A COMPARATIVE STUDY ON THE DESIGN OF RUBBLE MOUND BREAKWATERS

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ABSTRACT

A COMPARATIVE STUDY ON THE DESIGN OF RUBBLE MOUND BREAKWATERS

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Rubble mound breakwaters are one of the most common coastal defense structures constructed around the world. Hudson (CERC, 1977; CERC, 1984), Van der Meer (1988) and Van Gent et al (2003) are the major stability formulas that are used to find the armour stone weight of rubble mound breakwaters.

In the first part, a comparative study on major stability formulas is carried out to discuss the discrepancies in application of these design formulas. A computational tool Design Armour Stone (DAS) is developed within this study by defining the application limits of Van der Meer and Van Gent et al formulas tested by physical model experiments under appropriate design conditions.

In the second part, design water level which is an important parameter affecting the armour stone size is investigated. Components of change in mean water level that affects design water level within economic life of a coastal structure are discussed. Effect of design water level on armour stone size of a coastal structure is analyzed by a computational tool developed in this study called Design Water Level Determination (DWLD) considering Hudson, Van der Meer and Van Gent et al approaches. Current deterministic approach is upgraded by the use of DWLD in order to find the most critical design water level in economic life of a rubble mound coastal structure.

In the final part of this study, a case study is conducted in Aliaga, Izmir, Turkey. DWLD and DAS are applied to this region in order to show discussions in a real case study.

The outcomes of these studies can be used as a guide in design of rubble mound breakwaters for practical purposes.

Keywords: Rubble mound breakwaters, Hudson, Van der Meer, Van Gent, Design water level, Physical model experiments

ÖZ

TAŞ DOLGU DALGAKIRANLARIN TASARIMI ÜZERİNE
KARŞILAŞTIRMALI BİR ÇALIŞMA

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Taş dolgu dalgakıranlar dünya ölçeğinde inşa edilmiş en yaygın olarak kullanılan kıyı koruma yapılarından biridir. Hudson (CERC, 1977; CERC, 1984), Van der Meer (1988) ve Van Gent vd (2003) yaklaşımları taş dolgu dalgakıranların zırh tabakasında kullanılan taşların ağırlığının belirlenmesinde kullanılan başlıca stabilite formülleridir.

İlk kısımda, başlıca stabilite formüllerindeki tutarsızlıkların gösterilmesi amacıyla karşılaştırmalı bir çalışma yapılmıştır. Bu amaçla, Van der Meer ve Van Gent et al formüllerinin uygulama limitlerini tanımlayan, Design Armour Stone (DAS) isimli bir hesaplama aracı geliştirilmiş ve uygun tasarım koşulları altında fiziksel model deneyleriyle test edilmiştir.

Çalışmanın ikinci kısmında, zırh tabakasının tasarımını etkileyen önemli bir parametre olan tasarım derinliği incelenmiştir. Taş dolgu kıyı yapılarının ekonomik ömrü boyunca göz önüne alınması gereken tasarım derinliğini etkileyen su seviyesi değişiminin bileşenleri tartışılmıştır. Tasarım derinliğinin taş dolgu kıyı yapısının zırh tabakasında kullanılan taşların büyüklüğüne etkisi, tez çalışması kapsamında geliştirilen "Design Water Level Determination (DWLD)" isimli Hudson, Van der Meer ve Van Gent vd yaklaşımlarını göz önüne alan hesaplama aracı yardımıyla analiz edilmiştir. Şu an kullanılan deterministik (belirlenirlecilik) yaklaşım, taş dolgu kıyı yapısının ekonomik ömrü boyunca en kritik tasarım derinliğini belirlemek amacıyla, DWLD kullanılarak geliştirilmiştir.

Çalışmanın son kısmında Aliağa, İzmir, Türkiye bölgesinde bir uygulama yapılmıştır. DWLD ve DAS bu bölgeye uygulanarak, tez boyunca yapılan tartışmalar gerçek bir örnek üzerinde gösterilmiştir.

Bu çalışma sonucunda elde edilen veriler, taş dolgu dalgakıranların tasarımında pratik amaçlara yönelik bir rehber oluşturmaktadır.

Anahtar Kelimeler: Taş dolgu dalgakıranlar, Hudson, Van der Meer, Van Gent, Tasarım derinliği, Fiziksel model deneyleri

To my beloved family…

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ABBREVIATIONS

1D	One dimensional
DAS	Design Armour Stone Code
DWLD	Design Water Level Determination Code
ECMWF	European Centre for Medium-Range Wave Forecasts
ES	Example Study
HWL	High Water Level
LWL	Low Water Level
NSW	Near Shore Wave Transformation Model
NW	North West
RMBW	Rubble Mound Breakwater
SLR	Sea Level Rise
SWL	Sea Water Level
SSW	South South West
SW	South West
W	West
WNW	West North West
WSW	West South West
WT	Wave Transformation and Regular Wave Breaking

NOMENCLATURE

γ_{stone}	Unit Weight of Armour Stone
γ_{water}	Unit Weight of Sea Water
g	Gravity of acceleration
H_0	Deep Water Wave Height
H_{s0}	Deep Water Significant Wave Height
$H_{1/10}$	Wave Height exceeded by 10% of waves in a storm
$H_{s,toe}$	Significant wave height at the toe of the structure
H_{design}	Design wave height
$H_{2\%}$ (m)	Wave Height Exceeded by 2% of the waves
H_{tr}	Transmitting Wave Height
H_{rms}	Root Mean Square Wave Height
H_b	Breaking Wave Height
T	Wave Period
T_s	Significant Wave Period
T_m	Mean Wave Period
$T_{m-1,0}$	Spectral Mean Energy Wave Period
T_p	Peak Wave Period
L_0	Deep Water Wave Length
L	Wave Length
L_{op}	Peak Wave Length
H_{s0}/L_0	Deep Water Significant Wave Steepness
S_{op}	Peak Wave Steepness
h	Water Depth
K_s	Shoaling Coefficient

K_r	Refraction Coefficient
α_0	Deep Water Wave Approach Angle
θ	Mean Wave Approach Angle
K_D	Stability Parameter
ξ_m	Mean Surf Similarity Parameter
$\zeta_{m-1,0}$	Spectral Mean Energy Surf Similarity Parameter
ζ_{cri}	Critical surf similarity parameter
m	Foreshore Slope
$\cot(\alpha)$	Structure Face Slope
S	Damage level
Δ	Relative Buoyant Density
P	Porosity or notional permeability of the structure
C_{pl}	Plunging coefficient
C_s	Surging coefficient
N	Number of Incident Waves at the toe of the structure
D_{n50}	Nominal armour stone diameter assuming a 50% cumulative distribution
Fr	Froude Number
u	Water Particle Velocity
λ	Scale
U_{10}	Wind Velocity measured 10 m above of sea level
η_w	Wind Set-Up
η_{max}	Maximum Wave Set-Up
η_{min}	Wave Set-Down
E	Total Wave Energy
C_g	Group Velocity
D_b	Dissipation due to the random wave breaking,
k	Wave Number
f_p	Peak Frequency

erf	Error Function
$\bar{\eta}$	Mean Water Level Fluctuation
F_x	Sum of Radiation Stresses
τ_{wx}	Wind Shear Stress
S_{xx}	Radiation Stress acting in x-direction
E_{sr}	Kinetic Energy of Surface Rollers
C	Wave Celerity
n	Ratio of Group Velocity to Wave Celerity
A_{sr}	Area of Surface Roller
m_0	Total Energy Density
K_{sr}	Rate of Dissipation of Surface Roller Energy
C_D	Drag Coefficient
F	Fetch Length
ΔH	Change in Mean Water Level
Δh	Increment of Water Level
R_p	Return Period

CHAPTER 1

INTRODUCTION

Coastal areas are the most attractive and valuable parts of world due to both economic and social objectives. Coastal areas offer a lot of opportunities considering urbanization, industry, tourism, recreation, agriculture, fisheries, aquaculture, energy production, mineral-petroleum-natural gas resources and transportation. Furthermore, coastal areas are the places where land, water and air meet; thus, it is a hard duty to explain complex processes of coastal areas.

Ports, harbors and marinas are the main examples of coastal structures that people need for efficiently using the coastal areas. These coastal structures usually have defense units named as breakwater in order to protect the area from wave attack. Breakwaters can be constructed as rubble mound, piled, vertical wall or floating according to wave condition in the coastal area.

Rubble mound breakwaters are widely constructed around the world. It is the most common coastal defense structure in Turkey and in Europe. Design of rubble mound structures is a challenging issue because of the uncertainties and complexity of the coastal areas' nature. Assessment of wave conditions in the design area, selecting the most appropriate of coastal defense structure considering the conditions of the area, design of coastal defense structure according to importance of the coastal structure and constructing the coastal structure are the main steps in this process.

In this study, different aspects of design of rubble mound breakwaters are discussed. In Chapter 2, a brief literature survey is given in order to provide background information.

In Chapter 3, the major objective is to rise questions in designers' minds by showing discrepancies in design of rubble mound structures. Hudson (CERC, 1977; CERC, 1984), Van der Meer (1988) and Van Gent et al (2003) approaches are taken as major stability of rock slopes. An example problem is pointed out in order to define the discrepancies in design of rubble mound structures considering Van der Meer and Van Gent et al formulations as the main concern resulting in drastic armour stone size differences obtained in application ranges of these formulations provided by Rock Manual (2007). Hudson approaches are given at all steps of this part of the study in order to supply a common measure. Furthermore, these major stability formulas are compared to each other to visualize the difference results obtained from these formulations in wide ranges of application. Computed results using different approaches are compared by curves at different depths of construction and considering different deep water wave steepness values. At the end of first part of the study, a new application range is defined for Van der Meer and Van Gent et al methodologies. In the first part of the study, a design tool is developed in MATLAB environment named as Design Armour Stone (DAS).

The new application ranges of Van der Meer and Van Gent et al formulations are tested by physical model experiments conducted in METU Department of Civil Engineering, Ocean Engineering Research Center (OERC) Laboratory. Scaling, experimental setup, experiments and results are presented in Chapter 4.

Another important design parameter for rubble mound breakwaters is design water level. Design water level is determined according to sea level rise due to global warming, astronomic tides, seasonal variations, wave and wind setup occurs during a storm condition and barometric and Coriolis effects. These are the components of the change in mean water level. It is a challenging issue to determine design water level due to complexities in these components which affect the design of coastal structure deeply. In Chapter 5, components of mean water level change are defined and discussed in Turkish coasts. A numerical model, Near Shore Wave Transformation (NSW) (Baykal, 2012), is used as a part of a code developed in this study, namely, Design Water Level Determination (DWLD), to determine the most critical water level that would be taken as the design water level.

In Chapter 6, the approaches discussed throughout the study are applied to a case study in Aliaga, Izmir, Turkey and results are presented.

In Chapter 7, conclusions and future recommendations are given.

CHAPTER 2

LITERATURE SURVEY

2.1. Major Rock Slope Stability Formulas for Armour Layer of Rubble Mound Breakwaters

Rubble mound breakwaters are one of the most common coastal defense structures around the world and the most common in Turkey. It is widely studied among many researchers. Major rock slope stability formulas for armour layer of rubble mound breakwaters are Hudson (CERC, 1977; CERC, 1984), Van der Meer (1988) and Van Gent et al (2003) approaches.

2.1.1. Hudson Approach

Hudson formulation is proposed in 1959 based on experiments conducted with regular waves. Hudson's formula is given by Equation 2.1.

$$W_{50} = \frac{\gamma_{stone}\ H_{design}^{3}}{K_D\ \Delta^{3}\ cot\,\alpha} \qquad [2.1]$$

Parameters used in definition of Hudson approach are given in Table 2.1.

In Hudson's formula H_{design} is determined according to wave breaking condition. It is taken as breaking wave height, if the waves are breking at the toe of the structure. On the other hand, it is taken as wave height at the toe of the structure if the waves are not breaking. Finally, if the waves are broken, it is taken as broken wave height.

Table 2.1: Parameters of Hudson Approach

γ_{stone}	Unit weight of armour stone (t/m^3)
H_{design}	Design wave height (m)
K_D	Stability coefficient
α	Structure Face Slope angle, (·)
W_{50}	Nominal armour stone weight assuming a 50% cumulative distribution (tons)
Δ	Relative Buoyant Density

To find breaking condition, design wave height at the deep water is needed (H_0). In Hudson's approach, deep water wave height is considered as a wave height defining regular wave series. However, waves are randomly generated in nature. Therefore, H_0 is taken as different wave heights defined with an exceedance probability that obeys Rayleigh distribution. CERC (1977) defines this deep water wave height as significant wave height (H_{s0}); on the other hand, CERC (1984) gives this wave height as the wave height exceeded by 10% of waves in a certain storm ($H_{1/10}$). Both formulations are used in practice (Rock Manual).

Another important parameter in Hudson's formula is stability coefficient (K_D). This parameter is defined for each type of stone according to breaking condition. It is used as "4" for non-breaking wave condition and "2" for breaking wave condition considering rough, angular, randomly placed quarry stone traditionally.

In Figure 2.1, a sketch summarizes design of rubble mound breakwaters using Hudson's approach.

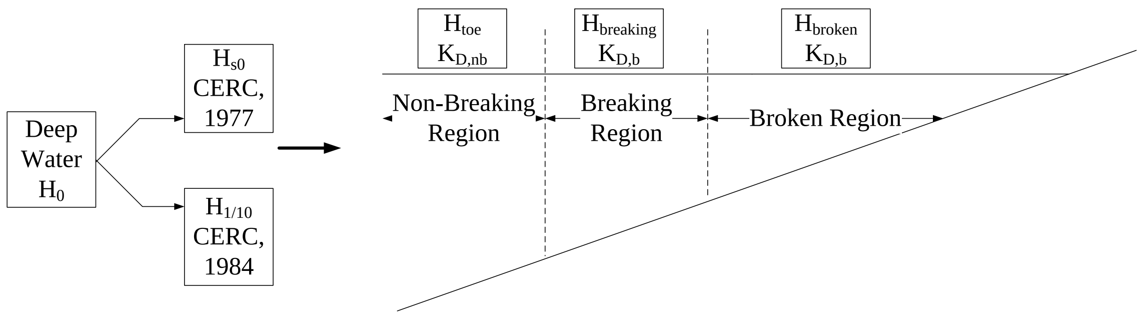


Figure 2.1: Summary for Design of Rubble Mound Breakwaters using Hudson's approach

2.1.2. Van der Meer Approach

Van der Meer (1988) suggested a formula based on based on earlier results by Thompson and Shuttler (1975) and physical model experiments conducted with irregular waves. This formula accounts for the type of wave breaking, number of waves attack coastal structure (implicitly wave period) and notional permeability in addition to other parameters given previously. In his approach, the surf similarity parameter (Equation 2.2) should be first calculated and compared to a critical surf similarity parameter (Equation 2.3) to determine the type of wave breaking. Then, armour stone diameter is calculated using the relevant formula given by Equations 2.4 and 2.5. In Table 2.2, the parameters used in the stability formula set are defined.

$$\zeta_m = tan\,\alpha \,/\, \sqrt{(2\pi / g)\; H_{s,toe} \;/\; T_m^2} \qquad [2.2]$$

$$\zeta_{cri} = \left(\frac{C_{pl}}{C_s} \; P^{0.31} \; \sqrt{tan\,\alpha} \right)^{\frac{1}{P+0.5}} \qquad [2.3]$$

Plunging waves ($\zeta_m < \zeta_{cri}$)

$$\frac{H_{s,toe}}{\Delta D_{n50}} = C_{pl} \;\; P^{0.18} \left(\frac{S}{\sqrt{N}} \right)^{0.2} \;\; \zeta_m^{-0.5} \qquad [2.4]$$

Surging waves ($\zeta_m \geq \zeta_{cri}$) [2.5]

$$\frac{H_{s,toe}}{\Delta D_{n50}} = C_s \ P^{-0.13} \left(\frac{S}{\sqrt{N}}\right)^{0.2} \ \sqrt{cot\,\alpha} \ \ \zeta_m^P$$

Table 2.2: Parameters in Van der Meer formula

Symbol	**Parameter**
$H_{s,toe}$	Significant wave height at the toe of the structure (m)
T_m	Mean wave period (sec)
g	Gravity of acceleration (m/s^2)
α	Structure Slope angle, (·)
D_{n50}	Nominal armour stone diameter assuming a 50% cumulative distribution
N	Number of incident waves at the toe of the structure
ζ_m	Surf similarity parameter using the mean wave period
ζ_{cri}	Critical Surf similarity parameter
Δ	Relative Buoyant Density
P	Porosity or notional permeability of the structure
C_{pl}	Plunging coefficient
C_s	Surging coefficient
S	Damage level

C_{pl} and C_s are plunging and surging calibration coefficients derived by physical model experiments as 6.2 and 1.0, respectively (Van der Meer, 1988).

Notional permeability (P) is taken into account to introduce the effect of permeability to design formula. It has no physical meaning; nevertheless, it is determined under different type of armour, filter and core layer designs as given in Figure 2.2 (Rock Manual, 2007). In the absence of data, it is usually assumed as 0.4.

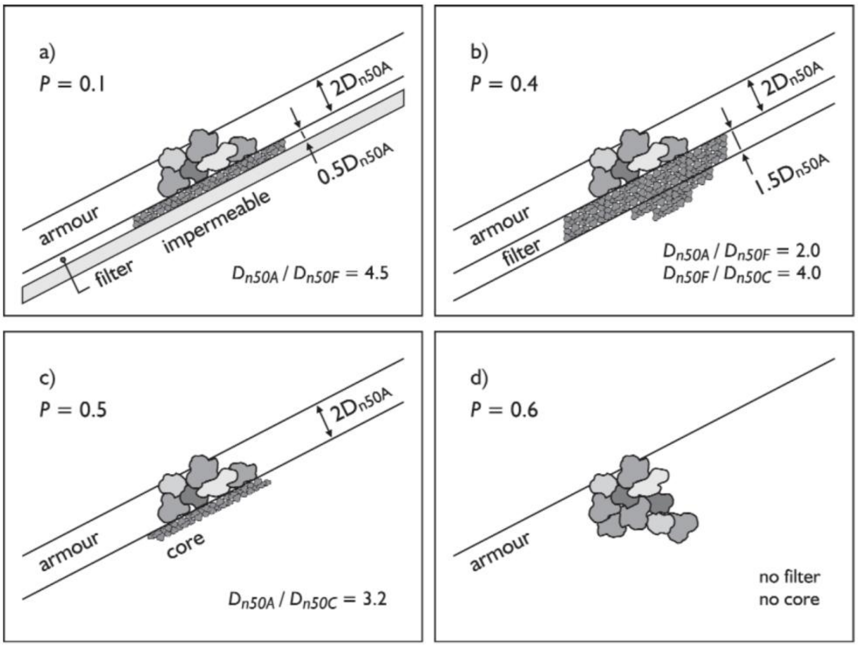


Figure 2.2: Notional Permeability (P) Coefficients

Damage parameter (S) is introduced to define damage level. It is a parameter gives opportunity to a designer to allow damage to a certain level. Damage levels according to damage parameter and structure face slope are given in Table 2.3 (Rock Manual, 2007).

Table 2.3: Damage Levels *(adopted from Rock Manual, 2007)*

Slope ($\cot\alpha$)	**Damage Level according to S**		
	Start of Damage	**Intermediate Damage**	**Failure**
1.5	2	3-5	8
2	2	4-6	8
3	2	6-9	12
4	3	8-12	17

Number of waves (N) parameter is introduced to include effect of storm duration implicitly into the formula (Van der Meer, 1988). It is defined as storm duration over mean wave period.

2.1.3. Van Gent et al Approach

Van Gent et al (2003) approach is a modifying formula of Van der Meer (1988) equations to extend its applicability to shallower water. The spectral mean energy wave period ($T_{m-1,0}$) is taken into account instead of mean wave period (T_m). The spectral mean energy wave period defines shallow water processes in a better way since it includes the influence of spectral shape (Guler, 2013). Similar to Van der Meer approach, breaking type is determined comparing surf similarity parameter calculated using spectral mean energy wave period given by Equation 2.6 to critical surf similarity parameter given by Equation 2.7. After that, armour stone diameter is found using relevant formula according to breaking type using Equations 2.8 and 2.9.

$$\zeta_{m-1,0} = tan\,\alpha / \sqrt{(2\pi / g)\, H_{s,toe} \;/\; T^2_{m-1,0}} \qquad [2.6]$$

$$\zeta_{cri} = \left(\frac{C_{pl}}{C_s}\, P^{0.31}\, \sqrt{tan\,\alpha} \right)^{\frac{1}{P+0.5}} \qquad [2.7]$$

Plunging waves ($\zeta_{\mathrm{m-1,0}} < \zeta_{\mathrm{cri}}$)

$$\frac{H_{s,toe}}{\Delta D_{50}} = C_{pl}\; P^{0.18} \left(\frac{S}{\sqrt{N}} \right)^{0.2} \left(\frac{H_{2\%}}{H_{s,toe}} \right)^{-1} \zeta_{m-1,0}^{-0.5} \qquad [2.8]$$

Surging waves ($\zeta_{m-1,0} \geq \zeta_{cri}$)

$$\frac{H_{s,toe}}{\Delta D_{50}} = C_{pl}\; P^{0.18} \left(\frac{S}{\sqrt{N}} \right)^{0.2} \left(\frac{H_{2\%}}{H_{s,toe}} \right)^{-1} \zeta_{m-1,0}^{-0.5} \qquad [2.9]$$

Parameters used in Van Gent et al formula are given in Table 2.4.

Plunging (C_{pl}) and surging (C_s) coefficients are calibrated using Van Gent et al's tests as 8.4 and 1.3, respectively.

Other parameters notional permeability (P), damage level (S) and number of waves (N) are defined as same as Van der Meer approach.

Table 2.4: Parameters in Van der Meer formula set

Symbol	**Parameter**
$H_{s,toe}$	Significant wave height at the toe of the structure, (m)
$T_{m-1,0}$	Spectral mean energy wave period (sec)
g	Gravity of acceleration (m/s^2)
α	Structure Slope angle, (·)
D_{n50}	Nominal armour stone diameter assuming a 50% cumulative distribution
$H_{2\%}$	Wave height exceeded by 2% of the waves in a certain storm
N	Number of incident waves at the toe of the structure
$\zeta_{m-1,0}$	Surf similarity parameter using the spectral mean energy wave period
ζ_{cri}	Critical surf similarity parameter
Δ	Relative Buoyant Density
P	Porosity or notional permeability of the structure
C_{pl}	Plunging coefficient
C_s	Surging coefficient
S	Damage level

In this formulation, another parameter is introduced, namely, wave height exceeded by 2% of the waves ($H_{2\%}$). It can be found using Battjes and Groenendijk (2000). In their study, Battjes and Groenendijk present wave height distributions on shallow foreshores. A simplified version of this study to find $H_{2\%}$ is given by Rock Manual (2007). A transmitting wave height, H_{tr}, and root mean square wave height, H_{rms}, are used to convert significant wave height, H_s, to wave height exceeded by 2% of the waves, $H_{2\%}$. H_{tr} and H_{rms} are given by Equations 2.10 and 2.11 where h is the water depth and tanθ is foreshore slope. Using the values for the ratio H_{tr}/H_{rms} given in Table 2.5., the ratio $H_{2\%}/H_{rms}$ is found. Thus, $H_{2\%}$ is approximated.

$$H_{tr} = (0.35 + 5.8 \tan\theta) h \qquad [2.10]$$

$$H_{rms} = [\,0.6725 + 0.2025(\,H_s\,/\,h\,)]\,H_s \qquad [2.11]$$

Table 2.5: Conversion of ratio H_{tr}/H_{rms} to ratio $H_{2\%}/H_{rms}$

H_{tr}/H_{rms}	0.05	0.50	1.00	1.20	1.35	1.50	1.75	2.00	2.50	3.00
$H_{\%2}/H_{rms}$	1.548	1.549	1.603	1.662	1.717	1.778	1.884	1.985	1.978	1.978

2.2. Physical Model Experiments

Physical model is defined as a physical system that is reproduced usually at a reduced size considering the main dominant forces available on the system in a correct proportion to actual physical system (Hughes, 1993).

Physical models have ability to visualize and observe the process in a closer view in detail (Price, 1978). However, a model can only be precise if the prototype is designed correctly; otherwise, measurements are meaningless regardless of instrumentation and measurement-methods (Yalin, 1989). Hughes (1993) states that "A model with poor scale determination would like a ruler with incorrect markings. The ruler can be used to make measurements, but the measurements are guaranteed to be wrong!".

2.2.1. Advantages and Disadvantages of Physical Model Experiments

Physical model experiments have both advantages and disadvantages considering modeling technique, measurement opportunities, scale effect, laboratory conditions and cost of the experiments.

Advantages of physical model experiments are summarized by Hughes (1993) using discussions of Dalrymple (1985), Kamphius (1991) and Le Mehaute (1990) as following:

- Physical models let one to integrate appropriate equations governing the process without assumptions made in analytical and numerical models Dalrymple (1985).

- Small scale of experiments permit easier data measurement compared to prototype scale with a reduced cost. Furthermore, prototype measurements, i.e. field measurements, take so much time and hard to achieve Dalrymple (1985).
- Physical process can be qualitatively understood immediately by an observer (Kamphius, 1991).
- Scaling a physical system compared to testing it in prototype scale is cost effective. By physical model experiments, decision making process gains reliability and credibility (Le Mehaute, 1990).
- Most of the coastal engineering phenomena are complex to reproduce by analytical and numerical techniques. However, complex turbulence effects can be modeled in an easier way by physical model experiments (Le Mehaute, 1990).

On the other hand, disadvantages of physical model experiments canalize researchers to other techniques. These disadvantages are summarized by Hughes (1993) as following:

- If all relevant variables are not simulated in correct relationship, scale effects occur (Le Mehaute, 1990).
- The process being simulated can be influenced by laboratory effects to the extent that suitable approximation is not possible (Hughes, 1993).
- It is sometimes impossible to include all forcing functions and boundary conditions (Hughes, 1993).
- Physical models are often more expensive than analytical and numerical models (Hughes, 1993).

2.2.2. Evaluation of Damage in the Cross-Section

In physical model experiments conducted on rubble mound breakwaters, the aim is to evaluate damage occurs under the effect of selected wave condition. According to damage measured in the cross-section, it is possible to determine a rubble mound breakwater safe or not.

There are various damage definitions given in the literature to define damage in the cross-section of rubble mound breakwaters. The general idea behind these definitions is to find number of stones that is moved or number of stones that can be inserted in eroded area along the cross-section.

Damage parameters given by Equations 2.12 and 2.13 are the most common damage parameters (Disco, 2013).

$$N = \frac{\# stones_{moved}}{\# stones_{active-zone}} 100 \qquad [2.12]$$

$$S = \frac{A_e}{D_{n50}^2} \qquad [2.13]$$

Equation 2.12 is found by counting stones. Moved stones over stones in active zone multiplied by 100 give the damage parameter. Water depth at the toe of the structure plus and minus wave height at the toe of the structure is defined as active zone.

Equation 2.13 is derived by Van der Meer (1988) using the study of Broderick (1984). Damage (S) is defined as the eroded area (A_e) over square of nominal diameter (D_{n50}) of armour stone. In Figure 2.3, definition of Van der Meer damage parameter is given.

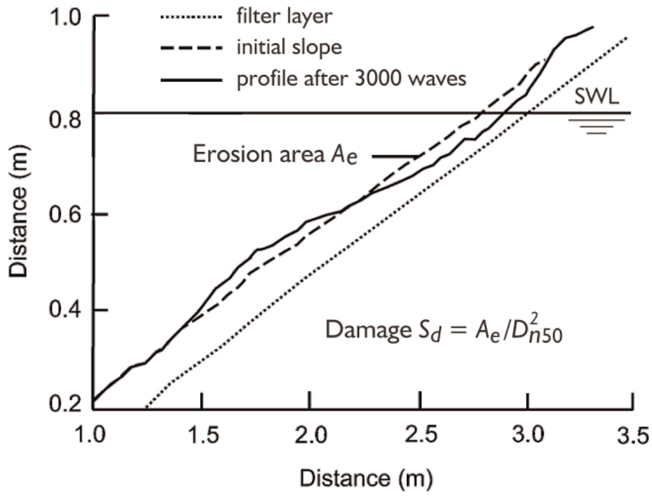


Figure 2.3: Definition of Damage

2.3. Design Water Level

Design water level is one of the most critical coastal engineering design parameter. This is simply because it affects wave conditions at the toe of the structure which is directly related to all other structural design parameters.

Design water level is decided traditionally by a risk-based approach of return period (FHA, 2008). Return period is related to encountered risk and economic life time of a coastal structure.

In coastal engineering design, design water level is determined considering mainly two groups of events. The first group is the expected mean water level changes throughout the coastal structure's lifetime such as sea level rise due to global warming, seasonal variations in the water level, astronomical tides, barometric and Coriolis effects and water level rise due to design storm, i.e. wave and wind setup for a certain storm. The second group is storm surges and tsunamis as extreme marine hazards. These two groups can be investigated separately or together, by a probabilistic or a deterministic approach. For both group of analyses, coastal defense structures are definitely designed considering stochastic behavior of extreme water level events (Jensen, 1985). Design water levels are determined using statistical analysis of extreme events (Dixon and Tawn, 1994).

CHAPTER 3

ANALYSIS OF MAJOR STABILITY FORMULAS ON RUBBLE MOUND BREAKWATERS

Rubble mound coastal defense structures are widely used around the world in order to protect coastal areas from wave attack. The stability of rubble mound structures has been studied by many researchers and there are number of outcomes of these studies that are used in practice.

Hudson (1959) developed a formula based on model tests with regular waves. A formula by Van der Meer (1988) is given for relatively deep water conditions at the toe of the structure with an application to moderately shallow water based on model tests with irregular waves. Van Gent et al (2003) proposed a modified formula to extend applicability of Van der Meer (1988) formula to shallower water depth. Hudson (1959) formula is different in both development methodology and application comparing to other two formulations.

Van der Meer and Van Gent et al formulas are recommended to be used together according to a criterion given by Rock Manual (2007). This criterion is defined as water depth at the toe of the structure (h) over significant wave height at the toe of the structure ($H_{s,toe}$) that defines shallow water. According to Rock Manual (2007), if this ratio is greater than 3, Van der Meer approach regarded as deep water approach should be used, on the other hand, if this value is less than 3, Van Gent et al approach regarded as shallow water approach should be used. Furthermore, it is also stated in Rock Manual that Van der Meer approach can be used in moderate shallow water together with Van Gent et al approach. However, moderate shallow water definition is not clear. There are also some other parameters that defines shallow water throughout Rock Manual that are going to be investigated in this chapter.

There are problems in application of these formulas if only the criterion given by Rock Manual is used. This chapter starts with a section that shows the problems in application of these formulas. After that, design constraints that are important in defining shallow water are described. Furthermore, a computational tool is developed to investigate the problems in application considering design constraints. Using this computational tool, major stability formulas on rubble mound breakwaters are compared and analyzed by extended example studies. Finally, a new design flowchart is proposed for using Van der Meer and Van Gent et al equations together.

Note that, main concern of this thesis study is to investigate Van der Meer and Van Gent et al formulations. Hudson approaches (CERC, 1977; CERC, 1984) are given in each step to give another well-known measure.

