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केन्द्रीय जल और विद्युत अनुसंधान शाला
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Government of India
Ministry of Jal Shakti
Department of Water Resources,
River Development & Ganga Rejuvenation
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No : CWPRS/CHS-II/ 2D Vadhavan /2021/ Dated : 04/10/2021

Chief Manager (PPD)
Jawaharlal Nehru Port Trust,
Administrative Building, Sheva,
Taluka-Uran, Navi Mumbai– 400 007
Maharashtra

Sub : Technical Report No. **5952** of **September 2021** entitled "Desk and 2-D wave flume studies for the design of revised breakwater cross-section for the development of port at Vadhavan, Maharashtra" -reg.

Sir,

Please find enclosed herewith Technical Report No. **5952 of September 2021 on** "Desk and 2-D wave flume studies for the design of revised breakwater cross-section for the development of port at Vadhavan, Maharashtra"

The receipt of the Report may please be acknowledged.

Thanking you,

Yours faithfully,

4/10/2021

(A.V.Mahalingaiah)
Scientist- E

Encl.: Technical Report No. **5952 of September 2021** in duplicate

CC: Director, CWPRS, Pune for kind information.

CC: Shri A.V.Mahalingaiah, Scientist- E/ Shri Uday B. Patil, Scientist "B", CHS-II / Library-WAPIS / OC

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GOVERNMENT OF INDIA

MINISTRY OF JAL SHAKTI

Department of Water Resources, River Development, & Ganga Rejuvenation

CENTRAL WATER AND POWER RESEARCH STATION

KHADAKWASLA, PUNE – 411 024

COASTAL HYDRAULIC STRUCTURES

TECHNICAL REPORT No. 5952

SEPTEMBER 2021

DESK AND 2-D WAVE FLUME STUDIES FOR THE DESIGN OF REVISED BREAKWATER CROSS-SECTION FOR THE DEVELOPMENT OF PORT AT VADHAVAN, MAHARASHTRA

A.K. AGRAWAL

DIRECTOR

REPORT DOCUMENTATION SHEET

TECHNICAL REPORT NO. 5952	SEPTEMBER 2021

Title : **DESK AND 2-D WAVE FLUME STUDIES FOR THE DESIGN OF REVISED BREAKWATER CROSS-SECTION FOR THE DEVELOPMENT OF PORT AT VADHAVAN, MAHARASHTRA.**

Officers Responsible for Conducting the Studies:

The studies have been conducted and Technical Report prepared under overall guidance and the supervision of Shri A.V.Mahalingaiah, Scientist-E. The studies have been carried out and assisted for preparation of Technical Report by Shri Uday B. Patil, Scientist-B and Shri Ganesh N.S., Research Assistant. Shri CV Ramana Murthy, Scientist-C was present during the course of studies.

Shri Prabhakar Chary, Scientist-C, Mrs. R. S. Erande, Scientist-B, Mrs. M. S. Deshpande, JE and Shri. A. Yognath, JE from RSWG group are responsible for the operation and maintenance of the RSWG system at Wave Flume under the overall guidance of Shri S. D. Ranade, Scientist- E.

Name and Address of Organization Conducting the Studies:

Coastal Hydraulic Structure Division
Coastal and Offshore Engineering Laboratory
Central Water and Power Research Station, Pune 411 024, India

Name and Address of Authority Sponsoring the Studies:

Chief Manager (PPD)
Jawaharlal Nehru Port Trust,
Administrative Building, Sheva,
Taluka-Uran, Navi Mumbai
Maharashtra – 400 007.

Synopsis:

The Government of India (GOI) has a proposal to develop a major Greenfield port at Vadhavan with joint venture between Jawaharlal Nehru Port working under Ministry of Surface transport, GOI and Maharashtra Maritime Board (MMB), Government of Maharashtra (GOM). It has been proposed to develop the modern all weather new port to handle deep draft vessels at Vadhavan and to prepare Detailed Project Report (DPR). In this context, M/s JNPT requested CWPRS to conduct the hydraulic model studies for the development of new Port at Vadhavan. The layout plan evolved based on mathematical model studies carried out at CWPRS. This Technical Report consists of the details of the desk and 2-D wave flume studies carried out for revised cross-sections of the breakwater with Accropode-II in the amour layer. The cross section consists of 11 Cu.m . and 13 Cu.m. Accropode-II placed from -6.4 m bed level to -19.0 m bed level. The top of the crest slab is at el.+12.5 m level with a parapet top at el.+15.0 m. A clear carriage way of 7.5 m width is provided on the crest slab. The hydraulic stability tests were conducted in the wave flume by reproducing the sections to a Geometrically Similar (GS) model scale of 1:56 for trunk portion. The allowable wave overtopping discharge observed at 2-D wave flume. The Design significant wave height (Hs) of 6.8 m to 7.5 m was considered for evolving the design of breakwaters.

Keywords: Accropode-II, Armour unit, Breakwater, Hydraulic Stability, Tides, Toe Berm, Wave Flume, etc.

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GOVERNMENT OF INDIA
MINISTRY OF JAL SHAKTI
DEPARTMENT OF WATER RESOURCES,
RIVER DEVELOPMENT & GANGA REJUVENATION

CENTRAL WATER AND POWER RESEARCH STATION

KHADAKWASLA, PUNE - 411 024

TECHNICAL REPORT NO. 5952	**SEPTEMBER, 2021**

DESK AND 2-D WAVE FLUME STUDIES FOR THE DESIGN OF REVISED BREAKWATER CROSS-SECTION FOR THE DEVELOPMENT OF PORT AT VADHAVAN, MAHARASHTRA

1.0 INTRODUCTION

Vadhavan is located on the West coast of India and is at about 110 km north of Mumbai City in the state of Maharashtra shown in Figure1. The Government of India (GOI) has a proposal to develop a major Greenfield Port at Vadhavan with joint venture between Jawaharlal Nehru Port working under Ministry of Surface transport, GOI and Maharashtra Maritime Board (MMB), Government of Maharashtra (GOM). It has been proposed to develop the modern all weather new Port to handle deep draft vessels at Vadhavan and appointed M/s Progen-Pentacle Consultants to prepare Detailed Project Report (DPR). In this context, M/s JNPT requested CWPRS to conduct the hydraulic model studies for the development of new Port at Vadhavan. Accordingly, various hydraulic model studies carried out at CWPRS. The layout plan for the breakwater for the development of new Port at Vadhavan as shown in Figure 2, was decided based on mathematical model studies. Based on the desk and wave flume studies, the design cross-sections of breakwater with Accropode-II have been evolved and submitted Technical Report No. 5648 in November 2018.

Further, the optimization of layout and crest level reduction to reduce cost of the project were discussed in several meeting with Officers of JNPT, M/s Royal

Haskoning DHV Consultant and CWPRS. During discussion , it was decided to optimize the breakwater crest level considering Design Water Level (DWL) of 6.9 m with respect to CD, In this regard, JNPT referred to CWPRS for conducting the 2-D wave flume studies for confirmation of hydraulic stability and allowance of wave overtopping discharge to reduce the crest level of breakwater.

Accordingly, 2-D Wave flume studies have been carried out at CWPRS. This Technical Report described the details of the desk and 2-D Wave flume studies carried out for confirmation of hydraulic stability and allowance of wave overtopping discharge to reduce the crest level of breakwater for proposed development of Port at Vadhavan, Maharashtra.

2.0 SCOPE OF STUDIES

Desk and 2-D Wave flume studies carried out for confirmation of hydraulic stability and allowance of wave overtopping discharge to reduce the crest level of breakwater for proposed development of Port at Vadhavan, Maharashtra.

3.0 SITE CONDITIONS

The location of proposed port at Vadhavan is in open sea (Latitude 19^0 55’ 08” N and Longitude 72^0 39’ 06”E) in Dahanu taluka, Palghar District of Maharashtra state (Figure1). The present port is proposed to be developed on the seaward side of the headland at Vadhavan & stack yard will be formed by creating an artificial land in the foreshore area with reclamation in the intertidal zone at Vadhavan Port.

4.0 DESIGN CONDITIONS

4.1 Tidal Levels

The tides in Vadhavan are semidiurnal in nature and also a macro tide with tidal range of more than 4m during spring. The tidal levels (w.r.t. Chart Datum) near Vadhavan as per the Admiralty Chart No.210 are as follows :

- Mean High Water Spring (MHWS) : + 4.7 m
- Mean High Water Neap (MHWN) : + 3.7 m
- Mean Sea Level (MSL) : + 2.8 m
- Mean Low Water Neap (MLWN) : + 2.0 m
- Mean Low Water Spring (MLWS) : + 1.2 m

The storm surge of about 2 m is predicted based on the extreme value analysis studies carried out at CWPRS (CWPRS Technical Report no. 5581 of March 2018). The Sea Level Rise (SLR) of about 0.2 m for 50 years. The computation of Design Water Level is as below;

- MHWS = + 4.7m CD
- Surge = 2.0m
- 50 year sea level rise = 0.2m
- Design WL = + 6.9m CD

The tide and surge is completely uncorrelated in the sense that there is a 0.005% probability of the Highest/Lowest astronomical tide occurring within 5 to 10 hours storm of return period of 200 years. As such, the Design Water Level (DWL) of + 6.9 m CD including tidal level, storm surge and Sea Level Rise have been considered for long duration tests in the wave flume for the design of breakwater as suggested by M/s Royal Haskoning DHV Consultant.

4.2 Design Wave Conditions

Based on the extreme value analysis studies carried out at CWPRS for Vadhavan (CWPRS Technical Report no. 5581 of March 2018), the significant wave height (H_s) varies from 6.8 m to 7.5 m for 100 years return period at different bed levels along breakwater. The waves are predominant from north-west and south-west direction. The design wave conditions for no damage with the significant wave height (H_s) of 6.8 m to 7.5 m were considered for evolving the design of breakwaters. The hydraulic stability of breakwater was assessed for regular wave height of H_{10} and also for extreme climate condition tests for endurance and reported in CWPRS Technical Report no. 5648 November 2018. The different significant wave heights (H_s) at various bed levels have been mentioned in the layout plan for the breakwater for the development of Port at Vadhavan as shown in Figure 3.

5.0 DESIGN OF BREAKWATER CROSS-SECTIONS

The proposed layout consists of about 10 km long offshore breakwater proposed from -6.4 m to -19 m contour depth as shown in Figure 2. It is proposed to design rubble mound protection structure/breakwater with Accropode^{TM} II as primary armour. The desk studies have been conducted for evolving cross-sections of protection structure/breakwater with Accropode^{TM} II in the armour at different bed levels based on empirical formulae as per the existing conditions at the site. Initially, the stable unit weight of Accropode^{TM} II for breakwater sections at various bed levels at suitable design wave conditions have been worked out using the equation by Hudson formula. The stability coefficient ($K_\mathbf{D}$) depends on the slope of the seabed and $K_\mathbf{D}$ values to be used in the design are for non breaking waves and based on safe engineering practice, guidelines by Concrete Layer Innovations (CLI), the License holder for the design of Accropode^{TM} II armour layers who recommended maximum values of Hudson's stability coefficient $K_\mathbf{D}$ of 16 for trunk section and 12 for round head section. The trunk and roundhead sections were designed for $K_\mathbf{D}$ values, corresponding to non-breaking waves and a seabed slope of 1% for conservative sides. According to BS: 6349-Part-7, the relevant configurations of the toe-berm for the rubblemound breakwater have been designed. The under layer is extended to form the toe mound and the same size of rocks to be assumed for design. The toe designs have been checked for relevant low and high water levels and also in corresponding wave conditions. The breakwater sections include design of wave wall, which resist the impact pressures from wave up-rush. The wave wall design also includes vehicular and non-vehicular loads as per IRC Class 70 R loading.

The weight of armour units are basically evaluated based on Hudson's formula (as given below):

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

Where,

- W = Weight of Armour
- H_s = Significant wave height (m)
- K_D = Stability coefficient,
- S_r = Specific Gravity of Armour relative to Water at the structure (w_r/W_w)
- w_r = Unit Weight of Armour ,
- W_w = Unit Weight of sea water
- Cot θ= Slope of armour

A conceptual design of protection structure/breakwater has been evolved based on the desk studies. The design of breakwater cross-sections at different bed levels with Accropode™ II in the armour have been finalized through 2D and 3D wave flume studies and are suggested in **CWPRS Technical Report No. 5648 in November 2018.**

5.1 Damage criteria

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

5.2 Permissible wave overtopping

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

The wave overtopping will be estimated using the following method by Eurotop 2008, (Equation 5.9) as presented below.

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

Where,

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

The crest width in front of wave wall consists of 3 units of primary armour.

