1. INTRODUCTION

Coastal structures are the marine structure located in coastal waters. Marine structures can be broadly classified into two types:

a) Offshore Structures and
b) Near-shore Structures

The design of Near-shore structures and offshore structures varies greatly in terms of environmental factors, depth of water etc. Offshore structures are located in deepwater 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 near-shore structures can be grouped into:-
Rigid structure – Sheet piles, Walls
Semi-rigid or Composite structures – Caissons or cells on rubble base
Flexible Structures – Rubblemound breakwaters, Sea walls

Fig 1: Types of Near-shore Structures
Thus these 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 and 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 / basins.

2. 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 & 3). 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.

![Figure 2: Typical Cross Section of Rubblemound Breakwater](image-url)
The breakwaters and the seawalls are generally constructed as rubblemound structures (Photo 1 & 2). 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. The seawalls are the coastal structures constructed along the coastline to protect the eroding shore. Wave is the important parameter in the design of these structures. Wave structure interaction needs to be thoroughly understood while designing the rubblemound structures.

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 structures.

The design of a seawall differs from that of a breakwater mainly in the following aspects:

- Run-up/overtopping conditions are generally important for seawalls
- Toe erosion of seawall has to be critically examined for stability of seawall as well as stability of coast along sides of seawall
- Seawalls are constructed parallel to the shoreline along the coast, whereas breakwaters are normal to the coast, protruding into the sea from the shore
- Construction of a seawall is comparatively simpler than that of the breakwater
3. DESIGN INFORMATION AND CONSIDERATIONS

Following information should be collected before the design of coastal rubblemound 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 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 breakwater should be decided accordingly by making sufficient allowance for freeboard
- It should be seen that the material in the foundation is not subject to scour

4. 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:

1) Hydraulic Response
2) Wave loadings and related structural response

4.1. Hydraulic Response

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

4.1.1. 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.

4.1.2. 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.

4.1.3. 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.

4.1.4. 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.

4.2. Wave loadings and related structural response

An important part of the design procedure for structures in general is the determination of the loads and the related stresses, deformations, and stability conditions of the structural members. In the case of rubblemound structures exposed to waves, such procedures cannot be followed because the wave loading on single stones or blocks cannot be determined by theory, by normal scale model tests, or by prototype recordings. Instead a black box approach is used in which experiments are used to establish relationships between certain wave characteristics and the structural response, usually expressed in terms of armour movements. The related stresses, e.g., in concrete armour blocks, are known only for a few types of blocks for which special investigations have been performed. For vertical-front monolithic structures like breakwater caissons and seawalls it is possible either from theory or experiments to estimate the wave loadings and subsequently determine stresses, deformations, and stability.

5. 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 analyzed 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.

\[
\frac{H_b}{d_s} = 0.78 \quad \text{............... (1)}
\]

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 \( \sigma \) of the sea surface elevation i.e. \( H_s = \sigma \).

\[
\text{Also, } H_s = 1.416 \, H_{\text{rms}} \quad \text{........... (2)}
\]

Where, \( H_{\text{rms}} = \text{Root mean square wave height} \)

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 \( H_{\text{max}} \) 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 \( H_5 \) \( (H_5 = 1.37 \, H_s \) i.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 \( H_{10} \) \( (H_{10} = 1.27 \, H_s \) i.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
6. 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 structure 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.

6.1. Factors Affecting Armour Unit Stability

6.1.1. 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.

6.1.2. 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 is then 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.

6.1.3. **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 steepeened to near the angle of repose of the cover layer units (usually 1 on 1.25 as a limit).

6.1.4. **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.

