COMPARISON OF DIFFERENT ARMOUR UNITS OF COASTAL STRUCTURES IN RIZE-ARTVIN AIRPORT UNDER OVERTOPPING AND STABILITY CONDITIONS

A THESIS SUBMITTED TO
THE GRADUATE SCHOOL OF NATURAL AND APPLIED SCIENCES
OF
MIDDLE EAST TECHNICAL UNIVERSITY

BY

ARİF UĞURLU

IN PARTIAL FULFILLMENT OF THE REQUIREMENTS
FOR
THE DEGREE OF MASTER OF SCIENCE
IN
CIVIL ENGINEERING

AUGUST 2017

Approval of the thesis:

COMPARISON OF DIFFERENT ARMOUR UNITS OF COASTAL STRUCTURES IN RIZE-ARTVIN AIRPORT UNDER OVERTOPPING AND STABILITY CONDITIONS

submitted by **Arif UĞURLU** in partial fulfillment of the requirements for the degree of **Master of Science in Civil Engineering Department, Middle East Technical University** by,

Prof. Dr. Gülbin Dural Ünver ____________________
Dean, Graduate School of **Natural and Applied Sciences**

Prof. Dr. İsmail Özgür Yaman ____________________
Head of Department, **Civil Engineering**

Prof. Dr. Ahmet Cevdet YALÇINER ____________________
Supervisor, **Civil Engineering Department, METU**

Examining Committee Members:

Assist. Prof. Dr. Gülizar ÖZYURT TARAKCIOĞLU ____________________
Civil Engineering Department, METU

Prof. Dr. Ahmet Cevdet YALÇINER ____________________
Civil Engineering Department, METU

Prof. Dr. Yalçın YÜKSEL ____________________
Civil Engineering Department, Yıldız Technical University

Prof. Dr. Esin ÖZKAN ÇEVİK ____________________
Civil Engineering Department, Yıldız Technical University

Assist. Prof. Dr. Cüneyt BAYKAL ____________________
Civil Engineering Department, METU

Assist. Prof. Dr. Gülizar ÖZYURT TARAKCIOĞLU ____________________
Civil Engineering Department, METU

Date: 08.08.2017

I hereby declare that all information in this document has been obtained and presented in accordance with academic rules and ethical conduct. I also declare that, as required by these rules and conduct, I have fully cited and referenced all material and results that are not original to this work.

Name, Last Name: Arif UĞURLU
Signature:

ABSTRACT

COMPARISON OF DIFFERENT ARMOUR UNITS OF COASTAL STRUCTURES IN RIZE-ARTVIN AIRPORT UNDER OVERTOPPING AND STABILITY CONDITIONS

Uğurlu, Arif

M.S., Department of Civil Engineering

Supervisor: Prof. Ahmet Cevdet YALÇINER

August 2017, 89 pages

In this study, the armour units which are considered to be used in the construction of Rize-Artvin Airport are compared due to their stability and overtopping conditions. The study followed the necessary procedure for the design of coastal structures starting from determination design wave characteristics. The wave transformations to the structure are performed using P025 numerical model which was developed by Port Airport Research Institute, Japan. During design computations global warming, wind set-up, wave set-up, seasonal tides, barometric and coriolis effect and tidal effect are also taken into account to cover all possible critical conditions during the service life of the structures. Using the design wave characteristics and 15 different structure alternatives, a series of physical model experiments are performed to obtain the most appropriate option for the cross-section of coastal defense structures of Rize-Artvin Airport. Finally all results are compared and the most reliable sections are introduced with discussions.

Keywords: Physical Modeling, Reclamated Area, Airport, Blacksea, Deep Structure, Accropode^{TM} II , Tetrapod

ÖZ

RİZE-ARTVİN HAVALİMANI KIYI YAPILARI KORUMA TABAKALARININ AŞMA VE STABİLİTE ŞARTLARI ALTINDA KIYASLANMASI

Uğurlu, Arif

Yüksek Lisans, İnşaat Mühendisliği Bölümü

Tez Yöneticisi: Prof. Dr. Ahmet Cevdet YALÇINER

Ağustos 2017, 89 sayfa

Bu çalışmada, Rize-Artvin Havalimanı inşaatında kullanılması olası koruyucu tabaka elemanları stabilite ve aşma yönünden karşılaştırılmıştır. Çalışmaya dizayn dalga karakteristiğinin elde edilmesiye başlanmıştır. Derin deniz dalgası, Japon Liman ve Havaalanı Araştırma Enstitüsü tarafından geliştirilen P025 sayısal modeli ile yapı önündeki derinliğe transforme edilmiştir. Transformasyon çalışması esnasında yapının servis ömrü boyunca karşılaşacağı doğa olaylarını doğru tanımlayabilmek için küresel ısınmadan kaynaklı su seviyesi değişimi, rüzgar kabarması, dalga kabarması, mevsimsel değişim, basınç, coriolis ve gel-git etkisi hesaba katılmıştır. Elde edilen dizayn dalgası ile Rize-Artvin Havaalanı koruma yapısında kullanılacak en uygun kesiti belirlemek için 15 alternatif içeren bir dizi fiziksel model deneyi yapılmıştır. Son olarak tüm sonuçlar karşılaştırılarak en iyi performans gösteren kesitler sunulmuştur.

Anahtar Kelimeler: Fiziksel Model, Dolgu Sahası, Havalimanı, Karadeniz, Derin Yapı, Accropode^{TM} II ,Tetrapod

TO MY DAD

ACKNOWLEDGEMENTS

First and foremost, i would like to thank to my supervisor Prof. Dr. Ahmet Cevdet YALÇINER for all his efforts he made during my master period. He gave me his valuable time whenever necessary.

My colleagues Şükrü Emrah ARIKAN, Cüneyt BİLEN, AND Olcay EĞRİBOYUN gave their best during the experiments. I am so grateful to them.

My lecturers Prof. Dr. Ayşen ERGİN, Dr. Işıkhan GÜLER, Assist. Prof. Dr. Cüneyt BAYKAL, Assist. Prof. Dr. Gülizar ÖZYURT TARAKCIOĞLU, Okan TAKTAK, Gökhan GÜLER always helped me to improve myself and become a coastal engineer. I am so grateful to all of them.

My wife Menekşe UĞURLU was always with me during this period and always supported me. I couldnt succeed this without her existence. All of this would be meaningless without her. My beloved daughter Ceylin. I have a special thank for her. She never complained during the time i studied instead of playing with her. I am so lucky to be her dad. Finally my mum made her all effort to raise me as a good person. I couldn’t be the person i am now without her.

TABLE OF CONTENTS

LIST OF TABLES

TABLES

LIST OF FIGURES

CHAPTER 1

INTRODUCTION

Breakwaters are the structures which shelter the area behind them to obtain calm area for the marine vessels. The design of breakwaters or coastal defense structures need careful investigations through physical modelling. There are numerous physical model studies performed for the design of important coastal defense structures. Each coastal defense structure needs special physical modelling investigations. Some of coastal defense structures are built to protect the reclamated area on which important infrastructure is constructed. Ordu-Giresun and Rize-Artvin airports are built on the reclamated area near the Black sea coast. This thesis aimed to present the physical model tests and their results of Rize Artvin Airport.

The purpose of this thesis is to understand the behaviour of a breakwater which will be constructed at a depth of 26 m on average. The marine structures in this depth are rare in the world. The stability of breakwater is also crucial to shelter the area of airport. The infrastructure of the reclaimed are contain runway, taxiway, apron and airport facilities (buildings with construction area of 50.000 m^2). The safety of all these components from the marine extreme conditions requires proper design of the coastal protection structure. Thus, 15 different cross section alternatives are selected and their stability are tested in laboratory under the design wave conditions. The damage levels of each alternative are compared and suitable cross sections are pointed out according to the experimental results. Furthermore, the level of overtopping on different alternatives are also measured in each test and the results are compared and discussed.

In the literature numerous studies were conducted about the physical model experiments for the stability of coastal defense structures and also overtopping on those structures. In Chapter 2 the selected publications are presented and discussed.

For the proper design of a stable breakwater, the design wave characteristics not only in deep water but also in front of the coastal defense structure. In Chapter 3 the location, coastal characteristics, wave climate studies from deep water to the shallow area through wave transformation are described and their results are presented and discussed.

The determination of the structural dimensions and armor units by the help of physical modeling reduce the construction cost and enables increase the stability of the designed structure. The experimental setup, selected structure alternatives, experimental procedure, damage levels of different structure alternatives in the experiments are presented and discussed in Chapter 4.

All experiments were performed in the hydraulic laboratory of The General Directorate of Infrastructure Investments. Information about the laboratory was given in Chapter 5.

Main conclusions of this study and recommendations for future studies are provided in Chapter 7.

CHAPTER 2

LITRATURE REVIEW

The stability of coastal structures under wave attack is the most important issue for assessment of their performance after the extreme marine events. Physical modeling is the most reliable way to investigate the stability of coastal structures and complete understanding of damage levels. In literature there are numerous studies related to the physical modeling of coastal structures. This study is focused on the stability of breakwaters. Therefore, only the literature related to this study is summarized in the following.

General Directorate of Infrastructure Investments (AYGM) published a new guidelines in 2016 (AYGM, 2016). It is also a road map for the structures to be built in Turkey. It gives solutions to probable questions under the titles of ; performance based design, wind waves, long waves and water level changes, currents, sedimentation and morphology, planning (terminals, fishery shelters, marinas, cruise ports), design (breakwaters, floating breakwaters, pipelines).

One of the most common formulae for the stability of breakwaters is the Hudson Formula. It was developed in 1959. The formula has the influence of wave height, structure slope, density of rock and water. It also contains K_D value which is stability coefficient in the formula.

Another formula for the stability of breakwaters was presented by Van der Meer (1987). The formula is based upon a series of model tests. This formula is given with the influence of wave period, number of waves, armor grading and permeability of core. Also a damage level parameter is introduced in the formula.

One of the recent examples is model experiments of HaydarpaĢabreakwater. Güler et al., (2015) showed that the stability of HaydarpaĢaPort especially crown wall was not stable under tsunami attack. A stable section was also recommended in the scope of that study. Scale effect should be considered to determine a realistic value of prototype. The example was conducted both in 2D and 3D for the yacht harbor of Rome at Ostia (Italy). The results showed a big difference between model and prototype according to Franco et al. (2008).

Crown walls are also important components for breakwaters. One study for wave loads on crown walls both in deep and shallow water wave conditions was performed by Norgaard et al. (2013). As a result of this study the formula by Pedersen was modified to a more accurate form.

To reduce overtopping discharge with the help of tetrapod and crown wall height a study was conducted by Park et al. (2014). A physical model based on Busan Yacht Harbor showed that increasing crown wall height and tetrapod size reduce overtopping discharge dramatically. To determine the effect of test duration on overtopping discharge which gives a result of 500 waves can be used to obtain an accurate overtopping discharge value with respect to recommended 1000 waves. (Romano et al.,2015)

Overtopping behaviors of different armor units were reported by Bruce et al. (2008) by using rock (two layers), cubes (single layer and two layers), Tetrapod, Antifer, Haro, Accropode, Core-Loc™ and Xbloc™. Consequently roughness factors (γ_f) and reflection coefficient of these armor units were obtained following 179 tests. The individual overtopping values were analyzed and compared with prediction formulae.

Vidal et al. (2006) studied stability formula by inserting wave height parameter and recommended to use H_{50} (average wave height of the 50 highest waves) in the stability relations. According to the authors, H_{50} describes the wave characteristics more realistically in the assessment of the stability of coastal structures.

From the EC-research projects OPTICREST and CLASH it is known that overtopping discharges determined from conventional Froude scale models of rubble mound breakwaters are smaller than measured in corresponding prototypes.

Andersen et al. (2011) examines this scale effect by comparing overtopping discharges in small scale and large scale tests and they identifies wave characteristics are very important for small overtopping discharges and suggests a new estimate of the scale effects is found by using H1/100 to make the freeboard and overtopping discharge dimensionless. .

Geeraerts et al. (2008) reported the comparison of overtopping between model and prototype for a steep rubble mound breakwater in Zeebrugge, Belgium. They focused on the wave speed to suggests to scale the wind speed o [0.8 to 0.5] for the 1:30 scale model for their cases.

Van Gent (2013) studied the stability of berm breakwaters by focusing on the influence of the slope angle (1:2 and 1:4), the width of the berm, the level of the berm, and the wave steepness. Based on the test results prediction formulae have been derived to quantify the required rock size for rubble mound breakwaters with a berm.

Rao et al. (2003) studied stability of berm breakwater with reduced armor stone weight. In the study it was indicated that wave period is very important for the stability of breakwater. As the period increases the design wave height for zero damage condition decreases.

CHAPTER 3

STUDY AREA

A new airport will be constructed at North East of Turkey between Rize and Artvin provinces. The general information and characteristics of Rize Artvin Airport are given in the following sections.

3.1. Rize-Artvin Airport

Rize-Artvin Airport will be constructed on the reclamated area near the coast in the black sea. It will serve as both domestic and international transportation. One of the main challenges of the Airport is to be the deepest reclaimed airport built in the World. The coastal defense structures of the airport will be constructed at an average depth of 26.00 m.

For the construction of the project, 88.500.000 tons of rocks will be used for reclaiming which is 2.5 times higher than used in Ordu-Giresun Airport. All airport facilities (runway of 3000 m X 45 m, apron of 300 m X 120 m, taxiway of 260 m X 24 m , superstructures including terminal building which will have 2 millions of passengers per year, flight tower, garages, parking lot, power centre, police station and installation channel and a ring road of 10.000 m) will be constructed on the reclaimed area which will cover more than 200 000 m^2 in which the closed areas will cover 50.000 m^2.

Airport facilities will be sheltered by the specially design coastal defense structures which will be denoted in this study as breakwaters. Hence the stability of breakwater and also physical modelling tests are crucial service to make proper design and construction of airport facilities and also safe operation of the Airport.

In this study a series of physical model experiments are carried out considering different structure alternatives of the breakwaters (coastal defence structures). In physical modelling; 15 different cross sections including rock, tetrapod and Accropode II units are tested.

3.2. Similar Structures Built in The World

In the World, Kansai, Ordu-Giresun and Hong Kong Airports are the examples of reclaimed structures. General characteristics of those airports are briefly described in the following

Ordu - Giresun Airport

Ordu Giresun Airport is another example on the Black Sea coast of Turkey. It is located at the coordinates 40.96° N and 38.08° E in between Ordu and Giresun provinces. (Figure 3.1)

Figure 3.1. The Location and General Layout of Ordu Giresun Airport

Water depth in front of the coastal defense structure is 10.67 m. Design wave height (H_s) for Ordu-Giresun Airport is selected as 7.40 m with a period of 11.93 sec. After seies of model test performed in the hydraulic laboratory of Ministry of Transport, Maritime Affairs and Communications, General Directorate of Infrastructure Investments. The armor unit is selected as rock with 10 – 12 tons of rock. Runway length of Ordu-Giresun Airport is 3000 m.

Kansai International Airport

Kansai Airport is an international aerodome located on an artificial island in the center of Osaka Bay. It is located at the coordinates 34.26° N and 135.13° E. (Figure 3.2) Kansai has passenger capacity of 23 millions per year.

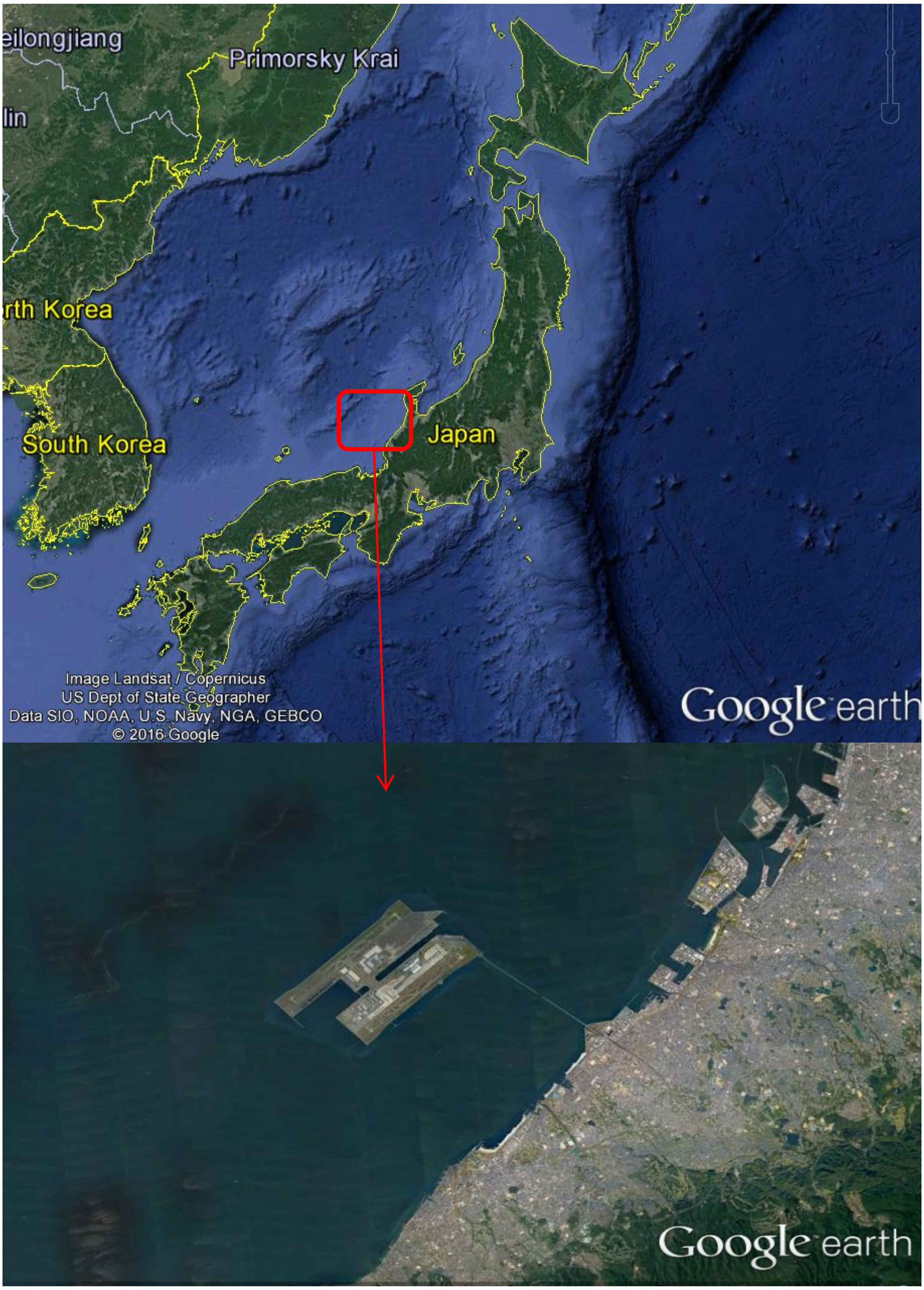


Figure 3.2. The Location and General Layout of Kansai Airport

Water depth in front of structure is 18.30 m. Design wave height (H_s) and wave period is 3.30 m and 7.10 sec. Water depth in front of the coastal defense structure is 10.67 m. Design wave height (H_s) for Ordu-Giresun Airport is selected as 7.40 m with a period of 11.93 sec. The armor unit is selected as tetrapod . There exists two runways and two parallel taxiways in Kansai Airport with a length of 4.000 m.

