NEW CONCEPTS OF PILE SUPPORTED BREAKWATER WITH BERTHING FACILITY

Project proposal submitted for research grant to

MINISTRY OF SHIPPING,

GOVT. OF INDIA

NEWDELHI

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PROJECT TITLE: NEW CONCEPTS OF PILE SUPPORTED BREAKWATER WITH BERTHING FACILITY

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National Technology Centre for Ports, Waterways and Coasts (NTCPWC)

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3.0. PROJECT SUMMARY

To cope with the drastic increase in the maritime trade, there has been a continuous progress in research and development for deep water berthing facility that demands with satisfactory tranquil conditions. This obviously cannot be achieved with the conventional rubble mound types of breakwater. Most of the major ports in India have an ambitious plan to expand and develop facilities to cater to the latest generation of vessels up to 18000 TEUS. In this regard, considerable work has been done on piled breakwaters and a few on skirt breakwaters. Breakwaters for relatively deeper waters should be stable in order to ensure safety and with minimum foot print in the Ocean floor, minimum or no wave overtopping but still keeping the crest elevation as low as possible. This may possibly be achieved by reshaping the shape of the sea front structure in such a way that it offers maximum resistance to the flow or enhances the dissipation of incident wave energy. With this as the background, four different types of which three are curved front face, while, the fourth being vertical pile supported breakwater models are taken up for a detailed investigation. These are termed as CP-PSB, FS-PSB, GW-PSB and VW-PSB and are projected in **Figs.1a** to **1d**. A detailed experimental investigation on the hydrodynamic pressures exerted on the different types of the front faces due to regular and random waves would be carried out. The motions of a vessel berthed on the leeside of the structure will also be measured.

The project includes experiments for the determination of the transmission, reflection, the run-up characteristics and the pressures exerted on the four different breakwater models in a wave flume (ref. **Fig. 2**) in Department of Ocean Engineering, Indian Institute of Technology Madras. The literature on the hydrodynamic performance characteristics of these types of pile supported

breakwaters to the best of investigators knowledge is nil. A perspective view of a harbour basin with the proposed breakwaters is shown in **Figs. 3a** to **3d**.

Key words (maximum 6) Breakwaters, berthing facility, dynamic pressures, reflection, transmission, wave run-up,

4.0 INTRODUCTION

Breakwaters are the most important harbour structures designed to withstand and dissipate the dynamic energy of waves. The estimation of wave forces on such structures form the major task in its design and stability. The main function of breakwater is to create tranquility condition on its leeward side for safe marine operations. Breakwaters further intercept the movement of longshore sediment and prevent them from deposition in the approach channel. As these structures need lot of initial investments, they should be properly designed, aligned and constructed.

Breakwaters can be divided into five types. They are Rubble mound breakwaters, Vertical breakwaters, Composite breakwaters, Composite breakwaters covered with wave dissipating blocks, and special type breakwaters. Special breakwaters include curtain wall breakwaters, floating breakwaters, etc., Special breakwaters are employed in relatively calm seas.

In the case of sloping or conventional rubble mound breakwater, the incident wave energy is dissipated by swashing over the slope surface and the wave height is reduced as the wave approaches from toe to top berm and there is considerable transmission and reflection from the breakwaters. In the case of vertical type, entire incident wave energy is dissipated by reflection. In the case of composite breakwater, the waves are reflected at high water and break against rock fill slope at low water.

Special breakwaters are breakwaters which have some kind of special features and are not commonly used. Special breakwaters can be divided into two kinds. One is the non gravity type breakwater such as pile type, floating, pneumatic etc. The other is the conventional breakwaters with special features conceived to improve the function and stability of the breakwaters. Due to the rapid increase in the size of vessels to handled requiring deeper drafts, alternate breakwaters with stability in deeper waters and also providing shelter to the maneuvering and berthing of deep draft vessels is the need of the hour. In order to fulfill this requirement a pile supported breakwater with its leeside serving as a berthing facility is considered for a critical study.

5.0 DEFINITION OF THE PROBLEM

The pile type breakwater is a typical non-gravity type breakwater. Pile type breakwaters include curtain wall breakwaters frame breakwaters etc. The pile type structures are normally built in relatively calm seas with soft soil foundation. The main advantage of this breakwater is that it allows the free passage of sediments.

The dissipation of the incident energy when waves act on coastal structures are mostly due to it run-up and reflection, in case of vertical wall, the amplitude of the waves near the wall magnifies to a maximum extent of twice the incident wave height. During the ingress of storms and extreme waves, the crest elevation has to necessarily be higher in order to avoid overtopping. In order to dissipate the incident wave energy gradually through shoaling, sloping walls were introduced. Although more stable, the sloping walls are reported to experience high pressures and run-up and thus requiring higher crest elevation and further it occupies more space compared to that of a vertical seawall.

Hence, it is evident that the protective measures should be safe with an optimum use of ocean floor with less or no wave overtopping even by maintaining a lower crest elevation. This objective may be achieved by considering a front shape of the structure. Curved front face structures was one of the options for the said purpose. The smooth curvature of the curved front wall will guide the wave over its curvature so as to allow it to return back into the sea by the process of which significant amount of energy get dissipated. The structures can be made of caissons so that, the weight & materials are reduced. As the caisson is a precast member, it can be erected easily by using a floating crane.

The model (model–CP-PBS) considered is a curved front wall from the concepts of Weber, (1934), which consists of a parabolic curve at the bottom with a quarter circle at the re-curved portion which was connected smoothly at the intersection to guide the waves smoothly towards the sea.

The (model- FS-PBS) is a modified version of the model proposed by Kamikubo *et al*., (2000). The cross section of the curved wall model proposed by Kamikubo *et al*., (2000) was formed with the deepest point from its base and from the vertical joining its toe and the crest of the wall are around 40% and 50% of its height respectively. The pressure experienced by such a wall is reported as some 3 to 4 times more than that on a vertical wall, and hence for the present study a modified section with the deepest point from the vertical joining its toe and the crest of the wall as 40% of its height was considered. The curved front wall was thus formed by adopting nine varying radii increasing from bottom towards top.

The seaside front face of curved wall (model–GW-PBS) is a combination of two radii of curvature as suggested in Coastal Engineering Manual., (2006) by US Army Corps of Engineers the curved section was adopted as seawall in Galveston, Texas, USA, during 1905.

The (model-VW-PBW) considered for the study for the purpose of comparison.

The type of breakwaters we considered is supported on piles. This is expected to give the combined benefits of both the curved sections as well as pile breakwaters. In addition the leeward side of the structure can be effectively used for berthing of vessels.

The advantages of pile supported curved wall breakwaters are:

(i) Free passage of sediments
(ii) dissipation of waves
(iii) Leeward side can be designed for berthing of vessels.
(iv) Good seaway enhancement is achieved compared to conventional breakwaters.

6.0. OBJECTIVES

The pile supported Curved front breakwater structures along with vertical front structure will be considered. The details of the model will be given later. The objectives of the study are

- To investigate the reflection and transmission characteristics of the breakwater models in regular and random wave fields.
- To study run up characteristics due to the action of regular and random wave fields.
- Qualitative assessment of wave overtopping.
- To investigate the effect of size of the piles and the spacing between them on the reflection and transmission characteristics.
- To investigate the motion behavior of a vessel berthed on the leeward side of the breakwater.
- To develop a finite element based linear diffraction model to estimate the reflection, transmission and pressure exerted on the four types of breakwaters.