6.2. **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. 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. 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 minimum weight and are described below.
6.2.1. **Iribarren Formula**: Until 1930’s, the design of rubblemound structures was based only on general knowledge and experience of site conditions. A first design formula to calculate weight of armour unit of rubblemound structure was developed by Iribarren in 1938. According to Iribaren’s formulae,

\[
W = \frac{K}{(f \cos \theta - \sin \theta)^3 (S_r - 1)^3} H^3 S_r \\
\text{............ (3)}
\]

Where, \( W \) = weight of armour unit (kg)
\( H \) = wave height breaking on the armour unit (m)
\( S_r \) = specific gravity of the armour unit
\( \theta \) = angle of breakwater slope with the horizontal
\( K \) = coefficient (15 for natural stones, 19 for artificial blocks)
\( f \) = coefficient of friction of armour units

6.2.2. **Hudson Formula**: Following the work of Iribarren, 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:

\[
W = \frac{W_r H^3}{K_D (S_r - 1)^3 \cot \theta} \\
\text{............ (4)}
\]

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
\( \theta \) = 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 to 1/5. 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 than 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 of 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 center 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.

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

<table>
<thead>
<tr>
<th>Armor Unit</th>
<th>n</th>
<th>Placement</th>
<th>Structure Trunk</th>
<th>Structure Head</th>
<th>Slope Cot ( \theta )</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>Breaking Wave</td>
<td>Non-breaking Wave</td>
<td>Breaking Wave</td>
</tr>
<tr>
<td>Quarry stones</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Smooth</td>
<td>2</td>
<td>Random</td>
<td>1.2</td>
<td>2.4</td>
<td>1.1</td>
</tr>
<tr>
<td>Rounded</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Smooth</td>
<td>&gt;3</td>
<td>Random</td>
<td>1.6</td>
<td>3.2</td>
<td>1.4</td>
</tr>
<tr>
<td>Rounded</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Rough Angular</td>
<td>1</td>
<td>Random</td>
<td>-</td>
<td>2.9</td>
<td>-</td>
</tr>
<tr>
<td>Rough angular</td>
<td>2</td>
<td>Random</td>
<td>2.0</td>
<td>4.0</td>
<td>1.9</td>
</tr>
<tr>
<td>Rough angular</td>
<td>&gt;3</td>
<td>Random</td>
<td>2.2</td>
<td>4.5</td>
<td>21</td>
</tr>
<tr>
<td>Rough angular</td>
<td>2</td>
<td>Special</td>
<td>5.8</td>
<td>7.0</td>
<td>5.3</td>
</tr>
<tr>
<td>Parallelepiped</td>
<td>2</td>
<td>Special</td>
<td>7 - 20</td>
<td>8.5 - 24</td>
<td>--</td>
</tr>
<tr>
<td>Graded angular</td>
<td>---</td>
<td>Random</td>
<td>2.2</td>
<td>2.5</td>
<td>--</td>
</tr>
<tr>
<td>Tetrapod &amp; Quadrupod</td>
<td>2</td>
<td>Random</td>
<td>7.0</td>
<td>8.0</td>
<td>5.0</td>
</tr>
<tr>
<td>Tribar</td>
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<td>Random</td>
<td>9.0</td>
<td>10</td>
<td>7.8</td>
</tr>
<tr>
<td>Dolos</td>
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<td>Random</td>
<td>15.8</td>
<td>31.8</td>
<td>8.0</td>
</tr>
<tr>
<td>Modified cube</td>
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<td>Random</td>
<td>6.5</td>
<td>7.5</td>
<td>--</td>
</tr>
<tr>
<td>Hexapod</td>
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<td>Random</td>
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<td>9.5</td>
<td>5.0</td>
</tr>
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<td>Random</td>
<td>11.0</td>
<td>22.0</td>
<td>--</td>
</tr>
<tr>
<td>Tribar</td>
<td>1</td>
<td>Uniform</td>
<td>12.0</td>
<td>15.0</td>
<td>7.5</td>
</tr>
</tbody>
</table>

*“Design & Construction Of Coastal Structures” by A.V. Sitarama Sarma, RO, CWPRS*
6.2.3. **Per Brunn’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 analyzed the flow conditions as a result of wave attack on the rubble mound 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:

\[
\xi = \frac{\tan \theta}{\sqrt{H/L_0}} = \sqrt{\frac{g}{2\pi}} (\tan \theta) \frac{T}{\sqrt{H}} \quad \text{...........(5)}
\]

