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Technical note

Stability of breakwater defenced by a seaward submerged reef

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Abstract

The stability of a uniformly sloped conventional rubble mound breakwater defenced by a seaward submerged reef is investigated using physical model studies. Regular waves of wide ranging heights and periods are used. Tests are carried out for different spacings between two rubble mound structures ($X/d = 2.5$–13.33) and for different relative heights ($h/d = 0.625$–0.833) and relative widths ($B/d = 0.25$–1.33) of the reef. It is observed that a reef of width (B/d) of 0.6–0.75 constructed at a seaward distance (X/d) of 6.25–8.33 breaks all the incoming waves and dissipates energy and protects the breakwater optimally.

Keywords: Reef; Regular waves; Wave breaking; Wave height attenuation; Breakwater; Stability

1. Introduction

Rubble mound breakwaters are the structures which are meant to reflect and dissipate energy of the wind generated waves and there by to prevent their incidence on a water area intended to protect; Submerged breakwater with its crest at or below still water level (SWL) can cause substantial wave attenuation and can be effectively [1]used in places where tidal variations are small and only partial protection from waves is required, like harbour entrance, beach protection, small craft harbours, etc.

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The wave breaking over submerged breakwater causes great turbulence on lee side. Current and turbulence together on lee side of submerged breakwater have a strong power of erosion on a sandy bottom and can thus prevent siltation. They also offer resistance through friction and turbulence created by breakwater interference in wave field causing maximum wave damping and energy dissipation, minimum wave reflection and bottom scour, and maximum sand trapping efficiency and are used for coastal protection. (Baba, 1985; Pilarczyk and Zeidler, 1996).

Submerged breakwaters are used as a protection to reclamation bund (Gadre et al., 1985). The submerged breakwaters are also used for protecting an already existing breakwater. It can be used as a rehabilition structure for a damaged breakwater, which is secured from storm waves (Gadre et al., 1989). The design of this type of combined structures requires detailed information on parameters such as wave loads on breakwater armour units, run up and run down on breakwater slope, damage, height of submerged structure, its crest width, its seaward location, wave transmission, armour weight, porosity, etc…The hydrodynamic performance of this structure is investigated based upon physical model study. The magnitude of various hydrodynamic parameters will give an indication of the suitability of this submerged breakwater as defense structure. Run up is required to select the crest elevation of the protected breakwater. Wave transmission at submerged breakwater gives the wave height that is going to impinge on the breakwater, which is then used to estimate the armour stone weight. The varying geometry and seaward location of submerged structure will help in designing an optimum structure, therefore the experimental investigation was carried out to determine the effect of submerged breakwater on wave transmission, wave run up and run down and ultimately stability of the protected breakwater and optimize its location with respect main breakwater and geometry.

2. Literature review

The review of literature revealed that the stability of breakwater armour, effect of porosity, and gradation wave transmission at submerged structure, wave propagation in surf zone, wave run up and run down etc has been studied by various investigators since Hudson (1959). The effect of slope, crest width and depth of submergence of various shapes of submerged breakwaters on wave transmission was studied by Johnson et al. (1951); Dick and Brebner (1968); Dattatri et al. (1978), Smith et al. (1996); Pilarczyk and Zeidler (1996); Twu et al. (2001). Many authors opine that the submerged structure is constructed in a water depth of 1.5–5 m with a slope of 1:2 to 1:3 and a height exceeding 0.7 times the depth of water. Baba (1985) writes that a reinforced concrete smooth submerged breakwater experimented in Russia with a seaward slope of 1:1.67 and vertical shoreward slope gives optimum wave transmission with minimum reflection for a tidal range less than 2 m and steepness greater than 0.075. But there as many opinions as the number of investigators on what should be the crest width of submerged breakwater.

Gadre et al. (1985), designed a submerged bund to break higher waves dissipate energy while protecting the revetment constructed at 100 m shoreward to retain land behind it.

Thus they reduced the required armour stone weight of 15 ton for a conventional reclamation structure in a depth of 8 to a stone weight of 2–3 ton by constructing the submerged bund protecting revetment at Madras port, India. Gadre et al. (1989) built a submerged breakwater at 80 m seaward to protect a damaged breakwater head of west breakwater at Veraval port in Gujarat, India. This submerged structure broke the storm waves protecting the damaged breakwater, which was repaired later.

Cox and Clark (1992) based on limited study, built a breakwater defenced by seaward submerged reef structure for protecting a marina harbour at Hammond, Indiana situated at southern tip of Lake Michigan. They developed the two breakwater system called tandem breakwater as least cost alternative to a single conventional breakwater. They designed the breakwater with 3 ton armour at 40 m and a seaward submerged reef constructed at as seaward distance of 4.0 m with 0–1 ton instead of 8 ton stone required for a conventional non overtopping breakwater. They could also lower the crest by 1.5 m and saved about 1.0 million dollars. Cornett et al. (1993) after conducting experimental investigation concludes that they may be a optimum location for submerged reef of relative height $h/d>0.6$ which protects the inner main breakwater. Ahrens (1989); Pilarczyk and Zeidler (1996), and Nizam and Yuwono (1996) have presented equations and graphs to calculate the armour weight of submerged reef breakwater.

Neelamani et al. (2002) experimentally investigated the hydraulic performance of a seawall defenced by a detached breakwater.

3. Problem selection

After studying the literature it is felt that, in depth physical model study is required regarding protective structures for the breakwater. The breakwater withstands extreme loads of storm waves can be safely designed with a protective structure on seaward side. Further it is decided that a submerged structure was one economical and effective option for protection of main breakwater as it breaks steep waves and attenuate them to a tolerable level.

A submerged reef is selected as the protective structure, as it is the optimized structure to a highest degree. It is economical, efficient in breaking the steep waves and is safe as it cannot fail catastrophically because it does not have a core. Also the reef while allowing some sediment to pass over it, retains sediments on its lee.

The literature survey also threw light on the designers' dilemma of choosing right geometry of a submerged structure especially regarding its crest width and location. Hence, it is decided that experimental work be taken up to study the influence of seaward location and crest width of reef, as a protective structure, on the stability of breakwater.

