A CASE STUDY ON THE SUBMERGED BERM TYPE COASTAL DEFENSE STRUCTURES

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ABSTRACT

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Coastal defense structures are built in order to protect valuable coastal regions from the destructive effects of the waves. Due to the cost of coastal defense structures and the economical potential of the coastal regions, failure of such structures could cause loss of high amounts of investment. Therefore in the design and construction of coastal structures, it is of vital importance to achieve an optimum design which is not neither underdesigned nor overdesigned.

In this study, Submerged Berm type coastal defense structures with several different cross-sections were tested for stability under storm conditions. Damage analyses of the different models were carried out to compare the structure characteristics under storm conditions and to obtain the most economical and stable cross-section.

For the model studies, 5 different models were constructed by using Van der Meer's approach and berm design guidelines. Models were constructed with a model scale of 1:31.08 in the wave flume in the Coastal and Harbor Engineering Laboratory, Civil Engineering Department, METU. The newly designed and optimized berm type structure was proved to be successful and economical.

Keywords: Rubble Mound Coastal Defense Structures, Damage, Stability Tests, Submerged Berm, Armor Layer

ÖZ

BATIK BASAMAK TİPİ KIYI KORUMA YAPILARI ÜZERİNE UYGULAMALI ARAŞTIRMA

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Dalgaların yıkıcı özelliklerinin kıyı alanları üzerindeki etkilerini azaltmak amacıyla kıyı koruma yapıları inşa edilmektedir. Kıyı koruma yapılarının maliyeti ve korunan kıyı bölgelerinin ekonomik potansiyeli göz önüne alındığında, bu tip yapıların yıkılması büyük miktarda yatırım kaybına sebep olmaktadır. Bu nedenle bu tip yapıların tasarım ve inşaası optimum tasarım koşullarını sağlamak açısından büyük önem taşımaktadır.

Bu çalışmada, değişik yapı kesitlerinin farklı fırtına koşulları altındaki davranışları test edilmiştir. Tüm kesitler için hasar analizleri deneylerden elde edilen verilere uygulanarak, yapı karakterleri, dalga koşulları ve analiz methodları göz önünde bulundurulup karşılaştırılarak, mümkün olan en stabil ve ekonomik kesit bulunmuştur.

Model çalışmalarında 5 değişik kesit "Van der Meer metodu" ve tasarım klavuzları kullanılarak tasarlanmıştır. Modeller 1:31.08 ölçeğiyle kurulmuş ve deneyler Orta Doğu Teknik Üniversitesi, İnşaat Mühendisliği Bölümü, Kıyı ve Liman Mühendisliği Laboratuvarı'nda gerçekleştirilmiştir. Yeni tasarlanan ve kullanım için en uygun hale getirilen ekonomik kesit denge testlerinde başarılı bulunmuştur.

Anahtar sözcükler: Topuk Tipi Taş Dolgu Kıyı Koruma Yapıları, Hasar, Denge Deneyleri, Batık Basamak, Anroşman

To My Family...

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LIST OF SYMBOLS AND ABBREVIATIONS

B_b	Berm width
CEM	Coastal Engineering Manual
d	Water depth
$D_{a,15}$	15% of the stones have a diameter less than D_{15}
$D_{a,85}$	85% of the stones have a diameter less than D_{85}
D_{50}	Equivalent cube length of median rock
D_l	Local damage (%)
D_t	Total damage (%)
F	Transportation cost (TL/Ton)
H	Wave height
$H_{D=0}$	Design wave height for zero damage
H_{des}	Design wave height
H_s	Significant wave height in front of the breakwater
H_0/L_0	Wave steepness
K	Transportation cost calculation constant
K_D	Stability coefficient
L	Life time of the structure in years
L_0	Deep water wave length (L)
M	Distance between two locations (km)
N_{dl}	Number of dislocated stones in a specified layer
N_{tl}	Total number of stones in a specified layer
N_{dt}	Number of dislocated stones within a cross-section
N_{tt}	Total number of stones within a cross-section

N_s	Stability parameter
N_w	Number of waves in storm duration
P	Porosity
P_b	Overall porosity of the breakwater
P_o	Occurrence probability
P_r	Fraction of the rounded stones
R_p	Return period in years
T	Wave period
T_r	Return period of design wave
T_s	Significant wave period
S_a	Damage level
SPM	Shore Protection Manual
SWL	Still water level
W	Weight of individual armor unit
γ_r	Specific weight of rock
γ_w	Specific weight of water
α	Angle of the front slope of the structure
Δ	Relative density of rock in water
Δ_a	Relative density of rock
γ_r	Specific weight of stones
γ_w	Specific weight of water
ξ_m	Surf similarity parameter
λ_L	Length scale
λ_T	Time scale
λ_V	Volume scale
λ_W	Weight scale

CHAPTER 1

INTRODUCTION

The coasts which do not have protective beaches fronting them will be directly attacked by the waves. The destructive effects of these waves can be minimized by constructing appropriate defense structures. Coastal defense structures are the structures that built in order to absorb the energy of incoming waves.

For the purpose of coastal protection, different types of breakwaters can be used. The availability of construction material, the severity of wave climate, water depth at the construction site and the foundation conditions are the factors effecting the selection of the breakwater type that will be used for the specified area.

The design of breakwaters should be able to withstand the forces due to severe waves. It should also meet other requirements like safety, stability, economy and serviceability. Most effective and safest design is achieved by optimizing the design where the total cost is minimized whereas the efficiency and stability increased to maximum. For this purpose the design should be supported with proper model investigations. Model studies help the designer to observe the effects of design storm to the structure and make necessary adjustments before the construction phase.

In Turkey, rubble mound breakwater is the most commonly used breakwater type for the protection of coastal zones. This is due to the fact that rubble mound structures are easier to construct than the other types of breakwaters. In addition, the availability of construction materials, which is the quarry stone in this case, and the easiness of maintenance can be stated as the other reasons of popularity of rubble mound structures. On the other hand, occurrence of critical damages due to the deficiencies in the design phase is another commonly seen scenario in Turkey. Improper designs lead the structure to be damaged heavily or it can even bring forth the total collapse of the structure. The highway defense structures built in Eastern Black Sea Coast can be given as a recent example.

The present study was based on the works of Dedeoğlu (2003) and Taşkıran (2003). In their studies, model tests on the stability of the implemented coastal protection structures of Eastern Black Sea Highway Project, proved that the structure is not safe under severe wave action. An alternative cross-section which was tested under design wave conditions is proposed in their studies together with economical analyses.

In the present study, further investigations of the proposed cross-section was carried out by using submerged berm which is placed 1 m below the still water level. The placement of the berm below design water level takes into consideration of the settlement of the stone layers and the storm surge. The effect of berm width was also taken into account and alternative cross-sections were tested with model studies carried out in a wave flume at the Coastal and Harbor Engineering Laboratory of the Middle East Technical University (METU), Ankara.

For the experiments, hydraulic models were constructed with a length scale of 1:31.08 in the wave flume using natural stones. Models were placed in a plexiglass channel of width 1.5 m, constructed in the wave flume. The foreshore slope was constructed as 1:30. The cross-sections had been tested under breaking and non-breaking waves at a constant water depth of 30 cm at the toe of the structure. Waves were generated as regular waves with a range of 3 – 7 m for wave height and 5 – 11 sec for wave period in prototype. Wave heights and periods were increased in each step of experiments in order to simulate storm conditions.

Review of the relevant literature and previous studies are given in Chapter 2. Breakwater types, information about applications and design procedures are mentioned in this chapter.

In Chapter 3, detailed information about cross-sections of models, model construction, experimental set-up, model scales, determination of design wave and experimental procedure is given.

The details of experiments are presented in Chapter 4. Reflection analysis results, percent damages of each model, corresponding damage curves and comparative economic analysis are given in this chapter. In Chapter 5, conclusions of this work and recommendations for future studies are given.

CHAPTER 2

LITERATURE REVIEW

Breakwaters are the most important coastal structures designed to withstand and dissipate the dynamic energy of waves. They function as barriers and create tranquility conditions on their leeward side. The forces due to dynamic energy of waves are the major components for the design and stability of such structures.

In general, breakwaters are classified by the shape of their seaward faces as sloping front breakwater, vertical front breakwater and combination of these two, termed as the composite breakwater. Conventional rubble-mound breakwater and berm breakwater are the examples of sloping front breakwaters. In the case of vertical front breakwaters, conventional caisson breakwater can be given as an example. Vertical composite caisson breakwater (the caisson is placed on a high rubble-mound foundation), horizontal composite caisson breakwater (the front of the caisson is covered by armor units or a rubble-mound structure) and sloping-top caisson breakwater (the upper part of the front wall above still water level is given a slope with the effect of a reduction of the wave forces) can be classified as composite breakwaters (Coastal Engineering Manual, 2003).

Various applications of breakwaters can be listed as follows;

- Protection of harbor facilities.
- Creation of calm water condition at the harbors to provide safe mooring and loading operations.
- Improvement of maneuvering conditions at harbor entrances.
- Protection of coastlines against tsunami waves.
- Protection of water intakes for the power stations.
- Shore protection.

In the design procedure, the application field is another important parameter for the selection of breakwater type together with the availability of construction material, wave climate and the foundation conditions of the selected region.

Being a commonly used coastal defense structure type, the rubble-mound breakwaters consist of random-shaped and random-placed armor units of selected stone sizes. The sloping face of the front armor layer provides a suitable surface to dissipate the incoming wave energy. The wave height is reduced as the wave approaches from toe to top berm and there is considerable dissipation of energy due to its passage through the porous medium and reflection from the structure.

For the design of rubble-mound breakwaters, stability and economy constitute the most important considerations. Recommended design guidelines mostly cover armor layer stability and upper boundary effects such as wave run-up and wave overtopping which are important criteria for designing the structure with the minimum crest elevation. Toe protection is another consideration that should be taken into account to preserve the stability of the structure in its lifetime.

Toe structures support the armor layer and prevent it from sliding. In addition, they protect the structure against scouring and prevent underlying material from leaching out. Moreover they provide structural stability against circular slip or failure. Toe stability could be a major source of damage, particularly when the structure is sited in relatively shallow water. In other words, the direct effect of hydraulic conditions to the bottom boundaries, for instance, wave breaking at the toe of the structure and the run-down effect on the toe stability, mostly occurs at relatively shallow water depths. Therefore, besides the hydraulic conditions, the geometrical and geotechnical properties of the bottom boundary are also functions of the structural design features in shallow water (Velioğlu, 1984).

Toe structure design is complicated by interactions between main structure, hydrodynamic forces and foundation soil. Design is often ad hoc or based on laboratory testing (CEM, 2003). Since toe failure often leads to major structural failure, a special care should be given to toe stability.

In the design process of the rubble-mound breakwaters, it is aimed to find the weight of a single armor unit for the specified conditions in the construction region. There are two common design schemes that are used in the design of rubble-mound breakwaters (CEM, 2003). These are Hudson formula and Van der Meer design scheme. It should be noted that the armor weight found through these criterion is not necessarily be the ideal one. Therefore, it is needed that the proper model studies should follow the calculation of the specified armor unit weight.

The stability formula developed by Hudson (1959) to determine the weight of an armor unit of relative uniform size is given below (CEM, 2003):

$$W = \frac{\gamma_r H^3{}_{D=0}}{K_D \left(\frac{\gamma_r}{\gamma_w} - 1 \right)^3 \cot\alpha} \tag{2.1}$$

where,

W	Weight of individual armor unit
γ_r	Specific weight of rock
γ_w	Specific weight of water
$H_{D=0}$	Design wave height for zero damage
K_D	Stability coefficient
α	Angle of the front slope of the structure

Although it is an easy to use formula, there are some important considerations not included in the Hudson formula. Such as wave period, type of breaking, structure permeability etc.

In 1987, Van der Meer suggested a new approach for the design of rubble-mound breakwaters. This contemporary method based on new studies includes the missing parts in the Hudson formula. The most important part that makes Van der Meer design scheme more preferable is the addition of incipient wave period to the calculations. The formula is also expanded to cover the different breaking type of incoming waves. The calculation of armor unit weight for a specified damage level is another advantageous part of the design scheme. Since Van der Meer approach includes many parameters, it is not a one step formula contrary to Hudson's. The equivalent cube length of median rock is represented by the following formula (CEM, 2003):

$$D_{50} = \frac{H_{des}}{\Delta N_s} \tag{2.2}$$

where,

D_{50}	Equivalent cube length of median rock
H_{des}	Design wave height
Δ	Relative density of rock in water
N_s	Stability parameter

The distinction between the plunging and surging waves is included in the definition of the stability parameter (N_s).

$$N_s = 6.2 P_b^{0.18} \left(\frac{S_a}{\sqrt{N_w}} \right)^{0.2} \sqrt{\cot\alpha}\, \xi_m^{-0.5} \qquad (\textit{for plunging breakers}) \tag{2.3}$$

$$N_s = 1.0 P_b^{-0.13} \left(\frac{S_a}{\sqrt{N_w}} \right)^{0.2} \sqrt{\cot \alpha}\, \xi_m^{P_b} \qquad (\textit{for surging breakers}) \qquad (2.4)$$

where,

P_b	Overall porosity of the breakwater
S_a	Damage level
N_w	Number of waves in storm duration
α	Slope angle
ξ_m	Surf similarity parameter

In Van der Meer design scheme, it is recommended that for practical purposes stability parameter (N_s) should be taken between 3 and 6. In other words, design can be done with allowing an acceptable level of damage at the toe of the structure.

Surf similarity parameter (ξ_m) is used to express the relation between slope angle (α), wave height (H) and offshore wave length (L_0). The usefulness of surf similarity parameter (ξ_m) has been demonstrated in the widely referenced work of Battjes (1975). A physical interpretation of surf similarity parameter (ξ_m) is given, showing that for waves that break on the slope, it is approximately proportional to the ratio of slope angle (α) to the local steepness of the breaking wave. There are two options for the determination of type of breaker. In the first alternative, range of surf similarity parameter (ξ_m) is considered.

Plunging breakers $0.5 < \xi_m < 3.3$

Surging breakers $\xi_m > 3.3$

In the second alternative ξ_m is compared with ξ_{mc} in order to determine the breaker type as plunging or surging;

where,

$$\xi_m = \left(\frac{H_0}{L_0} \right)^{-0.5} \tan\alpha \tag{2.5}$$

$$\xi_{mc} = \left(6.2P^{0.31} (\tan\alpha)^{0.5} \right)^{1/P+0.5} \tag{2.6}$$

Plunging breakers $\xi_m < \xi_{mc}$

Surging breakers $\xi_m > \xi_{mc}$

Van der Meer includes the effect of wave period by taking the number of waves (N_w) into account. Wave period is simply the proportion of storm duration to the number of waves that attack the structure in the storm duration.

According to Van der Meer (1995), the value of overall porosity of the breakwater (P_b) varies from a minimum of 0.1 for an armor layer with a thickness equal to $2D_{50}$ on an impermeable core, to a maximum of 0.6 for a homogeneous structure consisting only of rock fill. It also recommended that a sensitivity analysis be performed on all parameters as a part of the design process.

Recently, attention has shifted from statically stable rubble-mound breakwaters to dynamically stable berm breakwaters. As mentioned previously, conventional rubble-mound breakwaters are designed to withstand wave action without significant displacement of material. Therefore, each armor unit has to be put into its final position during the construction phase. The concept of berm breakwater, on the contrary, involves the use of relatively smaller armor stones which will be transported by the waves from the position where they were placed or dumped.

In other words, in statically stable structures (such as conventional rubble-mound breakwaters), the profiles are not permitted to change under severe wave conditions, whereas the profiles of dynamically stable structures (such as berm breakwaters) may change according to the wave climate. A berm breakwater is said to be dynamically stable when the net cross-shore transport of stones is zero and its profile has reached equilibrium under certain wave conditions. In fact, the originally built profile becomes dynamically stable when wave attack moves rock in the berm partly upward to the crest and partly downward to the toe; thus reshaping the seaward slope into a more statically stable S-shape profile (Noortwijk et al.,1996).

Berm breakwaters can be divided into three categories:

- In the static non-reshaping condition, only some few stones are allowed to move similar to what they are allowed to on a conventional rubble-mound breakwater.

- In the static reshaped condition, the profile is allowed to reshape into a profile, which is stable and where the individual stones are also stable.

- In the dynamical stable condition, the profile is reshaped into a stable profile, but the individual stones may move up and down the slope.

A berm breakwater has traditionally been constructed with a berm that has been allowed to reshape instead of constructing it with the reshaped profile directly (Torum et al., 2003). This is because it has been considered cheaper to construct the breakwater with a reshaping berm. In recent years, it has also been a drive to design a berm breakwater such that it will not reshape at all. However, there is not a universal design rule for berm type breakwaters.

For practical purposes, a design guideline proposed by Hall (1993) can be used to find the berm width (CEM, 2003).

$$B_b = D_{a,50}\left[-10.4 + 0.5\left(\frac{H_s}{\Delta_a D_{a,50}}\right)^{2.5} + 7.5\left(\frac{D_{a,85}}{D_{a,15}}\right) - 1.1\left(\frac{D_{a,85}}{D_{a,15}}\right)^{2} + 6.1P_r\right] \quad (2.7)$$

where,

B_b	Berm width
$D_{a,50}$	Equivalent cube length of median rock
H_s	Significant wave height in front of the breakwater
Δ_a	Relative density of rock
$D_{a,85}$	85% of the stones have a diameter less than D_{85}
$D_{a,15}$	15% of the stones have a diameter less than D_{15}
P_r	Fraction of the rounded stones

When the berm width is increased to a sufficiently large value not larger than (B=L/4), the water flowing down from the top slope is not allowed to run off the berm completely before the next wave has broken on the lower slope and is running up. Therefore, a slug of water remains on the berm to offer resistance to the oncoming run-up.

CHAPTER 3

MODEL STUDIES

3.1 Hydraulic Modeling

Hydraulic modeling can simply be defined as the reproduction of the hydraulic phenomenon in a laboratory environment. Hydraulic forces that are involved in the phenomenon should be known in order to decide the modeling technique. Hydraulic forces can be named as inertia force, gravitational force, viscous force, surface tension force, pressure force and elastic compression force. The major forces on a rubble-mound breakwater are gravity force and inertia force. Therefore the modeling should be done according to Froude model scaling.

In Froude model, the relations between length, time, volume and weight scales can be given as:

$$\lambda_T = \sqrt{\lambda_L} \qquad (3.1)$$

$$\lambda_V = \lambda_L{}^3 \qquad (3.2)$$

$$\lambda_W = \lambda_L{}^3 \frac{(\gamma_r)_m}{(\gamma_r)_p} \left[\frac{\dfrac{(\gamma_r)_p}{(\gamma_w)_p} - 1}{\dfrac{(\gamma_r)_m}{(\gamma_w)_m} - 1} \right]^3 \tag{3.3}$$

where,

λ_L Lenght scale

λ_T Time scale

λ_V Volume scale

λ_W Weight scale

γ_r Specific weight of stones

γ_w Specific weight of water

subscript “p” denotes the prototype and “m” denotes the model scales.

3.2 Aim of Model Studies

Shore protection structures are built to supply adequate protection of coasts against the destructive effects of waves. Both the protection structure and the protected coast itself could cost a high amount of investment. Therefore failure of a shore protection structure is not acceptable. Model studies are needed to test and to find the most optimum and stable cross-section before the beginning of construction stage.

The referenced works of Dedeoğlu (2003) and Taşkıran (2003) have chosen to be a threshold point for the thesis work at hand. They have studied the stability of the coastal defense structures of Eastern Black Sea Highway at Ordu-Giresun area experimentally. In the experiments models were constructed in Coastal Harbor Laboratory wave flume with a scale of 1:31.08 and tested under storm waves where prototype values ranging between 3 – 7 m. for wave height and 5 – 11 sec. for wave period. Accumulated damage curves were drawn for the cross-sections after each experiment where wave heights increased in steps.

In their work Dedeoğlu (2003), Taşkıran (2003), they have obtained an optimum cross-section for the coastal defense structures of Eastern Black Sea Highway as given in Figure 3.1. Both stability and economical analysis have been carried out in order to find the optimum cross-section of a berm type rubble-mound coastal defense structure. In these works, modifications had been made by changing the stone sizes of appropriate layers and keeping the berm width same throughout the tested sections.

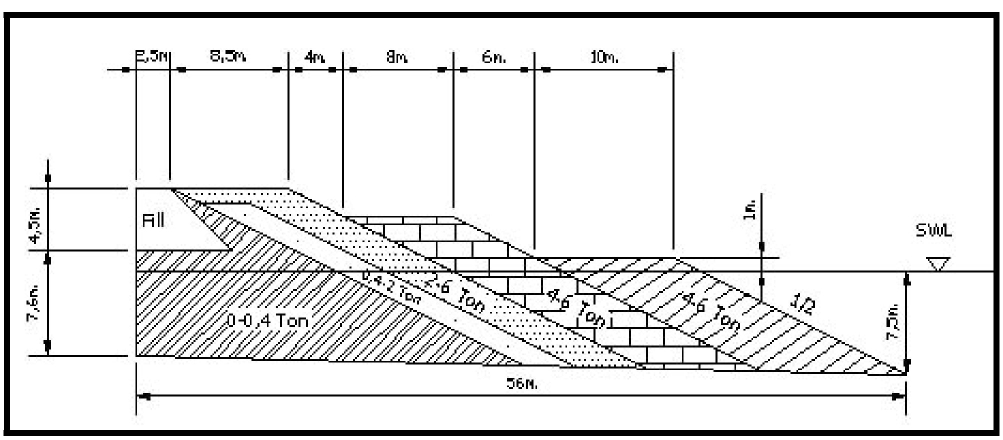

Figure 3.1 Proposed model section by Dedeoğlu, 2003 and Taşkıran, 2003

In the present work the cross-section given in Figure 3.1 was taken as reference and experiments were carried out by changing the stone sizes of appropriate layers as well as the berm width to optimize the cross-section which will yield a section both stable and economical.

The stability and damage criteria of the rubble mound coastal protection structures and the continuity of their functionality were studied under two main topics:

i) Stability of the coastal protection structure:

- Failure of the armor layer of the structure by having a certain percentage of dislocated rocks more than the acceptable damage percentage and due to this, loss of stability and serviceability in the whole structure, which will cause total failure.

ii) Functional properties of the coastal highway:

- Excessive run-up and water spraying during the storm damaging the highway and hindering the traffic flow as well as causing loss of life due to traffic accidents caused by the improper road conditions.

