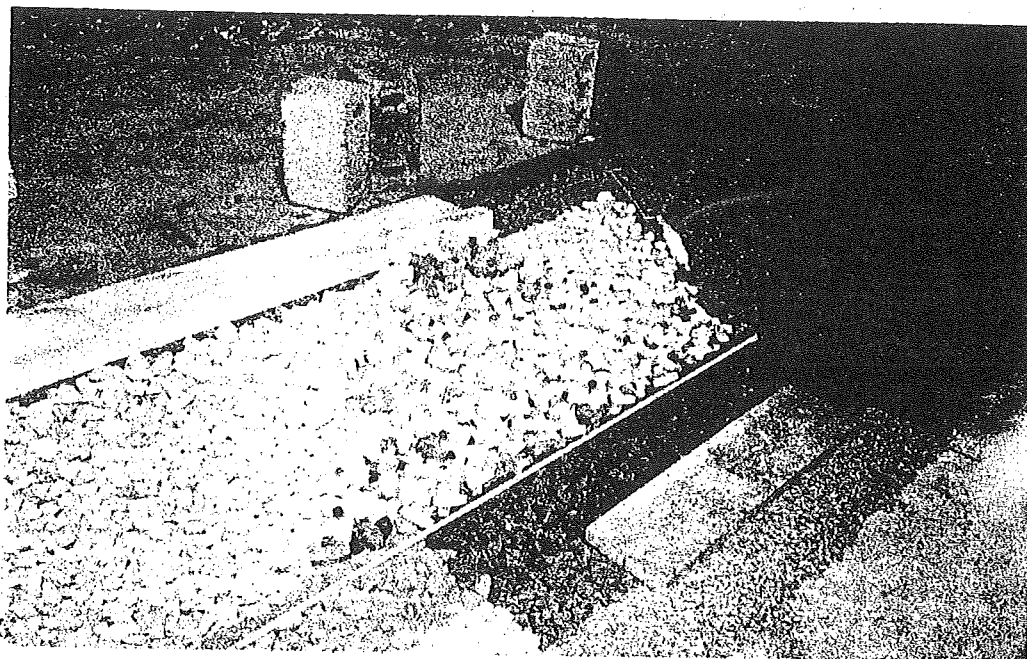
SCALE 1:500

A4-size

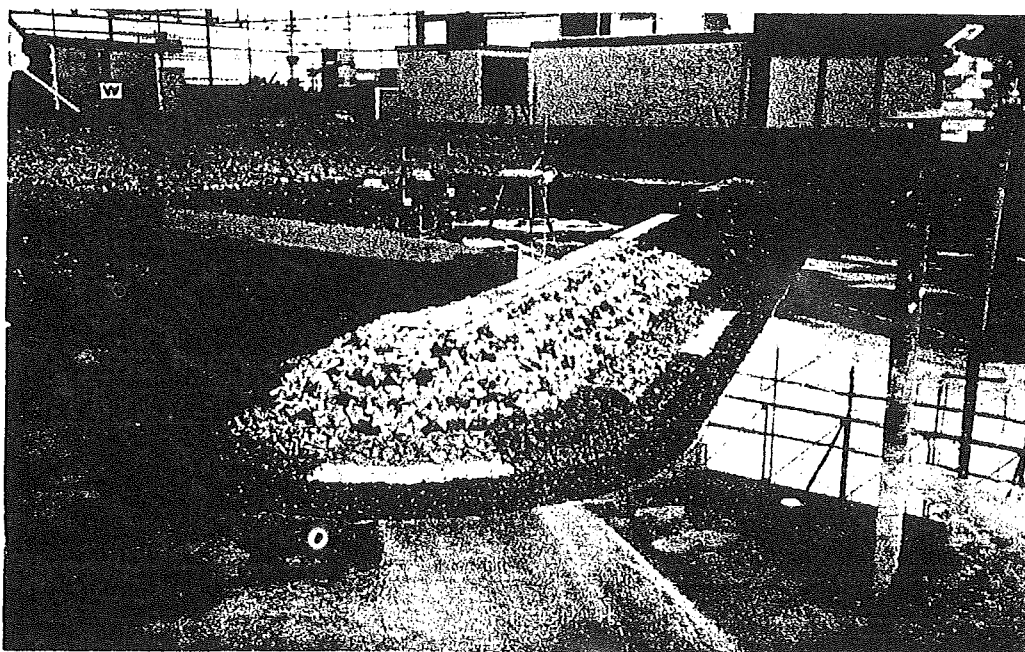
Project No 9019.A5528.21

DRWG. No

Fig. 4



Photograph 1 Placing of Accropodes on trunk of model according, to placing pattern advised by Sogreah



Photograph 2 Breakwater model, series A, before test

Chennai Port Trust
Ennore Coal Port Project



HASKONING
Consulting Engineers
and Architects

in association with

ROUTE
of India

date	drawn	revision	checked

SCALE

A4-size

Project No 9019.A5528.21

DRWG. No

Photographs 1 & 2



175

176



CONSTRUCTION OF BREAKWATERS FOR ENNORE PORT

S. Pearson
Haskoning
Chennai
Email: hasko_ch@satyam.net.in

MAIN ELEMENTS OF BREAKWATER CONSTRUCTION

- I) Rock Quarry, Transport and Stockpile at Ennore.
- II) Soil Improvement beneath breakwaters.
- III) Place rock in Breakwater by sea and land.
- IV) Accropodes as primary armour.
- V) Concrete crest and wave walls.
- VI) Upper layers of Armour and Accropodes behind wave walls.

I. Rock Quarry, Transport and Stockpile at Ennore

The quarry, of Gneissic Granite, was located 120 km west of Ennore. The quarry Contractor, produced 3 million tonnes of various grades of rocks from September 1996 to November 1999, as below.

Type	grading	tonnes
A	5.0 to 12.0 t	68,000
B	2.5 to 7.0 t	120,000
C	1.0 to 5.0 t	166,000
D	0.5 to 2.5 t	166,000
E	300 to 1000 kg	235,000
F	100 to 500 kg	245,000
G	1.0 to 50 kg	508,000
H (Quarry Run)	1 to 1000 kg	1,655,000

The rock was loaded into skips which for the first 20 km were trucked to a rail-head at Melpakkam. The skips were then loaded by gantry onto rakes of wagons, and

taken by rail, for a further 100 km, to the Ennore Site, where the rocks were stockpiled. Fig. 1 shows the layout of breakwaters and rock stockpile for Ennore coal port project.

II. Soil Improvement beneath Breakwaters

Over parts of each breakwater where the sea bed material was deemed to be unsuitable it was dredged out and deposited off shore. The resultant trench was backfilled to just above original seabed level with suitable sand dredged from a source on the seabed S.E. of Ennore. Fig. 2 shows the location of areas to be dredged and backfilled.

Typical Grading of dredged sand backfill

Sieve Size mm	% Passing
2	94
1	54
0.500	10
0.250	2.5
0.125	1.0
0.063	0
0.020	0

$D(20\%) = 0.62\text{mm} > 0.300\text{mm}$ specified requirement.

Dredging and backfill was carried out by 2 trailer suction dredgers "Volvox Hansa" (bottom doors), capacity 8000t, and "Orwell"(split), capacity 2500t, from 16 March 1998 to 23 August 1998 and involved the dredging of approx. 2.5 million cu metres to excavate the trenches and a similar amount to refill them.

The dredgers positions during operations were controlled by D.G.P.S. (Differential Global Positioning Systems).

Sea bed surveys were carried out before, during, and after operations by "Kamal" the survey vessel, also with D.G.P.S. and Atlas Deso 15 echo-sounder.

III. Rock Placement

The placement of rock was done in two separate operators.

a) By marine operations from 4 July 1998 to 20 April 1999, using the side stone dump vessel "Frans", capacity 1000t, to place the rock from seabed up to - 4 CD level: approx. 1,200,000 tonnes. Fig. 3 shows S.S.D.V. "Frans" for diagram of vessel.