4. Objective of the study

The objective the present experimental investigation is to study the influence of geometry of a submerged reef located at varying seaward distances on stability of the breakwater. During the model tests the location and crest width of submerged reef is varied and its influence on wave height attenuation, run up and run down and stability of breakwater is studied for a range of wave characteristics. In the course of test runs, wave breaking, wave attenuation, wave run up and run down and damage of breakwater is observed.

5. Experimental setup

5.1. Wave flume

The wave flume is 50 m long, 0.71 m wide and 1.1 m deep. It has a 42 m long smooth concrete bed. About 15 m length of the flume is provided with glass panels on one side. It has a 6.3 m long, 1.5 m wide and 1.4 m deep chamber at one end where the bottom hinged flap generates waves. The flap is controlled by an induction motor of 11 kW, 1450 rpm. This motor is regulated by an inventor drive (0–50 Hz) rotating in a speed range of 0–155 rpm. Regular waves of 0.08–0.24 m of periods 0.8–4 s can be generated with this facility.

5.2. Instrumentation

Two capacitance type wave probes were used one to measure incident wave height at about 1 m seaward of reef toe and another probe to measure transmitted wave height after breaking over the reef. The same probe was moved to measure wave height approaching the breakwater toe. During the experiment, the signals from wave channels are verified with digital oscilloscope along with computer data acquisition system. The main water surface elevation on seaward and shoreward side of the reef are converted into electrical signals. These are then stored as digital signals by software controlled 12-bit A/D converter with 16 digital input/output. During the experiment, every time after five waves pass the reef, transmitted waveform for 10 s duration is acquired using software ADTRIG-TC.

The cross section of the breakwater is surveyed using the profiler mounted over the rails fixed on top edges of the wave flume. The profiler consists of nine brass sounding rods with a ball and socket foot where the foot was circular with diameter of about 0.035 m.

6. Breakwater model

The breakwater with a uniform slope of $1V{:}2H$ is constructed on the flat bed of the flume with primary stone armour of nominal diameter D_{n50} equal to 0.0298 m. The secondary armour and core are designed as for a conventional breakwater Fig. 1.

A stable submerged reef of 0.25 m height is designed with homogeneous pile of stones with a nominal diameter D_{n50} equal to 0.0221 m (Ahrens, 1984; 1989; Pilarczyk and Zeidler, 1996; Nizam and Yuwono, 1996).The slope of the structure is $1V{:}2H$ uniformly. The reef is constructed with crest width 0.1, 0.2, 0.3, and 0.4 m at location 1, 2.5, and 4 m

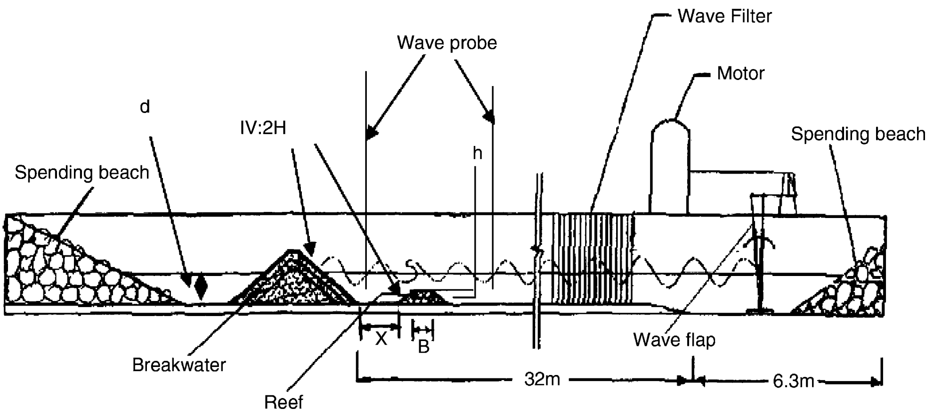

Fig. 1. Details of experimental setup.

seaward of the breakwater. The placement technique used for armour is keyed fitted placement. The model is tested for wave heights of 0.1, 0.12, 0.14, and 0.16 m of periods 1.5, 2.0, and 2.5 s in water depths of 0.3, 0.35, and 0.4 m. The armour units were painted with different colors and placed in bands of 0.2–0.3 m heights to track their movement during damage. The governing variables together with their possible range of application are listed in Table 1. The breakwater stability will be checked in response to non-dimensional reef and wave characteristics listed in Table 2.

Table 1
Range of governing variables

SI. No.	Variable	Expression	Range
1.	Wave height	H	0.10, 0.12, 0.14, 0.16 m
2.	Wave period	T	1.5, 2.0, 2.5 s
3.	Storm duration	N	3000 waves
4.	Water depth	d	0.30, 0.35, 0.40 m
5.	Angle of wave attack	F	90°
6.	Nominal diameter	D_{n50}	0.0298 m
7.	Mass density	ρ	2.77 gm/cm^3
8.	Armour weight	W_{50}	73 gm
9.	Porosity		55%
10.	Slope		$1V{:}2H$
11.	Reef Armour type		Angular quarry stone
12.	Nominal diameter for reef		0.0221 m
13.	Reef Armour weight	30 gm	
14.	Crest height of reef	h	0.25 m
15.	Crest width of reef	B	0.10, 0.20, 0.30, 0.40 m
16.	Porosity of reef		35%
17.	Location of reef	X	At 1.0, 2.5 and 4.0 m seawards of the breakwaters

Table 2
Non-dimensional reef and wave characteristics

SI. No.	Variable	Range
1.	Reef characteristics	
	Slope	1*V*:2*H*
	Relative height (h/d)	0.625–0.833
	Relative crest width (B/L_0)	0.01–0.114
	Relative crest width (B/d)	0.25–1.33
	Relative reef submergence (F/H_i)	0.312–1.5
	Seaward reef location (X/d)	2.5–13.33
2.	Wave characteristics	
	Steepness parameter (H_0/gT^2)	0.00163–0.00725
	Angle of wave attack	90°

6.1. Calibration of test facilities

Before the model tests are started the experimental set up is calibrated to find the proper wave height assigned to a particular combination of generator stroke and wave period for different water depths while the complete model setup is in place. The wave probes are also calibrated for temperature correction every morning and afternoon before the test begen and the wave characteristics are recorded. These are verified by manual observation.