- Severe storm wave action damaging the under layers of the highway structure causing total failure and flooding of the highway.

To carry out the model investigations following studies were performed in Coastal and Harbor Engineering Laboratory of the Middle East Technical University (METU), Ankara.

- Determination of the design wave height by studying the long term and short term wave statistics of the region.

- Determination of model scale

- Determination of alternative cross-sections

- Construction of model

- Calibration of wave measuring probes in wave flume

- Experiments

- Evaluation of the experiment results with respect to damage, run-up and water spray criteria

3.3 Determination of Design Wave

Selection of design wave height from the wave climate of the region is an important part of the model investigations. For the present study, extreme wave statistics of the region as well as deep water significant wave heights with their return periods (Dedeoğlu, 2003 and Taşkıran, 2003) obtained from Wind and Wave Atlas for Turkish Coasts Project Report (Özhan and Abdalla, 1999) were used as the design wave conditions. Return periods, significant wave heights and periods of the design waves for the Eastern Black Sea Region are represented in the Table 3.1.

Table 3.1 Return periods, significant wave heights and periods for the Eastern Black Sea Region (Özhan and Abdalla, 1999)

		25 Years		50 Years	
Coordinates	**Location**	**H_s (m.)**	**T_s (sec.)**	**H_s (m.)**	**T_s (sec.)**
42.00° N, 41.60° E	Poti	6.25	10.27	6.80	10.71
42.00° N, 41.30° E	Poti offshore	6.40	10.39	6.85	10.75
41.75° N, 41.30° E	Batum offshore	6.30	10.31	6.90	10.79
41.75° N, 41.00° E	Hopa offshore	6.30	10.31	7.00	10.87
41.50° N, 41.00° E	Hopa	6.30	10.31	7.00	10.87
41.50° N, 40.70° E	Çayeli offshore	6.25	10.27	6.75	10.67
41.25° N, 40.70° E	Çayeli	5.85	9.93	6.40	10.39
41.25° N, 40.40° E	Rize	5.80	9.89	6.35	10.35
41.25° N, 40.10° E	Sürmene	5.75	9.85	6.30	10.31
41.25° N, 39.80° E	Trabzon	5.80	9.89	6.40	10.39
41.25° N, 39.50° E	Akçaabat	5.80	9.89	6.45	10.43
41.25° N, 39.20° E	Vakfıkebir	6.00	10.06	6.60	10.55
41.25° N, 38.90° E	Tirebolu	6.10	10.14	6.75	10.67
41.25° N, 38.60° E	Çamburnu	6.20	10.23	6.80	10.71
41.25° N, 38.30° E	Giresun	6.20	10.23	6.80	10.71
41.25° N, 38.00° E	Ordu	6.25	10.27	7.00	10.87

The representative average storm duration for the Eastern Black Sea Coast was found as 8 hours which were considered as the mean duration of the storms occurring in this region. The deep water wave steepness (H_0 / L_0) was selected to be 0.038 in order to represent the general wave characteristics of the region (Özhan and Abdalla, 1999).

In the design of rubble-mound structures, waves that break at the toe create a critical condition on the stability of the structure. This is due to the fact that while breaking, waves almost dissipate all their energy on the sloping face and causing partial or total failures. Therefore in most of the cases testing of breaking wave condition becomes very important. In order to find the breaking wave properties and their deep water correspondents the water depth at the toe of the structures should be known. The maximum water depth which gives the most critical breaking condition at the toe was taken as 7.5 m. by considering the topographic maps of the Giresun region.

Breaking wave charts given in Shore Protection Manual (SPM, 1984) were used to find the wave characteristics together with the corresponding deep water wave height. For a sea bottom slope of 1:30 and water depth of 7.5 m. the breaking wave height (H_b) was computed as 6.5 m. which has correspondent deep water wave height (H_0) of 5.8 m. By using the wave steepness of 0.038 the wave period was calculated as 9.90 sec.

From Wind and Wave Atlas for Turkish Coasts Project Report (Özhan and Abdalla, 1999), the return period was found as 17 years for deep water wave height of 5.8 m. and significant wave period of 9.9 sec at Giresun region where the coastal protection structure is under construction.

The occurrence probability of a wave with a certain return period within the lifetime of the structure was found by the following formula.

$$P_o = 1 - (1 - \frac{1}{R_p})^L \tag{3.4}$$

where,

P_o Occurrence probability

R_p Return period in years

L Life time of the structure in years

Encounter probabilities of a wave with a return period of 17 years for different lifetimes were calculated and tabulated in Table 3.2. Since, in Turkey, the lifetime of coastal defense structures can normally be taken as 40 years computations in Table 3.2 are presented up to 40 years.

Table 3.2 Encounter probabilities of a wave with a return period of 17 years

Year	Encounter Probability	Year	Encounter Probability
1	0.059	**21**	0.720
2	0.114	**22**	0.736
3	0.166	**23**	0.752
4	0.215	**24**	0.766
5	0.261	**25**	0.780
6	0.305	**26**	0.793
7	0.346	**27**	0.805
8	0.384	**28**	0.817
9	0.420	**29**	0.828
10	0.454	**30**	0.838
11	0.487	**31**	0.847
12	0.517	**32**	0.856
13	0.545	**33**	0.865
14	0.572	**34**	0.873
15	0.597	**35**	0.880
16	0.621	**36**	0.887
17	0.643	**37**	0.894
18	0.664	**38**	0.900
19	0.684	**39**	0.906
20	0.702	**40**	0.911

The encounter probability of the design wave for 40 years of lifetime was computed as 91.1 % as can be seen in Table 3.2. Considering such a high level of encounter probability, it can be concluded that this storm will take place at least once in the lifetime. Causing most critical condition from stability point of view, it is appropriate to take this wave as a design wave height. Waves with greater wave height than 5.8 m. will break at larger depths than 7.5 m. Thus greater waves hit the structure as broken waves with less energy. Waves break directly on the slope of the structure are smaller than the waves breaking at the toe of the structure. In the experiments, it was decided to model the most critical condition that is the breaking of waves on the structure, as well. Therefore, the tests, started with smaller wave heights, increased up to the waves breaking at the toe of the structure.

3.4 Bathymetric Properties

Giresun region was selected as an application region for the model studies. From the bathymetric maps of the region, a representative value for the sea bottom slope was chosen as 1:30 and applied to the wave flume in the Coastal and Harbor Engineering laboratory.

3.5 Experimental Set-up

3.5.1 Wave Flume

For the model investigations, different cross-sections were tested in the wave flume of Coastal and Harbor Engineering Laboratory. The wave flume is

6.2 m. wide, 28.8 m. long and 1.0 m. deep. There is a separate inner channel made from plexiglass walls. This inner channel was placed in order to minimize the reflection effects from the sides of the wave flume. The inner channel where the models were built has a width of 1.5 m. and a bottom slope of 1:30.

To minimize disturbances formed by reflected waves, energy dissipaters were placed behind the model structure. Since reflected waves cause undesired agitation in the wave flume and disturbs the incoming waves, a special care should be taken in order to minimize the reflection of waves from the sides of the wave flume. The experimental set-up and the top view of the wave flume is given in Figure 3.2.

3.5.2 Measuring Apparatus

The waves generated in the wave flume were measured by separate measuring gauges also called probes placed at 8 different measuring points. The measuring gauges are of DHI type 202 which comprise two thin, parallel stainless steel electrodes. When immersed in water, the gauge measures the conductivity of the instantaneous water volume between the two electrodes, a conductivity that changes proportionally to changes of the water surface elevation, i.e. water height, between the electrodes.

Water height measured from the gauges are transferred to computer through a measurement cabinet and stored during the tests. The locations of 8 measuring points can be seen in the Figure 3.2.

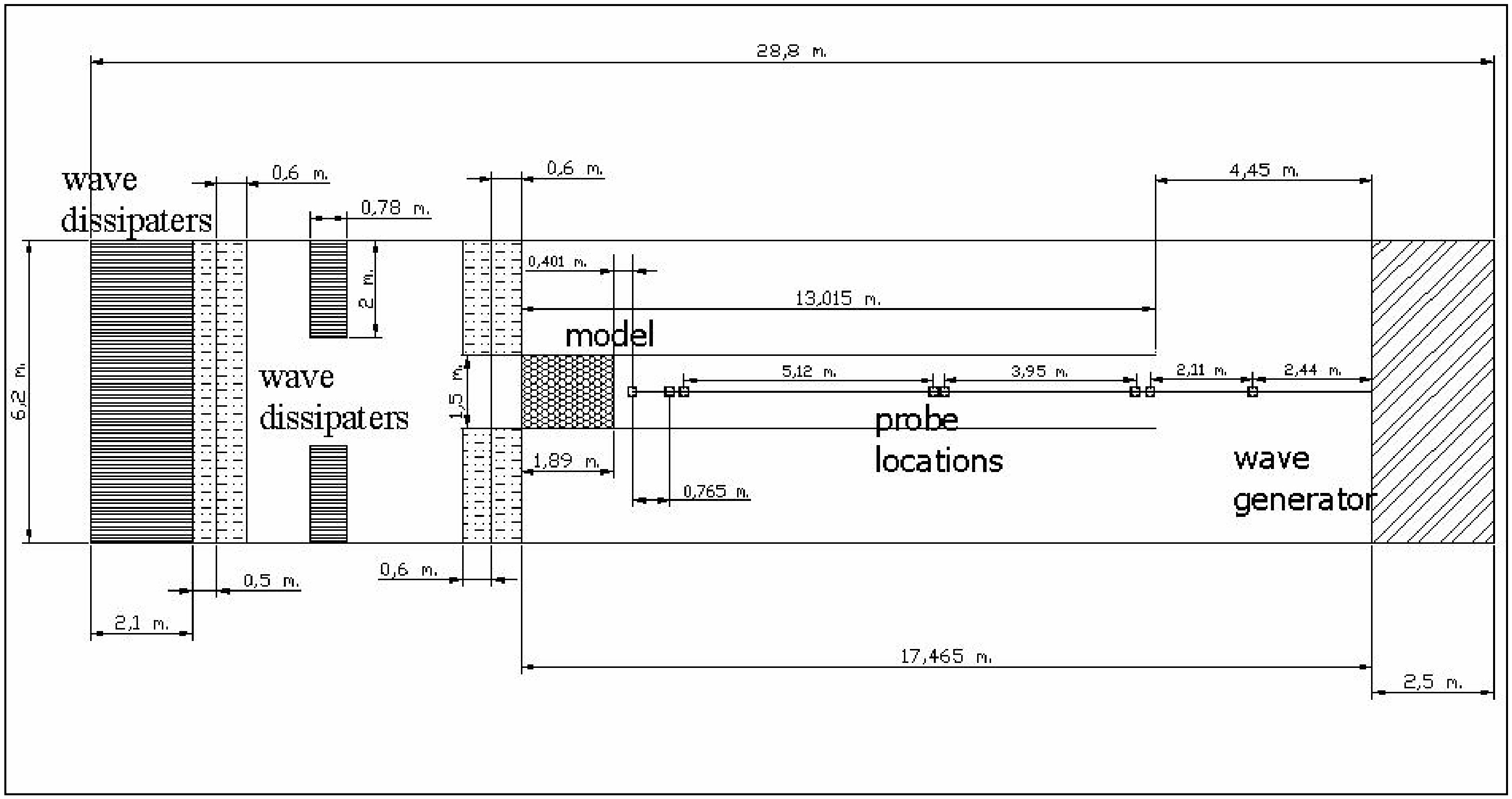

Figure 3.2 Layout of the wave flumeand the experimental set-up (model, dissipaters, generator and probes)

3.5.3 Wave Generator

For the generation of waves DHI Wave Synthesizer and DHI Hydraulic Power Pack type 301/22-PM were used. The wave generator is a paddle type wave generator composed of three main parts which are wave synthesizer software, hydraulic power pack and hydraulic servo actuator. Wave synthesizer software converts the digitized wave data into analog paddle motion signals. The hydraulic servo actuator transfers the analog signals to the piston while the necessary piston pressure maintained by hydraulic power pack.

The height and the period of the wave that can be generated in the flume depend on the water depth of the canal, as well as the technical capabilities of the wave generator. The waves that can be generated with 40 cm. of water depth in front of the generator paddle are the waves with heights ranging between 3-25 cm. and periods ranging between 0.5-15 seconds.

3.6 Selection of Model Scales

In the selection of model scale some considerations should be taken into account such as dimensions of the wave flume, available stone sizes and model wave characteristics. For the model studies length scale of 1:31.08 was found to be appropriate to use in the experiments.

As mentioned previously on part 3.1, Froude model scaling was used for the determination of time, volume and weight scales. Table 3.3 shows the scales used in the model study.

Table 3.3 Length, time, volume and weight scales used in the model

Scale of	$\lambda_m : \lambda_p$
Length (λ_L)	1:31.08
Time (λ_T)	1:5.58
Volume (λ_V)	1:30022.3
Weight (λ_W)	1:21297.2

3.7 Reflection Analysis

Due to reflection phenomenon, wave characteristics may change during the experiments. Although the energy dissipaters were placed in the channel, full prevention of wave reflection is impossible in a laboratory environment. For this reason, to find the actual wave characteristics in the channel, a special computer program (Öztunalı, METU 1998) was used. The data recorded through the wave measuring gauges, were converted to an input file in order to run the referred program. The water depth, length between measuring gauges and model scale are also needed as an input data for the execution of the program. The program gives reflection coefficient and reflected wave height as an output.

3.8 Construction of Models

The model cross-sections were built in the inner channel of the wave flume which has a bottom slope of 1:30. Quarry run material scaled with weight scale was used for building up the cross-sections.

The proportions of stone sizes for a given range were found by distributing around the mean value. To find the number of stones in each layer the net volume of a specific layer was divided by the volume of a single stone within that layer. The net volume was obtained by using porosity P=0.4 (CEM, 2003).

To be able to observe the movement of the stones within a layer, the stones were painted to form color stripes. Figure 3.3 shows the coloring of the related sections. The same color scheme was applied to all the cross-sections used in the experiments. The stones were randomly placed by releasing them about 15 cm height. The stones were not compressed or pressed by hand since it could cause over compaction of the sections and hence leading unreliable results. Iron bars were used as framework in the construction of all layers.

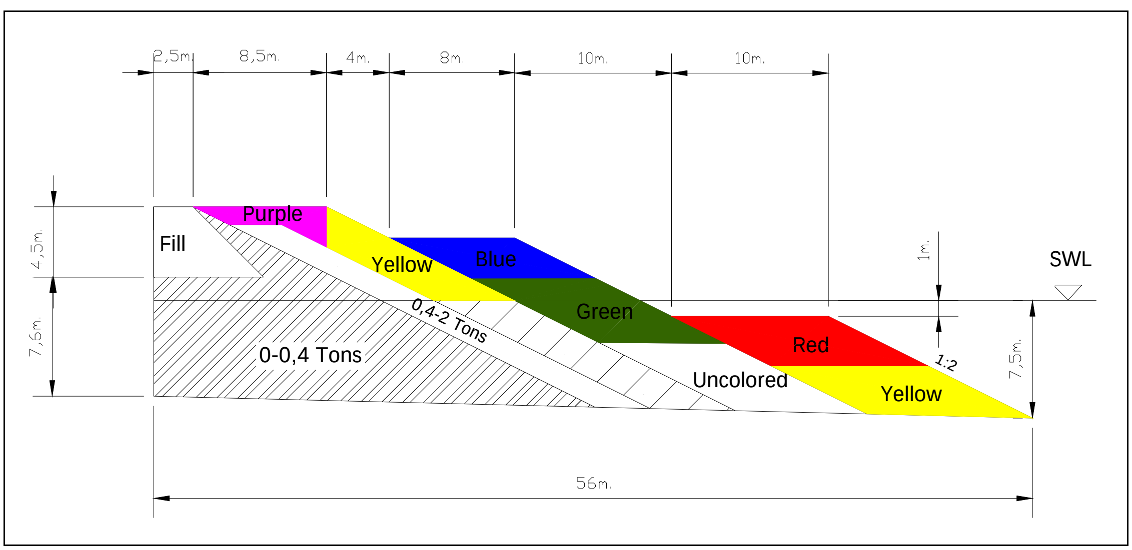

Figure 3.3 Colored stripes of models

3.9 Model Cross-Sections

Taking into consideration the construction methodologies in Turkey, the berm type coastal defense structures are in general designed with berm above still water level. In practice the height of the berm above the design water level (still water level) is normally taken as 1 m.

After the construction, during the lifetime of the structure due to settlement of stone layers, the berm might submerge into the sea. In case of excessive settlement, the berm might be submerged up to 1 m below the still water level.

Secondly, in Eastern Back Sea storm surge will cause a rise of design water level up to 2 m. Therefore in this study, the rise of water level due to storm surge is taken as 2 m above still water level. In that case a berm which is designed as 1 m above the still water level becomes a submerged berm placed 1 m below the still water level.

In the present study berm was constructed 1 m below the still water level to have comparative results with a case where berm was constructed 1 m above the still water level (Fışkın, 2004). In the model studies 5 different cross-sections were prepared by using Van der Meer design approach and tested with the design wave height.

All the cross-sections used in the experiments were made up of 5 different layers namely; front armor layer, back armor layer, two filters layers and core layer as given in Figure 3.4. Filter layers and core layer were kept the same throughout all the cross-sections since they have very little effect on the stability of breakwater in our case. However, these layers were checked for settlement after each model experiment.

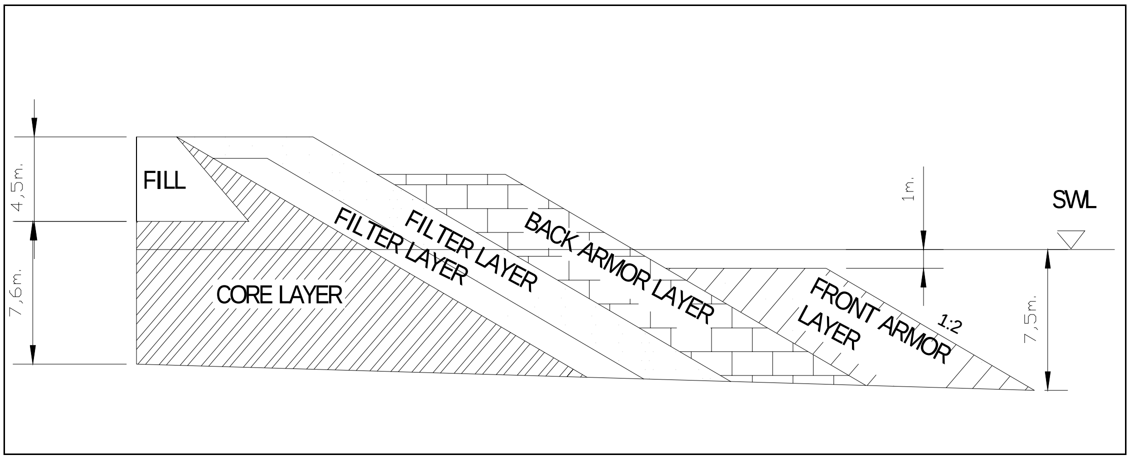

Figure 3.4 Layers of a cross-section

In the following parts, the cross-sectional views of alternative models are presented. In the figures dimensions are given for the prototype sections. Distribution and the numbers of stones of armor layers are also presented in the related tables separately for each model.

3.9.1 Model-1

In Model-1, the berm was placed 1 m. below the still water level and it had a width of 15 m. Cross-section of Model-1 can be seen in the Figure 3.5.

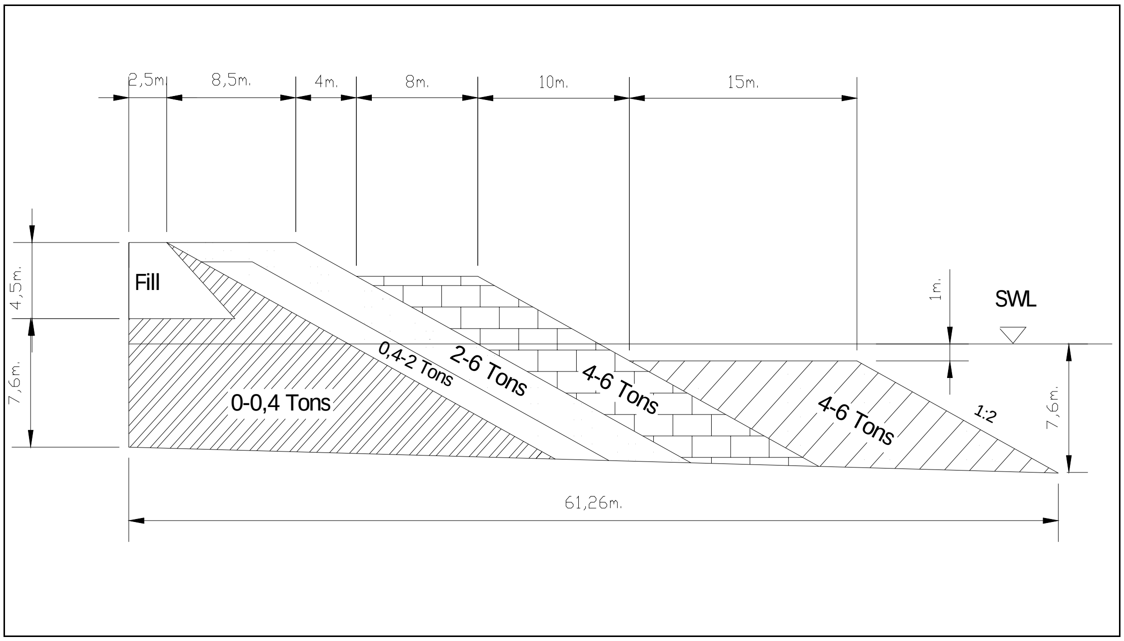

Figure 3.5 Cross-section of Model-1 (prototype values)

The stone sizes of each layer in Model-1 were as follows; core layer with 0 – 0.4 tons, filter layers with 0.4 - 2 tons and 2 – 6 tons, back armor layer with 4 – 6 tons, front armor layer with 4 – 6 tons. Distribution and number of stones used in back armor layer and front armor layer can be seen in Tables 3.4 and 3.5 respectively.

