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Review Article

Present Practices in Design of Rubblemound Breakwaters for Coastal Harbours-A Review

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Abstract

Harbour is defined as a place on a body of water that provides protection for the variety of ships from the coastal environmental parameters like waves, wind, tides and currents etc. Breakwaters are typically required for the harbours to provide desired tranquillity and protection for the ships approaching and mooring in the harbour. The harbour layout, the length and alignment of protective breakwater are decided through hydraulic model studies. Flexible rubblemound breakwater is the most commonly used type of breakwater. The hydraulic design of the breakwater structure is evolved through empirical methods and hydraulic model tests. Present methodology and model techniques used to design the cross-sections of rubblemound breakwaters are reviewed and illustrated with a case study of design of breakwaters for fisheries harbour. Suggestions are given to improve and optimize the design. Provision of a wider toe-berm and use of armour units with higher stability, significantly reduces the cost of rubblemound breakwater.

Keywords: *Coastal Harbours, Rubblemound Breakwaters, Wave Tranquility, Armour Units, Hydraulic Stability.*

1. Introduction

Rubblemound breakwater is the most common type of structure constructed to achieve tranquil conditions in the harbours against the waves and currents. The breakwaters also minimize the siltation in harbours and entrance channels. Thus, the breakwaters provide calm conditions and help in maintaining the depths in harbours for operations of boats. Design of flexible rubblemound structure is complex, as it involves various aspects such as wave-structure interaction, interlocking characteristics of armour, friction between armour and secondary layer etc.

Rubblemound breakwater is a flexible structure consisting of different layers of individual stones and a covering layer of larger stones or artificial concrete armour units. Stability of rubblemound breakwater depends upon the stability of individual armour units on its seaward slope. Therefore, a major aspect in the design of rubblemound structures is the determination of minimum weight of the armour units required on the seaward slope to withstand design wave condition.

The present methods and modelling techniques used to design the rubblemound breakwaters are reviewed and presented in the paper. The design procedure is illustrated with a case study of design of breakwaters for a fisheries harbour. The model studies were carried out at Central water and Power Research Station (CWPRS), Pune as a part of M. Tech. curriculum of first author.

2. Literature Review

Large number of research papers, reports and manuals are available for the design of rubblemound breakwaters, including wave flume studies for the confirmation of hydraulic stability under the attack of waves. Shore Protection Manual (1984) of US Army Corps of Engineers is commonly followed by the coastal engineers to design the rubblemound breakwaters. Many studies were carried out on the hydraulic stability of individual armour units on the seaward slope and several empirical formulae such as, Hudson's formula (1959) and Van der Meer formula (1988) have been derived for the estimation of the minimum weight. The present practice followed by the designers of rubblemound structures is to evolve the conceptual design by empirical formulae, which is confirmed and finalized by hydraulic model tests in wave flumes / basins.

Mahalingaiah *et al* (2014, 2016) have described the utility of wave flume studies in designing the rubblemound breakwater structures. Some of the new concepts have been introduced in the design, based on the hydraulic stability tests in wave flume/basin. It has

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been opined that 3-D model tests in a wave basin can further optimize the design of rubblemound breakwater. It is also stated that a concept of a breakwater with a wide toe-berm reduces the cost of construction of breakwater. This concept is also useful in rehabilitation of damaged rubblemound breakwaters. Studies indicated that a toe-berm of about 20 m width just at the low water level and having stone weight of 1/5 to 1/10 of the armour units of conventional breakwater, reduces the required weight of stable armour units by about 20-50%.

3. Methodology

Rubblemound breakwater consists of graded layers of stones and an armour layer of heavy stones or artificial concrete units. Main advantage of rubblemound structure is that, the failure of armour/cover layer is not sudden. The damage is gradual, and can be repaired. In some cases, it is economical to use smaller size armour units, expecting some damage during a storm, and making a provision for repairs.

Armour units must be having sufficient weight to resist wave forces. If the entire structure consists of units of this larger size, the structure would allow high wave energy transmission and finer material in foundation could easily get removed through the voids. Thus the stone sizes are graded, in layers, from the large exterior armour units to small quarry-run sizes at the core and bedding layer.