Surveys before during and after rock placement by the "Frans" were carried out by the "Kamal" survey vessel with D.G.P.S and echo-sounder.

b) The rest of the rock, approx. 1,400,000 tonnes is placed by land based operations using dump trucks, cranes, excavators etc, this started April 1998 and is still in progress.

Total (marine & land) rock placement is as Table below:

Type	grading	tonnes
A	5	57,000
B	2.5 to 7.0t	76,000
C	1.0 to 5.0t	130,000
D	0.5 to 2.5 t	110,000
E	300 to 1000 kg	147,000
F	100 to 500 kg	145,000
G	1.0 to 50 kg	298,000
H (Quarry Run)	1.0 to 1000 kg	1,569,000

IV. Accropodes

On the sea side of the North Breakwater for almost 2 km Accropodes of 4,5, and 6.3m³ are used as primary armour. Fig. 4 shows the view of stored accropodes.

The Accropodes are of unreinforced concrete cast in special moulds, on site. Concrete used is M25 with 50/50; GGBFS/OPC ratio and low w/c ratio.

4360 (4m³), 3200 (5m³) and 610 (6.3m³) Accropodes are cast using 9 moulds for the 4m³, 6 moulds for the 5m³ and 2 moulds for the 6.3m³. Casting started in June 1998 and is still in progress.

The Accropodes are placed using a 100t crane with an "Ascorel" positioning system. The crane operator is able to exactly position each Accropode with the assistance of the computer controlled "Ascorel" system using the slew angle of the crane and the angle of elevation of the boom, relative to the known position of the crane.

V. Crest Concrete and Wave Walls

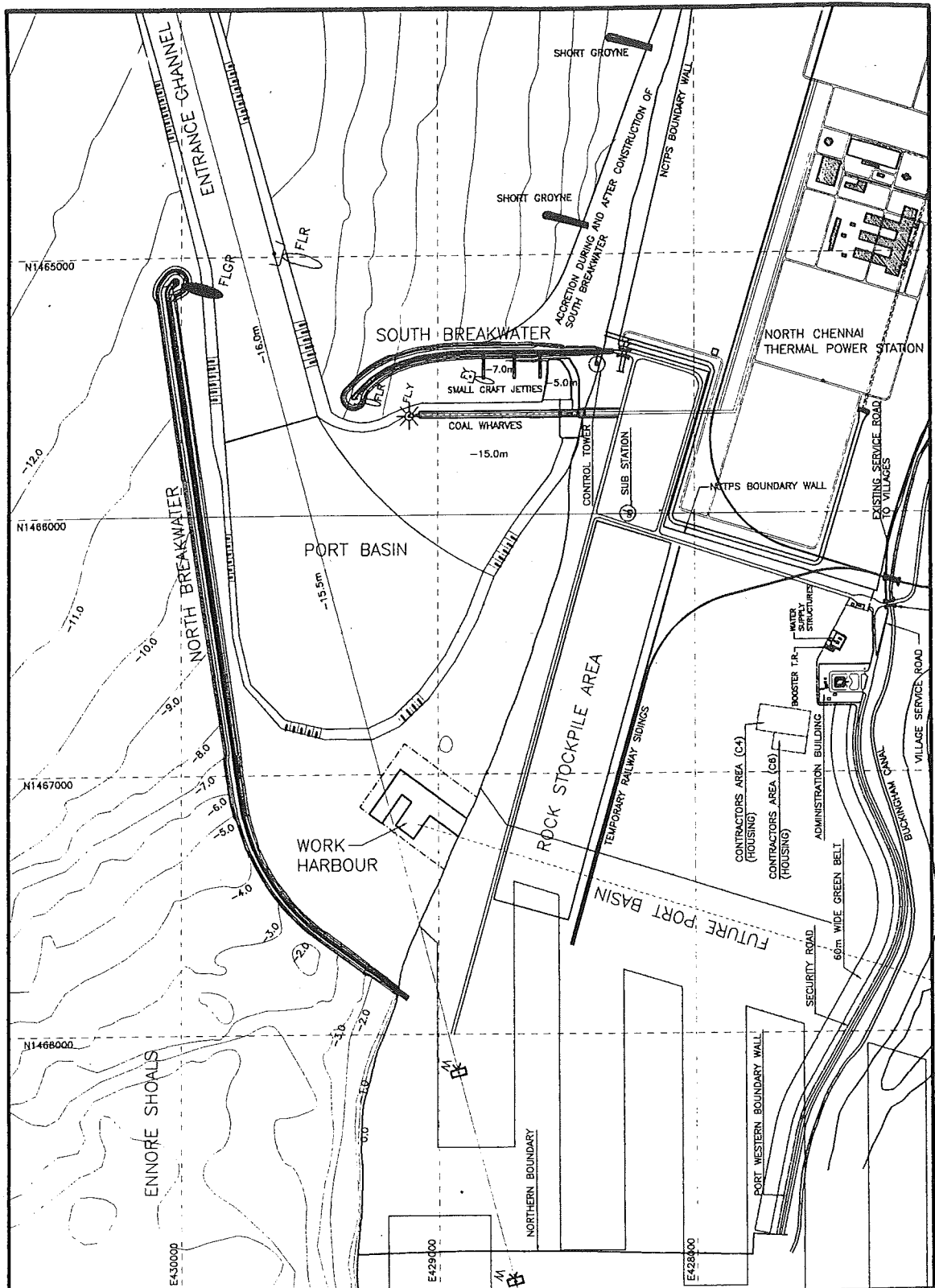
The top of the rock breakwaters are capped off with either a 1000mm unreinforced slab, in deeper water, or a 250mm reinforced slab, in shallower water, with a wave wall on the seaside.

The Concrete for thfse crest was M.30 using GGBFS/OPC at 66/34 ratio and a low w/c ratio with a retarder plasticizer additive. Concrete is batched on site and transported in transit-mixers, quantities required as below.

NBW		SBW	
<u>Reinforced m³</u>	<u>Unreinforced m³</u>	<u>Reinforced m³</u>	<u>Unreinforced m³</u>
1900	22000	500	8000

VI. Upper layers of Armour and Accropodes behind wave wall

After the construction of the concrete crest and wave wall the upper layers of rock armour and Accropodes are placed, by crane, behind the wave wall.