7.0. REVIEW OF STATUS OF RESEARCH AND DEVELOPMENT IN THE SUBJECT.

7.1. General

The major ports on the east coast of India in Chennai, Visakhapatnam and Paradeep are formed by construction of rubble mound breakwaters, in which the one on its southern side is sufficiently longer in order to prevent the sediments entering into the approach channel. Due to this a beach on its southern side is formed and the north side of the harbor starves for sediments resulting in significant erosion. In addition, the quantity of sediments depositing in the approach channel keeps increasing thereby, leading to huge investments in maintenance dredging.

Instead of blocking the long shore movement of sediments at an average rate of above 1 million m^3 per annum towards north, if the sediments are allowed to have its own course the problem of coastal erosion can be reduced, if not can be completely arrested. This can be accomplished by constructing a pile breakwater. The curved front face models will be quite effective in dissipating the incident wave energy. This type of breakwaters has certainly a better appearance and result in good enhancement of the scenic beauty of the harbour area than the conventional rubble mound breakwater.

The breakwaters chosen are a combination of closely spaced piles with curved front breakwaters. A search of literature has shows that this type of pile supported breakwater was reported by (Sundar, and Subbarao, 2000) on the effect of spacing between pile are discussed. The international & national status of the research and development on the present topic of curved front structures combined with piled breakwaters has been given under two categories, viz, piled breakwater and curved front structures.

7.2. International status

7.2.1 Piled breakwater

Piled breakwater consists of closely-spaced rigid, vertical piles. Vertical and horizontal slotted screens have also been used to form a breakwater. Examples of pile breakwaters in use are

i) Hanstholm, Denmark

ii) Marsa el Brega, Libya

iii) Port of Osaka, Japan

iv) Pass Christian, Mississippi

The flow behaviour through a number of piles resting on the seabed is quite complex. Modelling such a wave structure interaction is difficult and hence researchers focus on experimental investigation to understand the flow behaviour through group of piles. For the last several decades, there has been considerable research focused on examining group of piles as a breakwater by evaluating the transmission and reflection of waves. Notable published data include studies by Weigel (1961), Hayashi and Kano (1996), Allsop and Kalmus (1985), Truitt and Herbich (1987), Herbich & Douglas (1989), Grune and Kohlhase (1974), Hayashi et al (1968) and Khader and Rai (1981).Weigel (1961) derived an expression for wave transmission based solely on geometry of the piles

$$H_t \backslash H_i = b/(D+b) \quad (1)$$

The measured transmitted wave height was found to be almost 25% greater than the transmitted wave height predicted by this simple theory because of diffraction effects. When an array of piles is used which has more than one row, the phenomenon becomes complicated. The second row of piles transmits a portion of the energy that gets though the first row of piles and reflects a portion of the energy. The reflected portion is partially transmitted back through the first row of piles and partially reflected by it. In addition a portion of the energy is scattered, the scattered wave having a frequency that depends upon the pile diameters and a portion of the energy is dissipated by skin drag and form drag. Hayashi et al (1996) derived an expression for wave transmission based on water jets discharging through the pile gaps.

The dynamic pressure distribution due to waves on steel pipe and its sheltering effect on breakwater when the pipes are driven at close interval of 5 cm to 10cm were measured by Nagai (1966). It was observed that the ratio of the resultant pressures between the top face and the side of the pipe is observed to be 0.99 as compared to 0.94 to 1.14 obtained for the case of pipes placed close together. For the condition of more overtopping of waves, the transmission coefficient were found to be 6% to 11% higher than the calculated results. For the condition of overtopping of waves, K_T value were found to increase by about 20%.

Weele and Herbich (1972) performed experiments on determining the transmission and reflection characteristics on a group of rows of piles. Three cases by varying the spacing between the piles in the rows in the group and staggering the piles in the rows for constant water depth and wave length were studied. It was observed that the K_T and K_R for a particular pile group decreases with a decrease in wave steepness. Further, the K_R was found to decrease with an increase in the longitudinal and transverse spacing between the piles. From these experiments, it was concluded that the longitudinal spacing between the piles is of equal importance when compared to the spacing in the transverse direction as regard to the reflection characteristics of pile groups.

Truitt and Herbich (1987) conducted experimental studies on wave transmission through a breakwater consisting of a single row of piles for both regular as well as random waves. It was found that the following dimensionless ratios affect the transmission coefficient.

(i) b/D, ratio of breakwater spacing to pile diameter

(ii) d/H_s, ratio of water depth to wave height.

The influence of the second ratio on the transmission coefficient was observed to be more pronounced.

Herbich & Dougles (1989) conducted additional experimental studies with two staggered rows of piles for both regular as well as various random wave spectra. . Definition sketch for pile geometry is shown in **Fig. 4**. From the experiments it was concluded that

I. Wave transmission increases with increase in d/H (water depth/wave height) ratio

II. Wave transmission increases with increase in wave period for regular waves.

III. Wave transmission decreases with increase in H/L (wave steepness)

IV. For a 10% gap ratio, the addition of second row of piles reduced the wave transmission by 5 to 10%.

A field study on the performance of pile breakwater in Bay Marina, Auckland, New Zealand has been discussed in detail by Hutchinson et al (1984)

7.2.2. Curved front structures

Weber, (1934) has given a conceptual design of curved seawall with a combination of a parabolic and a circular arc that brings a smooth change in the direction of propagation from horizontal to vertical and vise versa to reduce the wave induced pressures.

Murakami *et al*., (1996) proposed a new type of circular arc non-over toping seawall and measured the pressures and forces due to regular waves. It was concluded that the critical crest elevation is much less compared to that for a vertical seawall. The maximum pressures occurred near the still water surface changing with relative water depth (water depth/ deep water wave height) and inducing large vertical force.

Kamikubo *et al*., (2000) investigated the characteristics of the curved seawall and the fluid flow near the seawall was reproduced through numerical simulation using finite volume method. The results obtained were almost similar to that of Murakami *et al*., (1996). Similar study was reported by Kamikubo *et al*., (2003) along with the investigation on the spray when the waves strike the wall. Murakami *et al*.,

(2008) reported the efficiency of a curved seawall under increased water level due to global warming.

The literature review reveals that, the concept of using curved front structures as seawalls with different curvatures instead of vertically faced seawall to reduce the overtopping, has been adopted at several locations, however, the literature on curved front structures as a composite breakwater was nil and hence the present study was taken up.

7.4. National status

7.4.1. Pile Breakwater

Mani and pranesh (1986) investigated the performance characteristics of piled breakwater-floating breakwater interaction. It was reported that increase in ds/d (ds: depth of submergence of the float, d: depth of water) from 0.0 to 0.15 resulted in th reduction in K_T from 0.6 to 0.4. However K_R was found to increase from 0.25 to 0.28. it was further observed that for ds/d = 0.6 the K_T was found to be 0.5 which was independent of wave steepness.