Table 3.4 Distribution and number of stones in back armor layer of Model-1

% of the total Volume	Number of stones	Weight	
		Prototype (tons)	Model (gr.)
40%	373	4	200
30%	226	5	250
30%	189	6	300
Color stripes of the layer and number of stones			
Royal Blue	263	∑788	
Green	263		
Uncolored	262		

Table 3.5 Distribution and number of stones in front armor layer of Model-1

% of the total Volume	Number of stones	Weight	
		Prototype (tons)	Model (gr.)
40%	376	4	200
30%	226	5	250
30%	188	6	300
Color stripes of the layer and number of stones			
Red	399	∑790	
Yellow	391		

3.9.2 Model-2

In Model-2, the berm was placed 1 m. below the still water level and it had a width of 10 m. Cross-section of Model-2 can be seen in the Figure 3.6.

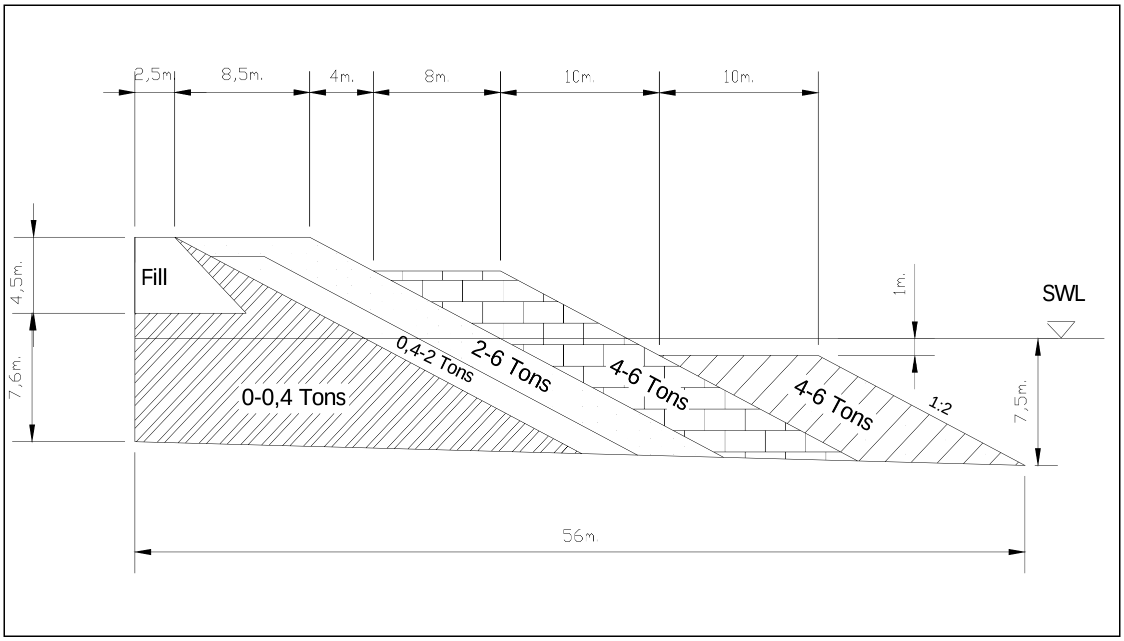

Figure 3.6 Cross-section of Model-2 (prototype values)

The stone sizes of each layer in Model-2 were as follows; core layer with 0 – 0.4 tons, filter layers with 0.4 - 2 tons and 2 – 6 tons, back armor layer with 4 – 6 tons, front armor layer with 4 – 6 tons. Distribution and number of stones used in back armor layer and front armor layer can be seen in Tables 3.6 and 3.7 respectively.

Table 3.6 Distribution and number of stones in back armor layer of Model-2

<table>
<tr><th rowspan="2">% of the total Volume</th><th rowspan="2">Number of stones</th><th colspan="2">Weight</th></tr>
<tr><th>Prototype (tons)</th><th>Model (gr.)</th></tr>
<tr><td>40%</td><td>373</td><td>4</td><td>200</td></tr>
<tr><td>30%</td><td>226</td><td>5</td><td>250</td></tr>
<tr><td>30%</td><td>189</td><td>6</td><td>300</td></tr>
<tr><th colspan="4">Color stripes of the layer and number of stones</th></tr>
<tr><td>Royal Blue</td><td colspan="2">263</td><td rowspan="3">∑788</td></tr>
<tr><td>Green</td><td colspan="2">263</td></tr>
<tr><td>Uncolored</td><td colspan="2">262</td></tr>
</table>

Table 3.7 Distribution and number of stones in front armor layer of Model-2

<table>
<tr><th rowspan="2">% of the total Volume</th><th rowspan="2">Number of stones</th><th colspan="2">Weight</th></tr>
<tr><th>Prototype (tons)</th><th>Model (gr.)</th></tr>
<tr><td>40%</td><td>262</td><td>4</td><td>200</td></tr>
<tr><td>30%</td><td>159</td><td>5</td><td>250</td></tr>
<tr><td>30%</td><td>132</td><td>6</td><td>300</td></tr>
<tr><th colspan="4">Color stripes of the layer and number of stones</th></tr>
<tr><td>Red</td><td colspan="2">284</td><td rowspan="2">∑553</td></tr>
<tr><td>Yellow</td><td colspan="2">269</td></tr>
</table>

3.9.3 Model-3

The berm was placed 1 m. below the still water level and it had a width of 5 m. Cross-section of the Model-3 can be seen in the Figure 3.7.

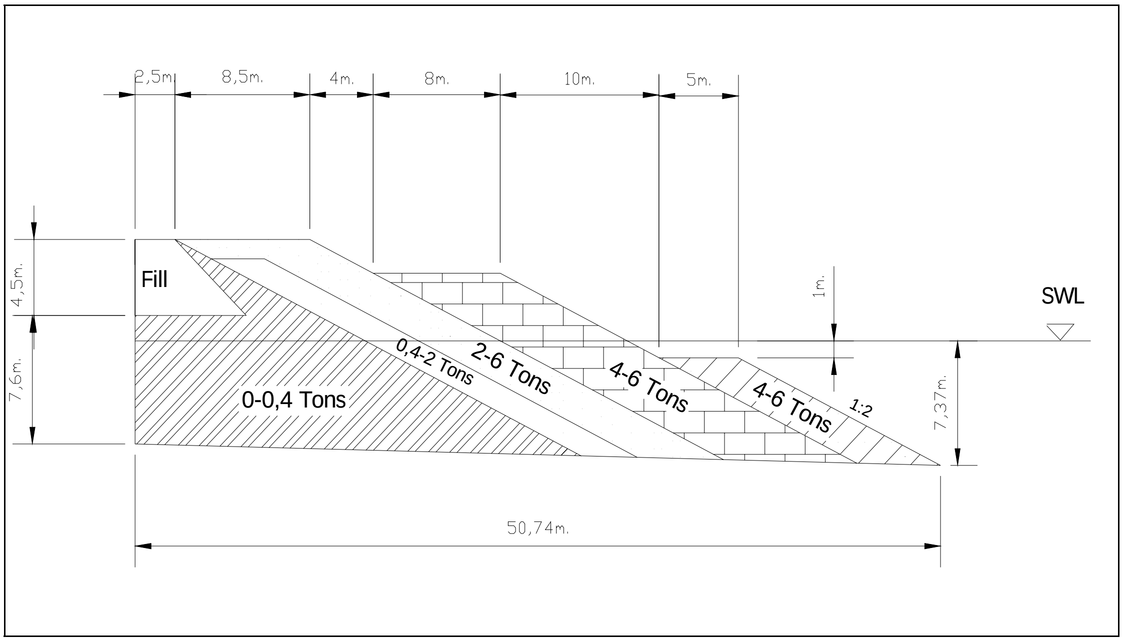

Figure 3.7 Cross-section of Model-3 (prototype values)

The stone sizes of each layer in Model-3 were as follows; core layer with 0 – 0.4 tons, filter layers with 0.4 - 2 tons and 2 – 6 tons, back armor layer with 4 – 6 tons, front armor layer with 4 – 6 tons. Distribution and number of stones used in back armor layer and front armor layer can be seen in Tables 3.8 and 3.9 respectively.

Table 3.8 Distribution and number of stones in back armor layer of Model-3

% of the total Volume	Number of stones	Weight	
		Prototype (tons)	Model (gr.)
40%	373	4	200
30%	226	5	250
30%	189	6	300
Color stripes of the layer and number of stones			
Royal Blue	263	∑788	
Green	263		
Uncolored	262		

Table 3.9 Distribution and number of stones in front armor layer of Model-3

% of the total Volume	Number of stones	Weight	
		Prototype (tons)	Model (gr.)
40%	137	4	200
30%	84	5	250
30%	69	6	300
Color stripes of the layer and number of stones			
Red	151	∑290	
Yellow	139		

3.9.4 Model-4

In Model-4, the berm was placed 1 m. below the still water level and it had a width of 10 m. Cross-section of Model-4 can be seen in the Figure 3.8.

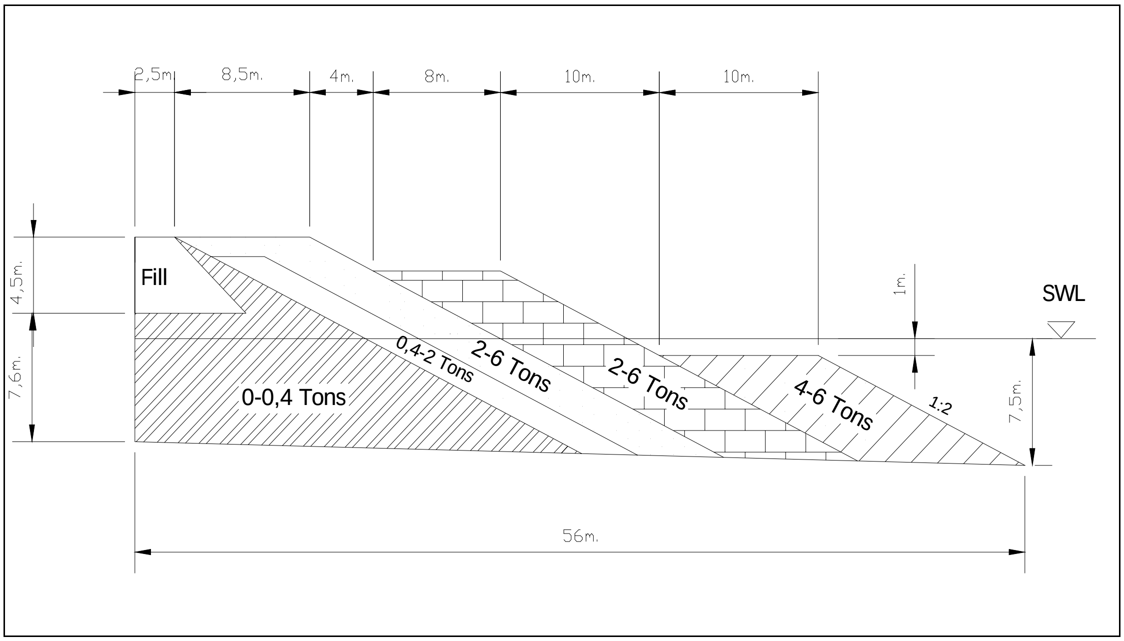

Figure 3.8 Cross-section of Model-4 (prototype values)

The stone sizes of each layer in Model-4 were as follows; core layer with 0 – 0.4 tons, filter layers with 0.4 - 2 tons and 2 – 6 tons, back armor layer with 2 – 6 tons, front armor layer with 4 – 6 tons. Distribution and number of stones used in back armor layer and front armor layer can be seen in Tables 3.10 and 3.11 respectively.

Table 3.10 Distribution and number of stones in back armor layer of Model-4

<table>
<tr><th rowspan="2">% of the total Volume</th><th rowspan="2">Number of stones</th><th colspan="2">Weight</th></tr>
<tr><th>Prototype (tons)</th><th>Model (gr.)</th></tr>
<tr><td>15%</td><td>299</td><td>2</td><td>100</td></tr>
<tr><td>35%</td><td>465</td><td>3</td><td>150</td></tr>
<tr><td>35%</td><td>342</td><td>4</td><td>200</td></tr>
<tr><td>15%</td><td>100</td><td>6</td><td>300</td></tr>
<tr><th colspan="4">Color stripes of the layer and number of stones</th></tr>
<tr><td>Royal Blue</td><td>430</td><td colspan="2" rowspan="3">∑1206</td></tr>
<tr><td>Green</td><td>394</td></tr>
<tr><td>Uncolored</td><td>382</td></tr>
</table>

Table 3.11 Distribution and number of stones in front armor layer of Model-4

<table>
<tr><th rowspan="2">% of the total Volume</th><th rowspan="2">Number of stones</th><th colspan="2">Weight</th></tr>
<tr><th>Prototype (tons)</th><th>Model (gr.)</th></tr>
<tr><td>40%</td><td>258</td><td>4</td><td>200</td></tr>
<tr><td>30%</td><td>156</td><td>5</td><td>250</td></tr>
<tr><td>30%</td><td>130</td><td>6</td><td>300</td></tr>
<tr><th colspan="4">Color stripes of the layer and number of stones</th></tr>
<tr><td>Red</td><td>275</td><td colspan="2" rowspan="2">∑544</td></tr>
<tr><td>Yellow</td><td>269</td></tr>
</table>

3.9.5 Model-5

In Model-5, the berm was placed 1 m. below the still water level and it had a width of 15 m. Cross-section of Model-5 can be seen in the Figure 3.9.

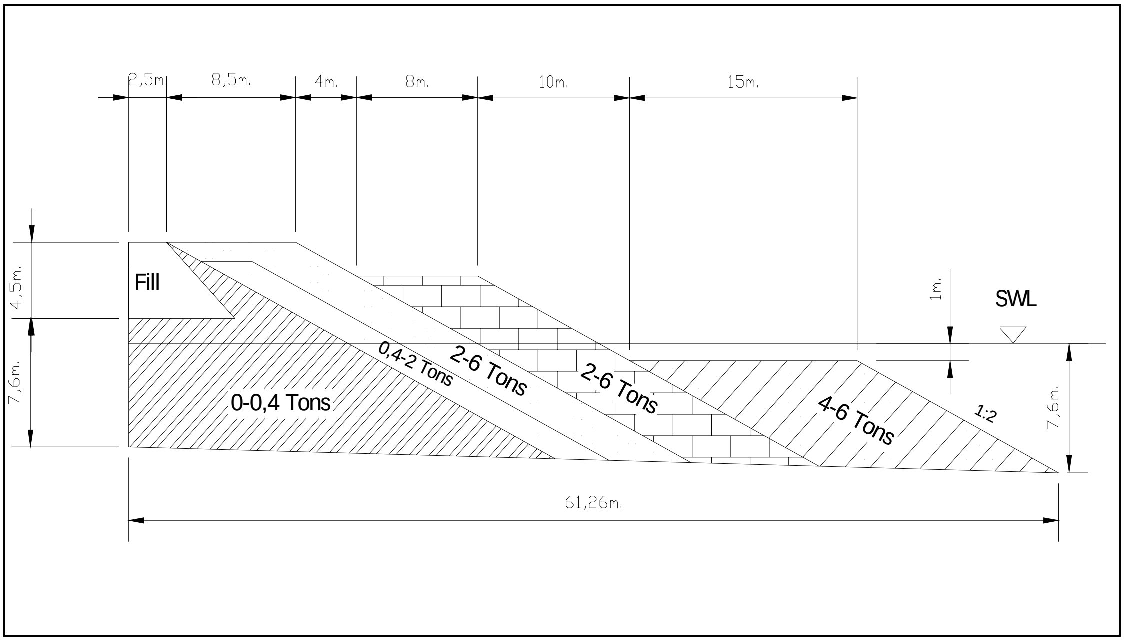

Figure 3.9 Cross-section of Model-5 (prototype values)

The stone sizes of each layer in Model-5 were as follows; core layer with 0 – 0.4 tons, filter layers with 0.4 - 2 tons and 2 – 6 tons, back armor layer with 2 – 6 tons, front armor layer with 4 – 6 tons. Distribution and number of stones used in back armor layer and front armor layer can be seen in Tables 3.12 and 3.13 respectively.

Table 3.12 Distribution and number of stones in back armor layer of Model-5

<table>
<tr><th rowspan="2">% of the total Volume</th><th rowspan="2">Number of stones</th><th colspan="2">Weight</th></tr>
<tr><th>Prototype (tons)</th><th>Model (gr.)</th></tr>
<tr><td>15%</td><td>299</td><td>2</td><td>100</td></tr>
<tr><td>35%</td><td>465</td><td>3</td><td>150</td></tr>
<tr><td>35%</td><td>342</td><td>4</td><td>200</td></tr>
<tr><td>15%</td><td>100</td><td>6</td><td>300</td></tr>
<tr><th colspan="4">Color stripes of the layer and number of stones</th></tr>
<tr><td>Royal Blue</td><td>430</td><td colspan="2" rowspan="3">∑1206</td></tr>
<tr><td>Green</td><td>394</td></tr>
<tr><td>Uncolored</td><td>382</td></tr>
</table>

Table 3.13 Distribution and number of stones in front armor layer of Model-5

<table>
<tr><th rowspan="2">% of the total Volume</th><th rowspan="2">Number of stones</th><th colspan="2">Weight</th></tr>
<tr><th>Prototype (tons)</th><th>Model (gr.)</th></tr>
<tr><td>40%</td><td>376</td><td>4</td><td>200</td></tr>
<tr><td>30%</td><td>226</td><td>5</td><td>250</td></tr>
<tr><td>30%</td><td>188</td><td>6</td><td>300</td></tr>
<tr><th colspan="4">Color stripes of the layer and number of stones</th></tr>
<tr><td>Red</td><td>399</td><td colspan="2" rowspan="2">∑790</td></tr>
<tr><td>Yellow</td><td>391</td></tr>
</table>

CHAPTER 4

EXPERIMENTS AND DISCUSSION OF RESULTS

The experiments carried out for each model cross-section together with the results and discussions are given in this chapter. Related tables and charts are provided for each test separately. Economical analysis was carried out in order to have a comparison between the cross-sections and it is given in the last part of this chapter.

In the experiments, regular waves were used for the generation of design storm. For the demonstration of the beginning and developing stages of the design storm, waves were generated in 11 time steps. Wave heights and periods were increased after each time step. Duration of each time step was chosen as 9 minutes with corresponding prototype value of 0.83 hours. Therefore, corresponding prototype value of storm duration was calculated as 9.13 hours. This value was chosen greater than the average storm duration which is 8 hours in order to be on the safe side concerning total duration of the storm.

Wave heights and corresponding wave periods with prototype and model durations that were used in the experiments are given in Table 4.1. The wave steepness was taken as 0.038 in the calculations (Özhan and Abdalla, 1999).

Table 4.1 Wave heights, wave periods and storm durations

H_0 (m.)	T (sec.)	Storm Duration	
		Prototype (hr)	Model (min)
3.69	8.21	0.83	9
3.97	8.47	0.83	9
4.19	8.71	0.83	9
4.45	8.95	0.83	9
4.70	9.18	0.83	9
4.97	9.41	0.83	9
5.23	9.63	0.83	9
5.50	9.85	0.83	9
5.77	10.06	0.83	9
6.04	10.27	0.83	9
6.31	10.47	0.83	9
		∑ 9.13 hr.	∑ 99 min.

During the experiments observations of water spraying, wave run-up and number of dislocated stones were recorded for each time step. As mentioned previously, each set of experiment consisted of 11 time steps with duration of 9 minutes. After each time step the wave generator was stopped in order to maintain a calm water surface. Water spraying and wave run-up observations were made during each time step whereas the numbers of dislocated stones were counted after the wave generator stopped.

In model studies, the structural damage was determined by the number of dislocated stones in a specified layer. In order to determine the dislocation of stones easily, the sections were painted with different colors as given in Figure 3.3. After each time step, the stones laying on different colored stripes were counted as dislocated stones and recorded for the subsequent computations. In an experimental set, number of dislocated stones was taken as accumulated number without repairing the damaged section for each test step with increased wave height. The movement of stones within the same colored stripe was not considered as dislocation. Local damage (%) was found by dividing the number of dislocated stones to the total number of stones within the specified layer as;

$$D_l = \frac{N_{dl}}{N_{tl}} \cdot 100 \qquad (4.1)$$

where,

D_l Local damage (%)

N_{dl} Number of dislocated stones in a specified layer

N_{tl} Total number of stones in a specified layer

Total damage (%) of the cross-section is then given as;

$$D_t = \frac{N_{dt}}{N_{tt}} \cdot 100 \qquad (4.2)$$

where,

D_t	Total damage (%)
N_{dt}	Number of dislocated stones within a cross-section
N_{tt}	Total number of stones within a cross-section

In the models total number of stones within specified layers and cross-sections are given in Tables 3.4 – 3.13.

During the experiments the maximum wave run-up (Figure 4.1) was also watched in order to inspect the overtopping phenomenon. The overtopping of waves may cause insignificant structural damage to the rubble mound highway defense structure, but vital damages occur in the defended structure which is a coastal highway in our case. The observed vertical level of rise of incoming wave on the surface of the breakwater was written down as run-up value using the colored stripes as reference levels.

Water spraying (Figure 4.1) due to the breaking of waves on the sloping face of the structure was another observed parameter during the experiments. The severity of water spraying classified in four different degrees namely; minor (M), tolerable (T), significant (S) and excessive (E). Run-up and water spray are given in Appendix. All of the observations gathered during the experiments were tabulated and given in number of dislocated stones, run-up and water spraying observations tables for each model in the following parts.