Hong Kong International Airport

Hong Kong International Airport is the main airport in Hong Kong. It is located on the island of Chek Lap Kok, which largely comprises land reclaimed for the construction of the airport itself. Hong Kong International Airport is located at the coordinates 22.19° N and 113.54° E. (Figure 3.3) The airport is also known as Chek Lap Kok Airport. Hong Kong International Airport contains two runways which are 3.800 m each.

Figure 3.3. The Location of Hong Kong International Airport

Water depth in front of structure is 15 m. Armour unit for this project is rock. Design wave height (H_s) is 3.9 m and wave period (T_m) is 5.5 sec.

Rize - Artvin Airport

Rize-Artvin Airport will be located at the coordinates 41.17° N and 40.85° E. This airport will be the second reclaimed structure in Turkey following Ordu-Giresun Airport. The location and general layout of Rize Artvin Airport is shown in Figure 3.4.

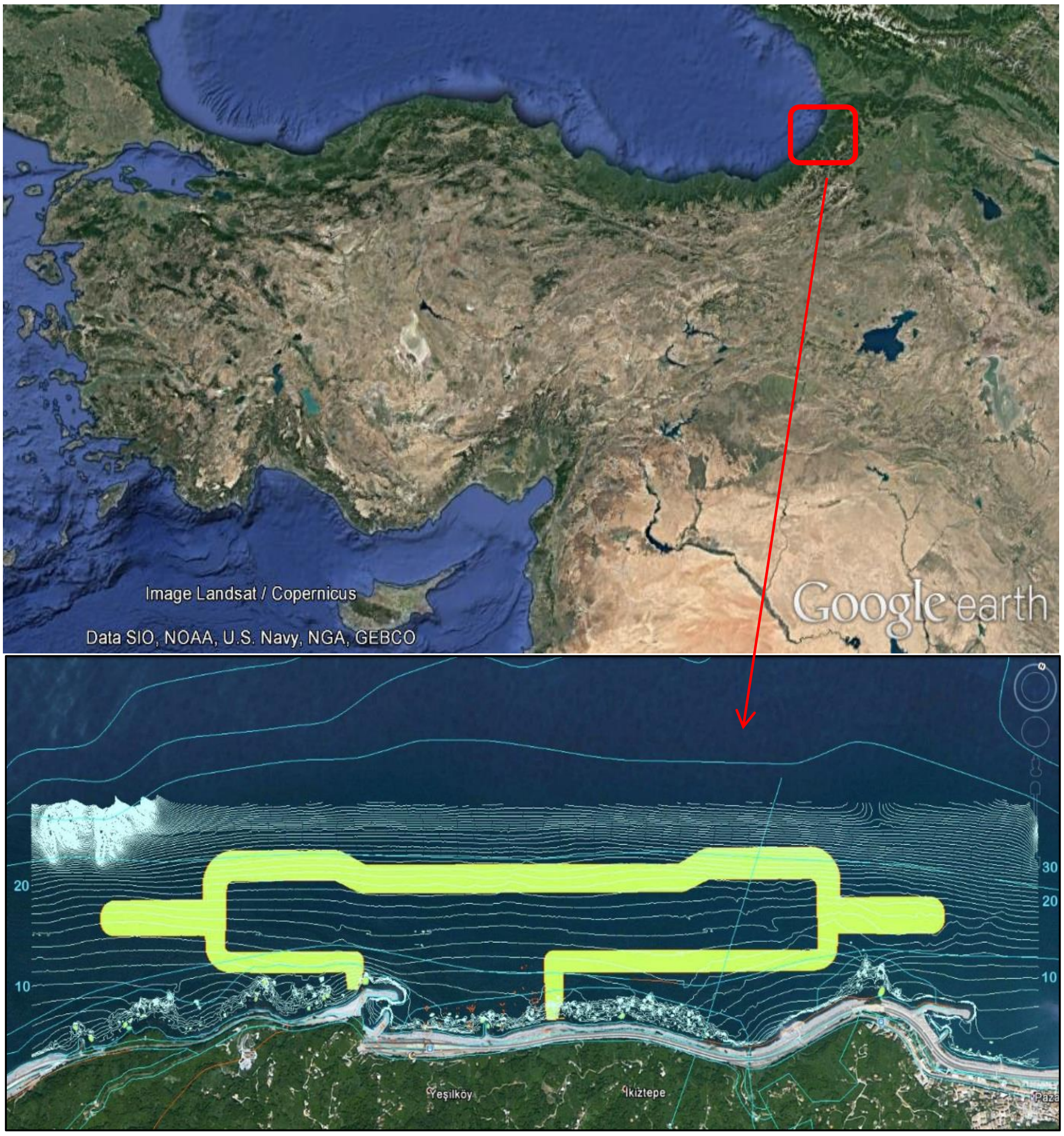


Figure 3.4. The Location and General Layout of Rize Artvin Airport

Water depth in front of structure is 27.60 m. Armour unit for this Project will be either tetrapod or Accropode II. Design wave height (H_s) is 7.70 m and wave period (T_m) is 11 sec.

The general characteristics of afroementioned airports constructed on reclaimed areas are given in Table 3.1. It is seen from Table 3.1 Comparison of Reclaimed Airports.

Table 3.1. Comparison of Reclaimed Airports

Airport	**Ordu-Giresun Airport**	**Kansai Airport**	**Hong-Kong International Airport**	**Rize-Artvin Airport**
Water Depth In Front Of Structure (m)	**10.67**	**18.30**	**15**	**27.60**
Design Wave Height (H_s)	**7.5**	**3.30**	**3.9**	**7.70**
Wave Period (Tm) (sec)	**11.93**	**7.10**	**5.5**	**11**

CHAPTER 4

DESIGN WAVE CHARACTERISTICS

In order to determine the design wave characteristics, the change in water level during the service life of structure, long term wind and wave hindcasting, wave setup, global warming, wave set-up, seasonal tides, barometric and coriolis effect and tidal effect have to be considered. Those are describe in the following.

4.1 Water Level Change

The water level change during the service of the structure are global warming, wind set-up, wave set-up, seasonal tides, barometric and coriolis effect and tidal effect.

4.1.1 Global Warming

In the “Coastal Structures Planning and Design Manual (AYGM, 2016)” it is recommended to take account the water level change results from global warming. Due to the last report of Intergovernmental Panel on Climate Change (IPCC) which was published in 2013 the predictions about global sea level increase that can occur until 2100 has been given in Figure 4.1. These predictions of IPCC is shown in Figure 2.4. IPCC considered various scenarios (Representative Concentration Pathway (RCP) Scenarios) including some uncertainty like global warming and changes in glacier mass. On Figure 2.3, RCP8.5 and RCP2.6 which are considered to be the most probable scenarios are shown in %5 and %95 confidence interval. In the light of this evaluation the sea level rise between 2007-2100 is predicted to be 0.53–0.98 m for RCP8.5 scenario and 0.28–0.61 m for RCP2.6 scenario.

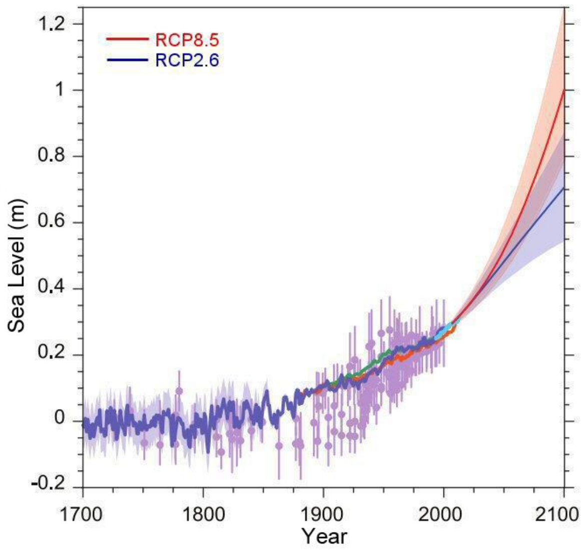


Figure 4.1 Prediction of sea level change until 2100(AYGM,2016)

4.1.2 Wind Set-up

In case of the wind blows from sea to the coast sea level rises in shore line. If θ is the angle between wind direction and the shore line normal, the sea level rise, η_0, in the shore line can be calculated with Equation 1.

$$\eta_0 = k\frac{F}{d}(U\cos\theta)^2$$ Equation 1

F : Fetch length (km)

U : Wind speed (m/s)

d : Average water depth (m)

k coefficient (OCDI, 2003). From research results obtained from Baltic Sea k value is determined as 4.8 x 10^{-2}.

From this equation for the location where Rize-Artvin Airport will be constructed, assuming that F=330 km, d=1300 m, wind speed for 100 years return period u=37 m/sec, η_0=16.7 cm wind set-up is estimated.

4.1.3 Wave Set-up

For the structures those will be constructed in breaking zone, wave set-up resulting from breaking should be considered .

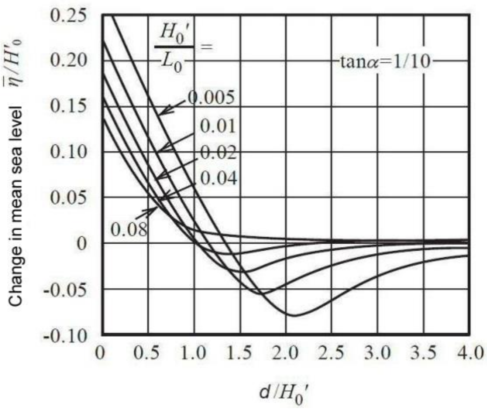


Figure4.2 Average rise at MWL in shoreline (η) (OCDI,2009, AYGM, 2016)

From Figure 4.2, considering Artvin-Rize Airport‟s 100 years return period design wave (H_s=7.70 m T_m=11.0 s) , average depth in front of structure d=-25 m, wave setup of η=7.2 cm is determined.

4.1.4 Seasonal Tides, Barometric and Coriolis Effect

In order to obtain seasonal tides, barometric and coriolis effect Figure 4.3 is used.

Table4.1 Average Water Level Changes; Güler, 2014

	Black Sea	**Marmara Sea**	**Aegean Sea**	**Mediterranean Sea**
Seasonal Tides	-9.5 – +9.5 cm	-9 – +9 cm	-8.5 – +8.5 cm	-4 – +4 cm
Barometric and Coriolis Effect	-2.5 – +7.0 cm	-2.4 – +12 cm	-2.4 – +6.7 cm	-1.9 – +6.2 cm
Total	-12 – +16.5 cm	-11.4 – +21 cm	-10.9 – +15.2 cm	-5.9 – +10.2 cm

4.1.5 Tidal Effect

To determine the water level changes due to tides, the data acquired from Trabzon Mareograph Station belonging to General Command of Mapping was used. In this data water level change due to tidal effect is given as 18 cm.

The water level change due to different constitutents presented above are given in Table 4.1 for 100 years duration as assumed the service time of the structures. As seen from Table 4.1, the High Water Level (HWL) becomes 1.31 m above the present Still Water Level (SWL). Therefore the water level in the physical model experiments is used at this level for considering future extreme conditions properly.

Table 4.2. Water Level Rise in Artvin-Rize Airport Project

Cause of Water Level Change	Rise of WL (cm)
Global Warming	73.0
Wind Set-up	16.7
Wave Set-up	7.2
Seasonal Tides	9.5
Barometric and Coriolis Effect	7.0
Tidal	18.0
TOTAL	131.4

4.2 Design Wave Characteristics

Determination of the design wave requires analysis of long term wind and wave data and their statistical analysis. The deep water wave characteristics for the region is taken from the previous studies mainly Ozhan and Abdalla, (2002) which is known as Wave Atlas. The point on the coordinate of 41.25N, 40.70° E was selected from Ozhan to Abdalla, (2002) (see Figure 4.4)

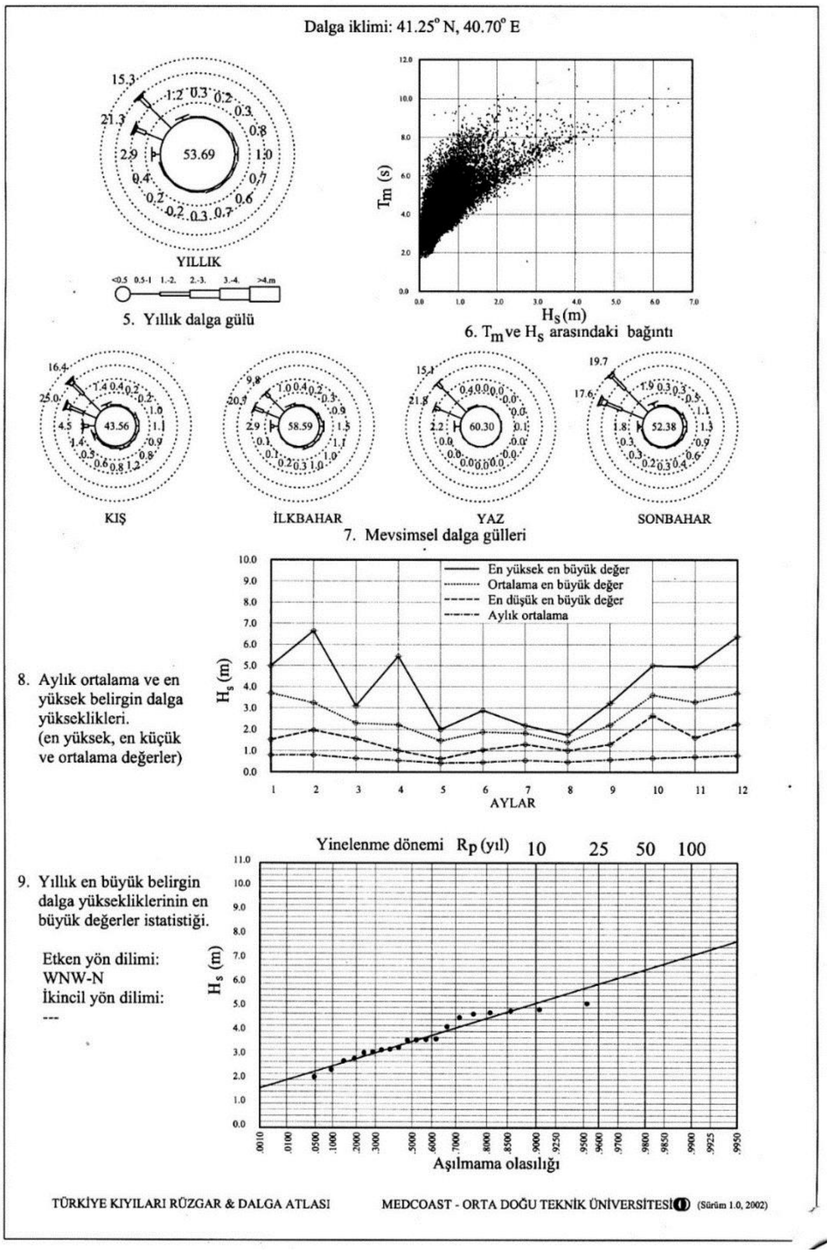


Figure 4.3 Wave Rose and Extreme Wave Analysis for the study region (Ozhan and Abdalla, 2002)

The deep water wave characteristics $H_s = 7.20$ m and $T_m = 11.0$ sec are obtained for 100 years return period from the wave atlas given in Ozhan and Abdalla, (2002). Dominant wave directions are also acquired from Ozhan and Abdalla, (2002) as WNW and N. Distribution of the wave height in 90 % confidence interval is 7.20 ± 1.04 m which was calculated by Goda 2010 method. The result is shown Figure 4.5.

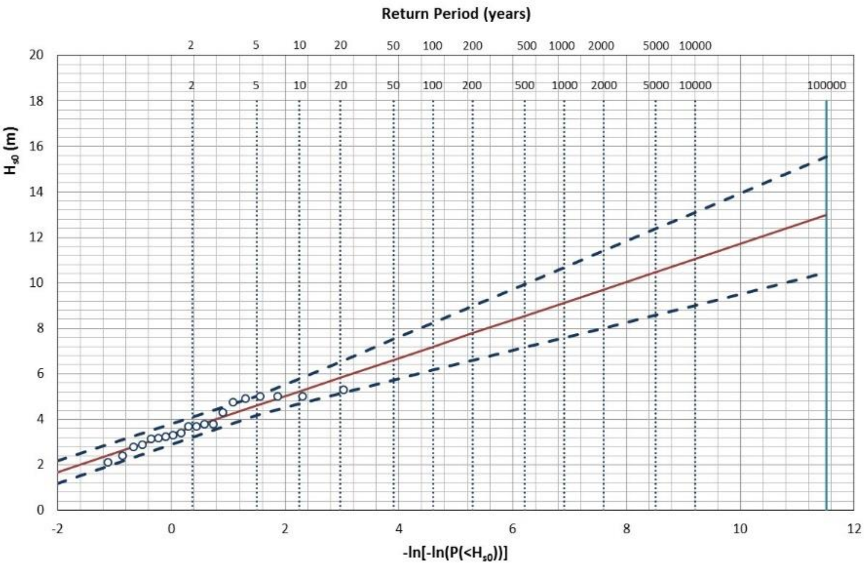


Figure 4.4 Distribution of the wave height in 90 % confidence interval

Another important issue is to determine the wave characteristics in front of the structure through wave transformation studies. Wave transformation from deep water of the toe of the structure where water depth is 26.7 m are performed by using the numerical code P025 developed by Port and Airport Research Institute (PARI) (reference). The area of interest for the wave transformation is selected in dimensions of 8000 m X 3500 m whose border extends to deep water. (see Figure 4.6).

Bathymetry data was provided by private sector for the section until 40 m depth. In the deeper parts the bathymetry map of The Office of Navigation, Hydrography and Oceanography is used (see Figure 4.6).

