# A COMPERATIVE ECONOMICAL ANALYSIS ON THE DESIGN OF BERM BREAKWATER: CASE STUDY ON ORDU-GIRESUN AIRPORT BERM BREAKWATER

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**A COMPERATIVE ECONOMICAL ANALYSIS ON THE DESIGN OF BERM BREAKWATER: CASE STUDY ON ORDU-GIRESUN AIRPORT BERM BREAKWATER**

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**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.**

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## ABSTRACT

### A COMPERATIVE ECONOMICAL ANALYSIS ON THE DESIGN OF BERM BREAKWATER: CASE STUDY ON ORDU-GIRESUN AIRPORT BERM BREAKWATER

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Berm breakwaters can be alternatives to the conventional rubble mound breakwaters as they can be designed for high stability, low wave overtopping rates and economic viability. However, fully and partly reshaped berm brekwaters require optimization of design and construction parameters which requires physical model experiments.

In this study, Ordu-Giresun Airport Berm Breakwater and 4 alternative cross-sections of fully and partly reshaped berm breakwaters were tested for stability and wave overtopping. Analyses of all sections were executed to compare their performances. The experiments were conducted in the Coastal and Harbor Engineering Laboratory of the Middle East Technical University, Civil Engineering Department. Models were constructed using different ranges of stone sizes with a model scale of 1:32.86. It was concluded that both fully reshaped and partly reshaped cross sections can be designed to provide the specifications of the project.

In the final part of study, cost analyses of the sections were carried out using two approaches: (a) unit prices listed by the government, (b) the implementation of the contractor. It is shown that construction of the alternative berm breakwaters to Ordu-Giresun Airport that ensures stability and wave overtopping conditions could be more economical.

**Keywords:** Berm Breakwater, Stability, Wave Overtopping, Economical Analysis

## ÖZ

**BASAMAK TİPİ DALGAKIRANLARIN TASARIMI ÜZERİNE KARŞILAŞTIRMALI EKONOMİK ANALİZ: ORDU-GİRESUN HAVALİMANI BASAMAK TİPİ DALGAKIRAN ÖRNEK ÇALIŞMASI**

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Yüksek Lisans, İnşaat Mühendisliği
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Basamak tipi dalgakıranlar denge performansları, düşük dalga aşma miktarı ve ekonomik uygunluk hususları açısından tek eğimli dalgakıranlara alternatif bir yapı tipidir. Kısmen ya da tamamen şekil değiştirebilen basamak tipi yapıların, tasarım ve inşaat parametrelerinin optimizasyona ihtiyacı bulunmaktadır.

Bu çalışmada Ordu-Giresun Havalimanı Basamak Tipi Dalgakıranı ve 4 adet alternatif kesit, denge ve dalga aşma miktarı performansları açısından test edilmiştir. Deneyler, Orta Doğu Teknik Üniversitesi, İnşaat Mühendisliği Bölümü, Kıyı ve Liman Mühendisliği Laboratuvarı'nda yürütülmüştür. Modeller farklı taş sınıfları kullanılarak ve 1: 32.86 model ölçeğiyle oluşturulmuştur.

Çalışmanın son kısmında tüm kesitler için iki ayrı ekonomik analiz yapılmıştır. Birim fiyatları temel alınarak yapılan yaklaşık maliyet hesabı ile yüklenici maliyet hesabı sonuçları kesitler aralarında kıyaslanmıştır. Şartnamelerdeki denge ve dalga aşma miktarlarını sağlayan ve Ordu-Giresun Havalimanı Basamak Tipi Dalgakıran'a alternatif daha ekonomik kesitlerin inşa edilebileceği tespit edilmiştir.

**Anahtar sözcükler:** Basamak Tipi Dalgakıran, Denge performansı, Dalga Aşması, Ekonomik Analiz

***To my beloved nephew Altan Çınar Atak…***

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| | |
|---|---|
| $H_s$ | Incident significant wave height |
| $\Delta$ | Relative buoyant density of the stone |
| $\rho_a$ | Density of stone |
| $\rho_w$ | Density of water |
| $D_{n50}$ | Nominal diameter of the armour stone |
| $M_{50}$ | Median stone mass |
| $S_d$ | Damage parameter |
| Rec | Recession of the berm |
| $T_m$ | Mean wave period |
| $f_g$ | Gradation factor |
| $f_d$ | Depth factor |
| d | Water depth in front of the breakwater |
| $D_{n15}$ | Diameter of the armour stone assuming a 15% cumulative distribution |
| $D_{n85}$ | Diameter of the armour stone assuming a 85% cumulative distribution |
| g | Gravitational acceleration |
| N | Number of waves |
| $h_{br}$ | Berm elevation above still water level |
| B | Berm width |
| $f_{H0}$ | Influence of stability index including wave period |
| $\alpha$ | Front slope angle |

| | |
|---|---|
| $f_N$ | Influence of number of waves |
| $f_{grading}$ | Influence of stone gradation |
| $h_b$ | Height of berm |
| $f_\beta$ | Influence of wave direction |
| $\beta$ | Angle between the wave direction and the breakwater trunk centreline |
| $f_{hb}$ | Influence of berm height |
| $h_s$ | Step height |
| $S_{0m}$ | Wave steepness |
| $h_t$ | Water depth above toe |
| $q$ | Average wave overtopping discharge |
| $\xi_0$ | Breaker parameter |
| $R_c$ | Free crest height above still water line |
| $\gamma_b$ | Influence factor for influence of berm |
| $\gamma_f$ | Influence factor for roughness elements |
| $\gamma_\beta$ | Influence factor for angle of wave attack |
| $\gamma_v$ | Influence factor for vertical wall on slope |
| $S_{op}$ | Peak wave steepness |
| $\gamma_{BB}$ | Influence factor for berm breakwater |
| $\gamma_r$ | Unit weight of stone |
| $\gamma_w$ | Unit weight of water |
| $A_e$ | Eroded area of the section |
| $T_{m-1,0}$ | Spectral wave period at the toe of the structure |

# CHAPTER 1

# INTRODUCTION

Coasts are among the most common used areas and it is expected that more than half of people is going to live on coasts in the future. Economic activities such as industry, transportation, tourism, agriculture, natural resources etc. make coastal areas attractive for people to settle. These types of activities may require coastal defence structures.

Sea walls, breakwaters are constructed as coastal defence structures. Breakwaters are the most widely used and they have different types. Rubble-mound breakwater is more common than reef, detached, floating breakwaters in our country.

Another type of breakwater named as 'berm breakwater' have been constructed for 30 years all over the world. Designing a berm breakwater considers stability, safety, serviceability and economy. At this point model studies gain great importance. Physical model experiments is a good way to show how test sections behave under given wave conditions. Even though the fact that doing experiments is costly and time-consuming, their results are priceless with respect to considerations for designing berm breakwater.

In this study, a newly constructed berm breakwater is physically modelled. The breakwater was tendered for a contract named as 'Ordu-Giresun Airport' by Ministry of Transport, Maritime Affairs and Communications. This study is performed to analyze behaviour of constructed section and then new sections were prepared to achieve the optimum stable, safe, economic and serviceable section. Additionally, economic analysis of all cross sections were performed to determine the most feasable structure.

All experiments were done with same significant wave height and wave period. These parameters were taken from 'Physical Model Experiments od Ordu-Giresun Airport Berm Breakwater Sections Report' prepared by Research Department of Hydraulic Branch Office in Mart 2012. It contains tender section and alternative sections experiments in it.

In Chapter 2, a brief literature survey is given. That includes information about berm breakwaters, necessary formulas and approach to design of berm breakwater. It is useful to comprehend the requirement and theoric procedure to follow.

In Chapter 3, all sections are examined in detail. Cross-sections, experiment set-up, wave characteristics, experiment properties, related pictures, models and prototypes drawings and experimental procedure to follow are given in the detail.

In Chapter 4, results of experiments, figures, graphs, tables are given. Moreover economic analysis for each section by the view of government and a contractor is prepared. Comparison of all the cross sections is discussed.

In Chapter 5, discussion and conclusion by comparing alternative sections to each other is given.

# CHAPTER 2

# LITERATURE SURVEY

## 2.1 Classification of Berm Breakwaters

Berm breakwaters have become popular than conventional rubble mound breakwaters in recent years due to several differences in the design and performance. The major difference is due to stability condition. In spite of the fact that rubble mound breakwater has to be nearly stable for the wave conditions which requies larger armor stones, berm breakwater can be designed to reshape to statically or dynamically stable profile as seen Figure 2.1 below. This condition enables the use of smaller stone sizes that decreases the cost. Additionally, the use of berm decreases the overtopping and run up significantly. The berm breakwaters are classfied into three catagories: (PIANC, 2003)

- Statically stable non-shaped: It is similar to conventional rubble mound breakwater with respect to allowance of stones to move such that only few stones can relocate. This type is called Hardly Reshaping (HR).
- Statically stable reshaped: The profile is allowed to reshape into a new profile but the breakwater and singular stones are stable. It is called Partly Reshaping (PR).
- Dynamically stable reshaped: In this classification, reshaped profile is stable. However, singular stones are unstable, they continue to move. It is called Fully Reshaping (FR).

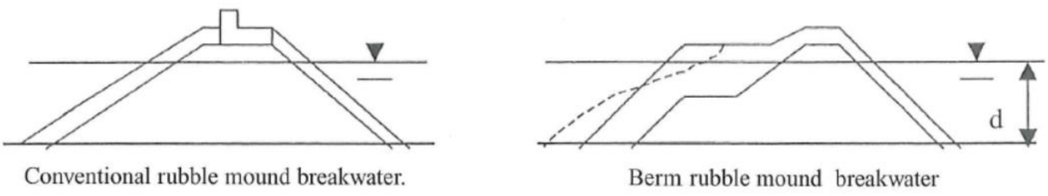


**Figure 2.1** Conventional and Berm rubble mound breakwaters (PIANC, 2003)

Further, berm breakwaters are categorised with respect to structural behaviour (Sigurdarson and Van der Meer, 2012), as shown in Figure 2.2:

- Mass armoured (MA): It consists of mainly one stone class and is homogenous with a wide range.
- Icelandic type (IC): It consists of more than stone classes with narrow grading.

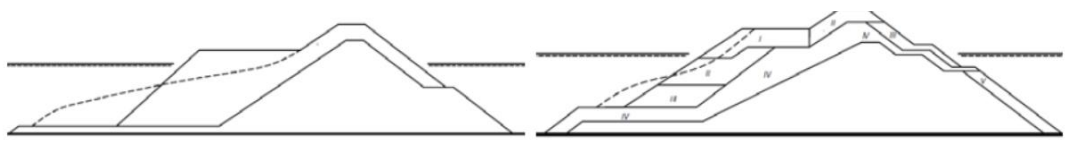

**Figure 2.2** Mass armoured and Icelandic-type berm breakwater (dotted line symbolises expected reshaping profile) (Sigurdarson and van der Meer, 2012)

Thus, the categorizations mentioned above create a new classification of berm breakwaters:

- Hardly reshaping Icelandic-type (HR-IC)
- Partly reshaping Icelandic-type (PR-IC)
- Partly reshaping mass armoured (PR-MA)
- Fully reshaping mass armoured (FR-MA)

## 2.2 Berm Breakwaters around the world

Throughout the world nearly a hundred berm breakwaters have been constructed. Table 2.1 by Sigurdarson et al. (2006) shows the number of constructed berm breakwaters in different countries.

**Table 2.1** List of constructed berm breakwaters (Sigurdarson et al., 2006)

| Country | Number of constructed berm breakwater | The year the construction of the first berm breakwater finished |
|---|---|---|
| Iceland | 29 | 1984 |
| Canada | 5 | 1984 |
| USA | 4 | 1984 |
| Australia | 4 | 1986 |
| Brazil | 2 | 1990 |
| Norway | 6 | 1991 |
| Faroe Islands | 1 | 1992 |
| Iran | 8 | 1996 |
| Madeira | 1 | 1996 |
| China | 1 | 1999 |
| India | 1 | 2003 |
| Denmark | 1 | 2003 |

## 2.3. Fundamental Parameters and Formulas for Design

The governing parameters that are used in design of berm breakwaters according to stability based on 100 years wave conditions are given in Sigurdarson and Van der Meer (2012).

**Table 2.2** Classification of berm breakwaters (Sigurdarson and van der Meer, 2012)

| | Abbreviation | $H_s/\Delta D_{n50}$ | $S_d$ | $R_{ec}/D_{n50}$ |
|---|---|---|---|---|
| Hardly reshaping Icelandic-type berm breakwater | HR-IC | 1.7 – 2.0 | 2 – 8 | 0.5 – 2 |
| Partly reshaping Icelandic-type berm breakwater | PR-IC | 2.0 – 2.5 | 10 - 20 | 1 – 5 |
| Partly reshaping mass armoured berm breakwater | PR-MA | 2.0 – 2.5 | 10 – 20 | 1 – 5 |
| Reshaping mass armoured berm breakwater | FR-MA | 2.5 – 3.0 | -- | 3 - 10 |

In Table 2.2., $H_s/\Delta D_{n50}$ is the stability number, $H_s$ is the incident significant wave height, $\Delta$ is the relative buoyant density of the stone ($\Delta = \frac{\rho_s}{\rho_w} - 1$) , $D_{n50}$ is the nominal diameter of the armour stone ( $D_{n50} = (\frac{M_{50}}{\rho_s})^{1/3}$ ), $S_d$ is the damage parameter and Rec is the recession of the berm.

For wave conditions with smaller return periods the values will be smaller and consequently, for more severe wave conditions, like overload tests, the values may be larger.

### 2.3.1 Recession

Recession is one of the most important design parameters for berm breakwater.

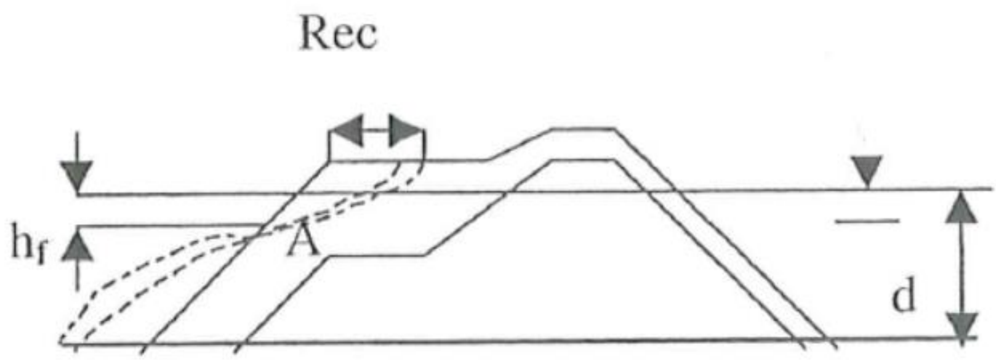


**Figure 2.3** Sketch of recession (PIANC, 2003)

Effect of rounded stones on the reshaping of the berm is founded by Hall and Kao (1991). According to them, the reshaping of a homogenous berm is as in Equation 2.1.

$$\frac{R_e C_{KH}}{D_{50}} = (-10.4 + 0.51\left[\frac{H_s}{(\frac{\rho_s}{\rho_w} - 1)D_{50}}\right]^{2.5}) + 7.52(\frac{D_{85}}{D_{15}}) - 1.07(\frac{D_{85}}{D_{15}})^2 + 6.12P_R \qquad (2.1)$$

D= sieve diameter ≡ 1.2 $D_n$ (Tvinnreim (1981))

$D_{85}$= 85% of the stones have a diameter less than $D_{85}$

$D_{15}$= 15% of the stones have a diameter less than $D_{15}$

$P_R$= percentage per number of rounded stones in the armour

Torum (1998) tested the $R_{ec}/D_{n50}$ which is nondimensional recession in conjunction with $H_oT_o$ for some scale model analyse projects in several labs (Danish Hydraulic Institute (DHI), Denmark, and SINTEF, Norway) on berm breakwaters with a congenerous berm. However, in seperate laboratories recession test results show scatter that Torum could not find any statement for the cause. Torum (1998) fitted quadratic polynomial and afterwards Torum et al (1999) fitted cubic polynomial to input. Onwards Menze (2000) and Torum and Krough (2000) attached terms in order to consider stone gradation and depth of water. The modified recession equation is :

$$\frac{R_{ec}}{D_{n50}} = 0.0000027(H_0T_0)^3 + 0.000009(H_0T_0)^2 + 0.11(H_0T_0) - (-9.9 f_g{}^2 + 23.9 f_g - 10.5) - f_d \tag{2.2}$$

$f_g$= $D_{n85}/D_{n15}$ gradation factor (formula is valid for $1.3<f_g<1.8$)

$H_0T_0$ = wave period stability number

$f_d$ = depth factor

The depth factor for the interval $12.5<d/D_{50}<25$ is

$$f_d = -0.16(\frac{d}{D_{n50}}) + 4.0 \tag{2.3}$$

d= the water depth in front of the berm breakwater.

Moreover, there is another term $h_f$ that is depth where original profile and reshaped profile intersects as shown in Figure 2.3. It is calculated or the interval 12.5< $d/D_{50}$<25 as:

$$\frac{h_f}{D_{n50}} = 0.2\frac{d}{D_{n50}} + 0.5 \tag{2.4}$$

Lykke Andersen (2006) described another way to calculate recession with dimensionless equation as follows:

$$\frac{R_{ec}}{D_{n50}} = f_{h_b} \cdot \left[ \begin{array}{l} \dfrac{(1+c_1)h - c_1 h_s}{h - h_b} \cdot f_N \times f_\beta \times f_{H_0} \times f_{skewness} \times f_{grading} \\ + \dfrac{\cot(\alpha_d) - 1.05}{2 \cdot D_{n50}} \cdot (h_b - h) \end{array} \right] \tag{2.5}$$

$h_b$= height of berm. ($h_b$ is negative when the berm is above the still water line)

h= water depth

$h_s$= step height

$$f_{h_b} = 1.18 \cdot \exp(-1.64\frac{h_b}{H_s}) \text{ for } h_b/H_s>0.1 \tag{2.6}$$

$$f_{h_b} = 1 \text{ for } h_b/H_s<0.1 \tag{2.7}$$

Since $h_b$ is negative when the berm is above the still water level, $f_{h_b}$ is equal to 1.

$$c_1 = 1.2 \tag{2.8}$$

$$f_\beta = \cos(\beta) \tag{2.9}$$

β is angle between breakwater trunk centerline and wave direction.

$$f_N = (N/3000)^{-0.0046 \cdot H_0 + 0.3} \text{ for } H_0<5 \quad (2.10)$$

$$f_N = (N/3000)^{0.07} \text{ for } H_0>5 \quad (2.11)$$

N is number of waves in a given storm.

$$h_s = 0.65 \cdot H_s \cdot s_{om}^{-0.3} \cdot f_N \times f_\beta \quad (2.12)$$

$$f_{H_0} = 19.8 \cdot \exp(-\frac{7.08}{H_0}) \cdot s_{om}^{-0.5} \text{ for } T_0>T_0^* \quad (2.13)$$

$$f_{H_0} = 0.05 \cdot H_0 T_0 + 10.5 \text{ for } T_0<T_0^* \quad (2.14)$$

$$T_0^* = \frac{19.8 \cdot \exp(-\frac{7.08}{H_0}) \cdot s_{om}^{-0.5} - 10.5}{0.05 \cdot H_0} \quad (2.15)$$

$$f_{skewness} = \exp(1.5 \cdot b_1^2) \quad (2.16)$$

$$b_1 = 0.54 \cdot Ur^{0.47} \quad (2.17)$$

$$Ur = \frac{H_s}{2 \cdot h \cdot (k \cdot h)^2} = \frac{H_s \times L_p^2}{8 \cdot \pi^2 \cdot h^3} \quad (2.18)$$

Ur is Ursells number and $L_p$ is the wave length corresponding to the peak period.

$$f_{grading} = 1 \text{ for } f_g \leq 1.5 \quad (2.19)$$

$$f_{grading} = 0.43 f_g + 0.355 \text{ for } 1.5<f_g<2.5 \tag{2.20}$$

$$f_{grading} = 1.43 \text{ for } f_g>2.5 \tag{2.21}$$

Differently from this, Moghim (2009) found another way to determine recession with dimensionless equation that was modified by Moghim et al. (2011) as follows:

$$\begin{aligned}\frac{R_{ec}}{D_{n50}} &= (10.4(H_0\sqrt{T_0})^{0.14} - 13.6)\{1.61 - \exp[-2.2(N/3000)]\} \\ &\times (h_{br}/H_s)^{-0.2}(d/D_{n50})^{0.56}\end{aligned} \tag{2.22}$$

is valid for $H_0\sqrt{T_0} <17$

$$\begin{aligned}\frac{R_{ec}}{D_{n50}} &= (0.089H_0\sqrt{T_0} + 0.49)\{1.61 - \exp[-2.2(N/3000)]\} \\ &\times (h_{br}/H_s)^{-0.2}(d/D_{n50})^{0.56}\end{aligned} \tag{2.23}$$

is valid for $H_0\sqrt{T_0} \geq 17$

In these equations, $h_{br}$ which is the height of the berm above still water level is negative when the berm is below SWL. Besides both of formulas are valid for these intervals:

$7.7< H_0\sqrt{T_0} <24.4$

$500<N<6000$

$0.12< (h_{br}/H_s) <1.24$

$8.0 < (d/D_{n50}) < 16.5$

$0.9 < (d/L) < 0.25$

$1.2 < f_g < 1.5$

Moghim and Alizadeh (2014) arrived at a formula to calculate the berm recession by using the maximum wave momentum flux near the structure toe.

$$\frac{R_{ec}}{D_{n50}} = (2.9N_m - 7.2)\{1.61 - \exp[-2.2(N/3000)]\}(\frac{h_{br}}{H_s})^{-0.2} \tag{2.24}$$

$$N_m = \left(\frac{[(M_F)_{\max}/\gamma_w d^2]}{\Delta}\right)^{1/2} \frac{d}{D_{n50}} \tag{2.25}$$

$N_m$= the stability number for the breakwater based on maximum wave momentum flux.

Lykke Andersen and Burcharth (2010) arrived at a formula considering shape of the reshaped profile with changing water depth, berm elevation and front slope.

$$\frac{R_{ec}}{D_{n50}} = f_{hb}\left[\frac{2.2 \cdot h_t - 1.2 \cdot h_s}{h_t - h_b} \cdot \frac{R_{ec1}}{D_{n50}} + \frac{[\cot(\alpha_d) - 1.05]}{2D_{n50}} \cdot [h_b - h_t]\right] \tag{2.26}$$

Lykke Andersen et al. (2014) indicated that $f_{hb}$ is an extra term that explains berms below SWL. In situation 1, natural slope angle is same with front slope, berm and SWL are at the same level (see Figure 2.4). On the other hand, in situation 2 the recession has to be calculated.

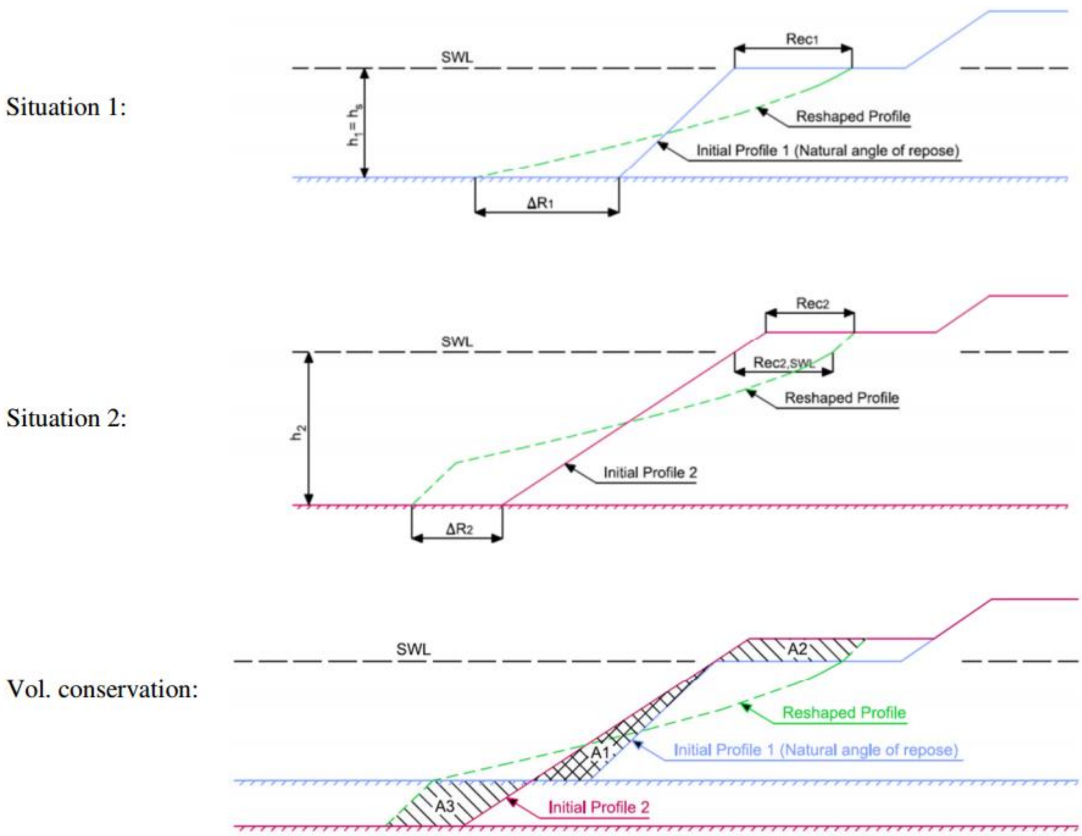


**Figure 2.4** Definition of situation 1 and 2 and areas A1, A2 and A3 (Andersen and Thomas Lykke, 2006)

$$\frac{R_{ec1}}{D_{n50}} = f_{H0} \cdot f_{\beta} \cdot f_{N} \cdot f_{grading} \cdot f_{skewness} \tag{2.27}$$

Lykke Andersen et al. (2014) arrived at a new modified formula of Lykke Andersen and Burcharth (2010). Lykke Andersen et al. (2014) specified that as in the original formula the recession is depending upon on recession in the simplified situation 1, but with new functions for $f_{H0}$ depending upon $H_0\sqrt{T_0}$ as recommended by Moghim et al. (2011). The ultimate modification in proportion to original formula is to revise mistaken calculated deposited volume at the toe in situation 2 in which reshaped profile ends above bottom however it coincides with the original profile (A3 area as defined in Figure 2.4 is incorrect in that case). An upper limit $h_t$ is needed to correct deposited volume calculation.

The formula and related equations that is mentioned is as below:

$$\frac{R_{ec}}{D_{n50}} = f_{hb}\left[\frac{2.2\cdot h_t^* - 1.2\cdot h_s}{h_t^* - h_b}\cdot\frac{R_{ec1}}{D_{n50}} + \frac{[\cot(\alpha_d) - 1.05]}{2D_{n50}}\cdot[h_b - h_t^*]\right] \quad (2.28)$$

$$h_s = 0.65 H_{mo} S_{om}^{-0.3} f_N f_\beta \quad (2.29)$$

$$f_N = \left(\frac{N}{3000}\right)^{\varphi} \quad (2.30)$$

φ = 0.30 for $H_0\sqrt{T_0}$ ≤24

φ = 0.64-0.0143 $H_0\sqrt{T_0}$ for 24< $H_0\sqrt{T_0}$ <40

φ = 0.07 for $H_0\sqrt{T_0}$ ≥40

$f_\beta$= cos(β),β = angle between the wave direction and the breakwater trunk centerline.

$f_{grading}$ = 1 for $f_g$≤1.5 (2.31)

$f_{grading}$ = 0.43$f_g$+0.355 for 1.5<$f_g$≤2.5

$f_{grading}$ = 1.43 for $f_g$>2.5

$f_{H0}$ is minimum of : (2.32)

$$-4.7\cdot 10^{-5}(H_0\sqrt{T_0})^4 + 1.6\cdot 10^{-3}(H_0\sqrt{T_0})^3 + 2.2\cdot 10^{-2}(H_0\sqrt{T_0})^2 + 3.8\cdot 10^{-2} H_0\sqrt{T_0}$$

$$0.429(H_0\sqrt{T_0}) + 12.0$$

$f_{hb} = 1$ for $\dfrac{h_b}{H_{m0}} \leq 0.1$ (2.33)

$f_{hb} = 1.18 \cdot \exp(-1.64 \cdot \dfrac{h_b}{H_{m0}})$ for $\dfrac{h_b}{H_{m0}} > 0.1$

$h_t^* = \min(h_t, \sqrt{\dfrac{2\,\mathrm{Re}\,c_1}{\cot(\alpha_d) - 1.05} \cdot [1.2h_s - 2.2h_{b*}] + h_{b*}^2})$ (2.34)

$h_{b*} = \min(h_b, 0.0)$ (2.35)

$\dfrac{R_{ec1}}{D_{n50}} = f_{H0} \cdot f_{\beta} \cdot f_N \cdot f_{grading}$ (2.36)

$h_s$= step height

$S_{om}$=wave steepness $S_{om} = \dfrac{H_{mo}}{L_o}$

$L_o$= deep water wave length $L_o = \dfrac{gT_{o,1}{}^2}{2\pi}$

$T_{o,1}$= spectral mean wave period $T_o = T_{o,1}\sqrt{\dfrac{g}{D_{n50}{}'}}$

N= number of waves

$h_t$= water depth above toe

$f_g$= stone grading $f_g = \dfrac{D_{n85}}{D_{n15}}$

$D_{n85}$ and $D_{n15}$= 85% and 15% of the stone material has a diameter smaller than $D_{n85}$ and $D_{n15}$.

