सत्यमेव जयते

भारत सरकार
जल शक्ति मंत्रालय
जल संसाधन, नदी विकास और गंगा संरक्षण विभाग

Government of India
Ministry of Jal Shakti
Department of Water Resources,
River Development & Ganga Rejuvenation

Technical Memorandum
on
DESIGN OF COASTAL HYDRAULIC STRCTURES

by
Shri A.V. Mahalingaiah,
Scientist 'E'

Dr. R. S. Kankara
Director

केन्द्रीय जल और विद्युत अनुसंधान शाला, खडकवासला
सिंहगड रोड, पुणे 411024

CENTRAL WATER AND POWER RESEARCH STATION
Khadakwasla, Sinhagad Road, Pune 411024

December 2024

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PREFACE

In coastal area, suitable engineering measures are of paramount importance for sustainable development of coastal stretches & marine environment. Knowledge of various coastal structures is important for development of Ports and for protecting the coast. The purpose of this technical memorandum is to cover recent trends in design of coastal hydraulic structures.

The coastal structures are subjected to various marine environmental forces due to waves, wind and currents. The wave forces are the dominant forces and are decisive in the design of coastal structures. The most popular and commonly used coastal structures for protection of harbour basin are rubblemound breakwaters, while for combating the erosion of the coastline are Groynes and seawalls.

The basic design and construction methods are generally available in coastal engineering books / manuals. However, this technical memorandum provides the essential technical information and basic requirements thorough knowledge on coastal hydraulic structures. This technical memorandum is prepared based on the vast experience of CWPRS in the field of Coastal Engineering and the field observation for more than 300 sites. I am sure that this technical memorandum would be useful for the initial designers and field engineers who are dealing with the design and construction of coastal structures.

Dr.R.S.Kankara
Director

ACKNOWLEDGEMENT

First and the foremost the authors express a deep sense of gratitude to Dr.R.S.Kankara, Director, CWPRS for his constant encouragement and support during preparation of this Technical Memorandum. His motivation helped in timely completion of the document in limited time period.

The author is thankful to Dr. Prabhat Chandra. Additional Director for his support for preparation of this Technical memorandum. The author is thankful to Shri B.R.Tayade, Scientist D, Shri N.V.Gohkale, & Shri Uday B Patil, Scientist-C and Shri. N.S. Ganesh, Assistant Research Officer for their timely support in preparation of this memorandum.

Special thanks to Dr. Kumaran Viswanathan, Scientist-B for editing and support in preparation of this memorandum.

Acknowledgement is extended to various central/state Government departments and Indian Navy officials for referring coastal projects involving design of coastal structures to CWPRS. The authors are immensely thankful to all retired and present research staff of Coastal Hydraulic Structure Division who have directly or indirectly contributed in conducting these studies. The memorandum would not have been a reality without their valued contribution.

Author is grateful to Officers and staff of CHS division for their support in conducting wave flume studies and preparation of this memorandum. Author is also thanks to WAPIS division for extending help in providing technical references and printing of the document.

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Definitions of symbols used

ρ= mass density of sea water

α_b= breaker angle with respect to coastline

ρ_s = mass density of the sediment

B = width of breakwater

C = a constant

d_c = depth of closure

F' = non-dimensional freeboard

g = acceleration due to gravity (9.81 m/s^2)

G = gap between breakwaters

H_b = breaking wave height

H_e = nearshore extreme significant wave height

H_i = incident wave height

Ho' = the un-refracted deep water wave height

Hs = Significant wave height

K = dimensionless constant relating sand transport to longshore energy flux

K_t = transmission coefficient

Ls = length of breakwater

m_0 = zeroth moment of the wave spectrum

m_2 = the second moment of the wave spectrum

p = porosity of sediment

Q = volume of longshore transport rate

R_c=Crest freeboard

T = wave period

Tp = Peak period

Tz = Mean wave period

CHAPTER- I

1.0 INTRODUCTION

The major mode of transport of goods, oil, iron ores between India and foreign countries is through waterways. It is the cheapest mode of transport than any other mode of transport. In order to cope-up with the existing waterborne traffic as well as to meet its future requirement, it is very essential to plan & built new harbours, which requires construction of various types of marine structures. The coastal structures are the marine structures located in the relatively shallower water depths. It has been observed that, a long coastline is continuously under threat of coastal erosion and it is the primary duty of engineers to protect the coastline of our country from the attack of severe ocean waves. The coastal structures are subjected to various marine environmental forces due to waves, wind and currents. The wave forces are the dominant forces and are decisive in the design of coastal structures. The most popular and commonly used coastal structures for harbour protection are rubblemound breakwaters, while for combating the erosion of the coastline are Groynes and seawalls. Design of these structures is complex due to complexity in the wave-structure interaction. As such, the design of these structures is mainly finalized by hydraulic model studies. Number of empirical/semi-empirical procedures based on the hydraulic model studies are also available.

Central Water and Power Research Station (CWPRS), Pune has been involved in design of more than 300 site specific design of coastal hydraulic structures along the Indian coast.

In the present Technical Memorandum, the details on the classification of coastal hydraulic structures are described in Chapter-II. Chapter-III deals with the design and stability aspects of rubble mound breakwater. Chapter- IV describes the hydraulic modelling of marine structures. Chapter-V elaborates the deviation in the construction of sea wall. Chapter-VI describes the various case studies for the design of marine structures including wave flume studies.

CHAPTER-II

2.0 CLASSIFICATION OF COASTAL STRUCTURES

Coastal structures can be broadly classified into two types:

- Offshore Structures
- Near-shore Structures

The design of offshore and Near-shore structures varies greatly in terms of environmental factors, depth of water etc. Offshore structures are located in deep-water and are subjected to forces due to short crested multi-directional waves, which are predominant, apart from other forces due to wind, ocean currents etc.

The different types of offshore structure are:-

- Gravity type Structures
- Pile supported Structures
- Floating Structures
- Submarine Pipelines

The different types of near-shore structures (Figure 2.1)

- Rigid structure – Sheet piles, Walls
- Semi-rigid or Composite structures – Caissons or cells on rubble base
- Flexible Structures – Rubblemound breakwaters, Sea walls

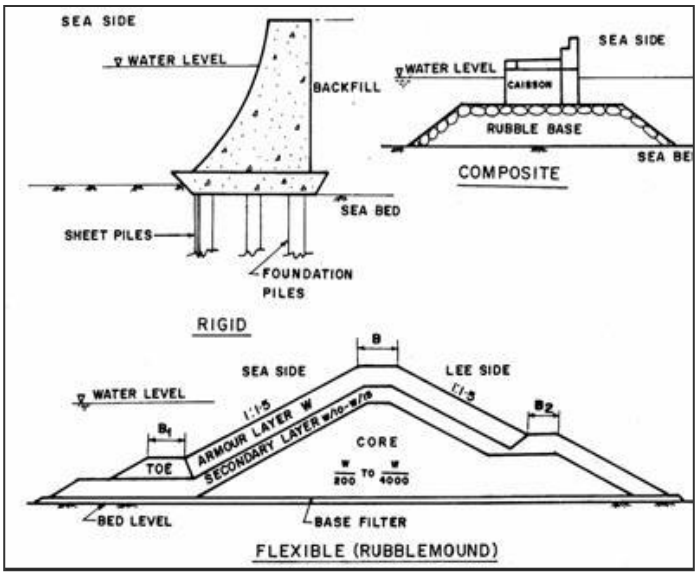


Fig 2.1: Types of Near-shore Structures

The near shore structures are subjected to various marine environmental forces due to waves, winds and currents. The wave forces are the dominant forces and are decisive in the design of near shore coastal structures. Rubblemound structures are the most commonly applied type for breakwater/seawall. The stability of rubblemound coastal structures depends primarily upon the stability of individual armour units on its seawards slope. The other hydrodynamic aspects of the effect of waves on the rubblemound are wave run-up, rundown, overtopping, reflection and transmission. Design of flexible rubblemound structures is complex as it involves various aspects such as wave-structure interaction interlocking, characteristics of armour, friction between armour and secondary layer etc. Though various empirical formulae are available, the designers/planners of rubblemound structure prefer to evolve the conceptual design by empirical formulae, which is confirmed and finalized by hydraulic model tests in wave flumes.

2.1 RUBBLEMOUND STRUCTURES

Rubblemound structure in its most simple shape, it is a mound of stones. However, a homogeneous structure of stones large enough to resist displacements due to wave forces is very permeable and might cause too much penetration not only of waves, but also of sediments if present in the area. Moreover, large stones are expensive because most quarries yield mainly finer material (quarry run) and only relatively few large stones. As a consequence, a rubblemound structure is normally composed of a bedding layer and a core of quarry-run stone covered by one or more layers of larger stone and an exterior layer or layers of large quarry stone or concrete armour units (Fig. 2.2).

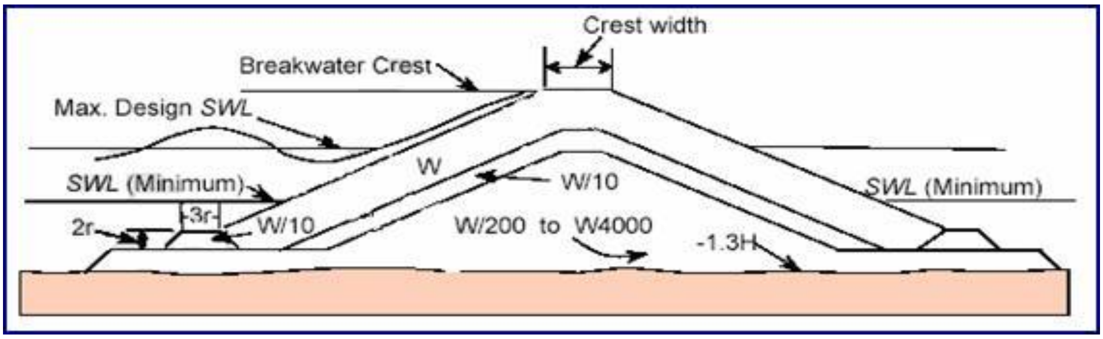


Fig 2.2: Typical Cross Section of Rubblemound Breakwater

Concrete armour units are used as armour blocks on the outer slopes of rubblemound structures in areas with rough wave climates or at sites where a sufficient amount of large quarry stones is not available.

2.2 BREAKWATER

Breakwaters are the structures constructed to protect the harbour facilities from the hostile forces of the waves and to provide tranquil conditions for the berthing of the ships. Breakwaters are generally categorized as fixed, floating, and special types. The most feasible one is chosen for construction based on the prevailing environment and depending on the required degree of shelter (Rajendra et al., 2017). The following are some of the types of breakwaters basically in use :

- Rubble mound breakwaters
- Vertical wall breakwaters
- Composite breakwaters
- Special type of breakwaters

Rubble mound breakwaters are further classified as below;

- Conventional breakwater
- Berm breakwater
- Reef breakwater
- Tandem breakwater

Rubblemound breakwaters are characterized by a core with some porosity or permeability, covered by a sloping porous armour layer consisting of large rock or concrete armour units (for example Xbloc/tetrapods/accropode). However, the formula for coastal and river dikes can be used for a wide range of slopes, and therefore allows for more flexible input parameters. Since the formula for a rubble mound breakwater is a simplified case of the coastal or river dike, the formula for the dike is implemented. The definitions of the variables are presented in Figure 2.3.

To prevent finer material being washed out through the armour layer, filter layers must be provided. The filter layer just beneath the armour layer is also called the under layer. Structures consisting of armour layer, filter layer(s), and core are referred to as multilayer

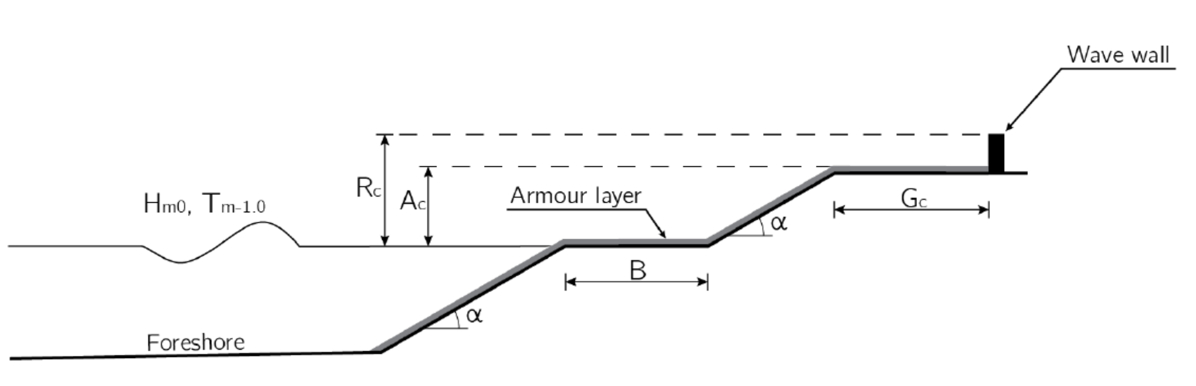


Fig 2.3: Schematization of a rubblemound breakwater with variables

Where,

A_c = armour crest freeboard of a rubble mound structure [m]

R_c = crest freeboard of structure [m]

H = wave height [m]

H_{m0} = estimate of significant wave height from spectral analysis $=4\sqrt{m_0}$[m]

$T_{m0,1}$ = average wave period defined by m_0/m_1 [s]

B = berm width, measured horizontally [m]

G_c = armour crest width for a rubble mound structure (over the full crest width or up to a crest wall) [m]

α = angle between overall structure slope and horizontal [°]

α = angle of parapet / wave return wall above seaward horizontal, Section 0 [°]

Rubble mound breakwaters have been used at ports of New Mangalore, Madras, Paradeep, and Tuticorin. They are economical up to depths of approximately 15 to 20 meters [Ramamurthy (1974)]. But there are cases where they have been used up to 50 meters, like Sines breakwater in Portugal.

2.3 VERTICAL CAISSON BREAKWATERS

Vertical caisson or wall-type breakwater is predominantly used to protect the inner harbour region from the high-water levels and waves to maintain tranquillity for safe marine operations, especially in deeper waters. The vertical-caisson structure breakwaters function as an effective barrier against the waves and maintain calm sea conditions on their leeward side for safe operations. These breakwaters are effective for water depths larger than 15–20 m; the most common type of vertical / caisson breakwaters are cellular reinforced concrete caissons, which are sunk with seawater ballast and then filled with sand. They are also called upright or vertical-caisson breakwater (Franco 1994).

Arrays of concrete caissons or vertical caisson are widely used as a breakwater in various countries (ex: Japan, Italy) when it is viable and cost-effective. The main concept of the vertical breakwater is to reflect waves, while for the rubblemound breakwater dissipate waves. Typical photography of Vertical breakwater is illustrated in Figure 2.4.

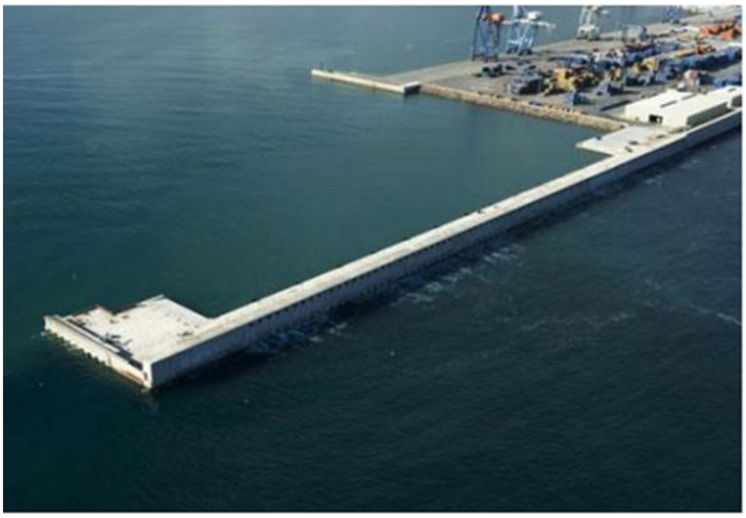


Fig 2.4: Vertical breakwaters

(Source:https://www.gravityeng.com/exp_tinnel_pid=122&type=Harbor)

The installation of caisson-type breakwaters is done by towing the caisson to the required location and ballasting it using rubble, concrete, or sand fill to lower it to the sea bed. Caisson-type breakwaters are the most suitable for rough sea conditions due to relatively fast and easier installation than rubble mound breakwaters. The main factors influencing the design and selection of vertical caisson breakwater are foundation stability and incident wave forces. Such kinds of breakwater are regularly designed as structures subjected to forces causing failure in the following ways:

- Sliding from one block to the another
- By overturning as a solid mass and continuous wave action leads to the uplifting of horizontal layers.
- Collapsing or fracture of massive blocks.

The installation of vertical wall breakwaters requires skilled labour, advanced construction equipment, and high knowledge of confidence. Many such breakwaters are failed in the past (e.g.) Japan, Italy, Algeria, etc. (Goda 1992)

A few of the merits and demerits of the caisson-type breakwater are listed below:

Merits

- Provide a larger harbour area and a narrower entrance.
- Reduce the amount of material
- Avoid dangers of unequal settlement
- Where rock is unavailable, it saves time and money

Demerits

- Difficult to repair if damaged
- Construction requires more extensive and heavier equipment
- Required formwork, quality concrete, and skilled labour, batching plants, floating crafts
- It can be constructed only where foundation conditions are favourable.

2.4 COMPOSITE BREAKWATERS

The combination of the RMB and the wall breakwaters are shown in Figure 2.5. The concrete caissons of different configurations are used to substitute the wall section to reduce the effect of wave reflection. Such kinds of breakwaters are significant in deeper waters or at sites where the variation of tidal is high. High mound composite breakwaters are unstable as the breaking waves induce impulsive pressure and scouring, due to which low mound breakwaters have commonly been used. These composite configurations function as mound breakwaters at low tide and vertical breakwaters at high tide. (Goda, 2000).

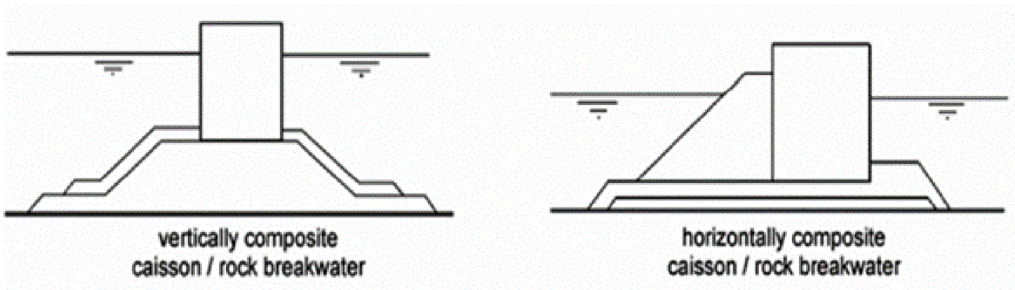


Fig 2.5: Types of the composite breakwater (CEM 2012)

The final choice of the type is governed by the equipment available and technical-know how in handling the job. During the low tide, the rubble mound will function as a rubble mound breakwater. During the high tide, the composite breakwater will perform the role of a caisson breakwater, and the rubble mound offers scour protection. It provides a platform for handling cargo. It makes it possible for ships to come close to the breakwater wall on the inner or harbourside for loading and unloading cargo.

2.5 SPECIAL TYPES OF BREAKWATERS

The special kind of breakwater is still in use though limited to special conditions. The curtain wall breakwater is used as secondary breakwaters to protect small craft harbours. Sheet pile or continuous pile vertical wall breakwaters are used to break small waves. A Horizontal plate breakwater can reflect and break waves. A floating breakwater is very useful as a breakwater in deep waters, but its effect is limited to relatively short waves. Some of the special types of breakwaters are explained in the subsequent sections.

Floating breakwaters is employed in low wave energy environments, as an alternative to conventional gravity breakwaters. It does not have any kind of bottom-founded structure. Furthermore, for practical considerations, it simply floats over the surface of the water, so it is not extended down to the sea bed, and also no penetration through the free surface of water as shown in Figures 2.6.

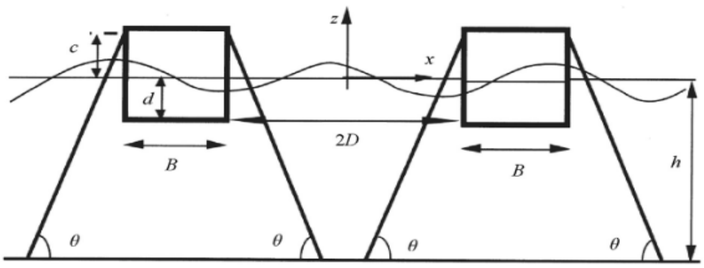


Fig 2.6: Floating double-box breakwater (Williams et al., 2000)

Mobile breakwaters are considerably used for their speedy installation at the site. The wave height is appreciably reduced on the leeward side of the structure which supports ready transportation.

Horizontal plate breakwater is preferable in a less energy-wave environment with weak and soft subsoil conditions. The structural configuration breaks and reflects the wave energy significantly. The steel jacket frames used to support these structures.

Pile breakwater, Figure 2.7 illustrates the configuration of pile breakwater formed by a series of piles arranged in rows. It has more advantages than the conventional rubble mound breakwaters in allowing the free passage of sediments and thus reducing coastline erosion on its down-drift side.

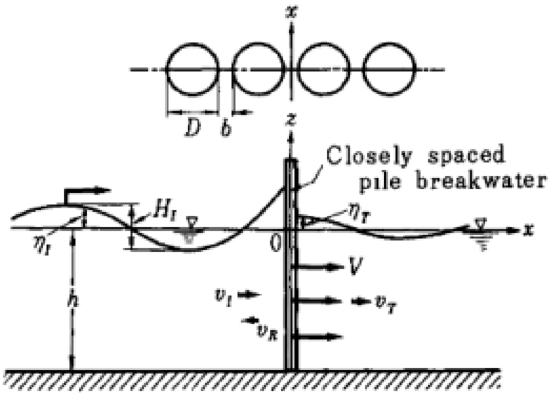


Figure 2.7: Pile breakwater (D'Angremond et.al., 2008)

Curtain wall breakwater, is frequently utilized as a supplemental breakwater to safeguard small vessel harbours as shown in Figure 2.8.

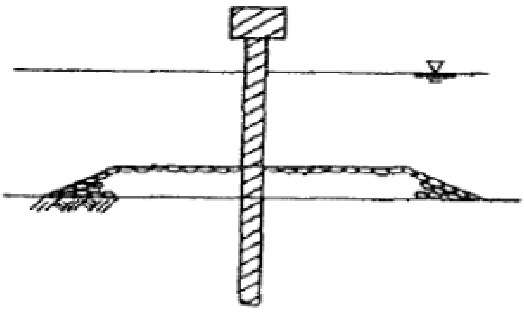


Figure 2.8: Curtain wall breakwater (D'Angremond et.al., 2008)

2.6 PROTECTIVE STRUCTURES

A) SEAWALL

Seawalls are the structures primarily designed to resist wave action along high value coastal property. They are either gravity or pile supported structures made of either concrete or stone. Seawalls have a variety of face shapes (Fig-2.9).

a) Curved face: designed to accommodate the impact and run-up of large waves while directing the flow away from the area being protected. Large wave force is resisted and redirected. This requires a massive structure with adequate foundation and toe protection.
b) Stepped face: designed to limit wave run-up and overlapping. They are generally less massive than curved-face seawalls, but the general design requirements for structural stability are the same as that of curved face.
c) Combination: incorporates the advantages of both curved and stepped face seawalls.
d) Rubble: it is a rubblemound seawall breakwater placed along the beach. The rough surface tends to absorb and dissipate wave energy with a minimum of wave reflection and scour.

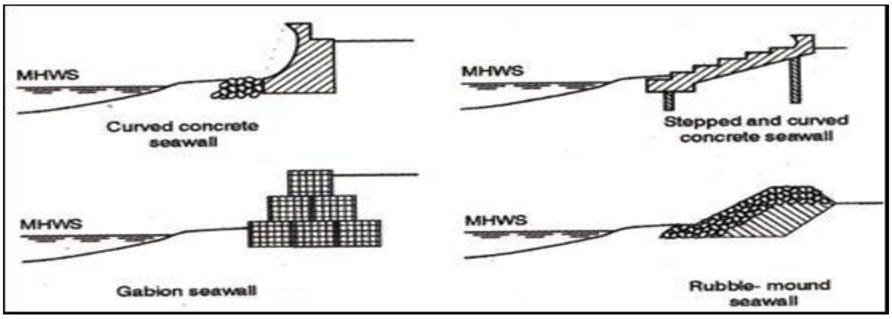


Fig.2.9 Types of Seawall

The toe protection is supplemental armouring of the bed surface in front of structure, which prevents waves from scouring and undercutting it. A typical rubblemound seawall is shown in Fig.2.10.