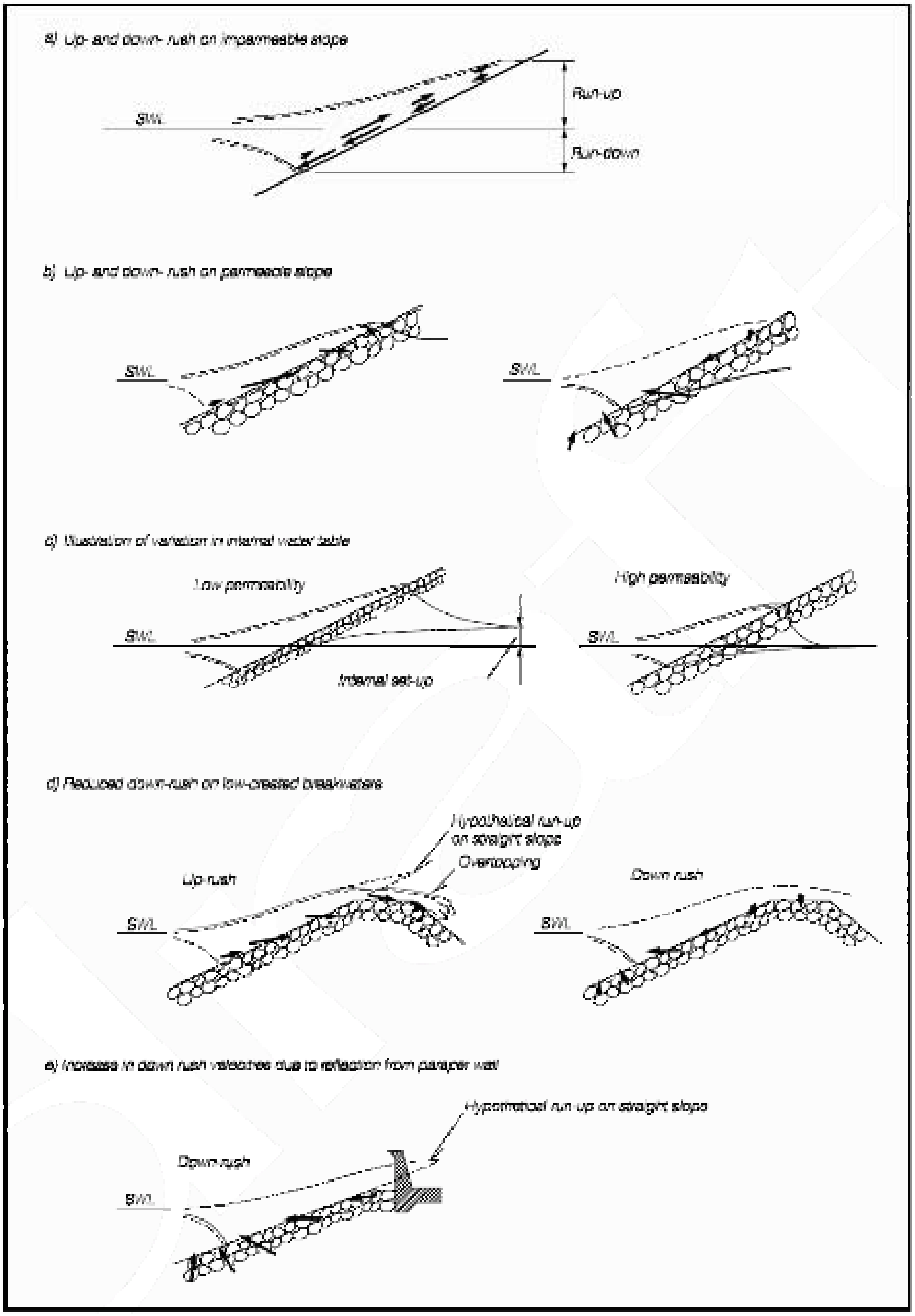

Figure 4.1 Illustration of run-up and water spraying (Burchart 1993, CEM)

4.1 Tests on Model-1

Cross-section of Model-1 is given in section 3.9.1. On this model, two sets of experiments were performed on August 11 and August 12, 2004. Wave data and the results of reflection analysis for Set 1 and Set 2 are given in Table 4.2 and Table 4.3 respectively.

Table 4.2 Test wave data and reflection analysis of Model-1, Set 1

Model# 1 (Set1) Experiment Date: August 11, 2004						
Computer Input		**Probe Measurements**		**Reflection Analysis**		
H (m.)	*T (sec.)*	*1. Probe*	*7. Probe*	*H(m.)*	*T(sec.)*	*Cr (ref. coeff.)*
3.69	8.21	3.26	2.89	3.25	8.12	0.176
3.97	8.47	3.29	3.04	3.40	8.34	0.088
4.19	8.71	4.19	3.26	2.92	8.46	0.197
4.45	8.95	3.92	3.41	4.32	8.64	0.170
4.70	9.18	4.44	4.04	3.75	8.84	0.129
4.97	9.41	5.22	4.66	3.81	8.75	0.140
5.23	9.63	4.95	4.78	4.21	9.22	0.193
5.50	9.85	5.12	4.81	4.45	9.51	0.121
5.77	10.06	5.69	5.48	5.23	9.58	0.182
6.04*	10.27	5.91	5.87	5.11	9.84	0.098
6.31**	10.47	6.37	6.46	4.66	10.04	0.099
* Breaking at the toe of the structure, ** Broken waves						

Table 4.3 Test wave data and reflection analysis of Model-1, Set 2

Model# 1 (Set2) Experiment Date: August 12, 2004						
Computer Input		**Probe Measurements**		**Reflection Analysis**		
H (m.)	*T (sec.)*	*1. Probe*	*7. Probe*	*H(m.)*	*T(sec.)*	*Cr (ref. coeff.)*
3.69	8.21	3.29	2.91	3.29	8.12	0.176
3.97	8.47	3.32	3.02	3.40	8.35	0.108
4.19	8.71	4.23	3.11	2.96	8.46	0.180
4.45	8.95	3.89	3.33	4.22	8.56	0.173
4.70	9.18	4.53	3.99	3.98	8.83	0.132
4.97	9.41	5.18	4.53	3.77	8.76	0.149
5.23	9.63	4.86	4.97	4.37	9.27	0.167
5.50	9.85	5.07	4.83	4.40	9.52	0.105
5.77	10.06	5.53	5.43	5.46	9.51	0.187
6.04*	10.27	5.84	5.79	5.05	9.87	0.105
6.31**	10.47	6.34	6.48	4.72	10.05	0.092
* Breaking at the toe of the structure, ** Broken waves						

Number of dislocated stones, wave run-up and water spraying during the experiments on Model-1 are tabulated in Tables 4.4 and 4.5 for Set 1 and Set 2 respectively.

Table 4.4 Dislocated stones, run-up and water spraying observations of Model-1, Set 1

DATE: August 11, 2004 (Set#1 Experiments)									
Ho (m.)	Duration	D A M A G E						Run-up	Water Spray
To (sec.)	(min.)	Purple	YellowI	Blue	Green	Red	YellowII		
3,69 8,21	9	-	-	-	-	-	-	mid blue layer	-
3,97 8,47	9	-	-	-	-	-	-	upper mid blue layer	-
4,19 8,71	9	-	-	-	-	-	-	upper blue layer	M
4,45 8,95	9	-	-	-	-	-	-	lower yellow layer	T
4,70 9,18	9	-	-	-	-	-	-	mid yellow layer	S
4,97 9,41	9	-	-	-	-	1(d)	-	upper mid yellow layer	S/E
5,23 9,63	9	-	-	-	-	1(d)	-	upper yellow layer	E
5,50 9,85	9	-	-	-	-	1(d)	-	upper yellow layer	E
5,77 10,06	9	-	-	2(d)	-	3(d)	-	lower purple layer	E
6,04 10,27	9	-	-	2(d)	-	4(d)	-	mid purple layer	E
6,31 10,47	9	-	-	2(d)	-	6(d)	-	upper yellow layer	E
NOTES:	1) At the last wave height which is 6,31 m. first 3-4 waves reached the lower 1/3 part of the purple layer but due to wave breaking which was between the first and second probes the run-up was reduced.								
	2) In the Run-up column, the level where the run-up reached is given in terms of the colored layers.								
	3) In the Water Spray column, the degree of water spray observed is described in terms of ; minor **(M)**, tolerable **(T)**, significant **(S)**, excessive **(E)**								
	4) In the Damage columns, the letters (u) & (d) represent "up" & "down" where; (u) means that the final position of the stone is an upper layer than its original position and (d) means the opposite.								

Table 4.5 Dislocated stones, run-up and water spraying observations of Model-1, Set 2

DATE: August 12, 2004 (Set#2 Experiments)									
Ho (m.)	Duration	**DAMAGE**						**Run-up**	**Water Spray**
To (sec.)	(min.)	Purple	YellowI	Blue	Green	Red	YellowII		
3,69 8,21	9	-	-	-	-	-	-	mid blue layer	-
3,97 8,47	9	-	-	-	-	-	-	upper mid blue layer	M
4,19 8,71	9	-	-	1(d)	-	-	-	upper blue layer	M
4,45 8,95	9	-	-	1(d)	-	-	-	lower yellow layer	T
4,70 9,18	9	-	-	1(d)	-	-	-	mid yellow layer	S
4,97 9,41	9	-	-	1(d)	-	-	-	upper mid yellow layer	E
5,23 9,63	9	-	-	1(d)	-	-	-	upper yellow layer	E
5,50 9,85	9	-	-	1(d)	-	1(d)	-	upper yellow layer	E
5,77 10,06	9	-	-	2(d)	-	3(d)	-	lower purple layer	E
6,04 10,27	9	-	-	2(d)	-	4(d)	-	mid purple layer	E
6,31 10,47	9	-	-	2(d)	-	4(d)	-	upper yellow layer	E
NOTES:	1) At the last wave height which is 6,31 m. first 3-4 waves reached the lower 1/3 part of the purple layer but due to wave breaking which was between the first and second probes the run-up was reduced.								
	2) In the Run-up column, the level where the run-up reached is given in terms of the colored layers.								
	3) In the Water Spray column, the degree of water spray observed is described in terms of ; minor **(M)**, tolerable **(T)**, significant **(S)**, excessive **(E)**								
	4) In the Damage columns, the letters (u) & (d) represent "up" & "down" where; (u) means that the final position of the stone is an upper layer than its original position and (d) means the opposite.								

As can be seen in Tables 4.4 and 4.5, maximum reach of run-up was up to mid purple layer, below the level of highway. Excessive water spray was observed after wave height increased to $H_s = 4.97$ m, with period $T_s = 9.41$ sec.

The tabulated values of corresponding percent damages in colored stripes and armor layers are given in Tables 4.6 and 4.7 for Set 1 and Set 2 respectively.

Table 4.6 Local and Total Damage (%) Table of Set 1 Experiments on Model 1 (August 11, 2004)

	Colored Sections										Date : August 11, 2004 (SET1)				
	Blue		Green (2/3)		Silver		Red		Yellow		FA		BA		Total Damage
# of stones	**263**		**176**		**0**		**399**		**391**		**790**		**439**		**1229**
Wave Height (m)	D.S.	SD	D.S.	SD	D.S.	SD	D.S.	SD	D.S.	SD	D.S.	SD	D.S.	SD	
3,69	0	0,00%	0	0,00%	0	0,00%	0	0,00%	0	0,00%	0	0,00%	0	0,00%	0,00%
3,97	0	0,00%	0	0,00%	0	0,00%	0	0,00%	0	0,00%	0	0,00%	0	0,00%	0,00%
4,19	0	0,00%	0	0,00%	0	0,00%	0	0,00%	0	0,00%	0	0,00%	0	0,00%	0,00%
4,45	0	0,00%	0	0,00%	0	0,00%	0	0,00%	0	0,00%	0	0,00%	0	0,00%	0,00%
4,70	0	0,00%	0	0,00%	0	0,00%	0	0,00%	0	0,00%	0	0,00%	0	0,00%	0,00%
4,97	0	0,00%	0	0,00%	0	0,00%	1	0,25%	0	0,00%	1	0,13%	0	0,00%	0,08%
5,23	0	0,00%	0	0,00%	0	0,00%	1	0,25%	0	0,00%	1	0,13%	0	0,00%	0,08%
5,50	0	0,00%	0	0,00%	0	0,00%	1	0,25%	0	0,00%	1	0,13%	0	0,00%	0,08%
5,77	2	0,76%	0	0,00%	0	0,00%	3	0,75%	0	0,00%	3	0,38%	2	0,25%	0,41%
6,04	2	0,76%	0	0,00%	0	0,00%	4	1,00%	0	0,00%	4	0,51%	2	0,25%	0,49%
6,31	2	0,76%	0	0,00%	0	0,00%	6	1,50%	0	0,00%	6	0,76%	2	0,25%	0,65%

NOTES 1) **D.S.**= # of displaced stones in that particular section 2) **SD** = Spatial damage in that particular colored section 3) **FA** = Front armor layer,including sections *Silver*, *Red* and *Yellow* 4) **BA** = Back armor layer, including sections *Blue* and 2/3 of *Green*

Table 4.7 Local and Total Damage (%) Table of Set 2 Experiments on Model 1 (August 12, 2004)

	Colored Sections										Date : August 12, 2004 (SET2)				
	Blue		Green (2/3)		Silver		Red		Yellow		FA		BA		Total Damage
# of stones	***263***		***176***		***0***		***399***		***391***		***790***		***439***		***1229***
Wave Height (m)	D.S.	SD	D.S.	SD	D.S.	SD	D.S.	SD	D.S.	SD	D.S.	SD	D.S.	SD	
3,69	0	0,00%	0	0,00%	0	0,00%	0	0,00%	0	0,00%	0	0,00%	0	0,00%	0,00%
3,97	0	0,00%	0	0,00%	0	0,00%	0	0,00%	0	0,00%	0	0,00%	0	0,00%	0,00%
4,19	1	0,38%	0	0,00%	0	0,00%	0	0,00%	0	0,00%	0	0,00%	1	0,13%	0,08%
4,45	1	0,38%	0	0,00%	0	0,00%	0	0,00%	0	0,00%	0	0,00%	1	0,13%	0,08%
4,70	1	0,38%	0	0,00%	0	0,00%	0	0,00%	0	0,00%	0	0,00%	1	0,13%	0,08%
4,97	1	0,38%	0	0,00%	0	0,00%	0	0,00%	0	0,00%	0	0,00%	1	0,13%	0,08%
5,23	1	0,38%	0	0,00%	0	0,00%	0	0,00%	0	0,00%	0	0,00%	1	0,13%	0,08%
5,50	1	0,38%	0	0,00%	0	0,00%	1	0,25%	0	0,00%	1	0,13%	1	0,13%	0,16%
5,77	2	0,76%	0	0,00%	0	0,00%	3	0,75%	0	0,00%	3	0,38%	2	0,25%	0,41%
6,04	2	0,76%	0	0,00%	0	0,00%	4	1,00%	0	0,00%	4	0,51%	2	0,25%	0,49%
6,31	2	0,76%	0	0,00%	0	0,00%	4	1,00%	0	0,00%	4	0,51%	2	0,25%	0,49%

NOTES 1) **D.S.**= # of displaced stones in that particular section 2) **SD** = Spatial damage in that particular colored section 3) **FA** = Front armor layer,including sections *Silver, Red* and *Yellow* 4) **BA** = Back armor layer, including sections *Blue* and 2/3 of *Green*

Tables 4.6 and 4.7 show that the maximum local damage of 1.50 percent occurred in the red section, which was the upper section of the berm. The total percent damage was calculated as 0.65 percent.

Wave height vs. total damage (%) curves of Set 1 and Set 2 experiments were plotted by using the tabulated values given in damage tables and can be seen in figure 4.2.

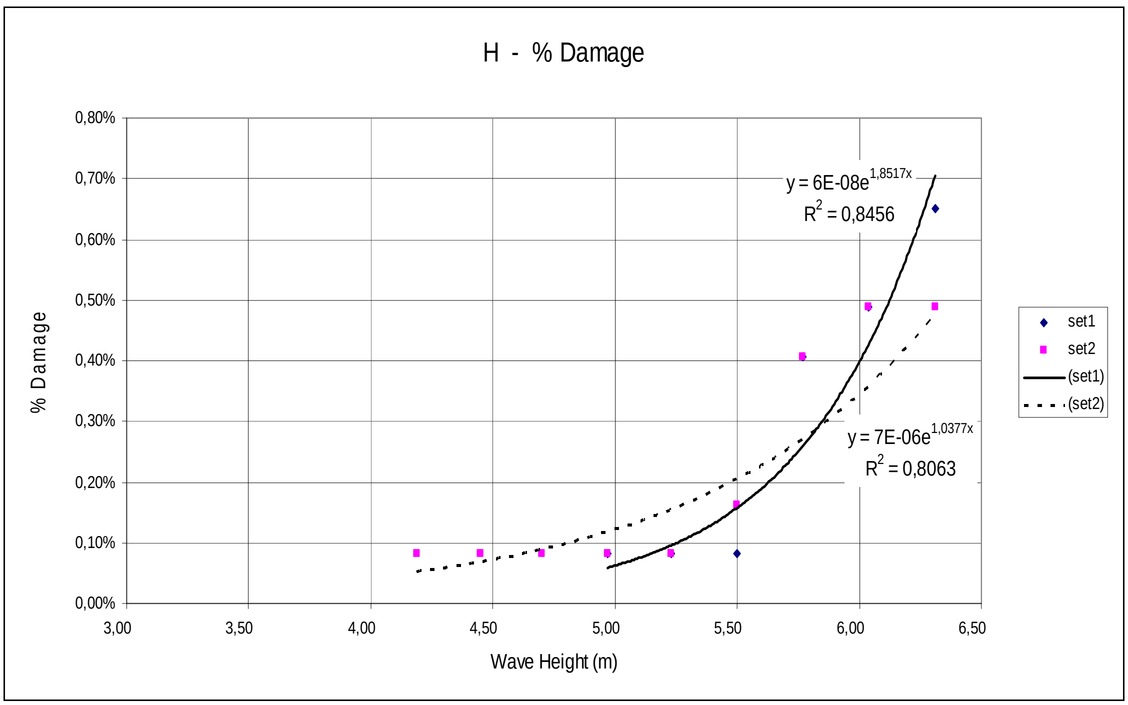

Figure 4.2 Wave height vs. total damage (%) curves for Model-1

Corresponding local damage curve for the critical section which came out to be red section is given in Figure 4.3.

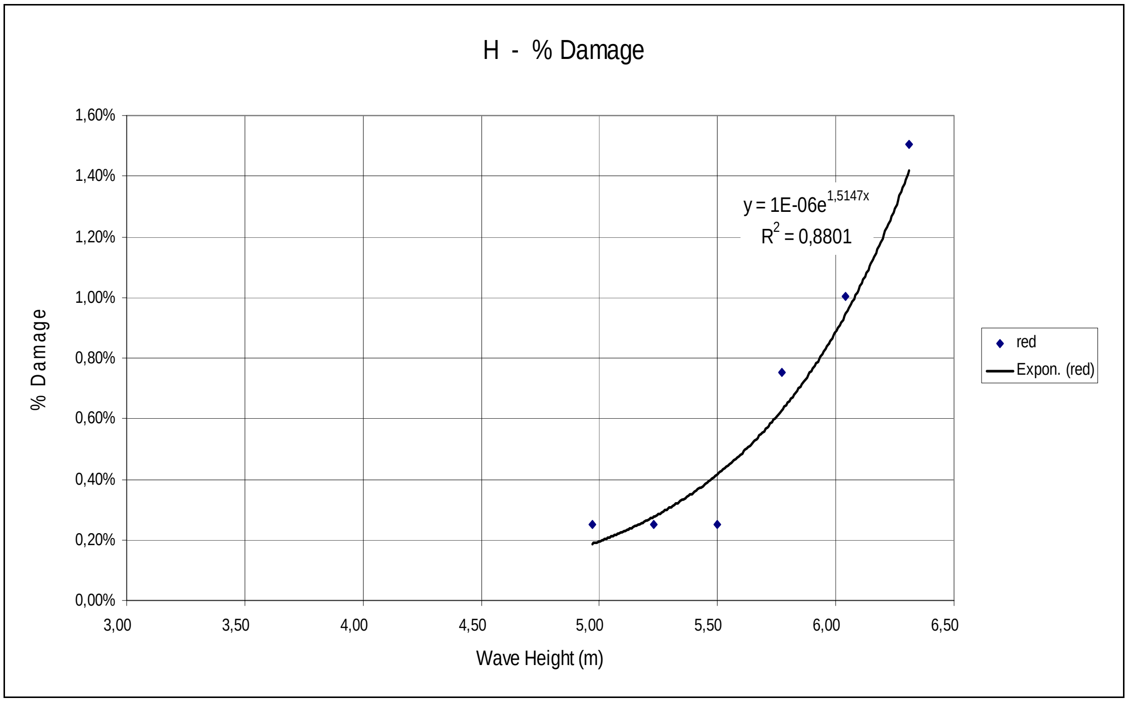

Figure 4.3 Wave height vs. local damage (%) curves for Model-1

The results showed that the cross-section was very stable and safe under design storm conditions. Maximum damage of 1.5 percent was seen in the local red section. Considering the allowable percent damage level which is 10 percent the cross-section was revised. The berm width changed to 10 m and model studies carried out with this new cross-section and the details are given in the following part.

4.2 Tests on Model-2

Cross-section of Model-2 is given in section 3.9.2. On this model, two sets of experiments were performed on August 19 and August 20, 2004. Wave data and the results of reflection analysis for Set 1 and Set 2 are given in Table 4.8 and Table 4.9 respectively.

Table 4.8 Test wave data and reflection analysis of Model-2, Set 1

Model# 2 (Set1) Experiment Date: August 19, 2004						
Computer Input		**Probe Measurements**		**Reflection Analysis**		
H (m.)	*T (sec.)*	*1. Probe*	*7. Probe*	*H(m.)*	*T(sec.)*	*Cr (ref. coeff.)*
3.69	8.21	3.04	2.99	3.17	8.11	0.152
3.97	8.47	3.23	3.01	3.22	8.34	0.088
4.19	8.71	4.04	3.32	2.87	8.41	0.170
4.45	8.95	4.17	4.28	4.05	8.59	0.105
4.70	9.18	4.31	4.35	3.68	8.79	0.091
4.97	9.41	4.81	4.98	3.74	8.78	0.150
5.23	9.63	4.74	5.12	4.16	9.24	0.134
5.50	9.85	4.86	5.23	4.24	9.48	0.072
5.77	10.06	5.84	5.91	4.89	9.52	0.155
6.04*	10.27	5.87	6.06	4.80	9.87	0.079
6.31**	10.47	5.95	6.37	4.63	10.06	0.079
* Breaking at the toe of the structure, ** Broken waves						

Table 4.9 Test wave data and reflection analysis of Model-2, Set 2

Model# 2 (Set2) Experiment Date: August 20, 2004						
Computer Input		**Probe Measurements**		**Reflection Analysis**		
H (m.)	*T (sec.)*	*1. Probe*	*7. Probe*	*H(m.)*	*T(sec.)*	*Cr (ref. coeff.)*
3.69	8.21	3.11	2.98	3.21	8.12	0.149
3.97	8.47	3.20	3.05	3.26	8.33	0.103
4.19	8.71	4.01	3.37	2.87	8.40	0.158
4.45	8.95	4.08	4.14	4.08	8.61	0.114
4.70	9.18	4.28	4.31	3.59	8.77	0.082
4.97	9.41	4.76	4.82	3.80	8.78	0.168
5.23	9.63	4.81	5.18	4.23	9.26	0.111
5.50	9.85	4.89	5.27	4.21	9.50	0.083
5.77	10.06	5.78	5.99	4.96	9.53	0.152
6.04*	10.27	5.90	6.01	4.77	9.88	0.085
6.31**	10.47	6.02	6.52	4.69	10.04	0.087
* Breaking at the toe of the structure, ** Broken waves						

Number of dislocated stones, wave run-up and water spraying during the experiments on Model-2 are tabulated in Tables 4.10 and 4.11 for Set 1 and Set 2 respectively.