5.3 Road on breakwater crest

The road on the breakwater crest shall have a minimum width of 10 m wide. The crest element shall be founded on the breakwater quarry run. The roadway on top of the breakwater shall be designed as Reinforced Cement Concrete (RCC) element for loading condition as per IRC 70R.

5.4 Design of secondary layer

As per BS: 6349-part 7, the weight of the secondary layer rock for concrete armour shall vary between W/7 to W/15 of the weight of Accropode™ II. However, 0.5 to 1 t, and 2 to 4 t stones are proposed as secondary layer and apron extended from underneath toe-berm to seabed for preliminary design.

5.5 Layer thickness

Rock armour and secondary layers shall have a minimum thickness equivalent to two rock layers for stability and to act as filter. The layer thickness shall be determined by the stone size as follows:

Thickness = 2 * D_{n50} * k_t,

Where k_t applied is 1.02 for Accropode™ II layers, the rock size recommended by CLI will be used as under layer.

5.6 Design of Filter criteria for various layers

The British Standard BS 6349, Part 7, clause 4.4.3 provides guidance for sizing of under layers. The functionality of the filter is described as:

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

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

› $D_{15a} / D_{85u} < 4$ to 5
› $D_{15a} / D_{15u} < 20\text{-}25$

Where

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

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

› $D_{50a} / D_{50u} < 7$

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

The suitable filter for bedding layer may be provided for leveling and also avoid the scouring below the breakwater. If the soft/loose soil exist below the breakwater, the geo-grid may be provided to stabilize foundation.

5.7 Design of Toe-berm

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

In shallow water, the toe of rubble mound breakwater is exposed to breaking wave action, which leads to high water particle velocity and reversal in the flow gradient. This can cause erosion of seabed material, as a result of which there will be a significant settlement in toe. Such settlement can be prevented by providing suitable toe protection. An important function of the toe mound is to provide the support to the armour. The width of the toe-berm should be provided to accommodate at least four rocks, and accordingly toe-berm on seaside consist of one Accropode^{TM} II unit and 3 units of rocks, which satisfy the BS standard for minimum 4 units as toe. The toe stability shall be checked by physical model test for design low water level.

The stability of toe berm formed by two layers of rocks on variable berm width & slope structure is given by Van der Meer et al (1995). The equation to calculate the toe size is given below as:

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

Where,

- H_s = Significant wave height (m)
- Δ = ($\rho r / \rho w$) − 1 = Relative buoyant density
- D_{n50} = Nominal diameter (m)
- h_t = Water depth on Toe (m)
- h = Water depth near sea bed (m)
- N_{od}= number of unit displaced

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

$$N_{od} = \begin{cases} 0.5 & \text{No damage} \\ 2.0 & \text{Acceptable damage} \\ 4.0 & \text{Severe damage} \end{cases}$$

The unit weight of the stones in toe-berm calculated based on Van der Meer improved formulae (Eq. 5.187 and 5.188) of CIRIA rock manual. Although, 3 to 6 t and 4 to 6 t stones are provided as toe berm for different sections which were tested successfully in physical the model test.

5.8 Design of Wave wall

The breakwater sections shall include a wave wall and the requirement of concrete and steel for the wave wall shall be as per BS:6349-Part 7, The wave wall on these sections shall resist impact pressures from wave up-rush. Wave wall design includes vehicular and non-vehicular loads as per IRC Class 70R loading. The structural design of crest slab and parapet wall is required to be carried out by **Project Authorities/Consultants.** The impact pressure on the crest element and the uplift forces is determined based on equation presented below.

Wave pressure,

$$P_w = K\, W_w\, L((H_S/H_c)-0.5)$$

Where,

- H_s = Significant wave height (m)
- K = Constant(-)'
- W_w = Seawater density(kg/cum)
- L = Wave length corresponds to significant wave period (m)
- H_c = Height of breakwater crest from design water level (m)

5.9 Design of Core

The core material shall consist of 10 to 100 kg well graded quarry run. Gradation of core will be estimated based on the filter criteria. The core rock with weight less than 10 kg shall be restricted to 1 and 5% of total volume respectively. The porosity of 37 % may be considered in the rock gradation.

5.10 Rear armour

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

A conceptual design of protection structure/breakwater has been evolved based on the desk studies. The design of breakwater cross-sections at different bed levels with Accropode^{TM} II in the armour was finalized through 2D wave flume studies. The design of cross-sections of breakwaters with Accropode^{TM} II in the armour considering the design wave condition at different bed levels are suggested in CWPRS Technical Report No. 5648 in November 2018.

6.0 POTENTIAL OPTIMIZATION OF BREAKWATER

The breakwater cross sections submitted by CWPRS vide TR No.5648 November 2018 was reviewed by M/s Royal Haskoning DHV as part of the update of the DPR and the following potential optimizations were identified:

- **Reduction in size of sea side primary armour** – A review of armour type and sizes has confirmed that the selected type and size of the units for the sea side slope of the breakwater are appropriate for the adopted design conditions. From this it was concluded that there was limited scope for reducing costs based on the optimization of sea side primary armour.

- **Reduction in crest level** –A reduction in the crest level of the breakwater would have a significant influence on the volume of quarry run and costs.

- **Removal wave wall and roadway -** The need for a roadway on the crest of the breakwater was discussed at the meeting on 23rd March 2021. RHDHV questioned whether a roadway was required as heavy maintenance equipment would not have access as the breakwater is not connected to the shore. JNPT advised that a roadway was required but the width could be reduced to 7.5 m to save costs.

6.1 Optimization of Crest Level

A reassessment of the overtopping rates indicates that the crest level of the breakwater could be reduced by 1 m to +15 m CD. As part of this optimization the roadway level has been lowered to +13 m CD and reduced in width to 7.5 m. The lowering of the crest requires a heavy wave wall and roadway due to the increase in the wave loading.

The reassessment found that the critical conditions for overtopping were under the 100 year storm conditions. The limit of 50 l/s/m under these conditions is to protect the crest of the lee side armour from damage under overtopping waves. Consideration was given to increasing this limit but there are concerns over the risk of damage to the rear of the crest under the large volumes of overtopping water associated with the long period waves. Further lowering of the crest level may, however, be possible subject to the findings from physical model tests. This is likely to involve heavier rock armour or an alternative type of armour to protect the upper part of the lee side slope.

In view of the above, the design cross-sections of breakwater with Accropode-II in the armour are worked out based on desk studies considering the design wave conditions and design water level of +6.9 m with respect to CD at different bed levels. The revised breakwater cross-sections from contour depths of -6.4 m to-19 m CD are as shown in Figure 4 to 7.

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

The section is designed for the roundhead portion of breakwater at -6.4 m bed level as shown in Figure4. This section consists of 13 cu.m Accropode-II in the armour with 1:1.33 slope on both the sides. A 5.25 m wide toe-berm consisting of

3 to 6 t stones is provided at -2.35 m with 1: 2 slope on both the sides. A secondary layer consists of 2 to 4 t stones provided on both the sides below the armour, crest slab and 0.3 to 1 t stones below the toe-berm. Core consists of 10-100 kg stones and a bedding layer of stones up to 10 kg weight is proposed. The crest slab is fixed at el.+12.5 m level with a parapet top at el.+15.0 m. A clear carriage way of 7.5 m width is provided on the crest slab.

6.3 Cross-section for the trunk portion of breakwater from - 8 m to -15 m bed level.

The section is designed at - 8.0 m level for the trunk portion from -8 m to -15 m bed level of breakwater as shown in Figure 5. This section consists of 11 cu.m Accropode-II in the armour with 1:1.33 slope on sea side and 2 to 4 t stones in the armour with 1:1.5 slope on lee side. A 6 m wide toe-berm consisting of 3 to 6 t stones provided at -3.0 m with 1: 2 slope on sea side. A secondary layer consists of 2 to 4 t stones provided below the armour units & crest slab. A layer of 0.3 to 1 t stones is provided below the toe-berm. Core consists of 10-100 kg stones and a bedding layer of stones up to 10 kg weight is proposed. The crest slab is fixed at el.+11.50 m level with a parapet top at el.+15.0 m. A clear carriage way of 7.5 m width is provided on the crest slab.

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

The section is designed at -19.0 m bed level for the trunk portion from -15 m to -19 m bed level of the breakwater as shown in Figure6. This section consists of 11 cu.m Accropode-II unit in the armour with 1:1.33 slope on sea side and 2 to 4 t stones in the armour with 1:1.5 slope on lee side. A 6.22 m wide toe-berm consisting of 4 to 6 t stones provided at -10 m with 1:2 slope on sea side. A secondary layer consists of 2 to 4 t stones provided below the armour units & crest slab. A layer of 0.3 to 1 t stones is provided below the toe-berm. Core consists of 10-100 kg stones and a bedding layer of stones up to 10 kg weight is proposed. The top of the crest slab is at el.+12.5 m level with a parapet top at el.+15.0 m. A clear carriage way of 7.5 m width is provided on the crest slab.

6.5 Cross-section of breakwater for the roundhead portion at -19 m bed level

The section is designed for the roundhead portion of the breakwater at -19.0 m bed level as shown in Figure7. This section consists of 13 cu.m Accropode-II in the armour with 1:1.33 slope on both the sides. A 6.22 m wide toe-berm consisting of 4 to 6 t stones is provided at -10 m with 1: 2 slope on both the sides. A secondary layer consists of 2 to 4 t stones provided on both the sides below the armour, crest slab and 0.3 to 1 t stones below the toe-berm. Core consists of 10-100 kg stones and a bedding layer of stones up to 10 kg weight is proposed. The top of the crest slab is at el +12.50 m level with a parapet top at el.+15.0 m. A clear carriage way of 7.5 m width is provided on the crest slab.

The technical specifications of Accropode^{TM} II armour units as shown in Figure 8.

7.0 HYDRAULIC MODEL STUDIES

To further develop and confirm the performance of the optimized sections, the Project Authorities and M/s Royal Haskoning DHV Consultant suggested physical modelling. In particular, 2D wave flume studies required to confirm the overtopping performance of the section and stability of the rear slope armour. The modelling would also establish if further optimization of the section is possible.

The overtopping calculations indicate that a relaxation in the 100 year overtopping limits would be required if the crest is to be lowered below +15 m CD. The limit of 50 l/s/m under these conditions is to protect the lee side armour from damage under overtopping waves. Higher overtopping rates would require heavier rock armour or an alternative type of armour (e.g. pattern placed concrete cubes) to protect the upper part of the lee side slope. This would need to be confirmed using physical modelling. An indication of the sensitivity of the overtopping rates to crest levels is provided in Table 1.

Table 1: Mean overtopping rates

Crest Level (m CD)	Mean Overtopping Rates (EurOtop 2018) (l/s/m)	Mean Overtopping Rates (EurOtop 2008) (l/s/m)
+15	51	40
+14.5	74	57
+14.0	107	80
+13.5	155	115

In view of the above, the wave flume studies conducted to confirm the wave overtopping discharges, leeward side and toe-berm stability of breakwater for different test conditions suggested by M/s Royal Haskoning DHV Consultant.

7.1 Model Scale for trunk portion of breakwater:

The model tests for the design of trunk portion of breakwater at -19 m bed level were conducted in 2-D random wave flume by reproducing the section to a Geometrically Similar scale of 1:56. The model was based on Froude's criterion of similitude. The various scales obtained are as follows:

Model Scale : 1:56

Length	- L	= 1:56
Area	- L^2	= 1:3,136
Volume	- L^3	= 1:1,75,616
Time	- $L^{1/2}$	= 1:7.48
Velocity	- $L^{1/2}$	= 1:7.48
Discharge (l/s/m)	- $L^{1.5}$	= 1: 419

7.2 Compensation for weight of stones

The density of stones in the prototype was considered as 2.6 t/cum and density of seawater is 1.025 t/cum. However, the density of stones in the model was 2.80 t/cum and fresh water with density 1.0 t/cum was used in the flume. As such, the weights of stones used in the model were compensated for these density differences by applying a weight factor, which was worked out as below:

$$\frac{W_1}{W_2} = \frac{2.6H^3 \Big/ \left(\frac{2.6}{1.025} - 1\right)^3}{2.8H^3 \Big/ \left(\frac{2.8}{1.0} - 1\right)^3}$$

Where,

$W_2 = 0.6678\,W_1$

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

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

The weight reduction factor of Accropode™ II is also can be calculated based on scale similitude by following Hudson formulae for buoyancy:

$$C = \frac{V_p}{V_m \times E^3} = \frac{H_{Sp}^{\,3}}{K_D\left[\frac{d_p}{d_{0p}} - 1\right]^3 \cot g\alpha \times E^3} \times \frac{K_D\left[\frac{d_m}{d_{0m}} - 1\right]^3 \cot g\alpha}{H_{Sm}^{\,3}} = \frac{\left[\frac{d_m}{d_{0m}} - 1\right]^3}{\left[\frac{d_p}{d_{0p}} - 1\right]^3}$$

Prototype volume = $V_p = V_m \times E^3 \times C$

"E" being the scale factor

The Accropode™ II model units in the range of 142 gram to 152 gram received at CWPRS Laboratory.