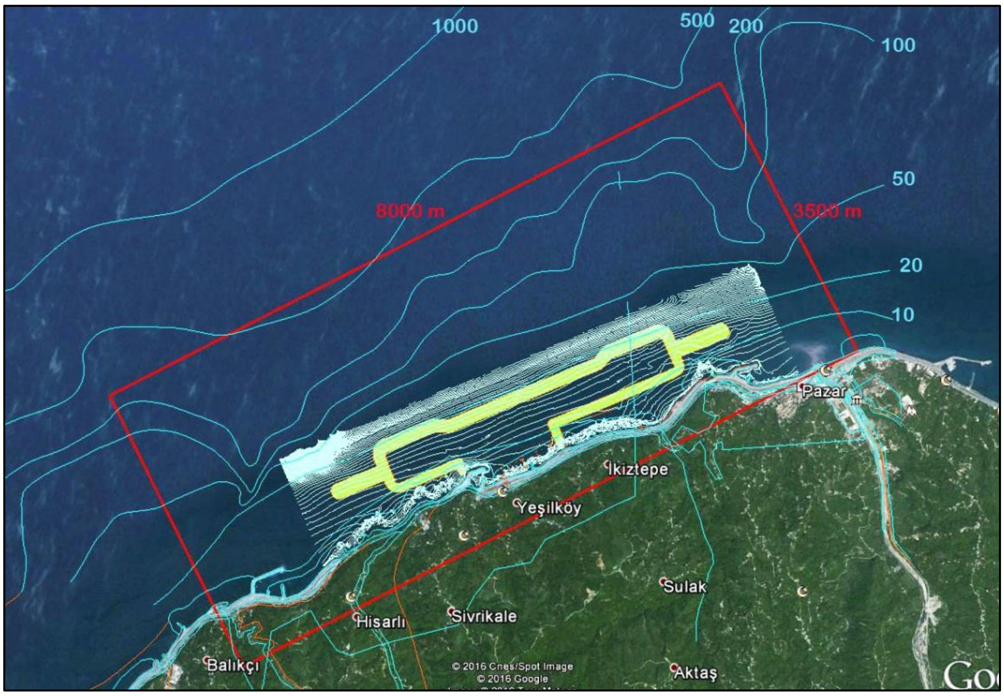


Figure 4.5 Boundary Condition for Wave Transformation

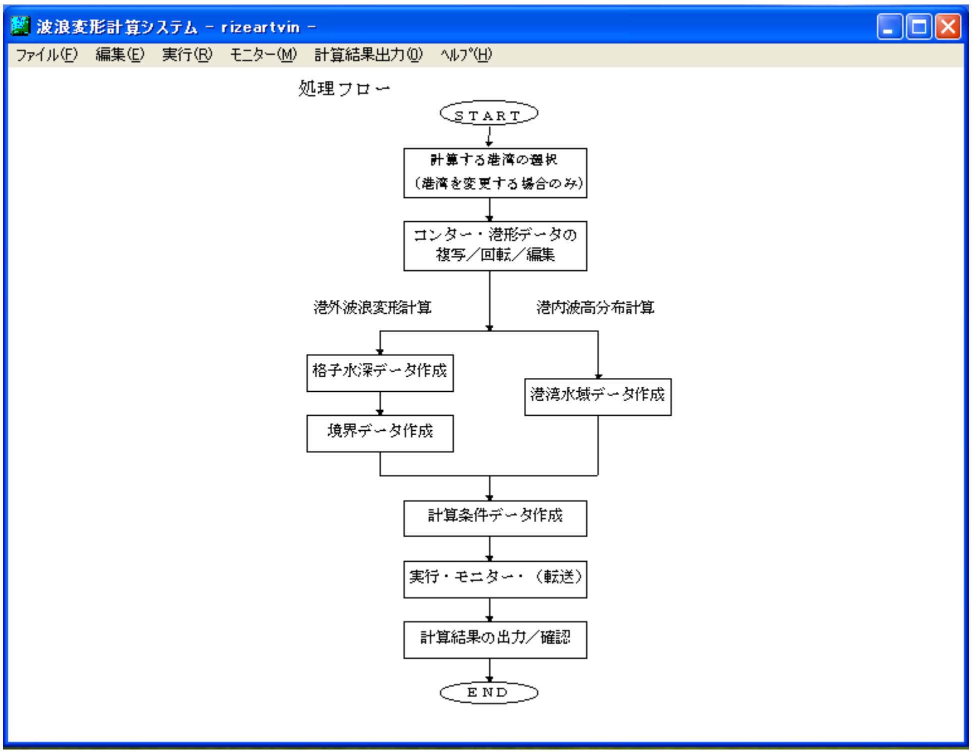


Figure 4.6 Interface of P025

As a result of the wave transformation simulation, the calculated wave parameters in front of the several parts of the breakwater are shown in Figure 4.8 and Table 4.2.

The wave conditions in front of different parts of the structure with %90 Confidence interval (CI) are given in Table 4.3.

As shown in Figure the main parts of the breakwater are trunk, east and west roundheads and east and west approach breakwaters. As shown in Table 4.2 the highest wave conditions at different parts are

i) 6.88 m at West – Roundhead, 6.74 m at trunk, 6.87 m at East – Roundhead, 6.60 m at West – AB, 6.83 m at East – AB for low water level condition.

ii) 6.87 m at West – Roundhead, 6.73 m at trunk, 6.86 m at East – Roundhead, 6.61 m at West – AB, 6.83 m at East – AB for high water level condition.

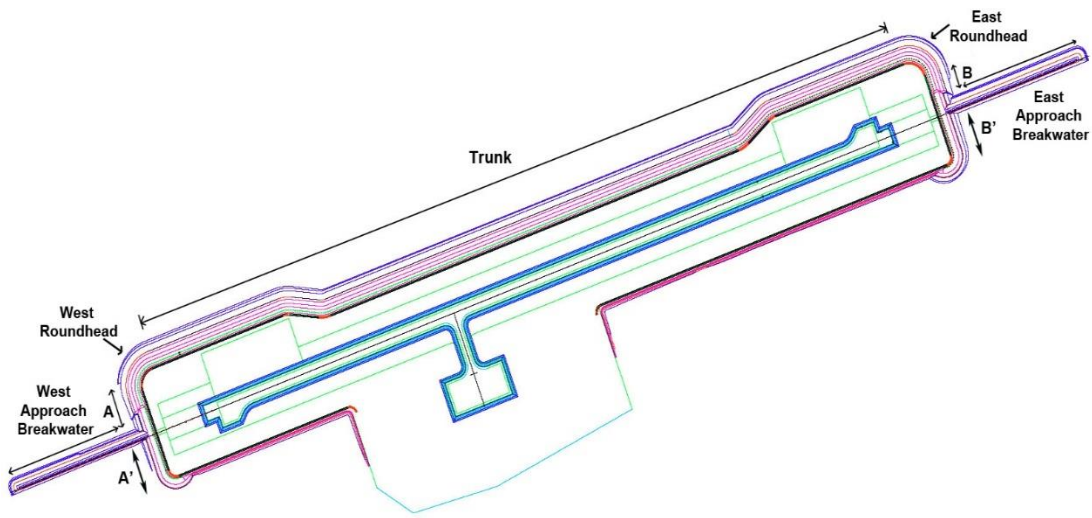


Figure 4.7 Parts of the Project

Table 4.3 Design Waves for Hs=7.20 m, Tm=11 s Deep Water Wave Conditions

			West - Roundhead	Trunk	East - Roundhead	A	B	West-AB	East-AB
	Direction	Water Depth In Front Of Structure (m)	24	23	25	19	18	15.5	16.7
LWL	WNW	Hs (m)	6.49	6.39	6.06	6.26	3.07	5.91	5.13
		Angle wrt N (^{o})	62	59	58	59	37	56	46
	N	Hs (m)	6.83	6.71	6.87	5.67	6.87	6.42	6.81
		Angle wrt N (^{o})	5	3	2	13	2	8	3
	NW	Hs (m)	6.77	6.67	6.56	6.67	4.82	6.45	6.36
		Angle wrt N (^{o})	44	42	42	44	30	42	35
	NNW	Hs (m)	6.88	6.74	6.82	6.56	6.27	6.60	6.83
		Angle wrt N (^{o})	24	23	22	26	17	24	21
HWL	WNW	Hs (m)	6.50	6.40	6.08	6.27	3.04	5.94	5.12
		Angle wrt N (^{o})	62	59	59	59	38	57	47
	N	Hs (m)	6.82	6.70	6.86	5.64	6.87	6.43	6.81
		Angle wrt N (^{o})	5	2	2	13	2	8	3
	NW	Hs (m)	6.76	6.67	6.57	6.67	4.81	6.47	6.36
		Angle wrt N (^{o})	44	42	42	44	31	42	36
	NNW	Hs (m)	6.87	6.73	6.81	6.55	6.26	6.61	6.83
		Angle wrt N (^{o})	24	23	22	26	17	24	21

Table 4.4 Design Waves for Hs=8.24 m, Tm=11 s Deep Water Wave Conditions with %90 CI

	Direction	Water Depth In Front Of Structure (m)	West - Roundhead	Trunk	East - Roundhead	A	B	West-AB	East-AB
			24	23	25	19	18	15.5	16.7
LWL	WNW	Hs (m)	7.42	7.30	6.94	7.09	3.51	6.66	5.84
LWL	WNW	Angle wrt N (°)	62	59	58	59	37	56	46
LWL	N	Hs (m)	7.80	7.66	7.85	6.43	7.77	7.19	7.64
LWL	N	Angle wrt N (°)	5	3	2	13	2	8	3
LWL	NW	Hs (m)	7.74	7.62	7.51	7.53	5.51	7.22	7.16
LWL	NW	Angle wrt N (°)	44	42	42	44	30	42	35
LWL	NNW	Hs (m)	7.86	7.70	7.80	7.41	7.11	7.38	7.65
LWL	NNW	Angle wrt N (°)	24	23	22	26	17	24	21
HW	WNW	Hs (m)	7.43	7.32	6.96	7.12	3.48	6.72	5.83
HW	WNW	Angle wrt N (°)	62	59	59	59	38	57	47
HW	N	Hs (m)	7.79	7.66	7.85	6.41	7.79	7.23	7.67
HW	N	Angle wrt N (°)	5	2	2	13	2	8	3
HW	NW	Hs (m)	7.73	7.63	7.52	7.56	5.49	7.28	7.18
HW	NW	Angle wrt N (°)	44	42	42	44	31	42	36
HW	NNW	Hs (m)	7.86	7.70	7.80	7.43	7.12	7.43	7.68
HW	NNW	Angle wrt N (°)	24	23	22	26	17	24	21

As shown in Table 4.3 the highest wave conditions at different parts of structure with %90 CI of design wave are i) 7.86 m at West – Roundhead, 7.70 m at trunk, 7.85 m at East – Roundhead, 7.38 m at West – AB, 7.65 m at East – AB for low water level condition.

ii) 7.86 m at West – Roundhead, 7.70 m at trunk, 7.85 m at East – Roundhead, 7.43 m at West – AB, 7.68 m at East – AB for high water level condition.

CHAPTER 5

PHYSICAL MODEL EXPERIMENTS

5.1 General Description of Physical Model Experiments

Physical model is a physical copy of the prototype phenomenon obtained by the similarity laws. The behaviour of structures under extreme conditions can easily be observed and measured in physical model experiments in terms of stability and overtopping. In some cases optimizations may be necessary because the conditions in nature can not be represented in the laboratory. The most important optimization for modeling is scale effect. It should be taken into account to obtain reliable results. Furthermore, efficient and low cost solutions are acquired with the help of physical modeling. Stability and overtopping problems can be predicted and solutions can be developed as a result of modeling.

In the following experimental facilities, wave basin, experimental set-up, measuring tecnique, model scale, rubble mound breakwater cross-sections are described.

5.2 Experimental Facilities

All of the physical model experiments are conducted in the wave basin of General Directorate of Infrastructure Investments (AYGM). Wave basin is a part of The Port Hydraulic Research Center.

The Port Hydraulic Research Center started operation in January 1995 for the duration of five years as part of technical cooperation program by the government of Japan towards the government of Turkey.

The General Directorate of Infrastructure Investments (AYGM), Turkish Ministry of Transport, Maritime Affairs and Communications took charge in the construction of the Center Building and the facilities such as the wave channel, wave basin and the electrical work.

The cooperation from the Japanese side was carried out through the Project type technical cooperation by the Japan International Cooperation Agency (JICA). The cooperation includes provision of the directional random wave generator system, work station and other equipment, dispatch of experts from the Port and Airport Research Institute (PARI) of te Japanese Ministry of Transport and other institutes. Training of Turkish counterparts in Japan was also part of technical cooperation.

The purposes of the Port Hydraulic Research Center are as follows :

- To make research on the projects of coastal and harbour structures which are planned and executed by The General Directorate of Infrastructure Investments (AYGM), from the coastal and harbour engineering point of view.
- To choose the most durable, stable and economical structure design among the alternative designs not only by conducting physical experiments but also by using numerical simulation methods.
- To develop new technologies on the coastal and harbour engineering.
- Firstly, getting the problems faced at coastal regions of Turkey, about the coastal and harbour structures (construction, stability, sedimentation, environment etc.) by the help of local directorates of AYGM, and then to produce solutions for these problems.

Port Hydraulic Research Center contains the following five main units of facilities.

1- Wave basin
2- Wave channel
3- Observation room
4- Water tank and machinery room
5- Work station

5.2.1. Wave Basin

Wave basin has dimensions of 40 m X 30 m X 1.2 m. Specifications of basin are listed below.

5.2.1.1 Wave Generator

The specifications of wave basin can be seen in Table 5.1. It contains 14 units with 56 paddles. Total length of the paddles is 28 m. Maximum significant wave height which can be generated by the system is 25 cm. It can generate wave periods between 0.7 – 2.0 secs.

Table 5.1 Specifications of Wave Basin

Type	Piston type (serpent type) multidirectional
Number of Units	14
Number of Paddles	56
Width of a Paddle	0.5 m
Total Width	28 m
Maximum Wave Height	0.25 m
Wave Period Interval	0.7-2.0 sec
Maximum Water Depth	1.0 m
Wave Generation	1-Regular (H,T,θ) $-30<\theta<30$ 2-Irregular a- One Directional $(H_{1/3},T_{1/3},\theta)$ b- Multidirectional $(H_{1/3},T_{1/3},\theta,S_{max})$ $10< S_{max}<75$

5.2.1.2 Wave Absorber System

Reflection from the borders must be controlled. In order to prevent the reflection in the borders of the basis, side walls and cross wall are covered by rubble which has ´ (at sides) and μ sl ope at the back of the basin.

Figure 5.1 General View of Wave Basin

5.2.2. Generation of Waves With Directional Random Properties

An observer on a beach or a boat in a offshore sea can easily notice that sea waves are quite random in height and period and that individual waves with short crests propagate in various directions. Sea waves of random nature are called “directional random waves”. As shown in Figure 5.2, directional random waves are expressed as the waves superposed of infinite number of component waves which have different heights, periods and propagation directions from each other. In order to reproduce “directional random waves” in a laboratory basin, “Serpent-Type Wave Generator” has been developed. As shown in the Figure 5.1 it consists of 56 segmented small wave paddles which are controlled by special software from the control room.

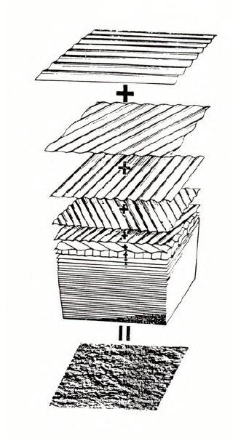


Figure 5.2 The sea surface obtained from the sum of many sinusoidal waves

5.3 Experimental Setup

The experiments are performed in the basin of General Directorate of Infrastructure Investments (AYGM). Inside the basin a flume with a width of 2m is constructed (Figure 5.3). 6 sensors are located in critical locations in the flume to measure the wave characterstics in front of the structure. Irregular wave train satisfying design wave characteristics are used in the experiments. The construction stages of the model is shown in Figure 5.4

Figure 5.3 Preparing the bottom topography for Rize – Artvin Airport physical modeling

(a) (b) (c) (d)

Figure 5.4 Creating Breakwater Section, (a) Core Layer, (b) 0.4-2 Ton Filter Layer, (c) 2-4 Ton Filter Layer, (d) 12-15 Ton Rock Armour Unit

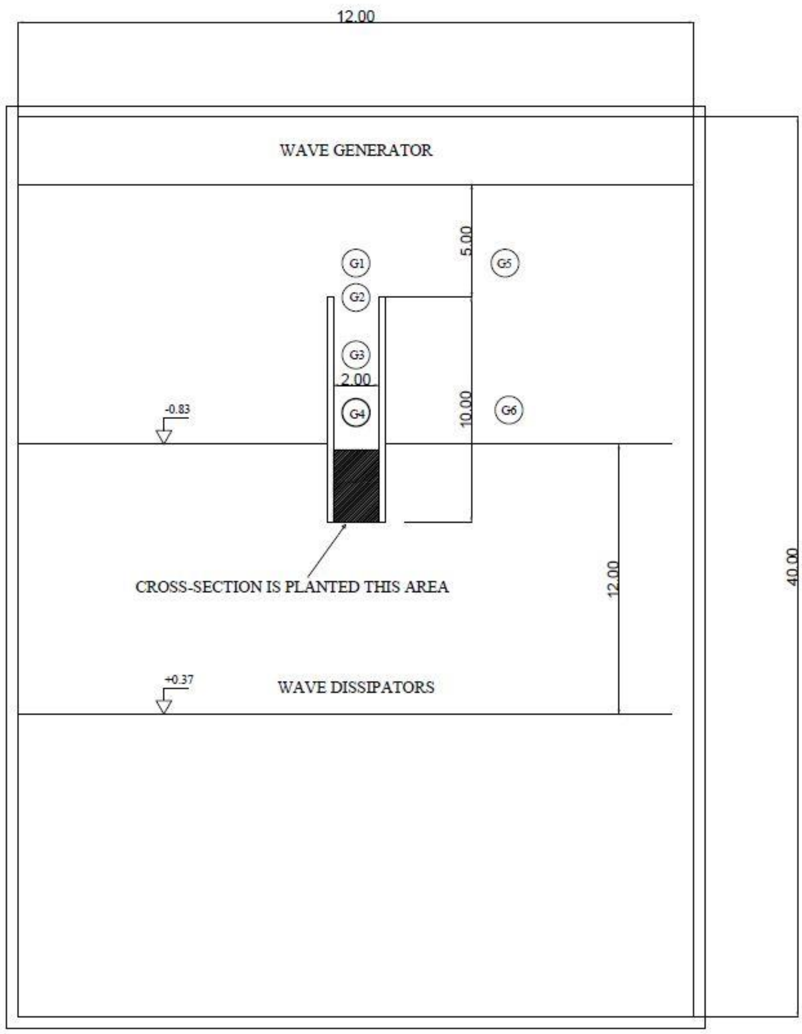


Figure 5.5 General Layout of Wave Basin in Physical Modeling (units are in m)

5.4.1 Measuring Technique

The measurement program is consisted of calibration and recording. Wave calibration process were conducted for both LWL and HWL using the system in operation room and sensors. In both water levels wave heights in front of the structure were acquired accurately. Wave gauges were installed in different locations starting from the wave paddle to the model. Two types of gauges were used during the tests. One of them was single sensor and the other one was array sensor which contains 4 single sensors to measure the wave direction, too. In Figure 5.6 array sensor which was placed to measure the wave height on wave propagation direction is seen.