Table 4.10 Dislocated stones, run-up and water spraying observations of Model-2, Set 1

DATE: August 19, 2004 (Set#1 Experiments)									
Ho (m.)	Duration	DAMAGE						Run-up	Water Spray
To (sec.)	(min.)	Purple	YellowI	Blue	Green	Red	YellowII		
3,69 8,21	9	-	-	-	-	-	-	lower blue layer	M
3,97 8,47	9	-	-	-	-	-	-	mid blue layer	M
4,19 8,71	9	-	-	-	-	-	-	upper blue layer	T
4,45 8,95	9	-	-	-	-	-	-	upper blue layer	S
4,70 9,18	9	-	-	-	-	-	-	lower yellow layer	E
4,97 9,41	9	-	-	-	-	-	-	mid yellow layer	E
5,23 9,63	9	-	-	-	-	1(d)	-	upper yellow layer	E
5,50 9,85	9	-	-	1(d)	-	1(d)	-	lower purple layer	E
5,77 10,06	9	-	-	1(d)	-	3(d)	-	mid purple layer	E
6,04 10,27	9	-	-	2(d)	-	1(u) 3(d)	-	lower purple layer	E
6,31 10,47	9	-	-	2(d)	-	1(u) 3(d)	-	lower purple layer	E
NOTES:	1) At the last wave height which is 6,31 m. first 3-4 waves reached the lower 1/3 part of the purple layer but due to wave breaking which was between the first and second probes the run-up was reduced.								
	2) In the Run-up column, the level where the run-up reached is given in terms of the colored layers.								
	3) In the Water Spray column, the degree of water spray observed is described in terms of ; minor **(M)**, tolerable **(T)**, significant **(S)**, excessive **(E)**								
	4) In the Damage columns, the letters (u) & (d) represent "up" & "down" where; (u) means that the final position of the stone is an upper layer than its original position and (d) means the opposite.								

Table 4.11 Dislocated stones, run-up and water spraying observations of Model-2, Set 2

DATE: August 20, 2004 (Set#2 Experiments)									
Ho (m.)	Duration	DAMAGE						Run-up	Water Spray
To (sec.)	(min.)	Purple	YellowI	Blue	Green	Red	YellowII		
3,69 8,21	9	-	-	-	-	-	-	lower blue layer	M
3,97 8,47	9	-	-	-	-	1(d)	-	mid blue layer	M/T
4,19 8,71	9	-	-	-	-	1(d)	-	upper blue layer	T
4,45 8,95	9	-	-	-	-	1(d)	-	upper blue layer	S
4,70 9,18	9	-	-	-	-	1(d)	-	lower yellow layer	S
4,97 9,41	9	-	-	1(d)	-	1(d)	-	mid yellow layer	E
5,23 9,63	9	-	-	1(d)	-	1(d)	-	upper yellow layer	E
5,50 9,85	9	-	-	1(d)	-	2(d)	-	lower purple layer	E
5,77 10,06	9	-	-	3(d)	-	2(d)	-	mid purple layer	E
6,04 10,27	9	-	-	4(d)	-	2(d)	-	lower purple layer	E
6,31 10,47	9	-	-	4(d)	-	3(d)	-	lower purple layer	E
NOTES:	1) At the last wave height which is 6,31 m. first 3-4 waves reached the lower 1/3 part of the purple layer but due to wave breaking which was between the first and second probes the run-up was reduced.								
	2) In the Run-up column, the level where the run-up reached is given in terms of the colored layers.								
	3) In the Water Spray column, the degree of water spray observed is described in terms of ; minor **(M)**, tolerable **(T)**, significant **(S)**, excessive **(E)**								
	4) In the Damage columns, the letters (u) & (d) represent "up" & "down" where; (u) means that the final position of the stone is an upper layer than its original position and (d) means the opposite.								

As can be seen in Tables 4.10 and 4.11, maximum reach of run-up was up to mid purple layer, below the level of highway. Excessive water spray was observed after wave height increased to $H_s = 4.97$ m, with period $T_s = 9.41$ sec.

The tabulated values of corresponding percent damages in colored stripes and armor layers are given in Tables 4.12 and 4.13 for Set 1 and Set 2 respectively.

Table 4.12 Local and Total Damage (%) Table of Set 1 Experiments on Model-2 (August 19, 2004)

	Colored Sections										Date : August 19, 2004 (SET1)				
	Blue		Green (2/3)		Silver		Red		Yellow		FA		BA		Total Damage
# of stones	**263**		**176**		**0**		**284**		**269**		**553**		**439**		**992**
Wave Height (m)	D.S.	SD	D.S.	SD	D.S.	SD	D.S.	SD	D.S.	SD	D.S.	SD	D.S.	SD	
3,69	0	0,00%	0	0,00%	0	0,00%	0	0,00%	0	0,00%	0	0,00%	0	0,00%	0,00%
3,97	0	0,00%	0	0,00%	0	0,00%	0	0,00%	0	0,00%	0	0,00%	0	0,00%	0,00%
4,19	0	0,00%	0	0,00%	0	0,00%	0	0,00%	0	0,00%	0	0,00%	0	0,00%	0,00%
4,45	0	0,00%	0	0,00%	0	0,00%	0	0,00%	0	0,00%	0	0,00%	0	0,00%	0,00%
4,70	0	0,00%	0	0,00%	0	0,00%	0	0,00%	0	0,00%	0	0,00%	0	0,00%	0,00%
4,97	0	0,00%	0	0,00%	0	0,00%	0	0,00%	0	0,00%	0	0,00%	0	0,00%	0,00%
5,23	0	0,00%	0	0,00%	0	0,00%	1	0,35%	0	0,00%	1	0,18%	0	0,00%	0,10%
5,50	1	0,38%	0	0,00%	0	0,00%	1	0,35%	0	0,00%	1	0,18%	1	0,13%	0,20%
5,77	1	0,38%	0	0,00%	0	0,00%	3	1,06%	0	0,00%	3	0,54%	1	0,13%	0,40%
6,04	2	0,76%	0	0,00%	0	0,00%	4	1,41%	0	0,00%	4	0,72%	2	0,25%	0,60%
6,31	2	0,76%	0	0,00%	0	0,00%	4	1,41%	0	0,00%	4	0,72%	2	0,25%	0,60%

NOTES 1) **D.S.**= # of displaced stones in that particular section 2) **SD** = Spatial damage in that particular colored section 3) **FA** = Front armor layer,including sections *Silver, Red* and *Yellow* 4) **BA** = Back armor layer, including sections *Blue* and 2/3 of *Green*

Table 4.13 Local and Total Damage (%) Table of Set 2 Experiments on Model-2 (August 20, 2004)

	Colored Sections										Date : August 20, 2004 (SET2)				
	Blue		Green (2/3)		Silver		Red		Yellow		FA		BA		Total Damage
# of stones	**263**		**176**		**0**		**284**		**269**		**553**		**439**		**992**
Wave Height (m)	D.S.	SD	D.S.	SD	D.S.	SD	D.S.	SD	D.S.	SD	D.S.	SD	D.S.	SD	
3,69	0	0,00%	0	0,00%	0	0,00%	0	0,00%	0	0,00%	0	0,00%	0	0,00%	0,00%
3,97	0	0,00%	0	0,00%	0	0,00%	1	0,35%	0	0,00%	1	0,18%	0	0,00%	0,10%
4,19	0	0,00%	0	0,00%	0	0,00%	1	0,35%	0	0,00%	1	0,18%	0	0,00%	0,10%
4,45	0	0,00%	0	0,00%	0	0,00%	1	0,35%	0	0,00%	1	0,18%	0	0,00%	0,10%
4,70	0	0,00%	0	0,00%	0	0,00%	1	0,35%	0	0,00%	1	0,18%	0	0,00%	0,10%
4,97	1	0,38%	0	0,00%	0	0,00%	1	0,35%	0	0,00%	1	0,18%	1	0,13%	0,20%
5,23	1	0,38%	0	0,00%	0	0,00%	1	0,35%	0	0,00%	1	0,18%	1	0,13%	0,20%
5,50	1	0,38%	0	0,00%	0	0,00%	2	0,70%	0	0,00%	2	0,36%	1	0,13%	0,30%
5,77	3	1,14%	0	0,00%	0	0,00%	2	0,70%	0	0,00%	2	0,36%	3	0,38%	0,50%
6,04	4	1,52%	0	0,00%	0	0,00%	2	0,70%	0	0,00%	2	0,36%	4	0,51%	0,60%
6,31	4	1,52%	0	0,00%	0	0,00%	3	1,06%	0	0,00%	3	0,54%	4	0,51%	0,71%

NOTES 1) **D.S.**= # of displaced stones in that particular section 2) **SD** = Spatial damage in that particular colored section 3) **FA** = Front armor layer,including sections *Silver, Red* and *Yellow* 4) **BA** = Back armor layer, including sections *Blue* and 2/3 of *Green*

Tables 4.12 and 4.13 show that the maximum local damage of 1.52 percent occurred in the blue section, which was the upper section of the back armor layer. Also, local damage of 1.41 percent occurred in the red section. The total percent damage was calculated as 0.71 percent.

Wave height vs. total damage (%) curves of Set 1 and Set 2 experiments were plotted by using the tabulated values given in damage tables and can be seen in figure 4.4.

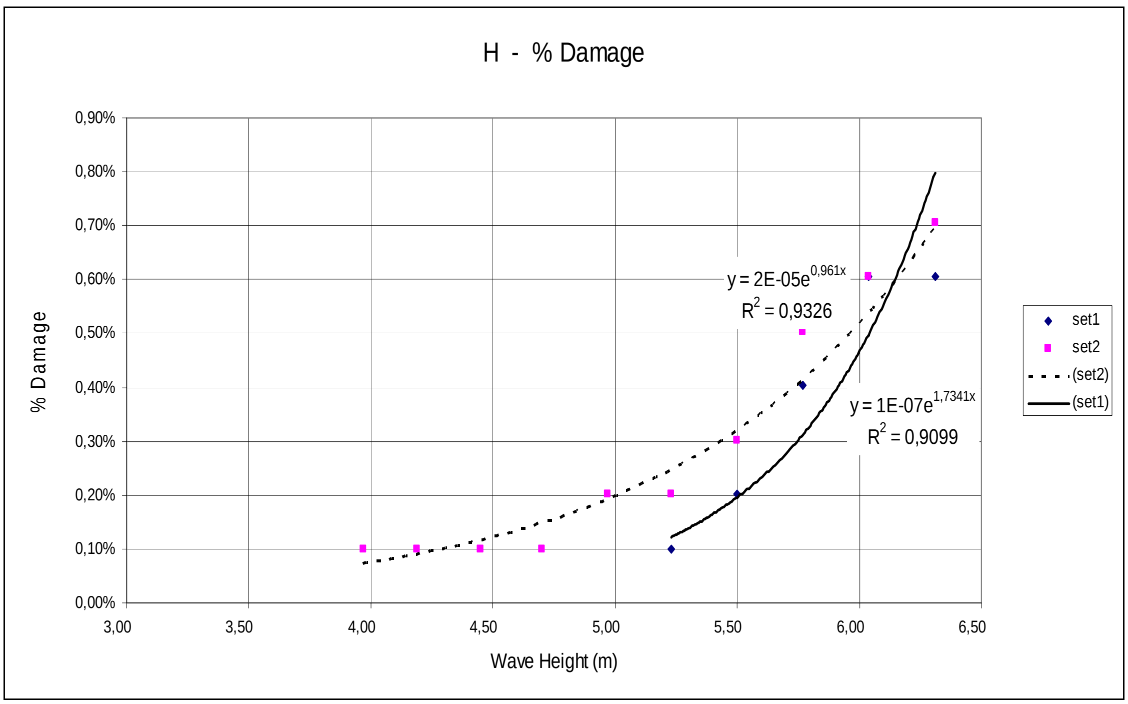

Figure 4.4 Wave height vs. total damage (%) curves for Model-2

Corresponding local damage curves for the critical sections which came out to be red and blue sections are given in Figure 4.5.

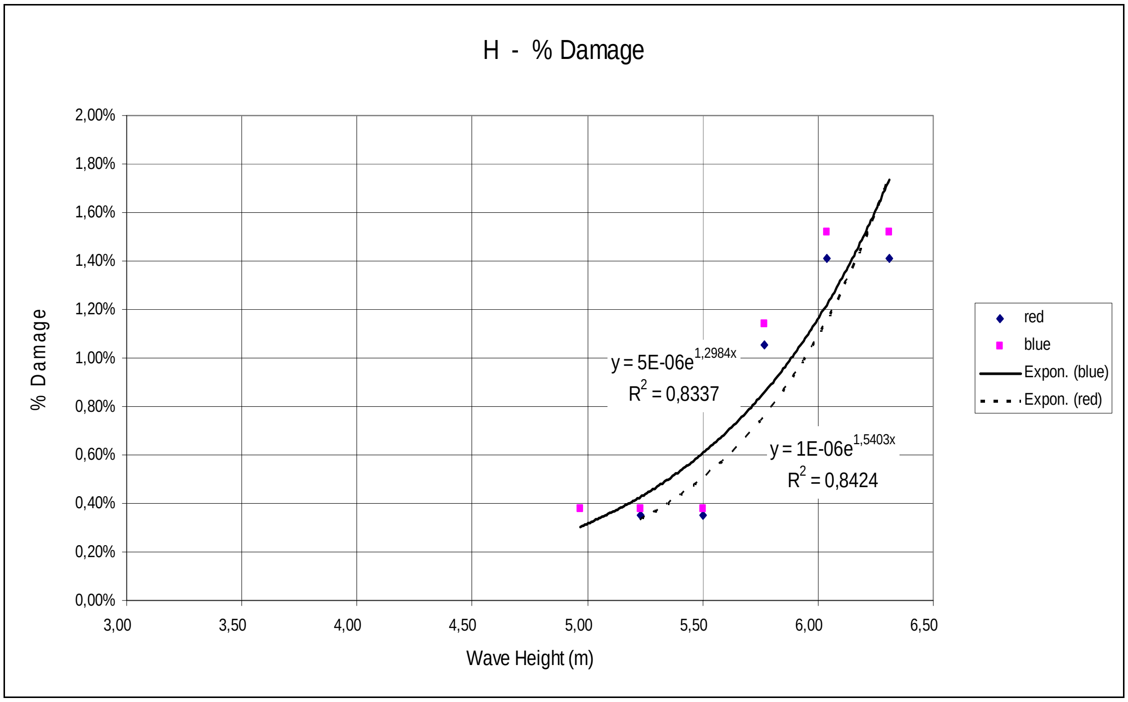

Figure 4.5 Wave height vs. local damage (%) curves for Model-2

The results showed that the cross-section was very stable and safe under design storm conditions. Maximum damage of 1.52 percent was seen in the local blue section. Considering the allowable percent damage level which is 10 percent the cross-section was revised further. The berm width changed to 5 m and model studies carried out with this new cross-section and the details are given in the following part.

4.3 Tests on Model-3

Cross-section of Model-3 is given in section 3.9.1. On this model, two sets of experiments were performed on August 13 and August 16, 2004. Wave data and the results of reflection analysis for Set 1 and Set 2 are given in Table 4.14 and Table 4.15 respectively.

Table 4.14 Test wave data and reflection analysis of Model-3, Set 1

Model# 3 (Set1) Experiment Date: August 13, 2004						
Computer Input		**Probe Measurements**		**Reflection Analysis**		
H (m.)	*T (sec.)*	*1. Probe*	*7. Probe*	*H(m.)*	*T(sec.)*	*Cr (ref. coeff.)*
3.69	8.21	3.11	3.07	3.07	8.11	0.157
3.97	8.47	3.38	3.12	3.14	8.34	0.104
4.19	8.71	3.76	3.56	2.98	8.43	0.144
4.45	8.95	3.82	3.73	3.81	8.56	0.165
4.70	9.18	4.28	4.04	3.62	8.79	0.069
4.97	9.41	4.46	4.21	3.64	8.81	0.145
5.23	9.63	4.22	4.41	4.32	9.28	0.102
5.50	9.85	4.85	4.69	4.29	9.46	0.160
5.77	10.06	5.93	5.28	4.57	9.50	0.155
6.04*	10.27	5.64	5.31	4.37	9.88	0.109
6.31**	10.47	5.61	5.71	4.80	10.06	0.082
* Breaking at the toe of the structure, ** Broken waves						

Table 4.15 Test wave data and reflection analysis of Model-3, Set 2

Model# 3 (Set2) Experiment Date: August 16, 2004						
Computer Input		**Probe Measurements**		**Reflection Analysis**		
H (m.)	*T (sec.)*	*1. Probe*	*7. Probe*	*H(m.)*	*T(sec.)*	*Cr (ref. coeff.)*
3.69	8.21	3.35	2.99	3.31	8.12	0.135
3.97	8.47	3.41	3.07	3.38	8.35	0.090
4.19	8.71	4.02	3.72	3.18	8.43	0.146
4.45	8.95	4.27	4.07	4.13	8.57	0.152
4.70	9.18	4.81	4.66	3.81	8.77	0.079
4.97	9.41	5.22	4.74	4.00	8.81	0.132
5.23	9.63	4.79	4.97	4.67	9.27	0.109
5.50	9.85	5.28	4.99	4.65	9.46	0.151
5.77	10.06	6.21	5.43	4.95	9.50	0.148
6.04*	10.27	6.07	5.84	4.76	9.87	0.102
6.31**	10.47	6.01	5.89	4.90	10.05	0.084
* Breaking at the toe of the structure, ** Broken waves						

Number of dislocated stones, wave run-up and water spraying during the experiments on Model-3 are tabulated in Tables 4.16 and 4.17 for Set 1 and Set 2 respectively.

Table 4.16 Dislocated stones, run-up and water spraying observations of Model-3, Set 1

DATE: August 13, 2004 (Set#1 Experiments)									
Ho (m.)	Duration	**D A M A G E**						**Run-up**	**Water Spray**
To (sec.)	(min.)	Purple	YellowI	Blue	Green	Red	YellowII		
3,69 8,21	9	-	-	-	-	-	-	mid blue layer	M
3,97 8,47	9	-	-	-	-	-	-	upper blue layer	T
4,19 8,71	9	-	-	-	-	-	-	upper blue layer	T/S
4,45 8,95	9	-	-	-	-	-	-	lower yellow layer	S
4,70 9,18	9	-	-	1(d)	-	-	-	upper yellow layer	E
4,97 9,41	9	-	-	1(d)	-	1(d)	-	upper yellow layer	E
5,23 9,63	9	-	-	1(d)	-	1(d)	-	lower purple layer	E
5,50 9,85	9	-	-	1(d)	-	1(d)	-	lower mid purple layer	E
5,77 10,06	9	-	-	1(d)	-	2(d)	-	mid purple layer	E
6,04 10,27	9	-	-	1(d)	-	4(d)	-	mid purple layer	E
6,31 10,47	9	-	-	2(d)	-	5(d)	-	upper mid purple layer	E
NOTES:	1) At the last wave height which is 6,31 m. first 3-4 waves reached the lower 1/3 part of the purple layer but due to wave breaking which was between the first and second probes the run-up was reduced.								
	2) In the Run-up column, the level where the run-up reached is given in terms of the colored layers.								
	3) In the Water Spray column, the degree of water spray observed is described in terms of ; minor **(M)**, tolerable **(T)**, significant **(S)**, excessive **(E)**								
	4) In the Damage columns, the letters (u) & (d) represent "up" & "down" where; (u) means that the final position of the stone is an upper layer than its original position and (d) means the opposite.								

Table 4.17 Dislocated stones, run-up and water spraying observations of Model-3, Set 2

DATE: August 13, 2004 (Set#1 Experiments)									
Ho (m.)	Duration	**DAMAGE**						**Run-up**	**Water Spray**
To (sec.)	(min.)	Purple	YellowI	Blue	Green	Red	YellowII		
3,69 8,21	9	-	-	-	-	-	-	mid blue layer	M
3,97 8,47	9	-	-	-	-	-	-	upper blue layer	T
4,19 8,71	9	-	-	-	-	-	-	upper blue layer	T/S
4,45 8,95	9	-	-	-	-	-	-	lower yellow layer	S
4,70 9,18	9	-	-	-	-	-	-	upper yellow layer	E
4,97 9,41	9	-	-	-	-	2(d)	-	upper yellow layer	E
5,23 9,63	9	-	-	-	-	2(d)	-	lower purple layer	E
5,50 9,85	9	-	-	1(d)	-	3(d)	-	lower mid purple layer	E
5,77 10,06	9	-	-	2(d)	-	6(d)	-	mid purple layer	E
6,04 10,27	9	-	-	2(d)	-	6(d)	-	upper purple layer	E
6,31 10,47	9	-	-	2(d)	1(d)	6(d)	-	upper mid purple layer	E
NOTES:	1) At the last wave height which is 6,31 m. first 3-4 waves reached the lower 1/3 part of the purple layer but due to wave breaking which was between the first and second probes the run-up was reduced.								
	2) In the Run-up column, the level where the run-up reached is given in terms of the colored layers.								
	3) In the Water Spray column, the degree of water spray observed is described in terms of ; minor **(M)**, tolerable **(T)**, significant **(S)**, excessive **(E)**								
	4) In the Damage columns, the letters (u) & (d) represent "up" & "down" where; (u) means that the final position of the stone is an upper layer than its original position and (d) means the opposite.								

As given in Tables 4.16 and 4.17, maximum reach of run-up was up to upper-mid purple layer, below the level of highway. Excessive water spray was observed after wave height increased to H_s = 4.70 m, with period T_s = 9.18 sec.

The tabulated values of corresponding percent damages in colored stripes and armor layers are given in Tables 4.18 and 4.19 for Set 1 and Set 2 respectively.