3.1. An Example Study Defining the Problems in Application of Major Stability Formulas

To point out the differences in usage of major stability formulas, an example study is carried for the given design parameters in Table 3.1. Using the given parameters, armour stone weight of rubble mound breakwater is calculated using Hudson, Van der Meer and Van Gent et al rock slope stability formulas.

Computational results are tabulated in Table 3.2. Under the same design conditions Hudson, Van der Meer and Van Gent et al formulas result in different armour stone size. Comparatively, Van der Meer approach results in minimum armour stone size, whereas Hudson approach that uses wave height exceeded by 10% of the waves results in maximum armour stone size. There is a 19% difference in calculated armour stone diameter between Van der Meer and Van Gent et al approaches which results in a 70% difference in weight of armour stone for the same design parameters.

Table 3.1: Design Parameters for Example Study

Parameters for all Approaches		
Deep Water Significant Wave Height	H_{s0} (m)	5.3
Significant Wave Period	T_s (sec)	8.5
Depth of Construction or Depth at the Toe of the Structure	h (m)	14
Structure Face Slope	cot(α)	2
Foreshore Slope	m	0.03
UnitWeight of Armour Stone	γ_{stone} (t/m^3)	2.7
Unit Weight of Sea Water	γ_{water} (t/m^3)	1.02
Deep Water Wave Approach Angle	α_0 (˚)	0
Deep Water Significant Wave Steepness	H_{s0}/L_0	0.047
Deep Water Peak Wave Steepness	S_{op}	0.04
Significant Wave Height at the Toe of the Structure	$H_{s,toe}$ (m)	4.87
Water Depth over Significant Wave Height at the toe of the structure	$h/H_{s,toe}$	2.88
Parameters for Hudson Approach		
Deep Water Wave Height Exceeded by 10% of the waves	$H_{1/10,0}$ (m)	6.73

Table 3.1 (continued)

Stability Parameter for Non-Breaking Case	$K_{D,non\text{-}breaking}$	4
Stability Parameter for Breaking Case	$K_{D,breaking}$	2
Parameters for Van der Meer and Van Gent et al. Approaches		
Notional Permeability	P	0.4
Damage Level	S	2
Number of Waves	N	1000
Mean Wave Period	T_m (sec)	6.89
Wave Height Exceeded by 2% of the Waves at the Toe	$H_{2\%}$ (m)	7.18
Spectral Mean Energy Wave Period	$T_{m-1,0}$ (sec)	8.33
Peak Wave Period	T_p	9.18
Surf Similarity Parameter, Van der Meer Approach	ξ_m	1.95
Critical Surf Similarity Parameter, Van der Meer Approach	$\xi_{c,vdm}$	3.77
Surf Similarity Parameter, Van Gent et al. Approach	$\xi_{m-1,0}$	2.36
Critical Surf Similarity Parameter, Van Gent et al. Approach	$\xi_{c,vg}$	3.95

Table 3.2: Results of Case Study

Approach	**Condition**	**Armour Stone Diameter, D_{n50} (m)**	**Weight of Armour Stone, W (tons)**
Hudson (H_s) *	Non-Breaking	1.49	8.9
Hudson ($H_{1/10}$) *	Non-Breaking	1.89	18.2
Van der Meer	Plunging	1.38	7.0
Van Gent et al	Plunging	1.65	12.0
* *Hudson formulation can be used by taking deep water wave height as significant wave height (CERC, 1977) or wave height exceeded by 10% of the waves (CERC, 1984). In this table, weight of armour stones are presented for both methodologies.*			

Water depth at the toe of the structure over significant wave height at the toe of the structure ($h/H_{s,toe}$) is a parameter that is given as a constraint in Rock Manual. Since it is 2.88 for this example study, Van Gent et al formulation should be used considering this constraint. However, there is a 70% difference in weight compared to Van der Meer formula. Such a difference has to be questioned since it results in drastic cost and application problems in practice.

3.2. Design Constraints

Difference obtained in Section 3.1 is questioned and it is found out that the reason for such a big difference is related to shallow water definitions since it is stated by the authors that Van der Meer approach is applicable deep water and moderate shallow water whereas Van Gent et al approach is applicable to shallow water.

In general, it is a design approach to use bigger armour stone sizes in shallow water due to the complexity of shallow water. Furthermore, it is more appropriate to use spectral wave energy period in processes occurring in shallow water such as wave run-up and wave overtopping (TAW, 2002a).

In the light of these discussions, three dimensionless design constraints (Rock Manual, 2007) that are used as shallow water definitions are taken into account and

Van der Meer and Van Gent et al formulations are investigated to observe the effect of these parameters in design.

Design constraints that are used to define shallow water are given as follows:

- *Constraint 1 ($h/H_{s,toe}$ < 3):* Water depth at the toe of the structure (h) over significant wave height at the toe of the structure ($H_{s,toe}$) should be less than 3.
- *Constraint 2 ($H_{2\%}/ H_{s,toe}$ < 1.4):* Wave height exceeded by 2% of the waves at the toe of the structure over significant wave height at the toe of the structure ($H_{s,toe}$) should be less than 1.4.
- *Constraint 3 ($H_{s,toe}/ H_{s0}$ < 0.9):* Significant wave height at the toe of the structure ($H_{s,toe}$) over deep water significant wave height (H_{s0}) should be less than 0.9.

These design constraints are given in literature and defines shallow water physically by using both water depth and wave height directly, wave period implicitly. Therefore, it should be noted that these parameters are different than the well-known shallow water definition water depth (h) over deep water wave length (L_0) which should be less than 0.0157 (Ergin, 2009) for regular waves.

Design constraints for the example study given in Section 3.1 are calculated as given in Table 3.3. It is seen that, only Constraint 1 ($h/H_{s,toe} < 3$) is satisfied in the example study.

Table 3.3: Parameters Used in Example Study (Section 3.1)

Parameter	**Value**
$h / H_{s,toe}$	2.88
$H_{2\%} / H_{s,toe}$	1.47
$H_{s,toe} / H_{s0}$	0.92

3.3. Computational Tool

Due to the discrepancy in the results obtained from armour stone weight calculations and computation of design constraints, an extended comparative study is required to see the picture from a wider view. This comparative study is performed using a computational tool that consists of Wave Transformation and Regular Wave Breaking (WT) and Design Armour Stone (DAS) parts. This computational tool is developed in MATLAB environment and parts of it can be used separately or together.

3.3.1. Wave Transformation and Regular Wave Breaking (WT)

Wave transformation is needed to calculate wave height at the toe of the structure for Van der Meer and Van Gent et al approaches. Since a comparative study is the goal for developing this computational tool, a wave transformation approach that does not use detailed bathymetry information is selected. The selection is done to reduce computational cost since the computations are planned in a wide range.

Van der Meer's (1990) 1D Energy Decay Numerical Model outputs given in graphical forms in Rock Manual (2007) are used for transformation of the design wave from deep water to the toe of the structure. If the case is out of the limits of the 1D Energy Decay Numerical Model outcomes, wave height at the toe of the structure ($H_{s,toe}$) is calculated by multiplying deep water significant wave height (H_{s0}) with shoaling (K_s) and refraction (K_r) coefficients obtained from small amplitude wave theory. A sample output of Van der Meer's (1990) 1D Energy Decay Numerical Model is given in Figure 3.1 as a graph for deep water peak wave steepness ($S_{op}=H_{s0}/L_{op}$) equals to 0.05 (Rock Manual, 2007). Deep water peak wave length (L_{op}) is calculated using peak wave period (T_p).

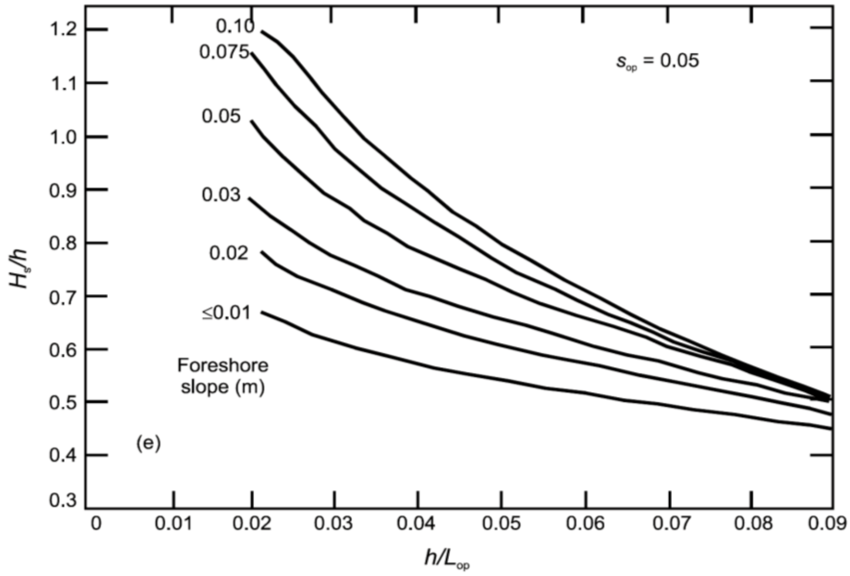


Figure 3.1: Sample Output of Van der Meer 1D Energy Decay Numerical Model for S_{op}=0.05 *(adopted from Rock Manual, 2007)*

Design wave height in Hudson approach is determined according to breaking condition. Since Hudson approach is developed by the use of regular waves, wave breaking definition given for regular waves (CERC, 1977) is used in this computational tool.

3.3.2. Design Armour Stone (DAS)

Design Armour Stone (DAS) is the second part of the computational tool that calculates armour stone weight using Hudson (CERC, 1977; CERC, 1984), Van der Meer (1988) and Van Gent et al (2003) formulations. In addition to these, DAS computes design constraints defined in Section 3.2.

The relationships between significant wave period, peak wave period, mean wave period and spectral mean energy period are needed in calculation of armour stone weights. The relationship between peak wave period and spectral mean energy period is taken from (Dingemans, 1987). Furhermore, this study is confined to parameters taken from Goda (2000) for a JONSWAP P-Type Spectrum with γ=3.3

which is a commonly used in defining the spectrum of the sea state around Turkish coasts.

Wave height exceeded by 2% of the waves ($H_{2\%}$) is used in Van der Meer and Van Gent et al approaches. It is calculated using the methodology proposed by Battjes and Groenendijk (2000).

3.4. Extended Example Studies Analyzing Major Stability Formulas

Extended example studies are selected specifically to show the trends of Hudson (CERC, 1977; CERC, 1984), Van der Meer (1988) and Van Gent et al (2003) formulations for different steepness values at different depths covering a wide range that can be encountered in practice.

In these example studies, for a given deep water significant wave steepness (H_{s0}/L_0), deep water significant wave height range with small increments and depth at the toe of the structure is taken. WT calculates significant wave period for each wave height, checks breaking condition (CERC, 1977) for Hudson approaches and transforms wave to the depth at the toe of the structure; after that, DAS calculates armour stone diameter using Hudson, Van der Meer and Van Gent et al approaches and design constraints for each case. Finally, curves showing change of $h/H_{s,toe}$ versus armour stone diameter and relative differences between Van der Meer and Van Gent et al approaches are drawn indicating design constraints.

Five example studies (ES) are carried out. In Table 3.4, parameters for each example study are presented. Parameters that are not given in Table 3.4 are taken as the same as the parameters given in Table 3.1. Results are presented in Figures 3.2 – 3.11

Table 3.4: Design Parameters Used in Example Study

Parameters	**ES 1**	**ES 2**	**ES 3**	**ES 4**	**ES 5**
H_{s0}/L_0	0.04	0.04	0.04	0.035	0.05
H_{s0} (m)	2.5 to 8	3 to 8	3 to 8	3 to 8	3 to 8
h (m)	8	10	12	8	8

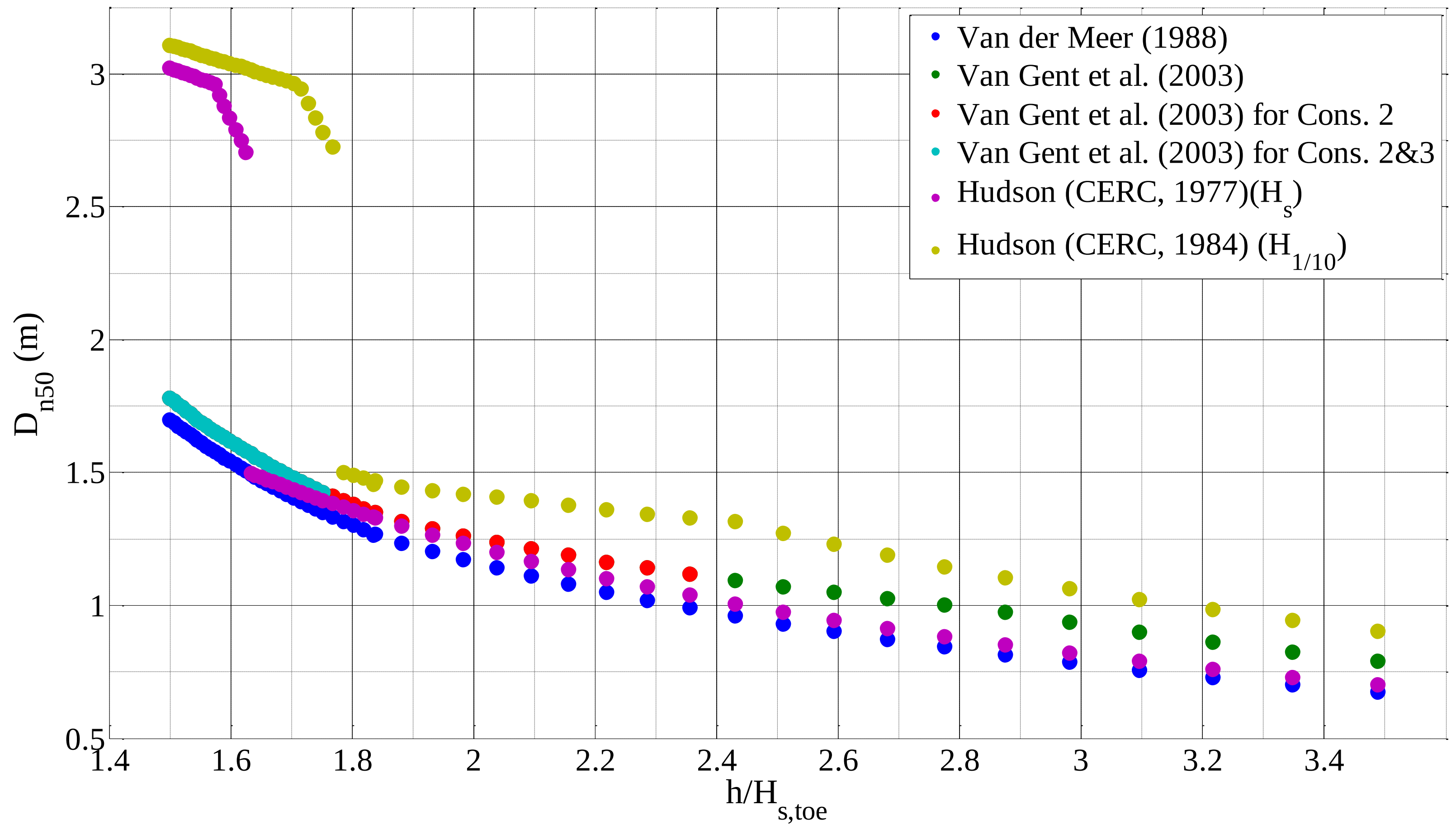


Figure 3.2: h/Hs,toe vs Dn50 (m) for Example Study (ES) 1

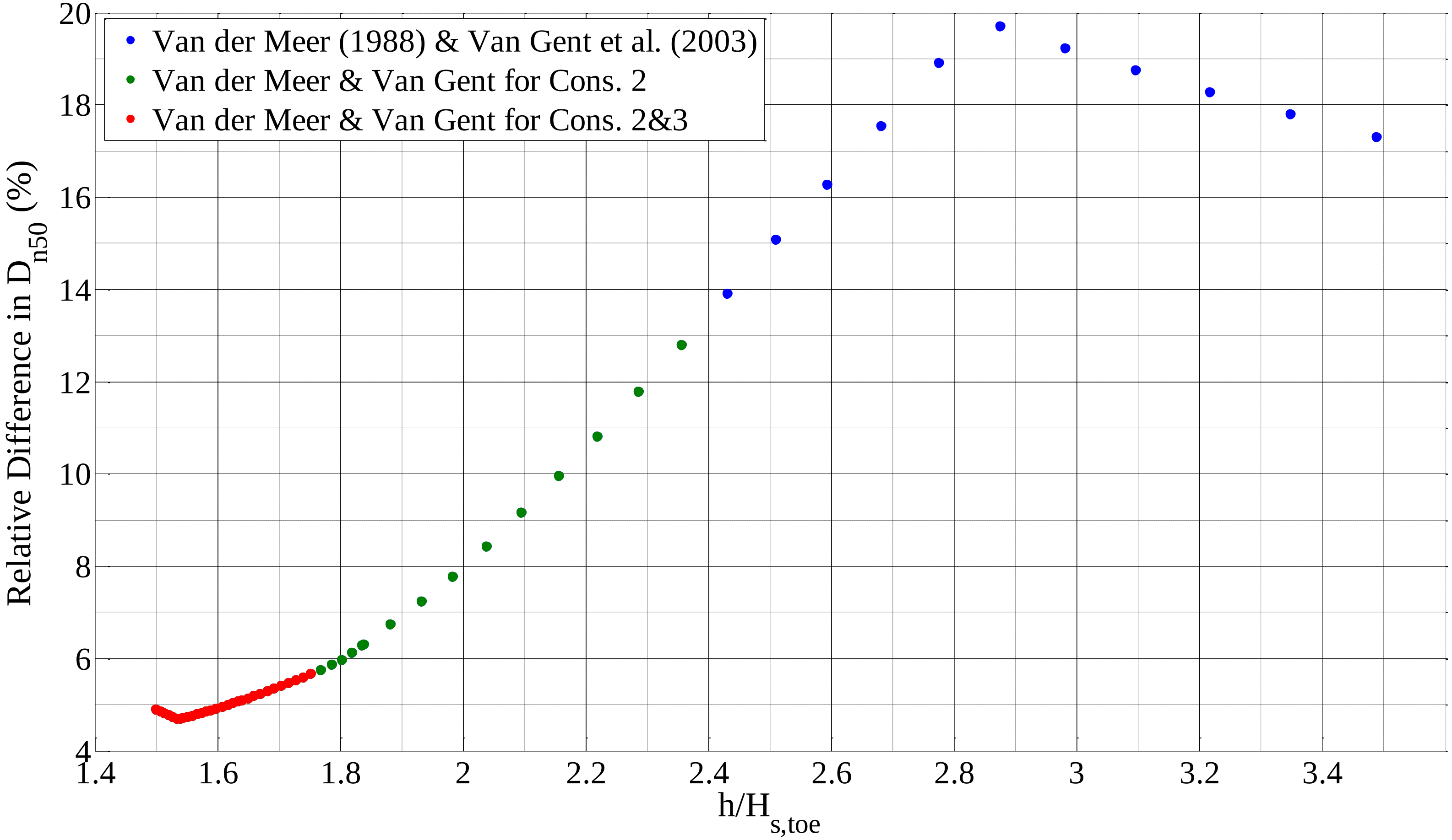


Figure 3.3: $h/H_{s,toe}$ vs Relative Difference in Armour Stone Diameter (%) for Example Study (ES) 1

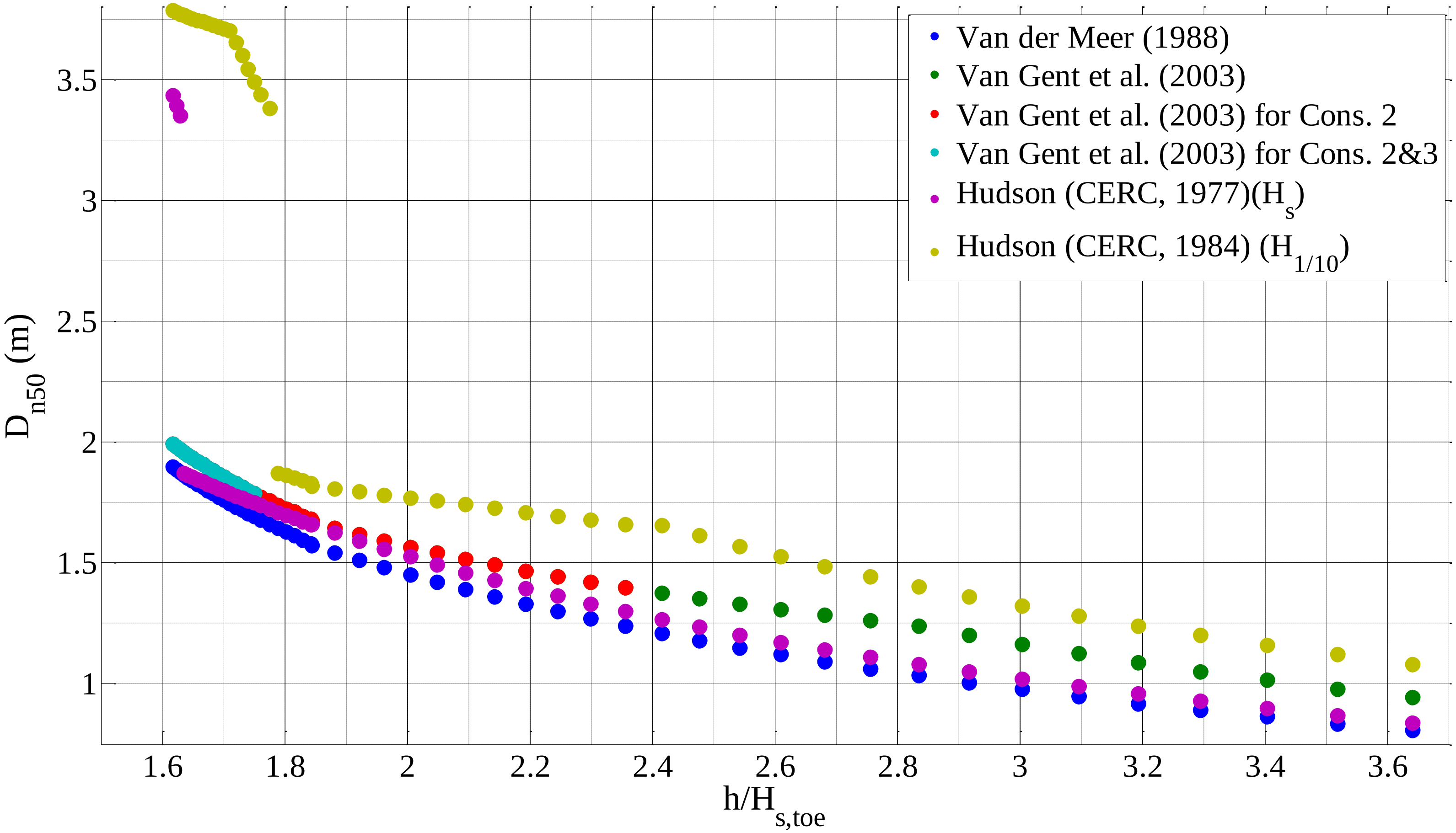


Figure 3.4: $h/H_{s,toe}$ vs D_{n50} (m) for Example Study (ES) 2

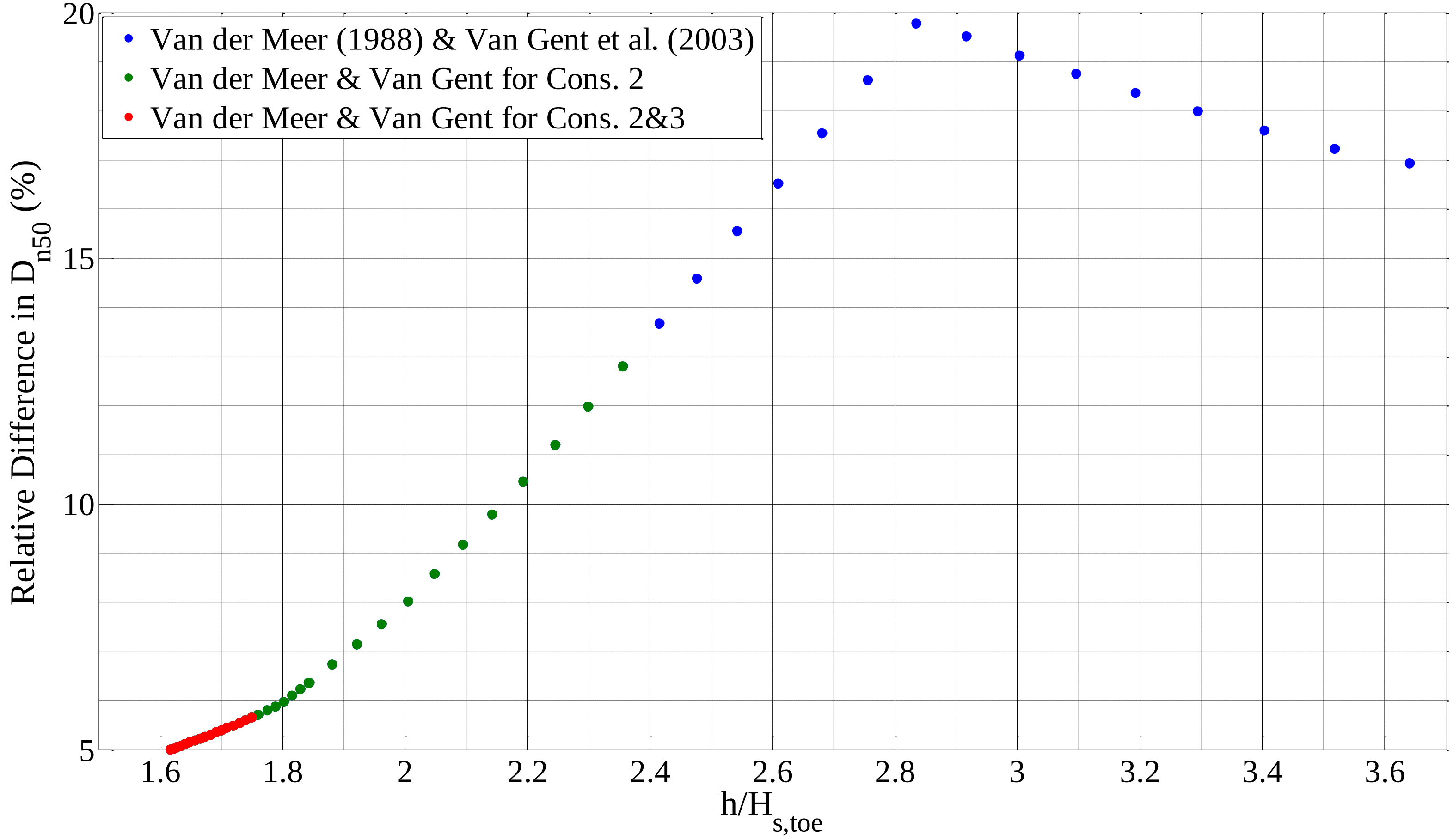


Figure 3.5: $h/H_{s,toe}$ vs Relative Difference in Armour Stone Diameter (%) for Example Study (ES) 2

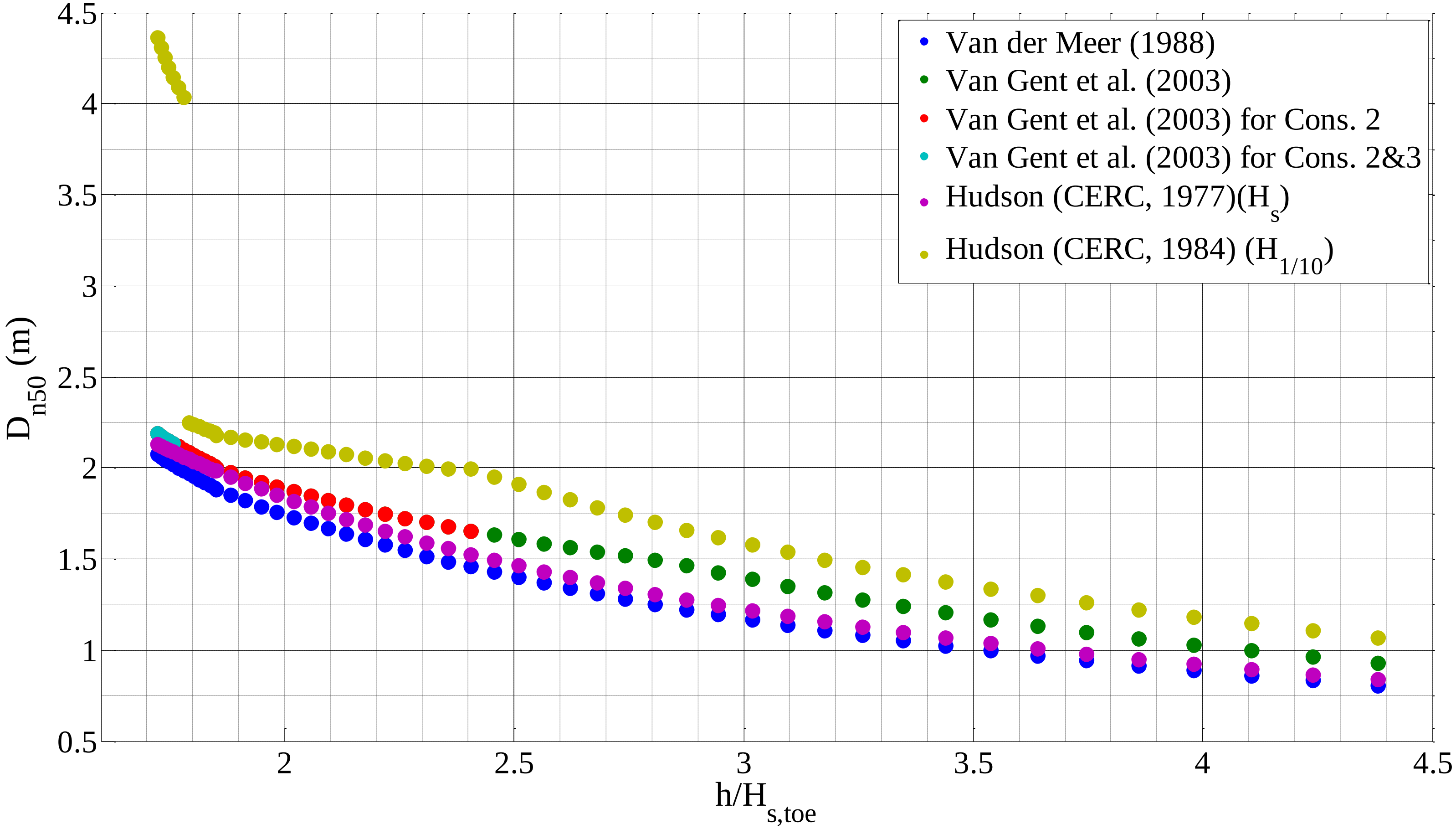


Figure 3.6: $h/H_{s,toe}$ vs D_{n50} (m) for Example Study (ES) 3

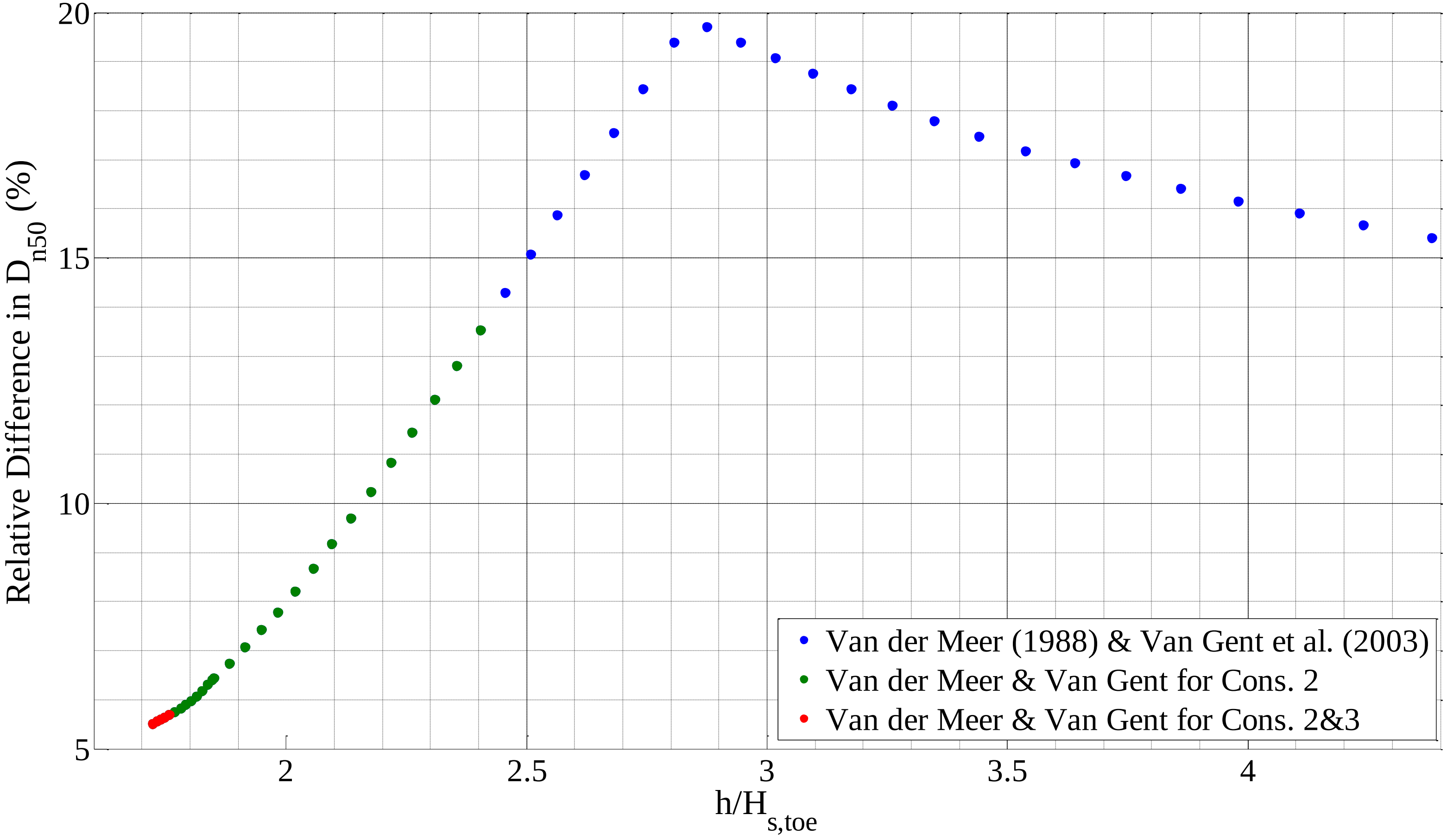


Figure 3.7: $h/H_{s,toe}$ vs Relative Difference in Armour Stone Diameter (%) for Example Study (ES) 3