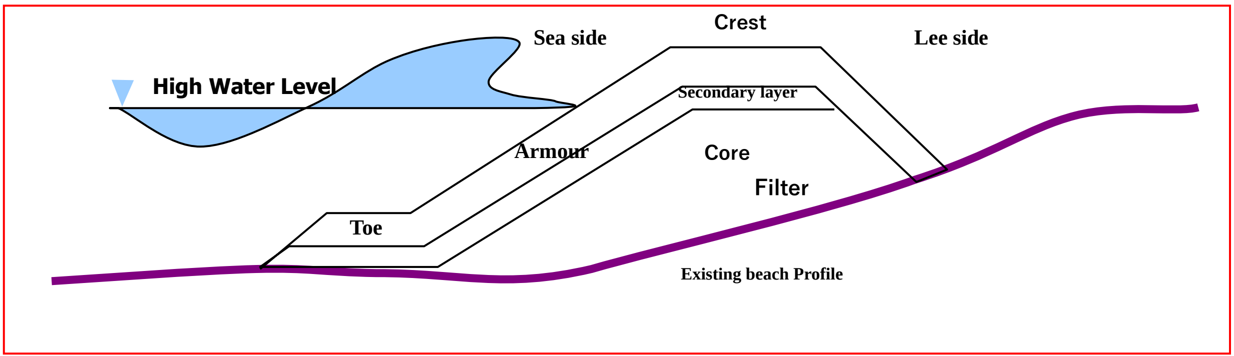


Fig.2.10 A typical rubblemound seawall

Design Procedure is as below:

1. Determine the tidal range for the site.
2. Determine the wave height.
3. Select suitable armour alternatives to resist the design wave.
4. Select armor unit size.
5. Determine potential run-up to set the crest elevation.
6. Determine amount of overtopping expected for low structures.
7. Design under-drainage features if they are required.
8. Provide for local surface runoff and overtopping runoff, and make any required provisions for other drainage facilities such as culverts and ditches.
9. Consider end conditions to avoid failure due to flanking.
10. Design toe protection.
11. Design filter and underlayers.
12. Provide for firm compaction of all fill and backfill materials. Also due allowance for compaction must be made in the cost estimate.
13. Develop cost estimate for each alternative.

B) REVETMENTS

Revetments are sloping hard structure designed to dissipate wave energy. They are built to protect embankment or other shoreline feature against erosion. Major components are armour layer, filter and toe. Armour layer provides the basic protection against wave action. Filter layer supports the armour, allows water to pass through the structure and prevents the underlying soil from washed through thearmour. The different types of toe protection prevent displacement of the seaward edge of the revetment as shown in Fig.2.11.

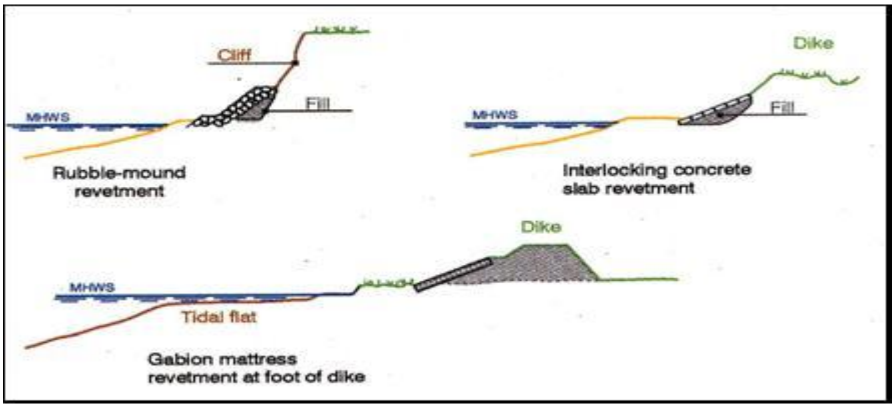


Fig. 2.11: Types of Revetments

Stone Revetment and Riprap:

a) The design practice for stone revetments is basically the same as for rubblemound breakwaters.
b) Since the primary function is to protect bank and preventing loss of upland material, more care should be exercised in filter design.
c) Application of geotextile filter is common.
d) Close attention should be paid to the hydraulic properties of the structure to prevent toe scouring, piping, bank instability and other hydraulically related failure modes.
e) Pressure build up in the soil behind the structure can result in leaching and loss of soil. Therefore, grading of the stone must be more tightly controlled than for breakwater design.

c) Bulkheads

Bulkheads are the retaining walls, which hold or prevent back fill from sliding and provide protection against light-to moderate wave action. They are used to protect eroding bluffs by retaining soil at the toe and increasing stability, or by protecting the toe from erosion and undermining. Bulkheads are used for reclamation projects, where a fill is needed seaward of the existing shore. Used in marinas and other structures where deep water is needed directly at the shore.

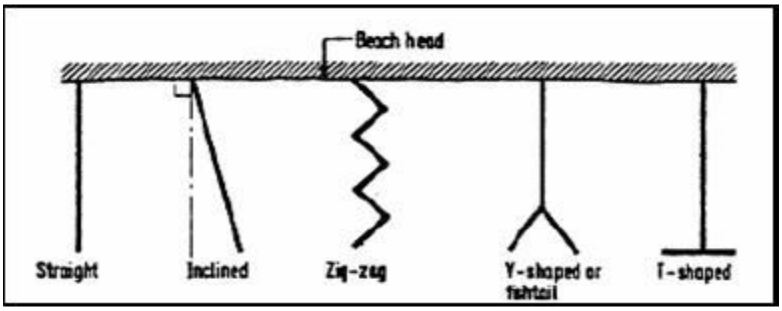
d) Detached seawall/bunds

The seawalls constructed in the inter tidal zone is called the detached seawall. A detached seawall consisting of seven segments with each segment of 67 m long is constructed at Udwada, Gujarat.

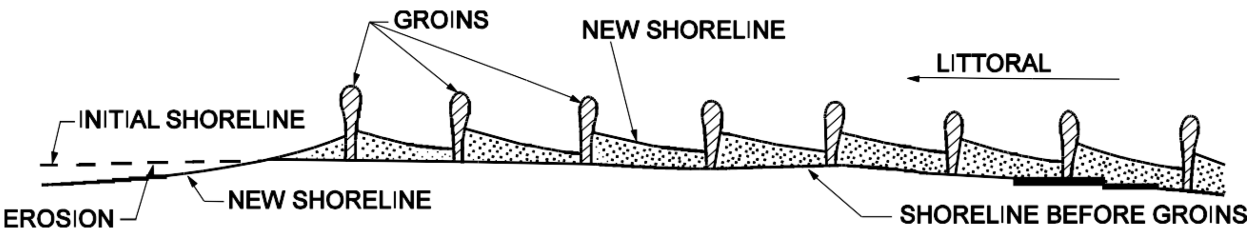
2.6.1 Structures to Trap Sediment Movement

a) Groynes

Groynes are structures placed perpendicular to the coastline to capture and hold sand that may be available in the littoral zone. Groynes are easy to implement and less costly than offshore structures. Groynes are built at few locations along the Kerala coast and North Chennai coast. If the understanding of the function of the Groyne and the location where it should be used is not clear, then the Groynes will become a disaster to the coastline.



Forms of Groynes



Groyne field, Groyne length, height and Groyne spacing

Step 1: Initially, provide the length of Groyne (L) equal to surf zone width.

Step 2: Assume S/L ratio in the range 2 to 3 and calculate spacing between Groynes in Groyne field.

Step 3: Length of the first Groyne (L_1) in the transition field is calculated by using the equation.

$L_1 = (1-(R/2)\tan 6^0)/(1+(R/2)\tan 6^0)$

where, L is the length of the previous Groynes in Groyne field R is the ratio of spacing to length of the Groyne

Step 4: Spacing between last Groyne in Groyne field and first Groyne in transition zone is given in the equation.

$S_1 = R \times L_1/(1+(R/2)\tan 6^0)$

Step 5: The length and spacing of remaining Groynes in transition zone is calculated using equation.

Step 6: Height of the Groyne is to be equal to the derived level considering the different factors.

Step 7: With the planned configuration, check whether the shoreline evolution using a numerical model (eg.: GENESIS, LITPACK). If the desired shoreline configuration is not achieved, then modify the length of the Groyne and repeat steps 2 to 7.

Structural design of Groyne

In the design of the rock armour structure, the following steps are considered.

1. Obtain the properties of the locally available stones with the range of size (gradation curve of a quarry).
2. Large fraction of the available stone can be used as material for the armour and the smaller fractions for core of the structure.
3. Determine the shape and dimensions of the armour protection, which typically involves increasing the thickness of the armour layer, so that during design wave condition, a stable structure is obtained. This is to be verified using physical model studies. As the dimensions of the armour protection are determined, the relative sizes of core material and armour material will vary to accommodate changes in the relative percentages of armour stone and core material required.
4. While designing the geometry of the cross-section of the armour, the availability of construction equipment at the location and the site characteristics are to be checked.

In most of the cases, rock is used as armour unit and the Groyne is designed for breaking wave condition. The details of design are available in Engineering Manual EM 1110-2-1100 (Part VI) of the US Army Corps of Engineers. Important steps are given below. Local wavelength at the toe of the structure α = Structural armour slope A, B, C_c are empirical coefficients and the values are provided in below Table.

Value of empirical coefficients used in Groyne design (EM 1110- 2-1100 Part VI)

Armor Type	A	B	Cc	Slope	Range of ξ	Wave condition
Stone	0.272	-1.749	4.179	1V to 1.5H	2.1-4.1	Breaking
Stone	0.198	-1.234	3.289	1V to 2.0H	1.8-3.4	Non-breaking

b) Offshore Breakwaters / Detached Seawalls

Offshore breakwaters are constructed parallel to the shore to reduce incoming wave energy and longshore transport of sand along the beach. Offshore breakwaters are mostly used in shore protection in an eroding coastline, where the loss of sediment occurs and a new recreational beach is required. Major types of offshore breakwaters used for coastal protection measures are single detached breakwater, multiple detached breakwaters, artificial headlands and submerged sill structure.

A salient will develop when sand is trapped behind the breakwater. This bulge in the shoreline can develop till it reaches the breakwater and this plan form is called tombolo. When a salient is formed, longshore transport can still go on (although on lower level) but a tombolo will act as a total barrier (like a Groyne) for longshore transport. Total barrier will cause downdrift erosion. Therefore, a salient seems a better shoreline response than a tombolo.

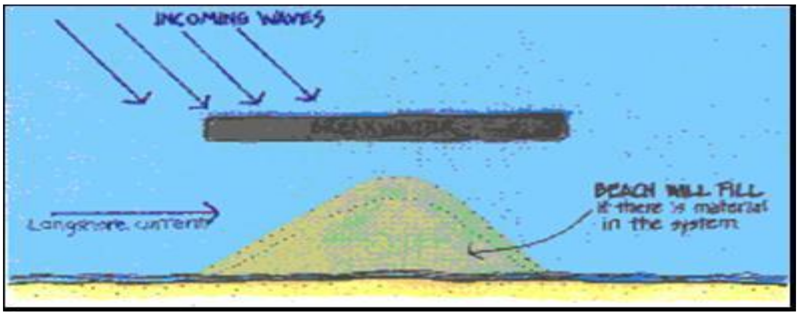
There are two reasons to use offshore breakwaters for beaches.

1) Reducing the volume of sand needed for the beach fill: this can be achieved by trapping sediment. When the offshore is close enough to the shore and enough sand is available (by longshore transport), tombolo will form behind the offshore breakwaters providing a wide recreational beach/bay. At these places (where a

tombolo will develop), the volume of sand needed for the beach is less.

2) Preventing transport of the beach sand (to elsewhere): without protection, the beach fill may be transported along the coast (of offshore) and the new beach will disappear. To avoid this transport of sand, offshore breakwaters can be used.

It is important to predict the beach response well. If the offshore breakwater is too short, then the beach can erode, but if the offshore breakwater is too long, a tombolo can develop resulting in negative effects.



Offshore Breakwater

The detached breakwater is constructed parallel to shore, which leads to formation of Salient/Tombolo on lee side.

L_B = Length of detached breakwater

x = Detached breakwater distance from the shoreline

x_{80} = Surf-zone width (Approx. 80% of littoral transport take place landward to this line)

$L_B{}^* = L_B / x$ $\qquad$ $X^* = x/X80$

$L_B{}^* < 0.6$ to 0.7 - Formation of Bell shaped Salient

$L_B{}^* > 0.9$ to 1.0 - Formation of Tombolo

2.7 DESIGN INFORMATION AND CONSIDERATIONS

Following information are required before the design of marine structure:

- Tidal levels
- Character of coastal currents
- Directions and force of prevailing winds
- Probable maximum height, force and intensity of waves
- Nature of seabed or foundation
- Cost and availability of materials of constructions

Following considerations are important in the design of coastal structure:

- The design should be based on the extreme phenomena of the wind and waves, and not on the mean or the average

- The height of coastal structures should be decided based on the provision for wave overtopping or non-overtopping conditions.

- The geotechnical investigation reports to confirm the foundation of the coastal structures.

2.8 DESIGN WATER LEVEL (DWL)

The design water level includes astronomical tides, storm surges and sea level rise are required to be consider for the design of coastal structures.

2.9 DESIGN WAVE CONDITIONS

Wind generated waves produce most powerful forces to which coastal structure are subjected. Wave characteristics are usually determined for deep water and then analytically propagated shoreward to the structure. Deep water wave heights and periods can be determined if wind speed, wind duration and fetch length data are available. Visual observations of storm waves may provide an indication of wave height, period, direction, storm duration and frequency of occurrence. Instruments such as wave rider buoys are available for recording wave height, period and direction of waves. Reliable deep-water wave data can be analysed to perform refraction and shoaling analysis to determine shallow water wave conditions.

The choice of design wave conditions for structural stability as well as functional performances of a rubble mound structure at any time depends critically on the water level at the site. Structure may be subjected to radically different type of wave action as the water level at the site varies. A given structure might be subjected to non-breaking, breaking and broken waves during different stages of a tidal cycle. The wave action may also vary along the length of the structure at a given time. Critical wave conditions that result in maximum forces on the structures like groins and jetties may occur at a location other than the seaward end of the structure. This possibility should be considered in choosing design wave and water level conditions. Generally, coastal structures are designed for breaking wave conditions, which exert maximum force on the structures. The breaking wave height (H_b) can be obtained from the depth of water (d_s), at the structure by the following relation.

$$\frac{H_b}{d_s} = 0.78 \tag{2.1}$$

If breaking in shallow water does not limit wave height, a non-breaking condition exists. A significant wave height (H_s) and significant wave period (T_z) would represent the characteristics of the real sea in the form of monochromatic or regular waves. To apply the significant wave concept, it is necessary to define the height and period parameters from wave observations. Munk (1944) defined significant wave height as the average height of the one-third highest waves ($H_{1/3}$ or H_s) and stated that it is about equal to the average height of the wave as estimated by an experienced observer. An alternative definition of H_s sometimes applied as 4 times standard deviation (σ) of the sea surface elevation i.e. H_s = σ.

$$H_s = 1.416 H_{rms} \tag{2.2}$$

Where, H_{rms} = Root mean square wave height

The selected design wave height depends on whether the structure is defined rigid, semi-rigid or flexible. As a rule of thumb, the design wave height is selected as follows:

For a rigid structure like sheet pile wall or concrete caisson, where a high wave within the wave train might cause failure of the entire structure, the design wave height is normally H_{max} or H_1 (H_1= 1.67 H_s i.e. average of highest 1 percent of all waves). For semi-rigid structures, the design wave height is selected from a range of H_1 to H_5 (H_5 = 1.37 H_si.e. average of highest five percent of all waves). For flexible structure such as rubble mound or riprap structure, the design wave height between H_s and H_{10} (H_{10} = 1.27 H_si.e. average of highest ten percent of all waves), which are based on the following factors:

- Degree of structural damage tolerable, associated maintenance & repair costs
- Availability of construction material & equipment
- Reliability of data used to estimate wave conditions

The design significant wave height (H_s) for 50 years, 100 years return period for deep water and maximum breaking wave height in shallow water depth may be consider for the design of coastal structures.

2.10 STATISTICAL PROCEDURE OF OCEAN WAVES

A comparison is made between the statistical behaviour of short term and long-term wave heights. The uncertainty of the maximum wave height is studied and results are compared with the design wave force load on the vertical barrier should be corresponding to the design wave height in eq.2.3, as recommended by Goda (1985).

$$Goda, H_{design} = 1.8H_s \tag{2.3}$$

$$H_b = 0.78\, h_b \tag{2.4}$$

(h_b= water depth at site)

Rayleigh distribution as an approximation to the distribution of individual wave heights according to Longuet-Higgins (1952).Figure 2.12, illustrates the definition of waves with ZUC (Zero Upcrossing)and ZDC (Zero Down crossing) methods.

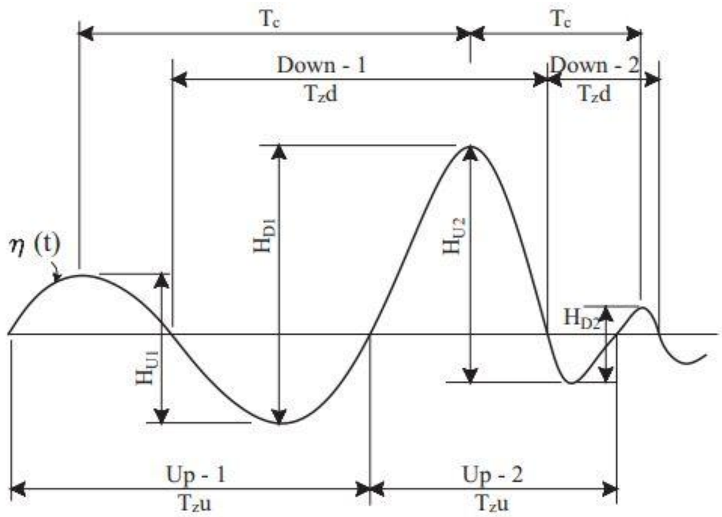


Fig 2.12: Definition of waves with ZUC and ZDC methods

****ZUC- Zero up crossing** *****ZDC-Zero down crossing**

$$H_{1/10} = 1.27H_s \tag{2.5}$$

$$H_{1/10} = 1.8,\ H_{max} = 2.172\ H_{rms} \tag{2.6}$$

These results represent the mean values of wave records taken together. Individual wave records containing less than 100 waves may result in deviations from the above mean relations.

2.11 WAVE STRUCTURE INTERACTION

A large segment of coastal engineering design requires an analysis of the functional and structural behaviour of a variety of coastal structures of paramount importance is the response of these structures to wave attack. Wave structure interaction can be divided in two parts:

i) Hydraulic Response
ii) Wave loadings and related structural response

Hydraulic Response

Design conditions for coastal structures include acceptable levels of hydraulic responses in terms of:

Wave Run-up & Run-down:

Wave run up level is one of the most important factors affecting the design of coastal structures, because it determines the design crest level of the structure in cases where no (or only marginal) overtopping is acceptable. Examples include dikes, revetments, and breakwaters with pedestrian traffic. Wind generated waves have wave periods which trigger wave breaking on almost all sloping structures. The wave breaking causes runup (R_u) and rundown (R_d) defined as the maximum and minimum water surface elevation measured vertically from the still water level (SWL) (Refer Fig. 2.13). Runup and Rundown characteristics depend on the height and steepness of the incident wave and its interaction with the preceding reflected wave, the surface roughness, and the permeability and porosity of the structure. The wave runup level is one of the most important factors affecting the design of coastal structures because it determines the design crest level of the structure in cases where no (or only marginal) overtopping is acceptable. The prediction of wave runup on a coastal structure is necessary in determining the crest height of the structure required for no overtopping of design waves (Shore Protection Manual, 1984).

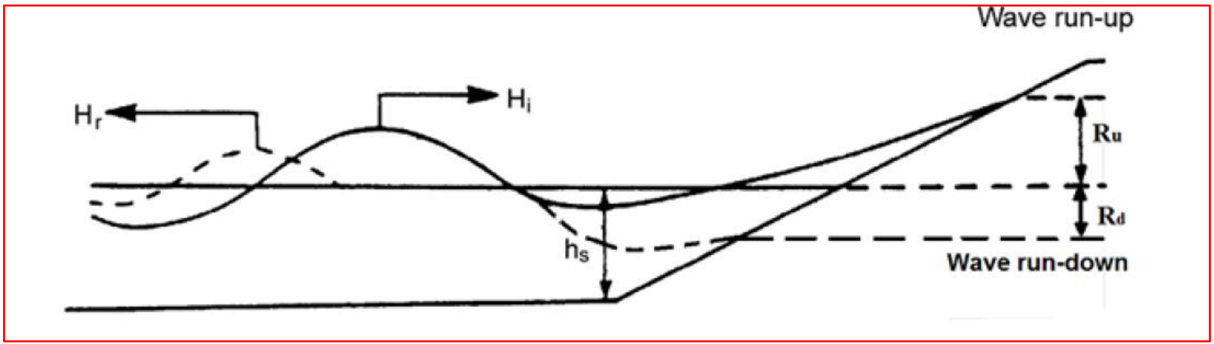


Fig 2.13: Wave runup and rundown

Wave Overtopping

It occurs when the structure crest height is smaller than the run up level. Overtopping discharge is an important design parameter because, it determines the crest level and the design of the upper part of the structure. Design levels of overtopping discharges frequently vary, from heavy overtopping of detached breakwaters and outer breakwaters without access roads, to very limited overtopping in cases where roads, storage areas, and moorings are close to the front of the structure.

Wave Transmission

Wave action behind a structure can be caused by wave overtopping and also by wave penetration as the structure is permeable. Waves generated from overtopping tend to have shorter periods than the incident waves. Generally, the transmitted wave periods are about half that of the incident waves. Permeable structures like single stone size rubble mounds and slotted screens allow wave transmission as a result of wave penetration. Design levels of transmitted waves depend on the use of the protected area.

To calculate the transmission coefficient, Kt in the physical model studies are done with help of wave Probes are analysed using Fast Fourier Transform (FFT) techniques to produce the spectral density histogram. The histogram plots the spectral energy against the wave frequency, resulting in the spectral variance, m_0. The time-domain parameter, H_t, known as the transmitted wave height, was calculated using the following equation.

$$H_{mo} = 4\sqrt{m_0} = H_t \tag{2.7}$$

Then Kt is calculated using the eq.2.8

$$K_t = \frac{H_t}{H_i} = H_t \tag{2.8}$$

Using the energy conservation law, the wave energy dissipation coefficient, Kl , is derived

$${K_t}^2 + {K_r}^2 + {K_l}^2 = 1 \tag{2.9}$$

$$K_l = \sqrt{1 - {K_t}^2 + {K_r}^2} \tag{2.10}$$

Wave Reflection

Coastal structures reflect some proportion of the incident wave energy. If reflection is significant, the interaction of incident and reflected waves can create an extremely confused sea with very steep waves that often are breaking. This is a difficult problem for many harbour entrance areas where steep waves can cause considerable manoeuvring problems for smaller vessels. Strong reflection also increases the sea bed erosion potential in front of protective structures. Waves reflected from some coastal structures may contribute to erosion of adjacent beaches.

The reflection coefficient, K_r , is the ratio of the reflected wave height, H_r , to the significant incident wave height, H_i in eq.2.11

$$K_r = \frac{H_r}{H_i} \tag{2.11}$$

CHAPTER-III

3.0 DESIGN OF RUBBLEMOUND STRUCTURES

Rubblemound structure consisting of graded layers of stone and an armour cover layer of stone or specially shaped concrete units are employed in the coastal zone as breakwater, jetties, groins, and seawalls. One advantage of rubblemound structure is that failure of armour cover layer is not sudden, but gradual, usually partial in extent, and spread over the duration of the storm. If damage does occur, the structure continues to function and the damage can be repaired after the storm abates during a period of lower waves. In some cases, it may be economical to use smaller size armour units, anticipate a certain degree of damage during a design storm, and provide provision for subsequent repair of structure.