6.2. Preliminary investigations

Initially the submerged reef is constructed at distance (X) of 8m and wave attunenation is observed for a distance of every meter shoreward of it. It is proved up to 4 m there is about 25–50% attenuation of waves after that there is uniform attenuation of 50%. Therefore, it is decided to locate the reef within a maximum distance of 4 m seaward of the breakwater.

6.3. Test procedure

Initially the newly constructed breakwater slope is surveyed with the profiler, which is, the reference survey for comparison of subsequent surveys. The waves are sent in short burst of five waves during the test so that generator would be shut off just before wave energy reflected from slope could reach the generator flap. Between wave burst there are brief intervals to allow reflected wave energy to dampen out.

The model is subjected to a series of smaller wave heights starting from 0.1 m and then gradually wave height is increased by 20% each time till it reached the highest value of 0.16 m for a selected period. Waves are run in bursts in the model until it appeared that no stones would be moved further by waves of this height or 3000 waves which is equivalent to a actual storm for 6–11 h, for each trial or the failure of the structure whichever occurred earlier. This is because 80% of the total damage would have already inflicted by that time and equilibrium would have established (Meer Van der and Pilarczyk, 1984). Damage level (S) is calculated as the ratio of area of erosion (A_e) to square of nominal diameter

D_{n50} of breakwater armour. The failure for these tests is defined as the displacement of primary armour stones so that filter layer is exposed to wave action and core material is actually being removed through the secondary filter layer as defined as Ahrens (1970).

The results are compared with those of the single conventional non-overtopping and non-defenced breakwater.

6.4. Test conditions

1. The sea bed is horizontal and sediment motions do not interfere with the wave motion and do not affect the model performance.
2. Secondary waves during wave generation are not considered.
3. Wave reflection from the structure does not interfere with freshly generated incident waves.
4. The density difference between freshwater and seawater is not considered.
5. Only hydraulic performance of the test model is considered.
6. Piling up of water behind the reef is negligible.

7. Scale effects

Scale effects occur mainly because of the differential hydraulic behavior of model and prototype. The scale effects are predominant if the Reynolds number in the model is too small. In the present investigation the Reynolds number was always maintained above 3.5×10^4 and therefore, scale effects are not significant (Owen and Briggs, 1986).

8. Measurements

The incident and transmitted wave heights are measured through capacitance type wave probes. They are checked by manual observation and average of 30 readings is taken. The run up and run down are measured usually using a scale fixed parallel to structure slope. The damaged structure is surveyed by taking sounding using profiler system. The cross section is surveyed by nine sounding rods at 0.1 m interval along the slope from crest to toe on the seaward side. There is no damage on the leeward side of the breakwater. The submerged reef is stable through out the investigations.

9. Results and discussions

9.1. To find optimum reef location

9.1.1. Influence of steepness parameter (H_0/gT^2) on wave height attenuation (WHA)

The submerged reef successfully trips the steeper waves and dissipates wave energy. The effectiveness of reef in damping of waves increases with an increase in wave

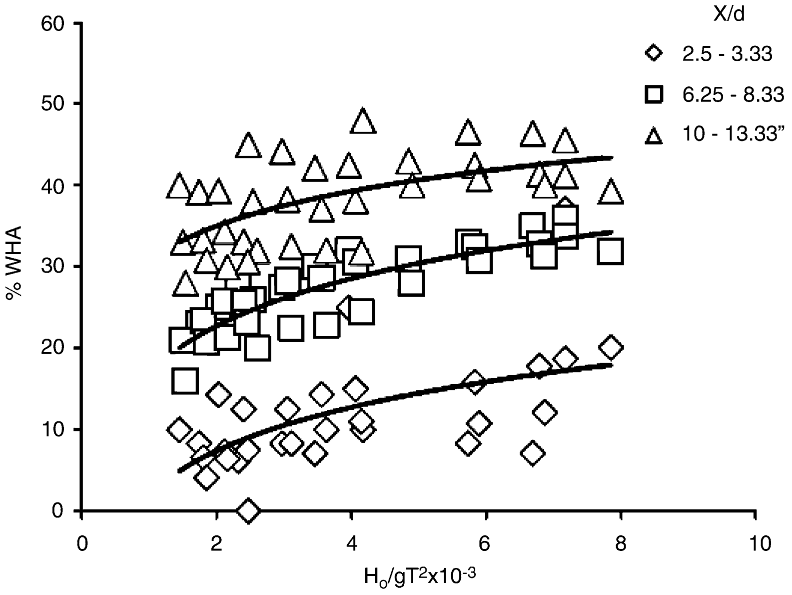

Fig. 2. Variation of wave height attenuation with deep waer wave steepness.

steepness. Further, as the distance between breakwater (X) increases, the waves those break over the reef, loose some more energy while propagating in the stilling basin (i.e. the energy dissipation zone), which is illustrated in Fig. 2. The reef located at $X/d = 2.5$–3.33 attenuates the waves by a maximum amount of 18% while for the reef located at $X/d =$ 6.25–8.33, the maximum attenuation of wave heights $(1 - K_t)$ is about 33% and for the reef located at $X/d = 10$–13.33, it is about 43%.

9.1.2. Influence of steepness parameter (H_0/gT^2) on run up (R_u/H_0) and run down (R_d/H_0)

The Influence of steepness parameter (H_0/gT^2) on run up (R_u/H_0) and run down (R_d/H_0) for single and defenced breakwater with different reef locations is clear from Fig. 3(a) and (b). The general trend is that, the run up and run down decrease with an increase in seaward distance of the reef for all ranges of X/d. However, it is observed that reef located 1 m seaward from the breakwater is unable to reduce the run up and run down over the defenced breakwater. This is because as the waves break on the reef, shoreward of the reef, there is a zone of length about 0.8–1.4 m of turbulence and high degree of mixing and the surface of the turbulent water fluctuated due to spreading of water. These causes more run up and run down (Diskin et al., 1970). For the reef located at a seaward distance of $X/d =$ 6.25–8.33 (i.e. $X = 2.5$ m), the run up and run down are reduced by about 30% and up to 20%, respectively, compared to single breakwater. Similarly, the reef located at a seaward distance of $X/d = 10$–13.33 (i.e. $X = 4.0$ m), reduces the run up and run down by about 50% and up to 40%, respectively, compared to single breakwater.