Chennai Port Trust
Ennore Coal Port Project

LAYOUT OF BREAKWATERS AND
ROCK STOCKPILE



HASKONING
Consulting Engineers
and Architects

in association with

OTIS
of India

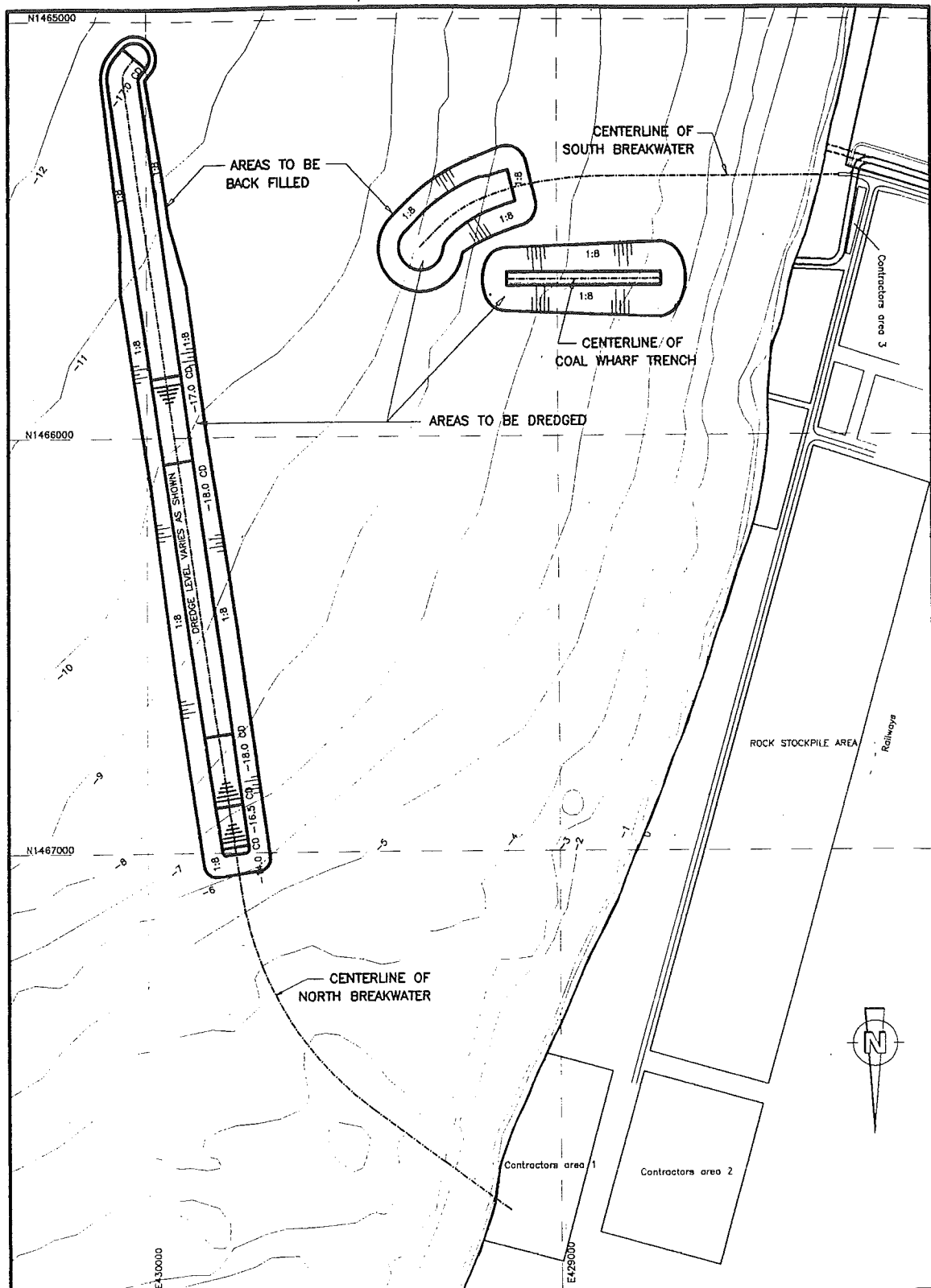
date	drawn	revision	checked
28-01-2000	R.K.Reddy		

SCALE 1:20000

A4-size

Project No 9019.A5528.21
DRWG. No CSC4-A4Layout

Fig. 1



Chennai Port Trust
Ennore Coal Port Project



HASKONING
Consulting Engineers
and Architects

in association with

CRIL
of India

LOCATION OF AREAS TO BE DREDGED
AND BACK FILLED

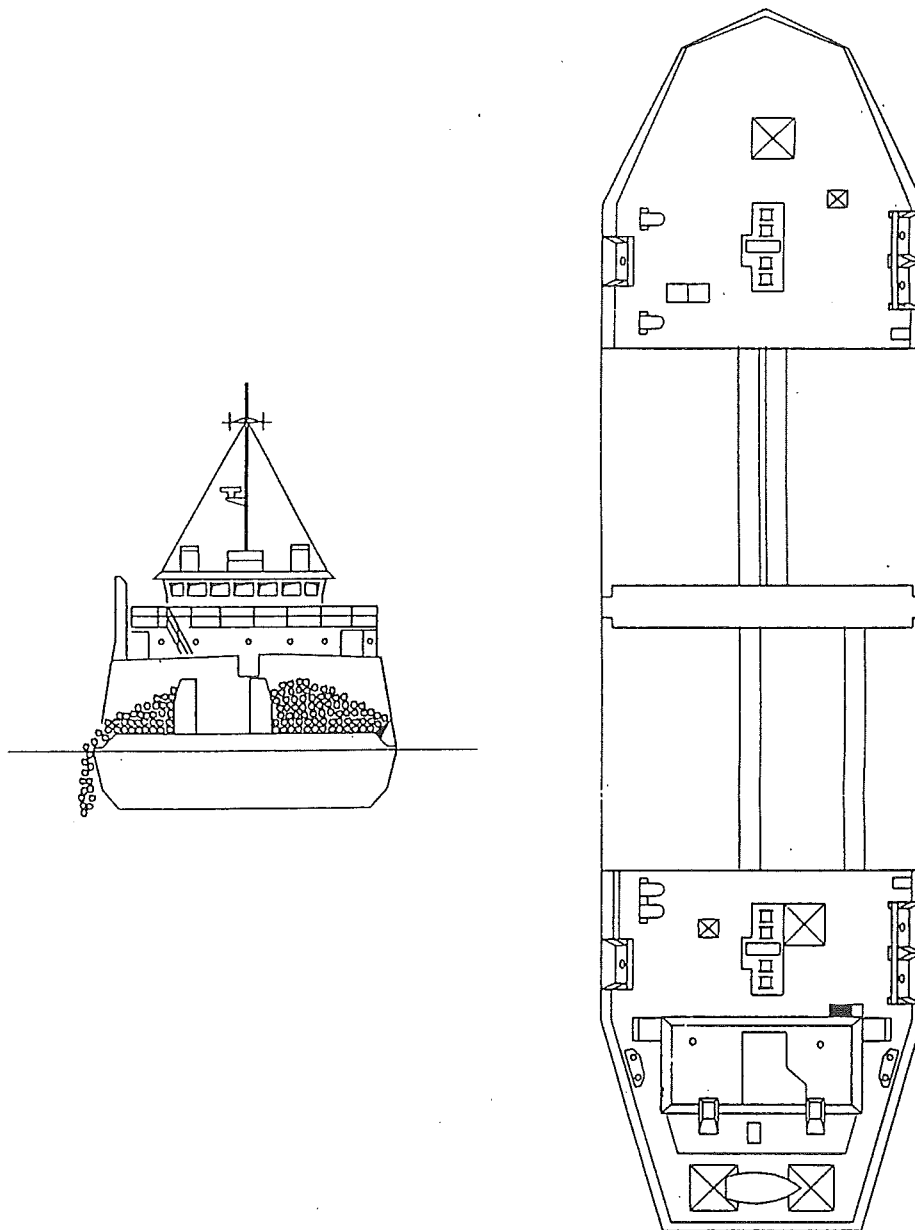
date	drawn	revision	checked
31-01-2000	R.K.Reddy		

SCALE 1:12500

A4-size

Project No 9019.A5528.21
DRWG. No CS-201A4

Fig. 2



Chennai Port Trust
Ennore Coal Port Project

SIDE STONE DUMP VESSEL 'FRANS'