The other considerations for design of rubblemound structure include prevention of scour at the seaward toe, spreading of structure load, so there is no foundation failure under excessive loads. Sufficient crest elevation and width shall be provided so that wave run-up and overtopping do not cause failure of the leeward slope of the structure. The crest width may be governed by minimum roadway width needed for construction vehicles.

The hydraulic stability of the rubblemound structure mainly depends on the stability of armour layer since it takes the major brunt of waves. Various factors affecting the stability of armour layer are:

3.1 Incident Wave Height

The significant wave height instead of maximum wave height is commonly used as the design wave height, since the failure of rubblemound structures is gradual. Sometimes more conservative wave height such as H_{10} is also used. Consideration should be given to the expected duration of wave attack, while selecting a design wave height. It is also important to determine, whether the design wave will break on the structure or before reaching the structure. If the water depth at the toe is sufficient for the design wave to sustain, then it is considered as Non-breaking wave condition. If the wave is breaking at the toe of the structure the armour layer should be stable for breaking wave condition.

3.2 Armour Unit Size, Weight, Shape, Location and Method of Placement

Armour unit stability formulae give the required stable weight of the armour unit for the given design wave height. The size depends on the various properties of armour units such as shape, interlocking property, and the specific weight of rock or concrete. Main resistance to hydrodynamic forces is developed by the interlocking of units, which depends on the shape, gradation and the method of placing. The artificial concrete armour units of various shapes have been developed to increase interlocking between the units. Armour unit stability also depends on location of units. Vulnerability to wave attack is higher at the head of the breakwater and is higher than along the trunk.

3.3 Armour Layer Thickness, Porosity & Slope

The armour units are placed in double layer to achieve the stability and prevention of removal of smaller size stones from the under layers. The single layer armour units are also in use, which effect overall savings in the quantity of concrete. The higher porosity of the layer helps in dissipating the wave energy. Layer porosity varies between 35-55 percent, depending on armour unit shape and placement method. Lower porosity causes reflection of waves and increased wave run-up. The armour unit stability is higher on flatter armour slopes. Typical seaward slopes of breakwater vary from 1 on 1.5 to 1 on 3. Leeward slope of a breakwater can be steeper, usually 1 on 1.5.

3.4 Allowable Damage

The degree of damage is defined as the percentage damage to the armour units. It is the number of armour units displaced from the armour layer expressed as percentage of total number of units in the layer. For the breakwater structures, 1% to 2% damage is allowable. Damage should not be allowed to the extent that interior layers are exposed to direct wave attack. The allowable damage depends on initial cost, maintenance cost and the allowable risk in case of failure of the structure.

4. Design Wave Conditions

Waves exert large forces on breakwaters and wave is main parameter in the design of cross-section of breakwater. As such, determination of design wave condition plays important role in the planning and design of coastal facility. The design conditions are determined from the analysis of wave data. There are three main sources of wave data:

a) Instrumentally collected wave data
b) Visually observed wave data (ship observed)
c) Hindcast storm wave data

Instrumentally collected wave data are available for larger coastal projects. However, these data are seldom

available for longer duration. For determination of correct design wave conditions, long term wave analysis is required for which wave data of larger duration (at least 15-20 years) are essential. As such, it is a general practice to use ship observed wave data in the deep sea. These visually observed data are transformed to nearshore conditions and then utilized to determine the design wave conditions. However, for determination of extreme wave conditions, hindcast storm wave data are used. Deep water wave heights and periods can be determined if the wind speed, wind duration and fetch length data are available.

Generally, breakwaters 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 at the structure (d_s), by the relation:

$$H_b = 0.78\ d_s$$

If breaking does not limit the 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. Significant wave height is the average height of the one-third highest waves ($H_{1/3}$ or H_s) and it is almost equal to the height of the visually observed waves as reported by an experienced observer.

An alternative definition of H_s, sometimes applied as 4 times of standard deviation (σ) of the sea surface elevations i.e.

$$H_s = 4\sigma.$$

Also, $H_s = 1.416\ H_{rms}$

Where, H_{rms} = Root Mean Square wave height

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

For a *rigid structures*, 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 (i.e. average of the highest 1 percent of all waves, also H_1= 1.67 H_s).

For *semi-rigid structures*, the design wave height is selected from a range of H_1 to H_5 (i.e. average of the highest five percent of all waves, H_5 = 1.37 H_s).