This shows that the reef should be located beyond $X/d = 2.5$–3.33 (i.e. $X = 1$ m) seaward of the breakwater to influence the run up and run down. Compared to reef at

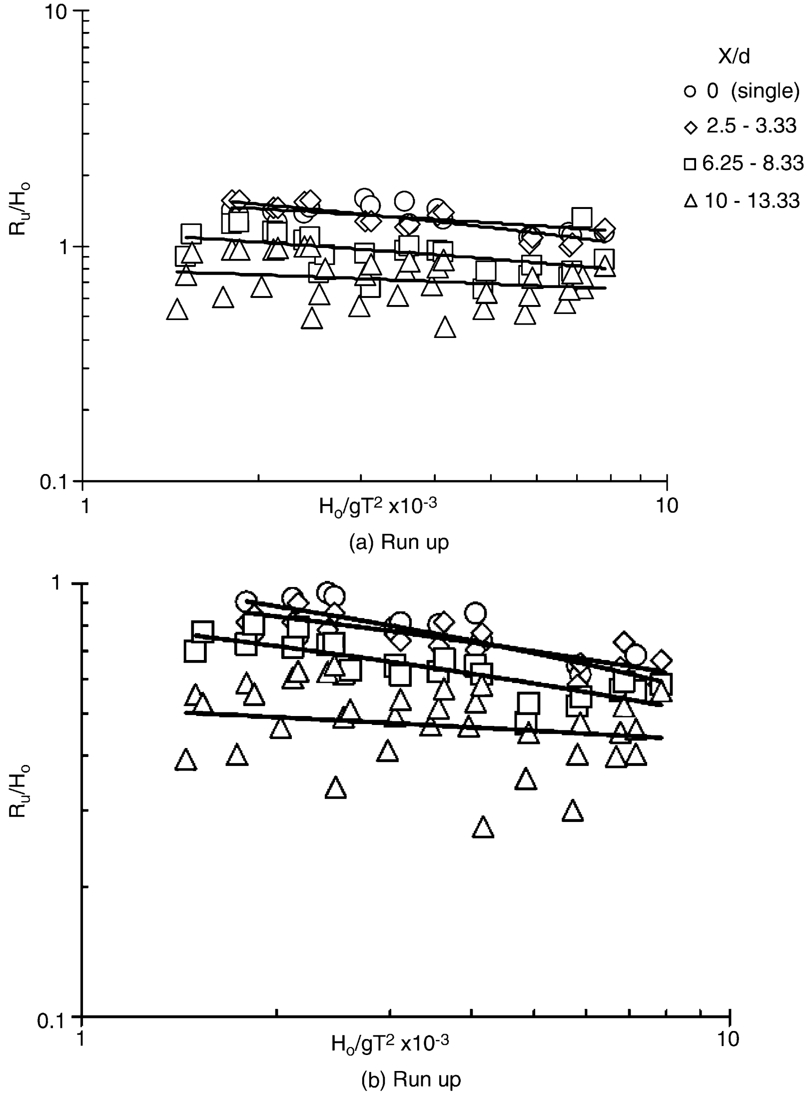

Fig. 3. Variation of relative run up and run down with deep waterr wave steepness parameter.

$X/d = 6.25$–8.33, the reef at $X/d = 10$–13.33 reduces the run up and run down by about 20% and up to 25%, respectively.

9.1.3. Influence of steepness parameter (H_0/gT^2) on damage level (S)

The damages of single as well as defenced breakwater with increasing steepness parameter (H_0/gT^2) for varying reef location are plotted in Fig. 4. The curves shown are envelops. It is seen that damage is highest for single breakwater, and decreases with

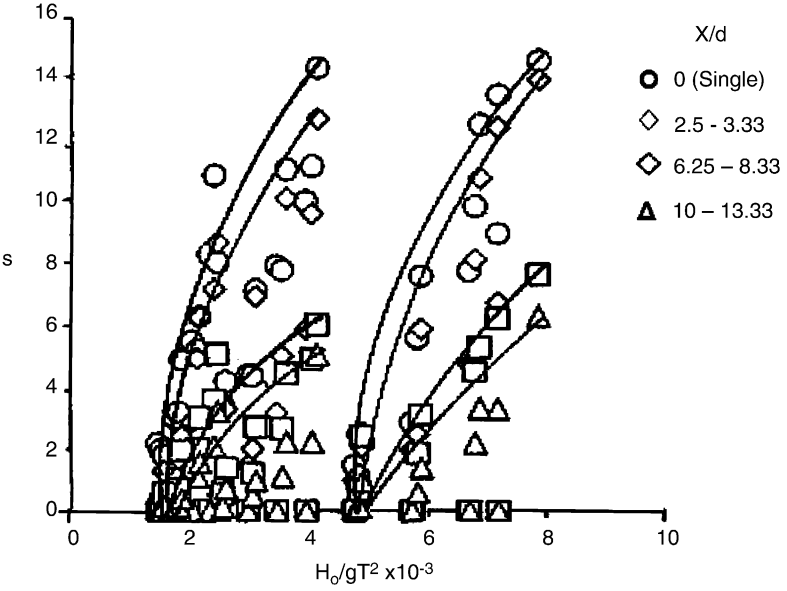

Fig. 4. 4Variation of damage level with deep water wave steepness parameter.

increase in breakwater spacing (*X*/*d*). This is because with increasing breakwater spacing the WHA increases causing high wave energy dissipation. It is observed that the damage at $H_0/gT^2 = 1.5\times10^{-3}$ and 5×10^{-3} the damages are minimum. This is because all these waves break over the submerged reef and lack enough energy to damage the breakwater. The damage of defenced breakwater with reef at $X/d = 2.5$–3.33 (i.e. $X = 1$ m) is up to 12% less compared to single breakwater with reef at $X/d = 6.25$–8.33 (i.e. $X = 2.5$ m) and $X/d = 10$–13.33 (i.e. $X = 4$ m) the damage is less up to 55 and 60%, respectively, compared to single breakwater. It is also seen the reef located at $X/d = 6.25$–8.33 reduces the damage of breakwater which is almost same as that for reef at $X/d = 10$–13.33.