7.4.2. Curved front structures

Anand and Sundar (2009 and 2010) investigated the performance of curved front structures as a seawall and compared the dynamic pressures and run-up with that on a vertical wall for random and cnoidal waves, and concluded that the dynamic pressures are high on curved walls. But the curved wall is efficient against overtopping and energy dissipation.

8.0. SUMMARY

The review on the earlier works carried out reveals that considerable amount of research work on the reflection and transmission characteristics of piled breakwaters and pressures on curved front structures have been reported. The literature on the hydrodynamic characteristics on curved front structures is scanty. This has necessitated the present research work.

9. WORK PLAN

9.2 Methodology

9.2.1. *Experimental Investigation*

The experiments will be carried out in the wave flume of 72.5m long and 2.0m wide in IIT Madras. The cross sectional view of the wave flume is shown in **Fig. 2**. The hydrodynamic characteristics of the four different types of pile supported breakwaters will be studied in the wave flume under the regular and random wave conditions. An empirical relation for wave overtopping would be proposed at the end of the study for the proposed model by considering the wave and structure parameters.

The measurements will be compared with the numerical model for the linear wave cases.

The following tables explain the different parameters to be considered for the proposed experimental study.

Table 1 Measuring parameters under different wave conditions

Tests	Types of structure	Measuring parameters
Regular and Random wave tests	1. CP-PSB 2. FS-PSB 3. GW-PSB 4. VW-PSB	♦ Reflection ♦ Transmission ♦ Dynamic pressures ♦ Total force ♦ Run up on the front face ♦ Wave overtopping

Wave Parameters

The proposed breakwater model systems will be tested in three different water depths, 0.5m, 0.75m and 1.0m. Regular waves of period ranging from 0.8 to 2.0sec at an interval of 0.2 sec will be generated. For each wave period, four different wave heights, 0.1m, 0.12, 0.15 and 0.18m will be adopted. The model will also be subjected to random waves of predefined significant wave height and period.

9.2.2. *Numerical Investigation*

Formulation of the problem

The state of the fluid can completely be described by the velocity potential, $\Phi(x,y,z,t)$ satisfying Laplace's equation if the fluid is assumed to be ideal, flow is irrotational and the application of linear wave theory is valid. The breakwater structure is assumed to be rigid.

$$\nabla^2 \Phi(x,z,t) = 0 \qquad (1)$$

A Cartesian coordinate system is employed, with the origin in the mean free surface, Oz directed positive upwards and Ox directed with positive in the direction of propagation of the waves. The boundary value problem thus presented here is the diffraction problem. The total velocity potential can be written in terms of incident potential, Φ_I, and scattered potential, Φ_s.

$$\Phi(x,z,t) = \Phi_I + \Phi_s \tag{2a}$$

$$= \mathrm{Re}\left[\left\{\phi_I(x,z) + \phi_s(x,z)\right\} e^{-i\omega t}\right] \tag{2b}$$

The incident velocity potential for the present wave structure interaction problem is represented by,

$$\Phi_I(x,y,z,t) = \mathrm{Re}\left[\frac{-iHg}{2\omega}\frac{\cosh k(z+h)}{\cosh kh}\exp\{i(kx-\omega t)\}\right] \tag{3}$$

where g is the gravitational constant; h is the water depth; and, H is the wave height. The wave number (k) satisfies the dispersion relation,

$$\omega^2 = gk\tanh kh \tag{4}$$

The boundary value problem for the scattered potential can be defined by the governing Laplace equation and the boundary conditions as defined below:

$$\nabla^2\phi_s = 0 \qquad \text{in the fluid domain } \Omega \tag{5}$$

$$\frac{\partial\phi_s}{\partial z} - \frac{\omega^2}{g}\phi_s = 0 \qquad \text{on the free surface, } \Gamma_F, z=0 \tag{6}$$

$$\frac{\partial\phi_s}{\partial z} = 0 \qquad \text{on the sea bed, } \Gamma_B, z=-h \tag{7}$$

$$\frac{\partial\phi_s}{\partial x} \mp ik\phi_s = 0 \qquad \text{at the radiation boundary, } \Gamma_\infty,\ x \to \pm\infty \tag{8}$$

The infinite boundary, Γ_∞ is fixed at a finite distance, $x = x_\Gamma$.

Kinematic body boundary condition

Since the breakwater is restrained from all its degrees of freedom,

$$\frac{\partial\phi}{\partial n} = 0 \qquad \text{on the surface of pipes, } \Gamma_o \tag{9}$$

From Eqn.(2), Eqn.(9) becomes

$$\frac{\partial\phi_s}{\partial n} = -\frac{\partial\phi_I}{\partial n} \qquad \text{on } \Gamma_o \tag{10}$$

where n is the unit inward normal to the body at the interface.

Hydrodynamic pressures and forces

The hydrodynamic pressure at any point in the fluid domain can be expressed as

$$p(x,y,z,t) = -\rho \frac{\partial \Phi}{\partial t} = i\omega\rho\Phi \qquad (11)$$

where ρ is the mass density of fluid. The hydrodynamic forces can be determined by integrating the pressure over the wetted body surface Γ_o.

$$F_j = \int_{\Gamma_o} p n_j \, d\Gamma \qquad (12)$$

The hydrodynamic forces, thus, evaluated can be called wave-exciting forces (${}^{e}F_j^{r}$).

$$F_j = {}^{e}F_j + {}^{h}F_j \qquad (13)$$

The wave exciting force, ${}^{e}F_j$ due to the diffracted potential can be expressed as

$$F_j\ (x,z,t) = i\omega\rho \int_{\Gamma_o} (\Phi_I + \Phi_s) n_j \, d\Gamma = \mathrm{Re}\left[\zeta_j^{r} e^{-I\omega T}\right] \qquad (14)$$

where, ζ_j is the complex force amplitude.

Numerical modeling

In order to solve the boundary value problem, solutions will be sought using a two dimensional finite element method. The infinite fluid domain will be made finite by incorporating either plane or higher order boundary dampers at the radiation boundary, which is at a finite distance, $x = \pm x_r$ from the breakwater. The optimum distance to radiation boundary from the body is large enough to neglect the local disturbance, and small enough to solve economically, and to neglect the significant accumulation of round-off errors.

The governing differential equation and the boundary conditions, including the far field boundary condition may be cast in the variational or weighted residual form to obtain the finite element equations. The velocity potential can conveniently be chosen as a field variable.

The diffraction problem (Eqns. 5 to 10) when recasted in an equivalent integral form by a variational formulation results in the functional, I(ϕ).

$$I(\phi) = \frac{1}{2} \int_{\Omega} \left\{ \left(\frac{\partial \phi}{\partial x}\right)^2 + \left(\frac{\partial \phi}{\partial z}\right)^2 \right\} d\Omega - \frac{\omega^2}{2g} \int_{\Gamma_F} \phi^2 \, d\Gamma \mp \frac{ik}{2} \int_{\Gamma_\infty} \phi^2 \, d\Gamma - Q' \qquad (15)$$

where ϕ denotes the diffraction potential. Q' term contributes from the external wave action through body boundary condition.