Armour units must be of sufficient size to resist wave attack. However, if the entire structure consists of units of this size, the structure would allow extremely high wave energy transmission and finer material in foundation or embankment could easily be removed. Thus the unit sizes are graded, in layers, from the large exterior armour units to small quarry-run sizes and finer at the core and at the interface with the native soil bed.

Other rubblemound structure design consideration includes prevention of scour at the seaward toe, spreading of structure load, so there is no foundation failure owing to excessive loads and providing sufficient crest elevation and width so wave run-up and overtopping do not cause failure of the armour units on the leeward side of the structure or regeneration of excessive wave action in the lee side of the structure. The crest width may be governed by minimum roadway width needed for construction vehicles.

3.1 FACTORS AFFECTING ARMOUR UNIT STABILITY

3.1.1 Incident Wave Spectrum

Since failure of rubblemound structures is gradual, the significant wave height is most commonly used in the design formulas, although more conservative heights such as H_{10} have been used. Some consideration should be given to the expected duration of wave attack. When selecting a design wave height. It is also important to determine, whether the design wave will break on or before the structure or the water depth is sufficient for the wave to reflect without breaking. If breaking on the structure does occur, armour unit stability is then dependent on the type of breaker, which, in turn, depends on the wave height and period and the structure slope.

3.1.2 Armour Unit Size, Weight, Shape, Location & Method of Placement

Armour unit stability formulas give the weight of a unit required for stability. The resulting size depends on the specific weight of rock or concrete. Resistance to hydrodynamic forces is also developed by unit interlocking, which depends on the unit shape, gradation and the method by which the units are placed during construction. One of the goals of design of artificial concrete armour unit is to develop shapes that exhibit a high degree of interlocking with sufficient porosity when in place. Armour unit stability also depends on location in the breakwater, as exposure to wave attack is usually greater at the head of a breakwater than at some point along the trunk.

3.1.3 Armour Layer Thickness, Porosity & Slope

Two layer of armour units are usually used to achieve an optimum trade-off between initial and reserve stability, prevention of removal of smaller sizes from the under layer, and structure costs. Layer porosities usually vary between 35-55 percent, depending on armour unit shape and placement method. Low porosities increase the level of wave reflection, an effect that can be undesirable in certain situations. Low porosities also cause increased wave run-up, as well as internal pressure builds up due to return flow of wave run-up. Internal pressure build up contributes to armour unit instability. Breakwater armour units are all of one or a small range of sizes (usually within ± 25 percent of the average size), but stone riprap revetments often has a much longer size range. The size range of successive layer breakwater should increase, to decrease breakwater permeability. Typical seaward of breakwater and seawall slopes vary from 1 on 1.33 to 1 on 3, whereas revetment slopes as flat as 1 on 5. A flatter slope increase armour unit stability. It may also increase costs, since more material is required even though run-up is lower and thus a lower crest elevation may be used. An economic trade-off between unit size (layer thickness) and slope length can often be made. Depending on the degree of wave overtopping anticipated, the leeward slope of a breakwater can be steepened to near the angle of repose of the cover layer units (usually 1 on 1.25 as a limit).

3.1.4 Allowable Damage

The degree of damage is usually defined as the percent damage based on the volume of armour unit displaced in the zone of wave attack. A certain amount of initial settling of armour units increases the stability of the armour layer. Allowance of up to 10 to 20 percent damage for a design wave will significantly decrease the required armour unit size. However, the damage should not be allowed to that extent interior layers are exposed to direct wave attack. The allowable damage should depend on initial costs versus maintenance costs, as well as on the allowable risk to areas protected by the structure.

The damage criteria associated environmental loads for the seaside and leeside rocks shall be limited to s = 2% as per BS Code, and CIRIA rock manual. Similarly, the single armour layer, designed for no damage criteria (N_{od} = 0).

3.2 Determination of Armour Unit Stability

The stability of the rubblemound under ocean wave attack is the most important aspect in the design of rubblemound breakwaters. The stability of rubblemound structures depends primarily upon the stability of individual armour units on its seaward slope (Fig .4). Design of flexible rubblemound structures is complex as it involves various aspects such as wave-structure interaction, interlocking characteristics of armour, friction between armour and secondary layer etc. A major aspect in the design of rubblemound structures is the minimum weight of the armour units on the seaward slope, required to withstand the design waves. The resisting action of armour units either stones or concrete blocks, is very complex. It is not possible theoretically to say when exactly the maximum force is exerted on the rubblemound to lift the individual armour unit.

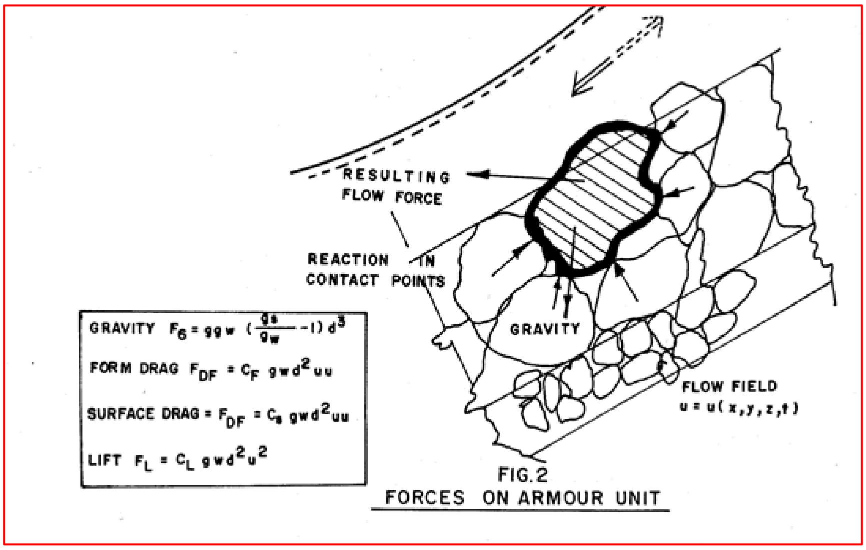


Fig 3.1: Forces on Armour Unit

Many studies were carried out on the hydraulic stability of individual armour unit on the seaward slope and several empirical formulae have been derived for the estimation of the weight of armour unit and are described below.

3.2.1 Hudson Formula

The comprehensive investigations were carried out by Hudson at US Army Corps of Engineers, Waterways Experiment Station, Vicksburg (USA). Based upon the experimental results, Hudson suggested the following formula for the armour units eq.3.1.

$$W = \frac{W_r}{(S_r - 1)^3 cot\theta} \tag{3.1}$$

Where,
W = weight of armour unit (kg)
W_r = unit weights of armour block (kg/cum)
H = wave height at the location of the proposed structure (m)
S_r = specific gravity of the armour units
θ = angle of breakwater slope measured with the horizontal
K_D = stability coefficient which varies with type of armour

Hudson had considered in his experiments wave periods varying from 0.8 sec to 2.65 sec and the armour layer slope from 1/1.25 to1/5. All the experiments were conducted for non-overtopping and non-breaking monochromatic waves. Hudson had also established K_D values for stones and artificially cast different types of blocks viz. Tetrapods, Tribars, etc. These values were worked out for no damage condition (i.e. the damage to the armour units would be less tan 1%). The Hudson formula is the most popular and has been in use for the last 50 years for the design of breakwaters, because of the fact that extensive K_D values are available based on the scale model tests. Most laboratory studies to evaluate K_D have used waves of constant period and height. For irregular waves, it is felt that the significant height is the most appropriate wave height to use for H in above equation. There have only been a limited number of evaluations of above equation using irregular waves.

More research with a variety of wave spectra is needed. The only effect of wave period on above equation is, in its effects on K_D through the breaking condition. Note that the required unit weight is a function of the wave height cubed, so armour unit weights increase rapidly with increased design wave height. Some values of are listed in table 1 below, as a function unit shape, location on the structure, and exposure to breaking or non-breaking waves (SPM 1984). These values are for zero allowable damage (less than 1%), units randomly placed in layers two units thick and minor or no wave overtopping.

A Tetrapod consists of four tapered legs extending outward from a common point at approximately equal angles to each other; a tribar has three parallel circular cylinders connected by a Y-shape member that connects to the centre point of each cylinder and is normal to axes of the three cylinders; and a dolos is like the letter H, with the vertical legs rotated 90° to each other. There is a significant effect of unit shape on the stability coefficient, which is inversely proportional to the armour unit weight. The stability coefficients given for riprap are for the weight of the median stone size in a gradation from 0.22W to 3.6W. The different types of concrete armour as shown in Figure 3.2.

Table 1: K_D Values for No-Damage Criteria and Minor Overtopping waves
(Refer SPM 1984 before using these values)

Armor Unit	n	Placement	Structure Trunk		Structure Head		Slope Cot θ
			Breaking Wave	Non-breaking Wave	Breaking Wave	Non-breaking Wave	
Quarry stones Smooth Rounded	2	Random	1.2	2.4	1.1	1.9	1.5 to 3.5
Smooth Rounded	>3	Random	1.6	3.2	1.4	2.3	
Rough Angular	1	Random	-	2.9	-	2.3	
Rough angular	2	Random	2.0	4.0	1.9	3.2	1.5
					1.6	2.8	2.0
					1.3	2.3	3.0
Rough angular	>3	Random	2.2	4.5	2.1	4.2	5
Rough angular	2	Special	5.8	7.0	5.3	6.4	5
Parallelepiped	2	Special	7 – 20	8.5 – 24	--	--	--
Graded angular	---	Random	2.2	2.5	--	--	--
Tetrapod & Quadripod	2	Random	7.0	8.0	5.0	6.0	1.5
					4.5	5.5	2.0
					3.5	4.0	3.0
Tribar	2	Random	9.0	10	8.3	9.0	1.5
					7.8	8.5	2.0
					6.0	6.5	3.0
Dolos	2	Random	15.8	31.8	8.0	16.0	2.0
					7.0	14.0	3.0
Modified cube	2	Random	6.5	7.5	--	5.0	5
Hexapod	2	Random	8.0	9.5	5.0	7.0	5
Toskane	2	Random	11.0	22.0	--	--	5
Tribar	1	Uniform	12.0	15.0	7.5	9.5	5

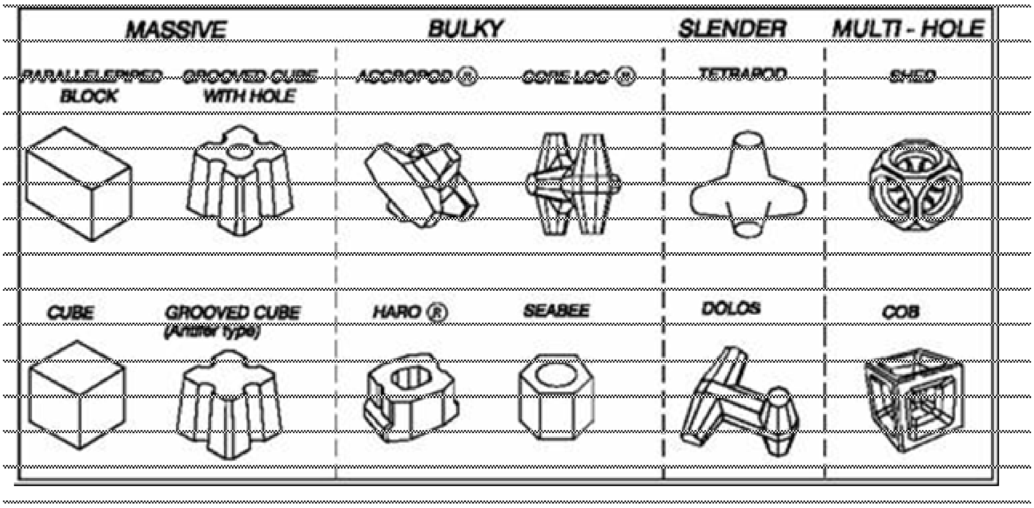


Fig 3.2 : Examples of concrete armour

3.2.2 PerBrunn's Formula

A number of formulae have been evolved by other investigators from time to time. Most of these formulae take into account the wave height, density of the armour units and angle of the breakwater slope. However, in the recent developments in the design of breakwaters, it is observed that weight of the armour unit is also related to wave period. Per Brunn et. al have analysed the flow conditions as a result of wave attack on the rubblemound structures - to determine the conditions which cause the maximum destructive force on the breakwater. They have considered the data available for slopes ranging from 1:1.5 to 1:5 from CERC and BEB tests. It has been concluded from their study that the breakwater slope (θ), the wave height (H) and the wave period (T) are the main parameters to be considered. A parameter called 'Surf Similarity parameter' comprising θ, H and T has been evolved as

$$\xi = \frac{tan\theta}{\sqrt{\frac{H}{L_o}}} = \sqrt{\frac{g}{2\pi}}\ tan\theta\ \frac{T}{\sqrt{H}} \qquad (3.2)$$

This parameter describes the overall flow characteristics like breaking waves, runup and run down and the effect of wave period. Per Brunn indicated that the forces trying to dislocate the armour units maximise with deep rundown conditions occurring simultaneously and repeatedly with collapsing, surging or plunging wave breaking conditions, thus corresponds to the range of ξ values between 2 & 3.

3.2.3 VanderMeer Formula

Van der Meer (1988) has given classification of coastal structure based on parameter which is called 'Stability Number'. The stability number is

$$N_s = \frac{H}{\Delta D}$$

Where as H = wave height,
Δ = relative mean density
D = Characteristics dimension of the armour unit (rock or concrete).

Small values of Ns give structure with large armour units whereas large values imply gravel beach and sand beaches. Two types of structure can be classified based on the response due to wave attack. These are 'statically stable structures' and, dynamically stable structures. Statically stable structure are structures where no or minor damage is allowed under design conditions. Damage is defined as displacement of armour units. The mass of individual units must be large enough to withstand the wave forces during design conditions. Caissons and traditionally designed breakwaters belong to the group of statically stable structures.

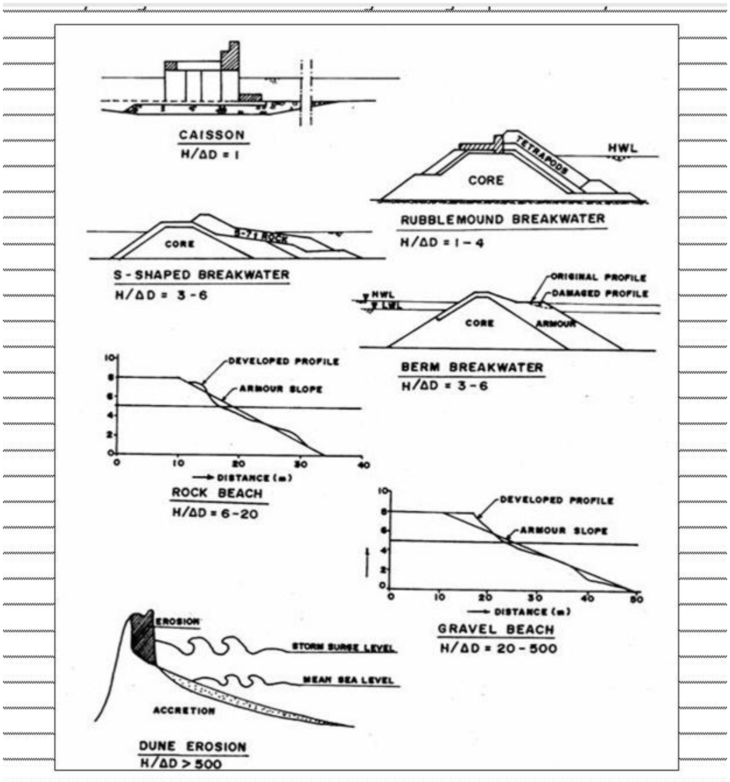


Fig 3.3 : Types of Structures as a Function of H/ΔD

The design is based on an optimum solution between design conditions, allowable damage and cost of construction and maintenance. Static stability is characterized by the design parameter 'damage' and can roughly be classified by H / ∆D = 1 – 4. Dynamically stable structures are structures where profile development is concerned. Units (stones, gravel or sand) are displaced by wave action until a profile is reached where the transport capacity along the profile is reduced to a very low level. Material around the still water is continuously moving during each run-up and run-down of water waves, but when the net transport capacity has become zero, the profile reaches an equilibrium state. Dynamic stability is characterized by the design parameter "profile" and can roughly classified by H / ∆D= 1 to 500. Types of structures with function of H / ∆D are shown in Fig. 3.3.

Van der Meer (1988) further examined dependence of wave period on the weight. He evolved stability formulae for rubble-mound breakwaters and revetments under random wave attack. The main shortcomings in the Hudson formula viz. wave period and randomness of waves have been solved in the investigations carried out by Van der Meer based on more than 250 laboratory tests, spectrum shape, groupness of waves and permeability of the core.

For Plunging Waves:

$$\frac{H_s}{\Delta D_{n50}} = 5.7\, P^{0.14} \left[\frac{S}{\sqrt{N}}\right] (\xi)^{-0.5} \tag{3.4}$$

For Surging Waves:

$$\frac{H_s}{\Delta D_{n50}} = 0.83\, P^{-0.2} \left[\frac{S}{\sqrt{N}}\right]^{0.2} (\xi)^{p} \sqrt{Cot\theta} \tag{3.5}$$

Where, Hs = Significant wave height
D_{n50} = Nominal diameter of the armour unit
ξ = Surf similarity parameter
P = Porosity
S = Damage Level
N = Number of waves
θ = Slope angle

Design values for the damage level S= 2–3 indicates ‘start of damage’ and is equivalent to ‘no damage’ criterion in the Hudson Formula. For the armour slope of 1:1.5 (cot ɵ = 1.5), S = 3 - 5 gives ‘intermediate damage’ where as S = 8 means ‘failure’. Based on the laboratory tests, Van der Meer (1988) concludes that the parameter such as grading of the armour, spectrum shape and groupness of wave have no influence on the stability of the breakwater.

3.3 Thickness of Armour Layer and Under Layer

The thickness of the cover under layers and the number of armour units required can be determined from the following formulae.

$$r = \mathrm{nK}\Delta \left(\frac{W}{w_r}\right)^{1/3} \tag{3.6}$$

Where, r = Average layer thickness (m)
n = No. of armour units in thickness comprising cover layer
K_Δ = Layer coefficient
W = Mass of armour unit in primary cover layer (kg)
w_r = Mass density of armour unit (kg / m3)

The placing density is given by

$$\frac{N_r}{A} = \mathrm{nK}\Delta \left[1 - \frac{P}{100}\left[\frac{w_r}{W}\right]^{\frac{2}{3}}\right] \tag{3.7}$$

Where, N_r = Required no. of individual armour units for a given surface.

A = Surface area
P = Average porosity of a cover layer in present.

3.4 Design of secondary layer

As per BS: 6349-part 7, the weight of the secondary layer rock for concrete armour shall vary between W/7 to W/15 the weight of concrete armour. However, 0.3 to 1.0 t, 1 to 3 t and 100 to 300 kg stones are proposed as secondary layer and apron extended from underneath toe mound to seabed for preliminary design.

Rock size gradation:

Layer Rock Size

Primary cover layer	W
Secondary layer	W / 10 to W / 15
Core	W /200 to W / 6000

Both the primary and secondary layers should be carried over to the crest and for a certain distance on the lee side so as to withstand any overtopping that may cause during severe storms.

3.5 Design of Filter criteria for various layers

The British Standard BS6349, Part 7, clause 4.4.3 provides guidance for sizing of under layers. The functionality of the filter is described as:

- To act as filter between core and armour layer,
- To provide a stable bed for the armour layer,
- To dissipate wave energy passing through the armour layer, and
- To protect the core material from moderate storms during construction.

The sizing of the underlayer material for armour units shall be as defined in BS6349 - Part 7, clause 4.4.3 and as recommended by the armour units developer. The filter stability between the core and filter shall be checked by the Terzaghi filter criteria (see BS 6349).

› $D_{15a} / D_{85u} < 4$ to 5

› $D_{15a} / D_{15u} < 20\text{-}25$

Where

- D_{xx} is the sieve rock diameter
- ‘u’ denotes under layer, and
- ‘a’ denotes armour

The Thomsen and Shuttler criteria shall be applied for filter criteria between filter and armour layer only:

› $D_{50a} / D_{50u} < 7$

The core material shall be quarry rock and well graded. It is important that the core material is not washed through the armour layers. From past experience in breakwater construction, the Terzaghi criteria shall be fulfilled between the size of armour and the filter or core material.

3.6 Design of toe-berm

According to BS:6349, Part-7, as per the toe configuration for rubblemound Breakwater in deep water, the secondary layer is extended to form the toe mound and the same size of rock may be assumed for preliminary design where the water depth exceeds twice the design H_s. The toe design is checked for relevant low and high water levels and corresponding wave conditions.

In shallow water, the toe of rubble mound breakwater is exposed to breaking wave action, which leads to high water particle velocity and reversal in the flow gradient. This can cause erosion of seabed material, as a result of which there will be a significant settlement in toe. Such settlement can be prevented by providing suitable toe protection. An important function of the toe mound is to provide the support to the armour. The width of the toe-berm should be provided to accommodate at least four rocks, and accordingly toe-berm on seaside consist of one armour unit and 3 units of rocks, which satisfy the BS standard for minimum 4 units in the toe. The toe-berm stability shall be checked through wave flume studies at design low water level conditions. The stability of toe berm formed by two layers of rocks on variable berm width & slope structure is given by Van der Meer et al (1995). The equation to calculate the toe size is given below as:

$$\frac{H_s}{\Lambda D_{n50}} = \left[2 + 6.2 \left(\frac{h_t}{h} \right)^{2.7} \right] n_{od}^{0.5}$$

Where,

H_s = Significant wave height (m)

$\Delta = (\rho r / \rho w) - 1$ = Relative buoyant density

D_{n50} = Median nominal diameter of rock (m)

h_t= Water depth on Toe (m)

h = Water depth near sea bed (m)

- N_{od}= number of unit displaced

The damage level for toe is measured in terms of N_{od} and is defined as number of unit displaced with in a strip width of D_{n50}. The stability of toe for design shall be as per the following conditions.

N_{od} = 0.5 No damage

N_{od} = 2.0 Acceptable damage

N_{od} = 4.0 Severe damage

The unit weight of the stones in toe-berm calculated based on Van der Meer improved formulae (Eq. 5.187 and 5.188) of CIRIA rock manual.

3.7 Design of Wave wall

The breakwater sections shall include a wave wall and the requirement of concrete and steel for the wave wall as per BS:6349-Part7, The wave wall on these sections shall resist impact pressures from wave up-rush. Wave wall design includes vehicular and non-vehicular traffic loads as per IRC Class 70R loading. The structural design of crest slab and parapet wall are required to be carried out by Project Authorities. The impact pressure on the crest element and the uplift forces is determined based on equation
presented below.
Wave pressure is given as,

- $P_w = KW_wL((H_S/H_C)-0.5)$

Where,

H_s = Significant wave height(m)

K = Constant(-)'

W_w= Seawater density(kg/cum)

L = Wave length corresponds to significant wave period (m)

H_c =Height of breakwater crest from design water level (m)

3.8 Design of Core

The core material shall consist of 10 to 500 kg well graded quarry run. Gradation of core will be estimated based on the filter criteria. The core rock with weight less than 10kg shall be restricted to 1% and 5% of total volume respectively. The porosity of 37 % may be considered for the rock gradation.

3.9 Lee side Armour of the Breakwater

The required stone size, Dn_{50} (m), at the rear side of coastal and marine structures for a given amount of acceptable damage, Sd, can be estimated with Van Gent Equation (2007).

$$D_{n50} = 0.036(cot\phi)^{0.5}(z_{1\%} - R_c)^{0.8}{R_c}^{0.2}\left(1 + 5\ \frac{R_{c2-rear}}{R_c - R_{c2-rear}}\right)^{0.4} \left(1 + \frac{R_{c2-front}}{H_s}\right)^{0.4}\left(\frac{S}{\sqrt{N}}\right)^{0.4} \tag{3.9}$$

$$\left(\frac{S}{\sqrt{N}}\right) = 0.00025(cot\phi)^{1.25}\left(\frac{z_{1\%}-R_c}{D_{n50}}\right)^2\left(\frac{R_c}{D_{n50}}\right)^{0.5}\left(1 + 5\ \frac{R_{c2-rear}}{R_c-R_{c2-rear}}\right)^{0.4} \left(1 + \frac{R_{c2-front}}{H_s}\right)^{0.4} \tag{3.10}$$

Were,

D_{n50} = nominal diameter of median rock (m)

Φ = slope angle of the armour layer on rear side (radians)

$Z_{1\%}$ = wave run -up

R_C = free board

R_{c2}-rear = free board rear

R_{c2}-front = free board front

S = Damage Number

N = No of Waves

3.10 Rear armour

Rear rock armour is designed based on the permissible wave overtopping and non-breaking incident waves propagating into the harbour, which are estimated from numerical modelling studies. The stability of rear armour under wave overtopping during the design storm event will be validated during 2D wave flume tests.