From the above dimensions it is seen that the optimum breakwater spacing is $X/d =$ 6.25–8.33 (i.e. $X = 2.5$ m). Further the analysis is carried out for the optimum breakwater spacing of $X/d = 6.25$–8.33 and effort will be directed towards finding the optimum crest width of the reef.

9.2. *To find optimum reef crest width*

9.2.1. *Effect of steepness parameter (H_0/gT^2) on wave attenuation (WHA)*

Fig. 5 shows the trends of wave height attenuation (WHA) against varying deep-water wave steepness parameter (H_0/gT^2) for a range of reef crest widths $B/d = 0.25$–1.33. For a given crest width, the WHA increases with an increase in steepness. The reason is that the reef breaks the steeper waves successfully, increasing wave damping, resulting in increased WHA. The wider reefs increase WHA. This is because after breaking, wider reef offer friction and waves increasingly shoal over them. The maximum WHA are 33, 41, 44

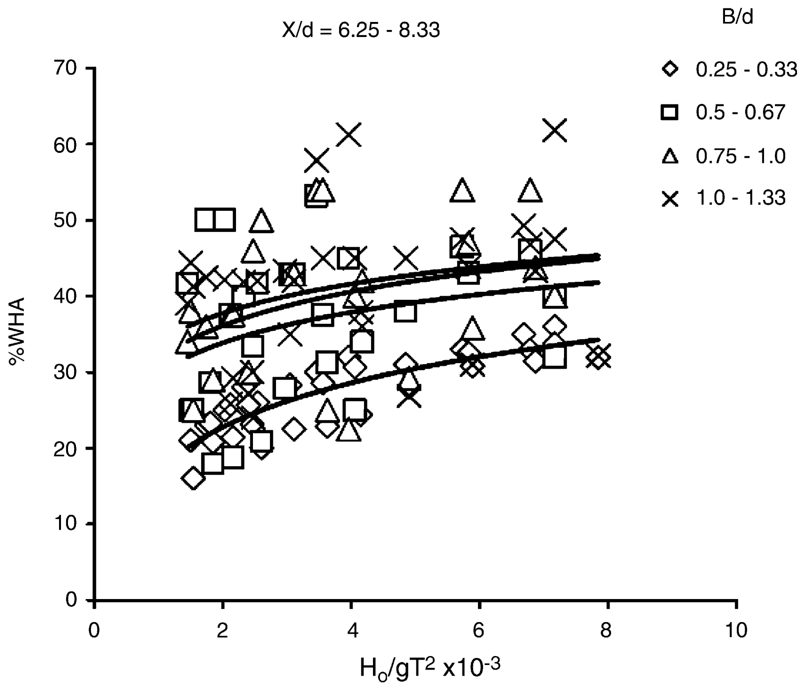

Fig. 5. Variation of wave height attenuation with the deep water wave steepness parameter.

and 46% for the ranges of crest widths (*B*/*d*) of 0.25–0.33, 0.5–0.67, 0.75–1.0 and 1.0–1.33, respectively.

9.2.2. Effect of steepness parameter (H_0/gT^2) on run up (R_u/H_0) and run down (R_d/H_0)

As the steepness parameter (H_0/gT^2) increases, the relative run up (R_u/H_0) and run down (R_d/H_0) decrease. These behaviors are shown in Fig. 6(a) and (b). It can be seen from the figures that run up over the defenced breakwater for all ranges of crest widths are about 15–30% less than that for a crest width of $B/d = 0.25$–0.33. Fig. 6(a) also shows that for all ranges of crest width except for $B/d = 0.25$–0.33, the run up is almost same and effect of the relative reef crest width is felt at higher deep water wave steepness (i.e. $H_0/gT^2 > 5 \times 10^{-3}$).

For run down in Fig. 6(b), except for crest width $B/d = 0.25$–0.33, the points for other crest widths are all equally scattered and show almost identical trend. Compared to $B/d = 0.25$–0.33, other crest widths reduce the run down by 40–60%.

9.2.3. Effect of steepness parameter (H_0/gT^2) on damage level (S)

Fig. 7 shows variation of the damage level for varying wave steepness parameter and reef crest widths. From the figure it is observed that the damage level decreases as the reef crest width increases. The damage level (*S*) of the breakwater is less than 2