For *flexible structures*, such as rubblemound or riprap structure, the design wave height is between H_s and H_{10} (i.e. average of the highest ten percent of all waves, H_{10} = 1.27 H_s).

Kudale and Bhalerao (2015) have described the design procedures for the coastal rubblemound structures, selection of design wave height and equivalence of the monochromatic and random waves. They suggested that the design wave height for rubblemound breakwaters shall be between H_{10} and H_{20}.

5. Armour Unit Stability

The stability of rubblemound structures depends primarily upon the stability of individual armour units on its seaward slope. Thus, determination of minimum weight of the armour units required to withstand the design waves is a major aspect in the design of rubblemound structures. The interaction of waves with the armour units is a complex phenomenon. It is difficult to predict the maximum forces exerted on the individual armour unit, which make it unstable in the armour layer of rubblemound structure. Many studies have been reported on the hydraulic stability of individual armour unit. Some empirical formulae have been derived for the estimation of the minimum weight of armour units, such as Iribarren formula (1938), Hudson's formula (1959) and Van der Meer formula (1988). The evolution of stability formulae is well described in Shore Protection Manual (SPM, 1984). Amongst these stability formulae, the Hudson's stability formula is the most commonly used formula by the coastal engineers.

5.1 Hudson's formula

Comprehensive investigations were carried out by Hudson (1959) at US Army Corps of Engineers, Waterways Experiment Station, Vicksburg. Based upon the experimental results, Hudson suggested the following formula for the armour units:

$$W = \frac{w_r H^3}{K_D (S_r - 1)^3 Cot\theta} \quad (1)$$

where,
W = Weight of armour units (kg)
w_r = Unit weight of armour block (kg/cum)
H = Design wave height (m)
S_r = Specific gravity of armour units
θ = Angle of breakwater slope
K_D = Stability coefficient

Stability coefficient (K_D) varies with the type of armour unit. It takes into account roughness, Sharpness of edges, quality of interlocking etc.

The Hudson formula is the most popular and has been in use for the last few decades for the design of breakwaters because of its simplicity. Hudson considered wave periods varying from 0.8 sec to 2.65 sec and the armour layer slope from 1/1.25 to 1/5 in his experiments. All the experiments were conducted for non-overtopping and non-breaking monochromatic waves. He also established K_D values for stones and artificial concrete armour units viz. Tetrapods, Tribars etc. These values were worked out for no damage condition (i.e. the damage to the armour units less than 1%). K_D values for stones and tetrapods are 2 and 7 respectively for breaking waves.

K_D values have been evolved by number of laboratories through the scaled model tests. These laboratory studies to evaluate K_D have considered waves of constant period. The significant height H_s is the most appropriate wave height to be used for H in above equation. The studies by Kudale and Bhalerao (2015) indicated that H_{10} is more appropriate than H_s.

Tetrapod is popular concrete armour unit used in coastal works in India. Recently, Accropodes, and Core-locs, which are single layer armour units are also being used because of savings in the overall cost.

5.2 Thickness of Armour Layer and Placing Density

The thickness of the cover layer and the number of armour units required for unit surface area are determined from the following formulae:

$$r = nK_\Delta \left[\frac{W}{w_r}\right]^{1/3} \quad (2)$$

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

The placing density, (i.e. number of units to be used in 100 m^2 surface area) is given by

$$\frac{Nr}{A} = nK_\Delta \left[1 - \frac{P}{100}\right]\left[\frac{wr}{W}\right]^{2/3} \quad (3)$$

Where,
N_r = armour units on sloping surface
A = Surface area,
K_Δ = Layer coefficient
P = Average porosity of a cover layer

6. Design of Under Layers

The rubble structure is normally composed of a bedding layer and a core of quarry stones covered by one or more layers of bigger stones and an exterior layer of armour stones or concrete armour units. Gradation of stones in each layer is given as:

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 are carried over to the crest and for a certain length of lee side slope, so as to withstand any overtopping that may cause during severe storms.

