

EFFECTS OF ANTIFER PLACEMENT METHODS ON WAVE LOADS
ACTING ON CROWN WALLS IN SHALLOW WATER

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



BY
FURKAN DEMİR

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

AUGUST 2023

Approval of the thesis:

**EFFECTS OF ANTIFER PLACEMENT METHODS ON WAVE LOADS
ACTING ON CROWN WALLS IN SHALLOW WATER**

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

Prof. Dr. Halil Kalıpçılar
Dean, Graduate School of **Natural and Applied Sciences**

Prof. Dr. Erdem Canbay
Head of the Department, **Civil Engineering, METU**

Prof. Dr. Ahmet Cevdet Yalçın
Supervisor, **Civil Engineering, METU**

Dr. Hasan Gökhan Güler
Co-Supervisor, **Civil Engineering, METU**

Examining Committee Members:

Assist. Prof. Dr. Gülizar Özyurt Tarakcıoğlu
Civil Engineering, METU

Prof. Dr. Ahmet Cevdet Yalçın
Civil Engineering, METU

Assoc. Prof. Dr. Elif Oğuz
Civil Engineering, METU

Assist. Prof. Dr. Cüneyt Baykal
Civil Engineering, METU

Assist. Prof. Dr. Doğan Kısacık
Civil Engineering, IZTECH

Date: 21.08.2023



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

Name, Last name: Furkan Demir

Signature:

ABSTRACT

EFFECTS OF ANTIFER PLACEMENT METHODS ON WAVE LOADS ACTING ON CROWN WALLS IN SHALLOW WATER

Demir, Furkan
Master of Science, Civil Engineering
Supervisor: Prof. Dr. Ahmet Cevdet Yalçınır
Co-Supervisor: Dr. Hasan Gökhan Güler

August 2023, 120 pages

In this study, the changes in wave forces on the crown walls of rubble mound breakwaters due to different placement methods of antifer units (closed pyramid, irregular and double pyramid) in the armour layer of rubble mound breakwaters are investigated through physical model experiments. In the experiments, the forces acting on the crown wall of a conventional breakwater section were measured under the influence of different irregular wave series in shallow water conditions. The ratio (R_c/H_{m0}) between the crest height of the breakwater (R_c) and the spectral significant wave height (H_{m0}) was set to vary between 0.85 and 1.80 in accordance with the literature. In the experiments, different situations were considered, in which the crown wall was fully and partially protected by the antifer units. The experimental measurements show that different antifer placement methods significantly influence the wave pressure distribution and the wave forces on the crown wall. When the forces acting on the front face of the crown wall are analysed, it is found that the irregular placement of the antifer units reduces the wave forces by 30% compared to the closed pyramid placement of the antifer units and by 15% compared to the double

pyramid placement of the antifer units. As a result, the relevant empirical coefficients in the formulae commonly used for the design of the crown wall were updated based on the measurements performed in the existing physical model experiments and reconciled with the experimental measurements.

Keywords: Rubble-Mound Breakwater, Crown-Wall, Antifer Placement Methods, Wave Loads, Pressure



ÖZ

ANTİFER YERLEŐTİRME YÖNTEMLERİNİN KRONMAN DUVARLARA SIĞ SUDA ETKİ EDEN DALGA YÜKLERİ ÜZERİNDEKİ ETKİLERİ

Demir, Furkan
Yüksek Lisans, İnşaat Mühendisliğı
Tez Yöneticisi: Prof. Dr. Ahmet Cevdet Yalçınır
Ortak Tez Yöneticisi: Dr. Hasan Gökhan Güler

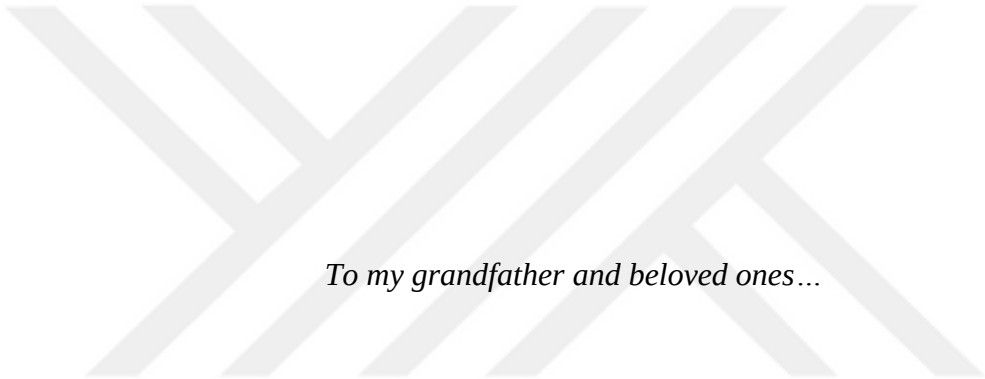
Ağustos 2023, 120 sayfa

Bu çalışmada taş dolgu dalgakıranların koruma tabakasında kullanılan antifer ünitelerin kapalı piramit, düzensiz ve ikili piramit dizilim metodları ile yerleştirilmesi sonucunda kronman duvarı üzerindeki dalga kuvvetlerinde oluşan değişimler fiziksel model deneyleri ile incelenmektedir. Deneylerde geleneksel bir dalgakıran kesiti üzerindeki kronman duvara etkiyen kuvvetler sığ su koşullarındaki farklı düzensiz dalga serileri etkisi altında ölçölmüştür. Deney programı dalgakıran kret kotunun (R_c) spektral belirgin dalga yüksekliğine (H_{m0}) oranı (R_c/H_{m0}) literatürle uyumlu olarak 0.85 ile 1.80 arasında değışecek şekilde belirlenmiştir. Deneylerde kronman duvarın antifer ünitelerce tamamen korunduğı ve bir kısmının korunduğı farklı durumlar da göz önüne alınmıştır. Deneysel ölçömler farklı antifer dizilim metodlarının kronman duvarı üzerindeki dalga basınç dağılımını ve dalga kuvvetlerini önemli miktarda etkilediğı gözlemlenmiştir. Kronman duvarının ön yüzüne etkiyen kuvvetler incelendiğinde düzensiz antifer diziliminin kapalı piramit antifer dizilimine göre dalga kuvvetlerini %30, ikili piramit dizilimine göre ise %15 mertebesinde azalttığı gözlemlenmiştir. Sonuç olarak kronman duvarı tasarımında yaygın olarak kullanılan formüllerdeki ilgili amprik katsayılar, mevcut fiziksel

model deneylerinde gerekleřtirilen lmler kullanılarak gncellenmiř ve deneysel lmlere uyumlandırılmıřtır.

Anahtar Kelimeler: Tař Dolgu Dalgakıranlar, Kronman Duvarı, Antifer Yerleřtirme Yntemi, Dalga Kuvvetleri, Basınc





To my grandfather and beloved ones...

ACKNOWLEDGEMENTS

I would like to express my deep thanks and gratitude to my advisor Prof. Dr. Ahmet Cevdet Yalçın, my co-advisor Dr. Hasan Gökhan Güler and Asst. Prof. Dr. Cüneyt Baykal, whom I can call my third advisor.

I would like to pay my respects to Prof. Dr. Ahmet Cevdet Yalçın, who supported my interest in coastal engineering from the first day he met me, who trusted me and provided me with opportunities, who included me in his projects and helped me develop in many areas.

I would like to thank Dr. Hasan Gökhan Güler with my most heartfelt feelings and express my great gratitude. He made the greatest contribution in my choice of coastal engineering, has been a very good advisor with patience, dedication and love while guiding a very curious and persistent person like me who constantly asked questions. I am also very grateful to him for the experiences he shared with me and for guiding me very well. While he was a very good advisor, he also taught me how to be a good instructor.

I would also like to thank Asst. Prof. Dr. Cüneyt Baykal, who trusted me in the projects in the laboratory, developed my engineering perspective and I enjoyed working with him very much. I had great pleasure to work with him.

I would like to extend my sincere thanks to Prof. Dr. Ayşen Ergin, to whom I pay my respects for her loving support and help in bringing our passion for coastal engineering to the highest levels. It was very enjoyable to work in her room as a group, brainstorming and producing something not only for coastal engineering but also for the laboratory.

I would like to express my special thanks to Dr. Işıkhan Güler for the different engineering perspectives he added to us, for emphasising the importance of being an

engineer who thinks about society and the environment, and for the engineering judgement skills he taught to me.

Asst. Prof. Dr. Gülizar Özyurt Tarakcıoğlu for the collaborative work she has done with us in the projects, for helping us and the projects to reach better points with the feedback she provided, and for always taking the time to answer my questions in detail.

I would also like to thank Assoc. Prof. Dr. Onur Pekcan. Although he was not a instructor in our laboratory, he helped me a lot by taking care of my thesis writing process and advising me about my career.

I would like to sincerely thank Dr. Gözde Güney Doğan for her trust and support in me. Since the first lesson I took in the laboratory, she has answered my questions in the sincerest way and taught me the quantities for preparing quality report and detailed project presentation.

I would like to thank Yusuf Korkut, Servet Şehirli and Eyüp Uğur, who have always been with me while doing my projects and thesis study. I was very lucky to have the chance to work with them. Whenever I called them and told them that I needed help, they always offered their support by helping me without hesitation. This thesis study could not have been completed without their valuable help. I would also like to extend my special thanks to Nuray Çimen Emre, who always gives me strength with her energy and support. I would also like to thank our laboratory staff Ayhan Karadeniz.

I would like to thank all my friends who worked day and night with me in the laboratory and with whom I had a lot of fun and made good friendships. I owe them all special thanks for the wonderful times I spent with them.

I would like to express my special thanks to Günay Gazaloğlu, who has taught me many things about experimental study since the first day I came to the laboratory, who has always helped me with his ideas whenever I needed help, and who helped

me day and night during my admission process to the UK. His friendship is very precious to me.