This parameter describes the overall flow characteristics like breaking waves, run-up and run down and the effect of wave period. Per Brunn indicated that the forces trying to dislocate the armour units maximise with deep run down conditions occurring simultaneously and repeatedly with collapsing, surging or plunging wave breaking conditions, thus corresponds to the range of ξ values between 2 and 3. Per Brunn concluded that the wave period is very significant parameter in the design and it is supported by a number of observations in the North Sea and Arctic Sea.

6.2.4. **Van der Meer Formula**: Van der Meer (1988) has given classification of coastal structure based on parameter which is called ‘Stability Number’. The stability number is

\[
Ns = \frac{H}{\Delta D} \quad \text{.......... (6)}
\]

Where as H = wave height,
\[\Delta = \text{relative mean density}\]
\[D = \text{Characteristics dimension of the armour unit (rock or concrete)}\].
Small values of $N_s$ 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.

![Figure 5: Types of Structures as a Function of $H/\Delta D$](image)
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 $H / \Delta D = 1 - 4$. 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 $H / \Delta D= 1$ to 500. Types of structures with function of $H / \Delta D$ are shown in Fig. 5.

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, groupiness of waves and permeability of the core.

For Plunging Waves:

$$\frac{H_s}{\Delta D_{n50}} = 5.7P^{0.14} \left( \frac{S}{\sqrt{N}} \right)^{0.2} (\xi)^{0.5}$$

............. (7)

For Surging Waves:

$$\frac{H_s}{\Delta D_{n50}} = 0.83P^{-0.2} \left( \frac{S}{\sqrt{N}} \right)^{0.2} (\xi) \sqrt{\cot \theta}$$

............. (8)

Where, $H_s$ = Significant wave height
$D_{n50}$ = Nominal diameter of the armour unit
$\xi$ = Surf similarity parameter
$P$ = Porosity
$S$ = Damage Level
$N$ = Number of waves
$\theta$ = 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 ($\cot \theta = 1.5$), $S = 3 - 5$ 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 groupiness of wave have no influence on the stability of the breakwater.
6.3. **Thickness of Armour Layer and Under Layer**

The thickness of the cover under layers and the no. of armour units required can be determined from the following formulae:

\[
 r = nK_{\Delta} \left( \frac{W}{W_r} \right)^{\frac{1}{3}}
\]

Where,
- \( r \) = Average layer thickness (m)
- \( n \) = No. of armour units in thickness comprising cover layer
- \( K_{\Delta} \) = 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

\[
 N_r = \frac{nK_{\Delta}}{A} \left[ 1 - \frac{P}{100} \right] \left( \frac{W_r}{W} \right)^{\frac{1}{2}}
\]

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.

6.4. **Design of Structure Cross Section**

The rubble structure is normally compound of a bedding layer and a core of quarry run stone covered by one or more layers of stones and an exterior layer or layers of quarry stone (armour) or concrete armour units.

Rock size gradation:

<table>
<thead>
<tr>
<th>Layer</th>
<th>Rock Size</th>
</tr>
</thead>
<tbody>
<tr>
<td>Primary cover layer</td>
<td>( W )</td>
</tr>
<tr>
<td>Secondary layer</td>
<td>( W / 10 ) to ( W / 15 )</td>
</tr>
<tr>
<td>Core</td>
<td>( W / 200 ) to ( W / 6000 )</td>
</tr>
</tbody>
</table>

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.

