

DESIGN OF BREAKWATERS FOR THE DEVELOPMENT OF FISHERY HARBOUR –A CASE STUDY

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Abstract: To achieve tranquil conditions in the harbor against the waves, the rubblemound breakwaters are commonly constructed. Their design is complex as it involves various aspects such as, wave-structure interaction, interlocking characteristics of armour, friction between armour and secondary layer etc. Various empirical formulae such as Hudson's Formula and Van der Meer's formula are available for conceptual design of the unit weight of armour. It is mandatory to finalise the section of breakwater based on hydraulic model test in wave flume and validate the conceptual design evolved using empirical methods. The hydraulic model tests simulate complex wave structure interaction as well as correct prototype site conditions of seabed slope, water level etc. These models are constructed to geometrically similar scale and the similarity is based upon Froude's number. The Central Water & Power Research Station (CWPRS), Pune has been associated with the design of most of the breakwaters in the country for last five decades or so. Design of breakwater cross sections considering design wave height for various bed levels, design of breakwater on sea/lee side, trunk/roundhead portion etc, for fishing harbour have been discussed in this paper.

Keywords: - Armour units, Breakwater, Design waves, Tides, Wave flumes etc.

1. INTRODUCTION

The rubblemound breakwaters are provided to achieve tranquil conditions in the harbour basin against the waves. A rubblemound breakwater is comprised of armour layer, toe berm, crest, secondary layers, core, and bedding layer. The toe berm acts as a seat to the armour layer. It helps in dissipating a part of wave energy incident on the breakwater. The top level of the toe is below the low water level. Since the armour layer in the conventional breakwaters bears the brunt of the wave attack, the stones should be massive enough to withstand high waves. The armour weight of breakwater worked out with empirical formula is required to be confirmed with wave flume studies for their hydraulic stability and also for further optimization. The safe and optimal design of rubblemound breakwaters at

various bed levels on seaside and leeside confirmed through wave flume experimentation are discussed in the paper.

2. OBJECTIVES

- 1) To study linear and non-linear wave theory.
- 2) Study of various armour units used for breakwater construction.
- 3) Understanding the design procedure of coastal structures (breakwater).
- 4) Conceptual designing of a breakwater using Empirical Formula (Hudson's Formula).
- 5) Construction of Geometrical Similar 2-D hydraulic model.
- 6) Carrying out experiments in a wave flume for different wave heights & periods.
- 7) Study and analysis of experimental data.
- 8) Confirming the design and making changes if needed.

3. METHODOLOGY

- 1) Studying literature related to marine and coastal structures.
- 2) Preliminary conceptual design of a breakwater section.
- 3) Construction of a geometrical similar model based on Froude's number similitude.
- 4) Carrying out tests in a wave flume.
- 5) Case study (Tuna fishing harbour, Tamil Nadu, India).

4. EXPERIMENTAL WORK

4.1 DESIGN OF CONVENTIONAL RUBBLEMOUND BREAKWATER:

The stability of rubblemound structures is dependent mainly upon the stability of individual armour units on its seaward slope. Design of rubblemound structures is complex involving various aspects such as wave-structure interaction, interlocking of armours, friction between armour and secondary layer, characteristics of armour etc. In the design of rubblemound structures the optimum weight of the armour units on the seaward slope, is a major aspect to be looked into. Extensive studies have been carried out in the previous century on the hydraulic stability of individual armour units on the seaward slope and several empirical formulae such as, Iribarren's, Hudson's, Per Brunn's and Van der Meer formula have been derived for the estimation of the

required stable weight. Based upon the experimental results, Hudson suggested the following formula for the armour units:

$$W = \frac{W_r H^3}{K_D (S_r - 1)^3 \cot \theta} \dots\dots\dots (Eq. No. 1)$$

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
 Most of the formulae derived, were corrected and modified with iterative process over the time. The researchers took into account the wave height, density of the armour units and angle of the breakwater slope. However, in the recent developments it has been 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 rubblemound structures - to determine the conditions which cause the maximum destructive force on the breakwater. Conclusion from their studies highlights 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 H, T and θ has been evolved as:

$$\xi = \frac{\tan \theta}{\sqrt{\frac{H}{L_0}}} = \sqrt{\frac{g}{2\pi}} \frac{(\tan \theta) T}{\sqrt{H}} \dots\dots\dots (Eq. No.2)$$

The Surf Similarity parameter (ξ) describes the overall flow characteristics like run-up, effect of wave period, breaking waves and run-down. The forces which try to dislocate the armour units maximise with deep run down conditions as was indicated by Per Brunn. These conditions occur simultaneously and repeatedly along with collapsing and breaking waves, thus corresponding to the range of ξ values between 2 - 3. Per Brunn concluded that the wave period is a very crucial parameter in the design.