Table 4.18 Local and Total Damage (%) Table of Set 1 Experiments on Model-3 (August 13, 2004)

	Colored Sections										**Date : August 13, 2004 (SET1)**				
	Blue		Green (2/3)		Silver		Red		Yellow		FA		BA		Total Damage
# of stones	***263***		***176***		***0***		***151***		***139***		***290***		***439***		***729***
Wave Height (m)	D.S.	SD	D.S.	SD	D.S.	SD	D.S.	SD	D.S.	SD	D.S.	SD	D.S.	SD	
3,69	0	0,00%	0	0,00%	0	0,00%	0	0,00%	0	0,00%	0	0,00%	0	0,00%	0,00%
3,97	0	0,00%	0	0,00%	0	0,00%	0	0,00%	0	0,00%	0	0,00%	0	0,00%	0,00%
4,19	0	0,00%	0	0,00%	0	0,00%	0	0,00%	0	0,00%	0	0,00%	0	0,00%	0,00%
4,45	0	0,00%	0	0,00%	0	0,00%	0	0,00%	0	0,00%	0	0,00%	0	0,00%	0,00%
4,70	1	0,38%	0	0,00%	0	0,00%	0	0,00%	0	0,00%	0	0,00%	1	0,23%	0,14%
4,97	1	0,38%	0	0,00%	0	0,00%	1	0,66%	0	0,00%	1	0,34%	1	0,23%	0,27%
5,23	1	0,38%	0	0,00%	0	0,00%	1	0,66%	0	0,00%	1	0,34%	1	0,23%	0,27%
5,50	1	0,38%	0	0,00%	0	0,00%	1	0,66%	0	0,00%	1	0,34%	1	0,23%	0,27%
5,77	1	0,38%	0	0,00%	0	0,00%	2	1,32%	0	0,00%	2	0,69%	1	0,23%	0,41%
6,04	1	0,38%	0	0,00%	0	0,00%	4	2,65%	0	0,00%	4	1,38%	1	0,23%	0,69%
6,31	2	0,76%	0	0,00%	0	0,00%	5	3,31%	0	0,00%	5	1,72%	2	0,46%	0,96%

NOTES 1) **D.S.**= # of displaced stones in that particular section 2) **SD** = Spatial damage in that particular colored section 3) **FA** = Front armor layer,including sections *Silver, Red* and *Yellow* 4) **BA** = Back armor layer, including sections *Blue* and 2/3 of *Green*

Table 4.19 Local and Total Damage (%) Table of Set 2 Experiments on Model-3 (August 16, 2004)

	Colored Sections										Date : August 16, 2004 (SET2)				
	Blue		Green (2/3)		Silver		Red		Yellow		FA		BA		Total Damage
# of stones	***263***		***176***		***0***		***151***		***139***		***290***		***439***		***729***
Wave Height (m)	D.S.	SD	D.S.	SD	D.S.	SD	D.S.	SD	D.S.	SD	D.S.	SD	D.S.	SD	
3,69	0	0,00%	0	0,00%	0	0,00%	0	0,00%	0	0,00%	0	0,00%	0	0,00%	0,00%
3,97	0	0,00%	0	0,00%	0	0,00%	0	0,00%	0	0,00%	0	0,00%	0	0,00%	0,00%
4,19	0	0,00%	0	0,00%	0	0,00%	0	0,00%	0	0,00%	0	0,00%	0	0,00%	0,00%
4,45	0	0,00%	0	0,00%	0	0,00%	0	0,00%	0	0,00%	0	0,00%	0	0,00%	0,00%
4,70	0	0,00%	0	0,00%	0	0,00%	0	0,00%	0	0,00%	0	0,00%	0	0,00%	0,00%
4,97	0	0,00%	0	0,00%	0	0,00%	2	1,32%	0	0,00%	2	0,69%	0	0,00%	0,27%
5,23	0	0,00%	0	0,00%	0	0,00%	2	1,32%	0	0,00%	2	0,69%	0	0,00%	0,27%
5,50	1	0,38%	0	0,00%	0	0,00%	3	1,99%	0	0,00%	3	1,03%	1	0,23%	0,55%
5,77	2	0,76%	0	0,00%	0	0,00%	6	3,97%	0	0,00%	6	2,07%	2	0,46%	1,10%
6,04	2	0,76%	0	0,00%	0	0,00%	6	3,97%	0	0,00%	6	2,07%	2	0,46%	1,10%
6,31	2	0,76%	1	0,57%	0	0,00%	6	3,97%	0	0,00%	6	2,07%	3	0,68%	1,23%

NOTES 1) **D.S.**= # of displaced stones in that particular section 2) **SD** = Spatial damage in that particular colored section 3) **FA** = Front armor layer,including sections *Silver, Red* and *Yellow* 4) **BA** = Back armor layer, including sections *Blue* and 2/3 of *Green*

Tables 4.18 and 4.19 show that the maximum local damage of 3.97 percent occurred in the red section, which was the upper section of the berm. The total percent damage was calculated as 1.23 percent.

Wave height vs. total damage (%) curves of Set 1 and Set 2 experiments were plotted by using the tabulated values given in damage tables and can be seen in figure 4.6.

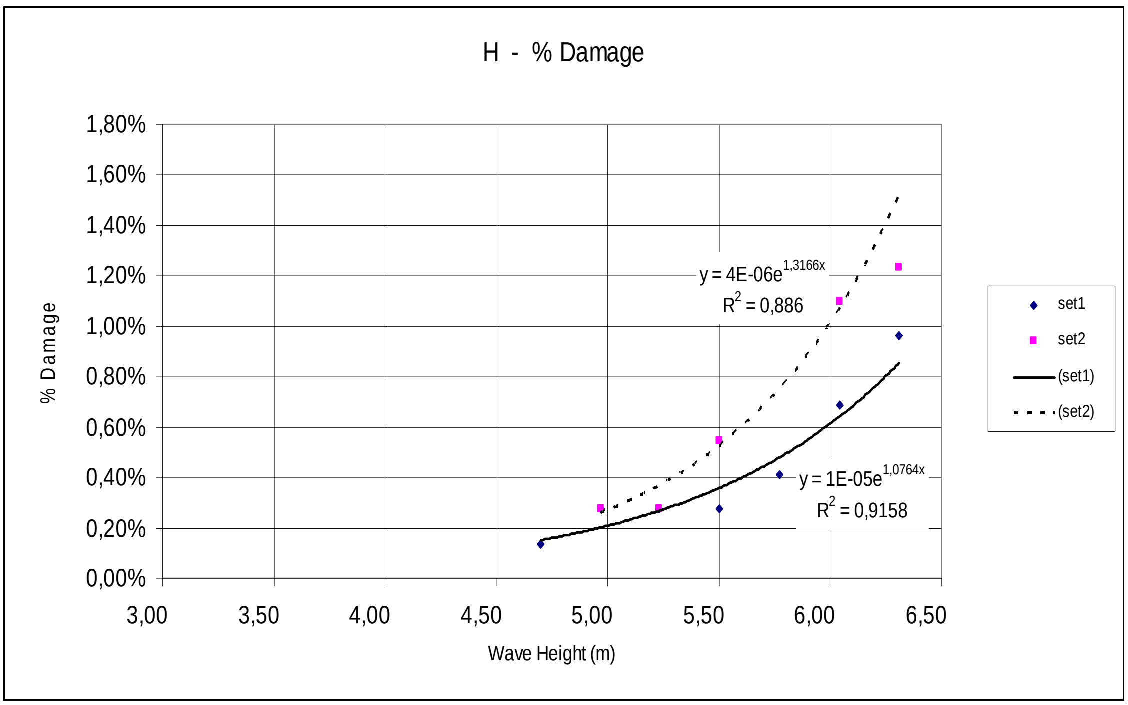

Figure 4.6 Wave height vs. total damage (%) curves for Model-3

Corresponding local damage curve for the critical section which came out to be red section is given in Figure 4.7.

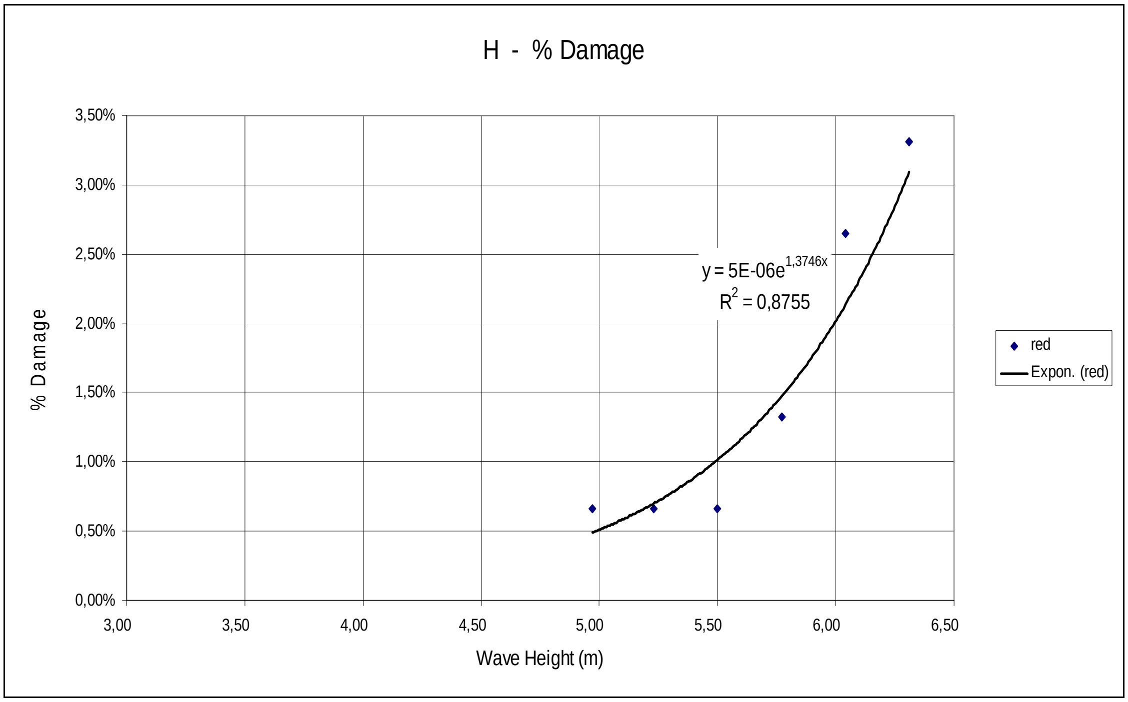

Figure 4.7 Wave height vs. local damage (%) curves for Model-3

The cross-section with a berm width of 5 m was also giving safe results from the structural safety point of view. Maximum damage of 3.97 percent occurred in the local red section. However excessive water spraying, which has significant effects on the functional properties of the coastal highway, was observed through the experiments.

Therefore the cross-section was revised by changing berm width to 10 m and stone sizes of the back armor layer to 2 – 6 tons from 4 – 6 tons. Model studies carried out with this new cross-section and the details are given in the following part.

4.4 Tests on Model-4

Cross-section of Model-4 is given in section 3.9.5. On this model, two sets of experiments were performed on August 30, 2004. Wave data and the results of reflection analysis for Set 1 and Set 2 are given in Table 4.20 and Table 4.21 respectively.

Table 4.20 Test wave data and reflection analysis of Model-4, Set 1

Model# 4 (Set1) Experiment Date: August 30, 2004						
Computer Input		**Probe Measurements**		**Reflection Analysis**		
H (m.)	*T (sec.)*	*1. Probe*	*7. Probe*	*H(m.)*	*T(sec.)*	*Cr (ref. coeff.)*
3.69	8.21	3.08	2.99	2.81	8.12	0.105
3.97	8.47	3.13	3.07	2.92	8.36	0.082
4.19	8.71	3.58	3.26	2.61	8.39	0.142
4.45	8.95	3.67	3.57	3.55	8.62	0.091
4.70	9.18	4.01	3.92	3.16	8.82	0.125
4.97	9.41	4.56	4.61	3.46	8.75	0.142
5.23	9.63	4.45	4.91	3.71	9.19	0.100
5.50	9.85	4.69	4.97	3.68	9.46	0.054
5.77	10.06	5.28	5.43	4.25	9.58	0.154
6.04*	10.27	5.13	5.59	4.37	9.88	0.087
6.31**	10.47	5.11	5.74	4.36	10.09	0.074
* Breaking at the toe of the structure, ** Broken waves						

Table 4.21 Test wave data and reflection analysis of Model-4, Set 2

Model# 4 (Set2) Experiment Date: August 30, 2004						
Computer Input		**Probe Measurements**		**Reflection Analysis**		
H (m.)	*T (sec.)*	*1. Probe*	*7. Probe*	*H(m.)*	*T(sec.)*	*Cr (ref. coeff.)*
3.69	8.21	3.11	3.04	3.35	8.12	0.108
3.97	8.47	3.17	3.07	3.46	8.34	0.103
4.19	8.71	3.72	3.41	3.12	8.42	0.126
4.45	8.95	3.91	3.57	4.17	8.62	0.101
4.70	9.18	4.26	3.63	3.81	8.78	0.102
4.97	9.41	4.84	4.38	4.10	8.76	0.167
5.23	9.63	4.72	4.78	4.43	9.23	0.083
5.50	9.85	4.75	4.80	4.46	9.43	0.082
5.77	10.06	5.31	5.37	4.92	9.58	0.144
6.04*	10.27	5.25	5.38	5.25	9.86	0.084
6.31**	10.47	5.21	5.77	5.12	10.09	0.081
* Breaking at the toe of the structure, ** Broken waves						

Number of dislocated stones, wave run-up and water spraying during the experiments on Model-4 are tabulated in Tables 4.22 and 4.23 for Set 1 and Set 2 respectively.

Table 4.22 Dislocated stones, run-up and water spraying observations of Model-4, Set 1

DATE: August 30, 2004 (Set#1 Experiments)									
Ho (m.)	Duration	DAMAGE						Run-up	Water Spray
To (sec.)	(min.)	Purple	YellowI	Blue	Green	Red	YellowII		
3,69 8,21	9	-	-	-	-	-	-	lower blue layer	-
3,97 8,47	9	-	-	-	-	4(d)	-	lower mid blue layer	-
4,19 8,71	9	-	-	-	-	7(d)	-	mid blue layer	-
4,45 8,95	9	-	-	-	-	8(d)	-	upper mid blue layer	M
4,70 9,18	9	-	-	-	-	8(d)	-	upper blue layer	M
4,97 9,41	9	-	-	-	-	9(d)	-	lower yellow layer	S
5,23 9,63	9	-	-	-	-	9(d)	-	mid yellow layer	S
5,50 9,85	9	-	-	1(d)	-	9(d)	-	upper yellow layer	E
5,77 10,06	9	-	-	6(d)	-	15(d)	-	lower purple layer	E
6,04 10,27	9	-	-	9(d)	-	23(d)	-	mid purple layer	E
6,31 10,47	9	-	-	9(d)	-	30(d)	-	lower purple layer	E
NOTES:	1) At the last wave height which is 6,31 m. first 3-4 waves reached the lower 1/3 part of the purple layer but due to wave breaking which was between the first and second probes the run-up was reduced.								
	2) In the Run-up column, the level where the run-up reached is given in terms of the colored layers.								
	3) In the Water Spray column, the degree of water spray observed is described in terms of ; minor **(M)**, tolerable **(T)**, significant **(S)**, excessive **(E)**								
	4) In the Damage columns, the letters (u) & (d) represent "up" & "down" where; (u) means that the final position of the stone is an upper layer than its original position and (d) means the opposite.								

Table 4.23 Dislocated stones, run-up and water spraying observations of Model-4, Set 2

DATE: August 30, 2004 (Set#2 Experiments)									
Ho (m.) To (sec.)	Duration (min.)	D A M A G E Purple	YellowI	Blue	Green	Red	YellowII	**Run-up**	**Water Spray**
3,69 8,21	9	-	-	-	-	3(d)	-	lower blue layer	-
3,97 8,47	9	-	-	-	-	3(d)	-	lower mid blue layer	-
4,19 8,71	9	-	-	-	-	4(d)	-	mid blue layer	-
4,45 8,95	9	-	-	-	-	4(d)	-	upper mid blue layer	M
4,70 9,18	9	-	-	-	-	5(d)	-	upper blue layer	M
4,97 9,41	9	-	-	-	-	6(d)	-	lower yellow layer	S
5,23 9,63	9	-	-	-	-	6(d)	1(d)	mid yellow layer	S
5,50 9,85	9	-	-	-	-	7(d)	1(d)	upper yellow layer	E
5,77 10,06	9	-	-	1(d)	-	22(d)	1(d)	lower purple layer	E
6,04 10,27	9	-	-	1(d)	-	1(u) 30(d)	1(d)	mid purple layer	E
6,31 10,47	9	-	-	3(d)	-	1(u) 31(d)	1(d)	lower purple layer	E

NOTES:
1) At the last wave height which is 6,31 m. first 3-4 waves reached the lower 1/3 part of the purple layer but due to wave breaking which was between the first and second probes the run-up was reduced.
2) In the Run-up column, the level where the run-up reached is given in terms of the colored layers.
3) In the Water Spray column, the degree of water spray observed is described in terms of ; minor **(M)**, tolerable **(T)**, significant **(S)**, excessive **(E)**
4) In the Damage columns, the letters (u) & (d) represent "up" & "down" where; (u) means that the final position of the stone is an upper layer than its original position and (d) means the opposite.

As can be seen in Tables 4.22 and 4.23, maximum reach of run-up was up to mid purple layer, below the level of highway. Excessive water spray was observed after wave height increased to H_s = 5.50 m, with period T_s = 9.85 sec.

The tabulated values of corresponding percent damages in colored stripes and armor layers can be seen in Tables 4.24 and 4.25 for Set 1 and Set 2 respectively.

Table 4.24 Local and Total Damage (%) Table of Set 1 Experiments on Model-4 (August 30, 2004)

	Colored Sections For Model # 10										Date : August 30, 2004 (SET1)				
	Blue		Green (2/3)		Silver		Red		Yellow		FA		BA		Total Damage
# of stones	***430***		***263***		***0***		***275***		***269***		***544***		***693***		***1237***
Wave Height (m)	D.S.	SD	D.S.	SD	D.S.	SD	D.S.	SD	D.S.	SD	D.S.	SD	D.S.	SD	
3,69	0	0,00%	0	0,00%	0	0,00%	0	0,00%	0	0,00%	0	0,00%	0	0,00%	0,00%
3,97	0	0,00%	0	0,00%	0	0,00%	4	1,45%	0	0,00%	4	0,74%	0	0,00%	0,32%
4,19	0	0,00%	0	0,00%	0	0,00%	7	2,55%	0	0,00%	7	1,29%	0	0,00%	0,57%
4,45	0	0,00%	0	0,00%	0	0,00%	8	2,91%	0	0,00%	8	1,47%	0	0,00%	0,65%
4,70	0	0,00%	0	0,00%	0	0,00%	8	2,91%	0	0,00%	8	1,47%	0	0,00%	0,65%
4,97	0	0,00%	0	0,00%	0	0,00%	9	3,27%	0	0,00%	9	1,65%	0	0,00%	0,73%
5,23	0	0,00%	0	0,00%	0	0,00%	9	3,27%	0	0,00%	9	1,65%	0	0,00%	0,73%
5,50	1	0,23%	0	0,00%	0	0,00%	9	3,27%	0	0,00%	9	1,65%	1	0,14%	0,81%
5,77	6	1,40%	0	0,00%	0	0,00%	15	5,45%	0	0,00%	15	2,76%	6	0,87%	1,70%
6,04	9	2,09%	0	0,00%	0	0,00%	23	8,36%	0	0,00%	23	4,23%	9	1,30%	2,59%
6,31	9	2,09%	0	0,00%	0	0,00%	30	10,91%	0	0,00%	30	5,51%	9	1,30%	3,15%

NOTES 1) **D.S.**= # of displaced stones in that particular section 2) **SD** = Spatial damage in that particular colored section 3) **FA** = Front armor layer,including sections *Silver, Red* and *Yellow* 4) **BA** = Back armor layer, including sections *Blue* and 2/3 of *Green*

Table 4.25 Local and Total Damage (%) Table of Set 2 Experiments on Model-4 (August 30, 2004)

	Colored Sections For Model # 10										Date : August 30, 2004 (SET2)				
	Blue		Green (2/3)		Silver		Red		Yellow		FA		BA		Total Damage
# of stones	**430**		**263**		**0**		**275**		**269**		**544**		**693**		**1237**
Wave Height (m)	D.S.	SD	D.S.	SD	D.S.	SD	D.S.	SD	D.S.	SD	D.S.	SD	D.S.	SD	
3,69	0	0,00%	0	0,00%	0	0,00%	3	1,09%	0	0,00%	3	0,55%	0	0,00%	0,24%
3,97	0	0,00%	0	0,00%	0	0,00%	3	1,09%	0	0,00%	3	0,55%	0	0,00%	0,24%
4,19	0	0,00%	0	0,00%	0	0,00%	4	1,45%	0	0,00%	4	0,74%	0	0,00%	0,32%
4,45	0	0,00%	0	0,00%	0	0,00%	4	1,45%	0	0,00%	4	0,74%	0	0,00%	0,32%
4,70	0	0,00%	0	0,00%	0	0,00%	5	1,82%	0	0,00%	5	0,92%	0	0,00%	0,40%
4,97	0	0,00%	0	0,00%	0	0,00%	6	2,18%	0	0,00%	6	1,10%	0	0,00%	0,49%
5,23	0	0,00%	0	0,00%	0	0,00%	6	2,18%	1	0,37%	7	1,29%	0	0,00%	0,57%
5,50	0	0,00%	0	0,00%	0	0,00%	7	2,55%	1	0,37%	8	1,47%	0	0,00%	0,65%
5,77	1	0,23%	0	0,00%	0	0,00%	22	8,00%	1	0,37%	23	4,23%	1	0,14%	1,94%
6,04	1	0,23%	0	0,00%	0	0,00%	31	11,27%	1	0,37%	32	5,88%	1	0,14%	2,67%
6,31	3	0,70%	0	0,00%	0	0,00%	32	11,64%	1	0,37%	33	6,07%	3	0,43%	2,91%

NOTES 1) **D.S.**= # of displaced stones in that particular section 2) **SD** = Spatial damage in that particular colored section 3) **FA** = Front armor layer,including sections *Silver, Red* and *Yellow* 4) **BA** = Back armor layer, including sections *Blue* and 2/3 of *Green*

Tables 4.24 and 4.25 show that the maximum local damage of 11.64 percent occurred in the red section, which was the upper section of the berm. The total percent damage was calculated as 3.15 percent.