Note that $h_b$ is negative when the berm is above the still water level.

According to Lykke Andersen and Burcharth (2010), for the first 3000 waves, the equation that is formed based on the data Van der Meer (1998) has to be used.

$$f_N = (\frac{N}{3000})^{-0.046 \cdot H_0 + 0.3} \quad \text{for } H_0<5$$

$$f_N = (\frac{N}{3000})^{0.07} \quad \text{for } H_0>5 \tag{2.37}$$

There is also a terminology named as resilience (P %) which is a way to calculate berm width.

$$B = R_{ec} / (P\% / 100) \tag{2.38}$$

There are ranges as shown in Table 2.4 for chosing P% value and it has to be selected by designer and client for a optimum section.

**Table 2.3** Resiliency ranges for breakwaters (Sigurdarson and van der Meer, 2014)

| | | | |
|---|---|---|---|
| Very resilient | hardly reshaping | IC HR | P% = 10 - 20 % |
| Good resiliency | partly reshaping | IC PR or MA PR | P% = 20 - 40 % |
| Minimum resiliency | fully reshaping | MA FR | P%≤ 70 % |

Moreover, for a minimum berm width;

$$B_{min} = R_{ec} + 1D_{n50} \text{ with a minimum of at least 3 } D_{n50} \tag{2.39}$$

Another important design parameter is horizontal armour width ($A_h$). It is the horizontal distance between the point where front layer touches the water and core material.

$$\frac{A_h}{H_s} = 2\frac{H_s}{\Delta D_{n50}} \qquad (2.40)$$

For hardly reshaping structures $A_h$ =3.4 to 4.0 $H_s$, partly reshaping structures $A_h$ =4.0 to 5.0 $H_s$, fully reshaping structures $A_h$ =5.0 to 6.0 $H_s$.

### 2.3.2 Wave Overtopping

Wave overtopping has always been considered as a fundamental factor for design of defense coastal structures. It is the mean discharge consisting of the waves that succeed to exceed crest level and is calculated $m^3$/s per m.

Van der Meer and Jansen (1995) arrived at a formula for wave overtopping as below:

For breaking waves;

Average discharge is $Q_b = 0.06\exp(-5.2R_b)$ (2.41)

Recommended discharge is $Q_b = 0.06\exp(-4.7R_b)$ (2.42)

$$Q_b = \frac{q}{\sqrt{gH_s^3}}\sqrt{\frac{s_{op}}{\tan\alpha}} \qquad (2.43)$$

$$R_b = \frac{R_c\sqrt{s_{op}}}{H_s\tan\alpha}\frac{1}{\gamma_b\gamma_d\gamma_f\gamma_\beta} \qquad (2.44)$$

For non-breaking waves;

Average discharge is $Q_n = 0.2\exp(-2.6R_n)$ (2.45)

Recommended discharge is $Q_n = 0.2\exp(-2.3R_n)$ (2.46)

$$Q_n = \frac{q}{\sqrt{gH_s^3}} \tag{2.47}$$

$$R_n = \frac{R_c\sqrt{s_{op}}}{H_s \tan\alpha}\frac{1}{\gamma_b\gamma_d\gamma_f\gamma_\beta} \tag{2.48}$$

$\gamma_b$= reduction factor taking into account a stepped slope

$\gamma_d$= depth reduction factor = $1-0.03(4-d/H_s)^2$ for $d/H_s<4$

= 1 for $d/H_s>4$

$\gamma_f$= friction reduction factor

$\gamma_\beta$= reduction factor for oblique wave attack = 1-0.0033β

In TAW (2002), wave overtopping calculation is presented in two formulae and they are intercorrelated. The first one is for breaking waves and second one is for the maximum that is achieved for non-breaking waves.

$$\frac{q}{\sqrt{gH_{m0}{}^3}} = \frac{0.067}{\sqrt{\tan\alpha}}\gamma_b\cdot\xi_0\cdot\exp(-4.3\frac{R_c}{H_{m0}}\frac{1}{\xi_0\cdot\gamma_b\cdot\gamma_f\cdot\gamma_\beta\cdot\gamma_v}) \tag{2.49}$$

and a maximum of : $\frac{q}{\sqrt{gH_{m0}{}^3}} = 0.2\cdot\exp(-2.3\frac{R_c}{H_{m0}}\frac{1}{\gamma_f\cdot\gamma_\beta})$ (2.50) q= average wave overtopping discharge

g= acceleration due to gravity

$H_{m0}$= significant wave height at toe of dike

$\xi_0$ = breaker parameter

$s_0$= wave steepness

$T_{m-1,0}$= spectral wave period at toe of dike

tanα= slope

$R_c$= free crest height above still water line

γ= influence factors for influence of berm, roughness elements, angle of wave attack, and vertical wall on slope.

In EurOtop Manual (2007), an overtopping formula of Lykke Andersen (2006) appears. It is used for berm breakwaters; however it has difficulty in implementation. Moreover the formula represents that wave period has an important effect on overtopping. Afterwards, Sigurdarson and Van der Meer (2013) arrived at a new formula for overtopping including a new term that is called as influence factor $\gamma_{BB}$.

$$\frac{q}{\sqrt{g \cdot H_{m0}^3}} = 0.2 \cdot \exp\left(-2.6 \frac{R_c}{H_{m0} \cdot \gamma_{BB} \cdot \gamma_\beta}\right) \tag{2.51}$$

$$\gamma_{BB} = 0.68 - 4.5 s_{op} - 0.05 B / H_s \text{ for HR and PR} \tag{2.52}$$

$$\gamma_{BB} = 0.70 - 9.0 s_{op} \text{ for FR}$$

$B/H_S$ are valid for a 100 years return period.

It is seen that for fully reshaping there is no term including ‘B’. This is sensible because for fully reshaping berm breakwaters, berm will disappear.

Clash (Verhaeghe, 2005) that is a neural network method taking place in literature is used for estimating wave overtopping discharges of coastal structures.

According to Hydralab III (2007), neural network modelling is compulsory to acquire real-like discharges. On the occasion of excessive parameters that impact on wave overtopping, it is not easy to define the effects of related parameters. By the virtue of Clash, a user can utilize many parameters as inputs as shown in figure below and that advantage ensures Clash to be one step ahead of other methods with respect to reliability.

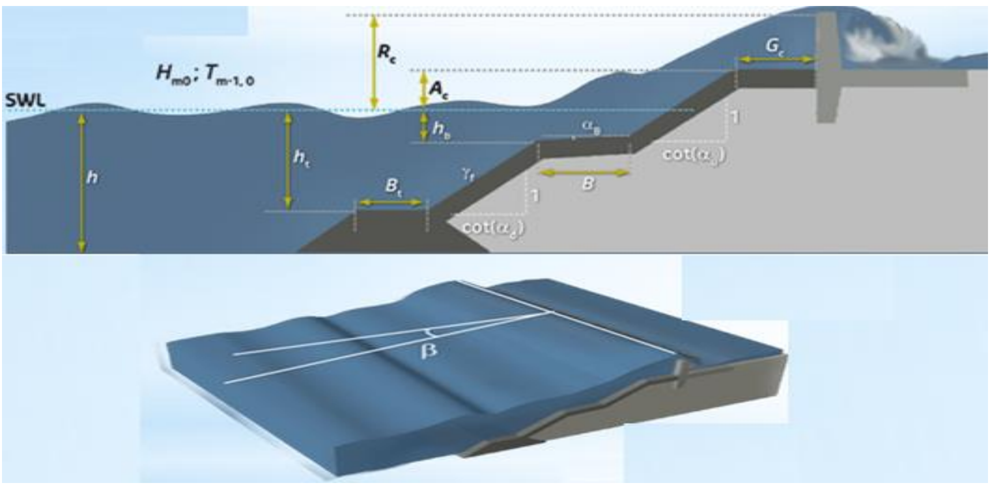


**Figure 2.5** Neural network prediction of wave overtopping at coastal structures. (Verhaeghe, H., 2005.)

β (degree) = angle of wave attack

h (m) = water depth in front of structure

$H_{m0}$ (m) = significant wave height at the toe of the structure

$T_{m-1,0}$ (s) = spectral wave period at the toe of the structure

$h_t$ (m) = water depth at toe of the structure

$B_t$ (m) = width of the toe

$\gamma_f$ = roughness coefficient

cot ($\alpha_d$) = angle of the down slope

cot ($\alpha_u$) = angle of the upper slope

$R_c$ (m) = crest freeboard relative to SWL

B (m) = berm width

$h_b$ (m)= water depth on the berm

tan($\alpha_B$) = berm slope

$A_c$ (m) = armour freeboard relative to SWL

$G_c$ (m) = armour width

# CHAPTER 3

# MODEL STUDIES

Hughes (1993) defined physical model as a physical system which is derived usually at a reduced size regarding principal dominant forces present on the system are available in the model in correct proportion to the actual physical system. There are some advantages and disadvantages of using physical model. The advantages of physical model studies are:

- It can be selected to get results in order to confirm or contradict theoretical results.
- It can be selected to get results for complex cases that are difficult to be explained by using theoritical approaches.
- It can be selected to get results for any case that is hard to obtain data in field.
- It gives a chance to make direct observation into phenomena that is not identified or understood.
- There is less need to make assumptions compared to analytical or numerical models.

Le Mehaute (1990) specified six causes supporting the selection of physical model studies in coastal engineering field as below:

- Model studies are cost-efficient method when compared to prototype projects.
- Natural limits of deterministic fluid mechanics due to turbulence give chance to model studies to be one of the most useful tools.

- By the help of the new techniques, model studies show better performance.
- Model studies provide opportunity to observe and measure the physics in a controlled environment.
- Scale models allow renewal of complex boundary conditions beyond the accuracy of finite step differences.
- In contradistinction to computer or numerical model, physical model may create instinctive behaviour in order to find new engineering approaches.

On the other hand, Hughes (1993) indicated disadvantages of physical model studies as follows:

- If all relevant variables in prototype is not compatible with model, scale effects ocur. The widespread for this situation in coastal models is viscous forces that are proportionally larger in scale model than in prototype.
- Laboratory effects are one of the major disadvantages of physical model studies. One of the widely effect occurs when unilateral waves are generated in the model to approximate lateral waves that arise in nature.
- In model studies, some boundary conditions or forces in nature may not be applied so it is important to consider deficient factors in models.
- When compared to analytical and numerical models, physical models are more costly.

### 3.1 Model Scale

Hughes (1993) remarked that ‘Scale selection for all models of coastal defense structures involves a compromise between the desire to model at as large as possible to avoid potential scale effects and the economics of conducting tests at smaller scales.’. While model scale is determined, many parameters should have to be taken into consideration. These are facility availability, interest area when it is compared to facilty dimensions, armour unit size and availability of stone, past experiences with physical models of analog nature, level of affordability, wave generator capacity, design wave conditions and water levels.

There are many scale model laws e.g. Froude, Reynolds, Weber, Mach, Cauchy, Richardson, Euler and Strouhal number. Hughes (1993) indicated that Froude Model Law is mostly used for scaling of hydraulic models in coastal engineering.

$$F_r = u^2 / gd \tag{3.1}$$

u= velocity of a water particle

g= gravitational acceleration

d= water depth

$F_r$= Froude number

Froude number of prototype and model has to be equal to each other in this law.

$$(F_r)_p = (F_r)_m \tag{3.2}$$

$(F_r)_p$= froude number of prototype

$(F_r)_m$= froude number of model

Accordingly, model scale factors are calculated as follows. Length scale of the model is:

$$\lambda_L = L_m / L_p \tag{3.3}$$

$$\lambda_L = 1 : 32.8625$$

Time scale of the model is :

$$\lambda_T = (\lambda_L)^{1/2} \tag{3.4}$$

$\lambda_T =$ 1:5.7326

Weight scale of the model is :

$$\lambda_w = \lambda_L{}^3 \frac{(\gamma_s)_m}{(\gamma_s)_p} \left[ \frac{\dfrac{(\gamma_s)_p}{(\gamma_w)_p} - 1}{\dfrac{(\gamma_s)_m}{(\gamma_w)_m} - 1} \right]^3 \tag{3.5}$$

$\lambda_w = 1:50002.0$

Where $\gamma_s$ is specific weight of the stones and $\gamma_w$ is specific weight of water.

Using the model scale factors, values of the main parameters of the study for model and prototype are given in Table 3.1 and Table 3.2.

**Table 3.1** Values of parameters in prototype and model

| | Prototype | | | | Model | | |
|---|---|---|---|---|---|---|---|
| **Incident significant wave height** | $H_s$ | = | 6.680 m | | $H_s$ | = | 0.203 m |
| **Spectral wave period** | $T_{m-1,0}$ | = | 10.31 s | | $T_{m-1,0}$ | = | 1.798 s |
| **Water depth at toe** | $h_{toe}$ | = | 12.120 m | | $h_{toe}$ | = | 0.369 m |
| **Stone unit weight** | $g_s$ | = | 2.550 t/m³ | | $g_s$ | = | 2.700 t/m³ |
| **Water unit weight** | $g_w$ | = | 1.025 t/m³ | | $g_w$ | = | 1.000 t/m³ |

**Table 3.2** Scaled values of stone class intervals

| Range | Prototype Weight | Prototype Diameter | Model Weight | Model Diameter |
|---|---|---|---|---|
| | **$W_{15}$-$W_{50}$-$W_{85}$** | **$D_{15}$-$D_{50}$-$D_{85}$** | **$W_{15}$-$W_{50}$-$W_{85}$** | **$D_{15}$-$D_{50}$-$D_{85}$** |
| (0.4-2) tons | 0.4 tons | 0.539 m | 8.0 grams | 0.014 m |
| | **1.2 tons** | **0.778 m** | **24.0 grams** | **0.021 m** |
| | 2.0 tons | 0.922 m | 40.0 grams | 0.025 m |
| (2-4) tons | 2.0 tons | 0.922 m | 40.0 grams | 0.025 m |
| | **3.0 tons** | **1.056 m** | **60.0 grams** | **0.028 m** |
| | 4.0 tons | 1.162 m | 80.0 grams | 0.031 m |
| (4-6) tons | 4.0 tons | 1.162 m | 80.0 grams | 0.031 m |
| | **5.0 tons** | **1.252 m** | **100.0 grams** | **0.033 m** |
| | 6.0 tons | 1.330 m | 120.0 grams | 0.035 m |
| (6-8) tons | 6.0 tons | 1.330 m | 120.0 grams | 0.035 m |
| | **7.0 tons** | **1.400 m** | **140.0 grams** | **0.037 m** |
| | 8.0 tons | 1.464 m | 160.0 grams | 0.039 m |
| (8-10) tons | 8.0 tons | 1.464 m | 160.0 grams | 0.039 m |
| | **9.0 tons** | **1.523 m** | **180.0 grams** | **0.041 m** |
| | 10.0 tons | 1.577 m | 200.0 grams | 0.042 m |
| (2-6) tons | 2.0 tons | 0.922 m | 40.0 grams | 0.025 m |
| | **4.0 tons** | **1.162 m** | **80.0 grams** | **0.031 m** |
| | 6.0 tons | 1.330 m | 120.0 grams | 0.035 m |
| (2-8) tons | 2.0 tons | 0.922 m | 40.0 grams | 0.025 m |
| | **5.0 tons** | **1.252 m** | **100.0 grams** | **0.033 m** |
| | 8.0 tons | 1.464 m | 160.0 grams | 0.039 m |
| (4-10) tons | 4.0 tons | 1.162 m | 80.0 grams | 0.031 m |
| | **7.0 tons** | **1.400 m** | **140.0 grams** | **0.037 m** |
| | 10.0 tons | 1.577 m | 200.0 grams | 0.042 m |
| (6-10) tons | 6.0 tons | 1.330 m | 120.0 grams | 0.035 m |
| | **8.0 tons** | **1.464 m** | **160.0 grams** | **0.039 m** |
| | 10.0 tons | 1.577 m | 200.0 grams | 0.042 m |
| (12-15) tons | 12.0 tons | 1.676 m | 240.0 grams | 0.045 m |
| | **13.5 tons** | **1.743 m** | **270.0 grams** | **0.046 m** |
| | 15.0 tons | 1.805 m | 300.0 grams | 0.048 m |

## 3.2 Experimental Set-up

Physical model experiments of stability and overtopping of berm breakwater of Ordu-Giresun Airport and alternative cross section designs were performed in the wave flume of METU Department of Coastal and Harbor Engineering Laboratory. The flume's length, width and depth dimensions are 28.80 meters, 6.20 meters and

1.00 meters respectively. Inside the wave flume, there is also an inner channel with dimensions 18.00 meters, 1.50 meters and 1.00 meters. Inner channel is made of plexiglass walls and inside the channel is not affected by wave reflection due to side walls of wave flume. Throughout all experiments, inner channel is also divided into two using plywoods in order to reduce the cross section area and minimise the amount of stone materials needed for physical models.

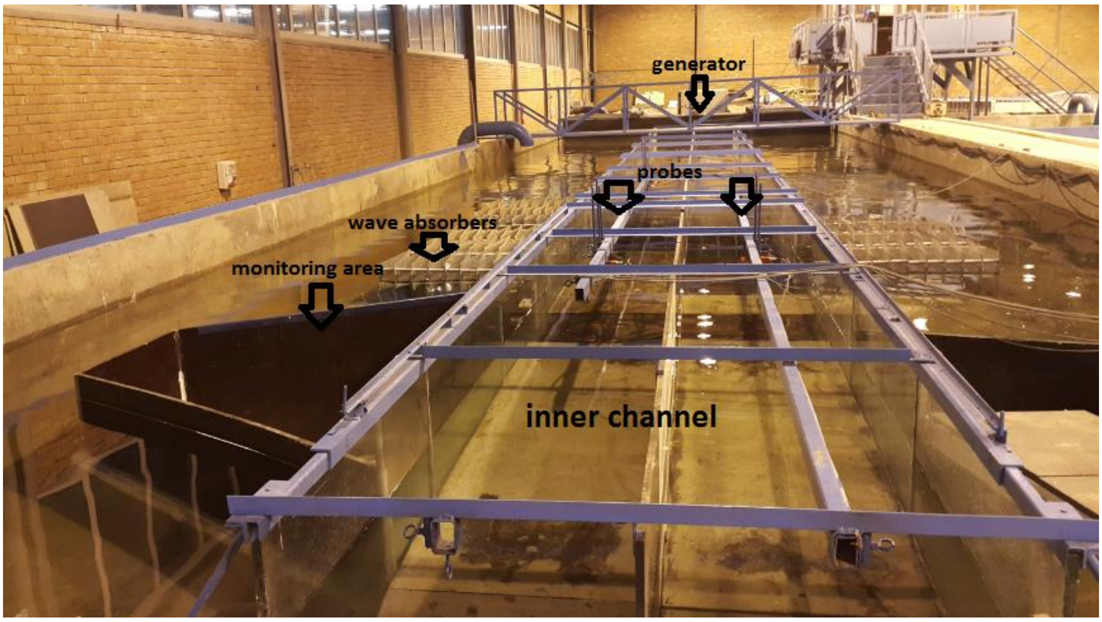


**Figure 3.1** Layout of wave flume

Piston type wave generator system has three stage which are power pack, computer with a software and hydraulic servo actuator. Power pack, DHI Hydraulic Power Pack type 301/22-PM, is the energiser so that movement of paddles can be ensured. Software, DHI Wave Synthesizer, is necessary to transform digital wave data to analog signals. Lastly, servo actuator is assgined to transmit the analog signals to the piston.

Eleven wave gauge probes that are formed from two thin stainless steel electrodes are placed along the wave flume as shown in Figure 3.2. These probes are of DHI type 202 and used for water volume conductivity measurements. Voltage differences caused by waves during the experiment are measured by probes and these data are transferred to a computer. A Matlab code is used with the calibration factor adjustment in order to determine the water levels. The calibration procedure was done at the beginning of each test by recording data at three different water levels (above still water level, at still water level and below still water level).

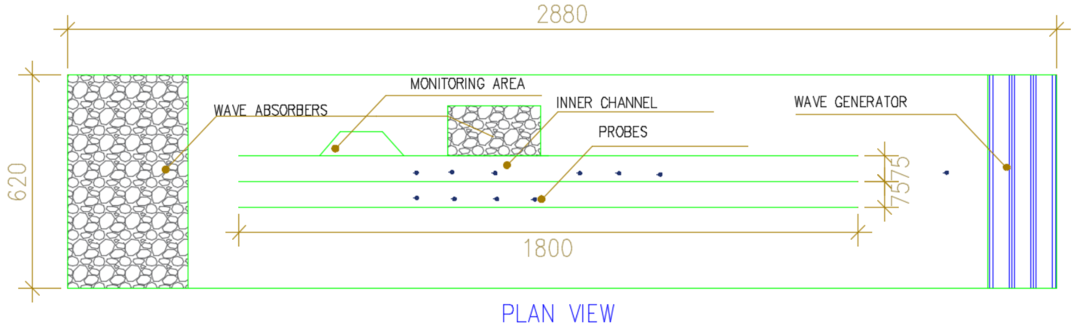


**Figure 3.2** Layout of wave channel (top view)

(Dimensions are in centimeters and figure is not to scale.)

There are also wave absorbers at the end of flume between the inner channel and wave flume. However, it is nearly impossible to prevent reflection of waves by the help of wave absorbers in laboratory conditions. Thus, a Matlab code using Goda and Suzuki (1976) method is used to find the realistic wave characteristics with respect to reflection analysis. Moreover, wave absorbers were needed near the inner channel in where waves with a high energy had capability of climbing over the monitoring area.

A 1:20 slope is placed in the wave channel. The length of plywood in x-direction is 4.10 meter. In wave channel water depth at toe is 37 cm and due to the 1:20 slope, water depth in wave flume is 57.50 cm.

### 3.2.1 Wave Generation and Analysis

Design wave height and period for the study is taken from Ordu-Giresun Airport Berm Breakwater Stability Calculations Final Report prepared by Yuksel Project. In reference to document, spectral specific wave height with a 100-year recurrence period that is calculated considering high water level at construction face is 6.68 m. and specific wave period is 10.80 sec. In that document high water level is determined as 1.12 m and still water level is 11.0 m.

In order to generate the wave climate of the study region, wave generator requires a time series paddle position. First, a target frequency spectrum is selected as Bretschneider-Mitsuyasu type to generate model irregular waves.

Bretschneider-Mitsuyasu type spectrum:

$$S(f) = 0.257 H_{1/3}{}^{2} T_{1/3}{}^{-4} f^{-5} \exp\left\{-1.03 (T_{1/3} f)^{-4}\right\} \tag{3.6}$$

Where

$H_{1/3}$= significant wave height

$T_{1/3}$= significant wave period

To ensure that the design wave characteristics are reproduced in the wave flume, several tests were performed in empty channels (no cross section) with the sea bottom slope only. After 36 empty channel wave tests, 3 different wave series which have the same specific wave characteristics have been obtained. They are all obtained at the onshore end of bottom slope, where the toe of the structure will be located. These are shown in Table 3.3.

**Table 3.3** Wave Series

| | **$H_s$ (m)** | **$T_s$ (sec)** |
|---|---|---|
| **1** | 8.53 | 11.00 |
| **2** | 8.70 | 11.00 |
| **3** | 8.93 | 10.80 |

All 3 wave series are used for each cross-section. First and second experiment for each cross-section is done for 1000 wave series. Then, the third experiment is done for 10000 wave series continiuing the wave series of the second set.

### 3.2.2 Profile Measurements

Profile measurements for damage parameter calculations are ensured by laser metre along three lines of cross-section at 5 cm intervals for Section 1 and 2 cm intervals for the other sections. Profile measurements are done after and before 1000 waves for the first and second sets of experiments for each cross section tested as well as after and before 1000, 3000, 5000, 7000 and 10000 waves for the third set of experiments.

Profile measurements after and before wave series is the method that Hudson (1959) used to find out the proportion volume of eroded stones to volume of total stones on layer. In Coastal Engineering Manual (2003), a dimensionless damage parameter, $S_d$, based on Broderick (1983) and introduced by Van der Meer (1988b) as follows:

$$S_d = \frac{A_e}{{D_{n50}}^2}$$

(3.8)

Where

$A_e$= cross sectional eroded area ($m^2$)

$D_{n50}$= nominal diameter of armourstone size (m)

$S_d$ is an important identification for physical model experiments to obtain the damage level of the structure. In the calculations, damage parameter is obtained by averaging the damage parameters of all three lines. In Figure 3.3 there is a schematic view of eroded area which occurs as a result of damage.

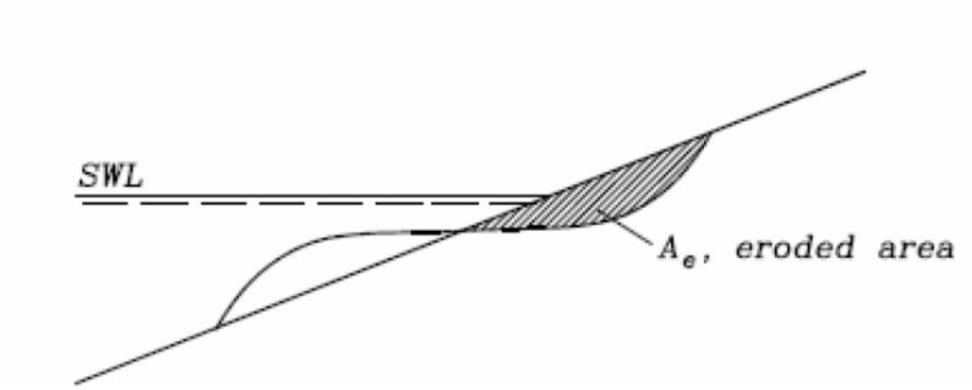


**Figure 3.3** Schematic view of eroded area (CEM, 2003)

### 3.2.3 Overtopping Measurements

Wave overtopping quantity is acquired by putting a conduit on the crest of breakwater and an overtopping tank is placed behing the model. The conduit over the crest line acts like a catch drain and the overtopping quantity is accumulated in the overtopping tank. After each 500 waves, water in the overtopping tank is weighed on precision scales and the data is converted to overtopped discharge for prototype.

## 3.3 Construction of Models

The foremost aim for testing alternative models is to assess the economic benefit of different alternatives. Reduced dimensions of cross-section and stone class are one way to design berm breakwaters under same wave conditions. At the same time, damage and wave overtopping criteria (<10lt/s) have to be procured strictly in the alternative cross-section designs.

In this study, there are 5 different cross-sections. In all cases,

- Filter consists of 0.0-0.4 ton.
- Core material consists of 0.4-2.0 ton.

- Layout in cross-sections for filter and core is all similar.
- Toe usage is preferred.
- Crest elevation is 7.50 m.
- Berm elevation is 3.00 m.
- Berm is unsubmerged.
- Face slope of berm breakwater is 1/1.5 except Section 1 that has a face slope of 1/3.

Alternative cross section designs were developed by selecting stone ranges to be used in models. Recession is calculated by using the formula 2.36. Division of recession value to median stone diameter is necessary to specify classification of section in Table 2.3. Formula 2.38 and Table 2.4 is used to find berm width and it has to be higher than the value calculated from Formula 2.39. Lastly, horizontal armour width is found by using Formula 2.40. Except for Section 2, in all sections horizontal armour width distances are ensured.

Stones from quarry were taken for model studies to laboratuary and all of them were classified according to their weights and diameters. After finalising classification as in Figure 3.4, they were painted in different colours in order to distinguish stone classes during and after the experiments as shown in Figure 3.5.

**Figure 3.4** Sieving of stones

**Figure 3.5** Painted stones according to their range

**Table 3.4** Colour choice for stone classification

| Range | Model Weight | Colour |
|---|---|---|
| 2-4 tons | 40-80 grams | Yellow |
| 4-6 tons | 80-120 grams | Green |
| 6-8 tons | 120-160 grams | Brown |
| 8-10 tons | 160-200 grams | Blue |
| 12-15 tons | 240-300 grams | Grey |

### 3.3.1 Section 1

Section 1 is the primary model for all studies and it is based on the implemented design of Ordu-Giresun Airport Berm Breakwater. By improving and modifying the characteristics of Section 1 with the intent of more economical section, other sections were acquired.

Section 1 can be classified as an Icelandic-type berm breakwater and hardly reshaping. Berm length is 15 m. Model stone weights of each layer in Section 1 are

0-8 grams for core layer, 8-24 grams for filter layer, 40-80 grams for first armour layer, 120-160 grams, 160-200 grams and 240-300 grams for second armour layer as can be seen in Figure 3.8.