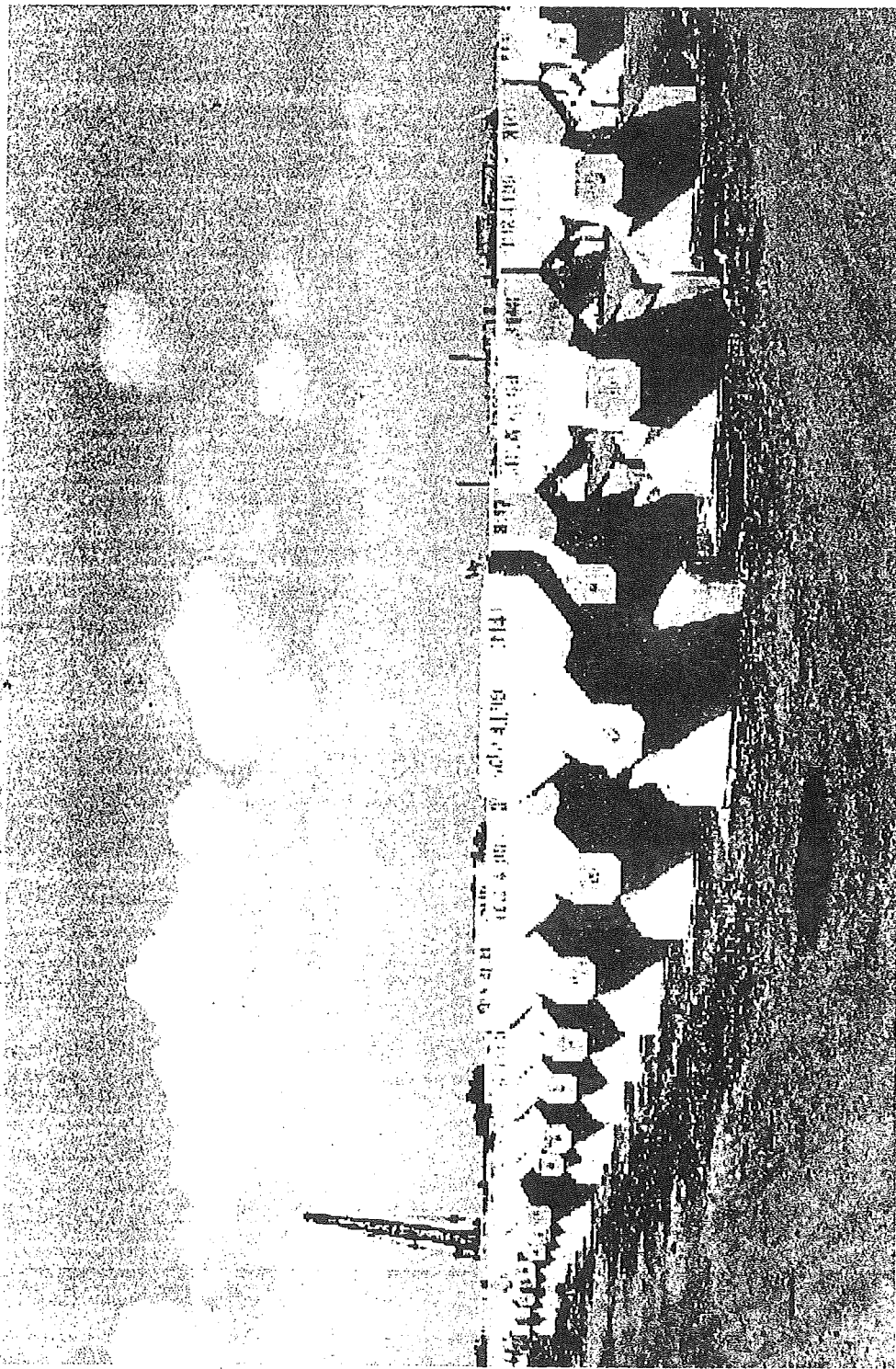


HASKONING
Consulting Engineers
and Architects

In association with

COOTERS
of India

date	drawn	revision	checked	SCALE	A4-size
				Project No 9019.A5528.21	
				DRWG. No C4-CS01	
				Fig. 3	



Chennai Port Trust
Ennore Coal Port Project

VIEW OF STORED ACCROPODES



HASKONING
Consulting Engineers
and Architects

In association with

COYATES
of India

date	drawn	revision	checked
02-02-2000	R.K.Reddy		

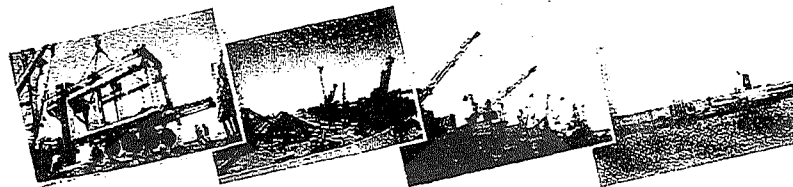
SCALE	A4-size
Project No 6018.A5828.21	
DRWG. No C4-CS02	

Fig. 4

PROFILE OF CO-SPONSORS

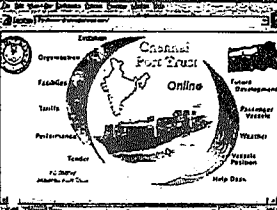
A multi-dimensional port marching ahead

- ↓ A unique Port with 24 alongside berths with facilities for handling all types of cargo.
- ↓ Chennai Port was the first to construct modern container terminals matching to international standard. Handling of container is totally mechanised.
- ↓ The world class container terminal works 24 hrs. a day, even during recess time by introducing certain incentives to the workforce at the terminal, this has been made successful.
- ↓ Chennai Port was the first in India to introduce the concept of single window clearance and it is functioning effectively.
- ↓ Providing safety and security to vessels on arrival. "Customer care and Cargo care" is our motto.
- ↓ 13 feeder lines are operating dedicating feeder services to Singapore / Colombo / Port Kelang / Bangkok offering everyday sailings. Providing an excellent opportunity for exporters to avail and keep upto their committed delivery schedule.
- ↓ Port of Chennai acts as the Gateway Port to ICDs, with excellent facilities offered by way of intermodal transportation. Fixed day dedicated container trains are run between Chennai and Bangalore & Chennai and New Delhi.
- ↓ Cabotage Law is relaxed for Mother Container Ships as well as Lash Barge vessels to call at the Port.
- ↓ The Port has a deep harbour basin which helps handle Panamax and Suezmax Vessels.
- ↓ Dedicated terminals for Dry Bulk and Liquid Bulk.
- ↓ A Port committed for efficiency through innovation and continuous modernisation.
- ↓ Port has hosted Website www.chennaiporttrust.com for access to Port Users' worldwide.
- ↓ Two rail-mounted gantry cranes at the terminal can handle 4th generation post Panamax container vessels of 140 feet beam width and adequate 14m draft is available to handle these vessels.
- ↓ EDI connectivity has been established, pilot projects successfully tested and message transfer has been done with 12 Mainline Operators.
- ↓ The Green Channel facility is available in the Port and Customs have identified certain industries for extending this facilities.



CHENNAI PORT On Website www.chennaiporttrust.com

The site contains information on Port facilities, services, tariff, development projects, besides daily updated berthing / sailing / waiting details of vessels.

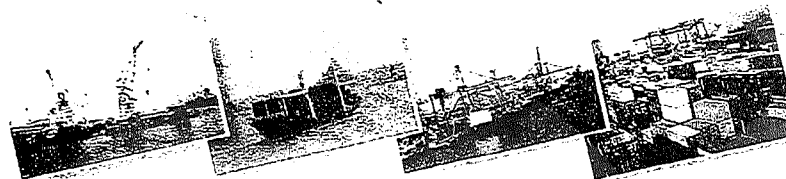


ADDITIONAL INFORMATION PROVIDED ON OUR WEBSITE

EDI Enabled	Harbour Map	Maritime Map	Press Release	What's new
Container Port Location	Tariffs	Evolution	Weather Information	Vessels Position
Tenders	Facilities	Performance	Development	Organisation

The Eastern Gateway In The South of India

A Port committed for efficiency through innovation and continuous modernisation.



CHENNAI PORT TRUST

Rajaji Salai, Chennai - 600 001.

Phone : (044) 5362201-(044)5360151 Telex : 041-8331 Fax : 044-5361228 Grams : PORTRUST

Website : www.chennaiporttrust.com E-mail : mmschpt@md3.vsnl.net.in

YOUR GLOBAL LINK TO THE WORLD MARKETS

Highlights of 1999 - 2000

(For the period upto Sept. '99)

An overall traffic of 18.48 Million Tonnes handled

Container traffic handled - 159770 TEUs

Iron Ore traffic handled - 2.86 million tonnes

Containerised Cargo traffic handled - 1.69 million tonnes

Railway traffic handled - 7.76 million tonnes

Record handling of 2831 tonnes of Steel on 21.04.1999 discharge from the vessel m.v. Golden Glint in a single shift with a single gang

Record tonnage of 15300 tonnes of Barytes was loaded into the vessel m.v. Asian Glory on 04.06.1999 surpassing the previous record of 12299-tonnes handled on 03.04.1998

Single day record tonnage of 201984 tonnes was handled on 22.06.1999 surpassing the previous record of 201914 tonnes handled on 31.01.98

Monthly record performance of 30382 TEUs of containers were handled during the month of August 1999 surpassing the previous record of 29577 TEUs handled during July 1998.