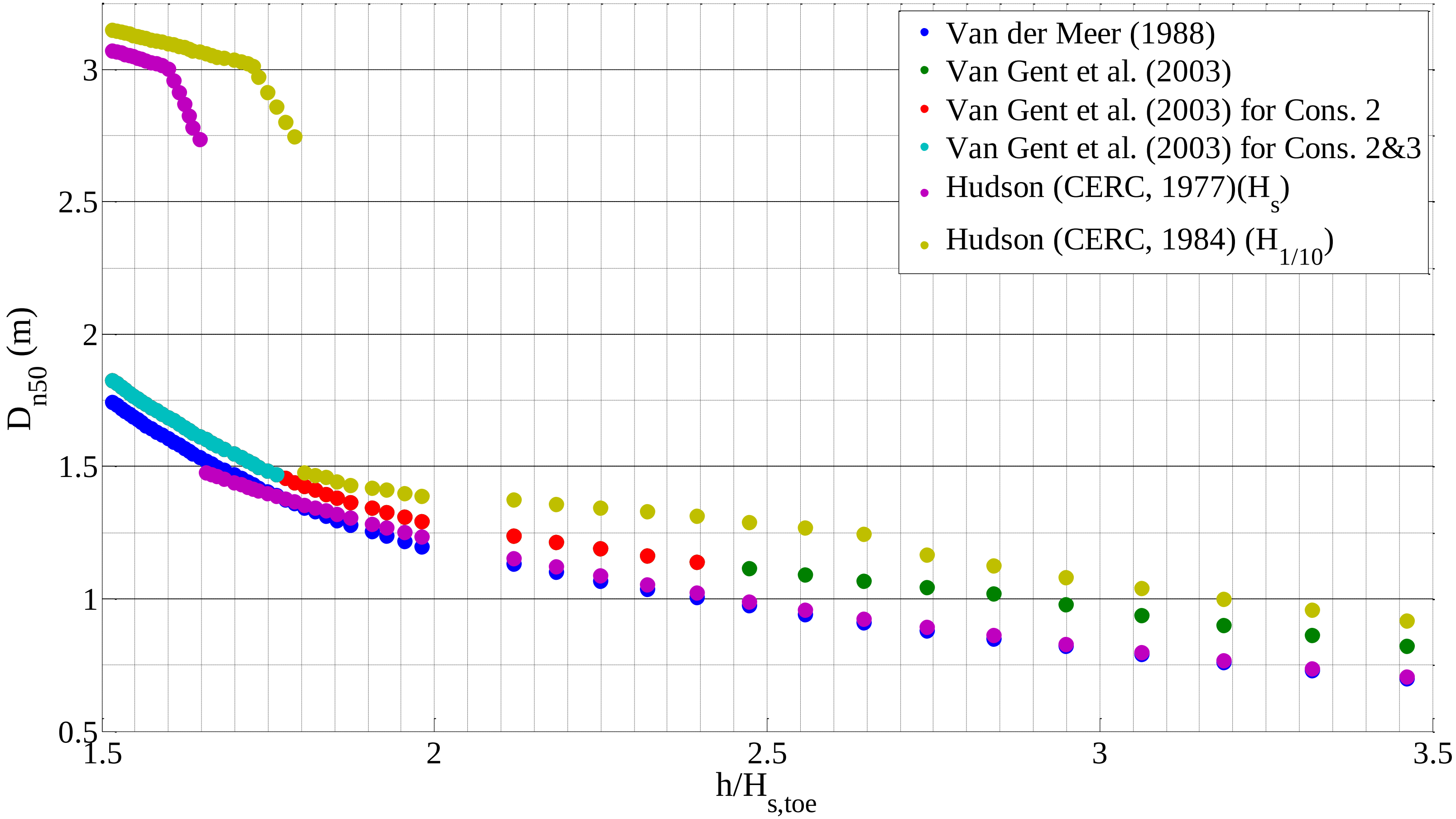


Figure 3.8: $h/H_{s,toe}$ vs D_{n50} (m) for Example Study (ES) 4

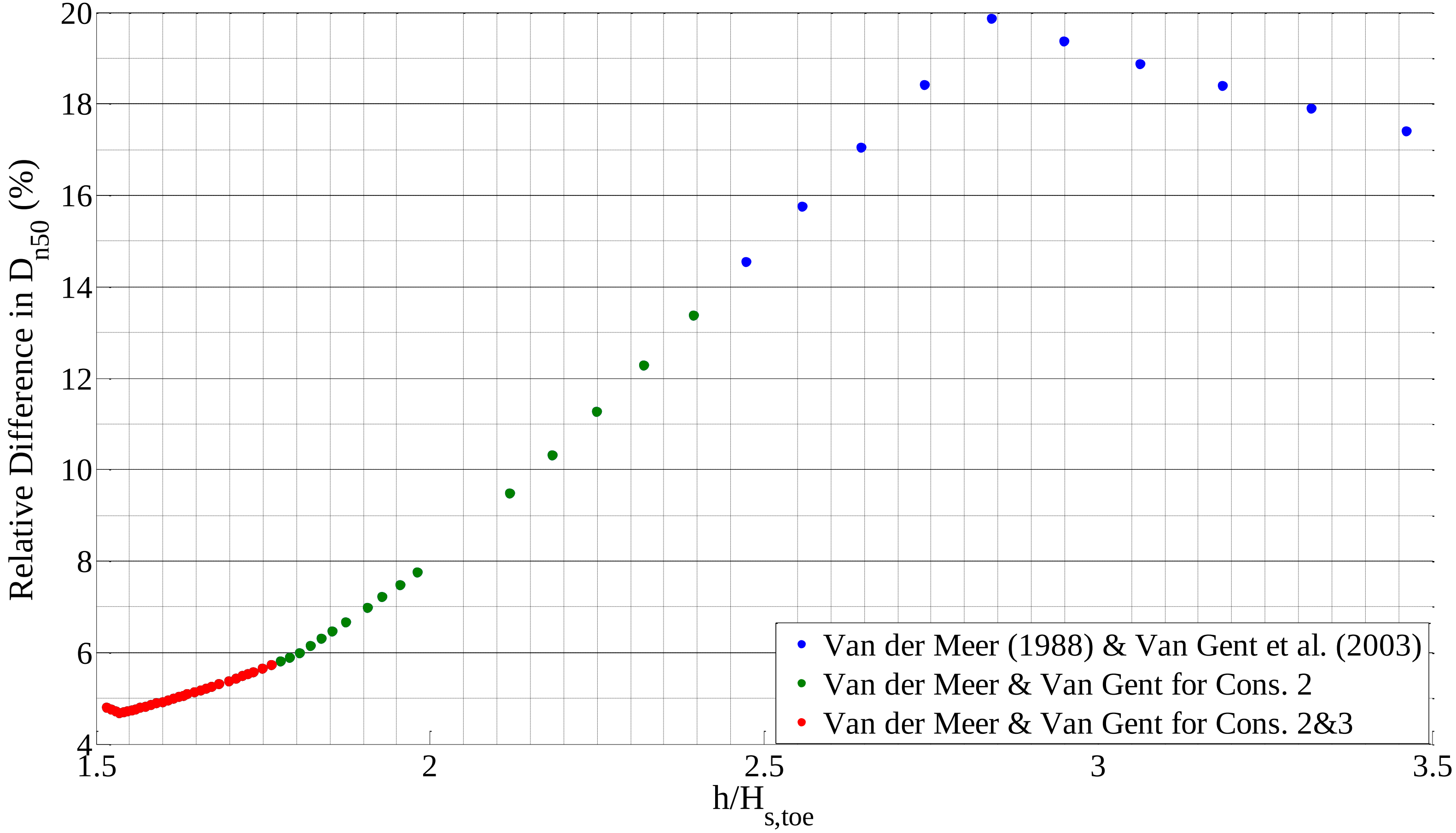


Figure 3.9: $h/H_{s,toe}$ vs Relative Difference in Armour Stone Diameter (%) for Example Study (ES) 4

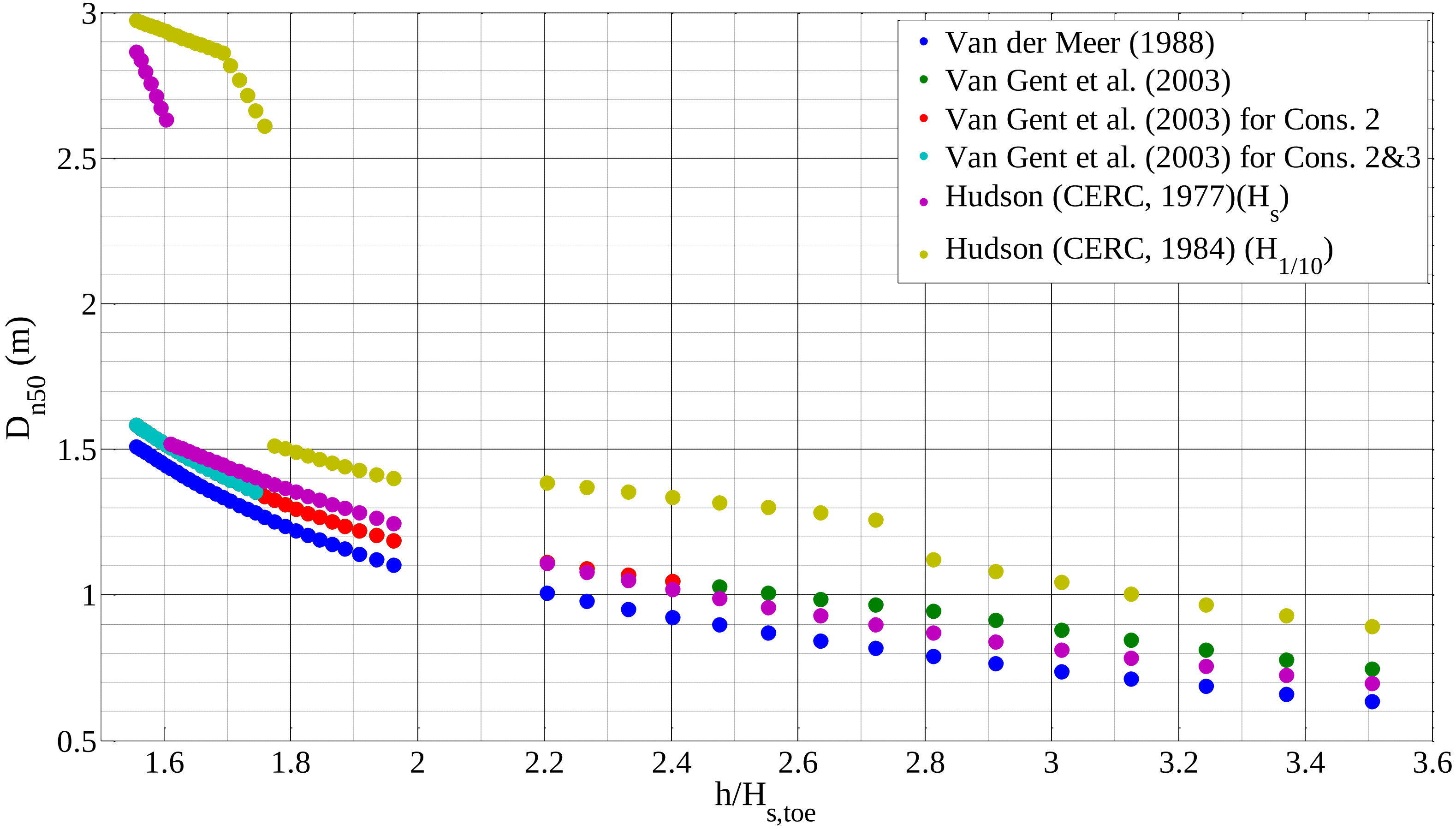


Figure 3.10: $h/H_{s,toe}$ vs D_{n50} (m) for Example Study (ES) 5

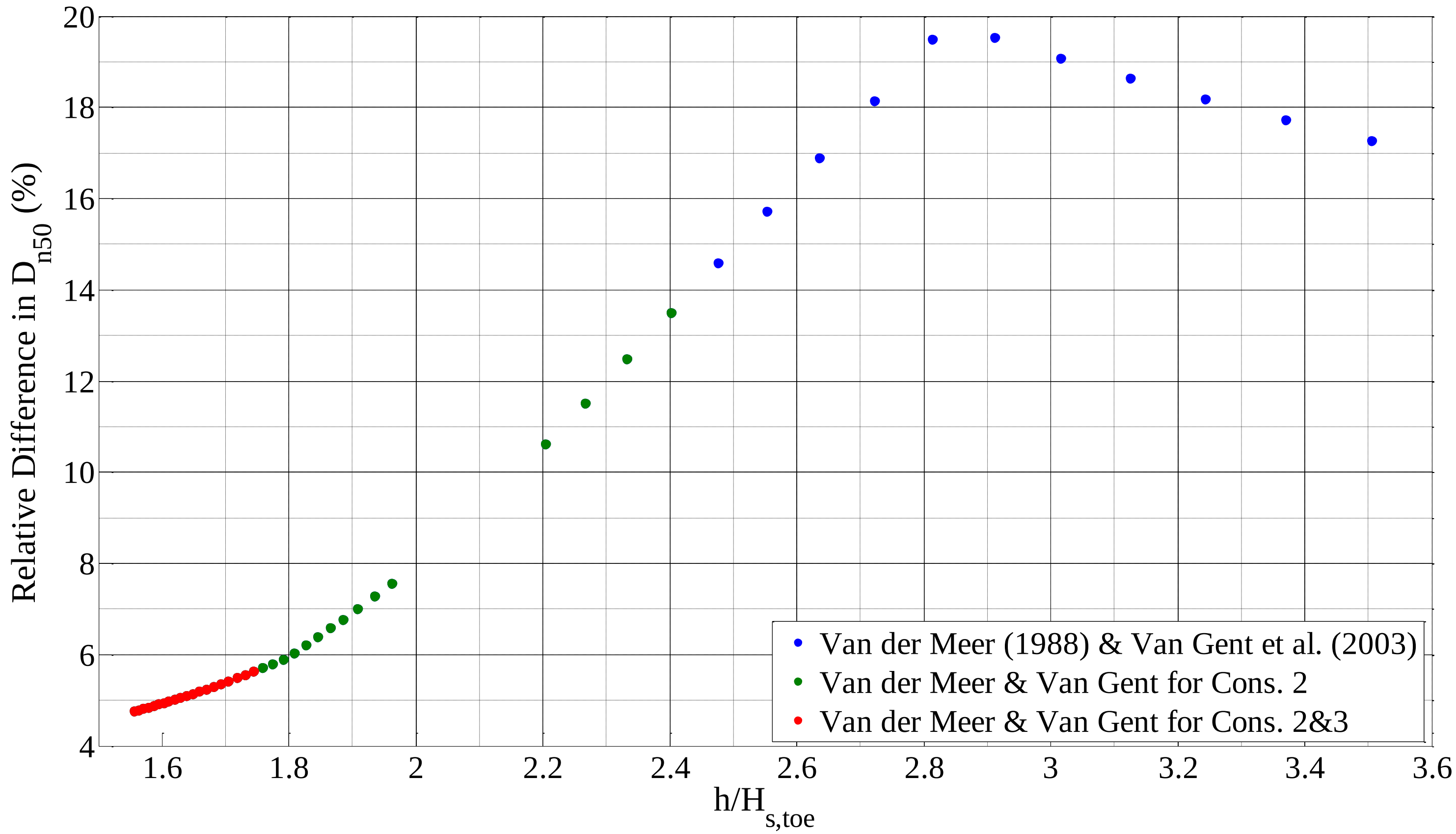


Figure 3.11: $h/H_{s,toe}$ vs Relative Difference in Armour Stone Diameter (%) for Example Study (ES) 5

Figures 3.2-3.11 give the trend of Van der Meer and Van Gent et al formulations where effect of design constraints is indicated by colors. Note that, x-axis of Figures 3.2-3.11 is Constraint 1 itself. From the figures giving relative differences in D_{n50}, it is observed that relative differences are between 4% and 20% for all selected example studies and decreases when design constraints are considered. It is shown by Figures 3.2-3.11 that the relative difference is between 4% and 6% when all design constraints are satisfied. In other words, relative difference obtained when all design constraints are satisfied is similar to the difference taken when a rubble mound structure is designed in shallow water due to complexity which makes sense.

To view Example Study 1 (ES1), Figures 3.12 and 3.13 are given. These figures show the results for Hudson (CERC, 1977; CERC, 1984), Van der Meer (1988) and Van Gent et al (2003) approaches; differently, constraints are indicated by arrows at the top of the figures this time. Furthermore, weights of the armour stones are indicated in the y-axis at the right hand side that allows comparing formulations with another meausure.

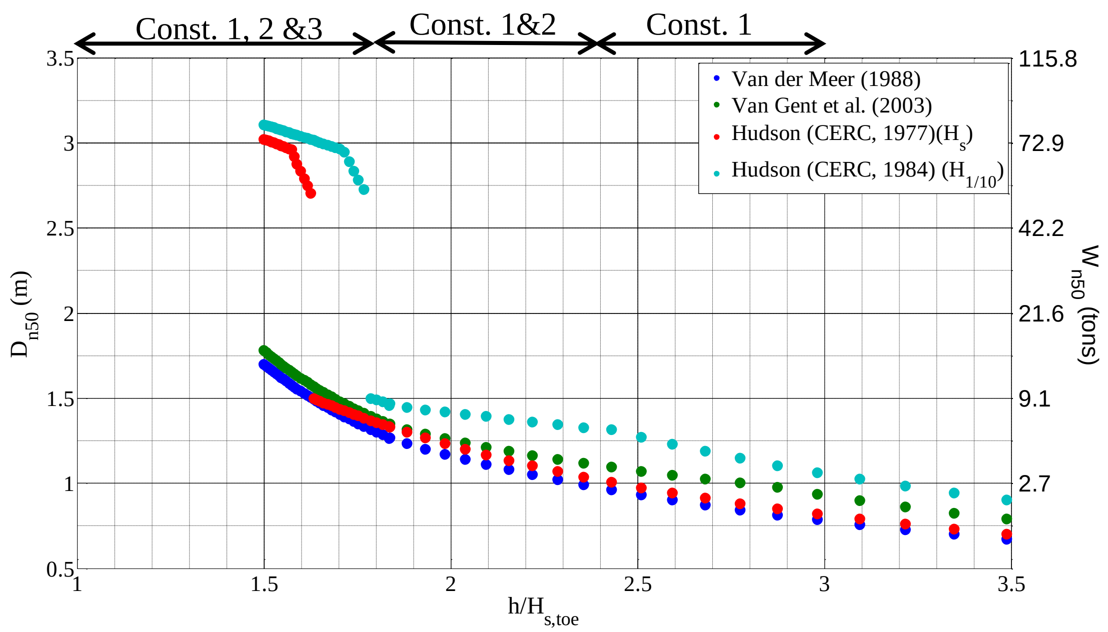


Figure 3.12: A Closer Look to Example Study 1

From the example studies carried out, it is concluded that, using Van Gent et al approach at shallow water seems to be more appropriate as a more conservative method noting that Van der Meer approach is not applicable in this region. Furthermore, Van Gent et al approach describes the process in shallow water in a better way since it uses spectral mean energy period (Tm-1,0).

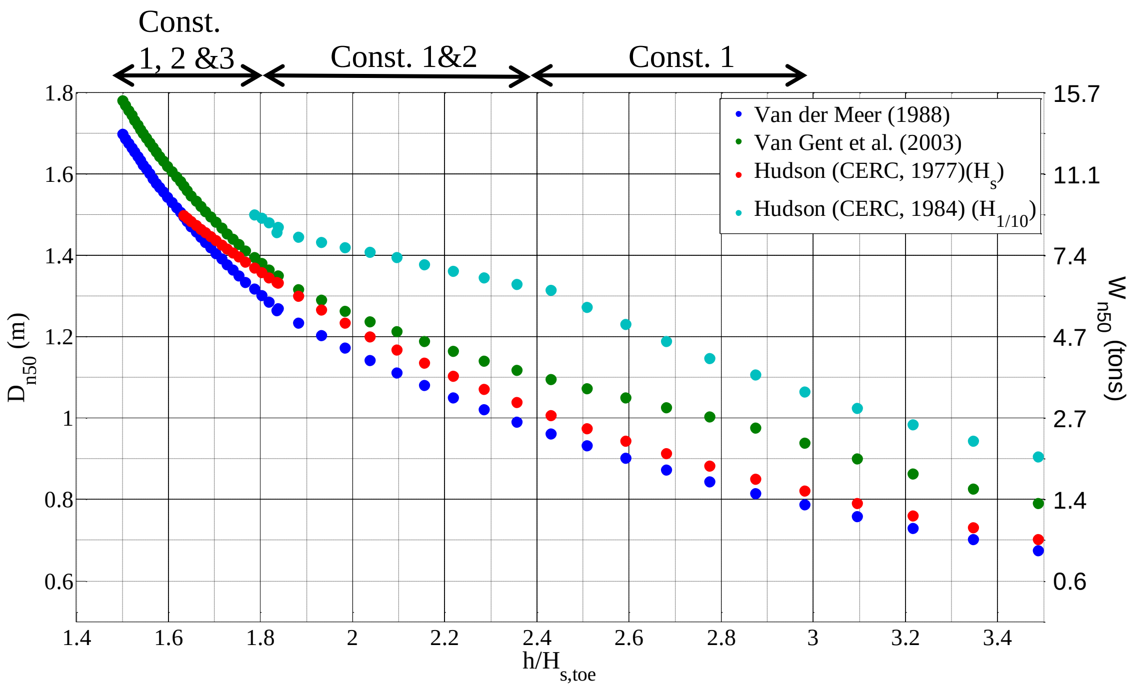


Figure 3.13: A More Closer Look to Example Study 1

Beyond very shallow water, Van Gent et al approach gives up to 70% larger armour stone weight in comparison to Van der Meer approach as seen from example studies. Due to its wide field application, Van der Meer approach is recommended for deep and moderate shallow water as also stated in Rock Manual.

It is observed from Figures 3.2-3.11 that Hudson (CERC, 1977) approach generally gives results between Van der Meer and Van Gent et al approaches if the breaking condition is non-breaking. However, if the design condition is breaking, Hudson (CERC, 1977) approach also gives higher results than Van der Meer and Van Gent et al approaches. This result is another indicator that Van der Meer approach is a more applicable approach in deep and moderate shallow water since it is known that

Hudson (CERC, 1977) approach is widely applied to countless number of cases as a conservative approach. Furthermore, it is observed that Hudson (CERC, 1984) is the most conservative approach giving the largest armour stone size.

3.5. Proposed Flow Chart in the Design of Armour Layer of Rubble Mound Breakwaters

Results obtained in Section 3.4 are indicators of the new design approach. Although this study is a mathematical approach, application of Van der Meer approach in deep and moderate shallow water and application of Van Gent et al approach in shallow water seem more appropriate considering the discussions given in Section 3.4. Since it is not possible to scan all the ranges that a rubble mound breakwater can be designed, chosen examples are assumed to cover the range for a comparative study of the trend of the major stability formulas. Furthermore, the conclusions drived in Section 3.4 are tested by physical model that are presented in Chapter 4.

Flow chart in the design of armour layer of rubble mound breakwaters is proposed as application of Van Gent et al approach at shallow water, i.e. when all the design constraints are satisfied. On the other hand, Van der Meer approach should be applied in any other case. This flowchart is summarized in Figure 3.12.

DAS code is updated according to this new flowchart for the use of Van der Meer and Van Gent et al equations according to design constraints.

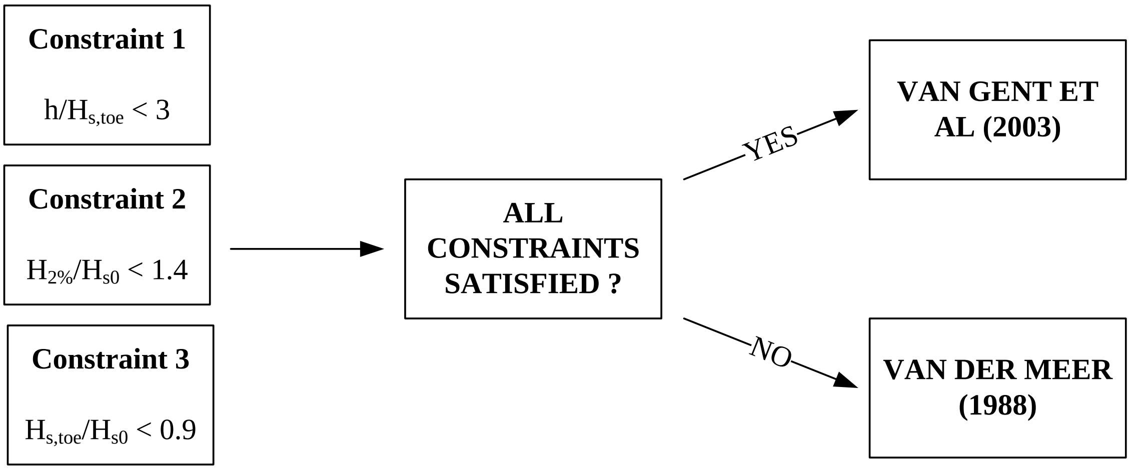


Figure 3.14: Proposed Flowchart in the Design of Armour Layer of Rubble Mound Breakwaters

CHAPTER 4

PHYSICAL MODEL EXPERIMENTS

In this chapter physical model experiments performed to test results obtained in Chapter 3 are explained. Construction of experimental cases, scaling approach, experimental setup, generation and analysis of waves and results of physical model experiments are clarified. Furthermore, physical model experiments are discussed in terms of new design approach proposed in Chapter 3.

4.1. General Overview of the Experiments

In Chapter 3, three design constraints are discussed to determine how Van der Meer (1988) and Van Gent et al (2003) approaches should be used. These design constraints are

- *Constraint 1 ($h/H_{s,toe}$):* Water depth at the toe of the structure (h) over significant wave height at the toe of the structure ($H_{s,toe}$).
- *Constraint 2 ($H_{2\%}/H_{s,toe}$):* Wave height exceeded by 2% of the waves at the toe of the structure over significant wave height at the toe of the structure ($H_{s,toe}$).
- *Constraint 3 ($H_{s,toe}/H_{s0}$):* Significant wave height at the toe of the structure

($H_{s,toe}$) over deep water significant wave height (H_{s0}).

In Chapter 3, the proposition that Van Gent et al (2003) equations should be used when all three design constraints are satisfied, otherwise, Van der Meer (1988) equations should be used are tested by physical model experiments.

In the construction of cross-section in the wave flume, design constraints, hence use of these design formulas, are controlled by changing water depth (h), significant wave height at the toe of the structure ($H_{s,toe}$) and significant wave period (T_s). Armour stone weight is fixed in 4-6 tons stone range.

Two critical cases are selected and tested for the physical model experiments. Case 1 is a case where all the design constraints are satisfied and armour stone weights are found using Van der Meer (1988) equations. Note that, it should be found with Van Gent et al (2003) equations according to proposition in Chapter 3. On the other hand, Case 2 is again a case where all the design constraints are satisfied, however, armour stone weight is found using Van Gent et al (2003) equations for this case. Hence, the expectations from these experiments are to obtain significant damage in Case 1 and no damage (or start of damage) in Case 2. In Table 4.1, design parameters used to construct these cases are given.

A typical single slope cross-section with a toe berm is designed using conventional design parameters. Weight of armour stones are fixed in 4-6 tons stone range as stated in Table 4.1. Height of the breakwater is determined considering run-up height. Calculations for run-up height are done using procedure given by TAW (2002a). Other structural properties and parameters used in Van der Meer (1988) and Van Gent et al (2003) approaches are provided in Table 4.2.