**Figure 3.6** Top view of Section 1

**Figure 3.7** Side view of Section 1

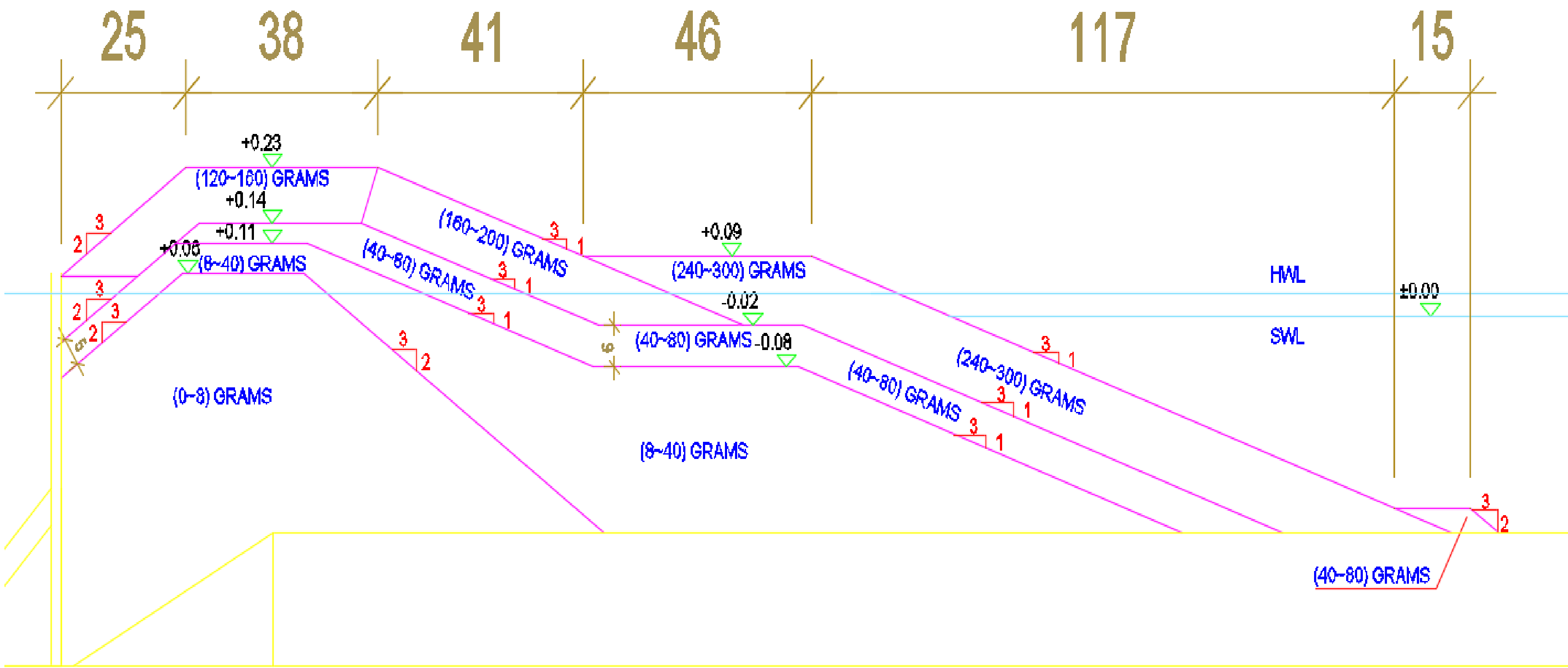


**Figure 3.8** Cross section of Section 1 (Model values are in centimetres and not scaled)

### 3.3.2 Section 2

Section 2 is designed to provide the minimum cross section area as well as the stone size. It is mass-armoured and fully reshaping with berm length of 35 cm and model weight of 40-80 grams stone material as can be seen in Figure 3.9.

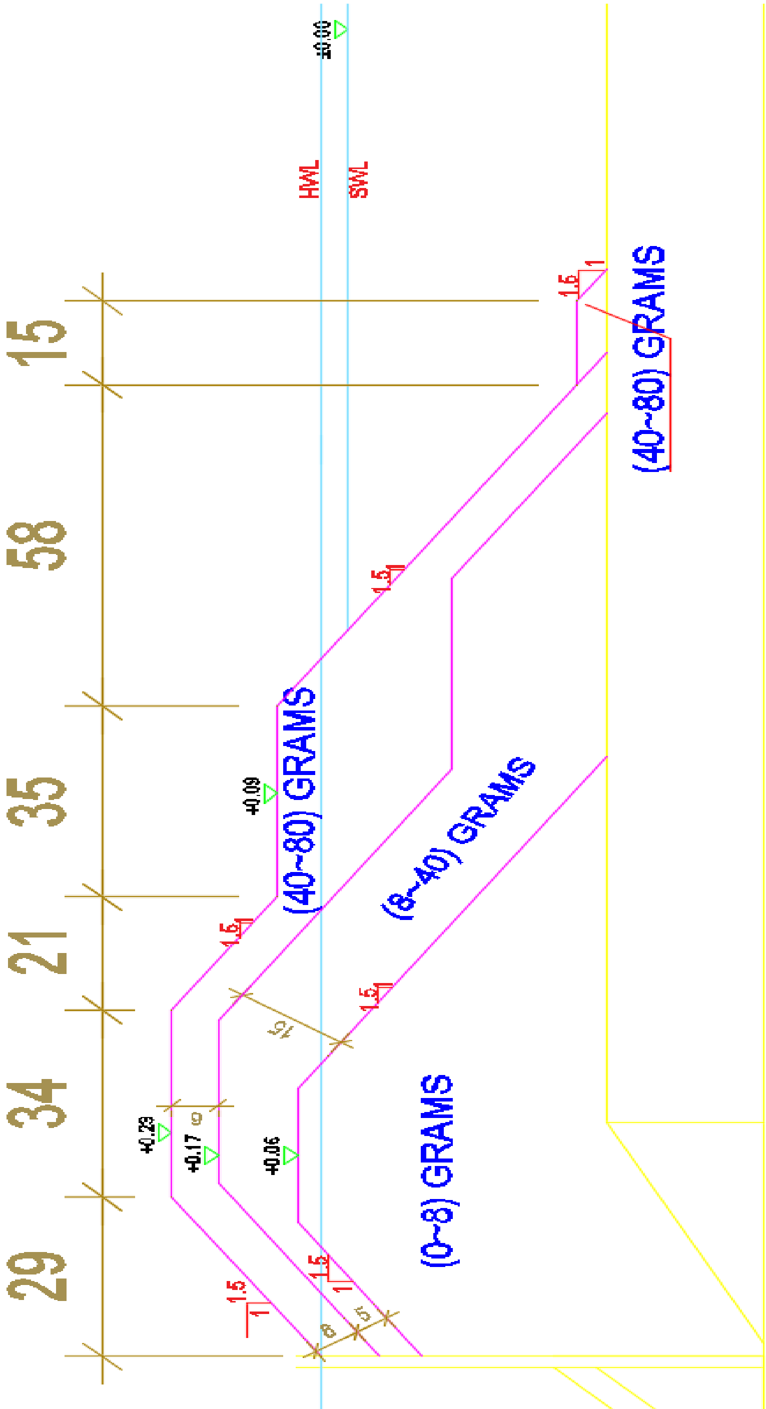


**Figure 3.9** Cross section of Section 2 (Model values are in centimetres and not scaled)

### 3.3.3 Section 3

Section 3 is an alternative to Section 2 where longer berm length is used in the event that Section 2 may not meet the standards in terms of overtopping and stability. Berm length is 79 cm. It is mass-armoured and fully reshaping. Armour layer consists of 40-80 gram stone material as seen in Figure 3.10. The possible effect of length of berm on damage and overtopping conditions are expected to be discussed by comparing Section 2 and Section 3.

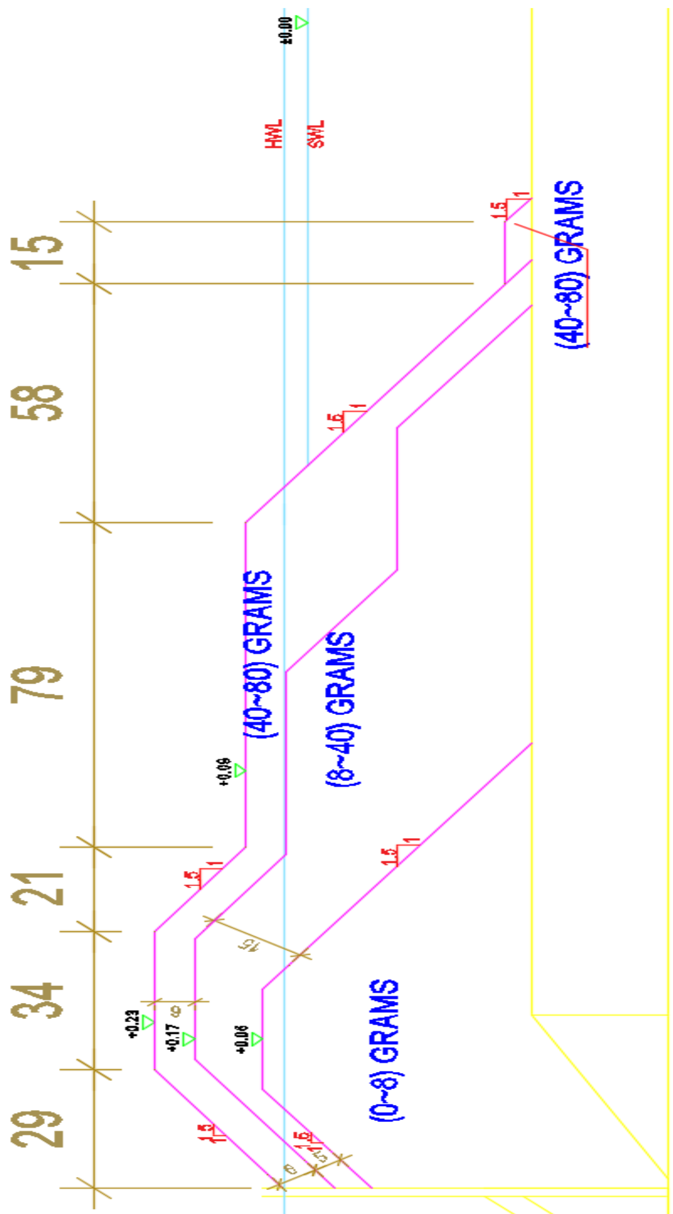


**Figure 3.10** Cross section of Section 3 (Model values are in centimetres and not scaled)

### 3.3.4 Section 4

Section 4 is planned as a section more reliable than Section 2 and Section 3 but expected to be more economic than Section 1. Wide stone range is preferred.

Berm length is again 79 cm. It is mass armoured and fully reshaping. Armour layer consist of 40-160 grams stone material as seen in Figure 3.11.

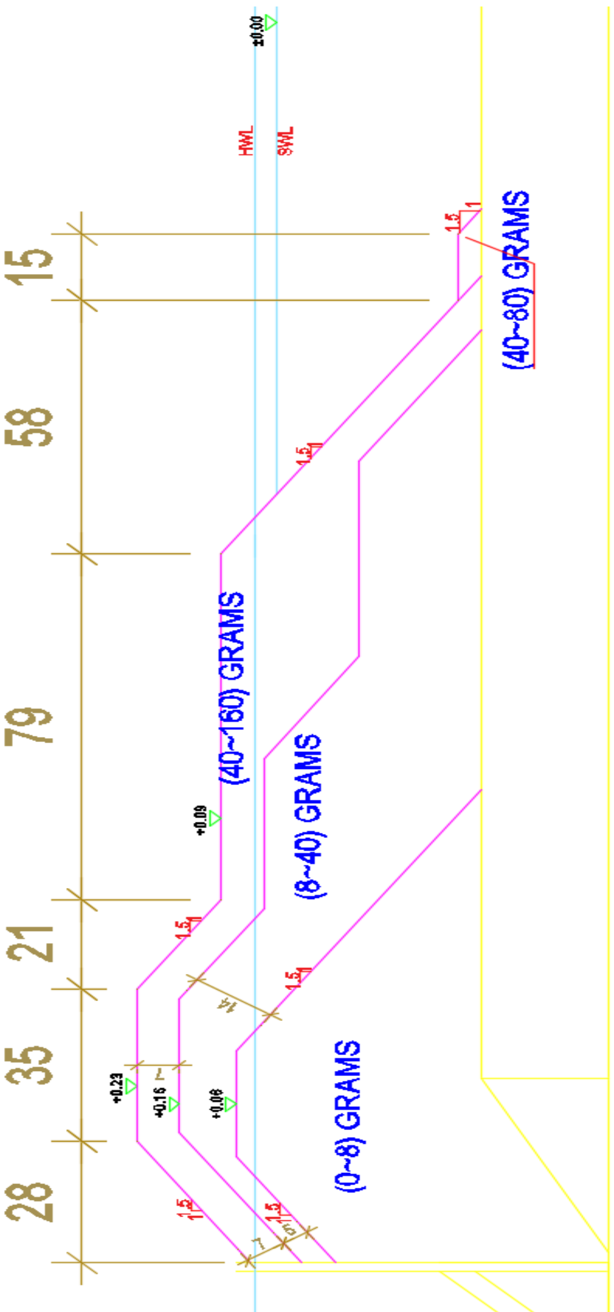


**Figure 3.11** Cross section of Section 4 (Model values are in centimetres and not scaled)

### 3.3.5 Section 5

Section 5 was intended as an alternative to Section 4 with respect to enlargement of stone range size.

Berm length is again 79 cm. It is mass armoured and partly reshaping. Armour layer is consisting of 120-200 grams stone material as seen in Figure 3.12. The possible effect of stone range enlargement on damage and overtopping conditions are expected to be discussed by comparing Section 4 and Section 5.

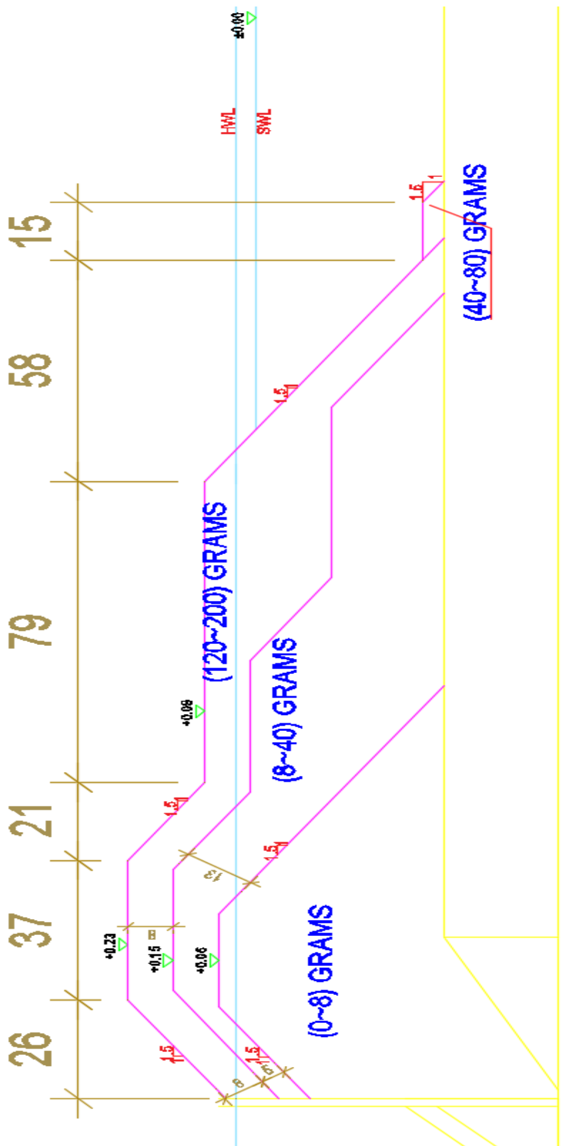


**Figure 3.12** Cross section of Section 5 (Model values are in centimetres and not scaled)

# CHAPTER 4

# EXPERIMENTS AND RESULTS

In this chapter, experiments conducted for each section along with results are presented. The research work consists of 3 main sections; results of physical model experiments on stability by damage parameter and recession, wave overtopping measurements and economical analysis of each cross-section tested.

## 4.1 Stability of Tested Cross-Sections

### 4.1.1 Section 1

Experiments were conducted between March 16 and March 25, 2016. Cross-section of Section 1 is presented in chapter 3.5.1. Three different experiment sets were performed with 1000 waves, 1000 waves and 10000 waves respectively under wave conditions discussed in Chapter 3. After every experiment set, the cross-section was reconstructed to the original cross-section.

The views of Section 1 before experiment sets and after set 1, set 2 and set 3 are demonstrated in Figures 4.1, 4.2, 4.3, 4.4, 4.5, 4.6 and 4.7.

**Figure 4.1** View of Section 1 before experiment set

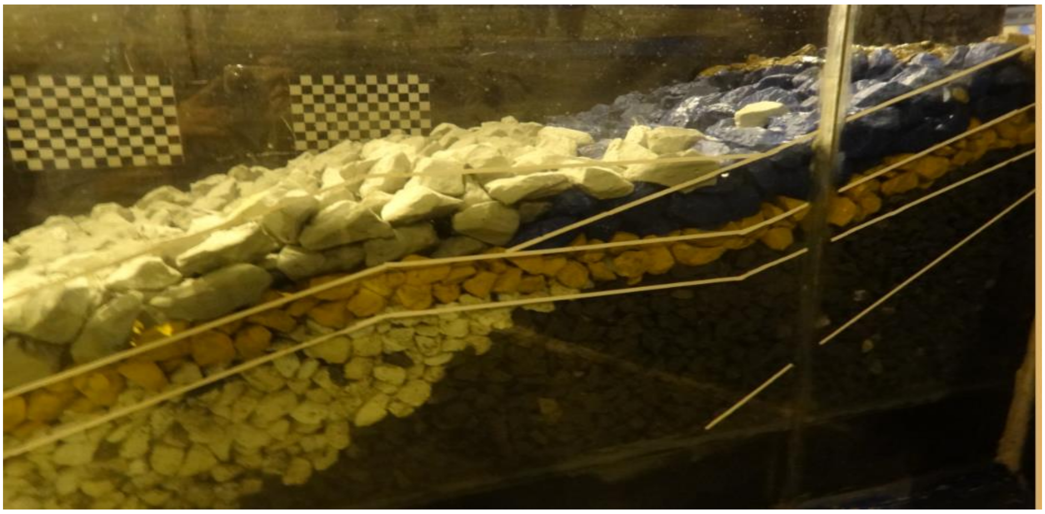

**Figure 4.2** View of Section 1 after Set 1 (1000 waves)

**Figure 4.3** View of Section 1 after Set 2 (1000 waves)

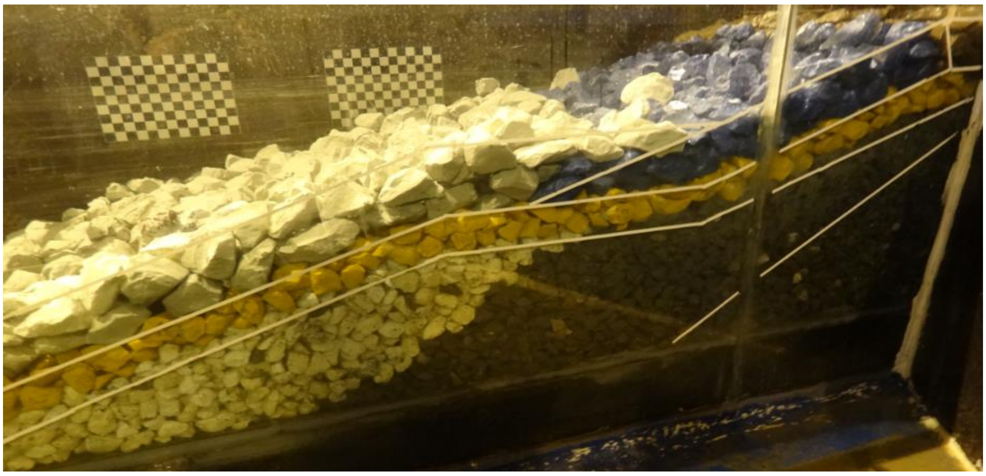

**Figure 4.4** View of Section 1 after Set 3 (3000 waves)

**Figure 4.5** View of Section 1 after Set 3 (5000 waves)

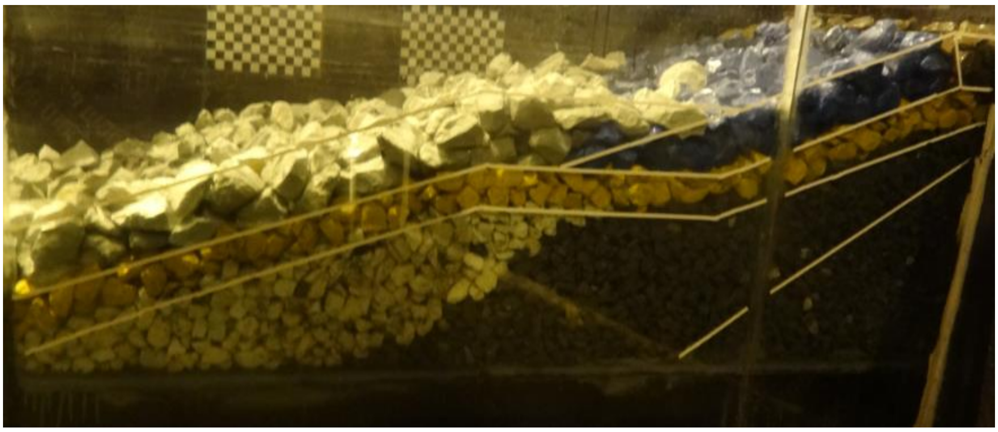

**Figure 4.6** View of Section 1 after Set 3 (7000 waves)

**Figure 4.7** View of Section 1 after Set 3 (10000 waves)

When images are examined, it is conluded that Section 1 does not suffer damage significantly in spite of the 10000 wave series. Also, the construction is extremely safe against reshaping of the profile. Only a few grey stones (12-15 tons) relocated after 3000 waves.

Table 4.1 indicates damage number, S, calculated from the experiment results along the three profile measurements shown in Figures 4.8, 4.9 and 4.10. Additionally, any recession observed is measured from the profiles as defined in Figure 2.3.

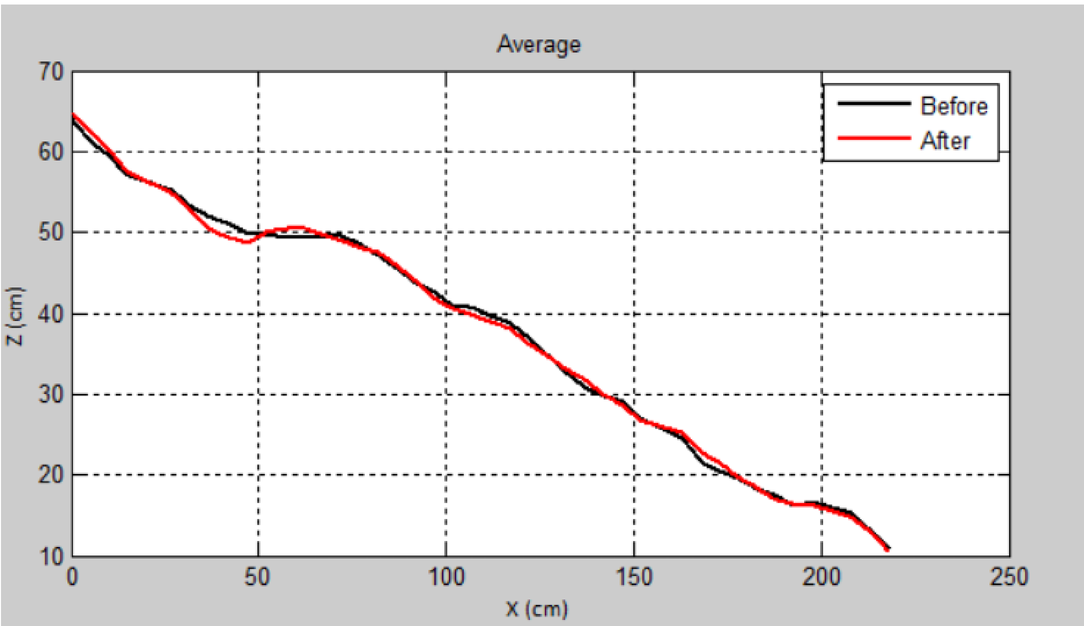


**Figure 4.8** Cross-section of Set 1 after and before experiment

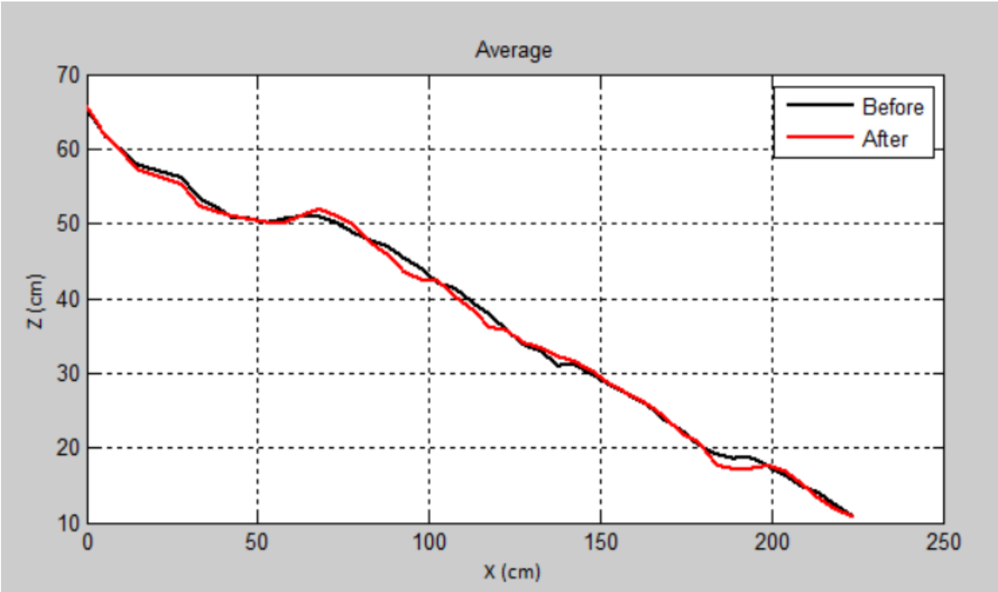


**Figure 4.9** Cross-section of Set 2 after and before experiment

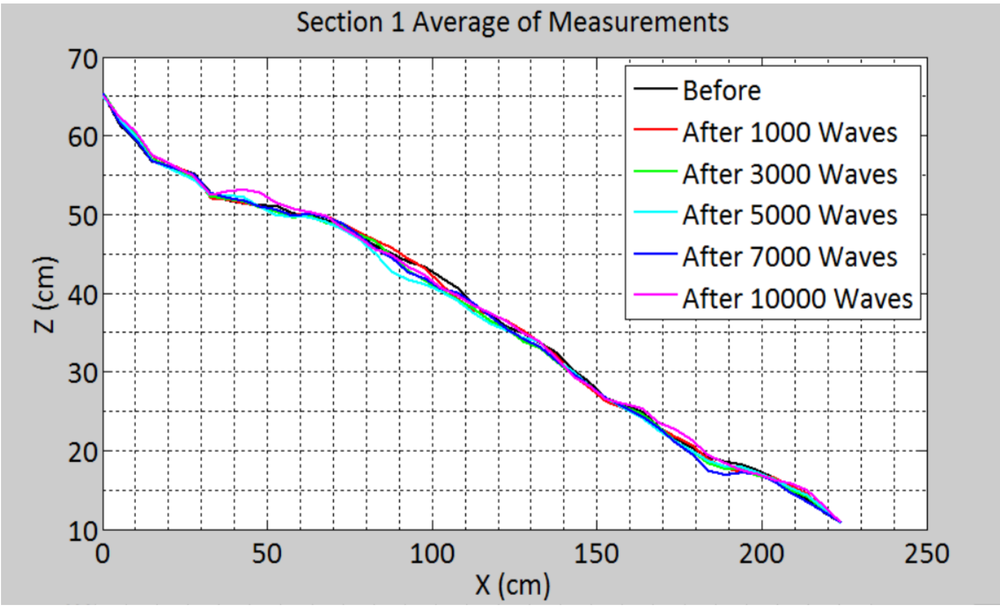


**Figure 4.10** Cross-section of Set 3 after and before experiment

**Table 4.1** Experiment Damage Results of Section 1

| EXPERIMENT DAMAGE RESULTS | | | | |
|---|---|---|---|---|
| **NAME** | **SECTION NO** | **WAVE NUMBER** | **AVERAGE S** | **$REC_{meas}$ (m)** |
| SECTION-1 (12-15 tons) | SET-1 | 1000 | 3.53 | 1.53 |
| | SET-2 | 1000 | 5.15 | 2.15 |
| | SET-3 | 1000 | 2.53 | 1.12 |
| | | 3000 | 4.49 | 1.66 |
| | | 5000 | 5.89 | 2.03 |
| | | 7000 | 7.20 | 2.68 |
| | | 10000 | 7.30 | 2.71 |

Additionally, the expected recessions that are calculated using the formulas given in Lykke Andersen et al. (2014) are presented in Table 4.2. Both damage number and measured recession values are compared to classification by Sigurdarson and Van der Meer (2014) given in Table 2.2.

**Table 4.2** Literature Stability Check of Section 1

| LITERATURE CHECK | | | | | | |
|---|---|---|---|---|---|---|
| **NAME** | **SECTION NO** | **WAVE NUMBER** | **$REC_{cal}$ (m)** | **BREAKWATER TYPE** | **S RANGE** | **STATUS** |
| SECTION-1 (12-15 tons) | SET-1 | 1000 | 2.35 | HR | 2-8 | **OK** |
| | SET-2 | 1000 | 2.35 | HR | 2-8 | **OK** |
| | SET-3 | 1000 | 2.35 | HR | 2-8 | **OK** |
| | | 3000 | 3.54 | HR | 2-8 | **OK** |
| | | 5000 | 3.54 | HR | 2-8 | **OK** |
| | | 7000 | 3.54 | HR | 2-8 | **OK** |
| | | 10000 | 3.54 | HR | 2-8 | **OK** |

When Table 4.1 and 4.2 are analyzed, primarily it is concluded that Section 1 conforms to the literature in terms of damage parameter. For 1000 waves set, average damage parameter is 3.53, 5.15 and 2.53 respectively. The differences may arise from reconstruction of the cross section where random placement of armor stones was maintained. As the shape of stones used in the armor layer showed variety, it is observed that flat stones on top layer tend to move more than the rounded stones.