I would like to express my great gratitude and thanks to Berkay Erler and Cem Sevindik, who I can call my big brothers, with whom we experimented and worked hard day and night, and who were always with me in the decisions I made about my career and life. Whatever the subject, they have always been there for me and have been a great friend and big brother to me.

I would like to express my special thanks to Bilge Karakütük. We have been perhaps the most dynamic and hardworking duo in the laboratory's history. It was always a pleasure to work with her, complete our courses and spend time with her. We have always pushed each other forward. I would like to thank her for her valuable support and help in many studies. I'm grateful for her energy.

I would like to give special thanks to Barış Ufuk Şentürk and Mert Yaman. In addition to the great friendship we have, their contributions to my courses, experiences and life are enormous. They were always just a phone call away and I am grateful for the unconditional support I received from them.

Although we met a little late, I would like to thank Alperen Korkmaz, who I can call my first student and very close friend. He helped me a lot with his diligent help in the tight schedule of my experiments, always willing to learn, and always bringing new ideas. I would also like to extend my thanks to Çağatay Uysal and thank him for the close friendship, emotional support, and fun environment he provided me. I enjoyed every time I spent with him, and I will never forget the days I spent with this duo.

I would also like to thank Irmak Özdemir for her wonderful friendship and her positive energy. She motivated us with her smiling face and jokes every time she came to the laboratory.

I would like to thank my other friends Akdeniz İnce, Aslıhan Devran, Berkay Akyol, Doğaç Sayar, Elif Ayşe Öztürk, İremnaz Kösem, Setenay Özaslan, Utku Uzun, Yağız Arda Çiçek, with whom I spent time in the laboratory.

I would like to thank our department canteen, Fatma Şahin and Ali Şahin, Çiğdem Asıl and Haydar Asıl, who always fed us with their good food and chatted with us.

I would like to express my thanks and gratitude to my dear friends Akınalp Fidan, Alper Hacıoğlu, Atakan Öztürk and Utku Berk Görgülü from my undergraduate education. They have always offered me their support and love unconditionally.

I would like to thank my dear friend Furkan Yeşiler and dear Dr. Taner Zorbay for the wonderful time they spent with me and the value they added to me.

I would like to thank my dear mother Işıl Aktaş, my dear sister Beyza Demir and my grandmother Birsen Aktaş for their support, love, and all kinds of help. I really could not have achieved the success I have without them.

Finally, I would like to express my respect, love and gratitude to my beloved grandfather Burhan Aktaş, to whom I dedicate this thesis study, whom I have always seen as my hero, whose footsteps I followed as I was growing up. His contribution to my personal development and support in all my education is invaluable.

This thesis study is partially funded by the Scientific and Technological Research Council of Turkey (TÜBİTAK) under grant number 122M053.

TABLE OF CONTENTS

ABSTRACT	v
ÖZ	vii
ACKNOWLEDGEMENTS	x
TABLE OF CONTENTS	xiv
LIST OF TABLES	xvii
LIST OF FIGURES	xix
LIST OF ABBREVIATIONS	xxiii
LIST OF SYMBOLS	xxiv
CHAPTERS	
1 INTRODUCTION	1
2 LITERATURE REVIEW	7
2.1 General Information About Crown Walls and Their Applications in Coastal Engineering.....	7
2.2 Stability Criteria for the Crown Walls.....	12
2.3 Methods for Calculating Wave Loads Acting on Crown Walls.....	13
2.3.1 Günbak and Gökce (1984)	14
2.3.2 Jensen (1984) and Bradbury et al. (1988).....	16
2.3.3 Pedersen (1996) and Norgaard et al. (2013)	18
2.3.4 Martin et al. (1999)	26
2.3.5 Berenguer and Boanza (2006)	30
2.3.6 Molines (2016).....	32
2.4 Comparison of the Existing Empirical Wave Load Formulations of Crown Walls in the Literature	33

2.5	Experimental Studies Investigating the Performance of Antifer Units	35
2.6	Summary of Literature Review	36
3	METHODOLOGY	39
3.1	Physical Model Experiments	39
3.1.1	Experimental Setup	39
3.1.2	Design of the Section and Wave Conditions to be Tested in Physical Model Experiments	41
3.1.3	Design of the Crown Wall Model and Location of the Pressure Measurement Sensors	51
3.1.4	Experimental Programme.....	55
3.1.5	Analysis of the Pressure Recordings	64
4	ANALYSES OF THE EXPERIMENTAL MEASUREMENTS AND DISCUSSION OF RESULTS.....	73
4.1	Analyses of the $F_{h0.1\%}$ Measurements	73
4.1.1	Modification of Empirical Coefficients for Antifer Unit Placements.....	73
4.1.2	Effects of the Packing Density	84
4.1.3	Effects of Staggered Placement.....	85
4.1.4	General Observation on $F_{h0.1\%}$ for Antifer Units.....	86
4.2	Analyses of the $M_{h0.1\%}$ Measurements	87
4.2.1	Modification of Empirical Coefficients for Antifer Unit Placements.....	87
4.2.2	General Observation on $M_{h0.1\%}$ for Antifer Units.....	92
4.3	Analyses of the $P_{b0.1\%}$ Measurements	92
4.3.1	Modification of Empirical Coefficient for Antifer Unit Placement.....	92
4.3.2	General Observation on $P_{b0.1\%}$ for Antifer Units.....	102
4.4	Case Study for the Conventional Breakwater Design.....	103

5	CONCLUSIONS	105
	REFERENCES	109
APPENDICES		
A.	Wave Height Distributions and Energy Spectrums	115



LIST OF TABLES

TABLES

Table 2.1. Empirical coefficients a and b to calculate the wave loads acting on the crown walls for the sections A-E. Adopted from CIRIA et al. (2012).	17
Table 2.2. Investigated parameters and their variations in Pedersen (1996) study.	18
Table 2.3. Limitations to apply the (2.21) in wave load calculations (Pedersen, 1996).	22
Table 2.4. Structural dimensions of the model cross-section. Adopted from Norgaard et al. (2013).	24
Table 2.5. Wave conditions of the experimental study. Adopted from Norgaard et al. (2013).	25
Table 2.6. The range of application in wave load calculations (Berenguer and Baonza, 2006).	32
Table 3.1. Unit and material sizes in the breakwater model cross-section.	44
Table 3.2. Structural dimensions of the rubble mound breakwater model section.	45
Table 3.3. The range of target wave parameters used in the test series.	47
Table 3.4. The range of target wave parameters used in the test series.	48
Table 3.4. Experimental programme for the pressure measurements.	56
Table 3.5. Antifer placement methods	60
Table 4.1. Statistical assessment of horizontal wave force measurements data to obtain the best fit for the modified empirical coefficients.	75
Table 4.2. Statistical assessment of the moment measurements data to obtain the best fit for the modified empirical coefficients.	89
Table 4.3. Statistical assessment of base pressure measurements data to obtain the best fit for the modified empirical coefficient.	94
Table 4.4. The range of application in wave load calculations (Berenguer and Baonza, 2006).	103

Table 4.5. Wave pressure calculations of the case study according to the three
different design formulations. 104



LIST OF FIGURES

FIGURES

Figure 2.1. (a) Wave screen, (b) Concrete plain face base slab, (c) Conventional L-shape crown wall, (d) Curved fronted/bull nosed crown wall, (e) Crown wall with a key. Adapted from British Standards Institution (1991) and Pedersen (1996).	9
Figure 2.2. (a) Crown wall with a chute block insertion, redrawn from CIRIA et al., 2012. (b) Innovative crown wall structure with frontal reservoir for generating energy. Adapted from Iuppa et al. (2016).	11
Figure 2.3. Geometrical locations of crest freeboard, R_c and armour freeboard, A_c	14
Figure 2.4. (a) Definition of wave run-up height, (b) distribution of stagnation and hydrostatic pressures. Redrawn from Günbak and Ergin (1983).	15
Figure 2.5. (a) Assumed pressure distribution, (b) hypothetical run-up wedge and design parameters. Adapted from Pedersen (1996).	19
Figure 2.6. (a) Crown wall in which armour and crest freeboard are equal, (b) Crown wall in which crest freeboard is higher than armour freeboard. Adopted from Norgaard et al. (2013).	24
Figure 2.7. Example pressure distributions of dynamic and reflecting pressures. Adopted from Martin et al. (1999).	27
Figure 2.8. Dynamic and reflecting pressure distributions and geometrical parameters of the Martin et al. (1999) study. Adopted from Martin et al. (1999). ..	28
Figure 3.1. Layout of the 2D model test in the wave flume.	40
Figure 3.2. Video snapshots from (a) front view, (b) side view.	41
Figure 3.3. Close up view of the experimental setup and cross-section.	46
Figure 3.4. Non-dimensionalized histogram of wave heights [D-6 Irregular Wave Condition]	49
Figure 3.5. An example comparison of three different wave height distributions for the irregular wave series. [D-6 Irregular Wave Condition]	51
Figure 3.6. Crown wall model designed with the kestamite material.	52
Figure 3.7. Aluminium apparatus position on the vertical face of the crown wall.	53