Table 4.1: Design Parameters for Case 1 and Case 2

Parameters		**Case 1**	**Case 2**
Deep Water Significant Wave Height (m)	H_{s0}	5.20	4.50
Significant Wave Period (sec)	T_s	8.10	7.60
Deep Water Wave Steepness	H_{s0}/L_0	0.050	0.049
Foreshore Slope	m	0.033	0.033
Water Depth at the toe of the structure (m)	h	8.00	7
Significant Wave Height at the toe of the structure (m)	$H_{s,toe}$	4.60	4.03
Stone Weight according to VdM*	$W_{50,VdM}$	4- 6 tons	2-4 tons
Stone Weight according to VG**	$W_{50,VG}$	6-8 tons	4-6 tons
Constraint 1	$h/H_{s,toe}$	1.739	1.738
Constraint 2	$H_{2\%}/H_{s,toe}$	1.301	1.301
Constraint 3	$H_{s,toe}/H_{s0}$	0.880	0.895
Design Formula used		VdM*	VG **
Design Formula that should be used according to New Design Flowchart		VG**	VG **
** VdM: Van der Meer (1988) equations* *** VG: Van Gent et al (2003)*			

Table 4.2: Structural Design Parameters

Paramater		Value	Paramater		Value
Unit weight of stone (tons/m^3)	γ_s	2.7	Structure Face Slope	cot(α)	2
Unit weight of water (tons/m^3)	γ_w	1.025	Notional Permeability	P	0.4
Damage Level	S	2	Number of waves	N	1000

Breakwater cross-section is constructed using three layers, namely, core, filter and armour layers. Crest width is determined using the length of three stone diameters in that layer. Furthermore, armour layer thickness is taken as two stone diameters. Dimensional requirements given by Ergin (2009) are satisfied by these implementations. In Figure 4.1, breakwater cross-section in prototype scale with required dimensions is given.

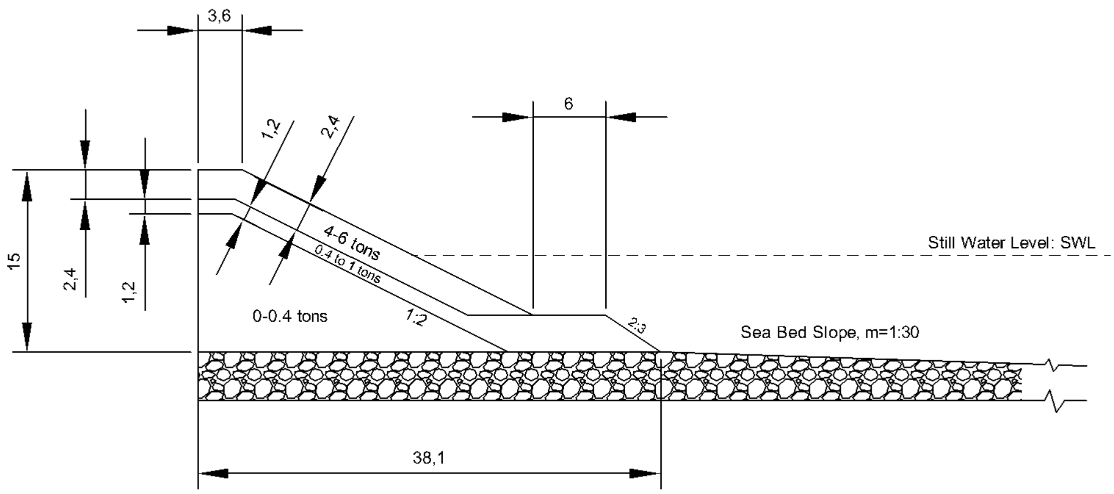


Figure 4.1: Breakwater Cross-Section in Prototype Scale
(Dimensions are in meters and figure is not to scale.)

4.2. Scaling of Breakwater Cross-Section

Froude Law is used in physical modelling of most of the coastal and ocean engineering problems since inertial and gravitational forces are dominant in wave motion and in wave effects on coastal structures. Froude number is square of water particle velocity (u) over water depth (h) multiplied by acceleration of gravity (g) given by Equation 4.1.

$$F_r^2 = \frac{u^2}{g\,h} \qquad [4.1]$$

Froude number in model denoted by "$\left(F_r\right)_m$" and Froude number in prototype shown by "$\left(F_r\right)_p$" must be equal as given in Equation 4.2.

$$\left(F_r\right)_p = \left(F_r\right)_m \qquad [4.2]$$

Length (L) and time (t) scales (λ) can be derived using Equations 4.1 and 4.2 as given in Equations 4.3 and 4.4.

$$\lambda_L = {L_m}/{L_p} \qquad [4.3]$$

$$\lambda_t = {t_m}/{t_p} = \sqrt{\lambda_L} \qquad [4.4]$$

Stability of armour units can be modelled correctly if the stability numbers in both prototype and the model are the same (CERC, 1984). By equating stability numbers, weight scale (λ_W) can be found as presented in Equation 4.5.

$$\lambda_W = \left(\lambda_L^3\right)\frac{\left(\gamma_r\right)_m}{\left(\gamma_r\right)_p}\left[\frac{\left(\gamma_r\right)_p / \left(\gamma_w\right)_p - 1}{\left(\gamma_r\right)_m / \left(\gamma_w\right)_m - 1}\right] \qquad [4.5]$$

In Equation 4.5, $\left(\gamma_r\right)_m$ and $\left(\gamma_r\right)_p$ are unit weights of stones that are used in model and prototype, respectively, which are taken as 2.7 t/m^3 and 2.7 t/m^3. Furthermore, $\left(\gamma_w\right)_m$ and $\left(\gamma_w\right)_p$ are unit weight of water that is used in model and unit weight of sea water in prototype. In the experiments, unit weight of water is taken as 1.0 t/m^3 instead of sea water which has a unit weight of 1.025 t/m^3.

Considering the limitations of wave channel and wave generator, length scale is selected as 1:20. Time scale and weight scale are determined with the use of Equations 4.4 and 4.5 accordingly. Length scale, time scale and weight scale are presented in Table 4.3.

Table 4.3: Model Scales

Length	$\lambda_L = 1:20$
Time	$\lambda_t = 1:4.4721$
Weight	$\lambda_w = 1.2 \times 10^{-5}$

Using length scale and weight scale, breakwater cross-section given in Figure 4.1 is scaled and given in Figure 4.2. Armour layer stones are colored with different colors in layers to observe the motion of stones. Colors of armour layer stones are indicated in Figure 4.3. Moreover, weight of stones in armour, filter and core layers are calculated and presented in Table 4.4.

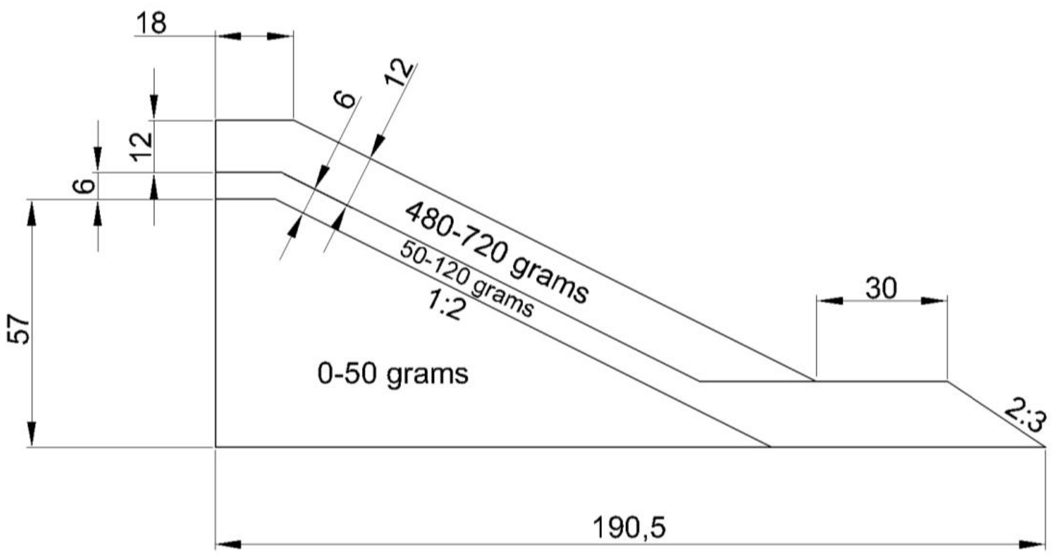


Figure 4.2: Scaled Cross-Section

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

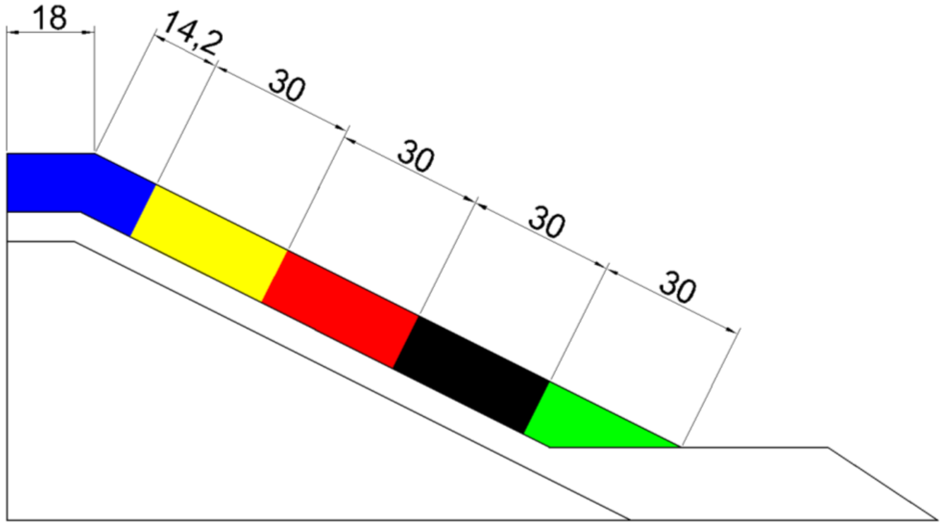


Figure 4.3: Colored Armour Layer

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

Table 4.4: Weight of Stones

Layer	**Original Weight of Stones**	**Scaled Weight of Stones**
Armour Layer	4-6 tons	480-720 grams
Filter Layer	0.4-1 tons	50-120 grams
Core Layer	0-0.4 tons	0-50 grams

4.3. Experimental Setup

Physical model experiments are conducted in the wave channel of METU Department of Civil Engineering Ocean Engineering Research Center. The dimensions of wave channel are 28.8 meters in length, 6.2 meters in width and 1.0 meters in depth. There are wave absorbers at one end of the flume, whereas, there is an irregular wave generator at the other end. There is an inner channel where the physical models and slope in front of the structure are constructed. Wave gauges are also placed in this inner channel. Inner channel has dimensions of 18m X 1.5m X 1.0m and separated by special glass material called Plexiglass walls from the bigger channel. The aim of the inner channel is to reduce reflection effect in the wave channel from side walls of bigger channel. Layout of wave channel is given in Figure 4.4.

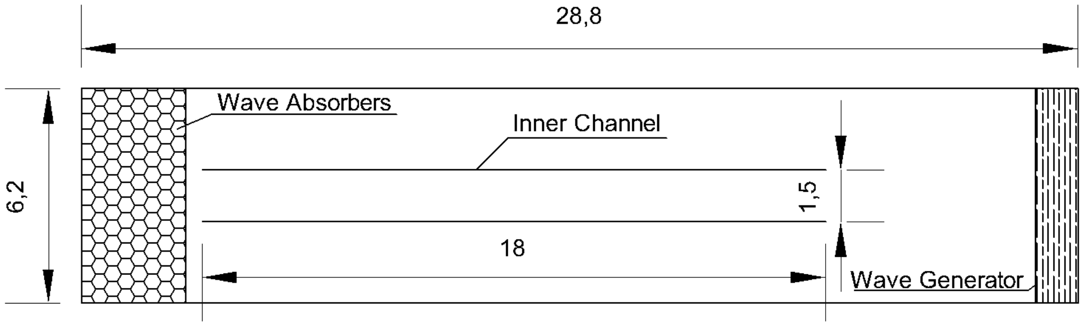


Figure 4.4: Layout of Wave Channel (top view)
(Dimensions are in meters and figure is not to scale.)

Wave generator of the flume is a piston-type wave maker. Digital information related to wave motion is converted to analog information by a wave synthesizer. Hydraulic equipment of piston-type wave maker produces the required motion by moved piston.

For wave measurements, six DHI 202 type wave gauges are used. Typically, wave gauges have two steel bars. These steel bars actually measure voltage differences and this information is interpreted as water level fluctuation time series data. Wave gauges in this experiment are placed as couples as shown in Figure 4.5. Wave gauges O and E are at the toe of the structure and other gauges are placed at the deeper region of the channel. These wave gauges are placed as couples to analyze reflection using Goda and Suzuki (1976).

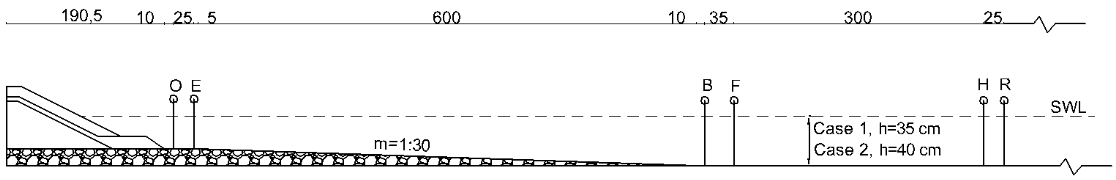


Figure 4.5: Side View of the Experimental Setup
(Dimensions are given in centimeters and figure is not to scale.)

A 1:30 slope is placed in the wave channel. Sea bed slope can be seen in Figure 4.6 which is constructed using two plates and small stones placed at the sides of plates.

Figure 4.6: 1:30 Slope in the wave channel

Cross-section placed in the wave channel is given in Figure 4.7a and 4.7b. Finally, experiments are recorded by video cameras from two angles.

(a) (b)

Figure 4.7: Cross-Section in the Wave Channel

4.4. Generation and Analysis of Waves

In the physical model experiments, waves in the wave flume are generated by a piston-type wave maker as stated in Section 4.3. A time series of piston position is inputted to software and by the help of this software digital signal is transformed to analog signal. This analog signal changes the position of the piston to generate irregular waves in the wave flume. Similarly, measurements from wave gauges are transformed to time series of water level fluctuations. After that generated wave characteristics are obtained by analyzing these time series. To do this, zero-up crossing method is used. Furthermore, spectral wave definition is used to produce and analyze an irregular wave time series.

For generating irregular waves using a certain wave spectrum and analyzing the measured data, MATLAB codes developed by Baykal (2010) are used. This code has options to produce waves using Bretschneider-Mitsuyasu type wave spectrum or JONSWAP type spectrum. For these experiments, JONSWAP type wave spectrum is used with shape parameter γ=3.3.

Shallow water defined by design constraints as given in Chapter 3 can be regarded as rather extreme cases where the rubble mound breakwater model tested as Case 1 and

Case 2. However, with the existing laboratory facilities e.g. wave flume and wave generator capacities, to model the waves which would satisfy the design constraints is a challenging issue. To achieve this goal, that is to find appropriate wave conditions for experiments, series of trials are carried out by changing scaling parameters. In physical model experiments, calibration of model waves in the wave flume which satisfies the design constraints is a time consuming process together with the calibration needed to interpret measurements by wave gauges repeated for each set throughout the experiments. Physical model experiments for both cases are repeated for three times.

4.5. Results of the Physical Model Experiments

Damage in the cross-section is evaluated using Van der Meer damage parameter (S) given by Equation 2.13 as eroded area over square of the diameter of the stone considering a 50% cumulative distribution (D_{n50}). Before and after each wave test, profile of the cross-section is measured along two lines that are indicated in Figure 4.8 by a pointed rod perpendicularly placed over the cross-section. Measurements are done with 5 cm intervals. Measurements are smoothed by fitting cubic splines with 0.01 cm intervals in x-axis and using this smooth curves eroded area along two lines are computed. The average of two eroded area is taken and divided by square of nominal armour stone diameter to obtain Van der Meer damage parameter (S).

Due to the randomness in placement of armour stones and small variations in measured wave properties compared to calculations described in Section 4.1, there are some differences in damaged profiles and computed damage parameters (S) in each repetition as expected. However, it is worth saying that these differences are small enough and in same ranges. In this section, details of each case and each set are presented.

4.5.1. Case 1

Case 1 is the case where the armour stone weight is found using Van der Meer (1988) formula when three constraints are satisfied. Therefore, considerable damage is expected for this case to approve results obtained in Chapter 3. All three sets showed that wave properties selected for Case 1 cause considerable damage in cross-section that is regarded as intermediate damage level in terms of Van der Meer damage parameter. Details are given throughout Section 4.5.1.

<u>Case 1 – Set 1:</u>

Cross-section is constructed as given in Figure 4.8 and profile is measured along two lines along the cross section as shown on the figure. Waves are applied to the cross-section using pre-determined time series. The resulting cross-section is given in Figure 4.9. Some of the stones are moved that are indicated in Figure 4.9.

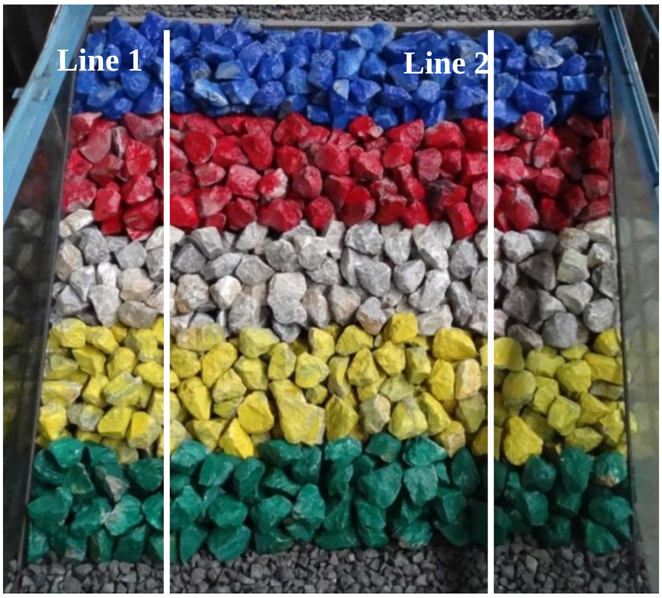


Figure 4.8: Before Case 1 – Set 1

Figure 4.9: After Case 1 – Set 1

Profile measurements given in Figure 4.10 are used to calculate Van der Meer damage parameter (S) which is found as 4.7971 which corresponds to “intermediate damage level” according to Table 2.3.

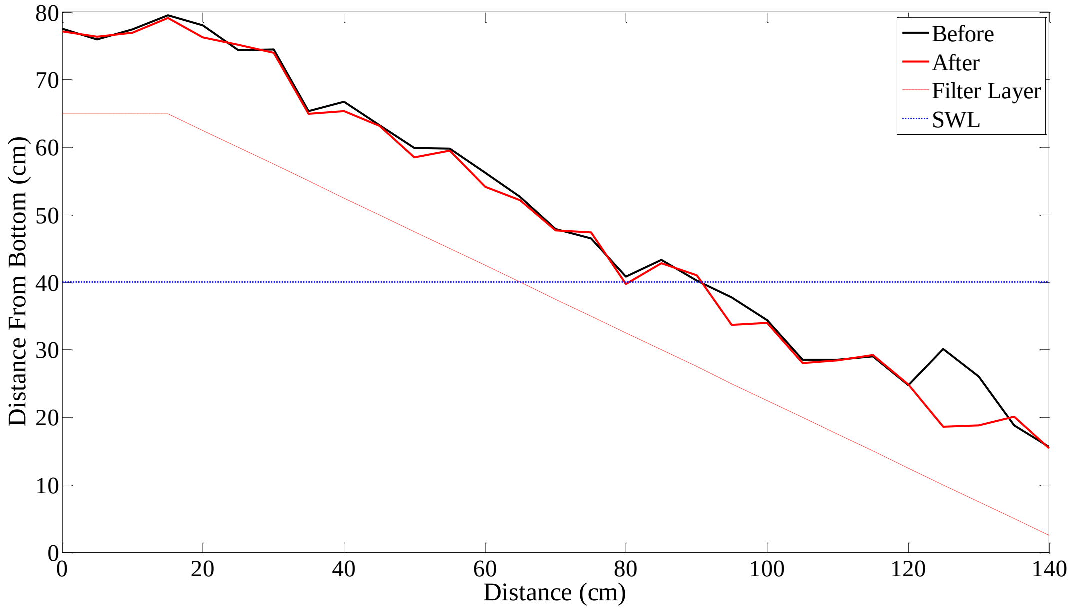


Figure 4.10: Profile Measurements for Case 1 – Set 1

Case 1 – Set 2:

Cross-section is reconstructed (Figure 4.11) for Case 1 – Set 2.

Figure 4.11: Cross-Section before Case 1 – Set 2

Appropriate wave series again applied to cross-section and the changes in cross-section are recorded. In Figure 4.12, the damaged cross-section after Case 1 – Set 2 is presented and some of the moved stones are indicated on the figure.

Figure 4.12: Cross-Section after Case 1 – Set 2

Van der Meer damage parameter (S) is calculated using profile measurements. It is found as 6.2534 for this set. Profile measurements are provided in Figure 4.13.

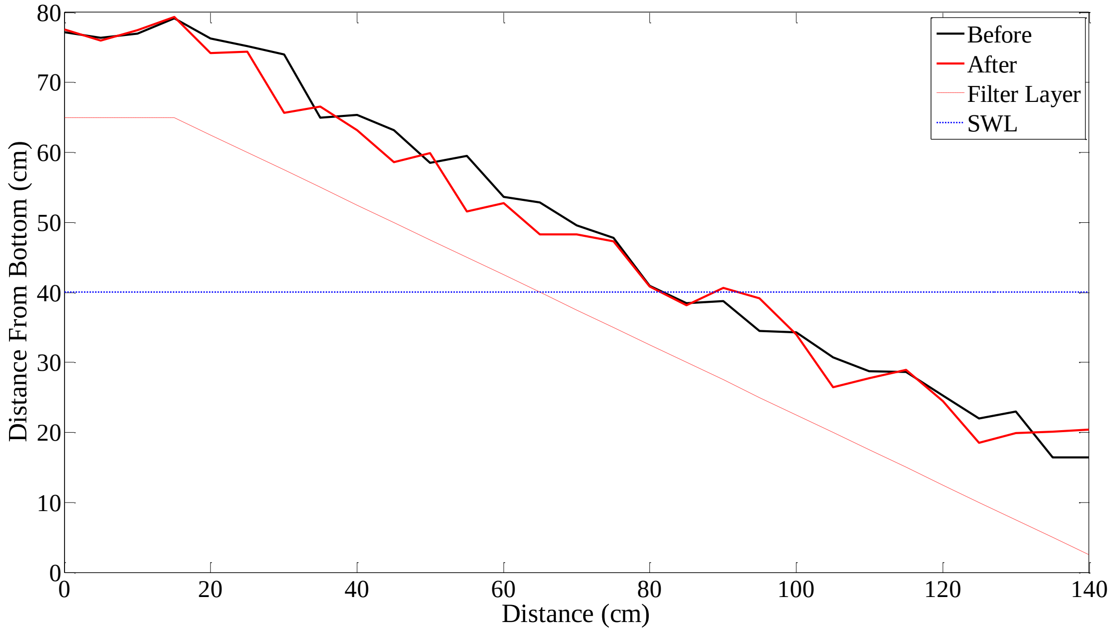


Figure 4.13: Profile Measurements for Case 1 – Set 2

<u>Case 1 – Set 3:</u>

Breakwater cross-section is reconstructed after profile measurements done for Case 1 – Set 2. In Figure 4.14, cross-section before final set is given.

Figure 4.14: Cross-Section before Case 1 – Set 3

After reconstruction, final wave set for Case 1 is applied to cross-section. Resulting cross-section is given in Figure 4.15. Some of the moved stones are shown on Figure 4.15.

Figure 4.15: Cross-Section after Case 1 – Set 3

Profile measurements provided in Figure 4.16 are used to calculate Van der Meer damage parameter (S). S is calculated as 4.9991 for Case 1 – Set 3.

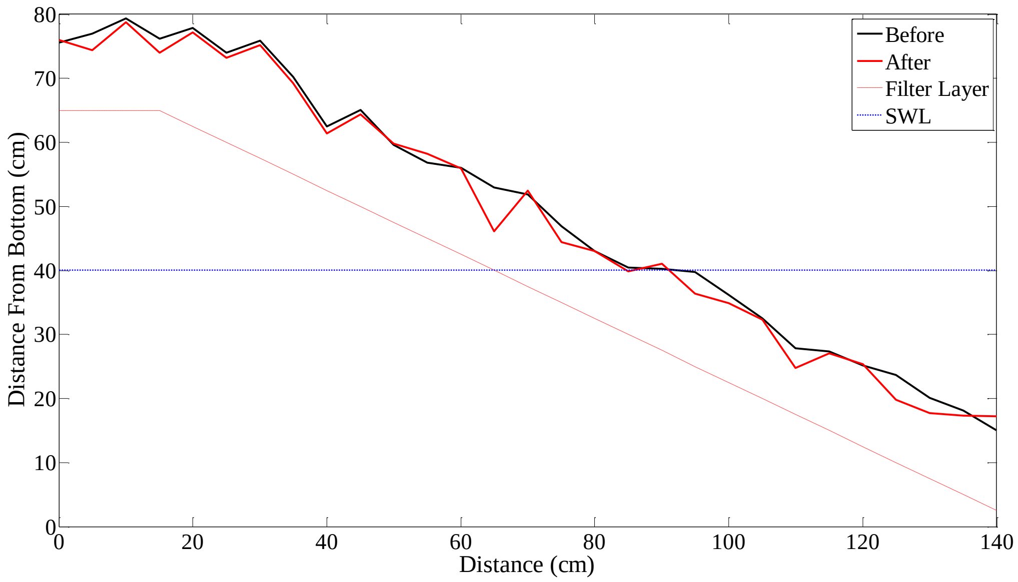


Figure 4.16: Profile Measurements for Case 1 – Set 3

4.5.2. Case 2

Case 2 is the case where the armour stone weight is found using Van Gent et al (2003) formula when three design constraints are satisfied. According to results obtained in Chapter 3, it is expected no damage level or starting damage level for Case 2. Similar to Case 1, three wave sets given in Table 4.1 are applied to the cross-section to test the results obtained in Chapter 3. All three sets result in "no damage criteria" according to Table 2.3. Details of the sets are given throughout Section 4.5.2.

<u>Case 2 – Set 1:</u>

Breakwater cross-section is constructed totally for Case 2. In Figure 4.17, a view before Case 2 is given.

Figure 4.17: Cross-Section before Case 2 – Set 1

Time series obtained for Case 2 is applied to breakwater cross-section. In Figure 4.18, cross-section after Case 2 – Set 1 is given and some of the moved stones are indicated on this figure.

Figure 4.18: Cross-Section after Case 2 – Set 1

To understand the damage level, Van der Meer damage parameter (S) is calculated using profile measurements given in Figure 4.18. It is found as 1.3367.

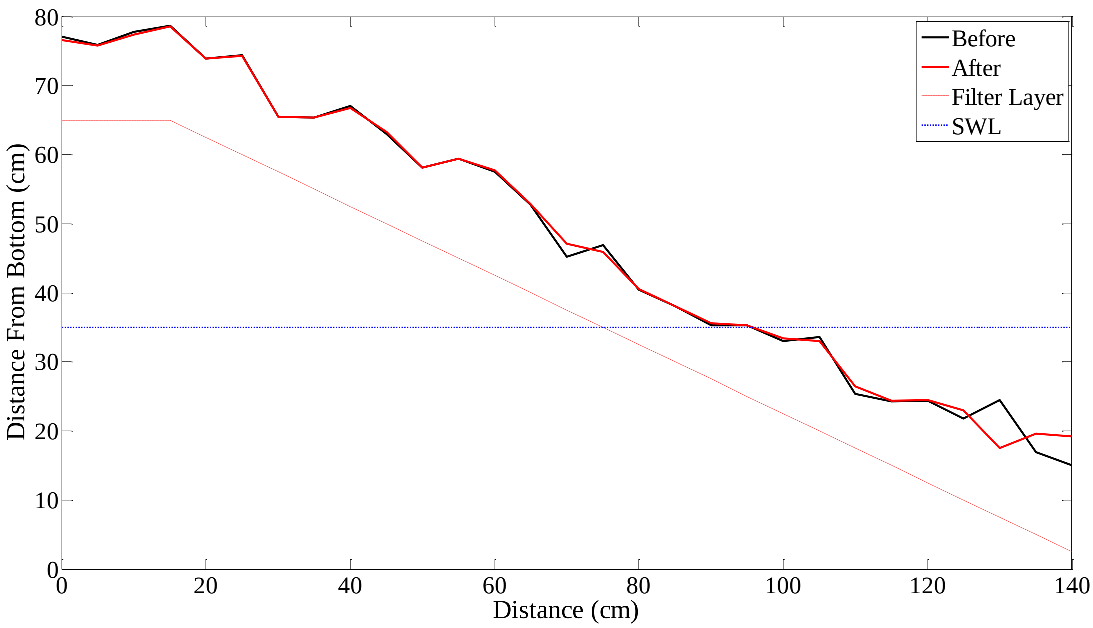


Figure 4.19: Profile Measurements for Case 2 – Set 1

Case 2 – Set 2:

After Case 2 – Set 1, the cross-section is reconstructed again for Case 2 – Set 2. In Figure 4.20, cross-section before Case 2 – Set 2 is given.

Figure 4.20: Cross-Section before Case 2 – Set 2

After reconstruction of cross-section, waves prepared for Case 2 – Set 2 are applied to the cross-section. In Figure 4.21, cross-section after Case 2 – Set 2 is given and some of the moved stones are indicated on the figure.

Figure 4.21: Cross-Section after Case 2 – Set 2

Profile measurements before and after Case 2 – Set 2 are used to calculate Van der Meer damage parameter (S) and found as 2.2150. In Figure 4.22, profile measurements are provided.