CHILIKA DEVELOPMENT AUTHORITY

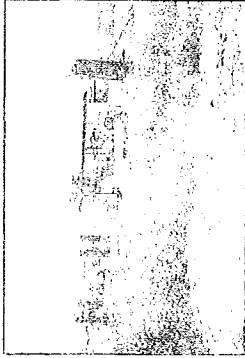
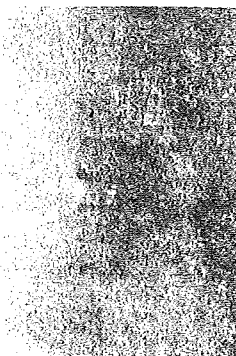
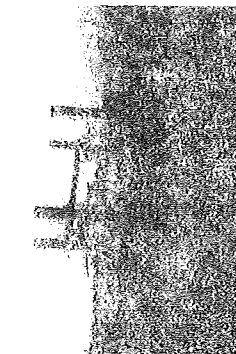
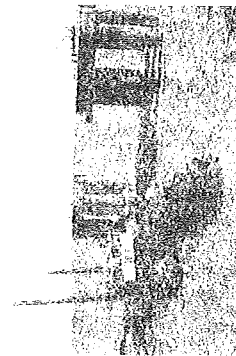
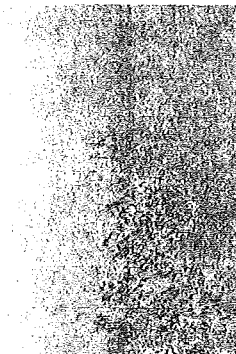
A Govt. of Orissa Agency

- ◆ To protect the lake eco-system with all its genetic diversity.
- ◆ To execute various multidimensional development activities either itself or through some other agency.
- ◆ To survey, plan and prepare the project proposal for integrated resource management for all round development in an around the Lake.
- ◆ To co-operate and collaborate with other institutions of the state, National and international institutions for all round development of the Lake.
- ◆ To establish management information system Lake.
- ◆ To promote long term multidisciplinary research, prepare environment status report and establish educations center for the Lake.

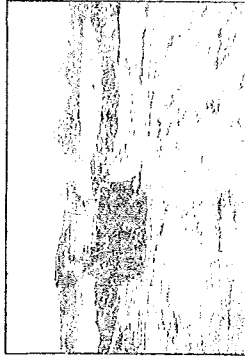
CHILIKA DEVELOPMENT AUTHORITY

BJ - 45, BJB NAGAR, BHUBANESWAR 751014, TEL : 91-674-434044, 435374, FAX : 91-674-434485

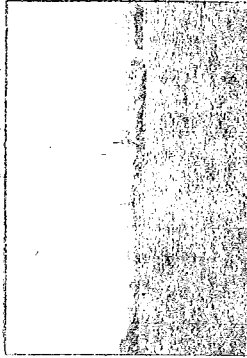
Email : chilika@chilika.com * Website : chilika.com



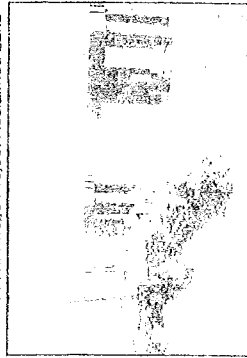
Noida Toll Bridge Project, Delhi



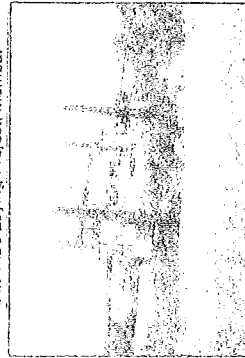
Noida Toll Bridge Project Reclaimed Land



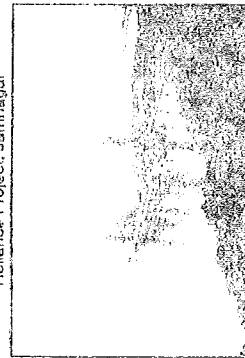
Gas Pipe Laying Project, Mumbai



Reliance Project, Jamnagar



Talagachol Project, Maharashtra



Dal Lake Project, Srinagar

Dharti The company involved in moving Earth

You want to move Earth contact us, as we are specialists in it Whether to deepen rivers, canals, lakes, fishing harbours, canals, drainage and irrigation systems, ports or reclamation of land, we are THE EXPERTS. We also are specialised in construction of marine structures, development of ports, building of roads and bridges along with allied civil works. Dharti, backed by a committed work force, is ready to meet the challenges of India's infrastructure development with environment friendly expertise in the ensuing millennium.

DHARTI DREDGING AND CONSTRUCTION LTD.

Fifth Floor, Varun Towers, 1-10-177A, Begumpet, Hyderabad - 500 016 India.
Phone : (040) 7761437 / 7761527 Fax : (040) 7760934

WITH BEST COMPLIMENTS FROM



HOCHTIEF

***HOCHTIEF (INDIA)
PRIVATE LIMITED***

No. 30, Khader Nawas Khan Road
3rd Floor, Nungambakkam
Chennai 600 006.
Tel. +44 - 821 0420 / 21 / 22
Fax +44 - 822 2379



"நீரின்றி அமையாது உலகெனில் யார் யார்க்கும்
வானின்றி அமையாது ஒழுக்கு" -குறள்

தமிழக வேளாண் பெருமக்களே!

நீர் வளம் காத்திட, பாசன மேலாண்மை செய்திடுவோம்!

பாசன மேலாண்மைக்கு, துவாக்குடி, திருச்சி-620 015 பாசன மேலாண்மை பயிற்சி நிலையம் ஆற்றி வரும் பணிகளாவன:

1. நீர்வளத்துறை, வேளாண் பொறியியல் துறை, வேளாண்மைத் துறை சார்ந்த அலுவலர்களுக்கு பாசன மேலாண்மை பயிற்சி அளித்தல்.

2. தமிழக வேளாண் பெருமக்களுக்கு பாசன மேலாண்மை பயிற்சி அளித்தல்.

3. பாசன நீர் சேமிப்பில் புதிய தொழில் நுட்ப உத்திகளைக் கடைப்பிடித்திட ஆலோசனை வழங்குதல்.

4. காவிரி மற்றும் ஏரிப்பாசன அமைப்பில் விவசாயிகளே நீர் பங்கிடு மற்றும் பராமரிப்பு செய்திட பயிற்சியளித்து, பாசன சங்கங்கள் அமைத்திட உதவுதல்.

5. பாசன சங்கங்களுக்கு ஏக்கருக்கு ரூ.100/- என்ற விகிதத்தில் சம.பட்டுத் தொகை வழங்குதல்.

6. பாசன மேலாண்மை பற்றிய புத்தகங்கள் வெளியிடுதல்.

மேலும் விவரங்களுக்கு அணுகுவிடுவீர்:-

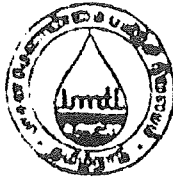
"இயக்குனர்,

தமிழ்நாடு பாசன மேலாண்மை பயிற்சி நிலையம்

துவாக்குடி, திருச்சி-620 015. தொலைபேசி எண்: 0431-552603 / 552088

Phone: { TIRUCHY { 552603
TIRUVARUR 23236

fax-0431-500103, phone-500003, 500088



**irrigation management
training institute-tamilnadu**

(Sponsored by the Government of Tamil Nadu)

"CAUVERY VALAGAM" THUVAKUDY, TIRUCHY-620 015.