Calculated recession values in Table 4.2 are found using Equation 2.28. However it is valid for maximum 3000 waves. For this reason, above 3000 waves, recession values are assumed to be same with values calculated for 3000 waves.

For 10000 wave set, at the end of experiment, damage parameter is found as 7.30 and for 7000 wave set, it is 7.20. This result could be interpreted such that Section 1 gains its stability around at 7000 waves and does not change much after. Moreover, the values are smaller than 8.00 and this structure can be classified as hardly reshaping structure.

Recession that is calculated using Equation 2.28 is 2.35 m. Similarly, measured recessions of experiments is in between 1.12 m and 2.71 m. Recession values that are computed on experiments show similarity with the literature.

### 4.1.2 Section 2

Experiments of Section 2 were conducted between April 5 and April 7, 2016. Cross-section of Section 2 is presented in chapter 3.5.2. Three different experiment sets were performed with 1000 waves, 1000 waves and 1000 waves respectively under wave conditions discussed in Chapter 3. After every experiment set, the cross-section was reconstructed to the original cross-section.

The views of Section 2 before experiment sets and after set 1, set 2 and set 3 are given in Figures 4.11, 4.12, 4.13 and 4.14.

**Figure 4.11** View of Section 2 before experiment set

**Figure 4.12** View of Section 2 after set 1 (1000 waves)

**Figure 4.13** View of Section 2 after set 2 (1000 waves)

**Figure 4.14** View of Section 2 after set 3 (1000 waves)

When Figures 4.12, 4.13 and 4.14 are examined, it is conluded that Section 2 suffered significantly more damage more than Section 1. It reaches to the S-shape profile that is expected with reference to literature search. It is mentioned that Section 2 is designed to have the minimum cross section area thus, the most exposed to danger in Chapter 3. There was significant deformation on the top layer to the extent that filters became exposed to the waves. Thus, the last set of experiments was concluded at 1000 waves instead of 10000 as it appeared that the section would take severe damage. Therefore, Section 3 is designed by increasing the berm length while keeping the stone size constant.

Table 4.3 indicates damage number, S, calculated from the experiment results along the three profile measurements shown in Figure 4.15, 4.16 and 4.17. Additionally, any recession observed is measured from the profiles as defined in Figure 2.3.

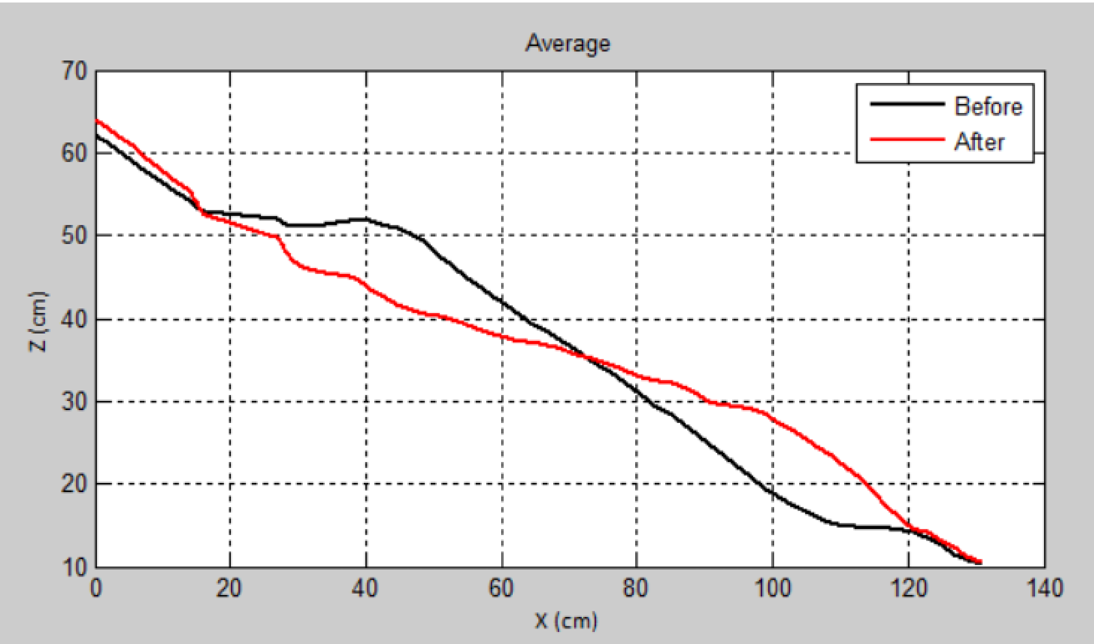


**Figure 4.15** Cross-section of Set 1 after and before experiment

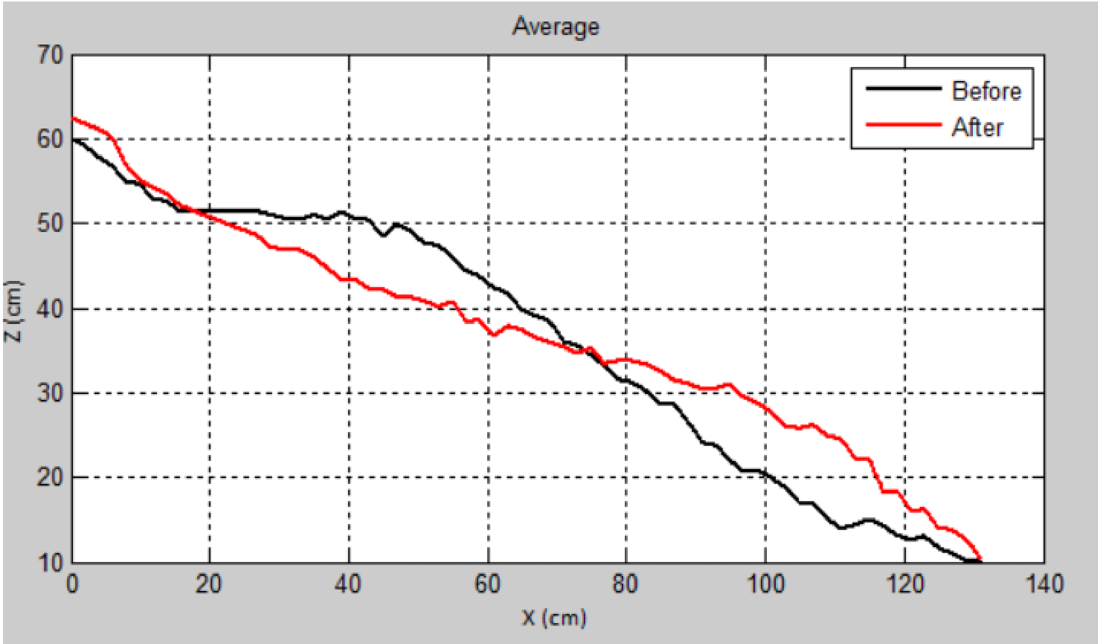


**Figure 4.16** Cross-section of Set 2 after and before experiment

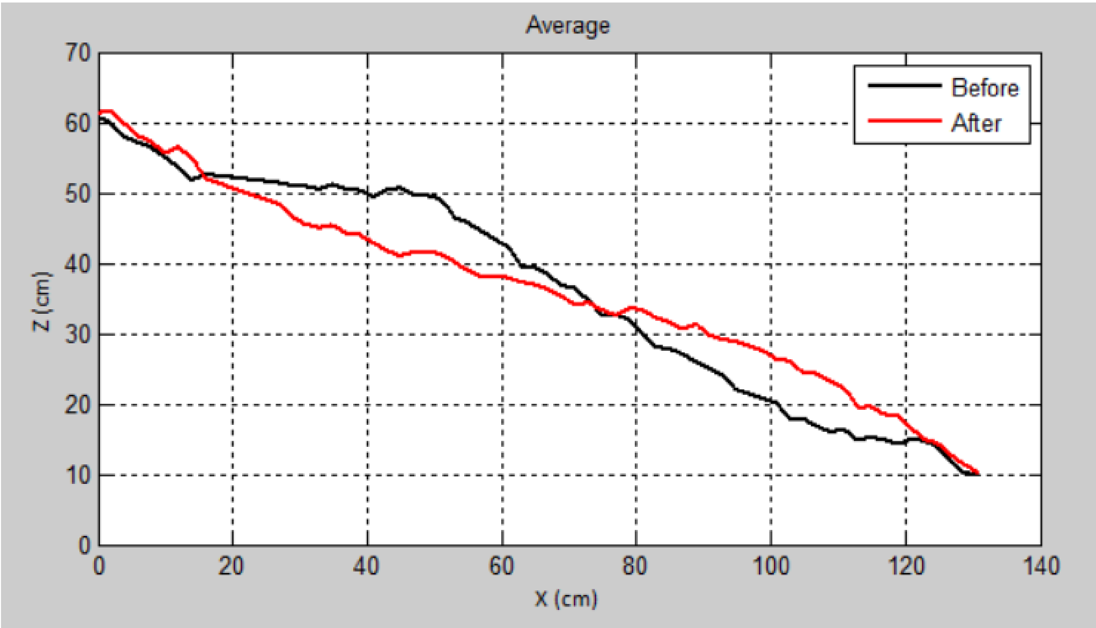


**Figure 4.17** Cross-section of Set 3 after and before experiment

**Table 4.3** Experiment Damage Results of Section 2

| EXPERIMENT DAMAGE RESULTS | | | | |
|---|---|---|---|---|
| **NAME** | **SECTION NO** | **WAVE NUMBER** | **AVERAGE S** | **$REC_{meas}$ (m)** |
| SECTION-2 (2-4 tons short) | SET-1 | 1000 | 32.88 | 8.06 |
| | SET-2 | 1000 | 31.75 | 8.25 |
| | SET-3 | 1000 | 34.18 | 8.87 |

Additionally, the expected recessions that are calculated using the formulas given in Lykke Andersen et al. (2014) are presented in Table 4.4. Both damage number and measured recession values are compared to classification by Sigurdarson and Van der Meer (2014) given in Table 2.2.

**Table 4.4** Literature Stability Check of Section 2

| LITERATURE CHECK | | | | | | |
|---|---|---|---|---|---|---|
| **NAME** | **SECTION NO** | **WAVE NUMBER** | **$REC_{cal}$ (m)** | **BREAKWATER TYPE** | **S RANGE** | **STATUS** |
| SECTION-2 (2-4 tons short) | SET-1 | 1000 | 8.29 | FR | >20 | **OK** |
| | SET-2 | 1000 | 8.29 | FR | >20 | **OK** |
| | SET-3 | 1000 | 8.29 | FR | >20 | **OK** |

When two tables are compared, it is concluded that Section 2 conforms to the literature in terms of damage parameter. For 1000 wave's sets, average damage parameters are larger than 20 which are expected. Moreover, all three experiment results of damage parameter are within 1 % thus the experiment setup has small uncertainty. In all three sets, it is seen that section 2 is a fully reshaping type berm breakwater.

Recession that is calculated using Equation 2.28 is 8.29 m. Similarly, measured calculations on experiments are in between 8.06 m and 8.87 m. Recession values that are computed on experiments show similarity with literature.

### 4.1.3 Section 3

Experiments of Section 3 were conducted between April 9 and April 15, 2016. Cross-section of Section 3 is presented in chapter 3.5.3. As discussed in Section 2, Section 3 is an upgrade of Section 2 with 44 cm. longer berm length in model. Two different experiment sets were performed with 1000 waves and 10000 waves respectively with wave conditions provided in Chapter 3. After every experiment set, the cross-section was reconstructed to the original cross-section. For discussions, the first 1000 wave of set 2 is also regarded as an independent set. Thus, after 1000 waves and taking the profile measurement, it is decided to continue with 9000 waves to complete 10000 waves set.

The views of Section 3 before experiment sets and after set 1 and set 2 are given in Figures 4.18, 4.19, 4.20 and 4.21.

**Figure 4.18** View of Section 3 before experiment set

**Figure 4.19** View of Section 3 after set 1 (1000 waves)

**Figure 4.20** View of Section 3 after set 2 (1000 waves)

**Figure 4.21** View of Section 3 after set 2 (10000 waves)

When Figures 4.19, 4.20 and 4.21 are examined, it is conluded that Section 3 suffered more damage than Section 1 and reached to an S-shape profile that is expected with reference to the literature search. When Section 2 is compared to Section 3, it seems that after 1000 waves, both sections reached to similar S shapes.

Table 4.5 indicates damage number, S, calculated from the experiment results along the three profile measurements shown in Figure 4.22 and 4.23. Additionally, any recession observed is measured from the profiles as defined in Figure 2.3.

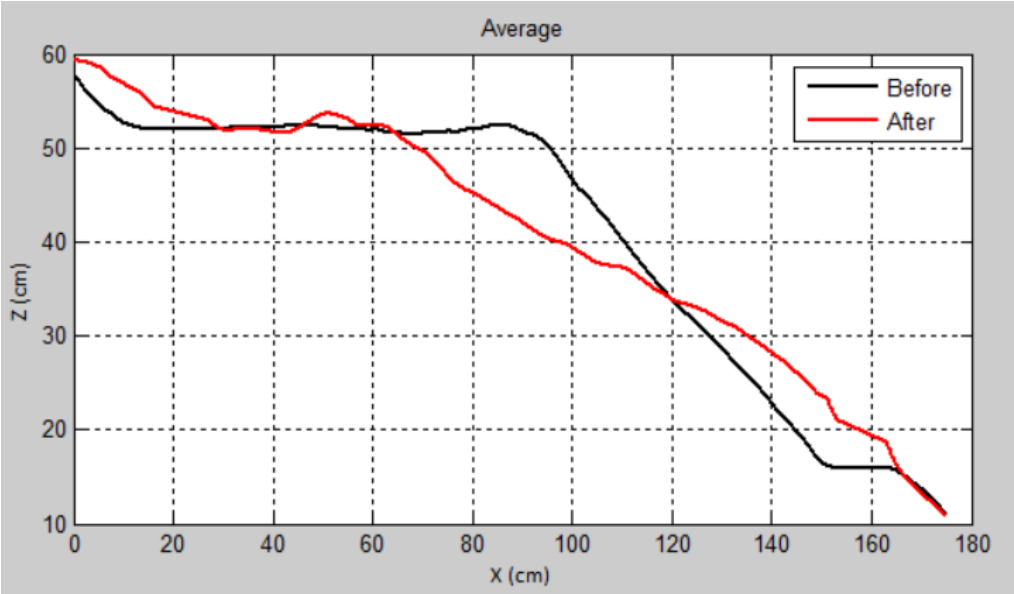


**Figure 4.22** Cross-section of Set 1 after and before experiment

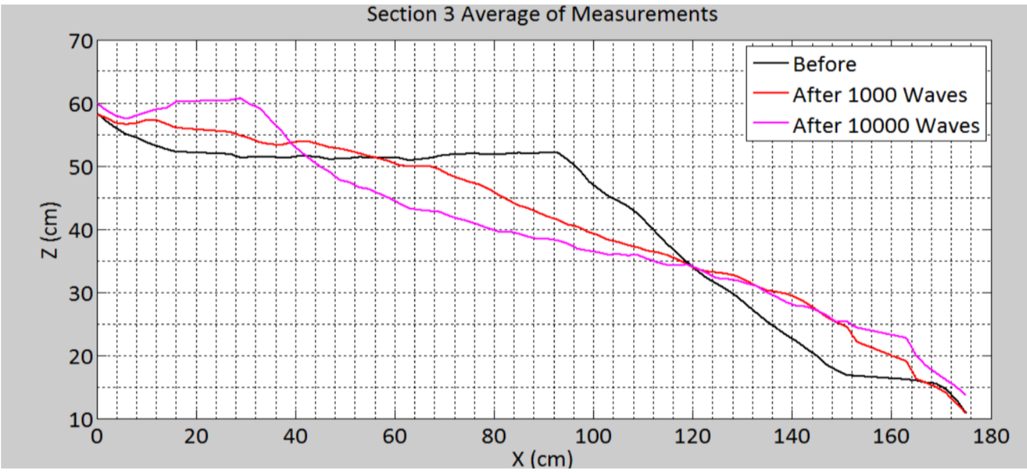


**Figure 4.23** Cross-section of Set 2 after and before experiment

**Table 4.5** Experiment Damage Results of Section 3

| EXPERIMENT DAMAGE RESULTS | | | | |
|---|---|---|---|---|
| **NAME** | **SECTION NO** | **WAVE NUMBER** | **AVERAGE S** | **$REC_{meas}$ (m)** |
| SECTION-3 (2-4 tons long) | SET-1 | 1000 | 38.79 | 7.58 |
| | SET-2 | 1000 | 41.33 | 8.53 |
| | | 10000 | 79.96 | 16.58 |

Additionally, the expected recessions that are calculated using the formulas given in Lykke Andersen et al. (2014) are presented in Table 4.6. Both damage number and measured recession values are compared to classification by Sigurdarson and Van der Meer (2014) given in Table 2.2.

**Table 4.6** Literature Stability Check of Section 3

| LITERATURE CHECK | | | | | | |
|---|---|---|---|---|---|---|
| **NAME** | **SECTION NO** | **WAVE NUMBER** | **$REC_{cal}$ (m)** | **BREAKWATER TYPE** | **S RANGE** | **STATUS** |
| SECTION-3 (2-4 tons long) | SET-1 | 1000 | 8.29 | FR | >20 | **OK** |
| | SET-2 | 1000 | 8.29 | FR | >20 | **OK** |
| | | 10000 | 10.67 | FR | >20 | **OK** |

When two tables are analyzed, it is concluded that Section 3 conforms to the literature in terms of damage parameter. For 1000 and 10000 waves set, average damage parameters are larger than 20 which are expected. Moreover, there are not significant differences between Section 2 and Section 3 with regard to damage parameters for 1000 waves set. However, after 10000 waves, the damage parameter is very high which is reflected in the profile measurements. Still, Section 3 is much more stable than Section 2 as the filter layer started to be exposed to waves after 10000 waves instead of 1000 waves which was the case for Section 2. In both sets, it is seen that Section 3 is a fully reshaping type berm breakwater.

Recession that is calculated using Equation 2.28 is 8.29 m. Similarly, measured recessions from the experiments are in between 7.58 m and 8.53 m for 1000 waves. Recession values that are computed on experiments show similarity with the literature. Furthermore, measured recession values show similarity with Section 2 for the 1000 waves like the damage parameter results. This result can show that the initial berm length does not have a significant effect on the initial profile changes. However, at the end of 10000 waves, recession value increased to 16.58 which show that longer berm lengths are needed to provide for stability which was not the case for Section 2 (thus more damage).

### 4.1.4 Section 4

Experiments of Section 4 were conducted between April 16 and April 21,2016. Cross-section of Section 4 is presented in chapter 3.5.4. This section uses a wider grading with 2-8 tons of stones. Three different experiment sets were performed with 1000 waves, 1000 waves and 10000 waves respectively with wave conditions given in Chapter 3. After every experiment set, the cross-section was reconstructed to the original cross-section.

The views of Section 4 before experiment sets and after set 1, set 2 and set 3 are given in Figures 4.24, 4.25, 4.26, 4.27, 4.28, 4.29 and 4.30.

**Figure 4.24** View of Section 4 before experiment set

**Figure 4.25** View of Section 4 after set 1 (1000 waves)

**Figure 4.26** View of Section 4 after set 2 (1000 waves)

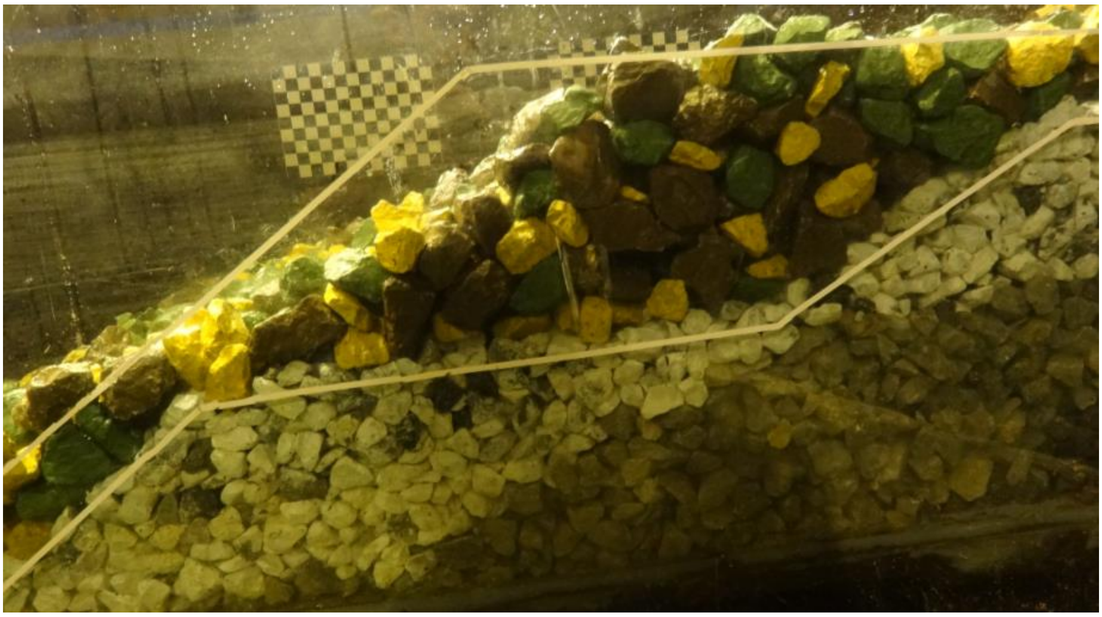

**Figure 4.27** View of Section 4 after set 3 (1000 waves)

**Figure 4.28** View of Section 4 after set 3 (4000 waves)

**Figure 4.29** View of Section 4 after set 3 (8000 waves)

**Figure 4.30** View of Section 4 after set 3 (10000 waves)

When Figures 4.25, 4.26, 4.27, 4.28, 4.29 and 4.30 are examined, it is conluded that Section 4 suffered damage more than Section 1 but less than Section 3 and reached to the expected S-shape profile. Due to the fact that grading is wider than Section 3, there is a recognizable recession difference which can be seen in Figures 4.25, 4.26 and 4.27.

Table 4.7 shows damage number, S, calculated from the experiment results along the three profile measurements shown in Figure 4.31, 4.32 and 4.33. Additionally, any recession observed is measured from the profiles as defined in Figure 2.3.

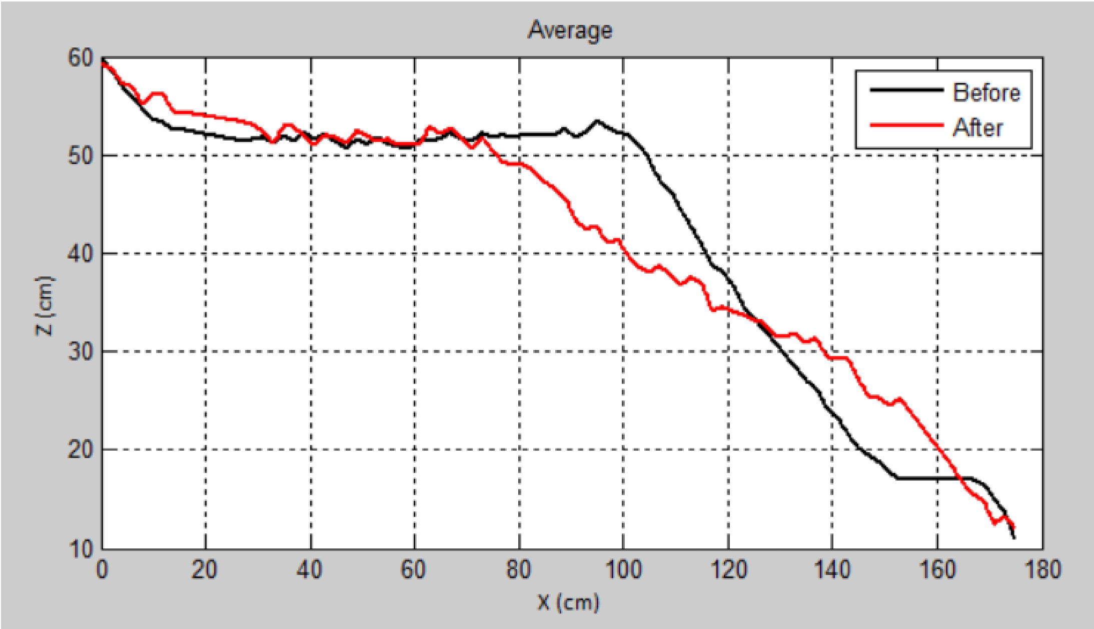


**Figure 4.31** Cross-section of Set 1 after and before experiment

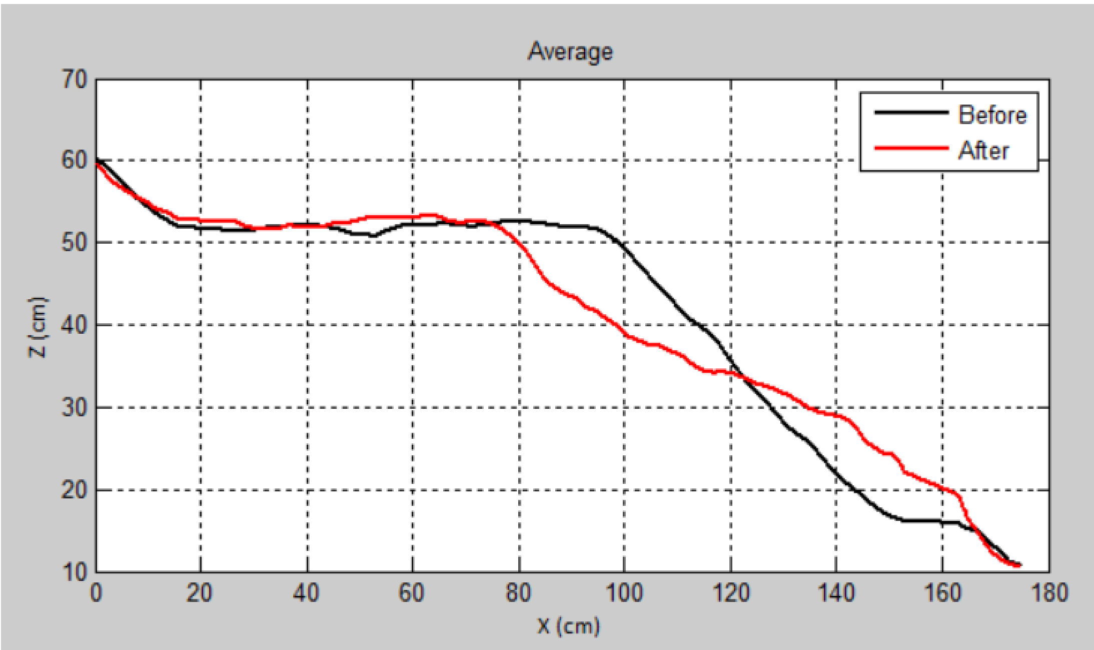


**Figure 4.32** Cross-section of Set 2 after and before experiment

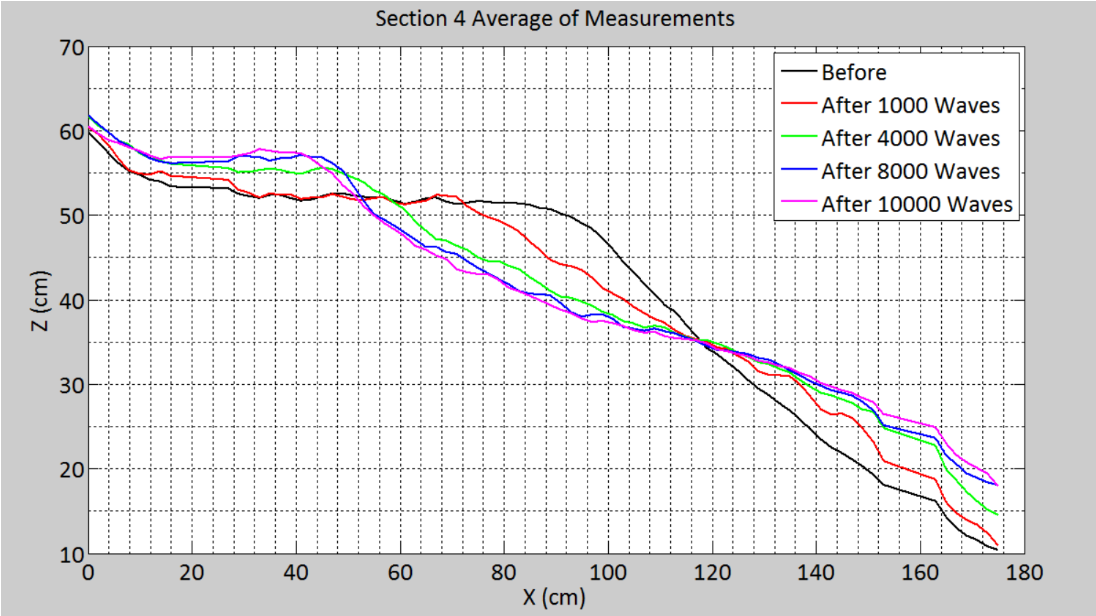


**Figure 4.33** Cross-section of Set 3 after and before experiment

**Table 4.7** Experiment Damage Results of Section 4

| **EXPERIMENT DAMAGE RESULTS** | | | | |
|---|---|---|---|---|
| **NAME** | **SECTION NO** | **WAVE NUMBER** | **AVERAGE S** | **$REC_{meas}$ (m)** |
| SECTION-4 (2-8 tons) | SET-1 | 1000 | 31.87 | 6.91 |
| | SET-2 | 1000 | 28.04 | 6.72 |
| | SET-3 | 1000 | 21.41 | 5.82 |
| | | 4000 | 31.81 | 10.56 |
| | | 8000 | 39.35 | 13.04 |
| | | 10000 | 42.47 | 14.10 |

Additionally, the expected recessions that are calculated using the formulas given in Lykke Andersen et al. (2014) are presented in Table 4.8. Both damage number and measured recession values are compared to classification by Sigurdarson and Van der Meer (2014) given in Table 2.2.