6.1 Crest Elevation and Width

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

$$B = nK_\Delta \left[\frac{W}{w_r}\right]^{1/3} \quad (4)$$

Where,
B = Crest width (m),
K_Δ = Layer coefficient
n = No. of stones in the top layer at the crest

6.2 Toe Berm for Cover Layer Stability

Structures exposed to breaking waves should have the primary cover layer supported by a toe berm of quarry stones. For preliminary design purposes, the quarry stones in the toe berm should weight about W/10 to W/15. The width of top of the berm is calculated with n=4 in the width formula. The height of the berm is calculated with n=2 in thickness formula.

6.3 Design of Filters

Appropriate filter layer on the sea bed and the stone layer of the rubblemound should be provided in order to prevent leaching of sediments from the base, due to wave action. In fact, all the layers of the rubblemound are designed as per the filter criteria.

6.3.1 Graded Stone Filter Criteria

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

$$\frac{d_{15}\,Filter}{d_{85}\,Base} < 5$$

$$4 < \frac{d_{15}\,Filter}{d_{15}Base} < 20$$

$$\frac{d_{15}\,Filter}{d_{50}\,Base} < 25$$

Where,
d_{15} = Diameter exceeded by 15% of layer material
d_{50} = Diameter exceeded by 50% of layer material
d_{85} = Diameter exceeded by 85% of layer material

7. Hydraulic Modelling

The conceptual design of breakwater can be carried out using available empirical methods. These empirical methods are based on field experience and physical model studies. The wave structure interaction is a complex phenomenon, which is difficult to simulate by

mathematical modelling. As such, hydraulic modelling of breakwater needs to be carried out in a physical model, in laboratory wave flume / basin facility to evolve safe and optimal design. The primary objective of model testing of rubblemound breakwater is to confirm the stability of structure and to get information on hydraulic performance of structure, in terms of reflection, run-up, over-topping and transmission.

The breakwater stability under the action of waves is simulated in Physical model according to the Froude's Law, since the inertia and gravity forces are predominant. The inertial forces are always present and viscous forces can be made negligible by selecting the proper model scale so that Reynolds number should be greater than 30,000. The ratio of inertial forces and gravitational forces is called as Froude's Number. It should be equal to one for dynamic similarity. Hence stability of rubblemound structure in model is based on Froude's law.

The wave models are Geometrically Similar, since water particle motion is horizontal as well as vertical in the wave propagation. As such, the model cannot be distorted. The Froude's Model Law is given as:

$$\frac{V_r}{\sqrt{G_r L_r}} = 1 \qquad (6)$$

Where,
L_r = Geometric Scale Ratio
V_r = Velocity Scale Ratio
G_r = Gravity Scale Ratio

From this basic model law the scale ratios for Geometrically Similar Froudian model are:

Length scale ratio = L_r
Area scale ratio = $(L_r)^2$
Volume scale = $(L_r)^3$
Velocity scale, V_r = $(L_r)^{1/2}$
Time Scale, T_r = $(L_r)^{1/2}$

If the density of stones, concrete and water is kept same in model and prototype, the force or weight scale is also equal to volume scale (i.e. $(L_r)^3$). Usual scales adopted for wave flume studies are, 1: 20 to 1: 60.

7.1 Wave Flume Facility

The hydraulic stability tests of breakwater model are carried out in a wave flume facility for normal attack of waves. Thus the model tests in wave flumes are called as 2-D model tests. A section of a breakwater with geometrically reduced dimensions and weights is reproduced in the flume. The density of stones and concrete armour units is kept same as in the prototype. Fresh water is used in the flume instead of sea water, since there is negligible difference in density of sea water and fresh water. The waves are generated in the flume with the help of wave generators. There are two types of flumes, regular wave flumes and random wave flumes. In the present studies tests were carried out in a regular wave flume. Since the design of breakwaters is governed by the breaking waves, which were reproduced in the wave flume.

8. A Case Study

The present study includes a case study of design of breakwaters for the proposed fisheries harbour. Presently, there is a small fishing port facility, operational with a wharf of 261 m length. The wharf is protected by a vertical breakwater of 282 m length. The vertical faces of the breakwater are also used as landing / outfitting berth by the fishermen. The present facility at the port is unable to meet berthing and navigational requirements of the fishing boats. Hence fisheries department have a proposal for development of the present harbour with additional facilities for berthing, landing and boat repairs etc by providing a north breakwater, extending the existing breakwater and keeping an entrance of about 100 m width. The draft requirement of the boats is 2.5 m. However, future needs of deeper drafts up to 4.5 m are also to be considered.