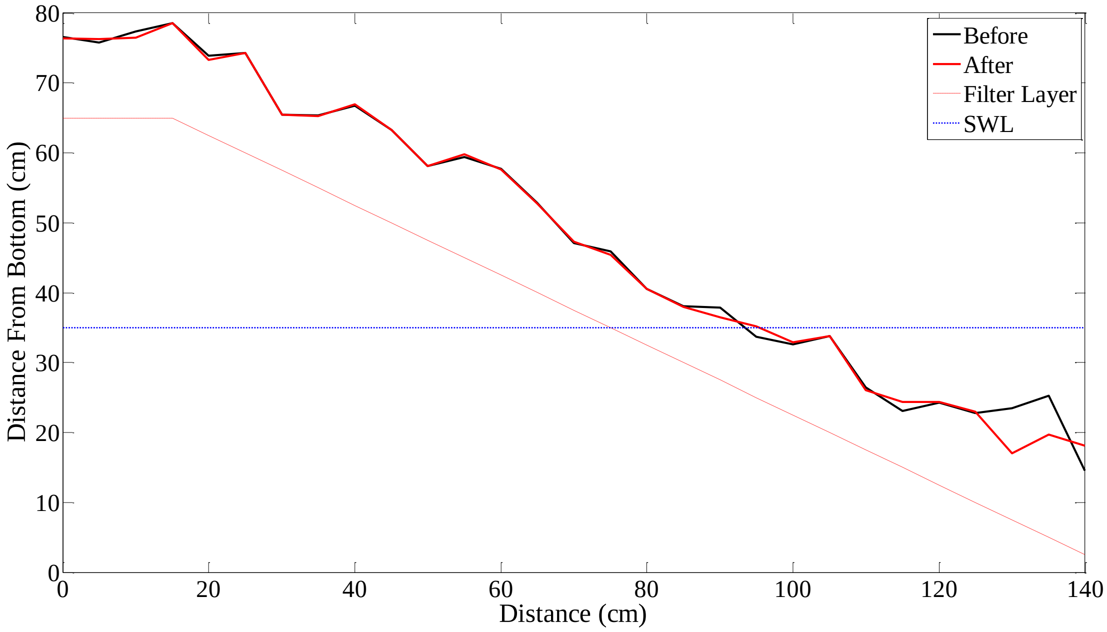


Figure 4.22: Profile Measurements for Case 2 – Set 2

<u>Case 2 – Set 3:</u>

Before final set of Case 2, the cross-section is reconstructed. In Figure 4.23, cross-section before the final set for Case 2 is given.

Figure 4.23: Cross-Section before Case 2 – Set 3

During Case 2 – Set 3, some of the stones are moved. Cross-section after the experiment and moved stones are indicated in Figure 4.24.

Figure 4.24: Cross-Section after Case 2 – Set 3

Using profile measurements, Van der Meer damage parameter (S) is calculated as 1.8477 for Case 2 – Set 3. Profile measurements are presented in Figure 4.25.

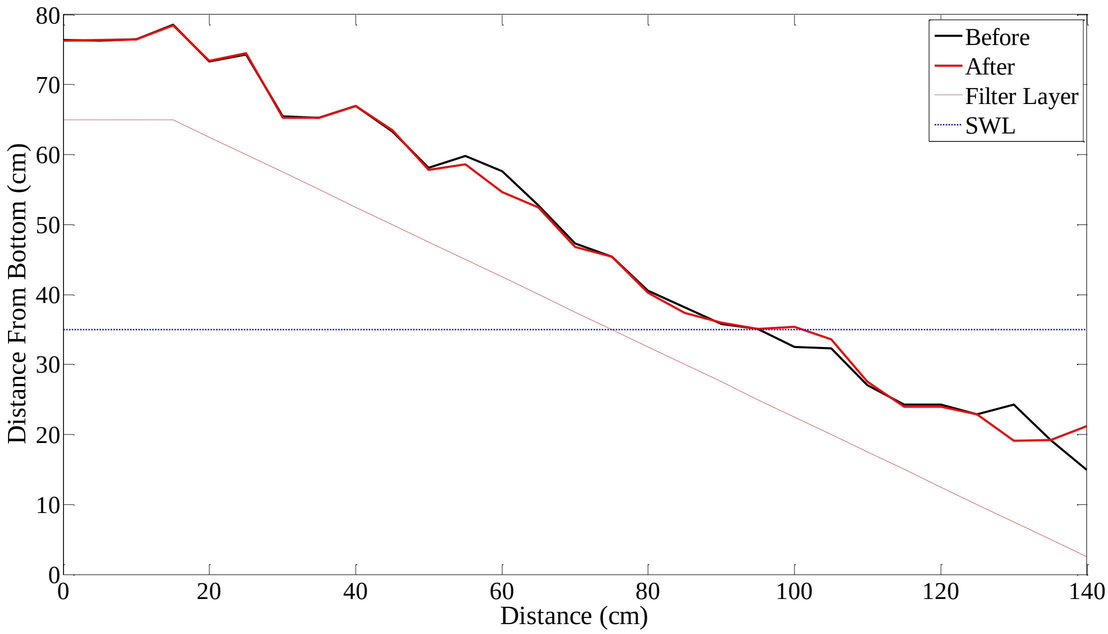


Figure 4.25: Profile Measurements for Case 2 – Set 3

4.5.3. Summary of the Experiments

Two different cases "Case 1" and "Case 2" are modelled by changing deep water significant wave height, significant wave period and water depth at the toe of the structure. These cases are selected to test the design flowchart presented in Chapter 3. Series of tests are carried out as Case 1 to test the validity of Van der Meer (1988) design approach to find armour stone size of rubble mound breakwaters when all the design constraints are satisfied. On the other hand, series of tests are carried out as Case 2 to find out the damage level in the cross-section when the armour weight of the cross-section is obtained using Van Gent et al (2003) formulas when the given design constraints are satisfied. Both cases are repeated for three times. Before and after each experiment, profile of the cross-section is measured to calculate Van der Meer damage parameter (S). Summary of the experiments including input and measured design parameters are given in Table 4.5.

Table 4.5: Summary of the Physical Model Experiments

			Case 1			**Case 2**		
	Parameters		**Set 1**	**Set 2**	**Set 3**	**Set 1**	**Set 2**	**Set 3**
INPUT	Deep Water Significant Wave Height (m)	H_{s0}	5.20	5.20	5.20	4.50	4.50	4.50
	Significant Wave Period (sec)	T_s	8.10	8.10	8.10	7.60	7.60	7.60
	Water Depth at the toe of the structure (m)	h	8.00	8.00	8.00	7.00	7.00	7.00
	Significant Wave Height at the toe of the structure (m)	$H_{s,toe}$	4.60	4.60	4.60	4.03	4.03	4.03

Table 4.5 (continued)

MEASURED	Significant Wave Height at the toe of the structure (m)	$H_{s,toe}$	4.59	4.62	4.64	3.99	4.02	4.04
	Significant Wave Period (sec)	T_s	8.07	8.11	8.12	7.57	7.64	7.68
	Constraint 1	$h/H_{s,toe}$	1.74	1.73	1.72	1.75	1.72	1.72
	Constraint 2	$H_{2\%}/H_{s,toe}$	1.31	1.30	1.32	1.31	1.31	1.32
	Constraint 3	$H_{s,toe}/H_{s0}$	0.88	0.89	0.89	0.89	0.89	0.89
	Number of Waves	N	1061	1082	1048	1012	1059	1067
	Van der Meer Damage Parameter	S	4.79	6.25	4.99	1.34	2.22	1.85
	Damage Level		Intermediate Damage			No Damage		

4.6. Interpretation of Experiments in terms of New Design Flowchart

Physical model experiments conducted in scope of this thesis study show that it is appropriate to use Van Gent et al (2003) equations when the design constraints given in Chapter 3 are satisfied. Van der Meer damage parameters (S) calculated for each set of the experiments are the indicators of this result.

Since Van der Meer damage parameter (S) is evaluated as "Intermediate Damage" when it is between 4-6 (Rock Manual, 2007), it is concluded that there is considerable damage in the cross-section for Case 1. Thus, it can be said that when all the constraints are satisfied, use of Van der Meer (1988) equations is found questionable which is also stated in Van der Meer (1988) also expresses that his equations are not applicable in "very shallow water" regarding the design constraints given as dimensionless ratios.

On the other hand, Case 2 indicates that when the design constraints are satisfied using Van Gent et al (2003) equations in finding armour stone weights is appropriate since Van der Meer damage parameter (S) is calculated as 2.22 at most. When Van der Meer damage parameter (S) is around 2, it is regarded as no damage (Rock Manual, 2007).

These results are the outcomes of two sets of experiments as Case 1 and Case 2. Therefore, the studies should be extended to observe the behavior of the structure under different design conditions such as different steepness ranges, different structure face slopes, different storm durations, different notional permeability construction styles, etc. However, based on physical model experiments, it can be concluded that, as a more conservative method, using Van Gent et al (2003) formulas when design constraints are satisfied is more appropriate since armour layer designed using Van der Meer (1988) equations suffered from “intermediate damage level” whereas armour layer designed by Van Gent et al (2003) equations is not damaged.

CHAPTER 5

EFFECT OF DESIGN WATER LEVEL ON STABILITY OF RUBBLE MOUND STRUCTURES

Design water level is one of the most important parameters for designing coastal structures since it is directly affects parameters such as wave height, wave period, wave approach angle, breaking wave properties, wave overtopping and wave transmission. The list of phenomenon can be extended considering design, construction and maintenance periods of a coastal structure in its economic life.

Design water level is determined considering various components of fluctuations in sea water level. These components are mainly sea level rise (SLR) due to global warming, seasonal variations in the water level, tides, wave and wind set-up (set-down) usually referred as storm components and the sea level rise originated from barometric and Coriolis effects.

In this chapter, components of mean sea water level change are defined, computational tool used to investigate the effects of change in mean water level on stability of rubble mound structures is clarified and the results obtained with the use of computational tool are discussed. Finally, a practical methodology to determine design water level is proposed and uncertainties involved in this methodology are discussed.

5.1. Components of Mean Water Level Change

In this part, components of change in mean water level are defined and investigated in the scope of this study including extreme marine events such as tsunamis and storm surges.

5.1.1. Global Warming

Sea level rise (SLR) due to global warming is one of the major components that should be taken into account when designing coastal structures. In general, coastal structures are designed with a long period of economic life. In that economic life time, increase in the mean water level due to global warming is the common scientific result which may affect design parameters of coastal structures. IPCC (2007) states that "By the end of the 21^{st} century, it is very likely that over about 95% of the world ocean, regional sea level rise will be positive, while most regions experiencing a sea level fall are located near current and former glaciers and ice sheets.".

IPCC (2007) gives the average global water level change due to global warming in a range of 18 cm to 59 cm for 2100. On the other hand, it should be stated that local water level change may be different from the global average. This situation is explained by IPCC (2007) as "Shifting surface winds, the expansion of warming ocean water, and the addition of melting ice can alter ocean currents which, in turn, lead to changes in sea level that vary from place to place". As a result, it is a better approach to use regional studies considering sea level rise if possible in addition to global average of increase in water level to determine design water level.

Along Turkish coasts, there are some studies to define regional water level change due to global warming. A summary of possible changes in mean water level due to global warming are given in Table 5.1 for coasts of Turkey:

Table 5.1: Summary of Possible Changes in Mean Water Level due to Global Warming

Region	Change in Mean Water Level	Reference
Black Sea	2.5-2.8 mm/year	Belokopytov and Goryachkin (1999)
Black Sea	3.5-4.5 mm/year	Shuisky (2000)
Black Sea	0.7-1.1 mm/year	Tsimplis et al. (2004)
Marmara Sea	8-9.6 mm/year	Yildiz and Demir (2002)
Aegean Sea	0.6-4.3 mm/year	Yildiz and Demir (2002)
Mediterranean	3.6 mm/year	Alpar et al. (1995)
Mediterranean	2.6-4.3 mm/year	Yildiz and Demir (2002)

5.1.2. Seasonal Variations

Mean water level changes from season to season due to the changes in water balance among oceans, river run off and floods, seasonal water density changes related to temperature and salinity. A few studies are done around Turkish coasts about seasonal mean water level variations that are summarized in Table 5.2:

Table 5.2: Summary of Possible Changes in Mean Water Level due to Seasonal Variations

Region	Change in Mean Water Level	Reference
Black Sea	19 cm	Alpar et al. (2000)
Marmara Sea	18 cm	Alpar and Yuce (1998)
Mediterranean	17 cm	Alpar et al. (2000)
Aegean	8 cm	Alpar et al. (2000)

5.1.3. Tides

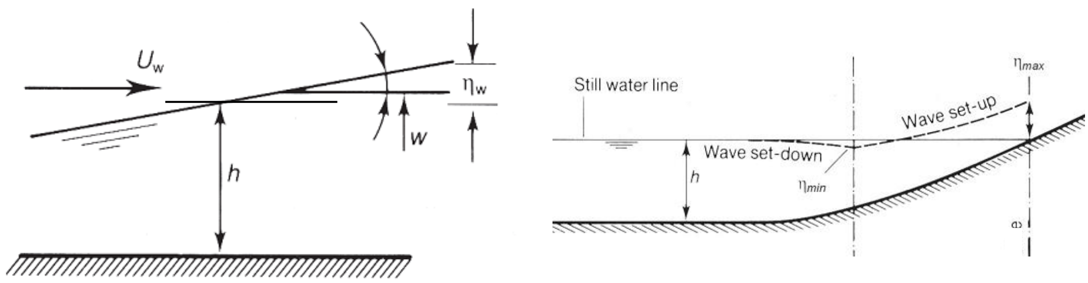
Tides, the rise and fall of sea levels originated from the combined effect of the motions of the Moon, Sun and Earth, are one of the most understood and studied phenomenon in the study area of coastal and ocean engineering. A mathematical theory of the tides including the tide-generating forces is given by Laplace in the end of 18th century. Non-uniformity of forces acting over the surface of the Earth is the reason why tides occur (Reeve et al, 2004).

Tides do not have very significant effect, along the Turkish coasts since the amplitudes are small compared to mean water level. Turkish coasts are said to be micro tidal coasts and micro tide amplitude is less than 50 cm. According to Alpar et al (2000), tide amplitude is in 10-40 cm range along Turkish Coasts.

5.1.4. Storm Components: Wave and Wind Set-Up (Set-Down)

Wave and wind set-up (set-down) are mean water level changes caused by storm conditions. Goda (2000) defines wave set-up as a quasi-linear rise in the mean water level toward to shoreline due to presence of wave motion. Wind set-up, on the other hand, is the shear stress exerted by wind on the water surface causes a slope in the water surface (Rock Manual, 2007). Wave set-down and wind set-down occurs at down wave and downwind boundaries, respectively.

In Figure 5.1, schematic representations of wind and wave set-up (set-down).



(a) Wind Set-Up and Set-Down

(b) Wave Set-Up and Set-Down

Figure 5.1: Schematic Representations of Wind Set-Up and Wave Set-Up (Set-Down) *(adopted from Rock Manual, 2007)*

In Figure 5.1a, U_w is the wind speed at an elevation of 10 m above mean water level (m/s), h is water depth (m) and η_w is wind set-up (m). Wind set-down can also be seen from Figure 5.1a at downwind boundary. In Figure 5.1b, components of wave set-up and set-down are shown where η_{max} is the maximum wave set-up and η_{min} is the wave set-down.

The equations used to express and calculate wave and wind set-up (set-down) are given and solved in Section 5.2.1 for one dimensional case.

5.1.5. Barometric and Coriolis Effects

Changes in the atmospheric pressure may affect mean water level. Rises in the mean water level may be caused by local low atmospheric pressures whereas high pressures cause drops in water levels (Rock Manual, 2007). Since effective storm events, thus wave action occurs at low pressure levels, rise in the mean water level due to changes in the atmospheric pressure should be taken into consideration when determining design water level.

Coriolis Effect can be defined as a deflection of moving objects in a rotating reference frame. Mean water level varies due to Coriolis Effect because of the Earth’s motion.

5.1.6. Other Parameters

Extreme, catastrophically hazardous and rare marine events such as tsunamis and storm surge should be taken into account due to increasing frequency of these types of events. Tsunami is a series of waves caused by displacement of large water bodies due to earthquakes, submarine landslides, asteroids and etc. The other extreme event storm surge is the increase in mean water level due to the high wind speeds and low pressure system which can be regarded as a long wave.

These events are usually studied different from wind waves. The resulting effects of these events on coastal structures, especially on rubble mound breakwaters, are newly studied topics in coastal and ocean engineering profession. It should be noted that combined effect of tides, wave and wind set-up and storm surges, on the other hand, tsunami propagation and tsunami heights at coastal zone can be modeled by numerically solving, in general, Navier-Stokes equations. It is left as a future study to investigate tsunamis and storm surges within the concept of changes in mean water level. Since the scope of this thesis is limited with wind waves, these events are not taken into consideration.

5.1.7. Deterministic Approach to Determine Design Water Level

After estimating deep water wind and wave characteristics of the region where you plan to build the coastal structure, as a general design practice in coastal and ocean engineering, the wave parameters at the toe of the coastal structure are tried to be determined by wave transformation studies. Wave transformation studies needs detailed bathymetry information, especially around the coastal structure. The bathymetry information can be obtained from various databases (usually with low resolution) and/or from measurements (usually with higher resolution). Nevertheless, the obtained bathymetry contains elevation values that should be updated by the possible changes in mean water level. The possible mean water level changes stated in this chapter are usually summed and added to current elevation values. The resulting elevation values are regarded as design water level. The idea behind this is that critical conditions occur at higher water levels in general. This idea is discussed in terms of stability of rubble mound structures in Sections 5.3 and 5.4.

Mean water level changes due to global warming, seasonal variations, tides, barometric and Coriolis effects can be accepted as constant within a range for a given return period. On the other hand, wave and wind set-up can be regarded as case specific and changes in the surf zone. Therefore, to determine a range for mean water level changes, the constant parameters should be summed and wave and wind set-up can be calculated for the given site specific case. Barometric and Coriolis effects can be calculated by numerically solving Navier-Stokes equations arranged with appropriate terms; but, in the scope of this thesis and most of design works barometric and Coriolis effects are taken as 10% of the summation of other parameters (Walton and Dean, 2009) since it is usually difficult to obtain atmospheric pressure changes and to reduce computational load. It should be noted that, if certain values obtained as barometric and Coriolis effects are available, they should be used in calculations. This assumption does not violate the scope of this thesis study, and in general, by this assumption a designer would be on the safe side.

In Figure 5.2, a schematic representation of the deterministic approach to determine design water level is given.

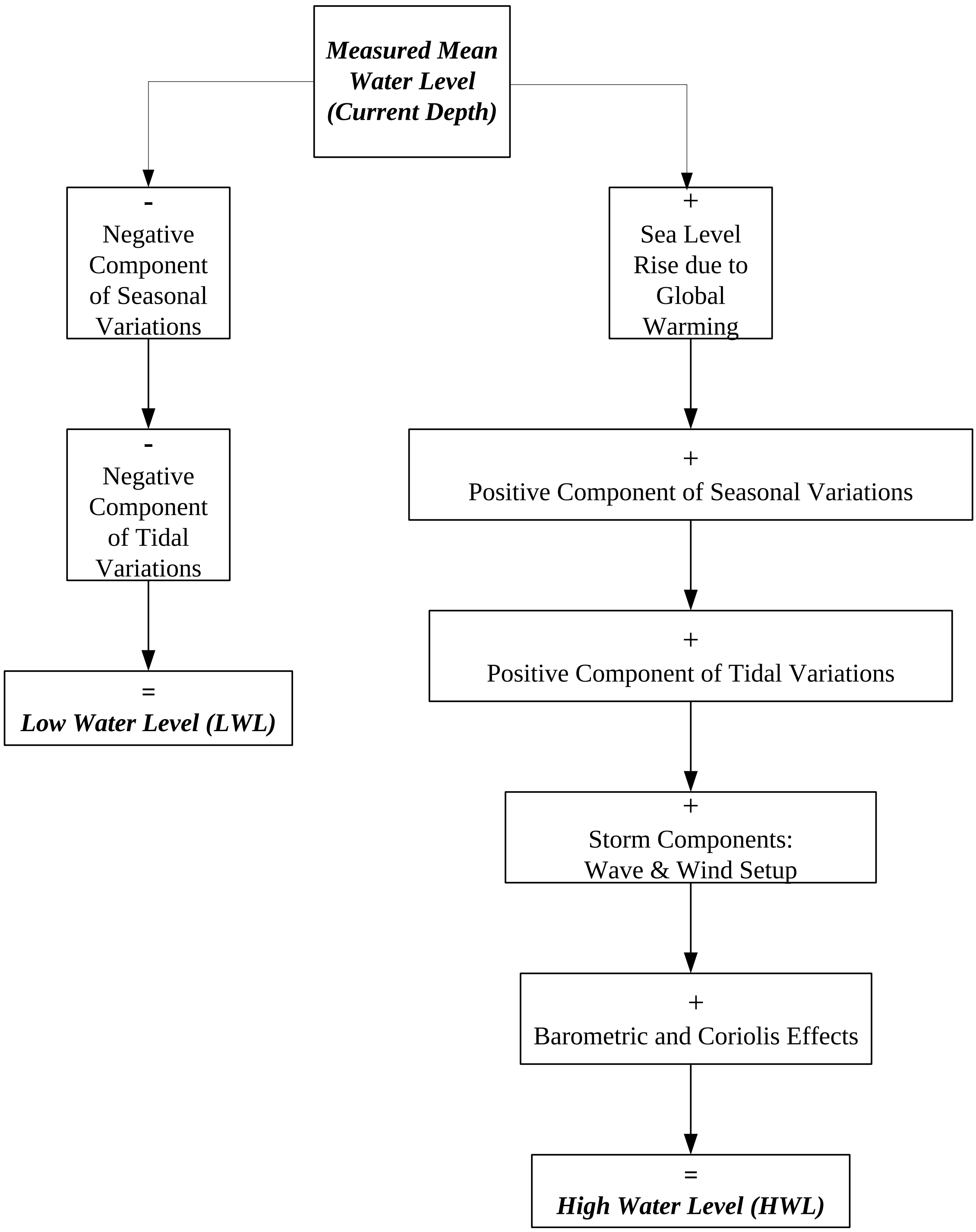


Figure 5.2: Description of LWL and HWL

5.2. Computational Tool

A computational tool is developed to understand the effect of design water level on stability of rubble mound structures. In this part, the components of the model are explained, equations used are defined and approaches used to determine design water level are clarified.

5.2.1. Overview of Computational Tool

The computational tool developed in MATLAB environment consists of three parts:

- 1D **N**ear **S**hore **W**ave Transformation Model (NSW, hereafter) (Baykal, 2012)
- **D**esign of **A**rmour **S**tone (DAS, hereafter)
- **D**esign **W**ater **L**evel **D**etermination (DWLD, hereafter)

The theoretical and numerical background in addition to benchmarking of NSW is given in Section 5.2.2.

DAS is a computational tool that finds armour stone sizes according to Hudson (1959), Van der Meer (1988) and Van Gent et al (2003) explained in Chapter 3.

DWLD is the final part of the computational tool that uses both NSW (to calculate near shore wave characteristics) and DAS (to find armour stone size). Theory of NSW is given in Section 5.2.2 and details of DWLD are given in Section 5.2.3.

The output of this computational tool may help a designer to determine the critical wave parameters and critical armour stone sizes throughout the economic life of the coastal structure.

5.2.2. One Dimensional Near Shore Wave Transformation Model (NSW)

The main structure of this model is given by Baykal (2012). Also a 2D version of NSW solving the same equations in two dimensions in addition to directional wave spectra formulation is explained in detail by Baykal (2012). In the scope of this thesis, 1D NSW developed by Baykal (2012) is used and upgraded by adding wind set-up relations.

In this part assumptions and limitations, theoretical and numerical background of NSW is given.

5.2.2.1. Assumptions and Limitations of NSW

- A phase-averaged approach is used to compute near shore wave parameters. Computed parameters are assumed to be constant during a single storm (Baykal, 2012).
- Linear wave shoaling, linear wave refraction and depth-induced random wave breaking are used for wave transformation over the arbitrary bathymetry (Baykal, 2012).
- Effects of tidal, atmospheric pressure and Coriolis terms are not taken into account (Baykal, 2012).
- Wave reflection from shore due to sharp bottom gradients, energy dissipation due to bottom friction, white-capping, bottom vegetation are neglected in NSW (Baykal, 2012).
- To calculate mean water level variations depending on specific storm conditions, i.e. wave and wind set-up (set-down), non-linear shallow water equations are solved iteratively to reach a steady state solution (Baykal, 2012). Since the main contribution to the solution is from radiation stress and wind stress in non-linear shallow water equations, bulk advection term, bottom shear stresses and lateral mixing terms are neglected.
- Straight and parallel bottom contours (1D case) are assumed. The effects of other dimensions are neglected.

5.2.2.2. Energy Balance Equation

Energy balance equation was introduced by Karlsson (1969) in order to compute wave shoaling and refraction of random waves. NSW is developed to solve energy balance equation numerically in order to estimate wave properties in the near shore region (Baykal, 2012). By adding terms for breaking and diffraction, Mase (2001) modified energy balance equation where 1D version of this equation is given by

Equation 5.1. Note that, diffraction term is not taken into consideration in Equation 5.1 since the model solves one dimensional case.

$$\frac{\partial(E\,C_g\cos\theta)}{\partial x} = D_b\,g\,\rho \qquad [5.1]$$

In Equation 5.1, E is the total wave energy, C_g is defined as propagation velocity or group velocity (m/s), θ is the wave approach angle taken counterclockwise with respect to x-axis (rad), D_b is the dissipation due to the random wave breaking, g is gravitational acceleration (9.81 m/s^2) and ρ is density of water (kg/m^3).

Total wave energy is defined as

$$E = \frac{\rho g H_{rms}^2}{8} \qquad [5.2]$$

where H_{rms} is the root-mean-square wave height.

Group velocity term, C_g, can be calculated using the following equations

$$C_g = \frac{gT}{4\pi}\tanh(kh)\,(1+\frac{2kh}{\sinh(kh)}) \qquad [5.3]$$

$$k = 2\pi / L \qquad [5.4]$$

$$L = \frac{g}{2\pi}T^2\tanh(kh) \qquad [5.5]$$

where k is defined as wave number (rad/m) and L is defined as wave length (m). Equation 5.5, widely called as dispersion relation, can be solved iteratively for a given period T (sec) at a water depth h (m).

Dissipation rate due to random wave breaking (D_b) is given by Janssen and Battjes (2007) based on the methodology proposed by Baldock et al. (1998). Baykal (2012) clearly explains that, the method proposed by Baldock et al. (1998) assumes distribution of random waves in the surf zone is assumed to be a full Rayleigh distribution with a weighting function which assumes the waves greater than a maximum depth limited wave height are broken. D_b is calculated by the relation given in Equation 5.6.

$$D_b = \frac{1}{B}\frac{3\sqrt{\pi}}{16} f_p \frac{H_{rms}^3}{h}\left\{1+\frac{4}{3\sqrt{\pi}}\left[\left(\frac{H_b}{H_{rms}}\right)^3+\frac{3}{2}\left(\frac{H_b}{H_{rms}}\right)\right]\exp\left[-\left(\frac{H_b}{H_{rms}}\right)^2\right]-erf\left(\frac{H_b}{H_{rms}}\right)\right\}$$

[5.6]

where B is taken unity and f_p is the peak frequency, *erf* is the error function and H_b is the maximum depth-limited wave height. H_b can be computed as

$$H_b = \gamma_b h \quad [5.7]$$

where γ_b is the breaker index might be taken as 0.78.

The peak frequency, f_p, is related to significant wave period using JONSWAP P-Type spectrum with $\gamma = 3.3$ (Goda, 2000) as

$$f_p \approx \frac{1}{1.07\,T_s} \quad [5.8]$$

Backward finite difference scheme is used to solve energy balance equations numerically by discretizing arbitrary bathymetry data in x direction (Baykal, 2012). The solution of the energy balance equation is given as

$$\frac{E_i\,C_{g_i}\cos\theta_i - E_{i-1}\,C_{g_{i-1}}\cos\theta_{i-1}}{dx} = D_{b_i}\,g\,\rho \quad [5.9]$$

$$E_i = \frac{D_{b_i}\,g\,\rho\,dx + E_{i-1}\,C_{g_{i-1}}\cos\theta_{i-1}}{C_{g_i}\cos\theta_i} \quad [5.10]$$

where dx is the grid spacing in x axis.

5.2.2.3. Numerical Modeling of Mean Water Level Fluctuations Depending on a Specific Storm: Wave and Wind Set-Up (Set-Down)

Mean water level fluctuations can be computed by solving non-linear shallow water equations. The non-linear shallow water equations are given for one dimensional case as

$$\frac{\partial\bar{\eta}}{\partial t} + \frac{\partial}{\partial x}\left[u\,(h+\bar{\eta})\right]=0 \quad [5.11]$$

$$\frac{\partial u}{\partial t}+u\frac{\partial u}{\partial x}+g\frac{\partial \overline{\eta}}{\partial x}+\frac{1}{\rho(h+\overline{\eta})}\left[F_x-\tau_{wx}\right]=0 \qquad [5.12a]$$

where $\overline{\eta}$ is the mean water level fluctuation (m), u is the depth averaged current velocity in x-direction, h is water depth from mean water level (m), F_x is the sum of radiation stresses acting on the water body due to surface rollers and τ_{wx} is the wind shear stress acting on the water surface. t is the time; however, as the equations are solved for a steady state, the time derivatives are assumed to be equal to zero. Equation 5.11 is called continuity equation in x-direction whereas Equation 5.12a is defined as conservation of momentum in x-direction. Terms related to tides and Coriolis forces are disregarded to simplify the equations. Within the scope of this study, one dimensional non-linear shallow water equations are linearized disregarding the advection terms $u(\partial u/\partial x)$. Therefore, $\overline{\eta}$ is solved only by considering the momentum equation given by 5.12b.

$$g\frac{\partial \overline{\eta}}{\partial x}+\frac{1}{\rho(h+\overline{\eta})}\left[F_x-\tau_{wx}\right]=0 \qquad [5.12b]$$

The sum of radiation stresses due to surface rollers, F_x, are given by Goda (2010) as

$$F_x=\frac{\partial S_{xx}}{\partial x}+\frac{\partial(2E_{sr}\cos^2\theta)}{\partial x} \qquad [5.13]$$

where S_{xx} is the radiation stress acting in x-direction (Equation 5.14), E_{sr} is the parameter given for the kinetic energy of the surface roller and θ is the mean wave approach angle calculated with respect to x-axis. The radiation stress, S_{xx}, given by Longuet-Higgins and Stewart (1964) can be calculated as

$$S_{xx}=E\left[n\left(\cos^2\theta+1\right)-0.5\right] \qquad [5.14]$$

where E is the total energy and n is the ratio of group velocity to wave celerity given by Ergin (2009) as

$$n=0.5\left(1+\frac{2kh}{\sinh(2kh)}\right) \qquad [5.15]$$

Baykal (2012) defines surface rollers as the white foams with a thickness of δ observed in front of breaking waves. Tajima and Madsen (2003) defines kinetic energy of surface roller, E_{sr}, as

$$E_{sr} = \frac{\rho A_{sr} C}{2T} \qquad [5.16]$$

In Equation 5.16, A_{sr} is the area of surface roller, C is the wave celerity, T is the wave period. In this one dimensional near shore wave transformation model, wave period (T) is taken as peak wave period (T_p) (Baykal, 2012).

Kinetic energy of surface roller, E_{sr}, can be computed by solving the evolution of E_{sr} over an arbitrary bathymetry. One dimensional evolution equation can be written as

$$\alpha\rho g \, \frac{\partial\left(m_0 \, C_g \, \cos(\theta)\right)}{\partial x} + \frac{\partial\left(E_{sr} \, C \, \cos(\theta)\right)}{\partial x} = -\frac{K_{sr} \, E_{sr} \, C}{h} \qquad [5.17]$$

where α is the energy transfer coefficient ($0 \le \alpha \le 1$) related to the energy transferred to surface roller, m_0 is given as total energy density, K_{sr} is the rate of dissipation of surface roller energy. K_{sr} is given by Tajima and Madsen (2003) as

$$K_{sr} = \frac{3}{8}(0.3 + 2.5m) \qquad [5.18]$$

where m is the bottom slope calculated at each grid interval.