3.11 Scour Calculation

The geometry of the toe berm is also checked against scouring and scour depth arrived as per "The Mechanics of Scour in Marine Environment" by Sumer B.M. and Fredsøe J. (Ref Fig 3.4)

$$\frac{S_m}{H_s} = \frac{f(\alpha)}{[Sink(kh)]^{1.35}} \tag{3.11}$$

Where

$f(\alpha) = 0.30 - 1.77 \exp(-\alpha/15)$

S_m= expected scour depth in front of toe (m)

H_s = significant wave height (m)

h = water depth at the toe of the structure (m)

k = wave number = 2π / L

L= wave length

α = slope angle of armour layer on seaside

The maximum scour depth under wave action for sloping revetment is arrived from above equation (3.11). For severe scour protection without any trench/excavation, minimum top width of thrice the scour depth is maintained for toe berm as per Cl. 6.3.4.1, 3d, CIRIA C683.

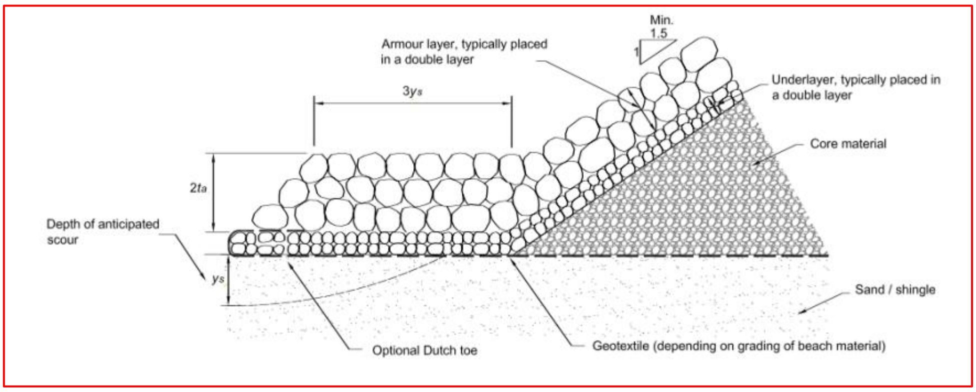


Fig 3.4: Severe Scour Protection – No Excavation

The maximum of ($3*D_{n50}$) and ($3*y_s$) is adopted as minimum toe-berm width to cater the function of toe for safe scour condition.

3.12 Permissible wave overtopping discharge

The allowable overtopping rate for safety and structural design shall be as per the specification in EurOtop manual 2007&2018 and no vehicle shall access the breakwater crest during the cyclone event. Therefore, the breakwaters shall be designed based on the wave overtopping criteria of 100 lit/s/m to estimate the crest level of the breakwater. The permissible overtopping for rubble mound breakwater in the range of 50-200 lit/s/m will not damage the crest and rear slope of the breakwater if these are well protected.

The EurOtop 2018 by Van der Meer et al. (2018) is widely renowned as the state of the art work and is used as design tool for various coastal structures (e.g. dikes, sea walls and rubble mound breakwaters). In the following section the improvements introduced compared to version 2007 (Pullen et al., 2007) will be discussed and an explanation will be given on how the overtopping discharge is obtained according to the modern EurOtop guidelines. Being the EurOtop 2018 approach, an improved version of the previous manual, the results concerning the EurOtop 2007 can be disregarded. Overtopping at low to zero freeboard conditions have often been overlooked in physical model studies, leading to a gap in available data on which empirical formulas were fitted. However, low to zero freeboard conditions are often encountered, e.g. breakwaters under construction, low-free board breakwaters, etc.

The EurOtop 2007 used a straight-line approach (see Equation 3.15), on log-linear paper, which in the low to zero free board region ($R_c/H_{mo} < 0.5$) often over-estimates the average overtopping. Where coefficients a and b are fitted, which depend on the type of coastal structure the formula is describing, e.g. dikes or rubble mound breakwaters. van der Meer and Bruce (2013) reanalysed old works providing a prediction to zero freeboard, which the EurOtop 2007 formula was not designed for. A curved line was proposed, introducing an exponent c, making the formula applicable to the full data range $R_c/H_{mo} > 0$. The formula widens the application area, but is very similar in the area with $R_c/H_{mo} > 0.5$, and has been introduced in the new EurOtop guidelines (2018).

The EurOtop 2007 approach,

$$\frac{qf_r}{\sqrt{\left(gH^3{}_{mo}\right)}} = \alpha \exp\left[-b\ \frac{R_c}{H_{m0}}\right] \tag{3.12}$$

The EurOtop 20018 approach,

$$\frac{qf_r}{\sqrt{\left(gH^3{}_{mo}\right)}} = \alpha \exp\left[-b\ \frac{R_c}{H_{m0}}\right] \tag{3.13}$$

The guidelines presented in the EurOtop for the latter case can be found in Chapter 6 (Armoured rubble slopes and mounds), where the formulas are based on Equation 2.1. for breaking wave height.

EurOtop (2018): Equations 6.9 and 6.10 – design and assessment approach

Mean value approach

$$\frac{q}{\sqrt{\left(gH^3{}_{mo}\right)}} = 0.09\exp\left[-\left(1.5\frac{R_c}{H_{mo}\gamma_f.\gamma_\beta.\gamma^*}\right)^{1.3}\right] \tag{3.14}$$

Where, γ_β is the influence factor for a berm breakwater. Note that Equation 6.9 EurOtop (2018): is similartoEquation6.5,but γ_f has been changed by γ_β.Equation 6.10 EurOtop (2018)with application of γ_β should be used if a design and assessment approach is needed.

Design & assessment approach

$$q = 0.1035\ \exp\left[-\left(1.35\frac{R_c}{H_{mo}\gamma_f.\gamma_\beta.\gamma^*}\right)^{1.3}\right]\sqrt{\left(gH^3{}_{mo}\right)} \tag{3.15}$$

EurOtop (2018): overtopping volumes. The prediction of the maximum overtopping volume can be determined by Equation 5.57, using further Equations 5.53 - 5.56.

$$V_{max} = a.\left[-(In(N_{ow}))^{\frac{1}{b}}\right] \tag{3.16}$$

$$a = \left(\frac{1}{\Gamma\left(1+\frac{1}{b}\right)}\right)\left(\frac{qT_m}{P_{ov}}\right) \tag{3.17}$$

$$b = 0.73 + 55\left(\frac{q}{gH_{mo}T_{m-1,0}}\right)^{0.8}\left(\frac{qT_m}{P_{ov}}\right) \tag{3.18}$$

$$P_{ov} = \frac{N_{ow}}{N_w} \tag{3.19}$$

$$P_{ov} = \exp\left[-\left(\sqrt{-In0.02}\,\frac{R_c}{R_{u2\%}}\right)^2\right] \tag{3.20}$$

Where,

Q = Discharge in m3/s/m
C_r = Crest width reduction factor (-): 3.06× exp (−1.5 × ($wHmo$))
C_w= Crest width (m)
H_{mo} = Spectral significant wave height (m)
R_C= Vertical distance between water level and rock crest (m)
γ_f = Roughness factor (-)
γ_β = Reduction factor due to oblique wave attack (-)

3.13 Wave runup (Eurotop 2018)

For relatively gentle slopes the breaker parameter is generally smaller than ξm-1,0 = 4. In case larger values are found for slopes of 1:2.5 or gentler, this can only be due to very small wave steepness, probably caused by severe wave breaking on a (very) shallow foreshore; very shallow foreshores. Steep slopes, say 1:2 up to vertical walls, give less wave run-up (and wave overtopping).

For a mean value approach, the wave run-up is expressed as:

$$\frac{R_{u2\%}}{H_{mo}} = 1.0\,.\,\gamma_f.\gamma_\beta\left(4-\frac{1.5}{\sqrt{\gamma_\beta.\xi_{m-1,0}}}\right) \tag{3.22}$$

where $Ru_{2\%}$ is the wave run-up height exceeded by 2% of the incoming waves [m], γb is the influence factor for a berm [-], γf is the influence factor for roughness elements on a slope [-], γβ is the influence factor for oblique wave attack [-] and $\xi_{m-1,0}$ is the breaker parameter [-].

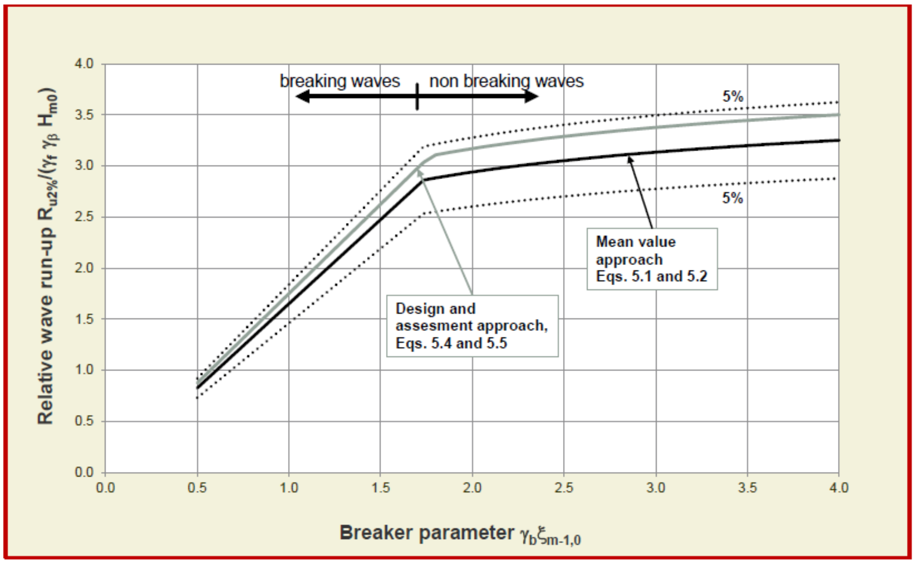


Fig 3.5: Relative 2%-wave run-up height Ru2%/Hm0 for relatively gentle slopes, as a function of the breaker parameter ξm-1,0 and other influence factors, (Source: Eurotop 2018)

The relative wave run-up height increases linearly with increasing $\xi_{m-1,0}$ in the range of breaking waves and small breaker parameters less than about $\xi_{m-1,0}$ = 1.8. For non-breaking waves and higher breaker parameter than this value, the increase is less steep as shown in Figure 3.4 and becomes more or less horizontal. In that area the influence of slope angle and wave steepness becomes much smaller. The relative wave run-up height $R_{u,2\%}$/Hm0 is also influenced by the geometry of the coastal dike or embankment seawall; the properties of the incoming waves; and possibly by the effect of wind.

3.14 Crest Elevation and Width

The maximum elevation on which water from breaking wave will run-up a given structure, determines the top elevation to which the structure must be built. The actual run up value depends on the characteristics of the structure (slope and roughness), the water depth at the toe of the structure and incident wave characteristics. The width of the crest depends greatly on the degree of allowable overtopping. Crest width be obtained from the following equation.

$$B = \mathrm{nK\Delta}\left[\frac{W}{w_r}\right]^{1/3}$$

Where, B = Crest width (m)

n = No. of stones, K_Δ = Layer coefficient

CHAPTER-IV

4.0 HYDRAULIC MODELLING OF COASTAL STRUCTURES IN WAVE FLUME

The conceptual design of breakwater is carried out using wave structure interaction is a complex phenomenon, which cannot be simulated by mathematical formulation. Hydraulic modelling of breakwater / seawall in the laboratory flume facilities evolves safe and optimal design of the structures. The primary objective of model testing of rubblemound structure is to check the stability of the structure up to and exceeding the design sea stage. However, modelling is also used to gather information on the hydraulic performance of the structure, in terms of reflection, run-up, over-topping and wave transmission. This information can then be used in the design process for the breakwater location, length and alignment to provide optimum wave protection for the harbour or other coastal installations. The hydraulic design of rubblemound structures needs to be finalized by physical model studies due to complex wave structure interaction. In addition to this as the flow conditions are not amenable to mathematical analysis, physical model study is the only source to finalise the cross section of the structure. Depending on type of phenomenon, the laws of similarity between hydraulic scale models and their prototype can be established based on dynamical consideration, dimensional analysis or differential equations. Dynamical similarity between model and proto involves geometrical and kinematical similarity and Newton's law of motion.

Physical models have scaled representations of reality in which a prototype system is duplicated as closely as possible on a smaller scale. Model studies have their own technical and practical limitations, but prove to be one of the best tools for the designer in arriving at a safe and stable design for breakwaters. The purpose of the model is to approximate and anticipate the prototype behavior through certain prescribed modelling laws. Many modelling approaches are followed in the study of natural systems. The physical model provides insight into a physical phenomenon that is not fully understood (Chakrabarthi, 1996). Froude's model law is applied because the essential forces involved are inertia, pressure and gravity whereas viscous and surface tension forces are neglected. The scale effects and uncertainty are the two major issues that decide the reliability of the model studies. To reduce scale effects the model should be as large as possible (Hughes, 1993) so that the Reynolds number of the flow is high and the flow is turbulent (Ouellet, 1970). And to minimize uncertainty the experiment has to be properly planned, experimental procedures and extrapolation methods should be standardized and sources of errors have to be minimized (Mishra, 2001).

4.1 FROUDE's MODEL LAW

The Froude criterion is a ratio of inertial forces to the gravitational forces, as follows.

$$\sqrt{\frac{Interia\ force}{Gravity\ Force}} = \sqrt{\frac{\rho L^2 V^2}{\rho L^3 g}} = \frac{V}{\sqrt{gL}} \qquad (4.1)$$

where 'V' and 'g' are the velocity of the flow and gravitational acceleration, respectively, and 'L' is the wavelength of the gravitational waves.

The Froude scaling law is applicable only when the predominant reaction force on the system is due to gravity, which controls the fluid flow in addition to the force of inertia. The application of the Froude scaling law in the physical model study requires that the Froude number in the prototype must be equal to the Froude number in the model.

$$\left(\frac{V}{\sqrt{gL}}\right)_p = \left(\frac{V}{\sqrt{gL}}\right)_m \qquad (4.2)$$

where the subscripts 'p' and 'm' denote the corresponding Froude number at prototype and model scale.

$$\lambda = \frac{L_p}{L_m} \qquad (4.3)$$

From the above equation, the parameter 'λ' is defined as the ratio of the prototype characteristic wavelength to the model wavelength. In this way, all the similitude parameters of the Froude scaling law can be defined. Based on Froude's scaling law, the main parameters used in the present research work.

4.2 PREDOMINANT VARIABLES

The present model study involves a complex structure comprising of a toe protected caisson breakwater and slotted barrier. The waves break over the barrier, loosing a major portion of energy and then loose some more energy while propagating in the zone between the structures. This phenomenon is difficult to express mathematically and one has to depend upon experimental investigations. The results of such investigations are more useful when expressed in the form of dimensionless relations. To arrive at such dimensionless relationships between different variables, dimensional analysis is carried out. The predominant variables considered for dimensional analysis in the present investigation are listed in Table 4.1.

Table 4.1 Predominant parameters influencing the performances of Vertical caisson Breakwater

Predominant Variable		**Dimension**
Wave Parameters	Incident Wave Height (H_i)	L
	Water Depth (d)	L
	Wave Period (T)	L
	Wavelength (L)	L
	Run-up (R_u)	L
	Particle Velocity (v)	LT^{-1}
Structural Parameter	Armour Unit Weight (W)	M
	Nominal Diameter (Dn_{50})	L
	Structural Height	L
	Relative mass density of Armour unit Weight (Δ)	$M^oL^oT^o$
Fluid Parameters	Mass Density (ρ)	ML^{-3}
	Dynamic Viscosity (υ)	$M L^{-1}T^{-1}$
External Effects	Acceleration due to Gravity (g)	LT^{-2}

4.3 DETAILS OF DIMENSIONAL ANALYSIS

For deep water wave conditions L and T are related by

The term gT^2 is used in the above equation to represent the wave length L, instead of taking L directly. This is because if L is used it would be depth specific while, gT^2 is independent of depth and represents the deep-water wave characteristics which can easily be transformed to shallow waters depending upon local bathymetry.

Considering the damage level S of the toe of caisson breakwater which is dependent on several independent parameters, their relationship can be expressed as follows

$$S = f\{H_i, T, d, L, D, d_1, \xi_{o,} \Delta, A_e, g, \rho, N_s, D_{n50}\}$$

By the application of Buckingham's π theorem, an equation of the form shown below is obtained.

$$S = A_e/D^2_{n50} = f\,[H_i/gT^2, H/\Delta Ns, d/L]$$

Similarly, wave force (F) on caisson breakwater which is dependent on several independent parameters, their relationship can be expressed as follows

$$F = f\{P, H_i, L, d, z, g, \rho,\}$$

By the application of Buckingham's π theorem, an equation of the form shown below is obtained.

$$F = f\{P/\rho gd, z/d, H_i/d, H_i/L, d/L\}$$

By the application of Buckingham's π theorem, an equation of the form shown below is obtained.

$$K_r = H_r/H_i = f\{H_i/d, H_i/L, d/L\}$$

Where,

A_e/D^2_{n50}	:	Dimensionless damage (S)
H_r/H_i	:	Transmission coefficient (K_r)
H_i/gT^2	:	Deepwater wave steepness
$H/\Delta D_{n50}$	:	Hudson's stability number (N_S)
d/L	:	Relative water depth
H_i/d	:	Relative water depth
H_i/L	:	Relative Wave Steepness
z/d	:	Relative depth parameter
$P/\rho gd$	:	Relative wave pressure

In case of rubblemound structures inertial forces are always present and viscous forces can be made negligible by selecting the proper model scale so that Reynolds number is greater than 3E+05. The ratio of inertial and gravitational forces is called as Froude's Number. It should be equal to one in case of dynamic similarity. Hence stability of rubblemound structure in model is based on Froude's law.

Breakwater/seawall sections are made geometrically similar based on Froude's model law. A section of the breakwater / seawall is placed normally across inside a wave flume, sufficiently away from the wave generator. The breakwater is subjected to attack by waves of probable maximum amplitude. Test whether displacement of structure material occurs. The wave processes is dependent upon depth, such as sloping, refraction and breaking height. Depth and breaking angle will be reproduced correctly in the wave flume models. Wave reflection from sloping surfaces and those containing rough or permeable surfaces like rubble mound structures are difficult to reproduce to scale unless no distortion is used.

Wave breaking is somewhat dependent upon beach slope or structure slopes so that distortion of these can influence this phenomenon. It may be possible to distort the major bed zone and revert to nearly zero distorted at boundaries where wave breaking is of greater importance.

The usual scales for wave action are,
Wave flume studies (no distortion) – 1: 20 to 1: 80
Basin studies – vertical 1: 60 & horizontal 1: 180
Seiching and surges (no distortion) – 1: 200 to 1: 1000

4.4 MODEL SCALE

The model was based on Froude's criterion of similitude. The model tests for the breakwater cross-sections conducted in 2-D wave flume for trunk portion and 3D wave flume for roundhead section with random wave generation by reproducing the section to suitable Geometrically Similar scales for different cross-sections. The physical model studies with Geometrically similar scale (GS) in the range of 1: 20 to 1:60 would be considered in the wave flume to test the hydraulic stability of the trunk (2D) and roundhead portion (3D) of the breakwater. The model scale depends on a number of factors varying from the prototype size, capacities of the testing facility in relation to the design wave and water level conditions as well as the available model armour units.

The final choice of the physical model is determined by the size of the available model armour unit. The various scales obtained are as follows:

$$\lambda = \frac{Dn_p . \Delta_p}{Dn_m . \Delta_m} \tag{4.4}$$

$$\Delta_p = \frac{\rho_{sp}}{\rho_{wp}} - 1 \tag{4.5}$$

$$\Delta_m = \frac{\rho_{sm}}{\rho_{wm}} - 1 \tag{4.6}$$

Where,

λ = Model length scale [-]
Dn_p= XblocPlus nominal diameter in prototype [m]
Dn_m =XblocPlus nominal diameter in model [m]
Δ_p = Relative density in prototype [-]
Δ_m = Relative density in model [-]
ρ_{sp} = Density of XblocPlus units in prototype [kg/m^3]
ρ_{sm} = Density of XblocPlus units in model [kg/m^3]
ρ_{wp} = Density of water in prototype [kg/m^3]
ρ_{wm} = Density of water in model [kg/m^3]

In the ideal situation the Δ_m of the model units is equal to the Δ_p of the prototype units. In this situation, the model scale will be geometrically most correct and there will be no influence on overtopping, wave run-up, wave run-down.

Typical Geometrically Similar scale of 1:60reproduced in the wave flume and other scale are as below

Model Scale 1:60	
Length – L	= 1:60
Area – L^2	= 1:3, 600
Volume – L^3	=1:2, 16,000
Time – $L^{1/2}$	= 1:7 746
Velocity – $L^{1/2}$	= 1:7 746

4.5 COMPENSATION FOR WEIGHT OF STONES

The density of stones in the prototype was considered as 2.60 t/cum and density of seawater is 1.025 t/cum. However, the density of stones in the model was 2.80 t/cum and fresh water with density 1.0 t/cum was used in the flume. As such, the weights of stones used in the model were compensated for these density differences by applying a weight factor, which was worked out as below:

$$\frac{W_1}{W_2} = \frac{2.6\,H^3 \,/\left(\frac{2.6}{1.025} - 1\right)^3}{2.6\,H^3 / \left(\frac{2.6}{1.025} - 1\right)^3} \qquad (4.7)$$

Where,

$W_2 = 0.6678\,W_1$

- W_1 = Weight of stones with density in prototype -- 2.6 t/cum
- W_2 = Weight of stones with density in model -- 2.80 t/cum

Hence considering the weight factor of 0.6678 for stones and 0.88 for concreter armour units in the model were worked out.

4.6 LIMITATIONS IN STUDIES OF FLUMES

Geometrically similar sectional models of hydraulic structure are investigated in flumes. Flumes can be horizontal or tilting. While interpreting results from studies in flumes, the following differences between “flume” and “field” conditions need be reckoned.

The standard practice in case of wave models till recently was to generate monochromatic waves (waves with fixed height and a fixed period). However, under actual sea conditions, a wave spectrum consists of waves of different heights and periods approach from different directions. Therefore, it would be necessary to generate wave of different heights and periods from different directions in the model producing similar wave spectra to that observed in nature. The wave spectrum varies from place to place and also from season to season in the same area of the sea. Various theoretical spectra have been suggested (JONSWAP, PM, OCHI, SCOTT etc). For reproduction of ocean waves in the model, it is necessary to adopt suitable theoretical spectra available to the particular area of the sea. Random sea wave generating facility in a model is used to generate appropriate wave spectra in the model. It is also possible to generate different wave spectra from different wave directions. For this purpose, a special multidirectional wave basin would have to be constructed.

Table 4.2 Comparison of Flume & Field conditions

Sr. No.	Flume conditions	Field conditions
1.	Limited range of depth and discharge can be investigated	Large range of depth and discharge is most common
2.	Slope can be varied between wide limits	Slopes relative constant over a particular reach
3.	Velocity can be varied over a wider range but can not be fluctuated over a short time period	Velocities highly variable over a short time period
4.	Variation in stream power and shear stress is principally the result of slope variation	Variation in stream power and shear stress is principally the result of depth variation
5.	Width is invariable due to rigid flume banks	Banks are susceptible to erosion and width is highly variation
6.	Similar bed configuration over entire flume length in equilibrium condition	Non-uniform velocity and depth in natural stream result in multiple bed configuration across and along a reach
7.	Experiments rarely run with large clay cloud	Large suspended clay is common

4.7 WAVE FLUME STUDIES

The alignments of breakwaters are finalized after studies in three-dimensional wave models. The finalized alignment indicates the portions of breakwater where there will be normal attack of waves and the portion where there will be angular attack. Similarly, the alignment indicates water depths along the breakwater length. After breakwater is designed by taking into consideration the above parameters, the sectional model of breakwater is laid in the wave flume to test its hydraulic stability. The hydraulic stability of trunk portion of breakwater section would be confirmed through 2D wave flume for normal attack of waves and roundhead portion of breakwater through 3D wave flume for oblique attack of waves.