The variational solution will be obtained by seeking the function ϕ that makes the functional, $I(\phi)$ stationary.

$$\delta \{I(\phi)\} = 0 \qquad (16)$$

In the finite element approach, the above functional will be adopted in a piecewise manner, i.e. on a typical finite element domain Ω_e, a sub domain of Ω. The velocity potential, ϕ will be approximated by a linear combination of interpolating functions over the element and may be represented as

$$\phi = \sum_{a=1}^{n_e} \phi_a N_a(x,z) \qquad (17)$$

for each element, where ϕ_a denotes the nodal variables, n_e: the number of element degrees of freedom and $N_a(x,z)$ the shape functions, which are explicitly written as functions of two isoparametric co-ordinates related to (x,z) [Bathe (1982)].

The fluid domain will be divided into discrete elements with a total of 'M_n' nodes with 'q' elements. The assemblage of the element matrices has been done in the usual manner [Zienkiewicz & Taylor (1989)] and the resulting simultaneous equations are solved for ϕ_s. The wave exciting force vector is executed from ϕ_s,

$$\left\{\zeta_j^r\right\} = i\rho\omega\left[\left\{P_{ja}^r\right\}^T\left\{\phi_{sb} + \phi_{Ib}\right\}\right], \qquad a,b = 1,...,M_n \qquad (18)$$

The numerical formulation based on two dimensional finite element method, discussed for the wave structure interaction problem will be programmed using FORTRAN code. The fluid domain will be discretised employing the well-known three-noded triangular linear elment or eight noded isoparametric elements. The generalized pressure force vector on the body surface will be evaluated once the velocity potentials are obtained.

9.3. Organization of work elements

Time schedule of activities giving milestones (bar chart provided under section 16)

9.4 Suggested plan of action for utilization of research outcome expected from the project.

a. Research papers in leading International Journals will be published.

b. It is expected that the scientific work of this project can lead to a Ph. D and a M.S. degree which is enhancement of scientific capability of not only the Investigators but also new scientific manpower.

c. The results obtained in this study will be shared with major ports in our country.
The immediate benefit will be to aid the design of the new concept of the breakwater envisage in this proposal.

10.0. BUDGET ESTIMATES: SUMMARY

	Item	BUDGET			In rupees
		1st Year	2nd Year	3rd Year	Total
A.	Recurring				
	1. Salaries/wages	11,04,000	11,76,000	12,48,000	35,28,000
	2. Consumables fabrication of models	5,00,000	3,00,000	1,00,000	9,00,000
	3. Travel	75,000	75,000	75,000	2,25,000
	4. Other costs/ conti.	2,00,000	2,00,000	1,50,000	5,50,000
B.	Equipment	16,00,000	0	0	16,00,000
	Total (A+B) Total FEC*				68,03,000
	Institute overheads (20%)				13,60,600
	Grand Total				**81,63,600**

11.0. BUDGET FOR SALARIES/WAGES

Designation & number of persons	**Monthly Emoluments**	**1st Year**	**2nd Year**	**3rd Year**	**Total (Rupees)**
Senior Project Officer (SPO)	39,000	4,68,000	4,98,000	5,28,000	
Project Associate (PAS)	25,000	3,00,000	3,18,000	3,36,000	
Project Assistant (P.A)	16,000	1,92,000	2,04,000	2,16,000	
Project Attendant (PAT)	12,000	1,44,000	1,56,000	1,68,000	
Total		**11,04,000**	**11,76,000**	**12,48,000**	**35,28,000**

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Justification for the grant manpower requirement.

One Sr. Project Officer Gr. I and one Project Associate requested for, will be in-charge of the design of structure, selection of model scale, purchase of materials and fabrication of the model in the flume. The wave maker is controlled by computer and is highly sophisticated, the operation of which needs a sound knowledge on control system and an exposure to wave Hydro dynamics that is essential. Senior Project Officer would be a candidate with M. Tech with one to two years experience in the field of wave structure interaction. The Project associate would be a fresh Engineering graduate who would be assisting the Senior Project Officer.

One Project Technician is required to handle the instrumentation set-up which includes the usage of wave gauges, fabrication of run-up meter, pressure transducers and device for the measurement of over topping in the flume tests.

Only one project attendant is requested for and is needed for Xeroxing research material, errection of model, control of water inflow and outflow into the flume, preparing the reports and other miscellaneous works.

12.0. BUDGET FOR CONSUMABLE MATERIALS

(in Rupees)

		BUDGET			
Item		1st Year	2nd Year	3rd Year	Total
	Q	Lump sum			Lump sum
Materials for models	B**	2,50,000	1,50,000	0	4,00,000
Fabrication of models		2,00,000	1,00,000	0	3, 00,000
Stationeries / computer accessories		50,000	50,000	1,00,000	2,00,000
Total	B	5,00,000	3,00,000	1,00,000	9,00,000
	F				

*Q: Quantity or number, **B: Budget

Justification for costly consumable

An amount of Rs. 5,00,000/-, during the first year, 3,00,000/-, during the second year and 1,00,000/-, during the third year is the barest minimum which has been requested for the project for the purchase of materials needed for the break water model and towards the computational requirements for analyzing the data. The cost also includes Fabrication of special type of steel frames and metal holders in the wave flume for mounting the wave gauges, pressure transducers and velocity probs.

13.0. BUDGET FOR TRAVEL

(in Rupees)

		BUDGET			
		1st Year	2nd Year	3rd Year	Total
	Travel (only inland travel)	75,000	75,000	75,000	2,25,000

Justification for intensive travel.

An amount of Rs. 75,000/- each year is allocated. This amount is necessary for the project coordinators to attend meetings and also a few of the National Conferences and review meetings

14.0. BUDGET FOR OTHER COSTS/CONTINGENCIES

(in Rupees)

		BUDGET			
		1st Year	2nd Year	3rd Year	Total
	Other costs (Computer related costs such as backup drive)	1,00,000	1,00,000	50,000	2,50,000
	Contingency Costs	1,00,000	1,00,000	1,00,000	3,00,000
	Total	2,00,000	2,00,000	1,50,000	5,50,000

15.0. BUDGET FOR EQUIPMENT

Sr. No	Generic name of the Equipment along with make & model	Imported/indigenous	Estimated Costs (in Foreign Currency also)*	Spare time for other users (in %)
1.	0.2 bar miniature Pressure transducers	Imported	Rs.50,000/- x 8 nos. = Rs.4,00,000/-	20% during project and after project completion, these will be used by all
2.	Force transducer	Imported	Rs.12,00,000	

Justification for the proposed equipment

The hydrodynamic pressure and total force measurements are an important project objective. In our department, we have been regularly using pressure transducers which are larger in size. It would be difficult to fix in the curved face models similar to the proposed models. Hence, miniature pressure transducers of underwater type are proposed to buy under this project. It is required to measure the pressure exerted on the front face of the breakwaters at various vertical levels. Hence, 8 nos. of pressure transducers are being planned.

In addition, the total force exerted on the structure is an important design data which is inevitable for structural design. Hence, the total force is planned to measure using specially fabricated (custom defined specifications) for underwater purpose for this project.