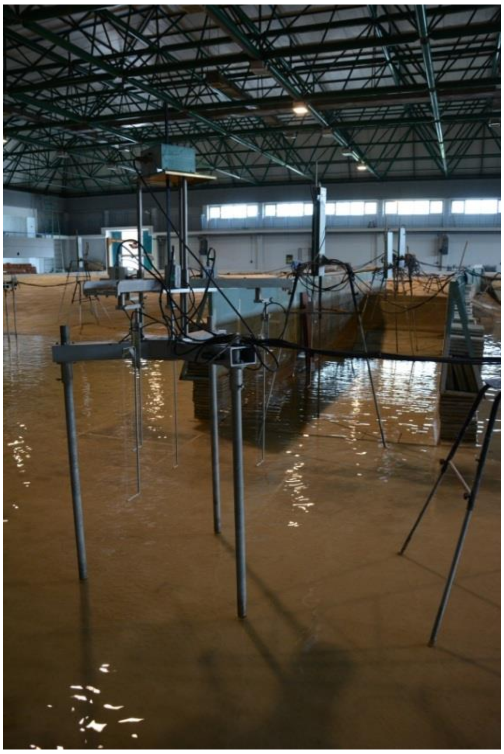


Figure 5.6 Positions of sensors

5.4.2 Model Scale

To obtain results of highest accuracy it is necessary to use the largest model. Large models have some disadvantages like high costs of construction and operation and longer test time. There is not an exact criteria for model scale but tests on breakwater stability are often performed with scales between an interval of 1/10 to 1/50.

Basic parameter in modelling is to determine the scale as great as possible to make the model realistic. However the dimensions of basin, limits of wave generators in producing wave, water depth, the dimensions of units which were used in armour units affect the scale factor. In the laboratory a flume with 2 m width was created within the basin of 40x30x1.2 m. All sections were inspected in this area. The period limit for wave generator is between 0.70 and 2.50 seconds. The maximum significant wave height can be produced by the generator is 25 cm.

In Rize-Artvin Airport model study, three types of armor units are tested. They are rock, tetrapod and Accropode II. The model scale was selected as $\lambda L = 1/43.06$ for rock and tetrapods, $\lambda L = 1/52$ for Accropode II considering model wave characteristics generated by irregular wave generator and water depth. The model was not distorted so the scale is the same in the horizontal and vertical directions. The scale of other variables involved in these series of model tests is given in Table (5.2)

In order to avoid boundary effects, the 30m distance from both sides of the flume (see different colord rocks in Figure 5.4.d) are not counted in the analysis of the experimental results and evaluation of damage ratio.

Table 5.2 Model unit categories used in physical modeling

	Prototype (ton)	Model (gr) Scale: 1/43.6	Model (gr) Scale: 1/52
Rock Categories	0 – 0.25	0 – 3.0	0 – 1.8
	0.4 – 2	4.8 – 24.1	2.8 – 14.2
	2 – 4	24.1 – 48.3	14.2 – 28.4
	8 – 10	96.5 – 120.6	56.9 – 71.1
	10 – 12	120.6 – 144.8	71.1 – 85.3
	12 – 15	144.8 – 180.9	85.3 – 106.7
Tetrapod	19	230.0	---
	28.4	343.0	---
	38	458.0	---
	32.5	---	230.0
Accropode ™ II	28.8 (12 m^3)	---	205.0

Stability inspection was conducted by applying the Return Period R_p=2, 10, 50 and 100 years return period waves cumulatively on the sections. Wave overtopping was evaluated for R_p=100 years return peripod wave both in still water level (SWL) and high water level (HWL) conditions. In the experiments every wave set was sent to the section under the condition of 1000 waves however the design wave with a return period of 100 years was given to the section under 3000 waves condition. This represents a 10.5 hours of storm. The wave height in front of the structure are given in Figure (Table 5.3).

Table 5.3 Wave height in front of the structure

	Prototype		Model Scale 1/43.6		Model Scale 1/52	
R_p (years)	H_s (m)	T_s (sn)	H_s (cm)	T_s (sn)	H_s (cm)	T_s (sn)
2	3.70	9.09	8.49	1.20	7.12	1.10
10	5.44	10.58	12.48	1.39	10.46	1.28
50	7.01	12.31	16.08	1.62	13.48	1.48
100	7.70	12.65	17.66	1.67	14.81	1.53

The average depth in front of the toe of Rize-Artvin Airport is about 25 m and average seabed slope is 1/10 which was obtained from bathymetric map. In the preliminary sections the crest elevation was determined as + 9.10 m for rock sectioned armour unit and + 8.40 m for tetrapod sectioned armour unit. Regarding to obstacle related to flight safety reasons, the crest elevation was kept as constant.

5.4.3 Rubble Mound Breakwater Cross-Sections

The two preliminary sections given below were determined as initial sections. These sections are provided by Port Research and Project Department of AYGM. The other cross-sections were generated according to the results and experience gained from ongoing experiments.

The section is composed by rock has a toe berm in the depth of -15.0 m with 10-12 tons of rock units as 3 layers, with a slope of 1/2, having a 15 m width berm at +5.35 m level and having a crest of 12.84 m width at +9.10 m level can be seen in Figure 5.7.

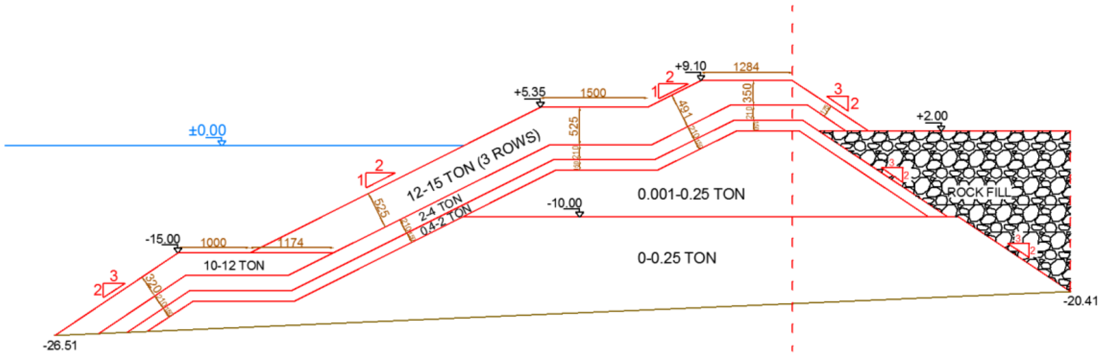


Figure 5.7 Preliminary rock cross – section

The section composed by 38 tons of tetrapod has a toe berm on -10.0 m elevation with a width of 18.0 m. On the front slope (2/3) 38 tons of tetrapods were used with a porosity %50. It has a berm on +5.0 m level with a width of 10 m and a crest on +8.40 m level with a width of 7.22 m. Tetrapods were used both in berm and crest in this section which can be seen in Figure 5.8.

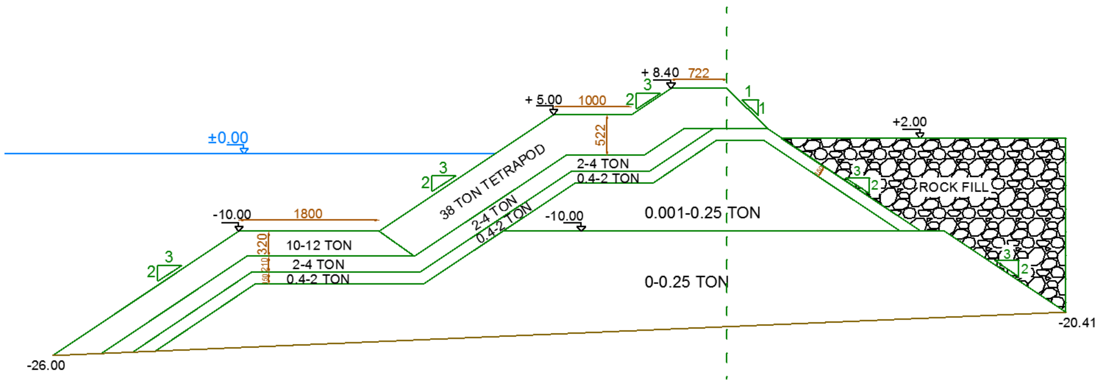


Figure 5.8 Preliminary tetrapod cross - section

5.5 Summary of Physical Model Experiments

5.5.1. Interpretation of Experimental Resutls

Determination of Damage Ratio

Remaining stable for its lifetime and creating a safe area is the first priority in the design of a breakwater. Section stability is inspected by applying the wave sets starting from the lowest one and evaluating damage ratio at the end of every set. Section of breakwater was not modified until the completion of all wave sets. Hence cumulative damage ratio can be determined as in nature.

Damage ratio is calculated as given below :

Damage (%)=(Replaced model units/Total number of model units)*100

Damage ratio of a section should be less than % 5 to regard the cross section as it stable. (Coastal Structures Planning and Design Manual (AYGM,2016))

Determination of Wave Overtopping

Another point that should be considered in breakwater section design is wave overtopping. Massive wave overtopping can cause damage on the inner side of breakwater which can affect the service of harbour. Acceptable wave overtopping level is determined related to the use of area behind the breakwater. In Coastal Structures Planning and Design Manual discharge tolerances are is given reffering to Wave Overtopping Manual, 2016.

In a meeting with the participation of Research Department, Airports Research and Project Department and Ports Research and Project Department have decided the wave overtopping value for Rize-Artvin Airport Breakwater “Rubble mound breakwaters; Hm0 > 5 m; rear side designed for wave overtopping Q < 5 – 10 lt/s/m” has been decided to be aimed.

5.5.2. Inspected Alternatives

It is crucial to design Rize-Artvin Airport Breakwater sufficient in stability, wave overtopping and applicability to use the sources of country efficiently. Hence to obtain the best alternative a series of alternatives were inspected in physical modelling. Three dffieren types of armour materials are used in the model experiments.

ROCK

Rock is the most common armour unit material used in the construction of breakwaters. It is easy to find from nature and easy to place but it is vulnerable under high wave heights and periods. Hence artifical units like tetrapod or accropode were developed to obtain more robust structures. In Figure 5.9 a breakwater built by rock is seen.

Figure 5.9 Breakwater built by rock

TETRAPOD

Tetrapod is a double layer armour unit for breakwater construction which was invented by Sogreah Consultants in France. Tetrapod is an artificial armour unit and made of concrete. It gives opportunity to build steeper slopes with compare to rock armour unit. In Figure 5.10 it is seen tetrapod armour units.

Figure 5.10 Tetrapod armor unit

ACCCROPODE II

Accropode II is a single layer armour unit which is used in the construction of breakwaters instead of double layer armour units. Single-layer systems consist of un-reinforced concrete armour units specifically designed for the protection of exposed coastal structures. Accropode II is Invented in 1981 by Sogreah Consultants in France. In figure 5.11 the pattern of the Accropode II can be seen.

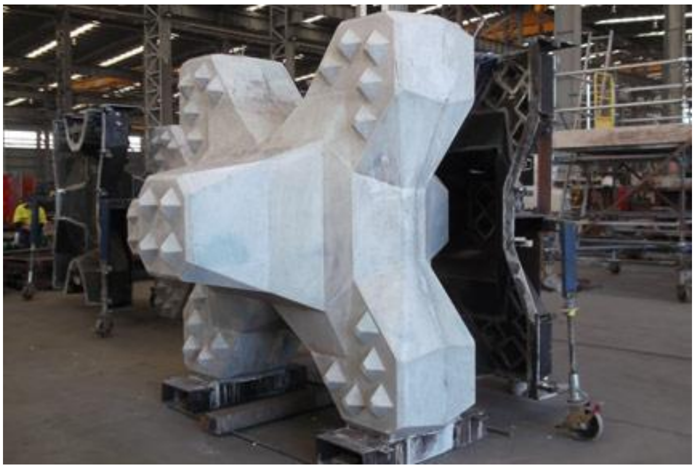


Figure 5.11 Accropode II armour unit

15 different structure alternatives are selected for the tests. They are :

1. **Rock Project**: This section was provided by Ports Research and Project Department. It has a toe berm in the depth of -15.0 m with 10-12 tons rock units, in armour unit 3 layers of 12-15 tons rock with a slope of 1/2, on +5.35

m level has a berm of 15 m width and has a crest of 12.84 m width on +9.10 m level.

2. **Rock-Alt-1** : On the 1/2 slope part of „Rock Project" section, between -6.90 m and +5.35 m level 28.4 tons of tetrapod was used, between -6.90 m and -15.00 m level 3 layers of 12-15 tons of rocks, on -15.0 m. level toe with 10-12 tons of rocks, the level above +5.35 m a crest with 10-12 tons of rocks were used.

3. **Rock-Alt-2** : On +9.10 m level of „Rock-Alt-1" section 8-10 tons of rock was used instead of 10-12 tons of rock.

4. **Rock-Alt-2-1**: A section expanding the crest of 12.84 m width to 17.34 m in „Rock-Alt-2" section on +9.10 m level and using 28.4 tons of tetrapods on 1/2 slope part.

5. **Rock-Alt-3** : 19 tons of tetrapods were used instead of 28.4 tons of tetrapods in „Rock-Alt-2" composite section .

6. **Rock-Alt-4**:Composing „Rock Project" section with a slope of 1/3 and using 8-10 tons of rocks in the crest.

7. **Rock-Alt-5**: Composing „Rock-Alt-2" composite section with a slope of 2/3 instead of 1/2.

8. **Rock-Alt-6**:Using 38 tons of tetrapods in the 2/3 slope of „Rock-Alt-5" section.

9.**Tetrapod Project**: This section was also provided by Ports Research and Project Department. It has a toe berm on -10.0 m elevation with a width of 18.0 m. On the front slope (2/3) 38 tons of tetrapods were used with a porosity %50. It has a berm on +5.0 m level with a width of 10 m and a crest on +8.40 m level with a width of 7.22 m. Tetrapods were used both at the berm and crest in this section.

10. **Tetrapod-Alt-1**: It was composed by moving the toe berm on -10 m elevation to -15 m in „Tetrapod Project" secion. Also by adding the berm on +5.0 m level to the crest the crest width becomes 17.22 m.

11. **Tetrapod-Alt-2**: 32.5 tons of tetrapods were used instead of 38 tons of tetrapods in „Tetrapod-Alt-1" section. In this section also the effect of overtopping was inspected by placing one layer 6-8 tons rocks on 0.4-2 tons filter layer with an elevation of +4.8 m and width of 10 m on the rear part of the section.

12. **Tetrapod-Alt-3**: In this section 8-10 tons of rocks were used on the 10.78 m width of crest in „Tetrapod-Alt-2" alternative.

13. **Accropode™ II Alt-1** : This section is created by using 12 m^3 one layer Accropode ™ II units with a porosity %54.58 and slope 2/3. Toe berm is on -15.0 m elevation and has 10-12 tons rock category. 3 layers of Accropode ™ II were used for the 7.80 m of crest on +8.40 m elevation and the 9.42 m is 10-12 tons of rocks.

14. **Accropode™ II Alt-2** : The width of 10-12 tons rock category was expanded to 17.20 m and total crest width became 25.00 m in „Accropode™ II Alt-1" section to reduce overtopping.

15. **AccropodeTM II Alt-3:** The width of 10-12 tons rock category was reduced to 12.80 m and total crest width became 20.60 m in „Accropode™ II Alt-2" section.

The descriptive Figures of each alternative are given in the next chapter in respective sections.

CHAPTER 6

RESULTS AND DISCUSSION

6.1 Results of Physical Model Experiments

Physical model experiments are conducted in different structure alternatives which are described in the previous Chapter.

1.Rock Project

The cross section of Rock Project alternative is shown in Figure 6.1. The views of section before and after experiment are also shown in Figure 6.2 (a) and (b). Damage ratios at the armour region and at the berm region, and also overtopping values are given in Table 6.1 In Table 6.2 overtopping values which were obtained from experiment under 50 years, 100 years LWL and 100 years HWL return period wave conditions are given.

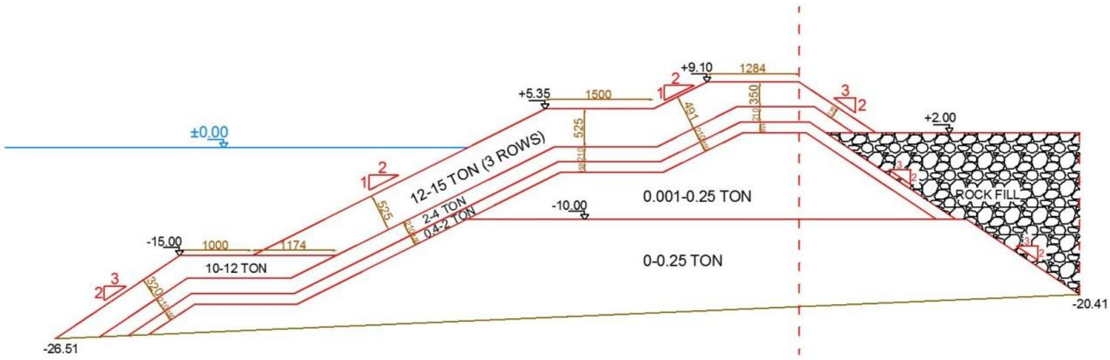


Figure 6.1 Cross-section of Rock Project

(a) *(b)*

Figure 6.2 (a) and (b) The views of section before and after the experiment.

Table 6.1 Damage ratio at the armor region and at the berm region of Rock Project

	Damage Ratio			Average Damage Ratio
	Trial 1	Trial 2	Trial 3	
12-15 ton Rock	20.60%	20.30%	21.20%	20.7%
Berm on -15 m (10-12 ton rock)	0.95%	0.85%	1.20%	1.0%

Table 6.2 Overtopping discharge of Rock Project

Rock Project	Overtopping (lt/s/m)
50 Years Rp LWL	2.86
100 Years Rp LWL	6.79
100 Years Rp HWL	---

For this section damage ratio varies between 20.30% and 21.20% with the average value as %20.7 in the armour region. The damage ratio of berm region varies between 0.85% and 1.20% with the average value of %1.0. Since the damage is high, then the filter layer came out the still water level. Most of the damage occured in the armour units around water level. Since the stability of this alternative is unacceptable, the overtopping analysis is excluded.

2.Rock-Alt-1

In this experiment cross-section that is given in Figure 6.3 was used. The views of section before and after experiment can be seen in Figure 6.4 (a) and (b). Stability conditons belong to the different parts of section is available in Table 6.3 . In Table 6.4 overtopping values which were obtained from experiment under 50 years, 100 years LWL and 100 years HWL return period wave conditions are given.