Initially it was proposed to extend the existing south breakwater by 258 m (Total length 540 m) and construct a 745 m long north breakwater. The harbour area is to be dredged up to -2.5 m depth. However, the initial studies and the review of requirements and the site conditions indicated that the proposed layout plan is not feasible. It would be difficult to dredge the rocky seabed up to -2.5 m. The harbour area created by the breakwaters would be in-sufficient and also would not fulfill the future requirements. As such, the initial layout plan was modified. In the modified layout plan, it is proposed to extend the existing 282 m long south breakwater by another 770 m length so that the total length of breakwater would be 1052 m up to -8 m contour. A north breakwater is to be provided with its length of 976 m up to -6 m contour (Fig. 1).

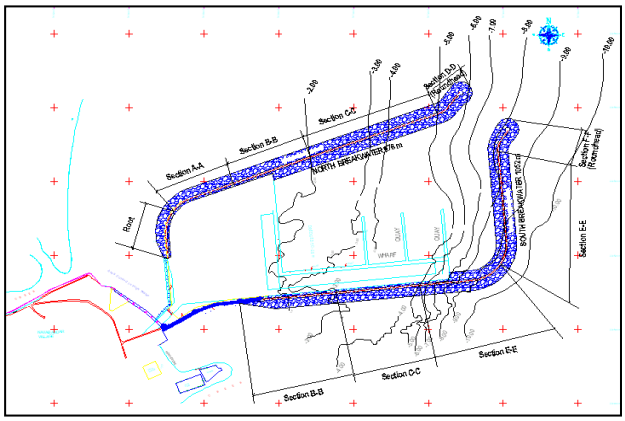

Fig.1 Layout of a Fisheries Harbour

A 150 m wide opening is provided at the harbour entrance. The extension of south breakwater and provision of north breakwater would enable to

accommodate various berthing facilities in the sheltered area. The extension of breakwater will also avoid dredging in the harbour area, which is difficult due to rocky seabed conditions. Furthermore, depths up to -4.5 m are available in the eastern portion of the harbour for the bigger boats.

The layout of the port as shown in Fig. 1 was finalized through mathematical model studies for wave tranquility, hydrodynamics and sedimentation. These studies showed that the required wave tranquility of 0.3 m is achieved in the harbour basin throughout the year. Also, there will not be any significant siltation in the harbour due to waves and tidal currents. The construction of harbour will not have significant impact on the coastline in the vicinity of harbour (Kudale et al, 2018).

8.1 Design of Breakwater Cross-Sections

The harbour layout consists of 976 m long northern breakwater and 1052 m long southern breakwater, having a clear gap width of 150 m at the entrance of the harbour. In order to design the breakwater cross-sections, each breakwater was divided into various ranges of bed levels. The breakwater cross-sections at various ranges of bed level were worked out, using empirical methods. The hydraulic stability of breakwater cross-sections was confirmed through a physical model tests under the attack of design waves in a "Wave Flume" facility at Central Water and Power Research Station, Pune.

8.2 Design Wave Conditions

Generally, breakwaters are designed for 100 year return period wave heights derived from the long term analysis of cyclonic waves at the site. In case of breaking wave conditions, which exert maximum force on the structures, the breaking wave height (H_b) is the governing wave for the design. The breaking wave height (H_b) can be obtained from the depth of water at the structure (d_s), by the relation, $H_b = 0.78\ d_s$. If breaking does not limit the wave height, a non-breaking condition exists. A design significant wave height (H_s) and significant wave period (T_z) represent the characteristics of the sea in the form of monochromatic or regular waves.

From the analysis of available wave data, the Consultants of the port evolved the following design wave conditions for the breakwaters:

1) Zero order damage (i.e. Less than 1%) with the wave height of the order of 7.5 m at the Mean High Water Level (MHWL) of +2.4 m.
2) First order damage (i.e. between 1% and 5%) with the wave height of the order of 8.0 m at the Mean High Water Level (MHWL) of +2.4 m.
3) The wave periods from 10 to 12 seconds.
4) The stability of the section is also to be assessed at low and intermediate water levels.