The breakwater section is constructed at the wave flume provided with glass to facilitate viewing the model as well as wave activity. The section as per the design, reduced to the model scales is first marked on the glass. The weight of graded stone and the weight of the model armour unit are to be worked out from the following law.

$$\frac{(W_a)m}{(W_a)p} = \frac{(\gamma_a)m}{(\gamma_a)p}\left(\frac{L_m}{L_p}\right)^3\left[\frac{(S_a)p-1}{(S_a)m-1}\right]^3$$

Where, subscript m denotes model and subscript p denotes prototype

W_a= Weight of armour unit

γ_a= Specific weight of armour unit

S_a= Specific gravity of an armour unit relative to sea water

L = Length scale

Stones of various weights are picked from ready stock and laid into the flume so as to follow the marked section. Artificial concrete armour blocks, which have been cast pre-hand, are also laid in the section in similar manner. Breakwater sections in a monochromatic wave flume are generally tested for significant waves. However, they are also tested for worst conditions of breaking waves at low water and high water. A typical test is required to run for about 2 to 4 hours. The various hydrodynamic parameters such as wave run-up, rundown, transmission, reflection, etc. will be observed. Actually, measuring of dislodged units and finding out its percentage to the total number of unit in the particular layer in the test section. The damage to the armour unit up to 1% is acceptable. First order damage (1-5%) is permitted in cases in order to reduce capital cost of the structure. However, maintenance of the structure is periodically carried out when damage occurs. For very fine material like core material, it is not possible to measure actual number of units dislodged. In this case, area of damage is measured and its percentage to design area is worked out. Sometimes the concrete model blocks would be so small in their size or the shape may be so peculiar that it is not possible to use concrete (it is not possible to use coarse aggregate in the concrete).

Under such circumstances iron filing, small pieces of nails and cement mortar are used in such a fashion that appropriate weight of the model block is obtained.The trunk section will be tested for finding out damage to armour units, measuring disturbance on lee side due to overtopping, deciding optimum length of toe berm, deciding level at which leeside armour should be stopped and stability of sub grades during construction phase. The round head section which laid to the scale in the hammer head portion of the wave flume, will be tested for different predominant wave directions namely SSE (South of South East) and ESE (East of South East). The damage is to be observed quadrant wise separately. The results of wave flume studies will be utilized to finales of trunk section and round head section.

4.8 WAVE MEASUREMENT AND CALIBRATION

The wave heights were measured by capacitance type wave probes. A gauge was fixed at about 30 m water depth in Prototype and at corresponding depth for geometrical scale adopted, in front of the model and was analysed by computer. The desired wave conditions in front of the model were obtained by matching of desired spectrum and achieved spectrum by iterative procedure. The typical wave spectrum (Pierson- Mosckowitz (PM) spectrum) generated during wave flume studies.

4.9 WAVE FLUME TEST PROCEDURE

The section of the breakwater was constructed to a geometrically similar model scale in the wave flume. The number of armour units provided on the seaside and number of stones in the toe-berm were counted initially, before starting the test. After conducting the tests for one-hour duration, the numbers of armour units displaced from its original position were recorded and percentage damage to the armour of breakwater was determined. During the test, extent of splashing / overtopping over the crest was observed.

CHAPTER-V

5.0 DEVIATIONS IN DESIGN AND CONSTRUCTION OF SEAWALLS

5.1 POSITION OF THE SEAWALL

For locating the seawall, beach profile and the water levels are important. The highest water level helps in deciding the exact crest level, while the lowest water level guides the location of the toe. The bed slope in front of a coastal structure also has an important bearing on the extent of damage to the structure and wave run up over the structure. The seawall should be located in such a position that the maximum wave attack is taken by the armour slope and the toe. It should be kept in mind that seawall is for dissipating the wave energy and not merely for avoiding inundation of the land.

5.2 UNDER DESIGN OF ARMOURS

In case of seawalls provided with a large percentage of undersized armour, there has been considerable displacement and dislocation of armours. The stones in the lower reaches have been excessively subjected to such forces. The displacement of the armours has resulted in the exposure of secondary layer, which is mostly removed from the section that has created small pockets of breaches completely exposed to the fury of waves.

5.3 TOE PROTECTION

Toe protection is supplemental armouring of the beach or bottom surface in front of a structure, which prevents waves from scouring and undercutting it. Toe stability is essential because failure of the toe will generally lead to failure throughout the entire structure. Design of toe protection for seawalls must consider geo-technical as well as hydraulic factors. Using hydraulic considerations, the toe apron should be at least twice the incident wave height for sheet-pile walls and equal to the incident wave height for gravity walls.

5.4 INADEQUATE OR NO-PROVISION OF FILTERS

Many rubblemound structures have failed due to no or inadequate provision of filter underneath (Photo- 5.1). As a consequence, the insitu soil is leached resulting in the collapse of the structure. In some cases, the toe of the seawall sank over the years due to inadequate filter and removal of insitu bed material.

Photo 5.1 Inadequate Filter Layer

With the failure of the toe, armours in the slope, which were otherwise intact, were dislodged by gravity and wave forces. These stones occupied the toe portion and sank further due to the absence of filter. Thus the failure is progressive and renders the seawall ineffective within a short period, if not attended promptly. It is necessary to provide a proper filter before reforming the section.

5.5 OVERTOPPING

Underestimation of design wave or the maximum water level leads to excessive overtopping of seawalls and eventual failure particularly of leeside slope and damage to reclamation, if any. The leeside fill and the seawall core (or secondary layer) should be sandwiched by an appropriate filter and adequate drain be provided for safe discharge of overtopped water.

5.6 ROUNDED STONES

The in-place stability of an armour unit is dependent on the interlocking achieved at placement of armours. In order to achieve efficient interlocking, the rock should be sound and the individual units should have sharp edges. Blunt or round edges result in poor interlocking and hence poor stability. Rounded stones result in lower porosity and are less efficient in dissipation of wave energy. The in-place stability of such units is highly precarious and sensitive to small disturbances. Hence such stones should not be used in rubblemound structures.

5.7 WEAK POCKETS

Several weak spots are often present in rubblemound structures, which may be attributable to reasons such as lack of supervision or deliberate attempts to dispose of undersized stones etc. (Photo-3.2). The failure thus initiated could lead to the failure of the structure as a whole. Concentration of stones much smaller than the required armour should therefore be avoided at any cost, otherwise the entire structure, though carefully executed, can become functionally ineffective.

Photo 5.2 Pockets in Armour Layer

5.8 DISCONTINUITIES IN SEAWALLS

The discontinuities in the seawalls are often forced to meet the needs of certain activities of the coastal population such as beaching of small crafts, providing pedestrian access to beach etc. (Photo-5.3). If the seawall on both sides is abruptly terminated without proper placement of armours in corners, in the event of severe wave attack this is one of the most vulnerable locations along the seawall and could be the first to fail. The area in the lee of the structure would experience considerable inundation.

Photo 5.3 Pockets in Armour Layer

These waters, while flowing back to the beach, would erode considerable in-situ soil which could undermine the stability of seawall on both sides of the opening. Where such gaps are unavoidable, proper care should be taken to terminate the seawall, which should be keyed with sufficient returns and by providing armours on the leeside to some length along the seawall depending on the expected level and extent of inundation.

5.9 ARMOUR IN SINGLE LAYER AND/ OR PITCHED

Several constructions in the country have been taken up with revetment type pitching of rubble (Photo-5.4) along the beach instead of normal type of rubblemound structures. Such structures result in poor dissipation of wave energy due to very low porosity of the top layer and higher wave run-up. This calls for increasing the crest level, which would upset the cost, thereby defeating the economy considerations. In the event of these armours being dislodged, there is no reserve or cover left to protect the secondary layer. It is therefore recommended to adopt 'two-layer pell-mell' type of rubblemound structures in marine environment.

Photo 5.4 Single Layer Armour Revetment

5.10 UNSOUND TEMPORARY MEASURES

When erosion is active, authorities at site are compelled to do 'something' which normally assumes the form of dumping available rubble (Photo-3.5). Often, such exercises end up in a fiasco. The benefits derived are only apparent and not even temporary. On many occasions, by the time the work commences, the fury of waves subsides and the situation is abated before any work is carried out. It is therefore necessary to give due technical consideration before affecting any protective measure, whether permanent or temporary.

Photo 5.5 Stones Dumped as Temporary Measure

5.11 PLANNING OF CONSTRUCTION PROGRAMME

From the bathymetry in the vicinity of the coastal structure and the data regarding littoral drift, the pattern of erosion/accretion can be anticipated. The construction of beach protection structures in such regions should be undertaken at the appropriate time. For example, construction of a seawall along the coast where considerable erosion has been taking place should be started immediately after the monsoon, when the eroded levels are the lowest and wave action is comparatively reduced. In an eroding coastline, if a long length of the coast, say about 500 m, is to be protected with a seawall and it is not possible to construct this seawall in one season, then it is best to start construction of the seawall from both ends and proceed towards the centre rather than constructing the seawall from one end only. With such planning, the extent of erosion along the beach and penetration into the beach in the coastline is reduced as compared to the extent and penetration of erosion when the construction of seawall is started from one end only.

5.12 MAINTENANCE OF COASTAL STRUCTURES

The most important aspect is the post construction maintenance of coastal structures. It is a general experience that once these structures are constructed, hardly any maintenance of the structure is undertaken. It must be remembered that no coastal structure is permanent, since it has to bear the brunt of coastal wave attack, which is random in nature and acts at different locations along the structure due to tidal fluctuations. The toe normally suffers initial damage, which leads to subsequent damage to structure. It is, therefore, essential to replenish the damaged toe periodically. Many times, the leeside slope and berm or the crest are gradually damaged due to constant overtopping and same should be repaired. If proper maintenance at regular interval is undertaken, it is possible to prevent these damages and improve the performance of the structure (Photo-5.6).

Photo 5.6 Well Maintained Seawall

CHAPTER-VI

CASE STUDIES 1

1.0 Design of deep-water sea dyke for the proposed dam of Kalpasar Project in Gulf of Khambhat, Gujarat

Kalpasar Project envisages construction of a proposed dam of about 26.70 km long across the Gulf of Khambat in deep water with bed level of -25.0 m with respect to MSL and extension of dam on either side in shallow water tidal flats having length of about 19.83 km on Bhavnagar side and 13.60 km on Dahej side. The proposed length of the dam is about 60.13 km for establishing a huge fresh water coastal reservoir for irrigation, drinking and industrial purposes (Fig-6.1).

Fig 6.1: Index and location/layout plan of sea dyke for the proposed dam of Kalpasar Project in Gulf of Khambhat, Gujarat

The desk and wave flume studies for the design cross-sections of sea dyke to the proposed Kalpasar dam considering the maximum Design Water Level (DWL) of +8.765 m with respect to MSL and maximum Significant Wave height (H_s) of8.1 m have been conducted with XblocPlus and Accropode-II armour units recommended by Kalpasar Department. The design cross-sections of sea dyke with XblocPlus and Accropode-II armour units of 0.75 cum, 3 cum, 6 cum and 14 cum with 1:1.33 slope on sea side at various bed levels from +5.0 m to -25.0 m with respect to MSL have been evolved based on desk and wave flume studies. The top of the parapet provided from el.+12.0 m to el. +19.0 m with respect to MSL. A clear carriage way width of 10 m is provided on the crest slab.

The sea dyke sections were designed considering Hudson's stability coefficient K_D of 12 for an armour slope steepness of 3V:4H (i.e. 1:1.33) corresponding to non-breaking waves and a seabed slope of 1% for conservative sides. The concrete armour layer of XblocPlus and Accropode-II were designed for no damage criteria (N_{od} = 0).According to BS: 6349-Part-7, the relevant configurations of the toe-berms for the rubblemound sea dyke have been designed. The sea dyke crest elevation fixed considering permissible wave overtopping in the range of 50-200 lit/s/mas per the specification in EurOtop manual 2008 & 2018. The design calculations of different cross-sections of sea dyke with XblocPlus and Accropode-II armour units are shown in Table-6.1.

Table 1: Design calculations of protection structure/breakwater to the dam at Kalpasar project

Sea bed level in mw. r. to		Depth of water In m	Design wave height in m	Armour volume in cum	Armour weight In ton	Weight of stones in Secondary	Toe-berm level in m	Weight of stones in Toe-berm
MSL	CD							
5.0	10.5	3.765	2.94	0.75	1.8	0.1 to 0.3 t	+7.40	0.3 to 1 t
2.0	7.5	6.765	5.28	3.0	7.2	0.3 to 1 t	+5.65	2 to 4 t
0.0	5.5	8.765	6.84	6.0	14.4	1 to 3 t	+4.10	4 to 6 t
-5.0	0.5	13.765	8.10	14.0	33.6	1 to 3 t	+0.10	6 to 10 t
-25.0	-19.5	33.765	8.10	14.0	33.6	1 to 3 t	-9.50	6 to 10 t

The typical cross-sections of sea dyke with XblocPlus and Accropode-II armour units at -25 m bed level are shown in Figs 6.2 & 6.3 respectively.

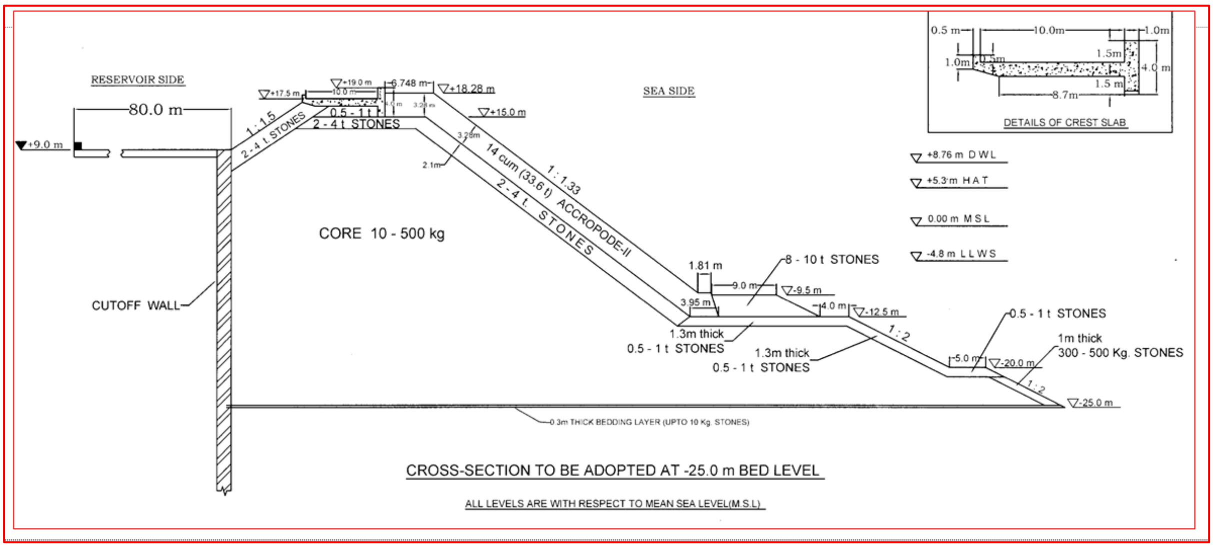


Fig 6.2. Cross-section of sea dyke with Accropode-II armour units at -25 m bed level for the proposed dam of Kalpasar Project, Gujarat

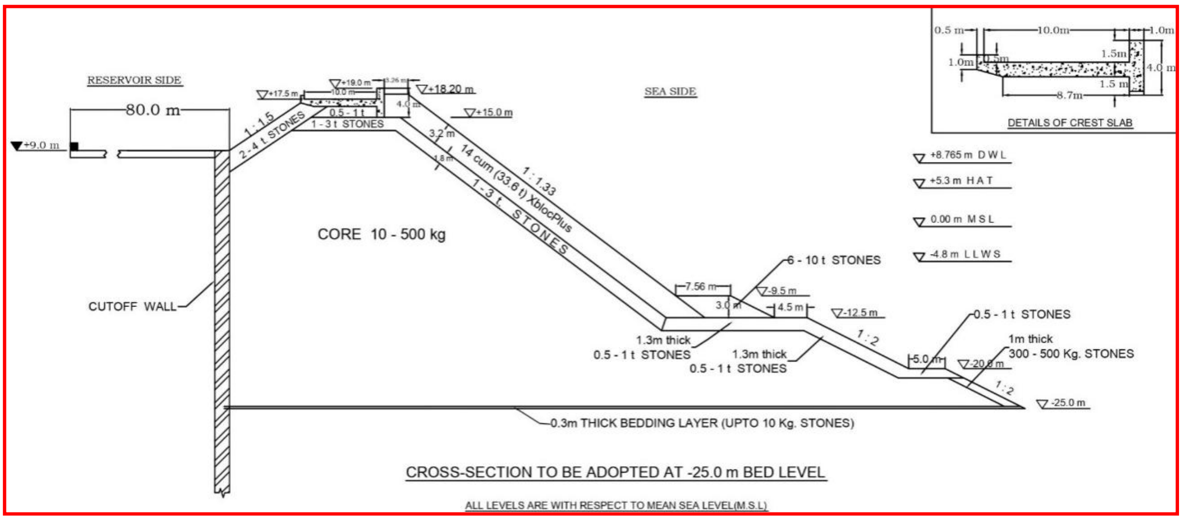


Fig 6.3. Cross-section of sea dyke with XblocPlus armour units at -25 m bed level for the proposed dam of Kalpasar Project, Gujarat

The wave flume model tests for the design of sea dyke have been conducted in 2-D random wave flume by reproducing the section to Geometrically Similar scales of 1:60, 1:44 and 1:35 for different cross-sections. The wave flume facility is equipped with a fully automated computerised random wave generating system comprising of hydro-servo system and wave board assembly (Fig.6.4). The placement of XblocPlus and Accropode-II armour model units for the sea dyke section carried out at wave flume studies as per the guidance of M/s DMC, the Netherland and M/s CLI, France (Fig.6.5 & 6.6).The Pierson-Mosckowitz (P-M) wave spectrum generated during wave flume studies (Fig.6.7).

Fig.6.4 The wave flume facility with random wave generating system

Fig.6.5 Placement of AccropodeTMII model units in the wave flume

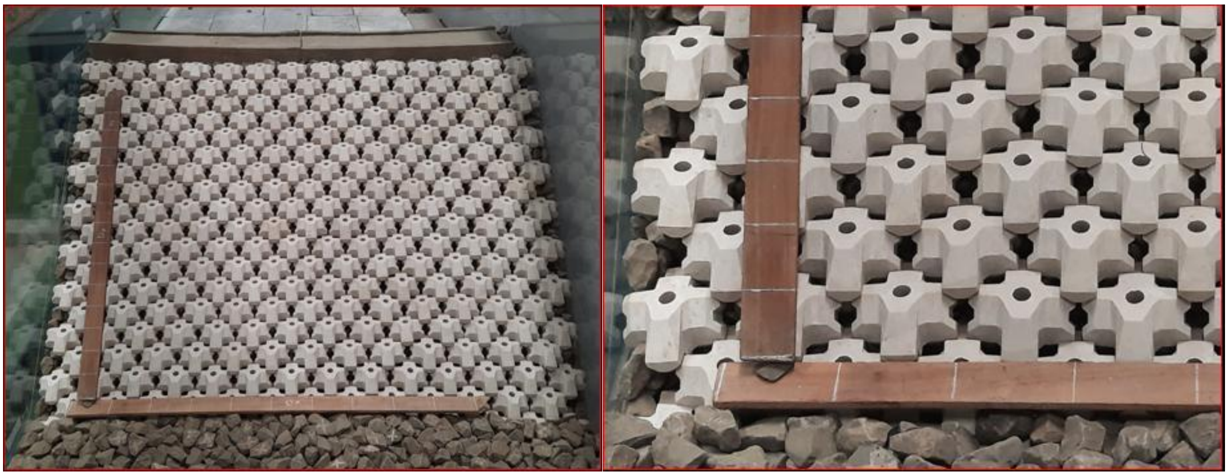


Fig. 6.6 Placement of XblocPlus model units in the wave flume

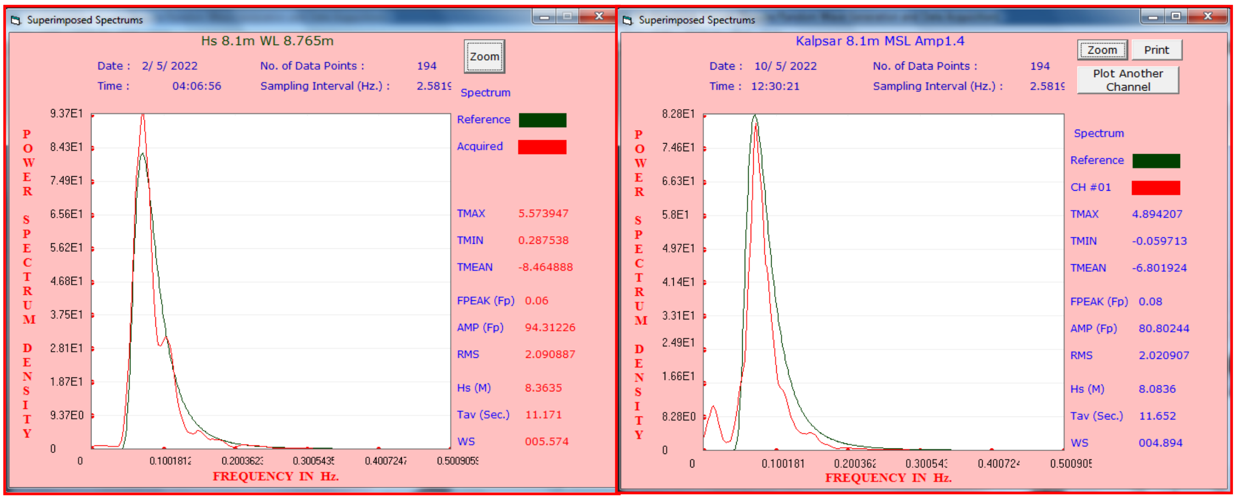


Fig.6.7 Typical Pierson-Mosckowitz (P-M) wave spectrum generated during wave flume studies

Fig.6.8 Wave action on the XblocPlus & Accropode-II armour units of sea dyke cross-section at -25.0 m bed level during (Hs) of 8.1 m at (DWL) of +8.765 m

CASE STUDIES - 2

2.0 Design of breakwater considering permissible wave overtopping discharge for the development of Port at Vadhvan, Maharashtra, India

The Government of India (GOI) has a proposal to develop a major Greenfield Port at Vadhavan with joint venture between Jawaharlal Nehru Port working under Ministry of Surface transport, GOI and Maharashtra Maritime Board (MMB), Government of Maharashtra (GOM). In this context, various hydraulic model studies have been carried out at CWPRS, Pune. The layout plan for the breakwater for the development of new Port at Vadhavan, Maharashtra as shown in Figure-1 was decided based on mathematical model studies. Based on the desk and wave flume studies, the safe and optimal design cross-sections of rubble mound breakwater with ACCROPODE^{TM}II at various bed levels have been evolved.

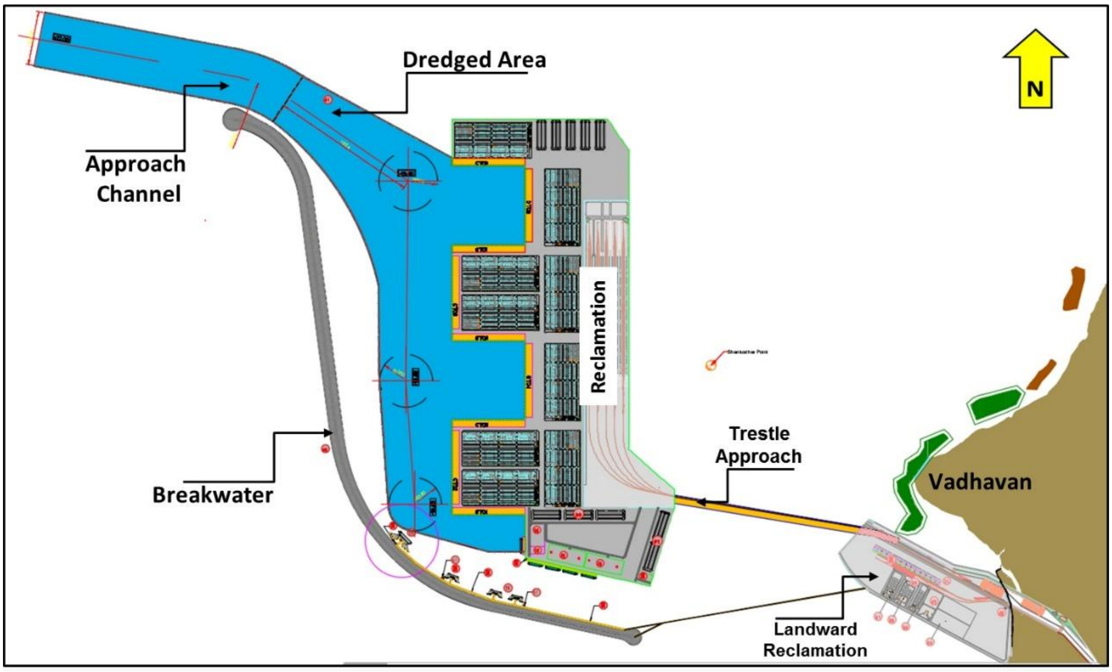


Fig 1. Layout of Proposed Vadhavan port.