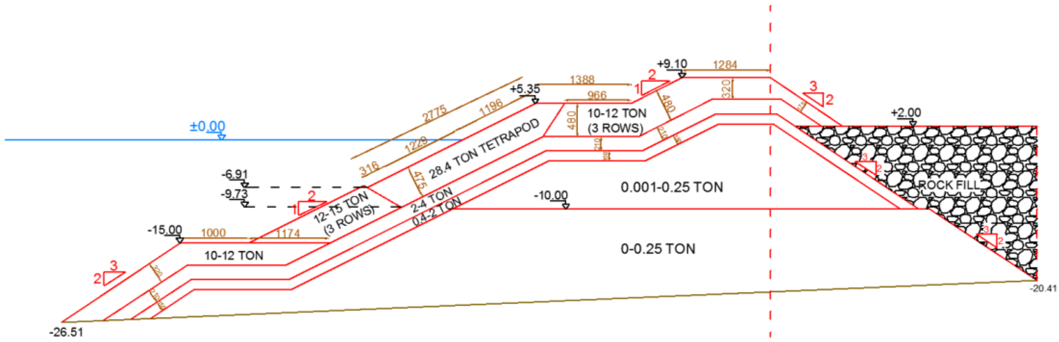


Figure 6.3 Cross-section of Rock-Alt-1

(a) *(b)*

Figure 6.4 (a) and (b) The views of section before and after the experiment

Table 6.3 Damage ratio at the armor region and at the berm region of Rock-Alt-1

	Damage Ratio			Average Damage Ratio
	Trial 1	Trial 2	Trial 3	
12-15 ton Rock	2.80%	2.22%	2.78%	2.6%
10-12 ton Rock on +5.35	1.80%	1.10%	1.60%	1.5%
28.4 Ton Tetrapod	3.85%	3.05%	3.64%	3.5%
Berm on -15 m (10-12 ton rock)	0.90%	0.90%	1.20%	1.0%

Table 6.4 Overtopping discharge of Rock-Alt-1

Rock Alt-1	Overtopping (lt/s/m)
50 Years Rp LWL	0.46
100 Years Rp LWL	2.83
100 Years Rp HWL	5.39

For this section damage ratio varies between 2.22% and 2.80% with the average value as %2.60 in the armour region for 12-15 ton rock. The damage ratio is 1.50% for 10-12 ton Rock on +5.35 which varies between 1.10% and 1.80%. 28.4 Ton Tetrapod has a damage ratio between 3.05% and 3.85% which is 3.5% on average. The damage ratio of berm region varies between 0.90% and 1.20% with the average value of %1.0. Overtopping discharge has values 0.46 lt/s/m for 50 years return period, 2.83 l/s/m for 100 years return period in low water level and 5.39 l/s/m for 100 years return period in high water level condition.

3.Rock-Alt-2

In this experiment cross-section that is given in Figure 6.5 was used. The views of section before and after experiment can be seen in Figure 6.6 (a) and (b). Stability conditons belong to the different parts of section is available in Table 6.5 . In Table 6.6 overtopping discharge values which were obtained from experiment under 50 years, 100 years LWL and 100 years HWL return period wave conditions are given.

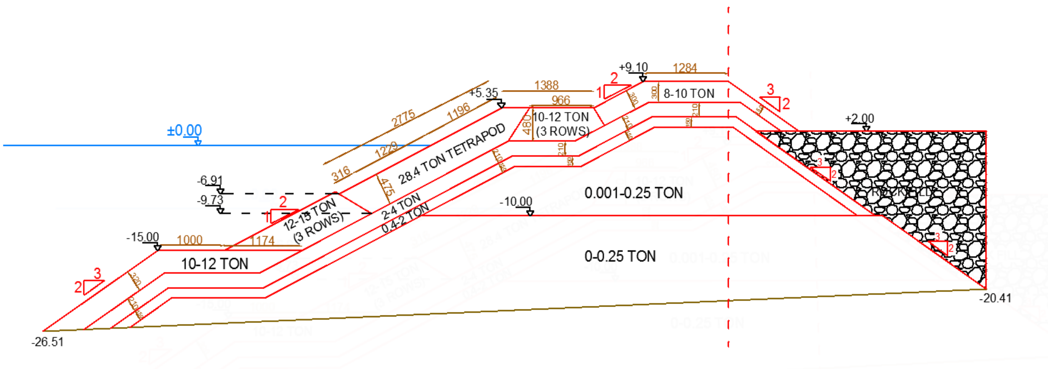
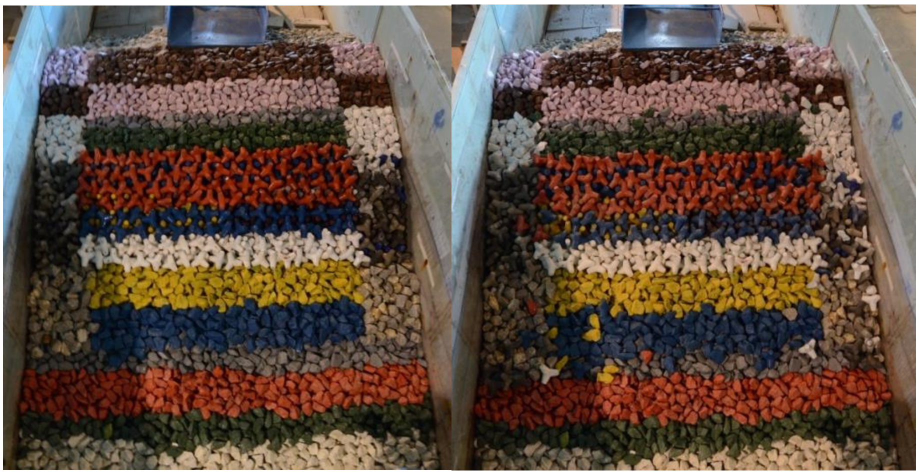


Figure 6.5 Cross-section of Rock-Alt-2



(a) *(b)*

Figure 6.6 (a) and (b) The views of section before and after the experiment.

Table 6.5 Damage ratio at the armor region and at the berm region of Rock-Alt-2

	Damage Ratio			Average Damage Ratio
	Trial 1	Trial 2	Trial 3	
12-15 ton Rock	1.86%	2.14%	2.05%	2.0%
10-12 ton Rock on +5.35	1.80%	1.80%	1.80%	1.8%
28.4 ton Tetrapod	2.84%	2.96%	3.20%	3.0%
Berm on -15 m (10-12 ton rock)	1.32%	1.28%	1.33%	1.3%
8-10 ton Rock on Crest	2.00%	2.25%	2.05%	2.1%

Table 6.6 Overtopping discharge of Rock-Alt-2

Rock Alt-1	Overtopping (lt/s/m)
50 Years Rp LWL	1.46
100 Years Rp LWL	2.74
100 Years Rp HWL	8.27

For this section damage ratio varies between 1.86% and 2.05% with the average value as %2.00 in the armour region for 12-15 ton rock. The damage ratio is 1.80% for 10-12 ton Rock on +5.35. 28.4 Ton Tetrapod has a damage ratio between 2.84% and 3.20% which is 3.0% on average. The damage ratio of berm region varies between 2.00% and 2.05% with the average value of %2.1. On crest 8-10 ton Rock were used and it has a damage ratio of 2.10%. Overtopping discharge has values 1.46 lt/s/m for 50 years return period, 2.74 l/s/m for 100 years return period in low water level and 8.27 l/s/m for 100 years return period in high water level condition.

4. Rock-Alt-2-1

In this experiment cross-section that is given in Figure 6.7 was used. The views of section before and after experiment can be seen in Figure 6.8 (a) and (b). Damage ratios belong to the different parts of section is available in Table 6.7. In Table 6.8 overtopping discharge values which were obtained from experiment under 50 years, 100 years LWL and 100 years HWL return period wave conditions are given.

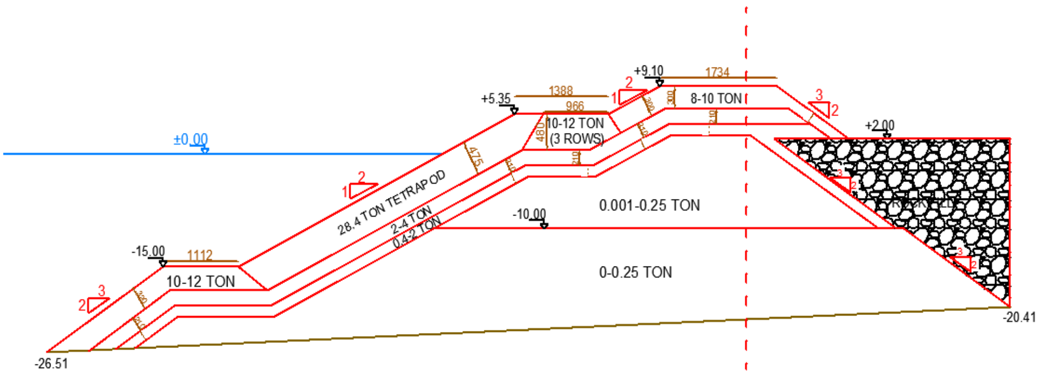


Figure 6.7 Cross-section of Rock-Alt-2-1

(a) (b)

Figure 6.8 (a) and (b) The views of section before and after the experiment.

Table 6.7 Damage ratio at the armor region and at the berm region of Rock-Alt-2-1

	Damage Ratio			Average Damage Ratio
	Trial 1	Trial 2	Trial 3	
10-12 ton Rock on +5.35	2.40%	2.70%	2.70%	2.6%
28.4 ton Tetrapod	0.90%	0.95%	1.14%	1.0%
Berm on -15 m (10-12 ton rock)	1.35%	1.10%	1.46%	1.3%
8-10 ton Rock on Crest	2.20%	2.30%	2.70%	2.4%

Table 6.8 Overtopping discharge of Rock-Alt-2-1

Rock Alt-2-1	Overtopping (lt/s/m)
50 Years Rp LWL	0.64
100 Years Rp LWL	1.33
100 Years Rp HWL	4.66

For this section damage ratio varies between 0.90% and 1.14% in 28.4 Ton Tetrapod with an average of 1.0%. 10-12 ton Rock on +5.35 has a damage ratio between 2.40% and 2.70% which is 2.6% on average. The damage ratio of berm region varies between 1.10% and 1.46% with the average value of %1.30. On crest 8-10 ton Rock were used and it has a damage ratio of 2.40%. Overtopping discharge has values 0.64 lt/s/m for 50 years return period, 1.33 l/s/m for 100 years return period in low water level and 4.6 l/s/m for 100 years return period in high water level condition.

5. Rock-Alt-3

In this experiment cross-section that is given in Figure 6.9 was used. The views of section before and after experiment can be seen in Figure 6.10 (a) and (b). Stability conditons belong to the different parts of section is available in Table 6.9 . In Table 6.10 overtopping values which were obtained from experiment under 50 years, 100 years LWL and 100 years HWL return period wave conditions are given.

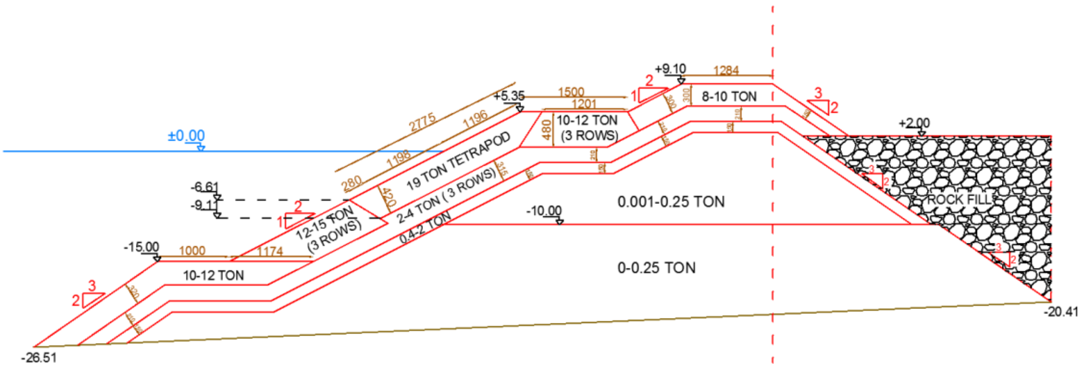
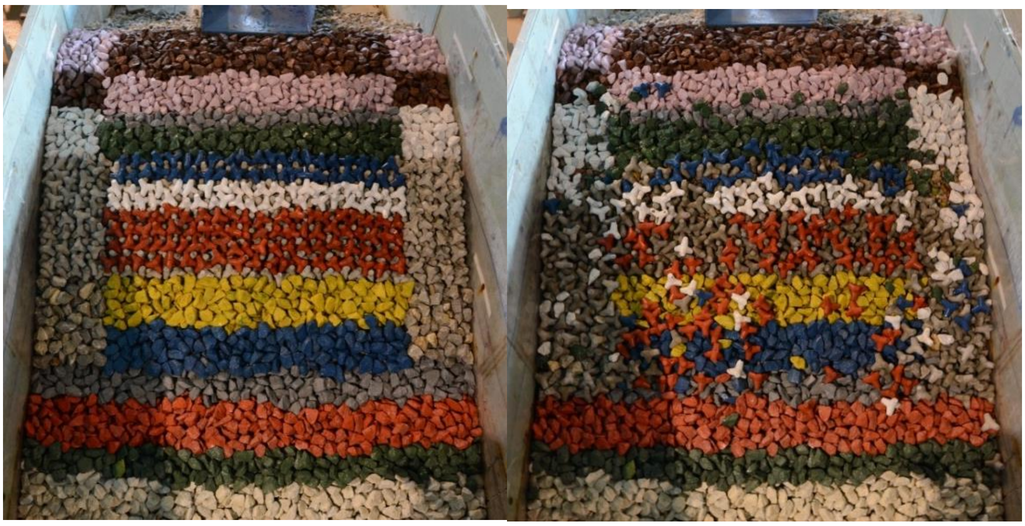


Figure 6.9 Cross-section of Rock-Alt-3



(a) *(b)*

Figure 6.10a) and (b) The views of section before and after the experiment.

Table 6.9 Damage ratio at the armor region and at the berm region of Rock-Alt-3

	Damage Ratio			Average Damage Ratio
	Trial 1	Trial 2	Trial 3	
10-12 ton Rock on +5.35	9.60%	9.00%	9.60%	9.4%
19 ton Tetrapod	22.0%	20.0%	20.5%	20.8%
Berm on -15 m (10-12 ton rock)	0.65%	0.75%	1.00%	0.8%
8-10 ton Rock on Crest	1.60%	2.15%	2.25%	2.0%
12-15 ton Rock	2.0%	2.20%	1.80%	2.0%

Table 6.10 Overtopping discharge of Rock-Alt-3

Rock Alt-3	Overtopping (lt/s/m)
50 Years Rp LWL	0.61
100 Years Rp LWL	3.98
100 Years Rp HWL	6.29

For this section damage ratio varies between 20.00% and 22.00% with the average value as %20.8 in the armour region. The damage ratio of berm region varies between 0.65% and 1.00% with the average value of %0.8. Since the damage is high, then the filter layer came out the still water level. Most of the damage occured in the armour units around water level. Since the stability of this alternative is unacceptable, the overtopping analysis is excluded.

6. Rock-Alt-4

In this experiment cross-section that is given in Figure 6.11 was used. The views of section before and after experiment can be seen in Figure 6.12 (a) and (b). Stability conditons belong to the different parts of section is available in Table 6.11 . In Table 6.12 overtopping discharge which was obtained from experiment under 50 years, 100 years LWL and 100 years HWL return period wave conditions are given.

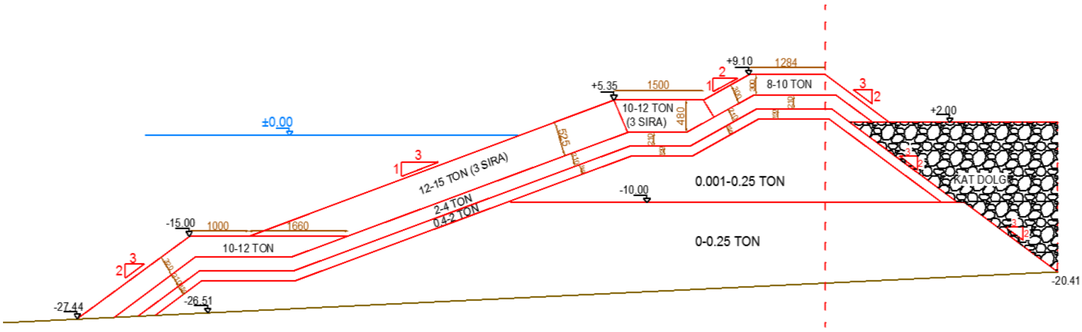


Figure 6.11 Cross-section of Rock-Alt-4

(a) *(b)*

Figure 6.12 (a) and (b) The views of section before and after the experiment.

Table 6.11 Damage ratio at the armor region and at the berm region of Rock-Alt-4

	Damage Ratio			Average Damage Ratio
	Trial 1	Trial 2	Trial 3	
10-12 ton Rock on +5.35	0.4%	0.4%	0.4%	0.4%
Berm on -15 m (10-12 ton rock)	1.0%	1.10%	1.30%	1.2%
8-10 ton Rock on Crest	0.25%	0.35%	0.30%	0.3%
12-15 ton Rock	14.80%	14.40%	13.70%	14.3%

Table 6.12 Overtopping discharge of Rock-Alt-4

Rock Alt-4	Overtopping (lt/s/m)
50 Years Rp LWL	0.44
100 Years Rp LWL	0.79
100 Years Rp HWL	2.15

For this section damage ratio varies between 13.70% and 14.80% with the average value as %14.3 in the armour region. The damage ratio of berm region varies between 1.00% and 1.30% with the average value of %1.0. Since the damage is high, then the filter layer came out the still water level. Most of the damage occured in the armour units around water level. Since the stability of this alternative is unacceptable, the overtopping analysis is excluded.

7. Rock-Alt-5

In this experiment cross-section that is given in Figure 6.13 was used. The views of section before and after experiment can be seen in Figure 6.14 (a) and (b). Stability conditons belong to the different parts of section is available in Table 6.13 . In Table6.14 overtopping discharge which was obtained from experiment under 50 years, 100 years LWL and 100 years HWL return period wave conditions are given.

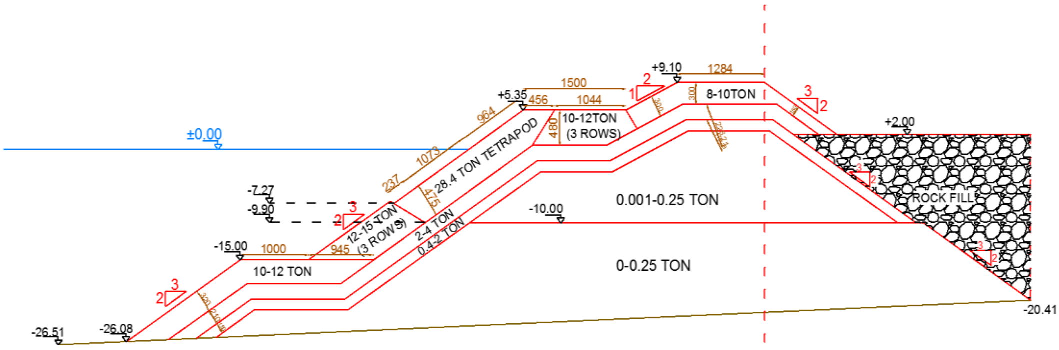


Figure 6.13 Cross-section of Rock-Alt-5 Project

(a) *(b)*

Figure 6.14 (a) and (b) The views of section before and after the experiment.

