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STUDIES ON TANDEM BREAKWATER

FINAL REPORT

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Abstract

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 water area intended to protect. In modern times the breakwaters are constructed for the purpose of protecting vital installations on the coast and offshore, shoreline stabilization, forming an artificial harbour with a water area so protected from the ocean waves so as to provide safe accommodation for ships and for preventing the siltation of river mouths.

In the beginning, primitive reefs and dykes of gentle slopes were built with natural stones. Later, to save the material, steeper sloped structures with rubble mound, concrete block mound, rock fill over mound, caisson type etc. were tried. A mound breakwater with concrete blocks was first constructed at Alger, Algeria in 1830's. The traditional and most commonly used breakwaters are of non-overtopping rubble mound type. These structures consist of essentially a core of fine material like quarry run and protected by armour layer of large rocks or artificial concrete blocks. Rubble mound breakwaters are suitable for all types of foundations. They do not require highly skilled labour for construction.

From the economic point of view, breakwaters represent a significant portion of capital investment in the development of the port and would require regular maintenance to retain their effectiveness.

Breakwater's vulnerability to extreme events such as storms is a reality. We get ample number of such cases in literature e.g. failures of breakwaters in American, European and African countries between 1950 and 1980. One of the things engineers can do is to design a seaward protective structure to the breakwater, which will withstand, resist and manage such destructive extreme events. Further it is decided that a submerged reef is a highly optimised structure which is stable, economical and could be an effective option for protection of (main) breakwater as it breaks steep waves and attenuate them to a tolerable level.

In depth physical model study is required to design such a system. Hence, it is decided to take up the experimental work to study the influence of the seaward location and crest width of the submerged reef, as a protective structure, on the stability of the (main) breakwater and design it for maximum protection of the main breakwater.

Firstly, a 1:30 scale model of a conventional breakwater, of trapezoidal cross section with a uniform slope of 1V:2H, is constructed on the flat bed of the flume with primary stone armour of weight of 73.2gms for a design wave of 0.1m. Its crest width is 0.1m and height is 0.70m. This model is tested for wave characteristics exceeding that of the design wave i.e. 0.12m, 0.14m and 0.16m in depth of water of 0.3m, 0.35m and 0.4m. It is found that the breakwater is severely damaged and/or failed for waves of 0.14m and 0.16m height.

Secondly, a 1V:2H sloped and 0.25m high submerged reef with armour stone weight varying from 15gm to 35 gm is tested for stability and it is found that an armour of 30gm is stable.

In the third phase, waves of heights of 0.1m to 0.16m and periods 1.5sec to 2.5sec are generated in depth of water of 0.3m, 0.35m and 0.4m pass over the submerged reef constructed with armour stone weight of 30gms. The transmitted wave heights are recorded for every 1m, up to a distance of 8m on the leeside. It is observed that up to a distance of 4m on the leeside of the reef, waves are attenuated by about 50%, beyond which there is no significant increase in wave attenuation. Therefore, it is decided to locate the reef within a maximum distance of 4m seaward of the main breakwater for major experimental work involving protection to the main breakwater.

Next, a protected breakwater is designed with a stable trapezoidal submerged reef having a slope of 1V:2H and height (h) of 0.25m. It is constructed with homogeneous pile of stones of weight 30gms on the seaward side of (main) breakwater at a spacing of 1m, 2.5m and 4m and these models are tested for similar conditions as those for conventional breakwater. The reef crest width of 0.1m is fixed for a spacing of 1.0m between the structures. The reef crest width is 0.1m, 0.2m, 0.3m and 0.4m for spacing of 2.5m and it is 0.1m and 0.2m for spacing of 4m. It is generally found that the breakwater damage is significantly reduced as the spacing between the structures and the reef crest width increased. The submerged reef of crest width 0.3m and located at a seaward spacing of 2.5m, completely protects the inner (main) breakwater.

Finally, after the model tests a design for defenced breakwater is evolved. It consists of a 0.5m high, 1V:2H sloped trapezoidal conventional breakwater with primary armour stones of 36gm and a 0.25m high submerged reef of 1V:2H sloped trapezoidal cross section with an armour stone of weight 30gms and located at a seaward spacing of 2.5m. It is found that, this particular defenced breakwater is totally safe under all the test conditions considered for the model study.

The cost analysis of the prototype of the defenced breakwater is undertaken. It shows that the defenced breakwater is about 21.08% to 27.2% economical than the conventional rubble mound breakwater (depending on the site conditions), both designed for the same operating conditions.

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Notations

A	Area of Cross Section of breakwater
A_e	Area of erosion of breakwater
B	Crest width of submerged reef
B_n	Armour bulk
c	Coefficient of variation
C	Cotangent of reef
d	Water depth
d_b	Breaking depth
d_{n50}	Nominal diameter of the reef armour unit
D_{n50}	Nominal diameter of the breakwater armour unit
e	surface roughness
F	Free board
F_r	Froude Number
g	Acceleration due to gravity
h	Height of the reef
h_c	Height of damaged reef
H	Wave height
H_b	Breaker height
H_i	Incident wave height
H_o	Deep water wave height
H_s	Significant wave height
H_t	Transmitted wave height
H_{zd}	Zero damage wave height
K	Wave number
K'	Experimentally determined coefficient
K_D	Hudson's stability coefficient
K_D'	Nizam's stability coefficient
K_D^*	Ahren's stability coefficient
K_Δ	Layer coefficient
K_r	Reflection coefficient
K_t	Transmission Coefficient (H_t/H_i)

L	Wave length
L_o	Deep water wave length
n	Number of layers of armour stones in the thickness comprising the armour
N	Number of waves
N_Δ	Damage parameter
N_s	Hudson's stability number
N_s^*	Spectral stability number
p	Porosity of armour layer
P	Permeability coefficient
P_l	Piling of water behind reef
r	Thickness of armour layer
R	Thickness of primary layer
R_d	Run-down on armour slope
R_e	Reynold's number
R_u	Run-up on armour slope
S	Damage level
S_p	Local wave steepness
S_r	Specific gravity of armour units
T	Wave period
v	Flow velocity
W	Weight of individual armour unit
W_{50}	Mean weight of individual armour unit
WHA	Wave height attenuation $(1 - K_t)$
X	Spacing between main breakwater and submerged reef
X^1	Crest edge distance
α	Breakwater slope angle
Δ	Relative mass density of armour units
γ_r	Specific weight of armour unit
γ_w	Specific weight of water
ρ_b	Bulk density of armour units
ρ_r	Mass density of armour units
ρ_w	Mass density of water
μ	Coefficient of friction

ϕ	Angle of repose
ξ	Surf similarity parameter
υ	Kinematic viscosity of water
B/d	Relative reef crest width
B/L_o	Relative reef crest width
B/X^1	Crest width parameter
d/gT^2	Depth parameter
F/H	Relative reef submergence
h/d	Relative reef crest height
h/F	Relative reef crest height
h_c/h	Dimensionless reef damage
H/L	Wave steepness
H_t/H_i	Wave transmission coefficient
H_o/gT^2	Deepwater wave steepness parameter
R_d/H_o	Relative breakwater rundown
R_u/H_o	Relative breakwater runup
X/d	Relative spacing between breakwater and reef

Chapter 1
Introduction

1.1 GENERAL

People knew that they can get protection from winds and waves behind headlands since time immemorial. They tried the same logic, by dumping small stones in rivers and lakes to protect the banks, called revetments and large stones/boulders in seas to safeguard life and property on the leeside. Such large structures were called Breakwaters. Our epic Ramayana contains the description of Lord Rama having built a bridge between Rameshwaram in the southern Indian state of Tamilnadu and northern Sri Lanka, by dumping boulders into Palk Strait. The first proof of such massive man made structure surfaced in late 1970's, when Jacques Cousteau while searching for the lost city of Atlantis discovered underwater remains of Minoan Breakwater which was 4000 years old (Price 1978). However, the first documented history of breakwaters can be traced to ancient times of 2000 to 3000 B.C. in Egypt and Mycenae the ancient name for Greece (Tanimoto and Goda 1992). Old breakwaters built by Romans are surviving even after 2000 years (Franco 2001).

In modern times the breakwaters are constructed for the purpose of protecting vital installations on the coast and offshore, for shoreline stabilization, forming an artificial harbour with a water area so protected from the ocean waves so as to provide safe accommodation for ships and prevent the siltation of river mouths. Most of the breakwaters constructed in harbours function only to provide protection against waves but some of them serve a dual purpose by providing berthing facilities along side for ships. The design of the breakwaters shows wide variety and there is no best solution which is a panacea for all situations. The design depends upon local, national and social situation. As each country has its own unique conditions, we find strong and solid caissons, for resisting severe wave attack (Kaldenhoff 1996), so also the submerged breakwaters are used for coastal protection in Japan (Goda 1996), block work walls in developing countries suitable for varying depths and rock foundations (Kaldenhoff 1996), low composite breakwater fitting best to the foundation problems in most of the European countries (Kaldenhoff 1996), perched beach, detached breakwaters and groynes mostly made of submerged rock mounds in Italy and other European countries (Van de Graf 1996 and Franco 2001).

Till 1933, the breakwaters were designed based on laboratory experiments, experience and comparative basis. Later various investigators gave simple design formulae based on limited laboratory investigations (Poonawala 1993). These designs were either over safe or under designed. Hudson (1959) first gave the design formula based on structure slope, wave height, density etc., for armour weight based on wide ranging experimental study of stone armoured breakwaters subjected to regular waves. From the economic point of view, breakwaters represent a significant portion of capital investment in the development of the port, and would require regular maintenance to retain their effectiveness. Hence, the selection, alignment, design and construction of the breakwater must be carefully undertaken after examining the important governing parameters like predominant direction of approach of waves and winds, tidal range, degree of protection required, magnitude and direction of littoral drift, and the possible effect of these breakwaters on the shorelines etc. Though breakwaters are being constructed with increased confidence, new ideas and developments, like berm breakwaters, submerged structures, breakwaters with new geometries etc., are in the process of being tested for reducing the wave load on the structures. Major research activities are necessary to gain better knowledge of the physical background of their performance so that the final selection will be based on the performance consistent with cost.

In this chapter breakwaters are classified in the conventional way and new ones like pile breakwaters, stabiplage and defenced structures are explained.

1.2 CLASSIFICATION OF BREAKWATERS

Broadly breakwaters can be classified as:

1. Rubble Mound Breakwater
2. Vertical Wall Breakwater.
3. Composite Breakwater.
4. Other breakwaters

Rubble mound breakwaters can be further classified into:

1. Conventional breakwaters
2. Berm breakwaters
3. Offshore breakwaters

1.2.1 Conventional breakwaters

The traditional and most commonly used breakwaters are of rubble mound type which consists of one or two layers of heavier armour stones, one or two filter layers consisting of relatively smaller stones and a core of quarry run. The design of the breakwater section, which is normally of a trapezoidal shape which is also optimum, is described in great detail by US Army Corps of Engineers (1984, 2001). The design involves the use of Hudson equation usually supported by physical model tests. The conventional breakwaters are designed in such a way that no wave overtopping and no damage or little damage is allowed. This criterion necessitates the use of large concrete mass or heavy rock or artificial concrete units for armouring. A more economical breakwater can be a structure with smaller armour unit and/or where, profile development is allowed in order to reach a stable configuration e.g. berm breakwater, submerged breakwaters and reef breakwaters. These structures can be statically stable or dynamically stable.

Statically stable breakwaters are rubble mound structures where no or minor damage is allowed under design conditions. Damage is defined and quantified as the displacement of armour units or using area of erosion of the breakwater. The mass of individual armour units must be large enough to withstand the wave forces during design conditions. Caissons and traditionally designed breakwaters belong to this group of statically stable structures. Static stability is characterized by the design parameter 'damage' and is roughly classified by Van der Meer (1989) as

$$\frac{H}{\Delta * D} = 1 - 4 \qquad \text{.......(1.1)}$$

where, Δ is relative mass density; H is wave height; D is characteristic diameter of armour unit (rock or concrete) stone. e.g. Vertical breakwater.

Dynamically stable structures are rubble mound breakwaters where, profile development is accepted. Units are displaced by wave action until a stable profile is reached, where the transport capacity along the profile is reduced to a minimum. Units around the still water level are moving continuously during each run-up and run-down of the waves. But when the net transportation capacity becomes zero, the profile has reached an equilibrium condition e.g. Rubble mound breakwater. Dynamic stability is characterized by the design parameter 'profile' and is roughly be classified by (Van der Meer 1989) as

$$\frac{H}{\Delta * D} = 6 - 20 \quad \ldots\ldots(1.2)$$

e.g. Rubble mound breakwater.

Many rubble mound breakwaters failed in Europe, Africa and America between 1950 and 1982 due to extreme wave loads of cyclonic waves (Davidson and Markle 1970, Edge and Magoon 1979 and Zwamborn 1979).

1.2.2 Vertical or upright wall breakwaters

Vertical or upright wall breakwaters are of types such as structures of huge concrete blocks, gravity walls, concrete caissons, rock filled timber cribs and concrete or steel sheet pile walls. The selection of the type of breakwater would be primarily based on the wave climate in that area, depth of water, availability of construction materials and local labour, geotechnical nature of seabed, function of breakwater, technical know-how and contractor potential available. Wall breakwaters are regularly designed as structures subjected to forces causing failure in the following four ways:

1. By the shear bonding of bed joints or by sliding of one block against the other.
2. By over turning as a solid mass.
3. By the uplifting of horizontal layers.
4. By fracture of blocks.

The main disadvantage of vertical wall breakwater is that, it is one of the most delicate and a difficult task to repair/rehabilitate once it fails. Because, it cannot be readily repaired and consequences of failure are catastrophic. The construction of this type of breakwater requires high level of technical knowledge, heavy construction equipment and skilled labour leading to large expenditure. Many such structures failed in 1930's e.g. Catania, Italy and Algeria (Tanimoto and Goda 1992).

1.2.3 Mound with superstructure or composite breakwaters

Composite breakwaters are combination of rubble mound and vertical wall. These are used in locations where either the depth of water is large or there is a large tidal range and in situations, the quantity of rubble stone required to construct a breakwater to the full height would be too large. In such conditions, a composite breakwater is constructed which is a structure with rubble mound base and a super structure of vertical wall.

1.3 BERM BREAKWATER

Rubble mound breakwaters are dynamically stable structures where, profile development is accepted. Units are displaced by wave action until a profile is reached and a berm is formed close to the SWL. Many times it is advisable for engineering and practical reasons to utilize the berm in the breakwater slope e.g. weak foundation conditions, non availability of large armour stones etc (Hunt 1959, Lundgren and Jacobsen 1987 and Gadre et al. 1991). It is not difficult to visualize the great economy that may be obtained under certain conditions by using berm. Berm is not cure alls or panacea but, when used judiciously, they can save a great deal of material and still ensure a safe structure. Hunt (1959) and Nagai (1970) opine that berm breakwater has lower crest elevation but require the most space. It is possible to design a stable and an economical berm breakwater with smaller sized stones instead of a continuous single sloped conventional breakwater. As this section requires smaller armour stones, quarry output can be optimally used, require simple design and construction methods speedy construction and over all economy can be the advantages of berm breakwaters (Bruun and Johannesson 1976, Ergin et al. 1989 and Gadre et al. 1991). Berm breakwater dissipates much of the wave energy reducing wave reflection. The effect of the berm is most marked over an area along the breakwater and increasing the berm width, wave damping will increase (Jacobsen et al. 1999). Hunt (1959) proved that wave reflection can be reduced if the structure slope is less than $\sqrt{(H/T^2)}$ for design waves. This slope also increased dissipation of energy by heat generated by the turbulence of breaking wave. Hunt opines that, if berm width is equal to or greater than 0.2 times the local wave length, wave up rush is reduced. Under similar conditions a three slope berm section suffers up to 90% less damage (Van der Meer 1988, Ergin 1989 and Hegde et al. 2002) and requires less volume of armour stones due to reduced run up relative to 1:2 single slope section. Also the berm width may be of the value of one fourth of the local wave length (Ergin et al. 1989).

Dynamic stability of the berm breakwater is characterized by the design parameter 'profile' and is roughly be classified by (Van der Meer 1989) as

$$\frac{H}{\Delta * D} = 3 - 6 \qquad (1.3)$$

Gadre et al. (1991) concluded that the weight of the armour in berm breakwater could be 20% to 30% lower than uniformly sloped breakwater structure. Baird and Hall (1984) opine that utilizing quarry yield economically; construction of a berm breakwater can save up to 50% to

70% of the cost compared to a conventional breakwater. Franco (2001) mentions that for a potential site in Italy, comparison of costs showed that, a berm breakwater could be 25% to 40% cheaper against a conventional structure. Van Gent and Vis (1994) developed a new mathematical approach to simulate the reshaping of berm breakwaters using hydrodynamic forces. Rao et al. (2004) through model tests concluded that 30% reduction of armour weight compared to conventional design may not result in stable structure for all berm widths tested but with 20% reduction in armour weight and a berm of 0.6m the structure is stable.

1.4 OFFSHORE BREAKWATERS

Offshore breakwaters (sometimes also called detached breakwaters) are shore- parallel coastal structures, constructed with trapezoidal cross-section, sited at a certain distance away from shoreline. Offshore breakwaters have been successfully built in USA, Japan, Israel, Spain, Singapore and Sri Lanka for shore protection. Offshore breakwaters are one of the most professional engineering measures often chosen, for the sake of shaping shores, from a variety of coastal protection structures, implemented over the world for various sites and destinations. According to Gourlay (1981) as quoted by Pilarczyk and Zeidler (1996), these structures trap the sand from the adjoining beaches and offshore areas to accumulate behind it and are effective for waves approaching both normal and at an angle to it. Their primary function is to dissipate wave energy and rearrange waves and currents or otherwise:

1. To reduce wave energy at shoreline structures and/or to modify the wave climate,
2. Redistribute sediment transport patterns so as to improve beach levels and create desirable beach features such as cusps or tombolos, and
3. Provide toe support for beach slopes known as perched beach, which would otherwise be unstable.

The functioning of offshore breakwaters depend upon their:

1. Geometrical proportions,
2. Interaction with the external hydraulic forcing and
3. Local sediment transport and morphodynamics.

The offshore breakwaters may be differentiated as emerging, low crested, submerged and reef structures. Some of these are explained in following sections.

1.4.1 Low crested breakwater

These structures are built with their crest at or below still water level (SWL) so that there is overtopping of design wave. The main advantage is in its economics. When only partial protection is required from the waves on lee side of the structure is necessary, a low crested breakwater may be best suited and resulting in substantial savings. As these structures allow some portion of incident wave energy to pass over, the armour units experience less wave force. Hence, depending upon wave and structural parameters these armour units may be designed for relatively lighter weight. There are two types of low crest structures namely overtopping breakwaters and submerged breakwaters. Some examples of overtopping breakwaters are at Pilgrim Plymouth, UK, Diablo Canyon California, and Honolulu Reef Runway Hawai and Dana Point harbour and Yarra Bay, Australia . These overtopping breakwaters are designed on the basis of overtopping volume (Kobayashi and Wurjanto 1989 and Gerloni et al. 1992). To estimate this volume, many formulae are available in the literature (US Corps of Engineers 1984 and Bradbury and Allsop 1988). All these authors considered the structure with a crown wall. Shirlal et al. (2001) calibrated the Bradbury Formula for a simple structure without crown wall. Shirlal and Rao (2001) conducted the model studies to test the stability of overtopping breakwater. Shirlal and Rao (2002) tested a similar structure with a reduced armour weight. They reported that the breakwater is stable without any significant damage even after a reduction of 26% in the armour weight.

1.4.2 Submerged breakwater

This is a type of low crested breakwater. Submerged breakwaters are the most commonly built structures to break waves, dissipate energy of the wind generated waves and thereby to prevent their incidence on a water area intended to protect. From an economic point of view, breakwaters represent a significant portion of capital investment, and would require regular maintenance to retain their effectiveness. This encouraged the researchers to investigate the feasibility of achieving economy in construction without sacrificing the utility of these structures depending upon their specific use. There are several sites where complete tranquility is not required and only partial protection from waves is desirable e.g. small craft harbours, coastal protection, protection of tourist/recreation spots on beaches etc. For such sites designing a conventional non-overtopping breakwater is uneconomical, unnecessary and uncalled for. One of the solution under such situations could be a submerged breakwater which is economical and efficient e.g. Townsville Harbour, Australia, Rockley Beach, Barbados, Japanese coasts, Civitavecchia, Rome and Sagar Island, West Bengal, India (Ahrens 1984, Goda 1996, Smith et al. 1996, Franco 2001 and Nagendra Kumar 2001).

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. At the same time, they also offer resistance through friction and turbulence created by breakwater's interference in wave field causing maximum wave damping and energy dissipation, minimum wave reflection and bottom scour, and maximum sand trapping efficiency and is used for coastal protection (Homa and Sakou 1959, Homa and Horikawa 1961, Baba 1985, Sorensen 1987, Van de Graf 1996, Pilarczyk and Zeidler 1996 and Smith et al. 1996).

Various investigators have studied different parameters of the submerged breakwater and spelt different specifications for the parameters like height, slope, crest width, armour stone weight and location of the submerged breakwater (Johnson et al. 1951, Hunt 1959, Dick and Brebner 1968, Dattatri 1978, Khader and Rai 1980, Ramamurthy and Sharma 1989, Cornett et al. 1993, Smith et al. 1996, Pilarczyk and Zielder 1996, Rambabu and Mani 2001, 2002 and Twu et al. 2001). Most of the investigators opine that crest width, crest elevation, depth of submergence and location are the most important parameters which decide the optimum functioning of the structure and needs further study.

1.4.3 Reef breakwater

The reef is a low crested structure which is little more than a homogeneous pile of stones whose weight is sufficient to resist the wave attack (Ahrens 1984). A submerged reef breakwater is an optimized structure to highest degree. The reef is fundamentally built to break the steep waves and as this structure is submerged and porous wave reflection is small and wave energy damping and wave transmission are the significant characteristics. Ahrens (1984) first studied the stability, transmission and reflection characteristics of the submerged reef breakwater. The simplicity of the reef could be the significant factor in keeping down the construction cost and it could be an optimum structure for many situations in the cases of beach protection (Fullford 1985). Ahrens (1984, 1989), Gadre et al. (1992), Nizam and Yuwono (1996) and Pilarczyk and Zeidler (1996) discuss the stability of the reef structure and have given equations and graphs to design the armour unit weight. Armono and Hall (2002) experimentally investigated the wave transmission at submerged breakwaters made of hollow hemispherical shaped artificial reef. They found that, for relative height (h/d) greater than 0.7, the effect of the reef crest width is visible. Bierawaski and Maeno (2002) conducted small scale experiments to investigate the water pressure fluctuation in the wave field over submerged structures of which one was the reef and another an impermeable breakwater. Out of the two structures tested they found that the reef was safer. Harris (2003) discusses about performance of submerged artificial reef constructed in Dominican Republic for beach erosion control.

1.5 OTHER TYPES OF BREAKWATERS

The other breakwaters are special structures like pneumatic breakwaters, hydraulic breakwaters, mobile breakwaters, semi circular breakwaters, floating breakwaters, pile/pipe breakwater. These are explained briefly in the following sub-sections.

1.5.1 Pneumatic breakwaters

This is very unorthodox type of breakwater. The essence of the idea is that waves may be dispersed in the undulatory stage by the introduction of a continuous stream of air under pressure from a pipe lying along the seabed. This was designed by Philip Brasher in 1907. Prototype structures were installed successfully at Million Dollar Pier in Atlantic city in 1908 and El Segundo Pier in 1915 (Straub et al. 1957).

1.5.2 Hydraulic breakwaters

William and Weigel in 1963 invented the hydraulic breakwater which was nothing but a series of water jets issued by forcing water through a number of nozzles mounted on a pipe installed at the water surface perpendicular to the direction of incident waves. The jets create a surface current which results in breaking of the incident wave and its attenuation. It was found that this concept was not economically feasible and doubts were also cast over its efficacy (Chen and Weigel 1976).

1.5.3 Mobile breakwaters

They can be defined as structures or devices that combine the stability to appreciably reduce the height of ocean waves on its leeside with a degree of mobility sufficient to permit its ready transportation for considerable distance and its speedy installation when arrived at the site. Such a device would find application wherever protection from wave is necessary but for limited periods, as in offshore drilling operations or where an installation is required to be completed in a very short time, as in amphibious military operations.

1.5.4 Semi-circular breakwaters

The semi-circular breakwater comprises a semi-cylindrical caisson, which is generally placed over a rubble mound breakwater. It is made of pre-stressed concrete and divided into a series of elements. Both its weight and necessary ballast material can be significantly reduced (compared to a conventional caisson), since the forces acting on the structure are much smaller. If the caisson is made up of pre-fabricated elements, it can be constructed relatively easily on site with the aid of a pontoon crane. We can distinguish four types of these structures which differ in their wave reduction in front of and behind the structure, and in their permeability for water exchange between the open sea and the protected zone:

1. The massive type with non-permeable seaward and rear sides,
2. The front dissipating type with perforated seaward side,
3. The permeable type with perforated seaward and rear sides and
4. The rear dissipating type with perforated rear side.

They display a much-reduced risk of sliding, since the wave pressure is divided into a vertical and a horizontal component by the curved surface. The additionally present vertical components are transmitted to the foundation soil via curved sidewall of the structure and thus enhance the static stability. A structure slides along its bed or along another lower plane in the foundation soil, if the forces acting parallel to this plane in the direction of displacement are greater than the resisting forces. These breakwaters remain excellently suited under poor foundation soil conditions, since the geometrical form of the caisson precludes the danger of overturning: the soil reaction is always evenly distributed. The wave pressure acting on the surface of the breakwater is always directed to the centre of the caisson bed, since the pressure of a liquid always acts perpendicularly to the surface of an object. A structure overturns, if the resultant forces acting on the structure are directed at a point too far removed from the centre of the bed area. A structure in water floats, if the forces acting on a downward direction (weight of the structure) are smaller than the forces acting upwards (buoyancy).

As the submerged semicircular breakwater represents an enclosed, hollow structure, it is necessary to prevent it from floating. This is normally achieved through increasing the weight of the structure, though in this case it may already be sufficient to provide perforations in the base plate of the caisson (Priya et al. 2004). Within the frame of a research project sponsored by the DFG (German Research Community) together with the Federal Ministry for Economic Cooperation (BMZ)- represented by the Society for Technical Co-operation (GTZ)- research group is working together with staff of the Ocean Engineering Department of the Indian Institute of Technology Madras, Chennai, India to determine the wave pressure acting on the submerged semicircular breakwater under different load conditions. They found that transmission and reflection coefficient vary between 0.5 to 0.75 and 0.15 to 0.5 respectively. Knowledge of the wave pressure loading is necessary for economic realization of this very promising type of coastal protection structure.

1.5.5 Floating breakwaters

These are the floating barriers meant to attenuate the ocean waves and are considered as cost effective and suitable alternatives for the conventional type of breakwaters in minor port and harbour applications, where the tranquility requirements are low e.g. floating breakwaters at Friday Harbour, Washington (Adee 1976). Floating breakwaters have gained significance in the recent years because of their basic advantages such as flexibility, easy mobilisation, installation and retrieval. In addition, the units can be fabricated on land, towed to the required site, and installed along any desired alignment with ease. The design is based on the principle that energy of oscillating waves is mostly concentrated near the surface and that a wall is needed to reflect or dissipate this wave energy. This type of breakwater does not interfere with local underwater currents or with the sediment movement and in consequence they enjoy the complete freedom from scour.

Air/liquid filled cylindrical bags placed just below SWL with longitudinal axis parallel to the direction of wave propagation cause wave damping. Such a breakwater was installed off Dorset coast in 1948 (Chen and Weigel 1976). Brebner and Obuya (1968) as quoted by Chen and Weigel (1976) developed several types of rigid floating breakwaters. These concepts successfully damped the waves with transmission coefficients of 0.2 to 0.7. However, the width requirements of most of the existing floating breakwaters, to achieve a desired transmission coefficient of less than 0.5, makes the system very expensive (Mani 1993). Cage floating breakwaters (Y-type breakwaters) with the ratio of width to wave length equal to 0.14, are most effective as the transmission coefficients are less than 0.5, where as, for other type of floating breakwaters like rectangular this ratio would be higher than 0.3 (Murali and Mani 1996).

1.5.6 Pile breakwaters

These are breakwaters, which can attenuate the energy of the steeper waves partially, so that the coastal erosion due to wave energy concentration can be prevented effectively and aesthetic condition of the beaches will not be hampered as it will be located away from the shore line. The pipe breakwater has minimum interference with the littoral drift. Depending on the tranquility requirements of the water area intended for protection and prevailing littoral movement conditions, the pipe breakwater can be designed suitably. These type of structures are likely to be economical compared to other type of conventional breakwaters (Hutchinson and Raudkivi 1984). Hayashi et al. (1966) first experimented on pile/pipe breakwater. Hutchinson and Raudkivi (1984) designed the pipe breakwater for Halfmoon Bay Marina Auckland, Australia. To reduce the wave load on piles a sloping precast slab or even a submerged breakwater can be built in the front (Lundgren and Jacobsen 1987).

Research in this area was continued by Mani (1993) and Mani and Jayakumar (1995) who experimented on pile breakwaters. Pile breakwater was built in 1995 at Pelangi Beach Resort, Langkawi Malaysia, to protect the beach by encouraging natural restoration and to have open sea view with minimum obstruction (Reedijk 2004).

Rao (2000) and Rao et al. (2002, 2003a, 2003b, 2004) tested single and double row of solid and perforated pipe breakwaters and computed the reflection, transmission and loss coefficients. They concluded that two rows of piles perform better than single row of piles and wave transmission is less for perforated pile breakwater. They studied the performance of a single row of semi-submerged perforated pipe breakwater.

1.5.7 Stabiplage

This patented technology was developed recently by the company ESPACE PUR of France, with a view to offer a method of soft coastal protection which is integrated perfectly into the natural environment as well from an aesthetic point of view and the sedimentary exchanges which governs the dynamics and the geomorphology of this environment. Its technique, developed from a sound knowledge of coastal engineering, uses high quality material. This structure does not hamper natural movements in the sea and uses the dynamic factors in the environment to find a positive balance and a positive sedimentary budget. Stabiplage uses minimal space while offering an optimal solution.

The fundamental principle of this technology consists to collect, accumulate and maintain the sediments in place. The structure of Stabiplage is manufactured and established according to needs i.e. submerged or emerged, perpendicular or parallel to the shore, and with or with out anchoring depending on the sea and tidal range. Stabiplage is a reply to the various needs for protection or restoration of the coastal zone i.e. protection of dunes and the back dunes, stabilizing coastal zone, raising beach profile, controlling sand drift and prevention of silting up of harbour.

Stabiplage structure is made up of several materials. It is a geocomposite structure injected with sand. The works generally consist of a woven polyester shell and a filter not woven. The composite materials used are more adapted to the physical constraints of the natural environment than a simple geotextile, in particular for the abrasion resistance. The permeable structure optimally dissipates the energy of swell waves and the anchoring is done in such way so as to avoid undermining. Stabiplage construction which is long lasting uses natural material available and is fast, efficient and does not require any heavy machinery. The length of work may reach several tens of metres, even 200m and the structure can be located at a depth of water of up to 4m. For a work of approximately 2m high and 4.5m broad, the weight is close to 16ton per linear metre (Espace Pur 2003).

1.6 PROTECTED STRUCTURES

Submerged breakwaters can be used for protecting an already existing breakwater (Groeneveld et al. 1984). They were used as a protection to reclamation bund at Bharathi Dock at Chennai Port, India (Gadre et al. 1985). It was used as a rehabilitation structure for a damaged breakwater at Veraval Port Gujarat, India, to secure it from storm waves (Gadre et al. 1989).

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 could lower the crest by 1.5m, use relatively lighter armour stones and save about 1.0 million dollars.

Cornett et al. (1993) after conducting experimental investigation concluded that a submerged reef of relative height (h/d) greater than 0.6 protects the inner main breakwater and there may be an optimum location for reef of a given geometry of tandem breakwater.

Neelamani et al. (2002) experimentally investigated the hydraulic performance of a plane seawall defenced by a detached breakwater and found that wave pressure, reflections and run up decreased. They concluded that with defenced breakwater, the plane seawall can be designed with higher degree of confidence.

1.7 THE PRESENT STUDY

Breakwaters have been built through-out the centuries but their structural development as well as their design procedure is still under massive change. New ideas and developments are in the process of being tested regarding breakwater layout for reducing wave loads and failures. Breakwater design is increasingly influenced by environmental, social and aesthetical aspects and new type of structures are being proposed and built.

B

At the same time, breakwaters' vulnerability to extreme events such as storms is a reality. And we get ample number of such cases in literature. One of the things engineers can do is to

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design a protective structure in the front, which will withstand, resist and manage such destructive extreme events ~~at least~~ to some extent, and ~~at the same time~~, mitigate catastrophic damage of the inner main breakwater. The design of such protection to the breakwater is the research work selected for the present study.

A protective submerged reef, which is an optimized structure to highest degree, can be located at a certain distance seaward of the main breakwater. The wave breaking over reef causes great turbulence on lee side. They also offer resistance through friction and turbulence created by reef's interference in wave field causing maximum wave damping, energy dissipation and attenuation of waves those attack the inner main breakwater. The design of this type of combined structure is complex and requires detailed information on parameters such as water level changes, wave loads on armour units of both the structures', run up and run down on breakwater slope, damage, height of submerged structure, its crest width, its seaward location, wave transmission, armour weight etc. Therefore, more research is needed to understand their mechanism, performance and design such complex structures confidently.

1.8 FORMAT OF PRESENTATION

The present thesis is presented in ten chapters. Chapter 1 describes different types of breakwaters and classifies them in the conventional way and new ones like pile breakwaters, stabiplage, and defenced structures ~~are explained~~.

~~In Chapter~~ 2 reviews the development in breakwaters through the literature survey and explains the evolution of breakwaters through the history of breakwaters, improvements in design formulae, impact of governing parameters on the design. Development of modern breakwaters is explained through improvements in their design and construction. Also performance appraisal and damage of rubble mound structures are discussed. The design and performance of submerged breakwaters, reef structures and protected structures ~~are enumerated at the end.~~

~~Chapter~~ 3 ~~while presenting developments in physical model testing of breakwater, explains the~~ 3.7 & importance of dimensional analysis and need for planning the experiment. It discusses model scale selection, limitations of model testing, scale effects, uncertainty and describes the standard test procedure of testing of breakwater models.

4.

The back ground, formulation of the present research problem, necessity and relevance of the present study and scope of work ~~are explained in Chapter 4.~~

5.

Details of present experimental investigation including the details of the laboratory conditions, dimensional analysis, hydraulic modeling, objectives and experimental procedure ~~are explained in Chapter 5~~.

6.

~~Chapter 6 describes~~ the stability studies conducted on the models of conventional breakwater and submerged reef.

7

Chapters 7, 8 and 9 explain the investigation of performance of protected breakwaters with spacings of 1 m, 2.5m and 4m respectively.

Summary of the work is sited, overall conclusions are drawn, the design of the defenced breakwater is explained and suggestions for future work are listed in chapter 10.

References, cost analysis of conventional and defenced breakwater, list of publications and brief resume are presented in appendixes 1, 2, 3 and 4 respectively.

Chapter 2
Development of Breakwaters

2.1 GENERAL

As per US Army Corps of Engineers (2001) "A mound of randomly shaped and randomly placed stones protected with a cover layer of selected or specially shaped concrete armour units built to protect shore areas of harbour anchorage or basin from the effects of wave action is called Rubble Mound Breakwater". It is a heterogeneous assemblage of natural rubble, undressed stone blocks, rip rap, supplemented in many cases by artificial blocks of huge bulk and weight, the whole being deposited without any regard to bond or bedding. Rubble mound breakwaters are traditionally trapezoidal in cross section and are widely used. This is the simplest type and is constructed by tipping or dumping of stones into the sea till the heap or mound emerges out of the water and can be economically built. The mound being consolidated and its side slopes regulated by the action of waves. The quantity of rubble depends upon water depth, rise of tides and wave exposure. Rubble mound breakwaters are the structures which, man has been constructing since ancient times for protection from high waves.

This chapter discusses the history of breakwaters, improvements in design formulae, impact of governing parameters on the design as well as developments in their design, construction, performance appraisal and damage of rubble mound structures. Also the design and performance of submerged breakwaters, reef structures and defenced structures are enumerated.

2.2 EVOLUTION OF BREAKWATERS

The history of breakwaters can be traced to 2000 to 3000 B.C. in Egypt and Myceanae, old name of modern Greece (Tanimoto and Goda 1992). But the proof of construction of breakwaters was unearthed by Jacques Cousteau in late 1970's. He discovered the under water remains of Minoan breakwater, which was 4000 years old, while searching for lost city of Atlantis (Price 1978). Fig. 2.1 shows the flow of historical development of breakwaters (Tanimoto and Goda 1992). Beginning from primitive reef and dyke of natural stones, as shown in Fig. 2.1, various sequences are noticed in the structural development towards the increase in the stability against waves and saving the material and cost.

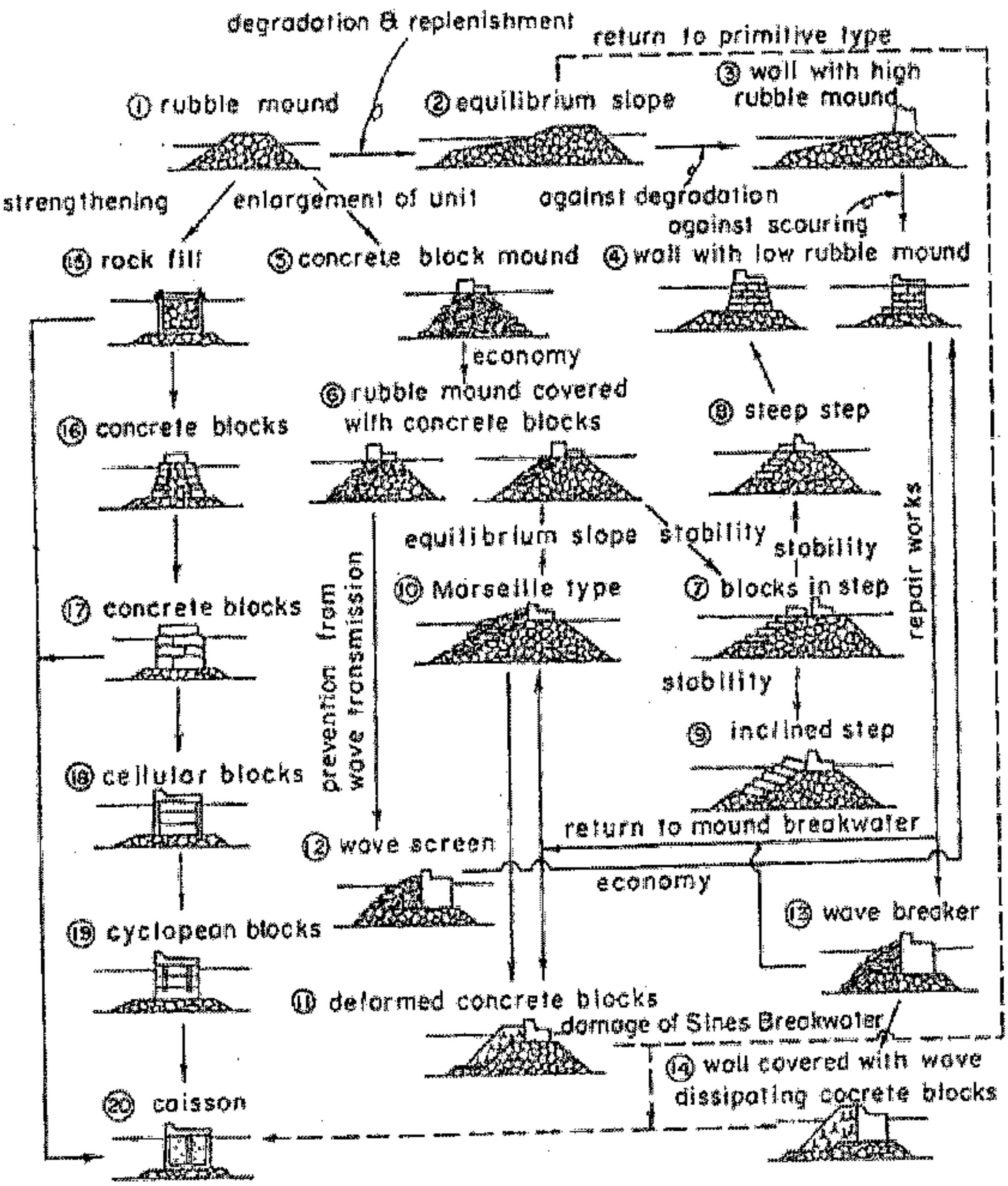

Fig. 2.1. Historical development of breakwaters

But the details of these structures have been discussed after the construction of the first modern breakwater at Cherbourg in France in 1784 – 1790 A. D (refer Fig. 2.2). This is a massive structure with a uniform slope of 1V: 3H which is 3.85Km long and has consumed 2.7Mm^3 of stones. Fig. 2.3 shows the first concrete mound breakwater built in Algeria in 1830, where, concrete units were simply dumped, which is called pell-mell placement. Then in 1879, the breakwater stability was improved by placing concrete blocks stepwise and Genoa breakwater (Italy) was constructed as shown in Fig. 2.4.

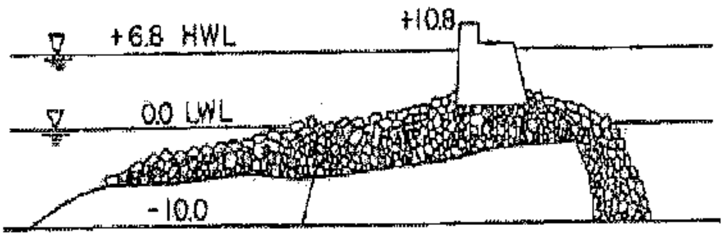

Fig. 2.2. Cross-section of Cherbourg breakwater

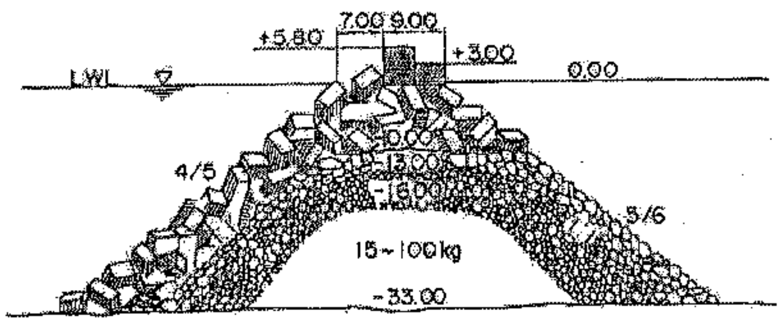

Fig. 2.3. Cross-section of Alger breakwater

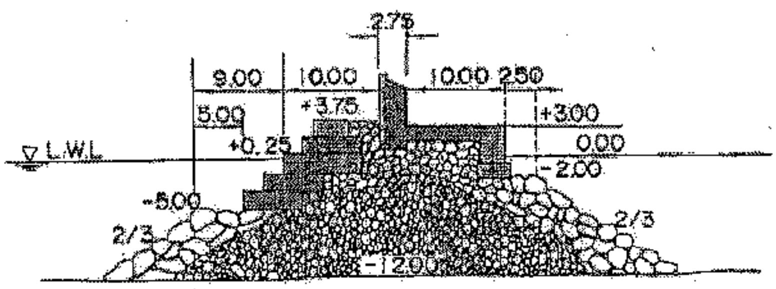

Fig. 2.4. Cross-section of Genoa breakwater

As blocks were overlapped more and more to increase stability, the structure eventually became a vertical wall breakwater. In this way a vertical breakwater with a low rubble mound was designed in Naples (Italy) in 1900 as shown in Fig. 2.5.

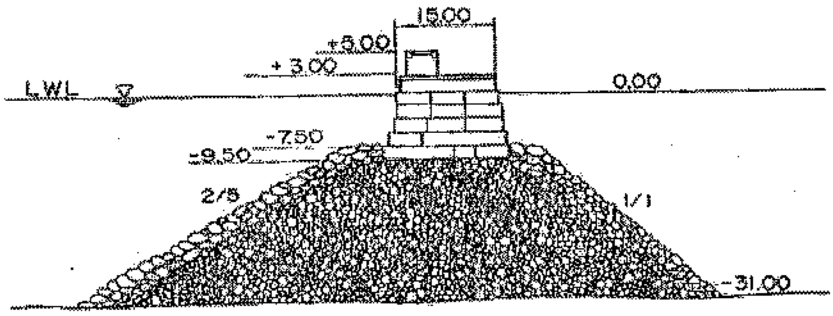

Fig. 2.5. Cross-section of Naples breakwater

2.3 CASE HISTORIES OF BREAKWATER FAILURES

In 1930's vertical wall breakwaters failed one after another, e.g. breakwater at Catania (Italy), as shown in Fig. 2.6. These were re-constructed as rubble mound breakwaters and then onwards such structures were predominantly adopted world wide as they were the most stable type of structures. In 1966 the dolosse block were first introduced by Merrifield and Zwamborn in the 10^{th} coastal Engineering conference at Tokyo as one which weighs $1/5^{th}$ to $1/6^{th}$ of a stone to resist the same wave height (Whillock and Price 1976).

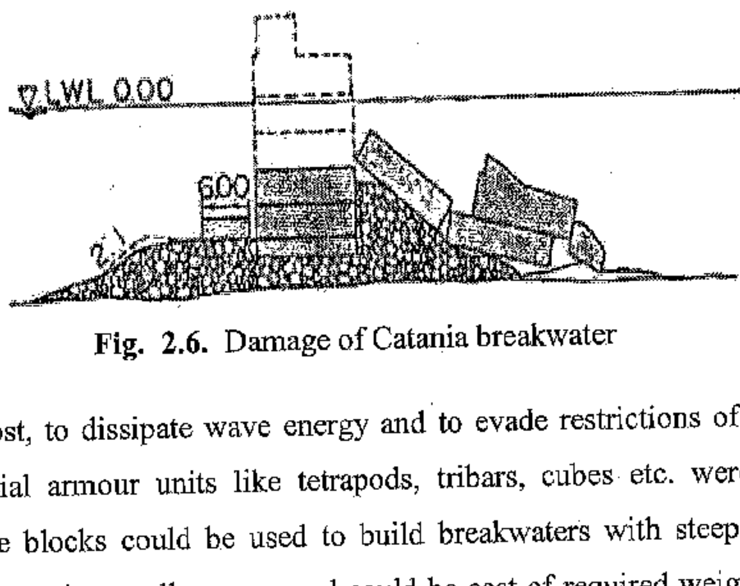

Fig. 2.6. Damage of Catania breakwater

To reduce the cost, to dissipate wave energy and to evade restrictions of patent soon other shapes of artificial armour units like tetrapods, tribars, cubes etc. were invented. These artificial concrete blocks could be used to build breakwaters with steeper slopes bringing hence, structures require smaller space and could be cast of required weight (Price 1978). In 1978, Sines breakwater (Portugal) which was a massive rubble mound structure built in deep waters, failed badly due to cyclonic waves. The other breakwaters which were damaged due to extreme waves in excess to design waves were at Kahulai, Hawaii, 1958, Gansbaai Cape Town, South Africa, 1970 and 1977, Torshavn 1972, Hirt Shab Harborn, 1973, Crescent city California, US 1974, Nawi Liurili Hawaii, 1976, Rosslyn Bay Queensland, 1976, Comeau, Qubec, Canada, 1976, Humbolt Jetty California, 1976, Azzawiya Libya, 1979, El Djedid Port of Arzew, 1980, Tipoli Libya, 1982 and Mogadishu Somaliya, 1982. Most of these structures had artificial concrete armour units. Most of these structures were badly damaged due high wave load, poor interlocking and insufficient structural strength of dolosse units and geotechnical instability, while, other breakwaters failed due to loss of toe support and morphological changes (Davidson and Marcle 1976, Edge and Magoon 1979; Zwamborn 1979, Linteringren and Chandrasekhar 1989, Sorensen and Jensen 1986 and Kaldenhoff 1996). These failures gave birth to safer breakwaters like berm breakwaters, submerged breakwaters and reef breakwaters.

2.4 MODERN RUBBLE MOUND BREAKWATERS

Rubble mound breakwaters are suitable for all types of foundations. They can be constructed up to 50m depth economically and can easily be repaired. They do not require skilled labour for construction. But for deep-water sites and at locations having large tidal ranges, the quantity of stones required may be large causing high expenditure. The traditional and most widely used are the rubble mound breakwaters.

The traditional rubble mound breakwaters generally have three layers of stones namely primary layer, secondary layer and core. Fig. 2.7 shows typical cross section of the rubble mound breakwater (Us Army Corps of Engineers 1984). Stone sizes are given by weight and the cross section shown in the figure illustrates typical dimensions for breakwaters exposed to waves on seaside and is intended to allow minimal wave transmission to the other (leeward) side. On exposed sites, the waves gradually drag down the mound, giving it a flat slope on the sea face. The disturbing action of the waves is most keenly felt around high water and low water levels. It is the region that the structure is most severely tested. Conventional breakwaters of this type are usually designed with crests elevated such that overtopping occurs only in the exceptional cases of very severe storms with long return periods. The primary layer is directly exposed to the severe wave action. So, it should be strong enough to

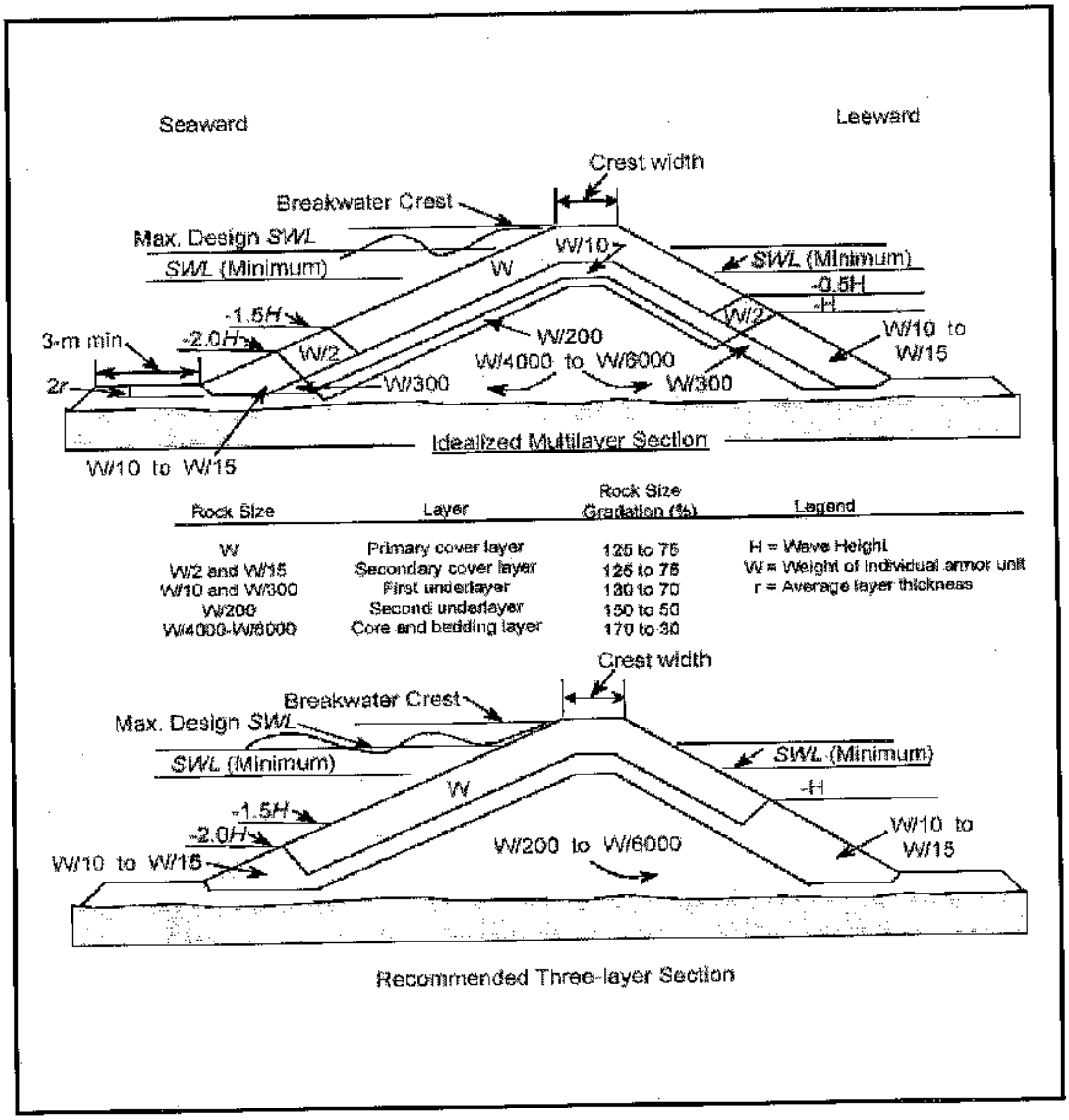

Fig. 2.7. Typical cross sections of non-overtopping rubble mound breakwater

resist the action of waves. Apart from dissipating the wave energy, it should act as protective cover layer for secondary layer and core. The core usually consists of quarry run. The main use of the core material lies in forming a huge mass or heap to act as a barrier. The core should be permeable enough to dissipate the wave energy by transmission conditions maintaining the necessary degree of tranquility conditions on lee side.

All the designers knew that to withstand high wave loads over the breakwater large stones are required. Till about 1933, there does not seem to be prevalent any recognized relationship between the height of waves, slope of breakwater, size and density of armour stones. The design of rubble mound structures was based only on experience and general knowledge of site conditions. The breakwaters were designed purely on the comparative basis i.e., considering the behaviour of a breakwater constructed earlier and using it as the basis for the design of next one without caring for the actual local conditions. Sometimes they were under designed or were over safe. Great attention was given to personal discretion and judgment, since those factors that might influence or standardize design were little understood. Understanding of the wave structure interaction is the primary requirement of design of any marine structure. However, Oumeraci (1984) as quoted by Hughes (1993) writes that in the case of rubble mound breakwaters, the wave structure interaction has not been completely explained and the stability of these structures cannot be modeled mathematically. The design of rubble mound breakwater has to be therefore semi empirical relying more on laboratory studies and field experience. This has made it difficult to arrive at a design, which is both, safe from the structural standpoint as well as being favourable from economic point of view. Although later several formulae were introduced, it was left to Hudson (1959) who through several model tests on stone armoured breakwater sections subjected to regular waves and evolved a simple design formula. Later, several mathematical models, software were developed for predicting sea states and derive wave characteristics, design and estimate the cost of the breakwater (Pranesh and Ramulu 1989, Haan 1991, Hughes 1993, Van Gent and Vis 1994, Cesar et al. 1995, Hegde 1996, Mai et al. 1999, Delft Hydraulics 2001, Mendez et al. 2001,Rambabu and Mani 2001, 2002 and Twu et al. 2001).

Subsequently, Hudson (1959) formula has been revised. Based on wave height, structure slope, and breaking or non-breaking waves, stability coefficient is selected, then, armour stone weight can be calculated using this formula. Van der Meer (1988) conducted wide ranging physical model tests on rock slopes using random waves. He improved Hudson formula by including storm duration, permeability, wave period and type of wave breaking and introduced new stability equations to calculate armour unit weight. Belfadhel et al. (1996) mentions about Sherbrooke University formula, Bertram formula and Koev formula which are field tested at some sites. However, there are difficulties in estimating some parameters in the new formulae given by Van der Meer and Belfadhel et al. have not been satisfactorily evaluated for large number of sites. Therefore, Hudson formula, which is simple, is very popular among the field engineers and is very widely used (Kudale and Dattatri 1994).

2.5 DESIGNING THE BREAKWATER ARMOUR UNIT WEIGHT

Many researchers through their studies proved that armour stability was a function of wave height, wave period, wave groupiness, wave breaking, water depth, structure slope, armour unit weight, armour gradation, storm duration and porosity. Each of them investigated structure stability with respect to different parameters (Hudson 1959, Ahrens 1970, Ergin and Pora 1971, Bruun and Gunbak 1976, Kondo et al. 1976, Johnson et al. 1978, Carver and Davidson 1982, Ahrens 1984, Timco et al. 1984, Van der Meer and Pilarczyk 1984, Gadre et al. 1985, Van der Meer 1988, Hall and Kao 1991, Poonawala et al. 1994 and Hegde and Samaga 1996). The design of rubble mound breakwater consists of determining the weight of the primary armour unit placed on a given slope to be stable under design wave conditions. The other dimensions such as secondary armour unit weight, core material weight, crest width etc., are determined in terms of the primary armour weight.

The various formulae presented by different authors (Poonawala 1993) are shown in Table. 2.1. These formulae give the required weight of primary armour stone as a function of slope angle; wave height, specific gravity of stone.

Several formulae have been developed over the past 40 years to evaluate and predict the stability of rubble mound structures against wind waves. Most of them have been derived from laboratory investigations using regular waves like the well-known Hudson (1959) formula. But, it is Hudson (1959) formula, which is still widely used around the world. These formulae are generally easy to use but do not consider all the parameters that influence the stability. With the advance of wave generation technology, another type of formulae have been developed based on irregular waves, thereby more accurately representing the prototype conditions (Thompson and Shuttler 1976; Van der Meer 1988). The important formulae are explained below.

Table. 2.1. Various stability formulae to compute armour unit weight

Author	Country	Formula
CASTRO	SPAIN	$W = \dfrac{0.704H^3\gamma_r}{(Cot\alpha+1)^2\Delta^3\sqrt{Cot\alpha-\dfrac{2}{\gamma_r}}}$
IRIBARREN	SPAIN	$W = \dfrac{K\mu^3H^3\gamma_r}{(\mu Cos\alpha - Sin\alpha)^3\ \Delta^3}$
EPSTEIN AND TYRREL	U.S.A	$W = \dfrac{K}{(\mu - Tan\alpha)^3}\dfrac{H^3\gamma_r}{\Delta^3}$
HUDSON	U.S.A	$W = \dfrac{\gamma_r H^3}{K_D Cot\alpha\Delta^3}$
HICKSON AND RODOLF	U.S.A	$W = \dfrac{0.0162H^2T\gamma_r}{Tan^3(45^\circ - \dfrac{\alpha}{2})\Delta^3}$
LARRAS	FRANCE	$W = \dfrac{K\left[\dfrac{2\pi H/L}{Sinh\dfrac{4\pi z}{L}}\right]H^3\gamma_r}{(Cos\alpha - Sin\alpha)^3\ \Delta^3}$
BEAUDEVIN	FRANCE	$KK_S\left[\dfrac{1}{Cot\alpha - 0.8} - 0.15\right]\dfrac{H^3\gamma_r}{\Delta^3}$
HEDAR	SWEEDAN	$W = \dfrac{KH^3\gamma_r}{(Cos\alpha - Sin\alpha)^3\Delta^3}$
SVEE	NORWAY	$W = \dfrac{KH^3\gamma_r}{Cos^3\alpha(\Delta)^3}$
SN-92-60	USSR	$W = \dfrac{KH^2L\gamma_r}{\sqrt{1+Cot^3\alpha}\ (\Delta)^3}$
RYBTCHEVSKY	USSR	$W = \dfrac{KH^2L\gamma_r}{Cos^3\alpha\ \Delta^3\sqrt{Cot^3\alpha}}$
METELICYNA	USSR	$W = \dfrac{KK_S H^3\gamma_r}{Cos^3\left(23^0 + \alpha\right)\Delta^3}$

2.5.1 Eqadro Castro formula

The first attempt to correlate the weight of armour stones with the wave height was done by the Spanish Engineer Eqadro Castro in 1933 (Carvalo and Vera Cruz 1960),

$$W = 0.704 \frac{H^3 \gamma_r}{(\cot\alpha + 1)^2 \Delta^3 (\cot\alpha - 2/\gamma_r)^{1/2}} \quad \text{.........(2.1)}$$

Where,

W is weight of the individual armour unit in metric tons, H is wave height in meters, S is specific gravity of armour units and α is angle of breakwater slope.

This formula was based on theoretical assumption that the destructive action of wave is proportional to its energy and the stability of the units under wave action is inversely proportional to a function of the angle of slope. This formula yielded small values for W and making the angle of repose dependent of specific gravity of the armour units. Harbour engineers rejected this formula, which, remained without practical application. Since then, the design of rubble mound breakwaters has been considerably rationalized. Extensive field and laboratory studies have been undertaken to correlate the wave characteristics and the size of the armour stone. The original purpose of laboratory investigation was to provide data from which an efficient design of rubble mound breakwater could be selected for different conditions of use and wave attack.

2.5.2 Irribarren formula

The very first rational approach of the modern age design of rubble mound breakwaters was given by a Spanish researcher called Irribarren who considered the principal resistant force as the one resulting from friction between the blocks. The formula suggested by Irribarren (Hudson 1953, 1959 and Poonawala 1993) was

$$W = \frac{K \gamma_r \mu^3 H^3}{\Delta^3 (\mu\cos\alpha - \sin\alpha)^3} \quad \text{.........(2.2)}$$

Where,

W is the weight of individual armour unit; K is coefficient depending on the armour units used (k = 15, for natural rock), H is wave height in meters, γ_r is specific weight of the armour units in tons per cubic meter, γ_f is the specific weight of the liquid in which the rock is submerged, α is angle measured from the horizontal, of the exposed breakwater slope and μ is the effective coefficient of friction between the armour units.

The basis of the Irribarren equation is that the hydrodynamic forces tending to replace the rock from the breakwater slope are proportional to wave height, area of the rock face on which the forces act and the specific weight of the liquid.

Experiments at the Waterway Experiment Station, Vicksberg, Washington, USA indicated that wave acting on the breakwater tends to do one of the following things:

1. The wave may completely break with the direction of jet approximately perpendicular to the slope.
2. It may be reflected forming a standing wave system seaward of the structure.
3. It may partially break with the resulting jet action, poorly defined with a portion of the wave energy reflected.

The effect of the variables such as voids in the rubble, shape of armour units and effective roughness which were not included in the formula, must have been included in the coefficient K. Irribarren assumed the coefficient K to be constant irrespective of the slope of the breakwater. Irribarren defined the coefficient of friction μ as the tangent of the angle of repose ϕ for quarry stone and took its value as unity. The tests at the Waterway Experiment Station, Vicksberg USA, showed that the value μ varied appreciably with the shape of armour unit and the method of placing these units in the cover layer. Hudson (1953, 1959) write that at that time it was concluded that the Irribarren formula could be used to correlate the test data and that it could be made sufficiently accurate for use in designing full-scale rubble mound breakwaters, if sufficient test data were available to evaluate the experimental coefficient K. But, tests conducted later showed that K couldn't be determined accurately unless comparative values of μ could be obtained for different shapes of armour units. Besides this Irribarren's formula does not represent the effects of impact forces on the armour units, nor does it consider stability of units subject to up rush (Mettam 1980).

2.5.3 Hudson formula

Hudson (1959) reviewed the Irribarren formula and found it to be inadequate. He conducted exhaustive laboratory studies and on the basis of experimental results, Hudson arrived at the new formula for the weight of the primary armour units. The stability formula was developed by equating the total drag and inertial force of regular waves to the relevant component of the weight of the individual units. According to Hudson, the weight of the individual armour unit is given by

$$W = \frac{\gamma_r H^3}{K_D \Delta^3 \cot\alpha} \quad \text{..........(2.3)}$$

Where,

γ_r is specific weight of amour units, H is design wave height, Δ is the relative mass density of armour and K_D is stability coefficient.

In deriving the stability formula Hudson neglected the friction between the armour units. He also assumed that the dynamic effect of the waves was to lift and roll the armour units from their initial positions on the breakwater slope.

Some of Hudson's conclusions are:

1. This formula could be used sufficiently accurately for no damage and no overtopping criteria. It means that a maximum of one percent of armour stones only can be displaced from their initial position when subjected to wave attack with the wave height equal to the design wave height.
2. K_D equal to 3.2 is adequate for quarry stone armour, which includes the safety factor also.
3. The stability of rubble bound breakwaters is not appreciably affected by the variation in relative depth (d/L) and relative wave height (H/L) for the conditions tested. However, when waves directly broke over the breakwater, these parameters are important.

The great merit of Hudson's dimensionless stability coefficient K_D lied in the fact that it was constant irrespective of the slopes. The safety factor should be determined from laboratory studies for each specific breakwater design (Carvalo and Vera-Cruz 1960). Booth (1960) felt that the K_D value given by Hudson could not be considered as final as such large scale model tests are required to negate the scale effects to conclusively arrive at the K_D value. Slichter (1960) opines that further research is required to explore relative effects of armour unit shape, porosity and reaction between units on armour stability. The stability coefficient K_D in the equation includes all the variables other than structure slope, wave height, and unit weight of the armour material and the fluid in which it is placed. The value of K_D depends primarily on the shape of the armour units, sharpness of the edges, surface texture, degree of interlocking obtained in placing the units and also whether the wave is breaking wave or a non-breaking wave. Hudson formula is empirically obtained from extensive tabulation of K_D values by scale model tests of non-overtopping conditions and for certain breakwater cross sections. Accepting the hydrodynamic phenomena, we cannot ignore the different flow characteristics occurring on the breakwater by assuming a constant stability coefficient K_D for the whole range of wave periods (Bruun and Gunbak 1976).

Hudson (1959) did not consider the influence of the variables such as wave period, wave steepness, wave breaking, up rush, down rush, duration of the storm, degree of overtopping, damage history, friction, porosity of armour units, permeability of core, armour gradation, structural strength and randomness of incident waves in his stability equation though he acknowledged the importance of the role of these critical variables played in the armour stability. Hudson's tests were for relatively deep waters (d/L = 0.063 to 0.5) and Hudson himself noted that wave period was important in shallow water when waves were directly breaking on the structure (Hughes 1993). It is also argued that the value of K_D is too simple to represent the stability of armour units and should only be accepted as a first approximation (Bruun and Gunbak 1976, Mettam 1980 and Pilarczyk and Zeidler 1996). Another view is that Hudson formula was designed for quarry stone armour and with in the scatter of his results there was probably a small dependence on the wave period and it was so small that he was justified in ignoring it. But for artificial concrete armour blocks wave period was also important (Price 1978). Baird and Hall (1984) listing a number of difficulties with the construction of breakwater designed with Hudson formula like large stone weights speed of construction etc., suggested berm breakwaters as an alternative which require smaller stone weights and simultaneously save about 50% to 70% of the construction cost. One more view is that Hudson's lab experiments covered a wide range of wave periods (0.88sec to 2.65sec) and most critical value of stability number obtained have been used to evaluate the stability coefficient K_D, thus incorporating the effect of the wave period (Maitra and Dattatri 1994). Hudson developed his formula for stone armour subjected to regular waves and does not include the effect of irregular waves and he did not specify what wave height of irregular wave train is to be used for the design.

As new types of armour units have been developed, their effectiveness has generally been measured by quoting the value of K_D in Hudson's formula in relation to their behaviour in model tests. However, Hudson's formula was originally developed to represent the behaviour of natural rock-type materials, which retain their stability under wave action principally by their own weight and without any significant interlocking with adjacent units (Mettam 1980). Development of new types of artificial armour units, which to varying degrees, do not behave in the same way as natural rock, has highlighted the limitations of Hudson's formula. The best known such unit is dolosse which relies principally upon its interlocking with a number of surrounding and underlying units to prevent displacement. For this reason dolosse units may be less than a third of the weight of the natural rocks required to resist similar wave load conditions. The Hudson K_D values for dollose and other interlocking units are convenient in comparing their weight- for - weight efficiency against more traditional forms of breakwater armour. This does not mean that Hudson formula represents their behaviour or such comparisons are valid as this equation was developed for natural rock (Mettam 1980). Further, the disadvantages of large interlocking blocks, like dollosse and influence of permeability of armour layer on its stability have led to development of new blocks with voids.

The great advantages of the Hudson's formula are its simplicity, and the wide range of armour units and configuration for which values of K_D have been derived (Pilarezyk and Zeidlar 1996). US Army Corps of Engineers (1973) recommended H_S and in 1984 the same agency recommended $H_{1/10}$ for the design and revised the K_D values e.g. the K_D value for breaking waves was revised from 3.5 to 2.0. The effect was that the armour stone weight increased by a factor of about 3.5 (Mettam 1980 and Pilarczyk and Zeidler 1996). However, the British code BS 6349 Part 7 recommends, Hudson formula with H_S and the same values of K_D for natural rock armour, but has introduced conservative K_D values for concrete armour (Mettam 1980). The values of K_D for different conditions are again revised in US Army Corps Engineers (2001) such as with respect to the types of armour units, their placement, section of the structure and types of waves. Due to the peculiar characteristics of damage to head section of the breakwater, different K_D values have to be used the head and trunk sections.

2.5.4. Van der Meer formula

Van der Meer (1988) established new stability formulae for rubble mound breakwaters subjected to random wave attack, with comprehensive model investigations at Delft Hydraulics, The Netherlands. Those include structure with a wide range of core, under layer permeability's and a wide range of wave conditions. Two formulae were derived for plunging and surging waves, respectively, which are now, known as the Van der Meer formulae. All the test results showed a clear difference between plunging and surging waves. Minimum stability was found for the transition from surging to plunging waves, referred to as collapsing waves. Using curve-fitting technique, Van der Meer (1988) specified two stability formulae, as follows:

For plunging waves,

$$\frac{H_s}{\Delta D_{n50}} = 6.2P^{0.18}\left(\frac{S}{\sqrt{N}}\right)^{0.2}\xi_m^{-0.5} \quad \text{.......(2.4)}$$

For surging waves,

$$\frac{H_s}{\Delta D_{n50}} = 1.0\,P^{-0.13}\left(\frac{S}{\sqrt{N}}\right)^{0.2}(\cot\alpha)^{0.5}\xi_m^{P} \quad \text{.......(2.5)}$$

Where,

Δ is the relative mass density of armour, S is the damage level, H_s is the significant wave height at the toe of the structure in meters, α is angle of the breakwater slope with horizontal, P is permeability coefficient, N is number of waves, ξ_m is the surf similarity parameter, D_{n50} $(= (W_{50}/\gamma_r)^{1/3})$ is nominal diameter of the armour unit in metres and W50 is mean weight of armour unit.

The transition from plunging to surging waves can be calculated with a critical value of

$$\xi_m = \left(6.2P^{0.31}\sqrt{\tan\alpha}\right)^{1/(P+0.5)} \quad \text{.......(2.6)}$$

For cot $\alpha \geq 4.0$ the transition from plunging to surging does not exist and for these slope angles only equation (2.4) should be used.

The variables in the Van der Meer formula and the possible range of each variable are listed in Table. 2.2.

Table. 2.2. Variables and their ranges used in Van der Meer formula

Variable	Expression	Range
Wave Height	$H/\Delta D_{n50}$	1.0 – 4.0
Wave Steepness	S_m	0.001 – 0.006
Surf similarity parameter	ξ_m	0.7 – 7.0
Damage as a function of no. of waves	$S/\sqrt{N}$	< 0.9
Breakwater Slope	Cot α	1.5 – 6.0
Grades of armour stones	D_{n85}/D_{n15}	1.0 – 2.5
Permeability of structure	P	0.1 – 0.6

These formulae included parameters such as wave period, storm duration, random wave conditions and a clearly defined damage level. The starting level of damage (i.e. S = 2 to 3), is equal to the definition of no damage in the Hudson's formula (2.3). The mass density value in the Van der Meer's equation is 2,000 to 3,100 Kg/m^3. Van der Meer allowed 5,000 to 7,000 waves during his investigations. The stability formulae specified by Van der Meer (1988) agree well with the results and are a substantial improvement in comparison with Hudson (1959) formula.

Rao and Raju (1992) conducted the sensitivity analysis of this formula for the wave heights ranging from 5m to 11m, for wave periods varying from 9sec to 11sec while slope increased from 1:2 to 1:5. They found that weight of the armour is directly proportional to wave period and varies inversely with permeability structure slope and density of the armour. They further report that Van der Meer formula gives smaller armour weight compared to that calculated using Hudson criteria with a K_D value of 3.5. And the percentage variation of weight of armour decreases with an increase in the wave heights.

2.5.5 About new design formulae

In developing new formulae, K_D has to be separated into a number of functions related to each element of the problem as a single K_D value of Hudson formula can't be expected to be a constant even for a particular armour unit under all circumstances. Due to scatter expected in the test results, it is necessary to have some sort of risk analysis. Impact of each of the critical parameters on the stability of the armour layer has to be studied individually and in combinations to take the worst scenario. Problems of breakwater stability in deep waters should be separately studied. Until new formulae are discovered, limitations of the present formulae should be recognized and they should not be used out of context. If this happens it is likely to be misleading and even dangerous (Mettam 1980).

Van der Meer formulae, though versatile, incorporate such parameters like permeability which is difficult to evaluate and quantify to a degree of accuracy and reliability required for design. This has limited the use of this new formula for the design of breakwaters. In spite of the use of this complex formula, the efforts required for a physical model testing has not been reduced (Maitra and Dattatri 1994).

2.5.6 Sherbrooke University formula

This formula was derived from the Hudson formula and based on regular wave test on both a steep (1V:1.5H) and flat slope (1V : 3H). The tolerated damage for a given design is clearly expressed by replacing the stability coefficient K_D used in the original form of the Hudson formula by the actual percentage of damage within the active zone (Belfadhel et al. 1996).

The formula expressed as,

$$W_{50} = 1.88\, \rho_r H_d^3 / (\Delta^3 \cot \alpha^{2.31} P_d^{0.6}) \quad \text{......(2.7)}$$

Where,

P_d is percentage damage within the active zone delimited by an elevation of one wave height (H_z) below and above water level, H_z being the wave height corresponding to the start of damage, H_d is design wave height. S is the ratio of eroded area A to square of the nominal diameter D_{n50} which is the number of cubic stones with a side of one nominal diameter D_{n50} eroded within a bandwidth of D_{n50},

$$P_d = (A \sin \alpha * 100) / (E_p\, 2H_z) \quad \text{......(2.8)}$$

Where,

A is eroded area and is given by $S*D_{n50}{}^2$ and E_p is riprap thickness

From the laboratory results, the start of damage percentage was characterized by the erosion of only few blocks and corresponding to about S = 1. For a riprap of a $2D_{n50}$ thickness, this criterion would correspond to a damage percentage P_d between 1.5% and 4.5% for slopes ranging from 1:3 to 1:1.5. Since the active zone becomes more extended for flat slopes, then for the same volume of eroded block, the intensity of damage is more important in case of steep slopes. Therefore, if 5% damage is considered, one should realise that the number of eroded blocks S, tolerated is greater for flat slopes than for steep slopes.

Nagaraj et al. (2002) carried out sensitivity analysis of Sherbrooke University formula for a breakwater of natural rock stones as armour units. The armour unit weights are calculated by varying the parameters like design wave height varying from 1.5m to 5.0m, slope of 1: 1.5 and 1:2 and percentage damage varying from 5 to 29.5. This is done by one by one while keeping the other parameters constant. Sensitivity coefficients are calculated taking the base value for the design wave height as 3.0m and base value for the stability coefficient as 15.50. However, from their study it is observed that armour unit weight is highly sensitive to design wave height, while it is sensitive to other two design parameters such as structure slope and percentage damage.

2.5.7 Bertram formula

Taylor first presented this formula at 13th Congress on Large Dams, 1973 (Belfadhel et al. 1996)

$$H_D = 0.388 W_{50}^{3/8} (b \cot \alpha)^{3/5} \tanh ((2\pi d)/L) a \qquad \text{.......(2.9)}$$

Where,

a and b are empirical coefficients, which depend on the riprap slope,

For $\cot \alpha = 5 - 10$, $a = 1/3$ and $b = 1$, For $\cot \alpha = 2 - 3$, $a = 1/5$ and $b = 0.75$,

H_D is design wave height (in ft) for zero damage, W_{50} is median stone mass (in lbs), d is water depth at the toe of slope, L is wave length.

Very little information is available on the development of this formula. It was used to design the riprap of Churchill Falls Complex and then, later was used for most of the riprap of the La Grande complex.

For deep water conditions ($d/L > 1/2$), the term $\tan (2\pi d/L)$ is equal to 1. For slopes ranging from 1:1.5: to 1:3:, the formula becomes,

$$H_D = 0.388\, W_{50}^{3/8}\, (0.75 \cot \alpha)^{3/5} \qquad \text{.......(2.10)}$$

2.5.8 Koev Formula

From a comparison point of view, this formula is interesting because it is derived from statistical analysis of 21 formulas developed using regular waves, representing most of the existing formulae considered for breakwater armour layer design. This formula is similar to the Hudson formula but includes the wave steepness (Belfadhel et al. 1996) as given below.

$W_{50} = (0.1421\ H_D^3\ \rho_r)/(\Delta^3 \cot \alpha^{1.5667}\ (H/L)^{0.3843})$ ....(2.11)

The formula is statistically valid for wave steepness varying between 0.04 and 0.1 and slope cot α between 1.11 and 2.0. According to Koev, as quoted by Belfadhel et al., it is more accurate than all other formulas considered in his analysis.

Nagaraj et al. (2002) carried out sensitivity analysis of Koev formula for a breakwater of natural rock stones as armour units. The armour unit weights are calculated by varying the parameters like design wave height varying from1.5m to 5.0m, slope of 1:1.5 and 1:2 and varying wave steepness. Sensitivity coefficients are calculated taking the base value for the design wave height as 3.0m. However, from their study it is observed that armour unit weight is relatively more sensitive to design wave height, compared to other two design parameters such as structure slope and wave steepness.

2.5.9 Melby and Hughes formula

Hudson (1959) and Van der Meer (1988) formulae are empirical and donot describe the variation in stability as waves shoal and become depth limited. Using the existing stability relationships, it is difficult to determine how variations in incident conditions, bottom bathymetry and structure slope will affect armour stability without performing extensive physical model studies.

Melby and Hughes (2003) used the maximum depth integrated wave momentum flux N_m to describe armour stability and proposed the following relationship:

$N_m = f\,(\Phi, \alpha, H_s/L, P, S, N, F/H_s)$ ....(2.12)

Where,

Φ is angle of repose of stone armour, α is structure slope, P is permeability of the structure, S is (A_e/D_{n50}^2), damage level of the breakwater, N is number of waves at mean period, F is freeboard.

Authors have also have given a graph between above parameters which indicates a nonlinear relationship.

2.6 STABILITY OF RUBBLE MOUND BREAKWATERS

The commonly used criterion for breakwater stability is given by the following expression.

$$N_s = \frac{H}{\Delta * D_{n50}} = (K_D \, Cot \, \alpha)^{1/3} \quad \text{.........(2.13)}$$

Where,

N_s is Hudson's stability number, H is wave height, Δ is relative mass density of armour (i.e. $(\rho_r - \rho_w)/\rho_w$), ρ_r and ρ_w are the mass densities of the armour units and water respectively. The design wave height that is commonly used in this formula is the significant wave height H_S.

The value of N_s is usually established by testing the breakwater until its damage reaches equilibrium. To attain this equilibrium, the time series of the sea state may have to be recycled many times. The number of times it is recycled is an important parameter to be taken into account. Unfortunately, this is not directly included in any stability formula. Van der Meer (1988) has taken this aspect into account indirectly by relating the damage attained after a certain number of waves to the damage anticipated after 5,000 waves.

2.6.1 Factors affecting stability

Many researchers have recognized that armour stability as a function of wave height, wave period, water depth, structure slope, armour weight, storm duration, porosity, armour gradation, wave breaker type and wave groupiness. They investigated the stability of a rubble mount breakwaters with respect to different parameters listed above (Ahrens 1970, Ergin and Pora 1971, Bruun and Gunbak 1976, Kondo et al. 1976, Johnson et al. 1978, Carver and Davidson 1982, Poonawala et al. 1994, Timco et al. 1984, Van der Meer and Pilarczyk 1984, Gadre et al. 1985, Hall and Kao 1991, Hegde and Samaga 1996). The above factors affecting the stability of breakwaters are classified in two groups and briefly explained below.

Environmental variables

1. Wave height
2. Wave period and duration of wave attack
3. Wave Breaking

4. Wave groupiness
5. Depth of water
6. Wave Steepness
7. Wave run-up and run down

Structural variables

1. Geometry of the Breakwater
2. Armour gradation
3. Thickness of the armour layer
4. Porosity and permeability of the structure
5. Seabed slope
6. Method of construction
7. Foundation

2.6.1.1 Wave height

This is the most important parameter affecting the stability of breakwater. Every investigator has recognized the importance of this parameter. Wave forces acting on armour are generally proportional to cube of wave height where other factors remain constant. Generally, damage of breakwater increases with an increase in wave height.

Rogan (1968) as quoted by Font (1970) concluded that effect of local wave storm is similar to that of periodic waves (i.e. regular waves) with height equal to storm H_s. Placing technique is important during initial damage but less reluctant for advanced damage (Font 1970).

Hudson's formula uses significant wave height (H_s) whereas some of the other formulae like Van der Meer's have been developed using random waves. Many researchers argue that H_S do not represent the affect of a train of random waves naturally occurring in sea. There are counter arguments too. Ergin and Pora (1971) argue that H_S in lab tests have same effect as irregular waves in nature with representative wave height of sea. Damage due to regular waves is same as that for irregular waves for design wave height and more than that of irregular wave train for waves higher than design wave height. Thus using regular waves for model tests would give conservative results.

Oullet (1972) studied effects of irregular wave trains on rubble mound breakwaters and concluded that effect of irregular wave train can be compared to that of periodic wave with height equal to H_S of wave spectrum and the effect depends upon the wave spectrum being used. The ratio of maximum wave height to H_S was an important parameter to be considered.

It is commonly assumed that rubble mound breakwaters are more stable to oblique wave attack since wave heights are reduced by refraction and armour units are effectively on a reduced slope (Whillock and Price 1976).

Carver and Davidson (1982) experimentally discovered that breakwater stability is minimal for $H/d > 0.9$ and wave steepness of 0.06 to 0.085.

The US Army Corps of Engineers (1984) recommends the use of $H_{1/10}$ (average of the highest one-tenth wave heights) instead of the H_S in the estimation of the stability number; but his parameter also suffers from the limitation that it cannot account for the total duration of the test.

Van der Meer (1986) concluded that wave spectrum shape has no influence on the stabilityof the breakwater. He also observed that for longer period waves (large surf similarity parameter ξ), more water percolates and flow through core. This reduces the wave forces and structure slope is stabilized.

Cesar et al. (1995) mentions that the description of a sea state just by its variance spectral density and duration is not enough to analyze the stability of rubble mound breakwaters, since it does not include an adequate characterization of the large waves in that sea state. Numerical simulation has been used by them to demonstrate that different time domain characteristics result from the simulation of a sea state solely defined by its spectrum and its duration. As a result, it was also possible to obtain drastically different damage values on a given breakwater. The results of numerical simulation were used to illustrate that a given sea state described only by its spectrum and by the duration of its time series can result in drastically different wave height statistics in physical models. Therefore, arguments were made for the use of a parameter that can adequately describe the wave height statistics actually realized in the model. This is followed by an illustration of hypothetical breakwater damage, justifying the use of a parameter that can account for the total number of storm cycles used in the model.

2.6.1.2 Wave period and duration of wave attack

Wave period is the parameter, which affects the stability indirectly, by affecting some other parameters like wave steepness, wave breaking characteristics, etc. The popularly used Hudson (1959) formula did not take into account the influence of wave period. The effect of wave period was first observed by Thompsen et al. (1972). They found that the stability was greatest at high wave periods and decreased as the wave period decreased. For $H_{zd}/L_0 > 0.03$ (where H_{zd} is zero damage wave height and L_0 is the deep water wave length,) the stability approached a limiting value and was relatively independent of wave period. As wave period increased, some damage of dollosse breakwater was observed for smaller wave height. This was because as wave period increased, wave surged on armour rather than break. This set high velocities over surface layer, causing damage to dollosse due to drag caused by surface flow (Whillock and Price 1976).

The number of waves or the storm duration brings the final damage that is likely to occur to the structure. For the given wave conditions, there is certain limit to the storm duration beyond which it will not have further influence in terms of damage. Then the structure is said to be in dynamic equilibrium. Thompsen et al. (1972) and Thompson and Shuttler (1976) studied the damage histories against number of waves. For smaller significant wave heights, the damage after 3,000 waves represented almost the total damage which was observed after 5,000 of higher waves. However, for higher waves it was nearly 80% of the total damage. Van der Meer and Pilarczyk (1984) observed that with random waves a stable profile is not found for less than 10,000 waves which is very different in the case of regular waves where equilibrium profile is established within 1,000 to 2,000 waves.

Shekappa (1994) measured the damage after every 1,000 waves up to 4,000 regular waves. He found that laboratory test with 2,000 waves would be adequate to give the final damage and the magnitude of change in breakwater profile after 2,000 waves was insignificant. Hegde (1996) concluded that the conventional breakwater will have a stable profile after about 3000 waves.

2.6.1.3 Wave breaking

The manner in which the waves break on the breakwater slope will influence the stability of the armour. When plunging breakwater hits the armour with great impact, this impact has only a small component which is tangential to the slope, so that it takes large wave to dislodge the armour units. The uprush following the plunge is turbulent, spongy mass of water with great quantity of entrained air and has little impact against stones. The reduction in wave height is seen to be more rapid for plunging breakers (due to instantaneous energy dissipation) compared to surging and spilling breakers (Narasimhan and Rao 1991). Also the return flow of uprush appears to lack energy to overturn the armour unit. In case of surging breakers, it is the down rush removed the armour stones. For the collapsing breaker, there is a strong uprush and down rush which dislodges the armour stones and breakwater has lowest stability (Ahrens 1970). That is the reason for which Battjes (1974) introduced surf similarity parameter ξ to study run up, reflection, wave breaking criterion and phase difference as:

$$\xi_b = \tan\alpha / \sqrt{(H_b/L_0)} \quad \text{.......(2.14)}$$

$$\xi_o = \tan\alpha / \sqrt{(H_0/L_0)} \quad \text{.......(2.15)}$$

Where,

ξ_b, ξ_o are the surf similarity parameters for breaker and offshore region respectively, α is angle of the breakwater slope with horizontal, H_b/L_0 is ratio of breaking wave height and deep water wave length, H_o/L_o is deep-water wave steepness.

Critical values of inshore surf similarity parameter, ξ_b for different types of breaker are listed below (Pilarczyk and Zeidler 1996)

Surging	$3.0 - 3.5 < \xi_b$;
Collapsing	$2.0 < \xi_b < 3.0 - 3.5$;
Plunging	$0.5 < \xi_b > 2.0$;
Spilling	$\xi_b < 0.5$.

The above figures seem to better apply to plane profiles and not natural barred ones.

Weisher and Byrne (1978) observed that breaking by plunging and non- plunging waves occurs as follows:

Surging $\xi_o > 3.3$

Plunging $0.5 < \xi_o < 3.3$

Spilling $\xi_o < 0.5$ for offshore and

Surging $\xi_b < 2$

Plunging $4 < \xi_b < 2$

Spilling $\xi_b < 0.4$ for inshore where, $H_d/d_b = 0.78$.

Later, Smith and Kraus (1991) as quoted by (Pilarczyk and Zeidler 1996) defined the following transition values:

Surging or Collapsing	$1.2 < \xi_b$
Plunging	$0.4 < \xi_b < 1.2$
Spilling	$\xi_b < 0.4$

Ahrens and Mc Cartnery (1975) investigated the run up and stability with the surf similiarity parameter ξ. From their experiments they found that the lowest stability was for the periods resulting in collapsing breaker. Bruun and Johannesson (1976) as well as Bruun and Gunbak (1976) studied the effect of wave period on stability and concluded that the lowest of down rush occurred with collapsing-plunging breakers when $2.0 < \xi < 3.0$.

In the range of $1.5 < \xi < 5.0$ (collapsing and surging breakers), the stones are expected to move down the slope and the rate of increase of damage shall be higher for higher ξ values. In the range of $\xi > 5.0$, where only surging breaker occur on the breakwater slope, the damage was expected to be fast and independent of ξ. Van der Meer (1988) has proposed different breakwater stability formula for plunging waves and surging waves.

2.6.1.4 Wave groupiness

Johnson et al. (1978) conducted experiments on dollosse armoured 1:1.5 sloped breakwater. They concluded that grouped wave trains caused more damage and stability is significantly affected by actual sequence of certain waves in a particular wave train, which tend to occur in wave groups. Van der Meer (1986) concluded that groupiness of waves have no influence on the stability.

2. 6.1.5 Water depth

Earlier to 1980, design of large breakwaters (in deep waters) relied mainly on extrapolating the existing state of knowledge, concerning design and construction of breakwater in relatively shallow waters, to deep waters. This naturally involved risk of enlarging uncertainties and inaccuracies inherent in breakwater design. When water depths are small, waves loose their energy due to bottom friction. Whereas, large water depths can sustain higher waves which can break close to the breakwater resulting in increased wave run-up and run down causing higher damage to the structure.

Weight of armour may be reduced by ¼ at depth below 1/3 of toe depth and by 3/4 below 2/3 depth (Palmer and Walker 1976).

A 2 km long breakwater was constructed with dollosse armour up to 42 tons in March 1972 at port of Sines, 160 km south of Lisbon, Portugal, to provide mooring facilities for 3,50,000 DWT ore carriers. This breakwater extended up to a depth of water of 50m. In February 1978, severe storm broke the dollosse units and caused extreme damage to this breakwater due to very large waves of 14m to 17m of periods varying from 18sec to 20sec (Zwamborn 1979).

Insufficient structural strength of heavy concrete armour combined with steep slope (1:1.33) caused failure of deepwater breakwater at Port Arjew El Djedid. This was initiated by compaction of armour due to vibration and due to run up and run down. A series of breakwater failures around American, European and African coasts from 1978 to 1982 (Edge and Magoon 1979 and Chandrashekhar et al. 1985) focused the attention on inadequacies of such an approach and resulted in series of analytical and experimental studies enhancing the knowledge in the field of deepwater (more than 15m to 20m of water depth) construction of breakwaters. Geotechnical stability of mound foundation is an important design consideration in deep water construction.

2. 6.1.6 Wave steepness

Wave steepness (H/L) and wave breaking characteristics depends on wave height, which in turn have a role in deciding the stability. The incident wave steepness has an important influence on the wave-breaking phenomenon. Steeper waves cause higher damage to conventional (single) breakwater.

2. 6.1.7 Wave run up and run down

Run-up is the vertical height reached by the up rushing wave on a slope above mean water level. Run-up is important in fixing the crest elevation for non-overtopping condition and mainly depends on the structure shape, roughness, porosity, water depth at the toe of the structure, bottom slope in front of the structure and incident wave characteristics (US Army Corps of Engineers 2001). Run-up is an important factor to be considered for stability of structure as run up level influences the inflow of water into the structure and also the elevation of water level within the structure core causing differential hydrostatic pressures.

Hunt (1959) experimentally proved that structure slope should be less than $\sqrt{(H/T^2)}$, then waves will definitely break on the structure, wave reflection will be less and energy dissipation increases by heat generated by turbulence of breaking wave. He feels that for waves breaking over structure, H/d has very little effect on wave run up. But water depth does have effect upon wave characteristics.

Le Mehaute (1976) indicated that the relative run-up (R/H) was a function of dimensionless parameters, tan α, the structure slope; $2\pi d/L$ or Kd, the relative depth; H/L, the wave steepness, d is the depth of water, L is the incident wave length and H is the incident wave height. Based on the theoretical investigations, Le Mehaute made the following observations:

1. For a given H/L and d/L the relative run-up had tendency to increase as the slope decreases up to the point where the waves begin to break.
2. The relative run-up of breaking waves decreased as the slope continued to decrease.
3. For non-breaking waves, the relative run-up increased with the wave steepness.
4. For breaking waves the relative run-up decreased with increased wave steepness.

Ouellet (1972) presented that the actual run-up depended on the water depth, the incident wave characteristics and the characteristics of the structure i.e., the shape and roughness of the armour unit.

$R_u/H = f\,(H/T^2,\ d/H,\ Cot\alpha,\ roughness\ of\ facing)$ ....(2.16)

Where,

R_u is run-up, H is wave height, T is wave period, d is depth of water, α is slope of the breakwater.

Thompsen et al. (1972) showed that, minimum stability of a 1V:2H sloped rubble mound breakwater occurred for $2 < \xi < 3$.

Bruun and Gunbak (1976) while explaining design principles for rubble mound structures write that when d/H > 3.0 effect of wave run up is negligible. As d/H is not always greater than 3.0, also run up and run down increase from spilling breakers towards plunging, collapsing and surging breakers and assume constant value for surging (ξ = 4 to 5). Permeability decreases wave run up. The effect increases as the slope angle decreases and R/H increases with increasing ξ. During rundown, the pressure forces and boundary resistance will all retard the rundown. Run up and rundown increase with an increase in ξ and remain constant for $\xi > 4.0$ and $d/H_o > 3.0$.

According to Bruun and Gunbak (1976), the failure of breakwater is caused by combinations of buoyancy, inertia and drag forces supported by the effect of hydrostatic pressure from the core. These forces all seem to reach their maximum value for lowest down rush which occurs at resonance. Resonance is the condition that occurs when rundown is in a low position and wave breaking takes place simultaneously and repeatedly at that location. Impact forces seem to maximise around resonance condition. Strong drag and inertia forces also occur on armour blocks due to high run up and rundown and accompanying large scale turbulence. At the same time mean water table elevation in the core rises due to high run up, causing an outward pressure on armour block. This effect will become even more significant when it is combined with the set down of mean water table outside breakwater. Resonance condition, maximum impact and suction forces seem to occur under breaking waves for $2 < \xi < 3.0$ and minimum stability of breakwater occurs at this stage. Therefore, the design wave for breakwater should be that which produces most dangerous resonance phenomena (Bruun and Gunbak 1976).
For breakwaters of slope 1:1.5, maximum run up could be up to 1.8 times depth of water (Palmer and Walker 1976).

Sollitt and Debok (1976) found that for a given wave period, T and steepness, shallow water produced more runup and the run up increased with increased wave steepness and wave period.

Armour damage due to displacement occur mainly due to rundown for steep slopes and run up is damaging for flatter stones i. e. $\cot\alpha > 3.5$ (Sorensen and Jensen 1986).

2.6.1.8 Geometry of breakwater

The overall geometry of the structure is decided based upon two layer thickness of each of the primary and secondary layer, crest elevation with respect to maximum water level, run up, overtopping, workability of section consistent with method and period of construction etc (Gadre et al. 1985).

Generally rubble mound breakwater is of trapezoidal shape which is also optimal (Pilarczyk and Zeidler 1996).

According to Harris (1996), the main parameters used to describe the general geometry of a breakwater include the structure slope, the height of the structure, water depth at the toe of the structure and the freeboard or depth of submergence of the structure, where, the freeboard is the difference between the height of a breakwater structure and the water depth at the seaward toe of the structure. The minimum crest width should be sufficient to accommodate three stones.

According to Caldwell (1954) as quoted by Hunt (1959), the thickness of the structure is important in wave energy absorption. He showed that the wave energy absorption by the structure continued up to a thickness of 2d, where d is the depth of water.

2.6.1.9 Armour gradation

The shape and surface texture of the armor units determine the interlocking, responsible for stability. The difference between armour unit size and filter material size is given by the ratio between D_{n50} (armour) and D_{n50} (filter). Thompson and Shuttler (1976) concluded that for a value of this ratio less than 4.5, no filter stone would be removed by erosion through armour layer and the stability of the armour layer was not dependent on this parameter. A wide grading of fine filter stones showed no influence on the stability of armour.

Gadre et al. (1985) conducted experimental investigation to study the influence of armour gradation stones and breakwater, recommend

1. D_{15} (above) / D_{85} (beneath) < 5
2. 4 < D_{15} (above) / D_{15} (beneath) < 20 and
3. D_{50} (above) / D_{50} (beneath) < 25

The above criteria be satisfied if secondary layer is two stone thick and its weight is W/10 to W/15 and core material should be W/200 to W/6000 where 'W' is the weight of primary armour.

If the fine material can't erode through the armour layer, the stability of the armour layer is not influenced by the grading and size of the filter layer (Van der Meer 1986, 1988).

2.6.1.10 Thickness of armour layer

Greater the thickness of armour layer better the stability of the structure i.e., failure would not occur in early stages but this thickness has vary little influence on the zero damage wave height. The thickness of the primary and secondary layer is determined by (Hudson 1959) as

$$r = n \times K_{\Delta} \times (W / \gamma_r)^{1/3} \qquad \text{.......(2.17)}$$

Where,

r is thickness of armour layer, n is number of layers of the armour units, K_{Δ} is layer coefficient (for rough quarry stones = 1.15), γ_r is specific weight of armour material, W is the weight of the armour unit.

Hall (1987) as reported by Hall and Kao (1991) concluded that thicker armour layer is very effective in reducing internal differential pressure in stable breakwater.