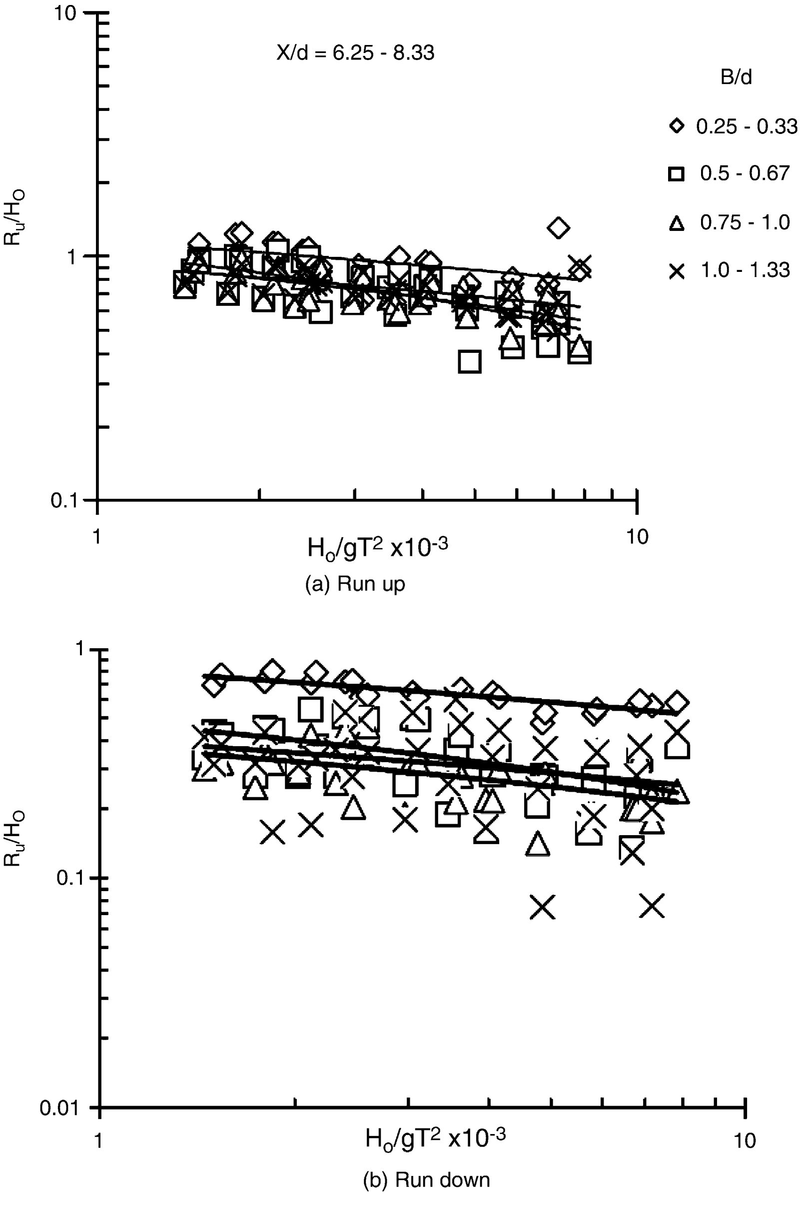

Fig. 6. Variation of relative run up and run down with the deep water wave steepness parameter.

(i.e. the breakwater is stable) for all reef crest widths except $B/d = 0.25$–0.33 for H_0/gT^2 less than 6×10^{-3}. It is also seen from the graph that the breakwater damage is minimum at $H_0/gT^2 = 1.5\times10^{-3}$ and 4.75×10^{-3} this is because the wave damping is maximum at thee points.

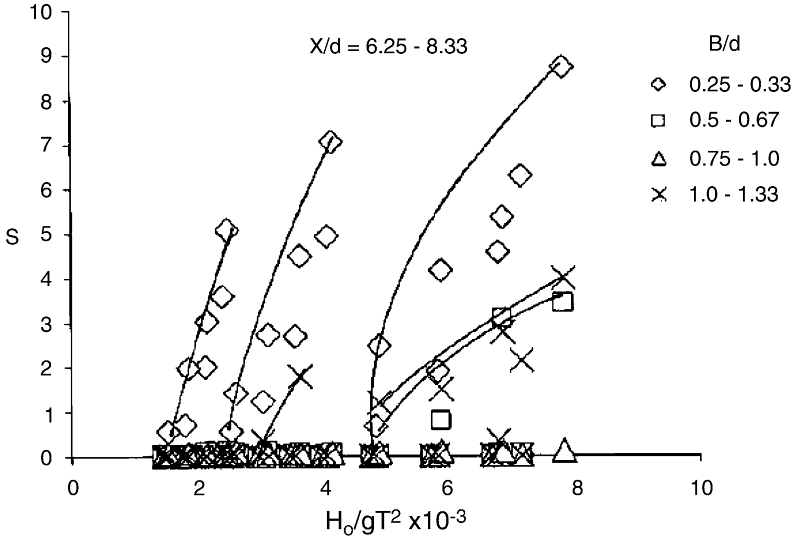

Fig. 7. Variation of damage level with deep water wave steepness parameter.

9.2.4. Effect of relative reef submergence (F/H_i) on damage level (S)

The waves generated in a water depth (d) of 0.3 m did not damage the breakwater irrespective of the crest widths of the submerged reef. This was because all the waves, irrespective of their heights and period broke over the reef and transmitted smaller waves, which did not posses sufficient energy so as to damage the breakwater. Therefore, we can infer that the breakwater stability is highest for a water depth (d) of 0.3 m. The damage level (S) for water depths (d) of 0.35 and 0.4 m are shown in Fig. 8(a) and (b), respectively. Figures indicated that for a particular water depth increases with increase in relative reef submergence (F/H_i) for various values of relative reef crest width B/L_0. As F/H_i increases the reef submergence (F) increases and this reduces wave breaking resulting in increased transmission, which ultimately results in increased damage. The impact of waves period is seen in increase of damage with an increase in B/L_0 for a given crest width (B). The reason is that for a given (B) with the increase in B/L_0 shows decreased wave period. As the wave period is reduced wave steepness increases and though the waves break over the reef the broken waves are still capable of inflicting damage on the breakwater. Thus from the Fig. 10 the damage of the breakwater is observed for $B/L_0=0.03$, 0.048 and 0.085 (i.e. $B=0.3$ m) is zero for water depth of 0.35 and 0.4 m.

9.2.5. Influence of reef crest width (B/L_0) and (B/d) on damage level (S)

The influence of relative reef crest width (B/L_0 and B/d) on damage level (S) is shown in Fig. 9(a) and (b) it can be seen that the optimum values of relative crest widths are $B/L_0=0.035$–0.045 for relative reef crest height h/d varying from 0.625 to 0.833 and $B/d=0.6$ to 0.75 for relative reef crest height $h/F=1.67$–5.0. From the figures, it is observed that for $h/d=0.833$ and $h/F=5.0$ (i.e. for depth $d=0.3$ m) the damage is zero and breakwater is stable for entire range of relative crest width of B/L_0 and B/d.