Table 6.13 Damage ratio at the armor region and at the berm region of Rock-Alt-5

	Damage Ratio			Average Damage Ratio
	Trial 1	Trial 2	Trial 3	
10-12 ton Rock on +5.35	2.80%	2.70%	3.20%	2.9%
Berm on -15 m (10-12 ton rock)	0.95%	0.90%	0.88%	0.9%
8-10 ton Rock on Crest	2.00%	2.30%	2.30%	2.2%
12-15 ton Rock	16.30%	16.10%	15.60%	16.0%
28.4 ton Tetrapod	9.40%	10.00%	10.00%	9.8%

Table 6.14 Overtopping discharge of Rock-Alt-5

Rock Alt-5	Overtopping (lt/s/m)
50 Years Rp LWL	1.97
100 Years Rp LWL	4.81
100 Years Rp HWL	20.48

For this section damage ratio varies between 15.60% and 16.30% with the average value as %16.0 in the armour region. The damage ratio of berm region varies between 0.88% and 0.95% with the average value of %0.90. Since the damage is high, then the filter layer came out the still water level. Most of the damage occured in the armour units around water level. Since the stability of this alternative is unacceptable, the overtopping analysis is excluded.

8. Rock-Alt-6

In this experiment cross-section that is given in Figure 6.15 was used. The views of section before and after experiment can be seen in Figure 6.16 (a) and (b). Stability conditons belong to the different parts of section is available in Table 6.15 . In Table 6.16 overtopping discharge which was obtained from experiment under 50 years, 100 years LWL and 100 years HWL return period wave conditions are given.

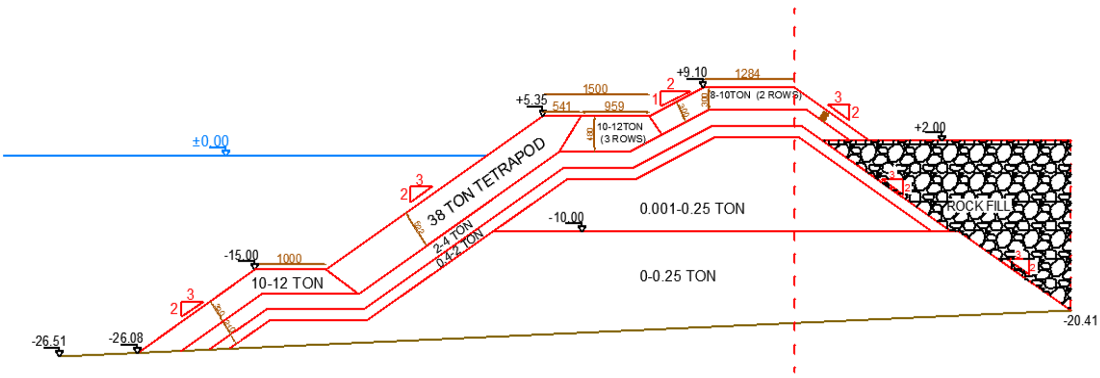


Figure 6.15 Cross-section of Rock-Alt-6

(a) *(b)*

Figure 6.16 (a) and (b) The views of section before and after the experiment.

Table 6.15 Damage ratio at the armor region and at the berm region of Rock-Alt-6

	Damage Ratio			Average Damage Ratio
	Trial 1	Trial 2	Trial 3	
10-12 ton Rock on +5.35	3.75%	3.70%	3.68%	3.7%
Berm on -15 m (10-12 ton rock)	1.9%	1.7%	1.9%	1.8%
8-10 ton Rock on Crest	1.8%	1.9%	1.9%	1.9%
38 ton Tetrapod	1.0%	0.9%	0.8%	0.9%

Table 6.16 Overtopping discharge of Rock-Alt-6

Rock Alt-6	Overtopping (lt/s/m)
50 Years Rp LWL	0.86
100 Years Rp LWL	5.14
100 Years Rp HWL	14.58

For this section damage ratio varies between 0.80% and 1.00% in 38 Ton Tetrapod with an average of 0.9%. 10-12 ton Rock on +5.35 has a damage ratio between

3.68% and 3.75% which is 3.7% on average. The damage ratio of berm region varies between 1.80% and 1.90% with the average value of %1.80. On crest 8-10 ton Rock were used and it has a damage ratio of 1.90%. Overtopping discharge has value of 0.86 lt/s/m for 50 years return period, 5.14 l/s/m for 100 years return period in low water level and 14.58 l/s/m for 100 years return period in high water level condition.

9.Tetrapod Project

In this experiment cross-section that is given in Figure 6.17 was used. The views of section before and after experiment can be seen in Figure 6.18 (a) and (b). Stability conditons belong to the different parts of section is available in Table 6.17 . In Table 6.18 overtopping discharge which was obtained from experiment under 50 years, 100 years LWL and 100 years HWL return period wave conditions are given.

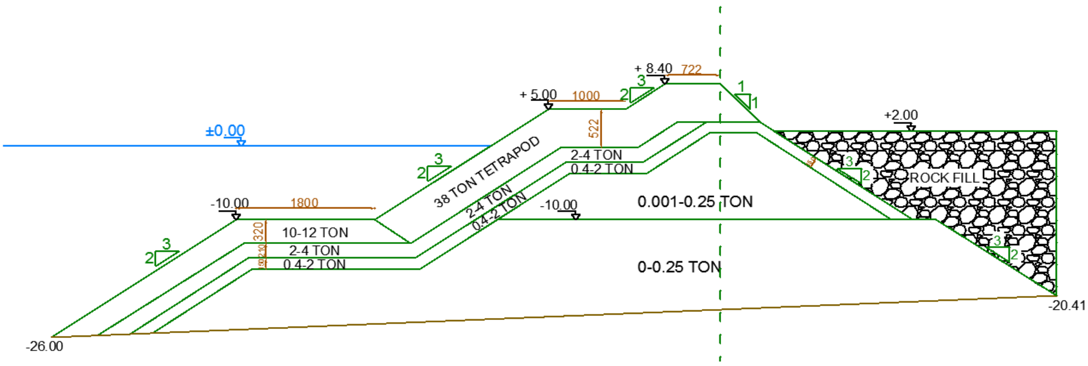


Figure 6.17 Cross-section of Tetrapod Project

(a) *(b)*

Figure 6.18(a) and (b) The views of section before and after the experiment.

Table 6.17 Damage ratio at the armor region and at the berm region of Tetrapod Project

	Damage Ratio			Average Damage Ratio
	Trial 1	Trial 2	Trial 3	
Berm on -10 m (10-12 ton rock)	9.0%	9.0%	8.72%	8.9%
38 ton Tetrapod	1.45%	1.56%	1.49%	1.5%

Table 6.18 Overtopping discharge of Tetrapod Project

Tetrapod Project	Overtopping (lt/s/m)
50 Years Rp LWL	0.90
100 Years Rp LWL	5.60
100 Years Rp HWL	21.60

For this section damage ratio varies between 1.45% and 1.56% in 38 Ton Tetrapod with an average of 1.5%. The damage ratio of berm region varies between 8.72% and 9.00% with the average value of %8.90. Since the damage is high, then the filter layer came out the still water level. Most of the damage occured in the armour units around water level. Since the stability of this alternative is unacceptable, the overtopping analysis is excluded.

10. Tetrapod-Alt-1

In this experiment cross-section that is given in Figure 6.19 was used. The views of section before and after experiment can be seen in Figure 6.20 (a) and (b). Stability conditons belong to the different parts of section is available in Table 6.19 . In Table 6.20 overtopping discharge which was obtained from experiment under 50 years, 100 years LWL and 100 years HWL return period wave conditions are given.

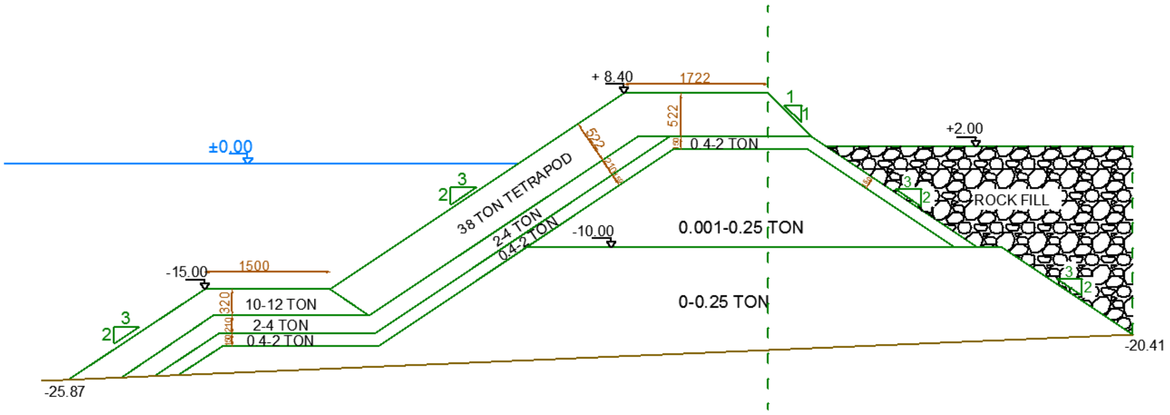


Figure 6.19 Cross-section of Tetrapod-Alt-1

(a) *(b)*

Figure 6.20 (a) and (b) The views of section before and after the experiment.

Table 6.19 Damage ratio at the armor region and at the berm region of Tetrapod-Alt-1

	Damage Ratio			Average Damage Ratio
	Trial 1	Trial 2	Trial 3	
Berm on -15 m (10-12 ton rock)	1.80%	1.60%	1.70%	1.7%
38 ton Tetrapod	0.40%	0.40%	0.40%	0.4%

Table 6.20 Overtopping discharge of Tetrapod-Alt-1

Tetrapod-Alt-1	Overtopping (lt/s/m)
50 Years Rp LWL	0.57
100 Years Rp LWL	2.21
100 Years Rp HWL*	5.72
100 Years Rp HWL**	10.80

* +8.40 elevation (over tetrapod)

** +3.20 elevation (below tetrapod)

For this section damage ratio varies between 1.60% and 1.80% in 38 Ton Tetrapod with an average of 1.70%. The damage ratio of berm region has the average value of %0.40. Overtopping discharge has values 0.57 lt/s/m for 50 years return period, 2.21 l/s/m for 100 years return period in low water level and 5.72 l/s/m for 100 years return period in high water level condition over tetrapod and 10.80 l/s/m below tetrapod.

11. Tetrapod-Alt-2

In this experiment cross-section that is given in Figure 6.21 was used. The views of section before and after experiment can be seen in Figure 6.22 (a) and (b). Stability conditons belong to the different parts of section is available in Table 6.21. In Table 6.22 overtopping values which were obtained from experiment under 50 years, 100 years LWL and 100 years HWL return period wave conditions are given.

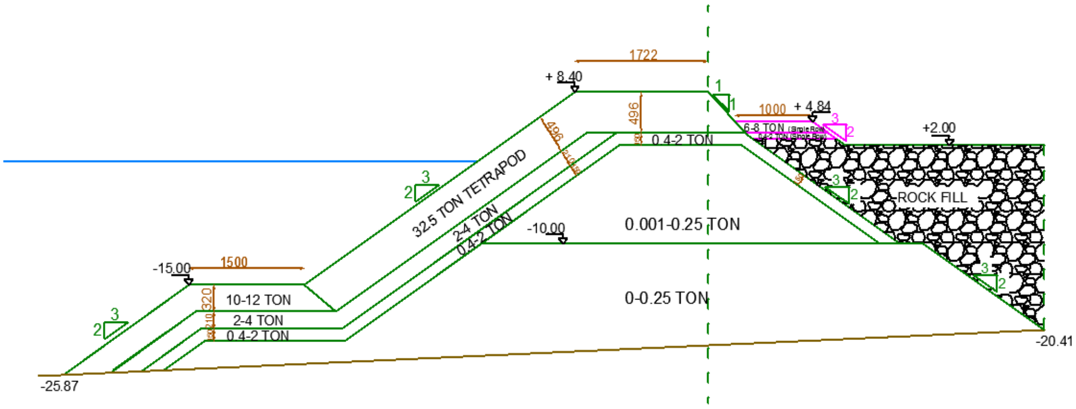


Figure 6.21 Cross-section of Tetrapod-Alt-2

(a) *(b)*

Figure 6.22 (a) and (b) The views of section before and after the experiment.

Table 6.21 Damage ratio at the armor region and at the berm region of Tetrapod-Alt-2

	Damage Ratio			Average Damage Ratio
	Trial 1	Trial 2	Trial 3	
32.5 ton Tetrapod	0.30%	0.25%	0.35%	0.3%
Berm on -15 m (10-12 ton rock)	2.0%	1.75%	2.25%	2.0%
6-8 ton Protection Behind Crest	2.60%	2.75%	2.75%	2.7%

Table 6.22 Overtopping discharge of Tetrapod-Alt-2

Tetrapod-Alt-2	Overtopping (lt/s/m)
50 Years Rp LWL	0.16
100 Years Rp LWL	1.43
100 Years Rp HWL*	5.61
100 Years Rp HWL**	19.26
Behind 6-8 ton	5.50

* +8.40 elevation (over tetrapod)

** +3.40 elevation (below tetrapod)

***Measurement behind 6-8 ton protection layer

For this section damage ratio varies between 0.25% and 0.35% in 32.5 Ton Tetrapod with an average of 0.30%. The damage ratio of berm region has the average value of %2.0. the damage ratio of 6-8 ton protection layer behind crest is 2.70 %. Overtopping discharge has values 0.16 lt/s/m for 50 years return period, 1.43 l/s/m for 100 years return period in low water level and 5.61 l/s/m for 100 years return period in high water level condition over tetrapod and 19.26 l/s/m below tetrapod.

12. Tetrapod-Alt-3

In this experiment cross-section that is given in Figure 6.23 was used. The views of section before and after experiment can be seen in Figure 6.24 (a) and (b). Stability conditons belong to the different parts of section is available in Table 6.23 . In Table 6.24 overtopping values which were obtained from experiment under 50 years, 100 years LWL and 100 years HWL return period wave conditions are given.

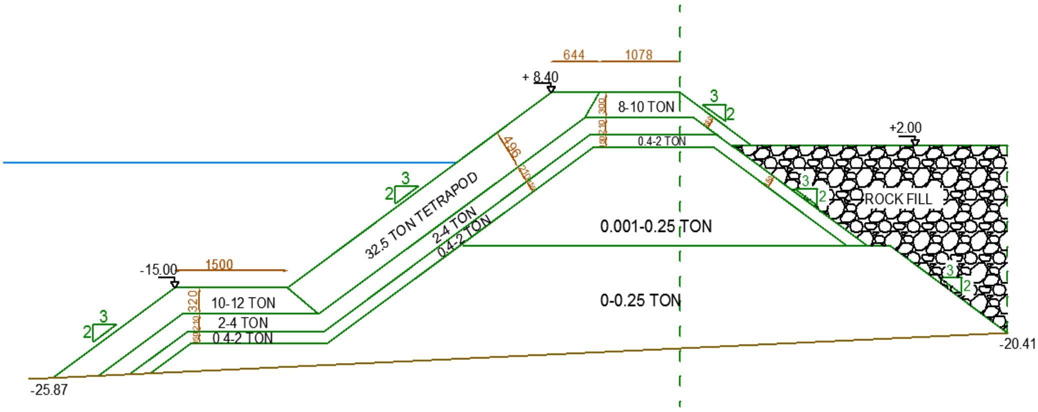


Figure 6.23 Cross-section of tetrapod-Alt-3

(a) *(b)*

Figure 6.24(a) and (b) The views of section before and after the experiment.

Table 6.23 Damage ratio at the armor region and at the berm region of Tetrapod-Alt-3

	Damage Ratio			Average Damage Ratio
	Trial 1	Trial 2	Trial 3	
32.5 ton Tetrapod	0.35%	0.26%	0.29%	0.3%
Berm on -15 m (10-12 ton rock)	1.80%	2.10%	1.80%	1.9%
8-10 ton Rock on Crest	4.50%	5.00%	5.30%	4.9%

Table 6.24 Overtopping discharge of Tetrapod-Alt-3

Tetrapod-Alt-2	Overtopping (lt/s/m)
50 Years Rp LWL	0.23
100 Years Rp LWL	2.34
100 Years Rp HWL*	5.59
100 Years Rp HWL**	7.49

* +8.40 elevation (over tetrapod)

** +3.30 elevation (below tetrapod)

For this section damage ratio varies between 0.26% and 0.35% in 32.5 Ton Tetrapod with an average of 0.30%. The damage ratio of berm region has the average value of %1.9. The damage ratio of 8-10 ton rock on crest is 4.90 %. Overtopping discharge has values 0.23 lt/s/m for 50 years return period, 2.34 l/s/m for 100 years return period in low water level and 5.59 l/s/m for 100 years return period in high water level condition over tetrapod and 7.49 l/s/m below tetrapod.

13. Accropode^{TM} II Alt-1

In this experiment cross-section that is given in Figure 6.25 was used. The views of section before and after experiment can be seen in Figure 26 (a) and (b). Stability conditons belong to the different parts of section is available in Table 6.25. In Table 6.26 overtopping discharge which was obtained from experiment under 50 years, 100 years LWL and 100 years HWL return period wave conditions are given.

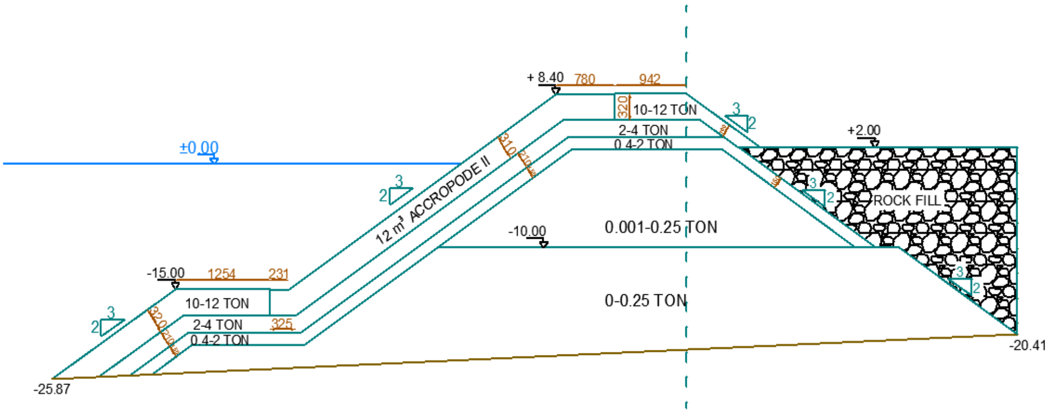


Figure 6.25 Cross-section of Accropode^{TM} II Alt-1

(a) *(b)*

Figure 6.26 (a) and (b) The views of section before and after the experiment.

Table 6.25 Damage ratio at the armor region and at the berm region of Accropode^{TM} II Alt-1

	Damage Ratio			Average Damage Ratio
	Trial 1	Trial 2	Trial 3	
12 m3 Accropode	0.0%	0.0%	0.0%	0.0%
Berm on -15 m (10-12 ton rock)	2.0%	2.40%	2.50%	2.3%
10-12 ton Rock on Crest	0.8%	0.8%	0.8%	0.8%

Table 6.26 Overtopping discharge of Accropode^{TM} II Alt-1

AccropodeTM II Alt-1	Overtopping (lt/s/m)
50 Years Rp LWL	1.36
100 Years Rp LWL	5.49
100 Years Rp HWL*	12.68
100 Years Rp HWL**	10.71

* +3.30 elevation (below accropode)

** +8.40 elevation (over accropode)

For this section damage ratio is 0.0% for AccropodeTM II units. The damage ratio of berm region has the average value of %2.3. The damage ratio of 10-12 tons of rock on crest is 0.80 %. Overtopping discharge has values 1.36 lt/s/m for 50 years return period, 5.49 l/s/m for 100 years return period in low water level and 12.68 l/s/m for 100 years return period in high water level condition over accropode and 10.71 l/s/m below accropode.