PORT OF KANDLA

THE NORTH-WESTERN GATEWAY OF INDIA



Number One Port Among the Eleven Major Ports Of India with huge Traffic handled

SPECIAL FEATURES

- ⇒ STRATEGIC location with vast hinterland
- ⇒ MOST ECONOMICAL rates with lowest cost per tonne
- ⇒ MINIMUM turn around time
- ⇒ CORDIAL AND PEACEFUL industrial relation
- ⇒ HUGE storage and back-up area for dry and containerised cargo
- ⇒ UNIQUE weather port with tranquil sea state condition
- ⇒ HIGH GROWTH rate of 9.8 % per annum
- ⇒ VADINAR handles one-third of country's imported crude oil by single buoy mooring
- ⇒ TRANSPERANT ADMINISTRATION and well streamlined procedures for customer facilitation
- ⇒ TRADER FRIENDLY port

ECONOMICAL FEATURES

- ☞ Demuuge rates nominal
- ☞ Stevedoring and cargo handling costs are economical

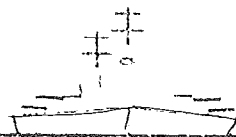
For Details, Please Write or contact

TRADE PROMOTION & PUBLICATION & PUBLIC RELATION OFFICER

P.B..50, A.O Building, Gandhidham (Kutch), Pin 370201, INDIA

Phone : 91-2836-33001 , Telex No. 0105 208 KPT IN

Fax No. 91-2836 32040



www.kandlaport.com

KVAERNER CEMENTATION INDIA LIMITED

(Formerly Trafalgar House construction Limited)

AN ISO 9002 COMPANY

for

Construction of Port and Harbour structures

OUR FOCUSSED MARKET :-

- ☆ Turnkey Construction of Marine Projects, Harbours, Jetties & Port Construction.
- ☆ Engineering & Construction of Industrial Projects including Bridges.
- ☆ Specialist works such as Drilling & Grouting, Pipe/Box Jacking, Diaphragm Walling, Piling, Soil Stabilization, Vibroflotation, Rehabilitation & Anchoring including Tunnelling and Mine Development.

BUILDING AN EVEN BETTER NAME FOR OURSELVES

CORPORATE OFFICE - BOMBAY

"APEEJAY HOUSE"
DINSHAW VAHHA ROAD
MUMBAI - 400 020

☎ : [022] 202-3201/7055 / 282-0738 / 283-6164/6165
Fax : 285 5032 / 2855446

REGIONAL OFFICE - CALCUTTA

"ANAR CHAMBERS"
5, CHOWRINGHEE APPROACH
CALCUTTA - 700 072

☎ : [033] 225 3050 / 5034 /35
Fax : [033] 225 8148 / 215 0376

IN A WORLD OF RISING STANDARDS, WE ARE SETTING STANDARDS FOR TOMORROW

With 1700 topflight engineers, a network of 30 offices spread over the country, experience in handling over 2300 consultancy assignments and EPC contracts worth Rs. 20 billion, an annual turnover of Rs. 2500 million and registration with WB, ADB, EBRD, MECON is one of the frontline design, engineering, consultancy and contracting organisations.

Engineering, Consultancy, Supply & Turnkey/EPC Execution

- ☐ Power Projects
- ☐ Oil & Gas, LPG/LNG
- ☐ Water Treatment & Supply
- ☐ Roads & Highways
- ☐ General Engineering
- ☐ IT, Industrial Automation
- ☐ Computer Applications
- ☐ Materials Handling/Ports & Harbours
- ☐ Mining, Mineral Beneficiation
- ☐ Ferrous Metallurgy
- ☐ Coke Oven Batteries
- ☐ Blast Furnace
- ☐ Rolling Mills & Processing Lines
- ☐ Continuous Casting Plants
- ☐ Gas Cleaning Plants
- ☐ Refractories
- ☐ Non-ferrous Metals
- ☐ Environmental Engineering
- ☐ Ocean Engineering
- ☐ Architecture & Town Planning



MECON LIMITED

(A Government of India Enterprise)

Ranchi - 834 002, Bihar, India

Phone : 0651-501002, 501216 Fax : 0651-502214, 502189

E.Mail : mecon-rnch@hub.nic.in

Expanding Capacities, Modernising Facilities

It has been a particularly satisfying year for the **Mormugao Port Trust**. Satisfying because, despite a slump in the iron ore markets of Japan and South East Asia - the largest importers of iron ore — that affected the throughput of the port, the **MPT** managed to not only present a good performance but also create new efficiency records. In addition, it also proceeded with determination to complete the many developmental projects with a view to transforming the **MPT from a mono-commodity to a multi-commodity port**.



LICENCE AGREEMENT ON BOOT BASIS

On April 11 this year, the **MPT** entered into a path-breaking agreement with an Indian private sector firm for construction and operation of two berths on build, own, operate and transfer basis. The **Rs 224 crore project** is the second of its kind to be undertaken by any major port in the country. According to the agreement, the private sector firm, **ABG Goa Pvt Ltd** will build the two berths in 30 months' time and operate it for a period of 30 years and thereafter the assets will be transferred to the **MPT** free of cost.

GENERAL CARGO RECORDS ALL-TIME HIGH

During the year 1998-99, general cargo traffic reached a **record 1.53 million tonnes**, as compared to the earlier high of 0.90 million

tonnes achieved in 1997-98, registering a **growth of 70 %**.

CONTAINERISED CARGO TOUCHES NEW FIGURE

Despite the slump, the **MPT** handled containerised **traffic of 36,022 tonnes**, a record level, the previous high being 33,223 tonnes reached in 1996-97.

MOHP CREATES NEW PERFORMANCE LEVEL

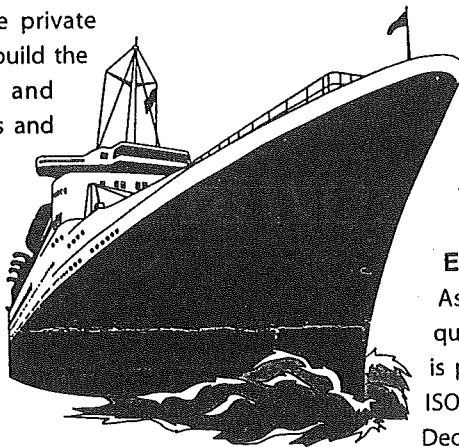
Even the Mechanical Ore Handling plant (**MOHP**) set a new record on January 15 this year by loading a quantity of **1,13,361 tonnes of iron on a single day**. It also unloaded a record figure of **53,054 tonnes on a single day** on January 14 this year.

EFFICIENCY PARAMETERS HIGHER

Average pre-berthing/waiting time (days) down to 1.40 from 1.87; average turn-round time (days) down from 5.80 to 4.81; average output per berth day (tonnes) up to 11,076 from 10,446.

EYE ON THE FUTURE

As a part of its mission for quality improvement, the **MPT** is preparing itself to attain the **ISO 9000 series certification** by December 1999.



MORMUGAO PORT TRUST

THE NATION'S PRIDE

HEADLAND SADA, GOA 403 804 INDIA. PH.: 0834-512911-16 FAX: 0834-513065

With best compliments from :

NEC

Navayuga Engineering Company Limited

GEO-TECHNICAL AND CONSTRUCTION ENGINEERS

Design & Construction of :

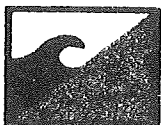
- ❖ Diaphragm Walls upto 1200 mm thick.
- ❖ Bored Piles (300 mm Ø to 2000 mm Ø)
- ❖ Well Foundations
- ❖ Soil Densification Works
- ❖ Drilling & Grouting
- ❖ Rock Anchors
- ❖ Marine Works: Jetties, Wharves, Piers, Sea Water Intake Systems, etc.
- ❖ Industrial Structures
- ❖ Bridges

48-9-17, Dwarakanagar, Visakhapatnam - 530 016 (A.P).