2.1 DESIGN CONDITIONS

The Design Water Level (DWL) of + 6.9 m CD including tidal level, storm surge and Sea Level Rise have been considered for the design of breakwater. Based on the extreme value analysis studies carried out at CWPRS for Vadhavan (CWPRS Technical Report no. 5581 of March 2018), the significant wave height (Hs) varies from 6.8 m to 7.5 m for 100 years return period at different bed levels along breakwater. The waves are predominant from north-west and south-west direction. The design wave conditions for no damage with the significant wave height (Hs) of 6.8 m to 7.5 m were considered for evolving the design of breakwaters.

2.3 DESIGN OF CONVENTIONAL RUBBLEMOUND BREAKWATER

Design of flexible rubble mound structures is complex as it involves various aspects such as wave-structure interaction, interlocking characteristics of armour, friction between armour and secondary layer etc. A major aspect in the design of rubblemound structures is the optimum weight of the armour units on the seaward slope, required to withstand the design waves. Extensive studies were carried out on the hydraulic stability of individual armour units on the seaward slope and several empirical formulae such as, Hudson's and Van der Meer formula have been derived for the estimation of the stable weight. The weight of armour units are basically evaluated based on famous Hudson's formula as given below.

$$W = \frac{w_r H_s^{\ 3}}{K_D \times (Sr-1)^3 \cot\theta}$$

Where,

W = Weight of Armour in kg.

Hs = Significant wave height (m)

KD = Stability coefficient, which varies with type of armour

Sr = Specific Gravity of Armour relative to Water at the structure (wr/Ww)

wr = Unit Weight of Armour block (kg/m3)

Ww = Unit Weight of sea water (kg/m3)

Cot θ = Slope of breakwater armour measured with the horizontal

A conceptual design of protection structure/breakwater has been evolved based on the desk studies. The design of breakwater cross-sections at different bed levels with ACCROPODE^{TM}II in the armour have been evolved and finalized through 2D and 3D wave flume studies.

2.4 OPTIMAL DESIGN OF BREAKWATER

The proposed layout consists of about 10 km long offshore breakwater from -6.4 m to -19 m contour depth for the development of new Port at Vadhavan, Maharashtra. The desk studies have been conducted for evolving cross-sections of breakwater with ACCROPODE™ II in the armour at different bed levels based on empirical formulae. Initially, the stable unit weight of ACCROPODE™II for breakwater sections at various bed levels at suitable design wave conditions have been worked out using the Hudson formula. The stability coefficient (KD) depends on the slope of the seabed and KD values to be used in the design are for non breaking waves and based on safe engineering practice, guidelines by Concrete Layer Innovations (CLI), the License holder for the design of ACCROPODE™II armour layers who recommended maximum values of Hudson's stability coefficient KD of 16 for trunk section and 12 for round head section. The trunk and roundhead sections were designed for KD values, corresponding to non-breaking waves and a seabed slope of 1% for conservative sides. According to BS: 6349-Part-7, the relevant configurations of the toe-berm for the rubblemound breakwater have been designed. The under layer is extended to form the toe mound and the same size of rocks to be assumed for design. The toe designs have been checked for relevant low and high water levels and also in corresponding wave conditions. The breakwater sections include design of wave wall, which resist the impact pressures from wave up-rush.

The details of design of breakwater cross-sections at different bed levels with ACCROPODE™II in the armour are as described below:

Cross-section for roundhead portion of breakwater at -6.4 m bed level

The section is designed for the roundhead portion of breakwater at -6.4 m bed level as shown in Figure 2. This section consists of 13 cum ACCROPODE™ II in the armour with 1:1.33 slope on both the sides. A 5.25 m wide toe-berm consisting of 3 to 6 t stones is provided at -2.35 m with 1: 2 slopes on both the sides. A secondary layer consists of 2 to 4 t stones provided on both the sides below the armour, crest slab and 0.3 to 1 t stones below the toe-berm. Core consists of 10-100 kg stones and a bedding layer of stones up to 10 kg weight is proposed.The crest slab is fixed at el.+12.5 m level with a parapet top at el.+15.0 m. A clear carriage way of 7.5 m width is provided on the crest slab.

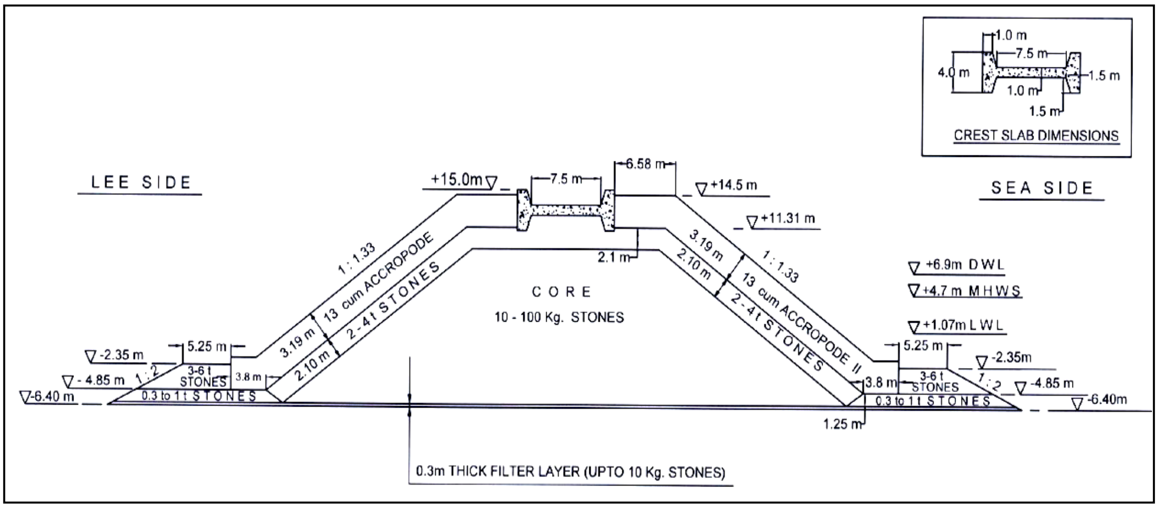


Fig 2. Roundhead cross section (-6.4 m) of breakwater.

Cross-section for the trunk portion of breakwater at - 15 m to -19 m bed level

The section is designed at -19.0 m bed level for the trunk portion from -15 m to -19 m bed level of the breakwater as shown in Figure3. This section consists of 11 cum ACCROPODE^{TM} II unit in the armour with 1:1.33 slope on sea side and 2 to 4 t stones in the armour with 1:1.5 slope on lee side. A 6.22 m wide toe-berm consisting of 4 to 6 t stones provided at -10 m with 1:2 slope on sea side. A secondary layer consists of 2 to 4 t stones provided below the armour units & crest slab. A layer of 0.3 to 1 t stones is provided below the toe-berm. Core consists of 10-100 kg stones and a bedding layer of stones up to 10 kg weight is proposed.The top of the crest slab is at el.+12.5 m level with a parapet top at el.+15.0 m. A clear carriage way of 7.5 m width is provided on the crest slab.

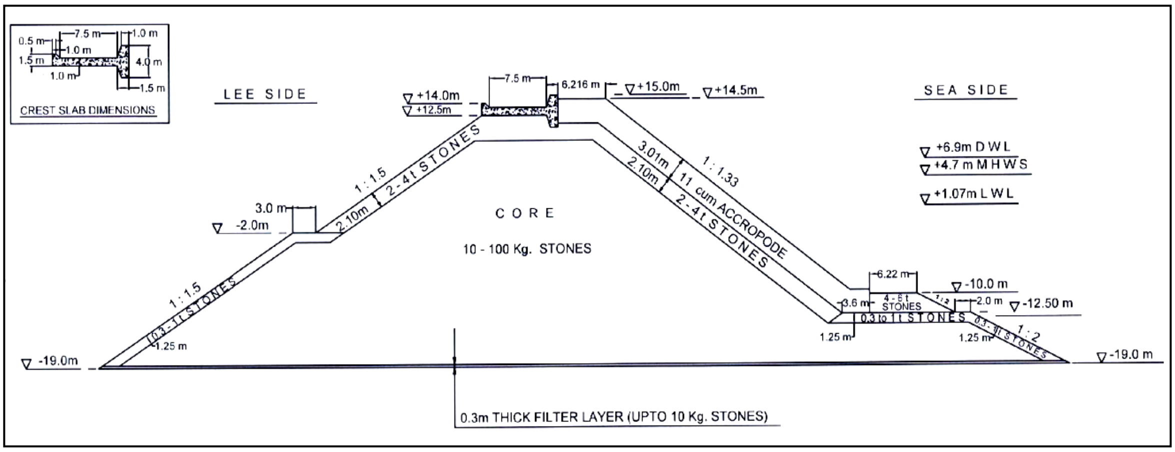


Fig 3. Cross-section of breakwater (-19.0 m bed level).

2.5 HYDRAULIC MODEL STUDIES

2.5.1 Model scale and Wave flume test procedure

The model tests for the design of trunk portion of breakwater at -19 m bed level were conducted in 2-D random wave flume by reproducing the section to a Geometrically Similar scale of 1:56 in the wave flume. The bed level of the representative section was at -19 m and the bed slope of 1:100 was reproduced in front of the structure. The model is based on Froude's criterion of similitude.

The trunk section of a breakwater is tested under a normal attack of waves in a 2-D random wave flume for its hydraulic stability. The models of breakwater cross-sections at -19 m bed level were constructed to a Geometrically Similar (GS) scale in a wave flume with random wave generation. In this wave flume, both regular as well as random waves of desired wave height & period and desired standard wave spectrum respectively could be generated. The numbers of ACCROPODE^{TM}II / stones provided in the armour and in the toe-berm was counted initially, before starting the test. After conducting the tests, the number of ACCROPODE^{TM}II / stones displaced from its original position was recorded and percentage of damage to the armour of breakwater was determined. During the test, extent of splashing/overtopping over the crest was also observed. The damage is expressed as percentage of number of ACCROPODE^{TM}II / stones displaced from their position. The overtopping discharges would be collected immediately before filled up in the discharge tray and measure the volume of water in the model during different test conditions. The filled overtopping water from the discharge tray was manually scooped out regularly during test to prevent it from overfilled. The volume of overtopping Discharge (l/s/m) as per model scale worked out to be 1:419 units, which was measured for 1000 waves in the model for different test conditions and converted overtopping discharge in to proto with model scale. The model tray for collecting the overtopping discharge was fabricated at CWPRS laboratory and the overtopping discharge were collected, measured and confirmed with computational values.

2.5.2 Wave flumes test conditions

The following are the test conditions considered in 2-D wave flume as shown in Table-1

Test Condition	Hs (m)	Tp (secs)	Water Level (m CD)
1	7.0	12	+6.9
2	7.0	12	+7.4
3	7.0	12	+7.9
4	7.5	12	+6.9
5	7.5	12	+7.4
6	7.5	12	+7.9
7	3.0	10	+6.9
8	3.0	10	+7.4
9	3.0	10	+7.9
10	7.5	12	0.0
Overload	8.5	14	tbc

Table 1. Wave flume test conditions

In ordered to confirm the hydraulic stability and overtopping discharge at different test conditions, the breakwater section designed at -19.0 m bed level for the trunk portion from -15 m to -19 m bed level as shown in Figure 5 is reproduced in the wave flume with Geometrically Similar scale of 1:56. The 2-D wave flume tests have been carried out in the Random Sea Wave Generation (RSWG) for trunk portion of breakwater at -19 m bed levels. This section consists of 11 cum ACCROPODE™ II in the armour with 1:1.33 slope on sea side and 2 to 4 t stones in the armour with 1:1.5 slope on lee side. A 6.22 m wide toe-berm consisting of 4 to 6 t stones provided at -10 m with 1:2 slope on sea side. A secondary layer consists of 2 to 4 t stones provided below the armour units & crest slab. A layer of 0.3 to 1 t stones is provided below the toe-berm. Core consists of 10-100 kg stones. The top of the crest slab is at el.+12.5 m level with a parapet top at el.+15.0 m. A clear carriage way of 7.5 m width is provided on the crest slab.

The wave flume test conducted for the period equivalent to 1000 numbers of random waves in the proto by generating random wave spectrum (PM-Spectrum). During the test, overtopping of waves of about 328 litres for 1000 waves in the model was observed. The number of ACCROPODE^{TM}II / stones provided in the armour and in the toe-berm was counted initially, before starting the test. After conducting the tests, the number of ACCROPODE^{TM}II / stones displaced from its original position was recorded and percentage of damage to the armour of breakwater was determined. However, there was no damage was seen on seaside of the breakwater. About 5 % of damage observed on leeward side of breakwater consists of 2 to 4 t stones after the generation of 1000 random waves in the model.

Fig 5. Placement of ACCROPODE^{TM} II model units and arrangement of overtopping discharge tray.

Fig 6. Wave action on trunk section armour layer (11 cum ACCROPODE^{TM} II) at -19.0m bed level with wave height of 7.5 m at +7.9 m water level.

2.6 Permissible wave overtopping discharge

The allowable overtopping rate for safety and structural design shall be as per the specification in EurOtop 2008 &EurOtop 2018 manuals and no vehicle shall access the breakwater crest during the cyclone event. Therefore, the breakwaters shall be designed based on the wave overtopping criteria of 100 lit/s/m to estimate the crest level of the breakwater. The permissible overtopping for rubble mound breakwater with ACCROPODE™ II as primary units in the range of 50-200 lit/s/m will not damage the crest and rear slope of the breakwater if these are well protected. The detailed overtopping discharge calculation for different design conditions are as follows

Deterministic Design or Design safety Assessment (EurOtop 2008, equation No. 6.5, page 118):

$$q = 0.2\exp\left(-2.3\left(\frac{R_c}{H_{m0}\gamma_f\gamma_\beta}\right)\right)\sqrt{gH_{m0}{}^3}$$

Average Overtopping Equation (EurOtop 2018, equation No. 6.6, page 174):

$$q_{f_r} = 0.1035\exp\left[-\left(1.35\frac{R_c}{H_{m0}\gamma_{f_{\bullet\mathrm{mod}}}\gamma_b\gamma_\beta}\right)^{1.3}\right]\sqrt{\left(gH_{m0}{}^3\right)}$$

Where,

- q= Mean wave overtopping discharge m^3/s
- q_{fr}= Average overtopping discharge (at front crest) in m^3/s
- R_c = Free board, (SWL to Crest Level) in m
- H_{m0} = Spectral Wave height in m
- Y_f = Roughness Coefficient =0.44)
- Y_β = Effect of Wave Angle = (1.0)
- Y_b = Berm Coefficient (1.0)
- g = Acc. due to gravity in m/s^2
- C_r = Over topping reduction due to additional crest width (0.883)

2.7 DISCUSSIONS OF RESULTS

The volume of wave discharge collected in the model and computed volume of wave discharge in proto and are compared well and overtopping discharges are within the permissible limit as shown in Fig.7. The hydraulic stability and overtopping discharge at different test conditions of breakwater cross-section with 11 cum ACCROPODE^{TM}II units in the armour at -19 m CD bed level have been confirmed through 2D wave flume studies. There was no damage observed on trunk and roundhead breakwater sections with ACCROPODE^{TM}II model units in the armour and 4 to 6 t stones in the toe-berm at various bed levels during the wave flume studies. The damage was observed on the leeside armour consist of 2 to 4 t stones during the test conditions Nos. 2, 3, 4, 5, 6 & 11 (Refer Table-1). This damage is mainly just below the crest slab and it may be repaired / maintained regularly. The infrastructure developments such as jetty/berths are not considered immediately on leeward side of breakwater. As such, minor damage on leeside stones may not be affected for jetty/berths. However, the maintenance on the leeside of the breakwater with 2 to 4 t stones is very much essential when damage occurs during extreme events. In order to optimize the breakwater sections, the crest level of breakwater have been reduced by 1 m to +15 m and crest width reduced to 7.5 m. Further, reduction of crest elevation is not suggested.

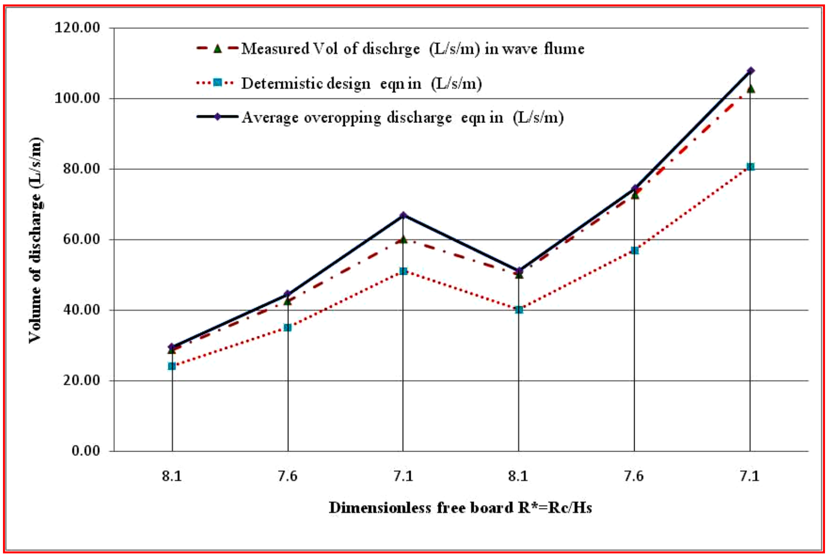


Fig 7: Dimensionless mean wave overtopping discharge

2.8 CONCLUSIONS

- The design cross-sections of breakwater with ACCROPODETM^{TM} II armour units have been evolved at various bed level up to -19 m bed levels.
- The hydraulic stability and overtopping discharge at different test conditions of breakwater cross-section with 11 cum ACCROPODE^{TM} II units in the armour at -19 m CD bed level have been confirmed through wave flume studies.
- The breakwater component such as ACCROPODE^{TM}II armour, toe-berm, secondary layer etc. are hydraulically stable and safe for construction at site.
- The breakwater sections have been designed considering optimization of crest level with allowable overtopping discharges within the permissible limit.
- The optimize breakwater section with 11 cum ACCROPODETM^{TM} II units in the armour, crest level at +15 m and crest width of 7.5 m at -19 m CD bed level have been suggested.

CASES STUDIES - 3

3.0 Design of Submerged Offshore Reefs for the Coastal Protection Measures

The erosion site at Ullal is located on south side of Mangalore, where Gurpur and Netravati rivers have their confluence just near to the mouth, where they meet the Arabian Sea (Fig.1). An integrated development plan prepared by the ADB Consultants for sustainable coastal protection includes i) Construction of two offshore reefs, ii) Construction of four inshore berms to trap the sediments, iii) Beach nourishment of the Ullal beach, iv) Re-habilitation of breakwaters to allow more sand movement towards south. Two delta shaped offshore reefs have been proposed as a part of sustainable coastal protection at Ullal. Design of cross-sections of these offshore reefs as low-crested rubblemound structures have been carried out at CWPRS. Based on desk and wave flume studies, the cross-sections of different alternative sections of offshore submerged reefs at -7m bed levels have been evolved.

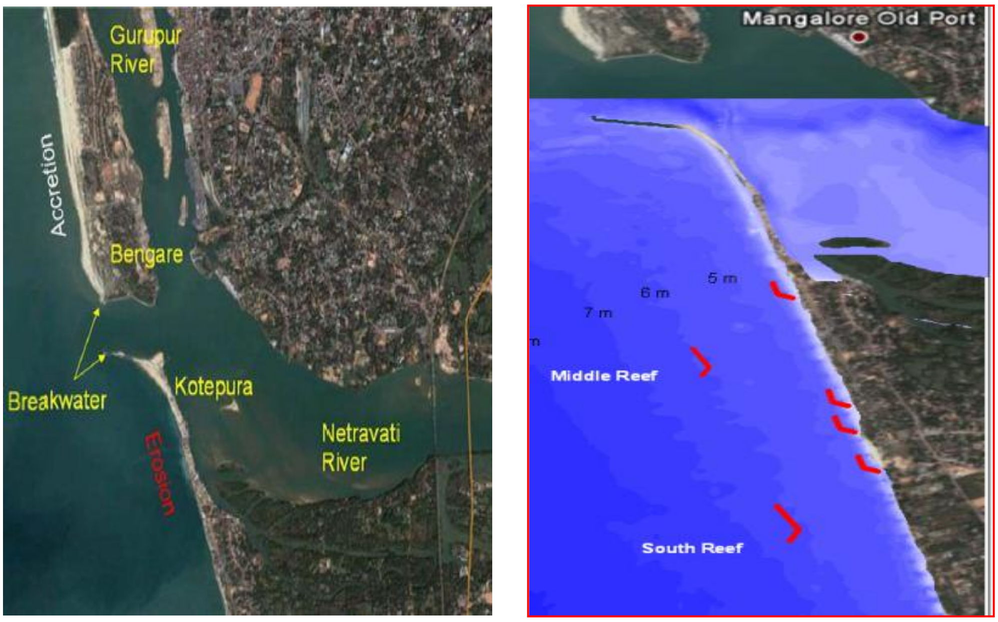


Fig.1. Location of proposed coastal protection to the erosion site at Ullal,

3.1 Design of rubblemound offshore submerged reefs

Low crested and submerged structures such as detached breakwaters and artificial reefs are becoming very common coastal protection measures. Their purpose is to reduce the hydraulic loading to a required level that maintains the dynamic equilibrium of the shoreline. It should be noted that the low-crested structures could be used not only for shoreline control but also to reduce wave loading on the coastal structures.

The layout plan of two offshore reefs at -7 m bed level has been evolved by ADB Consultants as shown in Fig.1. Rubblemound offshore reef comprises mound of stones having a bedding layer, core, secondary layer protected by an armour layer and a toe to prevent slippage of armour units. Armour layer consists of selected units of either quarry stones or artificial concrete blocks, which receives the wave impact. The stability of rubblemound structures depends primarily upon the stability of individual armour units on its seaward slope. A major aspect in the design of rubblemound structures is the minimum weight of the armour units on the seaward slope, required to withstand the design waves. Several empirical formulae such as, Hudson formula and Van der Meer formula are available for the estimation of the minimum stable weight of the armour unit.

The Stability number (Ns) derived from Hudson's formula is,

$$N_s = \frac{H}{\Delta . D_{50}}$$

Where $\Delta = s_r - 1$, and D_{50} = Characteristic diameter of the stone (m)

From Hudson's formula the relationship between K_D& N_s is $(K_D .\cot \theta)^{1/3} = N_s$

Some empirical relations are available for determining the stable weight of stones in the submerged breakwater. Brebner and Donnelly (1962) (recommended in Shore Protection Manual, 1984) tested the stability of toe to vertical faced composite breakwaters under monochromatic waves. A relationship is established between the ratio h_t/h and the stability number $H/\Delta D_{n50}$ (or N_s), where $h_t$is the depth of the toe below the water level and h is the water depth. A similar relationship has been established by Gadre et.al.(1990) for determining the stable weight of armour stones in the submerged reefs.