The wind shear stress acting on the water surface, τ_w, can be computed using Equation 5.19 for a bottom profile with straight and parallel bottom contours which is one dimensional case (Rock Manual, 2007).

$$\tau_w = \rho_{air} \, C_D \, U_{10}^2 \qquad [5.19]$$

In Equation 5.19, ρ_{air} is the density of air, C_D is drag coefficient and U_{10} is the wind velocity measured at an elevation of 10m above the mean water level in x direction. Drag coefficient, C_D, is given by Weaver and Slinn (2004) as

$$C_D = 0.001\,(0.75 + 0.067\,U_{10}) \qquad [5.20]$$

To solve non-linear shallow water equations, the equation given for evolution of kinetic energy of surface rollers (Eqn. 5.17) should be solved previously. Solution of

Equation 5.17 is given by Baykal (2012) using backward finite difference formulation as

$$\alpha\rho g\,\frac{\left(m_{0_i}\;C_{g_i}\;\cos(\theta_i)-m_{0_{i-1}}\;C_{g_{i-1}}\;\cos(\theta_{i-1})\right)}{dx}+\frac{\left(E_{sr_i}\;C_i\;\cos(\theta_i)-E_{sr_{i-1}}\;C_{i-1}\;\cos(\theta_{i-1})\right)}{dx}=$$
$$-\frac{K_{sr_i}\;E_{sr_i}\;C_i}{h_i}$$

$$\frac{\left(E_{sr_i}\;C_i\;\cos(\theta_i)\right)}{dx}+\frac{K_{sr_i}\;E_{sr_i}\;C_i}{h_i}=$$
$$\left[-\alpha\rho g\,\frac{\left(m_{0_i}\;C_{g_i}\;\cos(\theta_i)-m_{0_{i-1}}\;C_{g_{i-1}}\;\cos(\theta_{i-1})\right)}{dx}+\frac{E_{sr_{i-1}}\;C_{i-1}\;\cos(\theta_{i-1})}{dx}\right] \qquad [5.21]$$

$$E_{sr_i}=\frac{\left[-\alpha\rho g\,\frac{\left(m_{0_i}\;C_{g_i}\;\cos(\theta_i)-m_{0_{i-1}}\;C_{g_{i-1}}\;\cos(\theta_{i-1})\right)}{dx}+\frac{E_{sr_{i-1}}\;C_{i-1}\;\cos(\theta_{i-1})}{dx}\right]}{\left[\frac{C_i\;\cos(\theta_i)}{dx}+\frac{K_{sr_i}\;C_i}{h_i}\right]}$$

It is now possible to solve related non-linear shallow water equation, i.e. conservation of momentum (Equation 5.12). The main contribution to this equation is from radiation and wind stresses. Therefore, this equation is solved neglecting bulk advection term. Note again that, the equation is solved iteratively to achieve a steady state solution. Equation 5.12 is simplified and solved using forward finite difference formulation as

$$g\,\frac{\partial\bar{\eta}}{\partial x}+\frac{1}{\rho(h+\bar{\eta})}\left[\frac{\partial S_{xx}}{\partial x}+\frac{\partial(2\,E_{sr}\cos^2\theta)}{\partial x}-\tau_w\right] \qquad [5.22]$$

$$g\,\frac{\eta_{i+1}-\eta_i}{dx}+\frac{1}{\rho(h+\eta)}\left(\frac{Sxx_{i+1}-Sxx_i}{dx}+2\,\frac{E_{sr_{i+1}}\cos^2\theta_{i+1}-E_{sr_i}\cos^2\theta_i}{dx}-\tau_{w_i}\right)=0$$

[5.23]

$$\eta_{i+1}=\eta_i-\frac{(Sxx_{i+1}-Sxx_i+2\,E_{sr_{i+1}}\cos^2\theta_{i+1}-2\,E_{sr_i}\cos^2\theta_i-dx\;\tau_{w_i})}{\rho g\,(h_i+\eta_i)}$$

Effect from wind stress term is taken into account in this thesis study.

5.2.3. Design Water Level Determination (DWLD) Code

In this thesis, a code is developed in order to find the most critical water level in a region of interest. The main idea of this code is finding the most critical water level by increasing water level with small increments defined in a range. This code takes deep water wave characteristics, construction depth and range of change in mean water level as input parameters.

DWLD consists of two parts considering the different rubble mound breakwater design approaches. To find armour stone size, if Hudson equation is taken as the design approach, "DWLD-I" is used. On the other hand, if Van der Meer and Van Gent et al equations are considered to design rubble mound breakwater, second part of DWLD, "DWLD-II", is used. Inputs and algorithms of these codes are given below:

Inputs and Algorithm of DWLD-I:

INPUTS	1	Main input parameters: Deep water wave characteristics (wave height, H_0, wave period, T,) construction depth (h), 1D bathymetry and range of change in mean water level (ΔH).
	2	Other input parameters: Structure face slope, unit weight of stone and water, wind velocity, Hudson's stability coefficients for breaking and non-breaking conditions, etc.
	3	Define increment of design water level (Δh).
ALGORITHM	4	Find breaking depth, breaking wave height and breaker travel distance using CERC (1977) method.
	5	Determine wave setup and wind setup using NSW.
	6	Determine breaking condition for h+ Δh.
	7	According to breaking condition, find armour stone weight using DAS. If it is non-breaking case, transform wave using NSW. If it is broken case, broken wave height is found using CERC (1977).
	8	Increase h= h+ Δh. If h< h+ ΔH, go to step 5.
	9	Draw curves describing computations.

Inputs and Algorithm of DWLD-II:

INPUTS	1	Main input parameters: Deep water wave characteristics, 1D bathymetry, construction depth (h), 1D bathymetry and range of change in mean water level (ΔH)
	2	Other input parameters: Structure face slope, wind velocity, unit weight of stone and water, breaker index, storm duration (or number of waves), notional permeability, wind velocity, etc.
	3	Define increment of water level (Δh).
ALGORITHM	4	Transform wave to the toe of the structure using NSW.
	5	Find armour stone weight using flowchart described in Chapter 3 using DAS.
	6	If h< h+ ΔH, go to step 5.
	7	Draw curves describing computations

In this thesis study, to investigate the effect of design water level using examples in Sections 5.3 and 5.4, general values describing each type of change in mean water level are assumed as average values and evaluated for 100 years in Table 5.4. In practices, it is strongly recommended to use case specific values for these parameters.

Table 5.3: Range of Mean Water Level Changes along Turkish Coasts

	Black Sea	**Sea of Marmara**	**Aegean Sea**	**Mediterranean Sea**
Global Warming (1)	4.5 mm/yr	9.6 mm/yr	4.3 mm/yr	4.3 mm/yr
Seasonal Variations (2)	-9.5 to +9.5 cm	-9 to +9 cm	-8.5 to +8.5 cm	-4 to +4 cm
Tide (3)	-15 to +15 cm	-15 to +15 cm	-15 to +15 cm	-15 to +15 cm
Barometric and Coriolis Forces (4)	-2.5 to 7.0 cm	-2.4 to 12 cm	-2.4 to 6.7 cm	-1.9 to 6.2 cm
Total (5)	-27 to 76.5 cm	-26.2 to 132 cm	-25.9 to 73.2 cm	-20.9 to 68.2 cm

Table 5.3 (continued)

[(1)] Global warming values are taken as the maximum values for the regions from Table 5.1.
[(2)] Seasonal variations given in Table 5.2 are differences between lowest and highest months. It is assumed that mean water level is in the middle of these values.
[(3)] Tidal amplitude is assumed as 15 cm along Turkish coasts as a general value.
[(4)] Mean water level change due to barometric and Coriolis forces are calculated in a range of lowest level today and highest level 100 years later by summation of 10% of global warming and seasonal variations. Amount coming from wind setup is taken into consideration in NSW code.
[(5)] Total change in mean water level is determined as a range starting from the lowest level today to highest level 100 years later.

5.3. Effect of Design Water Level in Hudson (1959) Approach used for Stability of Rubble Mound Breakwaters

Hudson (1959) approach is one of the most common methodologies used to find armour stone weight of rubble mound breakwaters. Details of Hudson approach is given in Chapter 2. As stated in Chapter 2, design wave height in Hudson formula at deep water is recommended to be taken as deep water significant wave height (H_{s0}) by CERC (1977), on the other hand, it is recommended to be taken as the deep water wave height exceeded by 1/10 of waves ($H_{1/10,0}$) in a certain storm by CERC (1984). To investigate effect of design water level in design of armour layer of rubble mound breakwaters using Hudson approach, DWLD-I is used and throughout investigations conducted in Section 5.3, deep water significant wave height (H_{s0}) is used in Hudson approach.

DWLD-I is a computer code that calculates armour stone weight using Hudson approach for given input parameters. In this code, change in the mean water level is inputted as a range, increased by defined amount and computations for armour stone are performed for each water level separately. Outputs of the computer code are four graphs that give changing depths versus design wave height according to breaking condition, changing depths versus armour stone diameter, changing depths versus armour stone weight and distance from shoreline versus breaking region in terms of water depth. “Changing depths” mean water depth at the toe of the structure that is changed because of variations in water depth.

An example study is carried out for Black Sea coasts of Turkey. All the input parameters that are used for this study are arbitrary. Bathymetry for this example study is constructed assuming foreshore slope. Input parameters rather than foreshore slope are given in Table 5.5.

Table 5.4: Input Parameters for Example Study

Parameters		**Value**
Deep Water Significant Wave Height	H_{s0} (m)	4
Deep Water Significant Wave Steepness	H_{s0}/L_0	0.04
Deep Water Wave Approach Angle	α_0 (°)	0
Foreshore Slope	m	1/20
Wind Velocity	U (m/s)	15
Low Water Level*	LWL (cm)	-27
High Water Level*	HWL (cm)	76.5
Distance of Breakwater Section from Shoreline	(m)	100
Measured Depth at the toe of the Cross-Section	(m)	5
Increment of Change in Mean Water Level	Δh (cm)	1
Slope of Structure Slope	$\cot(\alpha)$	2
Stability Coefficient for Breaking Condition	$K_{D,b}$	2
Stability Coefficient for Non-Breaking Condition	$K_{D,nb}$	4
Breaker Index	γ_b	0.78
Unit Weight of Stones	γ_{stone} (t/m^3)	2.7
Unit Weight of Water	γ_{water} (t/m^3)	1.025
** Mean water level is taken as +0.0.*		

In Figures 5.4-5.7, output graphs of DWLD-I are given for the input parameters given in Table 5.5.

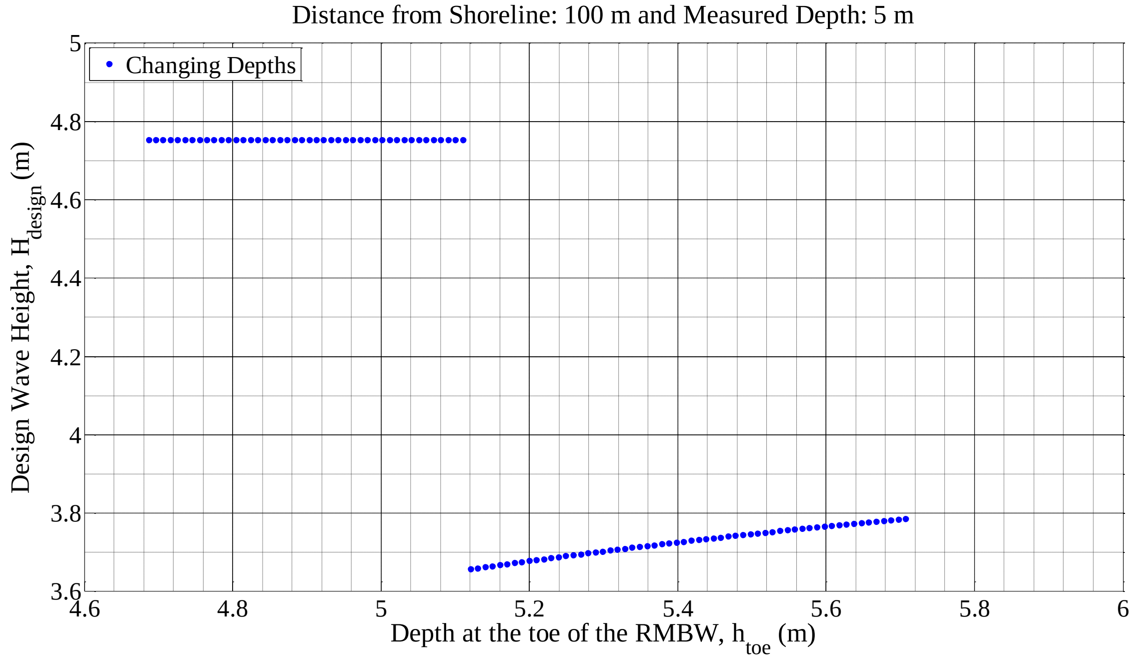


Figure 5.3: Depth at the toe of the RMBW vs Design Wave Height

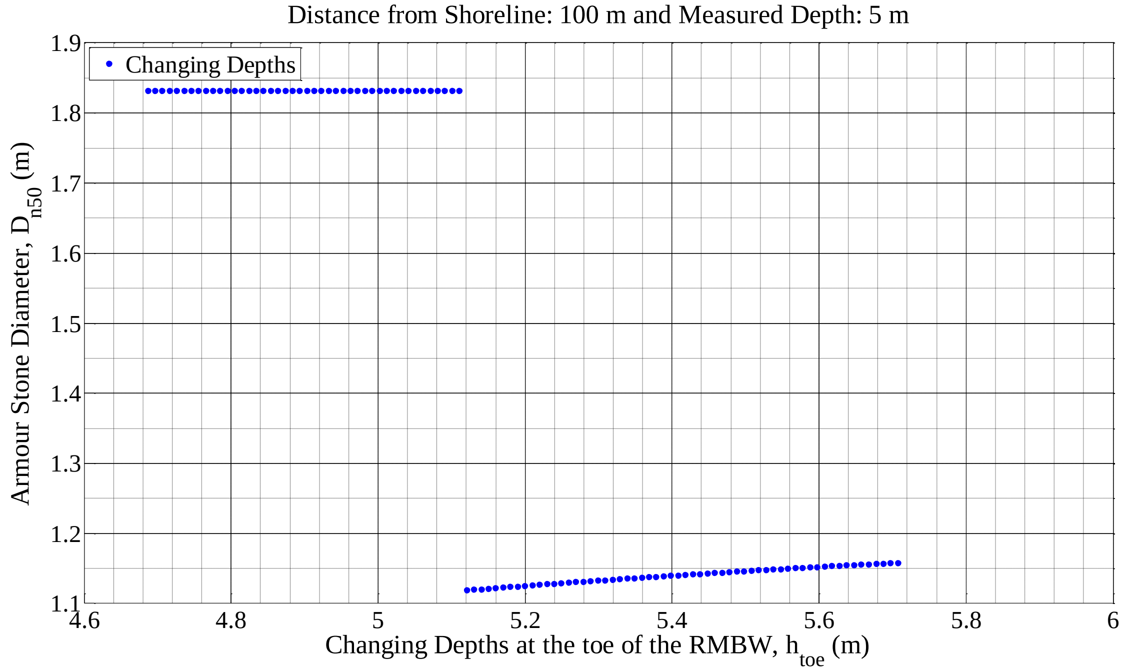


Figure 5.4: Depth at the toe of the RMBW vs Armour Stone Diameter

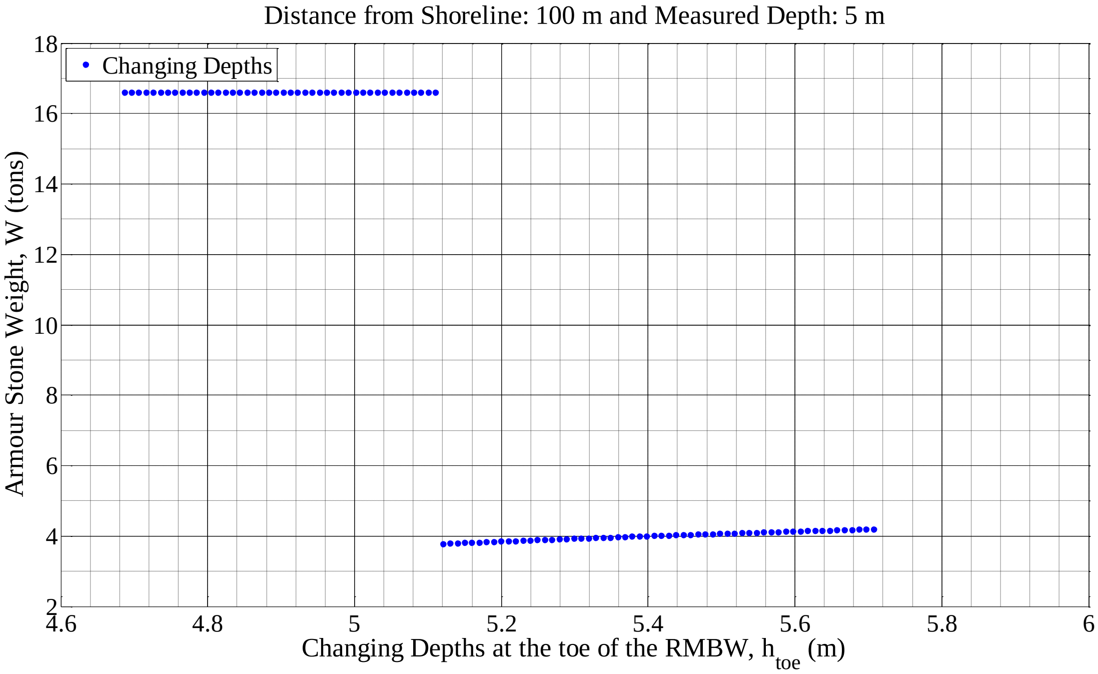


Figure 5.5: Depth at the toe of the RMBW vs Armour Stone Diameter

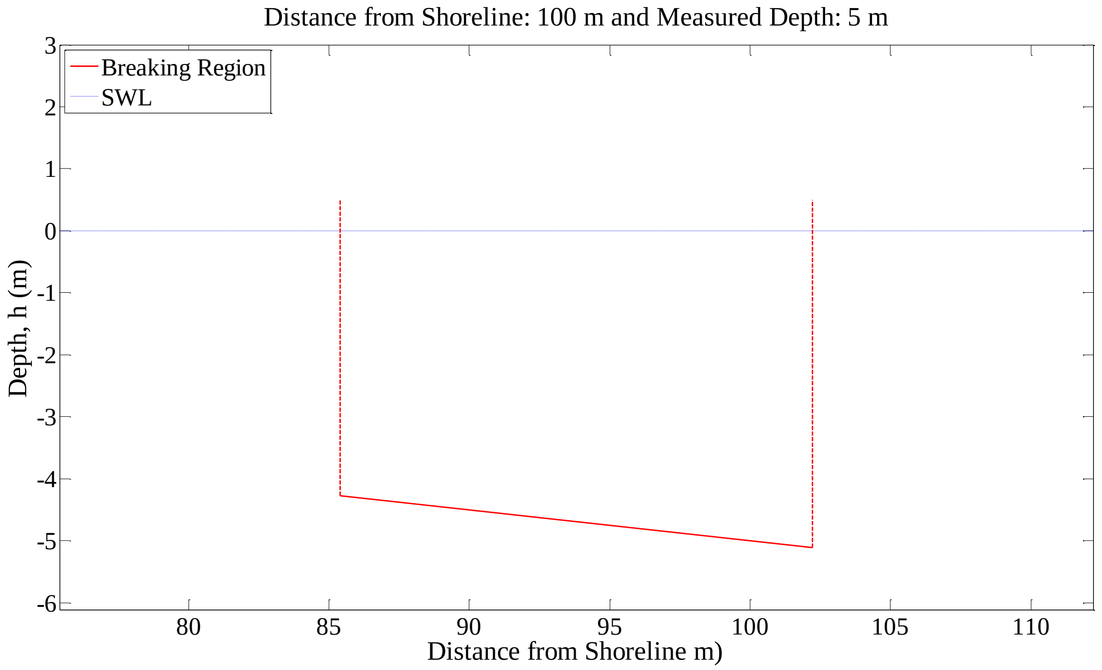


Figure 5.6: Breaking Region for the Example Study

Figures 5.4-5.7 show that differences in armour stone weights are really high throughout 100 years return period. Considering an average of 4 tons in non-breaking zone and 16.6 tons in breaking zone, relative difference defined by Equation 5.24 is calculated as 315%.

$$Relative\ Difference(\%)=\frac{(W_{breaking}-W_{non-breaking})}{W_{non-breaking}}*100 \qquad \textbf{(5.24)}$$

Such a big difference is resulted from the change in mean water level. This change leads to the change from breaking region to non-breaking defined by CERC (1977). In other words, water level increase throughout 100 years return period of rubble mound breakwater results in a safer situation, i.e. non-breaking condition, and water level measured currently is a much more critical condition, i.e. breaking condition. Therefore, it is concluded that increasing water level does not always cause a more critical condition opposing to common understanding.

5.4. Effect of Design Water Level in Van der Meer (1988) and Van Gent et al (2003) Approaches used for Stability of Rubble Mound Breakwaters

Application ranges of Van der Meer (1988) and Van Gent et al (2003) formulations are discussed in Chapter 3 and a new flowchart for application of these formulations depending on three design constraints is defined. Effect of design water level on these formulations that are used together by the procedure given by Chapter 3 is investigated using DWLD-II.

Similar to DWLD-I, DWLD-II takes inputs of range of change in mean water level, deep water significant wave characteristics, wind velocity and foreshore slope, etc. After increasing water level from the lowest level to highest level with pre-defined small increments, armour stone size is computed using the flowchart proposed in Chapter 3. The outputs of DWLD-II are four figures that describing the computations. The first figure is changing depths versus significant wave height at the toe of the structure, the second figure is changing depths versus armour stone diameter, the third figure is changing depths versus armour stone weight and finally the fourth figure is changing depths versus three design constraints that are defined to determine application regions of Van der Meer (1988) and Van Gent et al (2003) formulations. "Changing depths" again refers to decrease or increase of measured mean water level described in Section 5.1.

An example study is carried out by DWLD-II for Black Sea coasts of Turkey to observe the effect of design water level in armour stone size found by new design flowchart. Input design parameters of DWLD-II are given in Table 5.6.

Table 5.5: Input Parameters for Example Study

Parameters		**Value**
Deep Water Significant Wave Height	H_{s0} (m)	4.1
Deep Water Significant Wave Steepness	H_{s0}/L_0	0.04
Deep Water Wave Approach Angle	α_0 (°)	0
Foreshore Slope	m	1/20
Wind Velocity	U (m/s)	15
Low Water Level*	LWL (cm)	-27
High Water Level*	HWL (cm)	76.5
Distance of Breakwater Section from Shoreline	(m)	90
Measured Depth at the toe of the Cross-Section	(m)	4.5
Increment of Change in Mean Water Level	Δh (cm)	1
Slope of Structure Slope	$\cot(\alpha)$	2
Van der Meer Damage Parameter	S	2
Notional Permeability	P	0.4
Breaker Index	γ_b	0.78
Unit Weight of Stones	γ_{stone} (t/m^3)	2.7
Unit Weight of Water	γ_{water} (t/m^3)	1.025
** Mean water level is taken as +0.0.*		

In Figures 5.8-5.11, results obtained from DWLD-II using input parameters given in Table 5.6 are presented.

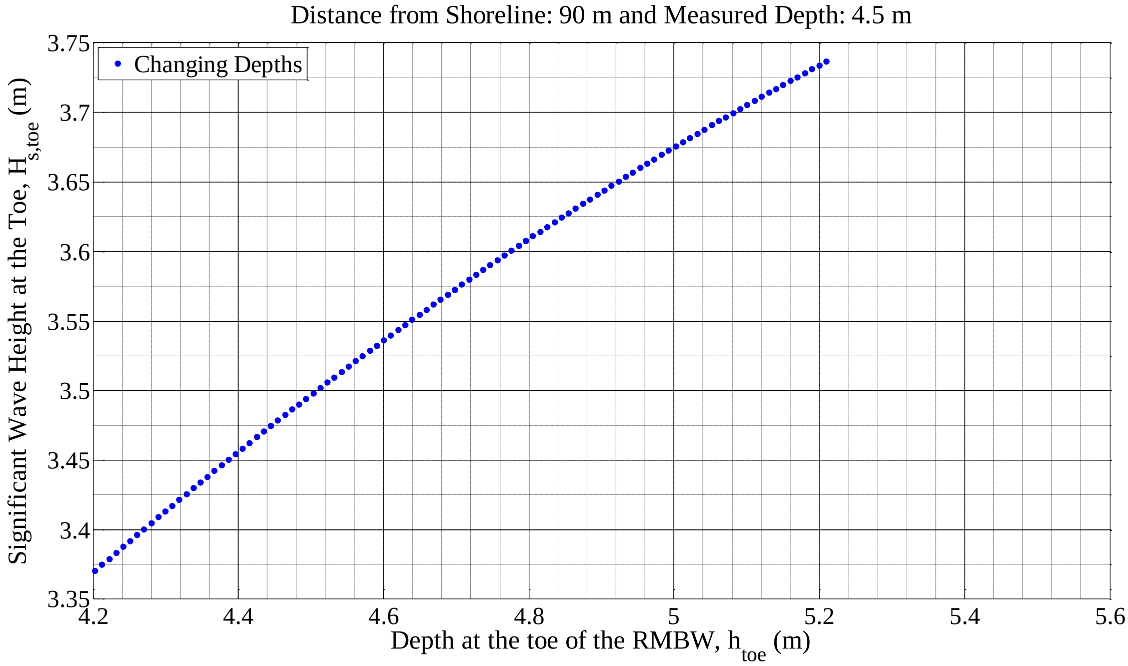


Figure 5.7: Depth at the toe of the RMBW vs Significant Wave Height at the Toe

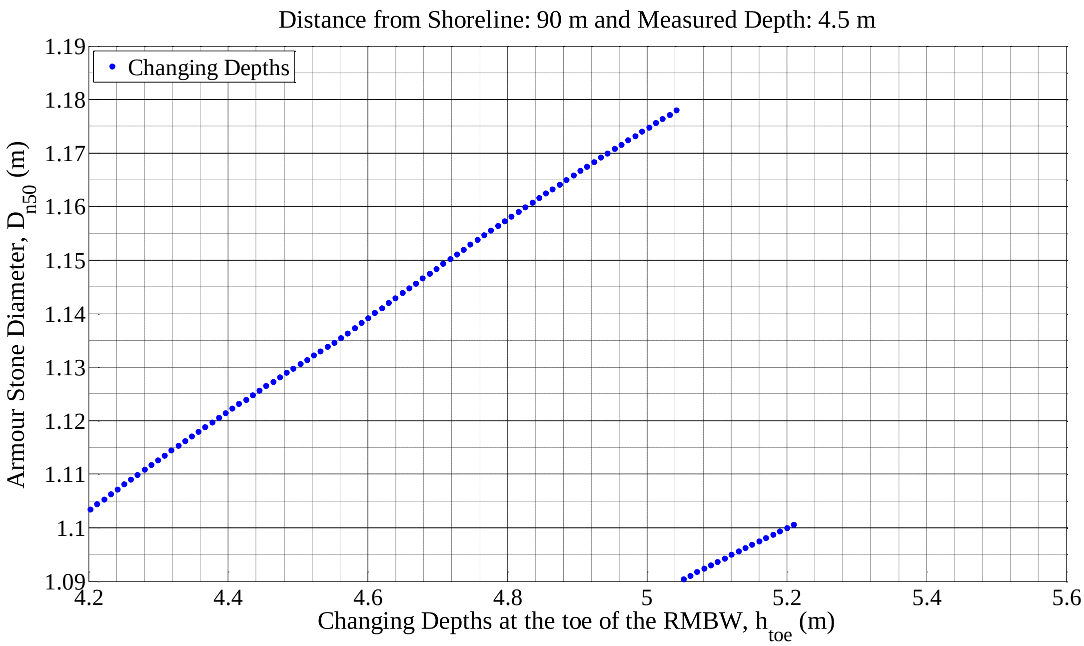


Figure 5.8: Depth at the toe of the RMBW vs Armour Stone Diameter

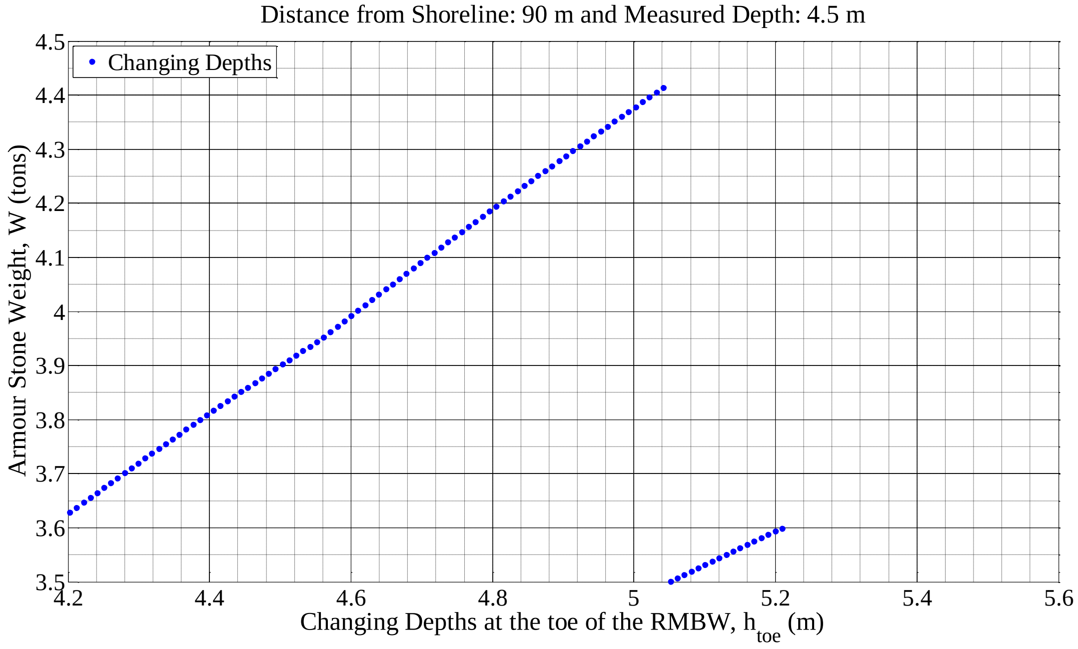


Figure 5.9: Depth at the toe of the RMBW vs Armour Stone Weight

The jump in armour stone diameter, and thus, in armour stone weight of rubble mound breakwater is because of the change in application region of Van der Meer and Van Gent et al formulations. As indicated in Figure 5.11, all design constraints are satisfied around a depth of 5.1 meters where the armour stone diameter and weight figures have a jump.