Tel: +91-891-554602 / 547335

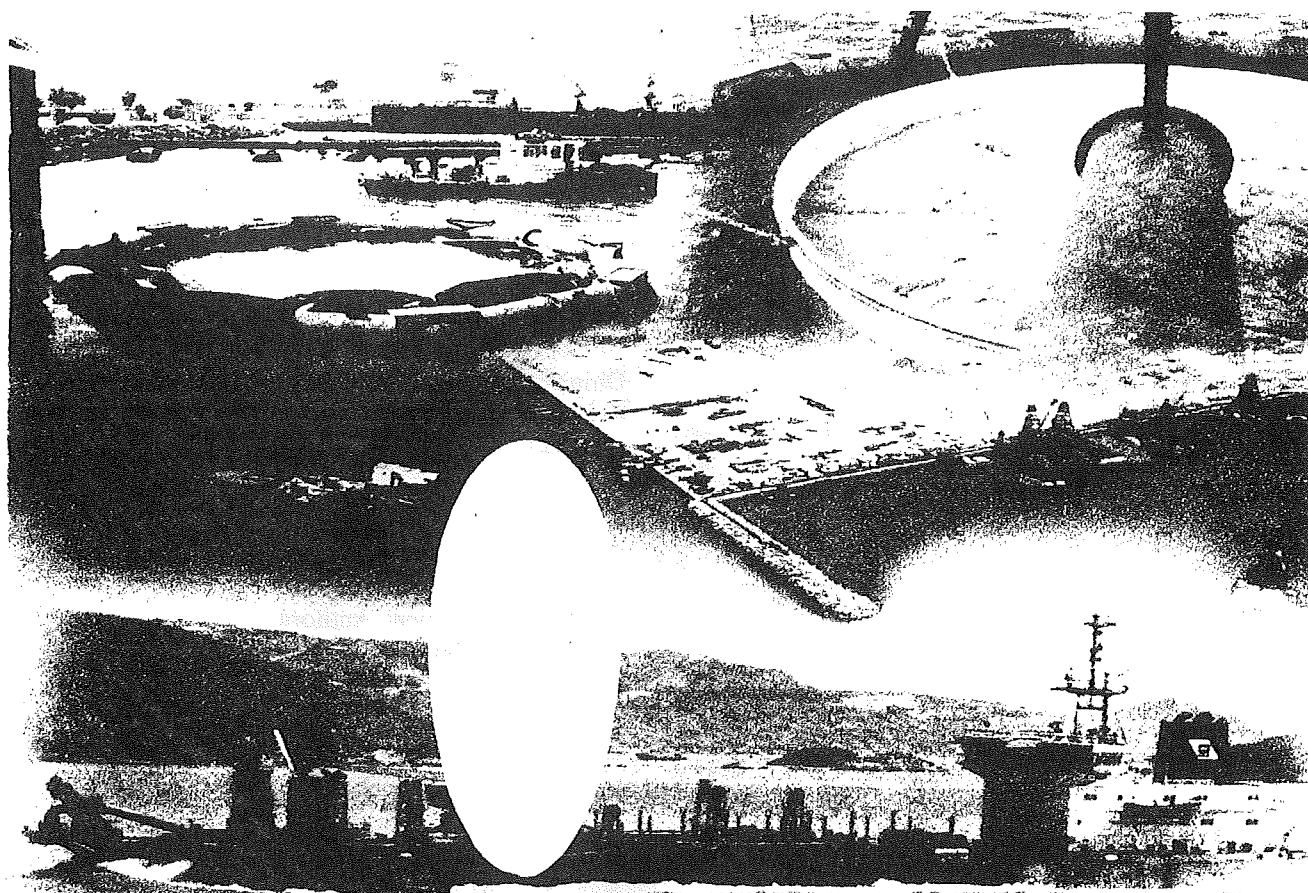
Fax: +91-891-547362 / 574056

E-mail: nec.vizag@rmx.sprintpg.ams.vsnl.net.in



Van Oord ACZ India

Marine and Dredging Contractors



Marine and Dredging contractor Van Oord ACZ have evolved, in half a Century of existence, from the individual companies to a fully integrated organisation. Having worked from Arctic to the tropics, the company is renowned around the world for its know-how, specialised services and versatile equipments.

They offer design, engineering and execution of work for private and public sector in three areas of expertise.

DREDGING

Capital Dredging
Maintenance Dredging
Presweeping
Beach Nourishments
Land Reclamation
Trench Back Filling
Artificial Islands
Deep Sea Dredging

MARINE CONSTRUCTION

Ports & Harbours.
Break water
Outfalls
Shore Approaches
Slope / Bottom Protection
River Training
Dykes / Embankments
Quaywalls / Concrete Work

OFFSHORE SERVICE

Pipe line intervention
Sea Preparation
Pipelines / Cables Protection
Platform Stabilization
Upheaval Bucking Prevention
Ballastic of Structure
Pipeline / Installation
Trenching

Van Oord ACZ takes pride in their cooperative approach towards clients, leading to a desired involvement and more economical solutions.

Service are performed on a single service contract, turnkey, lumpsum or alliance basis and may include project financing

Head office

The Netherlands
Van Oord ACZ B.V.
Jan Blankenweg 2
P. O. Box 458
4200 AI GORINCHEM
Phone : +31 183 642200
Fax : +31 183 628213
E-mail : voacz@voacz.com
J. De Ruyter - Area Manager • Jan Zwart - Area Representative

India Office

Flat No. 204, 2nd Floor,
Anurachal Building,
Barakhamba Road,
New Delhi 110 001
Tel. : 11 - 3357980 / 81
Fax : 11 - 3357982
E-mail : vanoord@nda.vsnl.net.in
A. P. Srivastava - Director

**List of Participants for the Inter Institutional Course on Coastal Engineering and
Workshop on Breakwaters, 6-10 March, 2000, IIT Madras**

Abel, H.J.
Hochtief (India) Pvt. Ltd.
30, Khader Nawaz Khan Road,
III floor,
Chennai - 600 006.
Ph : 0091-44-821 0420 / 0421 / 0422
Fax: 0091-44-821 0377

Anjaneyulu, V.S.R.
Visakhapatnam Port Trust,
Engineering Department,
Visakhapatnam - 530 035.
Ph: 564841

Balasubramanian, T.
Manager
Elcane Survey (Pvt.) Ltd.
D-222/30 T.T.C. Industrial Area
New Mumbai - 400 706
Ph: 7632348

Bas Van Dijk
Department of Civil Engineering and
Geo Sciences,
T.U.Delft,
The Netherlands.
Ph : 2785754
Fax: 2785124

Bhattacharjaya, P.
Kvaerner Cementation India Ltd,
Anar chamber,
5, Chowringhee Approach,
Calcutta -700 072

Bhaskar Rao,
Haskoning Consulting Engineers
and Architects,
NCTPS Post,
Post Box No. 3,
Chennai 600 120.
Ph: (04119) 69858 / 69859 / 69834
98400-76900
Fax: 98400-65901

Bose, D.
DE (Ports), MH Section,
MECON Ltd.,
Ranchi - 834 002.

Budda Yerranna,
Van Oord India,
No. 204, 2nd floor,
Arunachal Building,
Barakhamba Road,
New Delhi.
Ph : 3357980 / 81
Fax: 3357982

Dhinakaran, G.
Research Scholar
IIT Madras
Chennai.

Dhillip Ku. Nayak,
Junior Engineer,
Chilka Development Authority,
BJ - 45, BJB Nagar,
Bhubaneswar - 751 014,
Orissa.
Ph : 434044 / 435374
Fax: 434485

Dhiraj Gondane,
Van Oord ACZ India,
Delhi.
Ph : 91 11 33 57980 / 981 / 982
Fax: 91 11 33 57982

Geeta Arya,
Engineer (P & H),
Water and Power Consultancy services
(India) limited,
No. 9, Community Centre,
Saket, New Delhi - 110 017.
Ph : 6852025, 6560103
Fax: 6867930

James, S.
Mormugao Port Trust,
Administrative Building,
Headland sada, Goa - 403 804
Email: Prem-goah@hotmail.com

Josanto, V.
Scientist E-II,
National Institute of Oceanography,
Vidyanikethan Annene Building,
Provience Road,
Ernakulam, Cochin 682 018.
Email: sic@csnioc.ren.nic.in.