2.6.1.11 Porosity and permeability of structure

Hunt (1959) mentions that according to Caldwell (1954), permeability reduces wave run up. The wave energy absorbed by the structure varies linearly with porosity up to values of porosity of 50% at which time, 90% of wave energy would have been already absorbed.

Sollit and Cross (1970) proved through the mathematical model that, the transmission coefficient (K_t), through structure, decreases with decreasing permeability and reflection coefficient (K_r) decreases with increasing permeability.

As per Bruun and Johannesson (1976), the permeability/porosity of the structure has large influence on stability. This is also endorsed by Thompson and Shuttler (1976), Hedar (1986) and Van der Meer (1988). The permeability of the structure is influenced by the thickness of the armour layer, the sizes of the filter and core material. Permeability of core is an important parameter in describing the stability of the structure. If the core is made of finer particles, the hydrostatic pressure builds up in the core could be detrimental to the stability. Lower the core porosity greater will be the damage due to lower dissipation of wave energy and higher pore pressure in the core. At low porosity values, the damage was found to occur earlier i.e., number of waves required to cause a required damage level was small compared to that at high porosity.

The pervious breakwaters reflect fewer waves and reduce run up, but allow shoreward wave energy penetration. In rubble mound breakwater, transmission through the structure is primarily decided by core layer only, but wave reflection is very much influenced by armour. Pervious core layers installed in porous breakwaters effectively decrease transmitted wave heights and possibly reduce reflected wave height (Kondo et al. 1976).

For an impermeable core, there is considerable rocking, displacement and breakage of armour and in several places under layer is exposed during the model test. This is because the whole seaward face of breakwater was almost continually underwater during wave attack, as it takes larger time for water to drain off, than that for permeable core breakwater. In many instances of impermeable core, water from one wave would still be on the face of the breakwater, when next wave would hit. This may cause constant state of agitation of armour. Stability increases with core permeability but predicting permeability of core is difficult (Timco et al. 1984).

Bruun (1985) as quoted by Hall and Kao (1991) reported that for a stable breakwater with a range of 10% to 20% damage, for the core size varying by 500%, respective wave heights that produce same damage differ by approximately 15% and stability is not very sensitive to slight variation in porosity of core materials.

Water is temporarily stored in pores of structure, flows out with receding wave. This flow is capable of dislodging smaller stones from the interior of a poorly constructed structure. Then for a structure to remain intact and function effectively, the voids of armour layer should be enough to result in efficient dissipation of incident wave energy, but should be small enough to prevent the removal of stones from secondary layer through these voids. The filtering effect thus achieved in a multilayered structure, which is a result of porosity which in turn depends upon the gradation of armour, determines the eventual stability of the structure. Porosity of various components of breakwater is important in determining the intensity of inflow and net flow as well as elevation of water surface within breakwater etc. The phreatic surface within structure is higher or follow the wave action, depends upon whether the core is fine or coarse. Such different behaviour of phreatic surface causes different seepage forces which in turn causes armour units to behave differently leading to a varying degree of stability of breakwater (Hall and Kao 1991 and Hegde et al. 1997).

Hegde and Samaga (1996), through model tests concluded that Hudson formula is valid for core porosity of about 50%, core porosity remarkably affects the damage of breakwater. As porosity increases, stability too increases. They report that, armour weight is a function of fourth power of wave period and gave polynomial type of design formula.

2.6.1.12 Seabed slope

Sollit and Debok (1976) conducted experiment on rubble mound breakwater constructed on a flat as well as on a 1:12 sloped bottom. They found that for large scale (1:10 and 1:20) models, there was no difference in run up and run down of breakwaters constructed over flat and sloped bottom and represent prototype conditions. However, for small scale (1:100) models, they found that, run up is less by 20% and run down by 40% compared to large scale models.

Wave attenuation is more rapid on flatter slopes and less rapid on steeper slopes. Reduction in wave height seems to be more rapid for plunging breakers compared to surging and spilling breakers. The variations of particle velocities are seen to be rapid and continuous for flat slopes and gradual and discontinuous for steeper slopes (Narasimhan and Rao 1991).

Structures on steeper sea bed slopes, of the order of 1:10, suffer more damage compared to flatter seabed slopes, of the order of 1:100, particularly for higher wave heights, because, $H_b/d_b > 1.0$ for steeper slopes and larger waves break over the structures when seabed in steep sloped (Poonawala et al. 1994).

2.6.1.13 Method of Construction

Randomly placed armour stones or pell-mell construction would have less stability compared to those placed with special care. The method of placing units randomly, affects the stability and repeatability of tests, was proved by Thompson and Shuttler (1976) during their experiments. Also slope stability is high with stones placed with their longitudinal axis perpendicular to slope which is known as placed stones (Bruun and Johannesson 1976 and Sollitt and Debok 1976). Displacement of one armour unit does not lead to sudden massive failure; in fact the armour tends to heal unless it is grossly underweight. Keyed and fitted armour is several times more stable than loosely placed armour. Weight of armour may be reduced by ¼ at depth below 1/3 of toe depth and by 3/4 below 2/3 depth (Palmer and Walker 1976).

2.6.1.14 Foundation conditions

Breakwater over a weak foundation is less stable and has an undue settlement. If the underlying material is too weak to withstand the superstructure, wedges of material will be displaced which might result in uneven settlement due to which there will be instability of the rubble mound. PIANC (1976) suggested some methods to strengthen the foundation and improve the stability of rubble mounds like, removal of unsuitable material or replacing it by sand or gravel, increasing the compaction by injecting sand columns etc., Many researchers also suggest alternative structures like berm breakwater, pile breakwater, submerged structure etc.

2.7 DEVELOPMENTS IN CONSTRUCTION

Prior to 1980, design of large breakwaters (in deep waters i.e. more than 15m to 20m) relied mainly on extrapolating existing state of knowledge, concerning design and construction of breakwater in relatively shallow waters, to deep waters. And this naturally involved risk of enlarging uncertainties and inaccuracies inherent in breakwater design.

During construction of rubble mound breakwater with dollosse armour, care should be taken so as not to allow any breakage of armour units. As concentration of broken dollosse are more detrimental to stability of breakwater against wave attack than uniform or random breakage. This was proved from the test conducted on 1:45 scale model of Atlantic Generating Station (AGS) breakwater by U.S. Army Corps of Engineers, Waterway Experimentation Station Washington. This 1:2 sloped breakwater was 31.2m high, constructed with 37Ton to 62Ton dollosse in water depths of 9m to12m, protects two floating Nuclear Power Plants (Davidson and Markle 1976).

A rectangular parallelepiped shaped stone placed with its longitudinal axis perpendicular to structure slope is called placed stone construction technique. Placed stone breakwaters will have a densely packed surface and stability approaching that of dollosse armoured breakwater. This construction technique imparts a breakwater, higher stability than the structure with randomly placed stone armour (Sollitt and Debok 1976). The advantage of properly designed stone breakwater is that as heights build up, extent of damages occurs gradually. What would be acceptable damage to stone armoured breakwater would be serious damage for artificial unit armoured breakwater (Whillock and Price 1976).

A 2 km long breakwater with dollosse armour up to 42 tons was constructed in March 1972 at port of Sines, Portugal. This breakwater extended up to 50m deep water. In February 1978, severe storm caused extreme damage to this breakwater which was mainly due to large unbroken waves to the tune of 14m to 17m of period of 18 to 20sec which were much higher than design wave conditions, broke the dollosse armour around SWL (Zwamborn 1979).

Another series of breakwater failures, around American, European and African coasts from 1978 to 1982, focused the attention on inadequacies of such an approach and resulted in series of analytical and experimental studies enhancing the knowledge in the field of deepwater constructions (Edge and Magoon 1979 and Chandrashekhar et. al. 1985). Geotechnical stability of mound foundation is an important design consideration in deep water construction. Dynamic forces in the breakwater occur due to wave load, seismic loading, seepage forces and pore pressure. Such dynamic forces cause changes in internal stress yielding low effective inter-granular forces, internal deformation, breaking of interlocked armour units, settlement of crust and finally collapse of the structure. Large breakwaters (i.e. deep water breakwater) of steeper slope (1:1.5) are susceptible to geotechnical instability. Avoiding geotechnical risk of instability raises volume of breakwater significantly and increases construction cost. Edge and Magoon (1979) reviewed the damage and failure of many breakwaters in America, Canada, Europe, South Africa, Hawaii and found shortcomings in design and construction of rubble mound structures in deepwater and extreme environments.

Temperature stresses in artificial units lead to micro cracks. Effect of fatigue due to repeated loading reduces flexural tensile strength to 60% of its static value after 10^6 cycles which may be due to rocking behaviour of armour (Chandrasekhar et al. 1985).

Geotechnical stability of foundation and mound itself has received little attention in the past which later became an important design consideration due to breakwater construction in deepwater (Chandrasekhar et al. 1985). Constructional aspects of deep water breakwater are:

1. Requirement of large amount of quarry run for core,
2. Larger thickness of stone layer,
3. Larger armour units, big cranes, divers and GPS positioning are required, therefore costly (Chandrashekhar et al. 1985).

Geotechnical aspects of rubble mound breakwaters are important as they are concerned with mechanical behaviour of rock body and foundation subjected to seepage, pore pressure, and acceleration caused by waves and earthquakes (Barends 1986).

Hookway and Brinson (1986) list a number of steps undertaken for rapid construction of rubble mound breakwater at Ras Launf in Libya. These are:

1. Use of bottom dumping barges for core construction,
2. Optimizing quarry output,
3. Quality control,
4. Proper placement of armour, use of divers for proper armour placement and strict supervision.

Insufficient structural strength of heavy concrete armour combined with steep slope (1:1.33) caused failure of deepwater breakwater at Port Arjew El Djedid. This was initiated by compaction of armour due to vibration and due to run up and rundown. Damages of main breakwater in Tripoli, Libya and Somalia have been reported due to loss of reclamation material through core due to overtopping. Hence, using geotextiles in addition to gravel filters and using fine core fill over coarse quarry run can give stability (Sorensen and Jensen 1986). Securing the structure toe with heavy armour, digging a trench at toe and filling it with heavy armour stones; also filling voids with concrete were undertaken to increase armour stability of the breakwater at Azzawiya Refinery, Libya (Sorensen and Jensen 1986).

Hoedtden et al. (1987) speak of Bikon blocks which are nothing but concrete filled geotextile tubes which are positioned along breakwater slope from crest to toe and filled with liquid concrete. The geotextile tube is made up of high strength polyester fabric, so woven as to make concrete tight container which will not stretch unduly when filled. These blocks have a width to height ratio of 1.5 and length to width ratio of 2.0 and a void ratio of 5% to 10% between individual blocks with a tight interlocking fit of neighboring blocks adds greatly to the wave resistance. The construction requires less crest width and no crane is essential. Hence, its construction is 10% to 20% cheaper than conventional breakwater. Geotextile also may be spread over the seabed without recovering the soft top layer of weak soil. It improves over all factor of safety. It is used over the core to prevent sinking of secondary layer.

If there is weak soil on sea bed, ground improvement technique like dredging a trench and filling it with sand/gravel or small stones or replacing weak soil may be undertaken over which breakwater may be built e.g. breakwater built at Angra dos Reis, Brazil in1979 and breakwater constructed at Bintulu, Sarawak, Malaysia in 1980 (Lundgren and Jacobsen 1987). If there is inter bed sand layer and sufficient drainage, consolidation will take place during construction of breakwater. With this, the ground will acquire sufficient shear strength to resist weight of breakwater and wave load. Therefore, soil replacement is not required. But crest has to be raised to account for remaining settlement (Lundgren and Jacobsen 1987). Also a berm breakwater may be built with berm on one side or both sides and also to sometimes, to step their heights down at various distances from the breakwater. These may be constructed to avoid the risk of foundation failure due to weight of the breakwater in combination with wave forces. If the foundations are weak, pile breakwater or composite pile breakwater i.e. a pile breakwater with submerged breakwater in front may be built.

Baird and Hall (1984) list difficulties in construction with traditional approach which can be large size stones, wastage at quarry, costly quarry operation for large stones, difficulty in construction at deep water, speed of construction etc. Therefore, construction of berm breakwater with smaller size armour can be economical and may save a huge amount (50% to 70% in the construction cost) depending upon site conditions. It may be suitable for weak foundation condition too (Lundgren and Jacobsen 1987).

Construction of overtopping breakwater/submerged breakwater in small depths (say 2 to 3m) with stone/gravel filled bags/synthetic bags, concrete pipes, or chains of small concrete blocks totally weighing about 200kg may be easily constructed with small boats and 4 to 6 persons with locally available equipments very cheaply and economically (Kale and Gadre 1989).

In the construction of breakwater, core, secondary layer and primary layer accounts for about 60%, 20% and 20% of the costs respectively. For the core materials of 2 to 300Kg the workability is limited to a significant wave height of 1.2m, whose value is exceeded during approximately 20% of the time. Therefore, heavy stones may be used for core if waves are high and this can save some cost e.g. for a breakwater at Zubrugge and Belgium instead of a core of 2 Kg to 300Kg material, along with stones weighing 2 Kg to 300Kg, stones of weight 1000 Kg to 3000Kg were also used and these were stable up to 2.5m waves and thus saved 8% of the cost and 16% to 50% of time. During construction of seaward slope of breakwater, each armour block takes about the same time to place independent of its weight over a fairly wide range. Therefore, savings can be achieved by placing fewer and heavier concrete blocks (according to modified design) on a steeper slope than placing relatively lighter material stones over a greater slope depending upon site conditions. The disadvantage is that it may require bigger and expensive crane. Finally the cost of construction maintenance and repair should be optimized (Rietveld and Burcharth 1987).

Haan (1991) developed a computer program for determination of optimum design of rubble mound breakwaters using Van der Meer formula and quarry yield. The program evolved different alternatives and for each section, construction and maintenance costs are determined and total cost is minimized while quarry yield is economised.

John and Singh (1991) discuss about geotechnical analysis (slip circle) conducted on a breakwater designed for Seabird Project of Indian Navy at Karwar India. The analysis showed that a leeside slope of 1:15 was not stable and was modified to 1:2.

Hegde (1996) developed software called DORUB for deterministic optimum design of rubble mound breakwaters using Hudson and Van der Meer formulae.

Franco et al. (1996) discuss constructional aspects of three deep water rubble mound breakwaters in Morocco, Libya and Algeria in Northern Africa. These breakwaters were constructed with a slope of 1:1.33 to 1:1.5 and tetrapod armour of $10m^3$ to $22m^3$. These structures were built in a water depth of 16m to 28m and were exposed to severe waves of 7m to 25.9m. These heavy constructions brought following points into limelight:

1. Flexible design to accept modification at construction stage without extra cost.
2. Safe designs were adopted considering failure of breakwaters at Sines, Tripoli and Arzuo allowed some overtopping and heavy protection for head.
3. Optimum quarrying, improved, innovative planning and construction methods are necessary to reduce cost.
4. Improved wave forecasting for design and construction planning is vital.
5. Detailed risk analysis supported by probabilistic prediction of wave load is essential.
6. Long term feedback from contractors will go a long way in improving future construction.

A multiple pit breakwater was designed, by Mc Dougal et al. (1996) as quoted by Kaldenhoff (1996), to reduce wave loads. But this showed enormous diffraction and wave attenuation. Another approach to reduce wave loads and allow water exchange was concerned with permeable breakwaters of wave screens and semicircular caisson breakwaters for shallow water as in Miyazaki Port, Japan (Kaldenhoff 1996).

Physical model tests have proved that, replacement of 20% to 25% of artificial units (tetrapods) by rock armour in upper layer with their mean weight 20% higher than tetrapods is stable (Franco 2001).

2.8 APPRAISAL OF BREAKWATER PERFORMANCE

Belfadel et al. (1996) compared different new riprap stability equations on the basis of field tests and concluded that Hudson, Koev and Sherbroke University formulae gave realistic prediction of observed performance.

For shallow water condition, a semi circular caisson breakwater has performed well at Miyazaki Port in Japan (Kaldenhoff 1996).

The Permanent Technical Committee–II of PIANC in 1985 listed the following among the problems that need to be solved for rubble mound breakwaters: evaluation of safety and risk and asked for measurements of movements, forces and stresses in units of armour layers both in prototype and model, i.e. to monitor the behaviour in model and prototype and collect data regarding breakwater performance (Kaldenhoff 1996).

Franco (1996) writes that the long term feedback of performance of prototype breakwater from ports, contractors and Governments will go a long way in improving design of future structures.

Pirarczyk and Zeidler (1996) refer to model studies conducted by Ward and Ahrens (1992) to study the effect of dynamic rubble protection in front of vertical bulkhead which would reshape into an equilibrium profile under wave action. They observed that reflection was 27% to 50 % and this revetment dissipated 75% to 92% of incident wave energy.

In Italy, the breakwater design is increasingly influenced by environmental architectural and social issues, with greater attention to nature conservation, people recreation and sporting activities. The trend is towards adopting low crested and permeable structures with reduced use of visible concrete elements. Other important design aspects are safety, rapidity of construction and even the structure replaceability for a seasonal use. This typical approach, in this fast consuming modern age, is well in contrast with the aims of long term durability of old structures. Many new projects like Canmigione Marina, Ostia Harbour, Roma and other yacht harbours exhibit creative layout designs with a trend toward curvilinear plan shapes which have nautical, hydraulic, morphodynamic and aesthetical advantages such as reduced wave reflection at port entrance, safer navigational approach, improved water circulation in tideless sea and generally a more pleasant architectural image (Franco 2001).

2.9 DAMAGE OF BREAKWATERS

A meaningful definition for 'damage' is essential for qualitative analysis of stability of rubble mound structures. The traditional method of quantifying damage is physical counting of displaced units caused by wave action. In this process the question raised is that whether the stone dislodged from upper area, rolled and settled in a new position from where units get removed during early stages of damage should be counted or not. The above mentioned phenomenon affects the final profile of the damaged breakwater. But till now, there is no such definition for damage, which can explain the above-cited situation.

Iribarren (1953) as quoted by Hudson (1959), defined that a rubble mound reached its breaking level when the depth of damage on its main layer is equal to the length of the side of the equivalent cube i.e., exposing the armour units of secondary layer.

The damage parameter is defined by Hudson (1959) as the percentage of armour units displaced from the cover layer. The removal of up to 1% of the total number of armour units in the cover layer is considered as no damage. The definition of the design condition for 1% damage has advantage of being equivalent to no damage situation and thus numerically well defined (Font 1970). 1% to 5% damage is defined as 1st order damage (Phadnis 1985).

Rogan (1968) as quoted by Font (1970) pointed out that filter layer uncovering is simpler to observe and more significant than counting the number of displaced rocks.

The damage to the breakwater usually occurs in the area between SWL + wave height (H) and SWL - wave height (H) for both normal and oblique wave attack (Kreeke 1969 and Ouellet 1970). They defined the damage percentage as the number of displaced stones divided by the number of stones in the attacked area (SWL+H and SWL-H) times hundred. In computing the total number of stones in the attacked area a double layer armour stone was used. In the beginning, the damage in most of the cases occurs below SWL and for large waves the portion immediately above SWL is also strongly affected (Font 1970). Uncovering of filter layer in holes of diameter equal to two pieces (i. e. $2D_{n50}$) occurred for armour damage between 10% and 20% and total failure would follow for damage between 30% to 40% (Font, 1970). The main impact force can be expected at depths equal to half the design wave height below SWL, then secondary impact forces occur additionally around SWL and above (Fuhrboter et al. 1976).

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nless form as:

..................................(2.18)

....(2.19)

porosity of armour layer, J is total
width b, D is a representative linear
r unit, γ_r is unit weight of armour

ter, N_Δ as 'the number of D_{n50} sized
d by dividing the product of the bulk
tone'. The bulk density was used to

Table 2.3. Limits of damage level (S)

Cot α	Start of Damage	Failure (Filter layer visible)
1.5	2	8
2.0 - 3.0	2	8 - 12
4.0 - 5.0	2	17

Advantages of this representation are:

1. Since the limits are defined, it is easy to comprehend at what stage the damage is.
2. The stability number, N_s is widely used. Here the damage, S is related to N_s and hence other parameters studied with respect to N_s would also be compared with damage.

A physical description of the damage S was given as the number of squares with side D_{n50} that fitted into the erosion area. Another description was that the number of cubic stones with a side of D_{n50} eroded within a width of one D_{n50}. The actual number of stones eroded within this width of one D_{n50} could be more or less than S, depending on the porosity, the grading of the armour units and the shape of the stones. But generally, the actual number of stones eroded within a width of one D_{n50} was equal to 0.7 to 1.0 times the damage S (Pilarczyk and Zeidler 1996).

Van der Meer et al. (1996) studied breakwater toe and proposed the stability equation for toe as

$$\frac{H}{\Delta D_{n50}} = \{(0.24(d\text{-}h)/D_{n50}) + 1.6\}\, N_{od}^{\,0.15} \quad \text{..........(2.23)}$$

Where,

d is the depth of water, h is the height of the toe and N_{od} is the damage level measured as number of stones displaced out of a strip along longitudinal width of the structure with a width of one D_{n50}.

For zero damage N_{od} is 0.5, for some flattening N_{od} is 2.0 and for severe damage, N_{od} is 4.0. Authors concluded that wave steepness and toe width had no or only minor influence on stability of toe.

Melby and Mlakar (1997) state that typically the no-damage condition is represented by less than 2%, by count, of the stones displaced from the seaward face of the breakwater.

US Army Corps of Engineers (2001) defined zero damage wave height as the wave height corresponding to 0-5% damage. The percentage of damage is based on the volume of armour units displaced from active breakwater zone for a specific wave height. This zone may extend from middle of the breakwater crest down the seaward face to a depth equal to one zero-damage wave height, below the still water level.

2.9.1 Reasons for damage

Rogan (1968) as quoted by Font (1970) pointed out that, placing of armour, influences the damage of the breakwater. Further he observed that placing of armour is important during initial damage but is less relevant for advanced damage.

The dislocation of armour may be due to down rush of a surging breaker and the strong up rush and down rush of a collapsing breaker for which the breakwater has lowest stability (Ahrens 1972).

Resonance is the condition that occurs when rundown is in a low position and wave breaking takes place simultaneously and repeatedly at that location. They write that the resonance occurs for $2.0 < \xi < 3.0$. Impact forces due to wave breaking seems to maximise around resonance condition, strong drag and inertia forces also occur on the armour blocks due to high run up, rundown and accompanying large scale turbulence. At the same time the rise of water table in core due to high run up causes an outward pressure on the armour blocks. The suction forces occurring under a breaking wave due to interaction between the breaker forward velocity and random velocity generally maximise close to or a resonance condition. And dislocation of the armour stone of the breakwater is the result of existence of various forces mentioned above which may join in combination that cause maximum destructive forces (Bruun and Gunbak 1976).

Davidson and Markle (1976) through their experiments showed that concentrations of broken dollosse are more detrimental to stability of breakwater against wave attack than uniform or random breakage.

Displacement of one armour unit does not lead to sudden massive failure; in fact the armour tends to heel unless it is grossly under weight. Keyed and fitted armour is several times more stable than loosely placed armour. The weight of loosely placed may be two times that of a well placed stone for same stability (Palmer and Walker 1976).

Sollitt and Debok (1976) experimentally proved that placed stones armour were more stable than randomly placed stone armour. This technique has been successfully used since 1950's along northwest coast of U.S.A. The single layer of armour units placed in this manner provides a densely packed surface with stability approaching that of dollosse units.

Johnson et al. (1978) concluded that grouped wave trains caused more damage and breakwater stability appears to be affected by actual sequence of certain waves in a particular wave train, which tend to occur in wave groups.

Experiments conducted by Whillock and Price (1976) showed that what could be acceptable damage to quarry stone would be serious damage for dollosse, tetrapods occupying an intermediate position.

The review of damages of most of the rubble mound structures which were subjected to extreme damage well within life of the structure and which without major repairs would have subsequently been a total failure showed that they were built of artificial armour units. These failures showed the shortcomings in design and construction technology of rubble mound structures in deep water and in extreme environments (Edge and Magoon 1979). These breakwaters were at Rosslyn Bay, Queensland (1976), Qubec, Canada (1976), Bilbao, Spain

(1979), Cresent City, California (1960), Gausbai, Cape town South Africa (1970, 1977), Kahului, Hawaii (1958), Nawiliurle, Hawaii (1974-76) and Sines, Portugal (1978).

A 2 km long breakwater, with dollosse armour up to 42 tons, was constructed in March 1972 at port of Sines, Portugal. This breakwater extended up to a depth of 50m in water. In February 1978, severe storm caused extreme damage to this breakwater (Zwamborn 1979) due to:

1. Large waves to the tune of 14m to 17m which were much higher than design
2. Wave of 11m which reached the breakwater without breaking due to deepwater.
3. 42 ton dollosse armour broke around SWL.
4. Waves of longer period of 18 to 20sec lashed the structure and local refraction increased the wave heights further by 20%. This caused additional breakage of dollosse, causing collapse of super structure.
5. Inadequate structural strength of dollosse (Maitra and Dattatri 1994).

This was one of the greatest shocks to the design engineers. Since then, geotechnical stability of mound and foundation under static and dynamic conditions has become an important design consideration with increased construction of breakwaters in deepwater. Also earlier design of breakwater was based on shallow water considerations which were later extended to deepwater construction too. This naturally involved the risk of enlarging uncertainties as well as inaccuracies inherent in breakwater design.

Another series of failure of breakwaters in American, European and African countries from 1978 to 1982 focused the attention on inadequacies of such approach and resulted in series of analytical and experimental studies enhancing knowledge in deepwater constructions (15m to 20m). This catastrophe opened the eyes of the designers and planners to strengthening the structure with more reliable model tests, considering realistic wave condition and collection of prototype data regarding structural strength of artificial armour units, need for standardising the techniques and test procedures and encouraged engineers to think of inadequacies in the design of breakwater in deep waters, design of techniques of repair, rebuilding, rehabilitation and construction (Zwamborn 1979 and Chandrasekhar et al. 1985).

Considering various failures of breakwaters, Mol et al. (1983) concludes that the damages were caused by combination of aspects like wave climate, structural strength of armour, geotechnical stability and constructability and insufficient safety margins and inadequate traditional design approach.

For dollosse armour, as wave period increases, same damage was observed for smaller wave heights. Because, as period increased, wave surged rather than break and set up high drag due to increased surface flow. Authors suspect that, dollosse units are weak to resist this drag force. The advantage of a properly designed tipped stone breakwater is that as wave heights build up, the extent of damage will occur gradually. This is not the case with dollosse. Approaching the state of serious damage, small changes in wave height will produce large changes in damage and considerable rebuilding rather than repair is necessary for such a breakwater. Investigation of failures of large breakwaters brought into several issues, shown below, which were hitherto sidelined or unknown (Chandrasekhar et al. 1985): .

1. Rocking and collision of armour units causing very high impact loads, loads while casting, transport, placing, overlying armour etc.
2. Strength of concrete armour units
3. Quality of stones, quarry operations
4. Additional thickness of stone layer
5. Construction of core and
6. Armour placement.

A rubble mound breakwater can be damaged in several ways depending upon its configuration, the wave conditions and water level during the storm. The following are the main identified modes of damage (Sorensen and Jensen 1986):

1. Sliding/settlement of seaward face due to scouring or morphological changes of seabed or the toe of breakwater e.g. breakwater of Hirtshals harbour, Denmark (1973), due to change in seabed morphology and failure of Torshavn breakwater (1972) due to toe erosion.
2. Sliding of seaward armour layer due to unstable berm.
3. Damage due to geotechnical instability of subsoil or insufficient bearing capacity leading to serious settlements.
4. In special cases of steep slopes, wave forces may cause a sliding of whole armour layer for a single large wave rundown.

5. Damage to crown wall or superstructure due to wave force. This is an integrated problem occurring simultaneously with damage to armour in front and erosion of base of superstructure.
6. Damage to crest and rear side armour under excessive overtopping due to high waves and/or high water level.
7. Breaking of armour units due to large dynamic contact forces arising as a result of rocking or wave breaking.
8. Damage due to reclamation behind breakwater due to excessive transmission or overtopping.
9. Damage due to impact of floating objects like vessels, ice etc.
10. Poor interlocking due to improper placement of artificial armour unit leading to displacement of armour units e. g. Azzawiya breakwater, Libya (1979).

Vaidya (1989) identifies the most important failure mechanisms of breakwater as:

1. Breakwater settlement due to compressible sub soils and other layer
2. Failure of crest element
3. Sliding of crest element
4. Subsidence of crest element
5. Instability of armour
6. Loss of support to armour.

The major cause for this type of failure is extreme wave action.

Van der Meer and Heydra (1991) tried to analyze the rocking armour units, their number, location, impact velocity, accelerations etc., as these artificial armour units made of un-reinforced concrete caused extensive damage to breakwaters in European, African, Mediterranean and Atlantic Ocean countries.

2.10 IMPROVEMENTS IN BREAKWATER DESIGN

In 1966, Merrifield and Zwamborn described the dollosse blocks as those units whose design weight was 1/5 to 1/6 that of a natural stone to resist the same wave height. These artificial concrete blocks could be used to build breakwater with steep slopes hence required smaller space and could be cast of required weight. For stability these blocks should be placed at steep slope so that their interlocking capability could be utilized to the full extent (Whillock and Price 1976).

Permeability of breakwater reduces wave run up and wave energy absorbed is linearly varying with porosity up to 50%, at which 90% of energy is absorbed (Hunt 1959). Pervious core breakwaters can also reduce wave transmission and reflection (Kondo 1976). Research on permeable core breakwaters showed that these are stable and stability increased with core permeability (Timco et al. 1984 and Hegde and Samaga 1996).

Font (1970) proposed that, the damage function, rather than no damage or total failure condition alone be given to the design engineer as economic considerations in one hand and factor of safety in the other leading to design damage. These damage functions include storm and swell duration, placing technique, armour density and damage distribution and location.

Ahrens (1972) concluded that type of breakers could influence the stability of breakwater structure and collapsing breakers pose the highest damage. Hence, recommended that appropriate care should be taken against such cases.

All this created a new enthusiasm to research in related areas and investigators came up with new methods and/or alternatives. Researchers found that proper collection of data regarding wave climate and soil conditions prevailing at the site, behaviour of prototype structure, geomorphology of soil and quarrying operations would be immensely important inputs for improving the planning, design and construction of future breakwaters so that the whole project implementations could be optimized (Ouellet 1972).

Bruun and Gunbak (1976) concluded that resonance phenomena which occurs for $2 < \xi < 3$ could cause lowest stability for breakwater and hence, actual design wave considered should be that which produces most dangerous resonance phenomenon.

Davidson and Markle (1976) found out that, broken dollosse are more detrimental to stability of breakwater against wave attack than uniform or random breakage. They also found out that if uniform or random breakage exceeds 15% of total member of dollosse in top layer and/or cluster breakage exceeds 3 dollosse in a cluster, stability of breakwater suffers.

Failure of large breakwaters in many countries round the globe from 1950 to 1980 forced the researchers and engineers to introspect various design aspects. They noticed a number of shortcomings like neglect of impact of wave period, armour strength, wave grouping, armour gradation, method of placement, geotechnical stability and foundation condition on stability of structure and model testing under appropriate prototype wave climate etc. They also realized the difficulties regarding traditional approach of design and construction methods like large quantity of stones, large size of stones, slow constructions etc. (Edge and Magoon 1979).

Designs of rubble mound breakwater are generally bound on combination of experience, skill and hydraulic model studies. This is the breakwater design of level I which is based on quasi-probabilistic approach where a characteristic load was established through a design wave height of 50 years or 100 years of return period at which hardly any damage should occur. But between 1970 and 1982, many breakwaters suffered severe damage. The analysis showed that damage was caused by a combination of wave climate, structural strength of armour, geotechnical stability and constructability. The safety margin for these structures was not large enough and therefore, it was felt that traditional approach was inadequate. Hence, to establish a better approximation of risk of certain degree, probabilistic approach (level II) was proposed assuming a normal distribution of all stochastic variables influencing load and strength. It was hoped that this approach may help to diagnose priorities, in design studies, leading to a balanced approach. The failure of Sines west breakwater was analyzed using probabilistic approach (level II) with various assumptions concerning stability and wave climate (Mol et al. 1983).

Mol et al. (1984) opine that the traditional breakwater design based on design wave of 50 years to 100 years return period, may suffer 2% to 5% damage of armour units but this is defined as no damage condition (Melby and Mlakar 1997 and US Corps of Engineers 2001). They presented a level III approach, which considers all the possible failure modes and parameters like quality of materials, armour placement methods etc and arbitrary statistical distribution of all these variables and responses of breakwaters depending upon previous history of damage. They developed software to design the breakwater using this approach.

U.S Army Corps of Engineers (1984) recommended use of $H_{1/10}$ instead of H_S together with previous K_D values which resulted in doubling the armour weight. And for unbroken waves, it recommended a K_D value of 2.0, then armour weight will increase by factor of 3.5. But using H_s with partial safety factor is considered as rational. However, for rock armour with no great difficulties in repair, the partial safety factors are little more than 1.0 and therefore, design will not differ much (Mettam 1992).

Chandrashekhar et al. (1985) and Barends (1986) analyzed deepwater breakwater failures and stressed the importance of geotechnical stability analysis, streamlining of quarry operations, method of construction, construction of core and armour placement and stressed that larger thickness stone layer than recommended by U.S Army Corps of Engineers (1984) should be used.

Gadre et al. (1985) emphasized importance of armour gradation and its influence in enhancing armour stability and gave several criteria to select armour and filter layer stone size.

Hookway and Brinson (1986) explain the method used for rapid construction of rubble mound breakwater at Ras Lanuf, Libya. This method was adopted due to shortage of time. The salient feature are use of concrete tetrapods armour on a steep slope of 3V:4H using heavy duty cranes and divers under water, using bottom dumping barges for construction of core, optimizing quarrying operation, strict quality control and supervision.

Sorensen and Jenson (1986) suggested that accurate wave prediction using all means like measurements, mathematical modeling, wave refraction studies should be undertaken for proper estimation of wave climate at site and instead of designing the breakwater in a traditional way for a wave of 50 years to 100 years return period, where, there may be probability of exceedence of design wave by 20% to 63% in 10 to 50 years, it seems more appropriate to consider acceptable probability of exceedence of design situation and economic loss if it happens and then design accordingly. Probability of failure of different elements like front armour, super structure, rear armour etc., are different and design is optimum only if these different probabilities are same as even a small overtopping under extreme situation may change the scenario from zero damage to high damage.

Meanwhile to overcome difficulties of Hudson formula, Van der Meer (1988) came up with new design formula for plunging and surging waves. Based on sensitivity analysis of Van der Meer's formula, it was found that regardless of wave height and structure slopes, as the permeability of the core increased, the weight of the armour unit required for the stability decreased. It was observed that as the density of the structure increases the weight of the armour unit reduces (Rao and Raju 1992).

Pranesh and Ramulu (1989) developed a methodology for estimation of cost of rubble mound breakwater. They developed a computer program in FORTRAN 77 for estimating costs of breakwaters with different types of armour units for various water depths, design wave heights and structure slopes. The program is based on Hudson's formula for calculating armour weight enables the designer the choice of best suited armour unit, and slope for a particular water and design wave height.

Vaidya (1989) listed the uncertainties in breakwater design parameters like limited wave data, K_D value, design wave and different failure mechanisms which together can deliver a fatal knock to the structure. He suggests that, long term wave measurement, proper model tests to ensure adequate armour weights, introduction of failure probabilities in design, assessing safety of breakwater considering breakwater as a whole system, probabilistic analysis of each component of breakwater its failure mechanisms and determining combined probability of failure be incorporated in the design.

Haan (1991) developed a computer program in Turbo Pascal to design a breakwater and used AUTOCAD to draw the cross section. In this program, quarry yield is divided into number of categories, long term disturbance of deepwater wave heights are converted into wave heights at site. Van der Meer formulae for statistically stable structures are used and a set of alternative cross sections are compared based on both functional performance criteria and Van der Meer formula. Construction and maintenance costs are determined for each alternative. The optimum is derived by minimizing the of construction cost and maintenance cost. The program provides means to economize the use of quarry.

Hall and Kao (1991) also stressed the importance of armour stone gradation which in turn decides the porosity of armour and under layer in the stability of breakwater.

Van der Meer and Heydra (1991) conducted research on large cubes and tetrapod armour on a 1:1.5 sloped breakwater section and studied the number of units moved and number of impacts as a function of wave height, wave period and their location on steeper slope. They measured velocity and accelerations during impacts and described the distribution of impact velocities.

The permanent technical committee of PIANC listed in 1985 for rubble mound breakwaters among the problems that need to be solved are evaluation of safety and risk and asked for measurements of movements, forces and stresses in units of armour layers both in prototype and model (Mettam 1992).

The working group established by PIANC achieved a better understanding of safety aspects and developed a practical way of evaluating safety of main armour for using Hudson or Van der Meer formula. This enables a theoretical design to be evaluated more realistically in respect of varying probabilities of failure within specified design life. The group evolved a probabilistic design with the introduction of partial safety factors γ_z and γ_H for Hudson formula as,

$$G = \{(Z/\gamma_z)\, \Delta\, D_n\, (K_D \, Cot\, \alpha)^{1/3}\} - H.\gamma_H \qquad \text{.......(2.24)}$$

Where,

K_D is deterministic variable and all other parameters are stochastic variables. Z has a mean value of 1.0 and standard deviation of 0.18. The design condition is satisfied by $G \geq 0.0$. Formulas have been derived to calculate γ_z and γ_H. With design incorporating partial safety factors, the armour size D_{n50} will be increased by a factor of safety $\gamma_z \cdot \gamma_H$ (Mettam 1992).

Kudale and Dattatri (1994) write that till 1990's rubble mound breakwater were designed based on model tests and empirical formulae. The numerical modeling techniques were rarely used due to complexity of wave structure interaction. They feel that numerical techniques are needed to improve the understanding of the mechanism and enhance design capabilities. Authors extended the numerical model developed by Kobayashi and Wurjanto (1989) for the design of impermeable structure to permeable structures. Flow above rough permeable slope was represented by shallow water equation and solved by finite difference method. The model can compute reflection, run up and armour stability.

Oumeraci (1994) as quoted by Kaldenhoff (1996) expresses that the doubts of several scientists and practitioners, about the reliability and performance of our design concepts for breakwater, led to the recognition of the following five important lessons:

1. Design wave load concept
2. Breaking wave impact loads
3. Damage due to wave overtopping
4. Seabed scour, toe erosion and subsoil deformation
5. Integrated approach to design.

In order to use the above mentioned lessons as guidelines, we have to change the widely used black box concept and adopt the physical behaviour of all involved fields for design purposes increasingly. These factors are:

1. Static behaviour- forces, moments, stresses.
2. Dynamic behaviour- motion and movement of water and structural body
3. Energy flux- wave transmission and energy dissipation.
4. Geophysical behaviour- pore pressure and scour.

Van Gent and Vis (1994) developed a numerical simulation of wave interaction with coastal structures. This simulates breaking wave on various types of coastal structures and resulting movement of individual elements in the seaward slopes. In the numerical tool, wave dynamics are approximated by non-linear finite amplitude shallow water equations, often referred as long wave equations with steep wave fronts represented by forces. For this description of wave motion, the vortices and vertical acceleration are assumed to be negligibly small which leads to hydrostatic pressure. Depth averaged velocities are defined only at one free surface point per cross section are used.

Belfadhel et al. (1996) described new rip rap stability formula based on field tests like Koev and Sherbrooke university formulae and concluded that, these formulae estimate armour weights comparable with those given by Hudson formula.

Franco et al. (1996) explains the modern construction methods adopted in construction of large breakwater in North African countries of Libya, Morocco and Algeria during 1980 to 1984. These structures were constructed in deep waters (i.e. 16m to 28 m) after the failures of large breakwater at Sines, Tipoli etc. and safe designs were adopted by considering some overtopping. The new breakwaters were of steep slope of 1:1.33 to 1:1.5 and used heavy tetrapods. The breakwater lee side was also protected, core was protected during construction with filters and armour layer and design was flexible to adopt any change even during construction stage. During construction of these structures it was realised that mass scale production of artificial armour units, quality control, optimizing quarrying operation, innovative planning of construction, improved wave forecasting, detailed risk analysis and probability prediction of local wave statistics, long term feedback from contractors, ports and other user agencies regarding performance of these structures will go a long way in improving future designs.

Hegde (1996) also developed a mathematical model called DORUB to predict the armour stone weight of a rubble mound breakwater using the quarry yield optimally. He used programming language C and Visual Basic to draw the cross section.

The major design uncertainties are related to the process of defining and quantifying the entities. We always have to be aware that little changes of the design parameters may cause remarkable changes in structure dimension (Kaldenhoft 1996). The breakwaters have been built throughout the centuries, but their structural developments as well as their design procedure are still under massive change. The reason for this are limited reliability and high costs. Major research activities are necessary, some of them have already been installed to gain better knowledge of the physical background of the performance of the design concepts (Kaldenhoff 1996).

Meadowcroft et al. (1996) discussed the development and use of risk assessment and probabilistic design techniques for the analysis and/or design of sea and tidal defense schemes. He considered risk assessment can be done through:

1. Structure variable approach based on calculating single response variable from measured or collected data which can be extrapolated to extremes using an extreme value distribution.
2. Joint probability approach where several risk assessments are enlisted for the various failure mechanisms and their combined risk assessment is undertaken.

Melby and Mlakar (1997) observe that, though the basic design process is deterministic, some elements of probabilistic theory have been utilized. But many more stochastic variables must be included in order to adequately assess the reliability of breakwaters. The authors have derived the limit state equation for safety factor and safety margin of Hudson and Van der Meer design formulae for breakwater. They discuss level II reliability techniques and applied it to breakwater design. Using these methods, reliability and reliability index are determined for the dominant performance functions of a breakwater including armour stone stability, run up and overtopping. However, they caution that, though reliability can be a powerful tool in estimating uncertainty with respect to many failure modes, these methods do not guarantee a more reliable structure as response of rubble mound breakwaters vary widely, even in controlled laboratory conditions and limited local data, lack of design guidance for certain failure modes and unknown and somewhat unpredictable construction quality, among other things can reduce/vary structure's reliability from that of design estimate.

Delft Laboratories have developed a computer model called BREAKWAT. 3 (Delft Hydraulics 2001). Given the design inputs like wave characteristics, water depth etc., the model designs any type of breakwater using probabilistic method. Delft Hydraulics took up the project of developing of new beach zone, at Rubble Tip area between Eastern Beach and Catalan Bay, located at the eastern side of Gibraltar. A wide crested submerged breakwater was constructed parallel to the shoreline in front of the project site as a part of the structural measure. Its stability was tested using the software BREAKWAT. 3 and physical model study.

2.11 ECONOMICAL DESIGNS OF BREAKWATERS

New ideas and developments are always in the process of being tested for reducing the load on and failure of breakwaters. Ahrens (1984) defined reef breakwaters as low crested structures, with its crest at or below SWL, constructed through more or less uniform sized armour stones, of weight comparable to that of a conventional non-overtopping breakwater, but without a core and analyzed their stability.

Baird and Hall (1984) writes that by building berm breakwaters using smaller stones and utilizing quarry yield economically, there can be 50% to 70% cost savings for a site compared to traditional breakwater.

Fulford (1985) found that reef structures can be optimal solutions in many cases, like of beach protection, beach stabilization.

Baba (1985, 1986) concluded that a concrete submerged breakwater of seaward slope of 1:1.67 with a vertical shoreward side is the optimum geometry as it gives maximum damping, minimum reflection, minimum bottom scour and maximum sand trapping efficiency for steep waves for a site at which the tidal range is up to 2m.

Gadre et al. (1987) proposed sand filled rubberized coir bags for coastal protection which could be an effective and economical alternative depending upon site conditions.

In the same period various attempts were being made to arrive at cost effective building of breakwaters. Hoedtden et al. (1987) reported the invention of a Bikon block consisting of high strength polyester woven geotextile tube, which is positioned along the slope of breakwater from crest to toe and filled with liquid concrete. These blocks had a width to height ratio of 1.5 and length to width ratio of 2.0 with a void ratio of 5% to 10% between individual blocks with a tight interlocking knit of neighboring blocks. The construction is easy and economical and no crane is required. This construction saved at least 10% to 20% of the cost compared to conventional breakwaters.

Very often rubble mound breakwater design seems to be a result only of stability considerations corresponding to design wave conditions. Designers tend to put too little emphasis on practical problems related to construction, maintenance and repair. In construction of a breakwater, core takes about 60% of costs, super structure and armour layer costs about 20% each. For the core material of 2Kg to 300Kg, the workability is limited to wave height (H_S) of about 1.2cm, whose value is exceeded during approximately 20% of the time. Therefore, heavy size stone may be used when waves exceed the limit for quarry run and saving could be made. During construction of seaward slope of breakwater, each armour block takes about the same time to place, independent of its weight over a fairly wide range and savings can be achieved by placing fewer and heavier concrete blocks (according to modified design) on a steeper slope than a relatively lighter natural stones over a gentler slope depending upon the site conditions and wave climate (Rietveld and Burchardt 1987). Authors discuss case studies where, by using heavy core and armour materials, they could achieve a savings of about 8% in costs and 16% to 50% in time.

Ahrens (1989) improved his stability formula for reef given in 1984 and gave improved design guidelines. Gadre et al. (1992) and Nigam and Yuwono (1996) too gave stability criteria for submerged reef. Pilarczyk and Zeidler (1996) gave the design curve for estimating reef armour weight.

Due to special requirements of the equipment, offshore breakwaters are not normally constructed. On the contrary, the offshore breakwaters are more effective in providing beach protection without affecting littoral drift significantly. Offshore submerged breakwaters in small depth say 2m to 3m can be easily constructed with a variety of materials like stone filled boxes or synthetic bags or chains of small concrete blocks. These can be easily and economically transported to the site by boats or by rafts, where the total weight of chain of blocks is around 200Kg and can be lowered at site by 4 to 6 persons with locally available equipments. Such structures with crest width of 1 to 1.5m in a depth of 2m to 3m, can achieve a wave damping of 33% to 50% without significant damage to armour units (Kale and Gadre 1989).

Berm breakwater can be built with armour stones which are 20% to 30% lower in weight than that required for an equivalent traditional/conventional breakwater (Gadre et al. 1991).

Franco (2001) quotes Archelti and Lamberti (1991) comparing the cost of berm breakwaters at a potential site of a harbour in Italy showed a saving of 25% to 40% compared to design of traditional/conventional breakwaters.

Cox and Clark (1992) built a tandem breakwater system for Hammond Indiana using 3 ton stone armour instead of 8Ton armour stones for breakwaters defenced by a submerged reef of 0 to 1ton stones separated by a distance of 40.5m and saved $1 million.

To reduce wave load on structure and/or allow exchange of waters, the concept of underwater or submerged breakwaters is always attractive. These can be economically adopted for beach protection and for sediment trapping and retaining. Semicircular caisson breakwaters are suitable for shallow water conditions and offshore breakwaters to protect beaches from erosion (Kaldenhoff 1996).

Franco (2001) mentions that in Italy, the design of breakwaters and other maritime structures are increasingly being influenced by environmental, architectural and social issues with greater attention to nature conservation, people's recreation and sporting activities. The trend is towards low crested and permeable structures with reduced use of visible concrete elements. Other important design aspects are safety and speed of construction and even the structure replaceability for seasonal use. Franco, also opines that the complex hydromorphodynamic processes associated with submerged barriers are not yet fully known. Therefore, a specific European Research Project titled 'DELOS' has been initiated in 2001 which will also investigate their ecology and economical implications in order to provide more reliable design guidelines.

Beirawski and Maeno (2002) proved experimentally that for permeable submerged breakwaters or reefs, with 35% porosity, the pressure gradients are smaller compared to impermeable structure. They found that permeable structure was stable while the impermeable structure was badly damaged.

2.12 SUBMERGED BREAKWATER

This is an offshore rubble mound breakwater, with its crest at or below SWL, used for creation of a protected area of water or for protection of coastal structures and beaches from damage of erosion caused by wave action.

Since the crest of the structure is at lower elevation wave overtopping and/or breaking takes place due to which armour units are subjected to relatively smaller wave force thus the structure can be economically constructed with relatively smaller sized armour. Initially, Beach Erosion Board (BEB) of US in 1940s and Lamb in 1945 showed interest in submerged breakwaters (Johnson et al. 1951).

Johnson et al. (1951) studied the wave damping action of submerged breakwater and recognized that wave steepness (H/L), crest width (B), structure height (h) and water depth (d) are important parameters. They concluded that for a given height of the structure, steeper waves are effectively damped than flat waves due to breaking of waves. Gentle waves are increasingly broken if the crest width (B/L) is $\frac{1}{4}\sqrt{(1\text{-}h/d)}$. And for steeper waves in large depths, crest width (B) to be effective, should be equal to 0.1 to 0.24 times the wave length (L). They found for effective wave damping, d/L_o should be small, crest freeboard (F) should be zero (i.e. crest at SWL), structure should be located at one wave length or more seaward of the breaker line. They also observed that steeper waves in large depths require wider barriers for effective damping and transmission coefficient (K_t) is more sensitive to higher value of B/L. On natural beaches, seabed can be assumed to be horizontal over a distance of wave length or more. They also write that K_t is approximately equal $\sqrt{(1\text{-}h/d)}$.

The functions of a submerged breakwater according to Homma and Sakou (1959) are attenuation of waves by causing premature breaking and partial reflection and barring seaward movement of the bed material in the surf zone. They established that submerged breakwaters reduced regression of shoreline and scour depth varied with h/d and breakwater location with respect to breaker line. They observed minimum bed scour for the structure located offshore from breaker line. They studied the effect of the submerged breakwater on the coastline, deformation of the beaches and the scour around the wall placed in the vicinity of breaker zone through a model study. This was accomplished using an experimental wave flume 17m long, 0.7m wide and 0.6m deep. The water depth was maintained at 0.35m. They generated waves of 0.101m to 0.128m of periods varying between 1.1sec and 1.28sec. They concluded that h/d at the site, should not be more than one. They also concluded that the entire beach processes will change due to the effect of submerged backwater wall location and h/d ratio effects the variation of scour depth. Further they observed that, submerged backwater decreases the regression of the shoreline for the cases considered in the experimental study.

A fairly extensive work has been done correlating the field observations and the result obtained from model studies on the function of submerged backwater as a shore protection structure by Homma and Horikawa (1961) with reference to Nigata coast. Several types of breakwaters, all permeable in nature were considered and were tested by many investigators. From the previous experimental studies, it was found that, if the height of the submerged breakwater (h) is less than 70% to 80% of the water depth, where the structure is located, the damping action against the incoming waves is not significant. It was mentioned that the damping action of submerged breakwater had seen studied previously in relation with the height, width and shape of the structures and depth of water where the structure was located. But the authors have published only in Japanese languages and hence are not usable. Observations were made at two stations namely Ojoin beach and Kosoda beach. The height of waves damped and the steepness ratio of waves damped by submerged breakwater was found to be 30% to 70% and 10% to 70% of that of the original waves respectively. The attenuation of the waves was larger for steeper waves, which are responsible for beach erosion. The wave steepness was ranging from 0.02 to 0.08 and in both beaches. They studied the variation of beach profiles due to the presence of submerged breakwater by field observations and by experimental work. The laboratory experiments consist of models of hollow block and 'L' type submersible breakwaters, which were constructed at Nigata west coast. Authors found that the scour around the breakwater was considerable when the structure was located in the breaker zone and depended upon h/d. As h/d ratio reduced, the scouring increased when the structure is placed in surf zone. It was also observed that the increased scouring action was in place as h/d increased when the structure was placed in off shore zone.

Saville (1963) as quoted by US Army Corps of Engineers (1984) conducted investigations, which, were primarily aimed at studying the scale effects, and he concluded that the results of small-scale tests could be relied upon for prototype design. US Army Corps of Engineers (1984) has presented certain design curves based on the limited experimental data of Saville

(1963). However, comprehensive investigations of basic nature on the submerged breakwaters have been very limited.

The incident waves break over the submerged reef and propagate further. The waves while breaking create turbulence and air is entrained in the water. This takes away major portion of wave energy. Then as the waves propagate further, they loose a part of their energy in over coming bottom friction. Horikawa and Kuo (1966) has formulated an equation for wave energy loss of broken waves, while propagating over the horizontal bottom, based on the principle of energy conservation of solitary waves .

Dick and Berbner (1968) have investigated both solid and permeable breakwaters of the rectangular type where, the permeable breakwaters were made of nested tubes. The test results show significant scatter of data and hence only trend lines have been indicated. They write that in nature a wave striking a submerged barrier will result in some of its energy being reflected offshore where it is ultimately dissipated by wind and internal stress. And this is the simple model of nature and authors believe that mono-periodic waves could occur in nature. They observed during physical model study that, permeable and impermeable submerged rectangular breakwater behave differently. For permeable submerged breakwater, there was a well defined minimum K_t whereas, for the impermeable structure of the same size there was no such minimum. This was thought to be due to breaking and turbulent fluctuations. Both breakwaters caused loss of at least 50% of incident wave energy and 36% to 64% of energy is transferred to higher frequencies than incident waves (i.e. secondary waves) are generated. They experimentally found that, for a solid submerged breakwater of F/d of 0.075 and a B/L value of 0.5, K_t could be 0.3 while, K_t was greater than 0.5, for a permeable submerged breakwater with porosity of 0.4, F/d of 0.2 and B/L of 0.5. They observed that reflection coefficient K_r varied between 0.2 to 0.6 for F/H values between 0.05 to 0.2 and zero porosity.

Diskin et al. (1970) writes that submerged breakwater cause breaking of high waves at breakwater rather than at a point nearer to the shore, thus diminishing the height of waves transmitted to the shore. He lists the advantages of submerged breakwater as low cost, non-obstruction of the view of sea, which is important in cases of recreational and residential shore developments, nor do they convert the site into a completely still pool that would need artificial circulation of water if beach is used for swimming and submerged breakwaters are beneficial for artificial fish breeding areas due to small waves in a protected area. Piling up of water inside the protected area may take place due to breaking waves at submerged breakwater and spilling over into protected zone. Piling of water is important only in the case of submerged breakwater being very long or protected area being enclosed completely. They conducted the experiment on submerged breakwater of crest width 0.10m to 0.14m of slopes 1:2.5 to 1:3 with structure height (h) 0.15m to 0.40m constructed with the stones of 64gm to 96gm. They noticed that, for submerged breakwater with crest at SWL, piling is 0.35 times wave height, for $F/H_o = 0.5$, piling is 0.15 H_o and for $F/H_o > 1$, piling is less than $0.05H_o$ with an accuracy of 10% to 20%. Piling (P_l) is given by

$$\frac{P_l}{H_o} = 0.6\,Exp\,(0.7 + \frac{F}{H_o})^2 \qquad \text{....(2.25)}$$

for $-2 < F/H_o < 1.5$ and $0.1 < H_o/d < 0.83$

They also found that as the waves start breaking at submerged breakwater, piling up increases till equilibrium is established through the flows out over the breakwater and through the breakwater. As the waves break, the water mass spreads over a length of 4h to 6h immediate behind the submerged breakwater. This is the zone characterized by high degree of mixing and turbulence and the water there has high content of air bubbles entrapped during the wave breaking process. The surface of the water is turbulent and fluctuated due to spreading water mass and the secondary waves that traveled toward the shore may fully develop a short distance after the end of the turbulent zone.

Sollit and Cross (1970) through mathematical modeling proved that transmission coefficient K_t decreased with decreasing wave length L, porosity p and increasing wave height H and increasing breakwater width B. While reflection coefficient K_r decreased with decreasing B and L and increasing p. They found that for a layered structure of Kd (i.e. 2Πd/L) values of 1.5 and 2.01, K_r decreased from 0.3 to 0.15 and K_t decreased from 0.25 to 0.03, while, wave steepness increased from 0.005 to0.05. But for a steepness of H/L of 0.01 and 0.02, K_r decreased from 0.53 to 0.25 and K_t decreased from 0.2 to 0.03 for a variation of Kd (i.e. 2Πd/L) from 0.5 to 3.0.

Depth of siting submerged breakwater is generally from 2 to 5m according to Hueckel (1975) as quoted by Pilarczyk and Zeidler (1996). Submerged breakwaters should be sited at small distances from shoreline, parallel to it on a depth of 1.5m to 2.5m. For rubble mound breakwaters of smooth slope of 1:3, crest datum was generally located at 0.5m to 1.0m below SWL which was the practice till 1963. Adequate attenuation of waves was believed to be achievable if the elevation of submerged breakwater exceeded 0.7 times depth of water .i.e. $h/d > 0.7$ or $F/d < 0.3$

US Government took lot of interest in low cost shoreline protection measures such as revetment; bulkheads, groynes and offshore breakwaters, floating breakwaters, vegetation. The project was National Shoreline Erosion Demonstration Programme costing $8millions. This program adopted at number of locations, would demonstrate to public regarding various aspects of hard and soft options and would provide help for private land owners with an eroding shoreline (Edge et al. 1976).

Madsen and White (1976) developed analytical approach to determine reflection and transmission coefficients of flow through projecting multi layered trapezoidal rubble mound submerged breakwater of stone armour and compare them with experimental results. They obtained a K_r increasing from 0.4 to 0.6 and K_t decreasing from 0.35 to 0.15 for a H_i/L varying from 2.5×10^{-3} to 3×10^{-2}. They assumed that long incident waves do not break over breakwater and are described by linear wave theory. The external energy dissipation may be considered mainly due to bottom frictional effects.

Palmer and Walker (1976) write that for a breakwater, the weight of armour placed below 1/3 depth may be 3/4 of that of the armour units placed near water surface.

Thornton et al. (1976) write that as wave shoals, secondary waves start to grow and as the waves steepen rapidly, just before breaking, the secondary waves likewise develop rapidly on the back of primary waves. Development of secondary waves indicates transfer of energy to higher frequencies.

Dattatri et al. (1978) write that transmitted wave has the same fundamental period as incident wave but for cases with crest closer to SWL, higher harmonics are generated. The wave set up or piling up behind the submerged breakwater is significantly influenced by depth of submergence, crest width, shape, permeability and wave steepness. Wave steepness (H/L) and wave breaking characteristics depend on wave height, which in turn have a role in deciding the stability. They conclude that, incident wave steepness has an important influence on the wave-breaking phenomenon and steeper waves are likely to be attenuated more than the flatter waves. At higher wave steepness, the influence of incident wave steepness (H/L) on transmission coefficient (K_t) is not significant. Authors conducted experiment on different geometries of submerged breakwaters. They proved that crest width, depth of submergence/freeboard F and wave steepness are important parameters for deciding the wave transmission. They found rectangular and trapezoidal submerged breakwaters had same transmission. However, for trapezoidal shape wave reflection was less and transmission increased with porosity. But for large crest submergence ($F/d > 0.2$) porosity was unlikely to have any influence on transmission coefficient K_t. They summarised as follows:

1. For $F/d < 0.4$, K_t is small
2. At F/d of 0.4, K_t is 0.75 to 0.95.
3. $F/d > 0.4$ is not likely to have any practical significance.
4. Small crest widths do not break waves.
5. Optimum crest width (B/L) for rectangular breakwater is 0.2 to 0.3.
6. Sloping seaward shape with a vertical shoreward face is the optimum geometry.
7. Steeper waves are effectively attenuated, but in higher range of steepness, its influence on K_t is not significant.

Thornton and Schaeffer (1978) write that waves in the surf zone are highly nonlinear which is evident by the appearance of secondary waves.

At Massachusetts USA, sand filled Nylon fabric bag protective structure, supports the perched beach at Bay Road; Dead Nuk Island and Morris Island and Sunken Meadow Beach (Gutman 1979).

Khader and Rai (1980) continued the experimental work of Dattatri et al. (1978) and found that submerged breakwater absorbs wave energy, dissipates energy through premature breaking, partly reflected the remaining energy and partly transmitted it shoreward. Submerged breakwaters are effective in beach erosion control. They write that because of the complexity of the problem, theoretical development is over simplified, and therefore, greater reliance should be placed on lab and field studies. But these are surprisingly limited. And most reliable quantitative data on effect of submerged breakwater on wave action are those obtained in laboratory model studies. They found relatively wider barriers have a better wave damping action than narrower one. Other shapes of breakwaters effectively damp the waves only when structure height h/d is large (i.e. 0.43 to 0.66), where as, trapezoidal submerged breakwaters are effective even in the low ranges of h/d. They proved that for $F/d < 0.34$ to 0.57, the changes in K_t is more influenced by wave breaking. Rectangular shaped breakwater appeared most effective under all conditions.

Seelig (1980) according to US Army Corps of Engineers (1984) proposed the following empirical wave transmission coefficient,

$K_t = (0.51 - 0.11\ B/d)\ (1+F/R)$……………………………………………………………(2.26)

Where,

R is the run up on seaward slope of a breakwater in absence of wave transmission.

Aminti et al. (1983) according to Pilarczyk and Zeidler (1996) carried out an extensive series of experiments with various permeable and impermeable structures. They concluded that for wave breaking, the ratio of crest width to freeboard (B/F) should be 7 to 9. From the IBW PAN laboratory tests carried out in the seventies it was observed that.

1. In breakwaters a major portion of the wave attenuation results from losses caused by wave breaking creating turbulence at the breakwater slope.
2. Permeability of the submerged breakwater core and its shape has no significant influence on the transmission coefficient.
3. A higher submerged breakwater induces higher harmonics on the lee side.

They established experimentally for submerged breakwaters whose crest width is equal to water depth i.e $B/d=1$, K_t decreases from 0.9 to 0.69, when F/d decreases from 2/3 to 1/3. Raising the height of structure simultaneously reduces K_t by about 10% with respect to the structure with crest width equal to water depth.

U.S Army Corps of Engineers (1984) recognized the advantages of submerged breakwater compared to breakwaters include, low cost, aesthetics, effectiveness in triggering breaking of high waves without eliminating the landward flow of water, which may be important for water quality considerations.

Baba (1985, 1986) discuses about the Soviet design of reinforced concrete submerged breakwater as a coastal protection structure. This structure has an optimum seaward slope of 1:1.67 with vertical shoreward face give maximum wave damping, minimum reflection, minimum bottom scour and maximum sand tapping efficiency. These structures are very effective for a low tidal range of less than 2m and steep waves of H/L > 0.075. The reflection coefficient K_r decreased from 0.4 to 0.02 and K_t was fairly constant at 0.3 as d/h increased from 0.75 to 1.375.

Lundgren and Jacobsen (1987) write a submerged breakwater in front of a pile breakwater reduces wave breaking on piles and thereby reduce shock waves. The pile breakwater is suggested as an alternative structure on weak soils.

Sorensen (1987) suggests a perched beach concept for shore stabilization as a solution for shoreline conservation for developing countries. This beach may be protected by a submerged barrier reef.

Kobayashi and Wurjanto (1989) writes that submerged breakwaters are advantageous with respect to low cost, aesthetics, wave breaking, water flow and sediment movement over it therefore helping to maintain water quality. They also conducted mathematical model study of transmission over smooth impermeable submerged breakwaters and compared with physical model studies. The results agreed well. They used conservation equation of mass and momentum to compute flow field. An equation of energy is derived to estimate the rate of energy dissipation due to wave breaking. But Rojanakamthorn et al. (1989) found out that for a submerged breakwater with porosity of 39% and F/d=0.3, K_t varied from 0.1 to 0.7 depending upon wave climate and physical features of the structure.

Mani et al. (1991) states that submerged breakwater have been used to protect harbour entrances, to control wave action at inshore fishing grounds and to reduce littoral drift. Most applications have been designed on the basis of experimental findings as there is no proven theoretical or experimental work to permit reliable or safe design. He mentions that large amount of wave energy is lost by wave breaking, turbulence created and friction offered. In other combination of wave and structural characteristics a significant amount of energy is lost due to turbulence created by interference caused by breakwater to normal and transient fields in both horizontal and vertical directions. They conducted experimental investigations on rubble mound submerged breakwater of seaward slope of 1:2 and leeward slope of 1:1 and crest width of 0.5m and concluded that for (F/d) = 0.5 for a wave attenuation of 30%.

Van der Meer and d'Angremond (1992) gave the following equation for transmission for a low crested breakwater.

$K_t = 0.80$ for $-2.0 < F/H < -1.13$

$K_t = 0.46 - 0.3\ F/H$ for $-1.13 < F/H < 1.2$

$K_t = 0.10$ for $1.2 < F/H < 2.0$ ....(2.27)

For submerged breakwater they gave the following equations

$K_t = a\ F/D_{n50} + b$ ....(2.28)

Where,

$a = 0.031H/D_{n50} - 0.24$ ....(2.29)

$b = -5.42\ S_p + 0.0323H/D_{n50} - 0.0017(B/D_{n50})^{1.84} + 0.51$ ....(2.30)

and S_p is the wave steepness.

They obtained a minimum K_t of 0.075 and a maximum K_t value of 0.75.

Goda (1996) writes that emerging breakwaters to control beach erosion are aesthetically unacceptable to people of Japan due to obstruction to view open sea, beach users at tourist spots. Hence, they prefer submerged barriers for sites of moderate tidal ranges.

Kaldenhoff (1996) writes that to reduce wave load on structure and/or allow water to exchange, the concept of underwater structure i.e. submerged breakwater is always attractive.

Modeling of transformation and interaction of regular wave trains with submerged permeable structures is carried out and theoretical results are compared with the experimental data (Losada et al. 1996). The influence of wave characteristics including oblique incidence, structure geometry and porous material properties on the kinematics and dynamics over and inside breakwater is considered. Investigators concluded that crest width and crest height are more important parameters in controlling the wave reflection and transmission than the porosity, oblique wave incidence and structure slope for non-breaking waves but this may change for breaking waves.

The stability of the submerged breakwater is given by the formula (Pilarczyk and Zeidler 1996) as:

$$\frac{h_c}{h} = (2.1 + 0.1\,S)\, e^{-0.14\,N^*_s} \qquad \text{.....(2.31)}$$

Where,

h and h_c are the crest heights of the breakwater before and after the damage respectively, S is the damage level, N^*_s is spectral stability number, a parameter introduced by Ahrens (1984). N_s^* is given by

$$N_s^* = N_s\, S_p^{-1/3} = (H_S/(\Delta\, D_{n50}))\,(S_p^{-1/3}) \qquad \text{.....(2.32)}$$

Where,

S_p is the local wave steepness, it is calculated using local wavelength from the Airy theory.

Pilarczyk and Zeidler (1996) comments about conclusions of experimental works conducted in 1970s that in breakwater a major portion of wave attenuation results from energy losses caused by turbulence and breaking at breakwater and permeability of the structure core and its shape have no influence on K_t and higher the submerged breakwater higher harmonics on leeside. These views are quite opposite (Dattatri 1978). Pilarczyk and Zeidler write that energy damping mechanism due to wave breaking over a submerged breakwater is not well understood. These are similar to the views are expressed by Kaldenhoff (1996) and Franco (2001). Wave breaking over the structure induces changes in wave length and wave period and produces turbulence on shoreward side which should be taken into account for proper placement (location) of submerged breakwater. Pilarczyk and Zeidler (1996) comment that, in general it is recommended that, physical and mathematical model studies should be under taken at the design stage to optimise breakwater arrangement. Water exchange behind submerged breakwater is better than that for emerging breakwater. The advantages also include preservation of environmental and relatively low capital investments. But both breakwaters suffer from considerable settling due to scouring at the toe vibration and sediment transport at the toe. Submerged breakwaters like the emerging ones, are intended for prevention of beach erosion. This function is accomplished through entrapment of suspended sediment traveling offshore with the return flow and overtopping the breakwater in the onshore phase of the wave motion. The long shore transport is also partly intercepted by the growing cusp, sometimes arrested altogether if a tombolo rises. Hence, submerged breakwaters dissipate wave energy usually even more efficiently than their emerging counterparts do, prevent beach erosion, enhance the growth of accretion forms. They are less conspicuous than for emerging breakwaters as they do not spoil aesthetic aspect of beach and are economical. The distance of submerged breakwater from shoreline depends entirely on depth of foundation and sea bed slope. Submerged breakwater located in breaker line and surf zone cause higher scour if their height is higher. The submerged breakwater located seaward of breaker line cause higher erosion if their crest is lower. It is generally assumed that a trapezoidal cross section is better than rectangular one.

The best morphological effects, pronounced as accretion of sediment between the submerged breakwater and shoreline have reached for the slope of the seaward wall ranging 1:2 to 1:3. Better dissipation of waves, lower reflection and easier transport of sediment over the structure were observed for these slopes. Lower erosion at the foot of the structure was also noticed. In general, effectiveness of submerged breakwater in wave damping is lower than of emerging breakwaters having same geometry and layout. To achieve comparable results, the transmissibility of submerged breakwaters should be decreased by enlarging crest width or diminishing the free board (F). For the structures with sufficient crest width, the wave breaks resulting in greater energy dissipation and less transmission. Crest width is found in all investigation as a factor improving wave dissipation. Economic assessment must determine selection of this parameter. Further investigations on effect of crest width of submerged breakwater on transmission are needed. Pilarczyk and Zeidler (1996) write that wave propagation over submerged breakwater, generates higher harmonics on seaward side. This strongly influences wave decay and K_t. The higher harmonics are generated in shallow water larger over submerged breakwater and then transmitted to deep water as free waves. For regular waves this increases the transmitted wave energy which is at a higher frequency than that of offshore waves. The slope angle has largest influence on non-overtopped structures, but in the case of submerged structures, the wave attack is concentrated on the crest and less on the seaward slope. Therefore, excluding the slope angle of the submerged structures, being a governing parameter for stability, may be legitimate (Pilarczyk and Zeidler 1996).

Smith et al. (1996) mention about a three year feasibility study on coastal conservation for islands of Barbados, where, a submerged breakwater along with groyne and beach nourishment were designed to develop wider beach at the eastern end of Rockley beach. They studied wave breaking, sediment movement, beach building etc. The data obtained from physical model tests were used to calibrate various numerical models regarding sediment transports, performance of submerged breakwater etc. They concluded that optimum location was parallel to bed contours at a seaward distance X/d of 39.5 from beach, crest elevation h/d of 0.84 would cause beach accretion and if the submerged breakwater is located beyond at X/d $\geq$ 40, the beach gets eroded (refer Fig. 2.8). They found lowering of crest elevation h/d from 0.7 to 0.5, increased beach erosion due to offshore sediment transport. They also observed that critical conditions of stability were low water conditions.

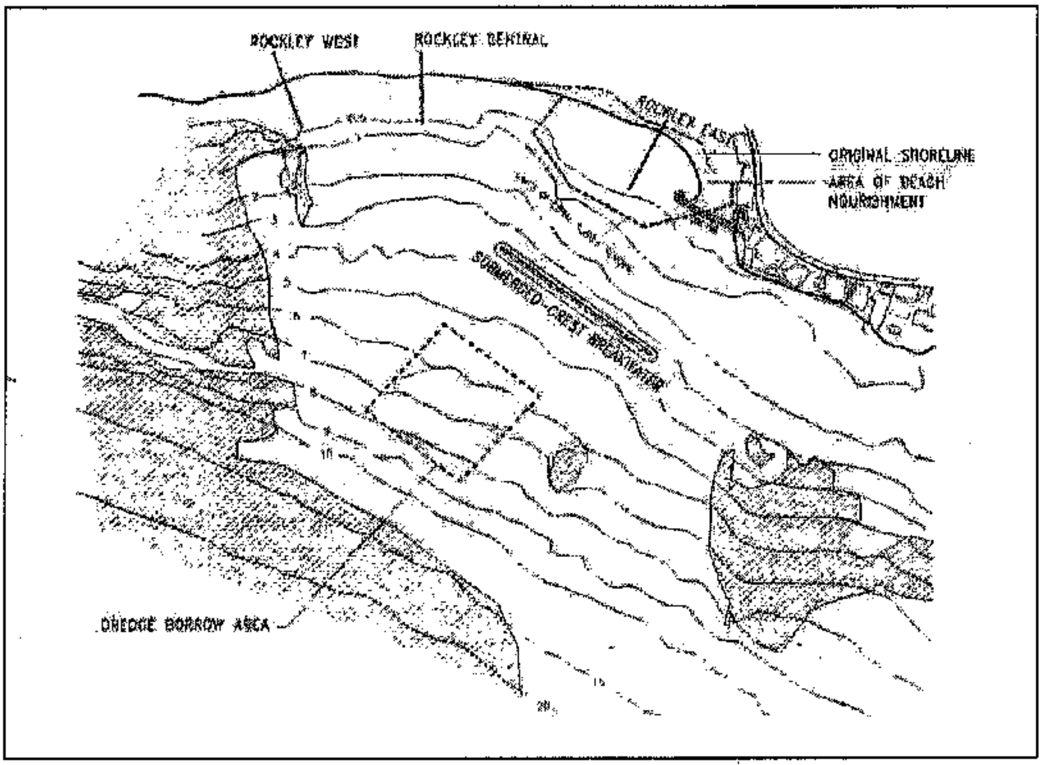

Fig. 2.8. Submerged breakwater at Rockley beach, Barbados

Van de Graff (1996) writes that a properly designed submerged breakwater is able to reduce wave heights on lee side and also reduce the rate of dune erosion during storm provided increase in sea water level does not exceed the design level. He further writes that, in order to restrict the volume of sediment needed for reclamation in zero option, one could consider a submerged breakwater with the help of which upper part of the cross shore profile can be supported and thereby avoiding the toe of profile while achieving reduction of large volume of sand. Author writes about experiments being carried out, in Fluid Mechanics lab of Delft University of Technology, The Netherlands, to study the effects of submerged breakwaters on coast and experiments are also being conducted to study the application of beach nourishment and shore parallel structures.

Seabrook and Hall (1997) presented the results of 2-dimentional tests defining the effect of depth of submergence, crest width and breakwater slope on K_t from regular and irregular waves and concluded that existing equations do not adequately predict the K_t for wide crested structures.

Mai et al. (1999) writes that submerged dykes are one of significant contributors to the protection and safety of the German coastal line. These dykes offer shoaling, refraction, wave breaking and bottom friction. They used mathematical modeling by HISWA, SWAN, MIKE 21and EMS software for simulating above processes and calibrated numerical models with experiments of large scale models in a wave tank.

Daemrich et al. (2001) concluded that the following design formula (given by d' Angremond et al. 1996) is good basis for analysis and control of measurements on K_t at submerged structures within the given range of validity.

$$K_t = -a\, F/H + (B/H)^{-b}\,(1-\exp(-c\xi))d \qquad \text{.....(2.33)}$$

Where,

for permeable structure, a = 0.4, b = 0.31, c = 0.5, d = 0.64 and for impermeable structure, a = 0.4, b = 0.31, c = 0.5 and d = 0.8.

Franco (2001) mentions that submerged rock mounds are fused for retaining that beach nourishment at the seaward end. Eg. Venetian Lidos. They have a variable submergence (F) of -0.5 to -2m where tidal range is less than 1m. He also writes that hydromorphodynamic processes associated with submerged barriers are not yet fully known. Therefore, a specific European research project called DELOS was started in 2001 and was coordinated by University of Bologna which will also investigate their ecologic and economic implications in order to provide more reliable design guidelines. In Italy perched artificial beach contained by L shaped rock groynes and submerged barrier, for enhancing recreation and urban social life, is being completed along the water front of Civitaveachia, Rome.

Mendez et al. (2001) conducted numerical model studies on permeable submerged breakwaters of slope 1:2 subjected to regular waves. They concluded that set up induced is due to radiation stress energy dissipation which in turn is the result of wave breaking and friction. The set up for non-breaking waves is 2% to 5% of incident wave height, while for breaking wave, set up reaches up to 20% of H_i or higher depending upon H_i.

Nagendra Kumar et al. (2001) designed a submerged breakwater along with a groyne for Pilot basin at Sagar Island in Hoogly Estuary, India to reduce the wave energy levels and siltation where, the average tidal range is about 6m The efficiency of submerged breakwater in dissipating wave energy was evaluated by numerical modeling of wave processes using parabolic mild slope (PMS) module of MIKE21 software. This model takes into account the effects of wave reflection, refraction, diffraction, energy dissipation due to bottom friction and wave breaking. The submerged breakwater was 330m long and design wave was 2.5m high and period 10sec. The model was run for regular waves. They concluded that, wave reflection from submerged breakwater is absent as the submerged breakwater dissipates incoming wave energy and transmits the remaining energy to the lee side. Further they write that if the submerged breakwater is oriented perpendicular to wave direction, the diffraction along wave direction is almost negligible. They found that the submerged breakwater creates friction to dissipate energy in addition to natural friction from bed and effect of bottom friction is cumulative and wave energy dissipation increases with distance of travel towards shore. The energy dissipation of monochromatic waves by submerged breakwater is less than irregular waves. This is very much required for conservative hydraulic design of submerged breakwater.

Rambabu and Mani (2001) proposed a numerical model to predict K_t using Laplace equation for certain boundary conditions. Their solution is in the form of velocity potential which is solved using Green's theorem. They observed that K_t decreases with increase in wave steepness and with decrease in F/d. They concluded that by restricting F/d to 0.5, K_t can be kept below 0.6.

Twu et al. (2001) proved through mathematical modeling that for deeply submerged rectangular breakwaters of B/d 20 and F/d of 0.5, K_t can be as low as 0.2 even when porosity is as high as 0.8.

Rambabu and Mani (2002) theoretically studied the effectiveness of a submerged breakwater in coastal protection. They aimed to investigate the effect of depth of submergence, crest width and incident wave conditions on transmission characteristics. The computational results showed that, K_t reduced by 45% for a wide range of wave steepness $0.0014 < H_i/gT^2 < 0.0081$ with F/d of 0.5. Further the results indicate an optimum width ratio B/d of 0.75.

In quite some situations low crested structures are not parallel to the coast and waves attack the structures obliquely. And oblique transmission takes place. The overtopping and transmission may reduce in such cases. Van der Meer et al. (2003) investigated this problem and have tried to find out whether the presently available equations accurately predict the transmission.

Diffraction effect of the tip of the permeable submerged breakwater on wave transmission has been studied in 3D model investigations by Dharma and Hall (2004). The results showed that, the parameters like F/H_i, H_i/D_{n50} and B/H_i are most important. A model to predict transmission was developed using statistical analysis. K_t was related to diffraction, wave breaking, overtopping, dissipation due to surface friction and transmission through breakwater. Regression analysis was performed and the following equation was derived.

$$K_t = \{-0.869\exp(F/H_i) + 1.049\exp(-0.003B/H_i) - 0.026(H_iF)/(BD_{n50}) - 0.005B^2/(L\,D_{n50}) + 0.003(h/d)(a/L)\cos(\Phi - \Phi_o)\} \quad (2.34)$$

Where,

d is the depth of water, F is freeboard, H is wave height, L is local wave length, B is breakwater crest width, h is structure height, a is radial distance of the point of interest from the tip of the structure on leeside, Φ is angle of the point of interest and Φ_0 is wave angle.

2.13 REEF BREAKWATER

Reef breakwater refers to a low crested rubble mound breakwater without the traditional multi layer cross section. This type of breakwater is little more than a homogeneous pile of stones with individual stone weights similar to those used in armour of conventional breakwaters (Ahrens 1984, 1989).

In recent years there has been a pronounced expansion of the role of rubble mound breakwaters in coastal engineering. Rubble mound jetties and breakwaters have traditionally protected navigation channels and harbours but now, rubble mounds are being widely used for beach stabilization and shore protection. Since reef structures do not have core, they cannot fail catastrophically. Hence, a logical strategy is to allow them to adjust and deform to some equilibrium condition because of high porosity, reef structures are more stable than rubble mound breakwaters and at the same time they would dissipate wave energy efficiently. It is also believed that their simplicity would be a significant factor in keeping down the construction cost and suggested that a reef breakwater would be an optimum structure type for many situations (Fulford 1985). He discusses the performance of reef breakwater as shoreline stabilization measure and response of the shoreline at Chesapeake Bay, USA against conventional methods like groynes, seawalls and revetment.

In Australia, at Rosslyn Bay, the breakwater was damaged by the cyclone called David in 1976. The crest height was reduced by 4m, but the structure still functioned efficiently as a submerged breakwater. Based on good performance, this damaged structure, a low crested design was chosen for reef breakwater at Townsville Harbour, Australia which was constructed with 3Ton to 5Ton stones (Bremner et al. 1980).

Ahrens (1984) worked on the reef breakwater model of slope 1:1.5, constructed with armour of 17gms to 71gms in water depths of 0.25m to 0.3m with irregular waves of heights varying from 0.01m to 0.18m of period 1.45 sec to 3.6 sec. The area of cross section (A) of the breakwater varied between 0.117 m^2 to 0.19 m^2. He passed 3500 to 4000 waves to get the equilibrium profile of the structure. Ahrens found that the reef was damaged more for waves of long period than short period. Therefore, he included the local wave steepness in a modified spectral stability number N_s^* defined by equation 2.32. He defined damage by

$$D' = A_d/(W_{50}/\gamma_r)^{2/3} \quad \text{.......(2.35)}$$

Where,

A_d is the damaged area of the reef and W_{50} and γ_r are the mean weight and specific weight of the armour respectively.

He found that for the spectral stability number (N_s*) less than or equal to 6, there was little or no stone movement and for the number grater than or equal to 8, there was noticeable stone movement and the damage of the structure. He found that for a given depth, the maximum stable height increased with wave period T and K_t was a function of crest width, depth of submergence and wave steepness. Further he reports that, maximum wave energy reflected was about 18% for long waves for crest at SWL and it was about 8% for short waves. The wave reflection reduced as depth of crest submergence F/H increased.

Sorensen (1987) writes that stabilizing the nourished beach with an offshore breakwater is the concept of perched beach. A low submerged stone reef is constructed offshore and parallel to the shore and sand fill is placed on landward side. The reef holds the perched beach, significantly reducing the sand fill that is required. The sill also acts as a large offshore bar to break strong waves and much of their energy, so that it can serve as a shore protection structure. He tested the model of the reef with a crest width of 0.076m, height 0.152m of slope 1:1.5 in a depth of water of 0.152m to 0.353m. Median armour size was 15gm with 5% stone lighter than 3.7gm and 95% stones lighter than 35gm. In the model test, he verified Ahrens (1984) stability number for regular waves of height 0.049m to 0.198m of period varying from 1.09sec to 2.55sec. He found that the damage increased with increasing wave period. He plotted damage (N_m) as a function of reef height (h/d).

$$N_m = (H^2/L)^{1/3}/((D_{n50})^{1/3}\Delta) \quad \text{.......(2.36)}$$

He observed that for a reef structure of h/d = 0.43 to 1.0, damage starts for $N_m > 5$ which is close the recommended value by Ahrens.

Sawargi et al. (1988) as quoted by Pilarczyk and Zeidler (1996) in their experimental work on submerged breakwater observed that decay of wave height after breaking becomes more rapid when structure is higher. They also discuss an adequate width of submerged breakwater to prevent severe erosion of nourished beach. They conducted experiments on small scale model of 1:2 sloped reefs with water depth of 0.10m on a 1:30 sloping beach. The relative reef height (h/d) was 0.25, 0.55 and 0.75 with relative wave height H/d of 0.5, 0.75, and 0.9. The wave periods T were 0.8sec, 1.1sec and 1.4sec for plunging breakers. They concluded that decay of broken wave height is more rapid for taller reef regardless of H_i/d and thus h/d was more important than H_i/d.

Ahrens (1989) further analyzed the stability of reef structures and calculated its response with respect to the variable called average cotangent of reef as

$$C = A/h_c^2 \qquad \text{(2.37)}$$

Where,

A is the cross section of the reef, h_c is the crest height at equilibrium after damage.

He found that for N_s^* less than 7, there was little or no adjustment of reef armour to the wave action. Then he defined the stability coefficient similar to Hudson's as

$$K_D^* = N_s^* / \log (C) \qquad \text{(2.38)}$$

C can also be calculated as

$$C = Exp(0.0945Ns^*) \qquad \text{(2.39)}$$

He concluded that reefs with greater bulk B_n resist the degradation better than small reefs and are stable if they are of low profile, where, bulk number is calculated as

$$B_n = A/D_{n50} \qquad \text{(2.40)}$$

Kale and Gadre (1989) write about offshore submerged breakwaters as efficient measures for beach protection without affecting the littoral drift significantly. They write about stable offshore submerged breakwater constructed in depths of 2m to 3m which can be economically constructed by chaining the small concrete blocks of weight less than 200kg which can be cast in situ, assembled and easily placed with the help of boats and 4 to 6 people with locally available equipments. They mention about lab test of such submerged breakwaters made of 30Kg to 40Kg concrete blocks with 1: 1.5 and 1:3 slopes with a crest submergence of 0.5 to

1m in water depth of 2 to 3m, the crest width was 1m to 1.5 m. They could achieve a K_t of 0.5 to 0.67 without significant damage to armour units.

Gadre et al. (1992) tested a reef structure as a protection measure for the buried submarine pipelines. The reef was constructed with rocks of weight 0.5Ton to 4Ton placed randomly at a slope of 1:3 in a water depth of 6m, 8m, 10m subjected to maximum wave height limited to breaking wave height in a particular depth, of period 10sec. The test was conducted for regular and irregular waves. They gave the following stability number

$N_S^* = \gamma_r H^3/(W_{50}\Delta^3)$ ....(2.41)

They also gave the deign graph of $(N_S^*)^3$ Vs F/d. The wave heights causing initiation of damage to armour of submerged reef were different for regular and irregular waves and $H_{reg}/H_{rand} = 1.12$.

Van der Meer and d'Angremond (1992) combined experimental data of many researchers, fitted the average line and gave the equations for K_t of submerged breakwater. But they observed that reef breakwaters differ considerably from conventional breakwaters and arrived at the following equation for K_t.

$K_t = a\, F/D_{n50} + b$ ....(2.42)

Where,

$a = 0.031H/D_{n50} - 0.24$ ....(2.43)

$b = -2.6\, S_p - 0.05H/D_{n50} + 0.85$ ....(2.44)

The above values are valid for $1 < H/D_{n50} < 6$ and $0.01 < S_p < 0.05$. They obtained a minimum K_t of 0.015 and a maximum K_t value of 0.6 for $F/D_{n50} < -2$, linearly increasing to 0.8 for $F/D_{n50} = -6$.

Goda (1996) proposed a longitudinal reef system for coastal protection where tidal range is moderate as shown in Fig. 2.9. This is a group of rubble mounds set normal to the coast and is

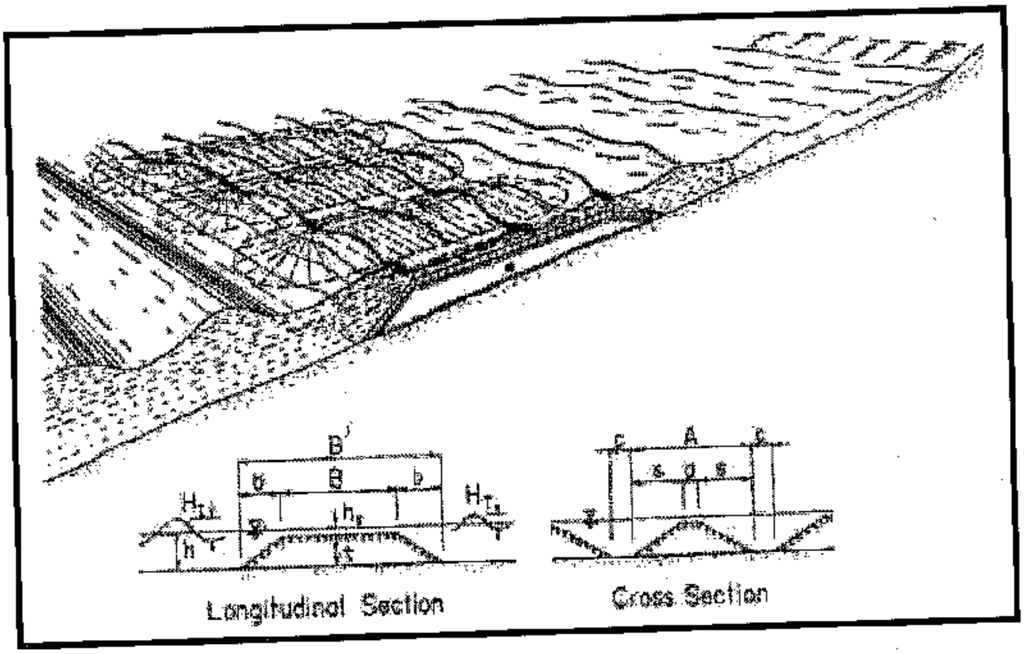

Fig. 2.9. Longitudinal reef system

less obtrusive to view open sea. It uses the wave refraction effect by the side slopes for enhancement of wave breaking and energy dissipation. The amount of rubble mounds required to build the longitudinal reef system is about same as that for submerged breakwater with broad crests. This system in addition of being less obstructive enhances exchange of seawater between offshore and near shore because of broad water passages between its units in contrast to diminished water exchange with detached breakwaters. Goda writes that a submerged reef with broad crest is usually designed with a K_t of 0.6 as a target value and longitudinal system can be designed with similar wave damping characteristics those comparable to submerged reef. He conducted experiments on a longitudinal reef system in a flume. He tested its model of length of 0.5m, 1m and 2m with a height of 0.1m, whose end slopes were 1:2 and side slopes 1:2.5. Because the sidewalls of the flume were regarded as perfect reflective boundary, the model arrangement represents the case with an infinite number of longitudinal reefs set along the coast. The tests were carried out with water depths of 0.08m to 0.14m with armour stone diameter of 0.02m to 0.03m for waves of 0.23m to 0.86m of periods varying from 0.64 sec to 1.4 sec. He concluded that crest width, depth of submergence and spacing between reefs are important parameters and by controlling these parameters, K_t can be brought down to 0.5 or less. He also concluded that this system does not cause piling up of water and reflection and construction costs were comparable with broad crested submerged reef as total volume of rubble was same.

Nizam and Yuwono (1996) conducted experiments on efficiency of artificial reefs as alternative beach protection. They assumed that reef can be designed using concepts similar to the berm breakwater i.e., to allow the structure to reshape and achieve its dynamic equilibrium with hydrodynamic loading. This makes the reef structure more attractive since, a much smaller material than the conventional design approach can be used. They conducted physical model studies on 0.2m to 0.45m high, 1:2 sloped model of reef breakwater with its crest width varying from 0.15m to 0.4m constructed with armour of mean diameter of 0.01m to 0.03m. The model was constructed in a depth of 0.2m to 0.7m and subjected it to regular waves of height 0.0191m to 0.2m with periods varying from 0.89 sec to 2.68 sec. They obtained a K_t of 0.1 to 1.0 and derived the following stability coefficient K to calculate the weight of reef armour.

$$K_D' = \gamma_r H^3/(W_{50}\Delta^3) \quad \text{.........(2.45)}$$

The authors studied the damage of the reef crest in detail and have provided a graph of stability number K_D' Vs relative distance B/X' (i. e. crest width parameter) on either side of the centre line of crest. They concluded that a significant economy can be achieved by zoning the crest width into several parts and reducing armour weights further away from the position of wave breaking.

Submerged reef breakwater is a structure that is optimized to the highest degree. For this structure, the critical conditions of stability are at low water level and it becomes more stable at increased depth of submergence (Smith et al. 1996). This is due to sheltering effect provided by the overlying water cushioning of the impact forces and attenuating the drag forces of the waves.

Franco (2001) writes about perched artificial beach contained by L shaped rock groynes and submerged barrier for enhancing recreation and urban social life was built along the water front of Civitaveachia, Rome. An interesting addition to this scheme was the novel design of artificial reef for surfing of rocky foreshore slope of 1:33.

Armono and Hall (2002) experimentally investigated the wave transmission at hollow hemispherical shaped submerged artificial reef. The model of slope 1:2, crest width 1.0m and height h of 0.22m was constructed with armour of nominal diameter (D_{n50}) of 0.0366m in depths of water d varying from 0.35m to 0.6m. The model was tested for regular and irregular waves of heights varying from 0.05m to 0.2m of periods 1sec, 1.5sec, 2.0sec and 2.5sec. They concluded that for relative height h/d > 0.7, the effect of the reef crest width is visible and as depth of water increases this effect becomes insignificant. They obtained the smallest K_t for h/d value of 0.7 while, it was highest for h/d value of 1.0 and observed that, the difference between wave transmission obtained for regular and irregular waves is not significant.

Bierawski and Maeno (2002) conducted physical model study of impermeable submerged breakwaters and reef breakwater to study the water pressure fluctuation around these structures. They conducted experiments on a 1:2 sloped 0.3m high and 0.3m wide crested breakwater constructed with armour of mean diameter of 0.015m with 35% porosity. The waves of 0.06 to 0.2m height and of period 2sec were generated in a depth of water of 0.3 to 0.6m. They found that largest water pressure and pore pressure fluctuations were in the area located near the seaward edge of the breakwater crest which had highest risk of damage. They assumed that the water exchange process between porous structure and wave field play a significant role in wave propagation inside structure. They found that, large pressure gradients appeared below toe and crest of impermeable structure was destroyed. However, for permeable reef breakwater, the pressure fluctuations were transmitted by moving particles throughout porous media and change in pressure took place on larger distance and the gradients were lesser and very far from horizontal. This made the permeable reef structure safer. They also reported that the influence of permeability of structure on wave attenuation to be minor importance and on wave reflection to be effective only for large relative heights and significance of any changes in structure characteristics becomes larger with freeboard ratio becoming close to zero.

Harris (2003) discusses about performance of submerged artificial reef ball breakwater constructed in Dominican Republic for beach erosion control. The author conducted beach survey 3years after the structure was put in place. The survey showed a shoreline advance of about 10m to 13m.

A cost effective, flexible yet tough Y shaped submerged artificial reef may be constructed with durable and environmentally inert High Density Polyethylene Pipe (HDPP) matrix for beach protection. Ross and Pleskunas (2003) showed that, such a structure breaks waves coming from all directions reducing the inshore wave energy encouraging beach development without disrupting the alongshore sediment transport. The structure is highly stable, can be installed and removed whenever desired. It can be deployed and anchored with a high level of precision. Anchoring can be through application of ballast of wet sand or concrete.

2.14 PROTECTED STRUCTURES

Designing breakwaters is primarily based on selection of design wave height. The designers' dilemma is to select the wave height from $H_{1/10}$, H_s or a wave height which creates a resonance condition or a wave height with 50 or 100 years return period. Even under such a design load, typical damage may occur between 2% to 5% of the armour units (Mol et al. 1984). Which one of this if selected for the design will guarantee against the hydraulic failure? In fact, one cannot rule out possibility of a cyclone creating a huge wave which may cause severe damage to the structure or initiate gradual morphological changes, changes in site conditions triggering the damage of the breakwater. However good the available data, there is always some doubt and considerable uncertainty about wave conditions during storm attack (Owen and Briggs 1986). Hence, there is always this element of risk inbuilt in the design of breakwaters. We can read many such cases of breakwater failure in the literature. So what is the solution? One solution could be the protection to breakwaters which can mitigate damage of storm waves to some extent. Though his concept was evolved in 1956 by Danel (Cox and Clark 1992), it was realised only in 1980's (Groeneveld et al. 1984 and Gadre et al. 1985). The submerged structure can be used for shore defense or protection to moderate the waves that impinge on shoreline or primary structure (refer Fig 2.10).

The wave energy transmitted past the submerged breakwater is dissipated through various mechanisms such as wave breaking, bottom friction and generation of turbulence. This process is known to depend upon the characteristics of the submerged breakwater such as its height, shape, permeability and roughness as well as wave climate (Cornett et al. 1993). A breakwater under wave attack gradually assumes an equilibrium shape of a berm breakwater. Danel recognized that berm is actually excess material which can be removed without much hassles and shifted seaward to form a submerged reef without impact on the breakwater stability. Thus, a breakwater and a submerged reef evolved as a concept of tandem breakwater, where, two structures act together and resist extreme wave loads (Cox and Clark 1992).