In addition Van der Meer (1988) also studied the dependence of wave period on the armour unit weight. The main shortcomings in the Hudson's formula were regarding the discrepancy to consider the variability of wave period and randomness of waves were solved in the investigations carried out by Van der Meer based on more than 250 laboratory tests, by differing spectrum shape, permeability of the core etc.

For Plunging Waves:

$$\frac{H_5}{\Delta D_{n50}} = 5.7 P^{0.14} \left[\frac{S}{\sqrt{N}} \right]^{0.2} (\xi)^{-0.5} \dots\dots\dots (Eq. No. 3)$$

For Surging Waves:

$$\frac{H}{\Delta D_{n50}} = 0.83 P^{-0.2} \left[\frac{S}{\sqrt{N}} \right]^{0.2} (\xi)^P \sqrt{\cot \theta} \dots\dots\dots (Eq. No. 4)$$

Where,

H_s = 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 1:2 (cot θ = 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.

A common practice followed for designing the rubblemound breakwaters is to use the existing design formulae for each individual component of the breakwater and evolve a conceptual design. The overall stability of rubblemound breakwater is however not only a function of the stability of each individual component but also of the interdependence of these parts. Therefore, in the next step, hydraulic model studies in a wave flume are carried out and the conceptual design of the breakwater is further optimized.

4.2 OPTIMAL DESIGN OF BREAKWATERS FOR THE DEVELOPMENT OF FISHERIES HARBOUR:

The design of rubblemound breakwaters for the proposed development of fishery harbour at Tuna Fishery harbour, Thiruvottriyur Kuppam, Tamil Nadu has been discussed in this paper in detail. The layout plan for the breakwaters was finalized based on physical model studies. The proposed layout consists of 1088 m long south breakwater and an 852 m long north breakwater as can be seen in (Figure 1). The layout plan of the breakwaters for the development of Tuna Fishery harbour was finalized based on mathematical model studies carried out by M/s Virgo Aqua Consulting engineers. The offshore wave climate of the project site at 13.16°N latitude and 80.3° E in the form of parametric quantities has been extracted from National Centre for Environmental Prediction (NCEP).

Based on the site specific data regarding bathymetry, wave conditions and tidal levels, conceptual design of breakwater were worked out using empirical methods. The wave climate during the entire year indicates that the predominant wave directions in deep water are from NE, E and SE with the maximum wave heights of the order of 5.5 m. Zero order damage (i.e. Less than 1%) with the breaking waves of the order of 5.3 m at the High Water Level (HWL) of + 1.5 m. First order damage (i.e. in between 1% to 5%) with the breaking waves of the order of 5.5 m at the High Water Level (HWL) of + 1.5 m. The wave periods from 10 to 12 seconds was considered. The stability of the section was also assessed at low and intermediate water levels.

Detailed desk studies were conducted at CWPRS to develop cross-sections at different bed levels along with extensive in-house wave flume studies for hydraulic stability of breakwaters. The cross sections at different bed levels are marked along the alignment of the breakwaters as shown in Figure 1. The breakwaters consists of 6 different sections (A-A to F-F) 2 t tetrapods in the armour placed at 1:2 slope from the root of breakwater to (-) 2 m bed level, 4 t tetrapods in the armour for (-) 2m to (-) 4m bed level, 6 t tetrapods in the armour for (-)4.0m bed level to (-)6.0m, 8 t tetrapods in the armour for (-)6.0m bed level to (-)8.0m, 8 t tetrapods in the armour for roundhead portion of North breakwater at (-)7.5 m m bed level, 10 t tetrapods in the armour for South breakwater for (-) 8.5 m bed level.