7.3 Random Sea Wave Flume:

The random sea wave flume (45 m x 1.0 m x 1.2 m) was utilised for the wave flume tests. The system is capable of generating maximum wave height of 0.4 m at a water depth of 0.6 m for wave periods ranging between 0.3 and 3 sec. The flume facility is equipped with a fully automated computerised random wave generating system comprising of hydro-servo system and wave board assembly. The command signals are sent by the computer to the servo-valve through which pressurised oil supplied by hydraulic power pack flows into the actuator. A *To* and

Fro motion of the piston drives the wave board, which in turn generates waves in the flume.

7.4 Wave Measurement and Calibration:

The wave heights were measured by capacitance type wave probes. A gauge was fixed in front of the model of breakwater section and was analysed by computer. The desired wave conditions in front of the model were obtained by matching of desired spectrum and achieved spectrum by iterative procedure. A typical wave spectrum generated in the model as shown in Figure 9 &10.

7.5 Wave flume test procedures

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

$$Q_r = \frac{A \bullet V}{L} = \frac{L^2 \bullet \sqrt{L}}{L} = L_r^{\ 1.5}$$

$$\frac{Q_p}{Q_m} = \left(\frac{L_P}{L_m}\right)^{1.5} = L_r^{1.5}$$

7.6 Test conditions of 2-D wave flume studies

The following are the test conditions considered in 2-D wave flume studies:

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

The details of the wave flume tests for various test conditions are described in the following paragraphs.

7.7 2-D Wave flume test for trunk portion of breakwater

The number of Accropode-II blocks required in the 2-D wave flume model was derived by packing density calculations. The armour geometry and packing density coefficient of 0.62 is considered for the calculations. Total 346 nos. of Accropodes-II blocks were placed in the armour of the breakwater. A wooden template consists of horizontal / vertical distance between the two consecutive gravity centres of the block has been made for guiding the placement of the

Accropode-II blocks (Dh=7.06 cm and Dv=3.53 cm). The bed slope of 1:100 in front of breakwater was considered during wave flume studies.

The crest level is fixed at +15.0 m CD for all breakwater sections. In ordered confirm the hydraulic stability and overtopping discharge at different test conditions the breakwater section designed at -19.0 m bed level for the trunk portion from -15 m to -19 m bed level as shown in Figure6 is reproduced in the wave flume with Geometrically Similar scale of 1:56. The 2-D wave flume tests have been carried out in the Random Sea Wave Generation (RSWG) for trunk portion of breakwater at -19 m bed levels as follows**:**

This section consists of 11 Cu.m Accropode-II in the armour with 1:1.33 slope on sea side and 2 to 4 t stones in the armour with 1:1.5 slope on lee side. A 6.22 m wide toe-berm consisting of 4 to 6 t stones provided at -10 m with 1:2 slope on sea side. A secondary layer consists of 2 to 4 t stones provided below the armour units & crest slab. A layer of 0.3 to 1 t stones is provided below the toe-berm. Core consists of 10-100 kg stones. The top of the crest slab is at el.+12.5 m level with a parapet top at el.+15.0 m. A clear carriage way of 7.5 m width is provided on the crest slab.

The wave flume test conducted for a period of 1336 seconds in the model equivalent to 1000 numbers of random waves in the proto by generating random wave spectrum (PM-Spectrum). The rubbles are provided backside of the breakwater section and also backside of wave paddle for dissipate /observe the wave energy to reduce the wave reflection. The wave flume studies observations for different test conditions are described in details as below;

7.7.1 Water Level of + 6.9 m , Significant wave height (H_S) of 7.0 m. (Ref Photo: 26 a)

Initially, the test was conducted at the Water Level of + 6.9 m under the attack of random waves with significant wave height (H_S) 7.0 m (PM-Spectrum). There was marginal overtopping of waves of about 92 litres for 1000 waves in the model.

However, there was no damage was seen to any component of the breakwater including Armour layer and Toe-berm on both the sides.

7.7.2 Water Level of + 7.4 m , Significant wave height (H_s) of 7.0 m. (Ref Photo: 26 b)

Another test at Water Level of + 7.4 m was conducted for random significant wave height ((H_s) 7.0 m (PM-Spectrum). There was marginal overtopping of waves of about 136 litres for 1000 waves in the model. However, there was no damage was seen to any component of the breakwater including Armour layer and Toe-berm on both the sides. Less than 1% damage observed on leeward side of breakwater consist of 2 to 4 t stones after the generation of 1000 random waves in the model.

7.7.3 Water Level of + 7.9 m , Significant wave height (H_s) of 7.0 m. (Ref Photo: 27 a)

Further at Water Level of + 7.9 m, another test was conducted with random waves with significant wave height (H_s) of 7.0 m (PM-Spectrum). During the test, overtopping of waves of about 192 litres for 1000 waves in the model was observed. However, No damage was seen to any part of the breakwater including Armour layer and Toe-berm on both the sides. About 1% damage observed on leeward side of breakwater consist of 2 to 4 t stones after the generation of 1000 random waves in the model.

7.7.4 Water Level of + 6.9 m , Significant wave height (H_s) of 7.5 m. (Ref Photo: 27 b)

The test was also conducted at Water Level of + 6.9 m, another test was conducted with random waves with significant wave height (H_s) of 7.5 m (PM-Spectrum). During the test, overtopping of waves of about 160 litres for 1000 waves in the model was observed. However, No damage was seen to any part of the breakwater including Armour layer and Toe-berm on both the sides. About 1% damage observed on leeward side of breakwater consist of 2 to 4 t stones after the generation of 1000 random waves in the model.

7.7.5 Water Level of + 7.4 m , Significant wave height (H_s) of 7.5 m. (Ref Photo: 28 a)

Another test at Water Level of + 7.4 m was conducted for random significant wave height ((H_s) 7.5 m (PM-Spectrum). There was overtopping of waves of about 232 litres for 1000 waves in the model. The number of Accropode-II / stones provided in the armour and in the toe-berm was counted initially, before starting the test. After conducting the tests, the number of Accropode-II/ stones displaced from its original position was recorded and percentage of damage to the armour of breakwater was determined. However, there was no damage was seen on seaside of the breakwater. About 3% damage observed on leeward side of breakwater consist of 2 to 4 t stones after the generation of 1000 random waves in the model.

7.7.6 Water Level of + 7.9 m , Significant wave height (H_s) of 7.5 m. (Ref Photo: 28 b)

Further at Water Level of + 7.9 m, another test was conducted with random waves with significant wave height (H_s) of 7.5 m (PM-Spectrum). During the test, overtopping of waves of about 328 litres for 1000 waves in the model was observed. The number of Accropode-II / stones provided in the armour and in the toe-berm was counted initially, before starting the test. After conducting the tests, the number of Accropode-II/ stones displaced from its original position was recorded and percentage of damage to the armour of breakwater was determined. However, there was no damage was seen on seaside of the breakwater. About 5 % of damage observed on leeward side of breakwater consists of 2 to 4 t stones after the generation of 1000 random waves in the model.

7.7.7 Water Level of + 6.9 m , +7.4 m and +7.9 m with Significant wave height (H_s) of 3.0 m.

The tests were also conducted at different Water Levels of + 6.9 m, +7.4 m and +7.9 m with random waves with significant wave height (H_s) of 3.0 m (PM-

Spectrum). During the test, no splashing and overtopping of waves in the model were observed. However, No damage were seen to any part of the breakwater including Armour layer and Toe-berm on both the sides. This test conditions may not give the stability of toe-berm. However, following test condition at Low water is required to assess the stability of toe-berm.

7.7.8 Water Level of + 0.0 m , Significant wave height (H_s) of 7.5 m. (Ref Photo: 30 & 31)

Another test at Low Water Level of + 0.0 m was conducted for random significant wave height ((H_s) 7.5 m (PM-Spectrum). The number of stones provided in the in the toe-berm was counted initially, before starting the test. After conducting the tests, the number of stones displaced from its original position was recorded and percentage of damage to the armour of breakwater was determined. There was no overtopping/splashing of waves of in the model. There was no damage was seen on any part of the breakwater including Armour layer and Toe-berm on both the sides.

7.7.9 Water Level of + 7.4 m , Significant wave height (H_s) of 8.5 m. (Ref Photo: 29)

The test was conducted for extreme conditions at Water Level of + 7.4 m with significant wave height of 8.5 m (Hs). There was heavy overtopping and no damage to 11 cum Accropode-II units have been observed during the test. However, more than 5 % damages was observed on leeward side of breakwater consist of 2 to 4 t stones.

Typical photographs of the 2D wave flume facilities and wave flume studies conducted for different test conditions are as shown in Photos 1 to 31.

8.0 DISCUSSIONS OF RESULTS

Now, the studies have been carried out for confirm the hydraulic stability and the overtopping discharge through 2-D wave flume considering the different test conditions for trunk portion at -19 m bed level as shown in Figure6. M/s Concrete Layer Innovations (CLI) has confirmed the placement of Accropode-II ™ model units during the wave flume studies. The trunk portion of breakwater section consists of 11 cu.m Accropode-II ™ armour units at -19 m bed level as suggested in Figure6 was reproduce in the wave flume with Geometrically Similar scale of 1:56. The bed slope of 1:100 in front of breakwater section was considered during wave flume studies.

The hydraulic stability of breakwater cross-section at -19 m CD bed level has been confirmed through 2-D wave flume and hydraulic stability of other cross-sections including trunk and roundhead of breakwater at different bed levels have already been confirmed earlier and suggested in CWPRS Technical Report No. 5648 in November 2018. There was no damage observed on trunk and roundhead breakwater cross-sections with Accropode-II ™ model units in the armour and 4 to 6 t stones in the toe-berm at various bed levels during the wave flume studies.

The detailed overtopping discharge calculation for different design conditions are as follows :

Deterministic Design or Design safety Assessment:

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

Where,

- q = Mean wave overtopping discharge m^3/s
- R_c = Free board, (SWL to Crest Level) in m
- H_{m0} = Spectral Wave height in m
- Υ_f = Roughness Coefficient =0.44)

- Y_β = Effect of Wave Angle = (1.0)
- g = Acc. due to gravity in m/s^2
- C_r = Over topping reduction due to additional crest width (0.883)

H_{mo} in m	Water Level In m	R_c in m	q In m3/s	$q_f = q * C_r$ In m3/s
7.0	6.9	8.1	27.39	24.19
7.0	7.4	7.6	39.79	35.13
7.0	7.9	7.1	57.80	51.03
7.5	6.9	8.1	45.46	40.14
7.5	7.4	7.6	64.42	56.88
7.5	7.9	7.1	91.28	80.60

Average Overtopping Equation:

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

Where,

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

H_{mo} in m	Water Level In m	R_c in m	q In m3/s	$q_{fr} = q * C_r$ In m3/s
7.0	6.9	8.1	33.83	29.48
7.0	7.4	7.6	50.44	44.54
7.0	7.9	7.1	75.59	66.75
7.5	6.9	8.1	57.79	51.03
7.5	7.4	7.6	84.29	74.42
7.5	7.9	7.1	122.02	107.74

The detail of test conditions with wave height, water level, volume of wave discharge collected in the model, computed volume of wave discharge in proto and damage conditions of breakwater sections are as tabulated in Table-II.

Table-II : Wave test results

Test conditions	Wave Height (Hs) in m	Water level in m	Volume of Discharge (Litre) in model	Volume of Discharge (L/s/m) in proto	Damage conditions
1	7.0	6.9	92	28.86	No damage was seen on any part of the breakwater (Photo-26- a)
2	7.0	7.4	136	42.66	No damage was seen on any part of the breakwater. Less than 1% damage observed on leeward side of breakwater consist of 2 to 4 t stones (Photo-26-b)
3	7.0	7.9	192	60.23	No damage was seen on any part of the breakwater. About 1% damage observed on leeward side of breakwater consist of 2 to 4 t stones (Photo-27-a)
4	7.5	6.9	160	50.19	No damage was seen on any part of the breakwater. About 1% damage observed on leeward side of breakwater consist of 2 to 4 t stones (Photo-27-b)
5	7.5	7.4	232	72.77	No damage was seen on seaside of the breakwater. About 3% damage observed on leeward side of breakwater consist of 2 to 4 t stones (Photo-28-a)
6	7.5	7.9	328	102.88	No damage was seen on seaside of the breakwater. About 5 % damage observed on leeward side of breakwater consist of 2 to 4 t stones. (Photo-28-b)
7	3.0	6.9	-		No damage was seen on any part of the breakwater
8	3.0	7.4	-		No damage was seen on any part of the breakwater
9	3.0	7.9	-		No damage was seen on any part of the breakwater
10	7.5	0.0	-		No damage was seen on any part of the breakwater including toe-berm on sea side of breakwater. (Photo-30 & 31)
11	8.5	7.4	626	196.36	More than 5 % damages were observed on leeward side of breakwater consist of 2 to 4 t stones. (Photo-29)

The damage was observed on the leeside armour consist of 2 to 4 t stones during the test conditions Nos. 2, 3, 4, 5, 6 & 11. This damage is mainly just below the crest slab and it may be repaired / maintained regularly. The infrastructure developments such as jetty/berths are not considered immediately on leeward side of breakwater. As such, minor damage on leeside stones may not be affected for jetty/berths. However, the maintenance on the leeside of the breakwater with 2 to 4 t stones is very much essential when damage occurs during extreme events. Further, reduction of crest elevation is not suggested.