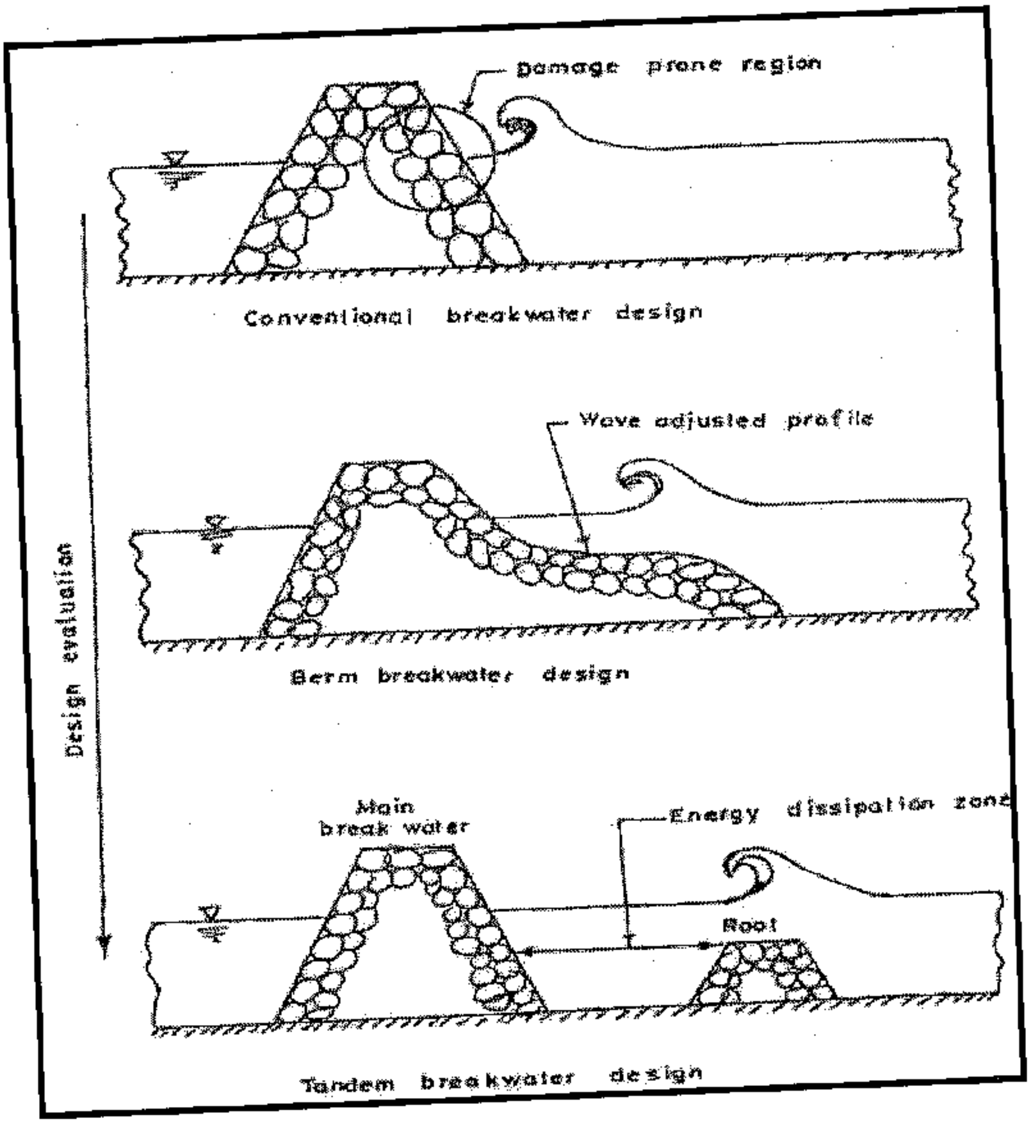

Fig. 2.10. Evolution of tandem breakwater

Groeneveld et al. (1984) recognized the construction of a submerged barrier in front of a breakwater or providing a submerged berm attached to the seaward face of the breakwater as rehabilitation measures for damaged breakwaters as submerged barriers break high waves and transmit the attenuated waves. K_t was always found to be greater than 0.4. But the submerged reefs are usually designed with a K_t of 0.6 (Goda 1996).

Based on limited study Gadre et al. (1985) designed a submerged bund in front a revetment for reclamation of land between outer harbour and fisheries harbour north of Bharathi dock of Madras Port, Chennai India. Refer Fig. 2.11.

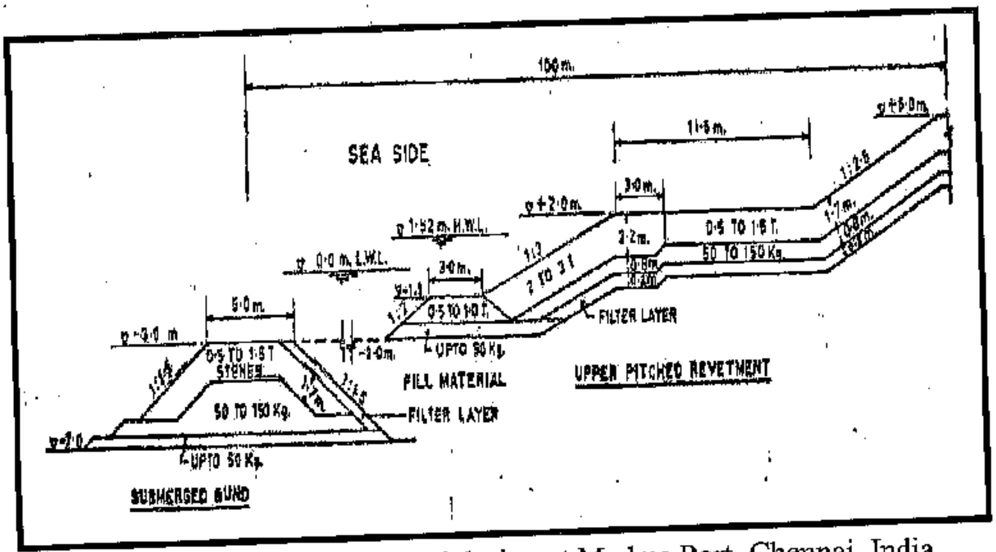

Fig. 2.11. Reclamation bund design at Madras Port, Chennai, India

The normal conventional type of reclamation bund in a water depth of 8m would require armour stones of 15Ton or tetrapods of 6.5Ton on the slope of 1:2 to withstand waves of 5m height in a depth of 7m considering the non-availability of stones over 2Ton to 3Ton, in the vicinity of port, it was suggested to construct a submerged bund, pump sand and then provide upper pitched revetment. This arrangement broke higher waves over the submerged bund and dissipated wave energy due to friction offered by the reclamation fill in between the two structures. This resulted in a low cost construction in comparison with conventional type of bund. The authors conducted a model study of scale 1:30 on a submerged breakwater of slope 1:1.5, crest width B/d of 0.625 and height h/d of 0.5 at a seaward distance X/d of 14.2 was used as a protection to reclamation bund such that the submergence F/H was -0.6 to -0.65, and wave steepness H/gT^2 of 0.0051 where, X, B, h and d are the spacing between two structures, crest width of the submerged breakwater, reef height of the submerged breakwater and depth of water respectively (Gadre et al. 1985). They tested it for regular as well as for irregular waves. The structure broke 5m design wave and transmitted a 4m wave ($K_t = 0.8$) and these waves further got attenuated by the friction offered by the reclamation fill between two bunds and only a wave of height of 3.5m impinged on the inner revetment. Hence, it now required armour of weight of 2Ton to 3Ton at a slope of 1:3 and stones of 0.5 ton to 1.5 ton at a slope of 1:2.5 at higher level where, it was constructed as a berm type structure. The submerged bund of 4m high was constructed in a depth of -7m to -3m with a crest width of 5m with armour of 0.5Ton to 1.5Ton at a slope of 1:1.5. There was no damage of submerged bund.

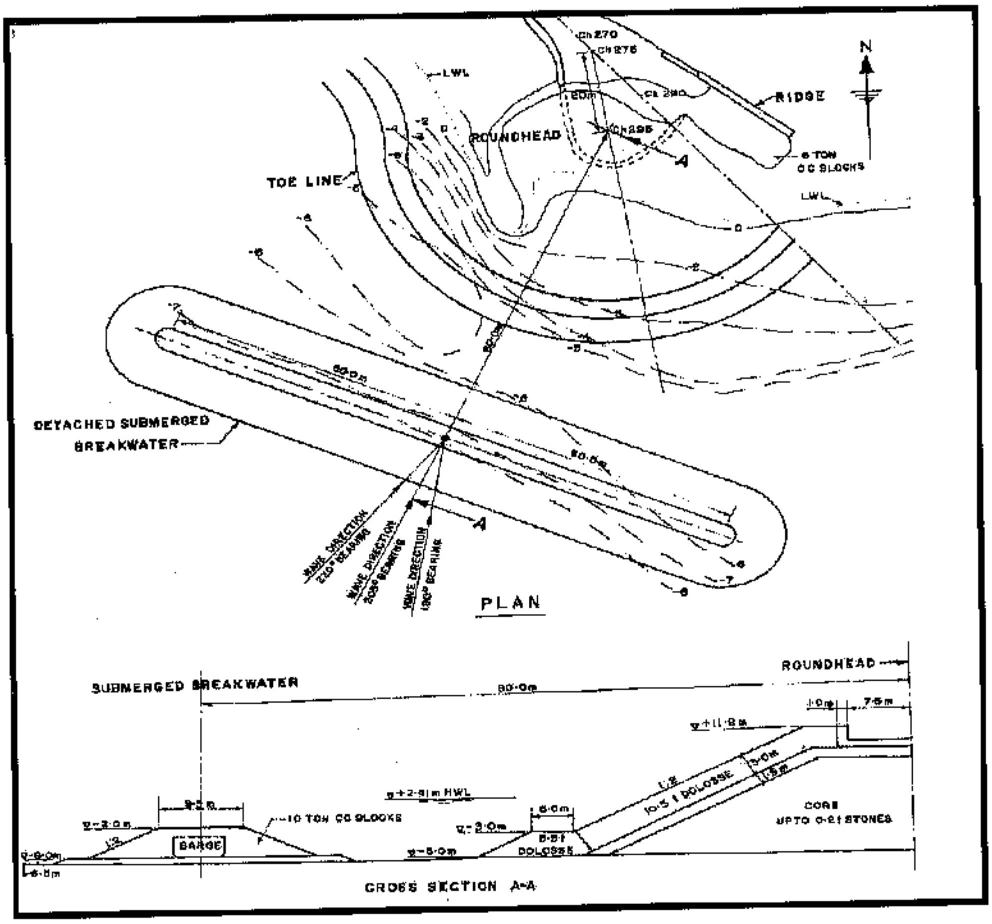

Fig. 2.12. Submerged breakwater protection at Veraval Port, Gujarat, India

In 1989, at Veraval Port in Gujarat, India, the breakwater head was damaged due to extreme waves crossing 9m in height. Several rehabilitation measures were tried. Ultimately it was decided to construct a submerged breakwater at 80m seaward of the damaged structure which was selected as a solution (refer Fig. 2.12). A submerged breakwater, of crest width B/d of 1.06 and height h/d of 0.5 at a seaward distance X/d of 8.88, with a crest submergence F/H of -0.22, was used as a rehabilitation structure for a damaged breakwater head at Veraval Port Gujarat, India, to secure it from storm waves (Gadre et al. 1989).

In 1990, it was decided to build a marina at Hammond, Indiana on the southern tip of Lake Michigan, USA. The normal armour weight required was 8Ton on a slope of 1:1.5 for a conventional non-overtopping structure. Cox and Clark (1992) conducted experimental studies and investigated the impact of a seaward submerged reef on the inner breakwater separated by a distance of 22m to 38m. They found that a submerged reef of 6.9m width and 3.9m to 4.95m height attenuated the incident waves up to 33% in water depth of 5.4m to 6.45m. Based on limited study, they found that a reef structure of crest width B/d of 1.53 and of height h/d of 0.86, constructed at a seaward distance X/d of 9.0, for protecting a breakwater, attenuated the waves considerably. According to them the transmission finally becomes depth limited and can be described by the empirical expression.

$$K_t = 0.8((-F/H)+0.5)^{0.5} \qquad \text{(2.46)}$$

Where,

K_t is transmission coefficient, F is free board (negative for submerged breakwater) and H is incident wave height.

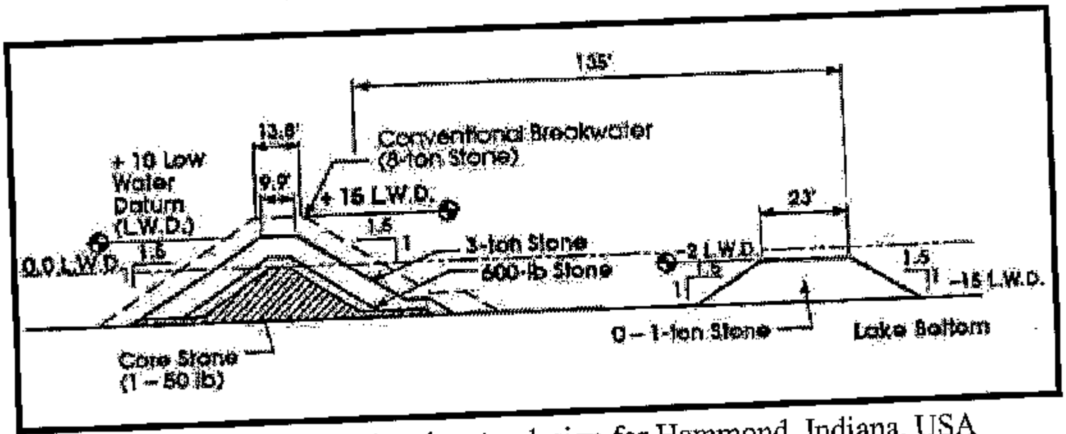

Fig. 2.13. Tandem breakwater design for Hammond, Indiana, USA

Cox and Clark (1992) finally recommended construction of a breakwater with an armour of 3Ton and lowered the crest by 1.5m with a seaward reef constructed with armour of weight up to 1Ton at about 40.5m from the breakwater as shown in Fig. 2.13. They called these structures as tandem breakwater, because, reef and breakwater acting together as a single unit resisted the onslaught of extreme waves. By constructing the tandem breakwater, they saved $1million. They concluded that depending upon site conditions; a tandem breakwater may prove to be an optimum structure.

Cornett et al. (1993) observe that submerged sand bars, shoals and reef are known to trigger shift wave energy from fundamental to higher frequencies through harmonic coupling of higher order wave components from their fundamental carriers. This effect is quite pronounced for a broad shallow sloping shoal according to Beji and Battjes (1993) as quoted by Cornett et al. (1993). Tests with regular waves indicate a strong spatial variation in wave characteristics down wave from submerged breakwater, depending upon phasing of free higher harmonics relative to fundamental wave components. But with low crested steeply sloping reef breakwaters, less dramatic transformations occur and even these subtle wave transformations can have important effects on near shore processes and performances of shore protection structures. The stability of protected structure is dependent on transmission of wave energy and transmission of extreme waves and extreme loading events. Cornett et al. (1993) conducted a limited set of small scale physical model tests on tandem breakwater system. They constructed a breakwater of medium armour diameter of 0.042m placed on a uniform slope of 1:1.75 in a water depth of 0.55m. They built reef breakwater with slope 1:1.5, a crest width of 0.1m (i.e.B/d of 0.18) and heights of 0.17m, 0.25m and 0.33m (i.e. h/d of 0.31 to 0.60) at a seaward distance of 2.11m from the breakwater (i.e. X/d of 3.83). The model was tested for regular and irregular waves of heights 0.12m and 0.2m of periods 1.5sec to 3sec. They passed 320 to 440 waves. Authors observed that at $F/H_i = 2.5$, K_t approaches 1.0 and then it remains unaffected for $F/H_i > 2.5$. The character of waves transmitted at reef breakwater has considerable effect on design and performances of additional protected structures such as breakwaters or revetments. They found that, the reflection coefficient K_r increased with reef submergence F/H. It was observed that maximum K_r was about 0.27 for F/H of -1.5 and decreased to zero for F/H < -3.0.

Authors gave alternative expression for stability in terms of number of stones eroded N_A from a particular test section of the breakwater with width B_{ts} as:

$$S = N_A D_{n50}/(B_{ts}(1-p)) \qquad \text{.........(2.47)}$$

Where,

S is the dimensionless damage given by A/D^2 n50 and Cornett et al (1993) quotes Vidal and Mansard (1993) that initiation of damage starts for S = 1 and start of destruction for $S \geq 4$ and p is the porosity,

Cornett et al. (1993) concluded that,

1. For a reef of h/d < 0.6, no significant wave attenuation occurred.
2. For reef of h/d > 0.6 and X/L = 2, the incident wave is reduced by 5% at the toe of breakwater, wave loading on breakwater armour is reduced by 15% while its damage is reduced by 50%.
3. For reef of h/d >0.6, largest wave heights are significantly reduced.
4. Wave period influences wave transmission at reef and also performance of tandem breakwater.

5. Depending upon geometry of tandem structures, an optimal spacing between the structures may exist.

Even modest reduction in wave heights and load levels can have dramatic effect on performance of breakwater armour. In conditions near the threshold of armour stone motion, such a reduction in loading can make the difference between extensive damage and no damage. Higher and broader reefs are likely to cause more pronounced wave transformation, greater reduction to extreme loading events and even more dramatic attenuation of damage. Such attenuation would allow more economic design of breakwater (Cornett et al. 1993). Further, they observe that considerably more research and testing of reef breakwaters and tandem breakwaters concept are required to develop a complete understanding of the transformation to transmitted waves, loading events and implications for the performance of diverse tandem breakwater systems.

For a new yacht harbour in Salerno, Italy, to lower the crest by about 2.5m for visual reasons, the designers chose tandem breakwater (Franco 2001).

Neelamani et al. (2002) experimentally investigated the hydraulic performance of an impermeable plain seawall, of varying inclination θ from 30° to 90°, defenced by a detached

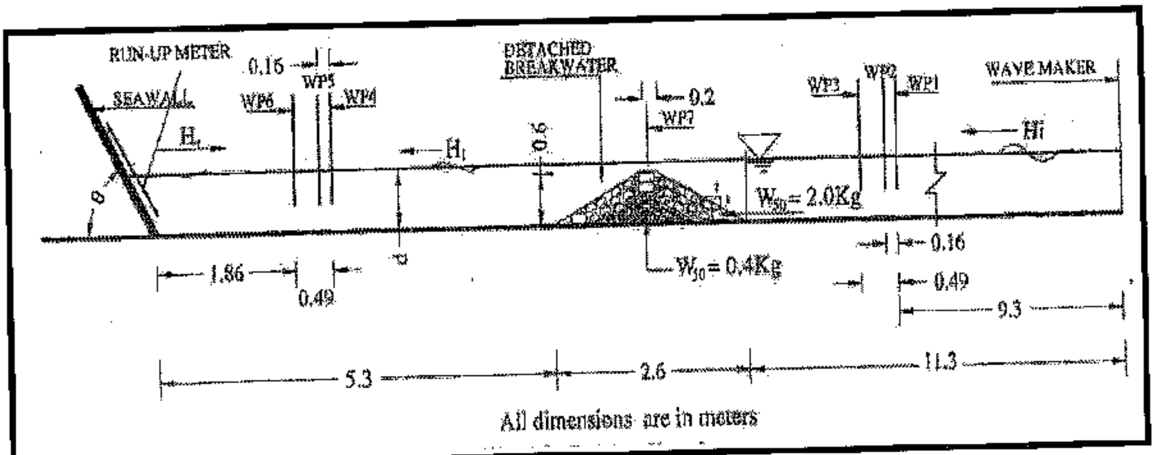

Fig. 2.14. Defenced seawall

breakwater as shown in Fig. 2.14. They concluded that, for a breakwater of width (B/d) of 0.25 to 0.40 and height (h/d) of 0.75 to 1.2, constructed a seaward distance (X/d) of 6.63 to 10.6 from the seawall, wave pressures on the seawall are reduced by 45%, wave run up and run down on the seawall are reduced by 10% to 40% and 10% to 70% respectively and reflections are reduced by about 10% for breakwater submergence of -0.33 < F/H < -4.0 and wave steepness of $0.004 < H/gT^2 < 0.115$.

Table 2.4 compares the parameters of field and model tests of protected structures as given by various investigators. Gadre et al. (1985), Gadre et al. (1989) and Cox and Clark (1992) report prototype studies and whereas Cornett et al. (1993) and Neelamani et al. (2002) conducted physical model studies in the laboratory.

In the design of the conventional breakwater, a wider toe berm of about 20m, instead of about 3m to 5m, constructed just at low water level with stones of 10% to 20% of the primary armour acts as a rehabilitation measure for a damaged breakwater and save the cost substantially say to the tune of 30% to 40% (Poonawala et al. 2004).

2.15 SUMMARY

Breakwaters have been built through out the centuries but their structural development as well as their design procedure is still under massive change. New ideas and developments are in the process of being tested regarding breakwater layout for reducing wave loads and failures. Breakwater design is increasingly influenced by environmental, social, aesthetical aspects and new type of structures are being proposed and built.

Till now the submerged breakwaters were used only for beach protection, harbour protection and rehabilitation of damaged breakwater. But they can also be used for the protection of rubble mound breakwater for stability. Breakwaters' vulnerability to extreme events such as storms is a reality. However, good the available data, there is always some doubt and considerable uncertainty about wave conditions during storm attack. Hence, there is always this element of risk inbuilt in the design of breakwaters which may cause damage or failure. We get number of such cases in the literature.

The failure or significant damage of conventional breakwater, due to onslaught of extreme waves, may have disastrous consequences. This may be catastrophic and extremely costly or in some cases impossible to rebuild the structure. One of the things engineers can do, is to design a protective structure in the front, which will withstand, resist and manage such destructive extreme events at least to some extent and at the same time, mitigate large damage of the inner main breakwater.

Table. 2.4 Comparison of field and model parameters of protected structures as tested by various investigators

Parameter	Gadre et al. (1985)	Gadre et al. (1989)	Cox and Clark (1992)	Cornett et al. (1993)	Neelamani et al. (2002)
X (m)	100	80	40.5	2.11	5.3
d (m)	7.0	9.0	4.5	0.55	0.5 – 0.8
B (m)	5.0	9.5	6.9	0.1	0.2
h (m)	4.0	4.5	3.87	0.17- 0.33	0.6
F (m)	3.0 - 4.52	2.0	0.6	0.1 – 0.38	0.1 – 0.2
H (m)	5.0	7.0 – 9.0	5.82 – 7.08	0.12 – 0.2	0.05 – 0.3
T (sec)	10.0	-----	10.3 – 11.7	1.0 – 3.0	1.0 – 3.0
X/d	14.2	8.88	9.0	3.83	6.63 – 10.6
B/d	0.714	1.06	1.5	0.18	0.25 – 0.4
h/d	0.57	0.5	0.86	0.3 – 0.6	0.75 – 1.2
F/H	0.6 – 0.9	0.22 – 0.29	0.085 - 0.103	0.5 – 3.17	0.33 – 4.0
H/gT^2	0.0051	-----	0.0043 – 0.0068	0.0014 – 0.02	0.00057 – 0.03

A submerged reef, which, is an optimized structure to highest degree is the one that may be located at a certain distance seaward of the breakwater as a protective structure. The wave breaking over reef causes great turbulence on lee side. They also offer resistance through friction and turbulence created by their interference in wave field causing maximum wave damping, energy dissipation and attenuation of the waves which then attack the breakwater within a tolerable level. The design of this type of combined structure is complex and requires detailed information on parameters such as water level changes, 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 etc.

Therefore, more research is needed to understand their mechanism and performance and design such protected structures confidently.

Chapter 3
Developments in Physical Model Testing

3.1 INTRODUCTION

Breakwaters have been built through out the centuries but their structural development as well as their design procedure is still under massive change. New ideas and developments are in the process of being tested regarding breakwater layout, reduction of wave load and failure of breakwaters. Designs are being increasingly influenced by environmental, social and aesthetical aspects transforming the breakwater into a complex structure. Under such situation where, the design/analysis of the prototype structure is very complicated, the use of physical models is particularly advantageous.

The stability of rubble mound breakwaters under wave attack is an important aspect of the design. Until early 1990s the design practices of breakwaters were based on hydraulic model tests and empirical formula. Most reliable and quantitative data, on effect of submerged breakwater on wave action, were those obtained in laboratory model studies. Because of the complexity of the problem, theoretical development is over simplified and therefore, greater reliance should be placed on lab and field (Khader and Rai 1980). Very few numerical techniques have been developed recently to check the stability of these structures (Kudale and Dattatri 1994 and Van Gent and Vis 1994). Very recently software such as HISWA, SWAN, EMS and MIKE21 and BREAKWAT.3 have been developed and is used for designing different types of breakwaters (Delft Hydraulics 2001, Nagendra Kumar 2001 and Mai et al. 1999). Many researchers have developed mathematical modeling to compute wave transmission at submerged breakwater using conservation of mass and momentum equations (Kobayashi and Wurjanto 1989). Sollit and Cross (1970) and Madsen and White (1976) developed a theory to predict wave reflection and transmission at permeable breakwaters. Mani et al. (1991) and Rambabu and Mani (2001 and 2002) also developed theories to predict transmission coefficients for the submerged breakwaters. Though numerical models have been developed as a design tool for many types of breakwaters after 1990, the necessity of physical model study has not diminished. They are still used to validate numerical models and are used when parameters involved are complicated or it is not possible to accurately represent them mathematically or to calibrate the parameters. Physical model studies are the only way out when the geometry of the structure to be tested is unusual and parameters involved interact in complex patterns (Chakrabarthi 1996 and Mai et al. 1999).

Physical models are a close representation of reality in which a prototype system is duplicated as closely as possible in a (generally) smaller scale. The purpose of the model is to approximate and anticipate the prototype behaviour through certain prescribed modeling laws. There are many modeling approaches that are followed in the study of the natural systems. The most important of these are physical models and mathematical models. The physical model provides insight into a physical phenomenon which is not fully understood (Chakrabarthi 1996).

Model tests have been considered as a powerful tool in breakwater design. In Italy, where, the design of breakwaters and other structures at sea is being increasingly influenced by nature conservation, peoples' recreation and sporting activities (Franco 2001). This calls for innovative shapes or structures with aesthetical advantages and conventional construction materials. In all these situations model study is the only design tool. Even in cases where mathematical modeling is possible, many researchers opine that to have a feel of the whole situation and confirm the observations made it is always better to supplement mathematical modeling with physical model study (Pilarczyk and Zeidler 1996).

Model results are normally transferred to prototype by applying Froude's model law. However, the extent of scale and model effects, involved when the Froude's model law is applied, has not been fully resolved. By comparing a number of known breakwater damages with associated model investigations, (Hudson 1975) as quoted by Torum et al (1979) considered that hydraulic scale models can be used to determine stability of rubble mound breakwater and that accuracy of model results will be within the limits required to design safe and economical full scale structures, if, models are designed and operated correctly and if test conditions are selected judiciously.

The scale effects and uncertainty are the two major issues those decide the reliability of the model studies. To reduce scale effects the model should be as large as possible (Hughes 1993), so that the Reynold number of flow is high and flow is turbulent (Ouellet 1970). And to minimize uncertainty the experiment has to be properly planned, experimental procedures and extrapolation methods should be standardized and sources of errors and magnitude of errors have to be minimized (Mishra 2001). Standardized procedures techniques used in model tests enable the comparison of test results of different labs (Zwamborn 1979).

This chapter explains the importance of dimensional analysis and need for planning the experiment, discusses model scale selection, limitations of model testing, scale effects and uncertainty. It also describes the standard test procedure of testing of breakwater models.

3.2 DIMENSIONAL ANALYSIS

Dimensional analysis is a rational procedure for combining physical variables in to dimensionless products, thereby reducing the number of variables those need to be considered (Hughes 1993). Dimensional analysis, a method by which we deduce information about a phenomenon, can be described by a dimensionally correct equation among certain variables. The generality of the method is both its strength and its weakness. With little effort, a partial solution to nearly any problem is obtained. On the other hand, a complete solution is not obtained, nor is the inner mechanism of a phenomenon revealed by dimensional reasoning alone.

In itself, dimensional analysis gives qualitative rather than quantitative relationships, but when combined with experimental procedures it may be made to supply quantitative results and accurate prediction equations. Dimensional analysis is for poor substitute for theory, which is able to group successfully a few experimental variables when the phenomenon is too complex to resort to a more theoretical approach. In the study of extremely complex problems with theoretical difficulties, dimensional analysis may be the only possible approach to classify experimental results. Hudson et al. (1979) as quoted by Hughes (1993) put it nicely when they said that the best method for solving coastal problems is by direct solution of the growing differential equations. However, when this is not possible, which is often the case, the method of dimensional analysis can be used to great advantage. The advantages of dimensional analysis are:

1. Forming dimensionless products reduces the number of variables that must be investigated, either experimentally, numerically or via field measurements.
2. Dimensionless graphs provide much more information than when dimensions are included because it is possible to cover a wider range of the parameters.

3. Points on dimensionless graphs can frequently be determined using models scaled in such a way that the dimensionless products are preserved at reduced scale.
4. Dimensionless products allow tests to be planned and experimental results to be presented in a condensed and systematic manner.
5. Dimensionless products can be used as the basis for scale model design and interpretation of results.

The dimensional analysis is an important tool correlating a particular trait of a model and other independent parameters which influence the trait. The particular trait may be influenced by a large number of parameters. Investigating such a large number of variables through model study is time consuming and laborious task. With the help of dimensional analysis we can club the different variables to form a non-dimensional PI terms. These PI terms are helpful in reducing the number of variables and also explaining the particular trait of the structure with respect to some familiar terms like Reynolds number, Froude number, relative depth, wave steepness parameter relative width etc.(Carver and Davidson 1982 and Hughes 1993). This trait can then be diagrammatically represented through the graphs depicting its variation with respect to the PI terms. The following examples indicate the dependence of performance characteristics of a breakwater and other independent variables.

Ouellet (1972) recognized that wave run up (R_u) was a function of structure slope (cot α), wave height (H), wave period (T), water depth (d) and surface roughness (e). This relationship can written as:

$$\frac{R_u}{H} = f\{H/gT^2,\ d/H,\ cot\ \alpha,\ e/D_{n50}\} \qquad (3.1)$$

Hudson (1959) recognized that weight of the breakwater armour (W) is a function of wave height, specific weight of armour stone (γ_r), specific weight of sea water (γ_w), and stability coefficient K_D. This relationship is expressed as

$$N_S = H/D_{n50} = f\{K_D,\ cot\ \alpha,\ \Delta\} \qquad (3.2)$$

Where,
Δ is relative mass density of armour which is a function of γ_a and γ_w, D_{n50} is nominal diameter of the armour given by $\left(\frac{W}{\gamma_r}\right)^{1/3}$.

Johnson et al. (1951) deduced the transmission coefficient K_t of the submerged breakwater as a function of wave height, wave period, crest width (B), height of the structure (h) and water depth. This can be written as:

$K_t = f\{H, T, B, h, d\}$ OR

$K_t = f\{H/L, B/L, h/d, d/L\}$ ....(3.3)

3.3 PHYSICAL MODEL LAWS

Modeling laws relate the behaviour of a prototype to that of a scaled model in a prescribed manner. The problem in scaling is to derive an appropriate law that accurately describes this similarity. A parametric approach of relating the model properties is used when little is known about the governing equations of the system (Chakrabarti 1996).

Hydrodynamic scaling laws are derived from the ratio of forces commonly encountered in a hydrodynamic model test. Generally wave structure interaction problems involve Froude Number ($F_r = V/\sqrt{(gL)}$) and Reynold Number ($R_e = Vd/\upsilon$). Over ninety percent of the coastal engineering problems are scaled according to Froude's Law (Hughes 1993 and Chakrabarti 1996). R_e is also important in many cases. However, Reynold's similarity is quite difficult, if not impossible to achieve in a small scale model. Froude's Law is the accepted method of modeling in hydrodynamics. Experiments have shown that the flow characteristics are most likely to be laminar for a critical R_e less than $1\text{x}10^5$, whereas, it is turbulent for R_e greater than $1\text{x}10^6$. But many researchers have stated smaller R_e of $3\text{x}10^3$ to $3\text{x}10^4$ for maintaining turbulent flow in the model (Owen and Briggs 1986 and Hughes 1993). Boundary layer can be made turbulent by introducing roughness on the model surface which trips the laminar flow. Once the flow region is turbulent, the drag effect is weakly dependent upon Reynolds number (Chakrabarti 1996).

3.3.1 Classification of models

Coastal engineers commonly employ two fundamental types of physical models: fixed bed models to study hydrodynamic phenomena in the coastal regime and movable bed models to study effects of water motion on deposition and transport of sediment. Only fixed bed models are discussed here.

Hydrodynamics involves waves and currents. Wave motions can be separated into two logical divisions: short waves which have wave periods in nature from 1sec to 20sec and long waves which can have wave periods in nature between few minutes and several hours. The division of waves, into two types by period, is well suited to modeling, because, in each case certain terms in the governing equations are dominant while other terms are less important. This simplifies model scaling requirements to better fit the assumption that dynamic similarity is achieved by a balance between only two dominant forces. Only short wave models will be discussed here.

3.3.1.1 Short wave models

They are used to study wind waves and swell effects on coastal structures, beaches and navigation. The following are a few problems those can be studied in a shortwave fixed bed physical model (Hughes 1993):

1. Wave transformations like shoaling, diffraction, refraction, breaking, wave force on structures, reflection, transmission, wave run up and overtopping.
2. Determination of most economical breakwater and/or jetty configurations that will provide wave protection.
3. Quantification of wave heights that penetrate into harbour.
4. Evaluation of proposed design modifications which could significantly reduce construction costs and still provide adequate harbour protection.
5. Development of qualitative information on effects of structures on the littoral processes.

Anytime one or more of the established similitude criteria are not satisfied in a model, the model is referred to as a distorted model. In an undistorted model, all dimensionless PI terms determined from important independent variables of the problem, must be the same in the model and prototype. For fluid flow problems of interest to engineers, similitude of all dimensionless PI terms is impossible. Even when considering only the Froude and Reynold criteria, hydraulic models conducted at reduced scale most likely will not satisfy both criteria simultaneously. Therefore, in a strict sense, practically all hydraulic models are distorted.

However, hydraulic model engineers have chosen to relax the definition for model distortion by writing the term distortion to consideration of only geometric similitude i.e. models that maintain geometric similitude are referred to as undistorted models (Hughes 1993). In these models the vertical and horizontal scales are the same, and they represent the true geometric reproduction (usually a miniature version) of the prototype. Examination of similitude requirements resulting form the fluid equation of motion reveals that, flow patterns and velocity distributions in short wave models are essentially governed by inertia and gravity effects and model must be geometrically undistorted.

Shortwave models are considered to be non-dissipative or fully turbulent. In other words, the scaling of these models assumes that waves experience negligible energy loss due to friction and surface tension effects prior to wave breaking and the models can have highly turbulent flow and energy dissipation over a relatively short distance such as during wave breaking. In reality, there will always be a small amount of wave attenuation due to viscosity and surface tension, but these can be minimized to the point that they are insignificant. Reynold's similitude is seldom invoked for most coastal processes or harbour models, instead it is recognized that gravity forces predominate in free surface flows discounting the viscous forces and consequently most models are designed using Froude's criteria (Hughes 1993).

3.4 LABORATORY AND SCALE EFFECTS IN SHORT WAVE MODEL

Laboratory effects and scale effects are the two most important factors affecting scale model results (Hughes 1993).

3.4.1 Laboratory effects

Laboratory effects in short wave models are primarily related to

1. Physical constraints of boundaries on the flow.
2. Unintentional nonlinear effects brought by using mechanical means of wave.
3. Simplification of prototype forcing conditions such as representing prototype wave conditions as unidirectional.

A not so obvious boundary effect is caused by reflection of waves by the wave board. Waves are generated and propagate down the flume until they reach the structure on the far end. Some wave energy is reflected seaward (toward the board), just as happens in nature.

However, in nature the reflected wave continue out into ocean, whereas in flume, they are again reflected back toward the beach. This can be dealt with in a number of ways (Hughes 1993).

1. Experiments are conducted as a series of wave bursts, with each burst of waves ending before re-reflected waves can again reach the testing section of the wave flume.
2. Active wave absorption is implemented at the wave board to detect and absorb unwanted reflected wave energy.
3. Energy dissipating beaches.

3.4.2 Scale effects

Scale effects in short wave models result from scaling assumption that gravity is the dominant physical force balancing the inertial forces i.e. Froude scaling. This incorrectly scales other physical forces of viscosity, elasticity, surface tension etc, with the belief that these forces contribute little to the physical processes.

In the models scaled according to Froude criterion, the non-similitude of viscous and surface tension forces can lead to scale effects involving wave transformations, wave energy dissipation and wave breaking.

Bruun (1970) as quoted by Hughes (1993) concluded that the reason for scale effects in small-scale tests could be that the downward velocities tend to be relatively higher. This might be due to the friction against flow and permeability of stone layer resulting in less interchange of water between uprush and water in mound. Also the scale effects are caused due to change in permeability, type of placing and degree of compaction achieved between the model and prototype. The scale effects are also because of the deviation in sea state produced in the model compared to the prevailing sea state of prototype. Rubble mound structure models must have turbulent flow conditions through out the primary armour layer. This is ensured when model is constructed at a large enough scale which then will be reasonably satisfy similarity of armour layer Reynolds number. To reduce the surface roughness of the armour in model and match it with that of prototype, technique of painting of model armour is used.

Oullet (1970) writes that the flow in under layer if not turbulent, use of regular instead of irregular waves, sidewall effects of flume etc., may contribute to scale effects.

Thompsen et al. (1972) showed that for $R_e < 3.0 \times 10^5$, the armour stability increased with the increase of R_e. The zero damage stability number obtained for the small scale models must be increased to 20 to 70%, depending on model Reynolds number so that the test results are on the safe side for the range of Reynolds number usually encountered.

The reflection coefficients for rip rap in the model are higher than experienced at full scale (Le Mehaute 1976) because, the flow through rip rap is influenced by viscous effects in the model and consequently the structure behaves as if it is less porous than prototype. This is corrected in the model by increasing the size of rip rap using larger scales. The larger scale assures that the flow into the structure is fully turbulent like in prototype. Froude criteria do not correctly simulate the viscous and frictional effects because Reynold number is different between the prototype and the model and waves are attenuated due to internal friction and by bottom boundary layer friction arising out of viscosity of the water. However, this is usually not very important over short distances modeled in short wave models (Le Mehaute 1976). Over short distances, internal friction is minimal and viscous dissipation effects in non-breaking waves are limited to the thin boundary layer.

The process of wave breaking on a coastal structure is important to coastal engineering physical models and it is important that wave breaking in the model produces the same hydrodynamic response as in prototype in breaking waves; entrained air bubbles are large in the model due to surface tension. Le Mehaute (1976) stated that the process of energy dissipation during wave breaking will be in similitude even if the finer details of the flow process are different.

Stive (1985) provided experimental confirmation of Le Mehaute's concept by conducting small scale and large scale tests of regular and irregular breaking waves. Tests were scaled according to the undistorted Froude criterion. He examined measured values of wave heights, wave setup and the vertical profiles of maximum seaward and shoreward and time mean horizontal water velocities. Stive's results indicated that there was no significant departure from Froude scaling in a wave height range of 0.1m to 1.5m with regard to the parameters measured. This result proved that observed difference in air entrainment between model and prototypes have no significant dynamic influence, which is similar to the observation of Le Mehaute (1976) that as the wave break on armour units in a physical model, the flow is turbulent so it is expected that pressure produced in the model are in similitude with the prototype.

Scale model experiments usually are conducted using fresh water. If the prototype condition is salt water, there is about a 3% difference in density and this changes wave force accordingly. This effect could cause as much as 10% to 15% error in breakwater stability studies (Le Mehaute 1976). Sharp and Khader (1984) as quoted by Hughes (1993) conducted 1:10 scale model tests where fresh water in the model represented salt water prototype. This particular example indicated that Hudson scaling called for model armour units about 8% lighter than suggested by Sharp and Khader's method. This means that scaling by Hudson stability parameter (i.e. keeping the same stability number between prototype and model) will produce more conservative designs for stable rubble mound structures because the lighter model armour units are more easily moved by waves.

Surface roughness of armour units in model may be large enough which may show higher stability than its prototype equivalent where surface roughness is negligible. This may give rise to friction scale effect. It can be reduced by painting the armour stones with enamel paints which provides a smoother surface (Hughes 1993).

3.5 SELECTION OF SCALE FOR THE MODEL

The best model is the prototype. But the limitations of the lab, most of the times, do not permit testing of a prototype. The choice of scale, for the model test, is often limited by constraints put by experimental facilities available. Within this constraint, an optimum scale should be selected by comparing the economies of the scale model with that of the experiment (Hughes 1993 and Chakrabarti 1996). As the general rule, the model should be as large as possible. This is advantageous on several counts.

1. Large models require high waves to be generated. This makes the flow turbulent whose Reynold number is high and viscosity effects can be conveniently neglected.
2. Large models are easily and accurately constructed to the dimensions derived from model law

3. The performance characteristics such as wave transmission, run up, run down, breakwater damage etc will be large enough to be measured accurately.
4. Instrumentation for measurements will be generally available
5. Observations of the model performance also can be easily recorded.
6. Researcher feels better of the whole situation
7. Scale effects will be minimal.

The disadvantages of large model are:

1. Model construction is costly.
2. Test duration increases.
3. Large laboratory space is required.
4. Larger resources are required.

Hughes (1993) quotes Hudson and Jackson (1953) in which it was demonstrated that prototype damage of San Pedro breakwater could be reasonably reproduced at scales of 1:30, 1:45 and 1:60.

Diephuis (1957) tested small scale glass models of tall and wide breakwaters. He found that considerable scale effects existed in transmission of energy for a wave breaking over the submerged breakwater. For same F/L_o and H_o/L_o, transmitted wave energy for higher period (i.e. 0.55 sec) was 3 to 4 times that of shorter period (i.e. 0.31sec). Decrease in wave does not remain constant for waves having same steepness but different periods. He concluded that it was impossible to obtain absolute quantitative data from small models because the side effects impose too many limitations.

Many scientists and investigators have adopted\recommended different scales for testing breakwater models in the lab. Dai and Kamel (1969) as quoted by Hughes (1993) combined a series of small scale tests with large scale test data to investigate the viscous scale effect in terms of flow Reynold number for smooth and rough quarry stone. They presented the results in terms of a graph of stability number (N_S) versus Reynold number (R_e). They found that, similarity of N_S was obtained for $R_e > 3x10^4$.

Oulett (1970 and 1972) writes that, if flow in under layer is not turbulent, use of regular waves instead of irregular waves, sidewall effects etc., may contribute to scale effects.

Treloar and Brebner (1970) studied wave energy losses due to interference of bottom and side walls separately with wave propagation in a flume. They conducted nine experiments to reduce experimental errors with wave periods ranging from 0.91sec to 1.21sec in water depths varying from 0.1m to 0.254m. The selection of these value ensured that L/d ratio would be between 0 and 10 for deep water waves to cnoidal waves. Maximum steepness was 0.05 and the boundary layer was laminar i.e. Reynold number was less than 160.

According to Hudson (1975) as quoted by US Army Corps of Engineers (1984), the scale effects will be negligible if $R_e > 3x10^5$. Hudson (1979) as quoted by Hughes (1993) writes that for the flow in armour of the breakwater to be turbulent if $R_e > 3x10^4$. Thomsen et al. (1972) write that, the model tests with $R_e > 3x10^5$ will avoid scale effects.

The breakwater, to protect the proposed runway of Honolulu International Airport, was tested with a model scale of 1:5, 1:35 and 1:45. All these tests showed similar results (Palmer and Walker 1976).

It is known that drag forces are an important mechanism for retarding run up, dissipating energy and loading armour units so that, in general R_e cannot be neglected in breakwater modeling. Since, drag coefficient increases at low R_e; small scale models produce high drag forces, less run up, less stable condition and less reflection. Therefore, extrapolating small model results to prototype scale could yield uneconomical over designed armour and an under designed crest elevation (Sollitt and Debok 1976). They conducted physical model tests on placed stone breakwaters constructed to a scale of 1:10, 1:20 and 1:100. They concluded that, the behavior of breakwater of scales 1:10 and 1:20 were almost similar to that of prototype. Whereas, the breakwater with scale 1:100 underestimate run up by approximately 20% overestimates run down by approximately 40%, reduces reflection by 10% and stability reduction is up to 40% compared to large scale models. At small models, drag coefficients increase at low R_e and therefore, drag forces will be relatively high, run up and reflection is less and stability is underestimated.

Johnson et al (1978) adopted a scale of 1:34.3 for a breakwater model constructed with dollosse armour.

Based on past successes, Hudson et al. (1979) as quoted by Hughes (1993), suggested that the approximate length scale range for rubble mound stability tests is between 1:5 and 1:70. The majority of tests conducted by Hudson at US Army Corps of Engineers, Waterways Experiment Station, Washington D. C., were in the model scales ranging between 1:40 and 1:50.

There is a need for detailed model tests for optimum breakwater design with sufficient repeat runs to increase reliability of tests (Zwamborn 1979). Standard test procedures/techniques should be used so as to reduce the errors and allow comparison of test results of different labs (Ouellet 1970 and Zwamborn 1979). Presentation of results also should be standardized (Oullet, 1970). Model test results are normally transferred to the prototype by applying Froude's law. However, the extent of scale and model effects involved have not been fully resolved (Torum et al. 1979). The interpretation of results, for given test conditions, should not be evaluated without considering the possible scale effects (Oullet 1970).

Torum et al (1979) to study scale effects, constructed a 1:80 scale model of breakwater at Bilbao Harbour, Spain which is 2.4Km long with 1:1.5 sea side slope standing in a water depth of 35m. The armour was 65Ton concrete blocks with priority of 40% to 45% placed pell mell type. They subjected the model to the storm conditions of 1st to 4th Dec 1976 which had damaged the prototype breakwater. They found that though model damage was comparable with that of the prototype damage, no damage in the model was greater than that of the prototype. This indicates that, scale or model effects if any, do not necessarily give conservative results.

Jensen and Klinting (1983) as quoted by Hughes (1993) found a lower value for critical R_e of $6x10^3$. This value was supported by Jensen himself later, who cited a successful model reproduction of an actual failure where good quality wave data were available. In the model, R_e for armour layer was $4x10^4$ and R_e for quarry run material was $5x10^3$. Authors also concluded that, the lower critical Reynold number remains uncertain, but it is likely to be lower than the value of $3x10^4$ as given by Dai and Kannel (1969) and quoted by Hughes (1993).

Baird and Hall (1984) write about the advantages of berm breakwater over conventional designs. They also mention about models constructed with scales of 1:30 and 1:50 which were tested for irregular waves.

Oumeraci (1984) as mentioned by Hughes (1993) stated that rubble mound structure models have linear scales ranging between 1:10 and 1:80, with 1:50 being the most common. However, Jensen and Klinting (1983) as quoted by Hughes (1993) give a somewhat larger scale of 1:30 as being the most common scale for rubble mound breakwaters whereas, Hughes (1993) gives typical scale range for laboratory tests on breakwater stability as 1:30 to 1:50.

When models test results are published through dimensionless formulae and diagrams – otherwise very desirable – it is absolutely necessary to mention the real dimensions.

The errors in extrapolation of model results to prototype may be due to

1. Scatter and inaccuracies of original data
2. Considerable extrapolation outside the available period of observations.

This results in a large confidence band around extrapolated functions and gives standard deviations in the order of 10% to 15% of average value (Mol et al. 1984).

Timco et al. (1984) investigated the effect of permeability of core on hydraulic and structural stability of breakwater through models scaled with 1:15 and 1:40 with R_e of $4x10^3$ to $8x10^3$ for permeable/open core and $0.2x10^3$ to $2x10^3$ for regular core. They found that for regular core, the damage inflicted on the breakwaters of scale 1:40 was similar to that of breakwater of scale 1:15. They found model scale and permeability of core has significant effect on stability of breakwater. So they concluded maintaining a R_c (> 10^3) will not guarantee against negative effects of viscosity on breakwater model test as a single value of R_e cannot represent the complex and unsteady flow which occurs in prototype and also permeability of prototype core is difficult to predict. Therefore, viscous flow in core should be scaled as correctly as possible. This makes the model tests more reliable. Other factors which contribute to scale effects are flow in under layer (if not turbulent or if they are not modeled to scale), regular or irregular waves, sidewall effects of the flume etc (Oullet 1970). Perhaps the most important sale effect associated with physical models of rubble mound breakwater are the viscous forces associated with the flow through the under layers and core of the structure. At typical scales employed for rubble mound stability models, viscous scale effects are not a problem in the primary and often secondary armour layer because the Reynold numbers of flow in these layers are sufficiently large enough to ensure fully turbulent flows. However, in some under layers and core materials, there is a possibility that the Reynold number of flow may fall below that value considered critical for avoiding scale effects.

To minimize scale effects and generate high Reynold number (greater than $3x10^4$), rubble mound structures are generally tested in the scales varying between 1:25 and 1:40. Such tests carried out for about an hour in the model correspond to about half the tidal cycle (Gadre et al. 1985a).

For many years, it has been considered that no modeling errors will arise as long as armour unit, Reynold number is greater than $3x10^4$. However, Owen and Briggs (1986) write that the model studies conducted with different scales and comparison with recorded damage suggest that Reynold number can be as low as $8x10^3$ or even $3x10^3$ before any significant errors rise. Gadre et al. (1987) conducted experiments on base of sand filled rubberized coir bags in coastal protection works. Authors conducted experiments to a scale of 1:5 to negate the scale effect whereas, the usual scales of experiments are 1:15 to 1:40.

Extensive model tests of armour stability, using irregular waves were taken up by Van der Meer (1988), exhibited no good correspondence between the large and small scale for R_e > $4x10^4$ without any viscous scale effects. This was the smallest R_e used in his tests.

Bruun (1985) as quoted by Losada (1991) writes that, generally it is accepted that, scale effects are avoided if model scale is larger than 1:60.

For evolving stability criteria of submerged breakwater, Gadre et al.(1992) tested 1:20 scale models constructed using Froude's law. They inferred that with model scales of 1:10 to 1:40, effects of viscous forces can be made insignificant and also when Reynold number was more than $3x10^4$ the model would have negligible scale effects.

Generally, flows with Reynold number above $1x10^4$ are turbulent where; viscous forces become independent of the Reynold number (Hughes 1993).

A study of Delft Hydraulics Laboratory where a 1:61 scale physical model of the final repair to the Arzew breakwater tested, produced same results as a 1:11 large scale model (Hughes, 1993).

Mol et al. (1983) discuss testing of models of Sines Breakwater at scales of 1:85, 1:78 and 1:12 which produced the same stability results where the $R_e > 3x10^4$.

Poonawala et al.(1994) conducted physical model studies to investigate the influence of sea bed slope stability of armour layer. They built breakwater models of uniform slope of 1V:2H with a scale of 1:35 on a sea bed of 1:50 and 1:500.

Selection of scale for rubble mound breakwater models involves a compromise between the desire to model it as large as possible to avoid potential scale effects and the economics of conducting tests at smaller scales. The range of length scales is dependent on wave flume and wave generation capability.

3.6 LIMITATIONS OF PHYSICAL MODELING

It is now generally accepted that physical modeling can be used as a powerful tool in the process of breakwater design, both for optimizing the layout and height of the breakwater and also to check the stability and sensitivity to damage in the finished state and during construction. However, the results obtained from modeling studies are limited by the quality of input data, by technical limitations of the physics of modeling and by the time and money available. When considering the performance and stability of rubble mound breakwaters, the exact importance of many parameters of breakwater itself, and of the environmental forces to which it will be subjected are incompletely understood despite many years of extensive research. In this situation it is important that every model should reproduce, as closely as possible, all details of breakwater construction and of environmental factors (Owen and Briggs 1986).

3.6.1 Wave climate

A full description of environment of breakwaters includes a definition of winds, waves, currents, water levels, sediment transport, sea bed topography and composition. Of these wave climate is usually by far the most important. In nature, a wave striking a submerged barrier will result in some of its energy being reflected offshore where it is ultimately dissipated by wind and internal stresses. In the lab, reflected waves strike the wave paddle and are almost totally reflected. This may go on and on. The net result is a wave system which differs considerably from simple model in the nature i.e. assuming mono-periodic waves could occur in nature (Dick and Brebner 1968).

It is essential to use same degree of wave grouping in the lab as actually observed in nature (Johnson et al. 1978). Simulating realistic sea states in lab and accurately modeling the mechanical properties of armour will predict the damage accurately (Timco and Mansard 1982 and Zwamborn 1979). But this is extremely difficult and costly. Collection of wave data is done for a minimum period of twelve months for reasonable statistical analysis. Limited size of wave data leads to higher statistical uncertainties (Vaidya 1989). In practice, all the breakwater modeling is carried out with random unidirectional waves as multi-directional sea wave generation facility is costly, complex and is not justified for breakwater testing. This may be reasonable for breakwater on shallow water where waves have only a narrow spread of direction. However, breakwater in deep water needs multi directional wave testing.

However good the available data be, there is always some doubt and considerable uncertainty about wave conditions during storm attack (Owen and Briggs 1986). Difficulty in accurately measuring wave characteristics in lab, errors in instrumentation etc., can lead to serious errors in the results (Misra 2001).

3.6.2 Modes of failure

Under normal wave conditions, the performance of the breakwater is characterized by wave run up/run down, wave transmission, reflection and over topping. These are easily measured in the model. Under extreme events such as storms, the breakwater may become additionally damaged in the following modes:

1. Fracture of individual armour units.
2. Displacement of armour
3. Erosion of under layer damage.
4. Stripping of back face due to excessive overtopping.
5. Fluidization of armour layer, giving sliding down slope.
6. Slumping of rubble mound.

For testing the damage of breakwaters, as many tests as possible should be considered for different armour placements, packing densities etc (Owen and Briggs 1986). However, this is costly and time consuming. In order to reproduce as many possible of these mechanisms as possible and also to measure the normal operating performance, the breakwater is usually modeled with strict geometric scales, i.e. all the relevant dimensions of breakwater like cross section, the size, shape of armour units, size and shape of rock, water depth etc are correctly scaled. Since breakwaters resistance to damage depends upon gravitational forces, the specific gravity of all breakwater materials is also reproduced accurately. Care is also taken to match model construction techniques with those employed in prototype breakwater so that model armour units are no better or no worse than the real thing. For artificial units, model armour units are proposed such that their fracture is not the mode of damage. For these models, shape, size, specific gravity and surface texture is sufficient. Other limitations are lab effects, scale effects, effects due to using fresh water instead of salt water. This causes the density difference of 3% and leads to an accuracy up to 15% in stability of the model (Le Mehaute 1976). Most aspects of behaviour of breakwater depend upon to a greater or lesser extent on the flow of water through, into or out of the armour and under layer. The breakwater porosity is usually specified and reproduced faithfully in the model however, permeability depends upon R_e of flow through voids. In the prototype, this flow is turbulent, but, in the model, R_e is reduced considerably and the flow may not be turbulent especially in under layers and core. In a scale model the permeability, therefore, is difficult to simulate/maintain correctly at all locations with in the breakwater and at all wave conditions. Similar arguments will also apply to lift and drag forces on the armour unit which depends upon R_e (Owen and Briggs 1986). In theory, this possible source of error can only be eliminated if the model scale is sufficiently large so that, the model flows are fully turbulent at all stages.

3.6.3 Practical limitations

The results from physical models can only be as good as input data collected prior to design, which will include sea bed topography, wave climate, tide, current sediment movement etc. In addition designer should also consider quarry condition, maximum size of stones processed, site accessibility. Other limitations are time, cost of model study, pressure from various quarters such as environmentalists, political level etc (Owen and Briggs 1986).

3.7 UNCERTAINTY IN EXPERIMENTATION

The hydrodynamic test facilities differ from one another with regard to facilities, instrumentation, experimental procedures and scale and very often it is observed that experimental results as well as extrapolated values for full scale obtained from different test facilities vary considerably in such event, it becomes necessary for a test facility to provide with possible lower and upper margins, which can be adopted with a fair confidence level. Such a study for an experimental test procedure in a particular facility is termed as uncertainty analysis. Uncertainty is an estimate of experimental error. It describes the degree of goodness of a measurement or experimentally determined result.

In a complicated hydrodynamic test set up like wave flume tests conducted on breakwater model there can be different error sources e.g. wave generation, wave period and wave height measurements, observing the run up and run down and measurement of breakwater damage. In each of these measurements error can be introduced and ultimately it may affect the result. With the help of uncertainty analysis it is possible to conduct experiments in a scientific manner and predict the accuracy of the result (Misra 2001)

3.7.1 Errors in experimentation

The different phases of an experimental procedure include planning, design, construction of model, observations, measurements, data analysis and report preparation. There is no such thing as a perfect measurement. All measurements of a variable contain errors. Error in a measurement is the difference between the true value and the measured or recorded value. Accuracy is closeness of agreement of a measured value and true value. The degree of inaccuracy is the error (δ).

Uncertainty (U) is estimation of error (δ) with certain percentage of confidence. Thus for a steady variable A, the true value of A lies with in the interval $\tilde{A} \pm U_A$ or $A = -U_A < \tilde{A} < +U_A$. Where, $\tilde{A}$ is the mean value of A for N trials or readings and U_A is uncertainty in A.

Each measurement system that is used to measure the value of individual variable A is influenced by following error sources (Abernethy et al. 1985 and Misra 2001):

1. Model accuracy and scale effect
2. Flow conditions (density, viscosity, temperature, turbulence)
3. Test conditions (test duration, measurement points)
4. Instrumentation and data collection (wave generation, recording instrument, it's calibration, visual observation and checks)
5. Data analysis (averaging and curve fitting)
6. Extrapolation.

The above error sources can be regrouped into

1. Error sources those can be removed before the experiment is started.
2. Error sources those can be neglected.
3. Error whose uncertainty is difficult to evaluate and therefore, error themselves should be minimised.
4. Error sources those remain and should be included in uncertainty analysis.

The experiment error sources should be identified and the error (δ) should be determined from manufacturer's brochures, from calibration and conducting simple experiments respectively (Kline 1985). Once this is done it must be ensured that a particular error source behaves in the same manner throughout the experiment and error does not cross the limits due to any reason.

Therefore, an experiment should be carefully planned and conducted. Error estimation should be made for full experiment. All this can be accomplished through standardizing the experimental procedures and extrapolation methods and this will ultimately reduce the uncertainty levels.

3.7.2 Uncertainty calculations

There is no single way to describe uncertainty in measurements and there are many different situations that demand some what differing description. Whatever may the method used for calculating uncertainty, but the method used should be reported in some appropriate way (Kline 1985).

3.7.3 Performance measures

There are several statistical measures that can be used to assess goodness of fit of a given model and to compare the performance of a suite of models. These statistical measures are used as indicators of the extent at which model predictions match observations.

Residual based measures are mean bias (B), the sum of square errors (SSE), mean of square errors (MSE) and root mean square errors (RMSE) provide quantitative estimates of the deviation of model prediction from the observation . On the other hand, measures of statistical association are correlation coefficient (R), the Nash Sutcliffe coefficient of efficiency (E) and coefficient of determination (D) provide quantitative estimates of the statistical co-variation between observed and predicted values.

For non-linear models in which the criterion variable Y is not transformed, the goodness of fit statistics are valid indicators of the reliability of the model. However, when the criterion variable is transformed, such as Log(Y) space the goodness of fit statistics are not necessarily a reliable indicator model reliability, especially since engineering decisions are made to predict Y and not Log(Y). Therefore, when a model requires a transformation of the criterion variable Y in order to calibrate the coefficients, the goodness of fit statistics that are included with the multiple regression output should not be used as measures of reliability (Billal and Richard 1997).

In many cases, scientists use various combinations of measures to test the accuracy of their models. When comparing different models, RMSE is useful in comparing the expected magnitude of error corresponding with each model. The correlation coefficient between observations and prediction, although still widely used is in fact one of the weakest measures of model performance (Bisher et al. 1999). It is felt that no single statistics can adequately describe the model performance, consequently, using more than one measure is highly recommended. Yet a large number of measures can be over whelming. Researchers must identify the objective of their comparison before selecting which performance measures to use (Bisher et al. 1999).

Mol et al. (1984) write that the estimated statistics for parameters such as armour weight is based on allowable tolerances in the specifications and on quality of supervision, specified measurements method and expected quality of workmanship. Standard test procedures/techniques should be used to ensure reliability and enable comparison of tests results of different labs but results from physical models can only be as good as input data collected prior to design (Owen and Briggs 1986) and the response of rubble mound structures vary widely, even in controlled laboratory conditions. But it must be kept in mind that various techniques of error analysis can be utilized to gain insight into the degree of uncertainty in the empirical design equations and they do not guarantee more reliable structures (Melby and Mlakar 1997).

3.8 OBJECTIVES OF MODEL TESTING

Rubble mound breakwaters are widely used through out the world. Economic designs of stable rubble mound breakwaters is a difficult problem involving the complex interaction of waves and structure optimizing a structure often can only achieved by systematic physical model tests (Hughes 1993). And tests of rubble mound structures are most frequently conducted for coastal structure models. The purposes of conducting stability model tests of rubble mound structures could be to:

1. Examine the stability of rubble mound armour layers when exposed to wave attack at different water levels. Studies can be aimed at verifying specific designs or toward developing general design guidance on armour size.
2. Determine hydrodynamic forces exerted.

3. Optimize the structure type, size, and geometry to meet performance requirements and budget constraints.
4. Investigate structure characteristics such as wave run up, random, overtopping, reflection, transmission, geometries and \ or construction methods etc.
5. Develop and/or test methods of repairing damage on existing structures or improving the performance of an existing structure.
6. Determine the effects of a proposed modification that might have on an existing structures stability and performance.
7. Examine alternate construction sequences under different wave conditions

Physical model studies directed at any of the above purposes will yield quantitative results provided the model is correctly scaled and operated and scale effects are determined to be minor.

Rubble mound structures must be geometrically undistorted in length scale. Flow hydrodynamics in a rubble mound structure model must conform to Froude criterion. Rubble mound structure models must have turbulent flow conditions through out the primary layer i.e. if armour layer Reynold number is large, viscosity effects can be reduced.

3.9 NEED FOR PLANNING AN EXPERIMENT

It is well known that the experimental errors due to any reason propagate and affect the final outcome. This becomes very important when error affects variables to which test results are very sensitive. Model experiments in a hydrodynamic test facility are expensive in terms of time and money. Therefore, it is necessary to plan an experiment and this should reduce the redundancy of experimental work and help to predict the final outcome with certain confidence level (Misra 2001). Also, while conducting tests of a relatively infrequent nature or a body of new geometry, it is necessary to study uncertainty during experimental planning stage and take adequate precautions. The steps in planning the experiments are as follows:

1. As a first step in planning an experiment, it is necessary to identify the sources of errors like wave generation wave measurements i.e. instrumentation, flow conditions model scale, test condition, data analysis and extrapolation.

2. Next, these errors can be classified as error which can be eliminated, errors which can be reduced, errors which can be neglected and errors those remain should be acknowledged and included in uncertainty analysis.
3. If errors can't be ignored, its magnitude should be determined from manufacturers' calibration data, or from repeated experiments of simplified nature.
4. Use of standardized methods/procedures of model testing and extrapolation reduces the uncertainties.
5. Once the errors' magnitudes are established, it must be ensured that the error sources behave in the same manner throughout the experiment. This may be ensured by changing/adjusting the experimental procedure and also instrumentation used.
6. Total error estimate must be made on experiment results as well as full scale prediction (Oullet 1970 and Misra 2001).

3.10 BREAKWATER MODEL TESTING

3.10.1 Model design

Laboratory model of rubble mound breakwater is scaled according to Froude's Law. This linear scale of the model is usually the largest possible given the restrictions imposed by the flume dimensions and wave generating capacity. The wave data is analyzed and design wave characteristics and water depth are decided and reduced to model scale.

The philosophy of design of rubble mound breakwater is that, the structure should remain intact and function effectively for the voids of primary armour layer, should be large enough to ensure efficient dissipation of incident wave energy but should be small enough to prevent the removal of stones from secondary layer through these voids. The filtering effect thus achieved in a multilayered structure determines the stability of the structure.

Using the design wave height in Hudson formula, the weight of rock armour (primary layer) unit is calculated. Once the primary armour weight W is evaluated, weight of the secondary armour unit and core is fixed according to the guidelines e.g. secondary armour units is W/10 and core is w/100 to w/6000 (Hudson 1959 and Gadre et al. 1985). The armour sizes thus calculated automatically ensure the safety against leaching out of core and secondary. The size distribution of the armour is determined in terms of weight. The armour stones are carefully hand picked so that they are roughly cubical shape. The weight of the stones ranged from 0.75W to 1.25W with 75% of the stones weighing W and above.

By comparing a number of known prototype breakwater damages with associated model investigations, Hudson (1975) as quoted by Torum et al. (1979) considers that hydraulic scale model can be used to determine stability of rubble mound breakwater and that, accuracy of results will be with in the limits required, to design safe and economical full scale structures, if models are designed and operated correctly and test conditions are selected judiciously. Therefore, model design, construction and testing should be planned and executed properly using proven and accepted techniques/procedures which can be standardized and repeated accurately (Oullet 1970 and Zwamborn 1979). The safety of structure must be determined by using conditions well in excess of design wave conditions because more often than not, these have a very low degree of reliability (Zwamborn 1979).

3.10.2 Scale model construction

Having computed the weights of primary armour, secondary armour and core materials, the model construction is then started with the placing of core at required slope. The core material is placed, on the flume bed, layer by layer, watered and lightly compacted by hand. Pipes are placed below the breakwater test section to balance the water levels on seaside and leeside (Diskin et al. 1970). The mound is completed to the desired slope. Then, secondary armour layer of designed weight and thickness is placed which is constructed again layer by layer. Each of these layers is also lightly compacted with hand. Lastly the primary armour stones are placed carefully by hand (Hudson 1959). Placing technique is important during initial damage (Font 1970). Also type of armour is a major variable and important factor as the weight of armour. The weight of loosely placed stone may be twice that of a well placed stone and keyed and fitted armour is several times more stable than loosely placed armour (Palmer and Walker 1976).

3.10.3 Test procedure

The common procedure adopted for stability tests of breakwater model sections (Ahrens 1970 and Palmer and Walker 1976) are as following:

1. Initially calibration of flume and wave generation facility are undertaken to determine the proper wave height assigned to a particular combination of generator stroke and wave period. The wave heights used in the test runs should be obtained during calibration. This will exclude the losses due to interference of flume bed and sidewalls and influence of reflection and therefore eliminates error sources.
2. Wave heights are measured at 1.5m on seaward of the breakwater.
3. Wave heights are measured before reflections from the structure set up a standing wave.
4. Waves are run in short bursts during the tests so that generator would be shut off just before wave energy reflected from the slope could reach generator blade.
5. Between wave bursts there are brief interludes to allow reflected wave energy to dampen out.
6. Tests are started by surveying newly constructed slope, with profiler system, which becomes reference survey for comparison of subsequent surveys.
7. The height of first waves would be normally smaller (about 30% lower) than those waves which could dislodge the armour and gradually increase the wave height by about 10%.
8. Waves are run, in bursts on the embankment until it appeared that no further stones would be moved by waves of this height and often over 1500 waves would be run before the slope was considered stable.
9. Failure for these tests is defined as having occurred when enough armour stones are displaced so that filter layer is exposed to wave action and core material is actually being removed through the filter (i.e. secondary) layer.

Zwamborn (1979) while analyzing the failure of Sines breakwater writes that:

1. Detailed model tests with sufficient repeat runs to increase reliability of tests are necessary.
2. Realistic wave conditions must be reproduced, particularly with the deep water structure.

3. Safety of structure must be determined by using conditions well in excess of design wave condition, because, more often than not, these have a very low degree of reliability.
4. Standard test procedures/techniques should be used to minimize errors and enable comparison of test results of different labs.

Van der Meer and Pilarczyk (1984) opines that breakwater armour slope would stabilize after about 1000 to 2000 regular monochromatic waves break on the structure, Hegde (1996) found that test should continue till the secondary layer is exposed (i.e. failure of breakwater section) or 3000 waves whichever is earlier. He found that at about 3000 waves about 80% to 90% damage would have already been inflicted.

Losada (1991) quoting Hudson and Kuelegan (1979) gives the following factors those present in model design and operation which influence the accuracy of model test results:

1. Type of wave generator and distance between generator and model.
2. Distance between test structure and wave absorber used in shoreward end of the wave flume.
3. Reflection coefficient of the wave absorber.
4. Type, magnitude and duration of attack of test waves.
5. Still water level (SWL) for testing.
6. Computation of damage of the test section.
7. Accuracy of measurements of wave characteristics.
8. Other conditions depending upon prototype data, experience of similar model tests previously conducted.

3.10.4 Data interpretation

After considering small scale tests of wave breaking over glass models of submerged barriers, Diephuis (1957) observed that:

1. Considerable scale effect exists in transmission of energy for a wave breaking on submerged barrier.
2. Small scale model tests due to scale effects impose too many limitations and it is impossible to obtain absolute quantitative data.

3. When results of models are published, even if this is done in dimensionless formulae and diagrams-otherwise very desirable it is necessary to mention the real dimensions.

Model results are normally transferred to prototype by applying Froude's model law. However, the extent of scale and model effects, involved when the Froude's model law is applied, has not been fully resolved. By comparing a number of known breakwater damages with associated model investigations, (Hudson 1975) as quoted by Torum et al (1979), considered that hydraulic scale models can be used to determine stability of rubble mound breakwater and that accuracy of model results will be within the limits required to design safe and economical full scale structures, if, models are designed and operated correctly and if test conditions are selected judiciously.

Fuhrboter et al. (1976) comparing the results of 1:10 scale model tests with that of field investigations, comment that, impact forces can't be reproduced in magnitude and transported quantitatively for prototype conditions due to different air entrainment in dependence of absolute magnitude of wave height. On the contrary, Le Mehaute (1976) opine that, there is no difference in the air entrainment in breaking waves and concluded that the process of energy dissipation during wave breaking will be in similitude even if the finer details of the flow process are different.

Palmer and Walker (1976) conducted model tests, of wave barrier to protect Honolulu International Airports reef runway, the breakwater model scales were 1:5, 1:35, 1:45. They commented that model tests should be made for all important rubble mound breakwaters subjected to breaking waves and scale effects should be considered when applying the model data to prototype design.

Sollitt and Debok (1976) write that in the small scale models, scale effects due to incorrect representation of flow (low R_e) cause less run up, less reflection and less stable condition and extrapolation of these results yield an uneconomical over designed armour and under designed crest elevation.

There is a need for detailed model tests for optimum breakwater design with sufficient repeat runs to increase reliability of tests (Zwamborn 1979). Standard test procedures/techniques should be used so as to reduce the errors and allow comparison of test results of different labs (Ouellet 1970 and Zwamborn 1979). Presentation of results also should be standardized (Oullet, 1970). Model test results are normally transferred to the prototype by applying Froude's law. However, the extent of scale and model effects involved have not been fully resolved (Torum et al. 1979). The interpretation of results, for given test conditions, should not be evaluated without considering the possible scale effects (Oullet 1970).

Mettam (1980) writes that until new design formulae are discovered, limitations of the present formulae should be recognized and they should not be used out of their intended context. This implies that the model test results are valid within the limitations of laboratory conditions under which models were tested and extrapolating the results or applying under different situations may not be correct.

For reliable results it is necessary that model tests accurately represent the prototype situation. For this, accurately representing the prototype breakwater armour in model (strength, texture and placement) and generation of realistic sea states in the model is important (Timco and Mansard 1982).

Mol et al. (1984) writes that, errors may be due to scatter of original data and considerable extrapolation outside the available period of observations. This results in a large confidence band around the extrapolated function and gives standard derivation of the order of 10% to 15% of the average value.

Owen and Briggs (1986) writes that the results from physical models can only be as good as input data collected prior to design.

Collection of prototype data regarding performance of structure in field is a great help in improving research regarding practical aspects and future constructions (Franco 1996).

Chapter 4
Problem Formulation

4.1 BACKGROUND

Conventional such as non-overtopping rubble mound breakwaters are in use since long time protecting the beach and/or land behind from flooding, erosion, silting or sheltering reclaimed land. Earlier, these breakwaters were of gentle slope consuming enormous quantity of material. Building these large structures was a mammoth task as it called for procurement, transportation and placement of massive blocks of rocks in the shallow/deep waters. This always encouraged engineers to research on economic design of breakwaters which gave maximum protection from sea waves. Thus the concept of optimization of breakwater section evolved. Till 1933, the breakwaters were designed based on laboratory experiments, experience and comparative basis. Later various investigators gave simple design formulae based on limited laboratory investigations (Poonawala 1993). These designs were either over safe or under designed. Many researchers like Hudson (1959); Van der Meer (1988); Belfadhel et al. (1996) and Melby and Hughes (2003) have given the design formulae for estimating the breakwater armour weight. Even though the design formulae are available, they have not eliminated the need for experimental verification of engineering design.

When these structures were not found compatible with the particular site conditions, as they failed due to extreme loads of cyclonic waves, the engineers settled for berm breakwaters. Gadre et al. (1991) suggested that the weight of the armour of berm breakwater could be substantially lower, i.e. 20% to 30%, than the conventional structure and this could save up to 50% to 70% of the cost depending up on the site conditions (Baird and Hall 1984).

Further the investigations found out that these protective structures need not project above SWL at all the sites to function effectively. Hence, they designed submerged breakwaters which would successfully trip the oncoming steep waves, dissipate the energy and attenuate the waves before further propagation, which could be effectively used at places requiring partial protection, where, tidal ranges are small. These structures being under water are subjected to smaller wave loads. This was an innovative method to further optimize the structure without compromising its performance. Thus many researchers (Johnson et al. 1951, Dattatri et al. 1978, Van der Meer and d'Angremond 1992, Rambabu and Mani 2001 and 2002 and Twu et al. 2001) attempted to design the slope, crest width, location and height of the submerged breakwater so as to reduce the transmission coefficient and arrived at various design criteria. Submerged breakwaters have been popular beach protection measures in Japan (Goda 1996).

Kale and Gadre (1989) demonstrated through physical model studies that, offshore submerged breakwaters in small depths of 2m to 3m could be easily and economically constructed with a variety of materials like stone filled boxes/synthetic bags, concrete pipes or chains of small concrete blocks. These can be easily transported to the site by boats or by raft and can be placed in water by 4 to 6 people as the total weight of chain of blocks is around 200Kg.

Meanwhile, Ahrens (1984, 1989), Fulford (1985), Gadre et al. (1992) and Nizam and Yuwono (1996) further optimized the submerged breakwater by proposing design criteria for reef structure. Ahrens (1984, 1989), Gadre et al. (1992) and Nizam and Yuwono (1996) have given the design formulae for the reef armour. A reef is a structure constructed of armour without a multilayer structure. This is just a porous structure which has the armour of sufficient weight so as to resist the wave load coming over it. This structure is an optimized structure to the highest degree which can successfully break the high waves and dissipate wave energy.

The design of breakwaters through out the world shows a great variety and there is no best solution, because, it depends not only upon hydrodynamic conditions but also on the local, national and social situation. These breakwaters have been built throughout the centuries but then structural development as well as their design procedure is still under massive change. Major research activities are necessary, some of them have been already been installed, to gain better knowledge of the physical back ground of the structure performance. They must lead to new design concepts and load adequate structures. New ideas and developments are in the process of being tested for reducing the load on and the failure of breakwaters. To reduce wave load on structure and/or allow water to exchange, the concept of underwater or submerged breakwaters is always attractive (Kaldenhoff 1996).

Franco (2001) writes that complex hydromorphodynamics associated with submerged barriers are not yet fully known. Therefore, in Italy, a specific European research project called DELOS was started in 2001, to investigate the ecological implications in order to provide more reliable design guidelines.

Until early 1990s the design practices of breakwaters were based on hydraulic model tests and empirical formula. Most reliable and quantitative data on effect of submerged breakwater on wave action, are those obtained in laboratory model studies. Because of the complexity of the problem theoretical development is over simplified and therefore, greater reliance should be placed on lab and field studies (Khader and Rai 1980). Very few numerical techniques have been developed recently to check the stability of these structures (Kudale and Dattatri 1994 and Van Gent and Vis 1994). Very recently software such as HISWA, SWAN, EMS and MIKE21 and BREAKWAT.3 has been developed and is used for designing different types of breakwaters (Mai et al. 1999, Delft Hydraulics 2001 and Nagendra Kumar 2001). Though numerical models have been developed as a design tool for many types of breakwaters after 1990, the necessity of physical model study has not diminished. They are still used to validate numerical models and are used when parameters involved are complicated or it is not possible to accurately represent them mathematically or to calibrate the parameters. Physical model studies are the only way out, when, the geometry of the structure to be tested is unusual and parameters involved interact in complex patterns (Chakrabarthi 1996 and Mai et al. 1999).

Off late designs are being increasingly influenced by environmental, social and aesthetical aspects transforming the breakwater into a complex structure. Under such situation, where, the design/analysis of the prototype structure is very complicated, the use of physical models, which are a close representation of reality in which a prototype system is duplicated as closely as possible, is particularly advantageous.

4.2 PROTECTION OF BREAKWATERS

The conventional breakwaters can not be economically designed to take on the wave loads due to extreme waves during cyclones and there is considerably uncertainty about data collected regarding the storm waves (Owen and Briggs 1986). Many deepwater breakwaters failed between 1950 and 1982 due to cyclonic waves. Hence, the idea of rehabilitation and/or design of some kind of protection to these conventional breakwaters evolved.

Breakwaters may be protected by providing a submerged berm attached to the seaward side of the breakwater or providing a detached underwater/submerged breakwater depending upon geometry of structure, type of damage, causes of failure, availability of construction material and equipment, financial constraints, future requirements for port expansion and other construction works (Groeneveld et al. 1984).

Gadre et al. (1985) designed a submerged berm seaward of revetment bund which protected the land reclaimed between outer harbour and fisheries harbour north of Bharathi Dock at Madras Port, Chennai, India. The submerged bund broke and attenuated high waves and the region between two structures dissipated the wave energy further. This facilitated construction of revetment with 2Ton to 3Ton stones, where, upper slope was constructed with stones of 0.5Ton to 1.5Ton at slopes of 1:3 and 1:25 respectively. This saved the material compared to conventional design of non-overtopping breakwater with armour stones of 15Tons or tetrapods of 6.5Ton on a slope of 1:2.

Gadre et al. (1989) economically rehabilitated a damaged head portion of the breakwater at Veraval Port Gujarat, India, by constructing a submerged breakwater at a seaward distance of 80m.

Cox and Clark (1992) through limited model studies designed a submerged reef to protect the inner shorter breakwater and called it a tandem breakwater. They designed a breakwater of armour weight of 3Ton and a submerged reef with stone armour of weight up to 1Ton at a seaward distance of 40m which was economical by 1million dollars compared to conventional breakwater design which otherwise required an armour of 8Ton. He concluded that such a tandem breakwater could be an optimum structure.

Cornett et al. (1993) through small scale model tests showed that, a low crested reef breakwater with height greater than or equal to 0.6 times the depth of water and crest width of more than 0.1m located seaward of main breakwater, can reduce wave loading and erosion of rock armour. They validated the tandem breakwater concept and concluded that, there was an optimum distance between the structures depending upon wave conditions and geometry of breakwaters and considerably more research and testing of tandem breakwater is required to develop a complete understanding of the transformation of waves, loading events and design.

4.3 PROBLEM FORMULATION

Breakwater's vulnerability to extreme events such as storms is a reality. One of the things engineers can do is to design a seaward protective structure to the breakwater, which will withstand, resist and manage such destructive extreme events. After going through the literature, it is decided that a submerged reef, which is a simple, porous and stable rubble mound structure without a layered cross section, is an optimised structure (Ahrens 1984, 1989, Fulford 1985, Gadre et al. 1992, Nizam and Yuwono 1996 and Bierawski and Maeno 2002). This could be an effective option as a protective structure for breakwater as it breaks steep waves and attenuate them to a tolerable level.

This concept is new and therefore, further information is needed to draw conclusions on the impact of geometry of submerged reef over the stability of the inner (main) breakwater and arrive at a stable and economical design of defenced breakwater. This calls for an in depth physical model study. Hence, it is decided that, an experimental study of breakwater protected by submerged reef (i.e. defenced breakwater) be taken up to investigate the influence of a submerged reef, on the stability of the inner breakwater and design it as a protective structure.

4.4 NECESSITY AND RELEVANCE OF PRESENT STUDY

After studying the literature, it is learnt that, the breakwaters, which are one of the important structures, are subjected to damage under extreme wave loading during storms. Extreme waves of storms can not be predicted accurately and there exists considerable uncertainty regarding storm wave data. Hence, breakwaters can't be economically designed for extreme waves. A seaward protective structure will withstand, resist and manage destructive steep waves of a storm at least to some extent, at the same time, mitigate catastrophic damage of the inner (main) breakwater. This could be an economical option for designing a structure which stands a fair chance of an encounter with such an extreme event. A submerged reef which is an optimized structure can be located at a certain distance seaward of the breakwater. The wave breaking over reef causes more turbulence on lee side. They also offer resistance through friction and turbulence created by reef's interference in the wave field causing maximum energy dissipation. This attenuates waves on leeside, which, then attack the inner (main) breakwater within the tolerable level.

The design of this type of protected structure requires detailed information on parameters such as water level changes, reef stability, height and crest width of the reef, seaward location and wave transmission. The literature survey also threw light on the need of investigating the submerged reef especially regarding its armour weight, crest width and its seaward location for maximum protection of the inner breakwater which could save an appreciable quantity of material and cost. Therefore, more research is needed to understand their mechanism, performance and design such complex structures confidently. These issues present an extremely complex picture, which, is difficult to model mathematically. In light of this, it is decided that experimental work be taken up to study the stability of the reef and influence of its crest width and seaward location on wave transmission and stability of breakwater to arrive at a design of a seaward reef that protects the inner breakwater. The design of such protection to the breakwater is the research work selected for the present study.

4.5 SCOPE OF PRESENT WORK

A protective submerged reef is located at a certain distance seaward of the main breakwater. This model is subjected to normal attack of regular waves of varying characteristics in changing water levels. In the present investigation, an attempt has been made to study the following aspects:

1. Armour stability of the conventional uniformly sloped breakwater.
2. Optimum armour stone weight for a stable uniformly sloped submerged reef.
3. Influence of a submerged reef located at varying seaward distances on the stability of inner (main) breakwater.
4. Influence of different crest widths, of a selectively located submerged reef, on the stability of inner breakwater.
5. Reef characteristics those protect the breakwater.

Chapter 5
Details of Present Experimental Investigation

5.1 GENERAL

The two dimensional wave flume available in Marine Structures Laboratory of Department of Applied Mechanics and Hydraulics of National Institute of Technology, Karnataka, India is used for the present study. It is decided that, an experimental study of breakwater protected by submerged reef be taken up to investigate the influence of a submerged reef, on the stability of the inner (main) breakwater and design it as a protective structure.

Keeping the laboratory conditions in mind, largest possible model scale is selected. The predominant non-dimensional variable affecting the performances of the chosen model study are obtained through dimensional analysis. The details of the laboratory conditions, experimental set up, dimensional analysis, hydraulic modelling and methodology and procedure adopted for the present experimental investigation are explained in this chapter.

5.2 DESIGN CONDITIONS

For the design of conventional rubble mound breakwater an equivalent of prototype design wave of 3m, a depth of water of 9m and a water level rise of 3m are assumed. While waves equivalent of height up to 4.8m and of period of 8sec to 13sec are considered to for model study.

5.3 DIMENSIONAL ANALYSIS

The present model study involves a complex structure comprising of a breakwater and a protective seaward submerged reef. The waves break over the reef, loosing a major portion of energy and then loose some more energy while propagating in the zone between the structures. This phenomenon is difficult to express mathematically and one has to depend upon experimental investigations. The results of such investigations are more useful when expressed in the form of dimensionless relations. To arrive at such dimensionless relationships between different variables, dimensional analysis is carried out.

5.3.1 Predominant variables

The predominant variables considered for dimensional analysis in the present investigation are listed in the Table. 5.1.

Table. 5.1. Predominant variables considered in the analysis of breakwater stability

Predominant Variable		Dimension
Wave Parameters	Incident Wave Height (H_i)	L
	Transmitted Wave Height (H_t)	L
	Deep water Wave Height (H_0)	L
	Water Depth (d)	L
	Wave Period (T)	T
	Wave Length (L)	L
	Run-up (R_u)	L
	Run-down (R_d)	L
	Particle Velocity (v)	LT^{-1}
Structural parameters	Armour unit Weight (W)	M
	Nominal Diameter (D_{n50})	L
	Reef submergence (F)	L
	Crest Width (B)	L
	Structure height (h_s)	L
	Relative Mass Density of Armour units (Δ)	$M^0L^0T^0$
	Mass Density of Armour units (ρ_r)	ML^{-3}
	Bulk Density of Armour units (ρ_b)	ML^{-3}
Fluid parameters	Mass Density (ρ)	ML^{-3}
	Dynamic Viscosity (υ)	$ML^{-1}T^{-1}$
External effects	Acceleration due to Gravity (g)	LT^{-2}

5.3.2 Details of dimensional analysis

For deep water wave conditions L and T are related by

$$L = \frac{gT^2}{2\pi} \qquad (5.1)$$

The term gT^2 is used in the above equation to represent the wave length L, instead of taking L directly. This is because if L is used it would be depth specific while, gT^2 is independent of depth and represents the deep water wave characteristics which can be easily transformed to shallow waters depending upon local bathymetry.

Considering the damage level S of the rubble mound breakwater which is dependent on several independent parameters, their relationship can be expressed as follows

$$S = f\,(H_o, T, d, X, L_o, B, F, h, \Delta, A_e, g, \rho, D_{n50}) \qquad (5.2)$$

By the application of Buckingham's π theorem, an equation of the form shown below is obtained.

$$S = A_e/D^2_{n50} = f\{H_o/gT^2, X/d, H/\Delta D_{n50}, B/d, B/L_o, h/d, F/H_i, d/gT^2\} \qquad (5.3)$$

Similarly, transmission coefficient (K_t) at the submerged reef can be expressed as,

$$K_t = H_t/H_i = f\{H_o/gT^2, X/d, H/\Delta D_{n50}, B/d, B/L_o, h/d, F/H_i, d/gT^2\} \qquad (5.4)$$

Where,

A_e/D^2_{n50}	Dimensionless damage S
H_t/H_i	Transmission coefficient K_t
H_o/gT^2	Deepwater wave steepness
X/d	Non-dimensional attenuation distance
$H/\Delta D_{n50}$	Hudson's stability number N_S
B/d	Relative reef crest width
B/L_o	Relative reef crest width
F/H_i	Relative reef submergence
h/d	Relative reef height
d/gT^2	Depth parameter

In the present work, the breakwater is modeled using Froude's law where, the dimensionless product Froude number ($F_r = v/\sqrt{(gL)}$) is kept identical for prototype and the model. For many years, it has been considered that no modeling errors will arise as long as Reynold number for flow through the armour layer is greater than $3x10^4$. However, Owen and Briggs (1986) write that the recent model studies conducted with different scales and comparison with recorded damage suggest that, this Reynold number can be as low as $8x10^3$ or even $3x10^3$ before any significant errors rise.

In the present work, Reynolds number is not considered, because, flow in the primary armour layer is considered to be turbulent and viscous effect becomes unimportant as $R_e > 3x10^3$. Similarly, reflection from the submerged reef and breakwater is also not considered. This is because the study is of fundamental nature and importance is attached to wave transmission at the reef, also reflection from porous submerged reef is relatively small (Ahrens 1984). In nature, a wave striking a submerged barrier will result in some of its energy being reflected offshore where it is ultimately dissipated by wind and internal stresses. In the lab, reflected waves strike the wave paddle and are almost totally reflected and create unduly high waves which may not represent the prototype condition truly. This may go on and on. The net result is a wave system which differs considerably from simple model in the nature i.e. assuming mono-periodic waves could occur in nature (Dick and Brebner 1968). Therefore, in the present experimental investigation, waves are sent in bursts of 5 waves, and generator is switched off between the bursts to allow sufficient time for the wave energy to dampen out.

5.4 SIMILITUDE AND MODEL SCALE SELECTION

A physical model should be designed correctly to get reliable results. In the present study, similitude is achieved by keeping the non-dimensional parameters in the same range for both model and prototype.

The best model is the prototype. But the limitations of the lab, most of the times, do not permit testing of a prototype. The choice of scale, for the model test, is often limited by constraints put by experimental facilities available. Within this constraint, an optimum scale should be selected by comparing the economies of the scale model with that of the experiment (Hughes 1993 and Chakrabarti 1996).

The results of Stive (1985) indicated that, there was no significant departure from Froude scaling in a wave height range of 0.1m to 1.5m with regard to the parameters measured. This result proved that observed difference in air entrainment between model and prototypes have no significant dynamic influence. When the wave break on armour units in a physical model, the flow is turbulent so it is expected that pressure produced in the model are in similitude with the prototype (Le Mehaute 1976).

To simulate the field conditions of wave heights, period and armour size, by application of Froude's law, different model scales are tried. This is presented in the Table. 5.2.

Table. 5.2. Selection of model scale

Scale	H (m)		T (sec)		D_{n50} (m)	
					Breakwater	Submerged reef (Range)
	3.0	4.8	8.0	13.0	0. 8970	0. 5270 - 0. 6980
1:10	0.300	0.48	2.53	4.11	0.0897	0.0527 - 0.0698
1:20	0.150	0.24	1.79	2.91	0.0449	0.0263 - 0.0349
1:30	0.100	0.16	1.46	2.37	0.0298	0.0175 - 0.0233
1:40	0.075	0.12	1.26	2.06	0.0221	0.0131 - 0.0174

In Marine Structures laboratory of Department of Applied Mechanics and Hydraulics of National Institute of Technology, with existing facilities of two dimensional wave flume, regular waves of heights ranging from 0.08m to 0.20m and period ranging from 1.0sec to 3.0sec can be produced. Keeping this in focus, a model scale of 1:30 is selected for the present experimental investigation. This scale is within the scale selected for rubble mound breakwater model tests conducted in majority of the laboratories around the world and is good enough to give reasonable and satisfactory results compared to those of the prototype (Le Mehaute 1976,Palmer and Walker 1976, Sollitt and Debok 1976, Gadre et al. 1985, Stive 1985,Gadre et al. 1987, Losada 1991and Hughes 1993).

5.5 STRATEGY FOR EXPERIMENTATION

It is decided to experimentally study the performance of a 1:30 scale model of submerged reef and protected breakwater under attack of varying wave climate in changing depths of water. The physical model study includes investigation of:

1. Armour stability of a trapezoidal submerged reef.
2. Wave transmission at reef and wave height attenuation.
3. Influence of varying reef geometry on stability of inner (main) breakwater.

5.6 OBJECTIVES OF STUDY

The objectives of the present experimental investigation are:

1. To study stability of the conventional uniformly sloped breakwater.
2. To arrive at optimum armour stone weight for a stable uniformly sloped submerged reef.
3. To study the influence of a submerged reef located at varying seaward distances on the stability of inner (main) breakwater.
4. To study the influence of different crest widths, of a selectively located submerged reef, on the stability of inner (main) breakwater.
5. To arrive at reef characteristics those protect the breakwater completely.

Therefore, the aim of the present physical model study is to evolve a design of defenced breakwater where, a seaward reef offers complete protection of the inner (main) breakwater for a range of selected wave characteristics and depths of water.

5.7 EXPERIMENTAL SETUP

5.7.1 Wave flume

The physical model is tested for regular waves in a two dimensional wave flume of Marine Structures laboratory of Department of Applied Mechanics and Hydraulics, National Institute of Technology Karnataka (NITK), Surathkal, India. Plate. 5.1 shows the distant view of wave flume.

The wave flume is 50m long, 0.71m wide and 1.1m deep. It has a 41.5m long channel with a smooth concrete bed. About 15m length of the flume is provided with glass panels on one side. It has a 6.3m long, 1.5m wide and 1.4m deep chamber at one end where the bottom hinged flap generates waves. The flap is controlled by an induction motor of 11Kw power at 1450rpm. This motor is regulated by an inventor drive (0 – 50Htz) rotating in a speed range of 0–155rpm. Regular waves of 0.08m to 0.20m of periods 1.0sec to 3.0sec can be generated with this facility. A gradual transition is provided between normal channel bed level and that of generating chamber by ramp. The wave filter consists of a series of vertical asbestos cement sheets spaced at about 0.1m centre to centre parallel to length of the flume. Fig. 5.1 gives a schematic diagram of experimental setup.

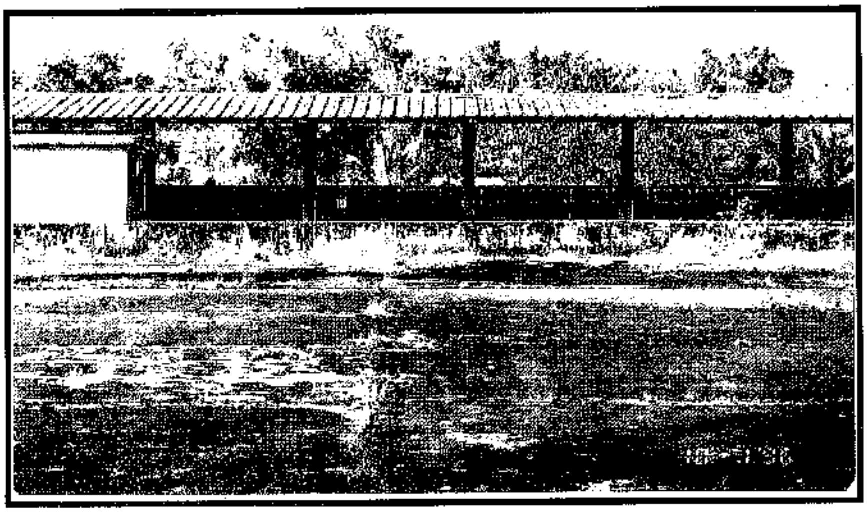

Plate. 5.1. Distant view of wave flume

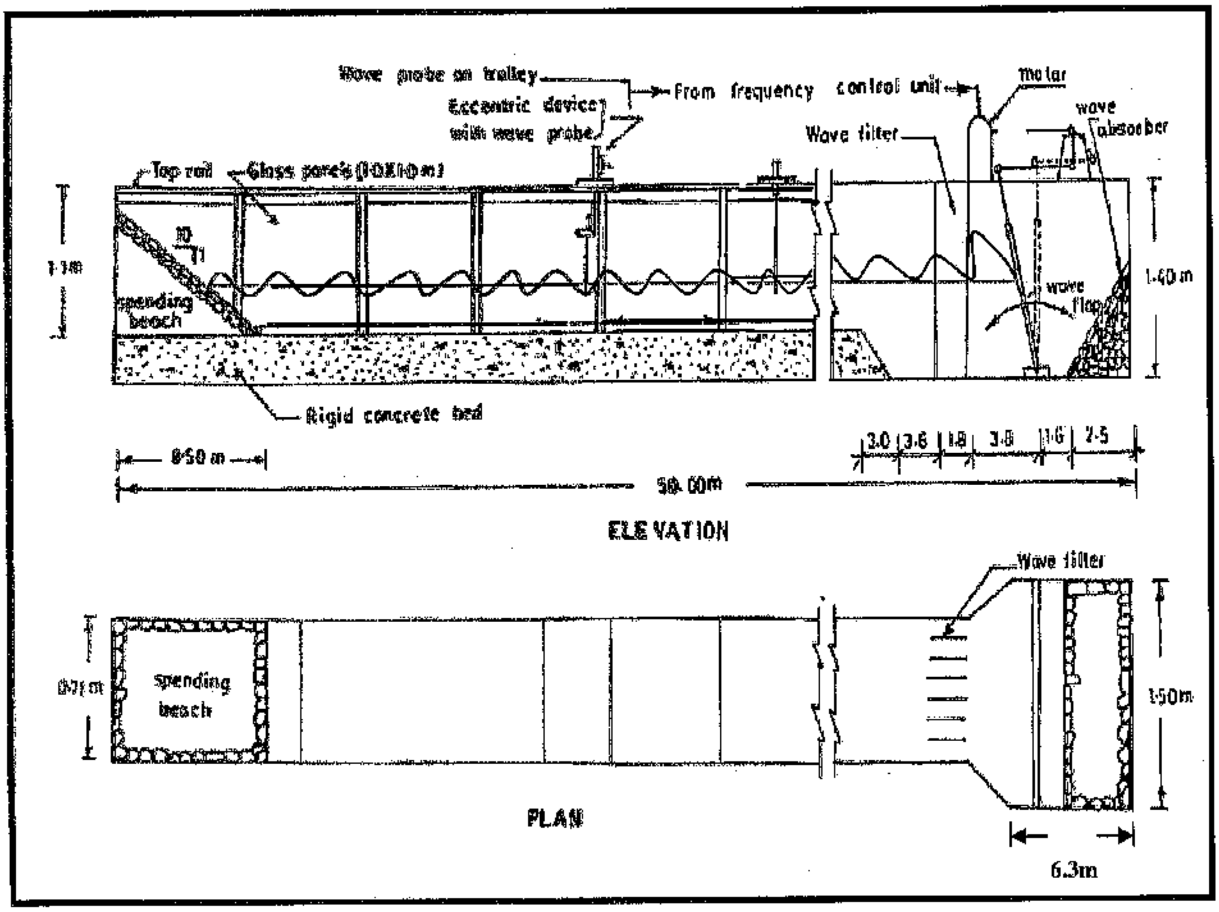

Fig. 5.1. Details of experimental setup

The changing the frequency through inverter one can generate the desired wave period. A fly-wheel and bar-chain link the mortar with flap. By changing the eccentricity of bar chain on the fly-wheel one can vary the wave height for a particular wave period.

5.7.2 Instrumentation

Two capacitance type wave probes are used where, first probe measures incident wave height (H_i) at about 1m seaward of reef toe and second probe measures transmitted wave height (H_t) after breaking over the reef. The same probe is moved to measure wave height approaching the breakwater toe. The water surface elevation is 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 waveform for 10sec duration is acquired using software ADTRIG-T.C. Plate. 5.2 shows the instrumentation setup.