Figure 3.8. Locations of the pressure sensors on the crown wall model. Cross-section view of the positions of the pressure sensors at (a) vertical front wall face, (b) horizontal base slab. Plan view of the positions of the pressure sensors at (c) horizontal base slab, (d) vertical front wall face.	54
Figure 3.9. Attachable apparatus of the 3 cm and 6 cm extension of the crown wall crest elevation.	55
Figure 3.10. (a) DP2A placement method configuration, (b) DP2B placement method configuration as an example for the arrangement in front of the pressure sensors.	57
Figure 3.11. Comparison of the maximum wave pressure of the DP2 antifer placement Method for the A and B arrangement type.....	58
Figure 3.12. Pressure sensor apparatus positions. (a) Vertical1 [V1], (b) Vertical2 [V2], (c) Horizontal1 [H1], (d) Horizontal2 [H2].	59
Figure 3.13. Results of the cross-correlation analysis between the pressure sensors.	65
Figure 3.14. Time dependent variation of the pressure obtained from one sensor during a single wave. Raw data is shown by blue line, processed data is shown by red line.	66
Figure 3.15. An example of average vertical base pressure measurements during the wave loading action at the crown wall base slab for the DP2A placement.	69
Figure 4.1. Comparison of Norgaard et al. (2013) and Pedersen (1996) formulation with all antifer placement method measurements. (<i>RMSE</i> and <i>MAE</i> values show the performance of present study with respect to Norgaard et al. (2013) formula with original coefficients.).....	77
Figure 4.2. Comparison of Norgaard et al. (2013) and Pedersen (1996) formulation with CP placement method measurements. (<i>RMSE</i> and <i>MAE</i> values show the performance of present study with respect to Norgaard et al. (2013) formula with original coefficients.).....	79
Figure 4.3. Comparison of Norgaard et al. (2013) and Pedersen (1996) formulation with IR placement method measurements. (<i>RMSE</i> and <i>MAE</i> values show the	

performance of present study with respect to Norgaard et al. (2013) formula with original coefficients.)	81
Figure 4.4. Comparison of Norgaard et al. (2013) and Pedersen (1996) formulation with DP placement method measurements. (<i>RMSE</i> and <i>MAE</i> values show the performance of present study with respect to Norgaard et al. (2013) formula with original coefficients.)	83
Figure 4.5. Comparison of $M_{h0.1\%}$ values for Pedersen (1996), Norgaard et al. (2013) and present study with the modified coefficients for all anti-fer placement method measurements. (<i>RMSE</i> and <i>MAE</i> values show the performance of present study with respect to Norgaard et al. (2013) formula with original coefficients.)..	91
Figure 4.6. Comparison of $P_{b0.1\%}$ Norgaard et al. (2013) formulation with all anti-fer placement method measurements. (<i>RMSE</i> and <i>MAE</i> values show the performance of present study with respect to Norgaard et al. (2013) formula with original coefficients.).....	96
Figure 4.7. Comparison of Norgaard et al. (2013) and Pedersen (1996) formulation with CP placement method measurements. (<i>RMSE</i> and <i>MAE</i> values show the performance of present study with respect to Norgaard et al. (2013) formula with original coefficients.)	98
Figure 4.8. Comparison of Norgaard et al. (2013) and Pedersen (1996) formulation with IR placement method measurements. (<i>RMSE</i> and <i>MAE</i> values show the performance of present study with respect to Norgaard et al. (2013) formula with original coefficients.)	100
Figure 4.9. Comparison of Norgaard et al. (2013) and Pedersen (1996) formulation with DP placement method measurements. (<i>RMSE</i> and <i>MAE</i> values show the performance of present study with respect to Norgaard et al. (2013) formula with original coefficients.)	101
Figure 4.10. Conventional breakwater design for the Project E.	103
Figure A.5.1. Wave height distribution comparison with Battjes and Groenendijk (2000), Rayleigh and Glukhovskii (1966) methods. [D-4 Irregular Wave Condition]	119

Figure A.5.2. Wave height distribution comparison with Battjes and Groenendijk (2000), Rayleigh and Glukhovskii (1966) methods. [D-5 Irregular Wave Condition] 120

Figure A.5.3. Wave height distribution comparison with Battjes and Groenendijk (2000), Rayleigh and Glukhovskii (1966) methods. [D-6 Irregular Wave Condition] 120



LIST OF ABBREVIATIONS

ABBREVIATIONS

ADV	Acoustic doppler velocimeter
CP	Closed pyramid placement
CVRMSE	Coefficient of variation of the root mean square error
DP	Double pyramid placement
IR	Irregular placement
MAE	Mean absolute error
NMAE	Normalized mean absolute error
OBREC	Overtopping breakwater energy conversion
RMSE	Root mean square error
PDF	Probability density function
SWL	Still water level
WG	Wave gauge

LIST OF SYMBOLS

SYMBOLS

α	Angle of breakwater armour slope
β	Angle of breakwater armour slope
γ	Peak enhancement factor for the JONSWAP spectrum
γ_β	Obliquity factor
γ_f	Roughness factor
γ_s	Packing density
γ_w	Specific gravity of water
θ	Angle of run-up wedge
λ	Empirical coefficient for the modification of the dynamic pressure
μ	Dimensionless parameter determined from monochromatic waves
v_0	Up-rushing velocity
ξ_0	Deep water surf similarity parameter
ξ_m	Local surf similarity parameter corresponding to mean wave period
$\xi_{m-1,0}$	Surf similarity parameter corresponding to spectral mean wave period
ξ_{m0}	Deep water surf similarity parameter corresponding to mean wave period
ξ_p	Local surf similarity parameter corresponding to peak wave period
ρ_s	Density of solid
ρ_w	Density of water

a	First empirical coefficient in wave load formulations
$a_{modified}$	Modified empirical coefficient of the $F_{h0.1\%}$ formulation of Norgaard et al. (2013)
A_c	Armour crest freeboard of structure
A_u	Empirical coefficient in Losada and Gimenez-Curto (1981) run-up formulation
b	Second empirical coefficient in wave load formulations
$b_{modified}$	Modified empirical coefficient of the $F_{h0.1\%}$ formulation of Norgaard et al. (2013)
B	Width of the armour crest
B_u	Empirical coefficient in Losada and Gimenez-Curto (1981) run-up formulation
c	Third empirical coefficient in Pedersen formulation
C_b	Length of the crown wall base
C_h	Height of the crown wall vertical face
d	Fourth empirical coefficient in Pedersen formulation
$d_{modified}$	Modified empirical coefficient of the $P_{b0.1\%}$ formulation of Pedersen (1996) and Norgaard et al. (2013)
D_{n50}	Nominal diameter of the breakwater units
e_1	Empirical coefficient in the Norgaard et al. (2013) formulation.
e_2	Empirical coefficient in the Norgaard et al. (2013) formulation.
f	Friction coefficient
f_c	Unprotected part of the crown wall front face

F_c	Foundation level of the crown wall
F_G	Weight of buoyancy-reduced crown wall structure
F_H	Wave-induced horizontal force on crown wall
$F_{h0.1\%}$	Horizontal wave force per meter acting on crown wall exceeded by 0.1% of the waves.
$F_{hl0.1\%}$	Horizontal wave force per meter acting on protected part of the crown wall exceeded by 0.1% of the waves.
$F_{hu0.1\%}$	Horizontal wave force per meter acting on unprotected part of the crown wall exceeded by 0.1% of the waves.
F_U	Wave-induced uplift force
g	Gravitational acceleration
G_c	Armour crest berm width
h_b	Length of the crown wall base
h_f	Height of the crown wall
h_{prot}	Protected part of the crown wall front face
H	Wave height
$H_{x\%}$	Wave height exceeded by x% of all wave heights
H_1, H_2	Characteristic normalized wave heights
$H_{1/3}$	Significant wave height
H_{m0}	Spectral significant wave height
H_{rms}	Root mean square wave height
H_s	Significant wave height

H_{tr}	Transition wave height in wave height distribution of Battjes and Groenendijk (2000)
K_D	Stability coefficient
L_0	Deep water wavelength
L_{0m}	Deep water wavelength corresponding to mean wave period
L_{0p}	Deep water wavelength corresponding to peak wave period
m_x	xth moment of the spectral density
$M_{0.1\%}$	Overturning moment exceeded by 0.1% of the waves
M_{50}	Median weight of the armour units in tons
M_G	Supporting moment caused by weight of crown wall
M_H	Moment caused by horizontal forces
M_U	Moment caused by wave-induced uplift forces
p_h	Hydrostatic wave pressure at crown wall face
p_m	Stagnation pressure at crown wall face due to wave impact
p_u	Uplift wave pressure at crown wall base
$P_{b0.1\%}$	Crown wall base pressure exceeded by 0.1% of the waves
P_d	Modified dynamic pressure acting on the vertical face of the crown wall in Martin et al. (1999) formulation
P_r	Reflecting pressure acting on the vertical face of the crown wall in Martin et al. (1999) formulation
P_{s0}	Dynamic pressure acting on the vertical face of the crown wall in Martin et al. (1999) formulation
R_c	Crest freeboard of structure

R_{ca}	Armour crest freeboard of structure
R_u	Wave run-up level
$R_{ux\%}$	Wave run-up level exceeded by x% of incident waves
$s_{m-1,0}$	Local wave steepness corresponding to spectral mean wave period
s_{om}	Deep water wave steepness corresponding to mean wave period
s_{op}	Deep water wave steepness corresponding to peak wave period
S_0	Geometrical parameter in Martin et al. (1999) formulation
T_{01}	Mean wave period defined by m_0/m_1
T_{02}	Mean wave period defined by $\sqrt{m_0/m_2}$
T_m	Mean wave period from time-domain analysis
$T_{m-1,0}$	Spectral mean wave period defined by m_{-1}/m_0
T_s	Significant wave period
T_p	Spectral peak wave period
V	Pedersen reduction factor for wave impact
V_1, V_2	Volumes in front of the crown wall for the Pedersen formulation
$w_{predicted,i}$	Expected value based on the analysis
$w_{true,i}$	Actual value obtained by the measurement
W_c	Foundation level of the crown wall
y	Vertical run-up wedge thickness
y_{eff}	Effective wave pressure impact zone height

CHAPTER 1

INTRODUCTION

The social order that emerged with the settled life of humanity started to use the positive benefits of living in coastal areas in economic, commercial, and cultural terms. For this reason, settlements established in coastal areas have gained importance in every age and have provided great returns to developing societies. Consequently, around 3 billion individuals in today's world, which accounts for roughly fifty percent of the global population, inhabit regions within a distance of 200 kilometers from a shoreline (Creel, 2003). Coastal communities have preferred to live on naturally sheltered coasts to be protected from storms, marine hazards, and other possible marine-induced negative factors in order to get the highest efficiency from the region they live in. Due to the crowdedness of the coastal zones and the increase in the usage areas, it has become difficult to find naturally sheltered areas, which has pushed people to build their own coastal protection structures.