Krishnamoorthy, M.R.
Senior Design Engineer (civil),
Fichtner Consulting Engineers
(India) Pvt. Ltd., Ganesh Chambers,
143, Eldams Road, Chennai – 600 018.
Ph : 4359158
Fax : 4344579
Email: fichtner@giasmd01.vsnl.net.in

Madhusudhan, ch.
Research Scholar,
Ocean Engineering Centre,
IIT.Madras, Chennai.

Mani, P.R.
Executive Engineer (Civil)
Tuticorin Port Trust, Tuticorin – 628 004.
Ph : 0461 – 352290
Fax : 0461 – 352301
Email: tutport@md2.vsnl.net.in

Martokm Van Rijsewijk
Department of Civil Engineering and
Geo Sciences,
T.U.Delft, The Netherlands.
Ph : 2785754
Fax: 2785124

Mehta A.K.
Assistant Engineer(Civil),
Gujarat Maritime Board,
Sector 10-A, Gandhi Nagar,
Gujarat – 382 043
Ph : 02712-38346-47-48
Fax : 02712-34703
Email: gmb@adl.vanl.net.in

Mehta, V.K.
Engineers India Limited,
1, Bhikaji Cama Place
New Delhi 110 066.
Ph : 331143
Fax : 320774 / 331134
Email: amm@eilrmd.ernet.in

Murthy, D.V.S.
Engineer (P&H),
Water and Power Consultancy services
(India) limited,
No. 9, Community Centre, Saket,
New Delhi – 110 017.

Nagaraj, C.
Hochtief (India) Pvt. Ltd.,
No. 30, Khadar Nawaz Khan Road,
3 rd floor, Chennai.

Nagaraju, K.
Project Associate Gr.1,
Ocean Engineering Centre,
IIT Madras,
Chennai.

Nanda, S.K.
Chilka Development Authority,
BJ – 45, BJB Nagar,
Bhubaneswar – 751 014,
Orissa.
Ph : 434044 / 435374
Fax : 434485

Natarajan, J.
RITES, Ennore Coal Port Project
NCTPS Post P.O Box 3
Chennai 600 120
Email: hasko_ch@satyam.net.in

Patel, C.N.
Dy. Executive Engineer,
Gujarat Maritime Board,
Office of the Dy. Ex. Engineer (C),
Fisheries Harbour Project Sub-Division,
Porbandar Port,
Porbandar – 360 575.

Pati, L.K.
Civil Engineer,
Darti dredging limited,
3rd floor, Khaleeli Centre,
149, Montieth Road. Chennai – 600 008.
Ph : 8553040 / 8553042
Fax: 8553655

Phani Kumar,
Water and Power Consultancy services
(India) limited, Pune.
Fax: 020-4392004

Prasad, P.S.R.
Navayuga Engineering Company Ltd.
48-9-17, Dwarakanagar nagar,
Visakhapatnam – 530 016.
Ph : 554602 / 547335
Fax: 547362 / 574056

Prema Kumar, J,
Assistant Executive Engineer,
Mourmagoa Port Trust, Admn. Building,
Headland Sada, Goa – 403 804.
Ph : 512911 – 16
Fax: 513065, 512721

Purushotham, S,
Research Scholar
Ocean Engineering Centre,
IIT Madras, Chennai.

Raghuveer, S.S.
Dy. General Manager (OPS),
Dredging Corporation India Ltd.,
Dredge House, Port Area,
Visakhapatnam – 530 035.
Fax NO. 44 – 2350509.

Rambabu. M.
Research Scholar,
IIT Madras, Chennai.

Ramanathan, R.
Haskoning, Post Box - 3
Chennai – 600 120
Ph : 69858 / 69859 / 69834
Fax : 98400 – 65901
Email: hasko_ch@satyam.net.in

Raijada, M.P.
Assistant Engineer(Civil),
Gujarat Maritime Board,
Construction Division,
Near S.T. Stand, Chowpati Road,
Porbandar – 360 575.
Ph : 02712-38346-47-48
Fax : 02712-34703

Rao, G.S.
Hydraulic Engineer, Kandla Port Trust
Port colony, At/po: Gopalpuri
Gujarat-370 240
Ph : 02836-70338
Fax : 02836-70338
Email: kpt@guil.guil.nio.in

Rao, M.V.
Dredging Corporation India Ltd.,
Dredge House, Port Area,
Visakhapatnam - 530 035.
Fax : 44 – 2350509

Ravi Kumar, B.
Civil Engineer,
Dharti Dredging and Construction Limited,
3rd floor, Khaleeli Centre,
149, Montieth Road,
Chennai – 600 008.

Roopsekhar, K.A.
Project Associate Gr. I
Ocean Engineering Centre,
IIT Madras,
Chennai.

Sahadevan, P.V.
Joint Chief Engineer,
(Plan Formulation),
Public Works Department,
Chepauk, Chennai.

Sarma, D.H.S.A.
Visakhapatnam Port Trust,
Visakhapatnam - 530 035.
Fax: 0891-565023

Satyanarayana Rao, T.V.S.
Navayuga Engineering Company Ltd.,
48-9-17, Dwarakanagar nagar,
Visakhapatnam – 530 016.
Ph : 554602 / 547335
Fax : 547362 / 574056
Email: nec.vizag@rmx.sprintpg.ams.vsnl.net.in

Sengupta, P.S.
Kvaerner
Calcutta.
E-mail : PS.SENGUPTA@kcind11.kvaerner.co.in

Sing, B.K.
Assistant Project Manager
PMU.CCE(R&D)MW
Balasore
Fax: 06782-62104

Sri Veda Kumar, K.
Research scholar,
Ocean Engineering Centre,
IIT Madras, Chennai.

Subba Rao, B.V.V.
Research scholar,
Ocean Engineering Centre,
IIT Madras, Chennai.

Suresh, P.K.
Assistant Engineer, W.R.O.,
Public Works Department,
I.H.H., Poondi.

Torsten Dose,
FB11,
Bergische University of Wuppertal,
Pauluksirch Street 7, 42285 Wuppertal,
Germany.
Email: dose@mail.urz.uni-wuppertal.de

Torsten Schlurmann,
University of Wuppertal,
Germany.
Email: schlurma@uniwuppertal.de

Vedgiri, K.
Engineers India Limited,
1, Bhikaji Cama Place
New Delhi 110 066.

Veluchamy, T.
Assistant Executive Engineer (Civil)
Tuticorin Port Trust
Tuticorin – 628 004
Ph : 0461 – 352290
Fax: 0461 – 352301
Email: tutport@md2.vsnl.net.in

Venkatachar, M.B.
Central Institute of Coastal
Engineering For Fishery,
64, Palace Road
Bangalore.
Ph : 2267841
Fax : 080 2258945
Email: cicefko@bgl.vsnl.net.in

Venkatesh Prasad, N.
Central Institute of Coastal
Engineering For Fishery,
64, Palace Road
Bangalore.
Ph : 2267841
Fax : 080 2258945
Email: cicefko@bgl.vsnl.net.in

Verma, R.K.
DE (Ports), MH Section,
MECON Ltd., Ranchi – 834 002.
Ph: 0651 - 501002 / 501216
Fax: 0651 – 502189/ 502214

Vijayasimhan, S.
Assistant Executive Engineer (Civil)
Tuticorin Port Trust
Tuticorin – 628 004
Ph : 0461 – 352290
Fax: 0461 – 352301
Email: tutport@md2.vsnl.net.in

Ziauddin Bin Abdul Latif,
Senior Engineer,
Department of Irrigation and Drainage,
Bahagian Kejuruteraan Pantai,
Ibu Pejabat,
JPS Malaysia.
E-mail: jps31@pop.moa.my