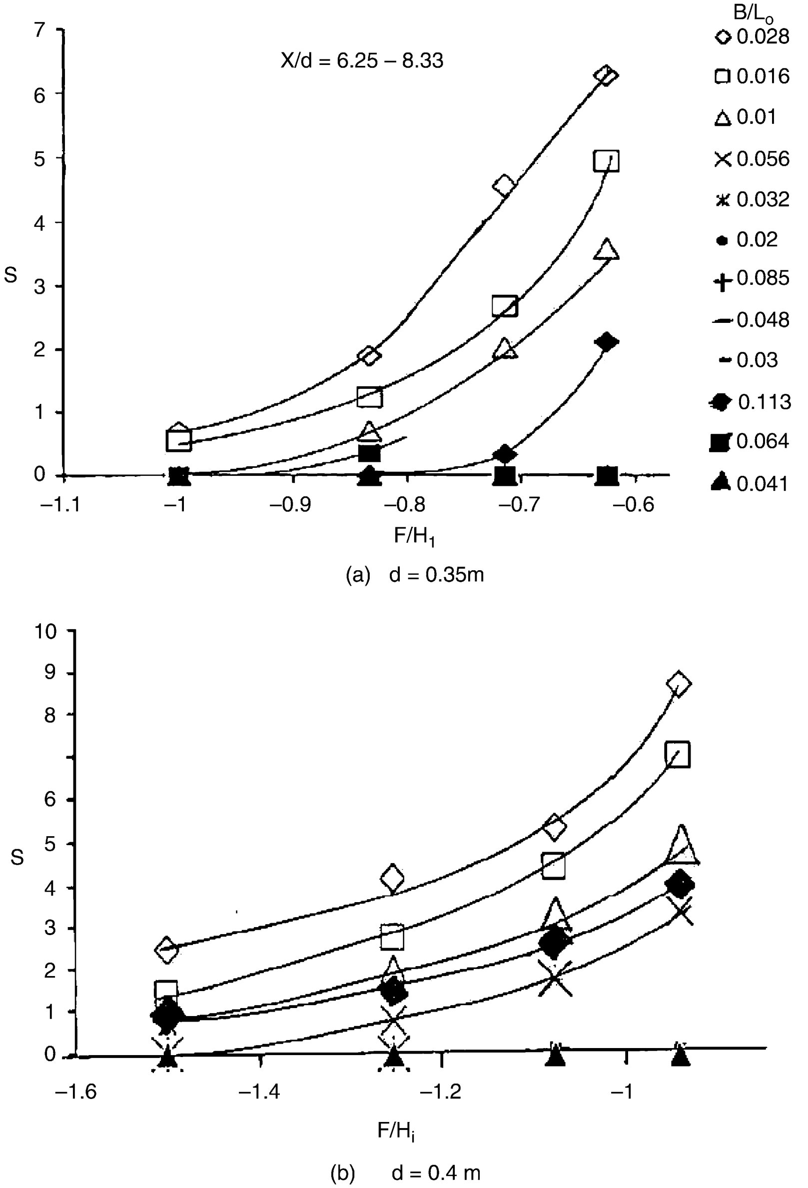

Fig. 8. Variation of damage level with reef submergence.

9.2.6. Influence of reef crest width on transmission coefficient

The general trend is that the transmission coefficient (K_t) decreases with an increase in reef crest width (B) is indicated in Fig. 10. Fig. 10(a) shows that variation of K_t with relative reef crest width (B/L_0) for varying relative reef height (h/d). For a given reef height K_t decreases with an increase in B/L_0. Also K_t decreases with an increase in reef height.

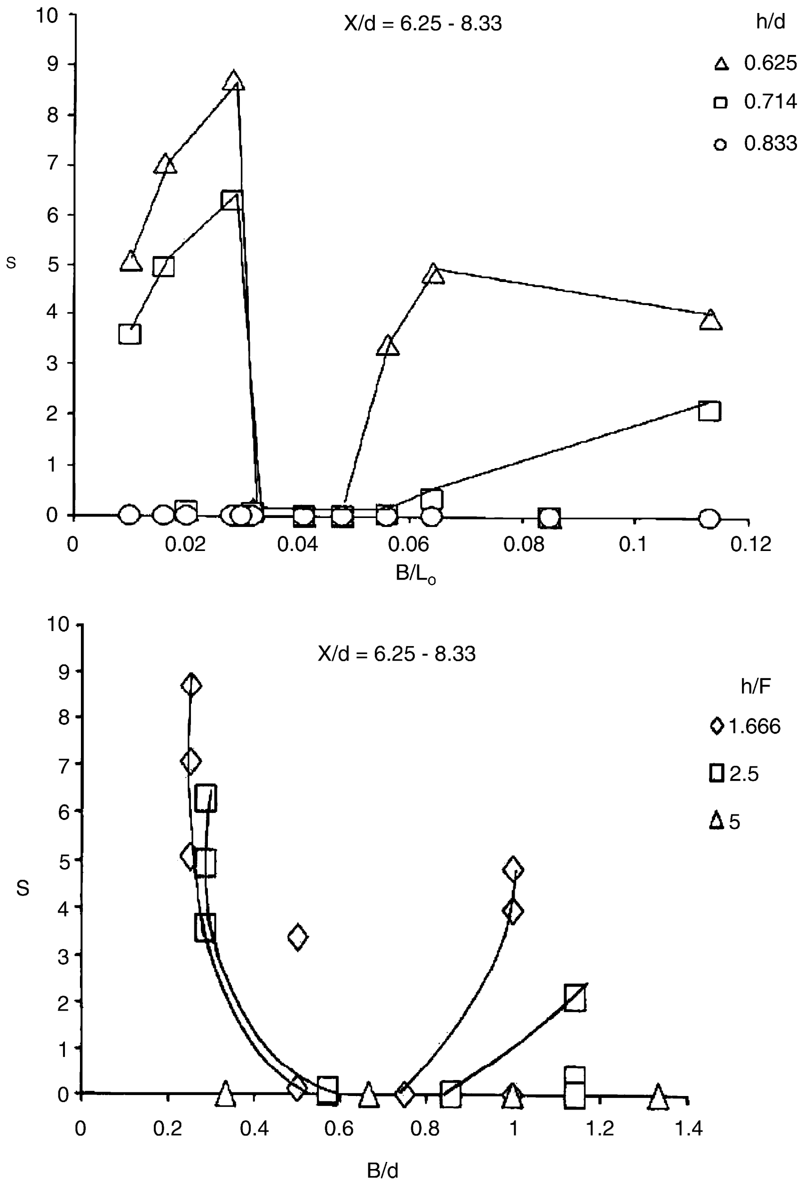

Fig. 9. Variation of maximum damage level with relative reef crest width.