8.3 Conceptual Design

Cross-sections of rubblemound breakwater at different bed levels were evolved based on empirical formulae, considering the wave condition at the respective bed levels. The sections at different bed levels are marked along the alignment of the breakwaters (Fig. 1). Tetrapods have been used in the armour layer. Stable weight of tetrapods was worked out using Hudson's formula (SPM, 1984). The other features of the breakwater were also worked out using the conceptual methods described earlier. In view of the utility of wider toe berm in dissipating the wave energy, wider toe berms at higher levels were considered in the design. Lighter tetrapods than those indicated by Hudson's Formula were provided in the armour(Table 1). However, for the cross-sections in shallower depths of water, it is difficult to accommodate a wider toe-berm at higher level.

Table 1 Armour Weight for Different Sections

Section	Bed Level (m)	Breaking Wave Height (m)	Weight of Tetrapod Formula	Weight of Tetrapod Actual
A-A	-1.0	2.66	1.3 t	2.0 t
B-B	-2.0	3.44	2.9 t	4.0 t
C-C	-5.0	5.78	13.8 t	15.0 t
D- D Roundhead	-6.0	6.56	28.3 t	20.0 t
E-E	-8.0	8.12 m Limited to 7.5 m	30.1 t	20.0 t
F-F Roundhead	-9.0	8.9 m Limited to 7.5 m	42.1 t	25.0 t

The leeside is assumed to be exposed to waves of the order of 1.0 m during construction. As such, the leeside slope designed for 1 m wave height. Typical design cross-section at -8.0 m bed level is shown in Fig. 2.

8.4 Cross-section at -8.0 m Bed Level (Section E-E)

Section E-E (Fig. 2) is designed for the portion of south breakwater from -5.0 m to -9.0 m bed level.

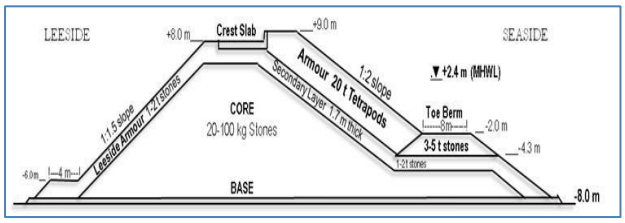

Fig. 2 Cross-Section of Breakwater at -8.0 m Bed Level (Section E-E)

The section is designed to withstand for design wave height 7.5 m, which would be breaking on the toe berm. This section consists of 20 t tetrapods in the seaside armour layer on 1:2 slope. 1 to 2 t stones are provided in the leeside armour on 1:1.5 slope. A 8 m wide toe-berm consisting of 3 to 5 t stones is provided at the level -2.0 m on sea side. A secondary layer of 1

to 2 t stones is provided below the armour layer. Core consists of 20-100 kg stones and a bedding layer of stones up to 10 kg weight is proposed. The top of the crest slab is at +8.0 m level with a parapet top at 9.0 m level. A clear carriage way width of 8 m is provided at the crest slab.

9. Hydraulic Model Tests

9.1 Model Scale

The model tests for the design of breakwaters were conducted in a wave flume. The model tests for a typical representative Section E-E at -8.0 m bed level (Fig. 2) are described herewith.

The section of breakwater was reproduced in a wave flume with a Geometrically Similar model scale of 1:44. The model was based on Froude's criterion of similitude. The Froude's law considers that gravity forces are dominant and other forces such as viscous forces, capillary forces etc., are insignificant. As per Froude's criterion, the various scales obtained are as shown in Table 2.

Table 2 Model Scales

Geometrically Similar Model scale 1:44	
Parameter	Scale
Length (L)	1:44
Area (L^2)	1:1936
Volume (L^3)	1:85184
Time ($L^{1/2}$)	1:6.6
Velocity ($L^{1/2}$)	1:6.6

9.2 Physical Model in Wave Flume

The trunk section of breakwater was tested under normal attack of waves in 2-D wave flume for its hydraulic stability. The section was constructed to a Geometrically Similar model scale of 1:44 in the wave flume. The number of tetrapods provided in double layer on the seaside armour and number of stones in the toe were counted initially, before starting the test. After conducting the tests for one-hour duration, the number of tetrapods 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 stones displaced from their position.