**Table 4.8** Literature Stability Check of Section 4

| LITERATURE CHECK | | | | | | |
|---|---|---|---|---|---|---|
| **NAME** | **SECTION NO** | **WAVE NUMBER** | **$REC_{cal}$ (m)** | **BREAKWATER TYPE** | **S RANGE** | **STATUS** |
| SECTION-4 (2-8 tons) | SET-1 | 1000 | 6.65 | FR | >20 | **OK** |
| | SET-2 | 1000 | 6.65 | FR | >20 | **OK** |
| | SET-3 | 1000 | 6.65 | FR | >20 | **OK** |
| | | 4000 | 8.66 | FR | >20 | **OK** |
| | | 8000 | 8.66 | FR | >20 | **OK** |
| | | 10000 | 8.66 | FR | >20 | **OK** |

When Tables 4.7 and 4.8 are analzyed, it is concluded that Section 4 conforms to literature in terms of damage parameter. For 1000 and 10000 waves set, average damage parameters are larger than 20 which are expected. Moreover, there are noticeable differences between Section 3 and Section 4 with regard to damage parameters for 1000 waves set. Even though both sets are regarded as fully reshaping type breakwater, the fact of wider grading which includes higher stone sizes makes Section 4 more reliable compared to Section 3. Section 4 has the same berm width as Section 3 so the only difference in performance is expected to be due to stone sizes and grading.

For 10000 wave set, at the end of experiment, damage parameter is found as 42.47 and for 8000 wave set it are 39.35. It can be concluded that Section 4 reaches to a stable S-shape profile around 8000 waves and does not change much after.

Recession that is calculated using Equation 2.28 is 6.65 m. Similarly, measured recession from the experiments is in between 5.82 m and 6.91m. Recession values of the experiments show similarity with the literature. Furthermore, measured recession values of Section 4 are less than the values of Section 3. That can be also concluded from pictures when amount of displaced stones on the berm is examined. Besides, at the end of 10000 waves, recession value increased to 14.10 which is less than 16.58 of Section 3 which corroborate that integrating some higher sized stones providing wider grading increases the stability of the reshaping berm breakwaters.

### 4.1.5 Section 5

Experiments of Section 5 were conducted between April 26 and May 2, 2016. Cross-section of Section 5 is presented in chapter 3.5.5 where a grading of 6-10 tons of stones was used as armor layer. Two different experiment sets were performed with 10000 waves and 1000 waves respectively. After every experiment set, the cross-section was reconstructed to the original cross-section.

The views of Section 5 before experiment sets and after set 1 and set 2 are given in Figures 4.34, 4.35, 4.36, 4.37, 4.38 and 4.39.

**Figure 4.34** View of Section 5 before experiment set

**Figure 4.35** View of Section 5 after set 1 (1000 waves)

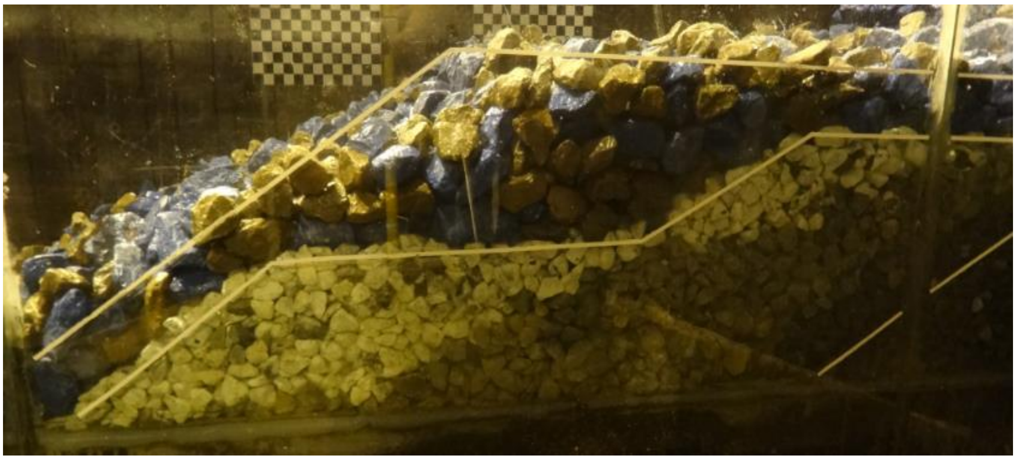

**Figure 4.36** View of Section 5 after set 1 (4000 waves)

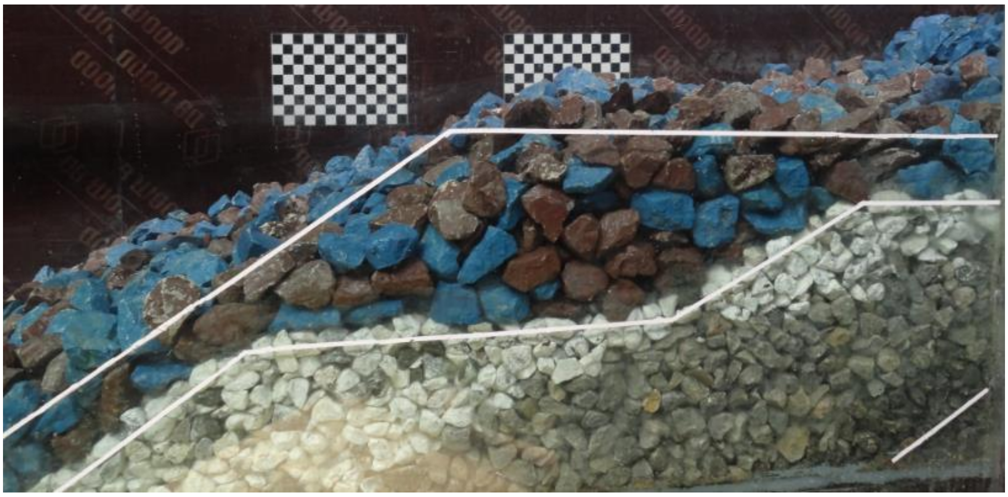

**Figure 4.37** View of Section 5 after set 1 (7000 waves)

**Figure 4.38** View of Section 5 after set 1 (10000 waves)

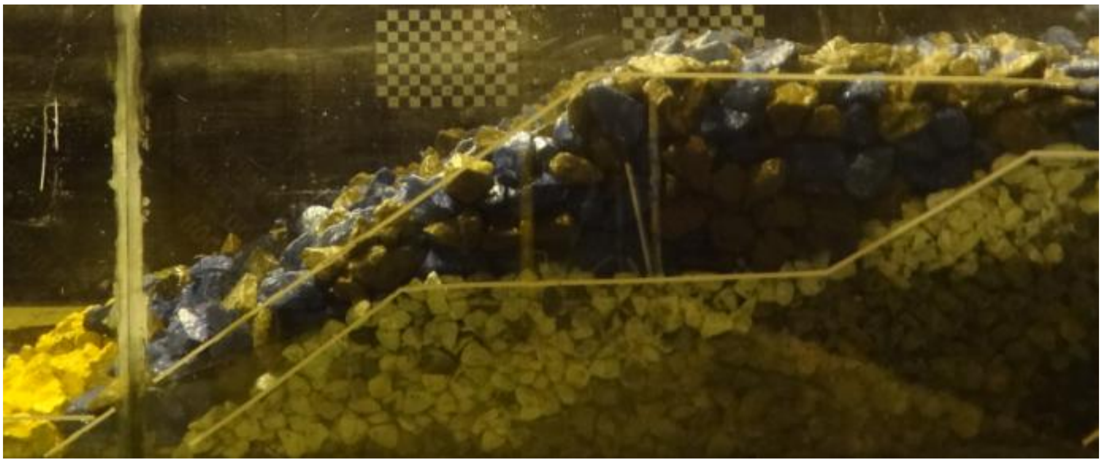

**Figure 4.39** View of Section 5 after set 2 (1000 waves)

When Figures 4.35, 4.36, 4.37, 4.38 and 4.39 are examined, it is conluded that Section 5 suffers damage more than Section 1 but significantly less than Section 4. Due to the fact that stone sizes are larger than Section 4, there is a recognizable recession difference that can be also seen from pictures. However, Section 5 is also classified as partly reshaping berm breakwater. Thus, with these 5 sections, all types of berm breakwaters according to Sigurdarson and Van der Meer (2014) classification were tested in this study.

Table 4.9 shows damage number, S, calculated from the experiment results along the three profile measurements shown in Figure 4.40 and 4.41. Additionally, any recession observed is measured from the profiles as defined in Figure 2.3.

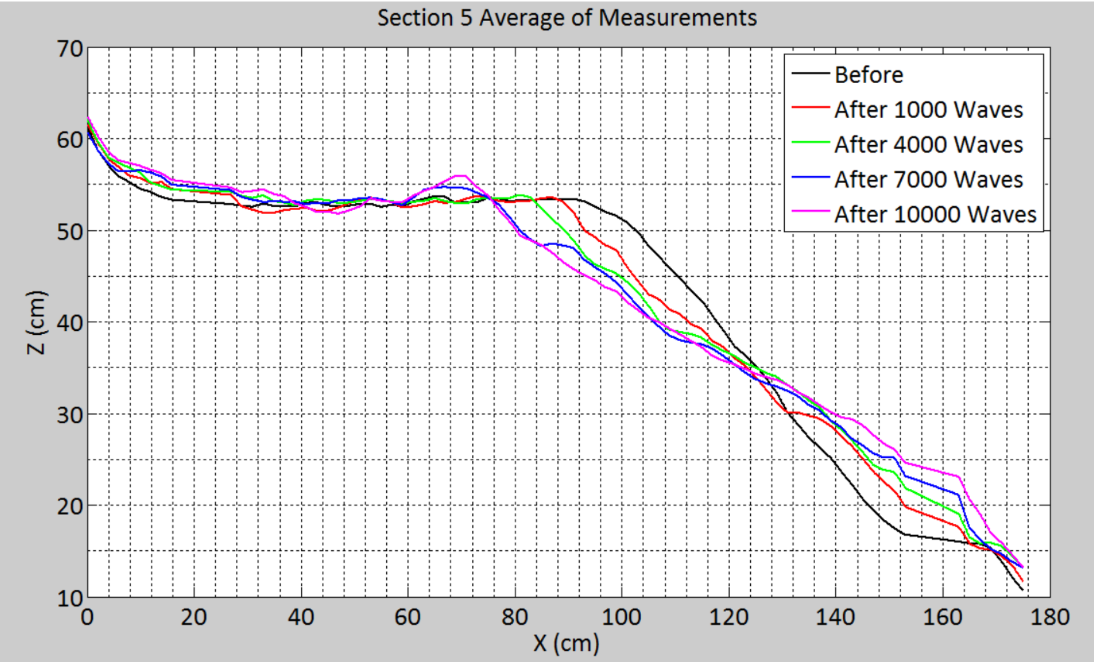


**Figure 4.40** Cross-section of Set 1 after and before experiment

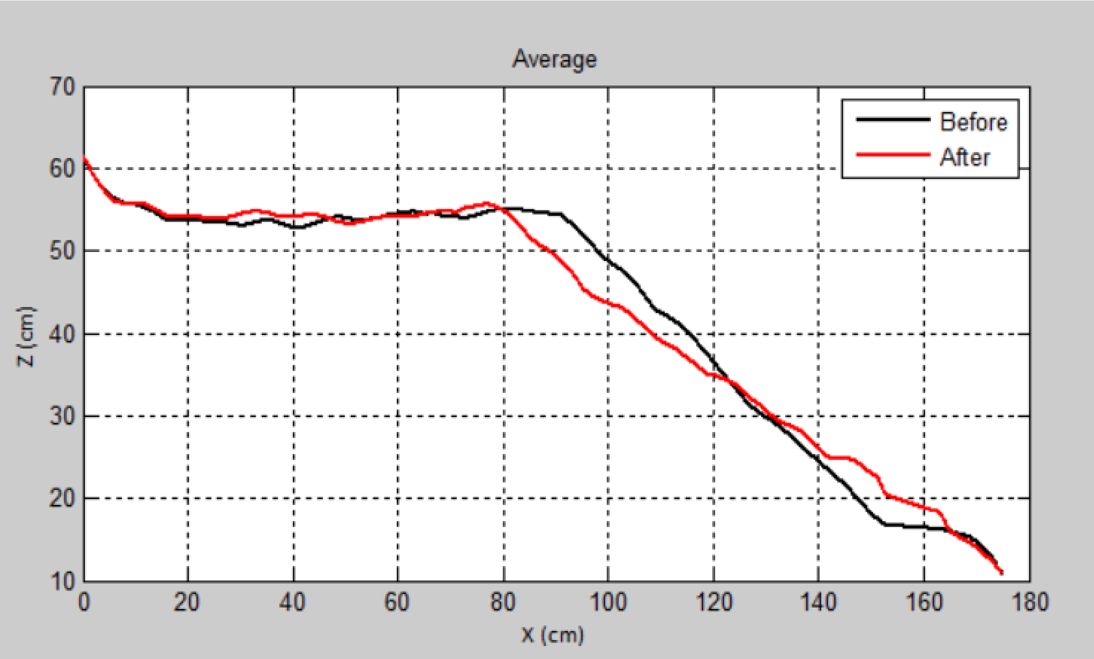


**Figure 4.41** Cross-section of Set 2 after and before experiment

**Table 4.9** Experiment Damage Results of Section 5

| EXPERIMENT DAMAGE RESULTS | | | | |
|---|---|---|---|---|
| **NAME** | **SECTION NO** | **WAVE NUMBER** | **AVERAGE S** | **$REC_{meas}$ (m)** |
| SECTION-5 (6-10 tons) | SET-1 | 1000 | 9.17 | 2.03 |
| | | 4000 | 12.10 | 3.49 |
| | | 7000 | 16.51 | 6.19 |
| | | 10000 | 18.46 | 6.81 |
| | SET-2 | 1000 | 11.25 | 3.46 |

Additionally, the expected recessions that are calculated using the formulas given in Lykke Andersen et al. (2014) are presented in Table 4.10. Both damage number and measured recession values are compared to classification by Sigurdarson and Van der Meer (2014) given in Table 2.2.

**Table 4.10** Literature Stability Check of Section 5

| LITERATURE CHECK | | | | | | |
|---|---|---|---|---|---|---|
| **NAME** | **SECTION NO** | **WAVE NUMBER** | **$REC_{cal}$ (m)** | **BREAKWATER TYPE** | **S RANGE** | **STATUS** |
| SECTION-5 (6-10 tons) | SET-1 | 1000 | 4.56 | PR | 10-20 | **OK** |
| | | 4000 | 6.11 | PR | 10-20 | **OK** |
| | | 7000 | 6.11 | PR | 10-20 | **OK** |
| | | 10000 | 6.11 | PR | 10-20 | **OK** |
| | SET-2 | 1000 | 4.56 | PR | 10-20 | **OK** |

When two tables are analyzed, it is concluded that Section 5 conforms to literature in terms of damage parameter. For 10000 and 1000 waves set, average damage parameters are between 10 and 20 which are expected. Moreover, there are noticeable differences between Section 4 and Section 5 with regard to damage parameters for 1000 waves set since Section 4 is fully reshaping and Section 5 is partly reshaping according to literature.

For 10000 waves set, at the end of experiment damage parameter is found as 18.46 and for 7000 wave set it is 16.51. It can be conlcuded that Section 5 reaches to a stable S shape profile around 7000 waves and does not change much after.

Recession that is calculated using Equation 2.28 is 4.56 m. Similarly, measured recessions from experiments are in between 2.03m and 3.46m. Measured recession values of experiments are less than the theoretical value. Also, they are less than the values that are measured in Section 4. That can be also concluded from pictures when amount of displaced stones on berm is examined. Similarly, at the end of 10000 waves, recession value increased to 6.81m which is less than 14.10m of Section 4 showing the difference in the performance of partly and fully reshaping berm breakwaters.

## 4.2 Wave Overtopping

Another important parameter on the design of breakwaters is serviceability of the structure which is governed by wave overtopping. In the design calculations, the allowable mean overtopping discharge has been determined as 10 l/s/m for Ordu-Giresun berm breakwater since the structure under consideration is a breakwater protecting an airport. Therefore, the same condition is taken as the upper limit for mean overtopping discharge of alternative models. The procedure for measurement of wave overtopping is given in Chapter 3. A summary of the results is provided in Table 4.11.

**Table 4.11** Wave overtopping summary for sections

| OVERTOPPING SUMMARY | | | |
|---|---|---|---|
| | **MAX Q (l/s/m)** | **MIN Q (l/s/m)** | **MEAN Q (l/s/m)** |
| **SECTION-1** | 3.17 | 1.00 | 2.09 |
| **SECTION-2** | 92.32 | 46.00 | 72.73 |
| **SECTION-3** | 8.24 | 5.61 | 6.73 |
| **SECTION-4** | 7.95 | 4.05 | 6.49 |
| **SECTION-5** | 1.91 | 1.47 | 1.64 |

In this section, measurements of overtopping for each cross section and comparison to CLASH, TAW and EUROTOP formulation results are provided with discussions.

### 4.2.1 Section 1

Wave overtopping values are presented set by set for Section 1 in Tables 4.12, 4.13 and 4.14:

**Table 4.12** Wave overtopping results for Set 1

<table>
<tr><th colspan="5">EXPERIMENT OVERTOPPING RESULTS</th><th colspan="5">LITERATURE CHECK</th></tr>
<tr><th>NAME</th><th>SECTION NO</th><th>WAVE NUMBER</th><th>MODEL QUANTITY</th><th>PROTOTYPE QUANTITY</th><th>OVERTOPPING RANGE</th><th>STATUS</th><th>CLASH RESULTS</th><th>TAW RESULTS</th><th>EUROTOP (by S&WdM) RESULTS</th></tr>
<tr><td rowspan="2">SECTION-1 (12-15 tons)</td><td rowspan="2">SET-1</td><td>0-500</td><td>3792 grams</td><td>3.17 l/s/m</td><td>< 10 l/s/m</td><td>OK</td><td rowspan="2">1.62 l/s/m</td><td rowspan="2">7.01 l/s/m</td><td rowspan="2">2.06 l/s/m</td></tr>
<tr><td>500-1000</td><td>3096 grams</td><td>2.59 l/s/m</td><td>< 10 l/s/m</td><td>OK</td></tr>
</table>

**Table 4.13** Wave overtopping results for Set 2

<table>
<tr><th colspan="5">EXPERIMENT OVERTOPPING RESULTS</th><th colspan="5">LITERATURE CHECK</th></tr>
<tr><th>NAME</th><th>SECTION NO</th><th>WAVE NUMBER</th><th>MODEL QUANTITY</th><th>PROTOTYPE QUANTITY</th><th>OVERTOPPING RANGE</th><th>STATUS</th><th>CLASH RESULTS</th><th>TAW RESULTS</th><th>EUROTOP (by S&WdM) RESULTS</th></tr>
<tr><td rowspan="2">SECTION-1 (12-15 tons)</td><td rowspan="2">SET-2</td><td>0-500</td><td>1434 grams</td><td>1.20 l/s/m</td><td>< 10 l/s/m</td><td>OK</td><td rowspan="2">1.62 l/s/m</td><td rowspan="2">7.01 l/s/m</td><td rowspan="2">2.06 l/s/m</td></tr>
<tr><td>500-1000</td><td>1196 grams</td><td>1.00 l/s/m</td><td>< 10 l/s/m</td><td>OK</td></tr>
</table>

**Table 4.14** Wave overtopping results for Set 3

| EXPERIMENT OVERTOPPING RESULTS | | | | | LITERATURE CHECK | | | | |
|---|---|---|---|---|---|---|---|---|---|
| **NAME** | **SECTION NO** | **WAVE NUMBER** | **MODEL QUANTITY** | **PROTOTYPE QUANTITY** | **OVERTOPPING RANGE** | **STATUS** | **CLASH RESULTS** | **TAW RESULTS** | **EUROTOP (by S&WdM) RESULTS** |
| SECTION-1 (12-15 tons) | SET-1 | 0-500 | 2846 grams | 2.38 l/s/m | < 10 l/s/m | **OK** | 1.62 l/s/m | 7.01 l/s/m | 2.06 l/s/m |
| | | 500-1000 | 2624 grams | 2.20 l/s/m | < 10 l/s/m | **OK** | | | |
| | | 1000-1500 | 3218 grams | 2.69 l/s/m | < 10 l/s/m | **OK** | | | |
| | | 1500-2000 | 4622 grams | 3.87 l/s/m | < 10 l/s/m | **OK** | | | |
| | | 2000-2500 | 4026 grams | 3.37 l/s/m | < 10 l/s/m | **OK** | | | |
| | | 2500-3000 | 3950 grams | 3.31 l/s/m | < 10 l/s/m | **OK** | | | |
| | | 3000-3500 | 3934 grams | 3.29 l/s/m | < 10 l/s/m | **OK** | | | |
| | | 3500-4000 | 3002 grams | 2.51 l/s/m | < 10 l/s/m | **OK** | | | |
| | | 4000-4500 | 3486 grams | 2.92 l/s/m | < 10 l/s/m | **OK** | | | |
| | | 4500-5000 | 3574 grams | 2.99 l/s/m | < 10 l/s/m | **OK** | | | |
| | | 5000-5500 | 4624 grams | 3.87 l/s/m | < 10 l/s/m | **OK** | | | |
| | | 5500-6000 | 4580 grams | 3.83 l/s/m | < 10 l/s/m | **OK** | | | |
| | | 6000-6500 | 3714 grams | 3.11 l/s/m | < 10 l/s/m | **OK** | | | |
| | | 6500-7000 | 3780 grams | 3.16 l/s/m | < 10 l/s/m | **OK** | | | |
| | | 7000-7500 | 3262 grams | 2.73 l/s/m | < 10 l/s/m | **OK** | | | |
| | | 7500-8000 | 2701 grams | 2.26 l/s/m | < 10 l/s/m | **OK** | | | |
| | | 8000-8500 | 2638 grams | 2.21 l/s/m | < 10 l/s/m | **OK** | | | |
| | | 8500-9000 | 2566 grams | 2.15 l/s/m | < 10 l/s/m | **OK** | | | |
| | | 9000-9500 | 2928 grams | 2.45 l/s/m | < 10 l/s/m | **OK** | | | |
| | | 9500-10000 | 2580 grams | 2.16 l/s/m | < 10 l/s/m | **OK** | | | |

When tables are analyzed, it is concluded that Section 1 conforms to the <10 l/s/m limit. The maximum and minimum wave overtopping amount is measured as 3.87 l/s/m and 1.00 l/s/m respectively. Moreover, CLASH results show similarity with prototype quantities measured in the experiments however the experiment values are higher. However, the difference between these values is not significant since CLASH is highly dependent on a limited database of berm breakwaters and overtopping measurements have higher uncertainty than stability calculations. Eurotop formulation provides a similar overtopping value whereas TAW formula predicts higher overtopping rates.

On the other hand, there is not a significant relation between the change in the profile during an experiment and wave overtopping amount. The damage of section 1 is very limited and the amount of wave overtopping changes between 2-4 l/s/m throughout the 10000 wave set for every 500 waves.

### 4.2.2 Section 2

Wave overtopping values are presented set by set for Section 2 in Tables 4.15, 4.16 and 4.17:

**Table 4.15** Wave overtopping results for Set 1

| EXPERIMENT OVERTOPPING RESULTS | | | | | LITERATURE CHECK | | | | |
|---|---|---|---|---|---|---|---|---|---|
| **NAME** | **SECTION NO** | **WAVE NUMBER** | **MODEL QUANTITY** | **PROTOTYPE QUANTITY** | **OVERTOPPING RANGE** | **STATUS** | **CLASH RESULTS** | **TAW RESULTS** | **EUROTOP (by S&WdM) RESULTS** |
| SECTION-2 (2-4 tons short) | SET-1 | 0-500 | 73094 grams | 61.20 l/s/m | < 10 l/s/m | **NOT OK** | 9.86 l/s/m | 121.10 l/s/m | 12.46 l/s/m |
| | | 500-1000 | 54945 grams | 46.00 l/s/m | < 10 l/s/m | **NOT OK** | | | |

**Table 4.16** Wave overtopping results for Set 2

| EXPERIMENT OVERTOPPING RESULTS | | | | | LITERATURE CHECK | | | | |
|---|---|---|---|---|---|---|---|---|---|
| **NAME** | **SECTION NO** | **WAVE NUMBER** | **MODEL QUANTITY** | **PROTOTYPE QUANTITY** | **OVERTOPPING RANGE** | **STATUS** | **CLASH RESULTS** | **TAW RESULTS** | **EUROTOP (by S&WdM) RESULTS** |
| SECTION-2 (2-4 tons short) | SET-2 | 0-500 | 95080 grams | 79.61 l/s/m | < 10 l/s/m | **NOT OK** | 9.86 l/s/m | 121.10 l/s/m | 12.46 l/s/m |
| | | 500-1000 | 110264 grams | 92.32 l/s/m | < 10 l/s/m | **NOT OK** | | | |

**Table 4.17** Wave overtopping results for Set 3

| EXPERIMENT OVERTOPPING RESULTS | | | | | LITERATURE CHECK | | | | |
|---|---|---|---|---|---|---|---|---|---|
| **NAME** | **SECTION NO** | **WAVE NUMBER** | **MODEL QUANTITY** | **PROTOTYPE QUANTITY** | **OVERTOPPING RANGE** | **STATUS** | **CLASH RESULTS** | **TAW RESULTS** | **EUROTOP (by S&WdM) RESULTS** |
| SECTION-2 (2-4 tons short) | SET-3 | 0-500 | 87920 grams | 73.61 l/s/m | < 10 l/s/m | **NOT OK** | 9.86 l/s/m | 121.10 l/s/m | 12.46 l/s/m |
| | | 500-1000 | 99920 grams | 83.66 l/s/m | < 10 l/s/m | **NOT OK** | | | |

When tables are analyzed, it is concluded that Section 2 does not conform to the limit set for the wave overtopping. The maximum and minimum wave overtopping amount is measured as 92.32 l/s/m and 46.00 l/s/m respectively. Moreover, CLASH results are significantly less than the measured values. This could be due to the fact that Section 2 also had the most damage under wave attack and the small cross section could not be represented by CLASH database. Eurotop formulation provides a similar overtopping value as CLASH databse whereas TAW formula predicts higher overtopping rates to which the experiment results are closer.

On the other hand, it can be said that there can be a relation between change in profile (damage) and wave overtopping amount as wave overtopping increases in the $2^{nd}$ 500 wave for two of the experiment sets. Nevertheless, there is not enough data to support this argument within the limits of this study. Section 2 is not suitable for construction since wave overtopping quantities is larger than overtopping range.

### 4.2.3 Section 3

Wave overtopping values are presented set by set for Section 3 in Tables 4.18 and 4.19:

**Table 4.18** Wave overtopping results for Set 1

| EXPERIMENT OVERTOPPING RESULTS | | | | | LITERATURE CHECK | | | | |
|---|---|---|---|---|---|---|---|---|---|
| **NAME** | **SECTION NO** | **WAVE NUMBER** | **MODEL QUANTITY** | **PROTOTYPE QUANTITY** | **OVERTOPPING RANGE** | **STATUS** | **CLASH RESULTS** | **TAW RESULTS** | **EUROTOP (by S&WdM) RESULTS** |
| SECTION-3 (2-4 tons long) | SET-1 | 0-500 | 6862 grams | 5.75 l/s/m | < 10 l/s/m | **OK** | 2.76 l/s/m | 121.10 l/s/m | 12.46 l/s/m |
| | | 500-1000 | 6704 grams | 5.61 l/s/m | < 10 l/s/m | **OK** | | | |

**Table 4.19** Wave overtopping results for Set 2

| EXPERIMENT OVERTOPPING RESULTS | | | | | | LITERATURE CHECK | | | |
|---|---|---|---|---|---|---|---|---|---|
| **NAME** | **SECTION NO** | **WAVE NUMBER** | **MODEL QUANTITY** | **PROTOTYPE QUANTITY** | **OVERTOPPING RANGE** | **STATUS** | **CLASH RESULTS** | **TAW RESULTS** | **EUROTOP (by S&WdM) RESULTS** |
| SECTION-3 (2-4 tons long) | SET-2 | 0-500 | 9844 grams | 8.24 l/s/m | < 10 l/s/m | **OK** | 2.76 l/s/m | 121.10 l/s/m | 12.46 l/s/m |
| | | 500-1000 | 8764 grams | 7.34 l/s/m | < 10 l/s/m | **OK** | | | |
| | | 1000-1500 | 5480 grams | 4.59 l/s/m | < 10 l/s/m | **OK** | | | |
| | | 1500-2000 | 5650 grams | 4.73 l/s/m | < 10 l/s/m | **OK** | | | |
| | | 2000-2500 | 5914 grams | 4.95 l/s/m | < 10 l/s/m | **OK** | | | |
| | | 2500-3000 | 6325 grams | 5.30 l/s/m | < 10 l/s/m | **OK** | | | |
| | | 3000-3500 | 7452 grams | 6.24 l/s/m | < 10 l/s/m | **OK** | | | |
| | | 3500-4000 | 8544 grams | 7.15 l/s/m | < 10 l/s/m | **OK** | | | |
| | | 4000-4500 | 9794 grams | 8.20 l/s/m | < 10 l/s/m | **OK** | | | |
| | | 4500-5000 | 9184 grams | 7.69 l/s/m | < 10 l/s/m | **OK** | | | |
| | | 5000-5500 | 8460 grams | 7.08 l/s/m | < 10 l/s/m | **OK** | | | |
| | | 5500-6000 | 7290 grams | 6.10 l/s/m | < 10 l/s/m | **OK** | | | |
| | | 6000-6500 | 8456 grams | 7.08 l/s/m | < 10 l/s/m | **OK** | | | |
| | | 6500-7000 | 6358 grams | 5.32 l/s/m | < 10 l/s/m | **OK** | | | |
| | | 7000-7500 | 6262 grams | 5.24 l/s/m | < 10 l/s/m | **OK** | | | |
| | | 7500-8000 | 6194 grams | 5.19 l/s/m | < 10 l/s/m | **OK** | | | |
| | | 8000-8500 | 8658 grams | 7.25 l/s/m | < 10 l/s/m | **OK** | | | |
| | | 8500-9000 | 8207 grams | 6.87 l/s/m | < 10 l/s/m | **OK** | | | |
| | | 9000-9500 | 8252 grams | 6.91 l/s/m | < 10 l/s/m | **OK** | | | |
| | | 9500-10000 | 7988 grams | 6.69 l/s/m | < 10 l/s/m | **OK** | | | |

When tables are analyzed, it is concluded that Section 3 conforms to the overtopping limit set for the experiments. The maximum and minimum wave overtopping amount is measured as 8.24 l/s/m and 4.59 l/s/m respectively. CLASH results are less than the prototype quantities but still within acceptable range. Also CLASH results for Section 3 is higher than Section 1 but less than Section 3. CLASH uses the geometry of the structure as input parameters not stone sizes, which is reflected in these results. Both Eurotop formulation and TAW formula predicts higher overtopping rates however Eurotop calculations are much closer to experiment results.