Plate. 5.2. Instrumentation

5.7.3 Calibration of test facilities

Before the model tests are started, the flume is calibrated to find the required wave heights which, are assigned to a particular combination of generator stroke and wave period, for depths of water (d) of 0.3m, 0.35m and 0.4m. The wave probes are also calibrated for temperature correction every morning and afternoon before the tests begin and the wave characteristics were recorded. Plate. 5.3 shows the calibration of wave probe.

Plate. 5.3. Calibration of wave probe

All the waves recorded are verified by manual observations. The combination which produce secondary waves in the flume are not considered for the experiments. This will exclude the losses due to interference of flume bed and sidewalls and influence of reflection and therefore, eliminates error sources. Plate. 5.4 and Plate. 5.5 show the recorded incident wave and transmitted wave forms respectively.

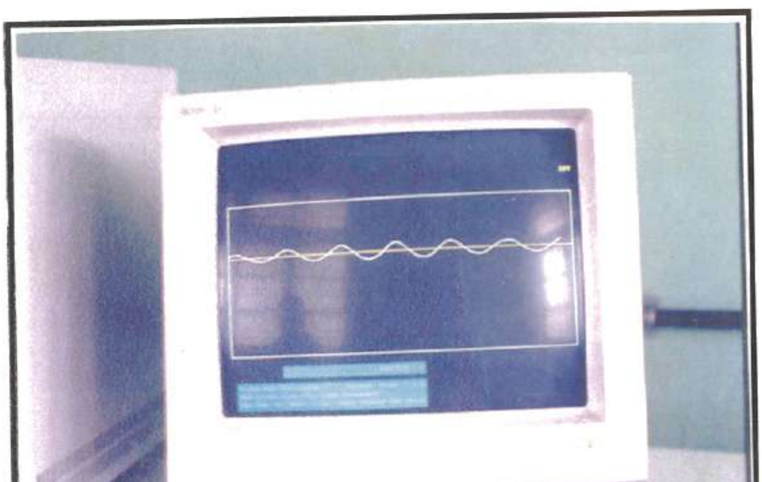

Plate. 5.4. Incident wave form

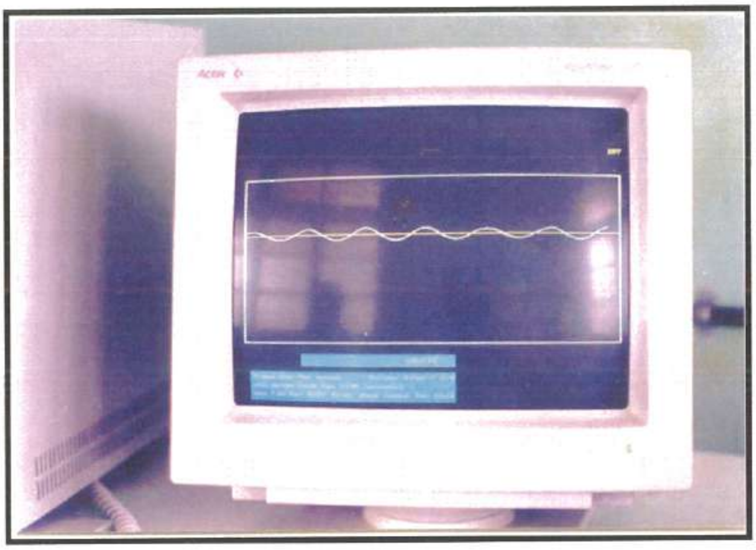

Plate. 5.5. Transmitted wave form

5.7.4 Surface profiler system

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 at 0.1m interval each one with a ball and socket foot where the foot is circular with diameter of about 0.035m. The damaged profile of the breakwater is plotted and the area of erosion (A_e) is measured with digital planimeter.

5.8 TEST MODELS

5.8.1 Conventional breakwater

A 1:30 scale model of a breakwater, of trapezoidal cross section with a uniform slope of 1V:2H, is constructed on the flat bed of the flume. The primary armour weight (W_{50}) for a design wave of 0.1m is determined using Hudson's formula to be 73.2gm (i.e. nominal diameter (D_{n50}) of 0.0298m). The actual weight of the armour stone used varied from 55gm to 91gm with a mean value of 73.2gm. The model characteristics and the standard deviation and coefficient of variation of armour stone weight are given in Table 5.3. The mean weight of the stones in the secondary layer is taken as 7.32gm. The model crest width is 0.1m and height is 0.70m. The core is designed with rock flour of size 300microns. The armour units are painted with different colours and placed in bands of heights of 0.2m to 0.3m to track their movement during damage. In the first phase, this conventional breakwater model is tested.

Table. 5.3. Model characteristics of conventional breakwater

Variable	Expression	Value
Slope		1V: 2H
Armour type		Angular quarry stone
Mass Density	ρ	2.8gm/cc
Nominal diameter	D_{n50}	0.0298m
Armour weight	W_{50}	73.2gm
Standard deviation	σ	8.12gm
*Coefficient of variation	c	0.11
Crest width		0.1m
Crest height		0.7m
Porosity of primary armour	p_1	43%
Porosity of secondary armour	p_2	39%
Porosity of core	p_3	36%

* The value is approximately equal to the value used by Melby and Mlakar (1997) in reliability analysis undertaken.

5.8.1.1 Model construction

The model of the main breakwater is constructed at a distance of 32m from the generator flap (refer Fig. 5.2). Plate. 5.6 shows the cross section of conventional breakwater. This test section is subjected to normal wave attack of 3000 regular waves of height ranging from 0.1m to 0.16m of periods varying from 1.5sec to 2.5sec in a depth of water (d) of 0.3m, 0.35m and 0.4m. The Table 5.4 lists the test wave conditions. Plate 5.7 captures the wave surging over the conventional breakwater. The cross section of breakwater denoting layers is drawn on the glass panel. Pipes are placed below the breakwater test section to balance the water levels on seaside and leeside. The core material is placed and formed to the required level and after lightly compacting by hand. Then, the secondary layer and the primary layer are constructed upto the marked level. The primary armour layers are painted to reduce the surface roughness (Hughes 1993). In the rubble mound breakwater model, the armour stones placed around SWL are painted in white and in the zone above, it is the armour stones with red colour while in the zone below, the stones were painted with black colour to identify the damage zone after the test. The armour stones are placed with keyed and fitted placement technique.

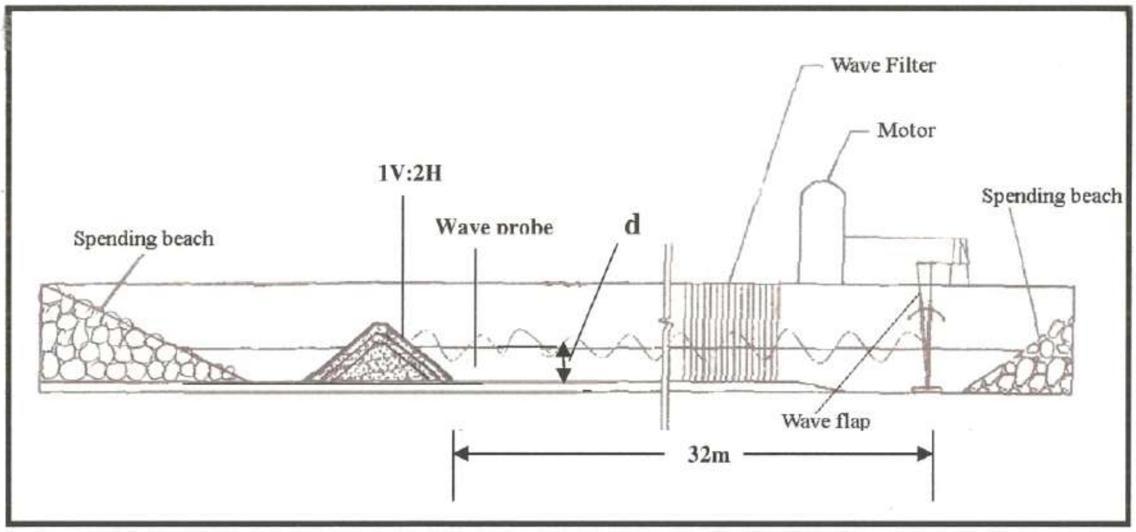

Fig. 5.2. Details of model testing of conventional breakwater

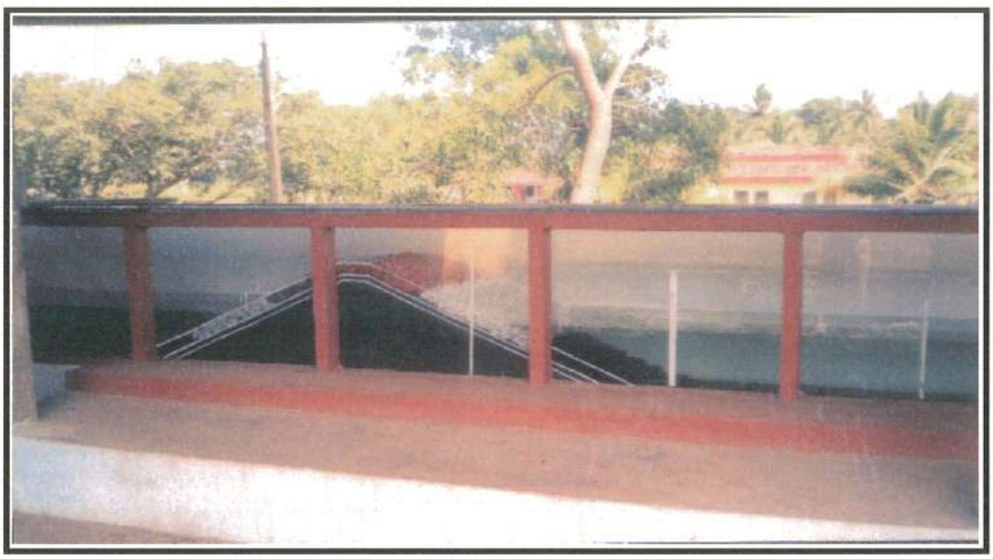

Plate. 5.6. Cross section of conventional breakwater

Plate. 5.7. Wave surging over the conventional breakwater

Table. 5.4. Wave characteristics

Variable	Notation	Range
Angle of wave attack	-	90^0
Wave height	H	0 .10, 0.12, 0.14, 0.16m
Wave period	T	1.5, 2.0, 2.5sec
Storm duration	N	3000 waves
Water depth	d	0.30, 0.35, 0.40m
Depth parameter	d/gT^2	0.004 - 0.013, 0.005 - 0.015, 0.006 - 0.018
Steepness parameter	H_o/gT^2	0.00145 - 0.00785

5.8.1.2 Physical properties of armour units

The specific gravity of the stones used as primary armour units is determined using pycnometer. Five samples are taken and the specific gravity calculated ranged between 2.71 and 2.81 with an average of 2.8. The standard deviation is 0.13 and coefficient of variation is 0.046. The bulk density of the armour unit is calculated as 1.6gm/cc.

The porosities of the primary armour, secondary armour and core are 43%, 39% and 36% respectively.

5.8.1.3 Gradation of armour units

The size distribution of the rubble material is determined in terms of weight. The aggregates are carefully hand picked so that they were angular or roughly cubical in shape. The weight of the stones used for the primary layer ranged from 0.75W to 1.25W with a mean weight W_{50} of 73.2gms is used. Fig. 5.3 shows the gradation curve for armour units.

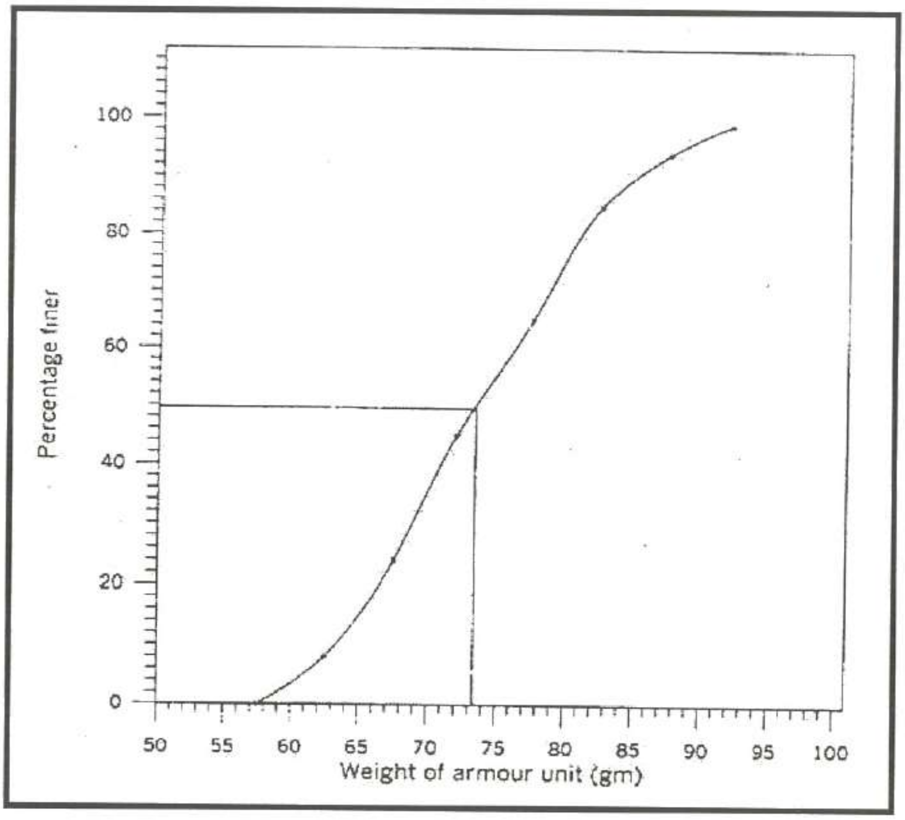

Fig. 5.3. Gradation curve for primary armour units

The thickness of the primary and secondary layer is determined by the equation

$$r = n \times K_{\Delta} \times (W / \gamma_r)^{1/3} \quad \text{.......(5.5)}$$

Where,

r is thickness of armour layers, n is number of layers of the armour units, K_{Δ} is layer coefficient (for rough quarry stones = 1.15), γ_r is specific weight of armour material.

The average layer thickness of 7.0 cm is provided for primary layer and an average thickness of 3.2 cm is provided for secondary layer. The minimum crest width should be sufficient to accommodate three stones. For the core, quarry run of size 300 microns was used.

As given by Van der Meer (1988) the following damage levels are used to measure the damage levels of the breakwater:

1. S = 2 (Incipient damage)
2. S = 5 – 8 (Intermediate damage)
3. S = 12 (Filter layer is visible).

5.8.2 Submerged reef structure

Fig. 5.4 shows a 1:30 scale model testing of a trapezoidal submerged reef, of slope 1V:2H, height (h) of 0.25m and crest width (B) of 0.1m. The Plate 5.8 shows the cross section of the actual submerged reef model constructed on the flume bed. The model is constructed at 28m from the generator flap. Five such reef models are designed with armour stones weighing 15gms, 20gms, 25gms, 30gms and 35gms as given by various design criteria (Ahrens 1984, 1989; Gadre et al. 1992; Nizam and Yuwono 1996 and Piarczyk and Zeidler 1996). The mean weights, standard deviation and coefficient of variation of the reef armour are given Table 5.5.

In the second phase, this test section is subjected to normal wave attack of 3000 regular waves of height ranging from 0.1m to 0.16m, of periods varying from 1.5sec to 2.5sec in a depth of water (d) of 0.3m as the reef stability is critical at the lowest water level (Smith et al. 1996). The damage to the structure is recorded in the form of reduction in crest height (h_c). From the model test optimum armour of weight for stable reef is derived.

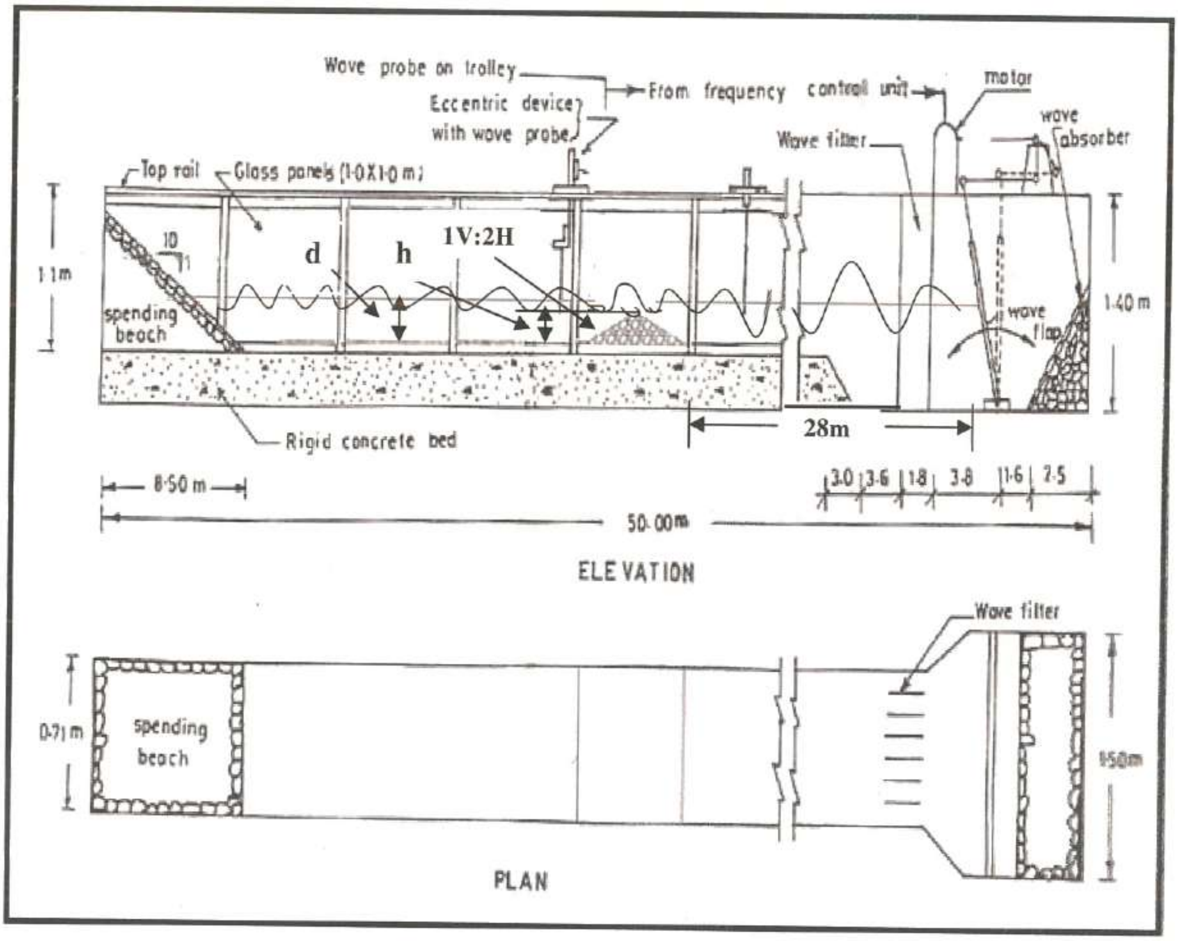

Fig. 5.4. Details of model testing of submerged reef

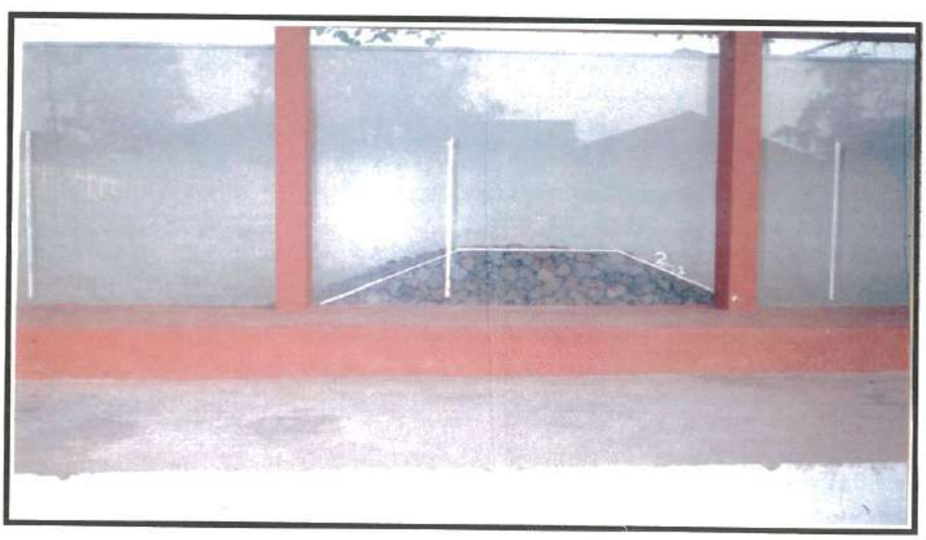

Plate. 5.8. Cross section of submerged reef

Table. 5.5. Model characteristics of submerged reef

Variable	Expression	Range
Slope		1V: 2H
Reef Armour type		Angular quarry stone
Nominal diameter for reef	d_{n50}	0.0221m
Reef Armour weight	w_{50}	15.4gm, 20.3gm, 25.5gm, 30.2gm, 35.3gm
Standard deviation	σ	2.61gm, 2.45gm, 3.42gm, 3.28gm, 4.56gm
Coefficient of variation	c	0.17, 0.12, 0.13, 0.11, 0.13
Crest height of reef	h	0.25m
Crest width of reef	B	0.10, 0.20, 0.30 and 0.40m
Porosity of reef		43%
Reef spacing	X	At 1.0m, 2.5m and 4.0m seaward of main breakwater
Relative height	(h/d)	0.625 to 0.833
Relative crest width	(B/L_o)	0.01 to 0.114
Relative crest width	(B/d)	0.25 to 1.33
Relative reef submergence	(F/H_i)	-0.312 to -1.5
Relative reef location	(X/d)	2.5 to 13.33

In the third phase, the submerged reef of same geometry is constructed with armour of optimum weight and subjected to waves of heights (H_i) of 0.1m to 0.16m of period (T) of 1.5sec to 2.5sec in the depth of water (d) varying from 0.3m to 0.4m. The incident waves are recorded at 1m seaward of the reef. The transmitted wave heights (H_t) on the leeside are recorded for a distance of every metre up to 8m and required reef spacing is derived depending upon the maximum wave height attenuation achieved. The wave breaking over the submerged reef is captured in Plate. 5.9 and Plate 5.10.

Plate. 5.9. Waves just breaking over the submerged reef (for d = 0.3m)

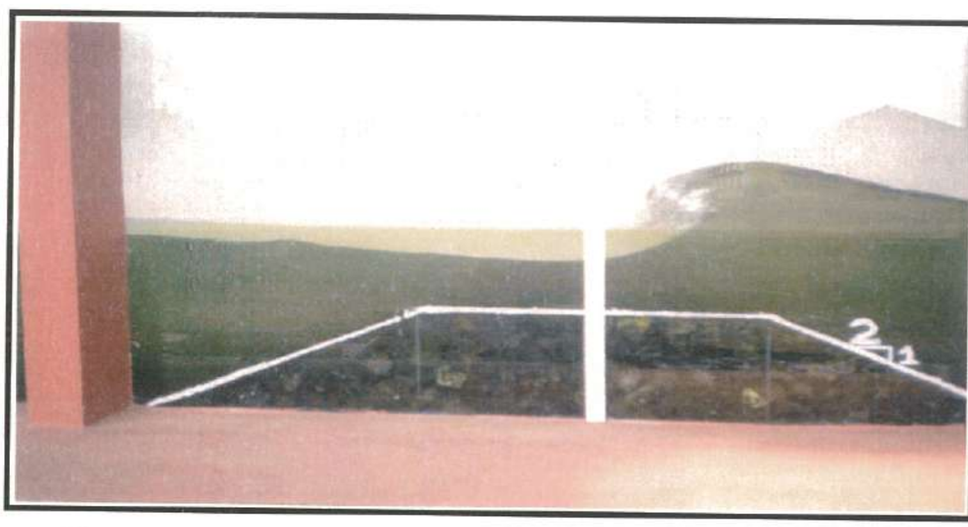

Plate. 5.10. Waves plunging over the submerged reef (for d = 0.4m)

5.8.3 Protected breakwater

In the fourth phase, a stable trapezoidal submerged reef having a slope of 1V:2H with a crest width (B) of 0.1m and height (h) of 0.25m is constructed, with homogeneous pile of stones of optimum weight at varying seaward distances (X) from the conventional (main) breakwater within the required spacing derived (refer Fig. 5.5).

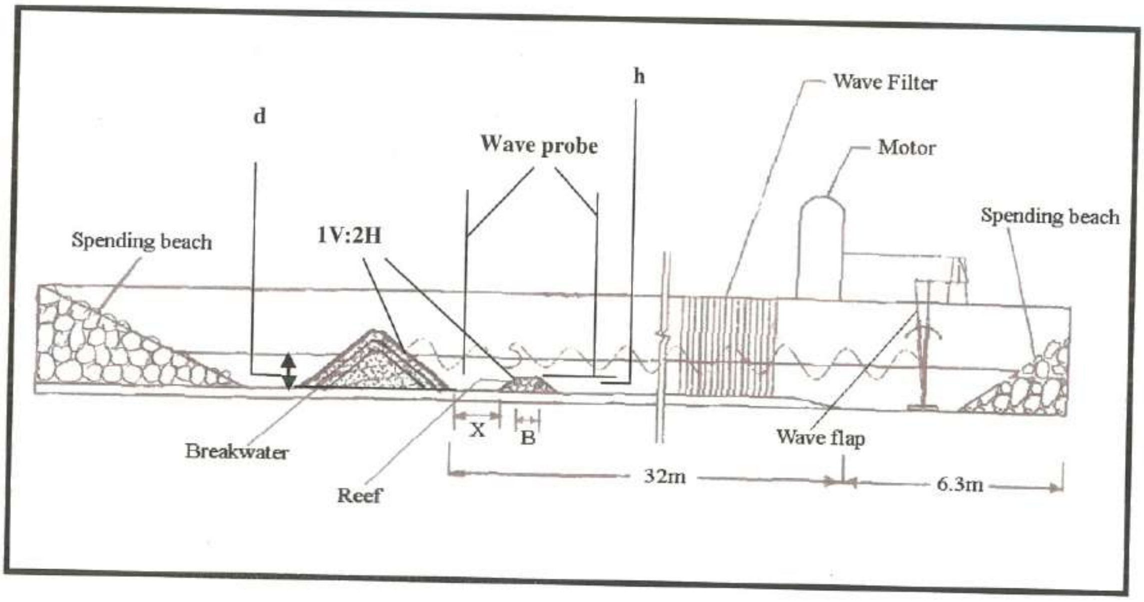

Fig. 5.5. Details of model testing of protected breakwater

Plate. 5.11 shows the view of the protected breakwater with a conventional breakwater and seaward submerged reef. Plate 5.12 exhibits the top view of the protected breakwater.

Plate. 5.11. View of the protected breakwater

Plate. 5.12. Top view of protected breakwater

The models are then tested. Once the location of the reef is selected, the tests are repeated, in the fifth phase, with a reef of different crest widths (B) of 0.1m, 0.2m, 0.3m and 0.4m to arrive at the crest width that protects the main breakwater. The model characteristics together with their possible range of application and wave characteristics are listed in Table. 5.4.

5.9 MODEL TEST CONDITIONS

By comparing a number of known prototype breakwater damages with associated model investigations, Hudson (1975) as quoted by Torum et al. (1979) considers that hydraulic scale model can be used to determine stability of rubble mound breakwater and that, accuracy of results will be with in the limits required, to design safe and economical full scale structures, if models are designed and operated correctly and test conditions are selected judiciously.

In nature, a wave striking a submerged barrier will result in some of its energy being reflected offshore where it is ultimately dissipated by wind and internal stresses. In the lab, reflected waves strike the wave paddle and are almost totally reflected and create waves which may not represent the prototype condition truly. This may go on and on. The net result is a wave system which differs considerably from simple model in the nature i.e. assuming mono-periodic waves could occur in nature (Dick and Brebner 1968). This can be dealt by conducting experiments with series of wave bursts, such that, each burst of waves ending before re-reflected waves can again reach the testing section of the wave flume (Hughes 1993).

In the present experimental facility reflected and re-reflected waves from the generator flap give rise to unduly high, complex and undefined waves. Therefore, to avoid wave reflection affecting the model performance, in the present experimental investigation, waves are sent in bursts of 5 waves, and generator is switched off between the bursts to allow sufficient time for the wave energy to dampen out.

The present physical model study is conducted in a two dimensional wave flume where, only the breakwater damage due to displacement of the armour stones in response to normal attack of regular waves is considered as the prime cause, and influence of sediment movement and currents on the structure performance is neglected.

Diskin et al. (1970) noticed that, for submerged breakwater with crest at SWL, piling is 0.35 times wave height, for F/H_o of 0.5, piling is $0.15H_o$ and for $F/H_o > 1$, piling is less than $0.05H_o$ with an accuracy of 10% to 20%. The set up for non-breaking waves is 2% to 5% of incident wave height, while for breaking wave, set up reaches up to 20% of H_i or higher depending upon H_i (Mendez et al. (2001).

In the present experimental work, in most of the test runs F/H_i values are greater than 0.5. Hence, if at all piling of water was taking place behind the reef, it may be between 0.05H and 0.15H. However, piling is discouraged by generating small number of waves (i.e. five waves) in bursts with sufficient time interval in between them. The present study is of fundamental nature and therefore, the breakwater damage due to armour stone displacement due to wave attack is the only mode of failure that is considered.

Sollit and Cross (1970) found that for a layered submerged structure of Kd (i.e. $2\Pi d/L$) values of 1.5 and 2.01, K_r increased from 0.2 to 0.3 while, wave steepness increased from 0.005 to 0.05. But for a given steepness of H/L of 0.01 and 0.02, K_r decreased from 0.53 to 0.25.

Stive (1985) conducted small scale and large scale tests of regular and irregular breaking waves. Stive's results indicated that there was no significant departure from Froude scaling in a wave height range of 0.1m to 1.5m with regard to the parameters measured. This result proved that observed difference in air entrainment between model and prototypes have no significant dynamic influence. When the wave break on armour units in a physical model, the flow is turbulent so it is expected that pressure produced in the model are in similitude with the prototype (Le Mehaute 1976). The wave heights used in the present study are within the range indicated by Stive's results.

Scale model experiments usually are conducted using fresh water. If the prototype condition is salt water, there is about a 3% difference in density and this changes wave force accordingly. This effect could cause as much as 10% to 15% error in breakwater stability studies (Le Mehaute 1976). Sharp and Khader (1984) as quoted by Hughes (1993) conducted 1:10 scale model tests where fresh water in the model represents a salt water prototype. This particular example indicated that Hudson scaling called for model armour units about 8% lighter than suggested. This means that sealing by Hudson stability parameter (i.e. keeping the same stability number between prototype and model) will produce more conservative designs for stable rubble mound structures because the lighter model armour units are more easily moved by waves.

The best morphological effects, pronounced as accretion of sediment between the submerged breakwater and shoreline have reached for the slope of the seaward wall ranging 1V:2H to 1V:3H. Better dissipation of waves, lower reflection and easier transport of sediment over the structure were observed for these slopes (Pilarczyk and Zeidler 1996). Hence, a slope 1V:2H is selected for the structures of present model study.

Considering all the above facts, the model tests of the present investigation are carried out with the following conditions:

1. The sea bed is rigid and horizontal and it is assumed that the sediment movement does not interfere with the wave motion and do not affect the model performance.
2. The waves are periodic and monochromatic.
3. Waves are generated in short burst of five waves.
4. Between wave bursts there are brief intervals to allow wave energy to dampen out.
5. Secondary waves generated during the test are not considered.
6. Wave reflection from the flume bottom or flume side walls is not considered.
7. Wave reflection from the structure does not interfere with generated incident waves.
8. The density difference between freshwater and seawater is not considered.
9. Only hydraulic performance of the model is tested.
10. Piling up of water behind the reef is negligible.

Table. 5.6 compares the parameters of field and model tests of protected structures as given by various investigators with those selected for present study. Gadre et al. (1985), Gadre et al. (1989) and Cox and Clark (1992) report prototype studies and whereas Cornett et al. (1993) and Neelamani et al. (2002) conducted physical model studies in the laboratory.

Table. 5.6. Comparison of field and model parameters of protected structures tested by various investigators with those selected for the present study

Parameter	Gadre et al. (1985)	Gadre et al.(1989)	Cox and Clark (1992)	Cornett et al. (1993)	Neelamani et al. (2002)	Present Study
X (m)	100	80	40.5	2.11	5.3	1.0 – 4.0
d (m)	7.0	9.0	4.5	0.55	0.5 – 0.8	0.3 – 0.4
B (m)	5.0	9.5	6.9	0.1	0.2	0.1 – 0.4
h (m)	4.0	4.5	3.87	0.17- 0.33	0.6	0.25
F (m)	3.0 - 4.52	4.5	0.6	0.1 – 0.38	0.1 – 0.2	0.05 – 0.15
H (m)	5.0	7.0 – 9.0	5.82 – 7.08	0.12 – 0.2	0.05 – 0.3	0.1 – 0.16
T (sec)	10.0	-----	10.3 – 11.7	1.0 – 3.0	1.0 – 3.0	1.5 – 2.5
X/d	14.2	8.88	9.0	3.83	6.63 – 10.6	2.5 – 13.33
B/d	0.714	1.06	1.5	0.18	0.25 – 0.4	0.25 – 1.33
h/d	0.57	0.5	0.86	0.3 – 0.6	0.75 – 1.2	0.63 – 0.83
F/H	0.6 – 0.9	0.5 – 0.64	0.085 - 0.103	0.5 – 3.17	0.33 – 4.0	0.312 – 1.5
H/gT^2	0.0051	-----	0.0043 – 0.0068	0.0014 – 0.02	0.00057 – 0.03	0.0016 – 0.0072

5.10 TEST PROCEDURE

By comparing a number of known prototype breakwater damages with associated model investigations, Hudson (1975) as quoted by Torum et al. (1979) considers that hydraulic scale model can be used to determine stability of rubble mound breakwater and that, accuracy of results will be with in the limits required, to design safe and economical full scale structures, if models are designed and operated correctly and test conditions are selected judiciously. Therefore, model design, construction and testing should be planned and executed properly using proven and accepted techniques/procedures which can be standardized and repeated accurately (Oullet 1970 and Zwamborn 1979). The safety of the structure must be determined by using conditions well in excess of design wave conditions because more often than not, these have a very low design reliability (Zwamborn 1979).

To minimize uncertainty the experiment has to be properly planned, experimental procedures and extrapolation methods should be standardized and sources of errors and magnitude of errors have to be minimized (Mishra 2001).

5.10.1 Steps undertaken to reduce errors

Following steps are undertaken during the present experimental investigation to ensure laboratory and scale effects are minimised and do not influence the model performance adversely (Ahrens 1970, Van der Meer and Pilarczyk 1984, Palmer and Walker 1976, Zwamborn 1979, Van der Meer 1984 and Hughes 1993).

1. The model is constructed, as per the standard procedure, with a scale of 1:30 which is the largest possible scale within the limitations of the laboratory constraints.
2. The depth of water in the flume is maintained exactly at the required level and is continuously monitored. Average variation of 2mm was found after a full day of model testing. Any drop in the water level of more than 3mm was immediately corrected.
3. Initially calibration of flume, wave generation facility and wave probes are undertaken to determine the proper wave height to assign to a particular combination of generator stroke and wave period. The wave heights used in the test runs are obtained during calibration. This will exclude the losses due to interference of flume bed, sidewalls and reflection and therefore, eliminates these error sources.
4. Waves are run in short bursts of five during the tests and the wave generator is shut off just before wave energy reflected from the slope could reach generator blade.
5. Between wave bursts there will be brief interludes to allow reflected wave energy to dampen out.
6. All the wave characteristics are measured with more than 30 readings and are statistically analysed. Similar exercise is repeated for other parameters such as wave run up/down over the breakwater.
7. Tests are started by surveying newly constructed slope, with profiler which becomes reference survey for comparison of subsequent surveys.
8. The height of first waves is normally smaller (i.e. 0.08m which is 20% lower) than the design wave height of 0.1m, which, could dislodge the armour and gradually the wave heights are increased by 0.02m and the model is tested for wave heights up to 0.16m which are well in excess of design wave condition.

9. Waves for each test run are generated in bursts until it appeared that no further stones from the armour would be moved by waves of this height and 3000 waves would be run before the slope was considered stable (which is equivalent to an actual storm of 6.85 hours to 11.41hours). This is because 80% to 90% of the total damage would have already inflicted by that time and equilibrium would have established (Van der Meer and Pilarczyk 1984 and Hegde 1996).
10. Failure for these tests is defined as having occurred when enough armour stones are displaced so that filter layer was exposed.

5.10.2 Test procedure

After carefully studying the model test conditions and procedures adopted by various investigators, the following procedure is evolved for the present study.

1. All the breakwater models are constructed, in the flume, at about 28m to 32m from the wave generating chamber.
2. The wave flume is filled with ordinary tap water to required depths of 0.3m, 0.35m and 0.4m and the entire experimental set up including the wave probes are calibrated or each depth of water.
3. In the beginning of the every test run, the breakwater is constructed to the standard size (if it is damaged) and its slope is surveyed with the profiler, which is, the reference survey (initial profile) for comparison of subsequent surveys (final profile after damage).
4. The model is tested for varying wave characteristics as shown in Table 5.4. The model is subjected to normal attack of regular waves. The waves are generated in bursts of five waves with brief interludes to allow reflected wave energy to dampen out.
5. 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 in 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.
6. Initially a series of smaller wave heights starting from 0.8m are passed and then gradually wave height is increased by 0.02m each time, till it reached the height of 0.16m for a selected period. 3000 waves are run for each trial or the exposure of the secondary layer whichever occurred earlier.

7. In the remaining phases, where the stability and wave transmission at the submerged reef and performance of the defenced breakwater is tested, the incident waves are measured at 1m seaward of the reef and the transmitted wave heights on the leeside are observed at 1m leeward of the reef and at the toe of the inner breakwater.

5.11 OBSERVATIONS

When conducting the experiment, the incident wave height, wave period, wave breaking over the submerged reef, transmitted wave height and wave at toe of the inner (main) breakwater are observed and recorded. The waves while breaking over the submerged reef produced lot of turbulence and lose a major portion of the energy. Then the remaining energy gets transmitted in the stilling basin (energy dissipating zone), and the waves may further lose some more energy. The initial displacement of stones is observed from the area around the SWL, which, is subjected to severe wave action. Plate 5.13 shows the waves breaking at the submerged reef of the protected breakwater.

Initially, waves caused the readjustment of stones to stable positions and as number of waves attacking the breakwater increased, the movement in the primary layer is observed due to rocking, lifting and pulling of the stones down the slope. It is observed that the stones are dislodged during run-up and then dragged down by run-down. The stones which are dragged, move down the slope and spread uniformly at the lower part of the slope where the influence of the wave activity is considerably less. The movement of stones started slowly with the impact of about 100 to 300 waves and continue rigorously up to about 1500 to 2000 waves after which it is only a trickle up to 3000 waves. After 3000 waves, the breakwater profile stabilised and there was no further displacement of stones.

Plate. 5.13. Waves breaking at the submerged reef of protected breakwater

The maximum damage of the breakwater occurred in the area between SWL + H_i and SWL - H_i due to run-down of the waves which is similar to the observation made by Kreeke (1969) and Font (1970). All the displaced stones formed a berm just below SWL.

5.12 MEASUREMENTS

5.12.1 Measurement of wave heights

The incident and transmitted wave heights are recorded at 1m seaward side and 1.0m leeside of the reef and wave heights are also recorded at toe of the inner (main) breakwater. The values of transmission coefficient (K_t) are computed as the ratio of H_t and H_i. The statistics, namely, the ranges of standard deviation and coefficient of variation of all wave heights (i.e. incident and transmitted waves) recorded for all depths of water in the entire model tests of the present study are listed in the Table. 5.7.

Table. 5.7. Statistics of all wave heights recorded

Depth of water d (m)	Range of statistical parameters of wave heights	
	Standard deviation σ (m)	*Coefficient of variation c
0.30	0.09 – 0.17	0.0126– 0.0229
0.35	0.11 – 0.2	0.0091– 0.0281
0.40	0.14 – 0.21	0.0086 – 0.024

* Melby and Mlakar (1997) used a value of 0.52 in the reliability analysis of the breakwaters.

5.12.2 Measurement of wave run-up and run-down

Wave run-up is measured as the maximum vertical distance above the SWL to which the wave up rushes over the structure slope when a wave impinges on inner (main) breakwater. The vertical distance below the SWL up to the minimum elevation attained by wave on the breakwater slope is recorded as wave run-down. The wave run up and run down recorded over the breakwater on the graduated scale fixed over the glass panels of the flume. The statistics of all run up and run down recorded for the entire model tests of the present study are listed in the Table. 5.8 and Table. 5.9 respectively.

Table. 5.8. Statistics of all run up values recorded

Depth of water	**Range of statistical parameters of run ups**	
d (m)	**Standard deviation σ (m)**	**Coefficient of variation c**
0.30	0.08 – 0.18	0.0074 – 0.0199
0.35	0.13 -0.18	0.0093 – 0.0173
0.40	0.1 – 0.13	0.0102 – 0.0112

Table. 5.9. Statistics of all run down values recorded

Depth of water	**Range of statistical parameters of run down**	
d (m)	**Standard deviation σ (m)**	**Coefficient of variation c**
0.30	0.1 – 0.18	0.0171 – 0.0350
0.35	0.12 – 0.17	0.0144- 0.0293
0.40	0.12 – 0.19	0.012 – 0.0261

5.12.3 Measurement of breakwater damage

The depth of erosion assesses the damage. Plate. 5.14 gives the picture of the damaged protected inner (main) breakwater. During sounding, the sounding rods of the surface profiler are lowered so that their feet rest on the armour units of the breakwater. The height of the markings on top of sounding rod above the wooden frame is measured. These heights are recorded as the initial profile of the breakwater. Plate. 5.15 demonstrate the measurement of damage of the breakwater using a surface profiler.

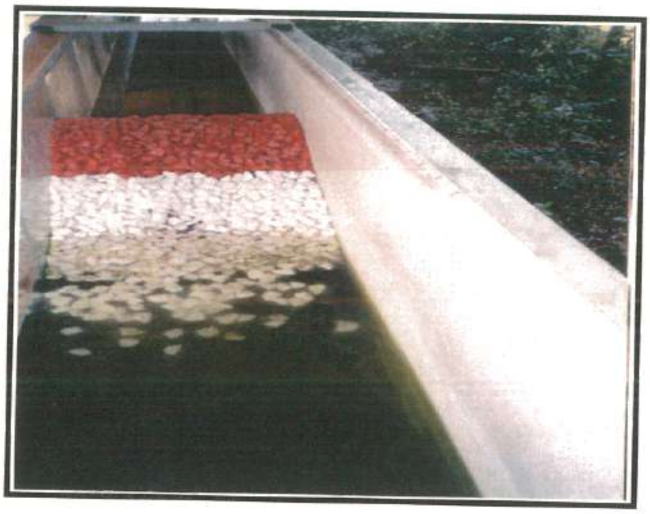

Plate. 5.14. Damaged inner (main) breakwater

A breakwater surface profiling is under taken after 3000 waves or the exposure of secondary layer whichever occurred earlier and is recorded as the final profile of the breakwater. The difference between the heights at a point is treated as the height of erosion or accretion depending on the height, measured after a test, is lesser than or greater than that before the test. By knowing the depth of erosion, the area of erosion along the path of each sounding rod is calculated. The average areas of erosion along the paths of nine sounding rods are taken as the average area of erosion (A_e). The damage level (S) of the main breakwater is calculated as A_e/D^2_{n50}.

Plate. 5.15. Measurement of breakwater damage

Chapter 6
Armour Stability of Conventional Breakwater and Submerged Reef

6.1 GENERAL

A conventional non-overtopping rubble mound breakwater of uniform slope of 1V:2H is designed and constructed with three layers namely primary armour, secondary armour and core. This model section is tested for wave loads more than that of the design values. The impact of higher waves (i.e. wave steepness) on the run up, run down, stability of armour is studied. Further this chapter explains the study conducted on the submerged reef to determine the armour stability and wave transmission characteristics.

6.2 DETAILS OF MODEL STUDY OF CONVENTIONAL BREAKWATER

A 1:30 scale model of a breakwater, of trapezoidal cross section with a uniform slope of 1V:2H, is constructed on the flat bed of the flume with primary stone armour of weight of 73.2gms (i.e. nominal diameter, D_{n50}, of 0.0298m) for a non-breaking design wave of 0.1m. Its crest width is 0.1m and height is 0.70m. The secondary armour of mean weight of about 7.32gm and core of size of about 300microns is designed. The porosities of the primary armour, secondary armour and core are 43%, 39% and 36% respectively. The primary armour units are painted with different colours and placed in bands of heights of 0.2m to 0.3m to track their movement during damage. In the first phase this conventional breakwater model is tested. The breakwater model characteristics are listed in Table. 5.3.

The model is constructed at a distance of 32m from the generator flap. The cross section of breakwater denoting layers is drawn on the glass panel. Pipes are placed over the flume bed, at the breakwater test section, to balance the water levels on seaside and leeside. The core material is placed and formed to the required level by light compacting with hands. Then, the secondary and the primary layer are constructed to the marked level. The technique adopted in the present work is keyed and fitted placement. The primary armour layers are painted to reduce the surface roughness. In the rubble mound breakwater model, the armour stones placed around SWL are painted in white and in the zone above, it is the armour stones with red colour while in the zone below, the stones were painted with black colour to reduce the friction between the armour stones and identify the damage zone after the test. Further, the details of breakwater model construction are explained in Chapter 5.

Before the model tests are started, the experimental set up along with the wave probes is calibrated to find the required wave heights which, are assigned to a particular combination of generator stroke and wave period, for depths of water of 0.3m, 0.35m and 0.4m. The model is subjected to regular waves (H_i) of height varying from 0.1m to 0.16m of a range of period (T) varying from1.5sec to 2.5sec generated in water depths (d) of 0.3m to 0.4m. The model is tested for varying wave characteristics as shown in Table. 5.4. The model is subjected to normal attack of regular waves. The waves are generated in bursts of five waves with brief interludes to allow reflected wave energy to dampen out. During the test the number of incident waves, their heights and periods, wave run up and run down over the breakwater slope and armour stone movements are recorded. Further, the details of test procedure are explained in Chapter 5.

6.3 ANALYSIS AND INTERPRETATION OF DATA

The data collected in the present experimental work, like wave characteristics, wave run up and run down and damage of armour stones is initially expressed in non-dimensional quantities. The graphs of relative run up and run down versus deep water wave steepness parameter (H_o/gT^2), damage level (S) versus steepness parameter and stability number and finally stability number versus surf similarity parameter (ξ) are plotted. The relationship between the parameters is analysed through the graphs.

6.3.1 Influence of deep water wave steepness on run up and run down

The influence of deep water wave steepness parameter ($H_o/(gT^2)$, on relative run up (R_u/H_o) and run down (R_d/H_o) for increasing ranges of depth parameter (d/gT^2) i.e varying wave climate in depths of water of 0.3m, 0.35m and 0.4m, is shown in Fig. 6.1 and Fig. 6.2 respectively. Both the relative run up and the run down, decrease with an increase in wave steepness and there is no significant impact of d/gT^2. For $1.45\text{x}10^{-3} < H_o/gT^2 < 7.85\text{x}10^{-3}$ and all ranges of d/gT^2, R_u/H_o vary from 0.96 to 1.57. The best fit lines show a decrease of R_u/H_o from 1.557 to 1.136 (27%). Similarly, from the Fig. 6.2, it is seen that, the best fit line

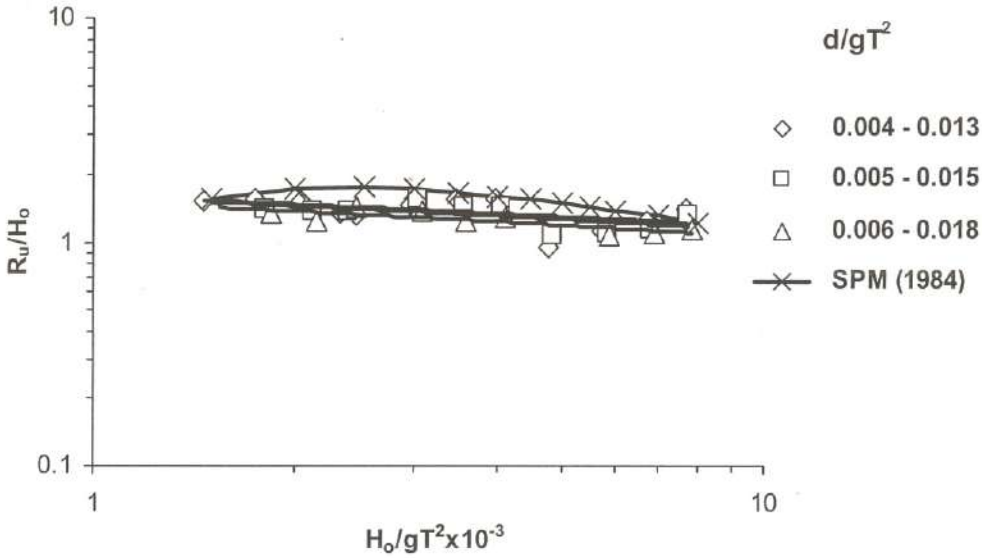

Fig. 6.1. Variation of R_u/H_o with H_o/gT^2

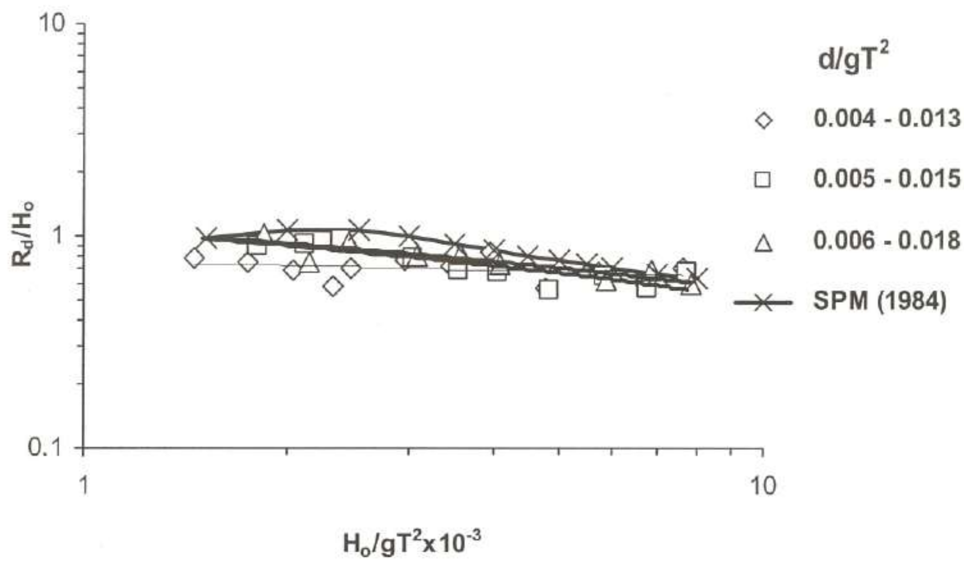

Fig. 6.2. Variation of R_d/H_o with H_o/gT^2

shows R_d/H_o drop from 0.95 and 0.6 (36.84%), while, the extreme values vary between 0.95 and 0.56 for the complete range of H_o/gT^2. Influence of depth is visible only for a depth of water of 0.3m. In the present study, it is observed that, run up and run down are relatively higher for $1.45x10^{-3} < H_o/gT^2 < 4x10^{-3}$, when, $4 < \xi < 4.25$. This is because, in this range of ξ, surging breakers occur. The relative run up and run down are up to 18% and 32% lower than those given for the conventional breakwater (for $d/H_o > 3.0$) by US Army Corps of engineers (1984).

6.3.2 Influence of various parameters on breakwater stability

6.3.2.1 Influence of deep water wave steepness

The trends of increasing damage level (S) with wave steepness parameter (H_o/gT^2) and ranges of depth parameter (d/gT^2), i.e. waves of different periods in each depth of water of 0.3m, 0.35m and 0.4m, are shown in Fig. 6.3. This is because steeper waves have higher energy and inflict increased damage on the breakwater and higher depths can sustain steeper waves.

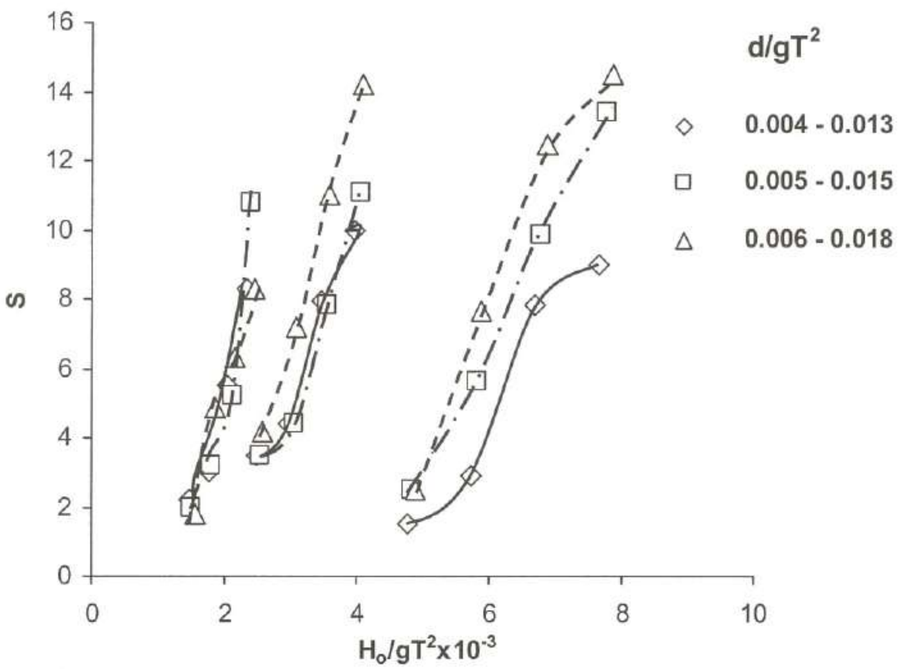

Fig. 6.3. Variation of S with H_o/gT^2

The impact of wave period also can be seen. Considering all the three ranges of d/gT^2, the damage levels (S) due to shorter period waves of 1.5secs (i.e. $4.77 \times 10^{-3} < H_o/gT^2 < 7.85 \times 10^{-3}$) are seen on right hand side of the figure, S for waves of period 2.5sec (i. e. $1.45 \times 10^{-3} < H_o/gT^2 < 2.46 \times 10^{-3}$) are seen on the left hand side and for period of 2sec (i. e. $2.47 \times 10^{-3} < H_o/gT^2 < 4.1 \times 10^{-3}$) S are in the middle of the figure. It is also seen that influence of depths of water on damage decreases with an increase in wave period. Figure shows large damage levels for short period waves and the damage levels decrease with an increase in period. Considering all the ranges of d/gT^2 (i.e. waves in all depths of water of 0.3m, 0.35m and 0.4m), the increase in damage levels are 9 to 14.5 (61.1%), 10 to 14.23 (42.3%) and 8.3 to 10.80 (30.1%) for waves of periods of 1.5 sec, 2.0sec and 2.5sec respectively. Ranges of damage level S decreases with an increase in period for all the ranges of d/gT^2.

6.3.2.2 Variation with stability number

The damage level (S) of the breakwater increases with an increase in stability number (N_s) for different depth parameters (d/gT^2), i.e varying wave periods in depths of water of 0.3m, 0.35m and 0.4m, as shown by best fit lines in Fig. 6.4, Fig. 6.5 and Fig. 6.6 respectively.

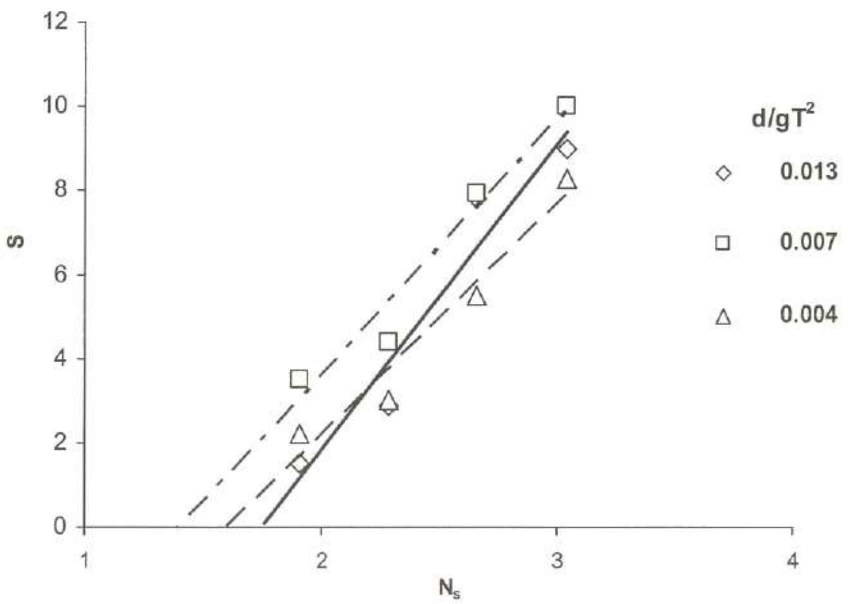

Fig. 6.4. Variation of S with N_s for d = 0.3m

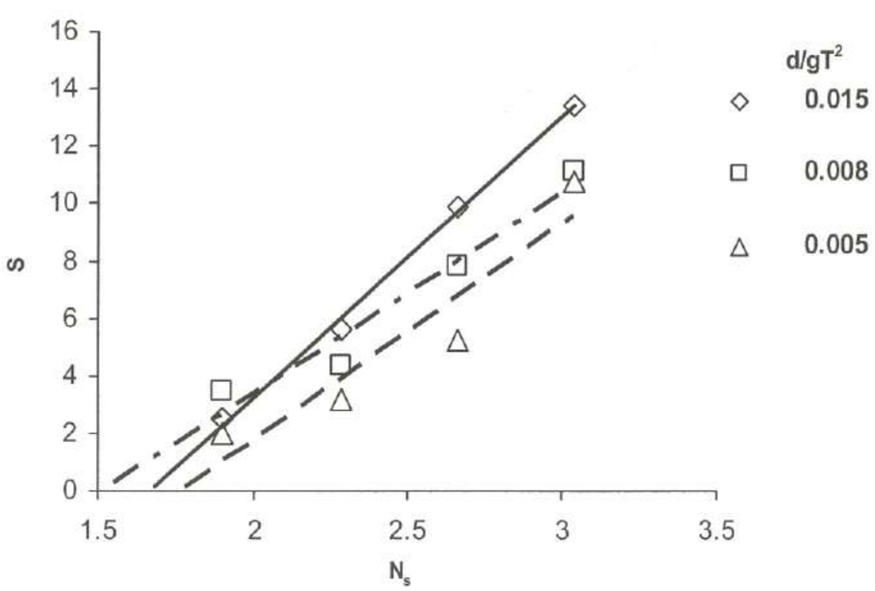

Fig. 6.5. Variation of S with N_s for d = 0.35m

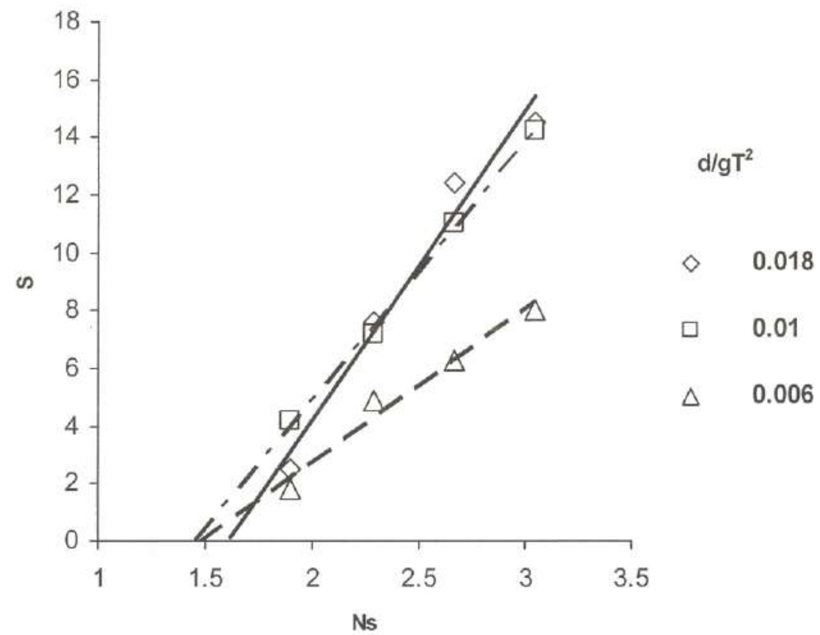

Fig. 6.6. Variation of S with N_s for d = 0.40m

Considering $0.004 < d/gT^2 < 0.013$ in the depth of water of 0.3m, maximum damage levels increase from 8.3 to 10.0 (i.e. 20.48%), for $0.005 < d/gT^2 < 0.015$ in depths of 0.35m, the maximum damage levels increase from 10.8 to 13.4 (i.e. 24.07%) and for $0.006 < d/gT^2 <$ 0.018 in the depth of 0.4m they increase from 8.05 to 14.5 (i.e. 80.1%).

The zero damage wave heights (H_{zd}), for $0.004 < d/gT^2 < 0.013$, (i. e a depth of 0.3m) and wave periods of 1.5 sec, 2.0sec and 2.5sec are 0.1071m, 0.0765m and 0.0997m respectively. The H_{zd}, for $0.005 < d/gT^2 < 0.015$ (i.e. for a depth of 0.35m) and wave periods of 1.5 sec, 2.0sec and 2.5sec are 0.0908m, 0.0828m and 0.1081m respectively and for $0.006 < d/gT^2 <$ 0.018 (i.e. for a depth of 0.4m) and for wave periods of 1.5 sec, 2.0sec and 2.5sec, the H_{zd} are 0.0828m, 0.0694m and 0.0908m respectively. It is found that, for any given depth of water (d), zero damage wave heights (H_{zd}) are minimal at a wave period (T) of 2sec indicating least breakwater stability. Zero damage wave heights are up to 30.6% lower and 8.1% higher than the design wave of 0.1m.

6.3.2.3 Influence of surf similarity parameter on stability number

Fig. 6.7 shows the variation of zero damage stability number (N_{zd}) with surf similarity parameter (ξ), for different ranges of depth parameter (d/gT^2) i.e. varying wave climate in depths of water of 0.3m, 0.35m and 0.4m for the present study. The results are compared with those given by Thompsen et al. (1972). They showed that, minimum stability of a 1V:2H sloped rubble mound breakwater occurred for $2 < \xi < 3$. According to Bruun and Gunbak (1976), the failure of breakwater is caused by combinations of buoyancy, inertia and drag forces supported by the effect of hydrostatic pressure from the core. These forces all seem to reach their maximum value for lowest down rush which occurs at resonance. Bruun and Gunbak (1976) observed that, this condition was reached for$2 < \xi < 3.0$. In the figure, it is observed that, for the present study, minimum stability of the conventional breakwater (i. e. S = 1.32 to 1.57 for waves of period 2sec in different depths of water) occurs for $4.3 < \xi < 4.8$. This is because, in this range of ξ (i.e. for $\xi > 3.3$), surging breakers occur (Weisher and Byrne 1978). In model study, during such condition,

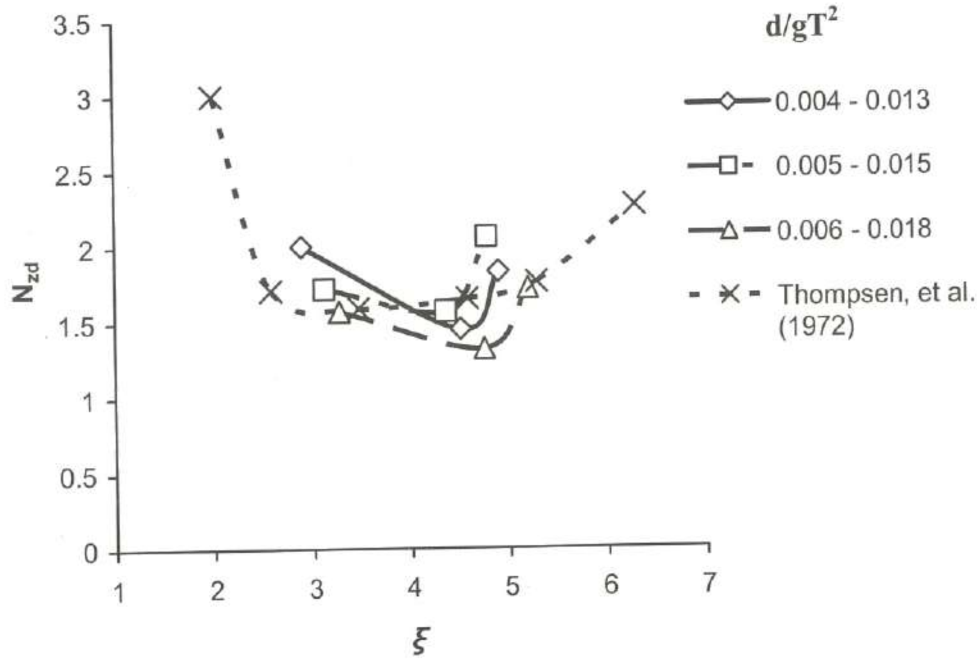

Fig. 6.7. Variation of N_{zd} with ξ

it is found that run up is maximum and run down is relatively high and the armour stones are disturbed and lifted by run up and pulled down the slope by the run down as stated by Ahrens (1970).

6.4 STUDIES ON SUBMERGED REEF

6.4.1 Study of reef armour stability

In the second phase, a 1:30 scale model of a trapezoidal reef, of slope 1V:2H, height (h) of 0.25m and crest width (B) of 0.1m is constructed over the flat bed of the flume with armour of mean weight varying from about 15gms to 35gms as given by various design criteria (Ahrens 1984 and 1989, Gadre et al. 1992 and Nizam and Yuwono 1996 and Piarczyk and Zeidler 1996).

This test section is subjected to normal wave attack of 3000 regular waves of height ranging from 0.1m to 0.16m, of periods varying from 1.5sec to 2.5sec in a depth of water of 0.3m as the reef stability is critical at the lowest water level. The damage to the structure is recorded in the form of reduction in crest height (h_c). The graphs of dimensionless damage (h_c/h) versus spectral stability number (N_S*) illustrated in Fig. 6.8 show increasing damage with an increase in stability number for armour stones of 15gm, 20gm and 25gm. The armour stones of 35gm are stable however, it is seen that the armour stones of 30gms are relatively safer in spite of some small damage. Therefore, it is concluded that, armour stone of mean weight 30gm (i.e. nominal diameter of 0.0221m) is safe for the submerged reef under test conditions.

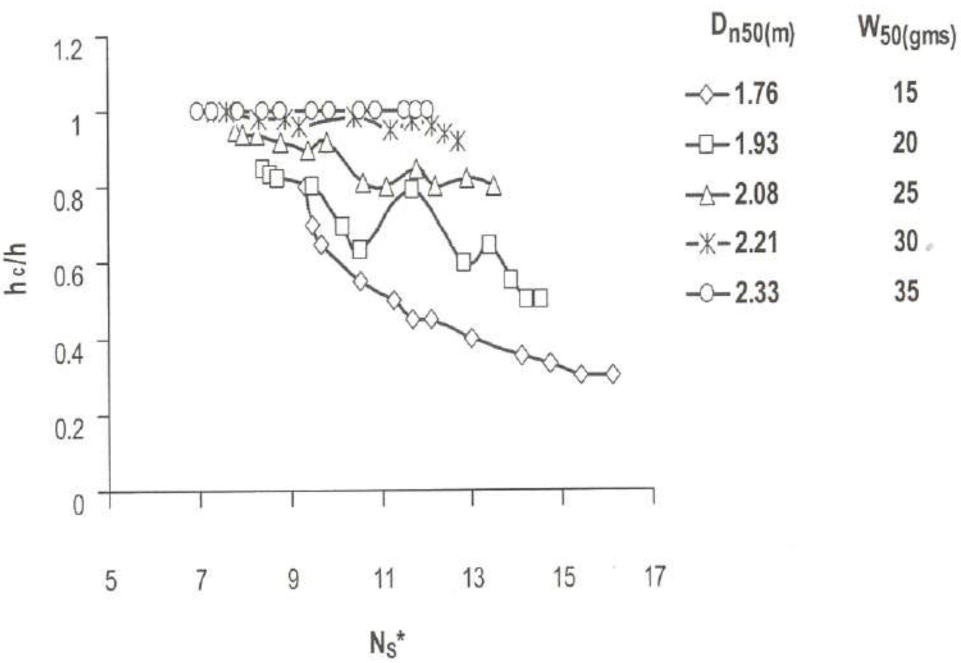

Fig. 6.8. Variation of h_c/h with N_S*

6.4.2 Study of wave transmission at the reef

The submerged reef successfully trips the steeper waves and dissipates a major portion of the wave energy. The effectiveness of reef in damping of waves increases with an increase in wave steepness. Further, as the shoreward distance (X) increases, the transmitted waves loose some more energy while propagating in the stilling basin (i.e. the energy dissipation zone). In the present study the waves of 0.1m to 0.16m height and of periods of 1.5sec to 2.5sec are generated in water depths of 0.3m to 0.4m and are passed over the submerged reef. The waves break over the reef and the transmitted wave heights are recorded for every metre up to 8.0m on the leeside (i. e. X/d of 2.5 to 26.67). It is observed that up to a distance of 4m (i. e. X/d of 10.0 to 13.33) on the leeside, there is a maximum wave attenuation of about 49.5% beyond which there is no significant increase in wave attenuation and the trend is similar. This is close to the maximum wave height attenuation (WHA = 1- K_t) of 50% observed in sea at a natural bar (Battjes and Jansen 1978 and Cox and Clark 1992). Fig. 6.9 shows the best fit lines for WHA obtained in the present study for varying deep water wave steepness parameter and different ranges of distances (X/d) on the leeside. It can be seen from the figure that, for $1.45 \times 10^{-3} < H_o/gT^2 < 7.85 \times 10^{-3}$, the trend lines show WHA increasing from 5% to 18%, 20% to 33% and 32% to 44% for ranges of X/d 2.5 to 3.33, 6.25 to 8.33 and 10 to 13.33 respectively.

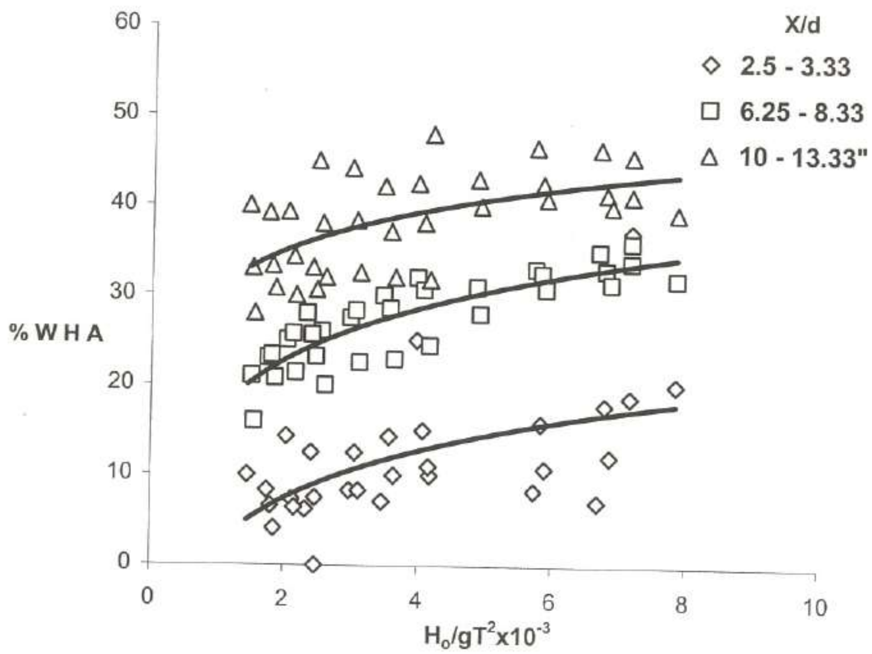

Fig. 6.9. Variation of WHA with H_o/gT^2

Therefore, it is concluded that the submerged reef should be located within a maximum seaward distance of 4m (i. e. X/d of 10 to 13.33) from the breakwater as a protective structure and further investigation could be carried out for determining the geometry and location of the reef that protects the inner (main) breakwater totally for the test conditions adopted.

6.5 CONCLUSIONS

From the analysis and the interpretation of the data presented in this chapter, the following conclusions are drawn.

6.5.1 Conclusions for conventional breakwater

1. Both the relative run up (R_u/H_o) and the run down (R_d/H_o), decrease with an increase in wave steepness and there is no significant impact of depth parameter d/gT^2.
2. Considering the complete ranges of H_o/gT^2 and d/gT^2, the maximum relative run up (R_u/H_o) and run down (R_d/H_o) are respectively 1.57 times and 0.95 times the deep water wave height.
3. The relative run up and run down are up to 18% and 32% lower than those given for the conventional breakwater by US Army Corps of engineers (1984).
4. The damage level S increases with increase in H_o/gT^2, d/gT^2 and N_S.
5. The breakwater damages are in the range of 1.5 to 10.0, 2.0 to 13.4 and 1.81 to 14.5 in the depths of 0.3m, 0.35m and 0.4m respectively.
6. Considering all the ranges of d/gT^2 (i.e. waves in all depths of water of 0.3m, 0.35m and 0.4m), the increase in damage levels are 9 to 14.5 (61%), 10 to14.23 (42.3%) and 8.3 to 10.80 (30.1%) for waves of periods of 1.5 sec, 2.0sec and 2.5sec (i.e. ranges of $4.7\times10^{-3} < H_o/gT^2 < 7.85\times10^{-3}$, $2.5\times10^{-3} < H_o/gT^2 < 4.0\times10^{-3}$ and $1.45\times10^{-3} < H_o/gT^2 < 2.46\times10^{-3}$) respectively.
7. As the depth of water increases from 0.3m to 0.35m (i.e. 16.67%) the maximum damage level of the breakwater increased from 10 to 13.4 (i.e. by 34%) and at a depth of 0.4m (i.e. an increase of 33.3% w.r.t 0.3m) damage level increases from to 14.5 (i.e. by 8.2%).
8. Considering $0.004 < d/gT^2 < 0.013$ i.e. in the depth of water of 0.3m, maximum damage levels increase from 8.3 to 10.0 (i.e. 20.48%), for $0.005 < d/gT^2 < 0.015$ i.e. in depths of 0.35m, the maximum damage levels increase from 10.8 to 13.4

(i.e. 24.07%) and for $0.006 < d/gT^2 < 0.018$ i.e. in the depth of 0.4m they increase from 8.05 to 14.5 (i.e. 80.1%).

9. Zero damage wave heights (H_{zd}) are minimal at a depth of 0.4m and a wave period (T) of 2 sec indicating least breakwater stability.
10. Zero damage wave heights are up to 30.6% lower and 8.1% higher than the design wave of 0.1m.
11. Minimum stability of the breakwater i. e. S = 1.32 to 1.57 occurs for $4.3 < \xi < 4.8$.

6.5.2 Conclusions for submerged reef

1. The optimum armour stone weight for a stable reef is 30gm.
2. It is observed that up to a distance of 4m on the leeside of the reef, waves are attenuated by about 49.5%, beyond which there is no significant increase in wave attenuation.
3. Reef may be located within a maximum distance of 4m (i. e. X/d of 13.33) seaward of the inner (main) breakwater for major experimental work of testing protected breakwater.

Chapter 7
Investigation of Protected Breakwater with a Reef at a Spacing of 1m

7.1 GENERAL

A conventional non-overtopping rubble mound breakwater when subjected to attack of waves higher than the design wave height, as in the case of storm waves, gets severely damaged. Therefore, a protective structure to the breakwater is proposed to be designed. In this context, it is decided to test a model of protected breakwater.

A conventional non-overtopping rubble mound breakwater of uniform slope of 1V:2H is designed and constructed with primary stone armour, secondary stone armour and core. A stable trapezoidal submerged reef is constructed on the seaward side of the main breakwater at a distance (X) of 1m and this model is tested for varying wave climate. In this chapter the influence of waves (i.e. wave steepness) on wave transmission at reef and on run up, run down, stability of armour of the inner main breakwater is investigated.

7.2 DETAILS OF PHYSICAL MODEL STUDY

A 1:30 scale model of breakwater, of trapezoidal cross section with a uniform slope of 1V:2H is constructed, at 32m from the generator flap, on the flat bed of the flume with primary armour stone of weight of 73.2gms. A stable trapezoidal submerged reef having a slope of 1V:2H with a crest width (B) of 0.1m and height (h) of 0.25m is constructed, with homogeneous pile of stones of 30gms weight (i.e. nominal diameter, d_{n50} of 0.0221m), on the seaward side of the main breakwater at a distance (X) of 1.0m (i.e. X/d of 2.5 to 3.33). Further, the details of breakwater model construction are explained in Chapter 5 and the model characteristics are listed in Table 5.3.

Before the model tests are started, the experimental set up along with the wave probes is calibrated to find the required wave heights which, are assigned to a particular combination of generator stroke and wave period for depths of water (d) of 0.3m, 0.35m and 0.4m. The model is subjected to regular waves of height varying from 0.1m to 0.16m of a range of period from 1.5sec to 2.5sec generated in water depths of 0.3m to 0.4m. Further, the details of breakwater model test conditions and non-dimensional wave characteristics are shown in Table 5.4.

During the test, the experimental data like incident wave characteristics, wave breaking, wave transmission at reef, wave propagation in the stilling basin (i.e. energy dissipation zone), wave run up and run down over the breakwater slope and armour stone movements are recorded. Further, the details of breakwater test procedure are explained in Chapter 5.

7.3 ANALYSIS AND INTERPRETATION OF DATA

The data collected in the present experimental work is expressed in non-dimensional quantities. The variation of transmission coefficient (K_t), relative run up (R_u/H_o) and run down (R_d/H_o), damage level (S) etc., for varying parameters like steepness $H_o/(gT^2)$ are studied through graphs with respect to changing depth parameter $d/(gT^2)$, relative reef width (B/L_o) etc. The relationship between the parameters is analysed through the graphs.

7.3.1 Influence of various parameters on transmission coefficient

7.3.1.1 Influence of deep water wave steepness

Fig. 7.1 shows the best fit lines for variation of transmission coefficient K_t with the deep water wave steepness parameter (H_o/gT^2) for varying relative reef height (h/d). K_t decreases with an increase in H_o/gT^2 and (h/d).

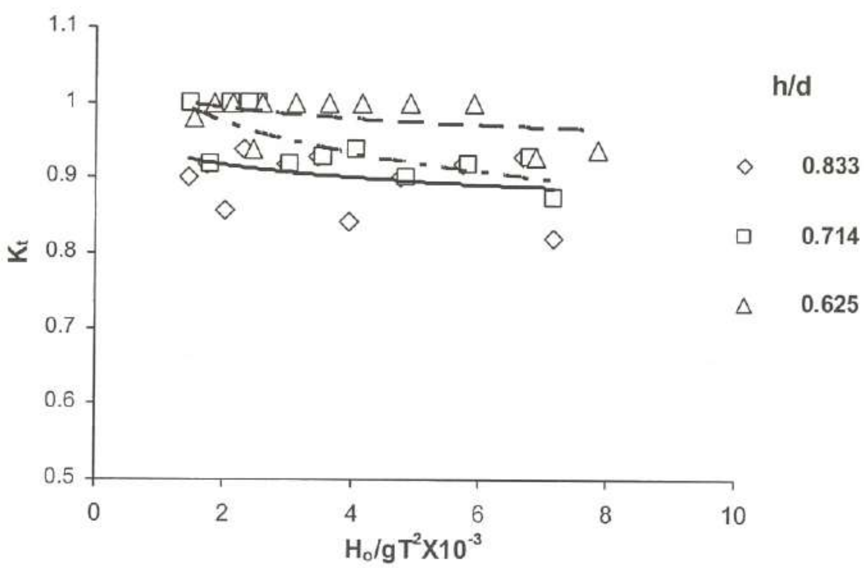

Fig. 7.1. Variation of K_t with H_o/gT^2

This is because submerged reef is efficient in breaking the steeper waves and efficiency in breaking the waves increases with the increase in reef height. The influence of H_o/gT^2 on wave breaking is minimal for h/d of 0.833 and 0.625 (i.e. the depths of 0.3m and 0.4m). K_t drops from 0.95 to 0.82 (13.68%), 0.95 to 0.87 (8.42%) and 1.0 to 0.94 (6.0%) for depths of 0.3m, 0.35m and 0.4m respectively. The average trend shows K_t values higher than 0.8 whereas, actual K_t varies between 0.82 and 1.0. This indicates that the wave height attenuation (WHA = 1 - K_t) is up to 18%.

7.3.1.2 Influence of relative reef submergence

The transmission coefficient K_t increases as relative reef submergence (F/H_i) and depth parameter (d/gT^2) increase. These trends are shown in Fig. 7.2, Fig. 7.3 and Fig. 7.4 for wave periods of 1.5sec, 2sec and 2.5sec respectively. This is because, in deeper waters and increasing reef submergence, waves break less at the submerged reef. For all wave periods, it is observed that K_t values are higher than 0.8. The present study shows gradually increasing K_t with the wave period. This is due to waves becoming less steep with increasing wave periods and breaking lesser and lesser at the submerged reef. It appears that F/H_i influences the K_t values in a depth of 0.3m more than other depths. From the figures it is seen that, for the present configuration of the defenced breakwater, K_t values are up to 20% higher than those given by Cox and Clark (1992) while it is 10% to 67% higher than the K_t values compared to those obtained by Van der Meer and d'Angremond (1992), Cornett et al. (1993) and d'Angremond (1996). For $-0.312 < F/H_i < -1.5$ and all ranges of d/gT^2, K_t increases from 0.826 to 1.0 (21%), 0.84 to 1.0 (19%) and 0.858 to 1.0 (16.55%) for 1.5sec, 2sec and 2.5sec respectively. The overall variation of K_t is from 0.82 to 1.0 (21.95%).

7.3.1.3 Influence of relative crest width

The variation of K_t with crest width (B/L_o) for increasing ranges of wave steepness parameter (H_o/gT^2) i. e. for increasing wave heights of 0.1m, 0.12m 0.14m and 0.16m and periods 1.5sec, 2sec and 2.5sec, is shown by the best fit lines in the Fig. 7.5, Fig. 7.6 and Fig. 7.7 for depths of water (d) of 0.3, 0.35 and 0.4m respectively. Generally it is observed that that for a given depth, K_t decreases with an increase in reef crest width (B/L_o) and in range of wave steepness. The reason is that as the reef crest widens the submerged reef increasingly interferes with the passing wave and offers larger friction and shoaling and wave breaking increases resulting in smaller wave transmission and as steepness increases, reef increasingly

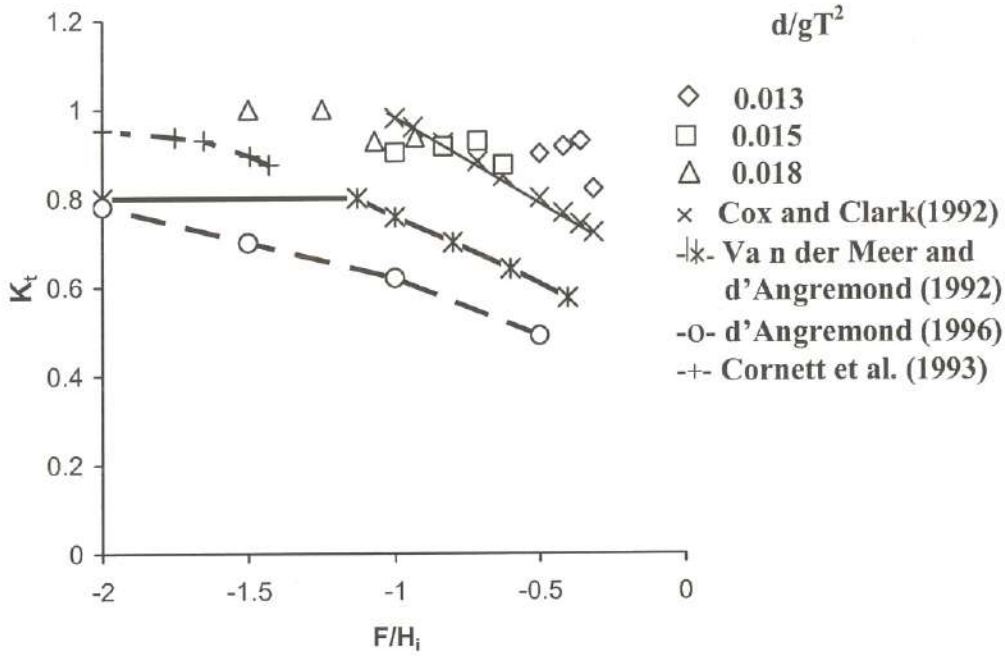

Fig. 7.2. Variation of K_t with F/H_i for T = 1.5 sec

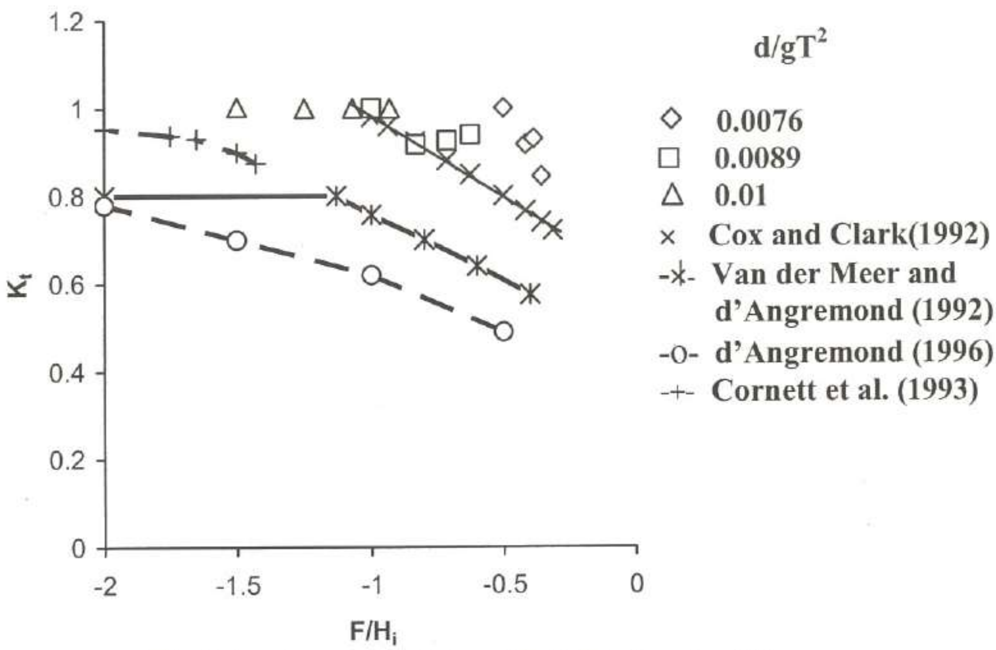

Fig. 7.3. Variation of K_t with F/H_i for T = 2.0 sec

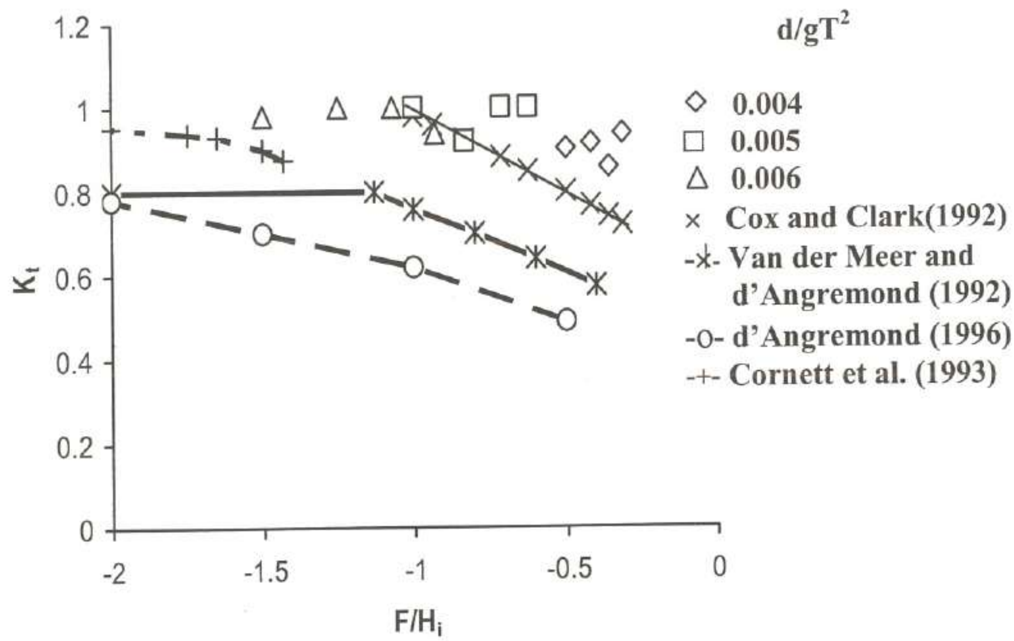

Fig. 7.4. Variation of K_t with F/H_i for T = 2.5 sec

breaks the waves reducing K_t. But for each depth, K_t shows a different trend for the present geometry of the protected breakwater. This may be due to smaller spacing X of 1m (i.e. X/d of 2.5 to 3.33) between the structures which is the zone characterized by high degree of mixing and turbulence, the water there has high content of air bubbles entrapped during the wave breaking process and the geometry of the protected breakwater may also be responsible for such a scenario.

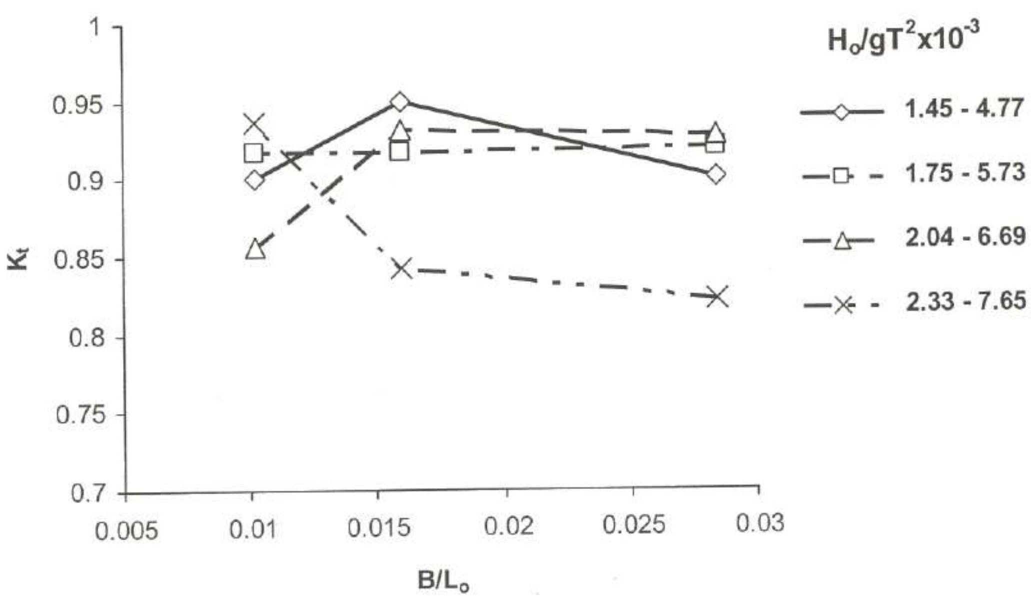

Fig. 7.5. Variation of K_t with B/L_o for d = 0.3 m

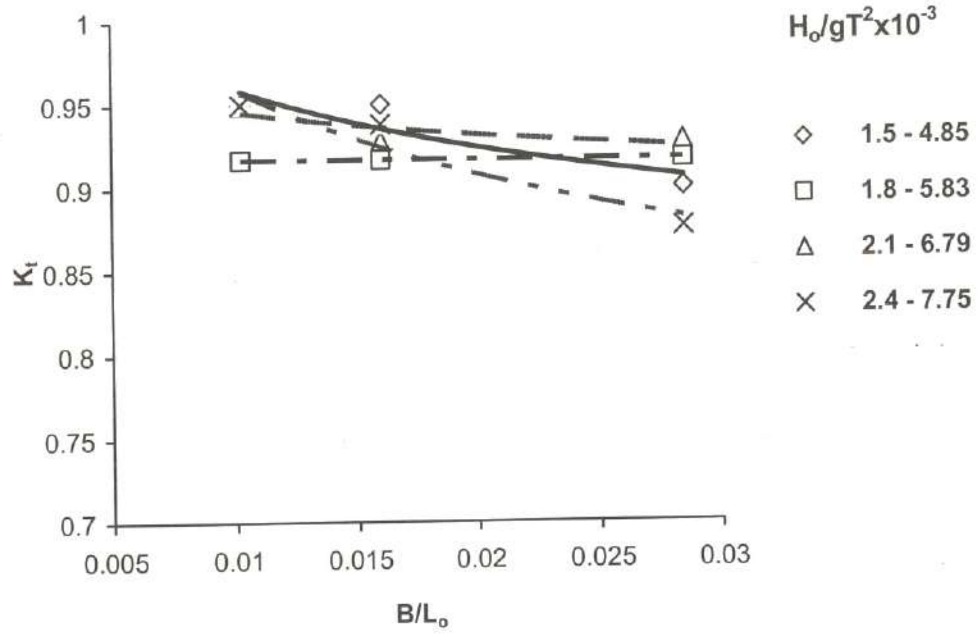

Fig. 7.6. Variation of K_t with B/L_o for $d = 0.35$ m

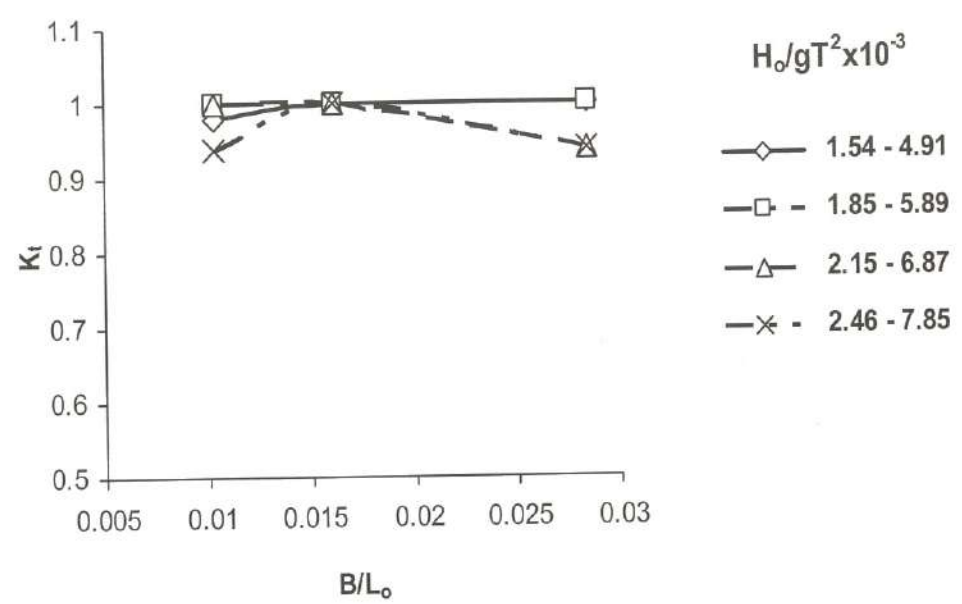

Fig. 7.7. Variation of K_t with B/L_o for $d = 0.4$m

As the depth of water increases, K_t values rise for all ranges of steepness parameter and difference between K_t for different steepness parameters reduces. This shows that as d/gT^2 (i.e. depth of water) increases the influence B/L_o and H_o/gT^2 on K_t gradually reduces.

7.3.2 Influence of deep water wave steepness on wave run up and run down

The influence of deep water wave steepness parameter (H_o/gT^2) on relative run up (R_u/H_o) and run down (R_d/H_o), for increasing ranges of depth parameter (d/gT^2) i.e. for varying wave climate in depths of water of 0.3m, 0.35m and 0.4m, is shown by best fit lines in Fig. 7.8 and Fig. 7.9 respectively. For $1.45 \times 10^{-3} < H_o/gT^2 < 7.85 \times 10^{-3}$, maximum R_u/H_o is 1.56 and the maximum R_d/H_o is 0.9. As H_o/gT^2 varies over its entire range, in a depth of water of 0.3m (i.e. $0.004 < d/gT^2 < 0.013$ and $-0.312 < F/H_i < -0.5$), R_u/H_o and R_d/H_o are 5% to15% lower and 17% to 28% lower respectively compared to that for a conventional (single) breakwater, while, for other water depths; these are not much different from that of the conventional breakwater.