A double layer of 20 t tetrapods was placed in the seaside armour on 1:2 slope and 1 to 2 t stones were placed in the leeside armour with 1:1.5 slope. The top level of the 8 m wide toe-berm consisting of 3 to 5 t stones was fixed at -2.0 m. A secondary layer of 1 to 2 t stones was provided below the tetrapods armour units and below the toe-berm. The top of the crest slab was fixed at +8.0 m level and parapet top was kept at +9.0 m level, with a clear crest width of 8.0 m. The sea bed slope of 1:100 was reproduced in front of the structure.

9.3 Model Tests

Initially a test was carried out with a wave height of 7.5 m at Mean High Water Level (MHWL) of +2.4 m for one-hour duration (corresponding to 6 hours in prototype). It was observed that the highest wave run-up was just above + 8.10 m level and rundown was up to - 0.50 m. The waves were breaking on the armour causing no damage to armour.

A test was also conducted with wave height of 7.0 m at Low Water Level (0.0 m). There was negligible splashing and no overtopping of the waves. It was observed that the highest wave run-up was just above +6.0 m and rundown was up to -2.4 m. The waves were breaking on the armour and toe-berm causing no damage to these components. However, a few stones in the toe berm were rocking during the test.

Another test was conducted with wave height of 8.0 m at MHWL of + 2.4 m. Marginal splashing was observed, however there was no overtopping of the crest. It was also observed that the highest wave run-up was just above +8.8 m and rundown was up to -0.10 m. The waves were breaking on the armour causing vibrations in few tetrapods. However there was no displacement of tetrapods. The stones in the toe-berm showed minor displacement.

A test with 7.5 m wave height was also carried out at the Mean Sea Water Level (MSL) of +1.37 m. All the tests were carried out for the wave periods of 10 seconds as well as 12 seconds. The section, in general, was found hydraulically stable with all the tests.

Typical photograph of wave flume test is shown in Photo 1.

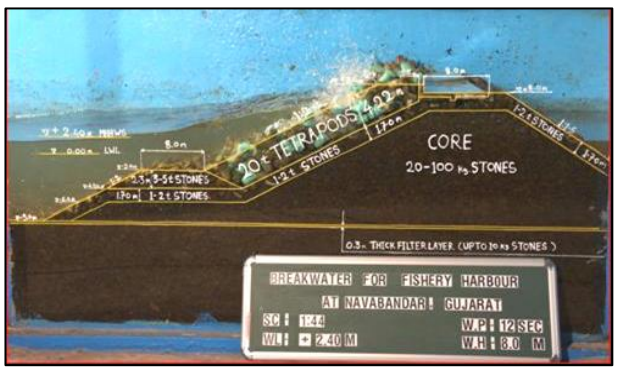

Photo 1 Action of 8 m waves at MHWL on armour layer of Breakwater

10. Results and Discussions

Hydraulic model tests indicated that the conceptually worked out designs of cross-sections are stable under the design wave conditions.

The stable weight of armour units is generally worked out using Hudson's Formula. However, this formula considers a rubblemound breakwater cross-

section without a toe berm. The toe-berm supports the armour as well as it dissipates some of the wave energy. A provision of a wider toe berm at higher level helps in reducing the required stable weight of armour unit (Poonawala et al, 2001). As such, this concept was used to design the cross-sections of breakwaters at various bed levels. The design was optimized through wave flume studies. The weight of the Tetrapod armour units required to withstand different wave heights worked out by empirical formula and the optimum weights evolved through hydraulic stability tests in wave flume (Table 1) are compared in Fig. 3. It is observed that, for the deeper sections the armour weight can be substantially reduced by providing a wide toe berm at higher level. For shallower depths of water, it is difficult to provide toe berm at higher elevation. A nominal toe berm was provided for shallower sections.