Looking at the history and distribution of coastal structures today, rubble mound breakwaters are one of the most prevalent structures frequently built by coastal communities. Rubble mound breakwaters, which were first built by the Phoenicians about 3000 years ago in the city of Atlit in Israel (Haggi, 2010), are fixed structures designed by coastal engineers with more modern techniques nowadays to create a sheltered area against the wind waves, tides, storm surges, and currents in coastal regions. These structures provide favourable conditions for ships and small vessels berthed and accommodated in the harbour. They also contribute to navigation for marine vessels by providing to the avoidance of sediment accumulation in the harbour. Rubble mound breakwaters are designed as porous structures consisting of natural or artificial units. With the help of this porous environment, the wave energy that threatens the protected region is absorbed and the wave effects inside the

sheltered area is reduced. The rubble mound breakwaters primarily consist of five components: main (seaward side) and secondary (landward side) armour layer, filter layer, core, crest (commonly together with a crown wall), and toe structure. Among these components, the importance of the crest for harbour operations and breakwater stability is considerably high. The breakwater crest should be wide enough and protected against waves for cranes, truck traffic, and various human activities during the construction process and the harbour operation phases (Jensen, 1984). Excessive wave overtopping leads to failures such as sliding of the inner slope, undesirable swash or inundation in the inner zone, and any problems that may be caused by water that cannot be discharged inside the harbour area. As a consequence, it is crucial to minimise the amount of wave overtopping and to be resistant to wave-induced forces along the crest to provide inner (i.e., rear armour) stability of the breakwater after the completion of the construction. For this purpose, structurally resistant and large superstructures are commonly constructed on the crest while constructing the rubble mound breakwater. These superstructures, named crown walls, located on the crest are cast-in-place mass concrete or reinforced concrete parts of mound breakwaters. Rubble mound breakwaters with crown walls are widely used in Turkey and worldwide. The main reasons for the widespread use of crown walls are to reduce wave overtopping, to increase the stability of the inner armour layer slope, and to ensure that the rear side of the breakwater structure can also be used efficiently for harbour operation purposes.

Due to the above-mentioned reasons, the construction of the crown walls in rubble mound breakwaters has significant importance especially for practitioners. Therefore, many studies in coastal engineering literature have focussed on this subject. Correspondingly, many theoretical design formulae have been presented to be included in various design manuals. To develop these empirical formulae, several different physical model experimental campaigns were carried out and inferences were made about the design of the crown walls by means of various parameters. These parameters significantly depend on serviceability which is one of the most important design criteria along with stability in the design of various types of

breakwater structures including rubble mound, vertical wall, or caisson-type composite breakwaters. Today, the serviceability of breakwaters is determined in accordance with wave overtopping. The amount of wave overtopping is affected by the breakwater crest elevation, armour layer properties, wave run-up height. significant effect on the forces acting on the crown wall. There are many experimental formulations in design manuals in Turkey and worldwide (e.g., EurOtop (2018), CIRIA et al. (2007) and AYGM (2016)) for the prediction of wave forces on the crown wall, which depend on many different variables such as the parameters mentioned above.

The most important pioneering studies in the derivation of these empirical formulae were carried out by Günbak and Gökce (1984), Jensen (1984), Pedersen (1996) and Martin et al. (1999) by relating wave pressures mainly to wave run-up height with a chance of being exceeded by 0.1% of the waves. In these studies, the wave forces acting on the crown walls were investigated experimentally and the pressure distribution on the superstructure was analysed by various formulations. As a result of the obtained results, the formulations are presented to the literature with experimental coefficients depending on various parameters. These studies have provided valuable knowledge to the literature on the design of crown walls in coastal engineering, specifically the formulation presented by Pedersen (1996) has been included in many design specifications such as CIRIA et al. (2007) and AYGM (2016). However, the effects of antifer armour units and its different placement methods on the wave forces acting on the crown-walls has not been studied widely in the literature. Moreover, as mentioned before, it is a general fact that the pressure distribution on the crown wall changes depending primarily on the wave run-up (EurOtop, 2018). In the studies conducted by Bruce et al. (2009), also included in the well-established CLASH database in the literature, different amounts of wave overtopping caused by different armour units in the breakwater section were analysed in detail. In the wave overtopping formula presented as a result of these studies, the performances of different units are associated with a coefficient called roughness factor (EurOtop, 2018). However, roughness factor parameter and the

design formulae of the crown-walls are not related in the physical modelling studies of crown-walls. Therefore, crown wall design formulas cannot fully reflect the characteristics of the different units of the armour layer of the breakwater for which it is designed. Among various armour layer unit types, no comprehensive study or comment has been made on the antifer units that are frequently used in the design process. Due to this deficiency, the horizontal wave forces acting on a crown wall to be calculated in the design process may be overestimated or underestimated when deriving empirical formulations. Consequently, during and after the design process, uneconomical and non-feasible design of the section may occur, or the breakwater section will be more susceptible to unexpected failures.

The aim of the physical modelling experiments conducted in this research study is to observe the effect of various antifer armour unit placement methods with different packing densities on wave-induced forces and pressure distribution on crown walls. With the data set and results obtained from these physical modelling experiments, the empirical coefficients belonging to the crown wall design formulas presented in the literature for shallow water condition will be calibrated for antifer units. It is also desired that this study will contribute to future numerical modelling simulations for the rubble mound breakwater crown wall design.

The main research questions expected to be answered within the scope of this study can be stated as follows depending on the research aim stated above:

- i. How will the formulations of Pedersen (1996) and Norgaard et al. (2013), developed for scenarios where the armour layer is mostly designed with natural stones, perform for antifer units?
- ii. What is the effect of antifer placement methods used in the armour layer of rubble mound breakwaters on the forces acting on the crown wall?
- iii. How does the packing density of the antifer units affect the development of wave forces acting on the crown wall?

Targeting to answer above-listed research questions following a logical sequence, the present thesis study was organized as follows:

In Chapter 2, the literature review is presented in detail. This chapter is divided into two main sections. In the first section, the empirical formulae used in design manuals today and the parameters on which these formulae are based on are discussed. In the second section, pressure measurements and analyses studies in the literature are discussed in detail.

Chapter 3 outlines the methodology of the physical model experiments utilized in this study. In this chapter, the experimental setup and the preparation of the setup, the process of taking measurements and the analysis phase, the scaling and dimensional analysis process, the wave sets, and the experimental cases are described.

Chapter 4 presents the pressure measurement results and analysis of the experimental data of the different antifer placement methods for the six different irregular wave sets. The results of the pressure distribution obtained as a result of the measurements and the wave-induced forces and the moments generated by these forces are presented, and the modification of the empirical coefficients based on these results is shown.

Chapter 5 summarises and concludes the study and the results of the physical modelling experiment. Contributions of this study and future remarks are also mentioned in this chapter.



CHAPTER 2

LITERATURE REVIEW

In this chapter of the thesis, various information in the literature about the crown walls and their applications in coastal engineering practice will be given in general, then experimental modelling studies in this field and the formulas obtained as a result of these studies will be discussed in detail, considering their advantages and disadvantages. In addition, the fundamentals, measurement techniques and parameters considered during the analysis of the wave forces acting on the crown walls with pressure sensors in the literature will be investigated.

2.1 General Information About Crown Walls and Their Applications in Coastal Engineering

Crown walls are common structures that are concrete superstructures in the form of crowns or parapets at the crest of rubble mound breakwaters (Pedersen & Burcharth, 1992). These structures are also called wave walls, wave parapets or wave screens in the literature (Martin et al., 1999). The main function of these structures is to help breakwaters reduce the amount of wave overtopping that occurs under wave action (Pereira et al., 2021). In addition, it is involved in managing transportation, maintenance, and construction operations on the crest in port activities. In common practice, berths are built immediately to the rear of the crown walls, resulting in cargo activities, port operations, conveyor lines, electrical and water supply connections being made on the horizontal base slab of the crown wall (CIRIA et al., 2007) [12]. Crown walls are placed on the crest of the breakwaters, i.e., on the upper layer of the breakwaters, providing fewer financial expenses to coastal engineers in practical application by limiting the usage of excessive amount of filter material and armour units to increase the crest freeboard level and the slope length of the breakwater

section. Manufactured in the form of mass concrete, crown walls allow the crest level of the breakwater to be easily raised, eliminating the need for more quarry stone and space that would be required if the same height were to be reached with the continuation of the armour slope. For the aforementioned reason, the use of crown walls also has a positive environmental impact by enabling the construction of shorter armour layers, filter layers and breakwater slopes (Berenguer & Baonza, 2006). Crown walls, due to their structure, weight, and size, give the breakwaters a great deal of structural stability. In addition, due to the frictional force generated at its base slab, it can work together with the armour layer of the breakwater as a whole against the forces generated by wave action (Gokce & Gunbak, 1984).

From a structural point of view, in coastal engineering breakwater construction applications, many different crown wall types can be used on the crest for different purposes. The structural design of the crown wall is shaped mainly for operational and stability purposes. The different crown wall sections have been historically developed in line with the observed failure modes and the new requirements needed. Various configurations of crown walls used in coastal engineering are shown in Figure 2.1.