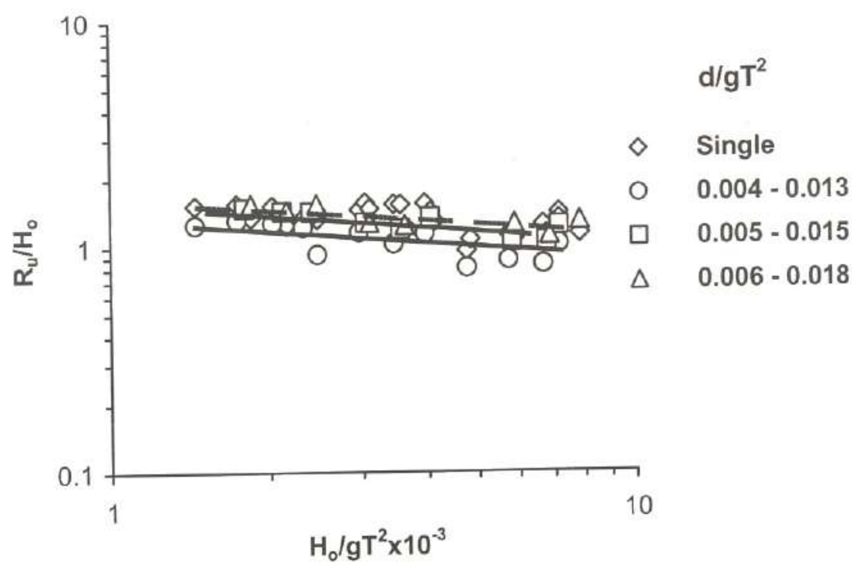

Fig. 7.8. Variation of R_u/H_o with H_o/gT^2

7.3.3 Influence of various parameters on damage level

7.3.3.1 Influence of deep water wave steepness

The trends of damage level (S) with varying wave steepness parameter (H_o/gT^2) for increasing ranges of depth parameter (d/gT^2) i.e. increasing depths of water of 0.3m, 0.35m and 0.4m and different wave periods of 1.5sec, 2sec and 2.5sec are shown in Fig. 7.10. The present damage levels are compared with those of the conventional breakwater. The damage increases with an increase in steepness for a particular range of d/gT^2. This is due to increasing energy

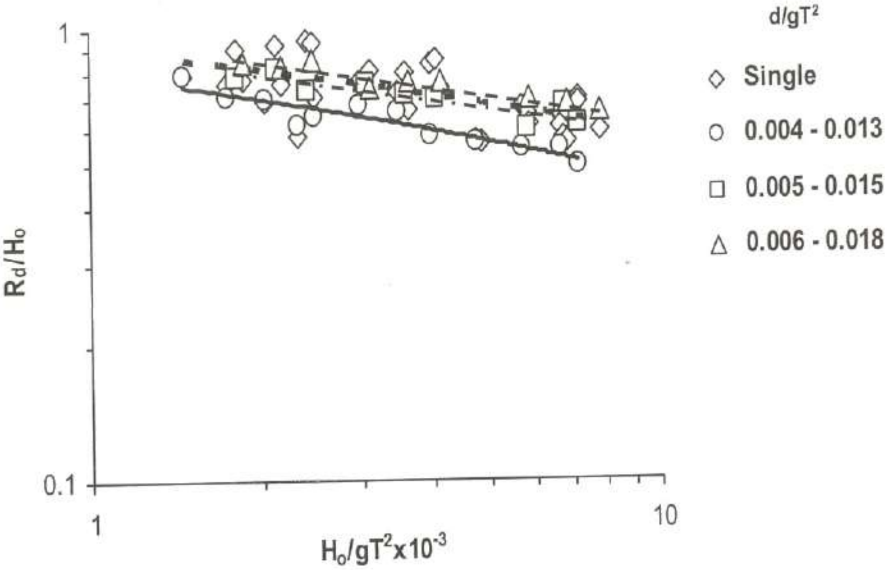

Fig. 7.9. Variation of R_d/H_o with H_o/gT^2

of steeper waves. The damage increases with an increase in depth of water and increases with a decreasing wave period. This is because higher water depths sustain larger waves and short period waves disturb the displaced stones within a smaller time interval without allowing them to settle.

The impact of wave period is clearly distinguishable. The damages in all depths of water due to shorter period waves of 1.5secs (i. e. higher values of H_o/gT^2) are seen on right hand side of the figure whereas, damage of longer period waves of 2.5sec (i. e. smaller values of H_o/gT^2) in all depths of water are seen on the left hand side and damages for waves of period of 2sec are in the middle of the figure. Considering all the ranges of d/gT^2 (i.e as the depth of water increases from 0.3m to 0.4m), for $4.77 \times 10^{-3} < H_o/gT^2 < 7.85 \times 10^{-3}$ i.e. wave periods of 1.5sec, the maximum damage level increases from 6.78 to 13.89 (i.e. by 104.8%), for $2.5 \times 10^{-3} < H_o/gT^2 < 4 \times 10^{-3}$ i.e. for 2sec, they rise from 5.9 to 12.6 (i.e. by 113.5%) and for $1.5 \times 10^{-3} < H_o/gT^2 < 2.46 \times 10^{-3}$ i.e. for 2.5sec, the maximum damage level increases from 5.0 to 8.67 (i.e. by 73.4%) respectively.

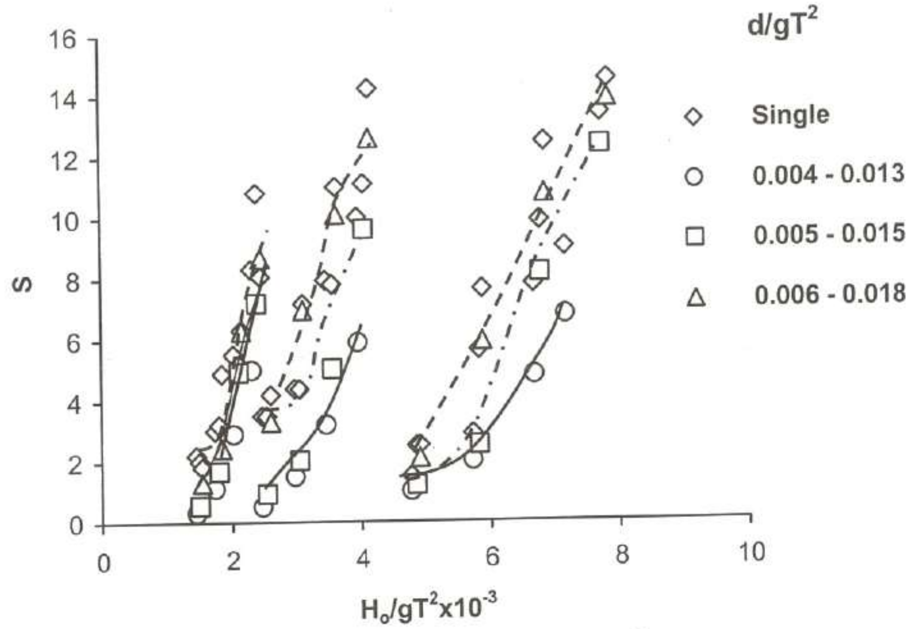

Fig. 7.10. Variation of S with H_o/gT^2

7.3.3.2 Influence of reef submergence

The graphs in Fig. 7.11 show an increasing damage level (S) with increasing the reef submergence (F/H_i) and depth parameter (d/gT^2). This is because increasing reef submergence indicated larger depths which sustain higher waves without much breaking. Also we can see that impact of wave period is clearly discernible only in higher water depths of 0.4m (on the extreme left side of the figure) where, it shows increasing damage with shorter period waves.

The impact of depth of water is clearly visible in the figure as graphs for damages in depths of 0.3m and 0.4m, are appearing on right and left sides respectively and the damages in depth 0.35m are located in the middle.

For $0.004 < d/gT^2 < 0.0013$ and $-0.312 < F/H_i < -0.5$ (i.e. for d = 0.3m and h/d = 0.833), the maximum damage levels increase from 5.0 to 6.78 (35.6%), for $0.005 < d/gT^2 < 0.0015$ and $-0.625 < F/H_i < -1.0$ (i.e. for d = 0.35m and h/d = 0.714), the maximum damage levels increase from 7.18 to 12.35 (72%) and for $0.006 < d/gT^2 < 0.0018$ and $-0.94 < F/H_i < -1.5$ (i.e. for d= 0.4m and h/d = 0.625), the maximum damage levels increase from 8.67 to 13.89 (60.21%). As the depth of water increases from 0.3m to 0.35m (i.e. 16.67%) or as the range of F/H_i increases from -0.312 to -0.5 to the range of -0.625 to -1.0, the maximum damage levels of the breakwater increased from 6.78 to 12.35 (i.e. by 82.1%) and at a depth of 0.4m (i.e. an increase of 33.3% w.r.t 0.3m) or as the range of F/H_i increases to -0.94 to -1.5, damage level (S) increases to 13.89 (i.e. by 104.8%).

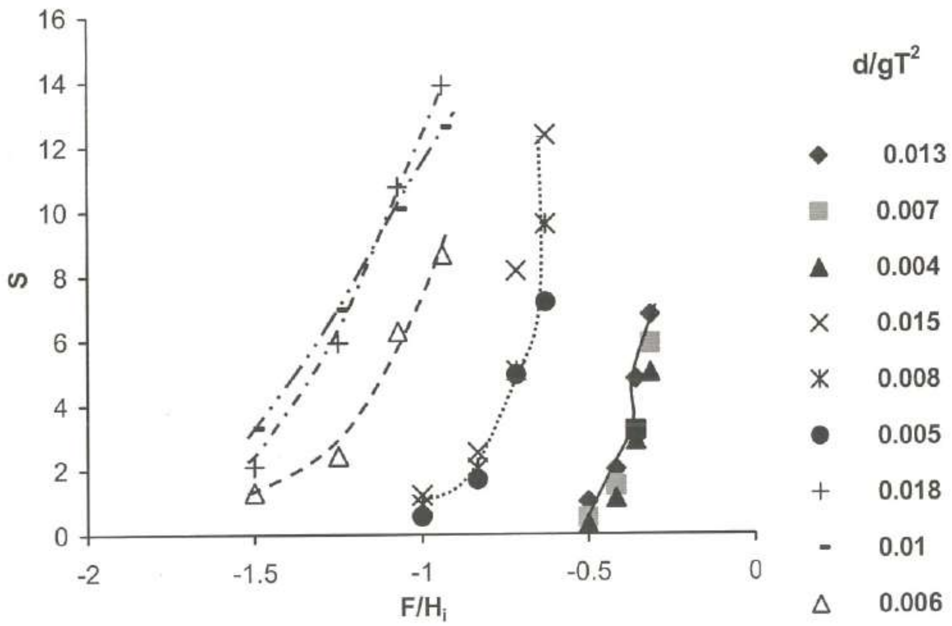

Fig. 7.11. Variation of S with F/H_i

The impact of depth of water is clearly visible in the figure as graphs for damages in depths of 0.3m and 0.4m, are appearing on right and left sides respectively and the damages in depth 0.35m are located in the middle.

For $0.004 < d/gT^2 < 0.0013$ and $-0.312 < F/H_i < -0.5$ (i.e. for d = 0.3m and h/d = 0.833), the maximum damage levels increase from 5.0 to 6.78 (35.6%), for $0.005 < d/gT^2 < 0.0015$ and $-0.625 < F/H_i < -1.0$ (i.e. for d = 0.35m and h/d = 0.714), the maximum damage levels increase from 7.18 to 12.35 (72%) and for $0.006 < d/gT^2 < 0.0018$ and $-0.94 < F/H_i < -1.5$ (i.e. for d= 0.4m and h/d = 0.625), the maximum damage levels increase from 8.67 to 13.89 (60.21%). As the depth of water increases from 0.3m to 0.35m (i.e. 16.67%) or as the range of F/H_i increases from -0.312 to -0.5 to the range of -0.625 to -1.0, the maximum damage levels of the breakwater increased from 6.78 to 12.35 (i.e. by 82.1%) and at a depth of 0.4m (i.e. an increase of 33.3% w.r.t 0.3m) or as the range of F/H_i increases to -0.94 to -1.5, damage level (S) increases to 13.89 (i.e. by 104.8%).

7.3.3.3 Influence of reef crest width

Fig. 7.12, Fig. 7.13 and Fig. 7.14 demonstrate the impact of reef crest width (B/L_o) on breakwater damage, for increasing ranges of H_o/gT^2 i.e for increasing wave heights of 0.1m to 0.16m of different periods of 1.5sec, 2sec and 2.5sec in water depths (d) of 0.3m, 0.35m and 0.4m respectively. For all depths of water, the general trend is that, increasing damage with reef crest widths (B/L_o) for any given range of wave steepness. This is because for constant reef crest width (B) of 0.1m, the increase in B/L_o indicates decrease in L_o i.e. decreasing wave periods and as already observed shorter period waves are relatively more damaging compared to longer period waves. Graphs indicate that, damage increases with increase in steepness. It is also seen that, the waves are more damaging with the increase in depth of water or decrease of h/d. As $0.01 < B/L_o < 0.0285$ and considering all ranges of H_o/gT^2, S increases from 0.32 to 6.78, 0.55 to 12.35 and 1.31 to 13.89 for depths of 0.3m, 0.35m and 0.4m respectively.

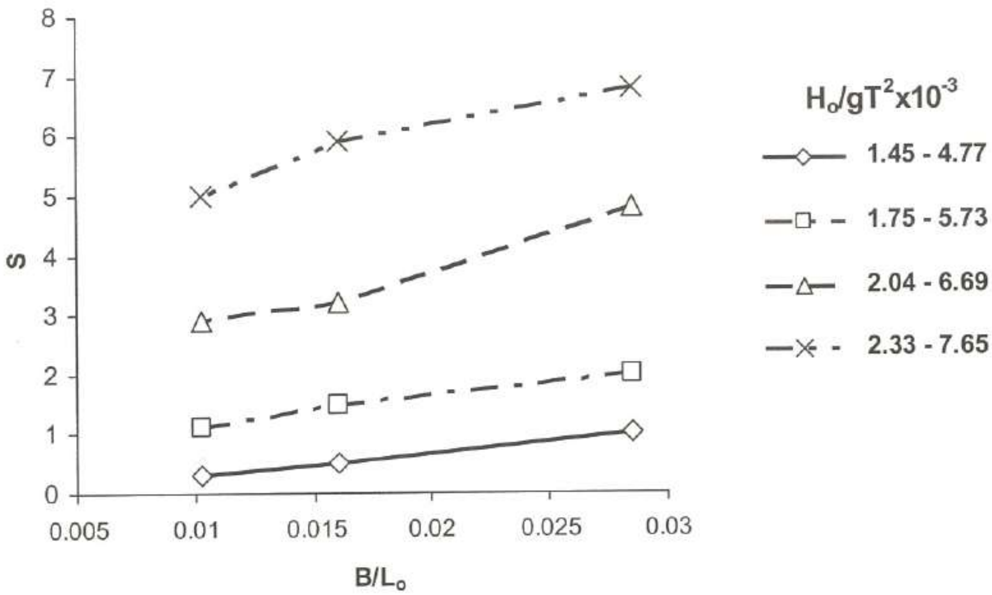

Fig. 7.12. Variation of S with B/L_o for d = 0.3m

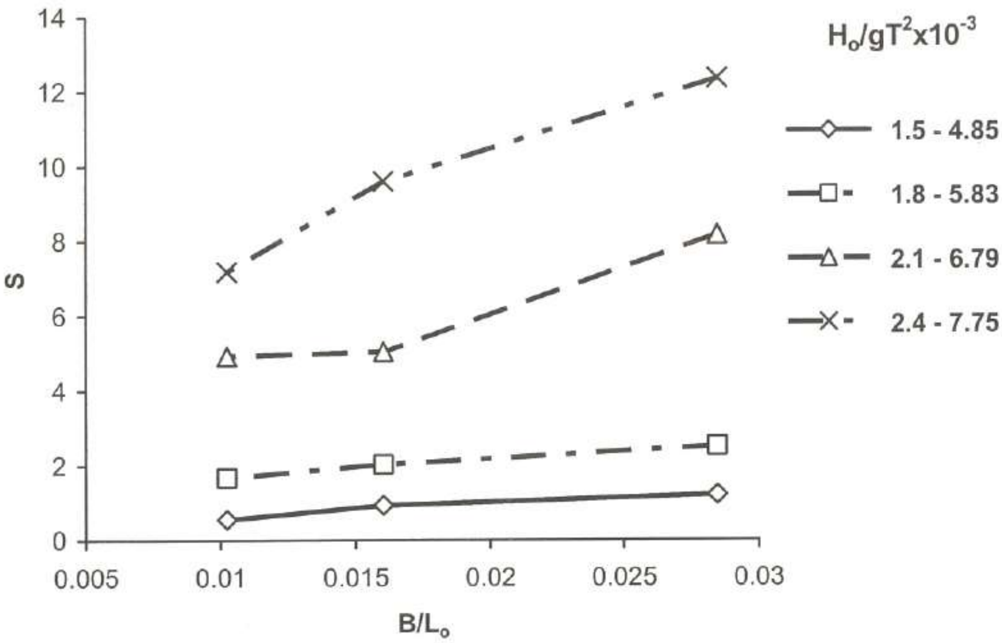

Fig. 7.13. Variation of S with B/L_o for d = 0.35m

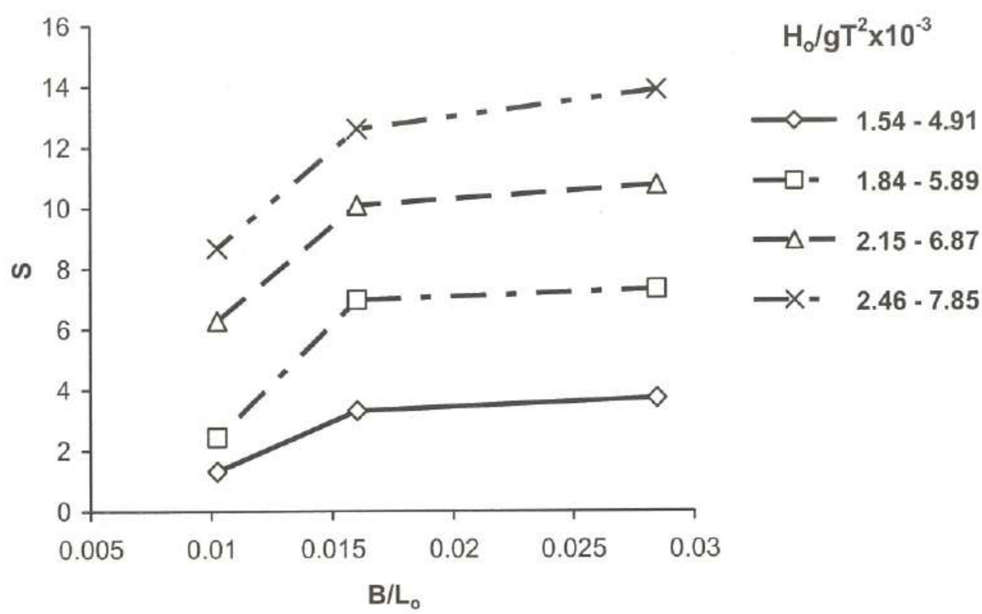

Fig. 7.14. Variation of S with B/L_o for d = 0.4m

7.3.3.4 Influence of stability number

The damage level (S) of the breakwater increases with an increase in stability number (N_s) for all the depths of water (d) of 0.3m, 0.35m and 0.4m as shown in Fig. 7.15, Fig. 7.16 and Fig. 7.17 respectively. Best fit lines show that,

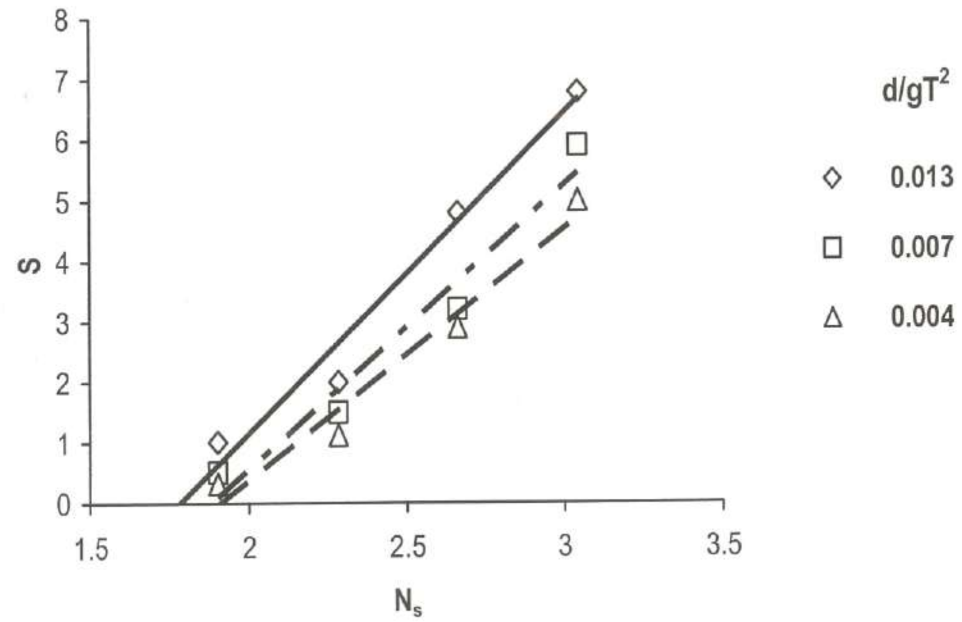

Fig. 7.15. Variation of S with N_s for d = 0.3m

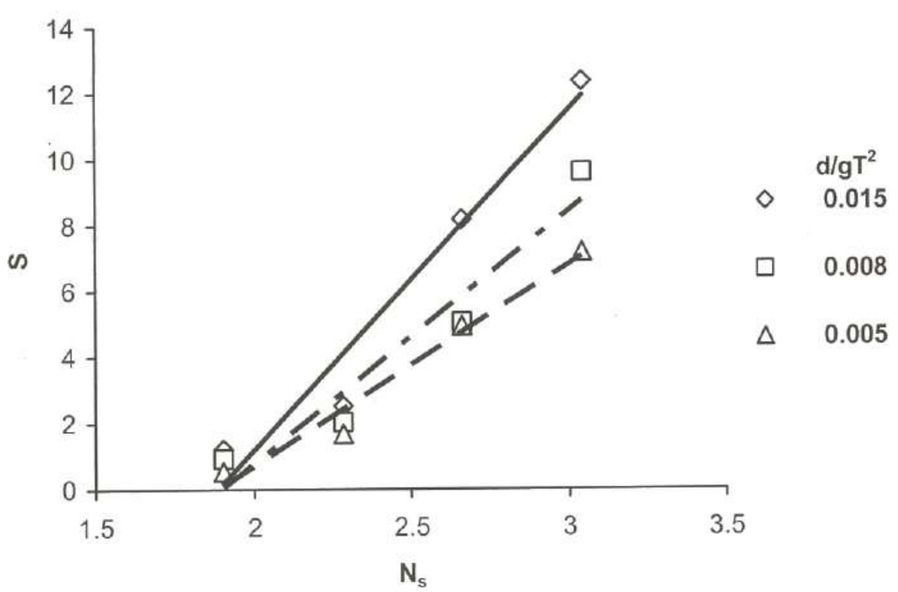

Fig. 7.16. Variation of S with N_s for d = 0.35m

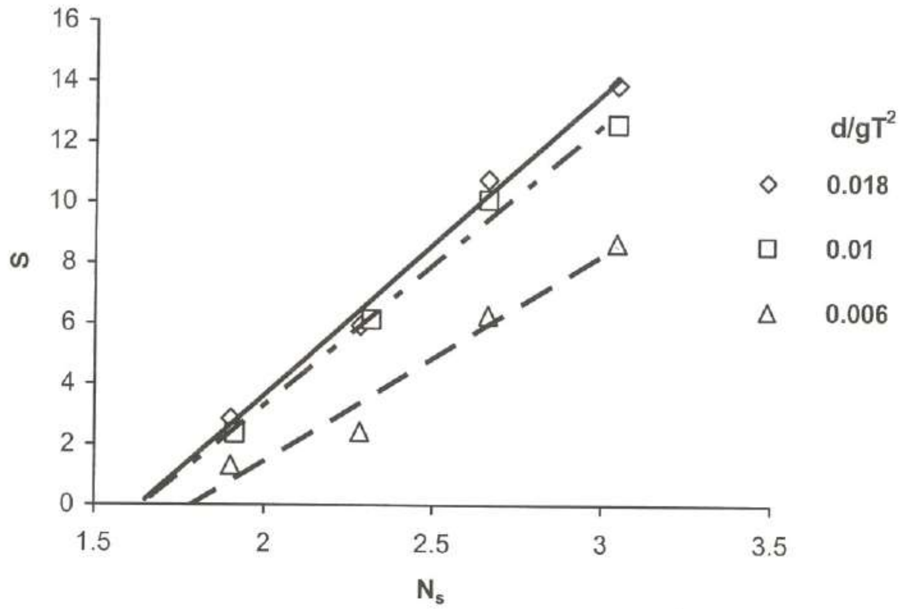

Fig. 7.17. Variation of S with N_s for d = 0.4m

The maximum damage levels increase from 5.0 to 6.78 (35.6%), 7.18 to 12.35 (72%) and 8.67 to 13.89 (60.21%) for the depths of 0.3m, 0.35m and 0.4m (i.e. h/d of 0.833, 0.714 and 0.625) respectively.

The zero damage wave heights (H_{zd}) decrease with an increase in depth of water (d) indicating higher damage with increased depth. The zero damage wave heights (H_{zd}), for a depth of 0.3m and for wave periods of 1.5 sec, 2.0sec and 2.5sec (i.e. d/gT^2 = 0.013, 0.007 and 0.004) are 0.113m, 0.1196m and 0.1262m respectively. These wave heights are 26.6% to 56% higher compared to conventional breakwater. The H_{zd} for a depth of 0.35m and for wave periods of 1.5 sec, 2.0sec and 2.5sec are (i.e. d/gT^2 = 0.015, 0.008 and 0.005) 0.1126m, 0.1187m and 0.1259m respectively which are 16.5% to 43% higher compared to conventional breakwater. H_{zd} for a depth of 0.40m and for wave periods of 1.5 sec, 2.0sec and 2.5sec (i.e. d/gT^2 = 0.018, 0.01 and 0.006) are 0.0847m, 0.0899m and 0.1133m respectively which are 2.3% to 29.5% higher compared to conventional breakwater. The damage levels of defenced breakwater for depths of water of 0.3m, 0.35m and 0.4m are respectively 24.6% to 41%, 7.8% to 33.5% and 4% to11% lower than the corresponding damages of conventional breakwater.

7.3.3.5 Variation of surf similarity parameter on stability number

Fig. 7.18 shows the variation of zero damage stability number (N_{zd}) and surf similarity parameter (ξ) for increasing ranges of depth parameter (d/gT^2) i.e. increasing depths of water

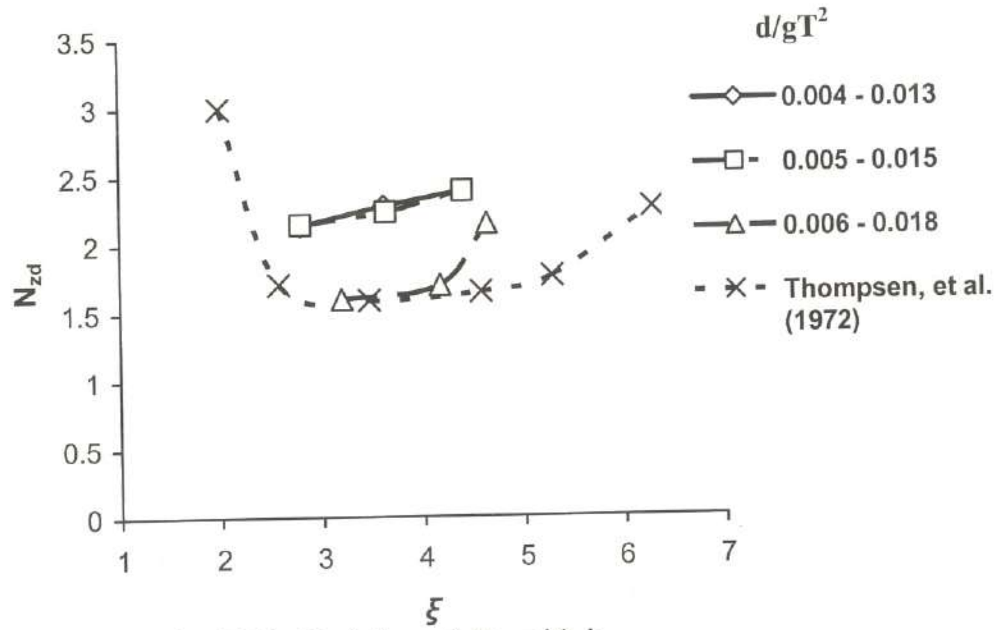

Fig. 7.18. Variation of N_{zd} with ξ

of 0.3m, 0.35m and 0.4m and different wave periods of 1.5sec, 2sec and 2.5sec. The values of N_{zd} for depths of water of 0.3m and 0.35m almost coincide which indicates that these depths have identical impact on the stability. The results are compared with those given by Thompsen et al. (1972). In the figure, it is observed that, minimum stability of the protected breakwater in a depth of 0.4m i. e. S = 1.6 occurs for $\xi = 3.2$.

7.4 CONCLUSIONS

From the present model study, the following conclusions are drawn.

1. The transmission coefficient (K_t) decreases with increase in H_o/gT^2 and h/d and decreases with decrease in F/H_i and d/gT^2. But the trend of variation of K_t with B/L_o is unclear.
2. The influence of H_o/gT^2 on wave breaking is minimal for h/d of 0.833 and 0.625 (i.e. the depths of 0.3m and 0.4m).
3. K_t drops from 0.95 to 0.82 (13.68%), 0.95 to 0.87 (8.42%) and 1.0 to 0.94 (6.0%) for h/d of 0.833, 0.714 and 0.625 i.e. depths of 0.3m, 0.35m and 0.4m respectively

4. K_t values are up to 20% higher than those given by Cox and Clark (1992) and 10% to 67% higher compared to those obtained by Van der Meer and d'Angremond (1992), Cornett et al. (1993) and d'Angremond (1996).
5. For $-0.312 < F/H_i < -1.5$ and all ranges of d/gT^2, K_t increases from 0.826 to 1.0 (21%), 0.84 to 1.0 (19%) and 0.858 to1.0 (16.55%) for 1.5sec, 2sec and 2.5sec respectively.
6. The maximum run up and rundown are respectively 1.56 times and 0.90 times the deep water wave height.
7. In a depth of water of 0.3m (i.e. $0.004 < d/gT^2 < 0.013$ and $-0.312 < F/H_i < -0.5$), R_u/H_o and R_d/H_o are 5% to15% lower and 17% to 28% lower respectively compared to that for a conventional (single) breakwater, while, for other water depths; these are not much different from that of the conventional breakwater.
8. Shorter period waves are relatively more damaging compared to longer period waves.
9. The damage level S increases with the increase in H_o/gT^2, F/H_i, d/gT^2, B/L_o and decrease in h/d.
10. As the depth of water increases from 0.3m to 0.35m (i.e. 16.67%) or as the range of F/H_i increases from -0.312 to -0.5 to the range of -0.625 to -1.0, the maximum damage levels of the breakwater increase from 6.78 to 12.35 (i.e. by 82.1%) and at a depth of 0.4m (i.e. an increase of 33.3% w.r.t 0.3m) or as the range of F/H_i increases to -0.94 to -1.5, damage level (S) increase to 13.89 (i.e. by 104.8%).
11. For $0.004 < d/gT^2 < 0.013$ and $-0.312 < F/H_i < -0.5$ (i.e. for d = 0.3m and h/d = 0.833), the maximum damage levels increase from 5 to 6.78 (35.6%), for $0.005 < d/gT^2 < 0.015$ and $-0.625 < F/H_i < -1.0$ (i.e. for d = 0.35m and h/d = 0.714), the maximum damage levels increase from 7.18 to 12.35 (72%) and for $0.006 < d/gT^2 < 0.018$ and $-0.94 < F/H_i < -1.5$ (i.e. for d= 0.4m and h/d = 0.625), the maximum damage levels increase from 8.67 to 13.89 (60.21%).
12. As $0.01 < B/L_o < 0.0285$, considering all ranges of H_o/gT^2 , S increases from 0.32 to 6.78, 0.55 to 12.35 and 1.31 to 13.89 for depths of 0.3m, 0.35m and 0.4m (h/d of 0.833, 0.714 and 0.625) respectively.
13. Considering the complete variation H_o/gT^2 and all the ranges of d/gT^2 (i.e as the depth of water increased from 0.3m to 0.4.), the maximum damage level increases from 6.78 to 13.89 (i.e. by 104.8%), 5.9 to 12.6 (i.e. by 113.5%) and 5.0 to 8.67 (i.e. by 73.4%) for wave periods of 1.5sec, 2sec and 2.5sec respectively.

14. Maximum damages to defenced breakwater are reduced by about 4% to 41% compared to conventional breakwater.
15. Zero damage wave heights are 2.3% to 56% higher than the conventional breakwater.
16. Minimum stability of the defenced breakwater i. e. $S = 1.6$ occurs for $\xi = 3.2$.

Chapter 8
Investigation of Protected Breakwater with a Reef at a Spacing of 2.5m

8.1 GENERAL

In the model of protected breakwater, with seaward submerged reef located at 1m from the breakwater, the reef does not dissipate enough wave energy so as to protect the main breakwater completely. Hence, it is decided to experiment with increased spacing (X) between the structures to find the proper configuration of the defenced breakwater.

A stable 1V:2H sloped trapezoidal submerged reef of height (h) of 0.25m with a varying crest width of 0.1m to 0.4m is constructed on the seaward side of the main breakwater at a distance (X) of 2.5m (i.e. X/d of 6.25 to 8.33). This model section of the protected breakwater is tested for regular waves and influence of waves (i.e. wave steepness) on wave transmission at reef as well as on wave run up, run down and stability of armour of the inner (main) breakwater is studied.

8.2 DETAILS OF PHYSICAL MODEL STUDY

A 1:30 scale model of a breakwater, of trapezoidal cross section with a uniform slope of 1V:2H is constructed, at 32m from the generator flap, on the flat bed of the flume with primary stone armour of weight of 73.2gms. A stable trapezoidal submerged reef having a slope of 1V:2H with a height (h) of 0.25m and crest width (B) of 0.1m, 0.2m, 0.3m and 0.4m is constructed, with homogeneous pile of stones of 30gms weight (i.e. nominal diameter, d_{n50} of 0.0221m), on the seaward side of the main breakwater at a distance (X) of 2.5m (i.e. X/d of 6.25 to 8.33). Further, the details of breakwater model construction are explained in Chapter 5 and the model characteristics are listed in Table 5.3.

Before the model tests are started, the experimental set up along with the wave probes is calibrated to find the required wave heights which, are assigned to a particular combination of generator stroke and wave period, for depths of water (d) of 0.3m, 0.35m and 0.4m. The model is subjected to regular waves of height varying from 0.1m to 0.16m of a range of periods of 1.5sec to 2.5sec generated in water depths of 0.3m to 0.4m. Further, the details of breakwater model test conditions and non-dimensional wave characteristics are shown in

Table 5.4. During the test, the experimental data like the incident wave characteristics, wave breaking, wave transmission at reef, wave propagation in the stilling basin (i.e. energy dissipation zone), wave run up and run down over the breakwater slope and armour stone movements are recorded. Further, the details of breakwater test procedure are explained in Chapter 5.

8.3 ANALYSIS AND INTERPRETATION OF DATA

The data collected in the present experimental work is expressed in non-dimensional quantities. The variation of transmission coefficient (K_t), relative run up (R_u/H_o) and run down (R_d/H_o), damage level (S) etc., for varying parameters like steepness $H_o/(gT^2)$ are studied through graphs with respect to changing depth parameter $d/(gT^2)$, reef width (B/L_o) etc. Their relationship is analysed through the graphs.

8.3.1 Protected breakwater with a reef of crest width (B) of 0.1m (i. e. B/d = 0.25 to 0.33)

8.3.1.1 Influence of various parameters on transmission coefficient

8.3.1.1.1 Influence of deep water wave steepness

Fig. 8.1 shows the best fit lines for the variation of transmission coefficient K_t with the deep water wave steepness parameter (H_o/gT^2) for different relative reef height (h/d). K_t decreases with an increase in H_o/gT^2 and increase in relative reef height (h/d). This is because submerged reef is efficient in breaking the steeper waves and efficiency of wave breaking increases with the reef height.

The influence of H_o/gT^2 on wave breaking is almost the same for h/d of 0.833 and 0.714 (i.e. for depths of water of 0.3m and 0.35m), and K_t drops from 0.76 to 0.66 (13.1%) and 0.79 to 0.66 (16.4%) respectively for these h/d values. Whereas, for h/d of 0.625 (i.e. depth of water of 0.4m), K_t drops from 0.84 to 0.68 (19%). The graphs shows that K_t values are higher than 0.66 whereas, actual K_t varies between 0.66 and 0.84. The wave height attenuation increases from 0 to 18% after breaking at the reef to 16% to 34% at the breakwater toe.

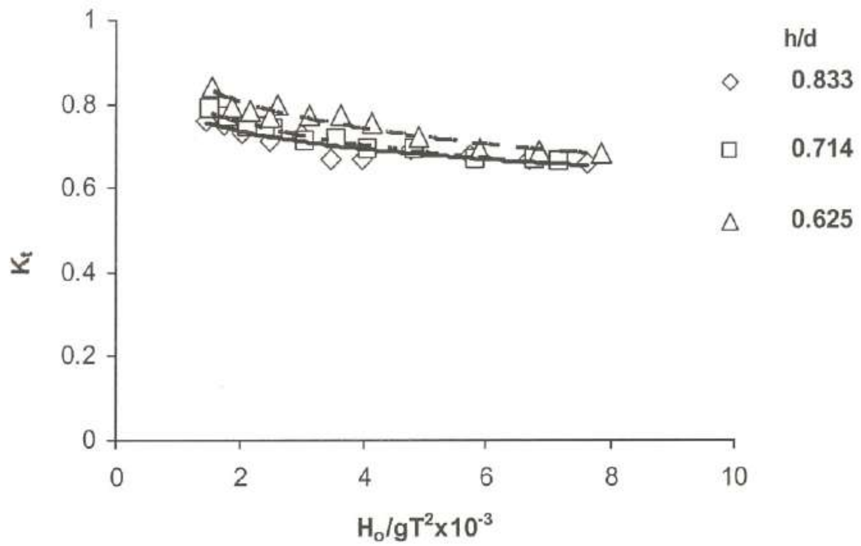

Fig. 8.1. Variation of K_t with H_0/gT^2

8.3.1.1.2 Influence of relative reef submergence

The transmission coefficient K_t increases as relative reef submergence (F/H_i) and range of depth parameter (d/gT^2) increase for wave periods of 1.5sec, 2sec and 2.5sec as shown in Fig. 8.2, Fig. 8.3 and Fig. 8.4 respectively. For the above periods and $-0.312 < F/H_i < -1.5$, K_t increases from 0.64 to 0.7 (9.4%), 0.696 to 0.8 (14.9%) and 0.71 to 0.84 (18.3%) respectively. Figures also show that the wave transmission gradually increases with wave period.This is due to waves becoming less steep with increasing wave periods and breaking lesser and lesser at the submerged reef. Considering the complete ranges of d/gT^2 and variation of F/H_i for all the wave periods, the present K_t values are 5% to 28% lower than Cox and Clark (1992), 9% to 23% lower than those given by Cornett et al. (1993), 30% to 55% higher than d'Angremond (1996) and distributed around the values given by Van der Meer and d'Angremond (1992) with an accuracy of 24%.

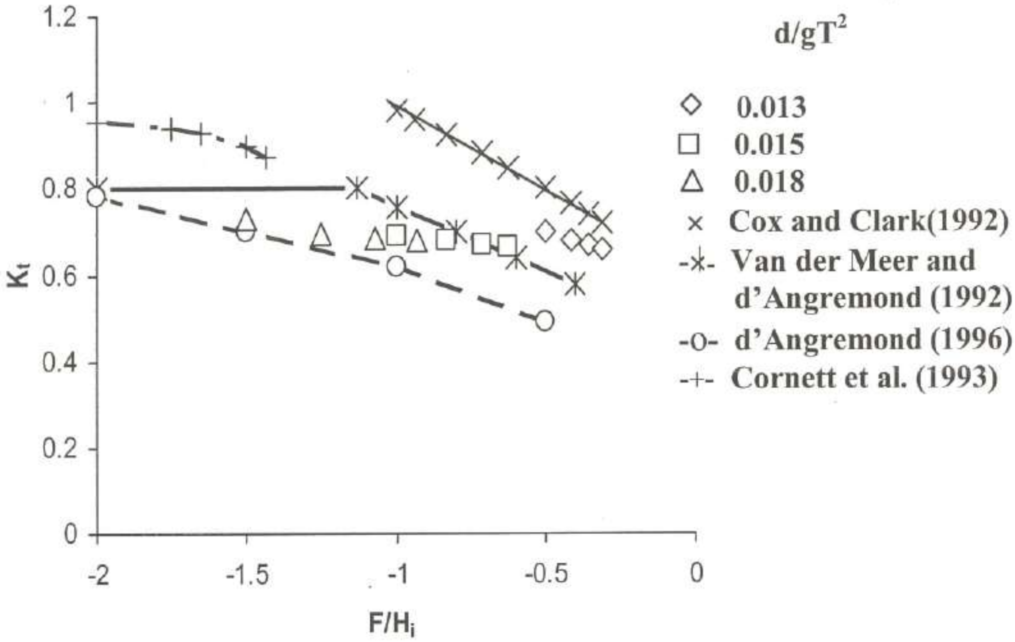

Fig. 8.2. Variation of K_t with F/H_i for T =1.5 sec

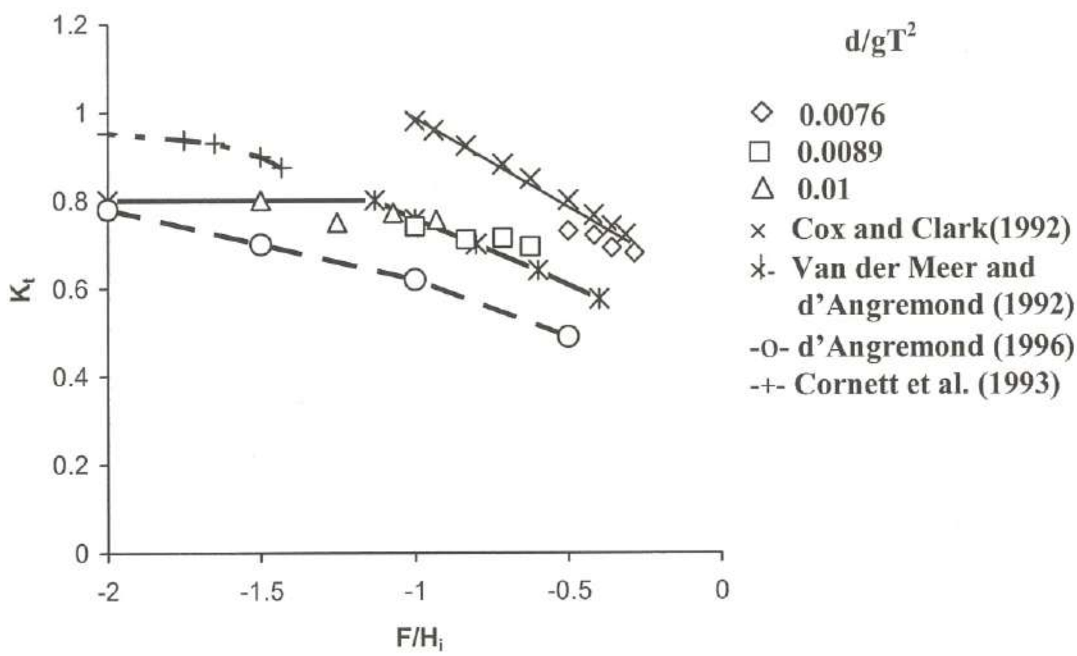

Fig. 8.3. Variation of K_t with F/H_i for T =2 sec

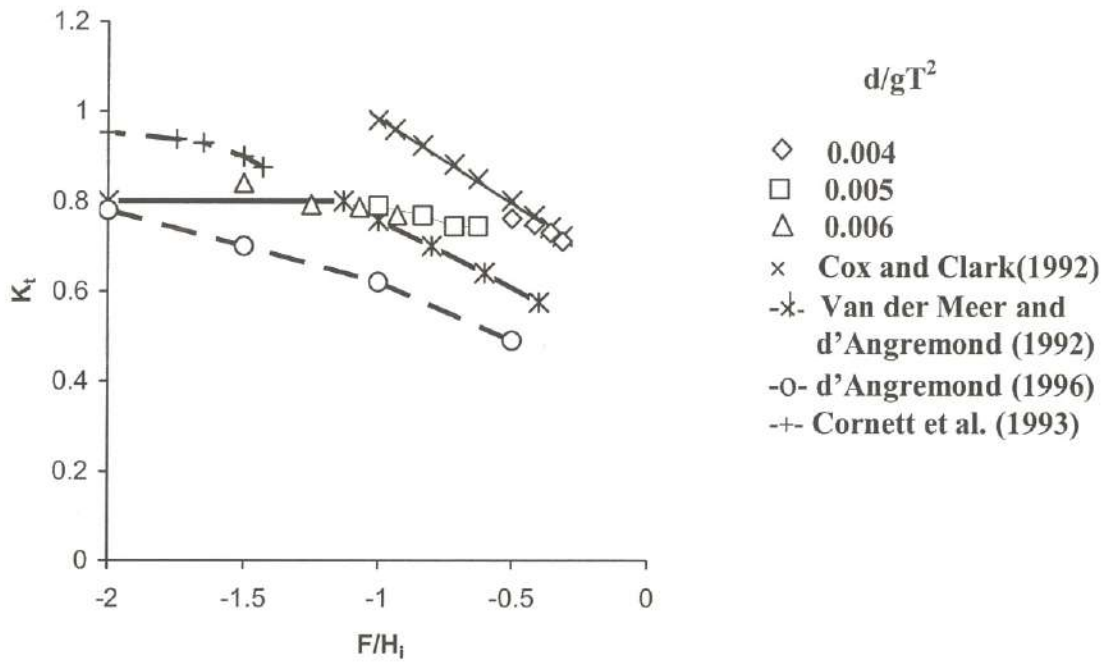

Fig. 8.4. Variation of K_t with F/H_i for T =2.5 sec

8.3.1.1.3 Influence of relative reef crest width

The variations of K_t with crest width (B/L_o) for increasing ranges of wave steepness parameter (H_o/gT^2), i.e. increasing wave heights of 0.1m, 0.12m 0.14m and 0.16m of periods 1.5sec, 2sec and 2.5sec, for depths of water (d) of 0.3, 0.35 and 0.4m are shown in Fig. 8.5, Fig. 8.6 and Fig. 8.7 respectively.

It is observed that for a given depth, K_t decreases with an increase in reef crest width (B/L_o) and with an increase in range of wave steepness. K_t drops from 0.76 to 0.66 (13.1%), 0.79 to 0.66 (16.4%) and 0.84 to 0.68 (19%) for for h/d of 0.833, 0.714 and 0.625 i.e. depths of 0.3m, 0.35m and 0.4m respectively. The trends of K_t are similar for depths of 0.3m and 0.35m whereas for a depth of 0.4m, it differs. As the depth of water increases, K_t values rise for all ranges of wave steepness for $0.01 < B/L_o < 0.0285$ and difference between the trends of K_t reduces for $1.75 \times 10^{-3} < H_o/gT^2 < 5.89 \times 10^{-3}$ and $2.04 \times 10^{-3} < H_o/gT^2 < 6.87 \times 10^{-3}$ i.e. for wave of height 0.12m and 0.14m.

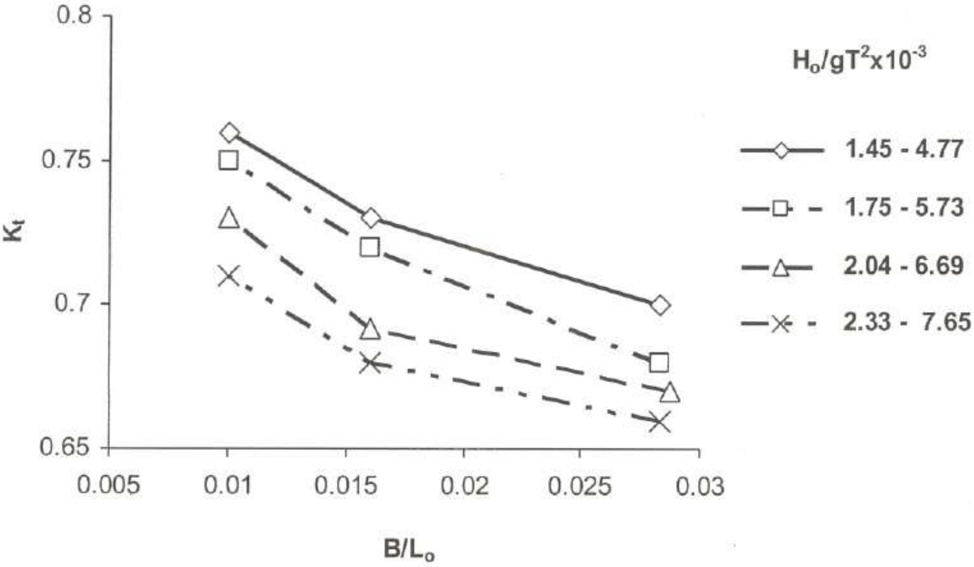

Fig. 8.5. Variation of K_t with B/L_o for d = 0.3m

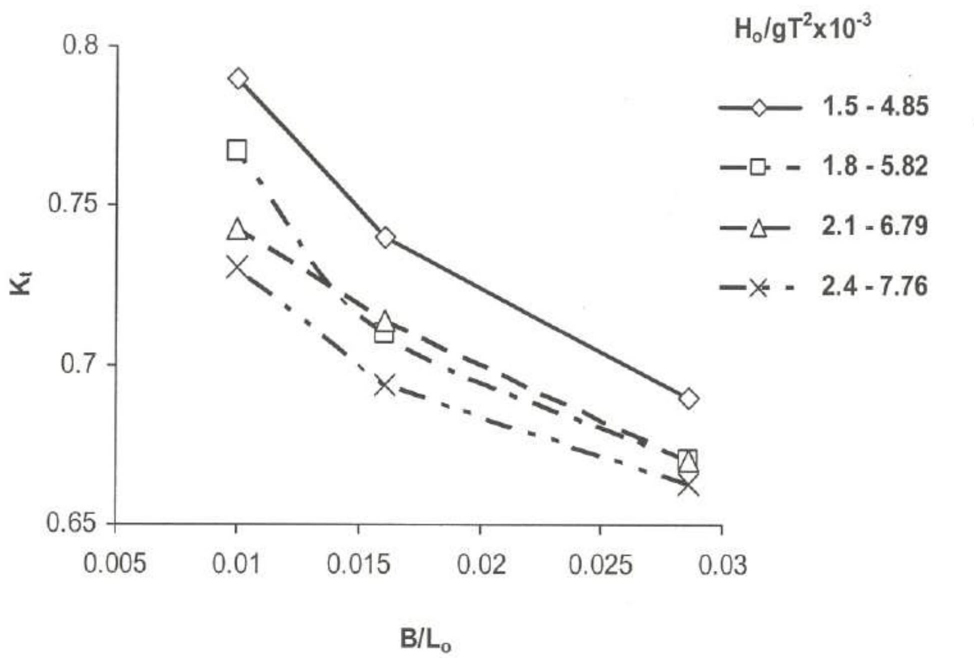

Fig. 8.6. Variation of K_t with B/L_o for d = 0.35m

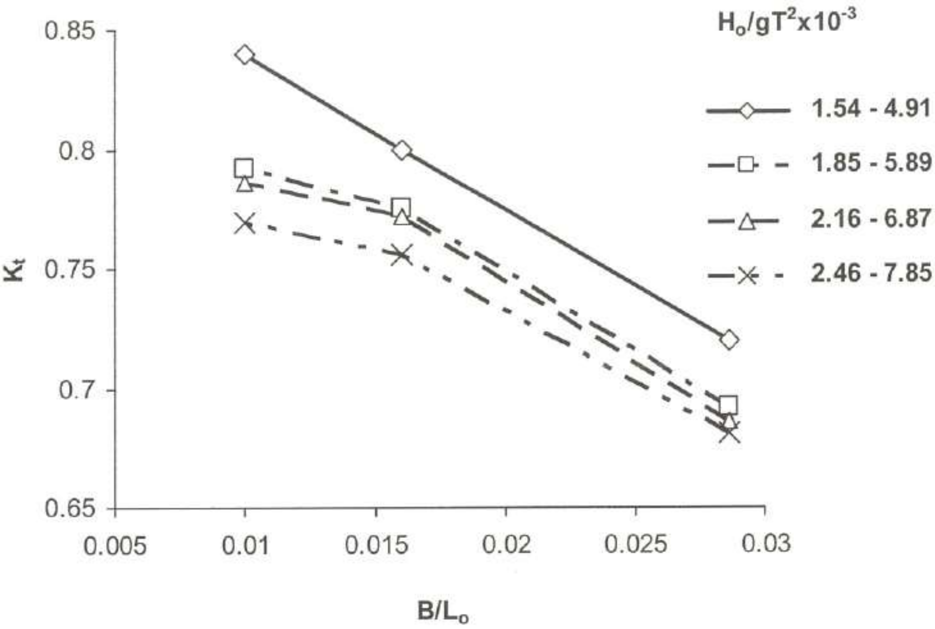

Fig. 8.7. Variation of Kt with B/Lo for d = 0.4m

8.3.1.2 Influence of deep water wave steepness on wave run up and run down

The influence of deep water wave steepness parameter (H_o/gT^2) on relative run up (R_u/H_o) and run down (R_d/H_o), for increasing ranges of depth parameter (d/gT^2) i.e. varying wave climate in depths of water of 0.3m, 0.35m and 0.4m, is shown by the best fit lines in Fig. 8.8 and Fig. 8.9 respectively. The results are compared with those for conventional (single) breakwater. Both the run up and the run down, decrease with an increase in wave steepness and decrease in the depth i.e. range of depth parameter.

The maximum relative run up and run down are respectively 1.24 times and 0.73 times the deep water wave height for the range of variables considered in the present study. Considering all the ranges H_o/gT^2, relative run up for depths of water of 0.3m (i.e $0.004 < d/gT^2 < 0.013$ and $-0.312 < F/H_i < -0.5$), 0.35m (i.e. $0.005 < d/gT^2 < 0.015$ and $-0.625 < F/H_i < -1.0$) and 0.4m (i.e. $0.006 < d/gT^2 < 0.018$ and $-0.94 < F/H_i < -1.5$), relative run up are about 35% to 38.6% lower than those of a conventional (single) breakwater. Similarly, relative rundown for depths of water of 0.3m, 0.35m and 0.4m are 23.5% to 30.8%, 16.6 to 17.6% and 6% to 15% lower than those of a conventional (single) breakwater respectively.

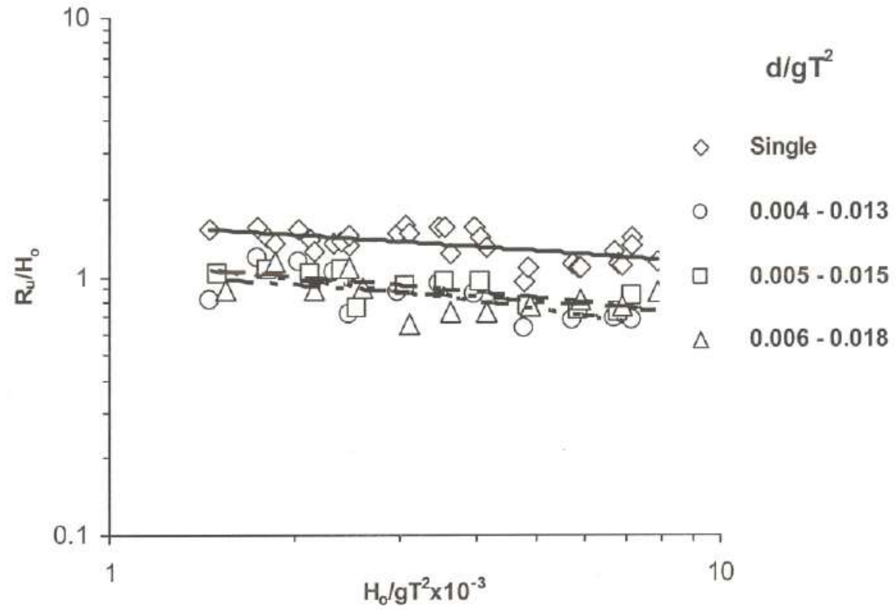

Fig. 8.8. Variation of R_u/H_o with H_o/gT^2

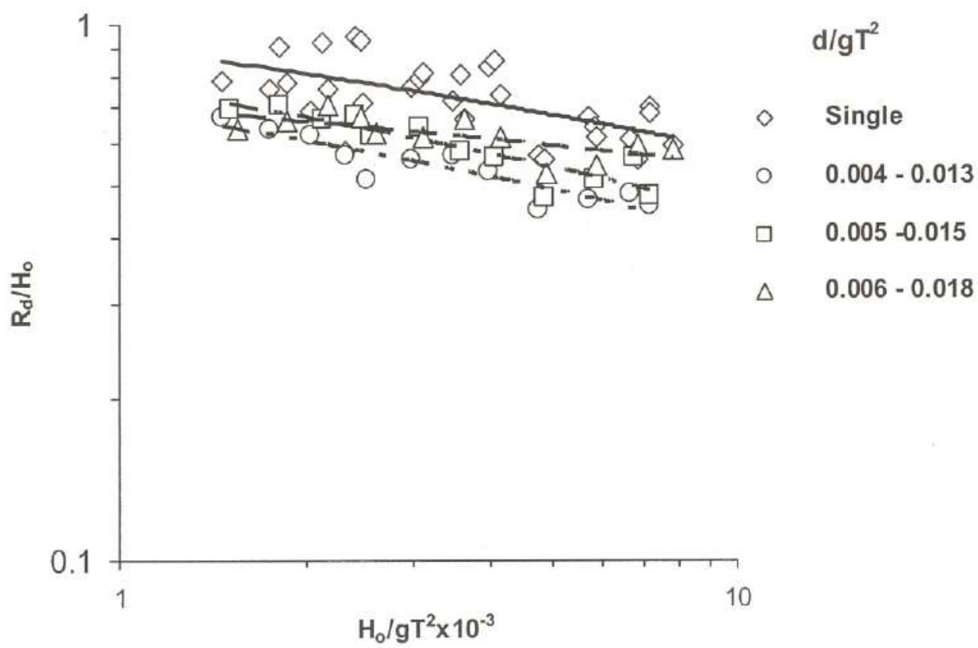

Fig. 8.9. Variation of R_d/H_o with H_o/gT^2

8.3.1.3 Influence of various parameters on damage level

8.3.1.3.1 Influence of deep water wave steepness

The graphs of Fig. 8.10 show increasing damage level (S) with increasing wave steepness parameter (H_o/gT^2) for increasing ranges of depth parameter (d/gT^2) i.e. for depths of water of 0.3m, 0.35m and 0.4m and different wave periods. This is due to increasing energy of steeper waves. The present damage levels are compared with those of conventional (single) breakwater. It also indicates that the breakwater damage increases with decrease in relative reef height h/d.

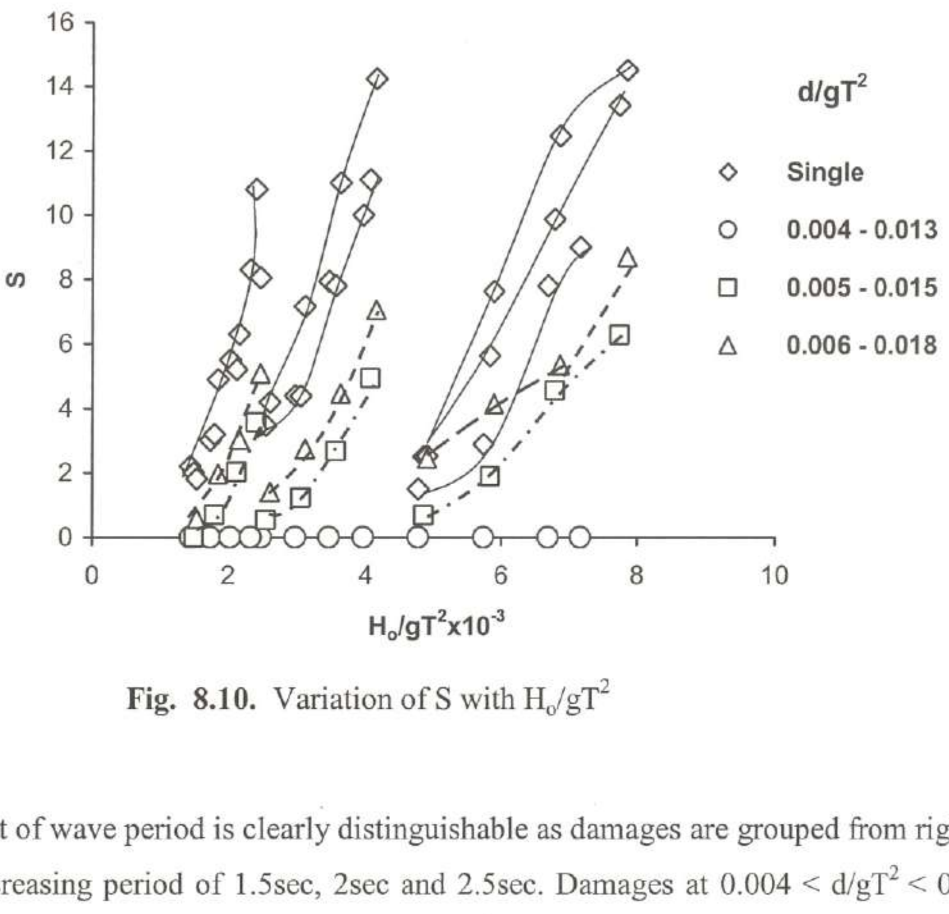

Fig. 8.10. Variation of S with H_o/gT^2

The impact of wave period is clearly distinguishable as damages are grouped from right to left for the increasing period of 1.5sec, 2sec and 2.5sec. Damages at $0.004 < d/gT^2 < 0.013$ i.e. depth of water of 0.3m are nil. For $0.005 < d/gT^2 < 0.015$ (i.e. depth of 0.35m) and $1.5x10^{-3} < H_o/gT^2 < 7.76x10^{-3}$, damage level (S) are 6.28, 4.94 and 3.58 for wave periods of 1.5sec, 2sec and 2.5sec respectively. Similarly, For $0.006 < d/gT^2 < 0.018$ (i.e. depth of 0.4m) and $1.54x10^{-3} < H_o/gT^2 < 7.85x10^{-3}$, damage level (S) are 8.7, 7.06 and 5.31 for wave periods of 1.5sec, 2sec and 2.5sec respectively. As the depth of water increased from 0.35m to 0.4m (i.e. 14.3%), for the wave period of 1.5sec, the maximum damage level S increases from 6.28 to 8.7 (i. e. a rise of 38.5%), and it increases from 4.94 to 7.06 (i. e. arise of 42.9%) for the wave period of 2.0sec while maximum damage level S rises from 3.58 to 5.31 (i. e. arise of 48.3%) for a wave period of 2.5sec.

8.3.1.3.2 Influence of reef submergence

The graphs in Fig. 8.11 show an increasing damage level (S) with the reef submergence (F/H_i) for varying depth parameter (d/gT^2). This is because increasing reef submergence indicated larger depths which sustain higher waves without much breaking. Also we can see that impact of wave period is clearly discernible, where, it shows increasing damage with shorter period waves for a given depth. The impact of depth of water on stability is clearly visible from the figure as damages are grouped from right to left for depths of 0.3m, 0.35m and 0.4m. Damages are nil for $0.004 < d/gT^2 < 0.013$ (i.e. a depth of 0.3m and h/d = 0.833) and $-0.312 < F/H_i < -0.5$. For $0.005 < d/gT^2 < 0.015$ (i.e. depth of 0.35m and h/d = 0.714) and $-0.625 < F/H_i < -1.0$, maximum damage levels increase from 3.58 to 6.28 (75.4%) and for $0.006 < d/gT^2 < 0.018$ (i.e. depth of 0.4m and h/d = 0.625) and $-0.94 < F/H_i < -1.5$, maximum damage levels increase from 5.31 to 8.7 (63.8%).

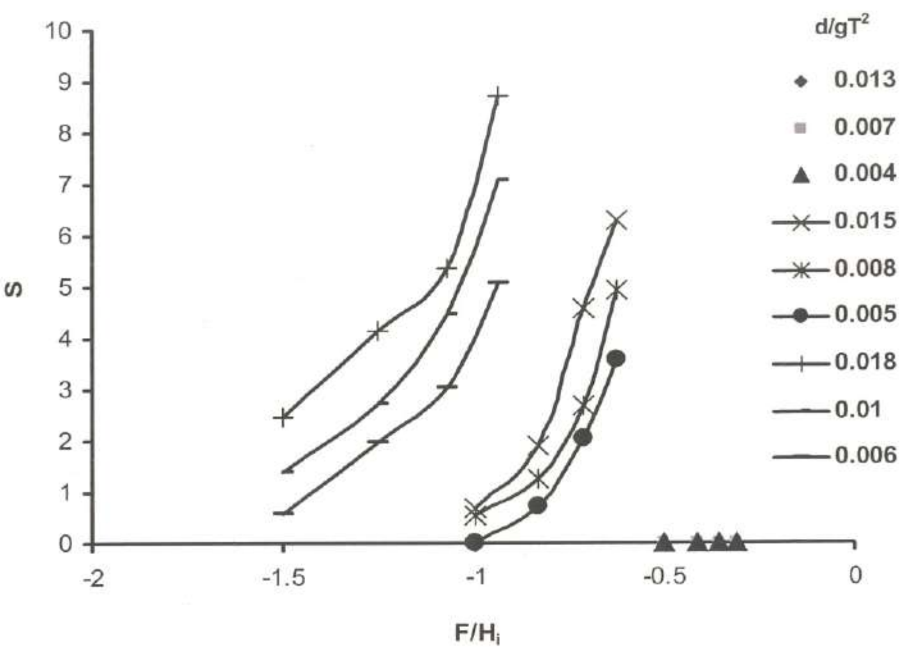

Fig. 8.11. Variation of S with F/H_i

8.3.1.3.3 Influence of reef crest width

Fig. 8.12 and Fig. 8.13 demonstrate the impact of reef crest width (B/L_o) on damage levels (S) of the breakwater, for increasing ranges of H_o/gT^2 i.e. for increasing wave heights of 0.1m to 0.16m of different periods of 1.5sec, 2sec and 2.5sec in water depths (d) of 0.35m and 0.4m respectively. For all depths of water, the general trend is increasing damage with reef crest widths (B/L_o) for any given range of wave steepness. This is because for a constant reef crest width (B) of 0.1m, the increase in B/L_o indicates decreasing L_o i.e. decreasing wave periods and as already observed shorter period waves are relatively more damaging compared to longer period waves.

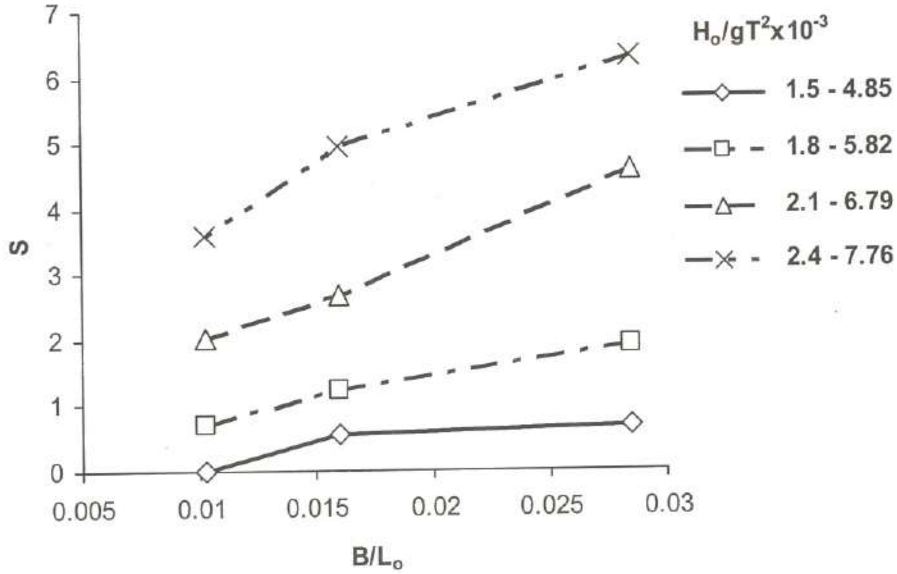

Fig. 8.12. Variation of S with B/L_o for d = 0.35m

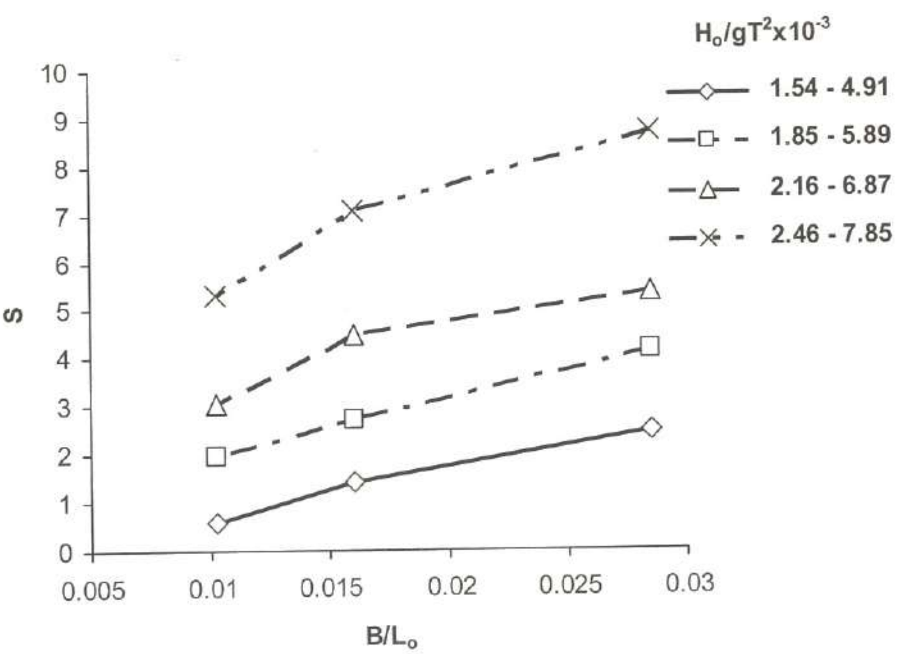

Fig. 8.13. Variation of S with B/L_o for d =0.4m

Graphs also indicate that, steeper waves are increasingly damaging the main breakwater. It is also seen that, the waves are more damaging with the increase in depth of water. For increasing ranges of H_o/gT^2 in a depth of 0.35m, where, $0.01 < B/L_o < 0.0285$, damage level S rises from 0.0 to 6.28. Similarly, in a depth of 0.4m S increases from 0.577 to 8.7.

8.3.1.3.4 Influence of stability number

Damages are negligible for a depth of water of 0.3m. The damage level (S) of the breakwater increases with an increase in stability number (N_s) for the depths of water (d) 0.35m and 0.4m as shown by the best fit lines in Fig. 8.14 and Fig. 8.15 respectively. The zero damage wave heights (H_{zd}) for depth of water of 0.35m and for wave periods of 1.5 sec, 2.0sec and 2.5sec are 0.1277m, 0.1457m and 0.1707m respectively which are 40% to 75% higher compared to conventional breakwater. H_{zd} for a depth of 0.40m and for corresponding wave periods of 1.5 sec, 2.0sec and 2.5sec are 0.09m, 0.1125m and 0.135m respectively which are 8.7% to 62% higher compared to conventional breakwater. It is found that, H_{zd} decrease with an increase in depth of water (d) from 0.35m to 0.4m indicating higher damage with increased depth. The damage levels of protected breakwater for depths of water of 0.35m and 0.4m are lower by 53% to 66% and 40% to 50% respectively compared to those of conventional breakwater.

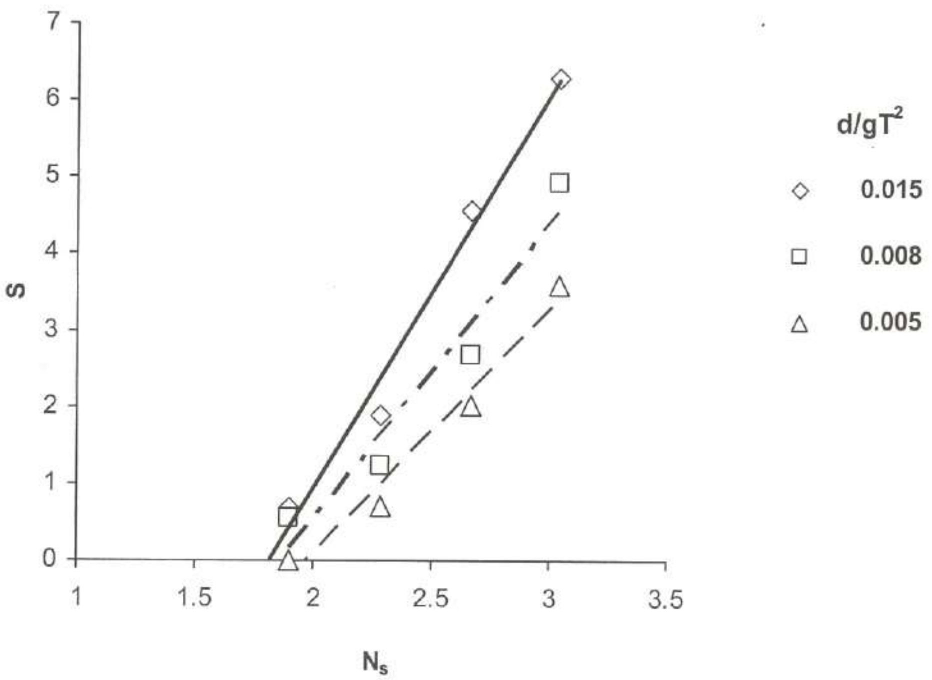

Fig. 8.14. Variation of S with N_s for d = 0.35m

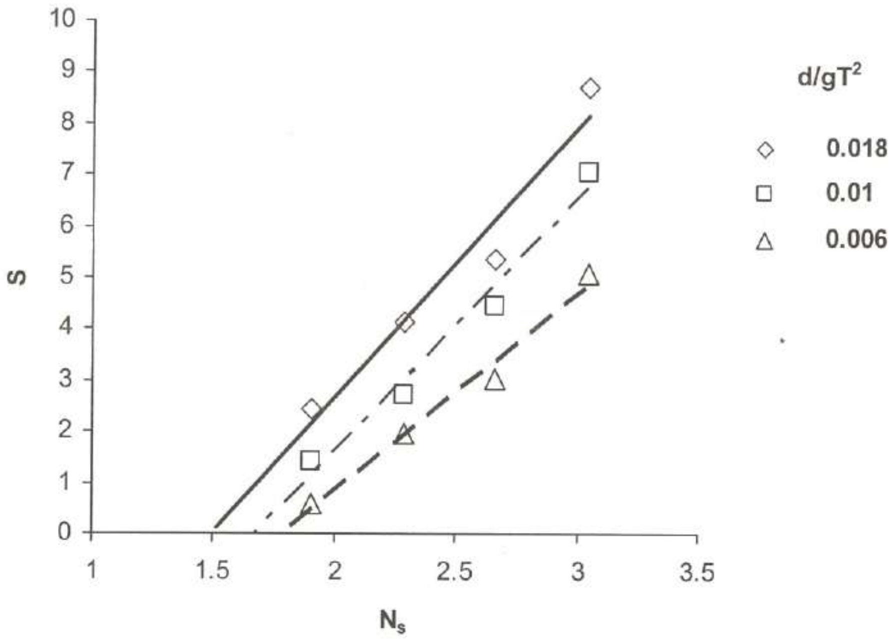

Fig. 8.15. Variation of S with N_s for d = 0.40m

8.3.1.3.5 Influence of surf similarity parameter on stability number

Thompsen et al. (1972) showed that, minimum stability of a 1V:2H sloped rubble mound breakwater occurred for surf similarity parameter $2 < \xi < 3$. Bruun and Gunbak (1976) write that the failure of the breakwater is caused by combinations of buoyancy, inertia and drag forces supported by the effect of hydrostatic pressure from the core and these forces reach their maximum value for lowest down rush which occurs at resonance for $2 < \xi < 3$. Fig. 8.16 shows the variation of zero damage stability number (N_{zd}) and surf similarity parameter (ξ) for varying wave climate in depths of water of 0.35m and 0.4m i.e. increasing ranges of depth parameter (d/gT^2). The results are compared with those given by Thompsen et al. (1972). For the present study, it is observed that N_{zd} increases with ξ. This is because H_{zd} increases with ξ. It is seen that minimum stability of the breakwater i. e. $N_{zd} = 1.71$ occurs for a ξ value of 3.11.

8.3.1.4 Conclusions

Based on the above discussions, the following conclusions are drawn.

1. The transmission coefficient (K_t) decreases with increase in H_o/gT^2, B/L_o and h/d and decreases with decrease in F/H_i and d/gT^2. K_t varies from 0.66 to 0.84.

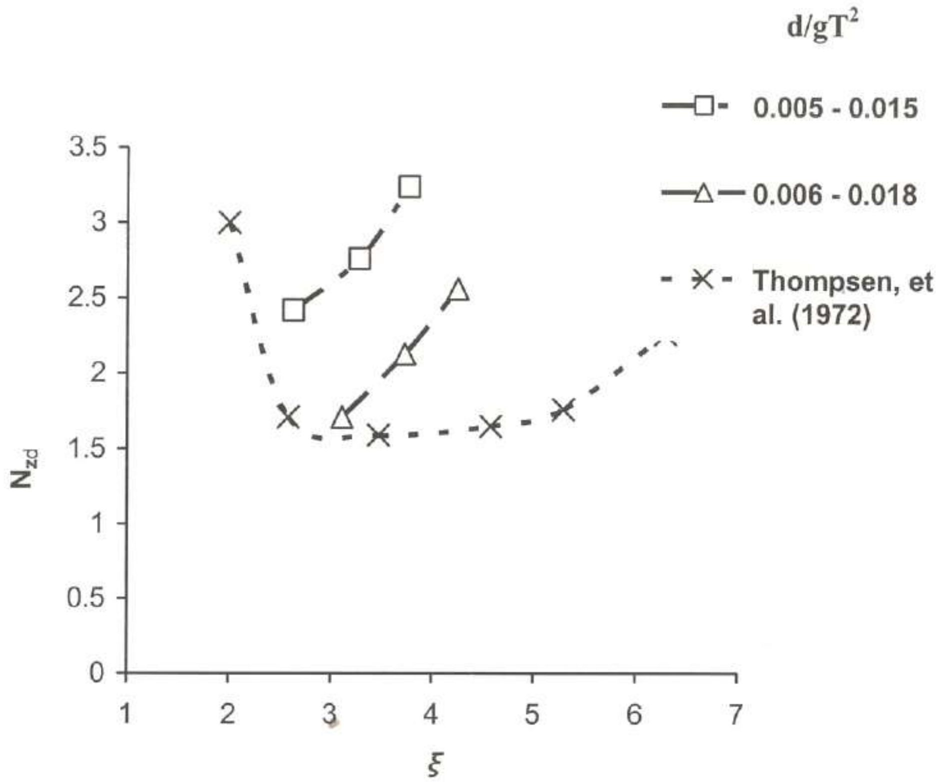

Fig. 8.16. Variation of N_{zd} with ξ

2. As $1.45x10^{-3} < H_o/gT^2 < 7.85x10^{-3}$ and $0.01 < B/L_o < 0.0285$, K_t drops from 0.76 to 0.66 (13.1%), 0.79 to 0.66 (16.4%) and 0.84 to 0.68 (19%) for h/d of 0.833, 0.714 and 0.625 (i.e. depth of water of 0.3, 0.35 and 0.4m) respectively.
3. For $-0.312 < F/H_i < -1.5$ and all depths, K_t increases from 0.64 to 0.7 (9.4%), 0.696 to 0.8 (14.9%) and 0.71 to 0.84 (18.3%) for wave periods 1.5sec, 2sec and 2.5sec, respectively.
4. Considering the complete ranges of d/gT^2 and F/H_i for all the wave periods, the present K_t values are 5% to 28% lower, 9% to 23% lower, 30% to 55% higher than Cox and Clark (1992), Cornett et al. (1993) and d'Angremond (1996) respectively and distributed around the values given by Van der Meer and d'Angremond (1992) with an accuracy of 24%.
5. The maximum run up and run down are respectively 1.24 times and 0.73 times the deep water wave height for the range of variables considered in the present study.
6. Run up and rundown are reduced 35% to 38.6% and 6% to 30.8% respectively compared to conventional breakwater.

7. The damage level S increases with the increase in H_o/gT^2, F/H_i, d/gT^2, B/L_o and decrease in h/d.
8. Damages at depth of water of 0.3m ($0.004<d/gT^2<0.013$, h/d = 0.833) are nil.
9. As the depth of water increases from 0.35m to 0.4m (i.e. $0.005<d/gT^2<0.015$ to $0.006<d/gT^2<0.018$) and 1.45x10-3 < H_o/gT^2 < 7.85x10-3, the maximum damage level increases from 6.28 to 8.7 (i. e. a rise of 38.5%), 4.94 to 7.06 (i. e. 42.9%) for 3.58 to 5.31 (i. e. 48.3%) for a wave period of 1.5sec, 2sec and 2.5sec respectively.
10. For $0.005<d/gT^2<0.015$ and -0.625 < F/H_i < -1.0 (i.e. for d = 0.35m, h/d=0.714), maximum damage level rises from 3.58 to 6.28 (75.4%) and for $0.006<d/gT^2<0.018$ and -0.94 < F/H_i < -1.5 (i.e. for d=0.4m, h/d=0.625), maximum damage level rises from 5.31 to 8.7 (63.8%).
11. As B/L_o increases from 0.01 to 0.0285 and considering all ranges of H_o/gT^2, damage level increases from 0 to 6.28, 0.577 to 8.7 for depths of 0.35m and 0.4m respectively.
12. At the depth of water of 0.35m the maximum damage level of the breakwater increases from 3.58 at 2.5sec to 6.28 at 1.5sec (i. e. a rise of 75.4%) and similarly at a depth of 0.4m (i.e. increase of 14.3% w.r.t 0.35m) damage level increases from 5.31 to 8.7 (i. e. 63.8%).
13. Damages at depths of water of 0.35m and 0.4m are 40% to 66% less compared to corresponding damages to conventional breakwater.
14. Zero damage wave heights are 8.7% to 75% higher than the conventional breakwater.
15. Minimum stability of the breakwater i. e. S = 1.71 occurs for a ξ value of 3.12.

8.3.2 Protected breakwater with a reef of crest width (B) of 0.2m (i. e. B/d = 0.50 to 0.67)

8.3.2.1 Influence of various parameters on transmission coefficient

8.3.2.1.1 Influence of deep water wave steepness

Fig. 8.17 shows the best fit lines for variation of transmission coefficient K_t with the deep water wave steepness parameter (H_o/gT^2) for varying relative reef height (h/d). K_t decreases with an increase in H_o/gT^2 and increase in relative reef height (h/d). K_t decreases from 0.57 to 0.43 (24.6%), 0.75 to 0.53 (29.3%) and 0.887 to 0.6 (32.6%) for h/d of 0.833, 0.714 and 0.625 i.e. for depths of water of 0.3m, 0.35m and 0.4m respectively where $1.45\text{x}10^{-3} < H_o/gT^2 < 7.85\text{x}10^{-3}$. This indicates that the wave height attenuation achieved is about 11% to 57%.

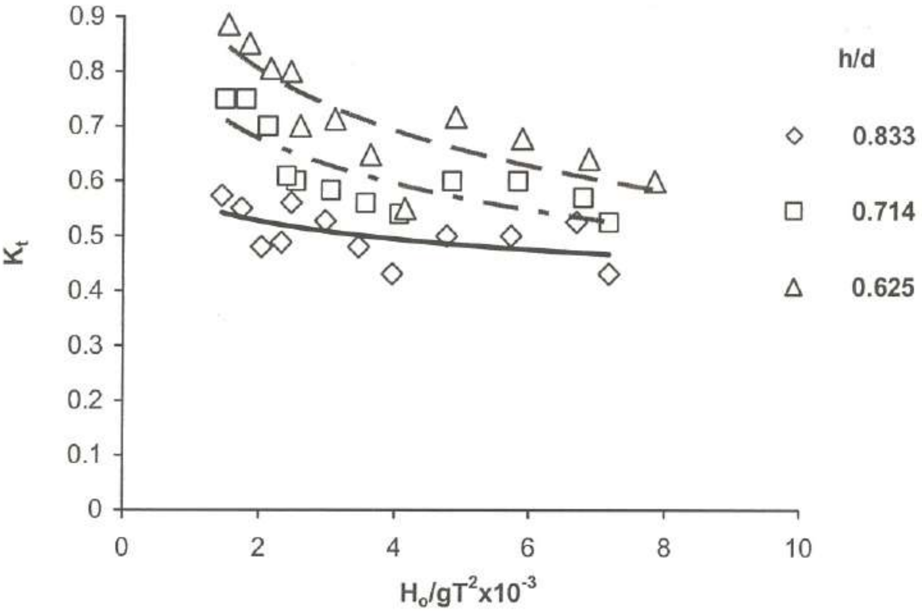

Fig. 8.17. Variation of K_t with H_0/gT^2

8.3.2.1.2 Influence of relative reef submergence

The transmission coefficient K_t increases as relative reef submergence (F/H_i) and the range of depth parameter (d/gT^2) increases for wave periods of 1.5sec, 2sec and 2.5sec as shown in Fig. 8.18, Fig. 8.19 and Fig. 8.20 respectively. This is because, in deeper waters and increasing reef submergence, waves break less at the submerged reef. Figures also show that the wave transmission gradually increases with wave period. The present study shows gradually increasing K_t with the wave period. The present K_t values are well predicted by criteria given by d'Angremond (1996) for periods of 1.5sec and 2sec and by Van der Meer and d'Angremond (1992) for 2.5sec. For 1.5sec and 2sec, K_t values are 38% to 41% lower, 23% lower and upto 17.3% lower than Cox and Clark (1992), Cornett et al. (1993) and Van der Meer and d'Angremond (1992) respectively. For 2.5sec, K_t values are 25% to 40% lower, 23% lower and 16% to 33% higher than Cox and Clark (1992), Cornett et al. (1993) and d'Angremond (1996) respectively. Considering the increasing ranges of d/gT^2 and F/H_i, K_t increases from 0.43 to 0.7 (62.8%), 0.43 to 0.72 (67.4%) and 0.49 to 0.887 (81%) for a wave period of 1.5sec, 2sec and 2.5sec respectively.

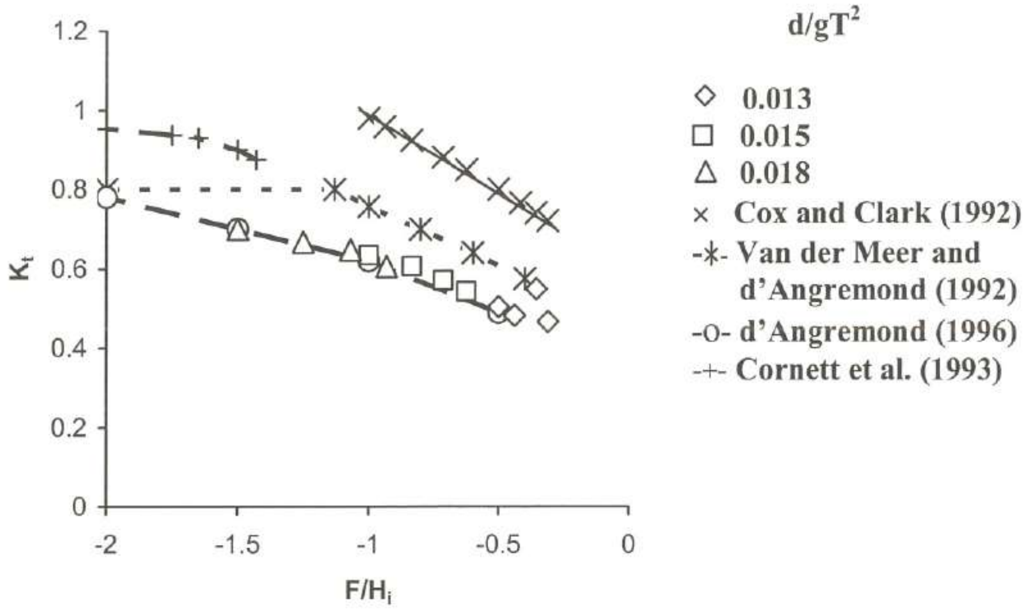

Fig. 8.18. Variation of K_t with F/H_i for T =1.5 sec

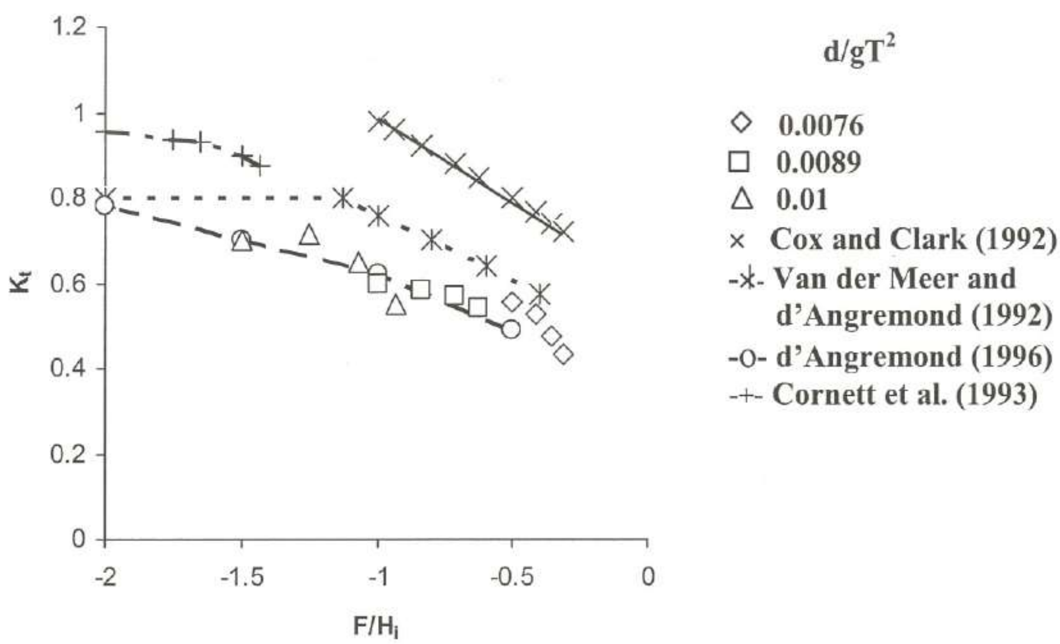

Fig. 8.19. Variation of K_t with F/H_i for T =2 sec

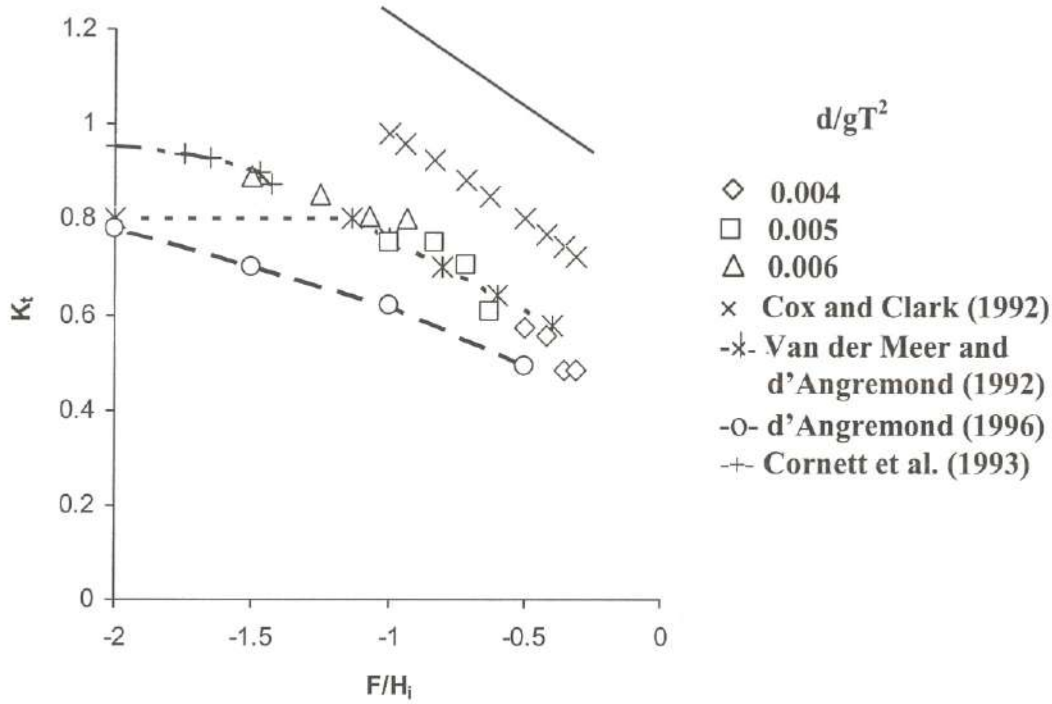

Fig. 8.20. Variation of K_t with F/H_i for T =2.5 sec

8.3.2.1.3 Influence of relative reef crest width

The variation of K_t with crest width (B/L_o) for increasing ranges of wave steepness parameter (H_o/gT^2) i.e. for increasing wave heights of 0.1m, 0.12m, 0.14m and 0.16m and periods 1.5sec, 2sec and 2.5sec in depths of water (d) of 0.3, 0.35 and 0.4m is shown in the Fig. 8.21, Fig. 8.22 and Fig. 8.23 respectively.

It is observed that for depth 0.3m and 0.4m, K_t decreases with an increase in B/L_o and with an increase in range of H_o/gT^2. But for a depth of 0.3m the freeboard (F) is quite small and wider reef crest increasingly interferes in the wave field resulting in different distribution of K_t. It is also observed that impact of B/L_o, for values higher than 0.0325, is not significantly felt by waves for depths of 0.35m and 0.4m. K_t decreases from 0.57 to 0.43 (24.6%), 0.75 to 0.53 (29.3%) and 0.887 to 0.6 (32.6%) for depths of water of 0.3m, 0.35m and 0.4m respectively. This shows that K_t values and its variation increases with an increase in depth of water.