16.0. Time Schedule of Activities through Bar Diagram

Year	1st Year				2nd Year				3rd Year			
Quarter	I	II	III	IV	I	II	III	IV	I	II	III	IV
Recruitment of project staff	■											
Fabrication of breakwater models	■	■										
Development of numerical model			■	■								
Wave flume experiments					■	■	■					
Analysis and interpretation of the measurements							■	■	■	■		
Validation of numerical model with flume experimental results								■	■			
Parametric study using numerical model										■	■	
Preparation of design charts based on the study											■	■
Final report submission and preparation of technical papers to referred journals												■

17.0. LIST OF FACILITIES BEING EXTENDED BY THE PARENT INSTITUTION(S) FOR THE PROJECT IMPLEMENTATION.

A) Infrastructural Facilities:

Sr. No	Infrastructural Facility	Yes/No/Not required Full or sharing basis
1.	Workshop facility	Yes
2.	Water & Electricity	Yes
3.	Laboratory Space/Furniture	Yes
4.	Power Generator	Yes
5.	AC Room or AC	Yes
6.	Telecommunication including email & fax	Yes
7.	Transportation	Yes
8.	Administrative/Secretarial support	Yes
9.	Information facilities like Internet/Library	Yes
10.	Computational facilities	
11.	Animal/Glass House	NR
12.	Any other special facility being provided	No

.

B. Equipment available with the Institute/Group/ Department/ Other Institutes for the project:

Equipment available with	Generic Name of Equipment	Model, Make & Year of purchase	Remarks including accessories available and current usage of equipment
PI & his group			
PI's Department	Wave Flume Wave Basin Wave Probes Pressure transducers	Nil Nil DHI make Kistler	

(PROF. S. A.SANNASIRAJ) **(PROF.V.SUNDAR)**

REFERENCES

1. **Allop, N.W.H**. and **Kalmus, D.C**., 1985. Plymouth marine events base: performance of wave screens, Report No. Ex 1327, Hydarulic Research, Wallingford, UK.
2. **Anand, K.V**., and **Sundar, V**., 2009. Dynamic Pressures on Vertical and Curved Seawall Models Due to Random Waves, Proceedings of National Conference on Hydraulics, Water Resources & Coastal and Environmental Engineering, Pune, India.
3. **Anand, K.**V., and **Sundar, V**., 2010. Dynamic Pressures on Vertical and Curved Seawall Models Due to Random Waves, Proceedings of National Conference on Coastal Processes, Resources and Management, Thiruvananthapuram, India
4. **Grüne, J.** and **Kohlhase, S**., 1974. Wave transmission through vertical slotted walls, Proceedings of *14th Coastal Engineering Conference,* ASCE, 3, 1906-1923.
5. **Hayashi,T., Hattori,M.** and **Shirai, M**., 1968. Closely spaced pile breakwater as protection structure against beach erosion, Proc. 11th Intl. conf. on Coastal Engg, 39, 606-621.
6. **Herbich, J.B** and **Douglas, B**. 1989. Wave transmission through a double-row pile breakwater, Proc. 21st International Conference on Coastal Engineering, ASCE, Chapter 165 pp. 2229-2241 Torremolinos, Spain.
7. **Kamikubo, Y.; Murakami, K.; Irie, I.**, and **Hamasaki, Y**., 2000. Study on Practical Application of a Non-Wave Overtopping Type Seawall, International Conference on Coastal Engineering, Proceedings of the Twenty-seventh International Conference held in Sydney Australia. Vol.126, pp. 2215-2228.
8. **Kamikubo, Y.; Murakami, K.; Irie, I.**, and **Hamasaki, Y.**, 2002. Transportation of Water Spray on Non-Wave Overtopping Type Seawall. Proceedings of the Twelfth International Offshore and Polar Engineering Conference held in Kitakyushu, Japan.
9. **Kamikubo, Y.; Murakami, K.; Irie, I.; Kataoka, Y.**, and **Takehana, N**., 2003. Reduction of Wave Overtopping and Water Spray with Using Flaring Shaped Seawall, Proceedings of The Thirteenth International Offshore and Polar Engineering Conference, Honolulu, Hawaii, USA.
10. **Khader, M.H.A.** and **Rai, S.P**. 1981. Wave attenuation due to closely spaced circular cylinders, Proc. International Association for Hydraulic Research, XIX Congress, New Delhi, India pp. 95-102.
11. **Mani, J.S.,** and **Pranesh, M.R**., 1986. Pile breakwater floating barrier interaction, 3rd Indian Confrernce on Ocean Engineering IIT, Bombay, pp B 217.
12. **Murakami, K.; Irie, I.,** and **Kamikubo, Y**., 1996. Experiments on a non-wave overtopping type seawall, International Conference on Coastal Engineering, Proceedings of the Twenty-fifth International Conference held in Orlando, Florida. Vol.122.
13. **Murakami, K.; Kamikubo, Y.**, and **Kataoka, Y**., 2008. Hydraulic Performances of Non-Wave Overtopping Type Seawall against Sea Level Rise Due to Global Warming.
14. **Nagai, S**., 1966. Researches on steel pipe breakwater “, Proc. 10th Coastal Engg Conf., Japan, pp-850-872 .

15. **Sundar, V.**, and **Anand, K.V.**, 2010. Dynamic Pressures and Run-up on Curved Seawalls Compared with Vertical wall under to Cnoidal Waves, Proceedings of National Conference on Coastal Processes, Resources and Management, Thiruvananthapuram, India pp 225-226.

16. **Sundar, V.** and **Subba Rao., B.V.V.** 2003. Hydrodynamic performance characteristics of Quadrant front face pile supported Breakwater. Journal of waterways, Port, Coastal and Ocean Engineering, vol.129, no.1, pp.22-33.

17. **Takahashi, S.**, 1994. Lecture Notes on design of breakwaters, Port and harbour research institute, Japan International cooperation agency.

18. **Truit, C. L.,** and **Herbich, J. B**., 1987. Transmission of Random waves through Pile breakwaters, Proc. 20th Coastal Engineering Conf., pp 2303-2313.

19. **Weber, C**., 1934. Seawall, United States patent office, s.no. 681005.

20. **Wheele, V. J**., and **J.B. Herbich.**, 1972. Wave reflection and transmission for pipe arrays, 13th Coastal Engineering Conference, 135.

21. **Wiegel, R.L**., 1960. Transmission of Waves Past a rigid vertical thin barrier, *Journal of Waterway, Port, Coastal and Ocean Engineering*, 80, 1, pp. 1-12.

22. ---------Coastal Engineering Manual., 2006. U.S. Army Coastal Engineering Research Centre.