A wide toe provided at the suitable water level of the toe-berm will reduce the wave energy attacking the armour layer. The construction of a toe berm results in reduction of the required stable weight of armour units. The secondary, core and bedding layers are designed with suitable gradation of the stones. The design cross-sections of breakwaters with tetrapods in armour at different bed levels was finalized through wave flume studies as shown in (Figure 2 to 7), considering the design wave conditions at different bed levels.

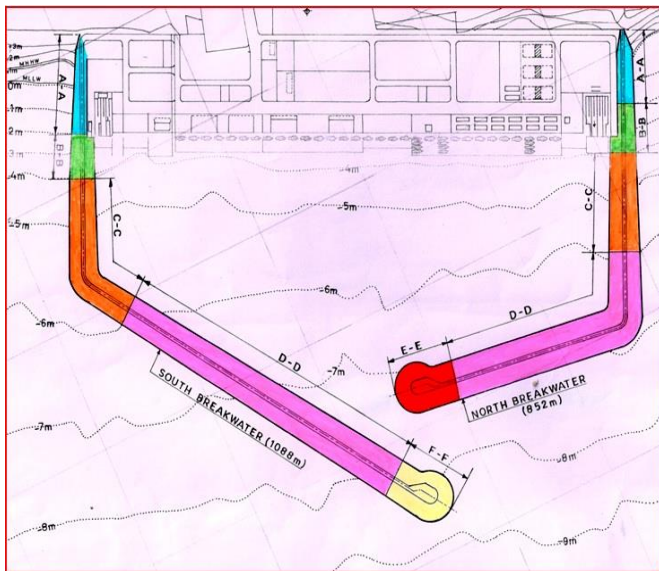


Fig-1: Layout Plan of Breakwaters for the development of Tuna Fishery Harbour at Thiruvottriyur Kuppam, Tamil Nadu

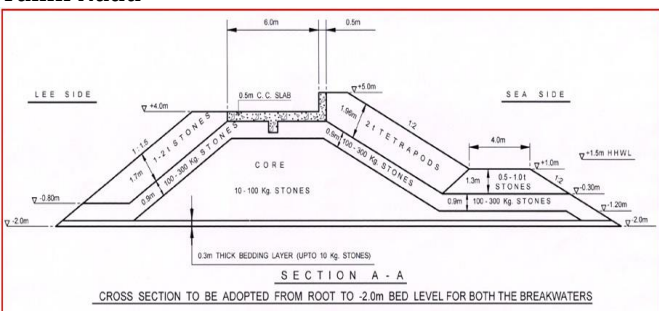


Fig-2: Cross-section to be adapted from root to -2 m bed level for both the breakwaters

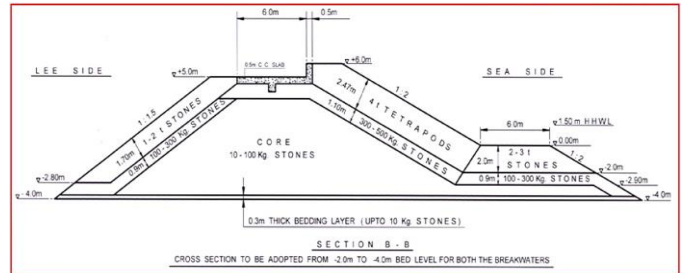


Fig-3: Cross-section to be adapted from -2 m to -4 m bed level for both the breakwaters

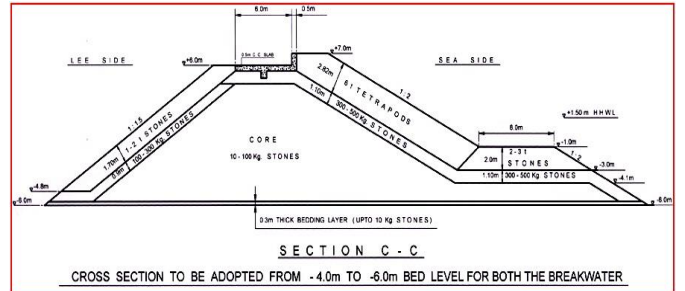


Fig-4: Cross-section to be adapted from -4 m to -6 m bed level for both the breakwaters