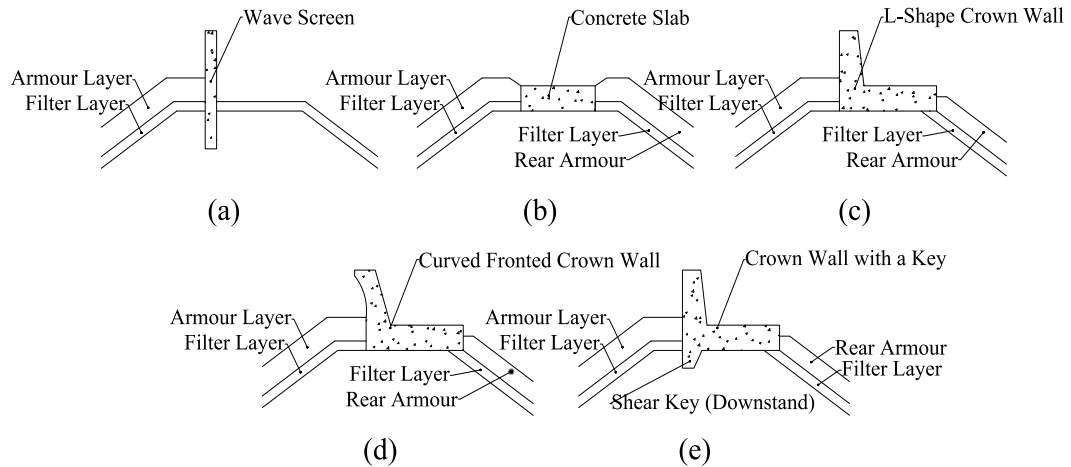


Figure 2.1. (a) Wave screen, (b) Concrete plain face base slab, (c) Conventional L-shape crown wall, (d) Curved fronted/bull nosed crown wall, (e) Crown wall with a key. Adapted from British Standards Institution (1991) and Pedersen (1996).

For example, a simple crown wall or wave screen, although not widely used today, is shown in Figure 2.1a with the logic of a typical retaining wall. The aim of this structure is to minimise the amount of wave overtopping discharge by simply raising the crest elevation (Pedersen, 1996). The second type of crown wall structure is the concrete slab placed on the crest for repair and maintenance works, construction activities to be carried out on the crest, pedestrian, and vehicle traffic requirements (British Standards Institution, 1991). This structure, shown in Figure 2.1b, consists of mass concrete and serves as a typical road. In the following stages, these two structures were combined to form the conventional L-shape crown wall structure shown in Figure 2.1c, which is still frequently used today. One of the biggest benefits of the L-shape design is that the amount of wave overtopping can be reduced together with providing the large crest flatness required for breakwater port operations. Another advantage of this structure shown in Figure 2.1c compared to Figure 2.1a and Figure 2.1b is that it can resist wave forces more stably. Especially compared to Figure 2.1a, it is more resistant to overturning moment failures. At later stages in the design of the crown wall, coastal engineers considered various solutions for the discharge of upcoming water towards the crest following the wave run-up. The most

used of these solutions is to design the vertical part of the crown wall as a curved shape structure instead of a straight wall. An example design of this structure is shown in Figure 2.1d. This design is also frequently used in vertical walls and caisson type composite breakwater structures. In some crown walls, a bullnose shaped parapet is also used instead of a recurved front face design, thus horizontal forces and wave overtopping ratio can be reduced (Molines et al., 2020). Wave forces on the crown wall due to wave action frequently cause sliding failure in addition to failure due to overturning moment. As a solution to this problem, coastal engineers have added a shear key at the foundation of the crown wall to ensure that the crown wall penetrates the layer on which it sits. With this design method shown in Figure 2.1e, the crown wall is anchored to the breakwater. Vos-Rovers et al. (2018) developed this idea and worked on reducing the magnitude of the overturning moment by extending the base slab of the crown wall towards the main armour layer.

Two of the problems arising from climate change are sea level rise and increased frequency of marine storms, which is expected to cause a large increase in wave overtopping in the future. In general, increased wave overtopping ratio will lead to increased wave-induced forces on the crown walls, more damage to the rear side armour and consequent damage or failure of the crown wall units. For this reason, coastal engineers have started various rehabilitation works on the already designed breakwaters and crown walls. One of the rehabilitation practices developed is the placement of castellated chute blocks or storm walls at the base of the crown wall (Kisacik et al., 2019). This reduces the energy of the overtopping jet and maintains the stability of the rear side armour (CIRIA et al., 2012). On the other hand, researchers have been working on various designs for the discharge of excess water encountered because of wave overtopping on the crown wall depending on the stated problem. The type of crown wall using chute blocks is shown in Figure 2.2a which contributes to energy dissipation by disturbing the overtopping jet flow caused by wave overtopping and protects the rear side layer by blocking the flow direction. Nowadays, for engineers focusing on sustainable structures and clean energy production, it is a trending topic to develop such an effective design of the crown

wall for wave energy generation. These structures are called Overtopping Breakwater Energy Conversion system, known as OBREC. As can be seen from the structure observed in Figure 2.2b, a reservoir and drainage system are designed in front of the vertical front face of the crown wall to drain excessive water. Pioneering studies in literature on generating energy from the water discharged through this reservoir are conducted by (Iuppa et al., 2016), (Kralli et al., 2019), (Contestabile et al., 2016), (Vicinenza et al., 2014).

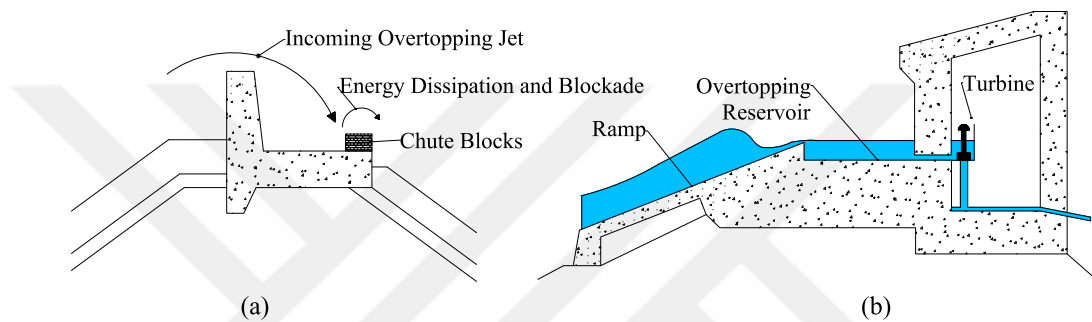


Figure 2.2. (a) Crown wall with a chute block insertion, redrawn from CIRIA et al., 2012. (b) Innovative crown wall structure with frontal reservoir for generating energy. Adapted from Iuppa et al. (2016).

For the design of the crown walls, there are various obligations specified in the technical specifications and design manuals. In particular, the crest elevation at which the base of the crown wall sits should be at least a meter above the high-water level to provide access for many land-based vehicles during construction (CIRIA et al., 2012). In addition, while determining the crest elevation, the settlements that will occur after construction and the future sea level rise due to climate change must be taken into consideration (CIRIA et al., 2012). CIRIA et al. (2012) also states that the construction of the crown should provide a minimum width of two meters for pedestrians and four meters for vehicles to provide the necessary traffic and access for port operations. In addition, the width of the crest on which the crown wall sits during the construction process should provide an area of two lorry widths or one crane-one lorry width (CIRIA et al., 2012). In addition, as a recommendation for

armour units placed in front of the crown, many specifications recommend a crest width design of $3D_{n50}$ (CEM, 2006; CIRIA et al., 2012; EurotOp, 2018).

2.2 Stability Criteria for the Crown Walls

The wave-induced horizontal forces and uplift pressure acting on the crown wall depend on the incident wave conditions, crown wall geometry and dimensions, armour layer parameters (armour slope, unit placement and type, layer thickness etc.), friction at the crown wall base and breakwater crest structure (CIRIA et al., 2012). Crown walls have four basic failure modes: overturning, sliding, cracking and geotechnical failure (Pedersen, 1996). However, when analysing the stability of these structures, the main focus is on overturning and sliding failure modes because the other two failures are significantly dependent on these ones. The stability criterion against sliding failure is shown in Eq. (2.1). In this formula, F_G is the weight of the buoyancy-reduced crown wall structure, F_U is the wave-induced uplift force, F_H is the wave-induced horizontal force on the crown wall, f is the friction factor. The magnitude of the f (-) is generally taken as around 0.5 (CIRIA et al., 2012).

$$f(F_G - F_U) \geq F_H \quad (2.1)$$

The stability criterion against overturning failure is shown in Eq. (2.2). In this formula, M_G represents the moment that supports the structure against the overturning moment caused by the weight of the crown wall, M_U represents the moment caused by the wave-induced uplift forces, and M_H represents the moment caused by the horizontal forces caused by the wave.

$$(M_G - M_U) \geq M_H \quad (2.2)$$

2.3 Methods for Calculating Wave Loads Acting on Crown Walls

In the literature, many experimental studies have been carried out and various formulations have been proposed to estimate the wave-induced forces acting on the crown wall. In this part of the literature review chapter, these experiments and formulations will be discussed. A quick summary of the various terms should be given before moving on to formulae and necessary discussions.

There are various parameters that indicate important levels during the design phase. One of them is the R_u parameter called run-up level, which is measured as the vertical distance from the still water level to the location that broken waves are able to reach through the armour layer. Depending on the incoming wave conditions, various run-up levels are used by coastal engineers utilizing statistical descriptions. $R_{u2\%}$ is used as wave run-up height in wave run-up calculations and design formulae developed accordingly (EurotOp, 2018). $R_{u2\%}$ represents the wave run-up level exceeded by 2% of incident waves. For the wave load calculations at the crown walls, on the other hand, 0.1% exceedance probability of incident wave heights is used when determining the wave run-up height. This fictitious wave run-up height level, $R_{u0.1\%}$ used in the calculation of wave forces was mentioned for the first time in Ergin and Günbak (1983), and then it was studied in detail by Pedersen (1996) and continued to be used. In the calculation of wave-induced pressures and forces on the crown wall, it is important to know the crest and armour freeboard levels as wave run-up height is significantly effective phenomena for the force distribution development. Crest freeboard, R_c , represents the crest height of the breakwater structure relative to the still water level. This level represents the elevation at which the incoming wave cannot flow back to the seaward side during the overtopping movement (EurotOp, 2018). Armour freeboard, A_c , is the highest horizontal flatness reached by the main armour layer of the breakwater relative to the still water level (EurotOp, 2018). R_c and A_c can be different or equal depending on the design. In each different case, the wave force distribution on the crown wall also varies. An example drawing of the parameters described in this paragraph is shown in Figure 2.3.