Even though the section is damaged, wave overtopping amount changes between 4-8 l/s/m which could be considered as constant. One reason for this constancy could be the impact of major waves being the main cause for overtopping and the amount of water they overtop does not change much even if there is an increase in damage.

### 4.2.4 Section 4

Wave overtopping values are presented set by set for Section 4 in Tables 4.20, 4.21 and 4.22:

**Table 4.20** Wave overtopping results for Set 1

| EXPERIMENT OVERTOPPING RESULTS | | | | | LITERATURE CHECK | | | | |
|---|---|---|---|---|---|---|---|---|---|
| NAME | SECTION NO | WAVE NUMBER | MODEL QUANTITY | PROTOTYPE QUANTITY | OVERTOPPING RANGE | STATUS | CLASH RESULTS | TAW RESULTS | EUROTOP (by S&WdM) RESULTS |
| SECTION-4 (2-8 tons) | SET-1 | 0-500 | 9494 grams | 7.95 l/s/m | < 10 l/s/m | **OK** | 2.76 l/s/m | 121.10 l/s/m | 12.46 l/s/m |
| | | 500-1000 | 9390 grams | 7.86 l/s/m | < 10 l/s/m | **OK** | | | |

**Table 4.21** Wave overtopping results for Set 2

| EXPERIMENT OVERTOPPING RESULTS | | | | | LITERATURE CHECK | | | | |
|---|---|---|---|---|---|---|---|---|---|
| NAME | SECTION NO | WAVE NUMBER | MODEL QUANTITY | PROTOTYPE QUANTITY | OVERTOPPING RANGE | STATUS | CLASH RESULTS | TAW RESULTS | EUROTOP (by S&WdM) RESULTS |
| SECTION-4 (2-8 tons) | SET-2 | 0-500 | 6362 grams | 5.33 l/s/m | < 10 l/s/m | **OK** | 2.76 l/s/m | 121.10 l/s/m | 12.46 l/s/m |
| | | 500-1000 | 4838 grams | 4.05 l/s/m | < 10 l/s/m | **OK** | | | |

**Table 4.22** Wave overtopping results for Set 3

| EXPERIMENT OVERTOPPING RESULTS | | | | | LITERATURE CHECK | | | | |
|---|---|---|---|---|---|---|---|---|---|
| NAME | SECTION NO | WAVE NUMBER | MODEL QUANTITY | PROTOTYPE QUANTITY | OVERTOPPING RANGE | STATUS | CLASH RESULTS | TAW RESULTS | EUROTOP (by S&WdM) RESULTS |
| SECTION-4 (2-8 tons) | SET-3 | 0-500 | 8652 grams | 7.24 l/s/m | < 10 l/s/m | **OK** | 2.76 l/s/m | 121.10 l/s/m | 12.46 l/s/m |
| | | 500-1000 | 7776 grams | 6.51 l/s/m | < 10 l/s/m | **OK** | | | |
| | | 1000-1500 | 7570 grams | 6.34 l/s/m | < 10 l/s/m | **OK** | | | |
| | | 1500-2000 | 4770 grams | 3.99 l/s/m | < 10 l/s/m | **OK** | | | |
| | | 2000-2500 | 4696 grams | 3.93 l/s/m | < 10 l/s/m | **OK** | | | |
| | | 2500-3000 | 5672 grams | 4.75 l/s/m | < 10 l/s/m | **OK** | | | |
| | | 3000-3500 | 4968 grams | 4.16 l/s/m | < 10 l/s/m | **OK** | | | |
| | | 3500-4000 | 4626 grams | 3.87 l/s/m | < 10 l/s/m | **OK** | | | |
| | | 4000-4500 | 5687 grams | 4.76 l/s/m | < 10 l/s/m | **OK** | | | |
| | | 4500-5000 | 6070 grams | 5.08 l/s/m | < 10 l/s/m | **OK** | | | |
| | | 5000-5500 | 5338 grams | 4.47 l/s/m | < 10 l/s/m | **OK** | | | |
| | | 5500-6000 | 5778 grams | 4.84 l/s/m | < 10 l/s/m | **OK** | | | |
| | | 6000-6500 | 6242 grams | 5.23 l/s/m | < 10 l/s/m | **OK** | | | |
| | | 6500-7000 | 6362 grams | 5.33 l/s/m | < 10 l/s/m | **OK** | | | |
| | | 7000-7500 | 5355 grams | 4.48 l/s/m | < 10 l/s/m | **OK** | | | |
| | | 7500-8000 | 5691 grams | 4.76 l/s/m | < 10 l/s/m | **OK** | | | |
| | | 8000-8500 | 4960 grams | 4.15 l/s/m | < 10 l/s/m | **OK** | | | |
| | | 8500-9000 | 6834 grams | 5.72 l/s/m | < 10 l/s/m | **OK** | | | |
| | | 9000-9500 | 5682 grams | 4.76 l/s/m | < 10 l/s/m | **OK** | | | |
| | | 9500-10000 | 4872 grams | 4.08 l/s/m | < 10 l/s/m | **OK** | | | |

When tables are analyzed, it is concluded that Section 4 conforms to limits set for this study. The maximum and minimum wave overtopping amount is measured as 7.95 l/s/m and 3.93 l/s/m respectively. Moreover, CLASH results are less than the prototype quantities but same as in Section 3. CLASH uses the geometry of the structure as input parameters not stone sizes, thus calculated overtopping rates do not change unless geometry of the structure changes. As the only change between Section 3 and 4 is stone sizes and grading, the CLASH overtopping rate stays the same. However, 2.76 l/s/m is much closer to the experiment results. This could be due to the fact that although Section 4 is still a fully reshaping berm breakwater, the damage number and recession are much less than Section 3 and this is reflected in the overtopping rates. Eurotop formulation provides a similar overtopping value whereas TAW formula predicts higher overtopping rates.

On the other hand, no significant relation between number of waves and wave overtopping amount can be detected for this section. Even though the section is damaged, wave overtopping amount changes within 4-6 l/s/m which could be considered as constant. Similar to Section 3, one reason for this constancy could be the impact of major waves being the main cause for overtopping and the amount of water they overtop does not change much even if there is an increase in damage.

### 4.2.5 Section 5

Wave overtopping values are presented set by set for Section 5 in Tables 4.23 and 4.24:

**Table 4.23** Wave overtopping results for Set 1

| EXPERIMENT OVERTOPPING RESULTS | | | | | LITERATURE CHECK | | | | |
|---|---|---|---|---|---|---|---|---|---|
| NAME | SECTION NO | WAVE NUMBER | MODEL QUANTITY | PROTOTYPE QUANTITY | OVERTOPPING RANGE | STATUS | CLASH RESULTS | TAW RESULTS | EUROTOP (by S&WdM) RESULTS |
| SECTION-5 (6-10 tons) | SET-1 | 0-500 | 2084 grams | 1.74 l/s/m | < 10 l/s/m | **OK** | 2.76 l/s/m | 121.10 l/s/m | 4.46 l/s/m |
| | | 500-1000 | 2684 grams | 2.25 l/s/m | < 10 l/s/m | **OK** | | | |
| | | 1000-1500 | 2430 grams | 2.03 l/s/m | < 10 l/s/m | **OK** | | | |
| | | 1500-2000 | 2457 grams | 2.06 l/s/m | < 10 l/s/m | **OK** | | | |
| | | 2000-2500 | 1412 grams | 1.18 l/s/m | < 10 l/s/m | **OK** | | | |
| | | 2500-3000 | 1658 grams | 1.39 l/s/m | < 10 l/s/m | **OK** | | | |
| | | 3000-3500 | 1908 grams | 1.60 l/s/m | < 10 l/s/m | **OK** | | | |
| | | 3500-4000 | 1586 grams | 1.33 l/s/m | < 10 l/s/m | **OK** | | | |
| | | 4000-4500 | 2170 grams | 1.82 l/s/m | < 10 l/s/m | **OK** | | | |
| | | 4500-5000 | 2496 grams | 2.09 l/s/m | < 10 l/s/m | **OK** | | | |
| | | 5000-5500 | 2188 grams | 1.83 l/s/m | < 10 l/s/m | **OK** | | | |
| | | 5500-6000 | 2294 grams | 1.92 l/s/m | < 10 l/s/m | **OK** | | | |
| | | 6000-6500 | 2670 grams | 2.24 l/s/m | < 10 l/s/m | **OK** | | | |
| | | 6500-7000 | 2488 grams | 2.08 l/s/m | < 10 l/s/m | **OK** | | | |
| | | 7000-7500 | 2774 grams | 2.32 l/s/m | < 10 l/s/m | **OK** | | | |
| | | 7500-8000 | 2565 grams | 2.15 l/s/m | < 10 l/s/m | **OK** | | | |
| | | 8000-8500 | 2448 grams | 2.05 l/s/m | < 10 l/s/m | **OK** | | | |
| | | 8500-9000 | 3046 grams | 2.55 l/s/m | < 10 l/s/m | **OK** | | | |
| | | 9000-9500 | 1760 grams | 1.47 l/s/m | < 10 l/s/m | **OK** | | | |
| | | 9500-10000 | 1988 grams | 1.66 l/s/m | < 10 l/s/m | **OK** | | | |

**Table 4.24** Wave overtopping results for Set 2

| EXPERIMENT OVERTOPPING RESULTS | | | | | LITERATURE CHECK | | | | |
|---|---|---|---|---|---|---|---|---|---|
| **NAME** | **SECTION NO** | **WAVE NUMBER** | **MODEL QUANTITY** | **PROTOTYPE QUANTITY** | **OVERTOPPING RANGE** | **STATUS** | **CLASH RESULTS** | **TAW RESULTS** | **EUROTOP (by S&WdM) RESULTS** |
| SECTION-5 (6-10 tons) | SET-2 | 0-500 | 1760 grams | 1.47 l/s/m | < 10 l/s/m | **OK** | 2.76 l/s/m | 121.10 l/s/m | 4.46 l/s/m |
| | | 500-1000 | 1988 grams | 1.66 l/s/m | < 10 l/s/m | **OK** | | | |

When tables are analyzed, it is concluded that Section 5 conforms to the limits of the experiment. The maximum and minimum wave overtopping amount is measured as 2.55 l/s/m and 1.18 l/s/m respectively. Moreover, CLASH results represent the performance of this structure better than Section 3 and 4. CLASH uses the geometry of the structure as input parameters not stone sizes, thus calculated overtopping rates do not change unless geometry of the structure changes. As the only change between Section 3, 4 and 5 is stone sizes and grading, the CLASH overtopping rate stays the same. However, performance of partly reshaping Section 5 is restricted in terms of change of geometry during the experiment. This could be a reason for less discrepancy between experimental and CLASH results. Eurotop formulation provides a similar overtopping value whereas TAW formula predicts higher overtopping rates.

On the other hand, no significant relation between number of waves and wave overtopping amount can be detected for this section. Both the structure being partly reshaping which limits the change of profile thus the damage and major waves being the main cause for overtopping could be the reasons for the amount of water overtopping the breakwater staying constant throughout the experiment for every 500 wave duration.

## 4.3 Economical Analysis

In this topic a comparison is made on sections alternative to Section-1 that was constructed in Ordu-Giresun Airport as a berm breakwater with respect to economical analysis. Section 2 is out of comparison list due to very high wave overtopping results.

Economical analysis is presented in two parts. Approximate cost of government using the unit prices listed every year for each section is given in the first part. Cost calculation approach of a contractor is assessed in the second part.

### 4.3.1 Approximate Cost of Government – Unit Price Approach

All calculations are made based on Year 2016 Bill of Quantities of Infrastructural Investment of Ministry of Transport, Maritime Affairs and Communications. There are many tables that are listed below. Volume of stones, unit cost of rockfill preparation for each stone class, unit cost of armour paving for each stone class, cost of each section, transportation cost for each section and total cost for each section are summarized in tables between 4.25 and 4.50, respectively.

**Table 4.25** Volume of Section 1

| SECTION 1 | | | | |
|---|---|---|---|---|
| **Range** | **Area ($m^2$)** | **Length (m)** | **Porosity** | **Volume ($m^3$)** |
| **0-0.4 ton** | 236.36 | 1,000.00 | 0.35 | 153,635.95 |
| **0.4-2 ton** | 265.32 | 1,000.00 | 0.35 | 172,457.35 |
| **2-4 ton** | 125.70 | 1,000.00 | 0.35 | 81,707.60 |
| **6-8 ton** | 53.12 | 1,000.00 | 0.35 | 34,529.30 |
| **8-10 ton** | 62.61 | 1,000.00 | 0.35 | 40,698.45 |
| **12-15 ton** | 147.60 | 1,000.00 | 0.35 | 95,941.30 |
| **2-4 ton (toe)** | 4.97 | 1,000.00 | 0.35 | 3,231.15 |
| **TOTAL** | | | | **582,201.10** |

**Table 4.26** Volume of Section 3

| SECTION 3 | | | | |
|---|---|---|---|---|
| **Range** | **Area ($m^2$)** | **Length (m)** | **Porosity** | **Volume ($m^3$)** |
| **0-0.4 ton** | 236.36 | 1,000.00 | 0.35 | 153,635.95 |
| **0.4-2 ton** | 374.98 | 1,000.00 | 0.35 | 243,736.35 |
| **2-4 ton** | 252.69 | 1,000.00 | 0.35 | 164,248.50 |
| **2-4 ton (toe)** | 6.91 | 1,000.00 | 0.35 | 4,490.85 |
| **TOTAL** | | | | **566,111.65** |

**Table 4.27** Volume of Section 4

| SECTION 4 | | | | |
|---|---|---|---|---|
| **Range** | **Area ($m^2$)** | **Length (m)** | **Porosity** | **Volume ($m^3$)** |
| **0-0.4 ton** | 236.36 | 1,000.00 | 0.35 | 153,635.95 |
| **0.4-2 ton** | 338.28 | 1,000.00 | 0.35 | 219,880.05 |
| **2-8 ton** | 300.64 | 1,000.00 | 0.35 | 195,417.95 |
| **2-4 ton (toe)** | 6.91 | 1,000.00 | 0.35 | 4,490.85 |
| **TOTAL** | | | | **573,424.80** |

**Table 4.28** Volume of Section 5

| SECTION 5 | | | | |
|---|---|---|---|---|
| **Range** | **Area ($m^2$)** | **Length (m)** | **Porosity** | **Volume ($m^3$)** |
| **0-0.4 ton** | 236.36 | 1,000.00 | 0.35 | 153,635.95 |
| **0.4-2 ton** | 319.32 | 1,000.00 | 0.35 | 207,559.95 |
| **6-10 ton** | 330.70 | 1,000.00 | 0.35 | 214,953.70 |
| **2-4 ton (toe)** | 6.91 | 1,000.00 | 0.35 | 4,490.85 |
| **TOTAL** | | | | **580,640.45** |

The volume of cross sections are calculated using the cross section designs provided in Chapter 3. The cross-sectional areas of stone ranges are multiplied by 0.35 which is the porosity value for experiments.

Items of rockfill preparation contain dynamite, tarry fuse, exploder, 2.5 $yrd^3$ excavator and compressor. It is the quarrying cost and does not include transportation cost of rock and armour paving on construction site. All quantities and unit prices are determined by government and stable for each year.

**Table 4.29** Unit Cost of Rockfill Preparation for 0-0.400 ton

| ITEM NO | ITEM DEFINITION | UNIT | QUANTITY | UNIT PRICE (TL) | TOTAL (TL) |
|---|---|---|---|---|---|
| **08.106** | **(0 - 0.400) ton CATEGORY ROCKFILL PREPARATION** | **TON** | | | **7.17** |
| 04.101 | DYNAMITE (GOM II) | KG | 0.085 | 10.21 | 0.87 |
| 04.104 | TARRY FUSE | MT | 0.850 | 1.48 | 1.26 |
| 04.105 | EXPLODER (ORDINARY) | NUMBER | 0.850 | 0.62 | 0.53 |
| 03.504 | 2.5 $YRD^3$ EXCAVATOR | HOUR | 0.012 | 155.39 | 1.86 |
| 03.517 | COMPRESSOR (210 Cfm HOSE and REVOLVER) | HOUR | 0.095 | 88.16 | 8.38 |
| **1 $M^3$ PRICE** | | | **A =** | | **12.90** |
| **1 TON PRICE** | | | **A / 1.8 =** | | **7.17** |

1 m$^3$ 0-0,400 ton category rockfill preparation requires 0.085 kg dynamite (GOM II), 0.85 m tarry fuse, 0.85 number exploder (ordinary), 0.012 hour work of 2.5 yrd$^3$ excavator and 0.095 hour work of compressor (210 Cfm hose and revolver). 1 m$^3$ price of 0-0.400 ton category rockfill preparation is found as 12.90 TL. 1 ton price of 0-0.400 ton category rockfill preparation is found as 7.17 TL by dividing m$^3$ price to 1.8.

**Table 4.30** Unit Cost of Rockfill Preparation for 0.400-2 ton

| ITEM NO | ITEM DEFINITION | UNIT | QUANTITY | UNIT PRICE (TL) | TOTAL (TL) |
|---|---|---|---|---|---|
| **08.108** | **(0.400 - 2) ton CATEGORY ROCKFILL PREPARATION** | **TON** | | | **7.67** |
| 04.101 | DYNAMITE (GOM II) | KG | 0.085 | 10.21 | 0.87 |
| 04.104 | TARRY FUSE | MT | 0.850 | 1.48 | 1.26 |
| 04.105 | EXPLODER (ORDINARY) | NUMBER | 0.850 | 0.62 | 0.53 |
| 03.504 | 2.5 YRD³ EXCAVATOR | HOUR | 0.015 | 155.39 | 2.33 |
| 03.517 | COMPRESSOR (210 Cfm HOSE and REVOLVER) | HOUR | 0.100 | 88.16 | 8.82 |
| **1 M³ PRICE** | | | **A =** | | **13.81** |
| **1 TON PRICE** | | | **A / 1.8 =** | | **7.67** |

1 m$^3$ 0.400-2 ton category rockfill preparation requires 0.085 kg dynamite (GOM II), 0.85 m tarry fuse, 0.85 number exploder (ordinary), 0.015 hour work of 2.5 yrd$^3$ excavator and 0.100 hour work of compressor (210 Cfm hose and revolver). 1 m$^3$ price of 0.400-2 ton category rockfill preparation is found as 13.81 TL. 1 ton price of 0,400-2 ton category rockfill preparation is found as 7.67 TL by dividing m$^3$ price to 1.8.

**Table 4.31** Unit Cost of Rockfill Preparation for 2-4 ton

| ITEM NO | ITEM DEFINITION | UNIT | QUANTITY | UNIT PRICE (TL) | TOTAL (TL) |
|---|---|---|---|---|---|
| **08.109/1** | **(2 - 4) ton CATEGORY ROCKFILL PREPARATION** | **TON** | | | **8.85** |
| 04.101 | DYNAMITE (GOM II) | KG | 0.085 | 10.21 | 0.87 |
| 04.104 | TARRY FUSE | MT | 0.850 | 1.48 | 1.26 |
| 04.105 | EXPLODER (ORDINARY) | NUMBER | 0.850 | 0.62 | 0.53 |
| 03.504 | 2.5 YRD³ EXCAVATOR | HOUR | 0.023 | 155.39 | 3.57 |
| 03.517 | COMPRESSOR (210 Cfm HOSE and REVOLVER) | HOUR | 0.110 | 88.16 | 9.70 |
| **1 M³ PRICE** | | | **A =** | | **15.93** |
| **1 TON PRICE** | | | **A / 1.8 =** | | **8.85** |

1 $m^3$ 2-4 ton category rockfill preparation requires 0.085 kg dynamite (GOM II), 0.85 m tarry fuse, 0.85 number exploder (ordinary), 0.023 hour work of 2.5 $yrd^3$ excavator and 0.110 hour work of compressor (210 Cfm hose and revolver). 1 $m^3$ price of 2-4 ton category rockfill preparation is found as 15.93 TL. 1 ton price of 2-4 ton category rockfill preparation is found as 8.85 TL by dividing $m^3$ price to 1.8.

**Table 4.32** Unit Cost of Rockfill Preparation for 4-6 ton

| ITEM NO | ITEM DEFINITION | UNIT | QUANTITY | UNIT PRICE (TL) | TOTAL (TL) |
|---|---|---|---|---|---|
| **08.109/2** | **(4 - 6) ton CATEGORY ROCKFILL PREPARATION** | **TON** | | | **10.18** |
| 04.101 | DYNAMITE (GOM II) | KG | 0.085 | 10.21 | 0.87 |
| 04.104 | TARRY FUSE | MT | 0.850 | 1.48 | 1.26 |
| 04.105 | EXPLODER (ORDINARY) | NUMBER | 0.850 | 0.62 | 0.53 |
| 03.504 | 2.5 YRD³ EXCAVATOR | HOUR | 0.027 | 155.39 | 4.20 |
| 03.517 | COMPRESSOR (210 Cfm HOSE and REVOLVER) | HOUR | 0.130 | 88.16 | 11.46 |
| **1 M³ PRICE** | | | **A =** | | **18.32** |
| **1 TON PRICE** | | | **A / 1.8 =** | | **10.18** |

1 $m^3$ 4-6 ton category rockfill preparation requires 0.085 kg dynamite (GOM II), 0.85 m tarry fuse, 0.85 number exploder (ordinary), 0.027 hour work of 2.5 $yrd^3$ excavator and 0.130 hour work of compressor (210 Cfm hose and revolver). 1 $m^3$ price of 4-6 ton category rockfill preparation is found as 18.32 TL. 1 ton price of 4-6 ton category rockfill preparation is found as 10.18 TL by dividing $m^3$ price to 1.8.

**Table 4.33** Unit Cost of Rockfill Preparation for 6-8 ton

| ITEM NO | ITEM DEFINITION | UNIT | QUANTITY | UNIT PRICE (TL) | TOTAL (TL) |
|---|---|---|---|---|---|
| **08.110/1** | **(6 - 8) ton CATEGORY ROCKFILL PREPARATION** | **TON** | | | **10.52** |
| 04.101 | DYNAMITE (GOM II) | KG | 0.085 | 10.21 | 0.87 |
| 04.104 | TARRY FUSE | MT | 0.850 | 1.48 | 1.26 |
| 04.105 | EXPLODER (ORDINARY) | NUMBER | 0.850 | 0.62 | 0.53 |
| 03.504 | 2.5 YRD³ EXCAVATOR | HOUR | 0.031 | 155.39 | 4.82 |
| 03.517 | COMPRESSOR (210 Cfm HOSE and REVOLVER) | HOUR | 0.130 | 88.16 | 11.46 |
| **1 M³ PRICE** | | | **A =** | | **18.94** |
| **1 TON PRICE** | | | **A / 1.8 =** | | **10.52** |

1 $m^3$ 6-8 ton category rockfill preparation requires 0.085 kg dynamite (GOM II), 0.85 m tarry fuse, 0.85 number exploder (ordinary), 0.031 hour work of 2.5 $yrd^3$ excavator and 0.130 hour work of compressor (210 Cfm hose and revolver). 1 $m^3$

price of 6-8 ton category rockfill preparation is found as 18.94 TL. 1 ton price of 6-8 ton category rockfill preparation is found as 10.52 TL by dividing $m^3$ price to 1.8.

**Table 4.34** Unit Cost of Rockfill Preparation for 8-10 ton

| ITEM NO | ITEM DEFINITION | UNIT | QUANTITY | UNIT PRICE (TL) | TOTAL (TL) |
|---|---|---|---|---|---|
| **08.110/2** | **(8 - 10) ton CATEGORY ROCKFILL PREPARATION** | **TON** | | | **11.18** |
| 04.101 | DYNAMITE (GOM II) | KG | 0.085 | 10.21 | 0.87 |
| 04.104 | TARRY FUSE | MT | 0.850 | 1.48 | 1.26 |
| 04.105 | EXPLODER (ORDINARY) | NUMBER | 0.850 | 0.62 | 0.53 |
| 03.504 | 2.5 YRD³ EXCAVATOR | HOUR | 0.033 | 155.39 | 5.13 |
| 03.517 | COMPRESSOR (210 Cfm HOSE and REVOLVER) | HOUR | 0.140 | 88.16 | 12.34 |
| **1 M³ PRICE** | | | **A =** | | **20.13** |
| **1 TON PRICE** | | | **A / 1.8 =** | | **11.18** |

1 $m^3$ 8-10 ton category rockfill preparation requires 0.085 kg dynamite (GOM II), 0.85 m tarry fuse, 0.85 number exploder (ordinary), 0.033 hour work of 2.5 $yrd^3$ excavator and 0.140 hour work of compressor (210 Cfm hose and revolver). 1 $m^3$ price of 8-10 ton category rockfill preparation is found as 20.13 TL. 1 ton price of 8-10 ton category rockfill preparation is found as 11.18 TL by dividing $m^3$ price to 1.8.

**Table 4.35** Unit Cost of Rockfill Preparation for 6-10 ton

| ITEM NO | ITEM DEFINITION | UNIT | QUANTITY | UNIT PRICE (TL) | TOTAL (TL) |
|---|---|---|---|---|---|
| **08.110/A** | **(6 - 10) ton CATEGORY ROCKFILL PREPARATION** | **TON** | | | **10.85** |
| 04.101 | DYNAMITE (GOM II) | KG | 0.085 | 10.21 | 0.87 |
| 04.104 | TARRY FUSE | MT | 0.850 | 1.48 | 1.26 |
| 04.105 | EXPLODER (ORDINARY) | NUMBER | 0.850 | 0.62 | 0.53 |
| 03.504 | 2.5 YRD³ EXCAVATOR | HOUR | 0.032 | 155.39 | 4.97 |
| 03.517 | COMPRESSOR (210 Cfm HOSE and REVOLVER) | HOUR | 0.135 | 88.16 | 11.90 |
| **1 M³ PRICE** | | | **A =** | | **19.53** |
| **1 TON PRICE** | | | **A / 1.8 =** | | **10.85** |

1 $m^3$ 6-10 ton category rockfill preparation requires 0.085 kg dynamite (GOM II), 0.85 m tarry fuse, 0.85 number exploder (ordinary), 0.032 hour work of 2.5 $yrd^3$ excavator and 0.135 hour work of compressor (210 Cfm hose and revolver). 1 $m^3$ price of 6-10 ton category rockfill preparation is found as 19.53 TL. 1 ton price of 6-10 ton category rockfill preparation is found as 10.85 TL by dividing $m^3$ price to 1.8.

**Table 4.36** Unit Cost of Rockfill Preparation for 12-15 ton

| ITEM NO | ITEM DEFINITION | UNIT | QUANTITY | UNIT PRICE (TL) | TOTAL (TL) |
|---|---|---|---|---|---|
| **08.110/4** | **(12 - 15) ton CATEGORY ROCKFILL PREPARATION** | **TON** | | | **13.34** |
| 04.101 | DYNAMITE (GOM II) | KG | 0.085 | 10.21 | 0.87 |
| 04.104 | TARRY FUSE | MT | 0.850 | 1.48 | 1.26 |
| 04.105 | EXPLODER (ORDINARY) | NUMBER | 0.850 | 0.62 | 0.53 |
| 03.504 | 2.5 YRD³ EXCAVATOR | HOUR | 0.041 | 155.39 | 6.37 |
| 03.517 | COMPRESSOR (210 Cfm HOSE and REVOLVER) | HOUR | 0.170 | 88.16 | 14.99 |
| **1 M³ PRICE** | | | **A =** | | **24.02** |
| **1 TON PRICE** | | | **A / 1.8 =** | | **13.34** |

1 m$^3$ 12-15 ton category rockfill preparation requires 0.085 kg dynamite (GOM II), 0.85 m tarry fuse, 0.85 number exploder (ordinary), 0.041 hour work of 2.5 yrd$^3$ excavator and 0.170 hour work of compressor (210 Cfm hose and revolver). 1 m$^3$ price of 12-15 ton category rockfill preparation is found as 24.02 TL. 1 ton price of 12-15 ton category rockfill preparation is found as 13.34 TL by dividing m$^3$ price to 1.8.