Influence of reef width (B/L_0) is more pronounced as the reef height (h/d) increases. This confirms the observation of Cornett et al. (1993) that reef with height (h/d) greater that 0.6 is beneficial in reducing the wave transmission. The reasons are wave breaking increases and transmission reduces as both the reef crest and reef height increase. For the optimum value of B/L_0 0.035–0.045 the K_t values are about 0.5–0.75.

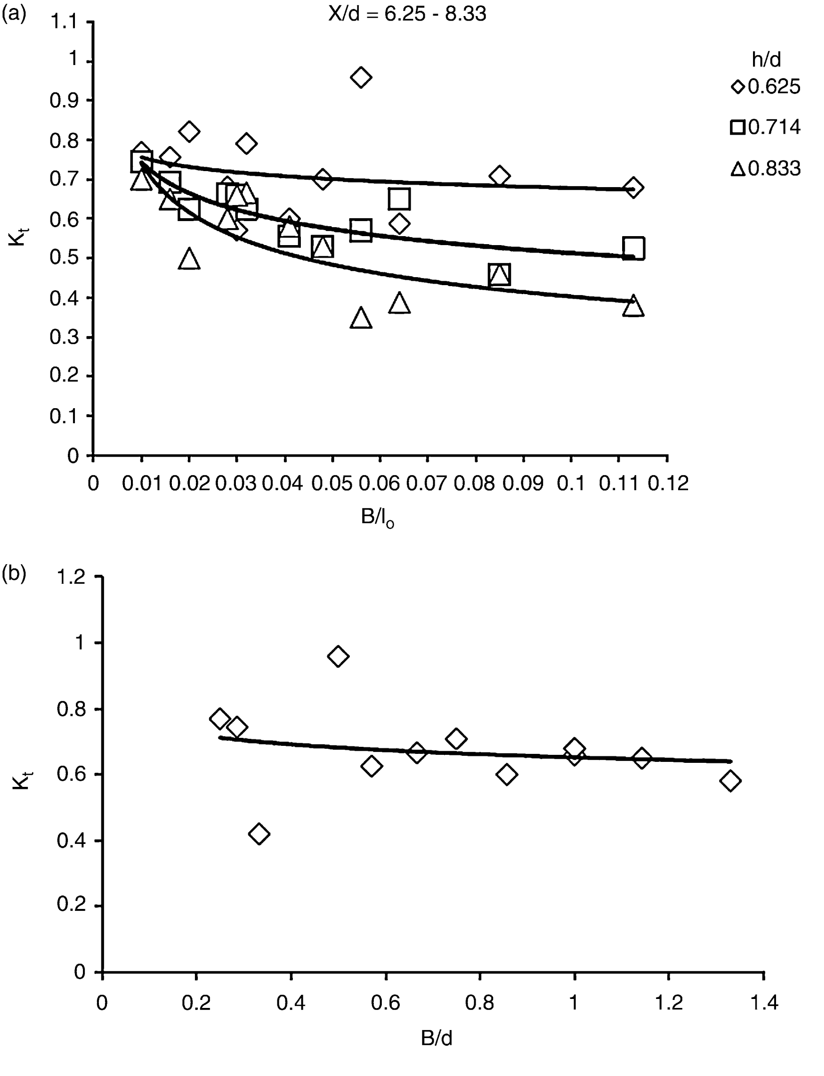

Fig. 10. Variation of transmission coefficient with relative crest width.

Table 3
Computations of reef crest width (B) in meters for varying B/L_0

Wave period T (s)	1.5	2.0	2.5
Deep water wave length L_0 (m)	3.51	6.24	9.75
$B/L_0=0.035$	0.123	0.218	0.341
$B/L_0=0.04$	0.14	0.25	0.39
$B/L_0=0.045$	0.158	0.281	0.439

Table 4
Computations of reef crest widths (B) in meters for varying B/d

Water depths d (m)	0.3	0.35	0.4
$B/d=0.6$	0.18	0.21	0.24
$B/d=0.65$	0.195	0.228	0.26
$B/d=0.75$	0.225	0.263	0.30

Fig. 10(b) shoes that decreases of maximum value of transmission coefficient K_t with an increase in relative reef crest width (B/d) For the optimum value of $B/d=0.6$–0.75 the K_t maximum values are 0.62–0.7.

9.2.7. Computation of optimum reef dimensions

For the optimum ratios of crest widths B/L_0 and B/d, the values of reef crest widths are calculated foe the test conditions. The relative reef crest heights h/d is 0.625–0.833, where the crest height (h) is 0.25 m, the structure slope $1V{:}2H$ and the crest width (B) is calculated as shown in Tables 3 and 4. From these tables the reef crest widths for $d=0.4$ m and $T=2$ s are comparable.

10. Conclusions

The physical model studies on stability of defenced breakwater with a seaward submerged reef as a protective structure were carried out in two-dimensional flume for regular waves, varying in wide range of heights, periods and water depths. Based on the above studies and analysis of the results, the following conclusions are arrived:

The submerged reef causes wave height attenuation, which may further be increased by increasing reef crest widths and length of the stilling basin. The optimum spacing between the structures is $X/d=6.25$–8.33. The optimum reef crest widths are $B/L_0=0.035$–0.05 and $B/d=0.6$–0.75. The run up and run down for the breakwater defenced by submerged reef are reduced up to 30 and 60%, respectively, compared to single breakwater not defenced by reef. The damage of the optimally defenced breakwater is reduced by 40–100% compared to a non-defenced (single) breakwater.

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