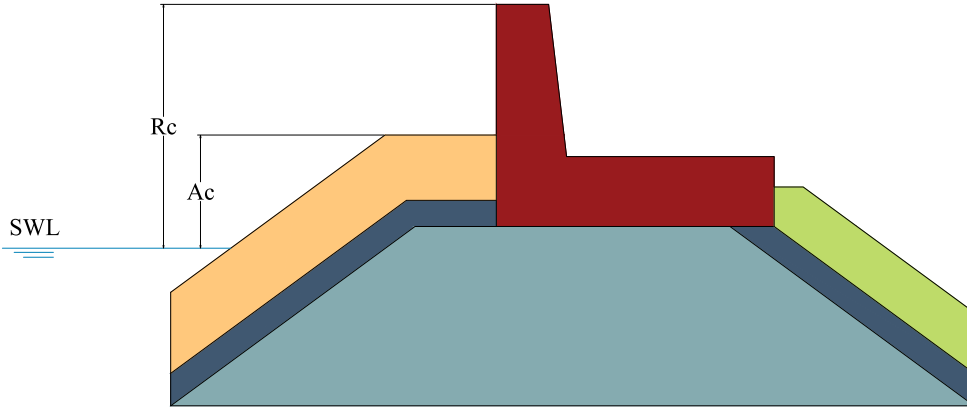


Figure 2.3. Geometrical locations of crest freeboard, R_c and armour freeboard, A_c .

The leading studies, which are also mentioned in the design specifications and pioneered other studies, are Günbak and Gökce (1984), Jensen (1984) and Bradbury et al. (1988), Pedersen (1996) and Martin et al. (1999). There are also additional studies that carry these studies further in specific cases. In the next section of the chapter, these formulations and studies will be mentioned.

2.3.1 Günbak and Gökce (1984)

In the study of Günbak and Ergin (1983) on the repair and observation of damage on the Antalya breakwater, $R_{u0.1\%}$ was used in the calculation of the wave pressures acting on the crown wall by considering a fictive wave run-up height concept. This study was then continued by Günbak and Gökce (1984) to derive a semi empirical formulation of the wave forces acting on the crown walls. Consequently, the derivation of these formulations is based on run-up calculations in the effect of incidence of regular waves. In this study, the authors mentioned two different types of pressure: stagnation and hydrostatic. When calculating the pressure distribution, the sum of stagnation pressure, p_m , and hydrostatic pressure, p_h , is considered. The definition of pressures and wave run-up height is shown in Figure 2.4.

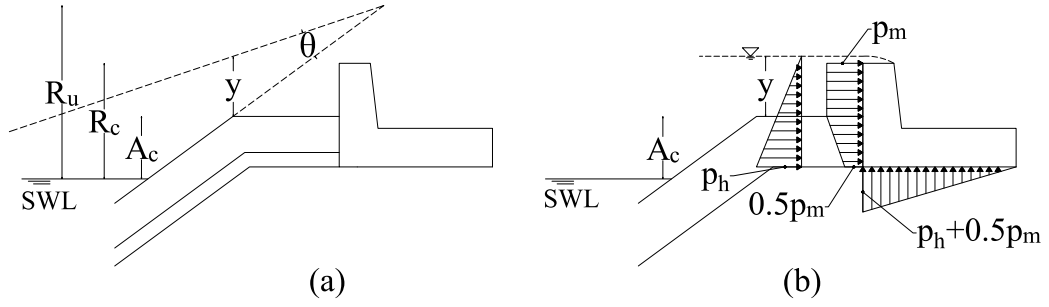


Figure 2.4. (a) Definition of wave run-up height, (b) distribution of stagnation and hydrostatic pressures. Redrawn from Günbak and Ergin (1983).

Figure 2.4a shows the run-up calculation assumptions used in finding the values of p_m and p_h pressures. The equations used in finding R_u are given in Eq. (2.3).

$$R_u = \begin{cases} 0.4 \xi H, & \xi < 2.5 \\ H, & \xi \geq 2.5 \end{cases} \quad (2.3)$$

As mentioned, the authors used $R_{u0.1\%}$ as a reference in their calculations. The angle of the run-up wedge is assumed to be 15 degrees. As can be seen in Figure 2.4.b, the wave stagnation pressure is p_m from the crest freeboard to the armour freeboard and $0.5p_m$ in the protected area along the armour layer. At the base of the crown wall, a triangular pressure distribution is assumed. Maximum pressure at the seaward side of the crown wall is assumed to be equal to the sum of the hydrostatic and stagnation pressures at that point and equal to zero at the harbour side base of the wall. Wedge thickness, y , is determined as shown in Eq. (2.4). where α shows armour layer slope.

$$y = \frac{(R_u - A_c)}{\sin \alpha} \cdot \frac{\sin \theta}{\cos(\alpha - \theta)} \quad (2.4)$$

Stagnation pressure, p_m , can be calculated as shown in Eq. (2.5). One can also determine hydrostatic pressure, p_h , as described in Eq. (2.6).

$$p_m = g \rho_w \cdot \frac{v_0^2}{2g} = g \rho_w \cdot \frac{y}{2} \quad \text{where } v_0 = \sqrt{gy} \quad (2.5)$$

$$p_h = g \rho_w \cdot (y + s) \quad (2.6)$$

v_0 shows the up-rushing velocity of the incident wave in Eq. (2.5). In Eq. (2.6), s represents the portion of the protected height of the crown wall front face.

2.3.2 Jensen (1984) and Bradbury et al. (1988)

Jensen (1984) conducted physical modelling experiments on 3 different sections and presented various results about the horizontal wave forces acting on the crown wall. Jensen (1984) conducted physical experiments utilizing irregular wave conditions for perpendicular waves. All the tests lasted at least 1000 or more waves duration. Dynamometer was used to measure horizontal forces by suspending wall model into this device. Although a precise formulation was not given, dimensionless analysis was performed on the horizontal force, F_H , and it was related to the significant wave height, H_s . As a result, a linear relationship between two dimensionless parameters was made by adding two empirical coefficients a and b . Dimensionless parameters are shown in Eq. (2.7).

$$\frac{F_H}{\gamma_w h_f h_b L_{op}} \propto \frac{H_s}{\Delta h} \quad (2.7)$$

In Eq. (2.7), F_H is the maximum horizontal force per meter of the crown wall face for 1000 waves. L_p represents the wavelength calculated with the peak period. γ_w is the specific weight of water in N/m^3 . The two parameters h_f and h_b in the denominator are respectively height and width of the wall in meters. Jensen (1984) concluded the model study with discussing influence of water level, wave period, angle of incidence and damage conditions. The dimensionless empirical relationship with two empirical coefficients a and b was also presented in this study and shown in Eq. (2.8).

$$\frac{F_H}{\gamma_w h_f L_{op}} = a + b \left(\frac{H_s}{A_c} \right) \quad (2.8)$$

Bradbury et al. (1988) established a wider experimental schedule. In this study, the crest elevation of the crown wall was tested at 4 different levels and a wider range of wave parameters and armour properties were scanned by performing the

experiments with an impermeable core. It was stated that both high frequency impact pressures and quasi-hydrostatic inertial pressures could be measured precisely with pressure transducers unlike Jensen (1984). It was also indicated that the non-dimensional parameters obtained in Jensen's study may be insufficient in some cases and that the effect of the wave period on the forces on the crown wall cannot be adequately reflected. Bradbury et al. (1988) fit the results of the physical experiments to the findings of Jensen's study by means of armour coefficients to better illustrate the armour related effects, which was not fully covered in Jensen (1984). Various armour coefficients were presented with the term defined as impact ratio and it was stated that Jensen's method could be extended for different crest geometries with these coefficients.

Although these studies were very valuable and pioneering studies in terms of determining the forces acting on the crown walls, they had some shortcomings because of not giving a clear formulation due to the poor instrumentation and methods were still developing at that time. Therefore, Burcharth (1993) extended the work of Jensen (1984) and Bradbury et al. (1988) and determined the empirical coefficients reported in these studies for five different cross-sections. The values of the coefficients from CIRIA et al. (2012) are shown in Table 2.1.

Table 2.1. Empirical coefficients a and b to calculate the wave loads acting on the crown walls for the sections A-E. Adopted from CIRIA et al. (2012).

Cross-sections	Parameters ranges in tests			0.1% exceedance values for empirical coefficients	
	R_{ca}	$s_{op} = H_s/L_{op}$	H_s/R_{ca}	A	b
A	5.60-10.60	0.016-0.036	0.76-2.50	0.051	0.026
B	1.50-3.00	0.005-0.011	0.82-2.40	0.025	0.016
C	0.10	0.023-0.070	0.90-2.10	0.043	0.038
D	0.14	0.040-0.050	1.43	0.028	0.025
E	0.18	0.040-0.050	1.11	0.011	0.010

As a result of these studies, the authors concluded that the base pressure has a linear, triangular distribution with a maximum at the heel of the crown wall on the seaward side and zero on the harbour side, as in Günbak and Gökce (1984).

2.3.3 Pedersen (1996) and Norgaard et al. (2013)

Pedersen (1996) has carried forward and slightly modified the run-up dependent wave load calculation method proposed by Günbak and Ergin (1983) and introduced semi empirical formulations to calculate horizontal and vertical wave forces and overturning moments. A total of 373 tests were completed with irregular wave series. Wave series lengths were set to be for a minimum of 5000 wave durations. Philips P13-OEM pressure transducers were used in the crown wall model made of aluminium plate. The study consisted of 12 distinct sections examined, with various combinations of berm width, armour slope, and breakwater crest height. In these tests, several armour layer components, including rocks, dolos, and cubes, were also used. The observed parameter variation for the experiments of Pedersen (1996), which investigated a wide spectrum of parameters, is given in Table 2.2.