This example study shows by Figures 5.8-5.11 that differences in armour stone weights due to changes in mean water level are effective when the application region changes from Van der Meer (1988) formulations' region to Van Gent et al (2003) formulations' region throughout 100 years return period. Differences obtained in Van der Meer (1988) and Van Gent et al (2003) approaches are smaller compared to Hudson (1959) approach for the selected examples. Obviously this trend would be same in general since the breaking definition used in Hudson (1959) is totally different than the other formulations. However, differences obtained in new flowchart proposed in Chapter 3 are also important since the range of the stones may change. In Figure 5.10, it is shown that armour stone weight is calculated in 4-6 tons stone range when the design water level is around 4.9 meters, on the other hand, armour stone weight is computed in 2-4 tons stone range when the design water level is around 5.1 meters considering the stone classes used in Turkey.

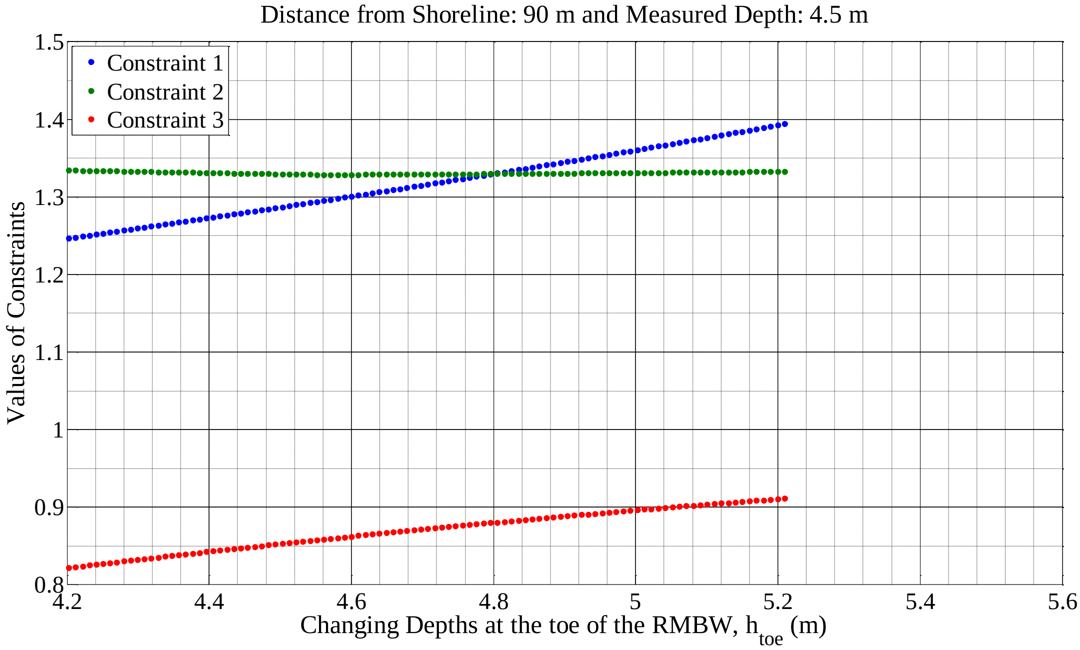


Figure 5.10: Depth at the toe of the RMBW vs Values of Constraints

Therefore, conclusion of this example study is increasing water level can result in a safer condition than current water level considering Van der Meer (1988) and Van Gent et al (2003) formulations.

5.5. Proposition of an Updated Deterministic Approach for Design of Rubble Mound Breakwaters

In this chapter the effect of design water level on stability of rubble mound breakwaters are discussed considering most common rubble mound breakwater design approaches, namely, Hudson (1959), Van der Meer (1988) and Van Gent et al (2003).

Design water level is determined considering changes in mean water level. Components of changes in mean water level are sea level rise due to global warming, seasonal variations, astronomic tides, storm components (wave & wind setup) and barometric and Coriolis effects. These parameters mainly affect the wave conditions at a certain depth, and these wave conditions may result in big differences in armour stone sizes considering different design approaches to find the most critical condition that occurs in the economic life of a coastal structure.

As a common deterministic approach, all the changes in mean water level are summed to define the most critical condition. However, high water level does not always refer to most critical condition as shown in two examples related to Hudson approach (Section 5.3) and the new flowchart proposed in Chapter 3 that uses Van der Meer and Van Gent et al approaches (Section 5.4). DWLD-I and DWLD-II are developed that uses NSW and DAS to transform a deep water wave to the construction depth and design armour stone size for changing mean water level. These codes allow finding the most critical condition. Lowest - highest water level and increment amount between these levels are inputs of DWLD-I and DWLD-II in addition to deep water wave characteristics and other parameters related to design of coastal structures. DWLD codes produces graphs that can help to find the most critical condition.

Computation cost of DWLD is really low since wave transformation methodology is one dimensional. For more accurate results in general, wave transformation should be two dimensional; however, computational cost would be really high this time. Since DWLD is a first approach to determine the most critical condition, probable critical conditions can be tested with more accurate wave transformation methodologies. Another method may be taking more cross-shore bathymetry data in the area of interest and comparing results to find the most critical condition.

Checking all the range between lowest and highest water level with small increments is an updated version of the deterministic approach that is used in general. Since the computation cost is low enough, it is appropriate to use algorithms' of DWLD-I and DWLD-II, especially, for the preliminary design.

CHAPTER 6

A CASE STUDY: ALIAGA, IZMIR, TURKEY

In this chapter, Design Armour Stone (DAS) code developed in Chapter 3 and Design Water Level Determination (DWLD) developed in Chapter 5 are applied to a case study. A breakwater that is planned to be constructed in Aliaga Bay near Izmir, Turkey in Aegean Sea is taken as a case study. This area is selected since it is an appropriate case to discuss results of DAS and DWLD.

A coastal structure is usually constructed with design wave characteristics considering a return period of 100 years. In this chapter, wave climate studies are done for the region to obtain design wave characteristics with a return period of 100 years. After wave climate studies, design wave height is transformed to near shore using Near Shore Wave Transformation (NSW) numerical model. Finally, DAS and DWLD are applied using near shore wave transformation characteristics and results are discussed.

6.1. Wave Climate and Wave Transformation Studies

In wave climate studies, wind data is obtained for the region and wave hindcasting studies are performed. The steps followed in wave climate studies are given below:

- Wind data is obtained from ECMWF (European Centre for Medium-Range Wave Forecasts) for the region.
- Effective fetch lengths are computed for the region.
- Wind data and effective fetch distances are used for wave hindcasting studies. For wave hindcasting, a mathematical model developed by METU Department of Civil Engineering Ocean Engineering Research Center called "Deep Water Wave Hindcasting Mathematical Model, W61" (Ergin & Ozhan, 1986) is used.
- Using hindcasted wave data, extreme term wave statistics are done using Goda (2000) methodology.

6.1.1. Wind Data

ECMWF (European Centre for Medium-Range Wave Forecasts) is a well-established international database that provides wind and wave data for all over the world. ECMWF provides wind data with 0.1 degree intervals. A point, 38.80N-26.50E, is selected in deep water near study area in Aliaga and wind data for this point is obtained for years between 1983 and 2010 (28 years) as wind velocity in x and y directions at 10 m above of sea level. The point selected from ECMWF database and the study area is given in Figure 6.1.

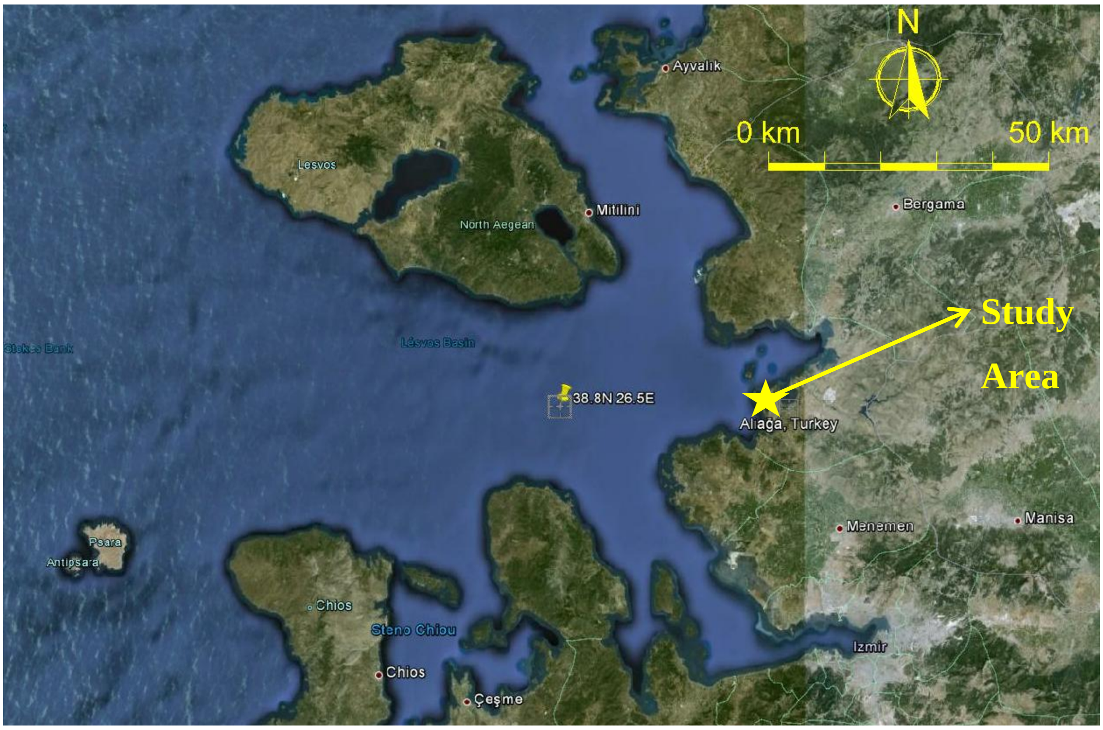


Figure 6.1: ECMWF data point and Study Area

6.1.2. Effective Fetch Length Calculations

Fetch is the sea area over which the wind blows to create waves. Fetch lengths are measured from an appropriate map on the sea area over which the wind is blowing. Effective fetch is the equivalent fetch of the region by weighted average process of the fetch of each direction. In effective fetch studies, two basic assumptions are taken into account:

1. Waves are generated over a range of 22.5˚ to either side of the wind direction and energy is transfer from wind to waves is proportional to cosine of the angle between waves wind and waves.
2. Wave growth is proportional to fetch length.

The effective fetch length calculation is measuring fetch distances drown out at 7.5˚ intervals over 22.5˚ to either side of the wind directions. Then, Equation 6.1 is used to calculate effective fetch distance.

$$F_{eff} = \frac{\sum F_i \cos^2 \gamma_i}{\sum \cos^2 \gamma_i} \qquad [6.1]$$

In this study, Google Earth images for Aegean Sea are used. Effective fetch lengths are calculated between North West (NW) and South West (SW) in counter clockwise direction. Effective fetch distances are given in Figure 6.2. A sample calculation for West (W) direction is given in Table 6.1.

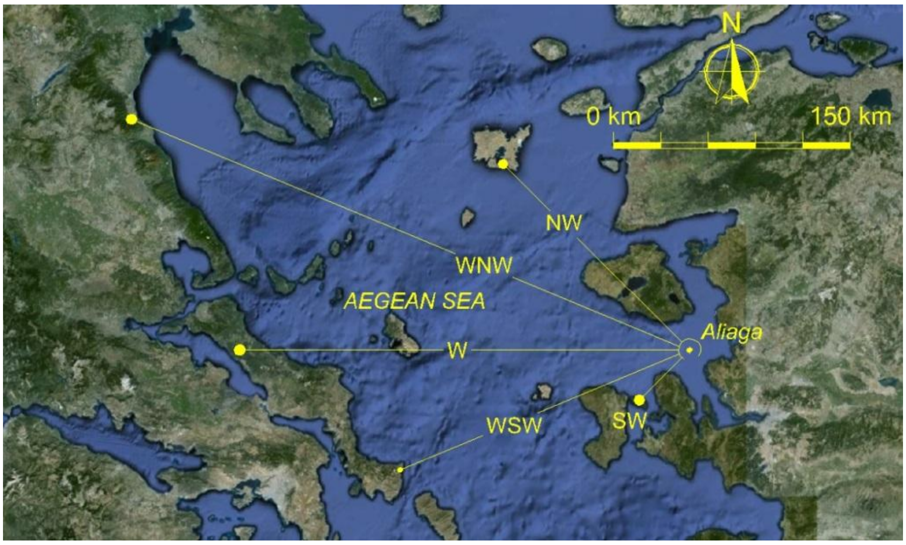


Figure 6.2: Effective Fetch Distances for Study Area

Table 6.1: Sample Effective Fetch Calculation for West Direction

β (deg)	cos(β)	(cosβ)^2	f (km)	f · cos(β)^2 (km)	
-22.5	0.92388	0.853553	262.65	224.185798	
-15	0.965926	0.933013	252.25	235.3524541	
-7.5	0.991445	0.982963	254.04	249.7118985	
0	1	1	255.35	255.35	
7.5	0.991445	0.982963	267.01	262.4609274	
15	0.965926	0.933013	288.00	268.7076581	
22.5	0.92388	0.853553	306.00	261.1873375	
Sum	6.7625	6.539058		1756.956074	***268.7 km***

Effective fetch distances for directions NW to SW in counter clockwise direction are presented in Table 6.2.

Table 6.2: Effective Fetch Lengths

Direction	Effective Fetch Lengths (km)
NW	137.0
WNW	255.0
W	259.0
WSW	143.6
SW	55.4

6.1.3. Wave Hindcasting Studies

Wind velocity at 10 m above of the sea level, wind direction and effective fetch lengths are the parameters that are used in wave hindcasting studies. Wind velocity data is taken from ECMWF database for 38.8N-26.5E point between 1983 and 2010. Direction is found using x and y components of wind velocity data and effective fetch lengths are calculated. These parameters are used as input parameters of the mathematical model, W61, to obtain significant wave heights hourly. This mathematical model uses input parameters to calculate energy produced by friction forces occurred on the sea surface due to wind that generates waves.

Using deep water wave characteristics obtained from mathematical model, deep water significant wave steepness (H_{s0}/L_0) is calculated as 0.0436. Deep water significant wave heights versus deep water significant wave lengths are given in Figure 6.3.

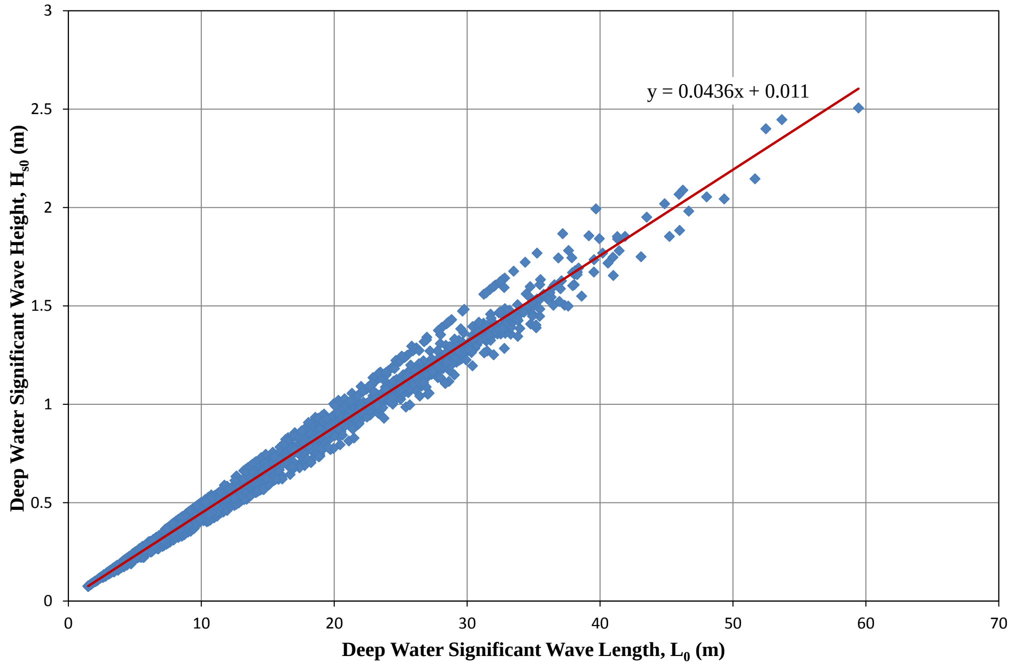


Figure 6.3: Deep Water Significant Wave Heights vs Deep Water Significant Wave Lengths

6.1.4. Extreme Term Wave Statistics

Previously in this chapter, deep water hourly averaged significant wave properties are obtained. Using this data, wave parameters needed to design coastal structures are determined by extreme term wave statistics. Yearly maximum deep water significant wave properties that are given in Table 6.3 are used to perform extreme term wave statistics.

Table 6.3: Yearly Maximum Significant Wave Properties

Year	H_{s0} (m)	T_s (sec)	Direction
1983	1.44	4.62	W
1984	2.11	5.46	WSW
1985	1.84	4.85	SW
1986	2.87	6.29	SW
1987	2.04	5.11	SW
1988	2.14	5.7	WNW

Table 6.3 (continued)

1989	1.74	5	WSW
1990	1.54	4.45	W
1991	2.08	5.46	WSW
1992	1.81	5.03	WSW
1993	1.76	4.81	NW
1994	1.36	4.34	SW
1995	1.48	4.82	WSW
1996	1.56	4.93	SW
1997	1.74	4.98	W
1998	1.74	4.87	W
1999	2.52	6.18	WNW
2000	1.28	4.33	W
2001	2.4	5.77	W
2002	2.91	6.27	W
2003	2.1	5.46	W
2004	2.14	5.45	W
2005	1.78	4.76	SW
2006	1.81	5.3	WNW
2007	3.43	7	W
2008	3.05	6.45	W
2009	2.35	5.75	NW
2010	2.87	6.29	WSW

Yearly maximum significant wave heights (H_{s0}) are fitted to FT-I (Gumbel), FT-II, Weibull and Log-Normal distributions in terms of extreme term wave statistics using the methodology given by Goda (2000). Details of these distributions are given in Appendix A. The best fit is determined using "Goodness of Fit Tests" given by Goda (2000) and the best distribution is selected as FT-I (Gumbel) distribution. In FT-I

(Gumbel) Distribution, non-exceedance probability is given by Equation 6.2 whereas distribution is defined by Equation 6.3.

$$P(<H_{s0}) = 1 - \frac{m}{N+1} \qquad [6.2]$$

$$H_{s0} = A\,(-\ln(-\ln(P(<H_{s0}))) + B \qquad [6.3]$$

“m” in Equation 6.2 is the order number of year if the deep water significant wave heights are sorted in decreasing order, N is the total number of years and $P(<H_{s0})$ is non-exceedance probability of H_{s0}.

A and B coefficients in Equation 6.3 is the slope and intercept with y-axis, respectively, of the best fit line found by linear least squares regression analysis when deep water significant wave heights (H_{s0}) are plotted with respect to $–\ln(-\ln(P(<H_{s0})))$.

To find deep water wave characteristics with a return period of 100 years, Equation 6.4 is used to relate non-exceedance probability with return period. After finding non-exceedance probability for a certain return period, deep water wave height is found using FT-I distribution equation given in Equation 6.3. Significant wave period is computed using deep water wave steepness.

$$R_p = \frac{1}{1 - P(<H_{s0})} \qquad [6.4]$$

Furthermore, 90% confidence intervals for FT-I (Gumbel) distribution is determined (Goda, 2000). In Figure 6.4, extreme term wave statistics is given and deep water significant wave heights within 90% confidence intervals are presented in Table 6.4 for return periods of 5, 10, 20, 50, 100, 200 and 500 years.

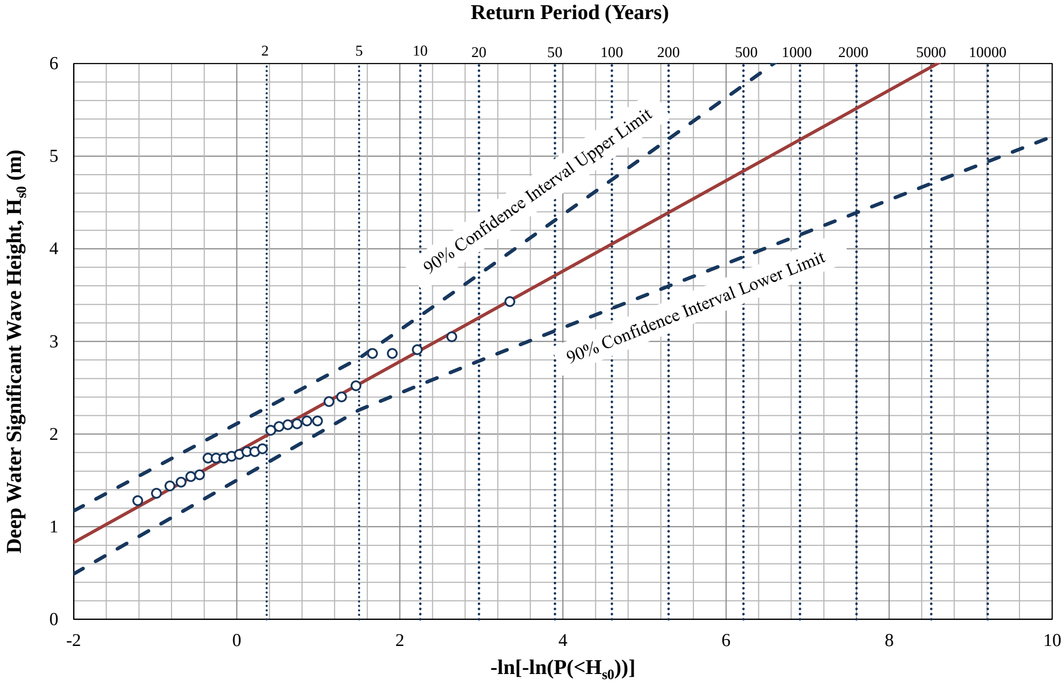


Figure 6.4: Extreme Term Wave Statistics

Table 6.4: Results of Extreme Term Wave Statistics (Deep Water)

Return Period (year)	Deep Water Significant Wave Height, H_{s0} (m)
5	2.54 ± 0.28
10	2.90 ± 0.37
20	3.26 ± 0.47
50	3.71 ± 0.60
100	4.05 ± 0.70

Deep water significant wave height with a return period of 100 years is found as 4.05 meters and considering deep water wave steepness that is found as 0.0436, significant wave steepness for the wave with a return period of 100 years can be found as 7.72 seconds.

6.1.5. Wave Transformation Studies

For wave transformation studies, 1D bathymetry is needed as an input of NSW. Detailed bathymetry of the project region is obtained given in Figure 6.5. 1D bathymetry along a line is taken from this data as shown in Figure 6.5.

Figure 6.5: Bathymetry for the Study Area

Deep water wave characteristics are found by extreme term wave statistics as given in Section 6.1.4. Wind velocity is another parameter needed for NSW. Wind velocity is found using S-M-B methodology (Wilson, 1965) as 16.67 m/s. Mean depth and fetch length is used when performing S-M-B for West direction which is 1006 meters and 259 kilometers, respectively.

Using these parameters and NSW, significant wave height at the toe of the structure is found when using DWLD and DAS.

6.2. Application of DWLD Code to Aliaga

DWLD code is applied to Aliaga as a case study. When applying DWLD, a breakwater is planned that starts from shoreline and goes up to 100 meters away from the shoreline .

Aliaga is a coastal town near Aegean Sea, therefore, the range of water level change is taken as -25.9 cm to 73.2 cm as discussed in Table 5.4.

All parameters that are used in application of both DWLD-I (Hudson Approach) and DWLD-II (Van der Meer and Van Gent et al approaches) are presented in Table 6.5.

Table 6.5: Input Parameters of DWLD Code

Parameter		**Value**
Deep Water Significant Wave Height	H_{s0} (m)	4.05
Significant Wave Period	T_s (sec)	7.72
Deep Water Wave Approach Angle	α_0 (°)	0
Wind Velocity	U (m/s)	16.67
Low Level Design Water Level	LWL (cm)	-25.9
High Level Design Water Level	HWL (cm)	73.2
Increment of Design Water Level	Δh (cm)	1
Breaker Index	γ_b	0.78
Hudson Stability Coefficient, Breaking	$K_{D,b}$	2
Hudson Stability Coefficient, Non-Breaking	$K_{D,nb}$	4
Van der Meer Damage Parameter	S	2
Notional Permeability	P	0.4
Unit Weight of Stones	γ_{stone} (t/m^3)	2.7
Unit Weight of Water	γ_{water} (t/m^3)	1.025
Structure Face Slope	$\cot(\alpha)$	2

<u>Application of DWLD-I:</u>

DWLD-I is run between shoreline to a distance of 100 meters away from shoreline increasing distance by 5 meters. It is seen that the most critical condition occurs near 70 meters away from the shoreline. Computation time when distance of breakwater to shoreline changed from 0 to 100 meters with 5 meters is 122.896 seconds, on the other hand, it is 4.581 seconds when DWLD-I is run just for a 70 meters distance.

The outputs of DWLD-I are given in Figures 6.6 to 6.9.

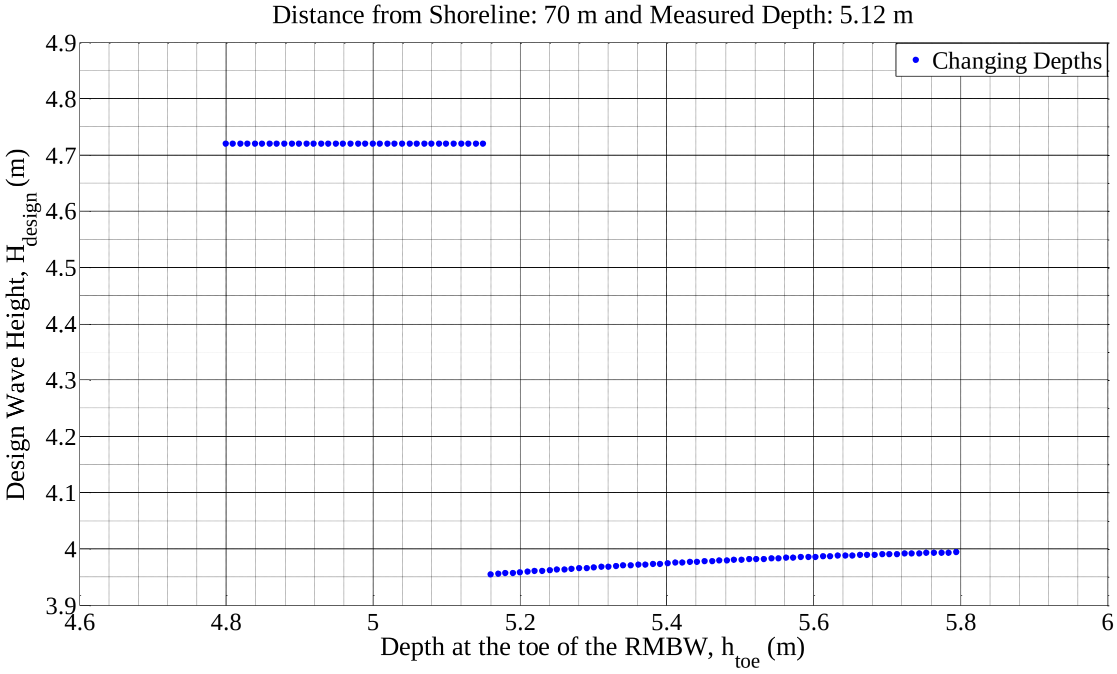


Figure 6.6: Changing depths vs Design Wave Height

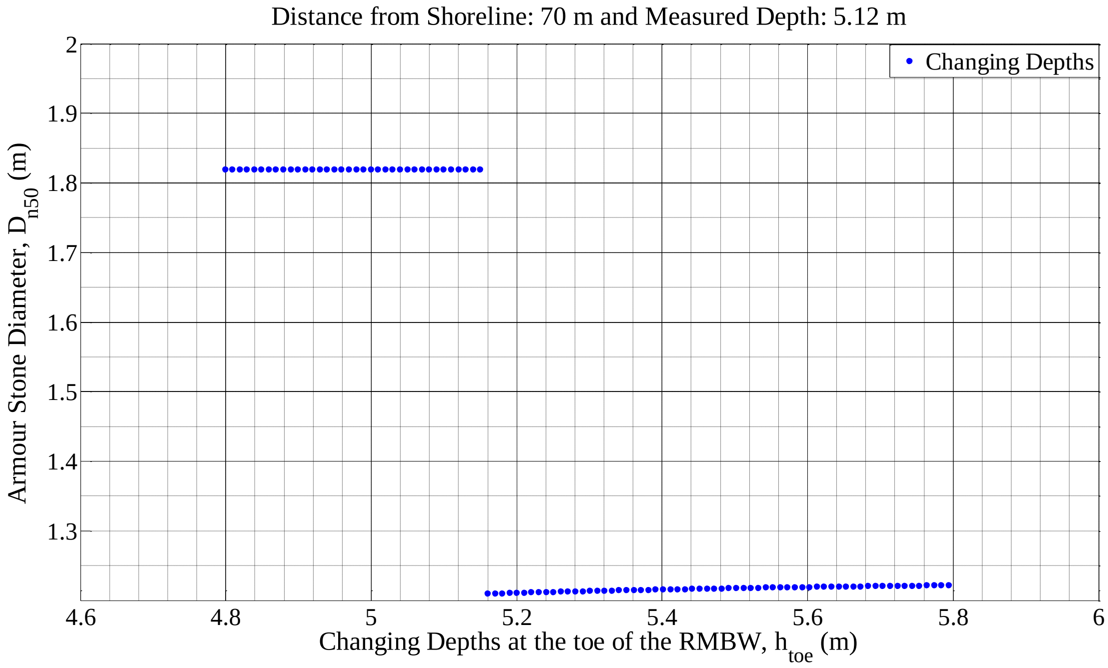


Figure 6.7: Changing Depths vs Armour Stone Diameter

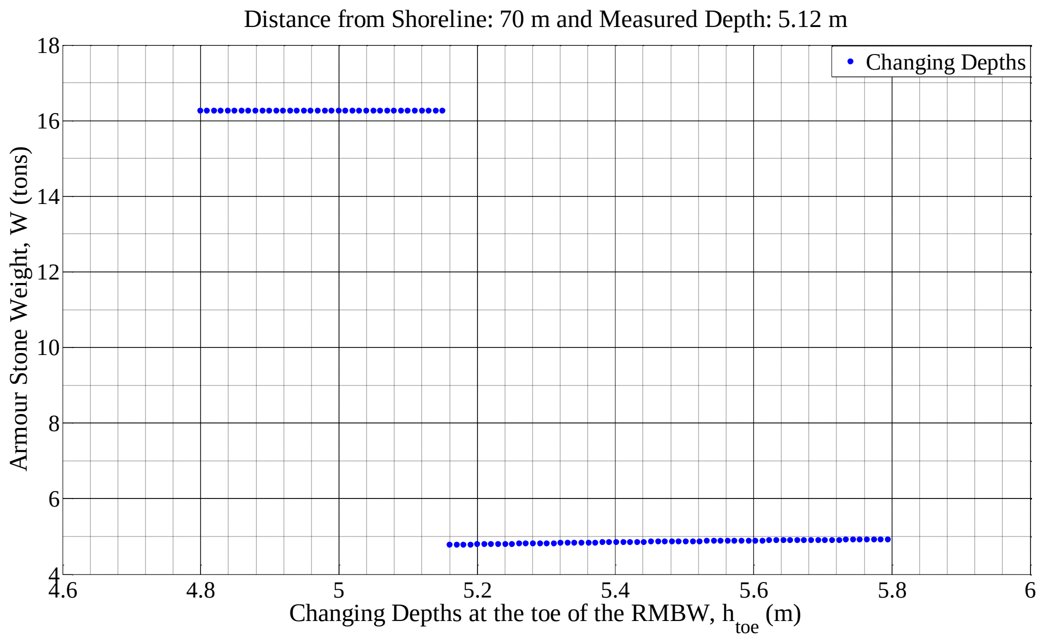


Figure 6.8: Changing Depths vs Armour Stone Weight

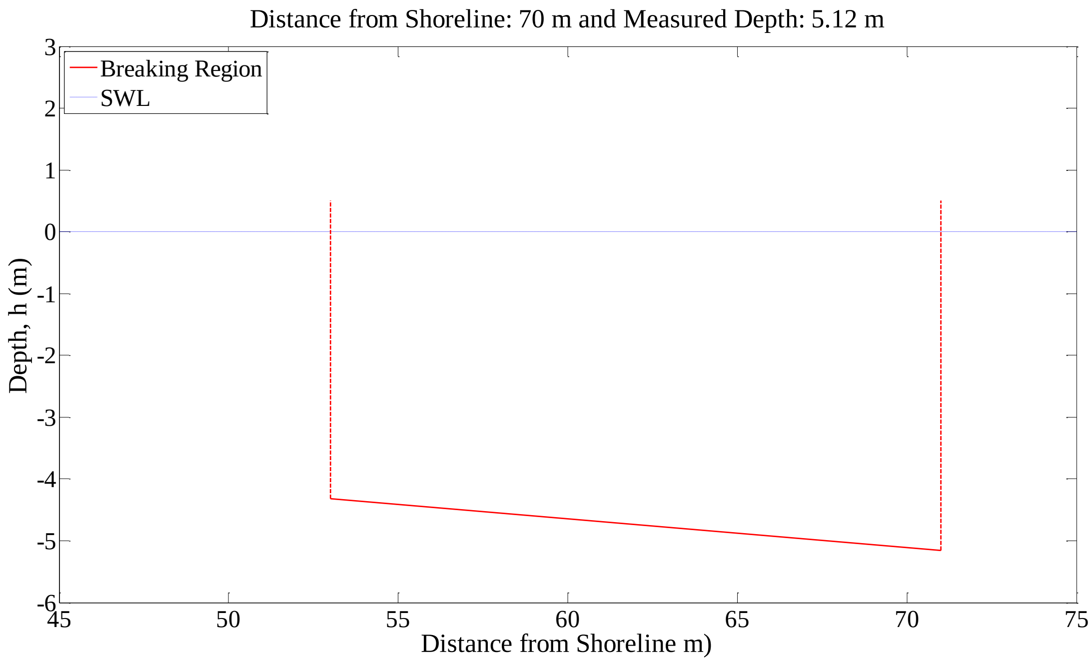


Figure 6.9: Breaking Region

It is observed from Figures 6.6 to 6.9 that design condition for Hudson approach change from breaking to non-breaking considering a measured depth of 5.12 meters.