14. Accropode^{TM} II Alt-2

In this experiment cross-section that is given in Figure 6.27 was used. The views of section before and after experiment can be seen in Figure 6.28 (a) and (b). Stability conditons belong to the different parts of section is available in Table 6.27 . In Table

6.28 overtopping discharge which was obtained from experiment under 50 years, 100 years LWL and 100 years HWL return period wave conditions are given.

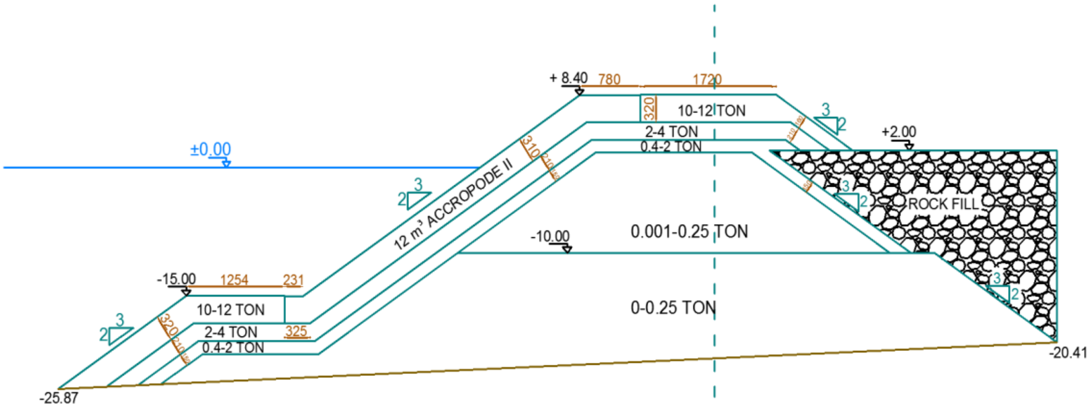


Figure 6.27 Cross-section of Accropode^{TM} II Alt-2

(a) *(b)*

Figure 6.28 (a) and (b) The views of section before and after the experiment.

Table 6.27 Damage ratio at the armor region and at the berm region of Accropode^{TM} II Alt-2

	Damage Ratio			Average Damage Ratio
	Trial 1	Trial 2	Trial 3	
12 m3 Accropode	0.0%	0.0%	0.0%	0.0%
Berm on -15 m (10-12 ton rock)	3.0%	3.0%	2.70%	2.9%
10-12 ton Rock on Crest	0.50%	0.50%	0.50%	0.5%

Table 6.28 Overtopping discharge of Accropode^{TM} II Alt-2

AccropodeTM II Alt-1	Overtopping (lt/s/m)
50 Years Rp LWL	0.10
100 Years Rp LWL	1.27
100 Years Rp HWL*	2.31
100 Years Rp HWL**	1.72

* +3.30 elevation (below accropode)

** +8.40 elevation (over accropode)

For this section damage ratio is 0.0% for AccropodeTM II units. The damage ratio of berm region has the average value of %2.9 varies between 2.70% and3.0%. The damage ratio of 10-12 tons of rock on crest is 0.50 %. Overtopping discharge has values 0.10 lt/s/m for 50 years return period, 1.27 l/s/m for 100 years return period in low water level and 1.72 l/s/m for 100 years return period in high water level condition over accropode and 2.31 l/s/m below accropode.

15. AccropodeTM II Alt-3

In this experiment cross-section that is given in Figure 6.29 was used. The views of section before and after experiment can be seen in Figure 6.30 (a) and (b). Stability conditons belong to the different parts of section is available in Table 6.29 . In Table 6.30 overtopping discharge which was obtained from experiment under 50 years, 100 years LWL and 100 years HWL return period wave conditions are given.

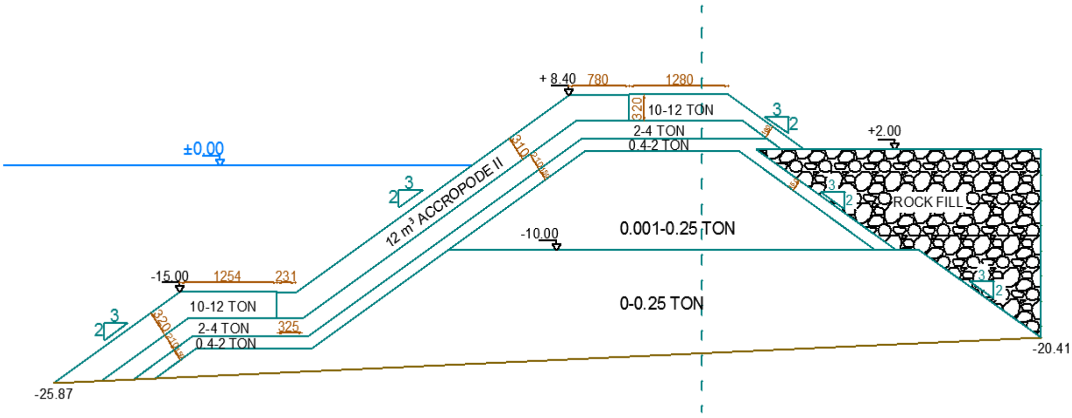


Figure 6.29 Cross-section of Accropode^{TM} II Alt-3

(a) *(b)*

Figure6.30 (a) and (b) The views of section before and after the experiment.

Table 6.29 Damage ratio at the armor region and at the berm region of Accropode^{TM} II Alt-3

	Damage Ratio			Average Damage Ratio
	Trial 1	Trial 2	Trial 3	
12 m3 Accropode	0.0%	0.0%	0.0%	0.0%
Berm on -15 m (10-12 ton rock)	2.50%	3.0%	2.60%	2.7%
10-12 ton Rock on Crest	0.60%	0.60%	0.65%	0.6%

Table 6.30 Overtopping discharge of Accropode^{TM} II Alt-3

AccropodeTM II Alt-1	Overtopping (lt/s/m)
50 Years Rp LWL	0.40
100 Years Rp LWL	2.31
100 Years Rp HWL*	7.11
100 Years Rp HWL**	4.91

* +3.30 elevation (below accropode)

** +8.40 elevation (over accropode)

For this section damage ratio is 0.0% for AccropodeTM II units. The damage ratio of berm region has the average value of %2.7 varies between 2.50% and3.0%. The damage ratio of 10-12 tons of rock on crest is 0.60 %. Overtopping discharge has values 0.40 lt/s/m for 50 years return period, 2.31 l/s/m for 100 years return period in low water level and 4.91 l/s/m for 100 years return period in high water level condition over accropode and 7.11 l/s/m below accropode.

6.2 Discussion of Results

In the scope of this study it is aimed to design the most appropriate cross – section will be used in the construction of Rize – Artvin Airport. Initially the design wave height is obtained after series of studies. Then modeling procedure was initiated in the wave basin. 15 different cross – sections were inspected during experiments.

The overall summary and discussion of the results are given in the following for each structure alternative.

Rock Project : For this section damage ratio is %20.7 in armour region. On water level filter layer appeared. Most of the damage occured in the armour units around water level. Since the stability of this alternative is unacceptable the overtooping analysis is excluded.

Rock-Alt-1: Most of the damage in Rock Project section occured around water level so in this part 28.4 tons of tetrapod was used for Rock-Alt-1. Stability was satisfied in this section and overtopping value is 5.39 lt/s/m.

Rock-Alt-2: In this alternative Rock-Alt-1 section was tried to be more economical by replacing 10-12 tons rock by 8-10 tons on the crest. Section succeeded stability but increased overtopping to 8.27 lt/s/m which is acceptable.

Rock-Alt-2-1: The damaged parts of Rock Project section were changed with 28.4 tons tetrapod and Rock-Alt-1 and Rock-Alt-2 sections were created. So Rock-Alt-1 and Rock-Alt-2 sections are composite. But the rock sections under tetrapod had a damage ratio of %2. It was considered to affect the stability of tetrapod section. It was decided to compose all slope with 28.4 tons of tetrapods. To reduce the overtopping the crest was expanded to 17.34 m and a value of 4.66 lt/s/m was obtained.

Rock-Alt-3: The 28.4 tons of tetrapods were replaced with 19 tons of tetrapods in Rock-Alt-2 and named as Rock-Alt-3. The damage ratio was %20.8 so this section failed.

Rock-Alt-4: The slope was created as 1/3 and by using just 12-15 tons of rock armour unit. The damage ratio was %14.3.

Rock-Alt-5: To make Rock-Alt-2 more economical the structure slope was changed to 2/3 . However damage ratio is %16 for 12-15 tons of rocks and %9.8 for tetrapods which are unacceptable. Overtopping is 20.48 lt/s/m .

Rock-Alt-6: In this section 38 tons of tetrapods were used on 2/3 slope. The section was stable but overtopping is 14.58 lt/s/m .

Tetrapod Project : The damage ratio in berm on -10.00 m elevation is %8.9 and overtopping value is 21.60 lt/s/m.

Tetrapod Alt-1: In this section the toe was created on -15.0 m elevation. Damage ratio occured at %1.7. To reduce the overtopping, berm on +5.00 m elevation was added to crest. In this alternative overtopping was measured both for +8.40m and +3.20 m elevations under 100 years return period waves (HWL) considering the conditions that whether a crown wall will be built or not.

On +3.20 m elevation 5.72 lt/s/m and on +8.40 m elevation 10.80 lt/s/m were measured.

Tetrapod Alt-2 : To design „Tetrapod Alt-1" in more economical way 32.5 tons of tetrapods were used instead of 38 tons of tetrapods and the stability was satisfied. Overtopping value on crest elevation is 5.61 lt/s/m and 19.26 lt/s/m under crest elevation. To satisfy the penetration of these waves and prevent the scouring behind tetrapods an area was created with a width of 10 m. It was composed by using 0.4-2 tons of filter layer and 6-8 tons of rocks. Overtopping behind this area is 5.50 lt/s/m. Damage ratio for 6-8 tons of rocks is %2.7.

Tetrapod Alt-3 : In this alternative 8-10 tons of rocks were used in crest. However the damage ratio for 8-10 tons is about % 5.

Overtopping on the crest is 5.5 lt/s/m and under the crest 7.49 lt/s/m.

Accropode™ II Alt-1, Accropode™ II Alt-2 ve Accropode™ II Alt-3 Alternatives

205 gr of Accropode™ II model units were used to inspect Rize-Artvin Airport Project Breakwater stability. Related to the design wave of Rize-Artvin Airport, dimension of Accropode™ II units were determined as 12 m^3 . In Accropode™ II Alt-1 alternative 10-12 tons of rocks were used in crest. The least requirement of CLI for crest is 3 rows of Accropode™ II units. They can be used in entire crest but for economical reasons the least requirement was satisfied. Damage ratio is % 0 for this alternative. Overtopping on crest 10.71 lt/s/m and 12.68 l/s/m under it. The section „Accropode™ II Alt-2" that was created to reduce overtopping with 25 m width has values of 1.72 l/s/m over the crest and 2.3 lt/s/m under the crest. In another alternative (Accropode™ II Alt-3) with a crst width of 20.60 m overtopping has occured as 4.91 lt/s/m over the crest, 7.11 lt/s/m under the crest. For all alternatives damage ratio is % 0.0. Besides Accropode™ II is a new technology for our country so to determine its damage trend a wave set 1.30 times larger than design wave was applied to the section and the damage ratio is still %0.0.

The Accropode™ II model units were placed with a porosity of %54.58. Placing method offered by CLI was applied that the distances between units were horizontally Dh=7.88 cm, vertically Dv=3.94. Gridded area necessary to design a stable cross section is given in Figure??

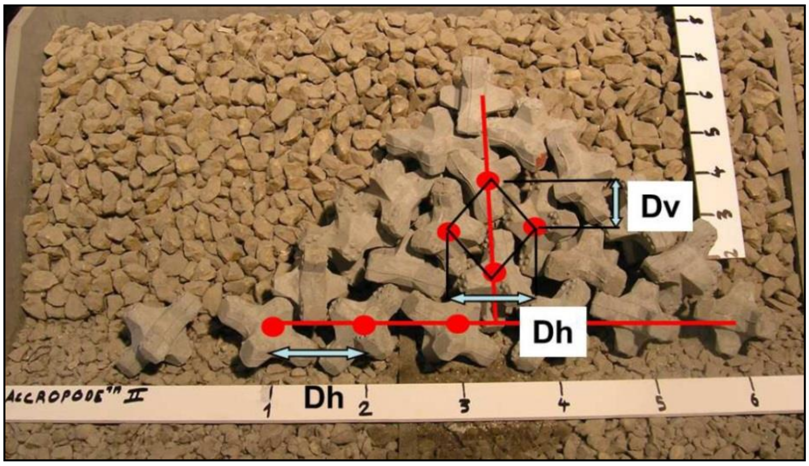


Figure 6.31 Accropode II placement is performed as in and units behave monolitic due to interlocking.

Figure 6.32 Accropode II Placement in Experiment Section

CHAPTER 7

ECONOMIC ANALYSIS AND COMPARISON

The necessary material amounts (concrete volume and rock categories) per unit meter of breakwater section for each above mentioned survived alternatives are given in Table 7.1. It is seen from Table 7.1 that the cross – sections where the Accropode^{TM} II are used require much less amount of concrete and less amounts of rocks and therefore, they are the most economical alternatives. In this part of study only the amount of materials used in cross-sections are considered and compared. The other expenses such as patent fee, transportation, formworks and other constructional requirements are excluded.

Table 7.1 The necessary material amounts per unit meter of breakwater section for each survived structure alternatives. (m^3/m).

Alternative	Concrete	Core	0.4-2 Ton	2-4 Ton	8-10 Ton	10-12 Ton	12-15 Ton
Rock Alt-1	64.85	1132.3	123.95	153.89	-	183.51	53.6
Rock Alt-2	64.85	1152.1	124.35	154.60	52.81	113.20	53.6
Rock Alt-2-1	108.89	1152.1	135.40	155.91	61.84	101.80	-
Tetrapod Alt-2	147.34	937.59	108.49	99.02	-	70.89	-
Tetrapod Alt-3	109.77	933.87	107.08	123.69	-	107.50	-
Accropode^{TM} II Alt-2	73.89	987.78	111.45	137.73	-	109.16	-
Accropode^{TM} II Alt-3	73.89	987.79	110.96	128.49	-	100.15	-

-Porosity for Tetrapod is %50, and for Accropode^{TM} II is %54.58; - Porosity for rock is %33 .

CHAPTER 8

CONCLUSIONS

The physical model expermiments for coastal protection structures of Rize - Artvin Airport are performed to determine the suitable cross section from 15 different alternatives. The design wave characteristics are determined from available studies and the same wave conditions are used in all of the tests. The water depth in front of the structure in average 22-23m and becomes 27.60 m as maximum. It is the deepest reclaimed airport built in the World. The water depth used in physical model experiments is selected as 25m which represent general dimensions of the modelled structure.

Design wave height used for the breakwater is H_s = 7.70 m with a period of Tm = 10 sec. Different slopes were tried for alternatives. Regarding to obstacle related to flight safety reasons, the crest elevation was kept as constant.

The results of model experiments for 15 different structure alternartives are tabulated in Table 8.1as a summary.

As seen from the Table8.1

- 7 of the 15 different structure alternatives are found to be acceptable in terms of national and international stability and overtopping criteria. All sections whose armor units are rock could not satisfy necessary stability conditions.

- AccropodeTM II units had zero damage during the experiments. In Turkey it was first time to use AccropodeTM II armor unit in physical modeling. AccropodeTM II was not also applied before to a structure at this depth.

Table 8.1 Result of Performed Experiments

	Damage				Overtopping (lt/s/m) 100 years HWL	
Alternative	***Toe Berm***	***Armour Unit***	***Berm***	***Crest***	***Measurement Over Crest***	***Measurement Under Crest***
Rock Project	% 1.0	% 20.7			---	
Rock-Alt-1	% 1.0	Tetrapod: % 3.5 12-15 Ton Rock: %2.6	% 1.5		5.39	
Rock-Alt-2	% 1.3	Tetrapod: % 3.0 12-15 Ton Rock: %2.0	% 1.8	% 2.1	8.27	
Rock-Alt-2-1	% 1.3	% 1.0	% 2.6	% 2.4	4.66	
Rock-Alt-3	% 0.8	Tetrapod: % 20.8 12-15 Ton Rock: %2.0	%9.4	% 2.0	6.29	
Rock-Alt-4	% 1.2	%14.3	% 0.4	% 0.3	2.15	
Rock-Alt-5	% 0.9	Tetrapod: % 9.8 12-15 Ton Rock: %16.0	% 2.9	% 2.2	20.48	
Rock-Alt-6	% 1.8	% 0.9	% 3.7	% 1.9	14.58	
Tetrapod Project	% 8.9	% 1.5			21.60	
Tetrapod-Alt-1	% 1.7	% 0.4			5.72	10.80
Tetrapod-Alt-2	% 2.0	% 0.3			5.61	Under Tetrapod: 19.26 Behind 6-8 Ton : 5.50
Tetrapod-Alt-3	% 1.9	% 0.3	---	% 4.9	5.59	7.49
Accropode II Alt-1	% 2.3	% 0.0	---	% 0.8	10.71	12.68
Accropode II Alt-2	% 2.9	% 0.0	---	% 0.5	1.72	2.31
Accropode II Alt-3	% 2.7	% 0.0	---	% 0.6	4.91	7.11

- For Rock Project damage ratio varies between 20.30% and 21.20% with the average value as %20.7 in the armour region. The damage ratio of berm region varies 0.85% and 1.20% with the average value of %1.0. Since the damage is high, then the filter layer came out the still water level. Most of the damage occured in the armour units around water level. Since the stability of this alternative is unacceptable, the overtopping analysis is excluded.
- For Rock-Alt-1 section damage ratio varies between 2.22% and 2.80% with the average value as %2.60 in the armour region for 12-15 ton rock. The damage ratio is 1.50% for 10-12 ton Rock on +5.35 which varies between 1.10% and 1.80%. 28.4 Ton Tetrapod has a damage ratio between 3.05% and 3.85% which is 3.5% on average. The damage ratio of berm region varies between 0.90% and 1.20% with the average value of %1.0. Overtopping discharge has values 0.46 lt/s/m for 50 years return period, 2.83 l/s/m for 100 years return period in low water level and 5.39 l/s/m for 100 years return period in high water level condition.
- For Rock-Alt-2 damage ratio varies between 1.86% and 2.05% with the average value as %2.00 in the armour region for 12-15ton rock. The damage ratio is 1.80% for 10-12 ton Rock on +5.35. 28.4 Ton Tetrapod has a damage ratio between 2.84% and 3.20% which is 3.0% on average. The damage ratio of berm region varies between 2.00% and 2.05% with the average value of %2.1. On crest 8-10 ton Rock were used and it has a damage ratio of 2.10%. Overtopping discharge has values 1.46 lt/s/m for 50 years return period, 2.74 l/s/m for 100 years return period in low water level and 8.27 l/s/m for 100 years return period in high water level condition.
- For Rock-Alt-2-1 damage ratio varies between 0.90% and 1.14% in 28.4 Ton Tetrapod with an average of 1.0%. 10-12ton Rock on +5.35 has a damage ratio between 2.40% and 2.70% which is 2.6% on average. The damage ratio of berm region varies between 1.10% and 1.46% with the average value of %1.30. On crest 8-10 ton Rock were used and it has a damage ratio of 2.40%. Overtopping discharge has values 0.64 lt/s/m for 50 years return period, 1.33 l/s/m for 100 years return period in low water level and 4.6 l/s/m for 100 years return period in high water level condition.
- For Rock-Alt-3 damage ratio varies between 20.00% and 22.00% with the average value as %20.8 in the armour region. The damage ratio of berm region varies 0.65% and 1.00% with the average value of %0.8. Since the damage is high,

then the filter layer came out the still water level. Most of the damage occured in the armour units around water level. Since the stability of this alternative is unacceptable, the overtopping analysis is excluded.