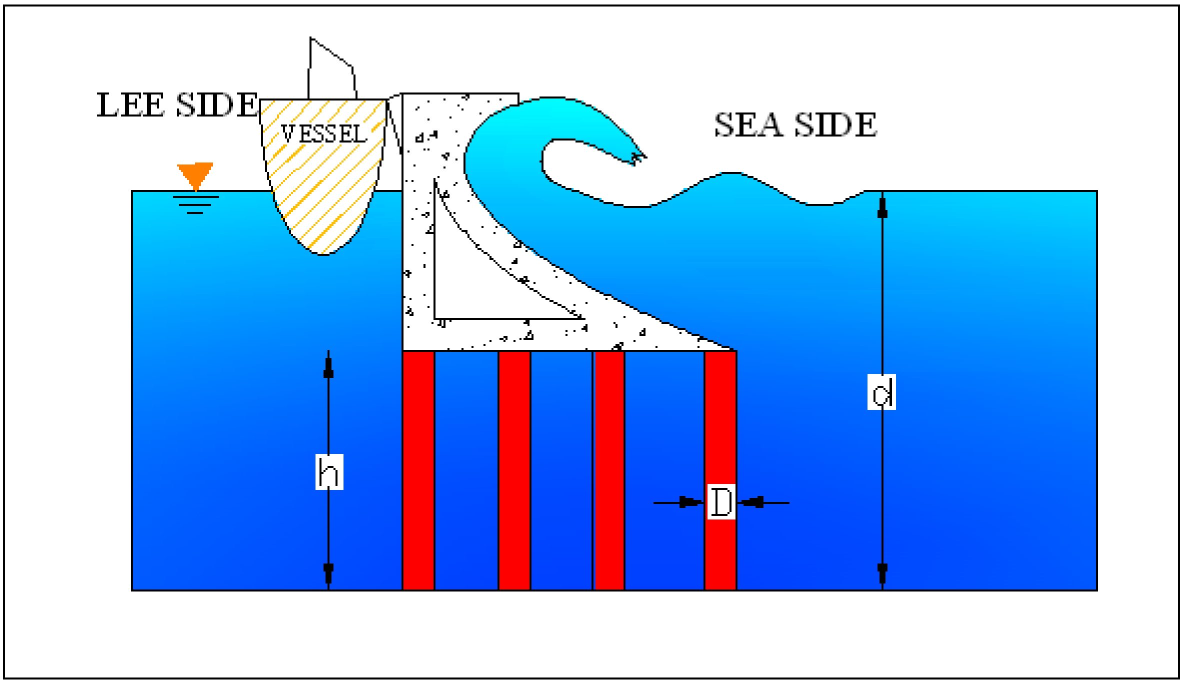

Fig. 1(a)- CP-PBW

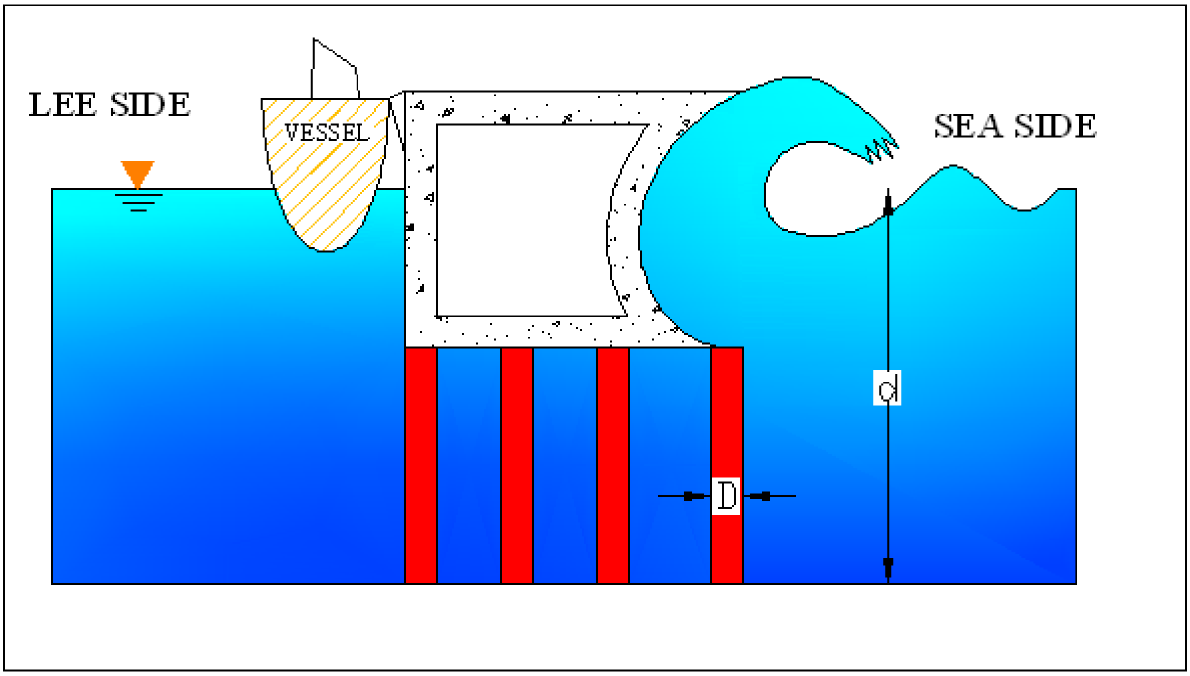

Fig. 1(b)- FS-PBW

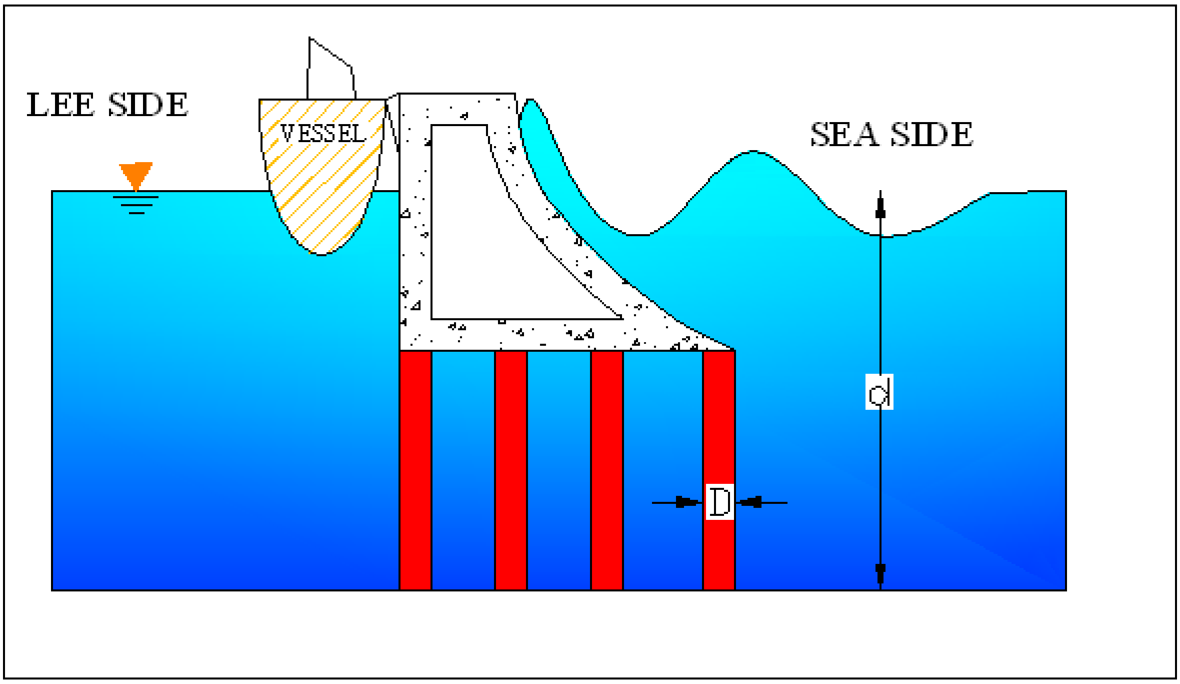

Fig. 1(c)- GW-PBW

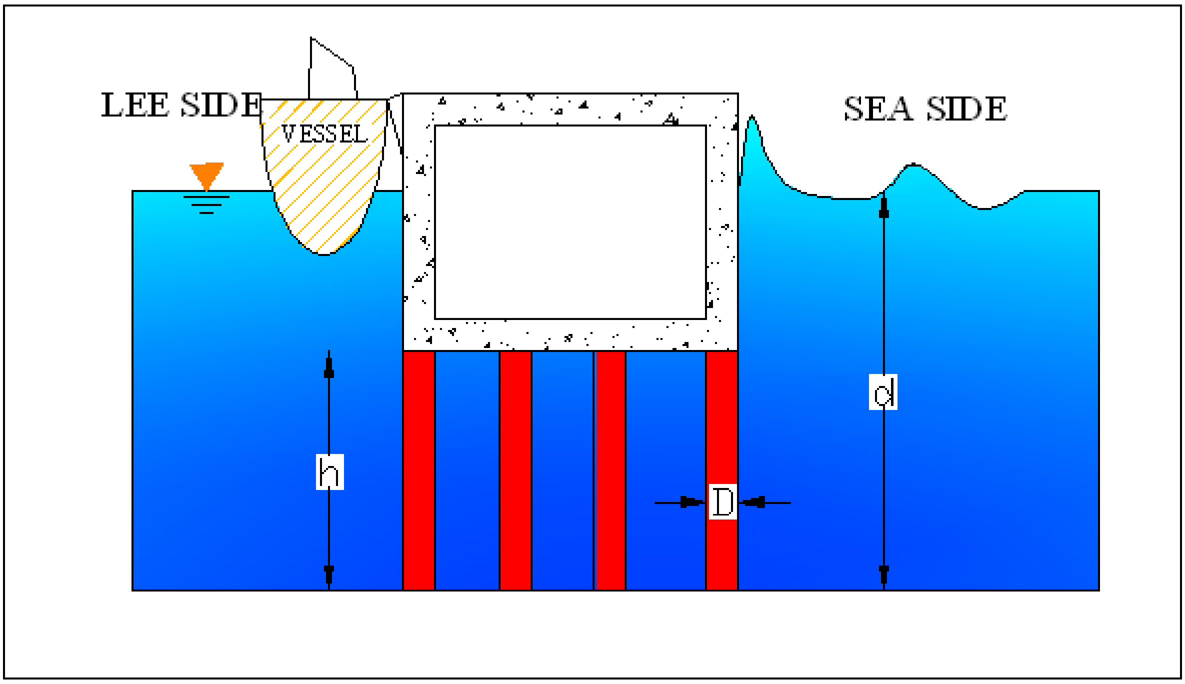

Fig. 1(d)- VW-PBW

Model-(**CP-PBW**)	Circular cum parabolic pile supported breakwater, (Weber, 1934)
Model-(**FS-PBW**)	Flaring shaped pile supported breakwater, (Kamikubo, et al., 2000)