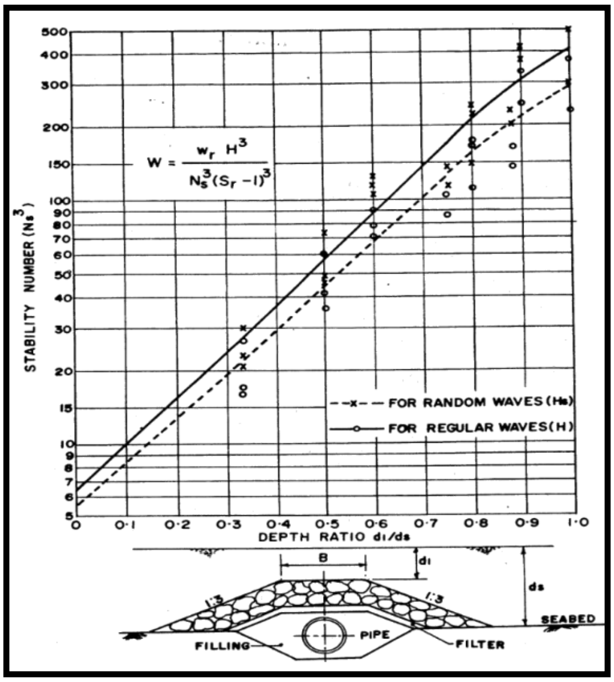


Fig.2 Design curve of Stability number(Ns) v/s Depth ratio for the submerged reefs

Based on the flume tests a relationship has been established by CWPRS (1990) for determining the stable weight of armour stones in the submerged rubblemounds (Fig.2). Using the design curves of CWPRS for submerged bund/reef the conceptual design of the offshore reefs was worked out.

$$W = \frac{w_r H^3}{N^3 s \left(S_r - 1\right)^3}$$

Though, theoretical formulae are available for designing coastal rubblemound structures, they can only be used for conceptual design. The hydraulic model tests are essential to simulate the complex wave structure interaction and the site conditions of seabed slope, water level etc. can be simulated in the wave flume or wave basin. These models are constructed to a Geometrically Similar (GS) model scale and are based on 'Froudian' criterion of similitude; thus, enabling proper simulation.

Design cross-sections of offshore submerged reef

Different alternative cross-sections have been worked out considering hydraulic stability, site conditions and availability of materials. Three alternative cross-sections of the trunk and roundhead portion for the offshore reefs at -7 m bed level have been evolved. In Alternative-I, the tetrapods in the armour and concrete blocks in the crest are considered. In Alternative-II, the tetrapods in the armour and stones in the crest are considered. Whereas, in Alternative-III, the stones in the armour and in the crest are considered.

Design of cross-section of offshore reef for the trunk portion

In Alternative–I, the section is designed for offshore reef with tetrapods in the armour and Cement Concrete blocks in the crest as shown in Fig.3. The section consists of 4 t tetrapods (in double layer) in the armour layers on 1:2 slope on both sides. The sea side toe level is fixed at –3.7 m with 4 m wide toe-berm consisting of 2 to 3 t stones. The Lee side toe level is fixed at –3.7 m with 2.5 m wide toe-berm consisting of 1 to 2 t stones. A secondary layer consists of 0.5 to 1 t stones below the armour on both sides and 200 to 500 kg stones below the toe-berm on sea side. Core consists of 20 to 100 kg stones and a bedding layer consists of stones 5- 20 kg weight are proposed. Concrete cubes (size 2.0m x 1.5m x 1.0 m) are provided in the crest at el. + 0.5 m.

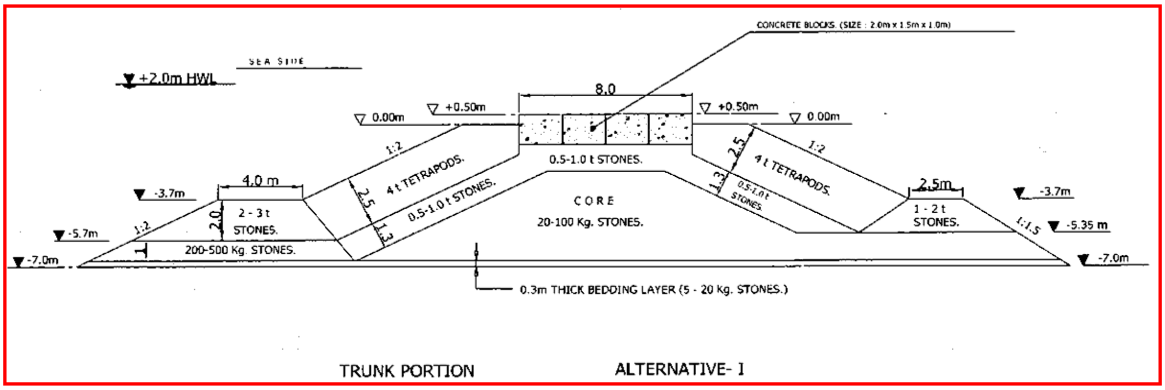


Fig.3 Cross-section of offshore reef with tetrapods in the armour and CC blocks in the crest- Alt–I,

In Alternative-II, the section is designed for offshore reef with tetrapods in the armour and stones in the crest as shown in Fig.4. The section consists of 4 t tetrapods (in double layer) in the armour layers on 1:2 slope on both sides. The sea side toe level is fixed at – 3.7 m with 4 m wide toe-berm consisting of 2 to 3 t stones. The Lee side toe level is fixed at –3.7 m with 2.5 m wide toe-berm consisting of 1 to 2 t stones. A secondary layer consists of 0.5 to 1 t stones below the armour on both sides and 200 to 500 kg stones below the toe-berm on sea side. Core consists of 20 to 100 kg stones and a bedding layer consists of stones 5 to 20 kg weight are proposed. 6 to 8 t stones are provided in the crest at el. + 0.5 m.

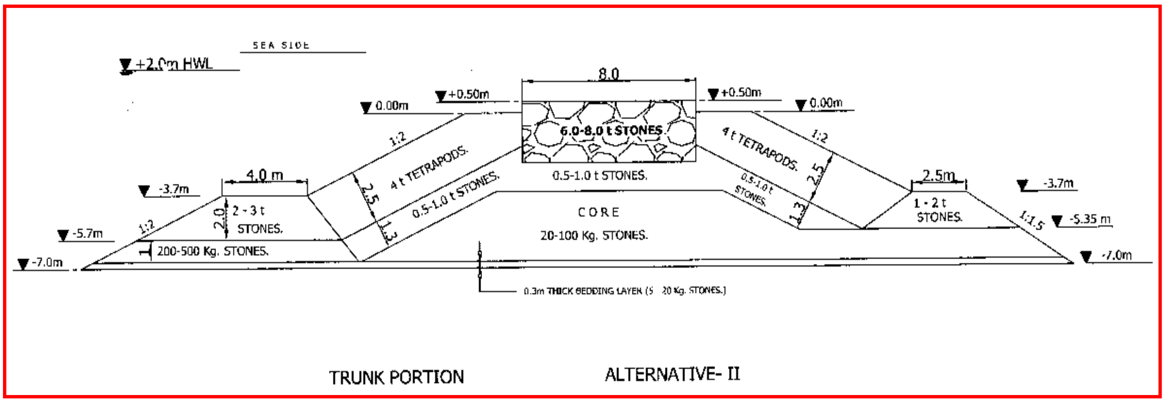


Fig.4 Cross-section of offshore reef with tetrapods in the armour and stones in the crest- Alt–II

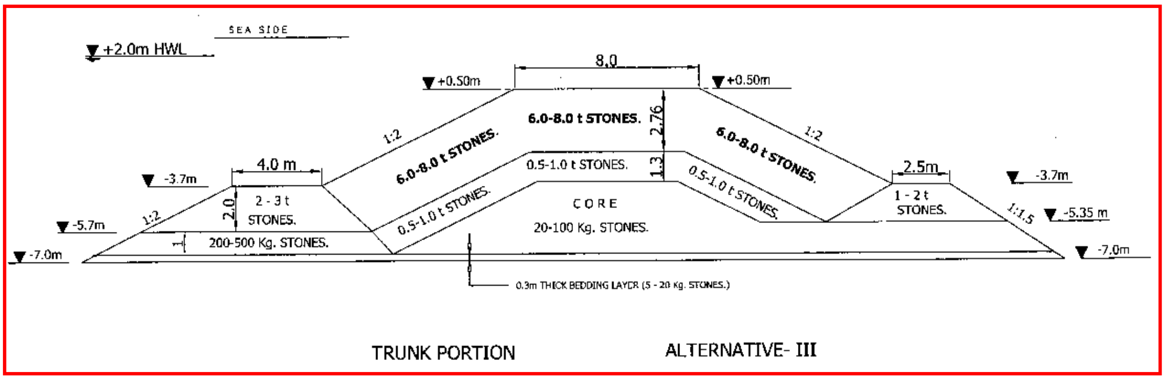


Fig.5 Cross-section of offshore reef with stones in the armour and crest- Alt–III

In Alternative–III, the section is designed for offshore reef with stones in the armour and crest as shown in Fig.5. The section consists of 6 to 8 t stones (in double layer) in the armour layers on 1:2 slope on both sides. The sea side toe level is fixed at -3.7m with 4 m wide toe-berm consisting of 2-3 t stones. The Lee side toe level is fixed at- 3.7 m with 2.5 m wide toe-berm consisting of 1to 2 t stones. A secondary layer consists of 0.5 to 1t stones below the armour on both sides and 200 to 500 kg stones below the toe-berm on sea side. Core consists of 20 to 100 kg stones and a bedding layer consists of 5 to 20 kg stones are proposed. 6 to 8 t stones are provided in the crest at el.+ 0.5 m.

Design of cross-section of offshore reef for the roundhead

The roundhead portion of the offshore reef is located at about –7 m bed level. The conceptual designs of the roundhead of the offshore reefs are evolved. Typical design cross-sections of offshore reef as shown Fig.6. The section consists of 4 t tetrapods (in double layer) in the armour layers on 1:2 slope on both sides. The toe level fixed at -3.7 m with 4 m wide toe-berm consisting of 2 to 3 t stones on both sides. A secondary layer consists of 0.5 to 1 t stones below the armour on both sides and 200 to 500 kg stones below the toe-berm on sea side. Core consists of 20 to 100 kg stones and a bedding layer consists of stones 5- 20 kg weight are proposed. Concrete cubes (size 2.0 m x 1.5 m x 1.0 m) are provided in the crest at el. + 0.5 m.

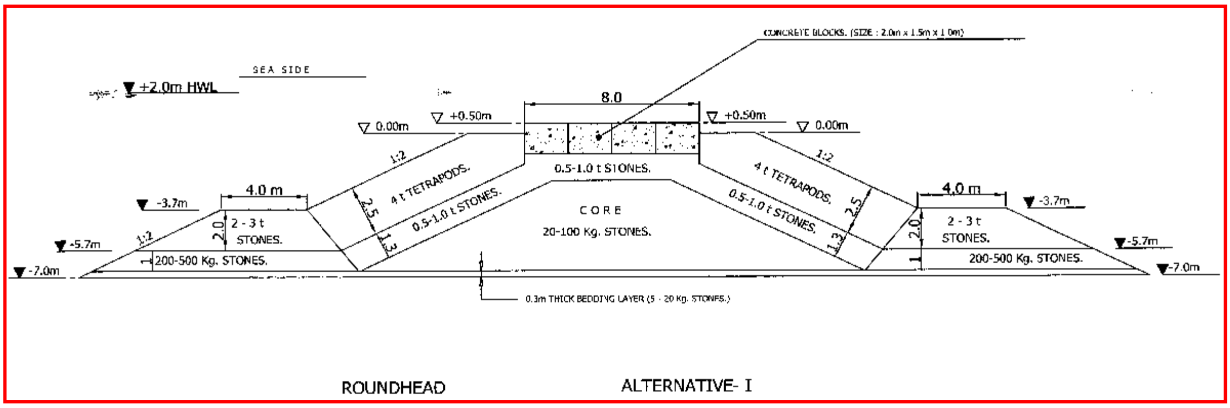


Fig.6 Cross-section of offshore reef with stones in the armour and crest for the roundhead portion.

3.2 HYDRAULIC MODEL STUDIES

The model tests for the design of offshore submerged reef were carried out in a wave flume by reproducing the section to a Geometrically Similar scale of 1:30. The bed level of the representative section was at - 7 m and bed slope of 1:100 was reproduced in front of the structure. The cross-sections was tested under regular waves for wave height of 5 m with High Water Level of +2.0 m (HWL) and Low Water Level (LWL) of 0.0 m for zero order damage (0-1%). Section was also tested for first order damage (between 1% to 5%) with the breaking waves of the order of 6.0 m at the High Water Level (HWL) of + 2.0 m and Low Water Level (LWL) of 0.0 m. A cross-section (Alternative –I) for the trunk portion at -7 m bed level as shown in Fig.3 was modelled in the wave flume to a model scale of 1:30 (GS). The bed level of the section was taken as –7.0 m and bed slope of 1:100 was reproduced in front of the structure. The section was tested at HWL of + 2.0 m and at LWL of 0.0 m.

Initially, a test was carried out with wave height of 5.0 m at +2 m water level (HWL). It was observed that a sheet of water of about 4.5 m thickness was passing over the crest of the reef. It was also observed that the rundown was up to 0.0 m. There was no damage to tetrapods in the armour as well as to the cement concrete blocks on the crest. The toe berm was also stable under this wave condition. The incident waves of 5 m height reduced to about 2.5 m after passing over the reef. The transmission co-efficient was of the order of 0.5 under these conditions.

Another test was carried out with wave height of 5.0 m at 0.0 m water level (LWL). It was observed that over topping sheet of water of about 1.8 m thickness was passing above the + 0.50 m crest level. It was also observed that the rundown was up to -1.5 m. The waves were breaking on the armour causing no damage to the armour & toe berm of offshore reef structure. The incident wave height of 5m reduced to about 1.0m after transmission of waves from sea side to leeside. The transmission co-efficient is in the order of about 0.2 for this condition.

The above tests were carried out with the wave period of 10 sec. 8 sec. and 12 sec. In order to observe the stability of offshore reef under the severe condition of 6.0 m waves was reproduced at LWL as well as HWL. The test at LWL with 6.0 m waves showed about 5% damage to the tetrapods as well as to the crest blocks.

The whole set of tests was repeated for the reef section with 6 to 8 t stones in the armour as well as in the crest for Alternative-II (Fig.4) and Alternative-III (Fig.5). The results are almost similar compared to the Alternative-I.

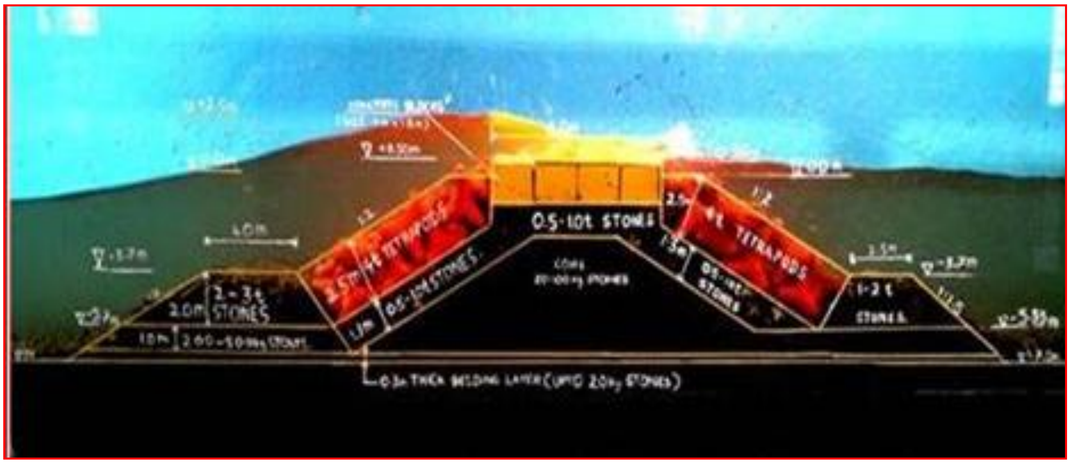


Fig.7 Wave action on the cross-section with tetrapods in the armour and CC blocks in the crest

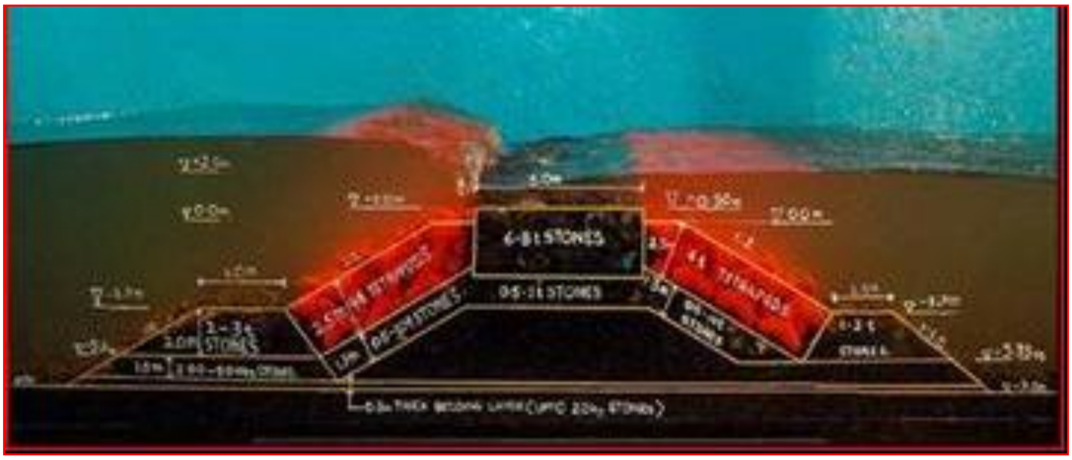


Fig.8 Wave action on the cross-section with tetrapods in the armour and stones in the crest

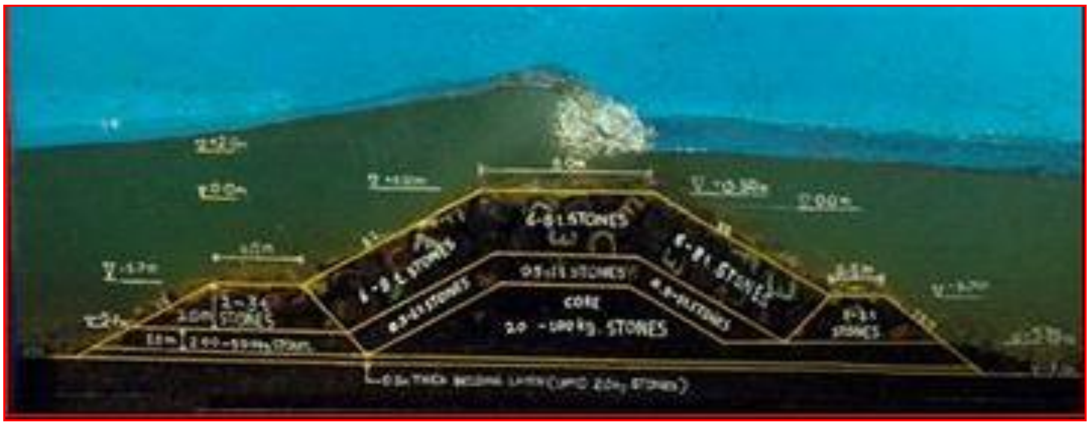


Fig.9 Wave action on the cross-section with stones in the armour and crest

Transmitted wave height

Transmission coefficient can be defined as the ratio of the transmitted wave height to the incident wave height. The incident wave height is measured on the seaside of the structure whereas the transmitted wave height is measured on the leeside.

$$C_t = \frac{H_t}{H_i}$$

where, C_t = Transmission coefficient

H_t = Transmitted Wave Height

H_i = Incident Wave Height

The wave flume tests were conducted at suitable interval of water level from low water to High water with respect to Chart Datum (+0.0, +0.5 m, +1.0 m, +1.5m, +2.0m and +2.5 m) with the regular incident wave heights of 5 to 6 m. The wave periods were taken as 10 second for each wave height. For each test condition, the transmitted wave height on the leeside of the submerged reef was measured in the wave flume. The graphical representation of the relation between wave transmission coefficient Ht/Hi and relative crest depth Rc/Hi as shown in Fig.10.

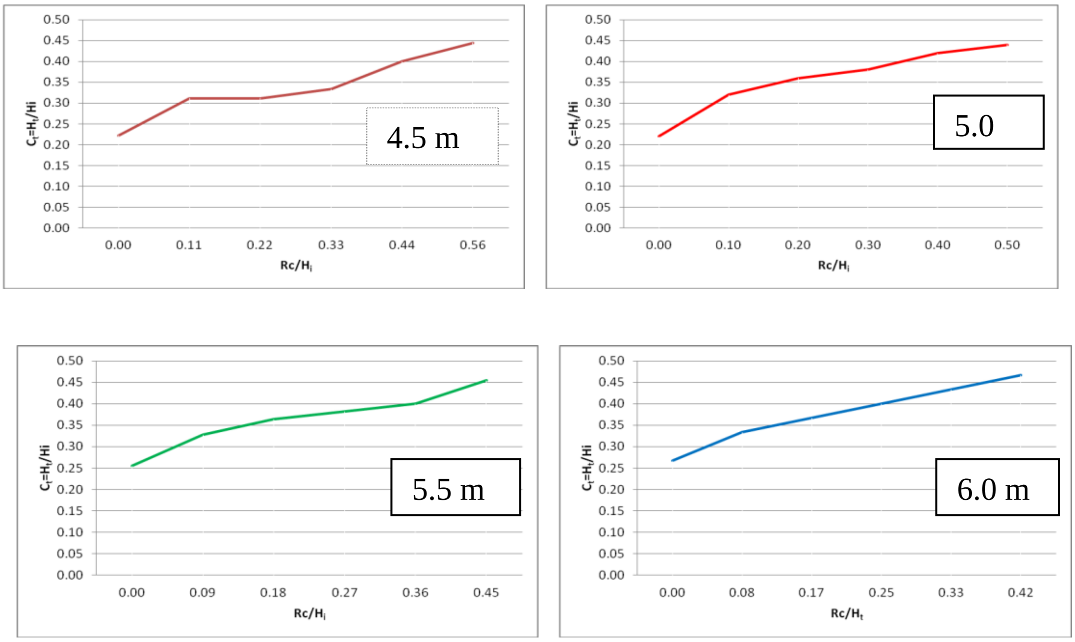


Fig.10 Relation between wave transmission coefficient Ht/Hi and relative crest depth Rc/Hi

3.4 Discussions of results

Based on the studies, three different alternative cross-sections of the trunk and roundhead portion for the offshore reefs at -7 m bed level have been evolved**.** The tetrapods in the crest are not considered because the tetrapods on horizontal slope are not stable**.** In Alternative-I, the tetrapods in the armour layer and concrete cubes in the crest are considered. In Alternative-II, the tetrapods in the armour layer and stones in the crest are considered. In Alternative-III, the stones in the armour layer and in the crest are considered**.** The maximum transmitted wave height of about 2.8 m was observed on the lee side and transmission co-efficient of the order of about 0.47 has been observed for the incident waves of 6 m height. It was suggested to consider suitable alternative comparing with the cost, availability of stones, site conditions for construction etc.

3.5 Conclusions

- The sections for the trunk and roundhead portions for the offshore submerged reefs have been evolved at -7m bed level with three alternatives viz. Alternative-I : Tetrapods in the armour and concrete blocks in the crest, Alternative-II : Tetrapods in the armour and Stones in the crest and Alternative-III: Stones in the armour and crest. These sections are hydraulically stable under the design wave height of 5m.

- While designing the offshore submerged reefs for the coastal protection, the wave transmission behaviour is important aspect and needs to be studied in the wave flume to ensure desire wave transmission.

- The wave transmission coefficient for all the alternatives was found to be in the range of 0.22 to 0.47 with the relative crest height of 0.0 to 0.5.

- The wave transmission coefficient of submerged offshore reef mainly depends on geometry of the structure and on the type of armour in the structures.

CASES STUDIES - 4

4.0 UTILIZATION OF STEEL SLAG IN COASTAL HYDRAULIC STRUCTURES

4.1 Introduction

The present case study describes the utilization possibilities of Electric Arc Furnace (EAF) steel slag as an alternative material in the construction of coastal hydraulic structures for the first time in India. Based on the EAF steel slag physical property tests, the average specific gravity of EAF samples is found to be 2.64 with values ranging from 2.25 to 3.0. Also, the average dry density of EAF samples is found to be 3494.66 kg/m^3 with values ranging from 3226.89 kg/m^3 to 3813.64 kg/m^3, which is required for the computation of the unit weight armour. In addition, the conceptual design cross-sections of different Coastal Hydraulic Structures (CHS) such as Seawalls, Groyne, Offshore reefs and breakwaters with the utilization of EAF steel slag for the coastal environment have been evolved and the hydraulic stability of those structures was confirmed through wave flume studies.