**Table 4.37** Unit Cost of Rockfill Preparation for 2-8 ton

| ITEM NO | ITEM DEFINITION | UNIT | QUANTITY | UNIT PRICE (TL) | TOTAL (TL) |
|---|---|---|---|---|---|
| **08.110/S** | **(2 - 8) ton CATEGORY ROCKFILL PREPARATION** | **TON** | | | **9.85** |
| 04.101 | DYNAMITE (GOM II) | KG | 0.085 | 10.21 | 0.87 |
| 04.104 | TARRY FUSE | MT | 0.850 | 1.48 | 1.26 |
| 04.105 | EXPLODER (ORDINARY) | NUMBER | 0.850 | 0.62 | 0.53 |
| 03.504 | 2.5 YRD³ EXCAVATOR | HOUR | 0.027 | 155.39 | 4.20 |
| 03.517 | COMPRESSOR (210 Cfm HOSE and REVOLVER) | HOUR | 0.123 | 88.16 | 10.87 |
| **1 M³ PRICE** | | | **A =** | | **17.73** |
| **1 TON PRICE** | | | **A / 1.8 =** | | **9.85** |

1 m$^3$ 2-8 ton category rockfill preparation requires 0.085 kg dynamite (GOM II), 0.85 m tarry fuse, 0.85 number exploder (ordinary), 0.027 hour work of 2.5 yrd$^3$ excavator and 0.123 hour work of compressor (210 Cfm hose and revolver). 1 m$^3$ price of 2-8 ton category rockfill preparation is found as 17.73 TL. 1 ton price of 2-8 ton category rockfill preparation is found as 9.85 TL by dividing m$^3$ price to 1.8.

The specific weight of the stone is accepted as 1.8 ton/m$^3$ in 2016 Unit Price List of Government. It has to be stayed loyal to that list in preparing tables and calculations.

Moreover, for 2-8 ton stone class there is not any unit price definition so interpolation is made using 2-4, 4-6 and 6-8 ton unit price explanations. The new item is described as 08.110/S in Table 4.37.

Item of armour paving contains rockfill preparation and buldozer. It is combination of quarrying cost and paving of stones on construction site according to any projects but does not include transportation cost. All quantities and unit prices are determined by government and stable for each year.

**Table 4.38** Unit Cost of Armour Paving for 0-0.400 ton

| ITEM NO | ITEM DEFINITION | UNIT | QUANTITY | UNIT PRICE (TL) | TOTAL (TL) |
|---|---|---|---|---|---|
| **34.006** | **(0 - 0.400) ton ARMOUR PAVING (OVERLAND)** | **TON** | | | **10.75** |
| 08.106 | (0 - 0.400) ton CATEGORY ROCKFILL PREPARATION | TON | 1.00 | 7.17 | 7.17 |
| 03.511 | BULDOZER | HOUR | 0.010 | 142.54 | 1.43 |
| **25 % CONTRACTOR'S PROFIT and GENERAL EXPENSES** | | | | | **2.15** |
| **1 TON PRICE** | | | | | **10.75** |

1 ton of 0-0.400 ton armour paving (overland) requires 1 ton of 0-0.400 ton category rockfill preparation which is summarised in Table 4.29 and 0.010 hour work of buldozer. 1 ton price is found as 10.75 TL by adding 25% contractor profit and general expenses.

**Table 4.39** Unit Cost of Armour Paving for 0.400-2 ton

| ITEM NO | ITEM DEFINITION | UNIT | QUANTITY | UNIT PRICE (TL) | TOTAL (TL) |
|---|---|---|---|---|---|
| **34.008** | **(0.400 - 2) ton ARMOUR PAVING (OVERLAND)** | **TON** | | | **11.53** |
| 08.108 | (0.400 - 2) ton CATEGORY ROCKFILL PREPARATION | TON | 1.00 | 7.67 | 7.67 |
| 03.504 | 2.5 $YRD^3$ EXCAVATOR | HOUR | 0.010 | 155.39 | 1.55 |
| **25 % CONTRACTOR'S PROFIT and GENERAL EXPENSES** | | | | | **2.31** |
| **1 TON PRICE** | | | | | **11.53** |

1 ton of 0.400-2 ton armour paving (overland) requires 1 ton of 0.400-2 ton category rockfill preparation which is summarised in Table 4.30 and 0.010 hour work of excavator. 1 ton price is found as 11.53 TL by adding 25% contractor profit and general expenses.

**Table 4.40** Unit Cost of Armour Paving for 2-4 ton

| ITEM NO | ITEM DEFINITION | UNIT | QUANTITY | UNIT PRICE (TL) | TOTAL (TL) |
|---|---|---|---|---|---|
| **34.009/1** | **(2- 4) ton ARMOUR PAVING (OVERLAND)** | **TON** | | | **13.20** |
| 08.109/1 | (2 - 4) ton CATEGORY ROCKFILL PREPARATION | TON | 1.00 | 8.85 | 8.85 |
| 03.504 | 2.5 YRD³ EXCAVATOR | HOUR | 0.011 | 155.39 | 1.71 |
| **25 % CONTRACTOR'S PROFIT and GENERAL EXPENSES** | | | | | **2.64** |
| **1 TON PRICE** | | | | | **13.20** |

1 ton of 2-4 ton armour paving (overland) requires 1 ton of 2-4 ton category rockfill preparation which is summarised in Table 4.31 and 0.011 hour work of excavator. 1 ton price is found as 13.20 TL by adding 25% contractor profit and general expenses.

**Table 4.41** Unit Cost of Armour Paving for 4-6 ton

| ITEM NO | ITEM DEFINITION | UNIT | QUANTITY | UNIT PRICE (TL) | TOTAL (TL) |
|---|---|---|---|---|---|
| **34.009/2** | **(4 - 6) ton ARMOUR PAVING (OVERLAND)** | **TON** | | | **15.25** |
| 08.109/2 | (4 - 6) ton CATEGORY ROCKFILL PREPARATION | TON | 1.00 | 10.18 | 10.18 |
| 03.504 | 2.5 YRD³ EXCAVATOR | HOUR | 0.013 | 155.39 | 2.02 |
| **25 % CONTRACTOR'S PROFIT and GENERAL EXPENSES** | | | | | **3.05** |
| **1 TON PRICE** | | | | | **15.25** |

1 ton of 4-6 ton armour paving (overland) requires 1 ton of 4-6 ton category rockfill preparation which is summarised in Table 4.32 and 0.013 hour work of excavator. 1 ton price is found as 15.25 TL by adding 25% contractor profit and general expenses.

**Table 4.42** Unit Cost of Armour Paving for 6-8 ton

| ITEM NO | ITEM DEFINITION | UNIT | QUANTITY | UNIT PRICE (TL) | TOTAL (TL) |
|---|---|---|---|---|---|
| **34.010/1** | **(6 - 8) ton ARMOUR PAVING (OVERLAND)** | **TON** | | | **16.06** |
| 08.110/1 | (6 - 8) ton CATEGORY ROCKFILL PREPARATION | TON | 1.00 | 10.52 | 10.52 |
| 03.504 | 2.5 YRD³ EXCAVATOR | HOUR | 0.015 | 155.39 | 2.33 |
| **25 % CONTRACTOR'S PROFIT and GENERAL EXPENSES** | | | | | **3.21** |
| **1 TON PRICE** | | | | | **16.06** |

1 ton of 6-8 ton armour paving (overland) requires 1 ton of 6-8 ton category rockfill preparation which is summarised in Table 4.33 and 0.015 hour work of excavator. 1 ton price is found as 16.06 TL by adding 25% contractor profit and general expenses.

**Table 4.43** Unit Cost of Armour Paving for 8-10 ton

| ITEM NO | ITEM DEFINITION | UNIT | QUANTITY | UNIT PRICE (TL) | TOTAL (TL) |
|---|---|---|---|---|---|
| **34.010/2** | **(8 - 10) ton ARMOUR PAVING (OVERLAND)** | **TON** | | | **17.48** |
| 08.110/2 | (8 - 10) ton CATEGORY ROCKFILL PREPARATION | TON | 1.00 | 11.18 | 11.18 |
| 03.504 | 2.5 YRD³ EXCAVATOR | HOUR | 0.018 | 155.39 | 2.80 |
| **25 % CONTRACTOR'S PROFIT and GENERAL EXPENSES** | | | | | **3.50** |
| **1 TON PRICE** | | | | | **17.48** |

1 ton of 8-10 ton armour paving (overland) requires 1 ton of 8-10 ton category rockfill preparation which is summarised in Table 4.34 and 0.018 hour work of excavator. 1 ton price is found as 17.48 TL by adding 25% contractor profit and general expenses.

**Table 4.44** Unit Cost of Armour Paving for 6-10 ton

| ITEM NO | ITEM DEFINITION | UNIT | QUANTITY | UNIT PRICE (TL) | TOTAL (TL) |
|---|---|---|---|---|---|
| **34.010/A** | **(6 - 10) ton ARMOUR PAVING (OVERLAND)** | **TON** | | | **16.86** |
| 08.110/A | (6 - 10) ton CATEGORY ROCKFILL PREPARATION | TON | 1.00 | 10.85 | 10.85 |
| 03.504 | 2.5 YRD³ EXCAVATOR | HOUR | 0.017 | 155.39 | 2.64 |
| **25 % CONTRACTOR'S PROFIT and GENERAL EXPENSES** | | | | | **3.37** |
| **1 TON PRICE** | | | | | **16.86** |

1 ton of 6-10 ton armour paving (overland) requires 1 ton of 6-10 ton category rockfill preparation which is summarised in Table 4.35 and 0.017 hour work of excavator. 1 ton price is found as 16.86 TL by adding 25% contractor profit and general expenses.

**Table 4.45** Unit Cost of Armour Paving for 12-15 ton

| ITEM NO | ITEM DEFINITION | UNIT | QUANTITY | UNIT PRICE (TL) | TOTAL (TL) |
|---|---|---|---|---|---|
| **34.010/4** | **(12 - 15) ton ARMOUR PAVING (OVERLAND)** | **TON** | | | **21.34** |
| 08.110/4 | (12 - 15) ton CATEGORY ROCKFILL PREPARATION | TON | 1.00 | 13.34 | 13.34 |
| 03.504 | 2.5 YRD³ EXCAVATOR | HOUR | 0.024 | 155.39 | 3.73 |
| **25 % CONTRACTOR'S PROFIT and GENERAL EXPENSES** | | | | | **4.27** |
| **1 TON PRICE** | | | | | **21.34** |

1 ton of 12-15 ton armour paving (overland) requires 1 ton of 12-15 ton category rockfill preparation which is summarised in Table 4.36 and 0.024 hour work of excavator. 1 ton price is found as 21.34 TL by adding 25% contractor profit and general expenses.

**Table 4.46** Unit Cost of Armour Paving for 2-8 ton

| ITEM NO | ITEM DEFINITION | UNIT | QUANTITY | UNIT PRICE (TL) | TOTAL (TL) |
|---|---|---|---|---|---|
| **34.010/S** | **(2 - 8) ton ARMOUR PAVING (OVERLAND)** | **TON** | | | **14.84** |
| 08.110/4 | (2 - 8) ton CATEGORY ROCKFILL PREPARATION | TON | 1.00 | 9.85 | 9.85 |
| 03.504 | 2.5 YRD³ EXCAVATOR | HOUR | 0.013 | 155.39 | 2.02 |
| **25 % CONTRACTOR'S PROFIT and GENERAL EXPENSES** | | | | | **2.97** |
| **1 TON PRICE** | | | | | **14.84** |

1 ton of 2-8 ton armour paving (overland) requires 1 ton of 2-8 ton category rockfill preparation which is summarised in Table 4.37 and 0.013 hour work of excavator. 1 ton price is found as 14.84 TL by adding 25% contractor profit and general expenses.

Overland unit prices are taken from list due to the fact that berms are above still water level. Construction is done by trucks on land and there is no need to use floating vehicles in building berm breakwater.

Cost of each section is tabulated in Table 4.47 and Table 4.50 by using unit costs of armour paving that are summarised in Table 4.38 and Table 4.46. It is acquired by multiplying weights of the stone ranges with the unit cost of armour paving.

**Table 4.47** Cost of Section 1

| RANGE | VOLUME ($m^3$) | WEIGHT (TON) | 2016 UNIT COST of ARMOUR PAVING (TON/TL) (OVERLAND) | TOTAL (TL) |
|---|---|---|---|---|
| **0-0.4 ton** | 153,635.95 | 276,544.71 | 10.75 | 2,971,583.53 |
| **0.4-2 ton** | 172,457.35 | 310,423.23 | 11.53 | 3,579,141.04 |
| **2-4 ton** | 81,707.60 | 147,073.68 | 13.20 | 1,941,242.05 |
| **6-8 ton** | 34,529.30 | 62,152.74 | 16.06 | 998,394.42 |
| **8-10 ton** | 40,698.45 | 73,257.21 | 17.48 | 1,280,317.72 |
| **12-15 ton** | 95,941.30 | 172,694.34 | 21.34 | 3,684,727.32 |
| **2-4 ton (toe)** | 3,231.15 | 5,816.07 | 13.20 | 76,766.96 |
| **TOTAL COST of SECTION 1 (TL)** | | | | **14,532,173.05** |

**Table 4.48** Cost of Section 3

| RANGE | VOLUME ($m^3$) | WEIGHT (TON) | 2016 UNIT COST of ARMOUR PAVING (TON/TL) (OVERLAND) | TOTAL (TL) |
|---|---|---|---|---|
| **0-0.4 ton** | 153,635.95 | 276,544.71 | 10.75 | 2,971,583.53 |
| **0.4-2 ton** | 243,736.35 | 438,725.43 | 11.53 | 5,058,449.37 |
| **2-4 ton** | 164,248.50 | 295,647.30 | 13.20 | 3,902,281.97 |
| **2-4 ton (toe)** | 4,490.85 | 8,083.53 | 13.20 | 106,695.42 |
| **TOTAL COST of SECTION 3 (TL)** | | | | **12,039,010.29** |

**Table 4.49** Cost of Section 4

| RANGE | VOLUME ($m^3$) | WEIGHT (TON) | 2016 UNIT COST of ARMOUR PAVING (TON/TL) (OVERLAND) | TOTAL (TL) |
|---|---|---|---|---|
| **0-0.4 ton** | 153,635.95 | 276,544.71 | 10.75 | 2,971,583.53 |
| **0.4-2 ton** | 219,880.05 | 395,784.09 | 11.53 | 4,563,341.08 |
| **2-8 ton** | 195,417.95 | 351,752.31 | 14.84 | 5,220,004.28 |
| **2-4 ton (toe)** | 4,490.85 | 8,083.53 | 13.20 | 106,702.60 |
| **TOTAL COST of SECTION 4 (TL)** | | | | **12,861,631.49** |

**Table 4.50** Cost of Section 5

| RANGE | VOLUME ($m^3$) | WEIGHT (TON) | 2016 UNIT COST of ARMOUR PAVING (TON/TL) (OVERLAND) | TOTAL (TL) |
|---|---|---|---|---|
| **0-0.4 ton** | 153,635.95 | 276,544.71 | 10.75 | 2,971,583.53 |
| **0.4-2 ton** | 207,559.95 | 373,607.91 | 11.53 | 4,307,652.50 |
| **6-10 ton** | 214,953.70 | 386,916.66 | 16.86 | 6,523,414.89 |
| **2-4 ton (toe)** | 4,490.85 | 8,083.53 | 13.20 | 106,702.60 |
| **TOTAL COST of SECTION 5 (TL)** | | | | **13,909,353.51** |

All items and costs that are mentioned above include cost of labour, machinery and equipment for placement of stones except for transportation. In that case, transportation of stone from quarry to construction site is called as transportation cost. It is calculated by using a formula in Year 2016 Bill of Quantities of Infrastructural Investment of Ministry of Transport, Maritime Affairs and Communications.

If the transport distance is less than 10 kilometres, the formula is

$$TP = 0.00017 \times K \times A \times \sqrt{M} \tag{4.1}$$

where

TP = Unit price (25% contractor profix excluding, TL/ton)

K = Year 2016 transport coefficient ( = 225)

A = Road difficulty coefficient (= 1.00)

M = Transport distance (meter)

The distance between stone quarry and construction area of Ordu-Giresun Airport Berm Breakwater is 5.6 km. All calculations are made based on that distance in Table 4.51 and Table 4.52.

**Table 4.51** Unit Price of Transportation

| | **QUANTITY** | **UNIT** |
|---|---|---|
| **M** | 5,600.00 | m |
| **K** | 225.00 | |
| **A** | 1.00 | |
| **TP** | 2.86 | TL/ton |
| **25% Contractor Profit** | 0.72 | TL/ton |
| **Unit Price** | **3.58** | **TL/ton** |

**Table 4.52** Transportation Cost of Sections

| SECTION | TOTAL VOLUME ($m^3$) | TOTAL WEIGHT (TON) | 2016 UNIT COST of ARMOUR PAVING (TON/TL) (OVERLAND) | TOTAL (TL) |
|---|---|---|---|---|
| **1** | 582,201.10 | 1,047,961.98 | 3.58 | **3,751,703.89** |
| **3** | 566,111.65 | 1,019,000.97 | 3.58 | **3,648,023.47** |
| **4** | 573,424.80 | 1,032,164.64 | 3.58 | **3,695,149.41** |
| **5** | 580,640.45 | 1,045,152.81 | 3.58 | **3,741,647.06** |

In Table 4.53, armour paving cost that is tabulated between Table 4.47 and Table 4.50 is added to transportation cost to find out total cost of each section.

**Table 4.53** Total Cost of Sections

| SECTION | ARMOUR PAVING COST (TL) | TRANSPORTATION COST (TL) | TOTAL (TL) |
|---|---|---|---|
| **1** | 14,532,173.05 | 3,751,703.89 | **18,283,876.94** |
| **3** | 12,039,010.29 | 3,648,023.47 | **15,687,033.76** |
| **4** | 12,861,631.49 | 3,695,149.41 | **16,556,780.90** |
| **5** | 13,909,353.51 | 3,741,647.06 | **17,651,000.57** |

The volume and weight of the cross sections are not very different however Section 1 has the weight and volume while Section 3 has the smallest as expected. These differences are reflected in the armor paving cost where Section 1 is 2.5 million TL expensive than Section 3. The other sections are calculated to cost less than Section 1 as smaller stone sizes were used even if the volumes were almost similar. The difference in stone sizes are not reflected in the transportation cost as significant as the paving cost. The total value calculated which also includes 25 % profit and general expenses is between 15-18 million TL.

### 4.3.2 Approach of Contractors to Cost Estimation

In this section, cost estimation is performed from the point of view of a contractor. Contractor's cost estimation procedure is different from approximate cost of government. Even though approximate cost of government is constant for any construction in Turkey since unit prices are set for a year without additional conditions, contractor's cost estimation is calculated according to construction site conditions. For instance, specific weight of stone is assumed as 1.8 ton/m$^3$ in government approximate cost, however for Ordu-Giresun Airport Berm Breakwater specific weight of stone is 2.55 ton/m$^3$ in quarry. All assumptions and approachment techniques are explained below where each number is also used in Tables to provide explanatory note.

(1) Estimated truck volume is 15 m$^3$. Because the placing of rock is diffucult for large stones, calculations are done considering number of stones that can be carried in one truck.

The trucks are assumed to carry 1 stone for 12-15 ton range, 2 stones for 8-10 ton range, 3 stones for 6-10 ton range, 4 stones for 6-8 ton range, 6 stones for 2-8 ton range, 11 stones for 2-4 ton range, 35 stones for 0.4-2 ton range and 270 stones for 0-0.4 ton range. Figures 4.42, 4.43, 4.44 and 4.45 show loading, transporting, unloading and arranging procedures.

**Figure 4.42** Loading of stones, Retrieved from İbrahim Ahmetoğlu, Project Manager of Cengiz Insaat.

**Figure 4.43** Transportation of stones to site by trucks, Retrieved from İbrahim Ahmetoğlu, Project Manager of Cengiz Insaat.

**Figure 4.44** Unloading of stones into the sea, Retrieved from İbrahim Ahmetoğlu, Project Manager of Cengiz Insaat.

(2) Estimated distance is taken as 12 km (5.6 + 5.6 km=11.2 km far from quarry and 0.8 km travel inside site) and loading time will be less for large stones than smaller ones. Resting time and driver fault is considered as well.

(3) Daily trip is calculated as 16 hours per day with two shifts.

(4) Loading & Unloading: It takes longer time for small stones than large stones. Duration of loading 0-4 ton is considered as 15 minutes while above 4 ton is

taken as 10 minutes. Excavator quantity is specified according to number of trucks. Small excavators (20 - 25 ton capacity) will be used for 0-4 ton, bigger excavators (35 - 50 ton capacity) will be used for larger stones.

(5) 2 small excavator will be enough for breaking of stones on quarry.

(6) 2 electrical cranes will be used for placing of rocks if excavator does not. Due to economical reasons electrical cranes have to be rented.

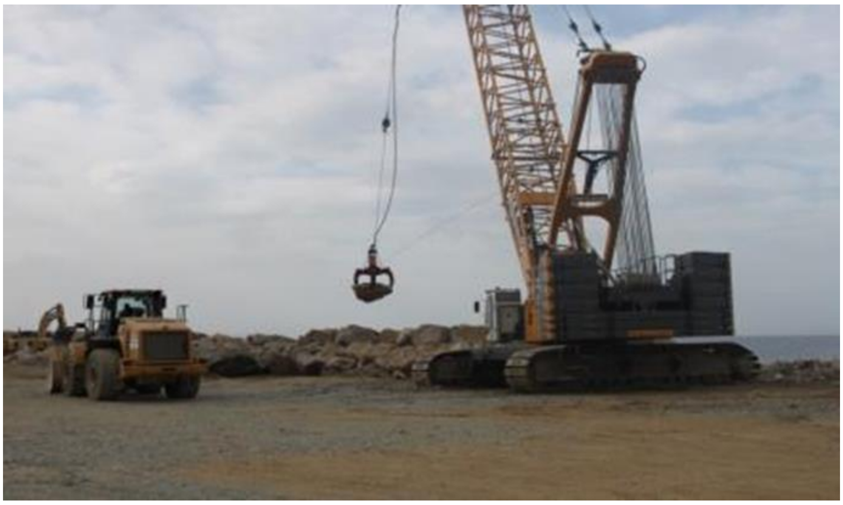

**Figure 4.45** Arrangement of rockfill with Crane, Retrieved from İbrahim Ahmetoğlu, Project Manager of Cengiz Insaat.

(7) Grader will be used for road opening works.

(8) Monthly cost is multiplied with 1,8 due to night shift works.

(9) Fuel-Oil cost was reduced with 18% due to tax and 14% for discount.

The list price of diesel fuel is 3.70 TL/liter. Cost estimation has to be calculated without any tax. Moreover, for this kind of projects, contractor can buy fuel-oil up to 14% discount for bulk purchase.

(10) Construction continues day and night. Since staff quantities are doubled.

(11) Cost of 100 TL/m is assumed and 1000 meter road is opened with respect to considering the past experiences.

(12) Contractor purchased two quarries for about 5,000,000 $ and they transported approximately 36,000,000 ton of stones, so interpolation is made between stone quantity and purchase price.

Divani that is shown in Figure 4.46 and Ayrilik that is shown in Figure 4.47 are the quarries 5.6 km and 6.0 km far from construction site, respectively. Divani with a 240,000 $m^2$ surface has 35,000,000 ton reservoir. On the other hand, Ayrilik with a 47,000 $m^2$ surface has 10,000,000 ton reservoir. Due to the low reservoir of Ayrilik, mostly Divani was used. Moreover, only filling materials (0-0.25 ton) for runway, apron and taxiway construction were acquired from Ayrilik. Therefore, Divani is considered for distance calculations for this study.

**Figure 4.46** Divani Quarry, Retrieved from http://documentslide.com/documents/ordu-giresun-havaalani-insaati-27012013.html

**Figure 4.47** Ayrilik Quarry, Retrieved from http://documentslide.com/documents/ordu-giresun-havaalani-insaati-27012013.html

(13) Blasting cost: 10% of blasting material supposed to be used as quarry refuse.

Blasting is the most important work in quarry in the beginning. Patterns, blaster and equipment quantity, consumables, fuel-oil show variety for acquiring of different stone varies. In Figure 4.47, there is a schematic view for blasting of 0-2 ton range and it can be seen that holes are on grid with dimensions of 3.12 m and 3.58 m. For 2-4 ton and 4 ton and above stone range, dimensions are 3.68 m and 4.23 m, 4.03 and 4.63, respectively.

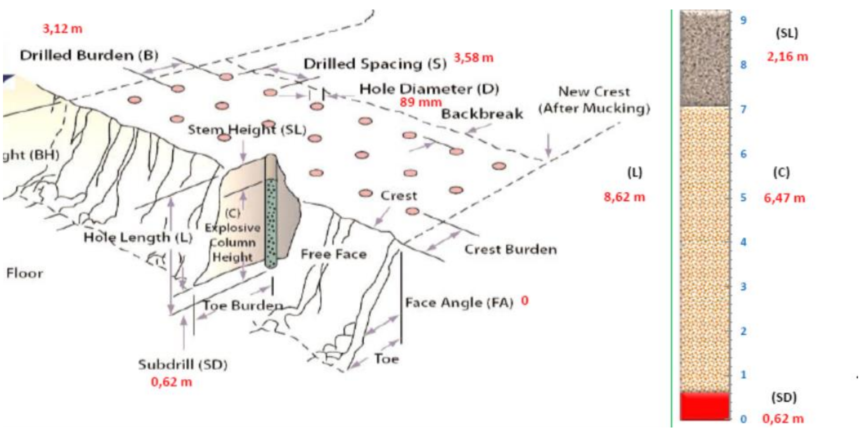


**Figure 4.48** Schematic view of blasting plan for 0-2 ton stone range, Retrieved from Uğur Kara, owner of blasting company.

Five main elements of blasting cost are summarised in Table 4.52 for different stone size ranges. Consumables consist of bit, shank, revolver, service, air filter, oil filter, diesel filter, hydraulic filter, compressor filter, dust filter and seperator. Fuel-oil cost includes diesel, engine oil, hyrdraulic oil, revolver oil, grease oil. Salary, social insurance institution, income tax and stamp tax are the third element of blasting cost. Mobilization and catering cost involves transport of machine, transport insurance, food expense, accommodation and communication expenses. Lastly, blaster and equipment cost contains all materials and expenditures related to blasting such as exploder, anfo, detonation cord, dynamite.

**Table 4.54** Blasting Cost Analysis for Stone Ranges

| BLASTING COST | | | |
|---|---|---|---|
| **Explanation** | **0-2 ton (TL/m³)** | **2-4 ton (TL/m³)** | **4 ton and above (TL/m³)** |
| **Consumables** | 0.14 | 0.10 | 0.09 |
| **Fuel-Oil** | 0.40 | 0.29 | 0.25 |
| **Staff and SSI Payment** | 0.09 | 0.06 | 0.05 |
| **Mobilization and Catering** | 0.13 | 0.12 | 0.11 |
| **Blaster and Equipment** | 1.27 | 1.22 | 1.20 |
| **Total Cost (TL/m³)** | 2.03 | 1.79 | 1.69 |
| **Contractor's Profit (10%)** | 0.20 | 0.18 | 0.17 |
| **Grand Total (TL/m³)** | **2.23** | **1.97** | **1.86** |

In Chapter 4.3.1, it is stated that specific weight of stone is taken as 1,8 ton/m$^3$ for the calculation of governmental cost estimation. However, in this chapter, specific weight of stone has to be taken as 2.55 ton/m$^3$ that is given in Table 3.1.

Machinery rental prices and fuel-oil consumption values are determined by regarding statement of market for year 2016.

Staff number is not changed because tonnage of all sections are close to each other.

#### 4.3.2.1 Contractor's Cost Estimation of Section 1

In Table 4.55, cross-section areas of all stone ranges calculated and total weight in ton is found by multiplying volume to 2.55.

**Table 4.55** Data of Section 1

| DATA | | |
|---|---|---|
| **RANGE** | **VOLUME ($m^3$)** | **WEIGHT (TON)** |
| **0-0.4 ton** | 153,635.95 | 391,771.67 |
| **0.4-2 ton** | 172,457.35 | 439,766.24 |
| **2-4 ton** | 81,707.60 | 208,354.38 |
| **6-8 ton** | 34,529.30 | 88,049.72 |
| **8-10 ton** | 40,698.45 | 103,781.05 |
| **12-15 ton** | 95,941.30 | 244,650.32 |
| **2-4 ton (toe)** | 3,231.15 | 8,239.43 |

In Table 4.56, estimation for working days is done. It is assumed that it takes 90 days to finish 1 km of cross section. Day and night shifts is assumed. Moreover, two Sunday in a month would be rest day for work site.