Model-(**GW-PBW**)	Galveston wall pile supported breakwater, (CEM, 2006)
Model (**VW-PBW**)	Vertical wall pile supported breakwater

Fig. 1(a) to (d). The models (CP-PBW) (FS-PBW) (GW-PBW) and (VW-PBW)

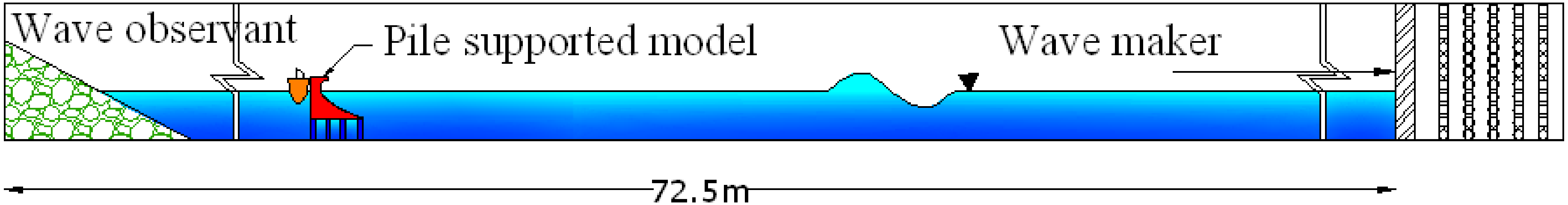

Fig. 2. The sectional view of the models positioned in the wave flume

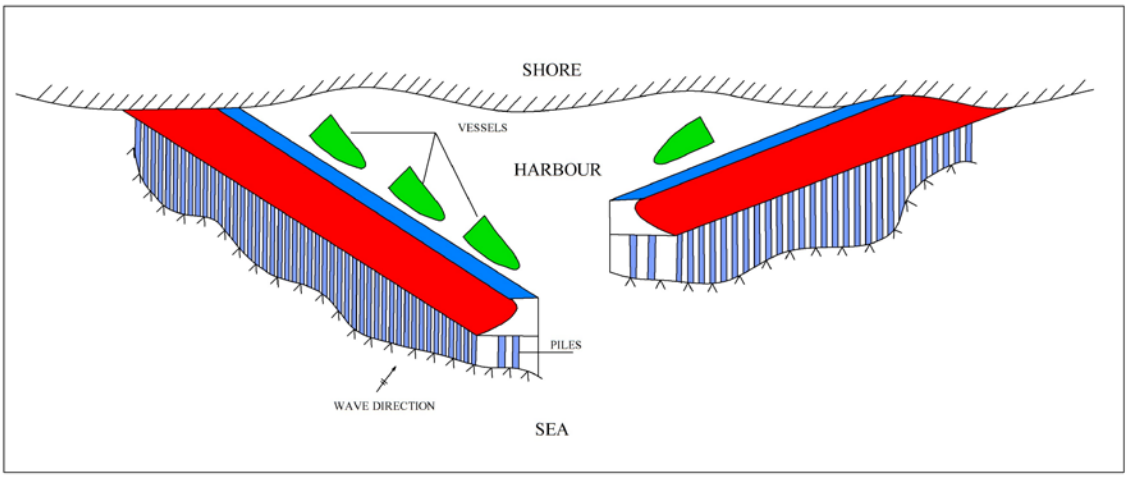

Fig. 3(a)- A perspective view of a harbour basin with the proposed CP-PBW as breakwaters