Table 2.2. Investigated parameters and their variations in Pedersen (1996) study.

Parameter	Range
Significant Wave Height, H_s (in m)	0.10 – 0.18
Peak wave period, T_p (in s)	1.20 – 2.20
Water depth, h (in m)	0.51 – 0.59
Height of crown wall face, h_f (in m)	0.15 – 0.33
Armour crest freeboard, A_c (in m)	0.11 – 0.19
Vertical distance from SWL to top of crown wall, R_c (in m)	0.11 – 0.37
Width of the armour crest berm, B (in m)	0.18 – 0.36
Armour slope, $cot\alpha$	1.50 – 3.50
Ratio	Range
ζ_m	1.1 – 5.1
H_s/A_c	0.5 – 1.7
R_c/A_c	1.0 – 2.6
A_c/B	0.3 – 1.1

A plot of the various design parameters and the assumed pressure distribution determined by Pedersen (1996) is given in Figure 2.5a.

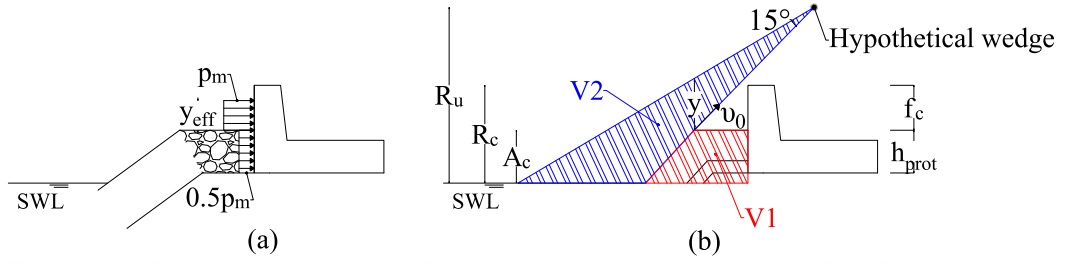


Figure 2.5. (a) Assumed pressure distribution, (b) hypothetical run-up wedge and design parameters. Adapted from Pedersen (1996).

While developing the method, Pedersen (1996), like G n bak and Ergin (1983), focused on two different types of pressure, namely impact and hydrostatic pressures. It was stated that impact pressures dominate the wave forces to be obtained in this case since an exceedance estimate of 0.1% is used when deciding the maximum forces and moments for all different types of crown walls and sections. In this case, it was indicated that the contribution of the hydrostatic pressure head can be neglected because it is too small. As applied in G n bak and Ergin (1983), impact pressures can be measured as stagnation pressure and the up-rush velocity at the armour crest is used to determine this. This velocity refers to the velocity at which the incident wave hits the wall face perpendicularly after rising along the armour (Pedersen, 1996). After deciding on $R_{u0.1\%}$, assuming no energy loss, the up-rush velocity, v_0 is found as in Eq. (2.9).

$$v_0 = \sqrt{2g(R_u - A_c)} \quad (2.9)$$

Depending on the found v_0 , the stagnation pressure, p_m , is calculated as in the Eq. (2.10).

$$p_m = \frac{g\rho_w v_0^2}{2g} \quad (2.10)$$

Pedersen used the method presented in the literature by Van der Meer and Stam (1992) to calculate $R_{u0.1\%}$ values. Van der Meer and Stam (1992) run-up formula shown in Eq. (2.11) is valid for deep water conditions and Pedersen used significant wave height, $H_{1/3}$, and mean period T_m .

$$R_{u0.1\%} = \begin{cases} 1.12H_{1/3}\xi_{m0} & \xi_{m0} \leq 1.5 \\ 1.34H_{1/3}\xi_{m0}^{0.55} & \xi_{m0} > 1.5 \end{cases} \quad (2.11)$$

For the surf similarity parameter ξ_{m0} , following (2.12) can be used,

$$\xi_{m0} = \frac{\tan\alpha}{\sqrt{\frac{2\pi}{g} \cdot \frac{H_{1/3}}{T_m^2}}} \quad (2.12)$$

The mean period used by Pedersen is the mean period referred to as T_{02} in CIRIA et al. (2012). The above-mentioned run-up formula, Eq. (2.12), can be used within the range of $R_{u0.1\%}/H_{1/3} \leq 2.58$. Gnbak and Ergin (1983) accepted the surface height affected by the impact pressure as the effective wedge thickness ($y=y_{eff}$). However, Pedersen (1996) reported that the actual impact zone observed during the experiments was approximately half of the calculated edge thickness. Pedersen's relation between unprotected height and wedge thickness when calculating the effective impact zone height is given in Eq. (2.13). In cases where the wedge thickness is negative (i.e., where $R_{u0.1\%}$ is lower than A_c), Pedersen suggested taking y_{eff} as 0.

$$y_{eff} = \min\left[\frac{y}{2}; f_c\right] \quad (2.13)$$

As a result, Pedersen (1996) divided the horizontal forces applied to the front face of the crown wall into two zones, protected and unprotected, and formulated the assumed pressure distribution as in Eq. (2.14) and Eq. (2.15).

$$F_{hu} = p_m \cdot y_{eff} \quad (2.14)$$

$$F_{hl} = 0.5 p_m \cdot h_{prot} \quad (2.15)$$

Pedersen (1996) stated that the up-rush velocity, v_0 , could not be obtained completely accurately in the first formulation, and completed the final formulation by including other wave parameters and structure dimensions that influence the horizontal forces. The effects of upper unprotected part and lower protected part were summed while obtaining the total maximum horizontal force. As a result, the relation in Eq. (2.16) is given by Pedersen (1996).

$$F_{h0.1\%} = a(b \cdot F_{hu0.1\%} + F_{hl0.1\%}) \quad (2.16)$$

Here, in order not to underestimate the influence of the wavelength on horizontal wave force, $\sqrt{L_{0m}/B}$ ratio was added to this relation while deciding on the empirical coefficients a and b during the formulation work. In this way, both the influence of armour crest width was added to the formula and the influence of wavelength was better reflected at the same time as shown in Eq. (2.17). Horizontal forces formulation for protected lower part and unprotected upper part are shown in Eq. (2.18) and Eq. (2.19), respectively.

$$F_{h0.1\%} = a \sqrt{\frac{L_{0m}}{B}} (b \cdot F_{hu0.1\%} + F_{hl0.1\%}) \quad (2.17)$$

$$F_{hl0.1\%} = \frac{1}{2} \cdot a \sqrt{\frac{L_{0m}}{B}} (p_m \cdot V \cdot h_{prot}) \quad (2.18)$$

$$F_{hu0.1\%} = a \sqrt{\frac{L_{0m}}{B}} (p_m \cdot y_{eff} \cdot b) \quad (2.19)$$

Pedersen (1996) also stated that the lower protected part overestimates the pressure values in milder wave conditions in which up-rushing wave hardly exceeds the

armour crest when analysing the tests results. Therefore, it is suggested to use the relation in Eq. (2.20) for the areas defined as V_1 and V_2 shown in Figure 2.5b.

$$V = \begin{cases} \frac{V_2}{V_1} & \text{if } V_2 < V_1 \\ 1 & \text{if } V_2 \geq V_1 \end{cases} \quad (2.20)$$

As a result of the fit study on 373 completed tests, empirical coefficients a and b were obtained as 0.21 and 1.60, respectively. The final equation for the calculation of the horizontal force, $F_{h0.1\%}$, is as in Eq. (2.21).

$$F_{h0.1\%} = 0.21 \sqrt{\frac{L_{0m}}{B}} (1.6 \cdot F_{hu0.1\%} + F_{hl0.1\%}) \quad (2.21)$$

Pedersen (1996) presented the ranges in which his formula is accepted as shown in

Table 2.3.

Table 2.3. Limitations to apply the (2.21) in wave load calculations (Pedersen, 1996).

Parameters	Range
ξ_m	1.1 – 4.2
H_s/A_c	0.5 – 1.5
R_c/A_c	1.0 – 2.6
A_c/B	0.3 – 1.1
$cot\alpha$	1.5 – 3.5

While analysing the results of Pedersen (1996), it was stated that there is a perfect linear correction between $F_{h0.1\%}$ and $M_{0.1\%}$ and therefore the overturning moment can be expressed by horizontal force. As a result of the analyses, the overturning moment formula shown in Eq. (2.22) was presented. In this equation, the empirical coefficient of c was identified as 0.55 by Pedersen (1996).

$$M_{h0.1\%} = c \cdot (h_{prot} + y_{eff}) \cdot F_{h0.1\%} \quad (2.22)$$

It has been reported that there is a weak correlation between the base pressure of the crown wall and the other two force components. Nevertheless, a simple but not entirely reliable formulation is given as shown in Eq. (2.23). In this equation, the empirical coefficient of d was identified as 1.00 by Pedersen (1996).

$$P_{b0.1\%} = d \cdot V p_m \quad (2.23)$$

The design formulae introduced by Pedersen (1996) have found a wide impact in coastal engineering literature and have been adopted by many design specifications and have become one of the most well-known formulae of crown wall design. The formulae of Pedersen (1996) were first included in the Coastal Engineering Manual (U.S Army Corps of Engineers, 2002). Today, it is recommended to be used in the Rock Manual (CIRIA et al., 2012) and AYGM (2016) design specifications.