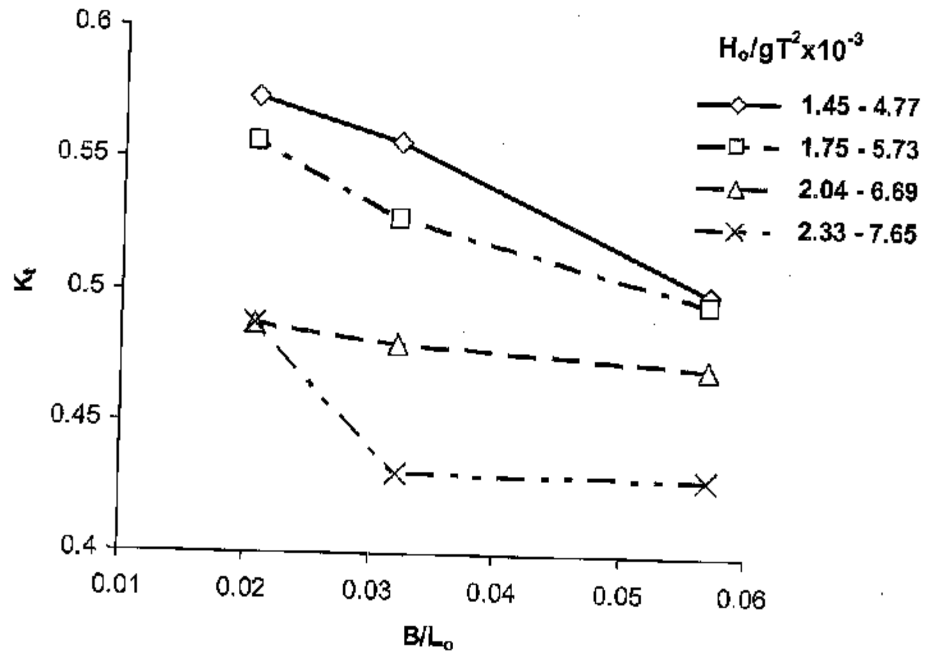

Fig. 8.21. Variation of K_t with B/L_o for d = 0.3m

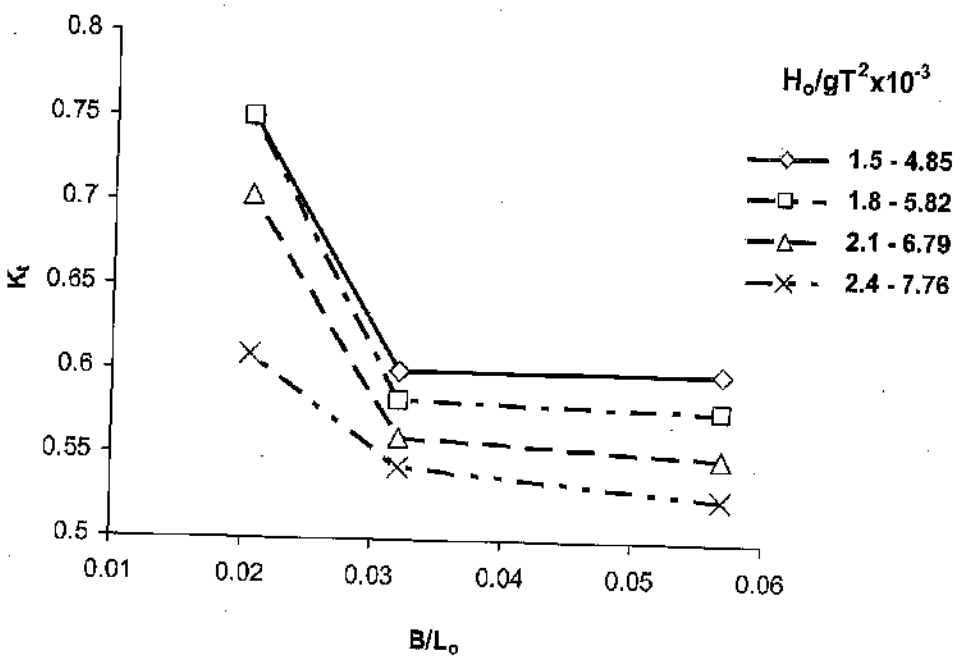

Fig. 8.22. Variation of K_t with B/L_o for d = 0.35m

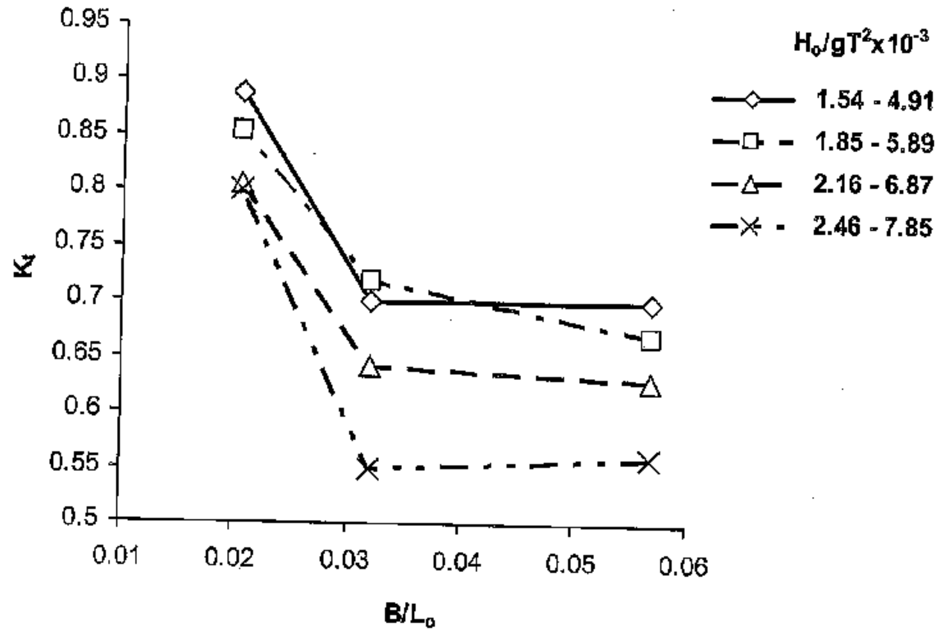

Fig. 8.23. Variation of K_t with B/L_o for d = 0.4m

8.3.2.2 Influence of deep water wave steepness on wave run up and run down

The influence of deep water wave steepness parameter H_o/gT^2 on relative run up (R_u/H_o) and run down (R_d/H_o) is shown by best fit lines in Fig. 8.24 and Fig. 8.25 respectively for varying

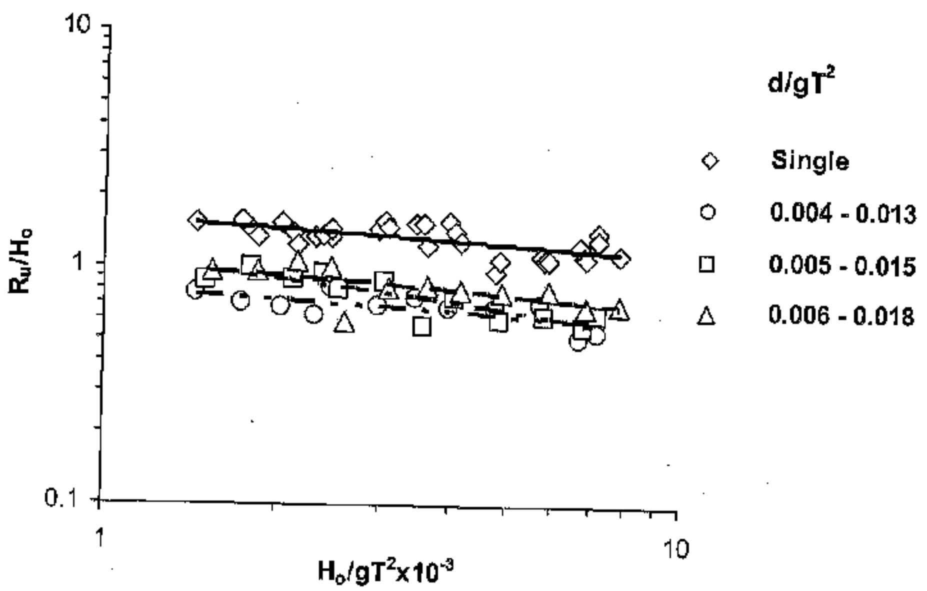

Fig. 8.24. Variation of R_u/H_o with H_o/gT^2

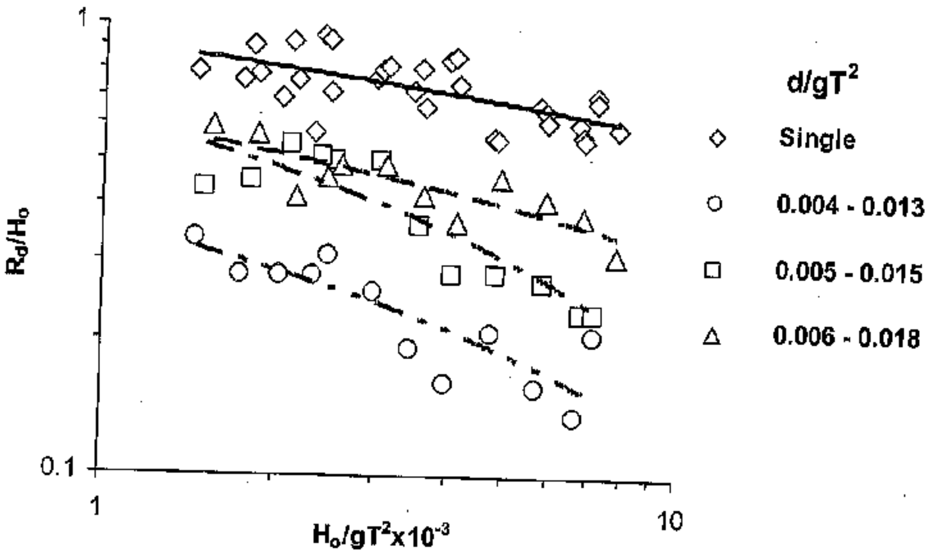

Fig. 8.25. Variation of R_d/H_o with H_o/gT^2

ranges of depth parameter (d/gT^2) i.e. varying wave climate in depths of water of 0.3m, 0.35m and 0.4m. Both the relative run up and the rundown, decrease with an increase in wave steepness and range of depth parameter. The maximum run up and run down are respectively 1.06 times and 0.54 times the deep water wave height for the range of variables considered in the present study. The results are compared with those of the conventional (single) breakwater. Considering all the ranges H_o/gT^2, relative run up for depths of water of 0.3m (i.e $0.004 < d/gT^2 < 0.013$ and $-0.312 < F/H_i < -0.5$), 0.35m (i.e. $0.005 < d/gT^2 < 0.015$ and $-0.625 < F/H_i < -1.0$) and 0.4m (i.e. $0.006 < d/gT^2 < 0.018$ and $-0.94 < F/H_i < -1.5$) relative run up are lower by 47% to 48.7%, 37.5% to 47.2%, 37.5% to 43.6% than corresponding values for conventional (single) breakwater. Similarly run down for depths of water of 0.3m, 0.35m and 0.4m are respectively lower by 61% to 75%, 38.8% to 61.6%, and 38.5% to 41.6% than that for conventional breakwater.

8.3.2.3 Influence of various parameters on damage level

8.3.2.3.1 Influence of deep water wave steepness

Fig. 8.26 shows the trends of damage level (S) with varying wave steepness parameter (H_o/gT^2) for increasing ranges of depth parameter (d/gT^2) i.e. increasing depths of water of 0.3m, 0.35m and 0.4m and different wave periods of 1.5sec, 2sec and 2.5sec.

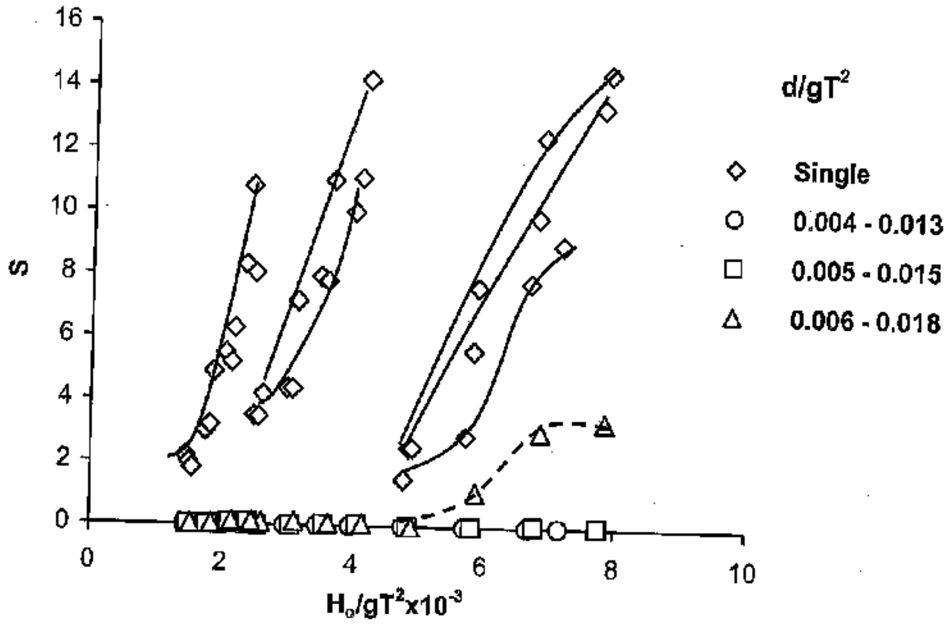

Fig. 8.26. Variation of S with H_o/gT^2

The results are compared with those of the conventional breakwater. At $0.004 < d/gT^2$ 0.013 and $0.005 < d/gT^2 < 0.015$ i. e. at depths of water of 0.3m and 0.35m and h/d of 0.833 and 0.714, damages to the breakwater are nil and negligible respectively. For the present configuration of the model, the damage occurs only at $0.006 < d/gT^2 < 0.018$ and $4.9 \times 10^{-3} < H_o/gT^2 < 7.85 \times 10^{-3}$ i.e. for waves of 1.5sec period in a depth of water of 0.4m and h/d of 0.625. At this depth, a damage level of 3.06 and 3.4 is observed respectively for waves 0.14m and 0.16m of period 1.5sec which damages are 75.42% to 76.55% lower compared to conventional breakwater.

8.3.2.3.2 Influence of reef submergence

The graph in Fig. 8.27 shows variation of the damage level (S) with the reef submergence (F/H_i) for varying depth parameter (d/gT^2). The breakwater damages for waves in depths of water of 0.3m and 0.35m (h/d = 0.833, 0.714) are nil and negligible respectively. The noticeable damage to the structure occurs only for d/gT^2 of 0.018, $-0.94 < F/H_i < -1.5$ and $4.9 \times 10^{-3} < H_o/gT^2 < 7.85 \times 10^{-3}$ in a depth of water of 0.4m (h/d = 0.625). Maximum damage (S) of 3.4 is observed.

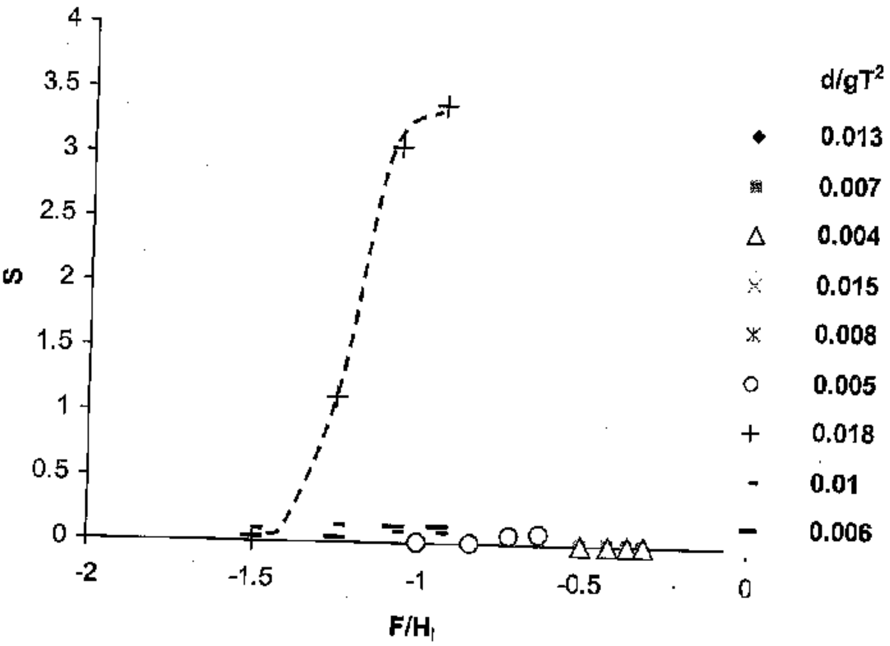

Fig. 8.27. Variation of S with F/H_i

8.3.2.3.3 Influence of reef crest width

Fig. 8.28 demonstrates the impact of reef crest width (B/L_o) on breakwater damage, for increasing ranges of H_o/gT^2 water depths (d) of 0.4m. From the figure it can be seen that for $0.02 < B/L_o < 0.057$, breakwater damage is observed only for a wave period of 1.5sec. It can also be seen that the damage is negligible for relatively smaller ranges of wave steepness of $1.54\text{x}10^{-3} < H_o/gT^2 < 4.91\text{x}10^{-3}$ and $1.85\ \text{x}10^{-3} < H_o/gT^2 < 5.89\text{x}10^{-3}$ and $B/L_o < 0.0325$ and then damage level increases to a value up to 3.06 and 3.4 for a wave of 0.14m and 0.16m respectively. Though the reef geometry successfully breaks the waves, it is not completely protecting the inner (main) breakwater.

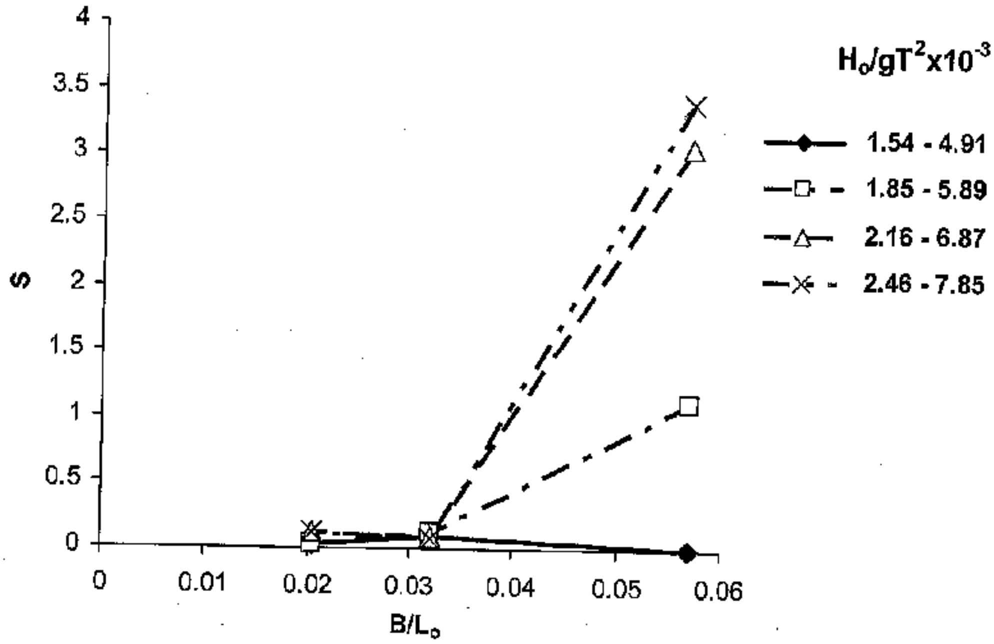

Fig. 8.28. Variation of S with B/L_o for d = 0.4m

8.3.2.3.4 Influence of stability number

The damage level (S) of the breakwater increases with an increase in stability number (N_s) for d/gT^2 of 0.018 i.e. waves of period of 1.5sec in a depth of water (d) of 0.4m, as shown by the best fit line in Fig. 8.29. The breakwater damage is noticeable only for waves of height of 0.14m and 0.16m. The zero damage wave height at this condition is 0.1328m which is 60.38% higher than that for conventional breakwater.

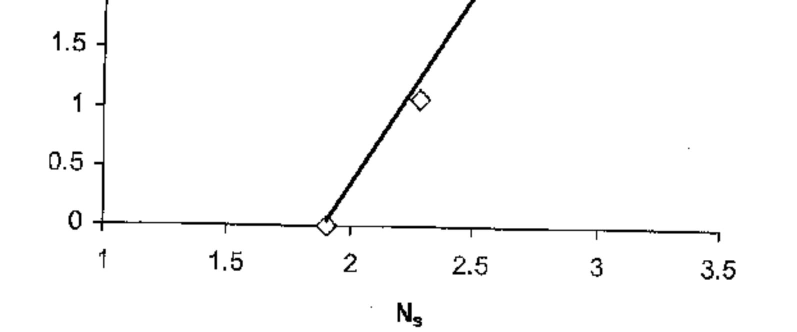

Fig. 8.29. Variation of S with N_s for d = 0.40m

8.3.2.3.5 Influence of surf similarity parameter on stability number

Fig. 8.30 shows the of zero damage stability number (N_{zd}) of 2.57 for a surf similarity parameter (ξ) of 2.52 obtained for waves of period 1.5sec in a depth of 0.4m. This result is compared with those given by Thompsen etal.(1972).

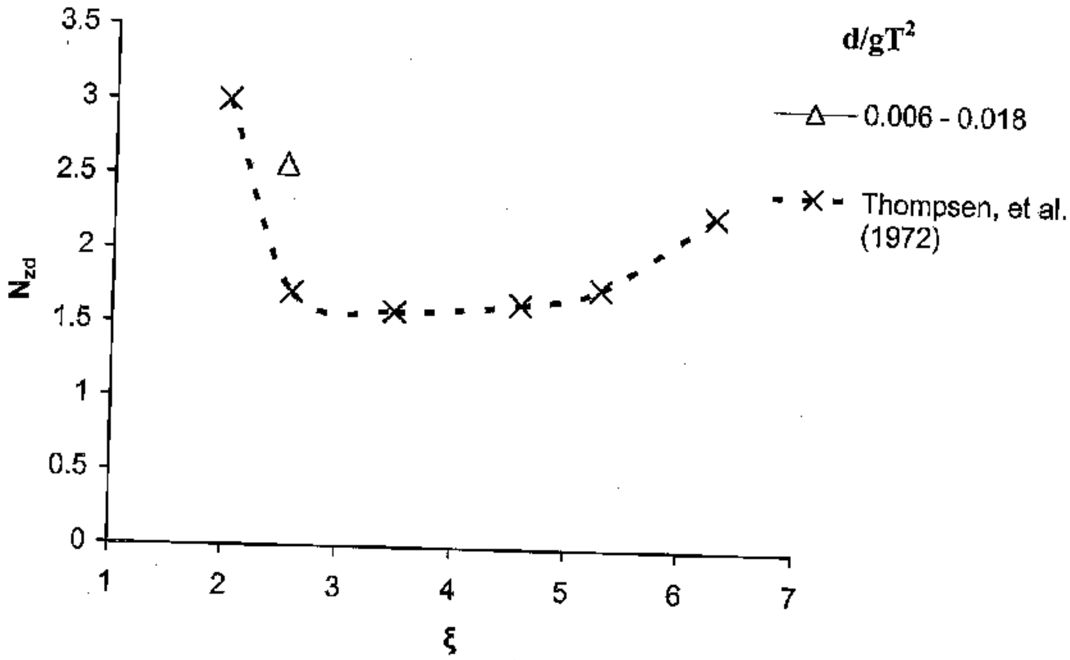

Fig. 8.30. Variation of N_{zd} with ξ

8.3.2.4 Conclusions

From the present model study the following conclusions are drawn.

1. The transmission coefficient (K_t) decreases with increase in H_o/gT^2, B/L_o and h/d and decreases with decrease in F/H_i and d/gT^2.
2. K_t drops from 0.57 to 0.43 (24.6%), 0.75 to 0.53 (29.3%) and 0.887 to 0.6 (32.6%) for h/d of 0.833, 0.714 and 0.625 (i.e. depth of water of 0.3, 0.35 and 0.4m) respectively for $1.45 \times 10^{-3} < H_o/gT^2 < 7.85 \times 10^{-3}$ and $0.02 < B/L_o < 0.057$.
3. K_t varies between 0.43 and 0.89 for the test parameters considered in the present study.
4. For $-0.312 < F/H_i < -1.5$ and all the depths of water, K_t increases from 0.43 to 0.7 (62.7%), 0.43 to 0.72 (67%) and 0.49 to 0.887 (81%) wave periods of 1.5sec, 2sec and 2.5sec respectively.
5. The present K_t values are well predicted by criteria given by d'Angremond (1996) for periods of 1.5sec and 2sec and by Van der Meer and d'Angremond (1992) for

2.5sec. For 1.5sec and 2sec, K_t values are 38% to 41% lower, 23% lower and up to 17.3% lower than Cox and Clark (1992), Cornett et al. (1993) and Van der Meer and d'Angremond (1992) respectively. For a wave period of 2.5sec, K_t values are 25% to 40% lower, 23% lower and 16% to 33% higher than Cox and Clark (1992), Cornett et al. (1993) and d'Angremond (1996) respectively.

6. The maximum run up and run down are respectively 1.06 times and 0.54 times the deep water wave height.
7. Relative run up and run down are 37.5% to 48.7% and 38.5% to 75% lower than those of conventional breakwater for the parameters considered in the study.
8. At depths of 0.3m and 0.35m (h/d=0.833, 0.714), damages to are nil and negligible respectively.
9. Damages S of 3.06 and 3.4 occur at $0.006 < d/gT^2 < 0.018$ i.e. in depth of water of 0.4m (h/d = 0.625) which are 75.42% and 76.55% lower compared to conventional breakwater.
10. Zero damage wave heights are 60.38% higher than the conventional breakwater.
11. Zero damage stability number (N_{zd}) of 2.57 for a surf similarity parameter (ξ) of 2.52 obtained for $0.006 < d/gT^2 < 0.018$ i.e. for waves of period 1.5sec in a depth of 0.4m.

8.3.3 Protected breakwater with a reef of crest width (B) of 0.3m (i. e. B/d = 0.750 to 1.0)

8.3.3.1 Influence of various parameters on transmission coefficient

8.3.3.1.1 Influence of deep water wave steepness

Fig. 8.31 illustrates the best fit lines for variation of transmission coefficient K_t with the deep water wave steepness parameter (H_o/gT^2) for varying relative reef height (h/d). K_t decreases with an increase in H_o/gT^2 and increase in relative reef height (h/d). K_t drops from 0.55 to 0.4 (27.27%), 0.62 to 0.48 (22.58%) and 0.8 to 0.48 (40%) for h/d of 0.833, 0.714 and 0.625 i.e. for depths of water of 0.3m, 0.35m and 0.4m respectively. Whereas, the average trend shows a variation of K_t in the ranges of 0.48 to 0.46, 0.62 to 0.5 and 0.75 to 0.56 for the same depths. The wave height attenuation achieved for the present configuration of the protected breakwater is 20% to 60%. It can be inferred that with an increase in relative reef height (h/d), wave damping increases and the influence of wave steepness on K_t gradually reduces.

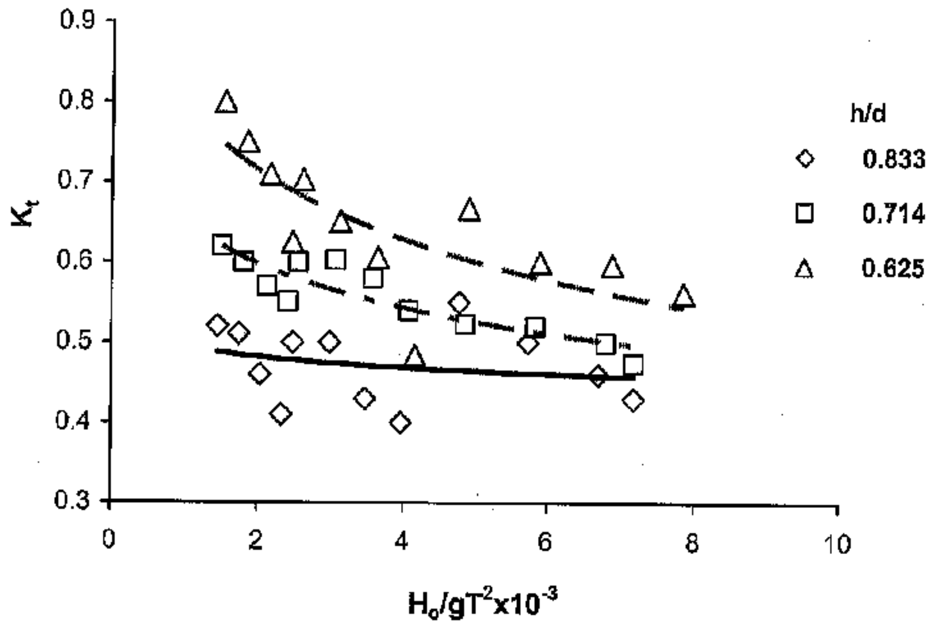

Fig. 8.31. Variation of K_t with H_o/gT^2

8.3.3.1.2 Influence of relative reef submergence

The transmission coefficient K_t increases as relative reef submergence (F/H_i) and the range of depth parameter (d/gT^2) increases for wave periods of 1.5sec, 2sec and 2.5sec which is illustrated in Fig. 8.32, Fig. 8.33 and Fig. 8.34 respectively.

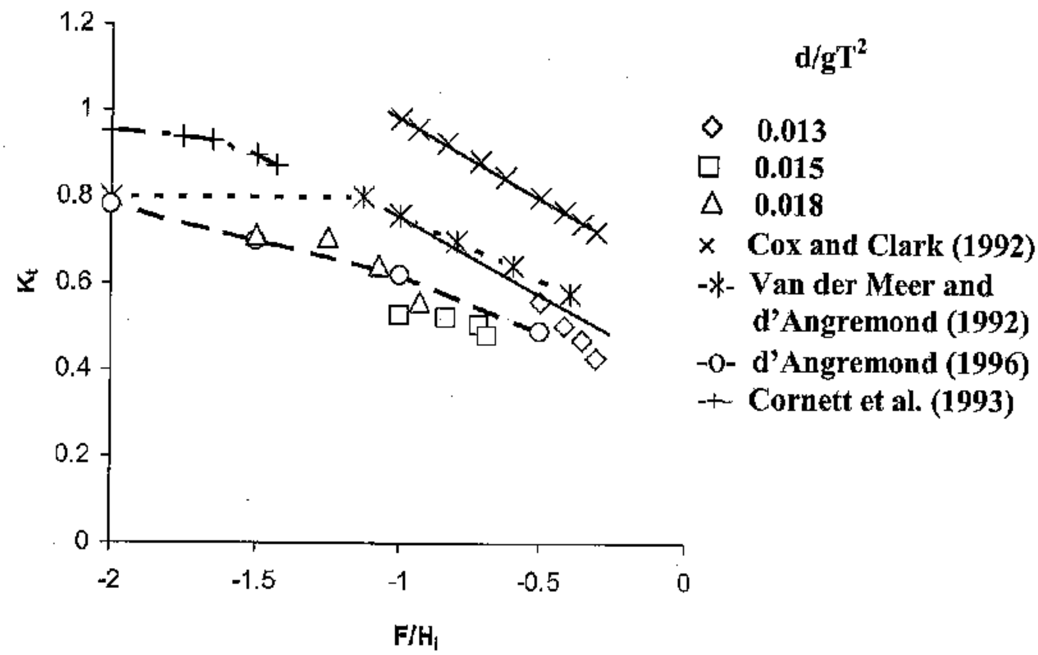

Fig. 8.32. Variation of K_t with F/H_i for T =1.5 sec

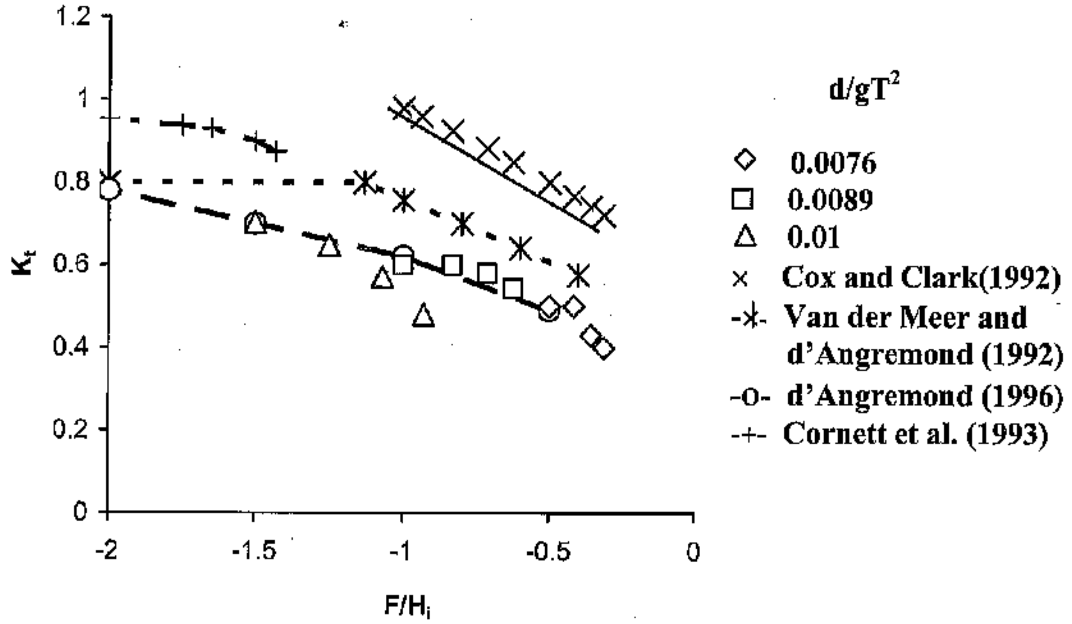

Fig. 8.33. Variation of K_t with F/H_i for T =2 sec

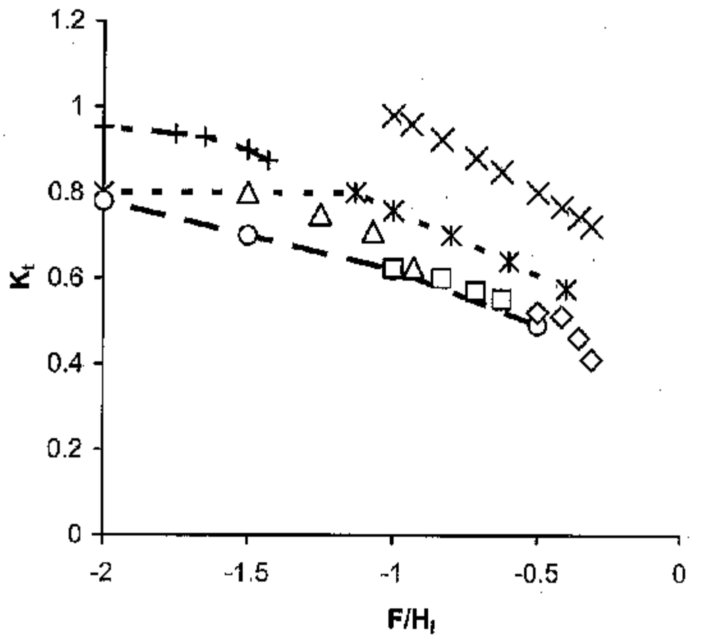

Fig. 8.34. Variation of K_t with F/H_i for T =2.5 sec

Figures show almost similar trends of K_t which is represented by d'Angremond (1996) with an accuracy of 20%. Present K_t values are lower by 36% to 46%, 11% to 21%, up to 30.5% than those of Cox and Clark (1992), Cornett et al. (1993) and Van der Meer and d'Angremond (1992) respectively.

For -0.312 < F/H_i < -1.5, all the ranges of the depth parameter (d/gT^2) considered, K_t increases from 0.43 to 0.71 (65.1%), 0.4 to 0.7 (75%) and 0.41 to 0.8 (95.1%) for wave period of 1.5sec, 2sec and 2.5sec respectively.

8.3.3.1.3 Influence of relative reef crest width

Fig. 8.35, Fig. 8.36 and Fig. 8.37 illustrate the variation of K_t with relative reef crest width (B/L_o) for different wave steepness parameters (H_o/gT^2) i.e. for increasing wave heights and periods for depths of water (d) of 0.3m, 0.35m and 0.4m respectively. It is observed that for a given depth, K_t decreases with increase in B/L_o and range of H_o/gT^2. But this is observed only for a depth of 0.35m as shown in Fig. 8.36. It is also seen that at value B/L_o of 0.048, the nature of variation of K_t changes. After this point, while K_t drops for a depth of 0.35m (refer Fig. 8.36), it remains almost constant for depths of 0.3m and 0.4m as demonstrated by Fig. 8.35 and Fig. 8.37.

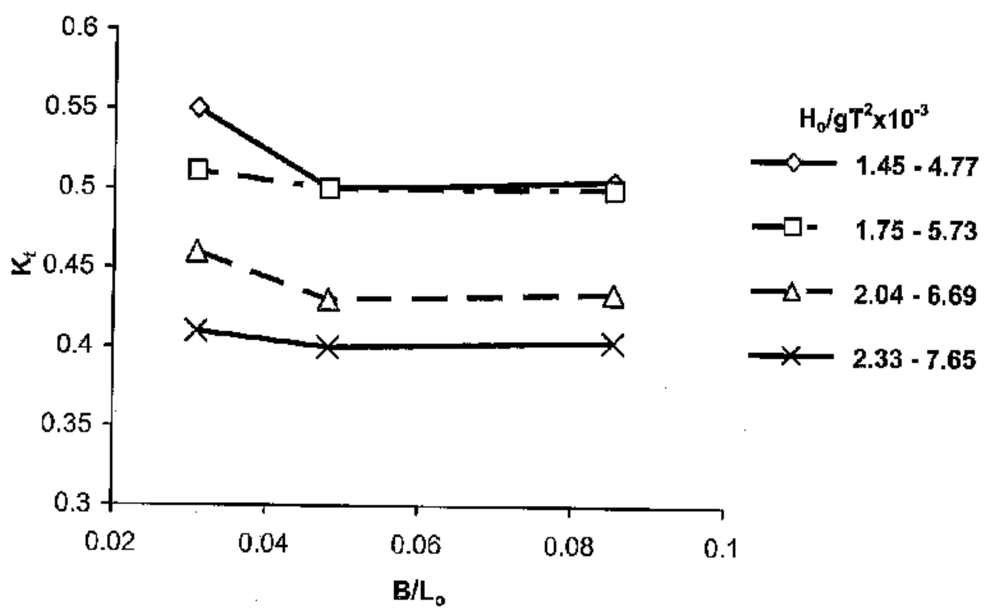

Fig. 8.35. Variation of K_t with B/L_0 for d = 0.3m

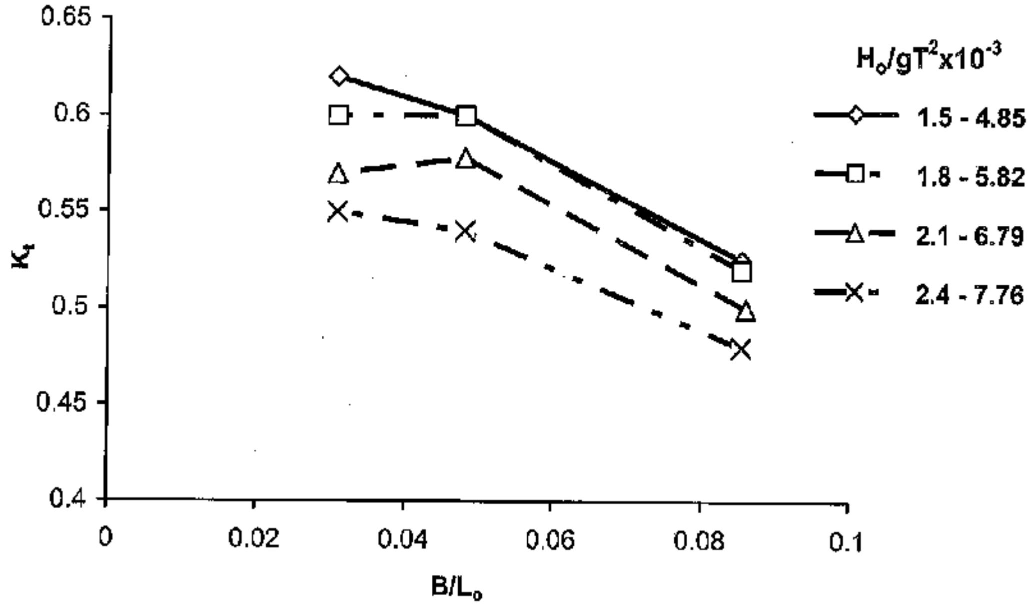

Fig. 8.36. Variation of K_t with B/L_o for d = 0.35m

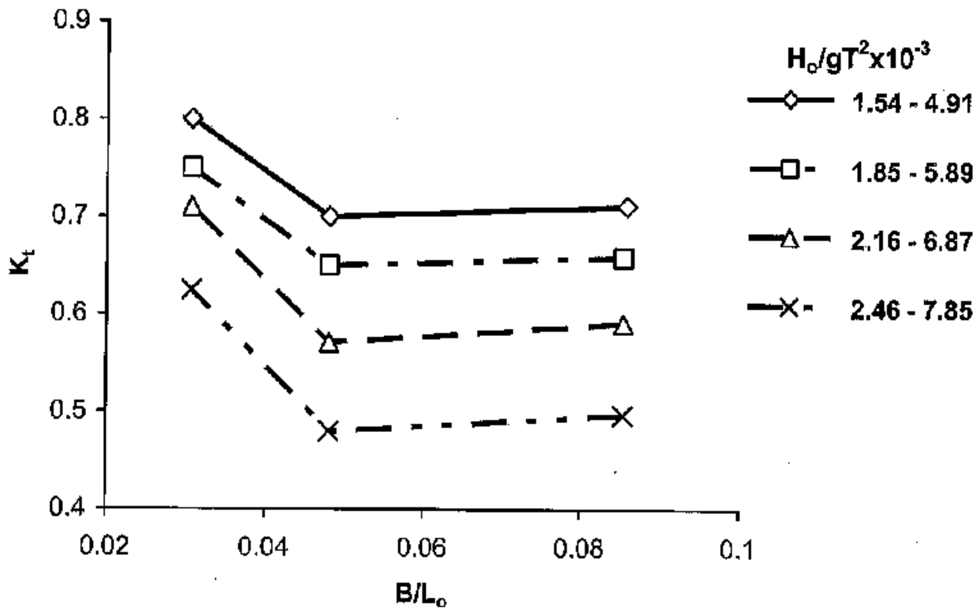

Fig. 8.37. Variation of K_t with B/L_o for d = 0.4m

8.3.3.2 Influence of deep water wave steepness on wave run up and run down

Fig. 8.38 and Fig. 8.39 reveal the influence of deep water wave steepness parameter ($H_o/(gT^2)$) on relative run up (R_u/H_o) and run down (R_d/H_o) respectively, by best fit lines, for varying wave climate in depths of water of 0.3m, 0.35m and 0.4m i.e. increasing ranges of d/gT^2. The results are compared with those for conventional (single) breakwater. The maximum run up and run down are respectively 0.97 times and 0.44 times the deep water wave height for the range of variables considered in the present study.

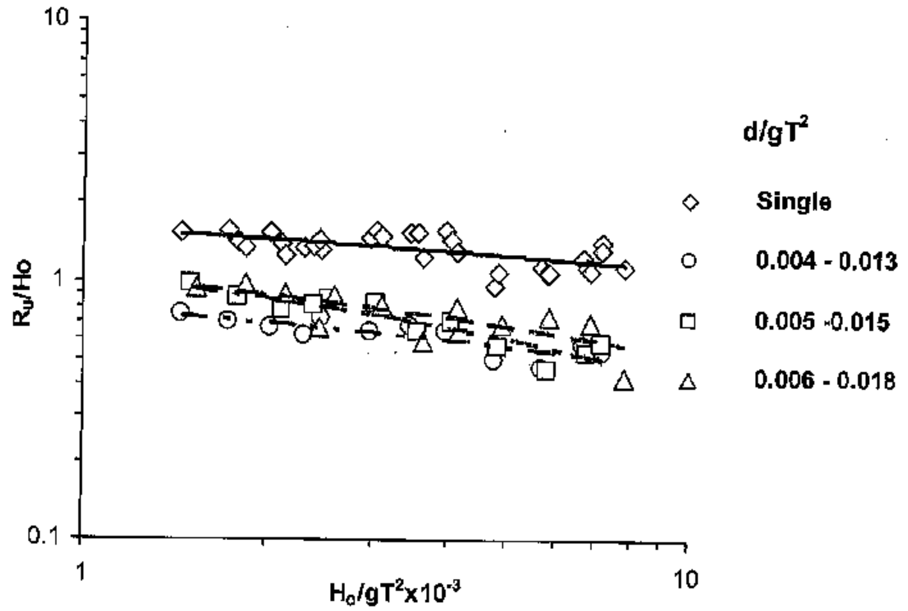

Fig. 8.38. Variation of R_u/H_o with H_o/gT^2

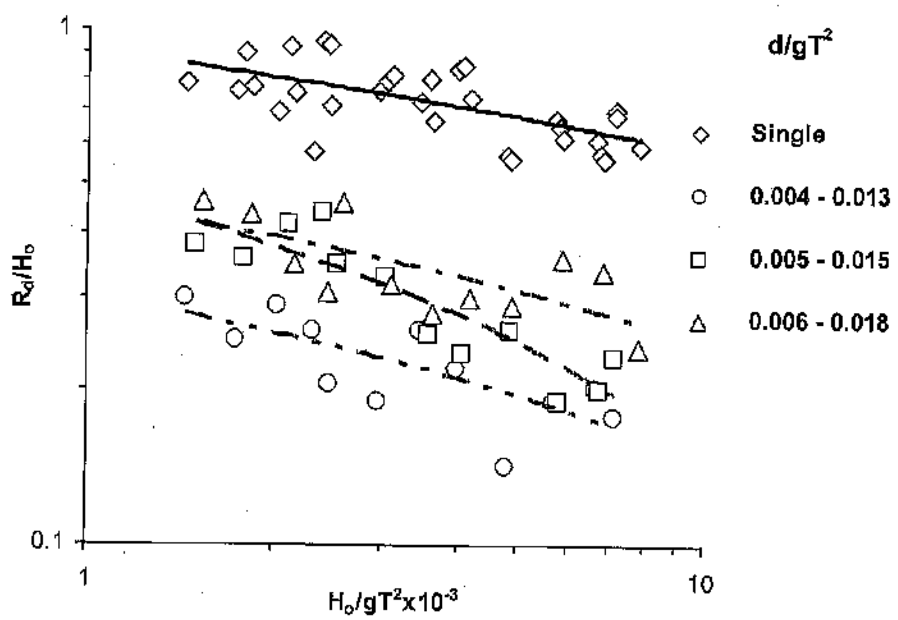

Fig. 8.39. Variation of R_d/H_o with H_o/gT^2

For $1.45 \times 10^{-3} < H_o/gT^2 < 7.85 \times 10^{-3}$ and $0.004 < d/gT^2 < 0.013$, $0.005 < d/gT^2 < 0.015$ and $0.006 < d/gT^2 < 0.018$ i.e. for depths of water of 0.3m, 0.35m and 0.4m and F/H_i is varied in different ranges between -0.312 and -1.5, relative run up are respectively 53% to 55%, 36% to 53% and 36% to 47% less than less than that for conventional breakwater. Similarly, for the same range of parameters, relative run down are 67% to 73%, 51% to 68% and 51% to 55% lower than that for conventional breakwater.

8.3.3.3 Influence of various parameters on damage level

8.3.3.3.1 Influence of deep water wave steepness

The trends of damage level (S) with varying wave steepness parameter (H_o/gT^2) for increasing ranges of depth parameter (d/gT^2) i.e. increasing depths of water of 0.3m, 0.35m and 0.4m and different wave periods of 1.5sec, 2sec and 2.5sec are shown in Fig. 8.40. From the figure it is seen that, at all depths of water, damages to the inner (main) breakwater are nil. Therefore, the present geometry of protected breakwater is such that, it is completely sheltered from all the waves and the structure is totally safe for test conditions selected.

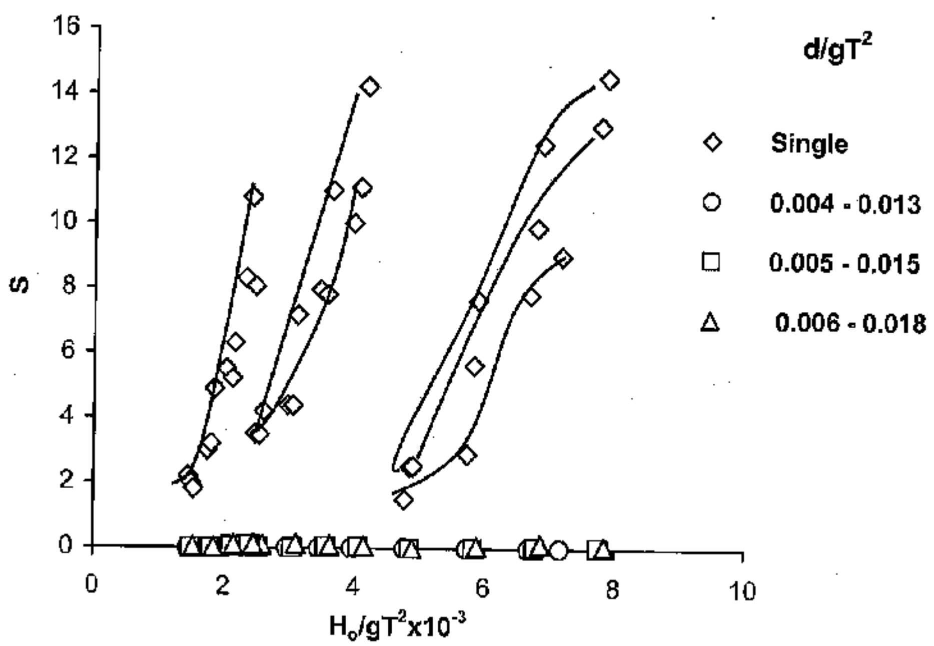

Fig. 8.40. Variation of S with H_o/gT^2

8.3.3.4 Conclusions

From the present study, the following conclusions are drawn.

1. The transmission coefficient (K_t) decreases with increase in H_o/gT^2, B/L_o and h/d and decreases with decrease in F/H_i and d/gT^2.
2. K_t drops from 0.55 to 0.4 (27.27%), 0.62 to 0.48 (22.58%) and 0.8 to 0.48 (40%) for h/d of 0.833, 0.714 and 0.625 (i.e. depth of water of 0.3, 0.35 and 0.4m) respectively as $1.45 \times 10^{-3} < H_o/gT^2 < 7.85 \times 10^{-3}$ and $0.03 < B/L_o < 0.0855$.
3. K_t varies between 0.4 and 0.8 for the test parameters considered in the present study.
4. For all the ranges of relative reef submergence F/H_i and depth parameter d/gT^2 considered, K_t increases from 0.43 to 0.71 (65.1%), 0.4 to 0.7 (75%) and 0.41 to 0.8 (95.1%) for wave period of 1.5sec, 2sec and 2.5sec respectively.
5. K_t values are represented by d'Angremond (1996) with an accuracy of 20%. Present K_t values are lower by 36% to 46%, 11% to 21%, up to 30.5% than those of Cox and Clark (1992), Cornett et al. (1993) and Van der Meer and d'Angremond (1992) respectively.
6. K_t decreases with increase in B/L_o and range of H_o/gT^2. But this is observed only for a depth of 0.35m as shown.
7. The maximum run up and run down are respectively 0.97 times and 0.44 times the deep water wave height.
8. For the parameters considered in the study, the relative run up and run down are 36% to 55% lower and 51% to 73% lower compared to that of the conventional breakwater.
9. The breakwater damages are nil for all the test parameters considered. Therefore, the protected breakwater is completely sheltered from all the waves and the structure is totally safe for test parameters considered.
10. This geometry of the protected breakwater could be an optimum as the inner (main) breakwater is totally sheltered from any kind of damage for the complete ranges of all the test parameters considered.

8.3.4 Protected breakwater with a reef of crest width (B) of 0.4m (i.e. B/d = 1.0 to 1.33)

8.3.4.1 Influence of various parameters on transmission coefficient

8.3.4.1.1 Influence of deep water wave steepness

Fig. 8.41 illustrates the variation of transmission coefficient K_t with the deep water wave steepness parameter (H_o/gT^2) through the best fit lines for varying relative reef height (h/d). K_t decreases with an increase in H_o/gT^2 and (h/d). K_t decreases from 0.55 to 0.38 (30.9%), 0.64 to 0.53 (17.19%),) and 0.708 to 0.55 (22.32%),) for h/d of 0.833, 0.714 and 0.625 i.e. for depths of water of 0.3m, 0.35m and 0.4m respectively while $1.45 \times 10^{-3} < H_o/gT^2 < 7.85 \times 10^{-3}$.

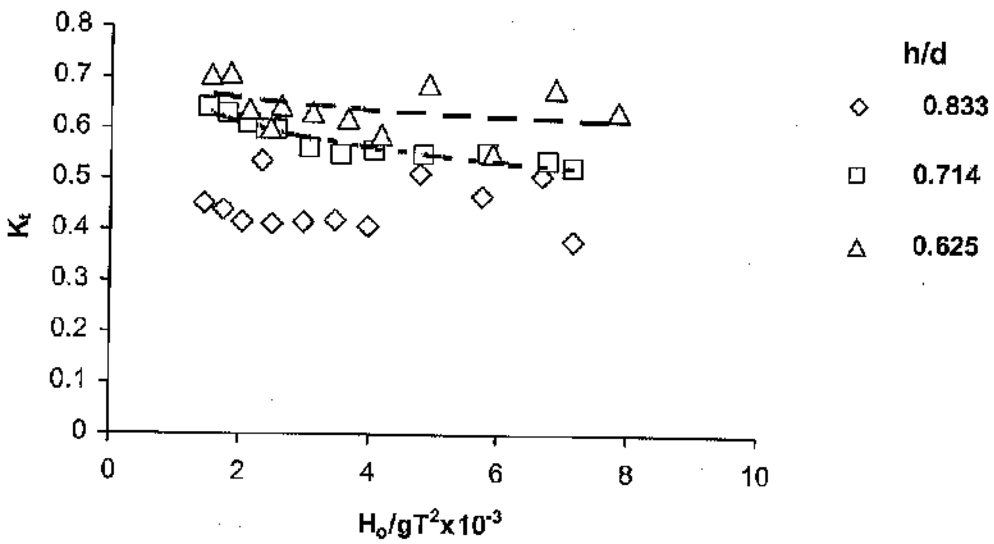

Fig. 8.41. Variation of K_t with Ho/gT^2

The wave height attenuation achieved is 29.2% to 62%. The average trends show a decrease in K_t from 0.45 to 0.4, 0.64 to 0.52 and 0.67 to 0.62 for the same depths. It can be observed that with an increase in relative reef height (h/d), wave damping increases and the influence of wave steepness on K_t is relatively more for a depth of 0.35m.

8.3.4.1.2 Influence of relative reef submergence

The transmission coefficient K_t increases as relative reef submergence (F/H_i) and the range of depth parameter (d/gT^2) increase for wave periods of 1.5sec, 2sec and 2.5sec as illustrated in Fig. 8.42, Fig. 8.43 and Fig. 8.44 respectively. As $-0.312 < F/H_i < -1.5$, K_t show almost similar trend for all the wave periods and is represented by d'Angremond (1996) with an accuracy of 16%.

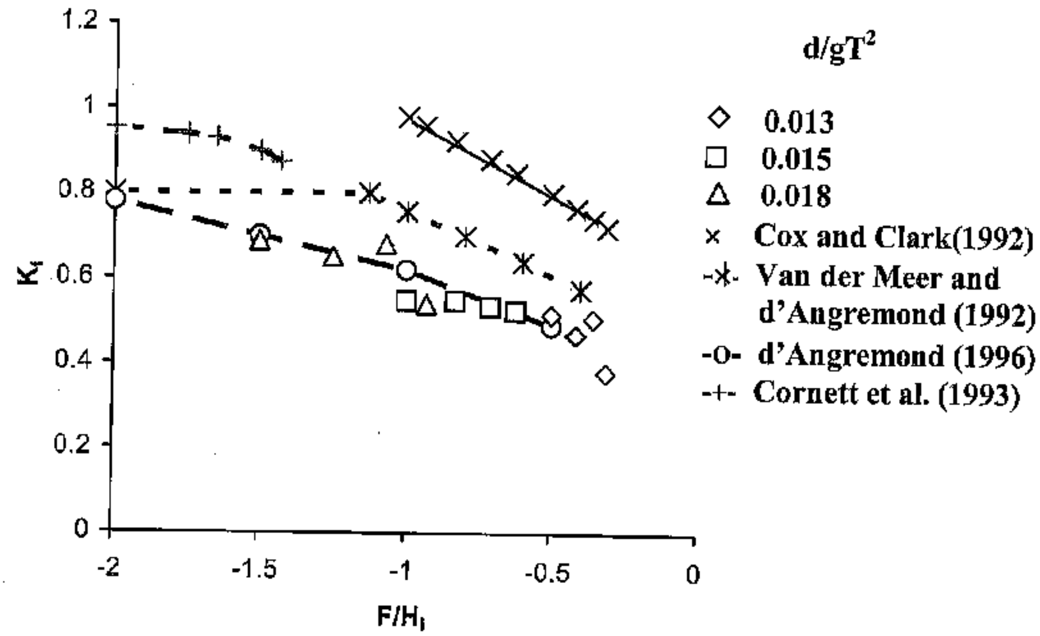

Fig. 8.42. Variation of K_t with F/H_i for T =1.5 sec

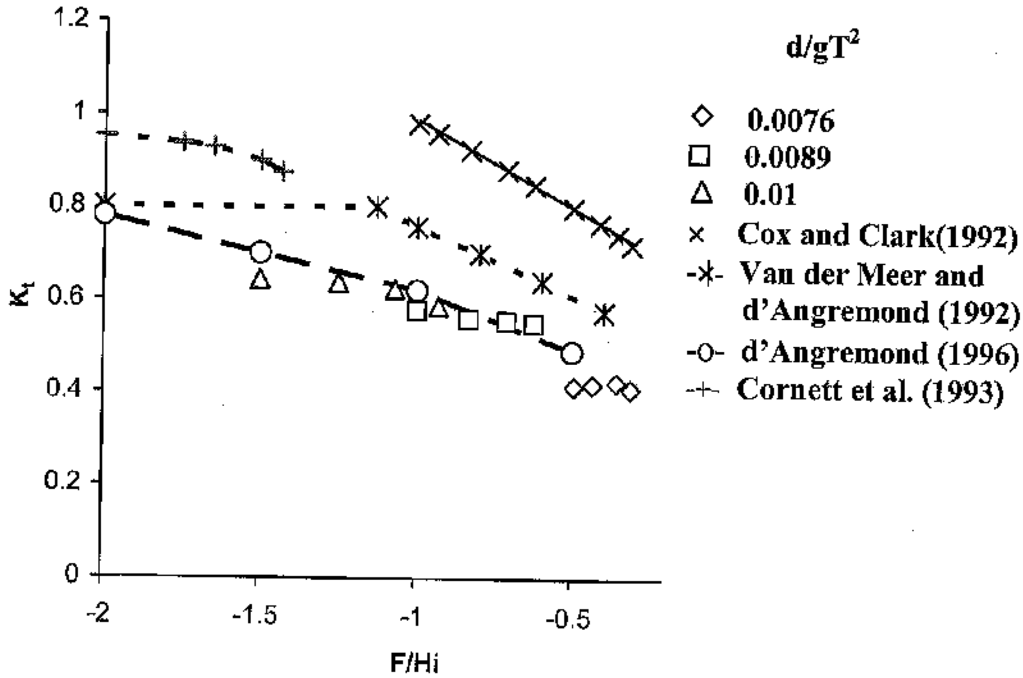

Fig. 8.43. Variation of K_t with F/H_i for T =2 sec

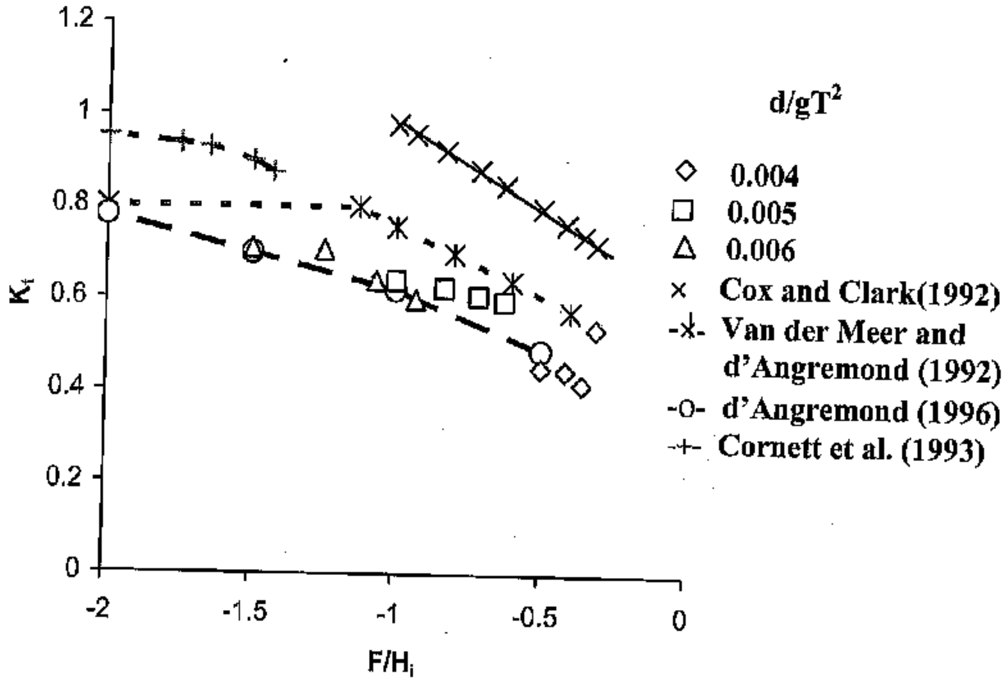

Fig. 8.44. Variation of K_t with F/H_i for T =2.5 sec

The K_t values are 37.5% to 48.3%, 16.67% to 31%, and 21% to 28.6% lower compared to Cox and Clark (1992), Van der Meer and d'Angremond (1992) and Cornett et al. (1993) respectively. For wave periods of 1.5sec, 2sec and 2.5sec K_t increases from 0.38 to 0.69 (81.6%), 0.4 to 0.65 (62.5%) and 0.42 to 0.708 (68.6%) respectively.

8.3.4.1.3 Influence of relative reef crest width

Fig. 8.45, Fig. 8.46 and Fig. 8.47 show the trends of K_t with relative reef crest width (B/L_o) for different ranges of wave steepness parameters (H_o/gT^2) for water depths of (d) of 0.3m, 0.35m and 0.4m in respectively. It is generally observed that for a given depth, K_t decreases with an increase in B/L_o and with an increase in range of H_o/gT^2. But this is clearly seen only for depth of water of 0.35m for $0.04 < B/L_o < 0.114$. It is also observed at value B/L_o of 0.064, the nature of variation of K_t changes. After this point, while K_t drops for a depth of 0.35m, it shows different trends for depths of 0.3m and 0.4m. This may be due to wider reef interfering in the wave field.

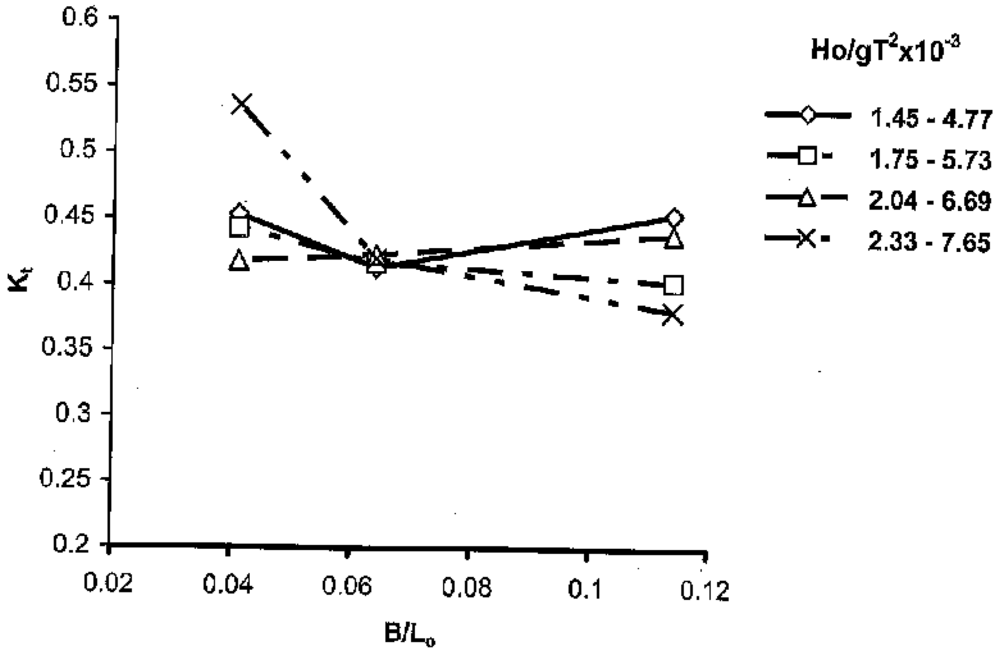

Fig. 8.45. Variation of K_t with B/L_o for d = 0.3m

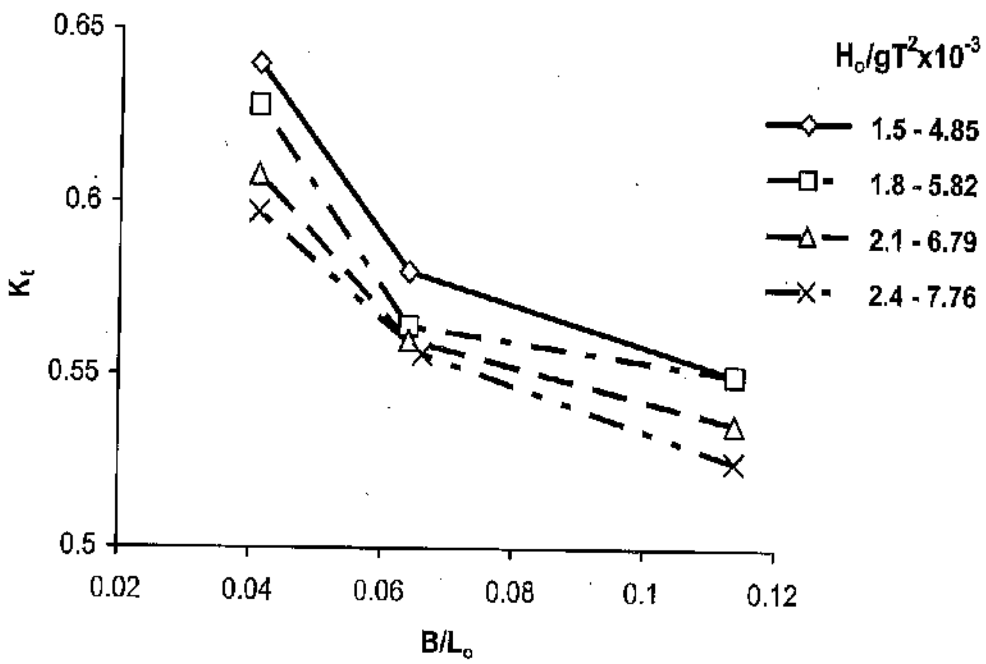

Fig. 8.46. Variation of K_t with B/L_o for d = 0.35m

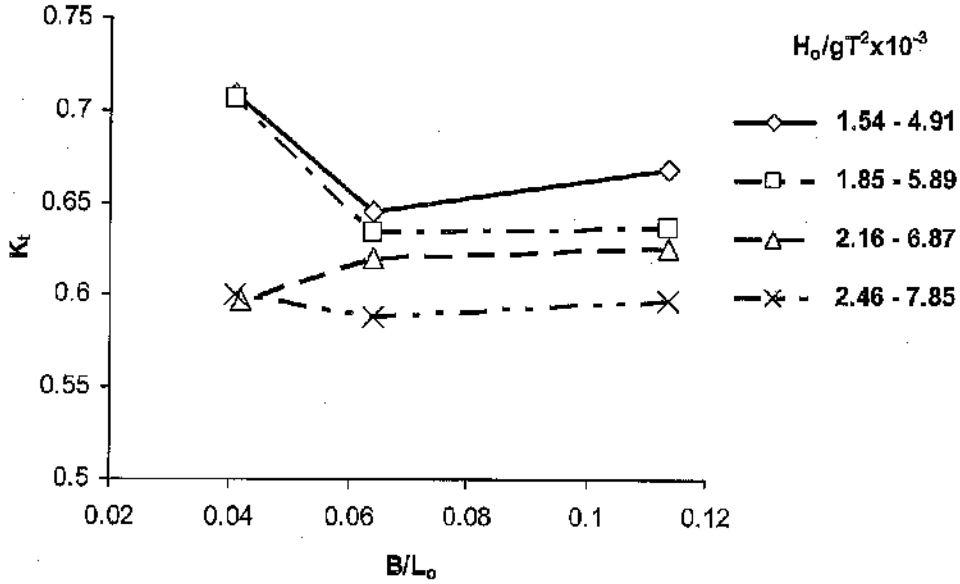

Fig. 8.47. Variation of K_t with B/L_o for d = 0.4m

8.3.4.2 Influence of deep water wave steepness on wave run up and run down

The influence of deep water wave steepness parameter ($H_o/(gT^2)$) on relative run up (R_u/H_o) and run down (R_d/H_o) for increasing ranges of depth parameter (d/gT^2), i.e. varying wave climate in depths of water of 0.3m, 0.35m and 0.4m, is shown by the best fit lines in Fig. 8.48 and Fig. 8.49 respectively.

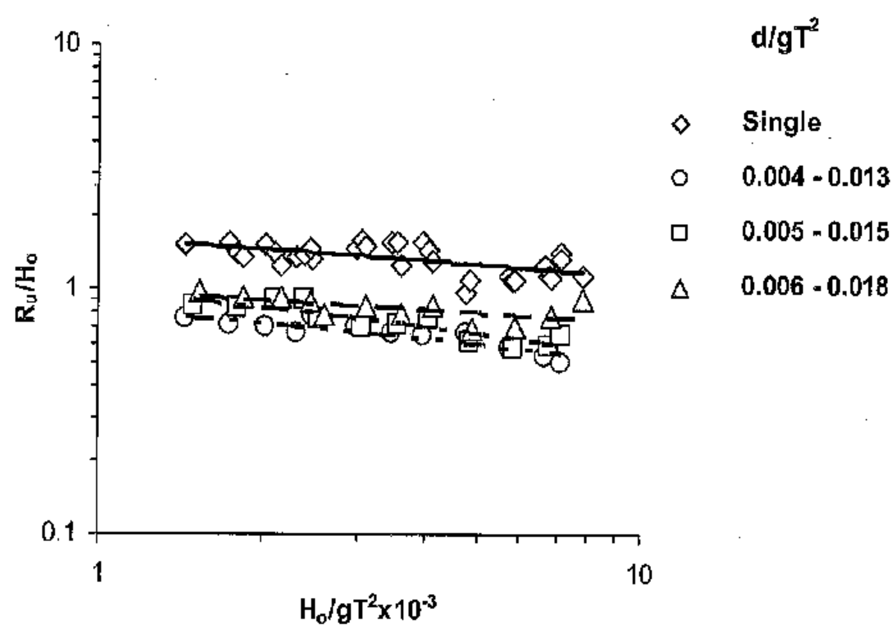

Fig. 8.48. Variation of R_u/H_o with H_o/gT^2

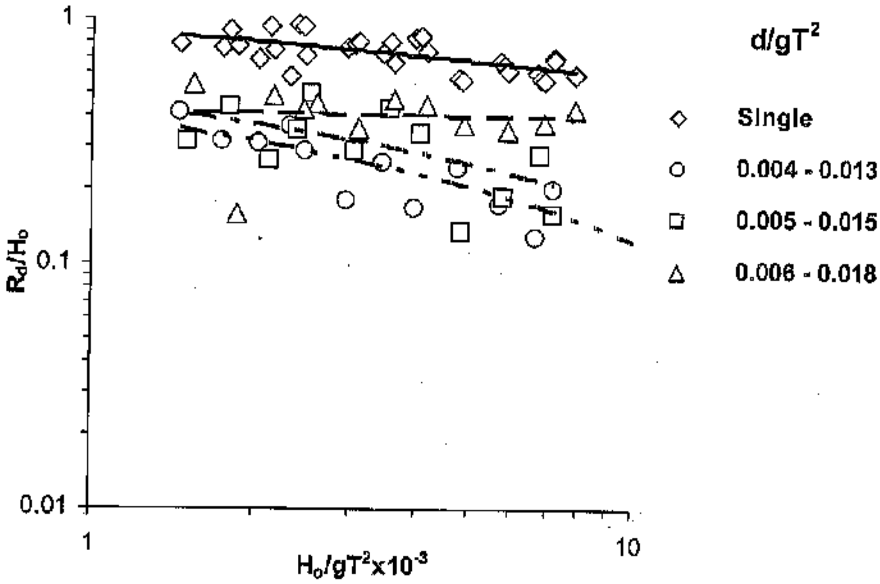

Fig. 8.49. Variation of R_d/H_o with H_o/gT^2

The maximum run up and run down are respectively 0.99 times and 0.53 times the deep water wave height. Considering all the ranges H_o/gT^2, relative run up for depths of water of 0.3m (i.e $0.004 < d/gT^2 < 0.013$ and $-0.312 < F/H_i < -0.5$), 0.35m (i.e. $0.005 < d/gT^2 < 0.015$ and $-0.625 < F/H_i < -1.0$) and 0.4m (i.e. $0.006 < d/gT^2 < 0.018$ and $-0.94 < F/H_i < -1.5$) are 51%, 42% to 47% and 36 to 38% lower than that for conventional breakwater. Similarly, for the same range of parameters, relative run down are 58% to 75%, 52 to 66% and 33% to 52% lower than that for conventional breakwater.

8.3.4.3 Influence of various parameters on damage level

8.3.4.3.1 Influence of deep water wave steepness

The trends of damage level (S) with varying wave steepness parameter (H_o/gT^2) for increasing ranges of depth parameter (d/gT^2) i.e. for increasing depths of water of 0.3m, 0.35m and 0.4m and different wave periods of 1.5sec, 2sec and 2.5sec are shown in Fig. 8.50. Damages at $0.004 < d/gT^2 < 0.013$ i.e. depth of water of 0.3m are nil. For $0.005 < d/gT^2 < 0.015$ and $1.5x10^{-3} < H_o/gT^2 < 7.76x10^{-3}$ (i.e. in depth of 0.35m), a maximum damage level (S) of 2.12 is observed for a wave period of 1.5sec while, for other wave periods damage is negligible. Similarly, for $0.006 < d/gT^2 < 0.018$ and $1.54x10^{-3} < H_o/gT^2 < 7.85x10^{-3}$ (i.e. in depth of 0.4m), noticeable damage levels (S) of 2.75 and 3.97 are observed only for the wave period of 1.5sec.

As the depth of water increased from 0.35m to 0.4m (i.e. 14.3%), for the wave period of 1.5sec, the maximum damage level S increases from 2.12 to 3.97 (i. e. a rise of 87.26%). Damages at depth of water of 0.35m and 0.4m are about 85.4% lower and 72.6% to 77.9% lower respectively when compared to those of the conventional breakwater.

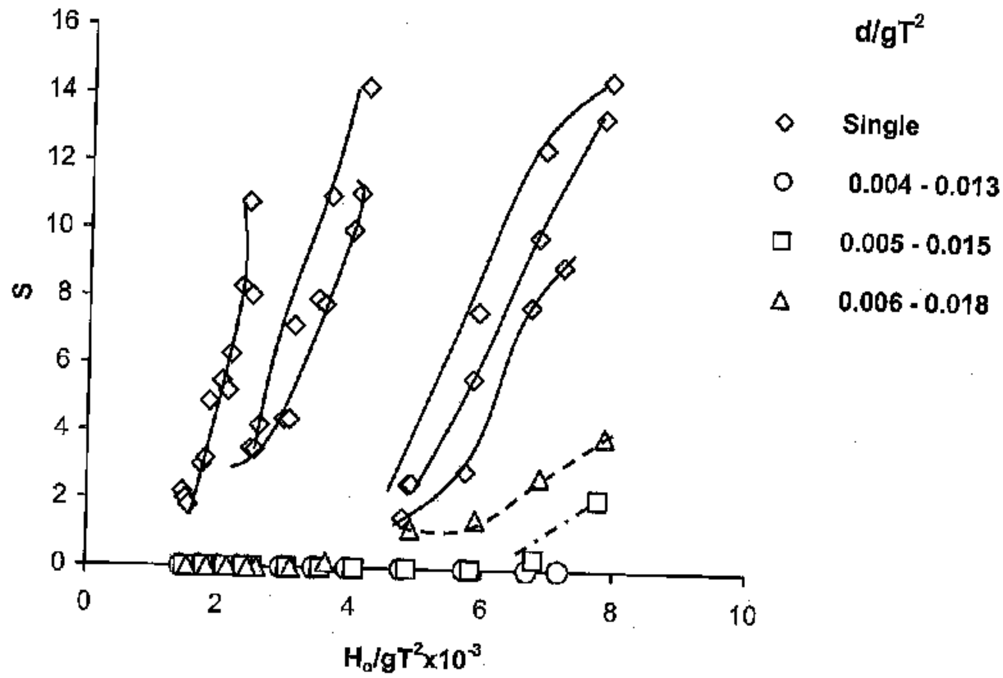

Fig. 8.50. Variation of S with H_o/gT^2

8.3.4.3.2 Influence of reef submergence

Fig. 8.51 shows variation of the damage level (S) with the reef submergence (F/H_i) for varying depth parameter (d/gT^2) i.e. for depths of 0.3m,0.35m and 0.4m and varying wave periods of 1.5sec, 2sec and 2.5sec. The impact of depth of water on stability is clearly visible from the figure with damages for depths of 0.3m, 0.35m and 0.4m are located at the right, centre and left side of the figure.

The damages to the inner (main) breakwater are nil for $0.004 < d/gT^2 < 0.013$ and $-0.312 < F/H_i < -0.5$ i.e. a depth of 0.3m, h/d = 0.833. For $0.005 < d/gT^2 < 0.015$ and $-0.625 < F/H_i < -1.0$ i.e. in a depth of 0.35m and h/d = 0.714, noticeable damage level of 2.12 occurs. And for $0.006 < d/gT^2 < 0.018$ and $-0.94 < F/H_i < -1.5$ i.e. in a depth of 0.4m h/d = 0.625, damage levels (S) of 2.75 and 3.97 are observed for wave heights of 0.14m and 0.16m of period 1.5sec while waves of height 0.1m and 0.12m cause negligible damage. As the depth of water increased from 0.35m to 0.4m (i.e. 14.3%), for the wave period of 1.5sec, the maximum damage level S increases from 2.12 to 3.97 (i. e. a rise of 87.26%).

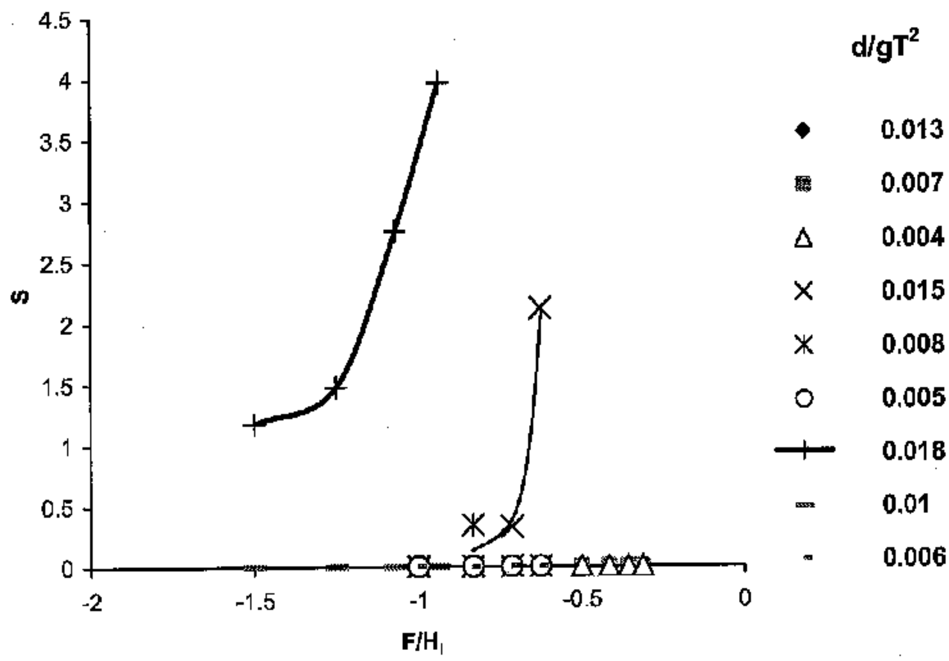

Fig. 8.51. Variation of S with F/H$_l$

8.3.4.3.3 Influence of reef crest width

Fig. 8.52 and Fig. 8.53 demonstrate the impact of reef crest width (B/L_o) on breakwater damage, for increasing ranges of H_o/gT^2 i.e. increasing wave heights of 0.1m to 0.16m of different periods of 1.5sec, 2sec and 2.5sec in water depths (d) of 0.35m and 0.4m respectively. The damage to the inner (main) breakwater for a depth of water of 0.3m is nil. It is seen that, the waves are more damaging with the increase in depth of water. In a depth of 0.35m, it is seen that the damage level of 2.12 occurs only for relatively higher range of wave steepness of $2.4\times10^{-3} < H_o/gT^2 < 7.76\times10^{-3}$ i.e. for a wave of 0.16m of period 1.5sec. Similarly, in a depth of 0.4m, noticeable damage levels (S) of 2.75 and 3.97 are observed for $2.16\times10^{-3} < H_o/gT^2 < 6.87\times10^{-3}$ and $2.46\times10^{-3} < H_o/gT^2 < 7.85\times10^{-3}$ i.e. for wave heights of 0.14m and 0.16m of period 1.5sec.

It can also be seen that, for both the depths, the damages are negligible up to a B/L_o value of 0.064 and then damage level increases indicating only steeper waves of period 1.5sec i.e short period waves are damaging the main breakwater. For increasing ranges of H_o/gT^2 in a depth of 0.35m, where, $0.04 < B/L_o < 0.114$, S rises from 0.0 to 2.12. Similarly, in a depth of 0.4m S increases from 0.0 to 3.97.

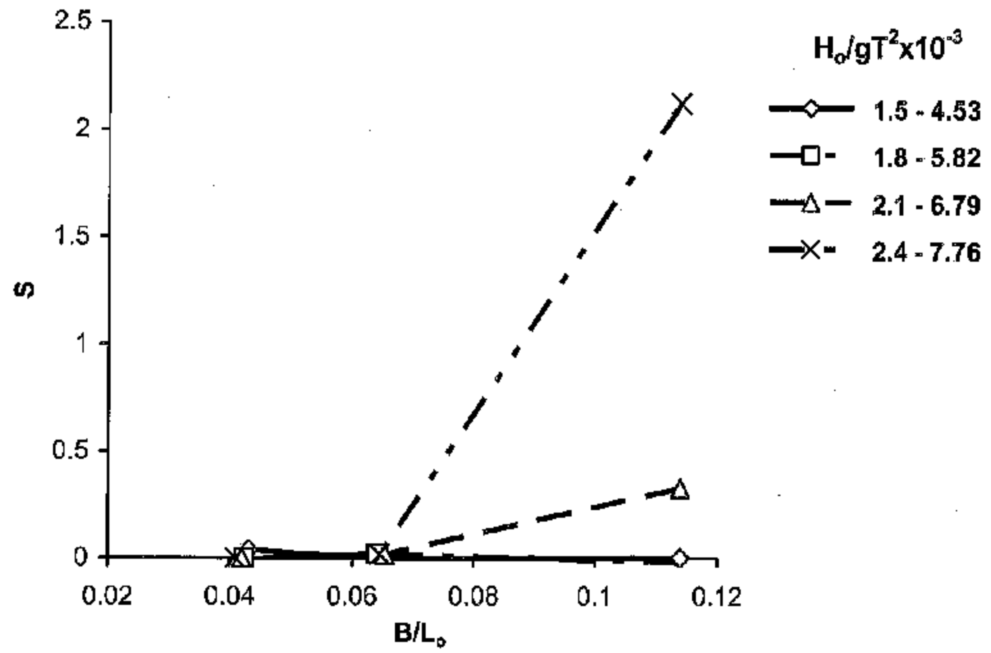

Fig. 8.52. Variation of S with B/L_0 for d = 0.35m

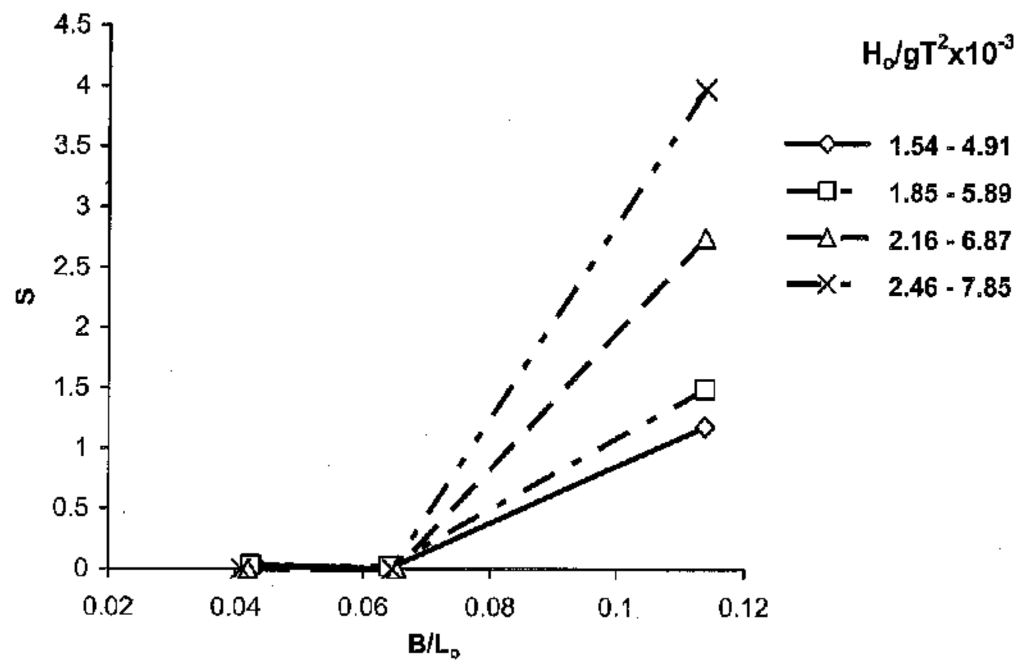

Fig. 8.53. Variation of S with B/L_0 for d = 0.4m

8.3.4.3.4 Influence of stability number

The damage level (S) of the breakwater increases with an increase in stability number (N_s) for $0.005 < d/gT^2 < 0.015$ and $0.006 < H_o/gT^2 < 0.018$ i.e. for depths of water (d) of 0.35m and 0.4m as shown in Fig. 8.54 and Fig. 8.55 respectively. Fig. 8.55 shows the best fit line.

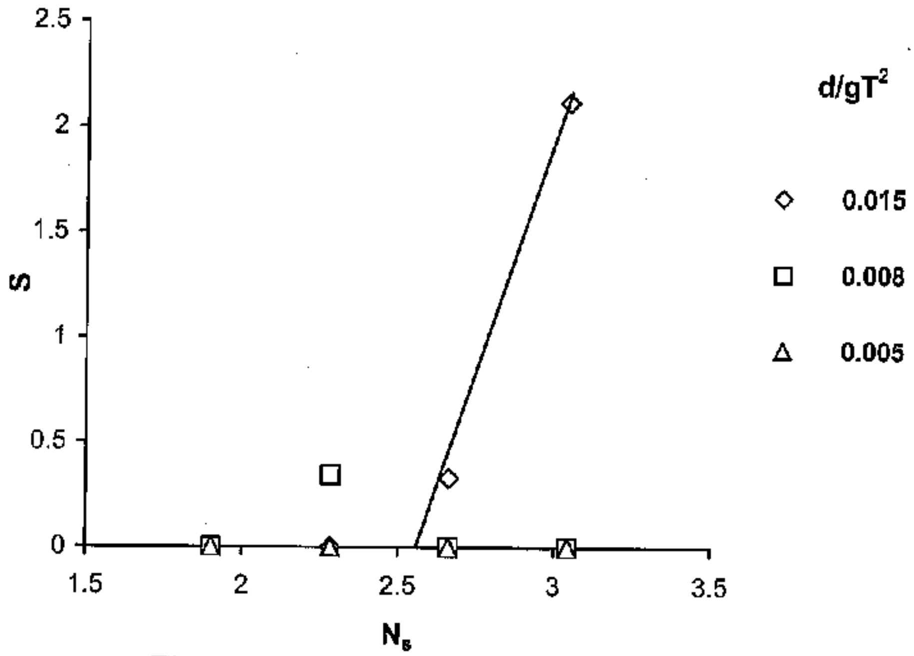

Fig. 8.54. Variation of S with N_s for d = 0.35m

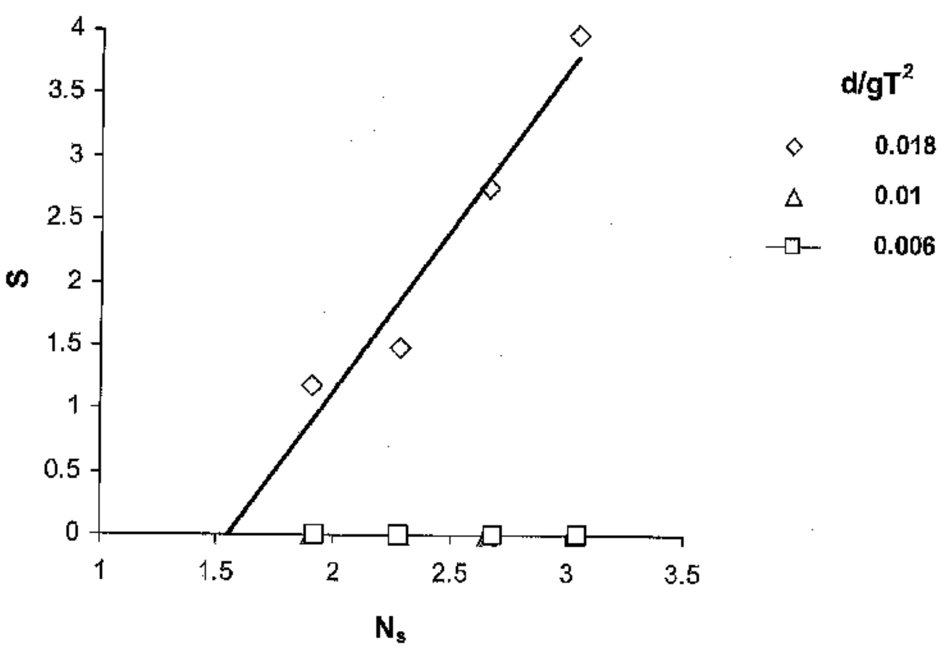

Fig. 8.55. Variation of S with N_s for d = 0.40m

The damage is observed only for waves of period of 1.5sec. The zero damage wave height (H_{zd}) for depths of water (d) of 0.35m and 0.4m are 0.158m and 0.1234m respectively and are 74% and 49.03% higher than those of the conventional breakwater.

8.3.4.3.5 Influence of surf similarity parameter on stability number

Fig. 8.56 shows the variation of zero damage stability number (N_{zd}) and surf similarity parameter (ξ) for increasing depths of water of 0.35m and 0.4m and wave period of 1.5sec i.e. ranges of depth parameter (d/gT^2). The results of the present study are compared with those of Thompsen et al. (1972). For the present study, it is observed that minimum breakwater stability (i.e. $N_{zd} = 2.32$) occurs for a ξ value of 2.68 in a depth of water of 0.4m.

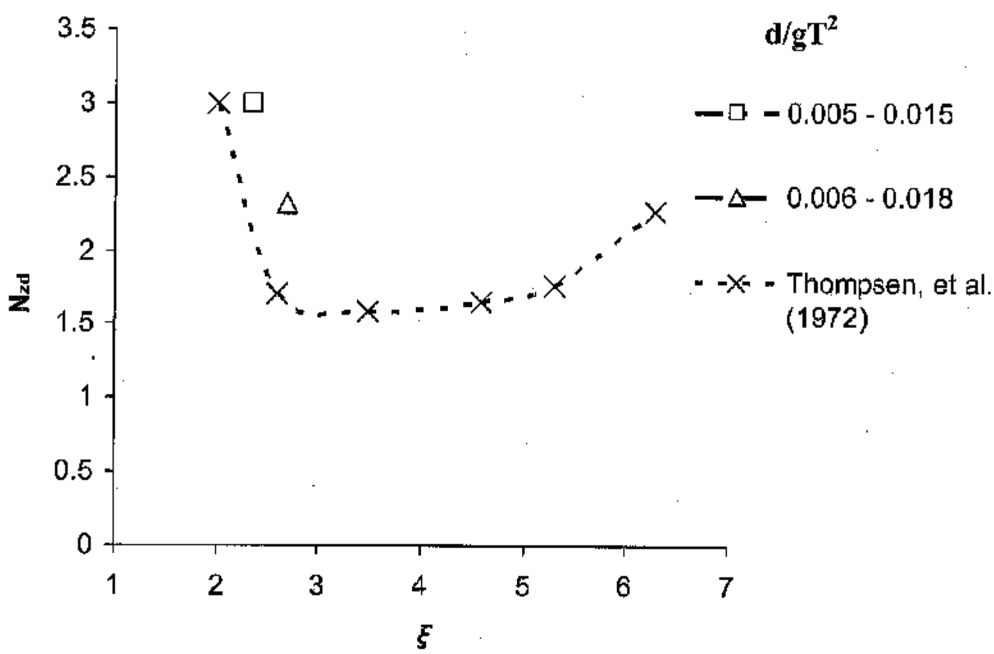

Fig. 8.56. Variation of N_{zd} with ξ

8.3.4.4 Conclusions

From the experimental investigation of the present geometry of protected breakwater, following conclusions are drawn.

1. The transmission coefficient (K_t) decreases with increase in H_o/gT^2 and h/d and decreases with decrease in F/H_i and d/gT^2.

2. K_t decreases from 0.55 to 0.38 (30.9%), 0.64 to 0.53 (17.19%),) and 0.708 to 0.55 (22.32%),) for h/d of 0.833, 0.714 and 0.625 i.e. for depths of water of 0.3m, 0.35m and 0.4m respectively for $1.45 \times 10^{-3} < H_o/gT^2 < 7.85 \times 10^{-3}$ and $0.04 < B/L_o < 0.0114$.
3. Kt for all the wave periods and is represented by d'Angremond (1996) with an accuracy of 16%. The K_t values are 37.5% to 48.3%, 16.67% to 31%, and 21% to 28.6% lower compared to Cox and Clark (1992), Van der Meer and d'Angremond (1992) and Cornett et al. (1993) respectively.
4. For $-0.312 < F/H_i < -1.5$ and all the depths of water, K_t increases from 0.38 to 0.69 (81.6%), 0.4 to 0.65 (62.5%) and 0.42 to 0.708 (68.6%) for wave periods of 1.5sec, 2sec and 2.5sec respectively.
5. K_t decreases with an increase in B/L_o and with an increase in range of H_o/gT^2. But this is clearly seen only for depth of water of 0.35m.
6. K_t varies between 0.38 and 0.708 for the test parameters considered.
7. The maximum run up and run down are respectively 0.99 times and 0.53 times the deep water wave height.
8. Relative run up and run down are 36% to 51% and 33% to 75% lower compared to that of the conventional breakwater.
9. The damage level S increases with the increase in H_o/gT^2, F/H_i, d/gT^2, B/L_o and decrease in h/d.
10. Damages at $0.004 < d/gT^2 < 0.013$ (i.e. depth of water of 0.3m and h/d of 0.833) are nil.
11. For $0.005 < d/gT^2 < 0.015$, $1.5 \times 10^{-3} < H_o/gT^2 < 7.76 \times 10^{-3}$ and $-0.625 < F/H_i < -1.0$ (i.e. h/d of 0.714 and for a wave height of 0.16m of period 1.5sec in depth of 0.35m), a maximum damage level (S) of 2.12 is observed while, for other wave periods damage is negligible. Similarly, for $0.006 < d/gT^2 < 0.018$, $1.54 \times 10^{-3} < H_o/gT^2 < 7.85 \times 10^{-3}$ and $-0.94 < F/H_i < -1.5$ (i.e. for h/d of 0.625 and wave heights of 0.14m and 0.16m of period 1.5sec in depth of 0.4m), noticeable damage levels (S) of 2.75 and 3.97 are observed.
12. For increasing ranges of H_o/gT^2 in a depth of 0.35m, where, $0.04 < B/L_o < 0.114$, S rises from 0.0 to 2.12. Similarly, in a depth of 0.4m S increases from 0.0 to 3.97.
13. As the depth of water increased from 0.35m to 0.4m (i.e. $-0.625 < F/H_i < -1.0$ increased to $-0.94 < F/H_i < -1.5$), for the wave period of 1.5sec, the maximum damage level S increases from 2.12 to 3.97 (i. e. a rise of 87.26%).

14. Noticeable breakwater damage levels of 2.12 and 2.75 and 3.97 occur for depths of 0.3m and 0.4m which are 85.4% lower and 72.6% to 77.9% lower respectively when compared to those of the conventional breakwater.
15. The zero damage wave height (H_{zd}) for depths of water (d) of 0.35m and 0.4m are 74% and 49.03% higher than those of the conventional breakwater.
16. For the present study, it is observed that minimum breakwater stability (i.e. N_{zd} = 2.32) occurs for a ξ value of 2.68 in a depth of water of 0.4m.

8.3.5 SUMMARY OF CONCLUSIONS FOR PROTECTED BREAKWATER

Table. 8.1 compares the conclusions of present model tests and conventional breakwater. It is seen from the table that the damage of the conventional breakwater reduced when a protective submerged reef is located at a distance (X) of 2.5m seaward of the breakwater. As the crest width (B) of the reef increased from 0.1m to 0.3m the damage of the main (conventional) breakwater reduced to zero. Further, when the reef crest width is increased to 0.4m, the waves damage the breakwater. The submerged reef of crest width 0.3m located at 2.5m seaward of the main breakwater completely protects it from the waves generated in the present experimental investigation.