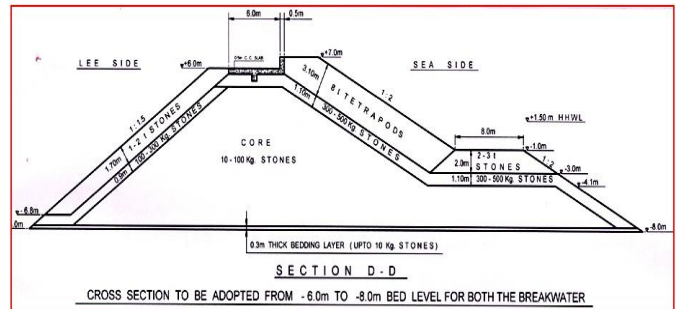


Fig-5: Cross-section to be adapted from -6 m to -8 m bed level for both the breakwaters

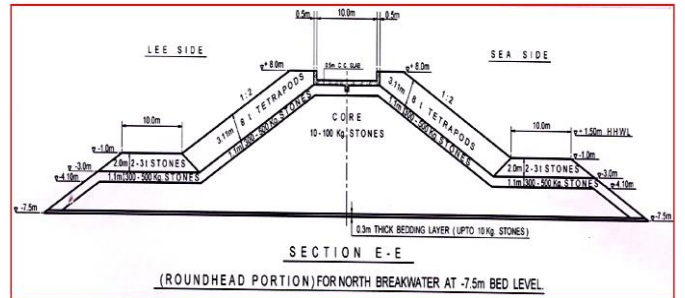


Fig-6: Cross-section to be adapted at -7.5 m bed level for rounded portion of the North breakwater

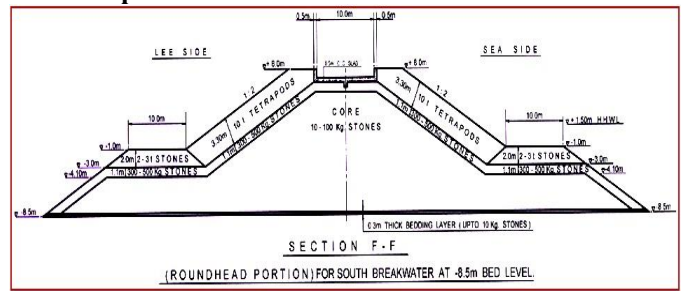


Fig-7: Cross-section to be adapted at -8.5 m bed level for rounded portion of the South breakwater

5. HYDRAULIC MODEL STUDIES

The wave flume facilities are available at CWPRS, Pune as shown in (Figure 8). The breakwater section consists of 10t tetrapods in the armour were constructed with a Geometrically Similar model scale of 1:37 in the wave flume.

The bed level of the representative section was at -8.5 m and the bed slope of 1:100 was reproduced in front of the structure. 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. The trunk section of breakwater was tested under normal attack of waves in 2-D wave flume for its hydraulic stability. The photos taken during the wave flume studies for different wave heights, wave periods and tidal levels are as shown in (Figure 9 and 10.)

The section D-D is designed to provide from -6.0 m to -8.0 m bed level and is shown in Fig. No.10. This section consists of 8 t tetrapods in the armour with 1:2 slope on sea side and 1.0 to 2.0 t stones in the leeside armour with 1:1.5 slope. The sea side toe consists of 2t to 3t stones. The top level of the toe on the sea side is fixed at -1.0 m with 8 m wide toe-berm. A secondary layer consists of 300 to 500 kg stones are provided below the armour, toe berm and crest whereas it consists of 100 to 300 Kg stones provided below leeside armour. Core consists of 10 to 100 kg stones and a bedding layer of stones up to 10 kg weight is proposed. The top of the crest slab is at +6.0 m level with a crest width of 6 m. The top of parapet wall is fixed at + 7.0 m.