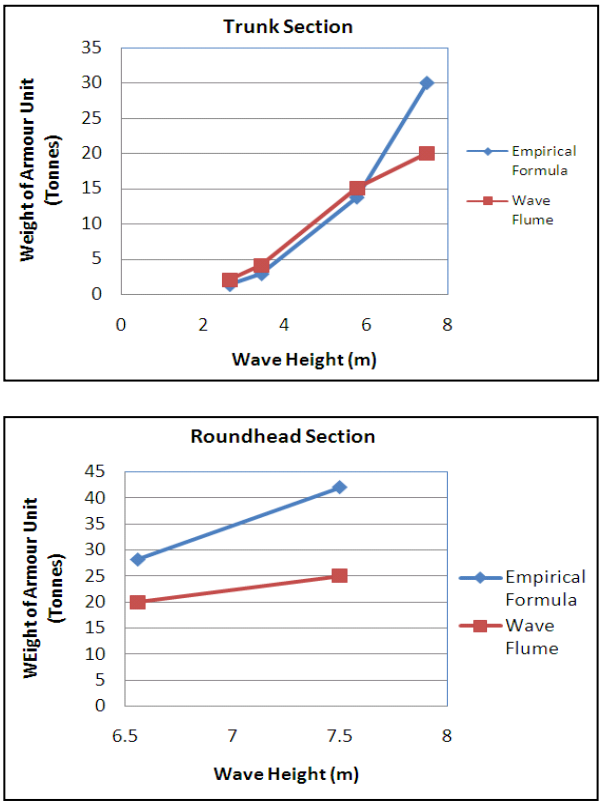

Fig. 3 Stable Weight of Tetrapods

Furthermore the section can be optimized by using different armour units having higher Stability Factor (K_D). The usual armour units like stones and tetrapods are laid in double layer in the armour. The section can be further economized by using single layer armour units like Accropods. Comparison of stable weights worked out by empirical methods is shown in Fig 4. It is seen that there is huge difference in the weights, since it varies with the cube of Wave Height.

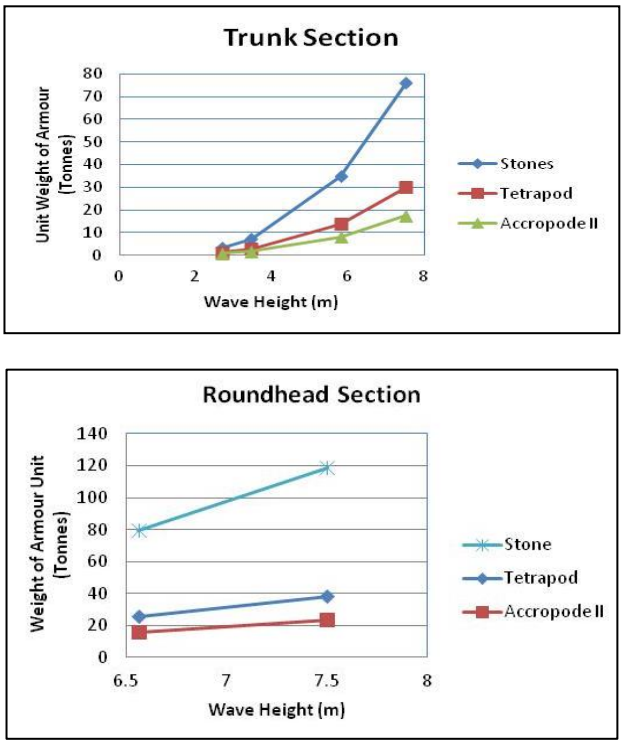

Fig. 4 Stable Weights of Different Armour Units

Conclusions

From the studies carried out to review and illustrate the methods and modelling technique for the design of rubblemound breakwaters, the following broad conclusions are drawn:

1) Present practice adopted in the design of breakwater cross-sections is 'to work out the conceptual design using empirical methods and confirm the stability through hydraulic model test in a wave flume'.
2) The cross-sections of breakwaters for a fisheries harbour were evolved using the empirical methods and a concept of wide toe-berm. Tetrapods up to 20 t weight have been used in the armour. These sections were found stable against the breaking waves up to 7.5 m height.
3) Provision of wider toe berm at higher level, reduces the stable weight of armour unit. The weight of the armour units and thereby the cost of the rubblemound breakwater can be reduced substantially by adopting a concept of wider toe berm.
4) The requirement of heavy armour units can be optimised by adopting concrete armour units with higher stability factor. Use of single layer armour units like Accropodes further reduces the quantity of concrete.
5) A concept of wider toe berm has been used to economize the breakwater section. Empirical relationship needs to be established between width and height of toe berm, water depth, wave height and the stable weight of armour.

6) Design of rubblemound breakwaters is presently based on the physical model studies Commercial Numerical models for wave-structure interaction are required to be developed for the design of rubblemound breakwaters.

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