Fig. 8.57 and Fig. 8.58 show the influence of relative reef crest width (B/L_o and B/d) on maximum damage level (S) of the defenced breakwater for different relative crest heights (h/d) of a submerged reef of crest width 0.3m located at a seaward distance of 2.5m. It can be seen from the graphs that the reef of relative crest widths B/L_o of 0.035 to 0.045 and B/d of 0.6 to 0.75, the damage levels are zero and the reef completely protects the main breakwater.

Table. 8.1. Comparison of conclusions for conventional breakwater (CBW) and protected breakwater

Sl. No	Parameter	Conventional breakwater (CBW)	Protected breakwater (PBW) with a seaward submerged reef of varying crest width B located at X = 2.5m (i.e. X/d of 6.25 to 8.33)			
			B			
			0.1m	0.2m	0.3m	0.4m
			B/d			
			0.25 –0.33	0.50 –0.67	0.75 –1.0	1.0 –1.33
1.	Max. R_u/H_o	1.57	1.24	1.06	0.97	0.99
2.	Present R_u/H_o compared with that of CBW (% Llower)	----------	35% - 38.6%	37.5 - 48.7%	36% - 55%	36% - 51%
3.	Max. R_d/H_o	0.95	0.73	0.54	0.44	0.53
4.	Present R_d/H_o compared with that of CBW (% Lower)	----------	6% - 30.8%	38.5% - 75%	51% - 73%	33% - 75%
5.	Transmission coefficient K_t	----------	0.66 - 0.84	0.43 - 0.887	0.40 - 0.80	0.38- 0.708
6.	K_t at d = 0.30m d = 0.35m d = 0.40m	----------	0.66 - 0.76 0.66 - 0.79 0.68 - 0.84	0.43 - 0.57 0.53 - 0.75 0.60 - 0.887	0.4 - 0.55 0.48 - 0.62 0.48 - 0.80	0.38 - 0.55 0.53 - 0.64 0.55- 0.708
7.	K_t at T = 1.5sec T = 2.0sec T = 2.5sec	----------	0.64 - 0.71 0.696 - 0.8 0.71 - 0 .84	0.43 - 0.7 0.43 - 0.72 0.49 - 0.887	0.43 - 0.71 0.40 - 0.70 0.41 - 0.80	0.38 - 0.69 0.40 – 0.65 0.42–0.708
8.	Max. damage level at d = 0.30m d = 0.35m d = 0.40m	8.3 - 10.0 10.8 - 13.4 8.05 - 14.5	Nil 3.58 - 6.28 5.31 - 8.7	Nil Negligible 0 - 3.4	Nil Nil Nil	Nil 0 - 2.12 0 - 3.97
9.	Max. damage level at T = 1.5sec T = 2.0sec T = 2.5sec	9.0 - 14.5 10.0 - 14.23 8.3 - 10.8	6.28- 8.7 4.94 - 7.06 3.58 - 5.31	0 - 3.4 Negligible Negligible	Nil Nil Nil	0- 3.97 Nil Nil
10.	Present damage level S compared with that of CBW (% Lower)	----------	40 - 66	75.42 – 76.55	No damage (PBW is totally safe)	72.6 - 85.4
11.	Present H_{sd} compared with that of CBW (% Higher)	----------	8.7% - 75%	60.38%	PBW is totally safe	49% -74%

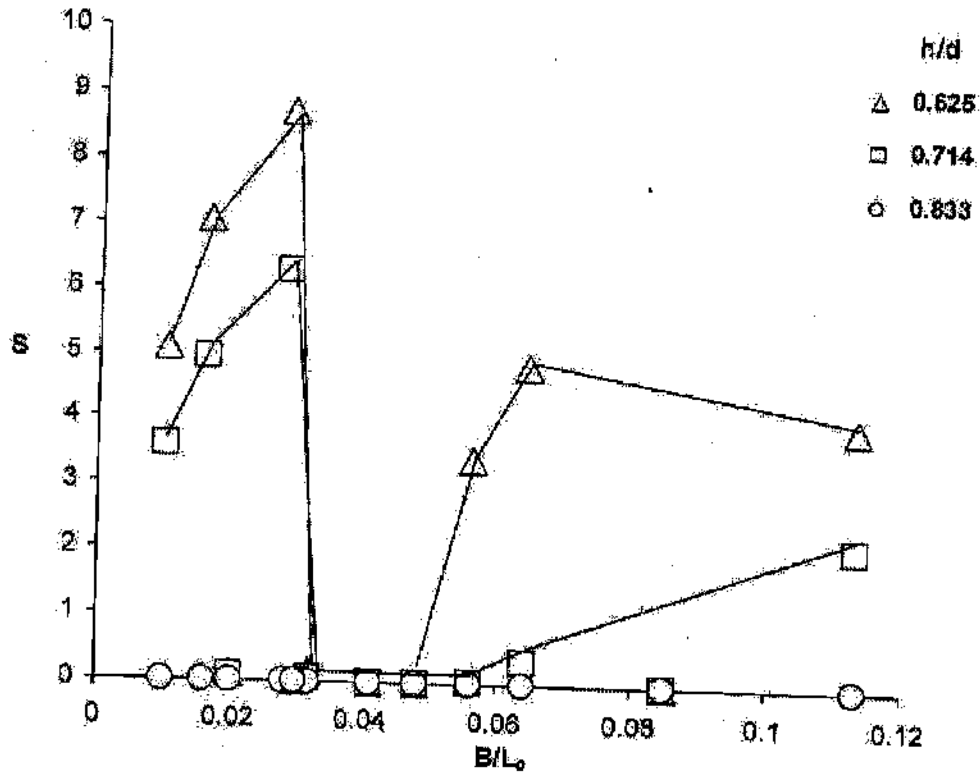

Fig. 8.57. Variation of maximum damage level with B/L_o

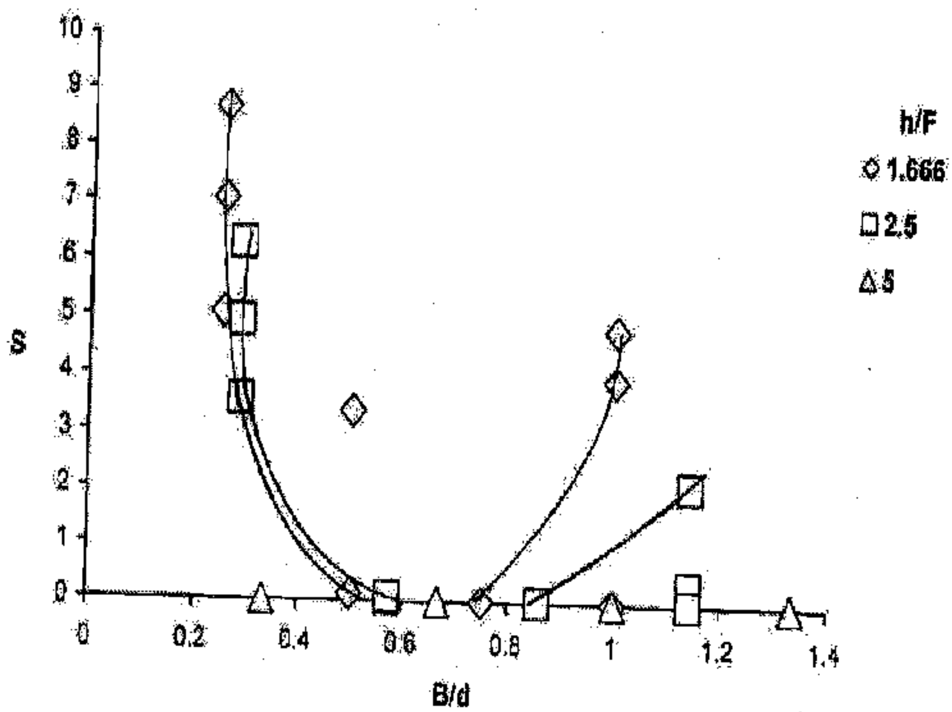

Fig. 8.58. Variation of maximum damage level with B/d

Chapter 9
Investigation of Protected Breakwater with a Reef at a Spacing of 4.0m

9.1 GENERAL

The submerged reef of crest width 0.3m located at 2.5m seaward of the main breakwater completely protects it from the waves generated in the present experimental investigation. Hence, it is decided to conduct the experiment with a reef of narrow crest width of 0.1m and 0.2m located seaward of the main breakwater at a farther distance (X) of 4m and investigate its efficacy as a protective structure to the main breakwater against the above model geometry.

This model section of the protected breakwater is tested for regular waves and impact of waves (i.e. wave steepness) on wave transmission at reef and on run up, run down, and stability of armour of the inner main breakwater is studied.

9.2 DETAILS OF PHYSICAL MODEL STUDY

A 1:30 scale model of a breakwater, of trapezoidal cross section with a uniform slope of 1V:2H is constructed, at 32m from the generator flap, on the flat bed of the flume with primary armour stone weight of 73.2gms. A stable trapezoidal submerged reef having a slope of 1V:2H with a height (h) of 0.25m and different crest widths (B) of 0.1m, and 0.2m is constructed, with homogeneous pile of stones of 30gms weight (i.e. nominal diameter, d_{n50} of 0.0221m), on the seaward side of the main breakwater at a distance (X) of 4m (i.e. X/d of 10.0 to 13.33). Further, the details of breakwater model construction are explained in Chapter 5 and the model characteristics are listed in Table 5.3.

Before the model tests are started, the experimental set up along with the wave probes is calibrated. The model is subjected to regular waves of height varying from 0.1m to 0.16m of a range of periods from1.5sec to 2.5sec generated in water depths of 0.3m to 0.4m. Further, the details of breakwater test procedure are explained in Chapter 5.

9.3 ANALYSIS AND INTERPRETATION OF DATA

The data collected in the present experimental work is expressed in non-dimensional quantities. The variation of transmission coefficient (K_t), relative run up (R_u/H_o) and run down (R_d/H_o), damage level (S) etc., for varying parameters like steepness $H_o/(gT^2)$ are studied through graphs with respect to changing relative depth $d/(gT^2)$, reef width (B/d) etc. The relationship between the parameters is analysed through the graphs.

9.3.1 Protected breakwater with a reef of crest width (B) of 0.1m (i. e. B/d = 0.25 to 0.33)

9.3.1.1 Influence of various parameters on transmission coefficient

9.3.1.1.1 Influence of deep water wave steepness

Fig. 9.1 shows the best fit lines for the variation of transmission coefficient K_t with the deep water wave steepness parameter (H_o/gT^2) for varying relative reef crest heights (h/d). K_t decreases with an increase in H_o/gT^2 and increase in relative reef height (h/d). K_t drops from 0.602 to 0.51 (15.8%), 0.67 to 0.56 (16.4%) and 0.72 to 0.59 (18%) for h/d of 0.833, 0.714 and 0.625 i.e. for depths of water (d) of 0.3m, 0.35m and 0.4m respectively. This indicates that the wave height attenuation (i.e. WHA = 1- K_t) achieved is 28% to 49%.

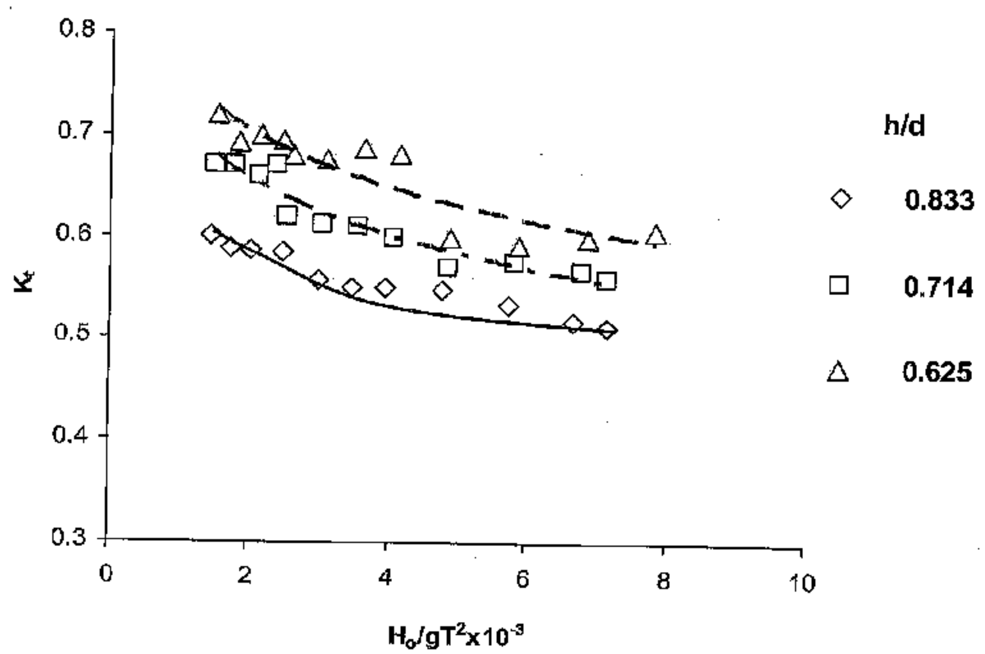

Fig. 9.1. Variation of K_t with H_o/gT^2

9.3.1.1.2 Influence of relative reef submergence

The transmission coefficient K_t increases as relative reef submergence (F/H_i) and range of depth parameter (d/gT^2) increase for wave periods of 1.5sec, 2sec and 2.5sec as shown in Fig. 9.2, Fig. 9.3 and Fig. 9.4 respectively. For above wave periods and $-0.312 < F/H_i < -1.5$, K_t increases from 0.51 to 0.606 (18.8%), 0.55 to 0.68 (23.6%) and 0.587 to 0.72 (22.6%) respectively. For the range of $-0.312 < F/H_i < -1.5$ and for a wave period of 1.5sec, the present K_t values are 31% to 40% lower, 10% to 26% lower, 33% lower than values of Cox and Clark (1992), Van der Meer and d'Angremond (1992), Cornett et al (1993) respectively and represented by criteria of d'Angremond (1996) with an accuracy of 20%. For same range of F/H_i and for a period of 2sec, the present K_t values are 29% to 36% lower, 6% to 18% lower, 24% lower and 0 to 16% higher than those given by Cox and Clark (1992), Van der Meer and d'Angremond (1992), Cornett et al (1993) and d'Angremond (1996) respectively. Similarly, for a period of 2.5sec, K_t values are 25% to 31% lower, 0% to 11% lower, 22% lower and up to 22% higher than those given by Cox and Clark (1992), Van der Meer and d'Angremond (1992), Cornett et al (1993) and d'Angremond (1996) respectively.

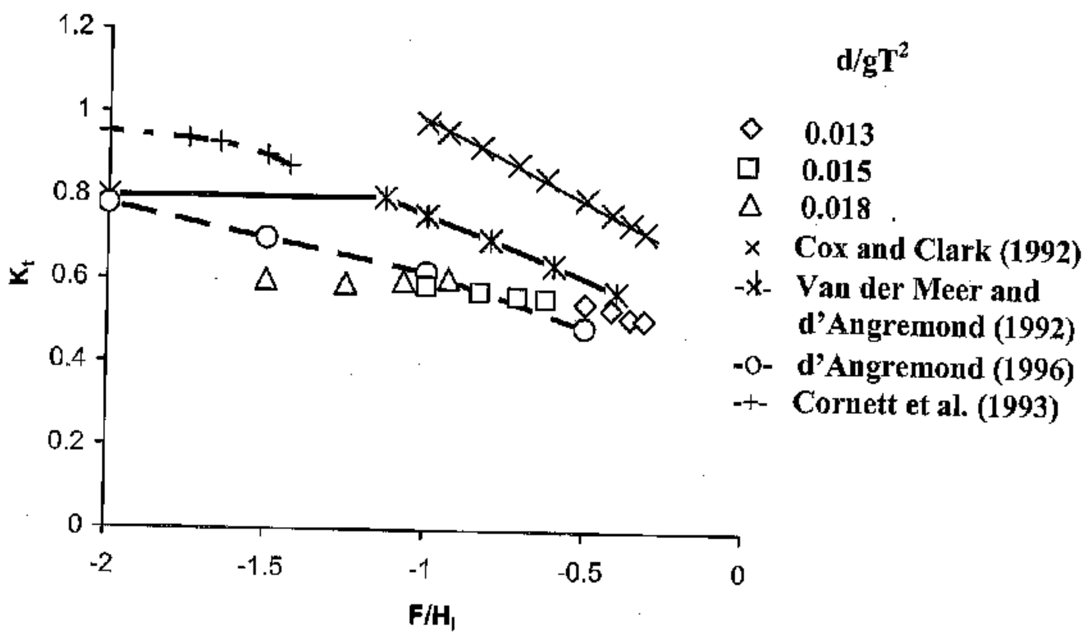

Fig. 9.2. Variation of K_t with F/H_i for T =1.5 sec

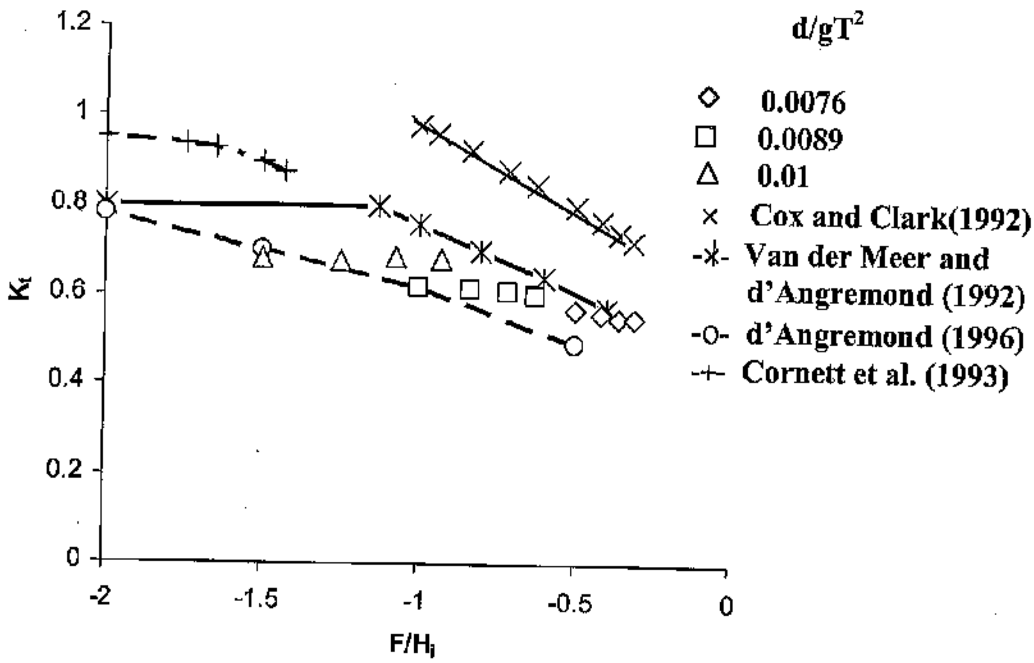

Fig. 9.3. Variation of K_t with F/H_i for T = 2 sec

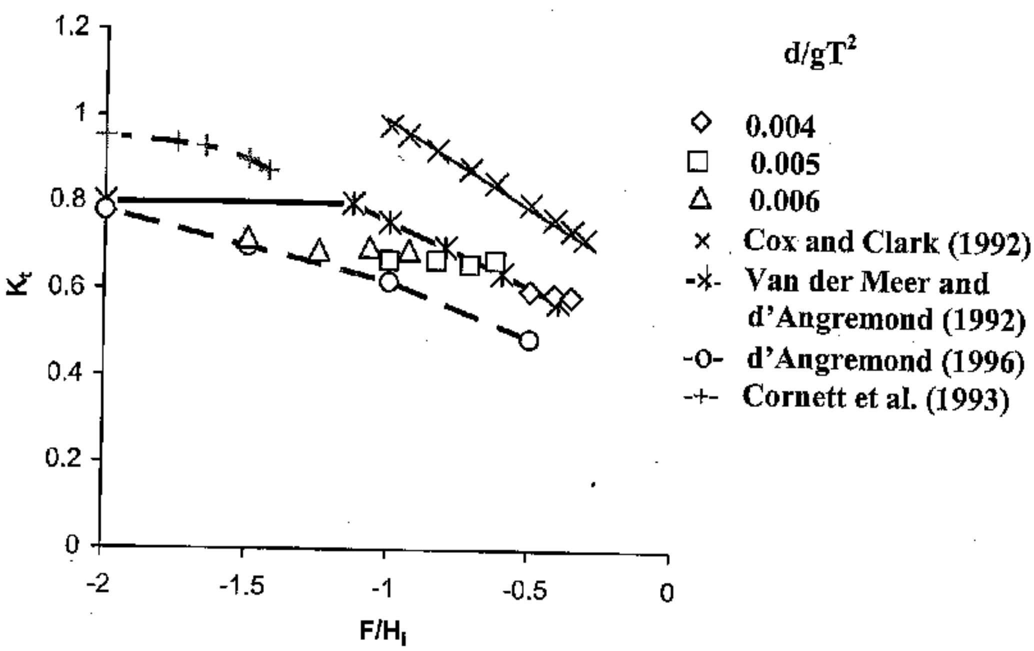

Fig. 9.4. Variation of K_t with F/H_i for T = 2.5sec

It can be seen from the figures that, for any given period, the K_t values are approximately constant for a given depth. But for a given period, the K_t values gradually increase with the depth.

9.3.1.1.3 Influence of relative reef crest width

Each of Fig. 9.5, Fig. 9.6 and Fig. 9.7 shows the decreasing trend of K_t with increasing relative crest width (B/L_o) and increasing ranges of wave steepness parameter (H_o/gT^2) (i.e. for increasing wave heights of 0.1m, 0.12m 0.14m and 0.16m and periods 1.5sec, 2sec and 2.5sec) for depths of water (d) of 0.3m, 0.35m and 0.4m respectively. As $0.01 < B/L_o < 0.0285$, the slope of trend lines of K_t changes at B/L_o of 0.016 and K_t values decrease from 0.602 to 0.51 (15.8%), 0.67 to 0.56 (16.4%) and 0.72 to 0.59 (18%) for h/d of 0.833, 0.714 and 0.625 i.e. for depths of water (d) of 0.3m, 0.35m and 0.4m respectively. As the depth increases and for $0.01 < B/L_o < 0.0285$, K_t rises and the influence of steepness parameter gradually reduces. And at a depth of 0.4m, the decreasing trend of K_t with the increase of B/L_o is almost similar for all ranges of H_o/gT^2.

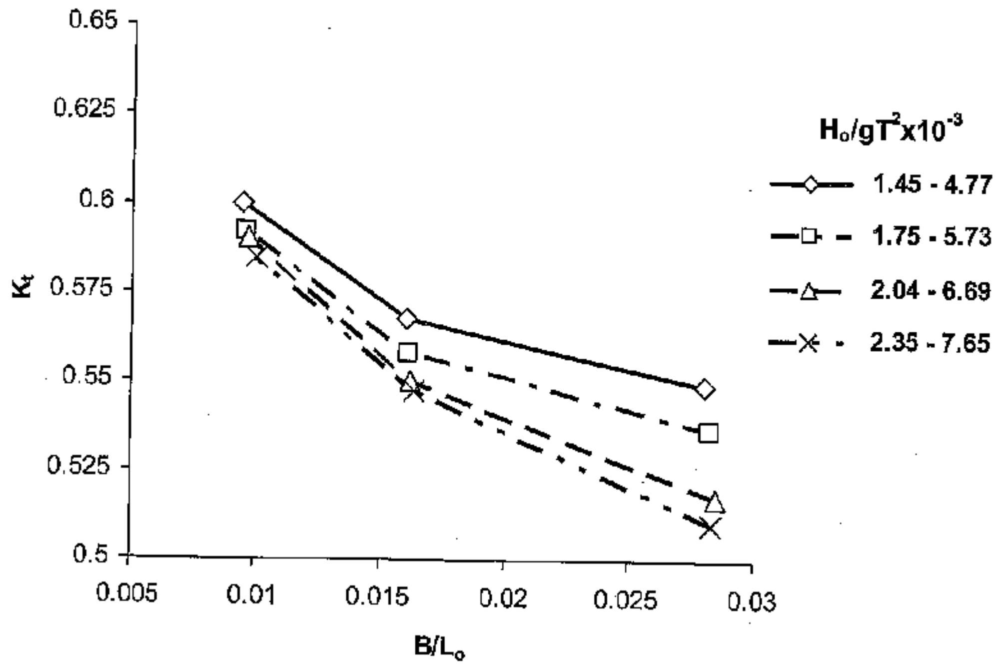

Fig. 9.5. Variation of K_t with B/L_o for d = 0.3m

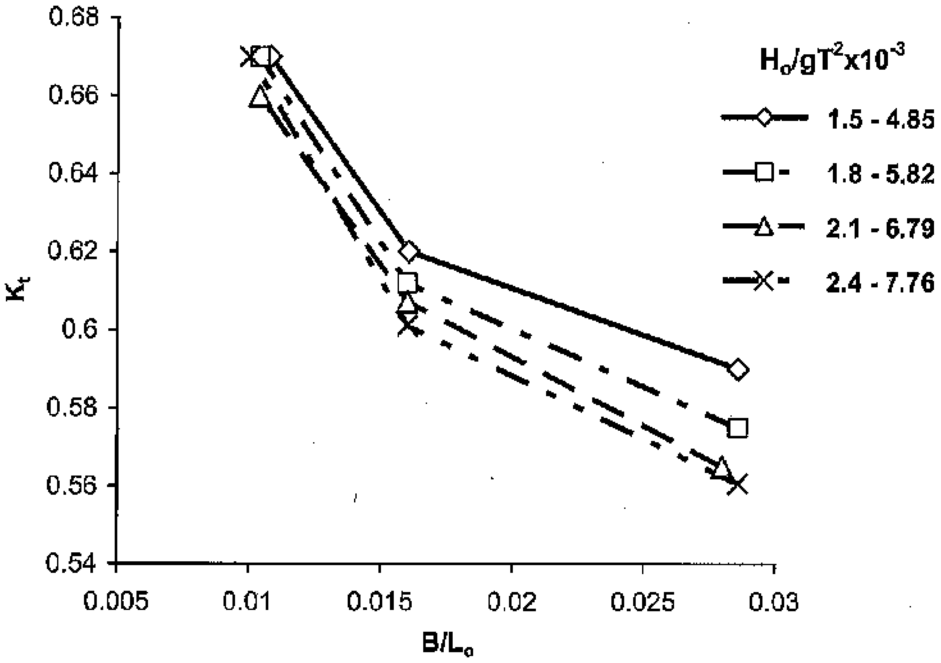

Fig. 9.6. Variation of K_t with B/L_o for d = 0.35 m

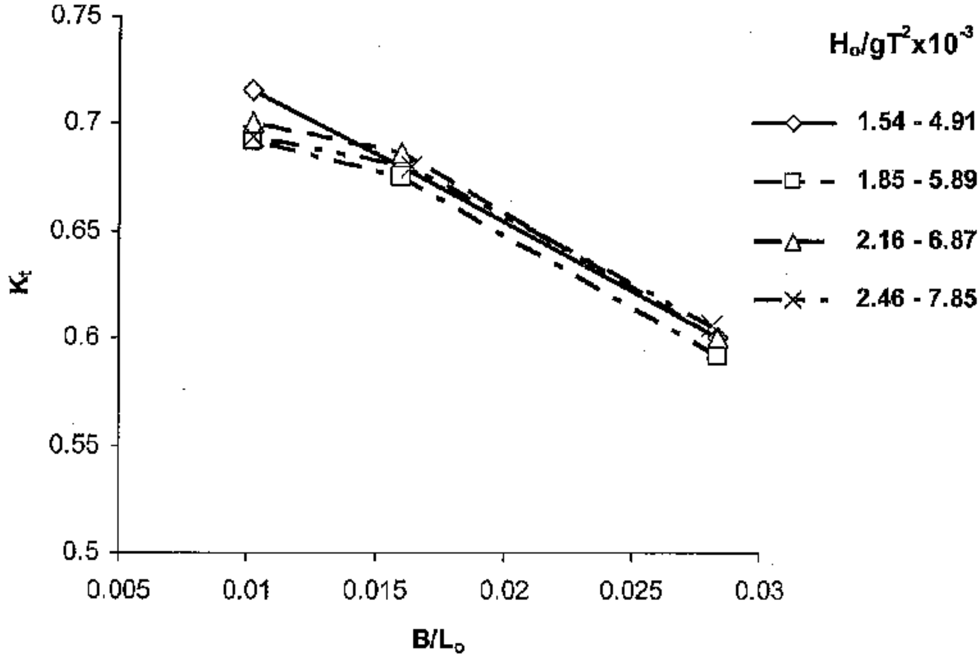

Fig. 9.7. Variation of K_t with B/L_o for d = 0.4 m

9.3.1.2 Influence of deep water wave steepness on wave run up and run down

The influence of deep water wave steepness parameter ($H_o/(gT^2)$) on relative run up (R_u/H_o) and run down (R_d/H_o), for increasing ranges of depth parameter (d/gT^2) i.e. varying wave climate in depths of water of 0.3m, 0.35m and 0.4m is shown in Fig. 9.8 and Fig. 9.9 respectively. The graphs show the best fit lines.

Both the relative run up and the run down, decrease with an increase in wave steepness and decrease in the depth i.e. range of depth parameter. The results are compared with those of the conventional breakwater. From the figures, it is observed that, R_u/H_o and R_d/H_o increase with the range of d/gT^2 and F/H_i.

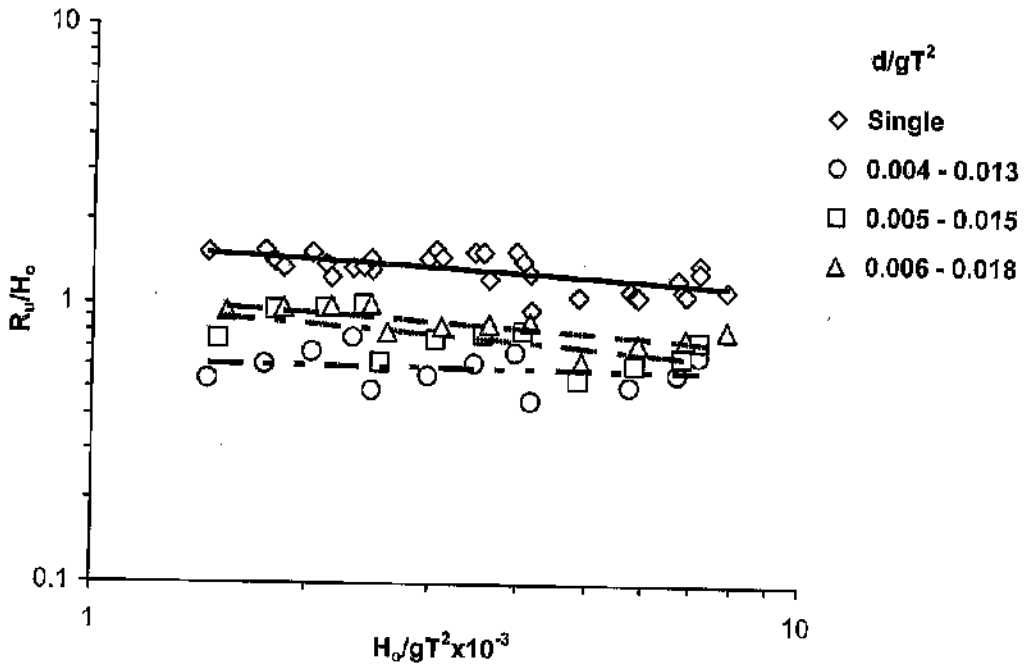

Fig. 9.8. Variation of R_u/H_o with H_o/gT^2

The maximum relative run up and run down are respectively 1.0 times and 0.65 times the deep water wave height for the range of variables considered in the present study. Relative run up for depths of water of 0.3m, 0.35m and 0.4m (i.e. when F/H_i varies in ranges of -0.312 to -0.5, -0.62 to -1.0 and -0.94 to -1.5) are respectively 47% to 61%, 42% and 33% to 36% lower than those for a conventional single breakwater. Similarly relative rundown for depths of water of 0.3m, 0.35m and 0.4m are respectively 41% to 49%, 16% to 39% and 15% to 30% lower than those for conventional breakwater.

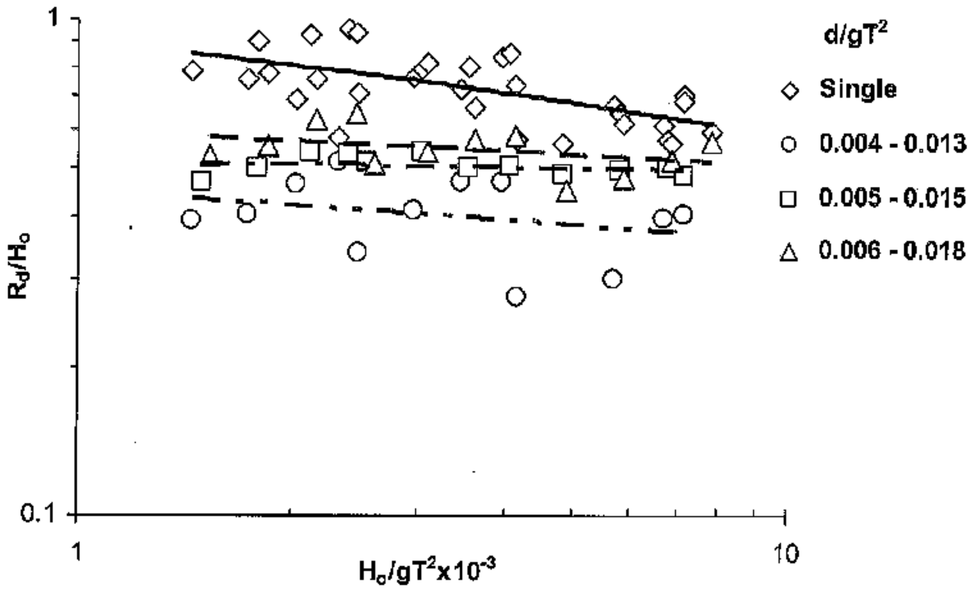

Fig. 9.9. Variation of R_d/H_o with H_o/gT^2

9.3.1.3 Influence of various parameters on damage level

9.3.1.3.1 Influence of deep water wave steepness

Fig. 9.10 shows increasing damage level (S) with increasing wave steepness parameter (H_o/gT^2) and increasing ranges of depth parameter (d/gT^2) i.e. for depths of water of 0.3m, 0.35m and 0.4m and different wave periods. The results of the present study are compared with those of the conventional breakwater. No damages are observed for $0.004 < d/gT^2 < 0.013$ i.e. depth of 0.3m. The breakwater damage increases with an increase in range of depth parameter (d/gT^2) i.e. depth of water of 0.35m and 0.4m and it decreases with the decrease in wave period for a given depth. The impact of wave period is clearly distinguishable as damages are grouped from right to left for the increasing period of 1.5sec, 2sec and 2.5sec.

For $0.005 < d/gT^2 < 0.015$ (i.e. depth of 0.35m) and $1.5x10^{-3} < H_o/gT^2 < 7.76x10^{-3}$, damage level (S) are 3.23, 2.26 and 2.1 for wave periods of 1.5sec, 2sec and 2.5sec respectively. Similarly, For $0.006 < d/gT^2 < 0.018$ (i.e. depth of 0.4m) and $1.54x10^{-3} < H_o/gT^2 < 7.85x10^{-3}$, damage level (S) are 5.35, 4.59 and 3.23 for wave periods of 1.5sec, 2sec and 2.5sec respectively. As the water depth increases from 0.35m to 0.4m (i.e.14.3%), the maximum damage level S for the wave period of 1.5sec, increases from 3.23 to 5.35 (i. e. a rise of 65.6%), and it increases from 2.26 to 4.59 (i. e. a rise of 103%) for the wave period of 2.0sec while maximum damage level S rises from 2.1 to 3.23 (i.e. a rise of 53.8%) for a wave period of 2.5sec.

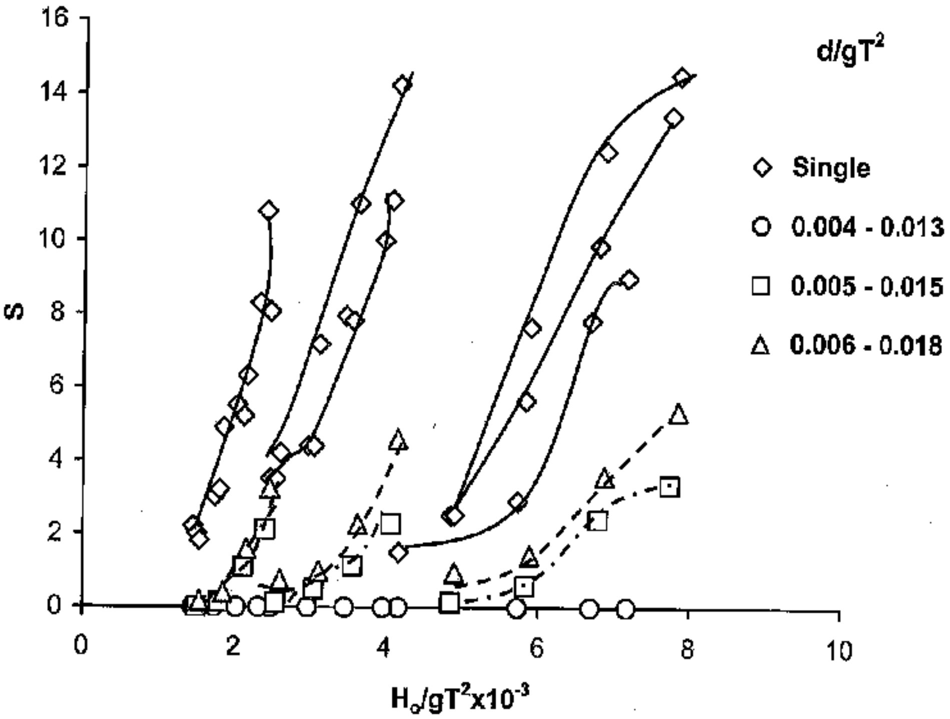

Fig. 9.10. Variation of S with H_o/gT^2

9.3.1.3.2 Influence of reef submergence

The graphs in Fig. 9.11 show an increasing damage level (S) with the reef submergence (F/H_i) for varying depth parameter (d/gT^2). This is because reduced wave breaking. Also we can see that influence of depth of water and wave period is clearly discernible, where, it shows increasing damage with shorter period waves for a given depth. Damages are nil for $0.004 < d/gT^2 < 0.013$ and $-0.312 < F/H_i < -0.5$ i.e. for d= 0.3m, h/d = 0.833. For $0.005 < d/gT^2 < 0.015$ and $-0.625 < F/H_i < -1.0$ i.e. for d= 0.35m, h/d = 0.714, damage levels, which are shown in middle of the figure, the maximum value varies from 2.1 to 3.23 (i. e. a rise of 53.8%) and for $0.006 < d/gT^2 < 0.018$ and $-0.94 < F/H_i < -1.5$ i.e. for d= 0.4m and h/d = 0.625, the maximum value increases from 3.23 to 5.35 (i. e. by 65.6%) as shown at left of the figure.

9.3.1.3.3 Influence of reef crest width

Fig. 9.12 and Fig. 9.13 demonstrate the impact of reef crest width (B/L_o) on damage level of the breakwater, for increasing ranges of H_o/gT^2,

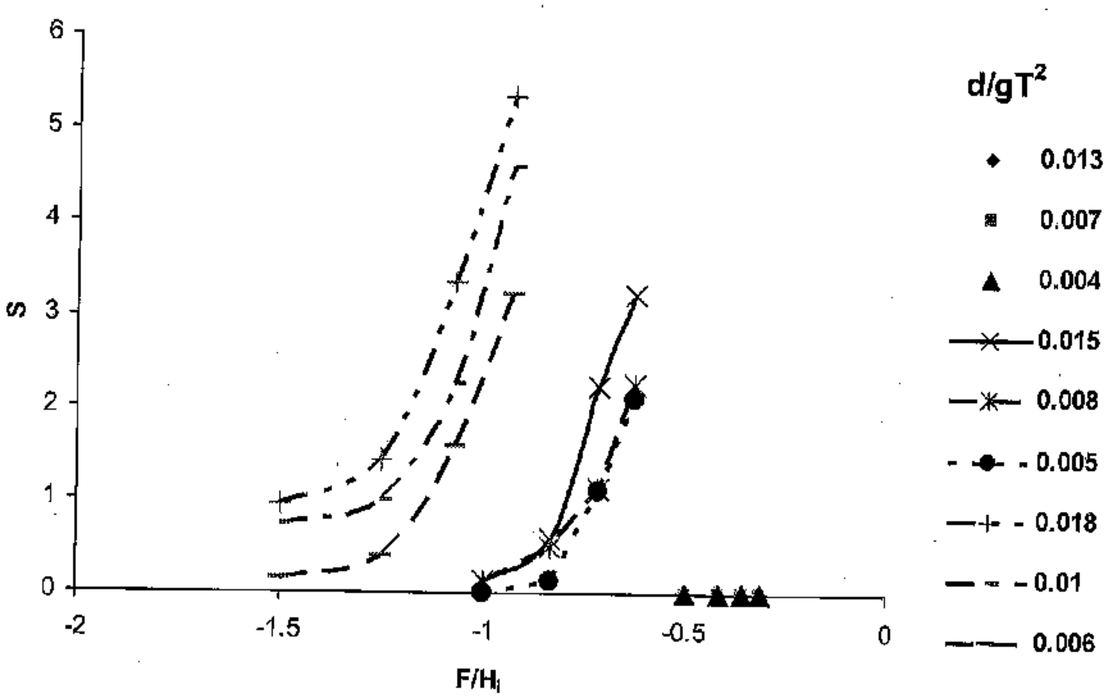

Fig. 9.11. Variation of S with F/H_i

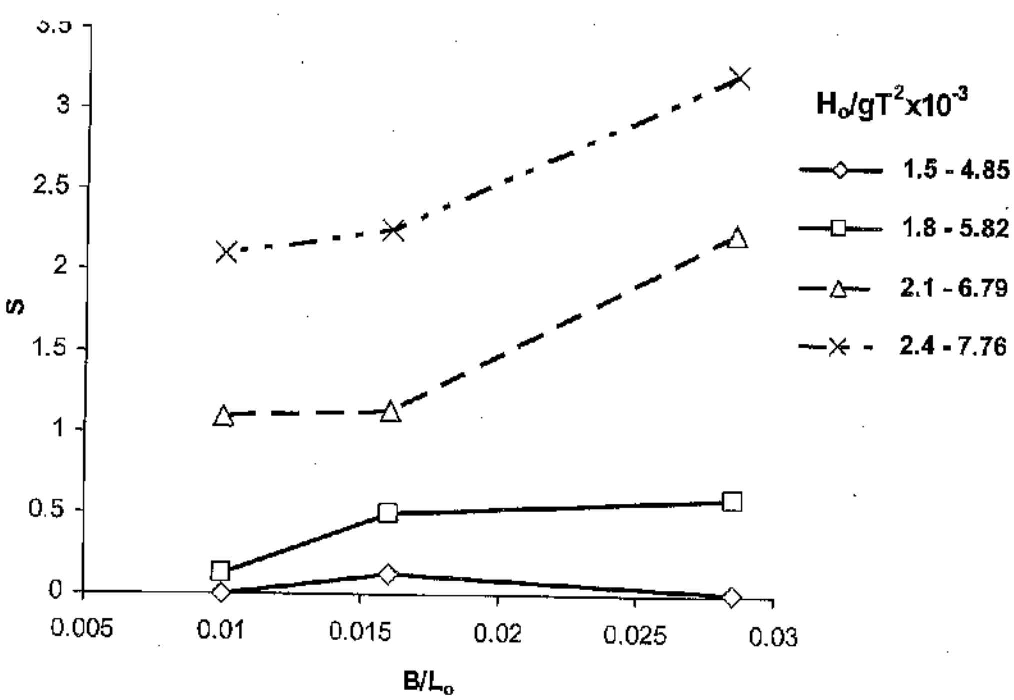

Fig. 9.12. Variation of S with B/L_o for d = 0.35 m

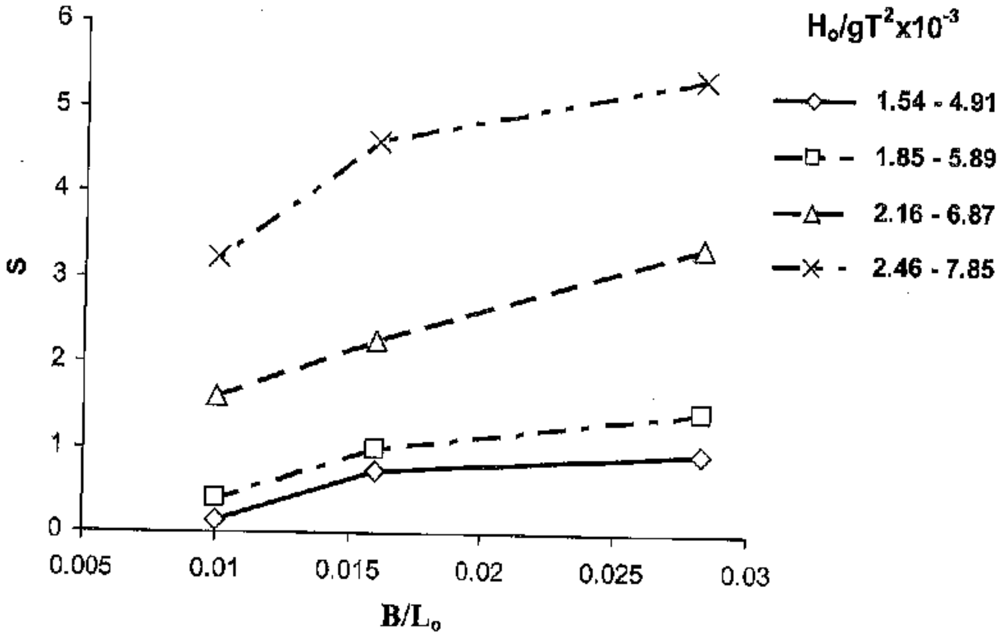

Fig. 9.13. Variation of S with B/L_0 for d = 0.4m

i.e. for increasing wave heights of 0.1m to 0.16m of different periods of 1.5sec, 2sec and 2.5sec, in water depths (d) of 0.35m and 0.4m respectively. The damages for a depth of 0.3m are nil. For depths of 0.35m and 0.4m, the general trend is that, damage level increases with reef crest widths (B/L_0) for any given range of wave steepness. This is because for a constant reef crest width (B) of 0.1m, the increase in B/L_0 indicates decreasing L_0 i.e. decreasing wave periods and as already observed shorter period waves are relatively more damaging compared to longer period waves. Graphs also indicate that, steeper waves are increasingly damaging the main breakwater. It is also seen that, the waves are more damaging with the increase in depth of water or decrease of h/d. As B/L_0 increases from 0.01 to 0.0285 and considering all ranges of H_0/gT^2, S increases from zero at the depth of 0.3m, 0.0 to 3.23 at the depth of 0.35m and 0.14 to 5.35 for depth of 0.4m.

9.3.1.3.4 Influence of stability number

Damages of the protected breakwater are nil for a depth of 0.3m. The damage level (S) of the breakwater increases with an increase in stability number (N_s) for all the depths of water (d) 0.35m and 0.4m as shown by the best fit lines in the Fig. 9.14 and Fig. 9.15 respectively. It is found that, the zero damage wave heights (H_{zd}) decrease with an increase in depth of water (d) from 0.35m to 0.4m indicating higher damage with increased depth. The H_{zd} in depth of water of 0.35m corresponding to wave periods of 1.5 sec, 2.0sec and 2.5sec are 0.1356m, 0.1582m and 0.1635m respectively which are 49% to 91% higher compared to conventional breakwater. H_{zd} in depth of 0.40m corresponding to wave periods of 1.5 sec, 2.0sec and 2.5sec are 0.1192m, 0.125m and 0.141m respectively which are 43.96% to 80% higher compared to conventional breakwater. The damage levels of protected breakwater for depths of water of 0.35m and 0.4m are respectively 75% to 80% and 63% to 67% lower than those of conventional breakwater.

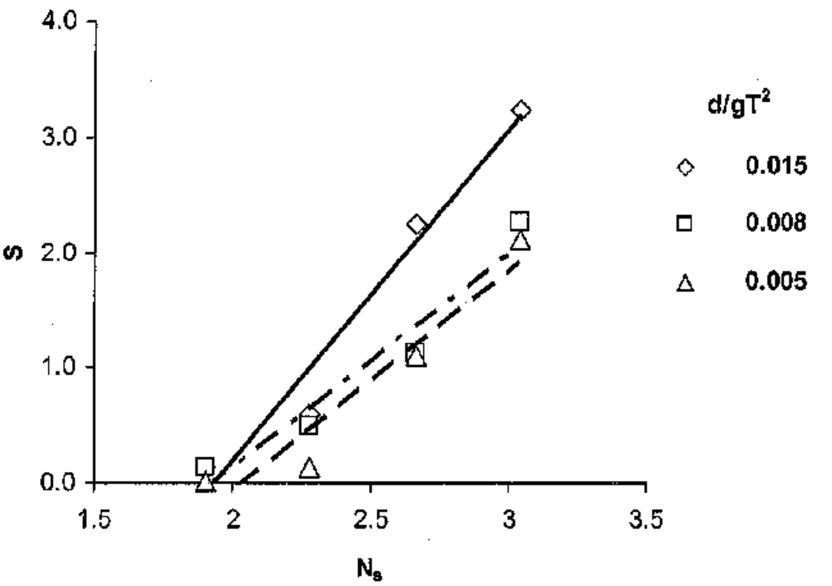

Fig. 9.14. Variation of S with N_s for d = 0.35m

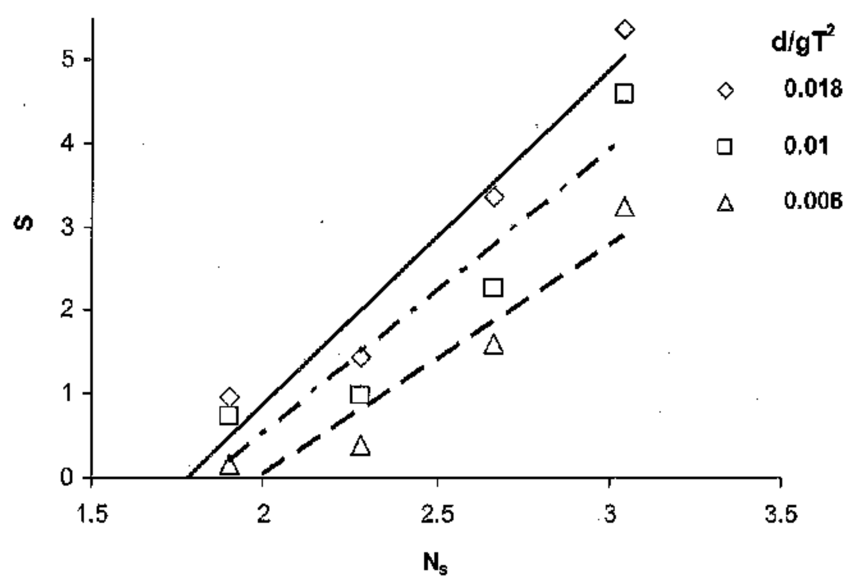

Fig. 9.15. Variation of S with N_s for d = 0.4 m

9.3.1.3.5 Influence surf similarity parameter on stability number

Thompsen et al. (1972) showed that, minimum stability of a 1V:2H sloped rubble mound breakwater occurred for surf similarity parameter $2 < \xi < 3$. Bruun and Gunbak (1976) write that the failure of the breakwater is caused by combinations of buoyancy, inertia and drag forces supported by the effect of hydrostatic pressure from the core and these forces reach their maximum value for lowest down rush which occurs at resonance for $2 < \xi < 3$. The damages for a depth of 0.3m are nil. Fig. 9.16 shows the variation of zero damage stability number (N_{zd}) and surf similarity parameter (ξ) for varying wave climate in depths of water of 0.35m and 0.4m i.e. increasing ranges of depth parameter (d/gT^2). The results are compared with those given by Thompsen et al. (1972). In the present study, it is observed that N_{zd} increases with ξ for a given range of d/gT^2. This is because H_{zd} increases with ξ. It is seen that minimum stability of the breakwater i. e. $N_{zd} = 2.26$ occurs for a ξ value of 2.71.

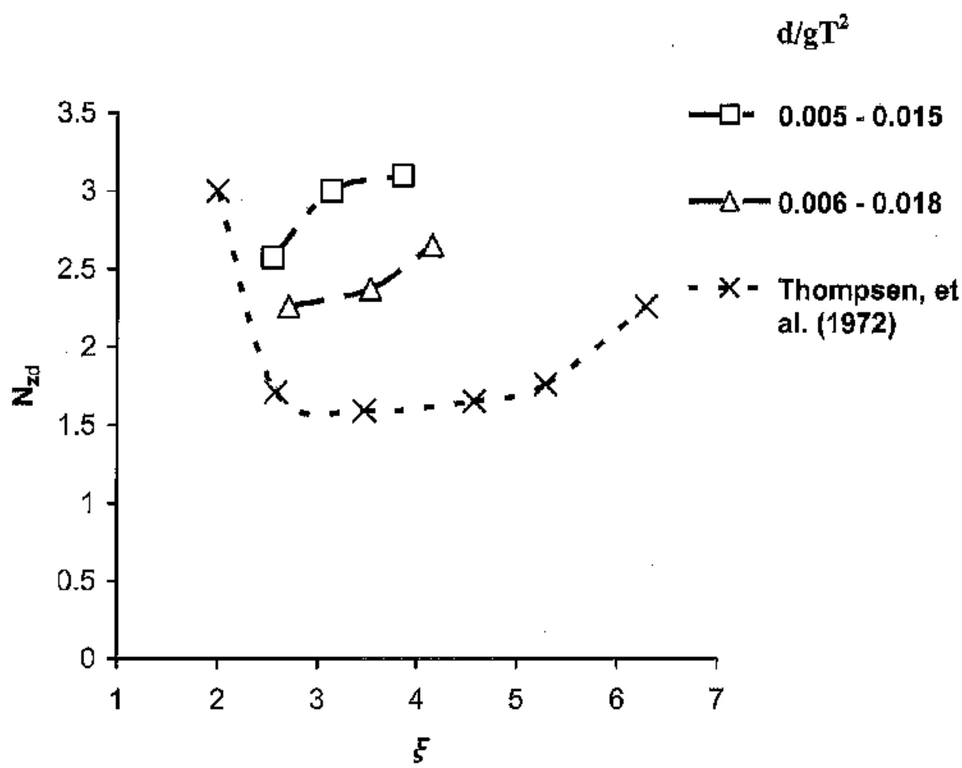

Fig. 9.16. Variation of N_{zd} with ξ

9.3.1.4 Conclusions

From the present study, the following conclusions are drawn.

1. The transmission coefficient (K_t) decreases with increase in H_o/gT^2, B/L_o and h/d and decreases with decrease in F/H_i and d/gT^2.
2. As B/L_o increases from 0.01 to 0.0285, K_t drops from 0.602 to 0.51 (15.8%), 0.67 to 0.56 (16.4%) and 0.72 to 0.59 (18%) for h/d of 0.833, 0.714 and 0.625 i.e. for depths of water (d) of 0.3m, 0.35m and 0.4m respectively.
3. K_t values are up to 25% to 40% lower, up to 26% lower, 22% to 33% lower and up to 22% higher than those given by Cox and Clark (1992) Van der Meer and d'Angremond (1992), Cornett et al. (1993) and d'Angremond (1996) respectively.
4. K_t increases from 0.51 to 0.606 (18.8%), 0.55 to 0.68 (23.6%) and 0.587 to 0.72 (22.6%) respectively for 1.5sec, 2sec and 2.5sec respectively for all the ranges of d/gT^2 and F/H_i.
5. K_t varies between 0.51 and 0.72 for the range of experimental parameters considered.
6. The maximum run up and run down are respectively 1.0 times and 0.65 times the deep water wave height. Run up and run down are reduced by 33% to 61% and 15% to 49% respectively compared to conventional breakwater.
7. The damage level S increases with the increase in H_o/gT^2, F/H_i, d/gT^2, B/L_o and decrease in h/d.
8. Damages at $0.004 < d/gT^2 < 0.0013$ and $-0.312 < F/H_i < -0.5$, i.e. in depth of water of 0.3m, are nil. As B/L_o increases from 0.01 to 0.0285 and considering all ranges of H_o/gT^2, damage level S is zero at $0.004 < d/gT^2 < 0.013$ and $-0.312 < F/H_i < -0.5$, i.e. for the depth of 0.3m and h/d of 0.833, for $0.005 < d/gT^2 < 0.015$ and $-0.625 < F/H_i < -1.0$, i.e. at the depth of 0.35m and h/d of 0.714, S increases from 0.0 to 3.23 and for $0.006 < d/gT^2 < 0.018$ and $-0.94 < F/H_i < -1.5$, i.e. at depth of 0.4m and h/d of 0.625, S increases from 0.14 to 5.35.
9. Considering the complete variation H_o/gT^2 and all the ranges of d/gT^2 (i.e as the depth of water increased from 0.35m to 0.4.), for the wave period of 1.5sec, the maximum damage level of the breakwater increases from 3.23 to 5.35 (65%), it increases from 2.26 to 4.59 (103%) for the wave period of 2.0sec while damage level rises from 2.1 to 3.23 (53.8%) for the wave period of 2.5sec.

10. For $0.005 < d/gT^2 < 0.015$ and $-0.625 < F/H_i < -1.0$ i.e. for d= 0.35m, h/d = 0.714, damage levels, which are shown in middle of the figure, the maximum value varies from 2.1 to 3.23 (i. e. a rise of 53.8%) and for $0.006 < d/gT^2 < 0.018$ and $-0.94 < F/H_i < -1.5$ i.e. for d= 0.4m and h/d = 0.625, the maximum value increases from 3.23 to 5.35 (i. e. by 65.6%).
11. Damages at depths of water of 0.35m and 0.4m are about 63% to 80% less compared to conventional breakwater
12. Zero damage wave heights are 43.96% to 91% higher than the conventional breakwater.
13. Minimum stability of the breakwater (i. e. $N_{zd} = 2.26$) occurs for a ξ value of 2.71.

9.3.2 Protected breakwater with a reef of crest width (B) of 0.2m (i. e. B/d = 0.5 to 0.67)

9.3.2.1 Influence of various parameters on transmission coefficient

9.3.2.1.1 Influence of deep water wave steepness

Fig. 9.17 shows decrease of transmission coefficient K_t with the increase in deep water wave steepness parameter (H_o/gT^2) and relative reef height (h/d). The figure shows the best fit lines for K_t. For $1.45x10^{-3} < H_o/gT^2 < 7.851.45x10^{-3}$, K_t decreases from 0.53 to 0.37 (30.2%), 0.62 to 0.4 (35.5%) and 0.7 to 0.55 (21.4%) for h/d of 0.833, 0.714 and 0.625 i.e. for water depth of 0.3m, 0.35m and 0.4m respectively. The wave height attenuation achieved is 30% to 63%.

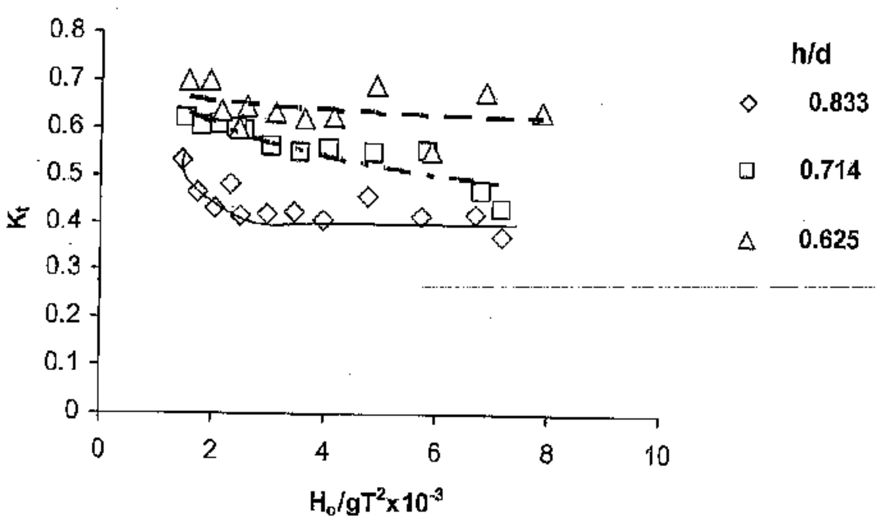

Fig. 9.17. Variation of K_t with H_o/gT^2

9.3.2.1.2 Influence of relative reef submergence

Each of Fig. 9.18, Fig. 9.19 and Fig. 9.20 show the increasing trend of transmission coefficient K_t with an increase in relative reef submergence (F/H_i) and range of depth parameter (d/gT^2) for wave periods of 1.5sec, 2sec and 2.5sec respectively. For -0.312 < F/H_i < -1.5, K_t increases from 0.37 to 0.63 (70.3%), 0.4 to 0.65 (62.5%) and 0.42 to 0.70 (66.7%) for periods of 1.5sec, 2sec and 2.5sec respectively. For all the wave periods, the K_t values are approximated by d'Angremond (1996) with an accuracy of 15%. Considering the full variation of F/H_i, for a wave period of 1.5sec, present K_t values are 42% to 48% lower, 26% to 35% lower, 30% lower than values of Cox and Clark 1992), Van der Meer and d'Angremond (1992) and Cornett et al. (1993) respectively. Similarly for a period of 2sec, present K_t values are 36% to 43% lower, 18% to 25% lower and 27% lower than those given by Cox and Clark 1992), Van der Meer and d'Angremond (1992) and Cornett et al. (1993) respectively. And for period of 2.5sec, present K_t values are 36% to 42% lower, 17% to 27% lower and 22% lower than those given by Cox and Clark (1992), Van der Meer and d'Angremond (1992) and Cornett et al. (1993) respectively. For wave periods of 1.5sec, 2sec and 2.5sec, K_t increases from 0.37 to 0.63 (70.3%), 0.4 to 0.65 (62.5%) and 0.42 to 0.70 (66.7%) respectively considering all the ranges of d/gT^2 and F/H_i.

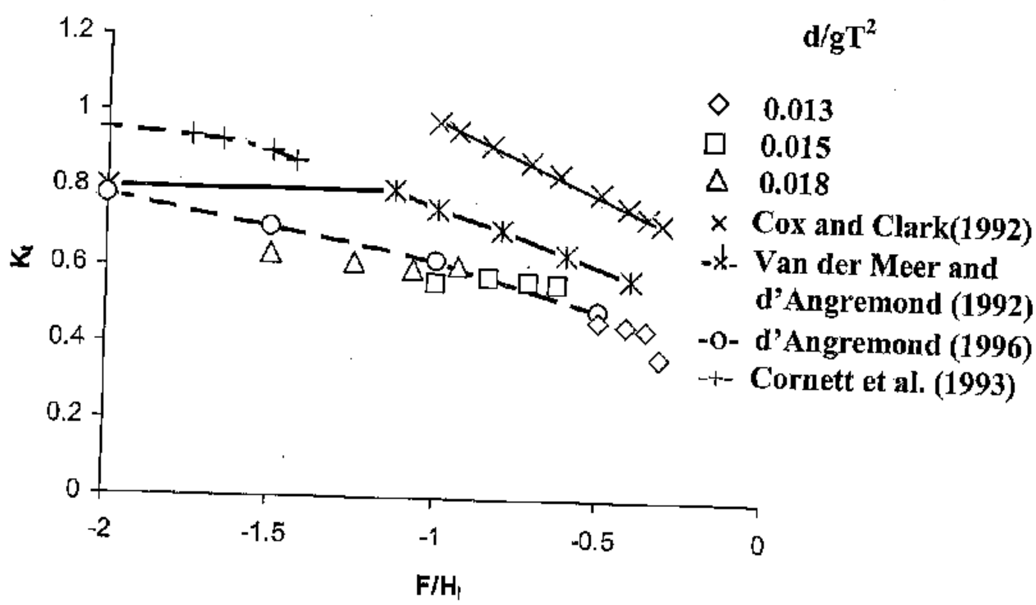

Fig. 9.18. Variation of K_t with F/H_i for T=1.5 sec

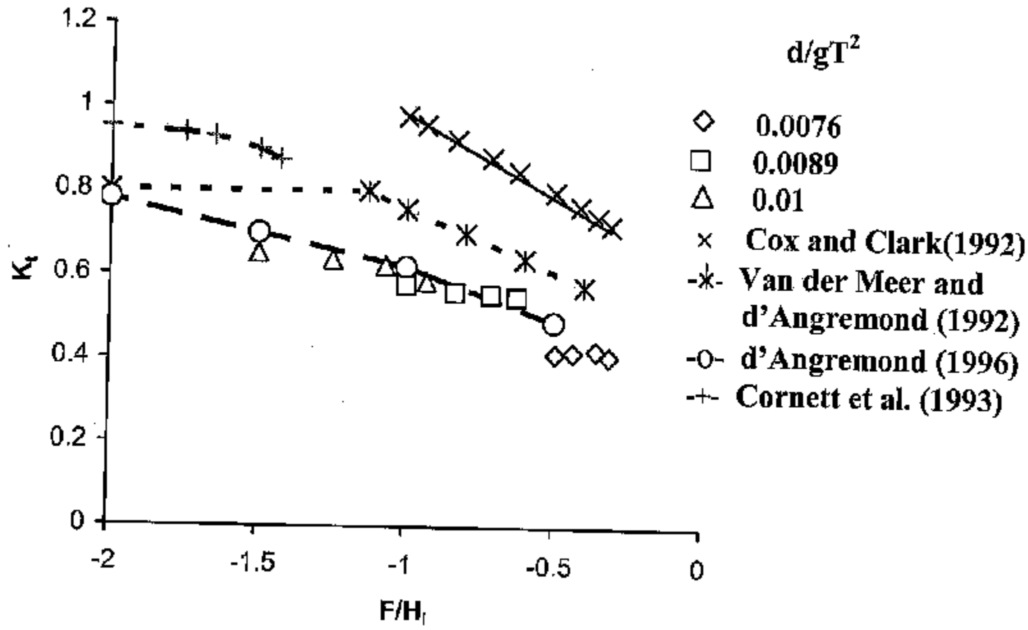

Fig. 9.19. Variation of K_t with F/H_i for T =2 sec

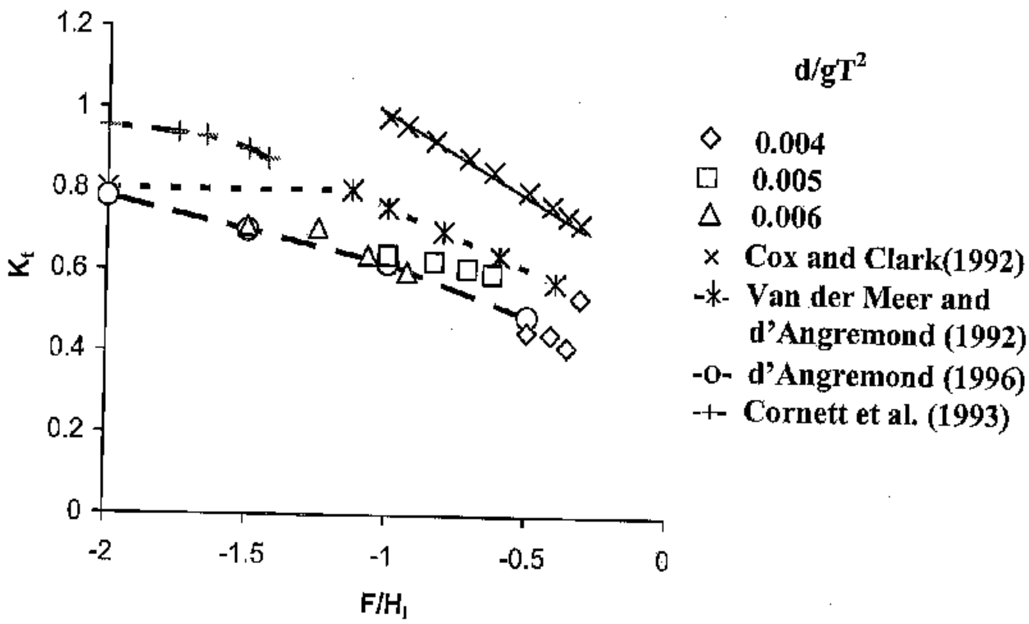

Fig. 9.20. Variation of K_t with F/H_i for T =2.5 sec

9.3.2.1.3 Influence of relative reef crest width

Fig. 9.21, Fig. 9.22 and Fig. 9.23 show variation of K_t with relative reef crest width (B/L_o) for different wave steepness parameters (H_o/gT^2) for water depths of (d) of 0.3m, 0.35m and 0.4m in respectively.

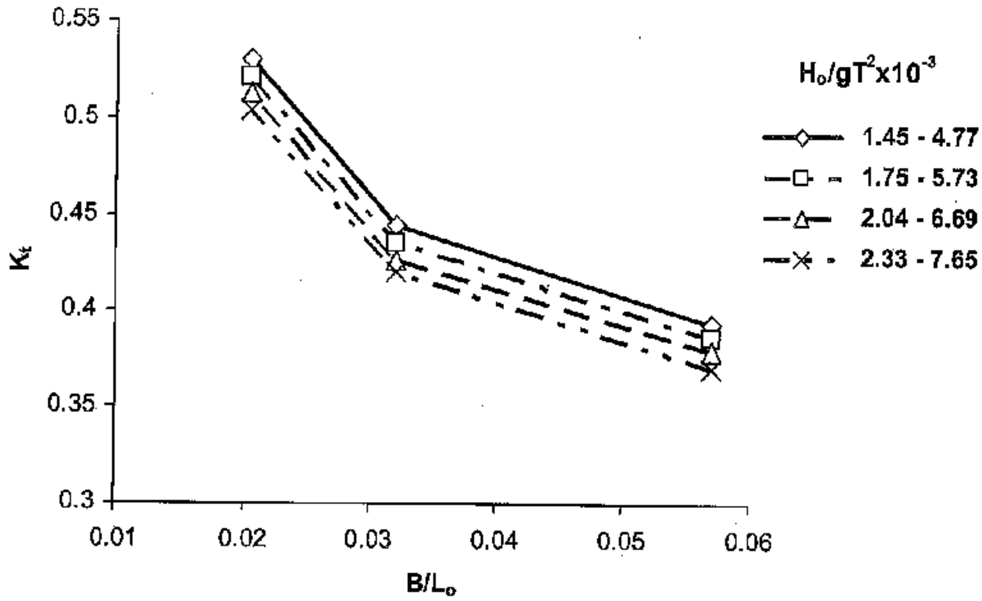

Fig. 9.21. Variation of K_t with B/L_o for d = 0.3m

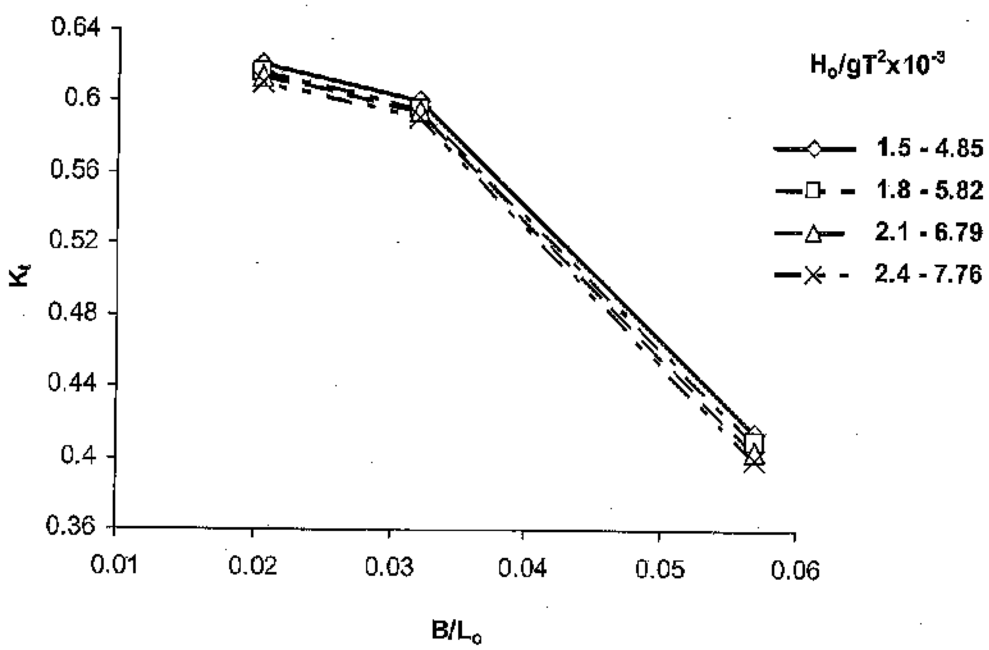

Fig. 9.22. Variation of K_t with B/L_o for d = 0.35m

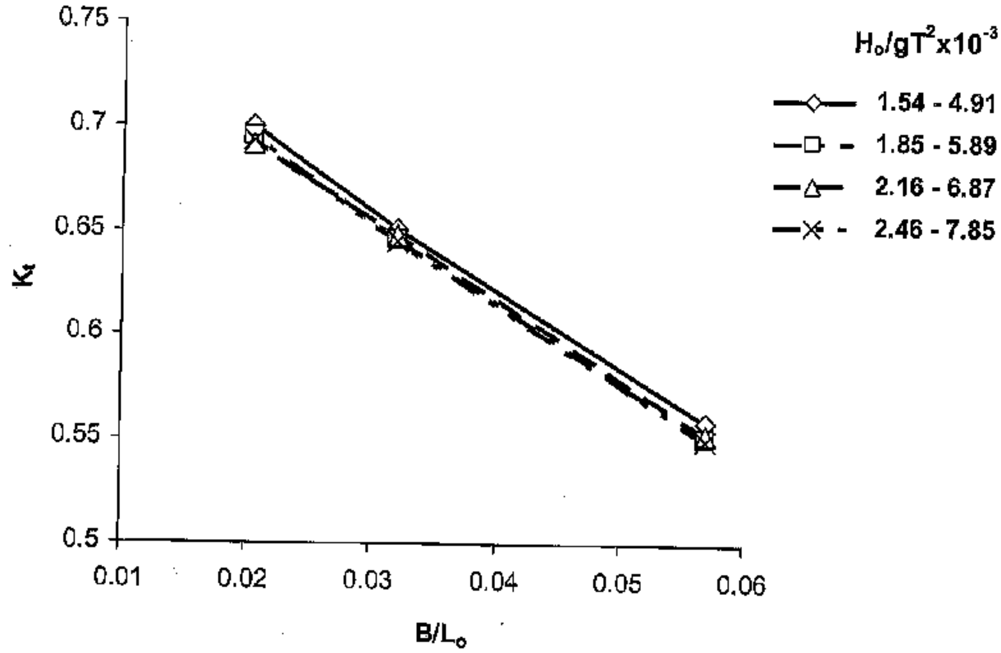

Fig. 9.23. Variation of K_t with B/L_o for d = 0.4m

K_t decreases with an increase in B/L_o and with an increase in range of H_o/gT^2 for a given depth of water. It is seen that influence of H_o/gT^2 on K_t is clearly visible for depth of water of 0.3m while for other depths of 0.35m and 0.4m, it is not significant. For depths of 0.3m and 0.35m, it is observed that the trend of K_t changes at a B/L_o value of 0.032 whereas, for a depth of 0.4m, K_t decreases uniformly as H_o/gT^2 increases. K_t varies in the ranges of 0.37 to 0.53, 0.4 to 0.62 and 0.55 to 0.70 for depths of water of 0.3m, 0.35m and 0.4m respectively.

9.3.2.2 Influence of deep water wave steepness on wave run up and run down

The influence of deep water wave steepness parameter (H_o/gT^2) on relative run up (R_u/H_o) and run down (R_d/H_o), for increasing ranges of depth parameter (d/gT^2) i.e. for varying wave climate in depths of water of 0.3m, 0.35m and 0.4m is shown in Fig. 9.24 and Fig. 9.25 respectively. The figures show the best fit lines. Both relative run up and the run down decrease, with an increase in wave steepness and decrease of range of d/gT^2. The maximum wave run up and run down are respectively 0.98 times and 0.53 times the deep water wave height for the range of variables considered in the present study. Run up for depths of water of 0.3m, 0.35m and 0.4m (i.e. when F/H_i varies in the ranges of -0.312 to -0.5, -0.62 to -1.0 and -0.94 to -1.5) for all the ranges of d/gT^2 are respectively lower by 48% to 50%, 31% to 45% and 38% than those for a conventional (single) breakwater. Similarly rundown for depths of water of 0.3m, 0.35m and 0.4m are respectively lower by 50% to 74%, 16% to 39% and 23% to 25% than those for conventional breakwater.

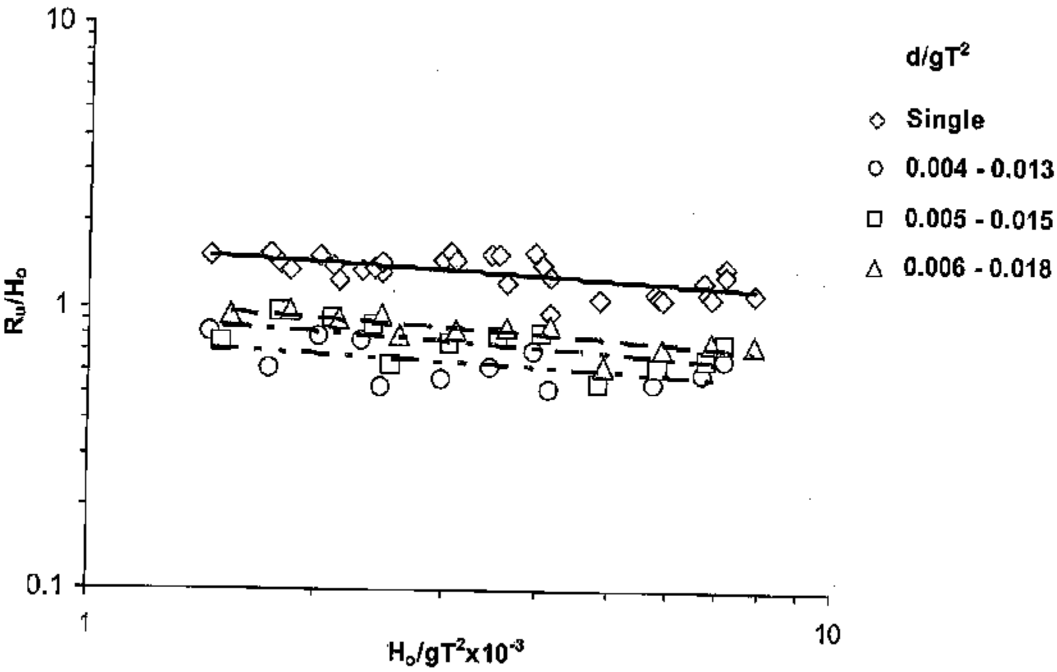

Fig. 9.24. Variation of R_u/H_o with H_o/gT^2

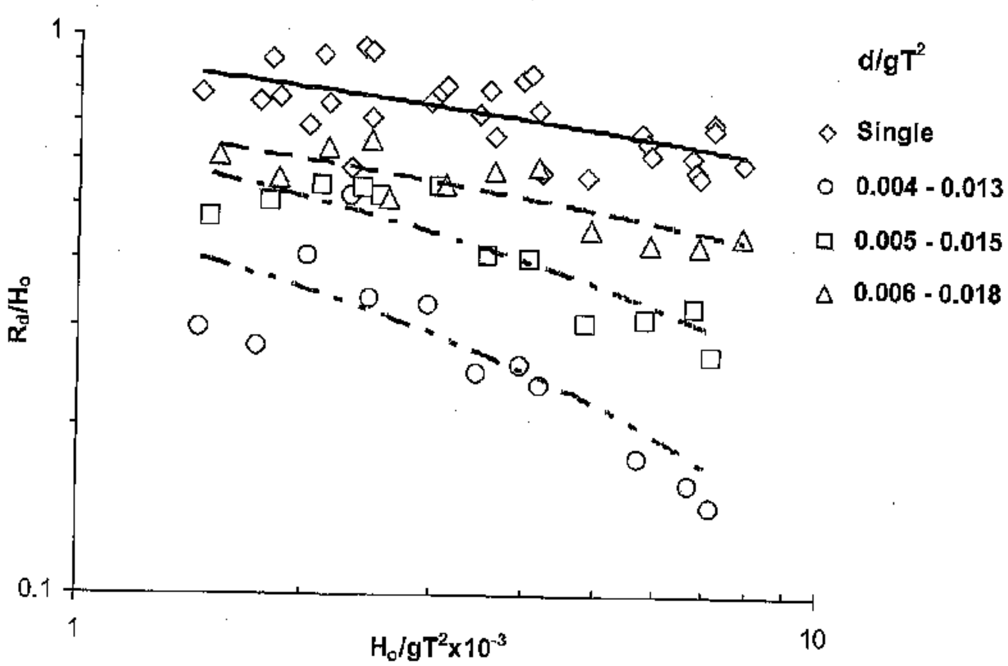

Fig. 9.25. Variation of R_d/H_o with H_o/gT^2

9.3.2.3 Influence of various parameters on damage level

9.3.2.3.1 Influence of deep water wave steepness

The graphs of Fig. 9.26 show increasing damage level (S) with increasing wave steepness parameter (H_o/gT^2) and increasing ranges of depth parameter (d/gT^2) i.e. for depths of water of 0.3m, 0.35m and 0.4m and different wave periods. The impact of wave period is clearly distinguishable as damages are grouped from right to left for the increasing period of 1.5sec, 2sec and 2.5sec. Damages at $0.004 < d/gT^2 < 0.013$ and $0.005 < d/gT^2 < 0.015$ i.e. depth of water of 0.3m and 0.35m, are nil. Some damage is observed at $0.006 < d/gT^2 < 0.018$ i.e. for the depth of 0.4m, the waves of period 1.5sec.

The damage at the depth of 0.4m (i.e. h/d of 0.625) increases with a decrease in wave period. The damage level of 0.512 to 2.31 is observed for $4.91 \times 10^{-3} < H_o/gT^2 < 7.85 \times 10^{-3}$ i.e. waves of 0.16m of period of 1.5sec, while for waves of period of 2.0sec and 2.5sec the damage is nil. The damage levels of protected breakwater for depths of water of 0.4m are 84% less compared to conventional breakwater.

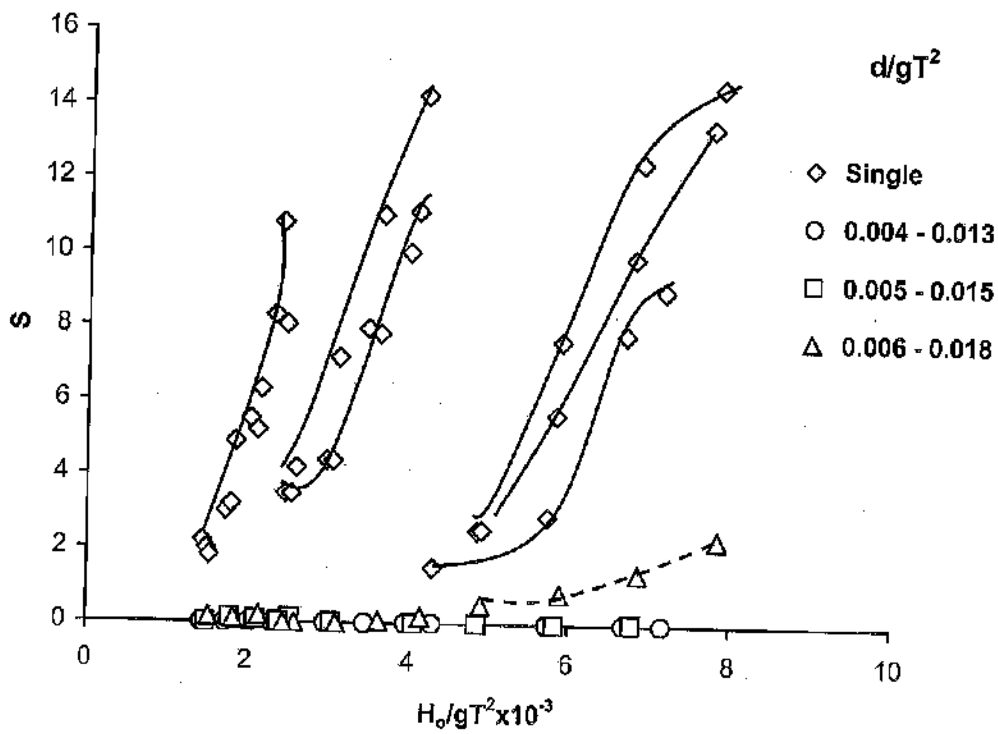

Fig. 9.26. Variation of S with H_o/gT^2

9.3.2.3.2 Influence of reef submergence

The graph in Fig. 9.27 shows variation of the damage level (S) with the reef submergence (F/H_i) for varying depth parameter (d/gT^2). The breakwater damages for waves in depths of water of 0.3m and 0.35m (h/d = 0.833 and 0.714) are nil. From the figure it can be seen that damage level of 0.512 to 2.31 occurs for d/gT^2 of 0.018 and for $-0.94 < F/H_i < -1.5$ i.e. waves of periods 1.5sec in a depth of water of 0.4m (h/d = 0.625).

9.3.2.3.3 Influence of reef crest width

Fig. 9.28 demonstrates the impact of reef crest width (B/L_o) on breakwater damage, increasing ranges of H_o/gT^2 in the water depth (d) of 0.4m i.e. for increasing wave heights of 0.1m to 0.16m of periods of 1.5sec, 2sec and 2.5sec. The breakwater damages for waves in depths of water of 0.3m and 0.35m are nil. It can also be seen that, the damages are nil up to a B/L_o value of 0.032 and then damage level increases indicating only steeper waves of period 1.5sec i.e. short period waves are damaging the main breakwater. From the figure it can be seen that, breakwater damage is initiated (i.e. at $S \geq 2$) only for $2.46 \times 10^{-3} < H_o/gT^2 < 7.85 \times 10^{-3}$ i.e. a wave of 0.16m and period of 1.5sec and the maximum damage level observed is 2.31.

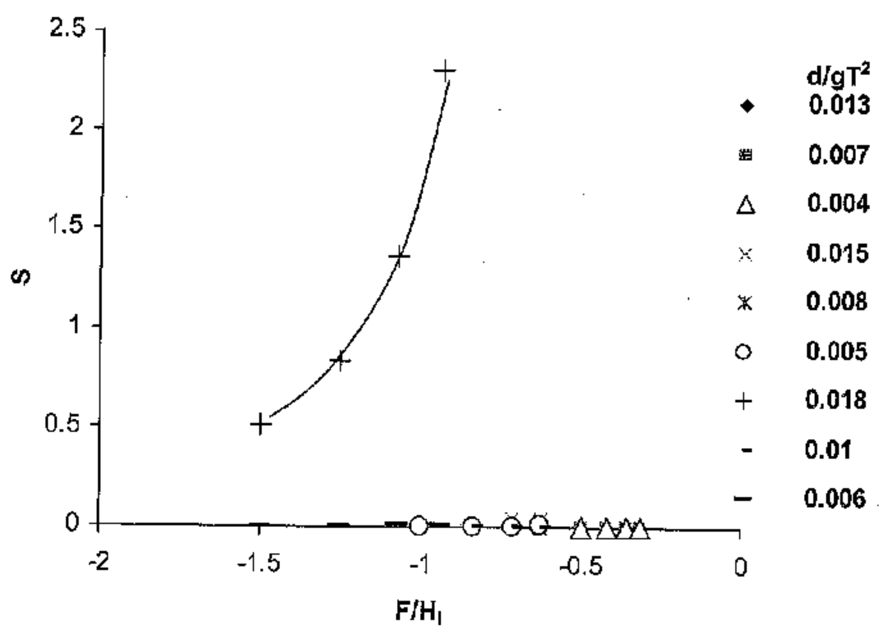

Fig. 9.27. Variation of S with F/H_i

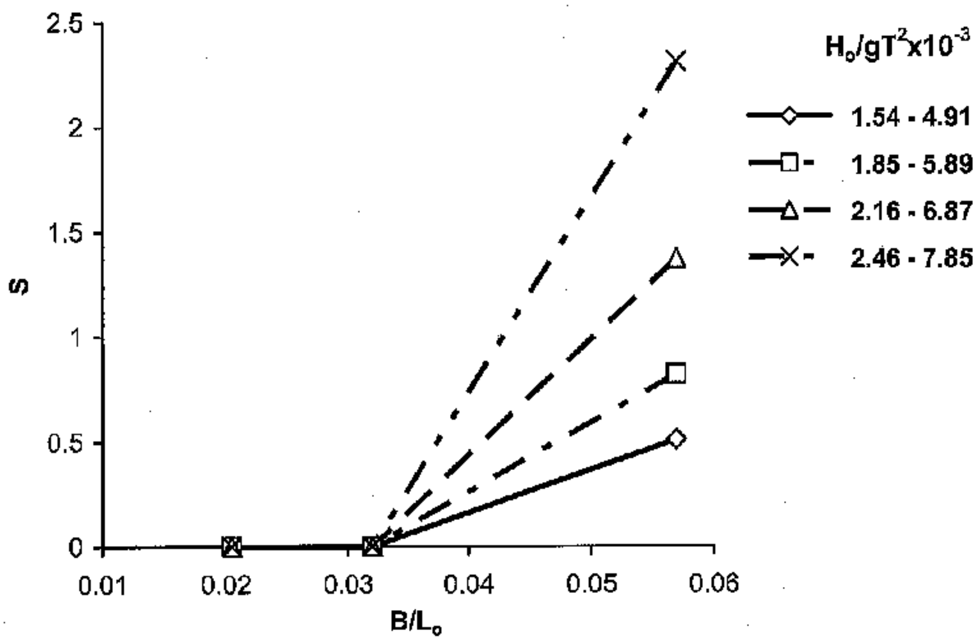

Fig. 9.28. Variation of S with B/L$_0$ for d = 0.4m

9.3.2.3.4 Influence of stability number

The damage level (S) of the breakwater increases with an increase in stability number (N_s) for the waves of period of 1.5sec in a depth of water (d) of 0.4m (i.e. h/d = 0.625) as shown by best fit line in Fig. 9.29. The breakwater damage is noticeable only for waves of height of 0.16m. The zero damage wave height (H_{zd}) at this condition is 0.1557m i.e. 88% higher than that of the conventional breakwater.

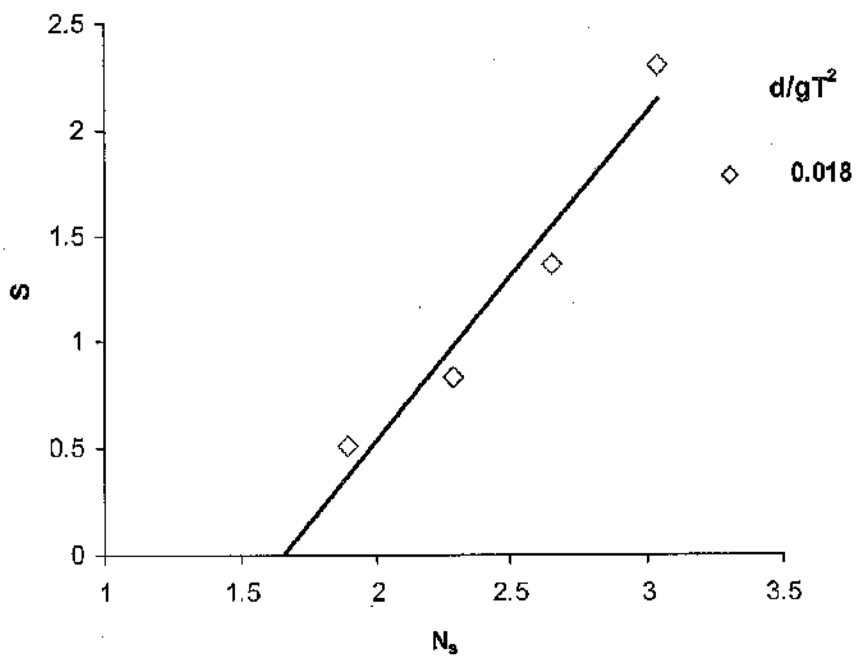

Fig. 9.29. Variation of S with N_s for d = 0.40m

9.3.2.3.5 Influence of surf similarity parameter on stability number

Fig. 9.30 shows the variation of zero damage stability number (N_{zd}) and surf similarity parameter (ξ) for varying wave periods in depth of water of 0.4m only (i.e. $0.006 < d/gT^2 < 0.018$) because the breakwater damages for waves in depths of water of 0.3m and 0.35m are nil. The result is compared with those given by Thompsen et al. (1972). In the present study, it is observed that a N_{zd} of 2.95 is observed for a ξ of 2.37.

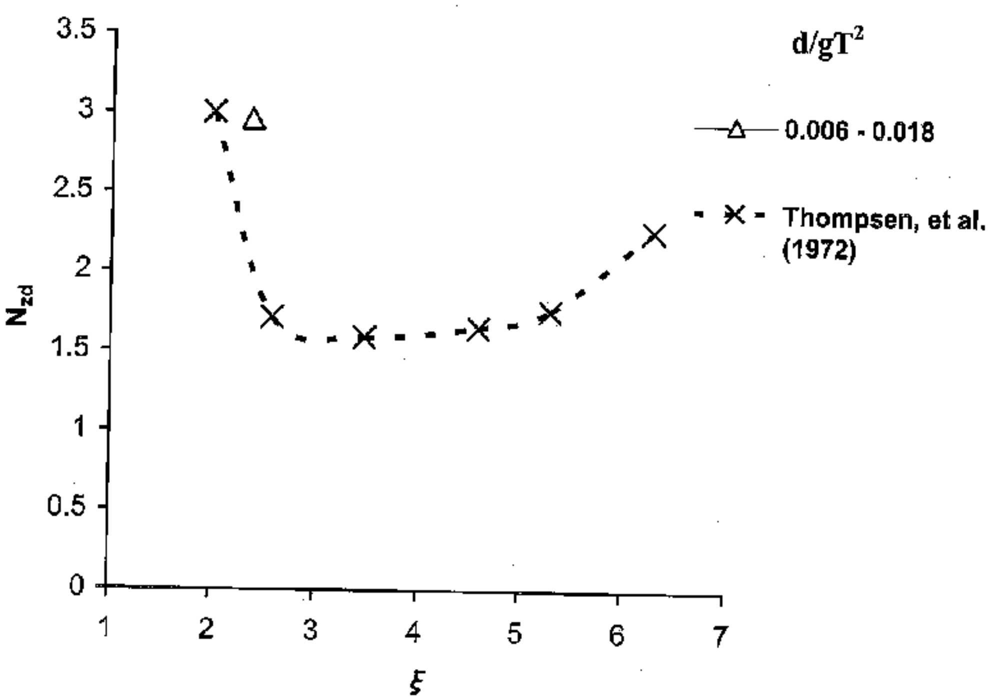

Fig. 9.30. Variation of N_{zd} with ξ

9.3.2.4 Conclusions

Based on the present study the following conclusions are drawn.

1. The transmission coefficient (K_t) decreases with increase in H_o/gT^2, B/L_o and h/d and decreases with decrease in F/H_i and d/gT^2.
2. For $1.45x10^{-3} < H_o/gT^2 < 7.851.45x10^{-3}$ and $0.02 < B/L_o < 0.057$, K_t decreases from of 0.53 to 0.37 (30.2%), 0.62 to 0.4 (35.5%) and 0.7 to 0.55 (21.4%) for h/d of 0.833, 0.714 and 0.625 i.e. for water depth of 0.3m, 0.35m and 0.4m respectively.
3. For $-0.312 < F/H_i < -1.5$, and all depths of water, K_t values are approximated by d'Angremond (1996) with an accuracy of 15%.

4. For -0.312 < F/H_i < -1.5, and all ranges of depth parameter, K_t values are lower by 36% to 48%, 17% to 35% and 22% to 30% than those given by Cox and Clark (1992) Van der Meer and d'Angremond (1992), Cornett et al. (1993) respectively.
5. For wave periods of 1.5sec, 2sec and 2.5sec, K_t increases from 0.37 to 0.63 (70.3%), 0.4 to 0.65 (62.5%) and 0.42 to 0.70 (66.7%) respectively for all the ranges of d/gT^2 and F/H_i.
6. For depths of 0.3m and 0.35m, it is observed that the trend of K_t changes at a B/L_o value of 0.032 whereas, for a depth of 0.4m, K_t decreases uniformly as H_o/gT^2 increases.
7. K_t varies between 0.37 and 0.70 for the range of experimental parameters considered.
8. The maximum run up and run down are respectively 0.98 times and 0.53 times the deep water wave height.
9. Relative run up and run down are lower by 31% to 50% and 16% to 74% compared to that of the conventional breakwater.
10. The damage level S increases with the increase in H_o/gT^2, F/H_i, d/gT^2 and B/L_o and decreases with h/d.
11. Damages at depths of water of 0.3m and 0.35m (h/d = 0.833 and 0.714) are nil.
12. Maximum damage level (S) of 2.31 occurs at depth of water of 0.4m (h/d = 0.625) which is about 84% less compared to conventional breakwater.
13. Zero damage wave height (H_{zd}) is 0.1557m i.e. 88% higher than that of the conventional breakwater.
14. Minimum stability of the protected breakwater (i. e. N_{zd} = 2.95) occurs for a ξ value of 2.37.