9.0 CONCLUDING REMARKS AND RECOMMENDATIONS

1) The design for the trunk and roundhead portions of breakwater cross-sections with Accropode-II ™ units in the armour from -6.4 m to -19 m bed levels for the development of Green field Port at Vadhavan, Maharashtra have been carried out. Their hydraulic stability also confirmed through 2-D & 3-D wave flume/basin studies and recommended vide CWPRS Technical Report No. 5648 in November 2018.

2) In order to optimize the breakwater sections, the crest level of breakwater have been reduced by 1 m to +15 m CD and crest width reduced to 7.5 m. The hydraulic stability and overtopping discharge at different test conditions of revised breakwater cross-section with 11 cu.m Accropode-II ™ units in the armour at -19 m CD bed level have been confirmed through 2D wave flume studies. The revised breakwater cross-sections of with Accropode-II ™ units at various bed levels for the development of Green field Port at Vadhavan, Maharashtra are as shown in Figs.4 to 7. These revised breakwater sections are hydraulically stable under the design wave conditions.

3) The hydraulic stability of toe-berm of breakwater at various bed levels also confirmed through wave flume studies by generating design wave height at low water. Under the extreme climatic conditions, some damage was noticed on the lee side of the breakwater trunk portion.

4) There is a very rare possibility of overtopping of waves in extreme climate conditions. However, about 5% damage may be occurred on the lee side of the breakwater under the extreme climatic conditions of HHWL including storm surge and significant wave height of 7.5 m. As such, maintenance on the lee side of the breakwater with 2 to 4 t stones are very much essential when damage occurs during extreme events. Further, reduction of crest elevation is not suggested.

5) The structural design of crest slab and parapet wall are required to be carried out by **Project Authority/Consultants**. The structural stability of the crest and parapet wall may be required to check before implementing on the site.

6) The density of concrete and stones to be used for the construction of the breakwaters should be about 2.4 t/cum and 2.6 t/cum respectively.

7) There should not be any deviation from the design during the construction for the breakwaters in respect of the levels, slopes and the weights of stones. The crest slab, the parapet wall and the key provided on the crest of the breakwaters should be constructed monolithically.

8) The construction of the breakwater may not be possible during one season. As such, a temporary roundhead may be provided wherever the work is curtailed.

9) The rubble mound structures are flexible structures and it is essential to monitor and maintain them regularly. Therefore, periodic survey and maintenance of the breakwater as and when damage occurs may be undertaken.

10) The grading of the stones to be used in the construction for the breakwater should be as follows :

4 to 6 t stones	- 50% stones should be higher than 5.0 t
3 to 6 t stones	- 50% stones should be higher than 4.5 t
2 to 4 t stones	- 50% stones should be higher than 3.0 t
0.3 to 1 t stones	- 50% stones should be higher than 650 kg
10 to 100 kg stones	- 50% stones should be higher than 55 kg

11) It is suggested to carry out geotechnical investigations to check the probable sinkage/settlement at the site before construction of the breakwaters. The suitable filter for bedding layer may be provided for leveling of bed and also avoid the scouring below the breakwater. If the soft/loose soil exists below the breakwater, the geo-grid may be provided to stabilize the soil foundation.

10.0 REFERENCES

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8. CWPRS Technical Report no. 5581 of March 2018
9. CWPRS Technical Report no. 5648 of November 2018

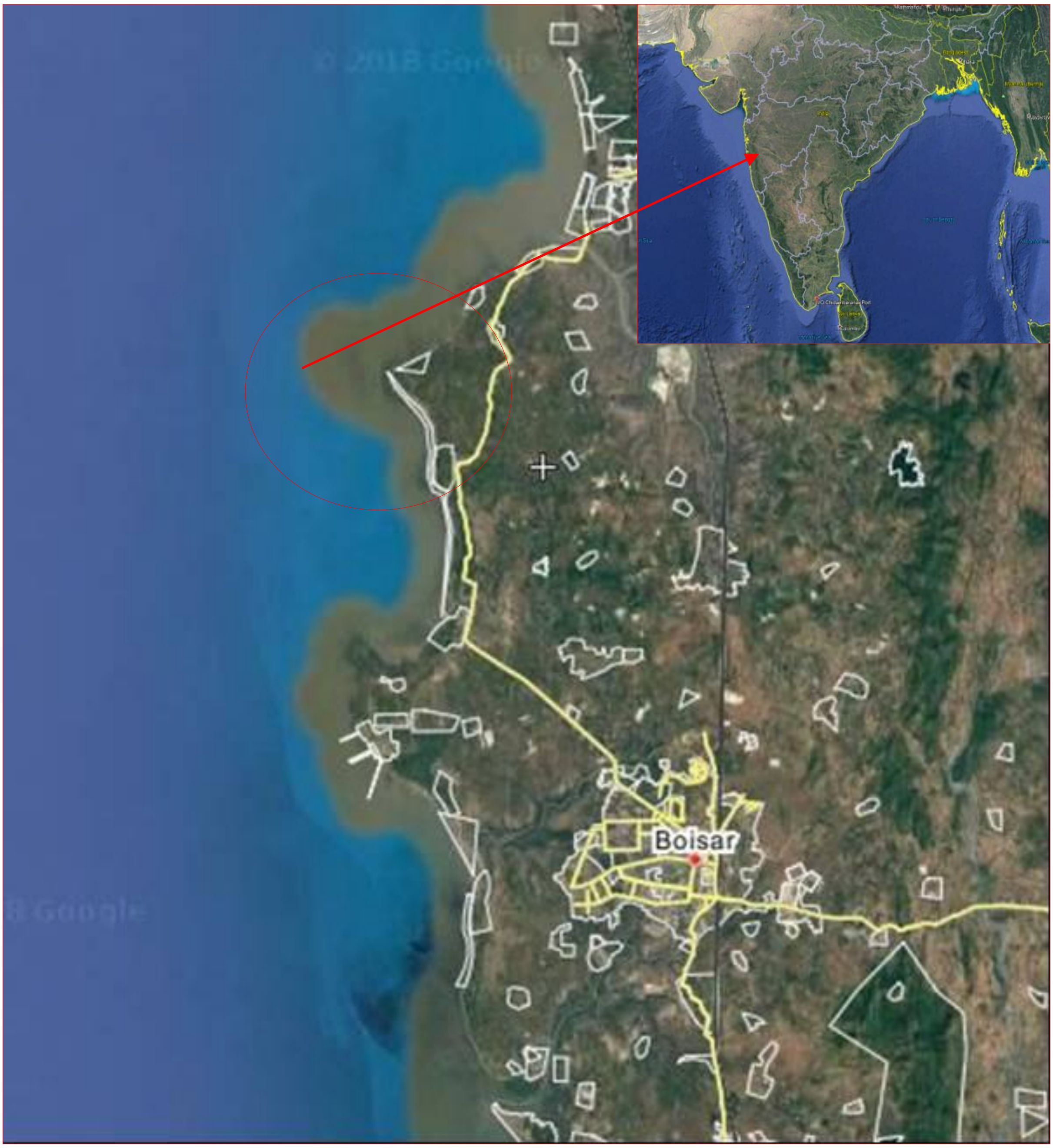

Figure 1: Location Map of proposed Port at Vadhavan, Maharashtra

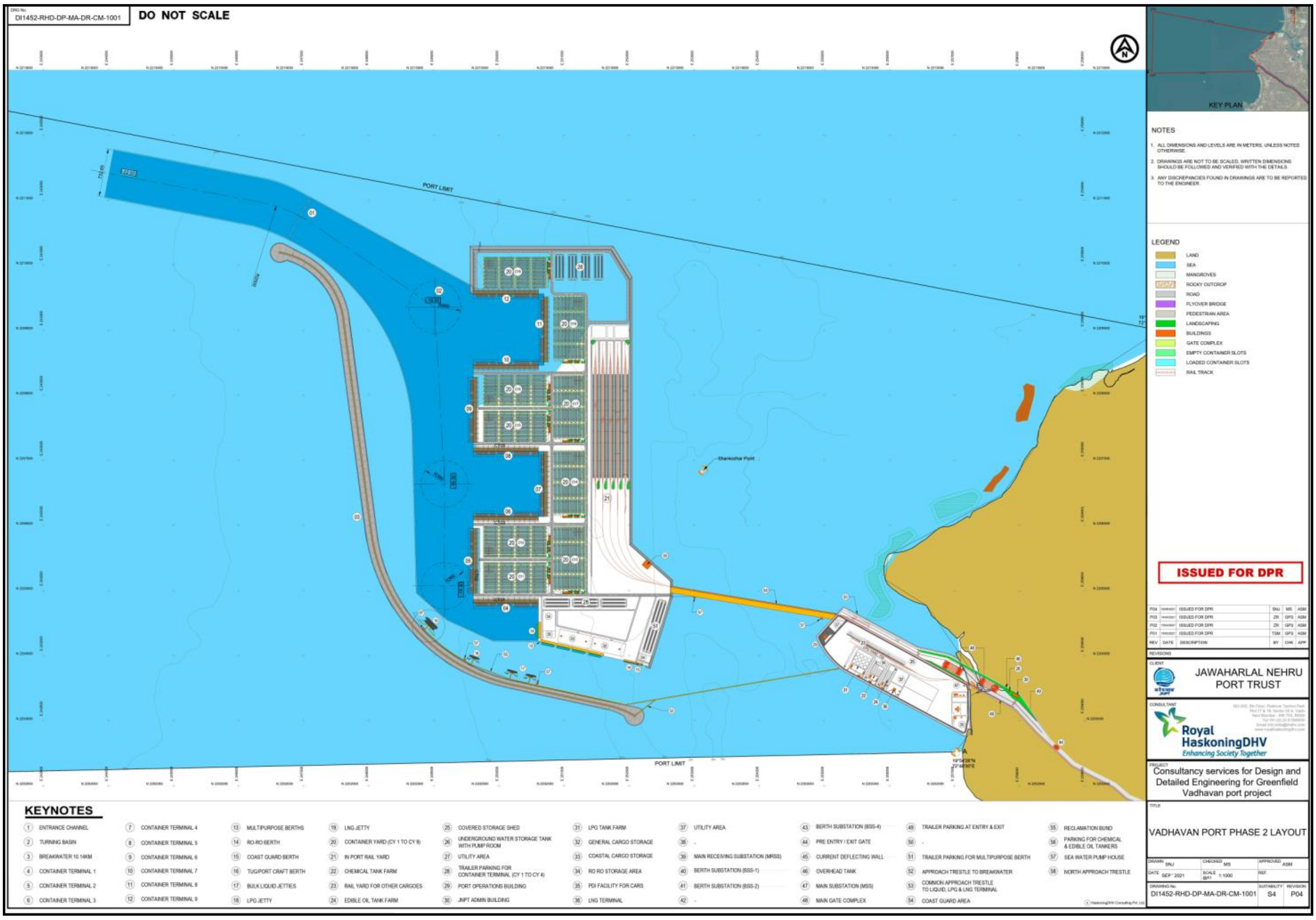

Figure 2: Layout plan of breakwater for the development of Port at Vadhavan, Maharashtra