6.5. **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 seawall must be built. The actual run up value depends on the characteristics of the seawall (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:

\[
 B = nK_{\Delta} \left( \frac{W}{W_r} \right)^{\frac{1}{2}}
\]

............ (11)
Where, $B =$ Crest width (m)  
$n =$ No. of stones  
$K_\Delta =$ Layer coefficient

### 6.6. Toe Berm for Cover Layer Stability

Structures exposed to breaking waves should have their primary cover layers supported by a quarry stone toe berm. For preliminary design purposes, the quarry stone in the toe berm should weight $W/10$. The width of top of the berm is calculated using equation (11) with $n = 3$. The maximum height of the berm is calculated using equation of (9) with $n = 2$.

### 6.7. Design of Filters

Appropriate filter layer on the land side between the backfill and the stone layer and below the rubblemound should be provided in order to prevent leaching of soil from the backfill as well as from the base, due to wave action.

#### 6.7.1. Graded Stone Filter:

The design of the filter material is based on Terzaghi – Vicksburg criteria,

\[
\frac{d_{15} \text{Filter}}{d_{85} \text{Base}} < 5 \quad \text{............... (12)}
\]

\[
4 < \frac{d_{15} \text{Filter}}{d_{50} \text{Base}} < 20 \quad \text{............... (13)}
\]

\[
\frac{d_{15} \text{Filter}}{d_{50} \text{Base}} < 25 \quad \text{............... (14)}
\]

$d_{15} =$ Diameter exceeded by coarsest 85% of layer immediately above under layer  
$d_{50} =$ Diameter exceeded by coarsest 50% of layer immediately above under layer  
$d_{85} =$ Diameter exceeded by coarsest 15% of layer immediately above under layer

#### 6.7.2. Geo-fabric Filter:

A) For Coarse grained soils:

i) $O_{90} \leq D_{85}$ of Soil; where $O_{90} =$ 90% opening size of filter mesh  
ii) $O_{90} \geq D_{15}$ of Soil  
iii) $O_{90} =$ always $\geq 0.05$ mm

B) For Fine grained soils:

i) $O_{90} \leq 0.12$ mm  
ii) $O_{90} \geq 0.05$ mm  
iii) Permeability $\geq 30$ lit./m²/sec, under 100 mm head
7.0 HYDRAULIC MODELLING OF RUBBLEMOUND STRUCTURES

The conceptual design of breakwater / seawall 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 / seawall location, length and alignment to provide optimum wave protection for the harbour or other coastal installations.

Breakwater / seawall models 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 h 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

Sometimes a pilot study is necessary to a small scale in order to obtain a picture of the complete problem. From this important aspects can be identified and separated into larger scale models. The generation of waves in models is a highly sophisticated procedure particularly in flumes where large-scales oscillations are required.

7.0 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 2 Comparison of Flume & Field conditions**

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

### 7.1 Testing of Breakwater Models

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 stability.
The breakwater model is constructed at the testing section of 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.

\[
\frac{(W_a)_m}{(W_a)_p} = \left( \frac{\gamma_a}{\gamma_p} \right) \left( \frac{L_m}{L_p} \right)^3 \left( \frac{(S_a)_p - 1}{(S_a)_m} \right)^3
\]

Where, subscript m denotes model and subscript p denotes prototype

\( W_a \) = Weight of armour unit
\( \gamma_a \) = Specific weight of armour unit
\( S_a \) = Specific gravity of an armour unit relative to water in which breakwater is situated
\( L \) = 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 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.

**8.0 DEVIATIONS IN DESIGN AND CONSTRUCTION OF SEAWALLS**

**8.1 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.

**8.2 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.

**8.3 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.
8.4 Inadequate or no-provision of filters

Many rubblemound structures have failed due to no or inadequate provision of filter underneath (Photo-5). 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. 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.

8.5 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.

8.6 Rounded Stones

The in-place stability of an armour unit is dependant 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.

8.7 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-6). 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: Inadequate Filter Layer

Photo 6: Pockets in Armour Layer
8.8 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-7). 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. 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.

8.9 Armour in Single Layer and / or Pitched

Several constructions in the country have been taken up with revetment type pitching of rubble (Photo-8) 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.

8.10 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-9). 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.

9.0 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.

10.0 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-10).

REFERENCES


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