9.4 SUMMARY OF CONCLUSIONS FOR PROTECTED BREAKWATER

The conclusions for protected breakwater, with a conventional inner main breakwater and a submerged reef of height (h) of 0.25m with a varying crest width (B) of 0.1m and 0.2m (i.e. B/d varying from 0.25 to 0.33 and 0.5 to 0.67 respectively) located seaward of the breakwater at a distance (X) of 4m (i.e. X/d of 10.0 to 1333), are summarized and compared with those of the conventional breakwater and are listed in Table. 9.1.

It can be seen from the table that the damage of the conventional breakwater reduced when a protective submerged reef is located at a distance (X) of 4m seaward of the breakwater. As the crest

Table. 9.1. Comparison of conclusions for conventional breakwater (CBW) and protected breakwater

Sl. No	Parameter	Conventional breakwater (CBW)	Protected breakwater with a seaward submerged reef of varying crest width B located at (X) of 4m (i.e. X/d of 10.0 to 1333)	
			B	
			0.1m	0.2m
			B/d	
			0.25 - 0.33	0.50 - 0.67
1.	Max. R_u/H_o	1.57	1.0	0.98
2.	Present R_u/H_o compared with that of CBW (% Lower)	-------------	33 - 61	31 - 50
3.	Max. R_d/H_o	0.95	0.65	0.53
4.	Present R_d/H_o compared with that of CBW (% Lower)	-------------	15 - 49	16 - 74
5.	Transmission coefficient K_t	-------------	0.51 - 0.72	0.37 - 0.7
6.	K_t at d = 0.30m d = 0.35m d = 0.40m	-------------	0.51 - 0.606 0.56 - 0.67 0.59 - 0.72	0.37 - 0.53 0.40 - 0.62 0.55 - 0.7
7.	K_t at T= 1.5sec T= 2.0sec T= 2.5sec	-------------	0.51 to 0.606 0.55 to 0.68 0.587 to 0.72	0.37 to 0.63 0.40 to 0.65 0.42 to 0.70
8.	Max. damage level at d = 0.30m d = 0.35m d = 0.40m	8.3 - 10.0 10.8 - 13.4 8.05 - 14.5	Nil 2.31 - 3.23 3.23 - 5.35	Nil Nil 0.512 – 2.31
9.	Max. damage level at T= 1.5sec T= 2.0sec T= 2.5sec	9.0 - 14.5 10.0 - 14.23 8.3 - 10.8	3.23 - 5.35 2.26 - 4.59 2.1- 3.23	0.512 - 2.31 Nil Nil
10.	Present S compared with that of CBW (% Lower)	-------------	63 - 80	84
11.	Present H_{zd} compared with that of CBW (% Higher)	-------------	43.96 - 91	88

width (B) of the submerged reef increases from 0.1m to 0.2m the reduction of damage of the protected breakwater compared to conventional structure increases from 63% to 80% to 84% respectively. But submerged reef of crest width of 0.2m does not safeguard the breakwater completely.

Chapter 10
Conclusions and Design of Defenced Breakwater

10.1 BACKGROUND

Rubble mound breakwaters are the structures built to reflect and dissipate energy of the wind generated waves and protect the area on the leeside. In modern times the breakwaters are constructed for the purpose of protecting vital installations on the coast and offshore, for shoreline stabilization, forming an artificial harbour with a water area so protected from the ocean waves as to provide safe accommodation for ships and for preventing the siltation of river mouths.

In the beginning, primitive reefs and dykes of gentle slopes were built with natural stones e.g. breakwater at Cherbourg in France built in 1784 – 1790 A. D. Later, to save the material, steeper sloped structures with rubble mound, concrete block mound, rock fill over mound, caisson type etc. were tried. A mound breakwater with concrete blocks was first constructed at Alger, Algeria in 1830's. The traditional and most commonly used breakwaters are of non-overtopping rubble mound type. These structures consist of essentially core of fine material like quarry run and are protected by armour layer of large rocks or artificial concrete blocks. Rubble mound breakwaters are suitable for all types of foundations. They do not require highly skilled labour for construction. They can be constructed up to 50m depth economically and can easily be repaired. But for deep-water sites and at locations having large tidal ranges, the quantity of stones required might be large causing high expenditure. On exposed sites, the waves gradually drag down the mound, giving it a flat slope on the sea face. The disturbing action of the waves is most keenly felt around high water and low water levels. It is in this region that the structure is most severely tested.

Till 1933, the breakwaters were designed based on laboratory experiments, experience and comparative basis which were either over safe or under designed. Though various investigators gave design formulae later, it was Hudson (1959) who gave the design formula for breakwater armour weight based on wide ranging experimental study involving structure slope, wave height, armour density etc.

From the economic point of view, breakwaters are cost intensive and would require regular maintenance to retain their effectiveness. Depending upon site conditions, a stable and economic structure can be with smaller armour units and/or where profile development is allowed in order to reach a stable configuration e.g. berm breakwater, submerged breakwater and reef breakwater. Hence, the selection, alignment, design and construction of the breakwater must be carefully undertaken after examining the important governing parameters like predominant direction of approach of waves and their characteristics, tidal range, degree of protection required, magnitude and direction of littoral drift and the possible effect of these breakwaters on the shorelines etc. All the time it is researchers' endeavour to design an innovative breakwater structure which optimally satisfies the requirements.

There are several sites where complete tranquility is not required and only partial protection from waves is desirable e.g. small craft harbours, coastal protection, protection of tourist/recreation spots on beaches etc. For such sites designing a conventional non-overtopping breakwater is uneconomical, unnecessary and uncalled for. One of the proposed solutions, under such situation, is a submerged breakwater which is economical and efficient. This is a type of low crested structure with its crest at or below the still water level (SWL). Submerged breakwaters are one of the most commonly built structures to dissipate energy of the wind generated waves and thereby to prevent their incidence on a water area intended to protect where the tidal ranges are small.

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. Depending upon the parameters like geometry, height, crest width, armour stone weight and location of the submerged breakwater, they offer resistance through friction and turbulence, causing maximum wave damping, energy dissipation, minimum wave reflection and bottom scour, and maximum sand trapping efficiency. These characteristics bestow the submerged breakwater the status of a structure that can be used for coastal protection based on specific model studies.

The reef is a low crested structure which is little more than a homogeneous pile of stones whose weight is sufficient to resist the wave attack. A submerged reef breakwater is an optimized structure to highest degree in the class of rubble mound structures as it has neither the multilayered cross section nor the core and therefore can't fail catastrophically and wave reflection is also relatively less due to its porous nature. Compared to impermeable structure, reef is safer. It is fundamentally built to break the steep waves and dissipate wave energy. The simplicity of the reef could be the significant factor in keeping down the construction cost and it could be an optimum structure for many situations in the cases of beach protection.

Based on limited studies it has been proved that a conventional breakwater may be protected by a suitable submerged structure located in the front. This protective structure breaks the steep waves and attenuates them before they propagate and impinge on the inner main breakwater. It is recognized that a submerged reef can offer such a protection to the inner main breakwater. This technique may be used to rehabilitate the damaged breakwater as well as to design an economical defenced breakwater as an alternative to the conventional structure. This topic of design of a defenced breakwater is selected for the present study.

10.2 SUMMARY OF MODEL TESTS

It is decided to experimentally study the performance of a 1:30 scale model of protected breakwater, where, a seaward submerged reef protects the inner main breakwater, under attack of varying wave climate in changing depths of water. The experiment would include investigation of:

1. Stability of conventional breakwater.
2. Armour stone stability of a trapezoidal submerged reef.
3. Wave transmission at reef and wave height attenuation.
4. Influence of spacing (i.e. energy dissipation zone) between the structures on stability of main breakwater.
5. Influence of varying reef crest width on stability of main breakwater for a reef placed at a selected seaward location.

Therefore, the aim of the present study is to evolve a design of defenced breakwater where, a seaward submerged reef offers complete protection of the main breakwater.

In the first phase, a 1:30 scale model of a breakwater, of trapezoidal cross section with a uniform slope of 1V:2H, is constructed on the flat bed of the flume, with primary stone armour of weight of 73.2gms (i.e. nominal diameter, D_{n50}, of 0.0298m) for a design wave of 0.1m. Its crest width is 0.1m and height is 0.70m. The secondary armour and core is designed as for a conventional breakwater (US Army Corps of Engineers 1984). This model is tested for armour stability for different waves of 0.1m to 0.16m of period varying from 1.5sec to 2.5sec generated in depth of water ranging from 0.3m to 0.4m.

In the second phase, a 1:30 scale model of a trapezoidal submerged reef, of slope 1V:2H, height (h) of 0.25m and crest width (B) of 0.1m is constructed over the flat bed of the flume with armour of weight varying from 15gms to 35gms. This section is tested for stability of its armour under varying wave characteristics in a critical depth of 0.3m of water. It is observed that reef with an armour of 30gms is stable.

In the third phase, a stable trapezoidal submerged reef having a slope of 1V:2H with a crest width (B) of 0.1m, height (h) of 0.25m and is constructed with an armour of 30gm. The waves of varying heights and periods are generated in depth of water of 0.3m, 0.35m and 0.4m. These waves pass over the reef constructed and transmitted the wave heights are recorded for every 1m upto a distance of 8m on the leeside. It is observed that up to a distance of 4m on the leeside of the reef, waves are attenuated by about 50%, beyond which there is no significant increase in wave attenuation.

The tests are repeated, in the fourth phase, with a reef of crest width 0.1m at different locations within the effective distance of 4m. From the wave height attenuation observed, it is found that reef at 2.5m could be the required spacing between structures that calls for further study with different reef crest widths. Then, at this particular location (of 2.5m seaward of the inner (main) breakwater), the reef crest widths (B) are varied from 0.1m to 0.4m as desired, to arrive at a crest width that completely protects the main breakwater.

The model tests are initiated by surveying the model cross section with the profiler, which is, the reference survey (initial profile) for comparison of subsequent surveys (final profile after damage). The models are subjected to a series of initial wave height (H_i) starting from 0.1m and then gradually wave height is increased by 20% each time till it reached the highest value of 0.16m for a selected period. 3000 waves are run in bursts for each trial (which is equivalent to an actual storm of 6.85 hours to 11.41hours) or the failure of the structure whichever occurred earlier. Damage level (S) of the breakwater is calculated as the ratio of area of erosion (A_e) to square of nominal diameter (D_{n50}) of breakwater armour unit. The failure in these tests is defined as the displacement of primary armour stones so that filter layer is exposed to wave action. The final profile of the damaged breakwater is again surveyed.

10.3 SUMMARY OF CONCLUSIONS

10.3.1 Conventional breakwater

1. The damage level S increases with the increase in H_o/gT^2, d/gT^2 and N_S.
2. The maximum run up and run down are respectively 1.57 times and 0.95 times the deep water wave height.
3. The breakwater damages are in the range of 1.5 to 10.0, 2.0 to 13.4 and 1.81 to 14.5 in the depths of 0.3m, 0.35m and 0.4m respectively.
4. Maximum damage levels in the depth of water of 0.3m, 0.35m and 0.4m increase from 8.3 to 10.0 (i.e. 20.48%), 10.8 to 13.4 (i.e. 24.07%) and 8.05 to 14.5 (i.e. 80.1%) respectively.
5. As the depth of water increases from 0.3m to 0.35m (i.e. 16.67%) the maximum damage level of the breakwater increased from 10 to 13.4 (i.e. by 34%) and at a depth of 0.4m (i.e. an increase of 33.3% w.r.t 0.3m) damage level increases from to 14.5 (i.e. by 8.2%).
6. Considering all the ranges of d/gT^2 (i.e. waves in all depths of water of 0.3m, 0.35m and 0.4m), the increase in damage levels are 9 to 14.5 (61%), 10 to14.23 (42.3%) and 8.3 to 10.80 (30.1%) for waves of periods of 1.5 sec, 2.0sec and 2.5sec respectively.
7. In the depth of water of 0.3m, 0.35m and 0.4m, maximum damage levels increase from 8.3 to 10.0 (i.e. 20.48%), 10.8 to 13.4 (i.e. 24.07%) and 8.05 to 14.5 (i..e. 80.1%)respectively.
8. Zero damage wave heights (H_{zd}) are minimal at a depth of 0.4m and a wave period (T) of 2sec indicating least breakwater stability. Minimum stability of the breakwater i. e. $S = 1.32$ to 1.57 occurs for $4.3 < \xi < 4.8$.
9. Zero damage wave heights are up to 30.6% lower and 8.1% higher than the design wave of 0.1m.

10.3.2 Submerged reef

1. The optimum armour stone weight for a stable reef is 30gm.
2. Reef may be located within a maximum distance of 4m (i. e. X/d of 13.33) seaward of the main breakwater for major experimental work of involving protected (defenced) breakwater as waves are attenuated by about 50%, beyond which there is no significant increase in wave attenuation.

10.3.3 Protected breakwater with a reef of crest width 0.1m (i. e. B/d = 0.25 to 0.33) located at a seaward spacing of 1m (i. e. X/d = 2.5 to 3.3)

1. The transmission coefficient (K_t) decreases with increase in H_o/gT^2 and h/d and decreases with decrease in F/H_i and d/gT^2. But the trend of variation of K_t with B/L_o is unclear.
2. K_t drops from 0.95 to 0.82 (13.68%), 0.95 to 0.87 (8.42%) and 1.0 to 0.94 (6.0%) for h/d of 0.833, 0.714 and 0.625 i.e. depths of 0.3m, 0.35m and 0.4m respectively.
3. For $-0.312 < F/H_i < -1.5$ and all ranges of d/gT^2, K_t increases from 0.826 to 1.0 (21%), 0.84 to 1.0 (19%) and 0.858 to1.0 (16.55%) for 1.5sec, 2sec and 2.5sec respectively.
4. The maximum run up and rundown are respectively 1.56 times and 0.90 times the deep water wave height.
5. For a depth of water of 0.3m the relative run up is 5% to 15% less and relative rundown is 17% to 28% less compared to conventional breakwater where as for other depths of water there is no significant reduction.
6. The damage level S increases with the increase in H_o/gT^2, F/H_i, d/gT^2, B/L_o and decrease in h/d.
7. The maximum damage levels increase from 5.0 to 6.78 (35.6%), 7.18 to 12.35 (72%) and 8.67 to 13.89 (60.21%) for the depths of 0.3m, 0.35m and 0.4m (i.e. h/d of 0.833, 0.714 and 0.625) respectively.
8. As $0.01 < B/L_o < 0.0285$, considering all ranges of H_o/gT^2 , damage levels increase from 0.32 to 6.78, 0.55 to 12.35 and 1.31 to 13.89 for depths of 0.3m, 0.35m and 0.4m (h/d of 0.833, 0.714 and 0.625) respectively.
9. Maximum damages to defenced breakwater are reduced by about 4% to 41% compared to conventional breakwater.
10. Zero damage wave heights are 2.3% to 56% higher than those obtained for conventional breakwater.

10.3.4 Protected breakwater with a reef of crest width 0.1m (i. e. B/d = 0.25 to 0.33) located at a seaward spacing of 2.5m (i. e. X/d = 6.25 to 8.33)

1. The transmission coefficient (K_t) decreases with increase in H_o/gT^2, B/L_o and h/d and decreases with decrease in F/H_i and d/gT^2.
2. K_t drops from 0.76 to 0.66 (13.1%), 0.79 to 0.66 (16.4%) and 0.84 to 0.68 (19%) for h/d of 0.833, 0.714 and 0.625 (i.e. depth of water of 0.3, 0.35 and 0.4m) respectively.
3. K_t increases from 0.64 to 0.7 (9.4%), 0.696 to 0.8 (14.9%) and 0.71 to 0.84 (18.3%) for wave periods 1.5sec, 2sec and 2.5sec, respectively.
4. The maximum run up and run down are respectively 1.24 times and 0.73 times the deep water wave height.
5. Run up and rundown are reduced 35% to 38.6% and 6% to 30.8% respectively compared to conventional breakwater.
6. The damage level S increases with the increase in H_o/gT^2, F/H_i, d/gT^2, B/L_o and decrease in h/d.
7. Damages at depth of water of 0.3m are nil.
8. At the depth of water of 0.35m the maximum damage level of the breakwater increases from 3.58 to 6.28 (i. e. a rise of 75.4%) and at a depth of 0.4m damage level increases from 5.31 to 8.7 (i. e. a rise of 63.8%).
9. As the depth of water increases from 0.35m to 0.4m (i.e. $0.005<d/gT^2<0.015$ to $0.006<d/gT^2<0.018$) and $1.45 \times 10^{-3} < H_o/gT^2 < 7.85 \times 10^{-3}$, the maximum damage level increases from 6.28 to 8.7 (i. e. a rise of 38.5%), 4.94 to 7.06 (i. e. 42.9%) for 3.58 to 5.31 (i. e. 48.3%) for a wave period of 1.5sec, 2sec and 2.5sec respectively.
10. Damages at depths of water of 0.35m and 0.4m are 40% to 66% less compared to conventional breakwater.
11. Zero damage wave heights are 8.7% to 75% higher than the conventional breakwater.

10.3.5 Protected breakwater with a reef of crest width 0.2m (i. e. B/d = 0.50 to 0.67) located at a seaward spacing of 2.5m (i. e. X/d = 6.25 to 8.33)

1. K_t drops from 0.57 to 0.43 (24.6%), 0.75 to 0.53 (29.3%) and 0.887 to 0.6 (32.6%) for h/d of 0.833, 0.714 and 0.625 (i.e. depth of water of 0.3, 0.35 and 0.4m) respectively.

2. K_t increases from 0.43 to 0.7 (62.7%), 0.43 to 0.72 (67%) and 0.49 to 0.887 (81%) wave periods of 1.5sec, 2sec and 2.5sec respectively.
3. The maximum run up and run down are respectively 1.06 times and 0.54 times the deep water wave height.
4. Relative run up and run down are 37.5% to 48.7% and 38.5% to 75% lower than those of conventional breakwater.
5. At depths of water of 0.3m and 0.35m, damages to the breakwater are nil and negligible respectively.
6. Damages S of 3.06 and 3.4 occur at $0.006 < d/gT^2 < 0.018$ i.e. in depth of water of 0.4m (h/d = 0.625) which are 75.42% and 76.55% lower compared to conventional breakwater.
7. Zero damage wave heights are 60.38% higher than the conventional breakwater.

10.3.6 Protected breakwater with a reef of crest width 0.3m (i. e. B/d = 0.75 to 1.0) located at a seaward spacing of 2.5m (i. e. X/d = 6.25 to 8.33)

1. K_t drops from 0.55 to 0.4 (27.27%), 0.62 to 0.48 (22.58%) and 0.8 to 0.48 (40%) for h/d of 0.833, 0.714 and 0.625 (i.e. depth of water of 0.3, 0.35 and 0.4m) respectively
2. K_t increases from 0.43 to 0.71 (65.1%), 0.4 to 0.7 (75%) and 0.41 to 0.8 (95.1%) for wave period of 1.5sec, 2sec and 2.5sec respectively.
3. The maximum run up and run down are respectively 0.97 times and 0.44 times the deep water wave height.
4. For the parameters considered in the study, the relative run up and run down are 36% to 55% lower and 51% to 73% lower compared to that of the conventional breakwater.
5. The breakwater damages, at all the depths of water, are negligible. Therefore, the defenced breakwater is completely protected from all the waves and the structure is totally safe for test parameters considered.

10.3.7 Protected breakwater with a reef of crest width 0.4m (i.e. B/d = 1.0 to 1.33) located at a seaward spacing of 2.5m (i. e. X/d = 6.25 to 8.33)

1. K_t decreases from 0.55 to 0.38 (30.9%), 0.64 to 0.53 (17.19%),) and 0.708 to 0.55 (22.32%),) for h/d of 0.833, 0.714 and 0.625 i.e. for depths of water of 0.3m, 0.35m and 0.4m respectively.

2. K_t increases from 0.38 to 0.69 (81.6%), 0.4 to 0.65 (62.5%) and 0.42 to 0.708 (68.6%) for wave periods of 1.5sec, 2sec and 2.5sec respectively.
3. K_t decreases with an increase in B/L_o and with an increase in range of H_o/gT^2. But this is clearly seen only for depth of water of 0.35m.
4. The maximum run up and run down are respectively 0.99 times and 0.53 times the deep water wave height.
5. Relative run up and run down are 36% to 51% and 33% to 75% less compared to that of the conventional breakwater.
6. Damages at depths of water of 0.3m are nil.
7. Noticeable breakwater damage levels of 2.12 and 2.75 and 3.97 occur for depths of 0.35m and 0.4m which are 85.4% lower and 72.6% to 77.9% lower respectively when compared to those of the conventional breakwater.
8. The zero damage wave height (H_{zd}) for depths of water of 0.35m and 0.4m are respectively 74% and 49.03% higher than those of the conventional breakwater.

10.3.8 Protected breakwater with a reef of crest width 0.1m (i. e. B/d = 0.25 to 0.33) located at a seaward spacing of 4.0m (i. e. X/d = 10.0 to 13.33)

1. K_t drops from 0.602 to 0.51 (15.8%), 0.67 to 0.56 (16.4%) and 0.72 to 0.59 (18%) for h/d of 0.833, 0.714 and 0.625 i.e. for depths of water (d) of 0.3m, 0.35m and 0.4m respectively.
2. K_t increases from 0.51 to 0.606 (18.8%), 0.55 to 0.68 (23.6%) and 0.587 to 0.72 (22.6%) respectively for 1.5sec, 2sec and 2.5sec respectively
3. The maximum run up and run down are respectively 1.0 times and 0.65 times the deep water wave height.
4. Run up and run down are reduced by 33% to 61% and 15% to 49% respectively compared to conventional breakwater.
5. Damages at depth of water of 0.3m are nil.
6. For depth of 0.35m and 0.4m (i.e. h/d = 0.714, 0.625), maximum damage varies from 2.1 to 3.23 (i. e. a rise of 53.8%) and 3.23 to 5.35 (i. e. by 65.6%) respectively.
7. Considering the complete variation H_o/gT^2 and all the ranges of d/gT^2 (i.e as the depth of water increased from 0.35m to 0.4.), for the wave period of 1.5sec, the maximum damage level of the breakwater increases from 3.23 to 5.35 (65%), it increases from 2.26 to 4.59 (103%) for the wave period of 2.0sec while maximum damage level rises from 2.1 to 3.23 (53.8%) for the wave period of 2.5sec.

8. Damages at depths of water of 0.35m and 0.4m are about 63% to 80% less compared to conventional breakwater
9. Zero damage wave heights are 43.96% to 91% higher than that of the conventional breakwater.

10.3.9 Protected breakwater with a reef of crest width 0.2m (i. e. B/d = 0.5 to 0.67) located at a seaward spacing of 4.0m (i. e. X/d = 10.0 to 13.33)

1. K_t decreases from of 0.53 to 0.37 (30.2%), 0.62 to 0.4 (35.5%) and 0.7 to 0.55 (21.4%) for h/d of 0.833, 0.714 and 0.625 i.e. for water depth of 0.3m, 0.35m and 0.4m respectively.
2. For wave periods of 1.5sec, 2sec and 2.5sec, K_t increases from 0.37 to 0.63 (70.3%), 0.4 to 0.65 (62.5%) and 0.42 to 0.70 (66.7%) respectively for all the ranges of d/gT^2 and F/H_i.
3. The maximum run up and run down are respectively 0.98 times and 0.53 times the deep water wave height.
4. Run up and run down are 31%p to 50% and 16% to 74% lower compared to that of the conventional breakwater.
5. Damages at depths of water of 0.3m and 0.35m are nil.
6. Breakwater damage level (S) of 2.31 occurs at depth of water of 0.4m which is about 84% less compared to conventional breakwater.
7. Zero damage wave height is 88% higher than that of the conventional breakwater.

10.4 CONCLUSIONS FOR PROTECTED BREAKWATER

The conclusions for protected breakwater, with a conventional inner (main) breakwater and a submerged reef of height (h) of 0.25m with a varying crest width (B) of 0.1m, 0.2m, 0.3m and 0.4m (i.e. B/d varying between 0.25 and 1.33) located seaward of the breakwater at a spacing (X) from 1.0m , 2.5m and 4.0m (i.e. X/d varying between 2.5 and 13.33), are summarized and compared with those of the conventional breakwater and are listed in Table. 10.1.

Table. 10.1. Comparison of conclusions for conventional breakwater (CBW) and. Protected breakwater

Sl. No.	Parameter	CBW	Protected breakwater with a submerged reef of crest width B (m) located at seaward spacing of X (m)						
			B=0.1 X=1.0	X=2.5				X=4.0	
				B=0.1	B=0.2	B=0.3	B=0.4	B=0.1	B=0.2
1	Max. R_u/H_o	1.57	1.56	1.24	1.06	0.97	0.99	1.0	0.98
2	Present R_u/H_o compared with CBW (% Lower)	--------	5 - 15	35- 38.6	37.5- 48	36- 55	36- 51	33 - 61	31 - 50
3	Max. R_d/H_o	0.95	0.9	0.73	0.54	0.44	0.53	0.65	0.53
4	Present R_d/H_o compared with that of CBW (% Lower)	--------	17 - 28	6- 30.8	38.5- 75	51- 73	33- 75	15 - 49	16 - 74
5	$K_t(X\ 10^{-2})$	--------	82 - 100	66 - 84	43 - 89	40 - 80	38 - 71	51 - 72	37 -70
6	K_t $(X\ 10^{-2})$ at d = 0.30m d = 0.35m d = 0.40m	--------	82-95 87-95 94-100	66 -76 66 - 79 68 - 84	43-57 53 -75 60 - 88.7	40- 55 48 -62 48 - 80	38 -.55 53 - 64 55 – 71	51–60.6 56 - 67 59 - 72	37 - 53 40 -62 55 - 70
7	K_t $(X\ 10^{-2})$ at T = 1.5sec T = 2.0sec T = 2.5sec	--------	82.6-100 84-100 85.8-100	64 - 71 69.6 - 80 71 -84	43- 70 43-72 49 -88.7	43 - 71 40 - 70 41 -80	38 - 69 40 - 65 42 -71	51– 60.6 55 - 68 58.7 -72	37 -63 40 - 65 42 -70
8	Max. S at d = 0.30m d = 0.35m d = 0.40m	8.3-10 10.8-13.4 8.05-14.5	5.0-6.78 7.2-12.4 8.7-13.9	Nil 3.6-6.28 5.3-8.7	Nil Neg 0-3.4	Nil Nil Nil	Nil 0-2.12 0-3.97	Nil 2.1-3.23 3.23-5.35	Nil Nil 0.5-2.3
9	Max. S at T = 1.5sec T = 2.0sec T = 2.5sec	9.0-14.5 10 -14.2 8.3- 10.8	6.8 -13.9 5.9 -12.6 5.0 -8.67	6.3- 8.7 4.9- 7.06 3.6- 5.31	0 to 3.4 Neg Neg	Nil Nil Nil	0 - 3.97 Nil Nil	3.23-5.35 2.26- 4.59 2.1 - 3.23	0.5-2.3 Nil Nil
10	Present S compared with that of CBW (% Lower)	--------	4 - 41	40 - 66	75.4–76.6	100 No damage	72.6-85.4	63 - 80	84
11	Present H_{zd} compared to CBW (% Higher)	--------	2..3 - 56	8.7 -75	60.38	PBW is safe	49 - 74	43.96 - 91	88

It can be seen from the table that the damage of the conventional breakwater reduced by 4% to 41% as the breakwater is protected by submerged reef of crest width 0.1m, built at a seaward spacing of 1.0m. Then, keeping the crest width of the submerged reef constant at 0.1m and increasing the spacing between the structures (X) from 1.0m to 2.5m, reduced the damage of the inner main breakwater up to 66%. Further, for the same spacing (X) of 2.5m between the structures, increasing the reef crest width from 0.1m to 0.3m reduced the damage of the protected conventional (main) breakwater to zero. Then, as the reef crest width is increased from 0.3m to 0.4m for the same spacing of 2.5m, damage of breakwater increased from zero to about 15% to 28% of the conventional breakwater (i.e. a reduction of 72.6% to 85.4%). It can also be seen from the table that the damage of the conventional breakwater increased to 20% to 37% (i.e. a reduction of 63% to 80%) and 16% (i.e. a reduction of 84%) of the conventional breakwater respectively for a submerged reef of crest width 0.1m and 0.2m, located seaward at a spacing (X) of 4m. The model tests show that a submerged reef of crest width 0.3m located at a seaward spacing of 2.5m completely protects the conventional (main) breakwater for the test conditions considered. For this scenario, it is observed that, the optimum range of relative reef crest widths are $0.035 \leq B/L_o \leq 0.045$ and $0.6 \leq B/d \leq 0.75$.

10.5 DESIGN OF DEFENCED BREAKWATER

From the model tests of the protected breakwater, it is concluded that the geometry of the protected breakwater with a conventional breakwater and a submerged reef of crest width 0.3m located at a seaward spacing of 2.5m is totally free from damage. This is because the reef breaks most of the incident waves, dissipates their energy while the transmission coefficient varies between 0.4 and 0.8 and completely safeguards the breakwater.

Considering the above details, a stable defenced breakwater is designed with a trapezoidal conventional breakwater and submerged reef separated by a spacing of 2.5m. The breakwater has a slope of 1V:2H and primary armour of stones of 36gm (i.e. nominal diameter of 0.0235m) which is designed for a wave height 0.08m (i.e. assuming a reduction of 20% in the design wave height of 0.1m for a K_t value of 0.8). This gives an armour weight of about 50% of that of original conventional breakwater. Considering the maximum run up of 0.1m for this geometry of the main breakwater (from the test observation), a crest height of 0.5m is fixed. The reef has trapezoidal cross section of slope 1V:2H, 0.25m high and a crest width of 0.3m is designed and the armour stone is weight 30gms.

10.5.1 Model test of defenced breakwater

This model is tested for the similar test conditions as the earlier model studies. It is found that the damages of the main breakwater are zero and the reef completely protects the breakwater. Further, sufficient number of waves of height 0.1m (design wave) are generated in depth of water of 0.3m and 0.4m to break over the inner (main) breakwater, so that reflection from the structure resulted a wave of height of about 0.13m breaking at the reef while a wave of 0.12m impinging on the inner (main) breakwater. It is found that, the damages to inner (main) breakwater are nil and this geometry of defenced breakwater is safe for these test conditions. Then, the same defenced breakwater is subjected to waves of height 0.1m (design wave) which are generated in a depth of water of 0.3m and 0.4m continuously, such that partial clapotis is formed. This resulted in a maximum wave height of 0.149m breaking at the submerged reef and transmitting a wave of about 0.13m which impinged on the inner (main) breakwater. It is found that, this geometry of defenced breakwater is safe for these test conditions even after passing 3,000 waves.

10.6 COST ANALYSIS

The cost analysis of the prototype of the defenced breakwater (details of which are given in Appendix II) is undertaken. It shows that the defenced breakwater is about 21.08% to 27.2% economical than the conventional rubble mound breakwater (depending on the site conditions), both designed for the same operating conditions.

10.7 SCOPE FOR FUTURE WORK

The similar model studies may be undertaken on the protected breakwaters with:

1. Varying structure slopes such as 1V:1.5H, 1V: 2.5H and 1V:3H and economical and safe geometries of the defenced breakwaters may be evolved.
2. Artificial armour units like concrete cubes etc.

Appendix 1
References

Abernethy, R. B., Benedict, R. P., and Dowdell, R. B. (1985). "ASME measurement of uncertainty", *J. of Fluids Engg.,* 107, June, 161 – 164.

Adee, B. H. (1976). "Floating breakwater performance", *Proc., 15th Conf. on Coastal Engg.,* 3, 2777 – 2791.

Ahrens, J. P. (1970). "The influence of breakwater type on rip rap stability", *Proc., Int. Conf. on Coastal Engg.,* 3, 1557 – 1566.

Ahrens, J. P. (1984). "Reef type breakwaters", Proc., *Conf. on Coastal Engg.*, 2648 – 2662.

Ahrens, J. P. (1989), "Stability of reef breakwater", *J. of Waterw., Port, Coastal and Ocean Engg.,* 115(2), 221 – 234.

Ahrens, J. P., and Mc Cartnery,. H. L. (1975*)* " Wave period effect on the stability of riprap", *Proc., Conf. on Coastal Engineering in the Ocean,* ASCE, 3.

Armono, H. D., and Hall, K. R. (2002). "Wave transmission on submerged breakwaters made of hollow hemispherical shape artificial reef", *Proc., Canadian Coastal Engg. Conf.,* www.Reefball.com/submerged breakwater/Armono %20 and %20 Hall. Pdf.

Baba, M. (1985). "Design of submerged breakwater for coastal protection and estimation of wave transmission coefficient", *Proc., 1st Nat. Conf. on Dock and Harbour Engg.*, I.I.T., Bombay, Dec, B-13 – B-26.

Baba, M. (1986). "Computation of wave transmission over a shore protecting submerged breakwater", *J. of Ocean Engg.,* 13(3), 227 – 237.

Baird, W. F., and Hall, K. R. (1984). "The design of breakwaters using quarried stones", *Proc., 19th Conf. on Coastal Engg.,* 3, pp 2580 – 2591.

Barends, F. B. J. (1986). "Geotechnical aspects of rubble mound breakwaters", *Developments in Breakwaters,* Thomas Telford Ltd., London, 155 – 174.

Battjes, J. A. (1974) "Surf similarity", *Proc., 14th Coastal Engg Conf.* 1, Ch. 26, Denmark, The Netherlands.

Battjes, J. A., and Jansen, J.P.F.M. (1978). "Energy loss and setup due to breaking of random waves", *Proc., !0th Coastal Engg. Conf.*, 569 – 587.

Belfadhel, M. B., Lefebure, G., and Rohan , K. (1996). "Comparision and evaluation of different rip rap stability formulas using field performance", *J. of Waterw., Port, Coastal and Ocean Engg.*, 122(1), 8 – 14.

Bierawski, L. G., and Maeno, S. (2002). "Water pressure fluctuations around a submerged breakwater", *Proc., Symposium on Dynamic Water Pressure and Bed response in water, DPRI,* Kyoto University, Japan, Dec., 45 -54.

Billal, A. M., and Richard, M. H. (1997). "Probability stati*stics and reliability for engineers*", *CRC Press,* LLC 2000 Corporate Blvd., Florida, USA.

Bisher, I., Soroosh, S., Thomas, M., Maral, S., Henk, W., and Bob. (1999). "Comparison of pedotransfer functions to compute water holding capacity using the Van Genuechters Model in inorganic soils", *Report to IGBP – DIS Soil Data Tasks,* , May, 5 – 17.

Booth, W. H. (1960). "Discussion on Laboratory investigation on rubble mound breakwaters", *J. of Waterw., Harbours and Coastal Engg. Division,* 1, 86.

Bradbury, A.P., and Allsop, W. H. (1988). "Hydraulic effects of breakwater crown walls", *Design of breakwaters,* Thomas Telford, London.

Bremner, W. D., Foster, N., Miller, C. W., Wallace, B. C. (1980). "The design concept of dual breakwaters and its application to Townsville, Australia", *Proc., 17th Coastal Engg. Conf.* Sydney, Australia, 2, 1898-19908.

Bruun, P. and Gunbak, Ali Riza. (1976). "New design principles for rubble mound structure", *Proc., 15th Coastal Engg Conf.*, 3, 2429 – 2473.

Bruun, P., and Johannesson, P. (1976). "Parameters affecting stability of rubble mound breakwaters", *J. of the Waterw., Harbours and Coastal Engg. Division,* 103.

Carvalho, J. R., and Vera-Cruz, D. (1960). "Discussion on Laboratory investigation on rubble mound breakwaters", *J. of Waterw., Harbours and Coastal Engg. Division,* 1, 86.

Carver, R. D., and Davidson, D. D. (1982). "Breakwater stability – Breaking wave data", *Proc., Coastal Engg. Conf.,* 2107 – 2128.

Cesar Vidal, Losada, M. A., and Mansard, E. P. D. (1995). "Suitable wave height parameter for characterizing breakwater stability", *J. of Waterw., Port, Coastal and Ocean Engg.*, 121(2), 88-97.

Chakrabarti, S. K. (1996). "Experimental techniques in offshore engineering", *Proc., Int. Conf. in Ocean Engg., COE-96,* I.I.T. Madras, Chennai, Dec., 631 – 642.

Chandrashekar, C. S., Ligteningen, H., and Mol, A. (1985). "Rubble mound breakwater in deep water structural design", *Proc., 1st Nat. Conf. on Dock and Harbour Engg.,* IIT Bombay, 1, B-207 – B-218.

Chen, K., and Weigel, R. L. (1976). "Floating breakwater for reservoir marinas", *Proc., 15th Conf. on Coastal Engg.,* 3, 1647 – 1666.

Contract C-2 (1994-1995). "Extension of northern and southern breakwaters- Detailed cost estimate", *Port Facilities for Handling Crude and POL Products for the New Refinery (MRPL) at Mangalore,* New Mangalore Port Trust, Panambur , Karnataka, India CER No-45/1994-95.

Cornett, A., Mansard, E., Funke, E. (1993). "Wave transformation and load reduction using a small tandem reef breakwater-Physical model tests", *Proc., Int. Conf. on Waves-93,* New Orleans, USA, 1008- 1023.

Cox, J. C., and Clark, G. R. (1992). "Design, development of a tandem breakwater system for Hammond, Indiana", *Coastal Structures and Breakwaters,* Thomas Telford Ltd., London, 111 – 121.

Daemrich, K. F., Mai, S., and Ohle Nino. (2001). "Wave transmission at submerged breakwater", *Proc., 4th Int. Symposium on Ocean Wave measurement and Analysis,* Sanfransisco, U.S.A.

Dattatri, J. (1978). "Analysis of regular and irregular waves and performance characteristics of submerged breakwaters", *Ph.D Thesis,* Dept. of Civil Engg. IIT Madras, Chennai, India.

Dattatri, J., Raman, H., and Jothi Shankar, N. (1978). "Performance characteristics of submerged breakwater", *Proc., 16th Coastal Engg. Conf,* Hamburg, 2153 – 2171.

Davidson, D. D., and Markle, D. G. (1976), "Effects of broken dolosse on breakwater stability", *Proc., 15th Conf. on Coastal Engg.,.* 3, 2544 – 2563.

Delft Hydraulics. (2001). "Gibraltar beach development". *Project description H590,* www.w/delft.nl/proj/4uk00091.scherm.pdf.

Dharma, S., and Hall, K. (2004). "Diffraction effect on wave transmission at submerged breakwaters", *Proc., 29th Int. Conf. on Coastal Engg.,* Lisbon, Sept.

Dick Milne, T., and Brebner, A. (1968), "Solid and permeable submerged breakwater", *Proc., 11th Conf. on Costal Engg.,* 1141 – 1158.

Diephuis, J. G. H. R. (1957). "Scale effects involving the breaking of waves", *Proc., 6th Conf. on Coastal Engg.,* 194-201.

Diskin, M. H., Vajda, M. L., and Amir, I. (1970). "Piling up behind low and submerged permeable breakwaters", *J. of Waterw., and Harbour Division,* 96(2), 359 – 371.

Edge, B. L., and Magoon, D. T. (1979). "A review of recent damages to coastal structures", *Proc., Conf. on Coastal Structures,* 333 – 346.

Edge, B. L., Housley, J. G., and Watts, G. M. (1976). "Low cost shoreline protection", *Proc., 15th Conf. on Coastal Engg.,* 3, 2888 – 2903.

Ergin Aysen and Pora Serif. (1971). "Irregular wave action on rubble mound breakwater", *J. of Waterw. Harbour and Coastal Engg. Division,* 97 (2), 279 - 293.

Ergin Aysen. (1989). "Rubble mound breakwater with S-shaped design", *J. of Waterw., Port, Coastal and Ocean Engg,* 115(5), 579 – 590.

Espace Pur. (2003). "Presentation of stabiplage technology", *unitrac.paris@wanadoo.fr., /* stabiplage@wanadoo.fr, 1-11.

Font, J. B. (1970). "Damage functions for a rubble mound breakwater under the effect of seawalls", *Proc., Conf. on Coastal Engg..* 3, pp 1567 – 1585.

Franco, L., Gentilomon, Noli, A., and Passcantando, G. (1996). "Constructional Aspects of some large rubble mound breakwaters in Northern Africa", *Advances in Coastal Structures and Breakwaters,* Thomas Telford, London, 159 – 173.

Franco, L. (2001). "New Italian experience in the design of maritime structures", *Proc., Int. Conf. in Ocean Engg., COE -01,* I.I.T. Madras, Chennai, Dec., K-81 – K-93.

Fuhrboter, A., Dette, H. H., and Greene, J. (1976). "Response of seadykes due to wave impacts", *Proc., 15th Conf. on Coastal Engg.,* 3, 2604 – 2622.

Fullford, E. T. (1985). "Reef type breakwater for shoreline stabilization", *Coastal Zone-'85,* 1776 – 1795.

Gadre, M. R., Poonawala, I.Z., Kale A.G., Kudale M.D.(1989). "Rehabilitation of rubble-mound breakwater", *Proc., 3rd Nat. Conf. on Dock and Harbour Engg,* Karnataka Regional Engineering College (K.R.E.C), Surathkal, Srinivasnagar, India, Dec, 387 - 393.

Gadre, M. R., Poonawala, I. Z., and Kudale, M. D. (1985). "Design of reclamation bunds for Madras Port", *Proc., 1st Congress of the Asian and Pacific Regional Division of the Int. Assn. for Hydraulic Research*, IIT Bombay, Oct, B-113 – B-123.

Gadre, M. R., Poonawala, I. Z., and Kudale, M. D. (1991). "Berm breakwater concept in the design of rubble mound breakwaters", *Proc. 4th Indian Conf. on Ocean Engg.*, INCOE – 91, Goa, Sept, 427 - 433.

Gadre, M. R., Poonawala, I. Z., and Kudale, M. D. (1992). "Stability of rock armour protection for submarine pipeline", *Proc., 8th Congress of the Asian and Pacific Regional Division of the Int. Assn. for Hydr. Res.*, CWPRS, Pune, India, Oct, 3, D-149 – D-160.

Gadre, M. R., Somayaji, N., and Kale, A. G. (1985a). "Stones in breakwaters and seawalls", *Proc., 1st Conf. on Dock and Harbour Engg*, IIT Bombay, Mumbai, India, Dec, 1, B-107 – B-112.

Gadre, M. R., Somayaji, N., Poonawala, I. Z., and Kudale, M. D. (1987). "Use of sand filled rubberized coir bags in coastal protection works", *Proc., Conf. on Coastal and Port Engg. In Developing Countries,* Indonesia, 1, 166 – 180.

Gerloni, M`. De., Cris, Enel., and Franco, P. G. (1992). "Safety of breakwaters against overtopping", *Coastal Structures and Breakwaters,* Thomas Telford, London.

Goda, Y. (1996). "Wave damping characteristics of longitudinal reef system", *Advances in Coastal Structures and Breakwaters,* Thomas Telford, London, 192 – 207.

Groeneveld, R. L., Mol, A., and Niuewenhuys. (1984). "Rehabilitation methods for damaged breakwaters", *Proc., 19th Conf. on Coastal Engg.*, 3, 2467 – 2486.

Gutman, M. L. (1979), "Low cost shoreline protection in Massachusetts", *Proc., Special Conf. on Offshore Coastal and Marine Structures",* 1, 373 – 387.

Haan de Wilbe. (1991). "Deterministic computer aided optimum design of rock rubble mound breakwater cross section", *Proc., Conf. on Coastal Engg.*, 15, 3 – 19.

Hall, K. R., and Kao, J. S. (1991). "The influence of armour stone gradation on dynamically stable breakwaters", *Proc. of Coastal Engg. Conf.,* The Netherlands, 15, 333 – 346.

Harris, L. E. (1996). "Wave attenuation by rigid and flexible submerged breakwaters", *Florida Atlantic University*, Boca Raton, FL, 158.

Harris, L. E. (2003). "Artificial reef structures for shoreline stabilization", *Proc., 3rd Int. Surfing Reef Symposium* Raglan, New-Zealand, June, 176 – 179.

Hayashi, T., Hattori, M., Kano, T., and Shirai, M. (1966). "Hydraulic research on the closely spaced pile breakwater", *Proc., 10th Conf. on Coastal Engg.,* Tokyo, Japan, 873 – 884.

Hedar, .P. A. (1986). "Armour layer stability of rubble mound breakwaters", *J. of Waterw., Harbours and Coastal Engg. Division,* 112(3).

Hegde, A. V. (1996). "Studies on deterministic optimum Computer Aided Design on non-overtopping rubble mound breakwaters", *Ph.D Thesis submitted to Mangalore University*, KREC, Surathkal, India.

Hegde, A. V., and Samaga, B. R. (1996). "Study on the effect of core porosity on rubble mound breakwater", *Proc., 10th Congress of the Asian and Pacific Regional Division of the Int. Assn for Hydr.Res.,*, Langkawi, Malaysia, 1, 287 – 294.

Hegde, A. V., Shirlal, K. G., and Rao, S. (1997). "Effects of fines in core on stability of rubble mound breakwaters - State of Art", *Proc., 2nd Nat. Conf. on Harbour and Ocean Engg.,* Thiruvananthpuram, Dec., 261 – 268.

Hegde, A. V., Shirlal, K. G., and Rajaraman, J. (2002). "Brerm breakwaters-Effects of berm width on stability", *Proc., Int. Conf. on Ship and Ocean Technology SHOT-02.,* I.I.T, Kharagpur, Indai, Dec.,

Hoedtden, G., Metz, H. E., and Voskamp, W. (1987). "Cost effective building of breakwater using geotextiles", *Proc., Conf. on Coastal and Port Engg. in Developing Countries,* Indonesia,. 1, 538 – 550.

Homma Masashi, and Sakou Toshitsugu. (1959). "An experimental study on the submerged breakwater", *Coastal Engg. Conf. in Japan*, 11, 103 – 109.

Homma Masashi and Horikawa Kiyoshi (1961) "A study on submerged breakwaters", Coastal Engineering *in Japan*, 4.

Hookway, D.W., and Brinson, A. G. (1986). "Construction of rubble mound breakwaters at Ras Lanuf, Libya ", *Developments in breakwaters,* Thomas Telford Ltd., London.

Horikawa, K., and Kuo, C. T. (1966). "Study on wave transformation inside surf zone", *Proc., Conf. on Coastal Engg.* in Japan, 217 – 233.

Hudson, R. Y. (1953). "Wave forces on breakwaters", *Transactions, ASCE,* 118(3), 653 – 674.

Hudson, R. Y. (1959). "Laboratory investigation of rubble mound breakwaters", *J. of Waterw. and Harbour Division,* 85(3), Sep, 93 – 121.

Hughes, S. A. (1993). "Physical models and laboratory techniques in Coastal Engg.", *Advanced series on Ocean Engg.,* , World Scientific, Singapore 7.

Hunt, I. A. (1959). "Design of seawalls and breakwaters", *J. Waterw. and Harbour Division,* 85(3), 123 – 152.

Hutchinson, P. S., and Raudkivi, A. J. (1984). "Case history of a spaced pipe breakwater at half-moon bay Marina Auckland, New-Zealand", *Proc., 19th Coastal Engg. Conf.,* Houston, Texas, 2530 – 2535.

John, E. J., and Singh Ajay Pal. (1991). "Stability analysis of rubble mound breakwaters for Karwar Naval Harbour", *Proc., 4th Indian Conf. on Ocean Engg.*, INCOE – 91, Goa, Sept, 435 – 439.

Johnson, J. W., Fuchs, R. A., and Morison, J. R. (1951). "The damage action of submerged breakwater", *Transaction, Americal Geophysical Union* 2 (5), 704 – 718.

Johnson, R. R., Mansard, E. P. D., and Ploeg, J. (1978). "Effect of wave grouping on breakwater stability", *Proc., 16th Coastal Engg. Conf.*, 2, 2228 – 2242.

Kaldenhoff, H. (1996). "Recent developments in the construction of breakwaters", *Proc., of Int. Conf. in Ocean Engg., COE,* I.I.T. Madras, Chennai, Dec., 681 – 693.

Kale, A. G., and Gadre, M. R. (1989). "Construction of offshore structures for beach protection", *Proc., 3rd Nat. Conf. on Dock and Harbour Engg.*, K.R.E.C Surathkal, India, Dec.,577 – 582.

Khader Abdul, M. H., and Rai, S. P. (1980). "A study of submerged breakwaters" *J. of Hydr. Res.,,* 18(2).

Kline, S. J. (1985). "The purposes of uncertainty analysis", *J. of Fluids Engg.,* 107, June, 153 - 160.

Kobayashi, N., and Wurjanto, A. (1989). "Wave transmission over submerged breakwater", *J. of Waterw., Port, Coastal and Ocean Engg.* 115(5), 662 – 680.

Kondo, H., Toma, S., and Yanno, K. (1976). "Laboratory study on pervious core breakwaters", *Proc., 15th Conf. on Coastal Engg,* 3, 2643 – 2661.

KREC Study Team. (1994). "Study on coastal erosion (D.K. District)", *Input to environmental master plan study* ,Danida-Dee- Mangalore, Department of Applied Mechanics and Hydraulics, KREC, Surathkal, India.

Kreeke Van De Jacobus. (1969). "Damage function of rubble mound breakwaters", *J. of Waterw., Harbours and Coastal Engg. Division,* 95, Aug.

Kudale, M. D., and Dattatri, J. (1994). "Mathematical modeling for the design of rubble mound structures", *Proc., Indian Nat. Conf. of Harbour and Ocean Engg.,* 2, J-115 – J-123.

Le Mehaute, B. (1976). "Similitude in coastal engineering", *J. of Waterw., Port, Coastal, and Ocean Engg., Division,* 102(3), 317-335.

Hookway, D. W., and Brinson, A. G. (1986). "Construction of rubble mound breakwaters at Ras Lanuf, Libya", *Development in breakwaters,* Thomas Telford Ltd., London.

Ligteringen, H., and Chandrasekhar, C. S. (1989). "Sines west breakwater, the final solution", *Proc., 3rd Nat. Conf. on Dock and Harbour Engg.* Dec., KREC Surathkal, India, 929 – 937.

Longuet Higgins, M. S. (1976). "Recent development in the study of breaking waves", *Proc., 15th Conf. on Coastal Engg.,* 1, 441 – 460.

Losada, M. A. (1991) "Recent developments in the design of mound breakwaters", *HandBook of Coastal and Ocean Engg.,* 1, Gulf Publishing Company, Houstan, Texas, USA.,

Losada, J., Silva, K., and Losada, M. A. (1996). "3D non-breaking regular wave interaction with submerged breakwaters", *Coastal Engg.,* Elsevier Science, 28, 229 -248.

Lundgren, H., and Jacobsen, H. C. L. (1987). "Breakwaters on weak soils", *Proc., Conf .on Coastal and Port Engg. in Developing Countries,* Indonesia, 1, 779 – 793.

Madsen, O. S., and White, S. M. (1976). "Wave transmission through trapezoidal breakwaters", *Proc., of 15th Conf. on Coastal Engg.,.* 3, 2662 – 2676.

Mai, S., Ohle, N., and Daemrich, K. F.(1999) "Numerical simulation of wave propagation compared to physical modeling", *www.fi.uni-hannover.de/~material/pdf.*

Maitra, S., and Dattatri, J. (1994). "Rubble mound breakwater stability formulae, An evolution" *Proc., Indian Nat. Conf. of Harbour and Ocean Engg.,* 2, J-205 – J-214.

Mani, J. S., Vijaykumar, S., and Mahadevan, R. (1991). "Diffraction of waves by submerged breakwater", *Proc., 4th Indian Nat. Conf. on Ocaen Engg.,* National Institute of Oceanography, Goa, Sept, 403-407

Mani, J. S. (1993). "Damping of waves by two rows of closely spaced piles", *J. of Institution of Engineers, India,* 73, 155 – 161.

Mani, J. S., and Jayakumar, S. (1995), "Wave transmission by suspended pipe breakwater", *J. of Waterw.,, Port, Coastal, and Ocean Engg. Division,* 121(6), 335 – 338.

Meadowcraft, I. C., Reeve, D. E., Allsop, N. W. H., Diment, R. P., and Cross, J. (1996). "Development of new risk assessment procedure for coastal structures", *Advances in Coastal Structures and Breakwaters,* Thomas Telford, London, 6 – 25.

Melby, J. A., and Mlakar, P. F. (1997). "Reliability assessment of breakwaters", *Technical report CHL-97-9,* U. S. Army Corps of Engineers, U.S. Govt. Printing Office, Washington, D.C., May.

Melby, J. A., and Hughes, S. A. (2003). "Armour stability based on wave momentum flux", *Proc., Int. Conf. on Coastal Structures*, Portland, Oregan, Aug.

Mendez Fernando, J., Losada Inigo, J., and Losada Miguel, A. (2001). "Wave induced mean magnitudes in permeable submerged breakwater", *J. of Waterw. Port, Coastal and Ocean Engg,* 127(1), 7 – 15.

Mettam, J. D. (1980). "Can we develop new breakwater formula", *Proc., Coastal Engg. Conf.*, 2304 – 2322.

Mettam, J. D. (1992). "Analysis of rubble mound breakwaters", *Coastal Structures and Breakwaters,* Thomas Telford, London, 533 – 543.

Misra, S. C. (2001). "Uncertainty analysis in hydrodynamics", *Proc., Int. Conf. in Ocean Engg.,* Dec., I.I.T. Madras, Chennai, Dec., 207 – 214.

Mol, A., Groeneweld, R. R., and Wanders, A. J. (1984). "Safety and reliability of breakwaters", *Proc., 19th Conf. on Coastal Engg.,* 3, 2451 – 2466.

Mol, A., Ligteringen, H., and Paape, A. (1983). "Risk analysis in breakwater design", *Delft Hydr. Laboratory*, Publication No. 301, The Netherlands, 81 – 86.

Murali, K., and Mani, J. S. (1996). "Response of a cage floating breakwater to regular waves", *Proc., of Int. Conf. in Ocean Engg., COE-96,* I.I.T. Madras, Chennai, Dec., 445 – 451.

Nagai, S. (1970). "Research on seawalls", *Proceedings of Coastal Engg.,* 3, 1431 – 1450.

Nagai, S., and Kakuno, S. (1976). "Slit type breakwater – Box type wave absorber", *Proc., 15th Conf. on Coastal Engg.,*. 3, 2697 – 2716.

Nagaraj, M. K., Shirlal, K. G., and Baliga, G. P. (2002). "Sensitivity analysis of design parameter for breakwater", *Proc., Int. Conf. on Ship and Ocean technology, SHOT-02.,* I.I.T, Kharagpur, India, Dec.,

Nagendra Kumar, B., Kalyani, M., and Kathiroli. (2001). "Studies on wave energy dissipation by submerged breakwater", *Proc., Int. Conf. on Ocean Engg.,* IIT Madras, Chennai, India, Dec, 553- 559.

Narasimhan, M. L., Rao Prabhakar. (1991). "Surf zone wave attenuation model through method of characteristics", *Proc., 4th Indian Conf. on Ocean Engg.,* INCOE – 91, Goa, 57 – 64.

Neelamani, S., Sumalatha, B.V., Rao Narasimhan S. (2002). "Hydrodynamics of seawall defenced by a detached breakwater", *Proc., Nat. Conf. on Hydr., Water resources, and Ocean Engineering,* HYDRO – 2002, I.I.T Bombay. Dec, 244-249.

Nizam, and Yuwono, N. (1996). "Artificial reef as an alternative beach protection", *Proc., 10th Congress of Asian and Pacific Division of Int. Assn. for Hydr. Res..*, Aug., Malaysia, 2, 422 – 430.

Ovellet Yuon. (1970). "Considerations on factors in breakwater model tests", *Proc., 12th Conf. on Coastal Engg.,* 3, 1809 – 1825.

Ouellet, Yuon. (1972). "Effects of irregular wave trains on rubble mound breakwaters", *J. of the Waterw., Harbours and Coastal Engg. Division,* 98(1), 1 - 14.

Owen, M. W., and Briggs, M. G. (1986). "Limitations of modeling", *Development in Breakwaters,* Thomas Telford Ltd., London, 91 – 101.

Palmer, R. Q., and Walker, J. R. (1976). "Honolulu reef runway dyke", *Proc., 15th Conf. on Coastal Engg.*, 3, 1629 – 1646.

PIANC. (1976). "Final report of the Int. Commission for the study of waves -Second part", Annex to Bull No. 25, III.

Phadnis, R. N. (1985). "Construction of breakwaters for fishing harbour at Mirkarwada Ratnagiri", *Proc., 1st Nat. Conf. on Dock and Harbour Engg.*, I.I.T., Bombay, Dec, C-183 – C-192

Pilarczyk, K. W., and Zeidler, R. B. (1996). "Offshore breakwater and shore evolution control", *A. Balkema Rotterdam,* The Netherlands.

Poonawala, I. Z. (1993). "Design of marine structures", *Lecture notes on training course on coastal erosion, protection and coastal zone management*. KREC, Surathkal, India, Dec.
Poonawala, I. Z., Kudale, M. D., and Purohit, A. A. (1994). "Effect of seabed slope on stability of armour layer", *Proc., Indian Nat. Conf. of Harbour and Ocean Engg.*, 2, B-97 – B-104.

Poonawala, I. Z., Kudale, M. D., and Purohit, A. A., kulkarni, S. P. (2004). Rehabilitation of rubble mound breakwater by providing a wide toe berm", *Proc., 3rd Indian Nat. Conf. of Harbour and Ocean Engg.*, 2, 599- 605..

Pranesh, M. R., and Ramulu, N. (1989). "Computer Aided Design of rubble mound breakwater", *Proc., 3rd Nat. Conf. on Dock and Harbour Engg*. KREC, Surathkal, India, Dec, 395 – 401.

Price, W. A. (1978). "Some ideas on breakwaters -Theme speech", *Proc., Conf. on Coastal Engg.*, 1, 37 - 48.

Priya, K. M., Sekhar, K. A. R., Sundar, V., Sanderavadivelu, R., and Graw, K. V. (2004). "Pressure exerted on submerged semi circular breakwater", www.uni-leepzig.de/~grw/lit.

Rao, S., and Raju S. (1994). "Sensitivity analysis of rubble mound breakwater", *Proc., 1st Indian Nat. Conf. of Harbour and Ocean Engg., INCHOE -94*, C.W.P.R.S., Pune, Dec., 2, B-57 – B-66.

Rao, S. (2000). "Studies on performance of perforated pile breakwaters", *Ph.D thesis submitted to Mangalore University*, KREC, Surathkal, India.

Rao. S., Rao, N. B. S., and Shirlal, K. G. (2003a). "Energy dissipation in suspended porous pipe breakwaters in laboratory", *J. of Water and Energy International*, 60(2), April – June, 24 – 31.

Rao. S., Shirlal, K. G., and Rao, N. B. S. (2002). "Wave transmission and reflection for two rows of perforated hollow pipes", *Indian J. of Marine Sciences*, 31 (4), Dec, 283 – 289.

Rao. S., Shirlal, K. G., and Rddy, G. R. (2003b). "Energy dissipation at single row of suspended perforated pipe breakwaters in laboratory", *J. of Institution of Engineers, India*, 84, Aug., 77 - 81.

Rao, S., Subramanya, K., Chandramohan, V. R., Radheshyam, B. and Rao, K. B. (2004). " Wave run up and run down studies on berm breakwater with reduced armour weight", *Proc., 3rd Indian Nat. Conf. on Harbour and Ocean Engg.*,INCHOE-2004, NIO, Goa, Dec, 81-88.

Rambabu Chiranjeevi, A., and Mani, J. S. (2001). "Wave transmission characteristics of submerged breakwater - A Numerical model study", *Proc., Nat. Conf. on Hydr. and Water Resources* – Hydro-01, Dec. CWPRS, Pune, India, 304 – 310.

Rambabu . Chiranjeevi, A., and Mani, .J.S (2002) "Effectiveness of submerged breakwater in Coastal protection", *Proc., Nat. Conf. on Hydr., Water Resources and Ocean Engineering*, HYDRO –02, IIT Bombay, Dec, 313-315.

Reedijk, J. S. (2003). "Shore parallel pile row breakwater - An example of an effective coastal protection system", Delta marine consultants by Gouda. *www.dmc.nl/publications/coastalprotection.pdf.*

Rietveld, F. W., and Brurcharth, H. F. (1987). "Construction, maintenance and repair as elements in rubble mound breakwater design", *Proc., Conf. on Coastal and Port Engg. in Developing Countries,* Indonesia, 1, 1042 – 1053.

Rojanakamthorn, S., Isoke, M., and Watanabe, A. (1989). "A mathematical model of wave transformation over a submerged breakwater", *J. of Coastal Engg. in Japan,* 32(2), 209 – 233.

Ross, G., and Pleskunas, S. (2003). "High density polyethylene pipe reef for surfing and shore protection", *Proc., 3rd Int. Surfing Reef Symposium*, Raglan, New-Zealand, June, 371 – 373.

Sathyanarayana, M., and Kumar Srinivasa, B. V. (1992). "Prediction of laboratory scale wave characteristics", *Proc., 8th Congress of Asian and Pacific Division of Int. Assn. for Hydraulic Research I.A.H.R.,* 3, CWPRS, Pune, Oct, D-75 – D-81.

Seabrook, S. R., and Hall, K. R. (1997). "Effect of crest width and geometry on submerged breakwater", *Proc., Canadian Coastal Conf.*– Abtract.

Shekappa, D..H. (1994). "Studies on stability of rubble mound breakwaters", *M.tech Dissertation, Mangalore University,* KREC, Surathkal, India.

Shiraishi, N., Palmer, R. Q., and Okamoto, H. (1976). "Quay wall with wave absorber Igloo", *Proc., 15th Conf. on Coastal Engg.,* 3, 2677 – 2696.

Shirlal, K. G., and Rao, S., (2001)."Laboratory investigation on stability of overtopping rubble mound breakwater", Proc., *Nat. Conf. on Hydr. and Water Resources-Hydro-2001,,* CWPRS, Pune, India, Dec, 291-297.

Shirlal, K. G., Rao, S., and Hegde, A. V. (2001)."Wave overtopping over rubble mound breakwaters", Proc., *Int. Conf. in Ocean Engg,* IIT Madras, Chennai, India, Dec, 309-313.

Shirlal, K. G., and Rao, S., (2002). "Sability of overtopping rubble mound breakwater with reduced weight", *Proc., Nat. Conf. on Hydr. and Water Resources and Ocean Engg.-Hydro-2002,,* IIT Bombay, Mumbai, India, Dec, 256-259.

Slichter, F. B. F. (1960). "Discussion on Laboratory investigation on rubble mound breakwaters", *J. of Waterw., Harbours and Coastal Engg. Division*, 1, 86.

Smith, D. A. Y., Warner Sorensen, R..M., and Atherley. (1996). "Submerged crest breakwater design", *Advances in Coastal Structures and Breakwaters*, Thomas Telford, London, 208 - 219.

Sollitt, C. K., and Cross, R. H. (1970). "Wave transmission through permeable breakwaters", *Proc., 12th Conf. on Coastal Engg.*, 3, 1827 – 1840.

Sollitt, K., and Debok, D. H. (1976). "Large scale model tests of placed stones breakwaters", *Proc., 15th Conf. on Coastal Engg.*, 3, 2572 – 2588.

Sorensen, R. M. (1987). "Evaluation of perched beach concept for shore stabilization", *Proc., Conf. on Coastal and Port Engg. in Developing Countries,* Indonesia, 1, 318 – 326.

Sorensen, T., and Jensen, O. J. (1986). "Experience gained from breakwater failures", *Development in Breakwaters,* Thomas Telford, London, 103 – 119.

Stive, M. J. F. (1985). "A scale comparison of waves breaking on a beach ", *J. of coastal Engg.,* 9,151-158.

Straub, L. G., Herbich, J. B., and Bowers, C. E. (1957). "An experimental study of hydraulic breakwaters", *Proc., 6th Conf. on Coastal Engg.*, 715 – 727.

Tanimoto, K, and Goda, Y. (1992). "Historical development of breakwater structures in the world", *Coastal Structures and Breakwaters*, Thomas Telford Ltd., London, 193 – 206.

Thompsen, L. A., Paul, E W., and Alfred, S. H. (1972) "Riprap stability on earth embankments tested in large and small scale wave tanks", *US Army Corps of Engrs, Coastal Engg. Res. Centre*, T. M. No. 37, U.S. Govt. Printing Office, Washington, D.C.

Thompson, D .H and Shuttler, R. M. (1976) "Design of riprap slope protection against wind waves", *CIRIA Rep. 61*, Hydr. Res. Station, Wallingford, England.

Thornton, E. B., and Schaeffer. (1978). "Probability density function of breaking waves", *Proc., Conf. on Coastal Engg.,*. 1, 507.

Thornton, E. B., Galvin, J. J., But, F. L., and Richardson, D. P. (1976). "Kinamatics of breaking waves", *Proc., 15^{th} Conf. on Coastal Engg.,*. 1, 461 – 476.

Timco, G. W., and Mansard, E.P.D. (1982). "Improvements in modeling rubble mound breakwaters", *Proc., Conf. on Coastal Engg.,* 3, 2047 – 2061.

Timco, G. W., and Mansard, E.P.D., and Ploeg, J. (1984). "Stability of breakwaters with variations in core permeability", *Proc., 19^{th} Conf. on Coastal Engg.,* 3, 2487 – 2499.

Torum, A., Mathlesen, B. J, and Escutia, R. (1979). "Reliability of breakwater model tests", *Proc., Special Conf. on Offshore Coastal and Marine Structure",* 1, 454 – 469.

Treloar, P. D., and Brebner, A. (1970). "Energy losses under wave action", *Proc., 12^{th} Conf. on Coastal Engg.,* 1, 257 – 267.

Twu Sheng Wen., Liu Cheng Chi., and Hsu Wen Hung. (2001). "Wave damping characteristics of deeply submerged breakwater", *J. of Waterw., Port,* Ocean Engg., 127(2), 97-105.

U. S. Army Corps of Engineers. (1973). "Shore Protection Manual (SPM)", *Coastal Engg. Res.Centre,* U.S. Govt. Printing Office, Washington, D.C.

U. S. Army Corps of Engineers. (1984). "Shore Protection Manual (SPM)", *Coastal Engg. Res. Centre,* U.S. Govt. Printing Office, Washington, D.C.

U. S. Army Corps of Engineers. (2001). "Coastal Engg. Manual (CEM)", *Coastal Engg. Res. Centre,* U.S. Govt. Printing Office, Washington, D.C.

Vaidya, G. T. (1989). "Statistical uncertainties in the breakwater design parameter", *Proc., 3^{rd} Nat. Conf .on Dock and Harbour Engg.* Dec., K.R.E.C Surathkal, 379 – 386.

Van de Graff, J. (1996). "Use of structures in coastal protection", *Proc., Int. Conf. in Ocean Engg.*, COE-96, I.I.T. Madras, Chennai, Dec, 461-468.

Van der Meer J.W. (1986). "Stability of rubble mound revetments and breakwaters under random wave attack", *Developments* in breakwaters, Thomas Telford Ltd., London, 141-154.

Van der Meer, J. W. (1988). "Rock slopes and gravel beaches under wave attack", *Ph.D Thesis, Delft University of Technology*, Delft, The Netherlands.

Van der Meer, J. W. (1989). "Stability of breakwaters – Delft hydraulics developments", *Proc., 3rd Nat. Conf. on Dock and Harbour Engg.* KREC, Surathkal, Dec, 889-899.

Van der Meer, J. W., and d'Angremond, K. (1992). "Wave transformation at low crested structure", *Coastal Structures and Breakwater*, Thomas Telford, London, 25 – 41.

Van der Meer, J. W., d'Angremond, K., and Gerding, E. (1996), "Toe structure stability of rubble mound breakwaters", *Advances in Coastal Structures and Breakwaters,* Thomas Telford Ltd., London, 308 – 321.

Van der Meer, J. W., and Heydra, G. (1991). "Rocking armour units, Number, location and impact velocity", *Proc., Conf, on Coastal Engg.*, Vol. 15, 21 – 39.

Van der Meer, J. W., and Pilarczyk K. W. (1984). "Stability of rubble mound slopes under random wave attack", *Proc., Coasta l Engg. Conf.,* 2620 – 2634.

Van der Meer, J. W., Wolters Ard, Zanuttigh Barbara, and Kramer Morten. (2003). "Oblique wave transmission over low crested structures", *Proc., Int. Conf. on Coastal Structures,* Portland, Oregan, Aug., 26 – 29.

Van Gent, M. R. A., and Vis, F. C. (1994). "Simulations of wave action on Coastal structures", *Proc., Indian Nat. Conf. of Harbour and Ocean Engg.,* 2, B-191 – B-196.

Weisher, L. L., and Byrne, R. J. (1978). "Field study of breaking wave characteristics", *Proc., Conf. on Coastal Engg.,*. 1, 487 – 507.

Whillock, A. F., and Price, W. A. (1976). "Armour blocks as slope protection", *Proc., 15th Conf. on Coastal Engg.,*. 3, 2564 – 2571.

Zwamborn, J. A. (1979). "Analysis of cause of damage to Sines breakwater", *Proc., Special Conf. on Ocean, Coastal and Marine Structures",* 1, 422 - 441.

Appendix 2
Cost Analysis

1. INTRODUCTION

From the model tests of the protected breakwater, it is concluded that the geometry of the protected breakwater with a conventional breakwater and a submerged reef of crest width 0.3m located at a seaward distance of 2.5m is totally free from damage. Considering this, the trunk section of a conventional and a defenced breakwater are designed for the same operating conditions. This chapter explains the details of the analysis and comparison of cost of prototypes of conventional and defenced breakwater.

2. DESIGN OF CONVENTIONAL RUBBLE MOUND BREAKWATER

2.1 Armour weight

Based on the investigations conducted in the wave flume, the trunk section of a conventional non-overtopping breakwater of primary armour weight (W) of 2.0Ton is designed using Hudson (1959) formula. However, stones of 2Ton to 3Ton are recommended. The weight of the stones in the secondary layer is W/10=200Kg but stones of 200Kg to 500Kg are used (Contract C-2 1994-95). A toe of 200Kg to 500Kg stones, of top width 4.8m and 3.6m high is provided at the bottom at both the ends.

2.2 Breakwater height

The maximum run up observed is about 1.6 times the wave height which is about 4.8m in a water depth of 12m. This results in a breakwater height of 16.8m. However, breakwater crest elevation is fixed at18m.

2.3 Thickness of layers

The thickness of primary and secondary layer is calculated as 2.2m and 1.0m respectively. The minimum crest width should be sufficient to accommodate 3 stones and crest width of 4.5m is provided.

2.4 Design parameters of conventional rubble mound breakwater

Design wave height, H	: 3.0m
Design Armour weight, W	: 2.0Ton
Armour weight range	: 2.0 to 3.0Ton
Specific gravity of armour, Sr	: 2.8
Mass density of armour unit	: 2.80 T/cum
Relative mass density of armour unit, ρ_a	: 1.8 T/cum
Layer coefficient K_Δ	: 1.15
Porosity of	
Primary	: 43%
Secondary	: 39%
Core	: 36%
Depth of water	: 12m
Breakwater height	: 18.0m
Crest width	: 4.5m
Breakwater slope	: 1V: 2H
Thickness of primary layer	: 2.2m
Thickness of secondary layer	: 1.0mts
Armour weight of secondary layer W/10	: 200Kg to 500Kg
Weight of core material W/200	: 5 Kg to 100kg
Armour weight of toe W/10	: 200 Kg to 500Kg

Fig. A gives the dimensions of the conventional breakwater designed.

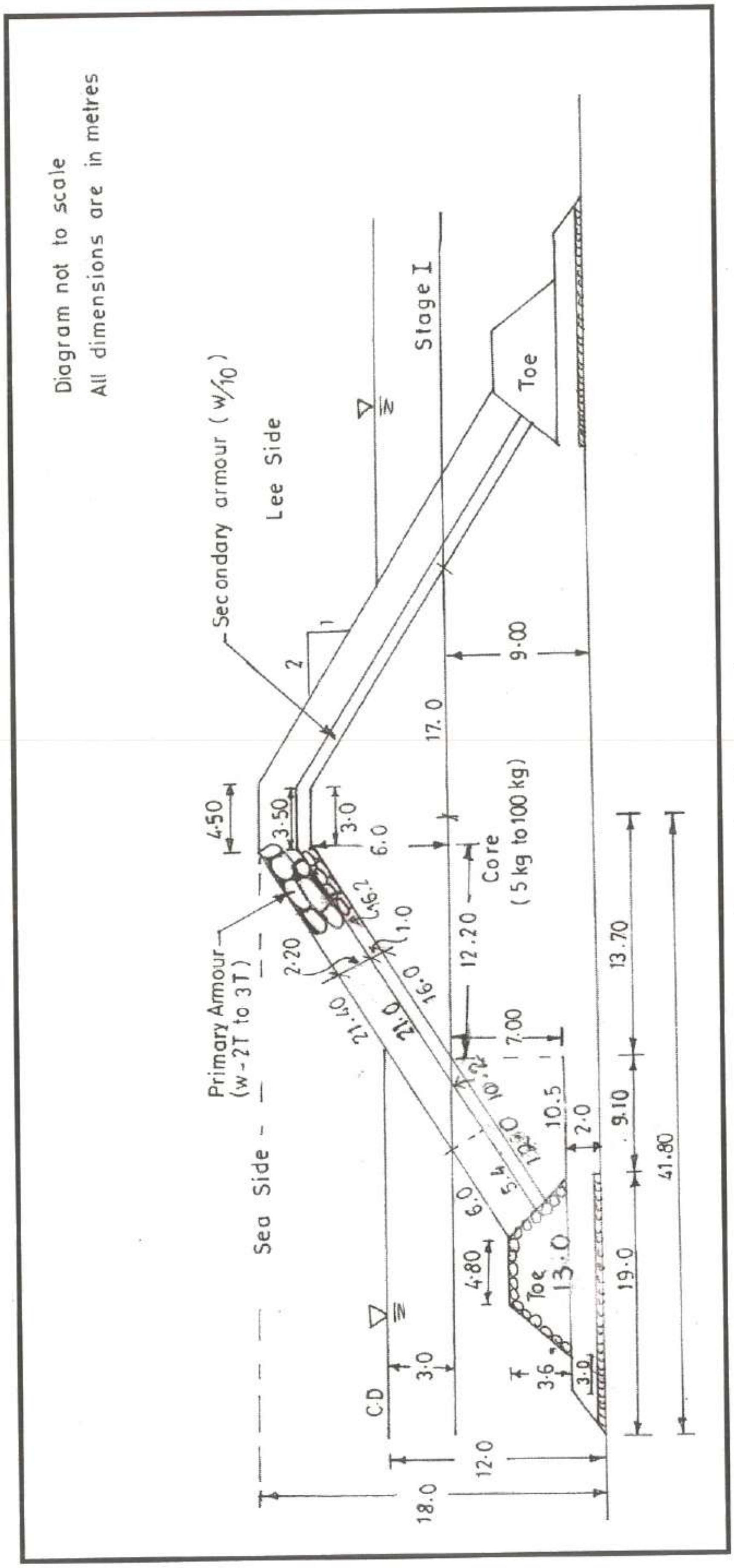

Fig. A. Conventional breakwater

3. DESIGN OF DEFENCED BREAKWATER

3.1 Introduction

Based on the conclusions of the present study, it is decided to deign the trunk section of a defenced breakwater with a inner (main) breakwater of armour stone weight of 50% of that of the conventional breakwater and a submerged reef located at 75m seaward from the breakwater.

3.2 Design

The transmission coefficient of the submerged reef of crest width of 0.3m is varying from 0.4 to 0.8. Assuming a conservative value of 20% for the maximum wave transmission, a design wave of 2.4m is fixed for the design of 1:2 sloped trapezoidal conventional breakwater. This gives a primary armour weight of 1.0Ton. However, stones weighing 1.0Ton to 2.0Ton are used for construction. Considering the maximum run up for this geometry of the defenced breakwater in a depth of 12m, a crest height of 15m is adopted. The crest a width of 4.5m is fixed. A 1:2 sloped trapezoidal submerged reef, of armour stone weight of 0.7Ton to 1.0Ton, height 7.5m and crest width of 9.0m, is built at a seaward distance of 75m from the conventional breakwater. Fig. B gives the dimensions of the submerged reef designed.

3.3 Design parameters of defenced breakwater

Main Breakwater

Design wave height, H	: 2.4m
Design Armour weight, W	: 1.0Ton
Armour weight range	: 1.0Ton to 2.0Ton
Specific gravity of armour, Sr	: 2.8
Mass density of armour unit	: 2.80 T/cum
Relative mass density of armour unit, ρ_a	: 1.8 T/cum
Layer coefficient K_Δ	: 1.15
Porosity of	
Primary	: 43%
Secondary	: 39%
Core	: 36%
Depth of water	: 12m
Breakwater height	: 15.0m

Crest width	: 4.5m
Breakwater slope	: 1V: 2H
Thickness of primary layer	: 1.6m
Thickness of secondary layer	: 0.75mts
Armour weight of secondary layer W/10	: 100Kg to 200Kg
Weight of core material W/200	: 1Kg to 5Kg
Armour weight of toe W/10	: 100Kg to 200Kg

Submerged reef

Side slope	: 1V:2H
Armour weight	: 0.7Ton to 1.0Ton
Porosity	: 43%
Crest height	: 7.5m
Crest width	: 9.0m

Fig. B gives the dimensions of the defenced breakwater designed.

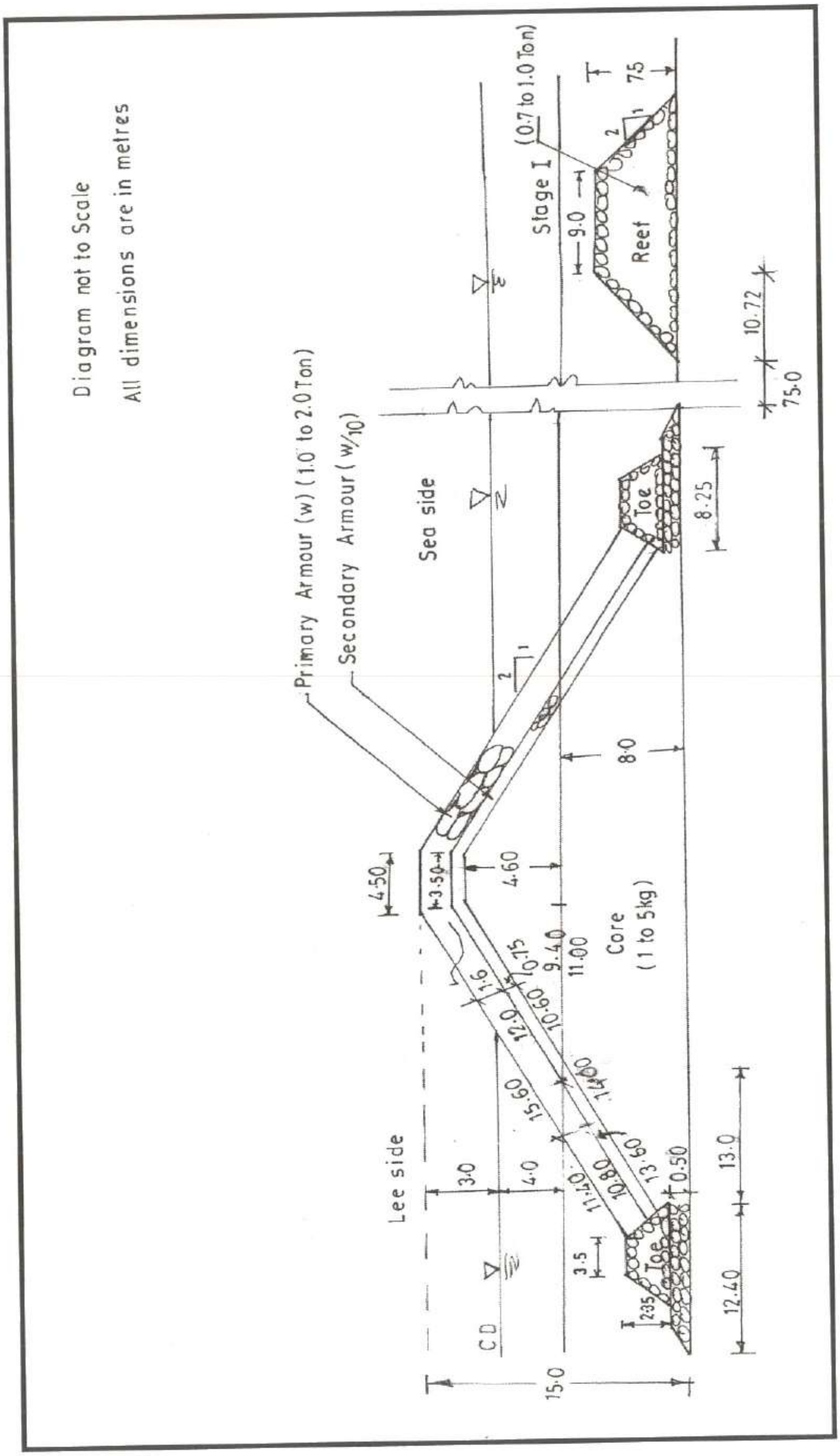

Fig. B. Defenced breakwater

4. QUANTITY ESTIMATION

The density of armour stones for the calculation of their quantity is estimated as follows (Contract C-2 (1994-1995).

4.1 Sample calculation

Porosity of the primary layer, n	= 0.43
Taking volume of primary layer	= $1m^3$,
Volume of voids V_v	= $0.43\ m^3$
Volume of stones in the layer	= 0.57
Weight of stones in $1m^3$ of the layer	= volume of stones × mass density of stones
	= 0.57 × 2.80
	= 1.6T
Density of primary layer	= $1.6T/m^3$
Similarly,	
Density of secondary layer	= $1.7T/m^3$
Density of core	= $1.8T/m^3$

4.2 Quantity estimation

The quantity of armour stones required for conventional and defenced breakwater are calculated in this section. The quantity of material of the main breakwater section are first calculated, up to certain level from the chart datum in Stage 1 and these are increased by 15% for possible settlement and/or soft clay conditions due to its large cross section and a heavy load on the foundation. In Stage 2 the remaining quantity of material is estimated and the total quantity is calculated.

4.2.1 Quantity estimation of conventional rubble mound breakwater

Table. A gives the estimation of quantity of primary armour, secondary armour and core per metre length of the breakwater as per the design parameters given in section 2.4 and with reference to Fig. A.

4.2.2 Quantity estimation of defenced breakwater

Table. B gives the estimation of quantity of primary armour, secondary armour and core per metre length of the breakwater as per the design parameters given in section 3.3 and with reference to Fig. B.

Table. A. Quantity estimation per metre length of conventional rubble mound breakwater

a) Stage-I Construction (-3.0m C.D.)

Sl. No.	Layer	Density (T/m^3)	No.	Length (m)	Breadth (m)	Depth (m)	Quantity (Ton)	Total Quantity (Ton)
1	Filter layer (1 to 5kg)	1.8	2	1.0	19.0	0.50	34.2	34.2
2	Core layer (5 to 100kg)	1.8	2	1.0	$\frac{(19+16)}{2}$	1.50	94.5	736.2
		1.8	2	1.0	9.1	2.00	65.52	
		1.8	2	1.0	$\frac{10.5}{2}$	7	132.3	
		1.8	2	1.0	13.7	9	443.88	
3	Secondary layer (200-500kg)	1.7	2	1.0	$\frac{(10.2+12)}{2}$	1.00	37.74	37.74
4	Primary layer (2.0 to 3.0 T)	1.6	2	1.0	$\frac{(6.0+10.2)}{2}$	2.20	57.02	57.02
5	Toe layer (200-500kg)	1.7	2	1.0	$\frac{(13.0+4.8)}{2}$	3.60	108.94	108.94

The quantities calculated above shall be increased by 15% for settlement and soft clay conditions.

1. Filter layer (1kg to 5kg) 34.2×1.15 = 39.33 Ton
2. Core layer (5kgto100kg) 736.2×1.15 = 846.63 Ton
3. Secondary layer (200kg to 500kg) 37.74×1.15 = 43.4 Ton
4. Primary layer (2.0 T to 3.0 T) 57.02×1.15 = 65.58 Ton
5. Toe berms (200kg to 500kg) 108.94×1.15 = 125.28 Ton

b) Stage–II Construction (+6.0m C.D)

Sl. No.	Layer	Density (T/m^3)	No.	Length (m)	Breath (m)	Depth (m)	Quantity (Ton)	Total quantity (Ton)
1	Core layer (5 to 100kg)	1.8	2	1.0	$\frac{12.2}{2}$	6	131.76	164.16
		1.8	2	1.0	1.50	6	32.4	
2	Secondary layer (200 to 500kg)	1.7	2	1.0	$\frac{(16.2+16.0)}{2}$	1.00	54.74	60.27
		1.7	1	1.0	$\frac{(3.5+3.0)}{2}$	1.00	5.53	
3	Primary layer (2.0 T to 3.0 T)	1.6	2	1.0	$\frac{(21.4+16.2)}{2}$	2.20	132.35	146.43
		1.6	1	1.0	$\frac{(4.5+3.5)}{2}$	2.20	14.08	

Table. B. Quantity estimation per metre length of defenced breakwater

a) Stage - I Construction (-4.0m C.D.)

Sl. No.	Layer	Density (Ton/m^3)	No.	Length (m)	Breath (m)	Depth (m)	Quantity (Ton)	Total Quantity (Ton)
1	Filter layer (1 to 5kg)	1.8	2	1.0	12.40	0.50	22.32	22.32
2	Core layer (1 to 5kg)	1.8	2	1.0	$\frac{13.0}{2}$	8.0	187.2	504.0
		1.8	2	1.0	11.0	8.0	316.8	
3	Secondary Layer (100 to 200kg)	1.7	2	1.0	$\frac{(14+13.6)}{2}$	0.75	35.19	35.19
4	Primary Layer (1Ton to 2Ton)	1.6	2	1.0	$\frac{(11.4+13.6)}{2}$	1.60	64.0	64.0
5	Toe berms (100to200kg)	1.7	2	1.0	$\frac{(3.5+8.2)}{2}$	2.35	46.74	46.74
6	Submerged breakwater (0.7Ton to 1.0Ton)	1.6	2	1.0	$\frac{10.72}{2}$	7.5	128.64	236.64
		1.6	2	1.0	4.50	7.5	108	

The quantities calculated above shall be increased by 15% for settlement and soft clay conditions.

1. Filter layer (1Kg to 5Kg) 22.32×1.15 = 25.67 Ton
2. Core layer (1Kg to 5Kg) 504×1.15 = 579.6 Ton
3. Secondary layer (100Kg to 200Kg) 35.19×1.15 = 40.47 Ton
4. Primary layer (1.0 T to 2.0 T) 65.02×1.15 = 73.6 Ton
5. Toe berms (100Kg to 200Kg) 46.74×1.15 = 53.75 Ton

b) Stage-II Construction(+3.0m C.D.)

Sl. No.	Layer	Density (Ton/m^3)	Nos	Length (m)	Breath (m)	Depth (m)	Quantity (Ton)	Total Quantity (Ton)
1	Core layer (1 to 5kg)	1.8	2	1.0	$\frac{9.40}{2}$	4.60	77.83	104.33
		1.8	2	1.0	1.60	4.60	26.50	
2	Secondarylayer (100 to 200kg)	1.7	2	1.0	$\frac{(12 + 10.6)}{2}$	0.75	28. 82	33.09
		1.7	1	1.0	$\frac{(3.5 + 3.2)}{2}$	0.75	4.27	
3	Primary layer (1 to 2 T)	1.6	2	1.0	$\frac{(15.6 + 12)}{2}$	1.60	70.66	80.9
		1.6	1	1.0	$\frac{(4.5+3.5)}{2}$	1.60	10.24	

5. ANALYSIS OF RATES

Unit rates of the all items of work are calculated for $10m^3$ of material which includes construction, quarrying, blasting of hard rock, depositing and stacking of useful stones within a lead of 50m, transportation, weighing and dumping/placing at specified location with a lead of 8Kms from quarry costs as shown in Table. C. Finally cost per ton of the armour stone is derived. The cost of preliminaries like equipments, mobilization, insurance, temporary construction of temporary jetties etc., are not included but are to be estimated separately and added to the cost of breakwaters. Table. D gives the final unit rates of armour stones of various sizes (Contract C-2 1994-1995).

Table. C. Calculation of unit rates of civil works

Sl. No.	Description of work	Quantity	Rate (Rs.)	Per	Amount (Rs.)
1	**Materials**				
	Country blasting powder	4Kg	120/-	1 kg	720.00
	Country fuse	12m	60/-	1 mt	720.00
2	**Labour**				
	Quarry men(for boring holes)	15Nos	150/-	each	2,250.00
	Quarry men(charging holes with Powder and tampering and firing)	2Nos	150/-	each	300.00
	Hammer men(Breaking big boulders)	2Nos	90/-	each	180.00
	Men (Removing blasted rock to a distance of 50 mts)	3Nos	90/-	each	270.00
3	Loading, Unloading and Transporting charge (8 kms).				657.00
					5,097.00
4	Royalties @ 10% on Rs 5,097/-				510.00
5	Weighing charge Lumpsum				75.00
6	Carriage charge of stone up to breakwaters site and placing				
	Hire charge of boat or any other transportation upto breakwater construction site into the sea.				375.00
	Loading and placing in position the stones with necessary lifting equipment.				300.00
7	Sundries, wastage – 10%				636.00
8	Add contractors overhead and profit 20%.				1,400.00
					8,393.00
9	So, cost per cubic meter (for average size & weight)		$\frac{8,393}{10}$		**840.00**
10	Cost per Ton		$\frac{(840 \times 1,000)}{2800}$		**300.00**

Table. D. Final unit rates of armour stones of various sizes

Sl. No.	Description of layer	Rate per Ton (Rs.)
1	Filter layer and core stones (1kg to 5kg)	300.00
2	Core and Secondary layer stones (5kg to 100kg) (Add 5% extra over '1')	315.00
3	Secondary layer stones (100kg to 200kg) (Add 5% extra over '2')	330.00
4	Primary layer stones (200kg to 500kg) (Add 5% extra over '3')	348.00
5	Primary layer stones (500kg to 700kg) (Add 5% extra over '4')	365.00
6	Primary layer stones (700kg to 1000kg) (Add 5% extra over '5')	383.00
7	Primary layer stones (1000kg to 2000kg) (Add 5% extra over '6')	402.00
8	Primary layer stones (2000kg to 3000kg) (Add 25% extra over '7')	503.00

6. COST OF CONSTRUCTION

6.1 Cost of construction per metre length of conventional breakwater

The cost of construction per metre length of the conventional breakwater is shown in Table. E. This is calculated considering the estimation of armour and other material quantities given in Table. A and the final unit rates calculated in Table. D.

Table. E. Cost of construction per metre length of the conventional breakwater

S.No.	Description of work	Quantity (Ton)	Rate (Rs/Ton)	Amount (Rs)
1.	Stage-I Quarrying, transporting, weighing and dumping/placing at the specified location by means of floating craft and or end method upto -3.0m CD (first stage construction) lines, dimensions and levels shown in drawing.			
a)	Filter layer (1kg to 5kg)			11,799
b)	Core stones (5kg to 100kg)	39.33	300	2,66,688
c)	Secondary layer (200kg to 500kg)	846.63	315	15,103
d)	Primary layer (2000kg to 3000kg)	43.4	348	32,986
e)	Toe berms (200kg to 500kg)	65.58	503	43,597
		125.28	348	3,70,175
2.	Stage-II Quarrying, transporting, weighing and dumping/placing at the specified location to by means of floating craft and or end on method from top level of first stage construction to lines, dimensions and levels shown in drawings.			
a)	Core layer (5 kg to 100kg)	164.16	315	51,710
b)	Secondary layer (200kg to 500kg)	60.27	348	20,974
c)	Primary layer (2000kg to 3000kg)	146.43	503	73,655
				1,46,340
3.	TOTAL COST (Stage-I + Stage-II)			5,16,514/-

6.2 Cost of construction per metre length of the defenced breakwater

The cost of construction of defenced breakwater is shown in Table. F. This is calculated considering the estimation of armour and other material quantities given in Table. B and the final unit rates calculated in Table. D.

Table. F. Cost of construction per metre length of the defenced breakwater

Sl.No	Description of work	Quantity Ton	Rate Rs/Ton.	Amount Rs.
	Stage-I			
1.	Quarrying, transporting, weighing and dumping/placing at the specified location by means of floating craft and or end-on method up to -4.00CD (first stage construction) to lines, dimensions and levels shown in drawing.			
a)	Filter layer (1Kg to 5Kg)	25.67	300	7,701
b)	Core stones (1Kg to 5Kg)	579.6	300	1,73,880
c)	Secondary layer (100Kg to 200Kg)	40.47	330	13,355
d)	Primary layer (1Ton to 2Ton)	73.6	402	29,587
e)	Toe berms (100Kg to 200Kg)	53.75	330	17,738
f)	Submerged reef (0.7Ton to 1Ton)	236.64	383	90,633
				3,32,894
2.	Stage-II			
	Quarrying, transporting, weighting and dumping/placing at the specified location to by means of floating craft or end on method from top level of first stage construction to liners, dimensions and levels shown in drawings.			
a)	Core layer (1Kg to 5Kg)	104.33	300	31,299
b)	Secondary layer (100Kg to 200Kg)	33.09	330	10,920
c)	Primary layer (1Ton to 2Ton)	80.9	402	32,522
				74,741
3.	TOTAL (Stage-I + Stage-II) Construction of defenced breakwater- system			4,07,635/-

7. COMPARSION OF CONSTRUCTION COST ESTIMATES OF CONVENTIONAL AND DEFENCED BREAKWATER

The construction costs estimated per metre length of conventional and defenced breakwater in sections 6.1 and 6.2 respectively are compared in Table. G below.

Table. G. Comparison of construction cost estimates per metre length

Sl. No	Type of structure	Primary armour weight (Ton)	Secondary armour weight (Kg)	Weight of core material (Kg)	Cost per metre length (Rs.)	Reduction in cost w.r.t conventional breakwater
1	Conventional Breakwater	2.0 to 3.0	200 to500	5 to 100	5,16,514/-/-	–
2	* Defenced Breakwater	1.0 to 2.0	100 to 200	1 to 5	4,07,635/-	21.08%

From the Table. G, it is seen that the defenced breakwater is 21.08% cost effective compared to conventional breakwater both designed for the same operating conditions.

* If the quantities estimated at Stage 1 of the defenced breakwater (refer Table. B of section 2.4.2.2) are not increased by 15% for settlement and/or soft clay conditions as the defenced breakwater is relatively lighter structure, then the cost of defenced breakwater drops and reduction of its cost with respect to conventional breakwater would be about 27.2%.

Appendix 3
Brief Resume of Co-ordinators

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Dr. Subba Rao is a Professor in the Department of Applied Mechanics& Hydraulics, National Institute of Technology Karnataka, Surathkal, since 1984. He has more than 120 research publications in the reputed International & National Journals and Conferences. He has been conferred with G.M.NAWATHE BEST PAPER PRIZE National award, constituted by Institution of Engineers (India), during the year 1999 at the National conference on Hydraulics - HYDRO `99 and the JAL VIGYAN PURASKAR FOR THE BEST PAPER published in the ISH Journal of Hydraulic Engineering during 2003. He is the member of many professional bodies. He has guided more than 38 M.Tech students and one Ph. D scholar. Presently he is supervising six Ph.D theses. He is actively involved in consultancy in the fields of Hydraulics, Coastal and Geotechnical Engineering. He was the organising secretary of Indian National Conference on Harbour and Ocean Engineering, INCHOE-2007.

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Dr. Kiran G Shirlal is a post-graduate in Offshore Engineering from IIT-Bombay and has a Doctorate degree in Coastal Engg from. Presently he is working as Assisatant Professor in the Dept. of Applied Mechanics and Hydraulics NITK, Surathkal. He has about 50 research publications in the reputed International / National journals / Conferences. He has been conferred with JAL VIGYAN PURASKAR FOR THE BEST PAPER published in the ISH Journal of Hydraulic Engineering during 2003. He has guided about 20 M.Tech candidates for their dissertation and presently guiding 3 PhD candidates. He is the member of various professional bodies. He is actively involved in R&D and Consultancy activities of the department.