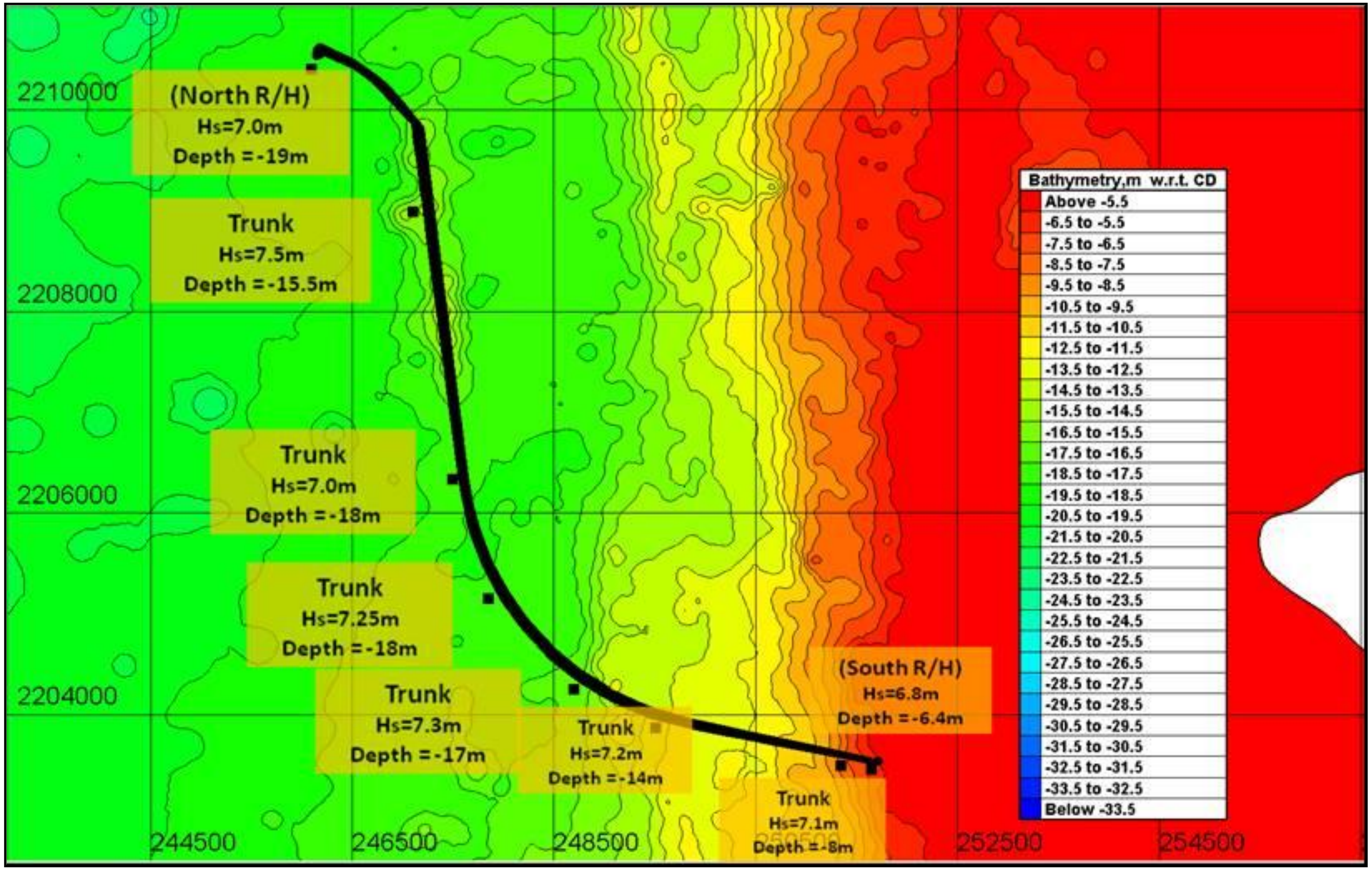

Figure3: Layout plan of breakwater with Significant wave height at different bed level for the development of Port at Vadhavan, Maharashtra

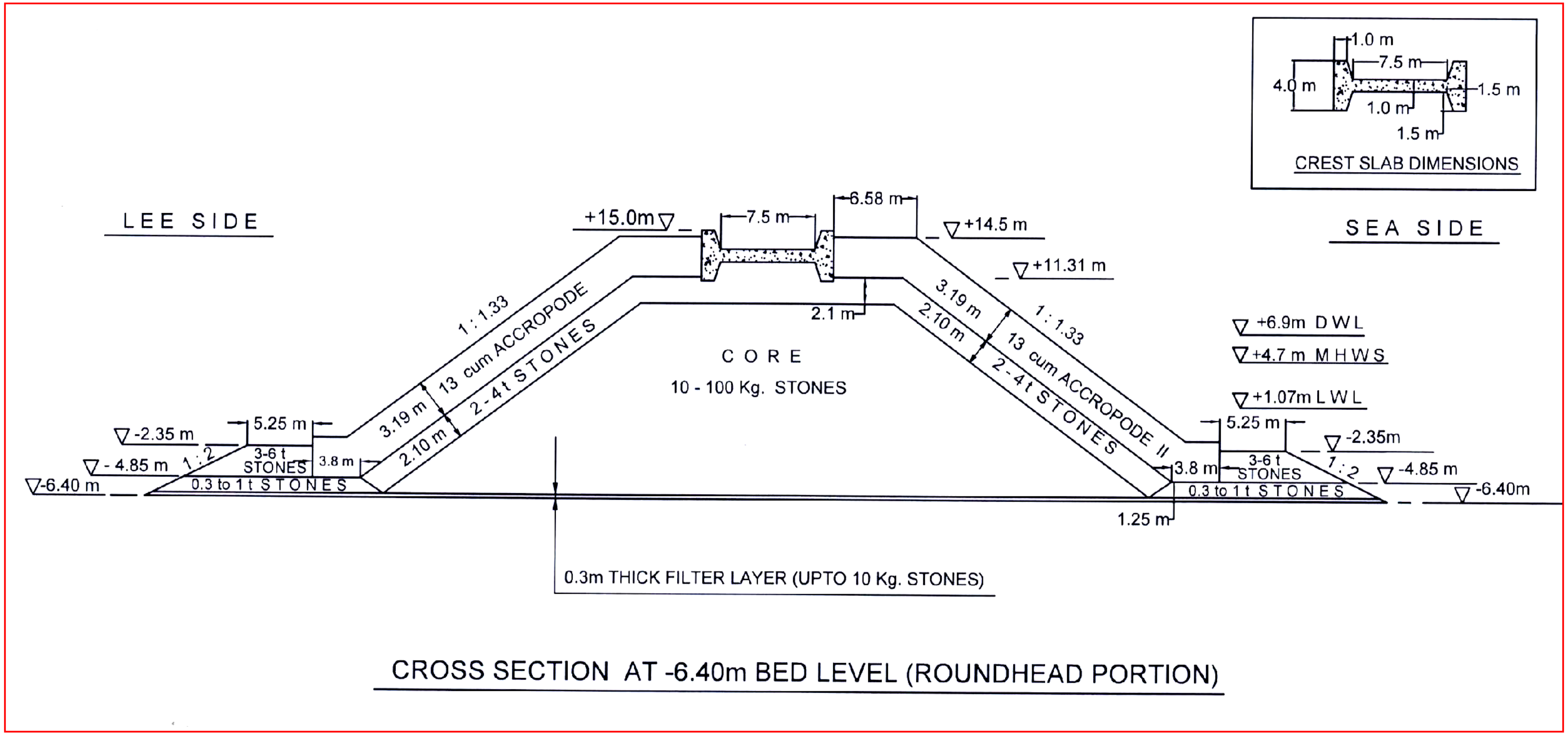

Figure 4 : Roundhead cross section (-6.4 m) of breakwater for the development of Port at Vadhavan, Maharashtra

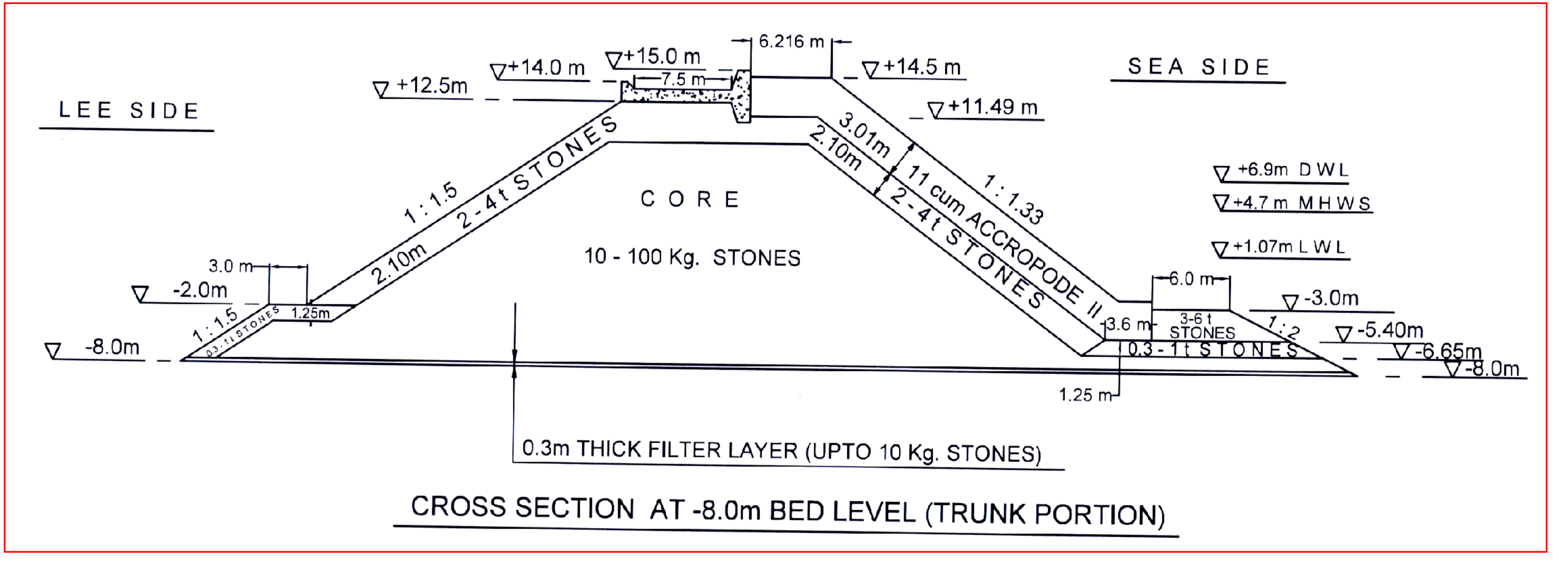

Fig 5 : Cross-section of breakwater (-8.0 m bed level) for the development of Port at Vadhavan, Maharashtra

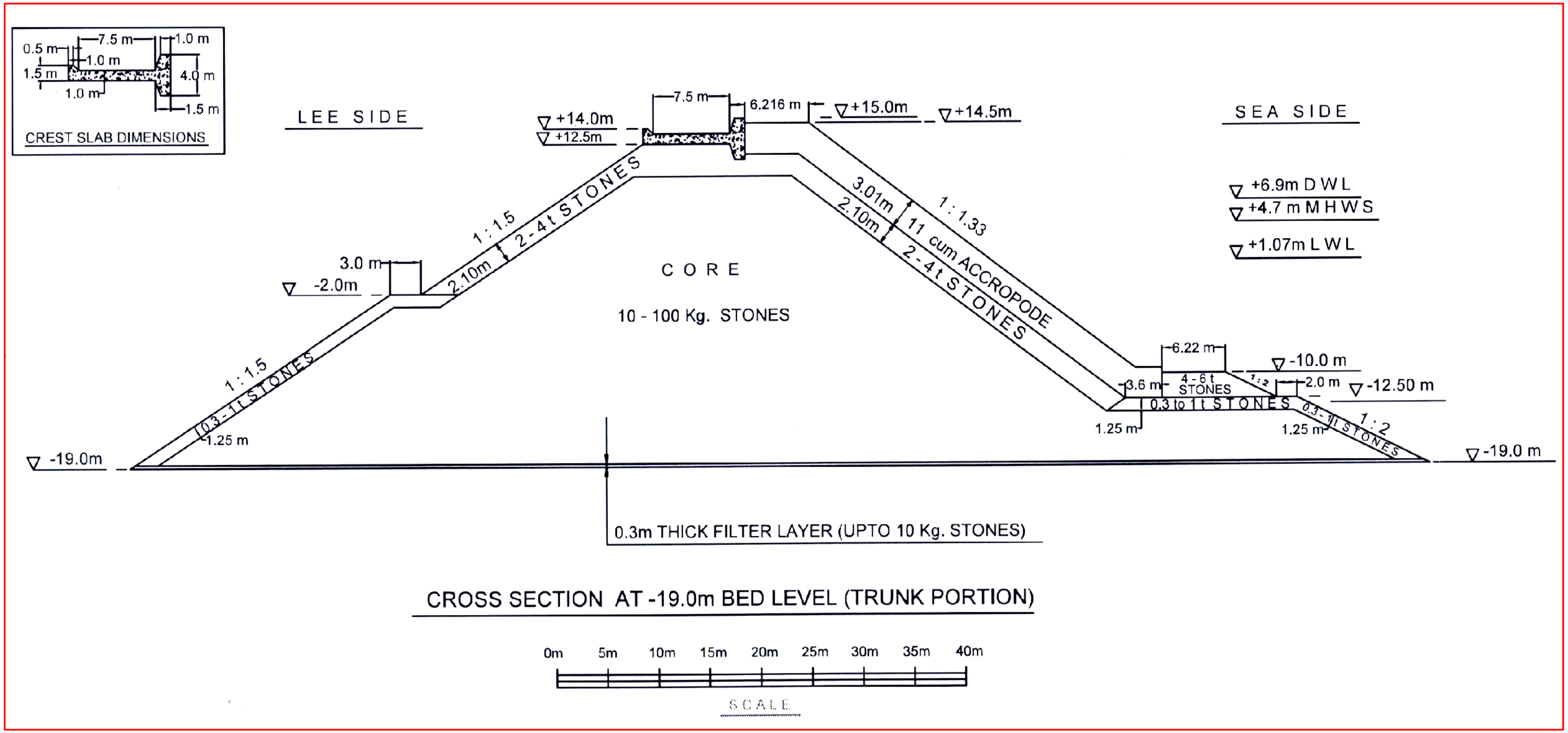

Fig 6 : Cross-section of breakwater (-19.0 m bed level) for the development of Port at Vadhavan, Maharashtra