Fig-8: Wave flume facilities available at CWPRS

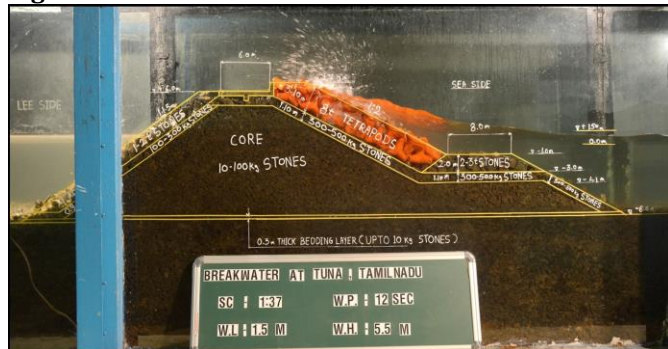


Fig - 9: Wave actions on armour layer of the breakwater at different wave heights during low water

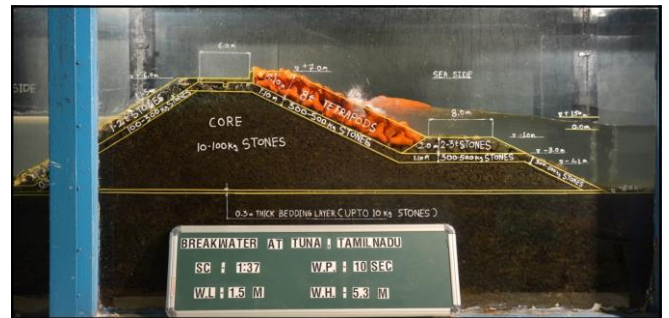


Fig - 10: Wave actions on armour layer of the breakwater at different wave heights during high water

6. TEST RESULTS & DISCUSSIONS

Through desk and wave flume studies the design of cross-sections of breakwaters at various bed levels was. It was presumed that, the seabed stratas underneath the construction of breakwaters are adequate to sustain the load of the breakwater structures. The weight of the armour units required for the breakwater for the different wave height have been worked out based on empirical methods and optimal design after confirmation of hydraulic stability test in wave flume as shown in graphical representation (Figure 11). It is observed that, the armour weight required for the breakwater at different wave height is more in empirical methods. The breakwater cross-sections consists of 1 to 5 t stones in the armour on lee side at various bed levels considering minimum wave height inside the harbour basin and also exposed to waves action during construction.

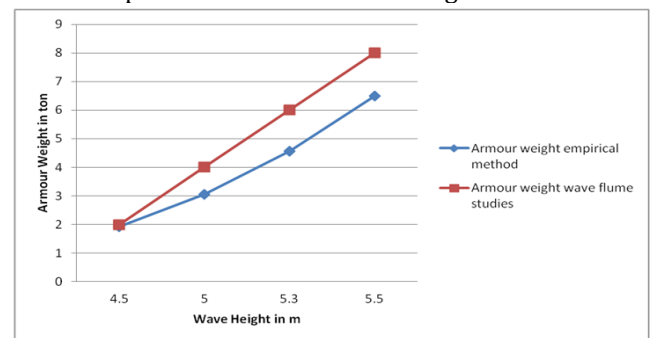


Fig - 11: Armour weight v/s wave height

7. CONCLUSIONS

1. Rubblemound type breakwater is efficient and most commonly adapted structure to protect port and harbor areas.
2. Wind generated waves produce most powerful forces to which coastal structure are subjected.
3. Wave attack and related structural responses is complex in nature and mere empirical formulae are not sufficient to finalize the section, which calls for hydraulic modeling for the design of structure.
4. A wide toe-berm reduces the cost of construction of breakwater by dissipating the energy of waves before striking the armour layer. Studies indicate that a wide toe-berm at the low water level of breakwater reduces the required weight of armour units by about 10-20%.

5. In order to optimize the design of breakwaters, the cross-sections should be evolved for seaside and leeside at various bed levels.
6. In the optimal design of breakwater, the total quantity of materials required for the breakwater can be saved more than 20%.
7. Basic studies are required for establishing relationship between the weight of armour units and the dimensions of the toe-berm in terms of width and depth of submergence.
8. The tetrapods of 2t, 4t, 6t, 8t and 10t weights in double layer in the armour of the breakwaters should be placed with correct slopes as specified, as per the packing density of 117 blocks/100 sqm, 74 blocks/100 sqm, 56 blocks/100 sqm, 47 blocks/100 sqm and 40 blocks/ 100 sqm respectively.
9. There should not be any deviation from the design during the construction of the breakwaters in respect of the levels, lopes and the weights of stones.
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