Wave height vs. total damage (%) curves of Set 1 and Set 2 experiments were plotted by using the tabulated values given in damage tables and can be seen in figure 4.8.

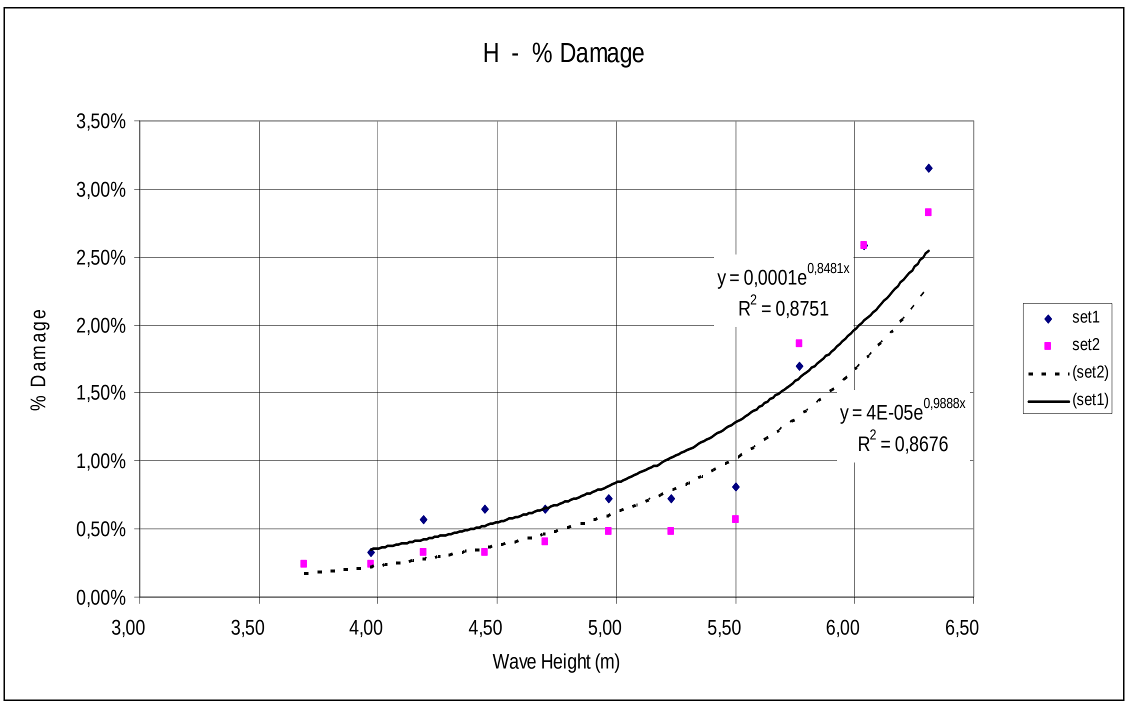

Figure 4.8 Wave height vs. total damage (%) curves for Model-4

Corresponding local damage curve for the critical section which came out to be red section is given in Figure 4.9.

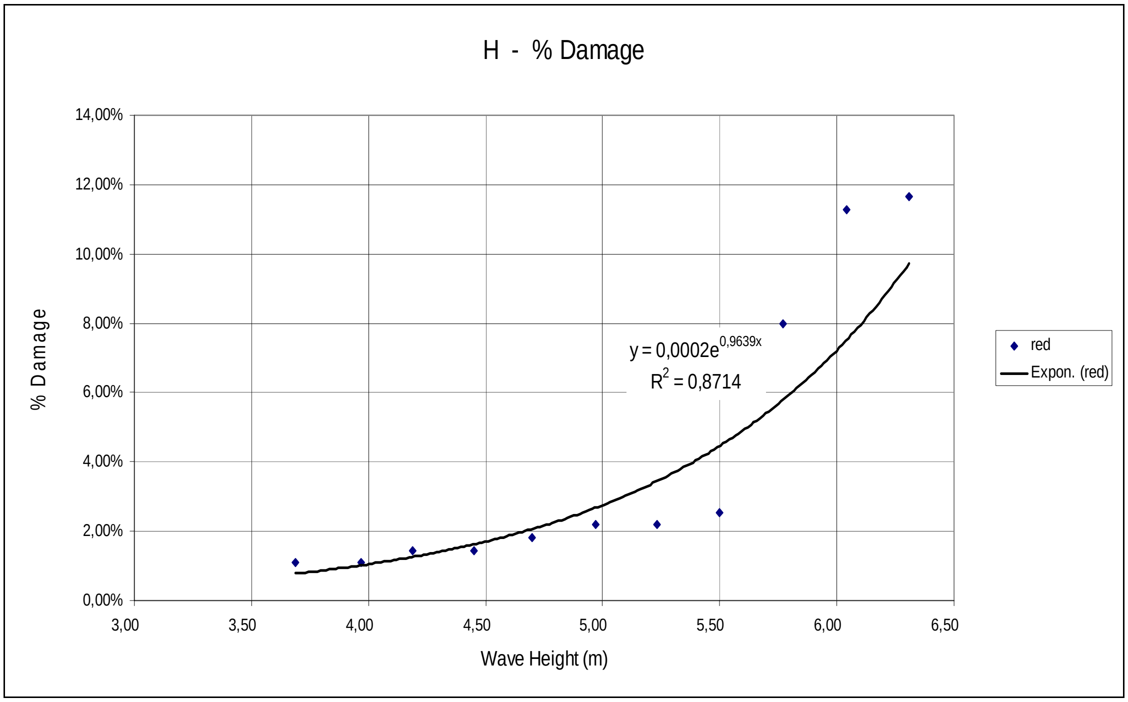

Figure 4.9 Wave height vs. local damage (%) curves for Model-4

The results show that the Model 4 was not stable under design storm conditions. Maximum damage of 11.64 percent occurred in the local red section. This value exceeds the allowable damage level which is 10 percent. As an alternative cross-section the berm width increased to 15 m and the stone sizes of back armor layer kept as 2-6 tons. Model studies carried out with this new cross-section and the details are given in the following part.

4.5 Tests on Model-5

Cross-section of Model-5 is given in section 3.9.4. On this model, two sets of experiments were performed on September 1, 2004. Wave data and the results of reflection analysis for Set 1 and Set 2 are given in Table 4.26 and Table 4.27 respectively.

Table 4.26 Test wave data and reflection analysis of Model-5, Set1

Model# 5 (Set1) Experiment Date: September 1, 2004						
Computer Input		**Probe Measurements**		**Reflection Analysis**		
H (m.)	*T (sec.)*	*1. Probe*	*7. Probe*	*H(m.)*	*T(sec.)*	*Cr (ref. coeff.)*
3.69	8.21	3.17	2.98	3.08	8.12	0.161
3.97	8.47	3.20	3.01	3.13	8.34	0.110
4.19	8.71	3.79	3.26	2.77	8.42	0.144
4.45	8.95	3.64	3.44	4.03	8.63	0.149
4.70	9.18	4.28	3.88	3.47	8.81	0.097
4.97	9.41	4.67	4.81	3.67	8.72	0.184
5.23	9.63	4.56	4.97	3.82	9.26	0.127
5.50	9.85	4.55	4.99	4.14	9.50	0.095
5.77	10.06	5.53	5.47	4.81	9.55	0.146
6.04*	10.27	5.66	5.91	4.81	9.83	0.129
6.31**	10.47	5.69	6.11	4.31	10.08	0.111
* Breaking at the toe of the structure, ** Broken waves						

Table 4.27 Test wave data and reflection analysis of Model-5, Set 2

Model# 5 (Set2) Experiment Date: September 1, 2004						
Computer Input		**Probe Measurements**		**Reflection Analysis**		
H (m.)	*T (sec.)*	*1. Probe*	*7. Probe*	*H(m.)*	*T(sec.)*	*Cr (ref. coeff.)*
3.69	8.21	3.08	2.95	3.10	8.10	0.173
3.97	8.47	3.14	2.99	3.13	8.32	0.109
4.19	8.71	3.79	3.21	2.80	8.43	0.143
4.45	8.95	3.51	3.43	4.01	8.63	0.152
4.70	9.18	4.19	4.01	3.49	8.81	0.096
4.97	9.41	4.54	4.78	3.64	8.74	0.177
5.23	9.63	4.53	4.91	3.86	9.27	0.129
5.50	9.85	4.45	4.97	4.15	9.48	0.102
5.77	10.06	5.28	5.53	4.85	9.56	0.145
6.04*	10.27	5.49	5.88	4.83	9.84	0.131
6.31**	10.47	5.61	5.99	4.31	10.08	0.108
* Breaking at the toe of the structure, ** Broken waves						

Number of dislocated stones, wave run-up and water spraying during the experiments on Model-5 are tabulated in Tables 4.28 and 4.29 for Set 1 and Set 2 respectively.

Table 4.28 Dislocated stones, run-up and water spraying observations of Model-5, Set 1

DATE: September 1, 2004 (Set#1 Experiments)									
Ho (m.)	Duration	DAMAGE						Run-up	Water Spray
To (sec.)	(min.)	Purple	YellowI	Blue	Green	Red	YellowII		
3,69 8,21	9	-	-	-	-	-	-	mid blue layer	M
3,97 8,47	9	-	-	-	-	1(d)	-	upper mid blue layer	M
4,19 8,71	9	-	-	-	-	1(d)	-	upper blue layer	M
4,45 8,95	9	-	-	-	-	1(d)	-	upper blue layer	S
4,70 9,18	9	-	-	-	-	1(d)	-	lower yellow layer	S
4,97 9,41	9	-	-	1(d)	-	1(d)	-	mid yellow layer	S
5,23 9,63	9	-	-	1(d)	-	1(d)	-	upper yellow layer	E
5,50 9,85	9	-	-	1(d)	-	1(d)	-	lower purple layer	E
5,77 10,06	9	-	-	3(d)	-	2(d)	-	lower mid purple layer	E
6,04 10,27	9	-	-	3(d)	-	6(d)	-	lower purple layer	E
6,31 10,47	9	-	-	3(d)	-	8(d)	-	lower purple layer	E
NOTES:	1) At the last wave height which is 6,31 m. first 3-4 waves reached the lower 1/3 part of the purple layer but due to wave breaking which was between the first and second probes the run-up was reduced.								
	2) In the Run-up column, the level where the run-up reached is given in terms of the colored layers.								
	3) In the Water Spray column, the degree of water spray observed is described in terms of ; minor **(M)**, tolerable **(T)**, significant **(S)**, excessive **(E)**								
	4) In the Damage columns, the letters (u) & (d) represent "up" & "down" where; (u) means that the final position of the stone is an upper layer than its original position and (d) means the opposite.								

Table 4.29 Dislocated stones, run-up and water spraying observations of Model-5, Set 2

DATE: September 1, 2004 (Set#2 Experiments)									
Ho (m.)	Duration	DAMAGE						Run-up	Water Spray
To (sec.)	(min.)	Purple	YellowI	Blue	Green	Red	YellowII		
3,69 8,21	9	-	-	-	-	-	-	mid blue layer	-
3,97 8,47	9	-	-	-	-	-	-	upper mid blue layer	M
4,19 8,71	9	-	-	-	-	-	-	upper blue layer	M
4,45 8,95	9	-	-	-	-	-	-	upper blue layer	M
4,70 9,18	9	-	-	-	-	-	-	lower yellow layer	S
4,97 9,41	9	-	-	-	-	-	-	mid yellow layer	S
5,23 9,63	9	-	-	-	-	1(d)	1(d)	upper yellow layer	E
5,50 9,85	9	-	-	-	-	1(d)	1(d)	lower purple layer	E
5,77 10,06	9	-	-	1(d)	-	2(d)	1(d)	lower mid purple layer	E
6,04 10,27	9	-	-	1(u) 1(d)	-	3(d)	1(d)	lower purple layer	E
6,31 10,47	9	-	-	1(u) 2(d)	-	3(d)	1(d)	lower purple layer	E
NOTES:	1) At the last wave height which is 6,31 m. first 3-4 waves reached the lower 1/3 part of the purple layer but due to wave breaking which was between the first and second probes the run-up was reduced.								
	2) In the Run-up column, the level where the run-up reached is given in terms of the colored layers.								
	3) In the Water Spray column, the degree of water spray observed is described in terms of ; minor **(M)**, tolerable **(T)**, significant **(S)**, excessive **(E)**								
	4) In the Damage columns, the letters (u) & (d) represent "up" & "down" where; (u) means that the final position of the stone is an upper layer than its original position and (d) means the opposite.								

As given in Tables 4.28 and 4.29, maximum reach of run-up was up to lower-mid purple layer, below the level of highway. Excessive water spray was observed after wave height increased to $H_s = 5.23$ m, with period $T_s = 9.63$ sec.

The tabulated values of corresponding percent damages in colored stripes and armor layers can be seen in Tables 4.30 and 4.31 for Set 1 and Set 2 respectively.

Table 4.30 Local and Total Damage (%) Table of Set 1 Experiments on Model-5 (September 1, 2004)

	Colored Sections										**Date : September 1, 2004 (SET1)**				
	Blue		Green (2/3)		Silver		Red		Yellow		FA		BA		Total Damage
# of stones	***430***		***263***		***0***		***399***		***391***		***790***		***693***		***1483***
Wave Height (m)	D.S.	SD	D.S.	SD	D.S.	SD	D.S.	SD	D.S.	SD	D.S.	SD	D.S.	SD	
3,69	0	0,00%	0	0,00%	0	0,00%	0	0,00%	0	0,00%	0	0,00%	0	0,00%	0,00%
3,97	0	0,00%	0	0,00%	0	0,00%	1	0,25%	0	0,00%	1	0,13%	0	0,00%	0,07%
4,19	0	0,00%	0	0,00%	0	0,00%	1	0,25%	0	0,00%	1	0,13%	0	0,00%	0,07%
4,45	0	0,00%	0	0,00%	0	0,00%	1	0,25%	0	0,00%	1	0,13%	0	0,00%	0,07%
4,70	0	0,00%	0	0,00%	0	0,00%	1	0,25%	0	0,00%	1	0,13%	0	0,00%	0,07%
4,97	1	0,38%	0	0,00%	0	0,00%	1	0,25%	0	0,00%	1	0,13%	1	0,14%	0,13%
5,23	1	0,38%	0	0,00%	0	0,00%	1	0,25%	0	0,00%	1	0,13%	1	0,14%	0,13%
5,50	1	0,38%	0	0,00%	0	0,00%	1	0,25%	0	0,00%	1	0,13%	1	0,14%	0,13%
5,77	3	1,14%	0	0,00%	0	0,00%	2	0,50%	0	0,00%	2	0,25%	3	0,43%	0,34%
6,04	3	1,14%	0	0,00%	0	0,00%	6	1,50%	0	0,00%	6	0,76%	3	0,43%	0,61%
6,31	3	1,14%	0	0,00%	0	0,00%	8	2,01%	0	0,00%	8	1,01%	3	0,43%	0,74%

NOTES 1) **D.S.**= # of displaced stones in that particular section 2) **SD** = Spatial damage in that particular colored section 3) **FA** = Front armor layer,including sections *Silver, Red* and *Yellow* 4) **BA** = Back armor layer, including sections *Blue* and 2/3 of *Green*

Table 4.31 Local and Total Damage (%) Table of Set 2 Experiments on Model-5 (September 1, 2004)

	Colored Sections										Date : September 1, 2004 (SET2)				
	Blue		Green (2/3)		Silver		Red		Yellow		FA		BA		Total Damage
# of stones	***430***		***263***		***0***		***399***		***391***		***790***		***693***		***1483***
Wave Height (m)	D.S.	SD	D.S.	SD	D.S.	SD	D.S.	SD	D.S.	SD	D.S.	SD	D.S.	SD	
3,69	0	0,00%	0	0,00%	0	0,00%	0	0,00%	0	0,00%	0	0,00%	0	0,00%	0,00%
3,97	0	0,00%	0	0,00%	0	0,00%	0	0,00%	0	0,00%	0	0,00%	0	0,00%	0,00%
4,19	0	0,00%	0	0,00%	0	0,00%	0	0,00%	0	0,00%	0	0,00%	0	0,00%	0,00%
4,45	0	0,00%	0	0,00%	0	0,00%	0	0,00%	0	0,00%	0	0,00%	0	0,00%	0,00%
4,70	0	0,00%	0	0,00%	0	0,00%	0	0,00%	0	0,00%	0	0,00%	0	0,00%	0,00%
4,97	0	0,00%	0	0,00%	0	0,00%	0	0,00%	0	0,00%	0	0,00%	0	0,00%	0,00%
5,23	0	0,00%	0	0,00%	0	0,00%	1	0,25%	1	0,26%	2	0,25%	0	0,00%	0,13%
5,50	0	0,00%	0	0,00%	0	0,00%	1	0,25%	1	0,26%	2	0,25%	0	0,00%	0,13%
5,77	1	0,23%	0	0,00%	0	0,00%	2	0,50%	1	0,26%	3	0,38%	1	0,14%	0,27%
6,04	2	0,47%	0	0,00%	0	0,00%	3	0,75%	1	0,26%	4	0,51%	2	0,29%	0,40%
6,31	3	0,70%	0	0,00%	0	0,00%	3	0,75%	1	0,26%	4	0,51%	3	0,43%	0,47%

NOTES 1) **D.S.**= # of displaced stones in that particular section 2) **SD** = Spatial damage in that particular colored section 3) **FA** = Front armor layer,including sections *Silver, Red* and *Yellow* 4) **BA** = Back armor layer, including sections *Blue* and 2/3 of *Green*

Tables 4.30 and 4.31 show that the maximum local damage of 2.01 percent occurred in the red section, which was the upper section of the berm. The total percent damage was calculated as 0.74 percent.

Wave height vs. total damage (%) curves of Set 1 and Set 2 experiments were plotted by using the tabulated values given in damage tables and can be seen in figure 4.10.

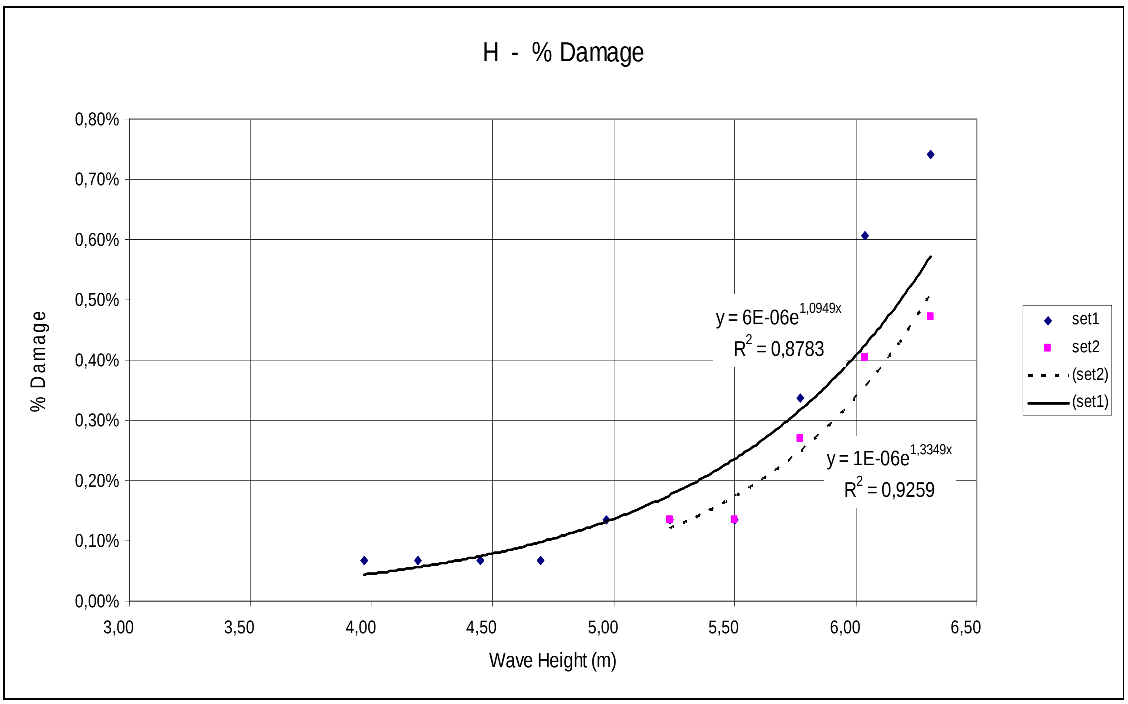

Figure 4.10 Wave height vs. total damage (%) curves for Model-5

Corresponding local damage curve for the critical section which came out to be red section is given in Figure 4.11.

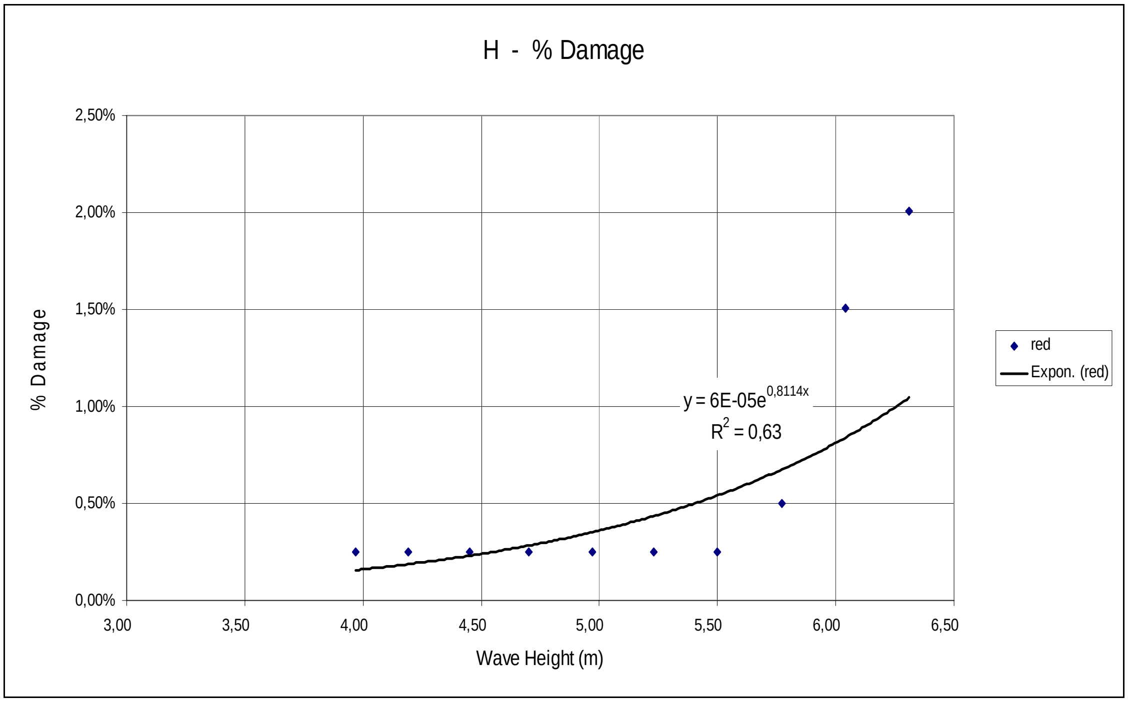

Figure 4.11 Wave height vs. local damage (%) curves for Model-5

Maximum damage of 2.01 percent occurred in the local red section. This damage level is much smaller than the maximum damage occurred in the Model 10. Water spraying was also at acceptable levels compared to other models.