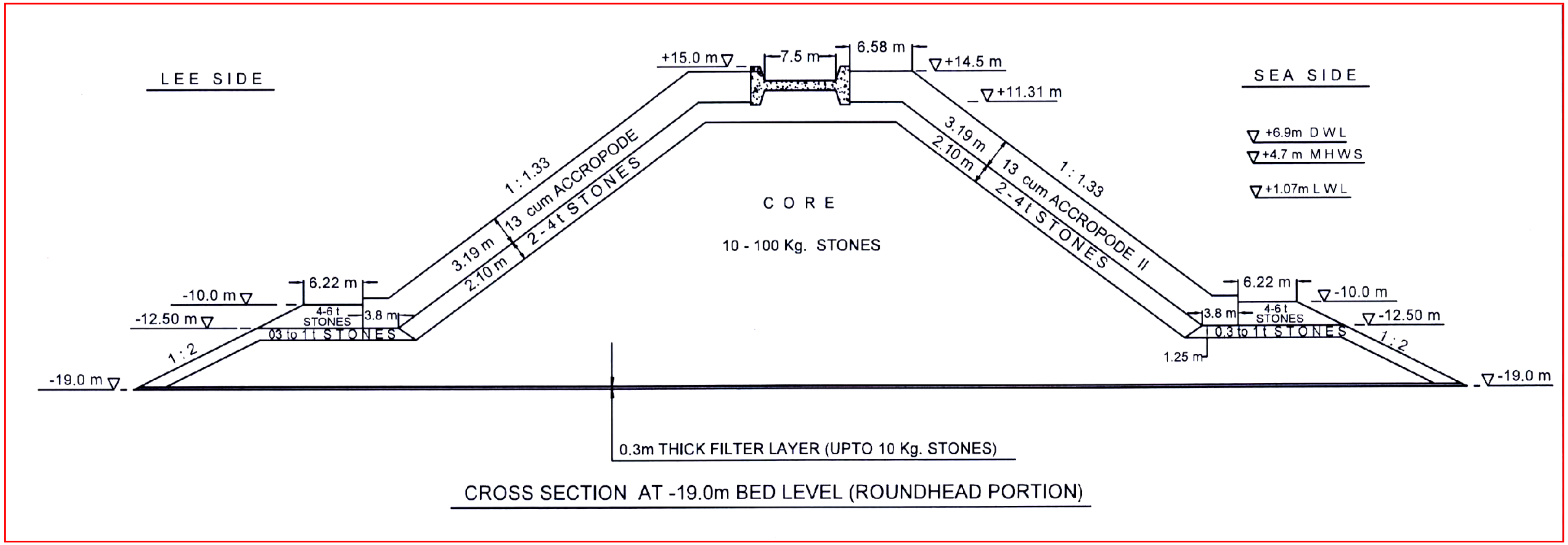

Figure 7 : Roundhead cross section (-19.0m) of breakwater for the development of Port at Vadhavan, Maharashtra

ACCROPODE™ II - ECOPODE™ Design Guide Table

The ECOPODE™ unit size is limited to 10m³ →

Unit Volume (m³)	**V = 0.2926H³**		**1.0**	**2.0**	**3.0**	**4.0**	**5.0**	**6.0**	**8.0**	**10.0**	**12.0**	**14.0**	**16.0**	**18.0**	**20.0**	**22.0**	**24.0**	**28.0**
Unit Height (m)	**$H = (V/0.2926)^{(1/3)}$**		1.51	1.90	2.17	2.39	2.58	2.74	3.01	3.25	3.45	3.63	3.80	3.95	4.09	4.22	4.34	4.57
Equivalent Cube Size (m)	**$Dn = V^{1/3}$**		1.00	1.26	1.44	1.59	1.71	1.82	2.00	2.15	2.29	2.41	2.52	2.62	2.71	2.80	2.88	3.04
Armour Thickness (m)	T = 1.36 Dn		1.36	1.71	1.96	2.16	2.33	2.47	2.72	2.93	3.11	3.28	3.43	3.56	3.69	3.81	3.92	4.13
Armour concrete consumption and coverage	Packing density Φ (-)		0.635	0.635	0.635	0.633	0.631	0.629	0.625	0.622	0.618	0.614	0.610	0.610	0.610	0.610	0.610	0.610
	Consumption (m³/m²)		0.635	0.800	0.916	1.005	1.079	1.143	1.251	1.339	1.414	1.479	1.537	1.599	1.656	1.709	1.760	1.852
	Number of units (u/m²)		0.635	0.400	0.305	0.251	0.216	0.191	0.156	0.134	0.118	0.106	0.096	0.089	0.083	0.078	0.073	0.066
	Porosity (%)		53.31	53.31	53.31	53.45	53.59	53.73	54.02	54.30	54.58	54.86	55.15	55.15	55.15	55.15	55.15	55.15
Filter stone underlayer - to meet the following requirement NUL/NLL < 3.0	**NLL (tons)**	**Standard**	0.17	0.34	0.50	0.67	0.84	1.01	1.34	1.68	2.02	2.35	2.69	3.02	3.36	3.70	4.03	4.70
		Min/Max*	0.1 0.2	0.2 0.4	0.4 0.7	0.5 0.9	0.6 1.1	0.7 1.3	0.9 1.7	1.2 2.2	1.4 2.6	1.6 3.1	1.9 3.5	2.1 3.9	2.4 4.4	2.6 4.8	2.8 5.2	3.3 6.1
	NUL (tons)	**Standard**	0.34	0.67	1.01	1.34	1.68	2.02	2.69	3.36	4.03	4.70	5.38	6.05	6.72	7.39	8.06	9.41
		Min/Max*	0.2 0.4	0.5 0.9	0.7 1.3	0.9 1.7	1.2 2.2	1.4 2.6	1.9 3.5	2.4 4.4	2.8 5.2	3.3 6.1	3.8 7.0	4.2 7.9	4.7 8.7	5.2 9.6	5.6 10.5	6.6 12.2
	Thickness (m) for standard NLL&NUL Specific density 2,6 t/m3	**Kt=1,15**	1.06	1.33	1.52	1.68	1.81	1.92	2.11	2.28	2.42	2.55	2.66	2.77	2.87	2.96	3.05	3.21
		Kt=0.9*	0.83	1.04	1.19	1.31	1.41	1.50	1.65	1.78	1.89	1.99	2.08	2.17	2.24	2.32	2.38	2.51

This table is to be used together with the note "Additional essential information regarding the tables" here appended.

: Geometrical characteristics of unit
: Recommended values for use at preliminary design stage
: (*)The information in this section is to be used with a compulsory analysis by a experienced coastal engineer even at preliminary stage - Ratio NUL/NLL should be kept between 2 and 3

This proprietary information of CLI is provided for preliminary guidance only. Hence, it is not a substitute for analysis by an experienced coastal Engineer. CLI provides assistance to the owners, developers, designers and contractors at all stages of projects. CLI reserves the right to make changes to the guidelines for improvement of its products. The validity of this document is therefore limited, but CLI will maintain accurate the version available online.

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Figure 8: Technical specification of Accropode™ II armour units

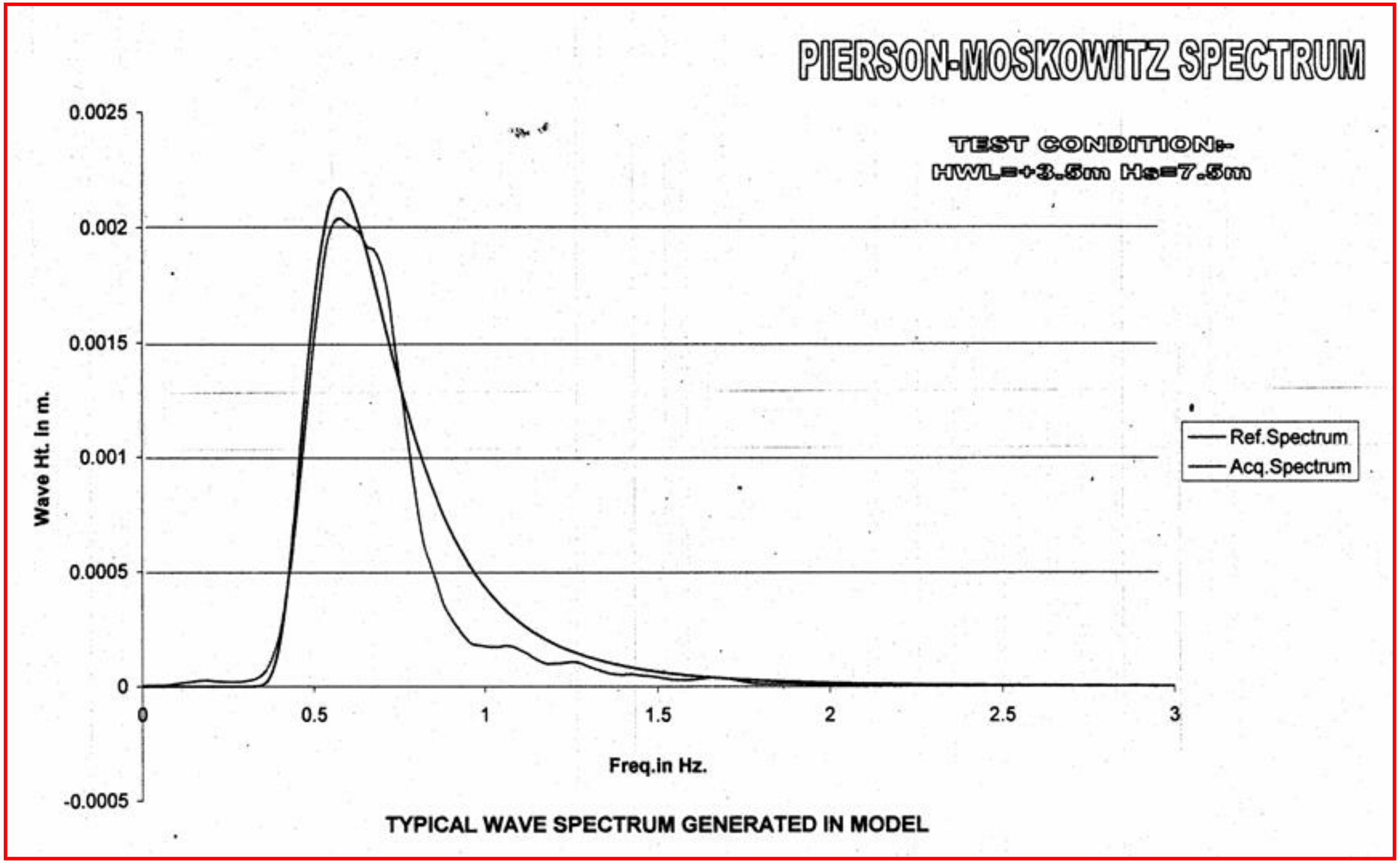
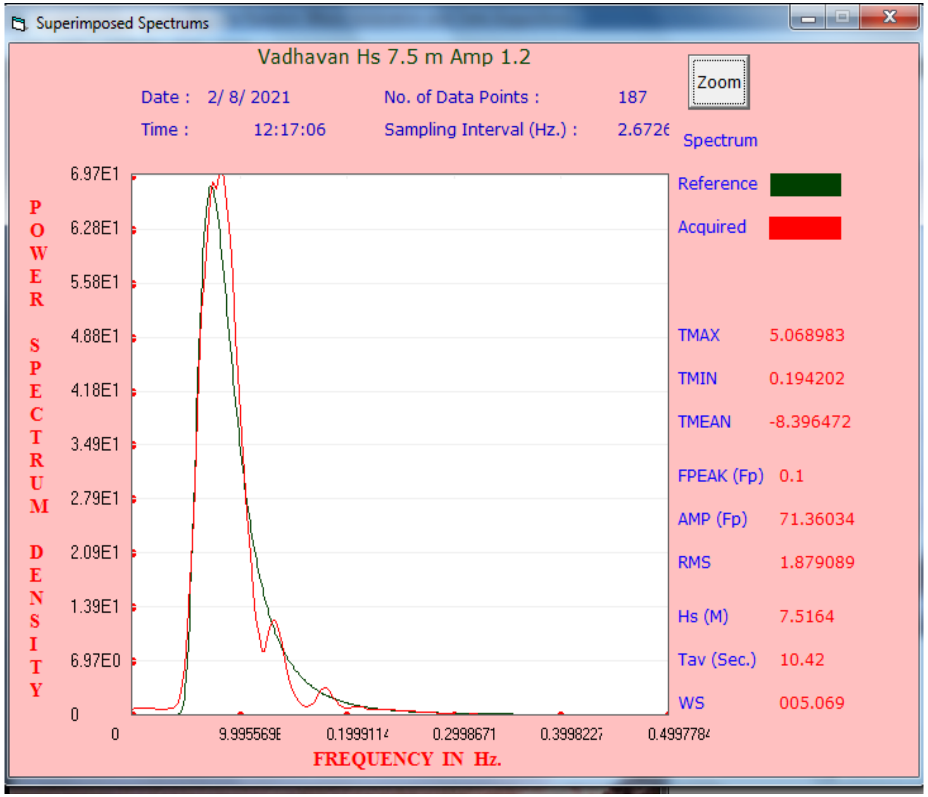