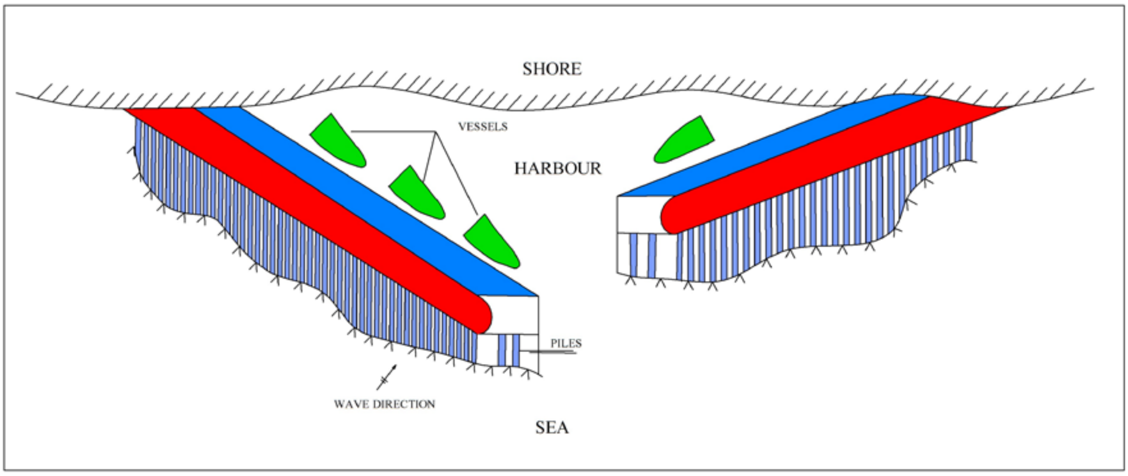

Fig. 3(b)- A perspective view of a harbour basin with the proposed CFS-PBW as breakwaters

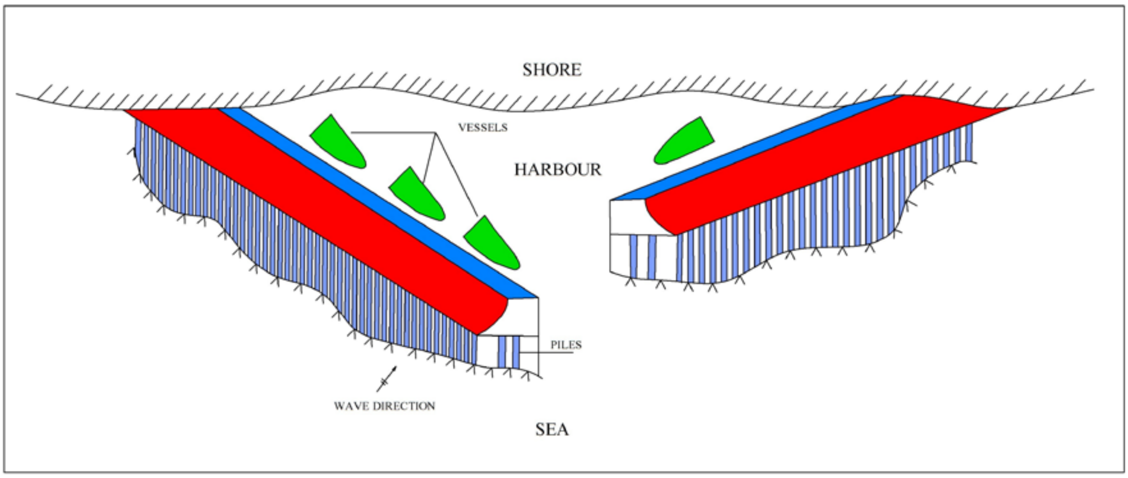

Fig. 3(c)- A perspective view of a harbour basin with the proposed GW-PBW as breakwaters

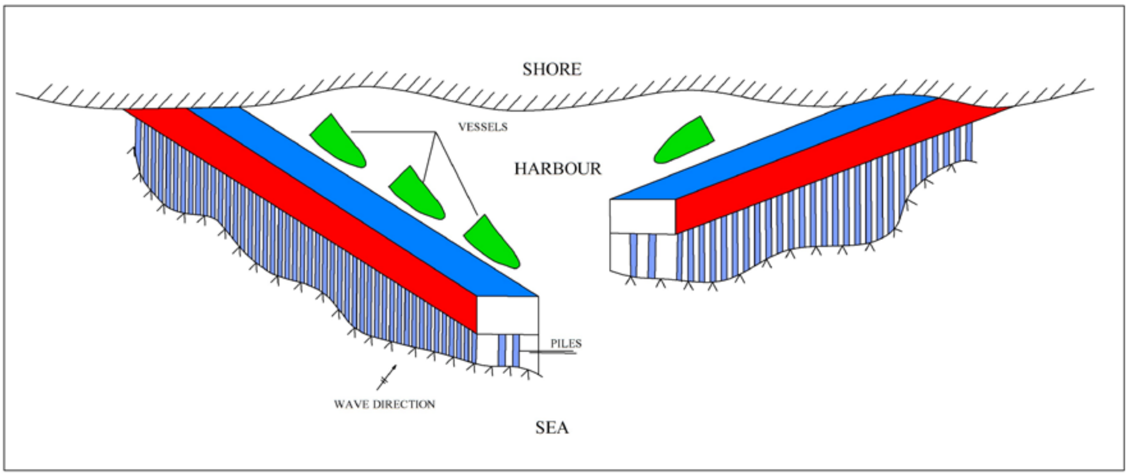

Fig. 3(d)- A perspective view of a harbour basin with the proposed VW-PBW as breakwaters

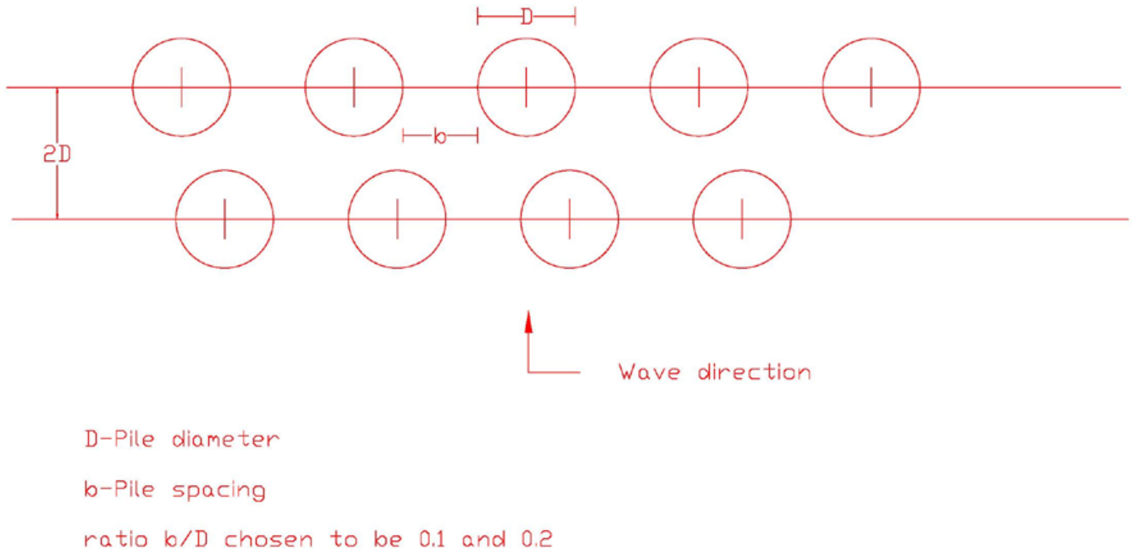

Fig. 4. Definition sketch for pile geometry