**Table 4.56** Estimation for Section 1

| ESTIMATION | | |
|---|---|---|
| **Starting Date** | 2.1.2017 | |
| **Working Days** | 90 | days |
| **Finishing Date** | 8.4.2017 | (Include Sunday & official holidays) |
| **Shift** | Day & Night | |

In Table 4.57, the needed machinery park is calculated. Weight of one truck to transport different stone ranges is expressed in (1) in Chapter 4.3.2. Required total trip for each range is found by dividing weight of stone to weight of one truck. Duration of transportation for each trip is expressed in (2) in Chapter 4.3.2. Required trip number for each range is found by dividing working hours in a day to duration of each trip which is commented in (3) in Chapter 4.3.2. Required days to transport each range by one truck is found by dividing required total trip to daily trip. Then, required days is divided to working days that is estimated in Table 4.56 to identify required truck quantities. The divisions are all rounded up. Calculation of excavator quantitites are expounded in (4) in Chapter 4.3.2. It consists of loading from quarry and unloading and paving armour in construction site. The assumptions are as in (4). Excavator quantities are found by dividing duration of loading (10 min. or 15 min.)

to division of duration of transportation for each trip to required truck quantities. The divisions are not rounded up however all decimal quantities are summed and then rounded up for cost calculation which is summarised in Table 4.58. Excavator necessity for breaking and sorting in quarry is assumed as 2 for each cross-section and expressed in (5) in Chapter 4.3.2. It does not change section to section because all cross-section weights are similar to each other.

**Table 4.57** Calculation of Machinery Park for Section 1

| CALCULATION OF MACHINERY PARK | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|
| **Range** | **Weight Of One Truck (ton) (1)** | **Required Total Trip** | **Duration Per Truck Each Trip (min) (2)** | **Daily Trip (3)** | **Required Days** | **Required Truck Quantites** | **Excavator Quantites** | | |
| | | | | | | | **Loading (4)** | **Breaking** | **Unloading (4)** |
| **0-0.4 ton** | 54.00 | 7,255.00 | 75.00 | 13.00 | 558.00 | 6.00 | 1.20 | 2 (5) | 1.20 |
| **0.4-2 ton** | 42.00 | 10,471.00 | 72.00 | 13.00 | 805.00 | 9.00 | 1.88 | | 1.88 |
| **2-4 ton** | 33.00 | 6,314.00 | 70.00 | 14.00 | 451.00 | 5.00 | 1.07 | | 1.07 |
| **6-8 ton** | 28.00 | 3,145.00 | 68.00 | 14.00 | 225.00 | 3.00 | 0.44 | | 0.44 |
| **8-10 ton** | 18.00 | 5,766.00 | 65.00 | 15.00 | 384.00 | 4.00 | 0.62 | | 0.62 |
| **12-15 ton** | 13.50 | 18,122.00 | 60.00 | 16.00 | 1,133.00 | 13.00 | 2.17 | | 2.17 |
| **2-4 ton (toe)** | 33.00 | 250.00 | 70.00 | 14.00 | 18.00 | - | - | | - |

In Table 4.58, cost calculation of machinery park is summarised. Small, big excavators and trucks quantities come from Table 4.57. Crane and grader are explained in (6) and (7) of Chapter 4.3.2, respectively. 1 water truck is required for construction site to prevent dust. 1 fuel-oil tank is enough to distribute fuel-oil to machines. 3 passenger car is needed for technical staffs. Rental cost is commented in (8) in Chapter 4.3.2. Montly rental prices and fuel-oil consumption average are determined according to market conditions. Moreover, non-working days are regarded. Fuel-oil unit cost is expressed in (9) in Chapter 4.3.2. Total cost is found by summing up total rental cost and total fuel-oil cost.

**Table 4.58** Cost Calculation of Machinery Park for Section 1

| MACHINERY COST | | | | | | |
|---|---|---|---|---|---|---|
| | Quantity | Rental Prices/Month (TL) | Rental Cost (TL) (8) | Fuel-Oil Consumption | Fuel Oil (TL/l) (9) | Fuel-Oil Cost (TL) |
| **Small Excavator (20-25 ton capacity)** | 10.00 | 12,500.00 | 720,000.00 | 15 lt/hr | 2.70 | 582,467.80 |
| **Big Excavator (35-50 ton capacity)** | 6.00 | 22,500.00 | 777,600.00 | 20 lt/hr | 2.70 | 466,560.00 |
| **Trucks** | 40.00 | 8,000.00 | 1,843,200.00 | 0,5 lt/ km | 2.70 | 831,432.60 |
| **Crane (6)** | 2.00 | 25,000.00 | 288,000.00 | 3000 TL/month | - | 9,600.00 |
| **Water Truck** | 1.00 | 6,000.00 | 34,560.00 | 0,2 lt/km | 2.70 | 2,430.00 |
| **Fuel-Oil Truck** | 1.00 | 6,000.00 | 34,560.00 | 0,2 lt/km | 2.70 | 2,430.00 |
| **Grader (7)** | 1.00 | 19,000.00 | 109,440.00 | 16 lt/hr | 2.70 | 38,880.00 |
| **Passenger Car** | 3.00 | 1,250.00 | 21,600.00 | 0,07 lt/km | 2.70 | 7,654.50 |
| **Total (TL)** | | | 3,828,960.00 | | | 1,941,454.90 |
| **Grand Total (TL)** | | | | | | **5,770,414.90** |

In Table 4.59, staff cost is determined according to day and night work. Their salaries are multiplied working month including non-working days. They are all same in each cross-section due to the fact that total weight of each section are similar to each other.

**Table 4.59** Cost Calculation of Staff for Section 1

| STAFF COST | | | |
|---|---|---|---|
| | Quantity (10) | Salary/ Month (TL) | Total Cost (TL) |
| **Reversing Assistant** | 8.00 | 1,500.00 | 38,400.00 |
| **Cartographer** | 6.00 | 3,250.00 | 62,400.00 |
| **Project Manager** | 2.00 | 15,000.00 | 96,000.00 |
| **Site Manager** | 2.00 | 7,500.00 | 48,000.00 |
| **Site Engineer** | 4.00 | 4,000.00 | 51,200.00 |
| **Unqualified Staff** | 4.00 | 1,500.00 | 19,200.00 |
| **Warehouse Staff** | 2.00 | 1,500.00 | 9,600.00 |
| **Technical Supervisor** | 2.00 | 2,500.00 | 16,000.00 |
| **Check Weigher Staff** | 2.00 | 1,500.00 | 9,600.00 |
| **Night Watchman** | 2.00 | 1,500.00 | 9,600.00 |
| **Accounting Staff** | 4.00 | 1,750.00 | 22,400.00 |
| **Total (TL)** | | | 382,400.00 |
| **Grand Total (TL)** | | | **382,400.00** |

In Table 4.60, mobilization for machinery park, setup cost, environmental impact assessment report are assumed as average values for contractor. Road opening works for quarry is expressed in (11) Chapter 4.3.2. The calculation of quarry operating cost is commented in (12) Chapter 4.3.2. It is found by multiplying weight of section to division of quarry rent cost (5,000,000 $ x 3.00 TL/$ = 15,000,000 TL) to 36.000.000 ton which was realised in Ordu-Giresun Airport Berm Breakwater. For this reason, quarry operating cost show little differences in each cross-section. Blasting of stone is expressed in (13) in Chapter 4.3.2. In Table 4.52, all blasting cost for each stone range is summarised. Total cost for blasting is found by multiplying blasting cost to weight of stone.

**Table 4.60** Calculation of Direct Cost for Section 1

| DIRECT COST | |
|---|---|
| | **Total Cost (TL)** |
| **Mobilization for Machinery Park** | 100,000.00 |
| **Setup Cost** | 150,000.00 |
| **Environmental Impact Assessment Report** | 50,000.00 |
| **Road Opening Works for Quarry (11)** | 100,000.00 |
| **Quarry Operating Cost (12)** | 618,333.33 |
| **Blasting of Stone (13)** | 1,334,692.67 |
| | |
| **Total (TL)** | 2,353,026.00 |
| **Grand Total (TL)** | **2,353,026.00** |

In Table 4.61, all costs that are found in Table 4.58, 4.59, 4.60 and contractor’s profit and general expenses are summed.

**Table 4.61** Cost Calculation Summary for Section 1

| SUMMARY | |
|---|---|
| | **Prices (TL)** |
| **Machinery Cost** | **5,770,414.90** |
| **Staff Cost** | **382,400.00** |
| **Direct Cost** | **2,353,026.00** |
| **Contractor's Profit & General Expenses (25%)** | **2,126,460.23** |
| | |
| **GRAND TOTAL (TL)** | **10,632,301.13** |
| **UNIT COST (TL/ton)** | **7.16** |

All cost calculation procedure for contractor is summarised above in topic 4.3.2.1. For other cross-section same approach is used. In 4.3.2.2, 4.3.2.3 and 4.3.2.4 contractor's cost estimation for other sections are tabulated.

### 4.3.2.2 Contractor's Cost Estimation of Section 3

In Table 4.62, cross-section areas of all stone ranges calculated and total weight in ton is found by multiplying volume to 2.55.

**Table 4.62** Data of Section 3

| DATA | | |
|---|---|---|
| **RANGE** | **VOLUME ($m^3$)** | **WEIGHT (TON)** |
| **0-0.4 ton** | 153,635.95 | 391,771.67 |
| **0.4-2 ton** | 243,736.35 | 621,527.69 |
| **2-4 ton** | 164,248.50 | 418,833.68 |
| **2-4 ton (toe)** | 4,490.85 | 11,451.67 |

Same assumption as in Section 1 is accepted where 90 days is the duration to finish 1 km of cross section. Day and night working is assumed.

In Table 4.63, the needed machinery park is calculated. All calculation methods are same with Section 1 and mentioned in Chapter 4.3.2.1.

**Table 4.63** Calculation of Machinery Park for Section 3

| CALCULATION OF MACHINERY PARK | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|
| **Range** | **Weight Of One Truck (ton) (1)** | **Required Total Trip** | **Duration Per Truck Each Trip (min) (2)** | **Daily Trip (3)** | **Required Days** | **Required Truck Quantites** | **Excavator Quantites** | | |
| | | | | | | | **Loading (4)** | **Breaking** | **Unloading (4)** |
| **0-0.4 ton** | 54.00 | 7,255.00 | 75.00 | 13.00 | 558.00 | 6.00 | 1.20 | 2 (5) | 1.20 |
| **0.4-2 ton** | 42.00 | 14,798.00 | 72.00 | 13.00 | 1,138.00 | 13.00 | 2.71 | | 2.71 |
| **2-4 ton** | 33.00 | 12,692.00 | 70.00 | 14.00 | 907.00 | 10.00 | 2.14 | | 2.14 |
| **2-4 ton (toe)** | 33.00 | 347.00 | 70.00 | 14.00 | 25.00 | - | - | | - |

In Table 4.64, cost calculation of machinery park is summarised. Small, big excavators and trucks quantities come from Table 4.63**.** All calculation methods are same with Section 1 and mentioned in Chapter 4.3.2.1.

**Table 4.64** Cost Calculation of Machinery Park for Section 3

| MACHINERY COST | | | | | | |
|---|---|---|---|---|---|---|
| | Quantity | Rental Prices/Month (TL) | Rental Cost (TL) (8) | Fuel-Oil Consumption | Fuel Oil (TL/l) (9) | Fuel-Oil Cost (TL) |
| Small Excavator (20-25 ton capacity) | 14.00 | 12,500.00 | 1,008,000.00 | 15 lt/hr | 2.70 | 815,454.92 |
| Big Excavator (35-50 ton capacity) | - | 22,500.00 | - | 20 lt/hr | 2.70 | - |
| Trucks | 29.00 | 8,000.00 | 1,336,320.00 | 0,5 lt/ km | 2.70 | 568,490.40 |
| Crane (6) | 2.00 | 25,000.00 | 288,000.00 | 3000 TL/month | - | 9,600.00 |
| Water Truck | 1.00 | 6,000.00 | 34,560.00 | 0,2 lt/km | 2.70 | 2,430.00 |
| Fuel-Oil Truck | 1.00 | 6,000.00 | 34,560.00 | 0,2 lt/km | 2.70 | 2,430.00 |
| Grader (7) | 1.00 | 19,000.00 | 109,440.00 | 16 lt/hr | 2.70 | 38,880.00 |
| Passenger Car | 3.00 | 1,250.00 | 21,600.00 | 0,07 lt/km | 2.70 | 7,654.50 |
| Total (TL) | | | 2,832,480.00 | | | 1,444,939.82 |
| Grand Total (TL) | | | | | | 4,277,419.82 |

In Table 4.65, mobilization for machinery park, setup cost, environmental impact assessment report are assumed average values for contractor. Cost calculation approachments for road opening works for quarry, quarry operating cost and blasting of stone are all same with Section 1 and mentioned in Chapter 4.3.2.1.

**Table 4.65** Cost Calculation of Direct for Section 3

| DIRECT COST | |
|---|---|
| | Total Cost (TL) |
| Mobilization for Machinery Park | 100,000.00 |
| Setup Cost | 150,000.00 |
| Environmental Impact Assessment Report | 50,000.00 |
| Road Opening Works for Quarry (11) | 100,000.00 |
| Quarry Operating Cost (12) | 601,250.00 |
| Blasting of Stone (13) | 1,340,841.92 |
| | |
| Total (TL) | 2,342,091.92 |
| Grand Total (TL) | 2,342,091.92 |

In Table 4.66, all costs that are found in Table 4.64 and 4.65 and contractor's profit and general expenses are summed. In Table 4.66, staff cost is taken as 382,400.00 TL, same as Section 1 as the same team can be used for all sections.

**Table 4.66** Cost Calculation Summary for Section 3

| SUMMARY | |
|---|---|
| | **Prices (TL)** |
| **Machinery Cost** | **4,277,419.82** |
| **Staff Cost** | **382,400.00** |
| **Directt Cost** | **2,342,091.92** |
| **Contractor's Profit & General Expenses (25%)** | **1,750,477.93** |
| | |
| **GRAND TOTAL (TL)** | **8,752,389.67** |
| **UNIT COST (TL/ton)** | **6.06** |

### 4.3.2.3 Contractor's Cost Estimation of Section 4

In Table 4.67, cross-section areas of all stone ranges calculated and total weight in ton is found by multiplying volume to 2.55.

**Table 4.67** Data of Section 4

| DATA | | |
|---|---|---|
| **RANGE** | **VOLUME ($m^3$)** | **WEIGHT (TON)** |
| **0-0.4 ton** | 153,635.95 | 391,771.67 |
| **0.4-2 ton** | 219,880.05 | 560,694.13 |
| **2-8 ton** | 195,417.95 | 498,315.77 |
| **2-4 ton (toe)** | 4,490.85 | 11,451.67 |

Same assumption as in Section 1 is accepted where 90 days is the duration to finish 1 km of cross section. Day and night working is assumed.

In Table 4.68, the needed machinery park is calculated. All calculation methods are same with Section 1 and mentioned in Chapter 4.3.2.1.

**Table 4.68** Calculation of Machinery Park for Section 4

| CALCULATION OF MACHINERY PARK | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|
| Range | Weight Of One Truck (ton) (1) | Required Total Trip | Duration Per Truck Each Trip (min) (2) | Daily Trip (3) | Required Days | Required Truck Quantites | Excavator Quantities | | |
| | | | | | | | Loading (4) | Breaking | Unloading (4) |
| 0-0.4 ton | 54.00 | 7,255.00 | 75.00 | 13.00 | 558.00 | 6.00 | 1.20 | 2 (5) | 1.20 |
| 0.4-2 ton | 42.00 | 13,350.00 | 72.00 | 13.00 | 1,027.00 | 11.00 | 2.29 | | 2.29 |
| 2-8 ton | 30.00 | 16,611.00 | 69.00 | 14.00 | 1,187.00 | 13.00 | 2.83 | | 2.83 |
| 2-4 ton (toe) | 33.00 | 347.00 | 70.00 | 14.00 | 25.00 | - | - | | - |

In Table 4.69, cost calculation of machinery park is summarised. Small, big excavators and trucks quantities come from Table 4.68**.** All calculation methods are same with Section 1 and mentioned in Chapter 4.3.2.1.

**Table 4.69** Cost Calculation of Machinery Park for Section 4

| MACHINERY COST | | | | | | |
|---|---|---|---|---|---|---|
| | Quantity | Rental Prices/Month (TL) | Rental Cost (TL) (8) | Fuel-Oil Consumption | Fuel Oil (TL/l) (9) | Fuel-Oil Cost (TL) |
| Small Excavator (20-25 ton capacity) | 11.00 | 12,500.00 | 792,000.00 | 15 lt/hr | 2.70 | 640,714.58 |
| Big Excavator (35-50 ton capacity) | 3.00 | 22,500.00 | 388,800.00 | 20 lt/hr | 2.70 | 233,280.00 |
| Trucks | 30.00 | 8,000.00 | 1,382,400.00 | 0,5 lt/ km | 2.70 | 608,520.60 |
| Crane (6) | 2.00 | 25,000.00 | 288,000.00 | 3000 TL/month | - | 9,600.00 |
| Water Truck | 1.00 | 6,000.00 | 34,560.00 | 0,2 lt/km | 2.70 | 2,430.00 |
| Fuel-Oil Truck | 1.00 | 6,000.00 | 34,560.00 | 0,2 lt/km | 2.70 | 2,430.00 |
| Grader (7) | 1.00 | 19,000.00 | 109,440.00 | 16 lt/hr | 2.70 | 38,880.00 |
| Passenger Car | 3.00 | 1,250.00 | 21,600.00 | 0,07 lt/km | 2.70 | 7,654.50 |
| Total (TL) | | | 3,051,360.00 | | | 1,543,509.68 |
| Grand Total (TL) | | | | | | **4,594,869.68** |

In Table 4.70, mobilization for machinery park, setup cost, environmental impact assessment report are assumed average values for contractor. Cost calculation methods for road opening works for quarry, quarry operating cost and blasting of stone are all same with Section 1 and mentioned in Chapter 4.3.2.1.

**Table 4.70** Cost Calculation of Direct for Section 4

| DIRECT COST | |
|---|---|
| | **Total Cost (TL)** |
| **Mobilization for Machinery Park** | 100,000.00 |
| **Setup Cost** | 150,000.00 |
| **Environmental Impact Assessment Report** | 50,000.00 |
| **Road Opening Works for Quarry (11)** | 100,000.00 |
| **Quarry Operating Cost (12)** | 609,166.67 |
| **Blasting of Stone (13)** | 1,338,096.70 |
| | |
| **Total (TL)** | 2,347,263.36 |
| **Grand Total (TL)** | **2,347,263.36** |

In Table 4.71, all costs that are found in Table 4.69, 4.70 and contractor's profit and general expenses are summed. In Table 4.71, staff cost is taken as 382,400.00 TL, same as Section 1 as the same team can be used for all sections.

**Table 4.71** Cost Calculation Summary for Section 4

| SUMMARY | |
|---|---|
| | **Prices (TL)** |
| **Machinery Cost** | **4,594,869.68** |
| **Staff Cost** | **382,400.00** |
| **Direct Cost** | **2,347,263.36** |
| **Contractor's Profit & General Expenses (25%)** | **1,831,133.26** |
| | |
| **GRAND TOTAL (TL)** | **9,155,666.30** |
| **UNIT COST (TL/ton)** | **6.26** |

### 4.3.2.4 Contractor's Cost Estimation of Section 5

In Table 4.72, cross-section areas of all stone ranges calculated and total weight in ton is found by multiplying volume to 2.55.

**Table 4.72** Data of Section 5

| DATA | | |
|---|---|---|
| **RANGE** | **VOLUME (m³)** | **WEIGHT (TON)** |
| **0-0.4 ton** | 153,635.95 | 391,771.67 |
| **0.4-2 ton** | 207,559.95 | 529,277.87 |
| **6-10 ton** | 214,953.70 | 548,131.94 |
| **2-4 ton (toe)** | 4,490.85 | 11,451.67 |

Same assumption as in Section 1 is accepted where 90 days is the duration to finish 1 km of cross section. Day and night working is assumed.

In Table 4.73, the needed machinery park is calculated. All calculation approachments are same with Section 1 and mentioned in 4.3.2.1.

**Table 4.73** Calculation of Machinery Park for Section 5

| CALCULATION OF MACHINERY PARK | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|
| **Range** | **Weight Of One Truck (ton) (1)** | **Required Total Trip** | **Duration Per Truck Each Trip (min) (2)** | **Daily Trip (3)** | **Required Days** | **Required Truck Quantites** | **Excavator Quantites** | | |
| | | | | | | | **Loading (4)** | **Breaking** | **Unloading (4)** |
| **0-0.4 ton** | 54.00 | 7,255.00 | 75.00 | 13.00 | 558.00 | 6.00 | 1.20 | 2 (5) | 1.20 |
| **0.4-2 ton** | 42.00 | 12,602.00 | 72.00 | 13.00 | 969.00 | 11.00 | 2.29 | | 2.29 |
| **6-10 ton** | 24.00 | 22,839.00 | 67.50 | 14.00 | 1,631.00 | 18.00 | 4.00 | | 4.00 |
| **2-4 ton (toe)** | 33.00 | 347.00 | 70.00 | 14.00 | 25.00 | - | - | | - |

In Table 4.74, cost calculation of machinery park is summarised. Small, big excavators and trucks quantities come from Table 4.73**.** All calculation approachments are same with Section 1 and mentioned in Chapter 4.3.2.1.

**Table 4.74** Cost Calculation of Machinery Park for Section 5

| MACHINERY COST | | | | | | |
|---|---|---|---|---|---|---|
| | **Quantity** | **Rental Prices/Month (TL)** | **Rental Cost (TL) (8)** | **Fuel-Oil Consumption** | **Fuel Oil (TL/l) (9)** | **Fuel-Oil Cost (TL)** |
| **Small Excavator (20-25 ton capacity)** | 8.00 | 12,500.00 | 576,000.00 | 15 lt/hr | 2.70 | 465,974.24 |
| **Big Excavator (35-50 ton capacity)** | 8.00 | 22,500.00 | 1,036,800.00 | 20 lt/hr | 2.70 | 622,080.00 |
| **Trucks** | 35.00 | 8,000.00 | 1,612,800.00 | 0,5 lt/ km | 2.70 | 697,296.60 |
| **Crane (6)** | 2.00 | 25,000.00 | 288,000.00 | 3000 TL/month | - | 9,600.00 |
| **Water Truck** | 1.00 | 6,000.00 | 34,560.00 | 0,2 lt/km | 2.70 | 2,430.00 |
| **Fuel-Oil Truck** | 1.00 | 6,000.00 | 34,560.00 | 0,2 lt/km | 2.70 | 2,430.00 |
| **Grader (7)** | 1.00 | 19,000.00 | 109,440.00 | 16 lt/hr | 2.70 | 38,880.00 |
| **Passenger Car** | 3.00 | 1,250.00 | 21,600.00 | 0,07 lt/km | 2.70 | 7,654.50 |
| **Total (TL)** | | | 3,713,760.00 | | | 1,846,345.34 |
| **Grand Total (TL)** | | | | | | **5,560,105.34** |

In Table 4.75, mobilization for machinery park, setup cost, environmental impact assessment report are assumed average values for contractor. Cost calculation approachments for road opening works for quarry, quarry operating cost and blasting of stone are all same with Section 1 and mentioned in Chapter 4.3.2.1.

**Table 4.75** Cost Calculation of Direct for Section 5

| DIRECT COST | |
|---|---|
| | **Total Cost (TL)** |
| **Mobilization for Machinery Park** | 100,000.00 |
| **Setup Cost** | 150,000.00 |
| **Environmental Impact Assessment Report** | 50,000.00 |
| **Road Opening Works for Quarry (11)** | 100,000.00 |
| **Quarry Operating Cost (12)** | 616,666.67 |
| **Blasting of Stone (13)** | 1,336,092.18 |
| | |
| **Total (TL)** | 2,352,758.85 |
| **Grand Total (TL)** | **2,352,758.85** |

In Table 4.76, all costs that are found in Table 4.74, 4.75 and contractor’s profit and general expenses are summed. Staff cost is taken as 382,400.00 TL, same as Section 1 as the same team can be used for all sections.

**Table 4.76** Cost Calculation Summary for Section 5

| SUMMARY | |
|---|---|
| | **Prices (TL)** |
| **Machinery Cost** | **5,560,105.34** |
| **Staff Cost** | **382,400.00** |
| **Direct Cost** | **2,352,758.85** |
| **Contractor's Profit & General Expenses (25%)** | **2,073,816.05** |
| | |
| **GRAND TOTAL (TL)** | **10,369,080.23** |
| **UNIT COST (TL/ton)** | **7.00** |

### 4.3.3 Comparison of Economical Analysis of Alternative Sections

In Table 4.77, there is a comparison of economical analysis with government and contractor. Also, comparison of unit costs of alternative sections to Section 1 are given. It is seen that except Section 2 all alternative sections are applicable and all of them are more economical than Section 1.

**Table 4.77** Comparison of Economical Analysis with Government and Contractor

| **SECTION** | **GOVERNMENT** | | **CONTRACTOR** | | **DISCOUNT PERCENTAGE of TOTAL COST (%)** |
|---|---|---|---|---|---|
| | Total Cost (TL) | Unit Cost (ton/TL) | Total Cost (TL) | Unit Cost (ton/TL) | |
| **1** | **18,283,876.94** | 17.45 | **10,632,301.13** | 7.16 | **41.85** |
| **3** | **15,687,033.76** | 15.39 | **8,752,389.67** | 6.06 | **44.21** |
| **4** | **16,556,780.90** | 16.04 | **9,155,666.30** | 6.26 | **44.70** |
| **5** | **17,651,000.57** | 16.89 | **10,369,080.23** | 7.00 | **41.26** |

Main difference among the sections relies on machinery cost in the case of the contractor approach. Since a whole quarry is purchased for the construction, the different sizes of stones used in the design is reflected in the number of machinery required. More than 40% difference is calculated between the contractor calculations and the government calculations. It is important to mention that both approaches include 25% profit.

# CHAPTER 5

# CONCLUSION

In the present study, Ordu-Giresun Airport Berm Breakwater and alternative cross-sections of fully and partly reshaped berm breakwaters were examined for stability and wave overtopping. Significant wave height and significant wave period that were used in design of Ordu-Giresun Airport Berm Breakwater were taken for design of alternative cross-sections. All cross-sections were assessed with model studies carried out in a wave flume at the Coastal and Harbor Engineering Laboratory of the Middle East Technical University, Ankara.

First of all, model of Ordu-Giresun Airport Berm Breakwater named as Section 1 in this study is constructed. Stability and wave overtopping calculations were made and related graphs and tables were given in Chapter 4. This section was very rigid and reliable as expected. The safe results enabled to model smaller cross-sections in terms of stone range and horizontal armour width.

Section 2 with the smallest stone range and horizontal armour width is modelled. All related calculations were summarised in Chapter 4. However, this section did not perform well with regard to wave overtopping. Therefore, by increasing horizontal armour width, Section 3 was acquired. The overtopping results were satisfactory in this case.

Section 4 was modelled with the same dimensions as Section 3. However, wide grading stone range (2-8 ton) instead of narrow grading stone range (2-4 ton) was preferred. Stability and wave overtopping quantity results showed better performance.

Lastly, an alternative to Section 4 in terms of wide grading stone range was modelled. The aim was also to have a partly reshaping berm breakwater design in the experiment sets. Section 5 was modelled with the same dimensions of Section 4. The only difference was armour stone layer was consisted of 6-10 ton stone range. Stability and wave overtopping quantity results showed better performance when it was compared to Section 4 as expected.

All experiments showed that different combinations of stone size and horizontal armour width can provide similar stability and wave overtopping performance. The most important issue is to use appropriate cross-sections in design dependent on technical specifications. In this study, except Section 2, all cross-sections can be constructed instead of Section 1.

Economical analysis serves a very important purpose to further distinguish these alternatives. In Chapter 4, all sections were accepted to be constructed in Ordu-Giresun Airport. Therefore, conditions of that region in terms of quarry characteristics were regarded. These characteristics are distance of quarry to construction site, specific weight of stone, blasting conditions etc.

Goverment cost was found out by using Year 2016 Bill of Quantities of Infrastructural Investment of Ministry of Transport, Maritime Affairs and Communications. That list is national and independent of design projects. In Chapter 4, all items for calculations of unit prices are summarised. Section 3 has the lowest cost while the constructed section costs 2.5 million TL more.

The contractor approach provided similar cost results with Section 3 having the lowest value. However, the difference between government cost and contractor cost is more than 40% which includes the additional 25 % of contractor profit. This significant difference is mostly due to renting or buying a stone quarry rather than buying stones from different quarries in very large projects such as this one.

As a contractor, Section 3 is preferred to be constructed because small stone range is always more applicable and advantageous than large stone ranges. For instance, as the size of stone increases, it becomes harder to employ trucks and excavators. In the sector, leasers of trucks and excavators do not prefer to work on larger rock placement projects due to maintenance costs. Moreover, the quarry distance is the main element to determine construction cost. In this experiments 5,6 km distance was taken into consideration. Therefore, it is not true to say there is a proportioning between government cost and contractor cost.

All cost calculations were conducted for a 1 km long structure. Ordu-Giresun Airport Berm Breakwater has 7.345 km length. The cost differences in TL which are mentioned in Chapter 4 can be multiplied by 7.345 in order to understand the cost advantage of alternative sections to Section 1 better.

In general, it can be stated that it could be a good idea to have designers and contractors decide together for selection of cross-section. Both sides would have crucial ideas to construct optimum structures whereas physical model studies should be required to design the optimum cross section especially for dynamic structures such as shaping berm breakwaters.

Maintanence of these types of structures especially for smaller stone sizes where lateral movement may occur should be considered for further study, both in experimental and economical analysis. Constant monitoring for small size design should be required as well as 3D modeling.

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