Figure 9 : Typical Wave Spectrum



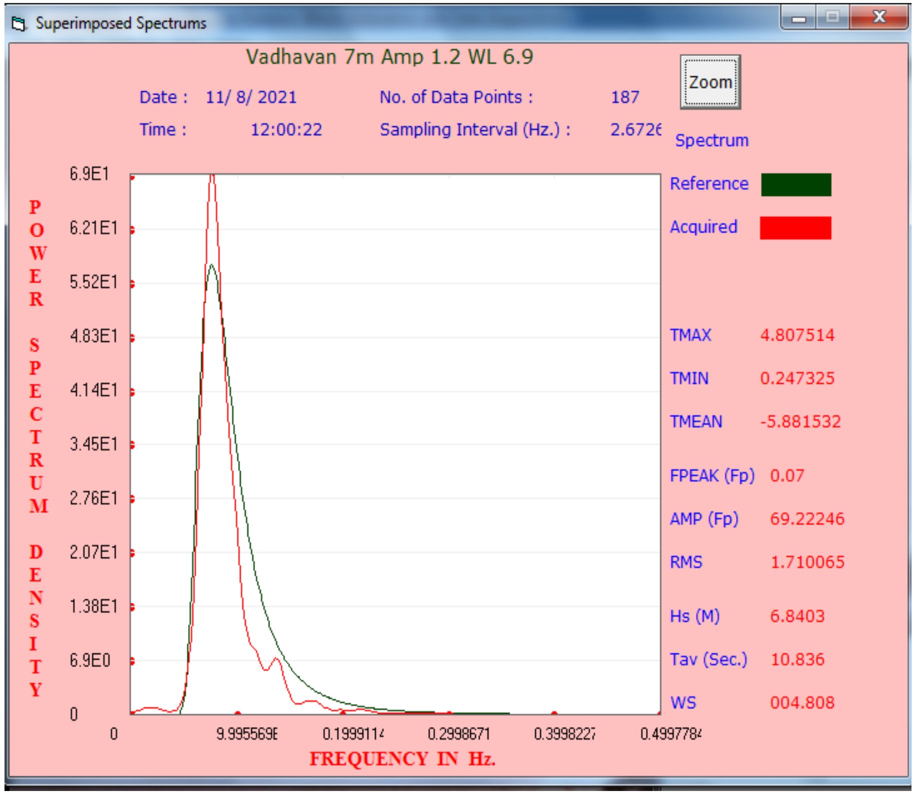


Figure 10 : Typical Wave Spectrums Generated in Model

Photo-1 : Wave flume Laboratory facilities at CWPRS

Photo-2 : RSWG with Servo Actuator

Photo-3 : Wave basin (45 m x 11 m x 1.1 m)

Photo-4 : Control room with wave data acquisition System

Photo-5 : Placement of Accropode™ II model units in the wave flume

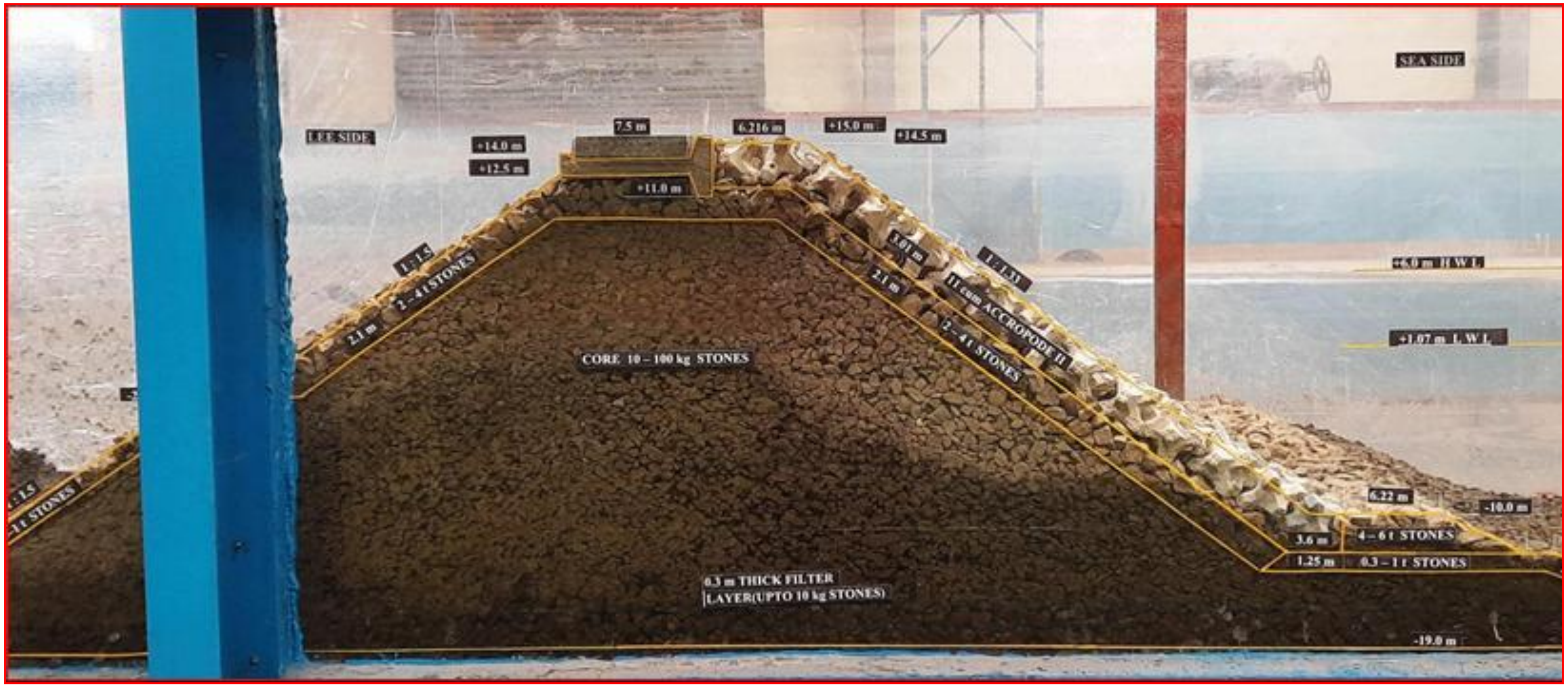


Photo-6 : Breakwater section with placement of Accropode^{TM} II model units in the wave flume at -19 m bed level

Photo-7 : Overtopping discharge tray provided in the wave flume for collection of volume of Overtopping discharge during the test

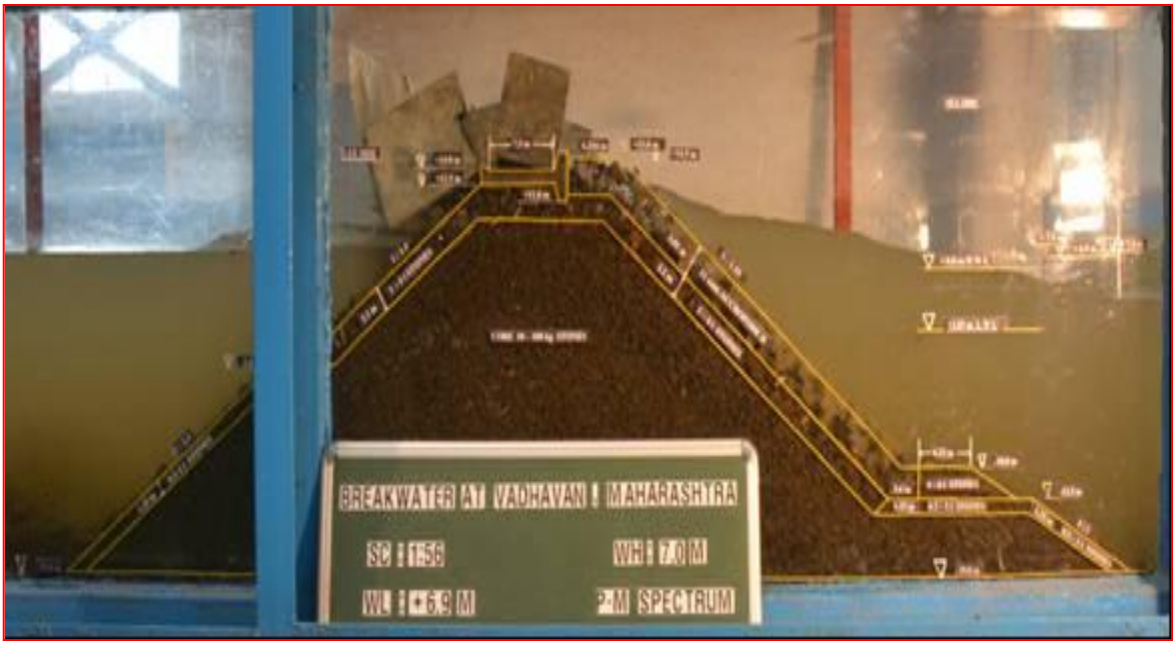


Photo-8 : Wave action on trunk section armour layer (11 Cu.m Accropode-II) at -19.0m bed level with wave height of 7.0 m at +6.9 m water level

Photo-9 : Wave action on trunk section armour layer (11 Cu.m Accropode-II) at -19.0m bed level with wave height of 7.0 m at +6.9 m water level

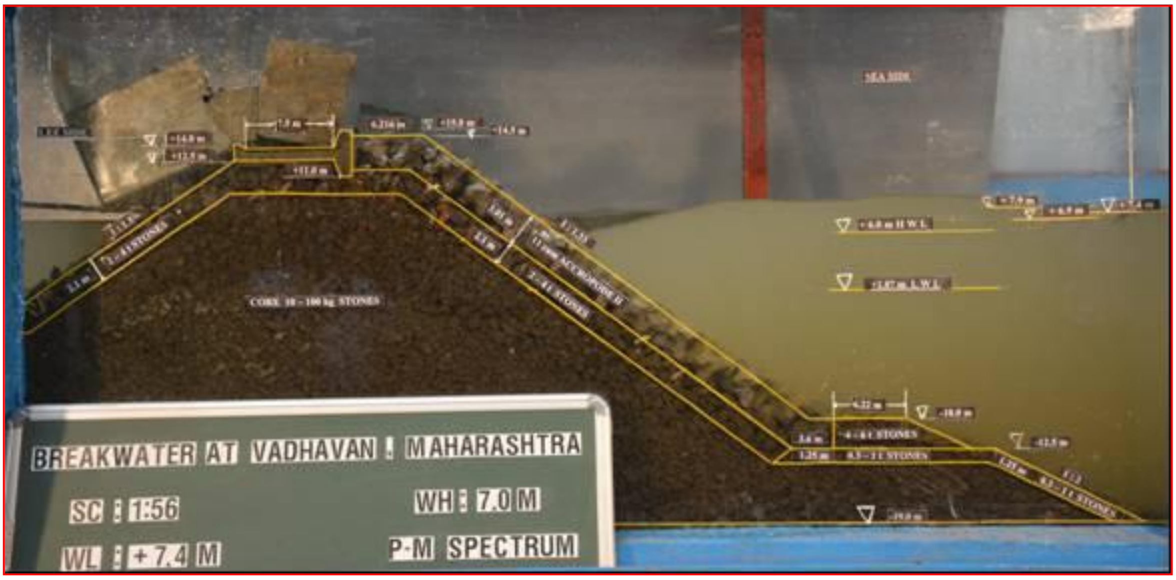


Photo-10: Wave action on trunk section armour layer (11 Cu.m Accropode-II) at -19.0m bed level with wave height of 7.0 m at +7.4 m water level