- For Rock-Alt-4 damage ratio varies between 13.70% and 14.80% with the average value as %14.3 in the armour region. The damage ratio of berm region varies between 1.00% and 1.30% with the average value of %1.0. Since the damage is high, then the filter layer came out the still water level. Most of the damage occured in the armour units around water level. Since the stability of this alternative is unacceptable, the overtopping analysis is excluded.
- For Rock-Alt-5 damage ratio varies between 15.60% and 16.30% with the average value as %16.0 in the armour region. The damage ratio of berm region varies between 0.88% and 0.95% with the average value of %0.90. Since the damage is high, then the filter layer came out the still water level. Most of the damage occured in the armour units around water level. Since the stability of this alternative is unacceptable, the overtopping analysis is excluded.
- For Rock-Alt-6 damage ratio varies between 0.80% and 1.00% in 38 Ton Tetrapod with an average of 0.9%. 10-12 ton Rock on +5.35 has a damage ratio between 3.68% and 3.75% which is 3.7% on average. The damage ratio of berm region varies between 1.80% and 1.90% with the average value of %1.80. On crest 8-10 ton Rock were used and it has a damage ratio of 1.90%. Overtopping discharge has values 0.86 lt/s/m for 50 years return period, 5.14 l/s/m for 100 years return period in low water level and 14.58 l/s/m for 100 years return period in high water level condition.
- For Tetrapod Project damage ratio varies between 1.45% and 1.56% in 38 Ton Tetrapod with an average of 1.5%. The damage ratio of berm region varies between 8.72% and 9.00% with the average value of %8.90. Since the damage is high, then the filter layer came out the still water level. Most of the damage occured in the armour units around water level. Since the stability of this alternative is unacceptable, the overtopping analysis is excluded.
- For Tetrapod-Alt-1 damage ratio varies between 1.60% and 1.80% in 38 Ton Tetrapod with an average of 1.70%. The damage ratio of berm region has the average value of %0.40. Overtopping discharge has values 0.57 lt/s/m for 50 years return period, 2.21 l/s/m for 100 years return period in low water level and 5.72

l/s/m for 100 years return period in high water level condition over tetrapod and 10.80 l/s/m below tetrapod.

- For Tetrapod-Alt-2 damage ratio varies between 0.25% and 0.35% in 32.5 Ton Tetrapod with an average of 0.30%. The damage ratio of berm region has the average value of %2.0. the damage ratio of 6-8 ton protection layer behind crest is 2.70 %. Overtopping discharge has values 0.16 lt/s/m for 50 years return period, 1.43 l/s/m for 100 years return period in low water level and 5.61 l/s/m for 100 years return period in high water level condition over tetrapod and 19.26 l/s/m below tetrapod.
- For Tetrapod-Alt-3 damage ratio varies between 0.26% and 0.35% in 32.5 Ton Tetrapod with an average of 0.30%. The damage ratio of berm region has the average value of %1.9. The damage ratio of 8-10 ton rock on crest is 4.90 %. Overtopping discharge has values 0.23 lt/s/m for 50 years return period, 2.34 l/s/m for 100 years return period in low water level and 5.59 l/s/m for 100 years return period in high water level condition over tetrapod and 7.49 l/s/m below tetrapod.
- For Accropode II Alt-1 damage ratio is 0.0% for AccropodeTM II units. The damage ratio of berm region has the average value of %2.3. The damage ratio of 10-12 tons of rock on crest is 0.80 %. Overtopping discharge has values 1.36 lt/s/m for 50 years return period, 5.49 l/s/m for 100 years return period in low water level and 12.68 l/s/m for 100 years return period in high water level condition over accropode and 10.71 l/s/m below accropode.
- For Accropode II Alt-2 damage ratio is 0.0% for AccropodeTM II units. The damage ratio of berm region has the average value of %2.9 varies between 2.70% and3.0%. The damage ratio of 10-12 tons of rock on crest is 0.50 %. Overtopping discharge has values 0.10 lt/s/m for 50 years return period, 1.27 l/s/m for 100 years return period in low water level and 1.72 l/s/m for 100 years return period in high water level condition over accropode and 2.31 l/s/m below accropode.
- For Accropode II Alt-3 section damage ratio is 0.0% for AccropodeTM II units. The damage ratio of berm region has the average value of %2.7 varies between 2.50% and3.0%. The damage ratio of 10-12 tons of rock on crest is 0.60 %. Overtopping discharge has values 0.40 lt/s/m for 50 years return period, 2.31 l/s/m for 100 years return period in low water level and 4.91 l/s/m for 100 years return period in high water level condition over accropode and 7.11 l/s/m below accropode.

Among the tested structure alternatives, „Rock- Alt-1‟, „Rock- Alt-2‟, „Rock- Alt-2-1‟, „Tetrapod-Alt-1‟, „Tetrapod Alt-2‟, „Tetrapod Alt-3‟, „Accropode II Alt-2‟, „Accropode II Alt-3‟ have satisfied stability and overtopping requirements.

- In Table 7.1 all materials used for cross sections are provided. As can be seen Accropode™ II is also the most economical solution for this study. In the future projects Accropode™ II can be considered as a reliable alternative both in stability and economy.

- All the experiments in this study are in hydraulic point of view. Slope stability is also another important issue for deep structures. It should additionally be inspected in numerical or physical modeling.

REFERENCES

Andersen, T.L., Burcharth, H.F., Gironella, X., (2010),"Comparison of new large and small scale overtopping tests for rubble mound breakwaters"

AYGM (2016),"Coastal Structures Planning and Design Manual"

Bruce, T., van der Meer, J.W., Franco, L., J.M. Pearson, J.M., (2008),"Overtopping Performance of Different Armour Units for Rubble Mound Breakwaters"

Franco,L., Geeraerts J., Briganti R., Willems M., Bellotti, G., De Rouck, J., (2008), "Prototype Measurements and Small-scale Model Tests of Wave Overtopping at Shallow Rubble-mound Breakwaters: the Ostia-Rome Yacht Harbour Case"

Geeraerts, J., Kortenhaus, A., González-Escrivá, J.A., De Rouck, J., Troch, P., (2008), "Effects Of New Variables On The Overtopping Discharge At Steep Rubble Mound

Breakwaters — The Zeebrugge Case"

Güler, G.,(2014)," A Comparative Study On The Design Of Rubble Mound Breakwaters"

Güler, G., Arikawa, T., Oei, T., Yalciner, A.C. (2015), "Performance of Rubble Mound Breakwaters Under Tsunami Attack, a case study: Haydarpasa Port, Istanbul, Turkey"

Nørgaard, J. Q. H., Andersen,T. L., Burcharth,H. F. (2013), "Wave Loads on Rubble Mound Breakwater Crown Walls in Deep and Shallow Water Wave Conditions"

Hiraishi, T., Minami, Y., Hasegawa, I. (2007), "Mitigation of Wave Overtopping Rate in Offshore Airport by Permeable Bed Type Seawall" (in Japanese)

Hudson, R. Y. (1959)"Laboratory Investigations of Rubble Mound Breakwaters", J. Waterways & Harbors Division, ASCE, Vol 85, No WW3, Paper No 2171, pp 93-121

OCDI (2009), Technıcal Standards And Commentarıes For Port And Harbour Facılıtıes In Japan

Park, S.K., Dodaran, A. A., Han, C.S., Shahmirzadi, M. E. M. (2014),"Effects of Vertical Wall and Tetrapod Weights On Wave Overtopping in Rubble Mound Breakwaters Under Irregular Wave Conditions"

Plant G.W., Craig S. C. (2012), "Site Preparation for the New Hong Kong International Airport"

Rao, S., Pramod, Ch., Rao, B., (2004),"Stability of berm breakwater with reduced armor stone weight"

Romano, A., Bellotti, G., Briganti, R., Franco, L., (2015), "Uncertainties in the physical modelling of the wave overtopping over a rubble mound breakwater: The role of the seeding number and of the test duration"

Uğurlu, A., Arıkan, Ş.E.,Eğriboyun, O.,Bilen, C. (2016), "Rize-Artvin Airport Wave Transformation Report" (in Turkish)(AYGM)

Uğurlu, A., Arıkan, Ş.E.,Eğriboyun, O.,Bilen, C. (2016), "Rize-Artvin Airport Physical Modelling Report" (in Turkish)(AYGM)

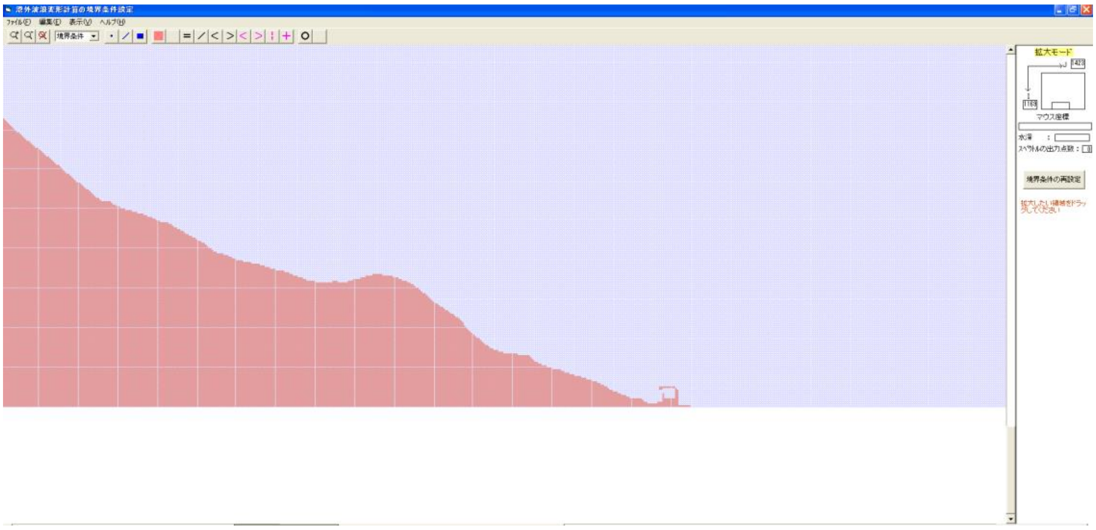
van der Meer, J.W.,(1987), "Stability of Breakwater Armor Layers-Design Formulae"

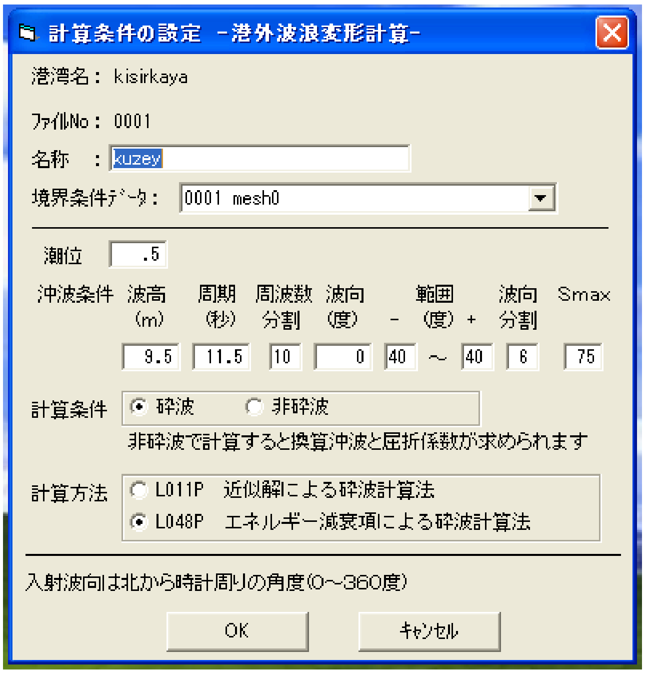
van Gent, M.R.A., (2013),"Rock stability of rubble mound breakwaters with a berm"

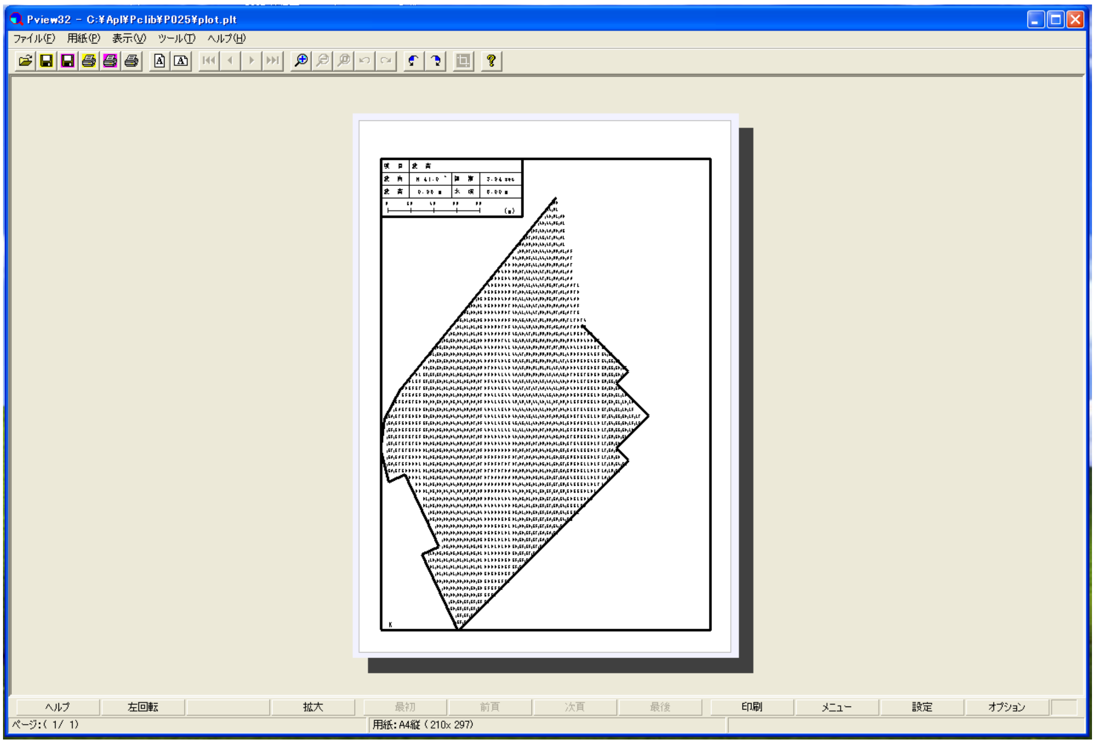
Vidal, C., Medina, R., Lomónaco, P., (2006), "Wave height parameter for damage description of rubble-mound breakwaters"

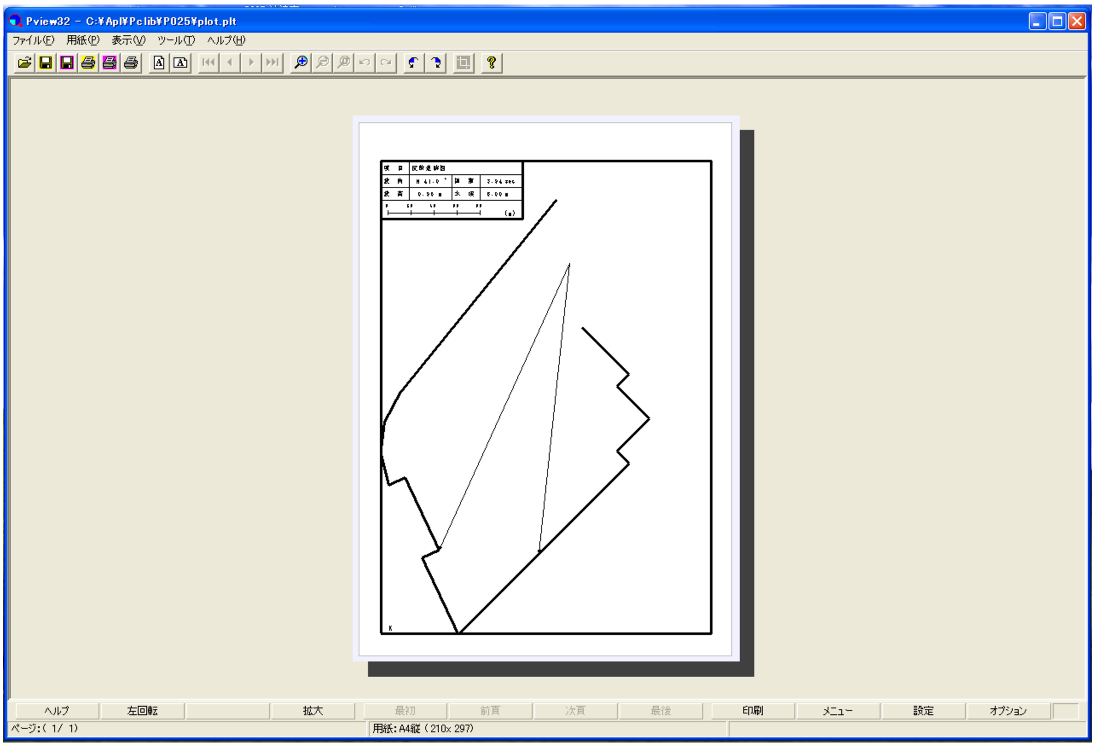
APPENDICES

User Interface Screen Shots






Pview32 - C:¥Apl¥Pclib¥P025¥plot.plt
ファイル(F) 用紙(P) 表示(V) ツール(T) ヘルプ(H)
ヘルプ
左回転
拡大
最初
前頁
次頁
最後
印刷
メニュー
設定
オプション
ページ:(1/ 1)
用紙: A4縦 (210x 297)


Pview32 - C:¥Apl¥Pclib¥P025¥plot.plt
ファイル(F) 用紙(P) 表示(V) ツール(T) ヘルプ(H)
ヘルプ
左回転
拡大
最初
前頁
次頁
最後
印刷
メニュー
設定
オプション
ページ:(1/ 1)
用紙: A4縦 (210x 297)