4.2 Hydraulic model studies

The present study highlights the conceptual design and hydraulic stability of the Offshore reef section with steel slag stones and breakwater section with steel slag tetrapods & stones in the armour evolved based on desk and wave flume studies. This will help with the appropriate use of results for a wide range of design wave conditions for different CHS sections. The various cases analyzed CHS sections are summarized below:

- Offshore reef section with steel slag stones
- Breakwater section with steel slag stones armour unit
- Breakwater section with steel slag tetrapod armour unit.

4.3 Offshore reef section with steel slag stones

The design cross-section of the Offshore Reef section using Steel slag stones was considered for confirmation of hydraulic stability through wave flume studies as shown in Figure-1. The wave flume test was carried out for the Offshore Reef section using Steel slag stones in the core, secondary and armour layer up to -3.00 m bed level to a Geometrically Similar model scale of 1:34 in the wave flume. Figure-2 (a) illustrates the Design Water Level (DWL) of +4.60 m including the storm surge and sea level rise and the maximum breaking wave height (H_b) of 5.0 m was considered. This section consists of 2.6 m thick 4 to 5 t steel slag stones in the armour layer with a 1:2 slope on both sides. The secondary layer of 1.3 m consists of 0.5 to 1 t steel slag stones provided below the armour layer. The core consists of 10 to 100 kg steel slag stones and a 0.3 m thick bedding layer with up to 10 kg steel slag is proposed. A 6.0 m wide crest with 4 to 5 t steel slag stones provided at el.+ 4.6 m. A toe-berm consists of 1 to 2 t steel slag stones provided at el. -1.20 m on both sides with a side slope of 1:2m on the seaside with a side slope of 1:2.Similarly figure 5 (b) shows the Design Water Level (DWL) of +3.60 m including the storm surge and sea level rise and the maximum breaking wave height (H_b) of 5.0 m.

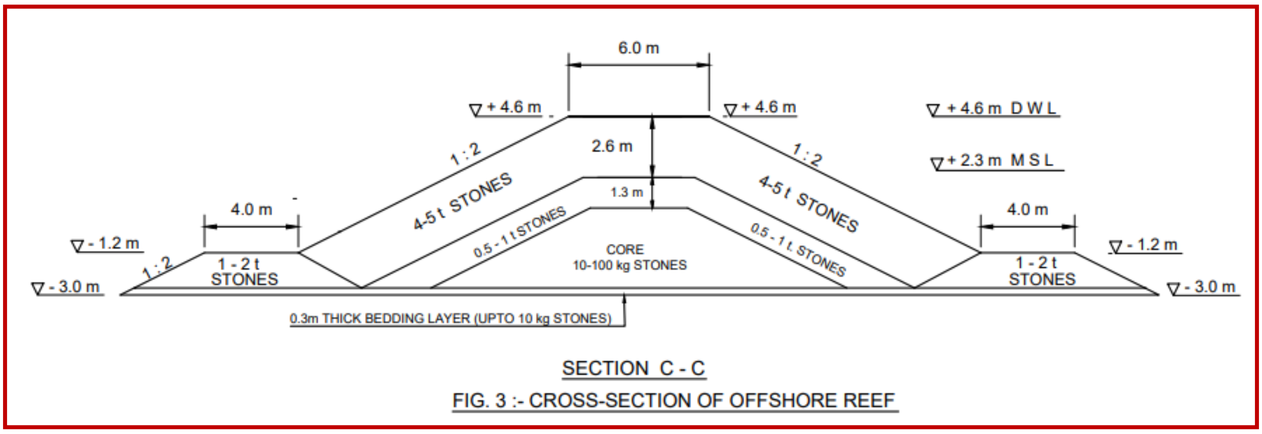
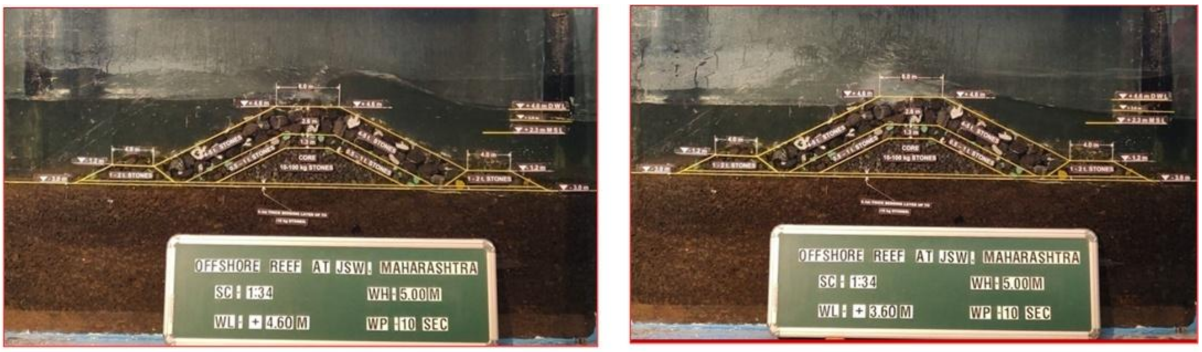


Figure 1. Typical Design Cross-Section Of Offshore Reef

The wave flume test was carried out with different breaking wave heights for the Offshore section as described below;

- Maximum breaking wave height (H_b) of 5.0 m at Design Water Level (DWL) of + 4.60 m at -3.0 m sea bed level,
- Maximum breaking wave height (H_b) of 5.0 m at a Water Level of + 3.6 m at -3.0 m sea bed level, and
- Maximum breaking wave height (H_b) of 4.0 m at Water Level of + 2.3 m at -3.0 m Sea bed level.



(a) (b)

Fig 2. Wave action on offshore reef section maximum breaking wave height (H_b) of 5.0 m during DWL of +4.60 m (a) and.DWL of +3.60 m (b)

The wave flume tests were carried out for a one-hour duration (corresponding to 5.92 hours in the prototype). Generally, in wave flume studies for Offshore reef/breakwaters, the Incident waves (H_i) and Transmitted waves (H_t) were measured with sophisticated instruments such as wave probes. The incident wave is the one that approaches the coastal structure but hasn't reached it yet and the wave action behind a structure can be caused by wave overtopping and also by wave penetration as the coastal hydraulic structure is permeable known as transmitted wave. Waves generated from overtopping tend to have shorter periods than incident waves. Generally, the transmitted wave periods are about half that of the incident waves. The transmission coefficient (K_t) is defined as the ratio of H_t (wave height in the lee of the wave array) to H_i (incident wave height) and is described below.

The Transmission Coefficient (K_t) is given as

$$K_t = \frac{H_t}{H_i}$$

Where,

H_t= Transmitted wave height in the Lee side

H_i = Incident wave height at the Seaside

The wave data regarding the Incident waves (H_i) and Transmitted waves (H_t) were recorded during the wave flume studies. It was observed that the waves were overtopping for different water level conditions. Based on the analysis of the wave-measured data, the Transmission Coefficient (K_t) was calculated as shown in

Table 1. The waves were breaking on the armour / transmitted through the section causing no damage to the armour, toe-berm and as a whole structure and the Offshore reef structure was found to be hydraulically stable with zero damage condition.

Table 1: Computation of Transmission Coefficient (K_t)

Sl. No.	Design Water Level (DWL)	Incident waves (H_i)	Transmitted wave (H_t)	Transmission Coefficient % (K_t)
	(m)	(m)	(m)	(m)
1	+4.60	5.0	1.60	32 %
2	+3.60	5.0	1.09	22 %
3	+2.30	4.0	0.30	7.5 %
4	+1.60	3.5	0.10	2.8 %
5	+0.60	2.5	0.08	3.2 %

4.4 Breakwater section with Steel slag stones as an armour unit

The design cross-section of the breakwater using the Steel Slag Stones armour unit was considered for confirmation of hydraulic stability through wave flume studies as shown in Figure 3.

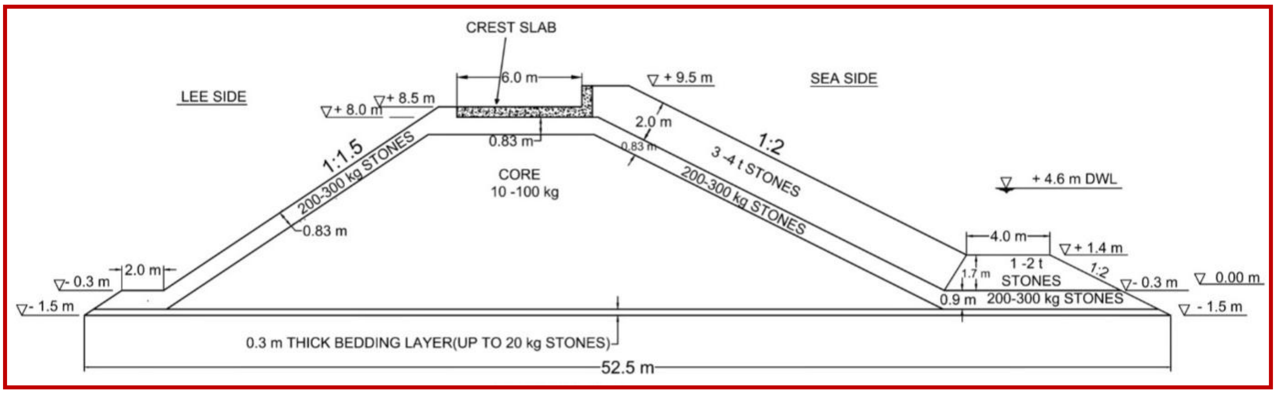
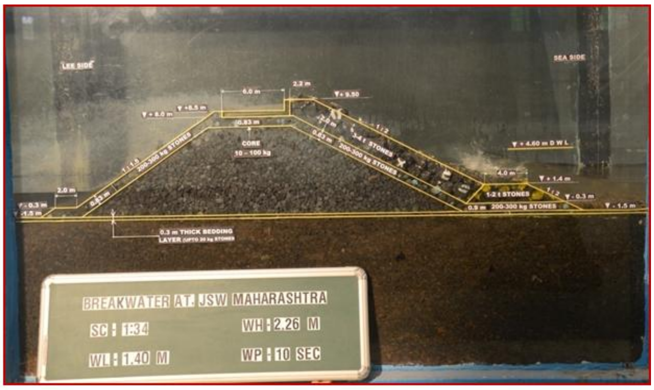


Fig 3. Typical design cross-section of breakwater using Steel Slag Stones

The wave flume test was carried out for the trunk section of the breakwater using Steel slag stones in the core, secondary and armour layer at -1.50 m bed levels Design Water Level (DWL) of +4.60 m including the storm surge and sea level rise and maximum breaking wave height (H_b) of 4.75 m was considered. This section consists of 2.0 m thick 3 to 4 t steel slag stones in the armour layer with a 1:2 slope on the seaside. The secondary layer consists of 0.2 to 0.3 t steel slag stones provided below the armour layer. The core consists of 10 to 100 kg steel slag stones and a 0.3 m thick bedding layer with up to 10 kg steel slag is proposed. A toe-berm consists of 1 to 2 t steel slag stones provided at el. +1.4 m on the seaside with side slope as 1:2. A 0.5 m thick & 6.0 m wide crest slab is provided at el.+8.5 m as crest slab top with parapet top at +9.5 m in wave flume are illustrated in Figure 4 (a).

(a)



(b)

Fig 4. Wave action on breakwater section(H_b) of 4.75 m during DWLof +4.60 m (a) and breaking wave height (H_b) of 2.26 m during DWLof +1.40 m (b).

The Figure 4 (b) shows the wave flume test was carried out for the normal attack of waves considering the maximum breaking wave height (H_b) of 2.26 m at a Low Water Level of +1.40 m at-1.50 m sea bed level for the one-hour duration (corresponding to 5.92 hours in prototype). There was marginal splashing and no overtopping of the waves. It was observed that the highest wave run-up was just above +8.15 m and the rundown was up to +3.60 m. The waves were breaking on the armour causing no damage to the armour and no damage to the toe-berm as a whole structure.

4.5 Breakwater section using Steel slag Tetrapod armour unit.

The design cross-section of the breakwater using a Steel Slag Tetrapod armour unit was considered for confirmation of hydraulic stability through wave flume studies as shown in Figure 5.

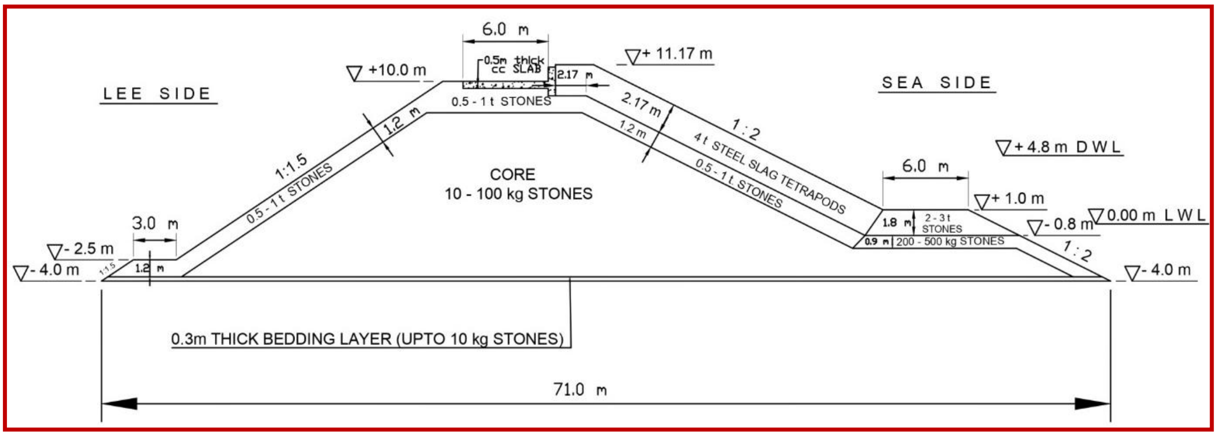


Fig 5. Typical design cross-section of breakwater using Steel Slag Tetrapod

The breakwater section uses steel slag in the core, secondary and Tetrapod armour unit at -4.00 m bed level to a geometrically similar model scale of 1:25 in the wave flume. The Design Water Level (DWL) of +4.80 m including the storm surge and sea level rise and maximum breaking wave height (H_b) of 6.5 m was considered. This section consists of 2.17 m thick 4 t steel slag tetrapods in the armour layer with a 1:2 slope on the seaside. The secondary layer consists of 0.5 to 1t steel slag stones provided below the armour layer. The core consists of 10 to 100 kg steel slag stones and a 0.3 m thick bedding layer with up to 10 kg steel slag is proposed. A toe-berm consists of 2 to 3 t steel slag stones provided at el. +1.0 m on the seaside with side slope as 1:2. A 0.5 m thick & 6.0 m wide crest slab is provided at el.+ 10.0 m as crest slab top with parapet top at + 11.0 m.

A total of 334 nos. of 230-gram Tetrapod model units were placed in the armour layer of the breakwater and a total of 211 no. of Steel slag stones ranging from 110 to 170 grams were placed in the toe-berm for conducting wave flume studies at different water levels and wave heights. Initially, the wave flume test was carried out for the normal attack of waves considering the Maximum breaking wave height (H_b) of 6.50 m at Design Water Level (DWL) of +4.80 m sea bed level for the one-hour duration (corresponding to 5.0 hours in prototype).

There was marginal splashing and no overtopping of the waves was observed during wave flume studies. It was observed that the highest wave run-up was just above +10.00 m and the rundown was up to +2.50 m as shown in Figure 6 a.

(a) (b)

Fig 6. Wave action on breakwater section (H_b) of 6.50 m during DWL of +4.80 m (a) and breaking wave height (H_b) of 3.10 m during DWL of 0.0 m (b).

Similarly, Figure 6 b illustrates the normal attack of waves considering the maximum breaking wave height (H_b) of 3.12 m at the Design Water Level (DWL) of +0.00 m sea bed level for a one-hour duration (corresponding to 5.00 hours in prototype). There was no splashing and overtopping of the waves observed during wave flume studies. It was observed that the highest wave run-up (R_u) was just above +1.50 m and the rundown (R_d) was up to -0.80 m. The waves were breaking on the armour causing no damage to the armour and no damage to the toe-berm as a whole structure and the structure is found to be hydraulically stable.

4.6 Comparison of Steel Slag vs Rubble Stone vs Concrete Tetrapod vs Steel Slag tetrapod armour Unit weight.

Figure 7, Illustrates the comparison of steel material with rubble stone and concrete tetrapod for different wave heights on the x-axis, and armour weight on the y-axis respectively. From the graph, it is observed that as such, the unit weight of steel slag stone/tetrapods required is less compared to the unit weight of rubble stone/ concrete tetrapods for the same wave height. In comparison to the armour weight of steel slag stones, the armour unit weight of rubble stone is required more than 2.7 times for coastal hydraulic structures such as Seawall, Groyne, Breakwater etc., whereas the concrete tetrapod armour weight of Breakwater is required more than 4 times of the steel slag tetrapod armour unit weight due to density difference of these materials (i.e the density of steel slag is about 3.50 t/cum compared to 2.60 t /cum for stones and 2.40 t/cum for concrete materials).

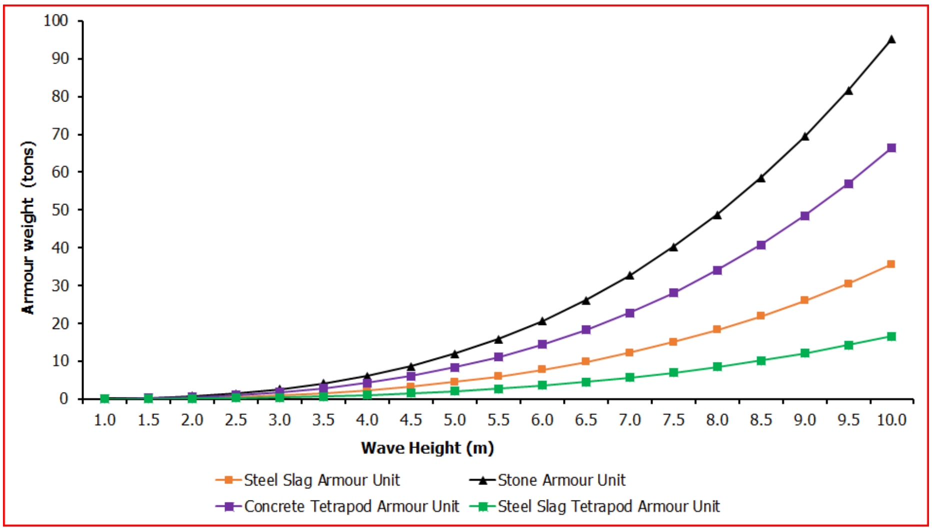


Fig 7. Comparison of armour Unit weight of Steel Slag stone vs Rubble Stone vs Concrete Tetrapod vs Steel SlagTetrapod

4.7 Comparison of Concrete vs Steel Slag Tetrapod Armour Thickness

Figure 8, Illustrates the comparison of a concrete tetrapod with a steel slag tetrapod for varying armour weight on the x-axis, and thickness of the armour layer on the y-axis respectively. Similarly, Figure 9 shows the inferences of steel slag stones with rubble stones. From both graphs, it was observed that the required thickness of armour layer is comparatively less for steel slag material. (steel slag stone and steel slag tetrapod) .

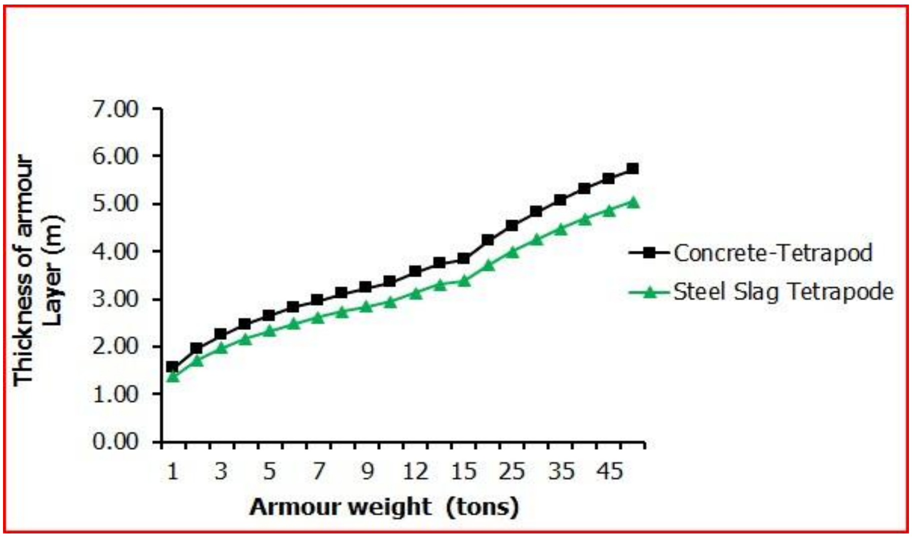


Fig 8. Comparison of Concrete vs Steel Slag Tetrapod Armour Thickness

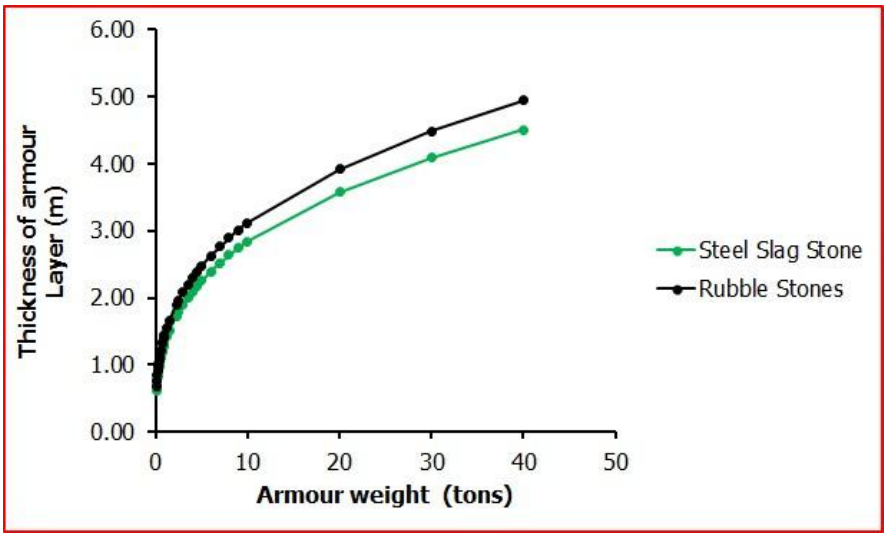


Fig 9. Comparison of Steel slag stones vs Rubble Stones

i.e. the thickness of steel slag for the double layer required for coastal structures is about 12 % less compared to rubble stone & concrete armour units for the same armour weight.

4.8. Discussion and Conclusions

Based on the desk and wave flume studies for the design of cross-sections of different coastal structures have evolved, the following conclusions are drawn:

- The conceptual design cross-sections of different coastal hydraulic structures such as seawalls, Groyne, Offshore reefs and breakwaters with the utilization of EAF steel slag material have been evolved based on empirical formulae considering the density of steel slag of 3.50 t/cum for different design conditions in the coastal environment.
- The hydraulic stability of the Offshore reef section with steel slag stone, breakwater section with steel slag stone &tetrapods have been confirmed under the different design wave and water levels in the wave flume and these sections were found to be hydraulically stable.
- The transmitted wave height (K_t) for the Offshore reef section is observed at a maximum of 32 % for the design water level of +4.6 m and a minimum of 2.8 % for the design water level of 1.6 m.
- It was observed that about 62 % of material reduction in armour weight of steel slag stones compare to rubble stones and 75 % in steel slag tetrapod compare to concrete tetrapod armour weight.

4.9. Limitations

- The steel slag materials may be available up to 20 kg only, whereas more than 20 kg weight of steel slag stone/block is required in different layers of CHS such as Seawall, Groynes and Breakwater etc. To utilize steel slag material in the core, secondary and armour layer of coastal structures, it is required to make suitable sizes/weights with different shapes by casting steel slag materials.
- The strength of steel slag material such as compression and tension strength of steel slag stones/tetrapods required to be ensured before utilization in the prototype construction of coastal hydraulic structures.
- The wave flume studies were conducted for the hydraulic stability of the Offshore reef and breakwater with the utilization of EAF steel slag material with different test conditions considering a 1:100 bed slope.
- The design cross-sections for site-specific coastal structures are needs to be evolved separately for that site conditions.

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