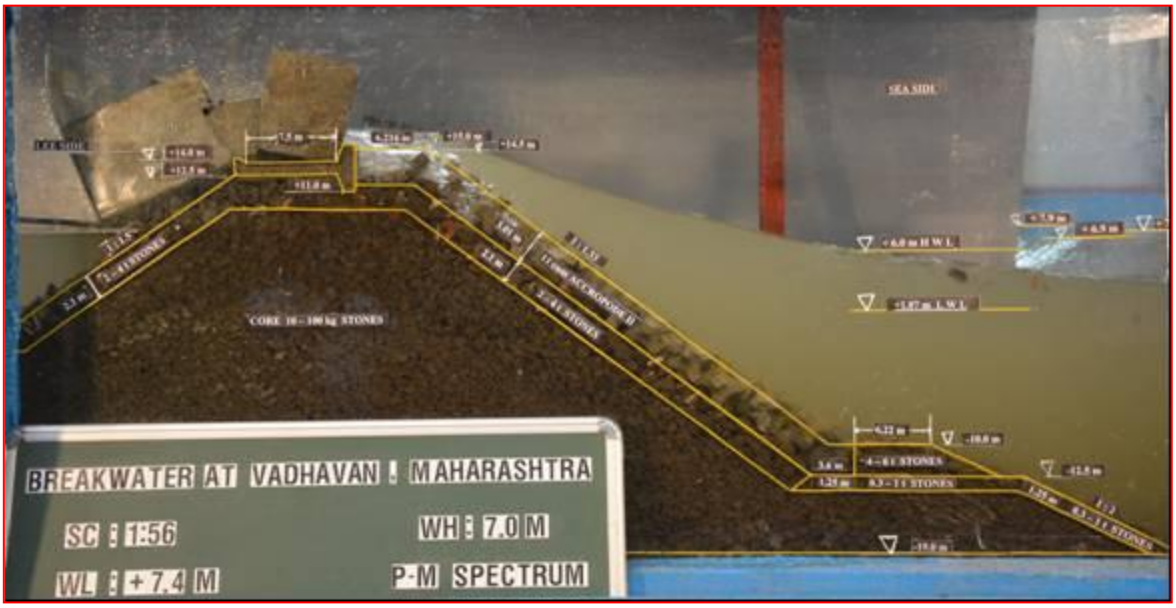


Photo-11: Wave action on trunk section armour layer (11 Cu.m Accropode-II) at -19.0m bed level with wave height of 7.0 m at +7.4 m water level

Photo-12 : Wave action on trunk section armour layer(11 Cu.m Accropode-II) at -19.0m bed level with wave height of 7.5 m at +6.9 m water level

Photo-13: Wave action on trunk section armour layer (11 Cu.m Accropode-II) at -19.0m bed level with wave height of 7.5 m at +6.9 m water level

Photo-14 : Wave action on trunk section armour layer(11 Cu.m Accropode-II) at -19.0m bed level with wave height of 7.5 m at +7.4 m water level

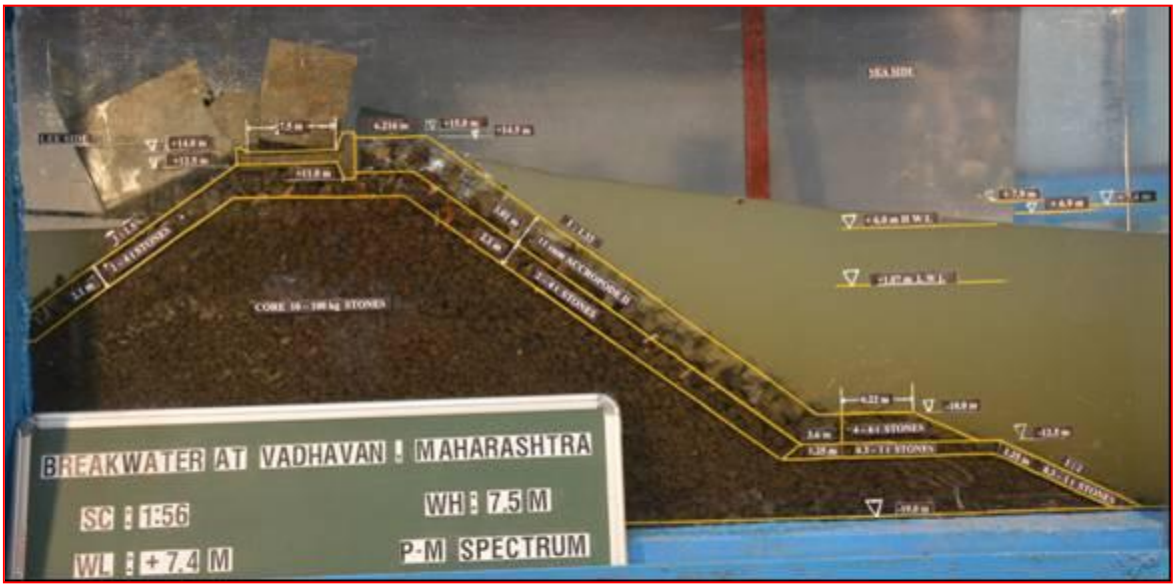


Photo-15: Wave action on trunk section armour layer (11 Cu.m Accropode-II) at -19.0m bed level with wave height of 7.5 m at +7.4 m water level

Photo-16: Wave action on trunk section armour layer (11 Cu.m Accropode-II) at -19.0m bed level with wave height of 7.5 m at +7.9 m water level

Photo-17: Wave action on trunk section armour layer (11 Cu.m Accropode-II) at -19.0m bed level with wave height of 7.5 m at +7.9 m water level

Photo-18: Wave action on trunk section armour layer (11 Cu.m Accropode-II) at -19.0m bed level with wave height of 3.0 m at +6.9 m water level

Photo-19: Wave action on trunk section armour layer (11 Cu.m Accropode-II) at -19.0m bed level with wave height of 3.0 m at +7.4 m water level

Photo-20: Wave action on trunk section armour layer (11 Cu.m Accropode-II) at -19.0m bed level with wave height of 3.0 m at +7.9 m water level

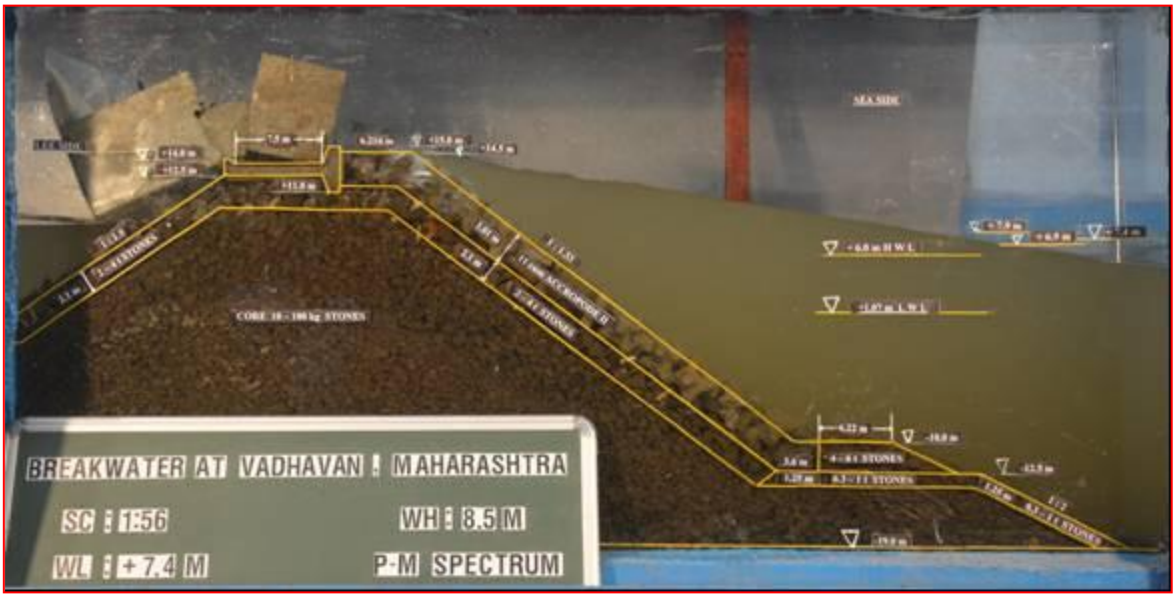


Photo-21: Wave action on trunk section armour layer (11 Cu.m Accropode-II) at -19.0m bed level with wave height of 8.5 m at +7.4 m water level

Photo-22: Wave action on trunk section armour layer (11 Cu.m Accropode-II) at -19.0 m bed level with wave height of 7.5 m at 0.0 m water level

Photo-23: Wave action on trunk section armour layer (11 Cu.m Accropode-II) at -19.0 m bed level with wave height of 7.5 m at 0.0 m water level

Photo-24: Placement of 2 to 4 t stones on leeside of breakwater section in the wave flume before wave generation

Photo-25: Overtopping of waves on leeside of breakwater section in the wave flume after wave generation

a) No damage, water level of + 6.9 m, wave height of 7.0 m (Hs),

b) less than 1% damage, water level of + 7.4 m, wave height of 7.0 m (Hs)

Photo-26: Leeside damage of breakwater section due to overtopping of waves during wave flume studies

a) About 1% damage, water level of + 7.9 m, wave height of 7.0 m (Hs), b) About 1% damage, water level of + 6.9 m, wave height of 7.5 m (Hs)

Photo-27: Leeside damage of breakwater section due to overtopping of waves during wave flume studies

a) 3% damage, water level of + 7.4 m, wave height of 7.5 m (Hs),   b) 5% damage, water level of + 7.9 m, wave height of 7.5 m (Hs)

Photo-28: Leeside damage of breakwater section due to overtopping of waves during wave flume studies

More than 5% damage, water level of + 7.9 m, wave height of 8.5 m (Hs),

Photo-29: Leeside damage of breakwater section due to overtopping of waves during wave flume studies

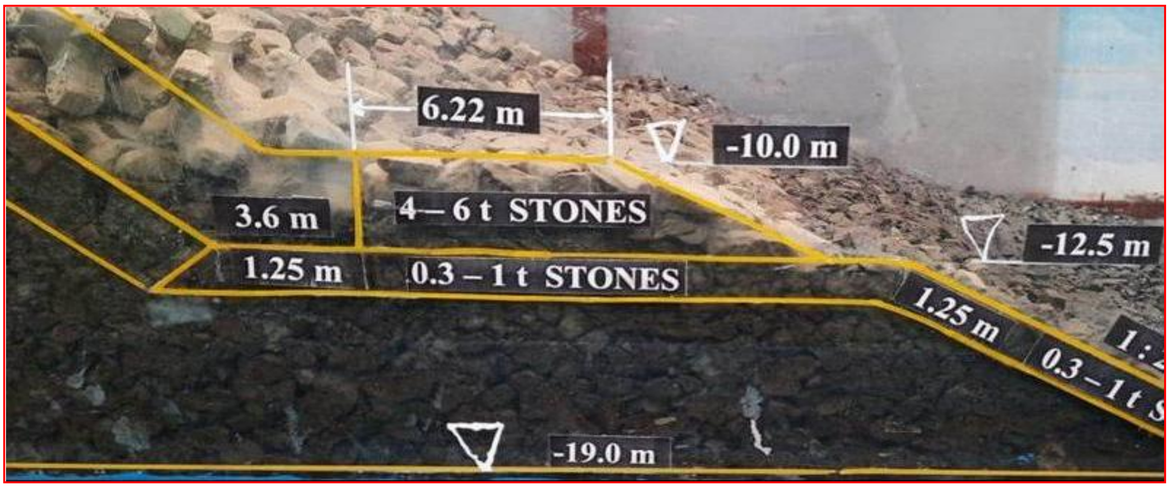


Photo-30: Toe-berm portion of breakwater section before wave generation

Photo-31: Toe-berm portion of breakwater section after wave generation (water level of + 0.0 m, wave height of 7.5 m (Hs))