4.6 Economical analysis

For detailed economical analysis, the physical and economic factors, such as design wave parameters versus the capital cost and costs of repair and maintenance, must be optimized. For the optimization, the annual cost (first cost plus maintenance cost) is plotted as a function of design wave height. As the design wave height increases, the first cost of the structure also increases. On the other hand, the annual maintenance cost will decrease if the structure is designed for a larger wave. This is usually based on some arbitrary assumptions made on a part of how many times in the life of the structure repairs will be carried out (Port Engineering: Planning, Construction, Maintenance and Security, 2004).

In this part of the study a simple comparative economical analysis on the alternative cross-sections of the coastal highway defense structures for Eastern Black Sea Coastal Highway (from Model 1 to Model 5) was carried out.

Economical analysis was carried out in two parts. In the first part, the initial costs of alternative structures were computed per unit length. Initial costs of structures included the construction cost and the cost of transportation of construction materials. In the second part, considering the possibility of damage, cost of repair and maintenance costs of the damaged alternative sections during the life time of the structure were calculated and added to the initial cost. Because of the uncertainties in the inflation rate, simple comparative economical analysis was done by using total repair and maintenance costs without considering annual values. Finally alternatives were compared in terms of total costs (per meter), which included construction, repair and maintenance, and transportation costs of the alternative cross-sections.

In the calculation of construction cost the unit prices that are given in List of Unit Prices (Year 2003 Ports List of Unit Prices, State Railways, Harbors and Airports General Directorate) were used. Unit prices for different stone sizes are given in Table 4.32. In the calculation of construction cost, the cross-sectional areas of different stone sizes were multiplied by porosity (P = 0.4). Obtained results were then multiplied with the specific weight of the stone which is 2.7 ton/m^3. These final values were the weights of the corresponding portions of the structure per meter. Multiplying these values with the unit prices of the corresponding stone sizes, construction cost of the alternative model was obtained.

Table 4.32 Unit Prices for different stone sizes (from Year 2003 Ports List of Unit Prices, State Railways, Harbors and Airports General Directorate)

	Unit Costs per ton	
Stone Sizes (ton)	**($/ton)**	**(TL/ton)**
Fill	0.94	1,409,537.00
0 – 0.4	2.31	3,465,931.00
0.4 - 2	2.48	3,723,801.00
2 – 6	3.03	4,543,748.00
4 – 6	3.23	4,843,353.00

In case of a breakwater constructed with a berm below still water level, the cost of placement and workmanship might be higher than the berm constructed above still water level. Construction of a berm above still water level can be one by trucks from the land, whereas construction of a berm below still water level has to be done from the sea by means of floating cranes. In the present work the difference in the cost could not be taken into consideration, because no information was available.

Construction cost includes all the costs like cost of material, workmanship, placement of stones, only except the cost of material transportation. Transportation cost is the cost of the material transportation from quarry to the construction site. The distance between the stone quarry and construction site was the governing parameter. Following formula was used in the calculation: (Year 2003 List of Unit Prices, State Highway General Directorate)

$$F = K \cdot (0.0007 * M + 0.01) \qquad (4.3)$$

where,

F	Transportation cost (TL/Ton)
K	70,000,000 TL for year 2003
M	Distance between two locations (km)

This formula is valid for distances greater than or equal to 50 km. Therefore M value which represents the distance between the stone quarry and the construction site was taken as 50 km in this pilot work considering the average distance that can be taken as a reference in Eastern Black Sea Coast. The result of this formula is in TL/Ton per meter. In order to calculate the transportation cost of models per meter, F value is multiplied with the total stone tonnage per meter of that model.

During the experiments it was observed that some of the model cross-sections were partially damaged. To avoid complete damage, these cross-sections have to be repaired after the damaging storm. For those cross-section alternatives, where maintenance is needed, the total cost must include construction cost, transportation cost and maintenance cost. Costs of repair and maintenance were calculated by using the maximum damage percentages obtained form the experiments for each model.

Considering the extreme value statistics which resulted in a design wave (Ho=5.80 m) with a return period of 17 years, it is most probable that this wave will strike the structure twice in structure’s lifetime which was taken as 40 years. Therefore cost of repair and maintenance calculations were based on this statistical approach.

Costs of repair and maintenance were calculated for the damaged area (displaced stones) of the cross-sections in terms of present day value. Transportation costs of construction materials were computed and added.

The economics of considering the smaller design wave must be evaluated, and the physical and economic factors, such as design wave parameters versus the capital cost and costs of repair and maintenance, must be optimized. The annual cost (first cost plus maintenance cost) is plotted as a function of design wave height.

4.6.1 Initial and Total Costs of Model-1

Cross-section of Model-1 can be seen in Figure 3.5. Initial construction cost of Model-1 was computed and presented in Table 4.33.

Table 4.33 Initial construction cost for Model-1

Cross-section	Stone Size	Unit Price (TL/ton*m)	Unit Price ($/ton*m)	Total stone weight (ton/m)	Construction Cost (TL/m)	Construction Cost ($/m)
Model 1	Fill	1,409,537	0.94	34.63	48,812,266.31	32.55
	0 - 0.4 ton	3,465,931	2.31	266.76	924,571,753.56	616.22
	0.4 - 2 ton	3,723,801	2.48	63.48	236,386,887.48	157.43
	2 - 6 ton	4,543,748	3.03	114.35	519,577,583.80	346.48
	4 - 6 ton	4,843,353	3.23	144.27	698,750,537.31	465.99
	4 - 6 ton	4,843,353	3.23	156.35	757,258,241.55	505.01
Notes: 1) Data in Unit price, Total stone weight and Cost columns are given per meter **2)** 1 U.S. Dollar is assumed to be equal to 1,500,000 TL for calculations					**3,185,357,270.01**	**2,123.68**

The transportation cost of Model-1 was obtained as 1,637.66 $/m (2,456,496,000 TL/m). Therefore the initial cost of Model-1 became 3,761.34 $/m.

Maximum total damage was obtained as 0.65 (%) as can be seen in Table 4.6. Maximum damages of 0.76 (%) and 0.25 (%) were obtained for front armor layer and back armor layer respectively. Using these damage percentages, maintenance cost of the structure for one storm was calculated as 8.25 $/m. If the structure is stroked by two storms within its lifetime, the maintenance is doubled and becomes 16.5 $/m.

4.6.2 Initial and Total Costs of Model-2

Cross-section of Model-2 can be seen in Figure 3.6. Initial construction cost of Model-2 was computed and presented in Table 4.34.

Table 4.34 Initial construction cost for Model-2

Cross-section	Stone Size	Unit Price (TL/ton*m)	Unit Price ($/ton*m)	Total stone weight (ton/m)	Construction Cost (TL/m)	Construction Cost ($/m)
Model 2	Fill	1,409,537	0.94	34.63	48,812,266.31	32.55
	0 – 0.4 ton	3,465,931	2.31	266.76	924,571,753.56	616.22
	0.4 - 2 ton	3,723,801	2.48	63.48	236,386,887.48	157.43
	2 - 6 ton	4,543,748	3.03	114.35	519,577,583.80	346.48
	4 - 6 ton	4,843,353	3.23	144.27	698,750,537.31	465.99
	4 - 6 ton	4,843,353	3.23	103.17	499,688,729.01	333.24
Notes: 1) Data in Unit price, Total stone weight and Cost columns are given per meter **2)** 1 U.S. Dollar is assumed to be equal to 1,500,000 TL for calculations					**2,927,787,757.47**	**1,951.91**

The transportation cost of Model-2 was obtained as 1,525.98 $/m (2,288,979,000 TL/m). Therefore the initial cost of Model-2 became 3,477.89 $/m.

Maximum total damage was obtained as 0.71 (%) as can be seen in Table 4.13. Maximum damages of 0.54 (%) and 0.51 (%) were obtained for front armor layer and back armor layer respectively. Using these damage percentages, maintenance cost of the structure for one storm was calculated as 7.12 $/m. If the structure is stroked by two storms within its lifetime, the maintenance is doubled and becomes 14.24 $/m.

4.6.3 Initial and Total Costs of Model-3

Cross-section of Model-3 can be seen in Figure 3.7. Initial construction cost of Model-3 was computed and presented in Table 4.35.

Table 4.35 Initial construction cost for Model-3

Cross-section	Stone Size	Unit Price (TL/ton*m)	Unit Price ($/ton*m)	Total stone weight (ton/m)	Construction Cost (TL/m)	Construction Cost ($/m)
Model 3	Fill	1,409,537	0.94	34.63	48,812,266.31	32.55
	0 – 0.4 ton	3,465,931	2.31	266.76	924,571,753.56	616.22
	0.4 - 2 ton	3,723,801	2.48	63.48	236,386,887.48	157.43
	2 - 6 ton	4,543,748	3.03	114.35	519,577,583.80	346.48
	4 - 6 ton	4,843,353	3.23	144.27	698,750,537.31	465.99
	4 - 6 ton	4,843,353	3.23	50.67	245,412,696.51	163.66
Notes: 1) Data in Unit price, Total stone weight and Cost columns are given per meter **2)** 1 U.S. Dollar is assumed to be equal to 1,500,000 TL for calculations					**2,673,511,724.97**	**1,782.33**

The transportation cost of Model-3 was obtained as 1,415.74 $/m (2,123,604,000 TL/m). Therefore the initial cost of Model-3 became 3,198.07 $/m.

Maximum total damage was obtained as 1.23 (%) as can be seen in Table 4.19. Maximum damages of 2.07 (%) and 0.68 (%) were obtained for front armor layer and back armor layer respectively. Using these damage percentages, maintenance cost of the structure for one storm was calculated as 10.82 $/m. If the structure is stroked by two storms within its lifetime, the maintenance is doubled and becomes 21.64 $/m.

4.6.4 Initial and total costs of Model-4

Cross-section of Model-4 can be seen in Figure 3.8. Initial construction cost of Model-4 was computed and presented in Table 4.36.

Table 4.36 Initial construction cost for Model-4

Cross-section	Stone Size	Unit Price (TL/ton*m)	Unit Price ($/ton*m)	Total stone weight (ton/m)	Construction Cost (TL/m)	Construction Cost ($/m)
Model 4	Fill	1,409,537	0.94	34.63	48,812,266.31	32.55
	0 – 0.4 ton	3,465,931	2.31	266.76	924,571,753.56	616.22
	0.4 – 2 ton	3,723,801	2.48	63.48	236,386,887.48	157.43
	2 - 6 ton	4,543,748	3.03	114.35	519,577,583.80	346.48
	2 - 6 ton	4,543,748	3.03	144.27	655,528,306.29	437.14
	4 - 6 ton	4,843,353	3.23	103.17	499,688,729.01	333.24
Notes: 1) Data in Unit price, Total stone weight and Cost columns are given per meter **2)** 1 U.S. Dollar is assumed to be equal to 1,500,000 TL for calculations					**2,884,565,526.45**	**1,923.06**

The transportation cost of Model-4 was obtained as 1,525.98 $/m (2,288,979,000 TL/m). Therefore the initial cost of Model-4 became 3,449.04 $/m.

Maximum total damage was obtained as 3.15 (%) as can be seen in Table 4.24. Maximum damages of 5.51 (%) and 1.30 (%) were obtained for front armor layer and back armor layer respectively. Using these damage percentages, maintenance cost of the structure for one storm was calculated as 39.92 $/m. If the structure is stroked by two storms within its lifetime, the maintenance is doubled and becomes 79.84 $/m.

4.6.5 Initial and total costs of Model-5

Cross-section of Model-5 can be seen in Figure 3.9. Initial construction cost of Model-5 was computed and presented in Table 4.37.

Table 4.37 Initial construction cost for Model-5

Cross-section	Stone Size	Unit Price (TL/ton*m)	Unit Price ($/ton*m)	Total stone weight (ton/m)	Construction Cost (TL/m)	Construction Cost ($/m)
Model 5	Fill	1,409,537	0.94	34.63	48,812,266.31	32.55
	0 – 0.4 ton	3,465,931	2.31	266.76	924,571,753.56	616.22
	0.4 - 2 ton	3,723,801	2.48	63.48	236,386,887.48	157.43
	2 - 6 ton	4,543,748	3.03	114.35	519,577,583.80	346.48
	2 - 6 ton	4,543,748	3.03	144.27	655,528,306.29	437.14
	4 - 6 ton	4,843,353	3.23	156.35	757,258,241.55	505.01
Notes: 1) Data in Unit price, Total stone weight and Cost columns are given per meter **2)** 1 U.S. Dollar is assumed to be equal to 1,500,000 TL for calculations					**3,142,135,038,99**	**2,094.83**

The transportation cost of Model-5 was obtained as 1,637.66 $/m (2,288,979,000 TL/m). Therefore the initial cost of Model-5 became 3,732.49 $/m.

Maximum total damage was obtained as 0.74 (%) as can be seen in Table 4.30. Maximum damages of 1.01 (%) and 0.43 (%) were obtained for front armor layer and back armor layer respectively. Using these damage percentages, maintenance cost of the structure for one storm was calculated as 11.60 $/m. If the structure is stroked by two storms within its lifetime, the maintenance is doubled and becomes 23.2 $/m.

4.6.6 Cost Comparison of Models

The values of initial and total costs of all models were calculated and represented in Table 4.38. Final results of economical analyses are represented in Figure 4.12 which gives the comparison of total costs of all models.

Table 4.38 Initial and total costs of all models

Model	Initial Cost		Total Cost	
	TL/m	$/m	TL/m	$/m
Model-1	5,642,010,000	3,761.34	5,666,760,000	3,777.84
Model-2	5,216,835,000	3,477.89	5,238,195,000	3,492.13
Model-3	4,797,105,000	3,198.07	4,829,565,000	3,219.71
Model-4	5,173,560,000	3,449.04	5,293,320,000	3,528.88
Model-5	5,598,735,000	3,732.49	5,633,535,000	3,755.69

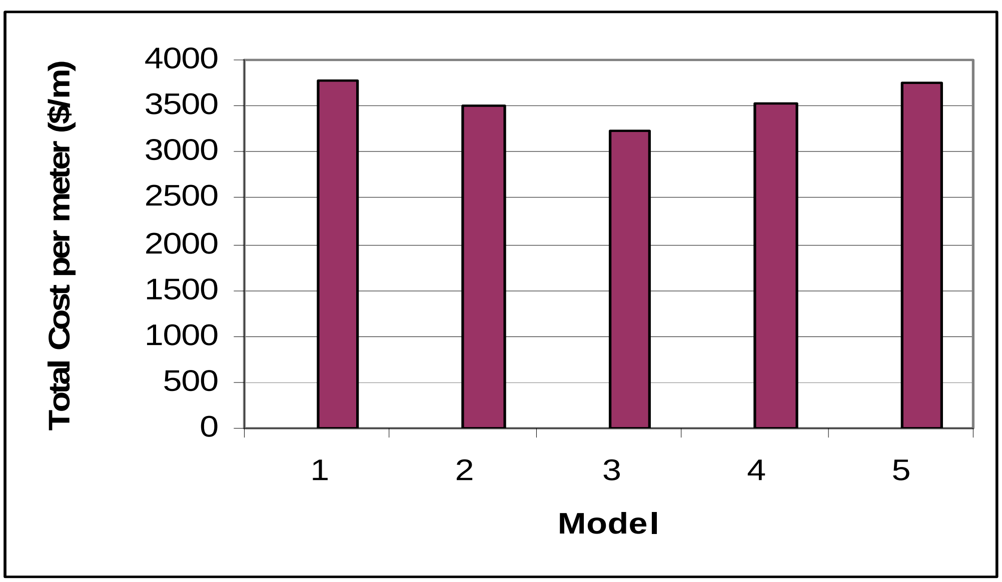

Figure 4.12 Comparison of total costs

The results of economical analysis showed that Model-1 was the most expensive cross-section. On the other hand Model-3 was the cheapest of all the cross-sections. However the final decision for the selection of most optimum cross-section have to be made by considering also the damage level, water spray and run-up parameters.

In the computation of total cost (per meter) of the coastal defense structures, cross-sections at a depth of d = 7.5 m in the model were tested and used in the economical analysis. However within the range of the project, because of the changing sea bed topography the depth of construction ranges between 3 to 9 m. The model experiments at the depth of construction of d = 7.5 m, covered the most critical wave condition imposed on the structure. Therefore, the economical analysis carried out for a depth of 7.5 m, resulted in maximum stone sizes in the armor layer yielding the maximum total cost per meter.

Since the depth of construction changes along the coastal highway, depths less than 7.5 m will give smaller stone sizes than the tested models. For example if an average depth of 5 m is taken, by using the same computational procedure and the model stability tests, the most critical condition would be again waves breaking at the toe of the structure, yet the armor stone sizes would be smaller compared to those for the construction depth of 7.5 m. To assess the overall project cost accurately, cross-sections designed with different armor layer stone sizes at different construction depths have to be taken into consideration (Taşkıran, 2004).

CHAPTER 5

CONCLUSIONS

In the present study further optimization of the coastal protection structures of Eastern Black sea Highway Project was carried out by using submerged berm which was placed 1 m below the still water level. The effect of berm width and stone sizes of back armor layer were taken into account and alternative cross-sections were tested with model studies carried out in a wave flume at the Coastal and Harbor Engineering Laboratory of the Middle East Technical University (METU), Ankara.

The pilot area had been selected as Ordu-Giresun region, where there is an ongoing highway coastal defense structure construction. Sea bottom slope of 1:30 was selected as the representative foreshore slope for the specified region. For the most critical stability condition waves breaking at the toe of the structure were taken into consideration. Breaking wave height was calculated as H_b = 6.5 m for a sea bottom slope of 1:30 at the depth of 7.5 m, which was the construction depth at the toe of the structure. The corresponding deep water significant wave height of the breaking wave was then obtained as H_{S0} = 5.8 m in deep water. Using a wave steepness of H_{S0} / L_{S0} = 0.038, corresponding design significant wave period was calculated as T = 9.90 sec.

For the model studies, hydraulic models were constructed with a length scale of 1:31.08 (Dedeoğlu, 2003 and Taşkıran, 2003) using Van der Meer equations. Regular waves were used for the generation of design storm. For the demonstration of the beginning and developing stages of the design storm, waves were generated in 11 time steps, where the significant wave heights were ranging between 3.69 m and 6.31 m, and corresponding wave periods were ranging between 8.21 sec and 10.47 sec.

5 different models were tested in order to find the most stable and economic cross-section. In all of the alternative cross-sections berm was constructed 1 m below the still water level. Different berm widths (5 m, 10 m and 15 m) were used to see the effects of berm width. Also the stone sizes of back armor layer were changed from 4 – 6 tons to 2 – 6 tons. Stone sizes of front armor layer (4 – 6 tons) and the under layers were same for all the alternative cross-sections.

The results of the experiments showed that the minimum cumulative total damage (%) of 0.65 % was occurred in the Model-1 (Figure 3.5) which was constructed with 15 m berm width and 4 – 6 tons of front and back armor layers. For this model excessive water spray (causing wetness on the highway) was observed before the fully developed storm condition reached at a wave height of H_S = 5.23 m with a period of T = 9.63 sec. However, run-up always remained below the level of highway (+6 m above the still water level). When Model-1 results compared with the similar model (Model 2) constructed with berm +1 m above still water level (Fışkın, 2004), it was seen that the cumulative total damage (%) was less. This may be due to the reason that waves are not directly attacking the armor stones on the berm due to the water depth existing. In both cases cumulative total damage (%) were below the no damage criteria (D = 5 %). On the other hand in the case of berm constructed above still water level both spray and run-up observed to be less (Fışkın, 2004).

Maximum cumulative total damage (%) of 3.15 % was occurred in the Model-4 (Figure 3.8) which was constructed with 10 m berm width and 2 - 6 tons of back armor layer. The local damage (%) of 11.64 occurred in this model exceeded the allowable damage percentage. For this model excessive water spray was observed at a wave height of H_S = 5.50 m with a period of T = 9.85 sec. Run-up always remained below the level of highway.

In the Present study, in all the sections tested with variable width and back armor layer stone sizes, the cumulative total and local damages (%) were not exceeding D_t = 4 % and D_l = 12 % respectively.

Excessive water spray were observed in all cases due to the fact that waves hitting the back armor layer without much loss of energy due to the water depth above the berm. However in all models run-up was not significant.

An economical analysis was carried out where the initial (capital) cost and cost of repair and maintenance were considered to obtain a cross-section which is both stable and economically optimized. In this respect, the cost analyses of all the models tested were carried out using both the initial cost and the cost of repair and maintenance.

The economical analysis was a first attempt to have a tool for comparison between the different cross-sections of almost the same damage level. It is however, strongly recommended that water spray and run-up as seen in the present experiments must be parameters taken into consideration for the final decision.

Model-3 was found to be the most economical section with a total cost of 4,829,565,000 TL/m (Table 4.38.) where the cumulative total damage (%) was less than 2 %. However, in this model the water spray was very excessive. Therefore considering total costs, water spraying and damage percentages Model-5 with a total cost of 5,633,535,000 TL/m (Table 4.38) were recommended where the cumulative total damage (%) was less than 1 %.

For further studies it can be recommended that the effect of wave irregularity should be studied in order to have more similarity with the prototype conditions.

It is also recommended that the alternative cross-sections with different under layer stone sizes should be tested for the further optimization of the cross-sections.

The construction cost of submerged berm should be added to the economical analysis in order to have more reliable results.

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APPENDIX

Figure A.1 Top view of Model-2

Figure A.2 Run-up and water spraying