This case study shows that the most critical condition occurs currently throughout the coastal structure's economic lifetime.

Application of DWLD-II:

Similar to DWLD-I, DWLD-II is run within an interval starts from shoreline to a distance of 100 meters away from the shoreline with an increment of 5 meters. The most critical condition is observed when distance of breakwater to shoreline is 60 meters which corresponds to 4.28 meters water depth. The outputs of DWLD-II are given in Figures 6.10-6.13.

Computation time for all conditions calculated by DWLD-II is 133.755 seconds whereas it is 4.774 seconds for the most critical condition.

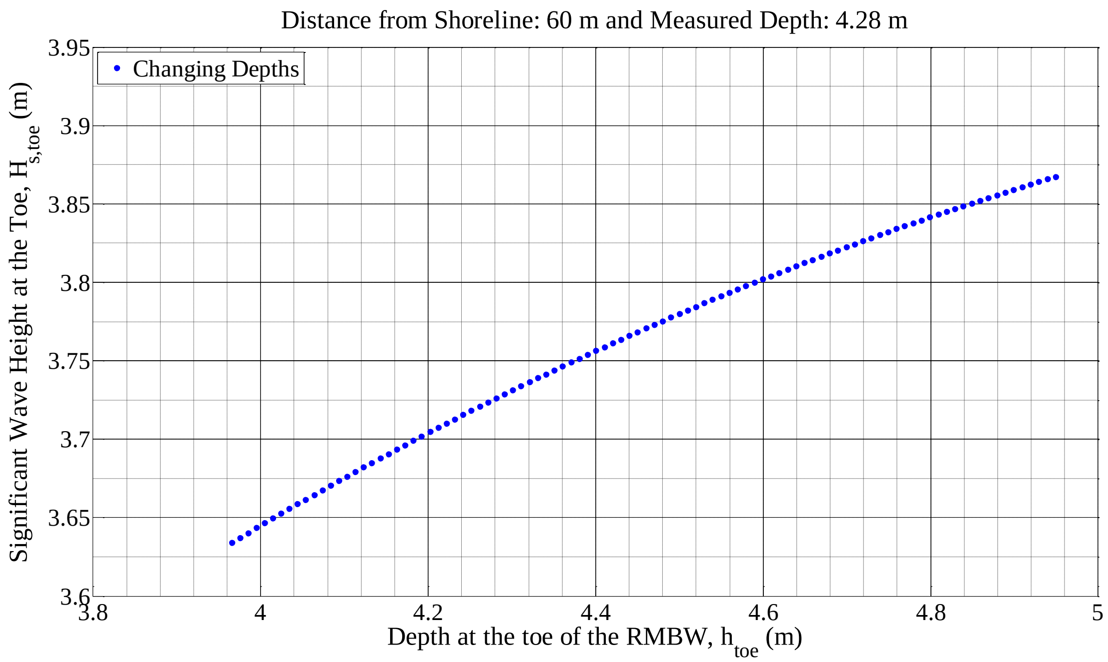


Figure 6.10: Changing Depths vs Significant Wave Height at the Toe of the Structure

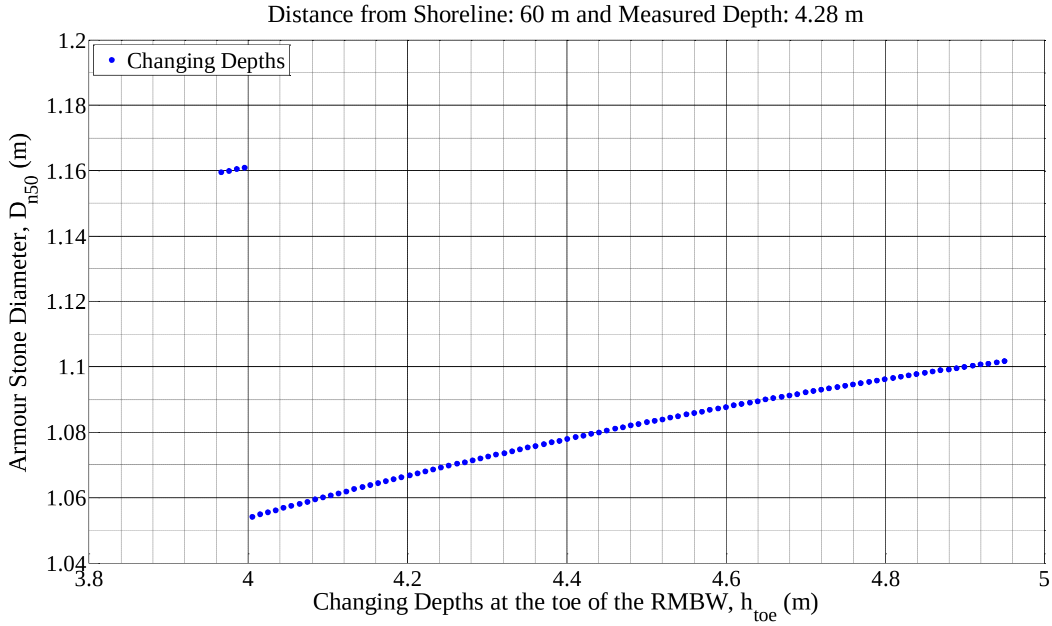


Figure 6.11: Changing Depths vs Armour Stone Diameter

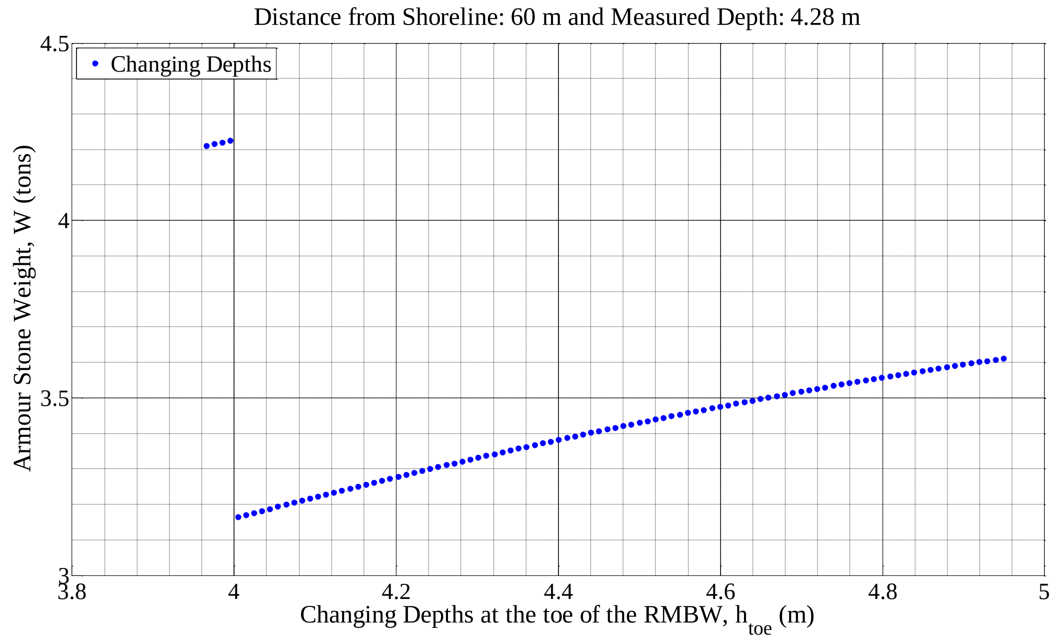


Figure 6.12: Changing Depths vs Armour Stone Weight

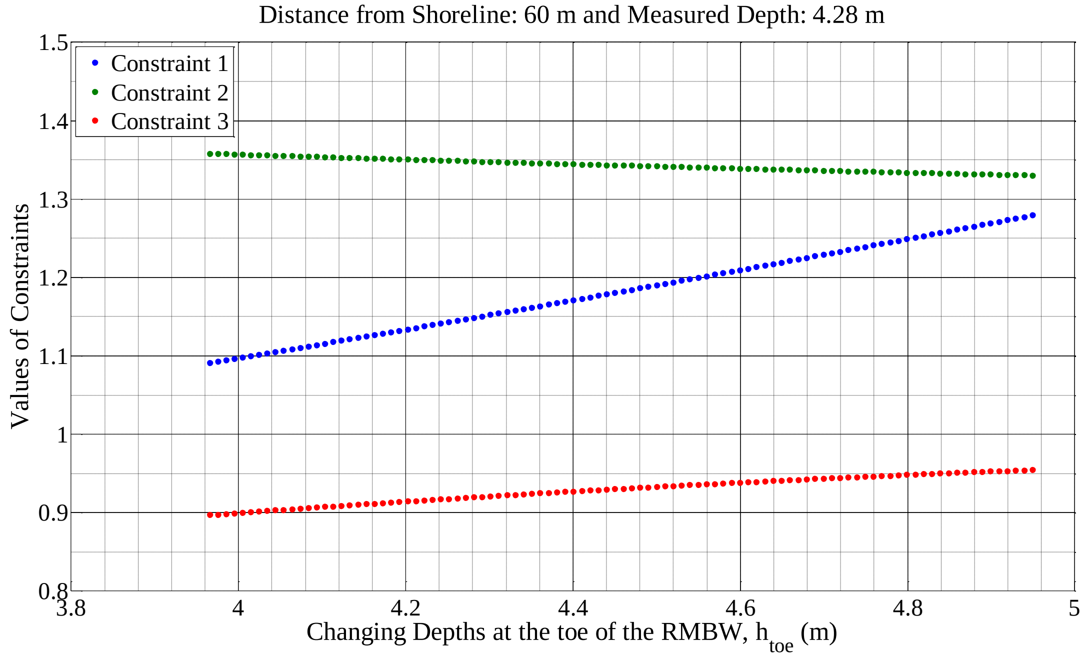


Figure 6.13: Changing Depths vs Constraints

In Figures 6.10-6.13, presented condition indicates that lowest design water level is the most critical condition. Note that the differences are not as significant as the results obtained by DWLD-I due to the methods' nature. However, the presented results show that stone classes may change (from 4-6 tons to 2-4 tons) by the effect design water level which is also important in design of coastal structures.

6.3. Application of DAS Code to Aliaga

Design Armour Stone (DAS) code is applied to Aliaga region in order to show the significance of new design flowchart proposed in Chapter 3. The new flowchart defines three design constraints and Van der Meer (1988) and Van Gent et al (2003) formulations are used simultaneously according to satisfaction condition of these design constraints.

Selected design parameters for application of DAS are given in Table 6.6. Note that wave transformation is done by NSW using the bathymetry given in Section 6.1.5.

Table 6.6: Input Parameters of DAS Code

Parameter		Value
Deep Water Significant Wave Height	H_{s0} (m)	4.05
Significant Wave Period	T_s (sec)	7.72
Deep Water Wave Approach Angle	α_0 (°)	0
Wind Velocity	U (m/s)	16.67
Water Depth at the Toe of the Structure	h_{toe} (m)	8
Significant Wave Height at the Toe of the Structure	$H_{s,toe}$ (m)	3.96
Breaker Index	γ_b	0.78
Hudson Stability Coefficient, Breaking	$K_{D,b}$	2
Hudson Stability Coefficient, Non-Breaking	$K_{D,nb}$	4
Van der Meer Damage Parameter	S	2
Notional Permeability	P	0.4
Unit Weight of Stones	γ_{stone} (t/m^3)	2.7
Unit Weight of Water	γ_{water} (t/m^3)	1.025
Structure Face Slope	cot(α)	2

DAS calculates armour stone weight using Van der Meer (1988), Van Gent et al (2003), Hudson (CERC, 1977) and Hudson (CERC, 1984) approaches in addition design constraints. Using the input parameters given in Table 6.6 and DAS, armour stone weights are found by different approaches. Results are presented in Table 6.7.

Table 6.7: Results obtained by DAS Code

Parameter		Value
Wave Height exceeded by 2% of the waves at the toe of the structure	$H_{2\%,toe}$ (m)	6.05
Design Constraint 1	$h/ H_{s,toe}$	2.02
Design Constraint 2	$H_{2\%}/ H_{s,toe}$	1.53
Design Constraint 3	$H_{s,toe}/ H_{s0}$	0.98
Armour Stone Weight: Van der Meer (1988) Approach	W_{VdM} (tons)	3.81
Armour Stone Weight: Van Gent et al (2003) Approach	W_{VG} (tons)	7.29
Armour Stone Weight: Hudson (CERC, 1977) Approach	$W_{Hud,Hs}$ (tons)	4.36
Armour Stone Weight: Hudson (CERC, 1984) Approach	$W_{Hud,1/10}$ (tons)	9.30

This selected case in Aliaga region showed that Van der Meer (1988) approach gives a stone size in the stone class of 2-4 tons whereas stone class is determined as 6-8 tons by Van Gent et al (2003) approach. This corresponds to 47.74% difference in weight. According to the new flowchart proposed in Chapter 3, since only Design Constraint 1 is satisfied by the selected design conditions, Van der Meer (1988) approach should be used as the design flowchart. However, according to the definition in Rock Manual (2007), Van Gent et al (2003) is recommended since $h/H_{s,toe}$ is smaller than 3. Therefore, the proposed flowchart for the design of armour stone of rubble mound breakwaters might have recommended to be considered in the design to reduce expenses.

CHAPTER 7

CONCLUSION

In this study, design of rubble mound breakwaters is investigated in terms of major rock slope stability formulas and design water level. Hudson (CERC, 1977; CERC 1984), Van der Meer (1988) and Van Gent et al (2003) formulations are compared to each other considering wide application ranges to show discrepancies in their application. A new flowchart is defined for Van der Meer and Van Gent et al approaches in order to cover both deep and shallow water ranges. This application method is tested by physical model experiments. On the other hand, changes in mean water level and their effect on design of rubble mound breakwaters are discussed. As one of the most important design parameter that differs throughout the economic life of a coastal structure, a basic deterministic method is provided to find the most critical design water level. A real case study is performed in Aliaga, Izmir, Turkey in order to show the importance of the methodologies defined for application of Van der Meer, Van Gent et al formulations and deterministic methodology to find the most critical design water level.

Results of this study are summarized as following:

- Van der Meer and Van Gent et al formulations may give different results in armour stone weight up to 70% relatively under same design conditions resulting in drastic cost and application problems.
- Differences between Van der Meer and Van Gent et al formulations decrease to 4-6% relatively, if design constraints given to define shallow water are applied. This difference is meaningful due to complexity of shallow water regions. On the other hand, knowing that Van der Meer formulas are derived for deep and moderate shallow water, Van Gent et al formulation is seemed to be more appropriate to apply in the shallow water defined by design constraints. Moreover, Van Gent et al formulations use spectral mean energy wave period that defines shallow water processes in a better way since it takes influence of spectral shape. In the light of these discussions, Van Gent et al formulations are recommended to be used in shallow water defined by given design constraints. Beyond shallow water, Van der Meer formulations are recommended since it gives an economic result and tested among many laboratory and field experiments.
- A conventional rubble mound breakwater is physically modeled and tested under design conditions satisfying design constraints. From physical model experiments, it is concluded that cross-section is not damaged when it is designed by Van Gent et al approach; on the other hand, it is suffered from intermediate damage level when it is designed by Van der Meer approach. This shows that Van Gent et al approach is more appropriate to use in shallow water defined by design constraints.
- Design water level is taken as the most critical water level in general. This critical water level is usually determined by adding components of mean water level change to the measured depth considering the end of economic life time. However, it is shown by this study that the most critical may or may not occur at the end of economic life. A computational tool is developed to find the most critical design water level which calculates armour stone size at the toe of the structure starting from low water level (LWL) up to high water

level (HWL) with small increments. By this deterministic approach, the most critical design water level is determined where the biggest armour stone size is computed. However, it should be noted that discussions related to design water level are limited to armour stone size. Most critical condition can be changed for phenomenon such as wave run-up and overtopping.

- Finally, a real case study shows that application of both methodologies given in this study emphasizing the importance of the comparative study on the design of rubble mound breakwaters.
- This study can be regarded as a guideline for practical designers. It summarizes design approaches and discrepancies in Van der Meer and Van Gent et al methodologies and defines clear application ranges for these formulations. Furthermore, it offers a basic deterministic approach to determine design water level.

However, there are obviously questions remaining on design of rubble mound structures. These questions are left as future studies which are itemized as following:

- Effect of number of waves (or storm duration or wave period) in Van der Meer and Van Gent et al approaches should be investigated.
- Notional permeability parameter in Van der Meer and Van Gent et al approaches should be investigated.
- Limited number of physical model experiments on the application of design constraints is performed in this study. Physical model experiments should be extended considering conditions such as different steepness ranges, different structure face slopes, different storm durations, different notional permeability construction styles, etc.
- Field applications of Van der Meer and Van Gent et al approaches should be comparatively studied.
- Design Water Level Determination (DWLD) should be extended considering probabilistic assessment. Furthermore, effect of extreme marine hazards such as storm surges and tsunamis should be included in DWLD probabilisticly.

This study forms a base for these abovementioned future studies that should be done in order to provide a better understanding on design of rubble mound structures.

REFERENCES

Alpar, B., Doğan, E., Yüce, H. (1995), *"On the Long Term (1935-1976) Fluctuations of the Low Frequency and Main Tidal Constituents and their Stability in the Gulf of Antalya",* Turkish Journal of Marine Science 1, 13-22

Alpar, B., Dogan, E., Yuce, H., Altiok, H. (2000) *"Sea Level Changes along the Turkish Coasts of the Black Sea, the Aegean Sea and the Eastern Mediterranean"*, Mediterranean Marine Science, Vol. 1, 141-156

Alpar, B., Yuce, H. (1998) *"Sea Level Variations and Their Interactions between the Black Sea and the Aegean Sea"*, Estuarine, Coastal and Shelf Science 46, 609-619

Baldock, T. E., Holmes, P., Bunker, S., Van Weert, P. (1998) *"Cross-Shore Hydrodynamics within an Unsaturated Surf Zone"*, Coastal Engineering, Vol 34, pp. 173-196

Battjes, J. A., Groenendijk, H. W. (2000) *"Wave Height Distribution on Shallow Foreshores"*, Coastal Engineering, Vol 40, No. 3, pp. 161-182

Baykal (2010) *"Irregular Wave Generation and Analysis"*, MATLAB Codes

Baykal, C. (2012) *"Two-Dimensional Depth-Averaged Beach Evolution Modelling"*, Ph. D. Thesis, Middle East Technical University, Ankara, Turkey

Belokopytov, V., Goryachkin, Y. (1999) "*Sea Level Changes in the Black Sea (1923-1997)"*, Mitchum, Gary (Ed.), IOC Workshop Report No: 171, Circulation Science Derived from the Atlantic, Indian and Arctic Sea Level Networks, Annex III, pp. 88-92

Broderick, L. L. (1984) *“Riprap Stability versus Monochromatic and Irregular Waves”*, MSc Thesis, George Washington University, USA

CERC (1977), *“Shore Protection Manual”*, USACE, Vicksburg, Mississippi, USA

CERC (1984), *“Shore Protection Manual”*, USACE, Vicksburg, Mississippi, USA

Dalyrmple, R. A. (1989) *“Physical Modelling in Littoral Processes”*, Recent Advances in Hydraulic Physical Modelling, R. Martins, Ed., Kluwer Academic Publishers, Dordrecht, the Netherlands

Dingemans, M. (1987) “*Verification of Numerical Wave Propagation Models with Laboratory Measurements, HISWA Verification in the Directional Wave Basin”*, Technical Report H228, Part 1B, Appendices A-G, Delft Hydraulics, Delft

Disco, M. J. (2013) *“A Generic Quantitative Damage Description for Rubble Mound Structures”*, MSc Thesis, Delft University, the Netherlands

Dixon, M. J., Tawn, J. A. (1994) *“Extreme Sea-Levels at the UK A-class Sites: Site by Site Analyses*” Proudman Oceanographic Laboratory Internal Document No: 65

Ergin, A. (2009) *“Coastal Engineering”*, METU Press, Ankara, Turkey

Ergin, A., Ozhan, E. (1986) *“Wave Hindcasting Studies and Determination of Design Wave Characteristics for 15 Regions – Final Report”*, Middle East Technical University, Department of Civil Engineering, Ankara, Turkey (in Turkish)

FHA (2008) *“Highways in the Coastal Environment: Second Edition”, US. Department of Transportation”*, USA

Goda, Y. (2000) *“Random Seas and Design of Maritime Structures”*, Advanced Series on Ocean Engineering, Vol 15, World Scientific, Sinagapore

Goda, Y. (2010) *“Random Seas and Design of Maritime Structures”*, 3rd Edition, World Scientific Publishing, ISBN-13: 978-981-4282-40-6, 708 pp.

Goda, Y., Suzuki, Y. (1976) "Estimation of Incident and Reflected Waves in Random Wave Experiments", Proceedings of 15th International Conference on Coastal Engineering, Honolulu, Hawaii, USA

Guler, H. G. (2013) *"A Comparative Study on Van der Meer (1988) and Van Gent et al (2003) Stability Formulae",* Proceedings of ASCE COPRI PORTS 2013, August 2013, Seattle, Washington, USA

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

Hughes, S. A. (1993) *"Physical Models and Laboratory Techniques in Coastal Engineering"*, Advanced Series on Ocean Engineering – Vol 7, World Scientific, Singapore

IPCC (2007) "Intergovernmental Panel on Climate Change (IPCC) Fourth Assessment Report: Climate Change 2007 Impacts, Adaptation and Vulnerability", M. L. Parry, O. F. Canziani, J. P. Palutikof, P. J. v. d. Linden, C. E. Hanson, eds., IPCC, Cambridge, United Kingdom and New York, NY, USA

Janssen, T. T., Battjes, J. A. (2007) *"A Note on Wave Energy Dissipation over Steep Beaches"*, Coastal Engineering, 54, 711-716

Jensen, J. (1985) *"Über Instationare Entwicklungen der Wasserstande an der Deutschen Nordseeküste"* Mitteilungen, Leichtweiss-Insitut der TU Breaunsschweig, Heft 88.

Kamphius, J. W. (1991) *"Physical Modeling"*, Handbook of Coastal and Ocean Engineering, J. B. Herbich, Ed., Vol 2, Gulf Publishing Company, Houstan, Texas, USA

Karlsson, T. (1969) *"Refraction of Continuous Ocean Wave Spectra"*, Proceedings of ASCE 95 (WW4), 471-490

Le Mehaute, B. (1990) *“Similitude”,* Ocean Engineering Science, B. Le Mehaute, Ed., Vol 9, Part B in the series The Sea, John Wiley and Sons, New York, pp 955-980.

Longuet-Higgins, M.S., Stewart, R. W. (1964) *“Radiation Stresses in Water Waves; A Physical Discussion with Applications”*, Deep-Sea Research 11: 529-562

METU OERC (2013), *“Maximum Water Level Computations due to Wind, Wave, Storm Surge and Tsunami Events at Adana Yumurtalık Thermal Power Plant Project Site”,* Technical Report, Ankara, Turkey

Price, W. A. (1978) *“Models – Can We Learn From the Past (theme speech)”*, Proceedings of the 16th Coastal Engineering Conference, ASCE, Vol 1

Reeve, D., Chadwick, A., Fleming, C. (2004) “*Coastal Engineering: Processes, Theory and Design Practice”*, Spon Press, Taylor&Francis Group, 461p.

Rock Manual (2007) *“The Use of Rock in Hydraulic Engineering”*, CIRIA-CUR-CETMEF (C683)

Shuisky, Y. D. (2000) *“Implications of Black Sea Level Rise in the Ukraine”* Proceeding of SURVAS Expert Workshop on European Vulnerability and Adaptation to Impacts of Accelerated Sea-Level Rise (ASLR), Hamburg, Germany, 19-21 June 2000, pp. 14-22

Tajima, Y., Madsen, O. S. (2003) *“Modelling Near Shore Waves and Surface Roller”* Proceedings of 2nd International Conference on Asian and Pacific Coasts (APAC 2003), Makuhari, Chiba, Japan, Paper No: 28

TAW (2002a) *“Wave Run-Up and Wave Overtopping at Dikes”*, Technical Report, Technical Advisory Committee on Flood Defense, Delft

Thompson, D. M. and Shuttler, R. M. (1975) *“Design of Riprap Slope Protection Against Wind Waves”*, HRS, Wallingford, CIRIA Report No:61, UK

Tsimplis, M. N., Josey, S. A., Rixen, M., Stanev, E. V (2004*) "On the Forcing of Sea Level in the Black Sea"*, Journal of Geophysical Research 109, C08015. doi: 10.1029/2003JC002185

Van der Meer, J. W. (1988) *"Rock Slopes and Gravel Beaches under Wave Attack"*, Ph. D. Thesis, Delft University, the Netherlands

Van der Meer, J. W. (1990) *"Extreme Shallow Water Conditions"*, Report H198, Delft Hydraulics, Delft

Van Gent, M. R. A., Smale, A. J., Kuiper, C. (2003*) "Stability of Rock Slopes with Shallow Foreshores"*, Proceedings of 4th International Coastal Structures Conference Portland, ASCE, Reston VA, USA

Walton, T. L., Dean, R. G. (2009) *"Landward Limit of Wind Setup on Beaches"*, Ocean Engineering, Vol 36, Issues 9-10, July 2009, Pages 763-766

Weaver, R. J., Slinn D. N. (2004) *"Effect of Wave Forces on Storm Surge"*, Coastal Engineering Journal, pp1532-1538

Wilson, B.W. (1965) *"Numerical Prediction of Ocean Waves in the North Atlantic for December 1959"*, Deut. Hydro. Zeit, Jahrg. 18, Ht. 3

Yalin, M. S. (1989) *"Fundamentals of Hydraulic Physical Modelling"*, Recent Advances in Hydraulic Physical Modelling, R. Martins, Ed., Kluwer Academic Publishers, Dordrecht, The Netherlands, pp 567-588

Yıldız, H., Demir, C. (2002) *"Mean Sea Level Changes and Vertical Crustal Movements at Turkish Tide Gauges for the period of 1984-2001"*, Workshop on Vertical Crustal Motion and Sea Level Change, 17-19 September 2002, Toulouse, France

APPENDIX A

EXTREME VALUE STATISTICS

Extreme value distributions that are used in extreme value statistics are given below (Goda, 2000). *f(x)* denotes the probability density function and *P(x)* defines cumulative distribution function, where x stands for the extreme variate(i.e. wave height, wind velocity).

1) Fisher-Tippet type I (FT-I) or Gumbel distribution:

$$P(x) = exp[-exp(-\frac{x-B}{A})] \qquad -\infty < x < \infty \qquad [A.1]$$

2) Fisher-Tippet type II (FT-II) or Frechet distribution:

$$P(x) = exp[-(1+\frac{x-B}{kA})^{-k}] \qquad B-kA < x < \infty \qquad [A.2]$$

3) Fisher-Tippet type III (FT-III) or Weibull distribution:

$$P(x) = 1 - exp[-(\frac{x-B}{A})^k] \qquad -B < x < \infty \qquad [A.3]$$

4) Log-normal distribution:

$$P(x) = \frac{1}{\sqrt{2\pi}Ax} exp[-\frac{(ln\,x - B)^2}{2A^2})] \qquad 0 < x < \infty \qquad [A.4]$$

As a general procedure for fitting an extreme value data set to one of the given distributions above, first of all, the data is ordered in descending order. After that, expected probability, *Pm*, is found using number or order (*m*) and total number of values in data set (*N*) in addition to plotting formula constants α and β as given in Equation A.5. Values of plotting formula constants for each distribution are given in Table A.1.

$$P_m = 1 - \frac{m-\alpha}{N+\beta} \qquad [A.5]$$

Table A1: Constants of plotting position formula, (Goda, 2000)

Distribution	α	β	Authors
FT-Ia	0	1	Gumbel, 1953
FT-Ib	0.44	0.12	Gringorten, 1963
FT-II	0.44+0.52/k	0.12-0.11/k	Goda, 1988; 1990
Weibull	$0.20+0.27/k^{0.5}$	$0.20+0.23/k^{0.5}$	Blom, 1958
Log-Normal	0.375	0.25	Blom, 1958

The shape parameters, *k*, given for FT-II distribution are assumed to be 2.5, 3.33, 5.0, 10.0 and for Weibull distribution; 0.75, 1.0, 1.4, and 2.0.

Methodology recommended by Goda (2000) is used to select best fitting distribution.