Norgaard et al. (2013) stated that the Pedersen (1996) formula overpredicts the forces on the crown wall in shallow water conditions and performed modification studies to extend Pedersen's method. Furthermore, Norgaard et al. (2013) expressed that the Pedersen (1996) formulae are limited for deep to intermediate water conditions and have not been validated in practice for the depth-constrained conditions ($H_{m0}/h > 0.2$) in most field conditions encountered today. In the physical model prepared with only quarry rocks, Norgaard et al. (2013) designed the cross-sections so that A_c and R_c were equal, and then increased their R_c at the model scale, first by 3 cm and then by 6 cm to obtain pressure measurements at these raised up locations. With 162 new experiments, these results were compared with those of Pedersen (1996) and Martin et al. (1999) (described in the forthcoming Section 2.3.4). The cross-sectional setup used by Norgaard et al. (2013) in their experimental study is shown in Figure 2.6. In these experiments, the *cota* value was determined as 1:1.5.

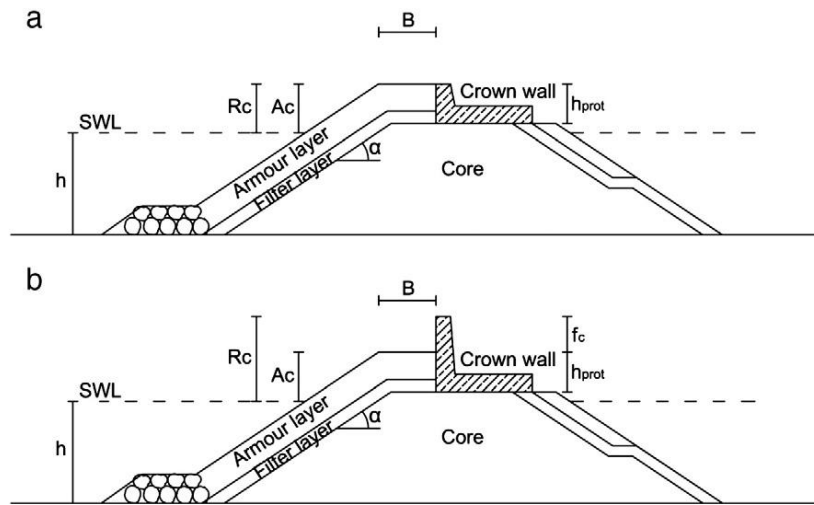


Figure 2.6. (a) Crown wall in which armour and crest freeboard are equal, (b) Crown wall in which crest freeboard is higher than armour freeboard. Adopted from Norgaard et al. (2013).

The structural dimensions of the model cross-section of Norgaard et al. (2013) are given in Table 2.4. The wave conditions of the experimental study are also given in Table 2.5. While generating the wave series, JONSWAP Spectrum was selected with a peak enhancement factor of $\gamma=3.3$ and wave series with at least 1250 wavelength were used (Norgaard et al., 2013).

Table 2.4. Structural dimensions of the model cross-section. Adopted from Norgaard et al. (2013).

Tests conditions	R_c (m)	A_c (m)	B (m)	R_c/A_c (-)	A_c/B (-)	f_c/A_c (-)
Shallow water	0.20-0.29	0.20-0.24	0.24	1.00-1.33	0.83-1.00	0-0.35
Deep water	0.10-0.19	0.10-0.14	0.17	1.00-1.70	0.59-0.82	0-0.70

Table 2.5. Wave conditions of the experimental study. Adopted from Norgaard et al. (2013).

Tests conditions	h (m)	H_{m0} (m)	$T_{m-1,0}$ (s)	H_{m0}/h (-)	A_c/H_{m0} (-)
Shallow water	0.30-0.36	0.150-0.180	1.826	0.500	1.00-1.33
Deep water	0.50-0.56	0.100	1.826	0.179-0.200	1.00-1.70

Norgaard et al. (2013) made the first modification to the calculation of $R_{u0.1\%}$ in the run-up formulation. Since the experimental conditions of Pedersen (1996) were in deep water, the Rayleigh distribution is applicable for wave height distribution. Norgaard et al. (2013) stated that it would be more appropriate to use $H_{0.1\%}$ instead of $H_{1/3}$ in the run-up formulation proposed by Van der Meer and Stam (1992) in shallow water conditions. Modified formulation of the run-up calculation is shown in Eq. (2.24).

$$R_{u0.1\%} = \begin{cases} 0.603H_{0.1\%}\xi_{m0} & \xi_{m0} \leq 1.5 \\ 0.722H_{0.1\%}\xi_{m0}^{0.55} & \xi_{m0} > 1.5 \end{cases} \quad (2.24)$$

Norgaard et al. (2013) also stated that if there is no data for $H_{0.1\%}$, the Battjes and Groendijk (2000) distribution should be used to estimate $H_{0.1\%}$ because the Rayleigh distribution function overestimates $H_{0.1\%}$ for shallow water conditions. Norgaard et al. (2013) made the second modification by comparing the Philips pressure transducers used by Pedersen (1996) with the Drück pressure transducers used in the new experiments when the up-rushing wave impacts directly on the front face of the crown wall in a slamming manner ($f_c > 0$). In the experiments carried out under the same conditions as Pedersen (1996) (i.e., deep water condition), it was observed that the old empirical coefficients overestimated the results of the measurements taken by the new sensors. Therefore, Norgaard et al. (2013) kept the value of the empirical coefficient a constant and updated the value of b to 1.0. Norgaard et al. (2013) made the third modification by multiplying the terms representing the unprotected and protected part wave forces in the formula of $M_{0.1\%}$ by various empirical coefficients

to calculate the overturning moment more accurately. A modified version of the $M_{0.1\%}$ is shown in Eq. (2.25). The empirical coefficients, e_1 and e_2 , have respective values of 0.95 and 0.40.

$$M_{0.1\%,mod} = \left(h_{prot} + \frac{1}{2} \cdot y_{eff} \cdot e_2 \right) \cdot F_{hu0.1\%} + \frac{1}{2} \cdot h_{prot} \cdot F_{hl0.1\%} \cdot e_1 \quad (2.25)$$

The research study of Norgaard et al. (2013) provided a valuable contribution to the literature by extending the formulae of Pedersen (1996) with modifications for shallow water conditions. The only possible shortcoming of this study is to perform tests with artificial armour units and to obtain empirical coefficients for varying armour layer conditions. For different roughness factors that will appear by using different artificial units in the armour layer, this formulation obtained by using only quarry rocks may overestimate or underestimate the wave loads acting on crown walls for armour layers constructed by artificial units.

2.3.4 Martin et al. (1999)

The formulae for estimation of wave pressures on the crown wall presented by Martin et al. (1999) are based on the wave run-up height level dependent pressure estimation formulae suggested by Günbak and Ergin (1983). In this study, it was stated that there are changes in the graphs of time-dependent pressure measurements in the sensors depending on the wave breaking types and wave steepness. Martin et al. (1999) used this as a basis for their formulation, showing that waves incident on the crown wall must make contact after breaking action. In other words, to apply the formulae of Martin et al. (1999), the incident waves must be broken on the armour slope as the formula does not include effects of excessive impact force. Martin et al. (1999) proclaimed that, there are two peaks in the pressure measurement time series of the tests in which the first pressure peak shows the dynamic pressure, P_d , the second pressure peak indicates the quasi-hydrostatic or reflecting pressure, P_r .

In the study of Martin et al. (1999), the linear assumption was repeated when estimating the uplift pressure as in Gnbak and Ergin (1983). The major difference of this assumption from Gnbak and Ergin (1983) and Pedersen (1996) is that the second pressure type, P_r , has a trapezoidal distribution instead of a triangular distribution. In other words, while the crown wall base uplift pressure under P_d pressure has a triangular pressure distribution with a maximum on the seaward side of the base and zero on the harbour side in accordance with the general assumption, under P_r pressure, contrary to the general belief, it is assumed to be maximum on the seaward side of the base and P_{ra} on the harbour side. Martin et al. (1999) presented the example scenario for both pressure actions as shown in Figure 2.7.

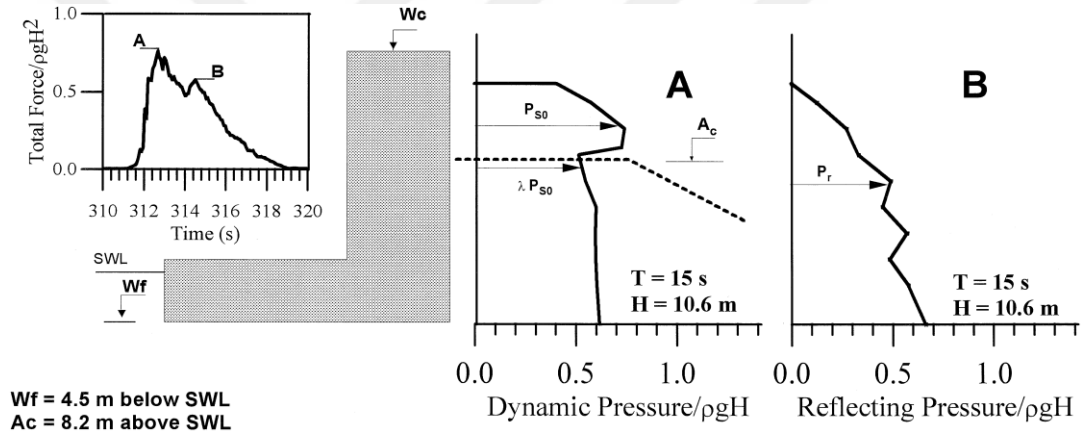


Figure 2.7. Example pressure distributions of dynamic and reflecting pressures. Adopted from Martin et al. (1999).

Martin et al. (1999) conducted modelling experiments mostly with monochromatic waves. Some random wave groups were also generated for the pressure measurement procedure. Eight strain type pressure gauges were placed to the vertical front face while four same type pressure gauges were located at the foundation of the crown wall. In this study, three different formulations are presented in total: first peak, second peak and uplift pressure. The pressure distribution and geometrical parameters on the crown wall are shown in Figure 2.8.

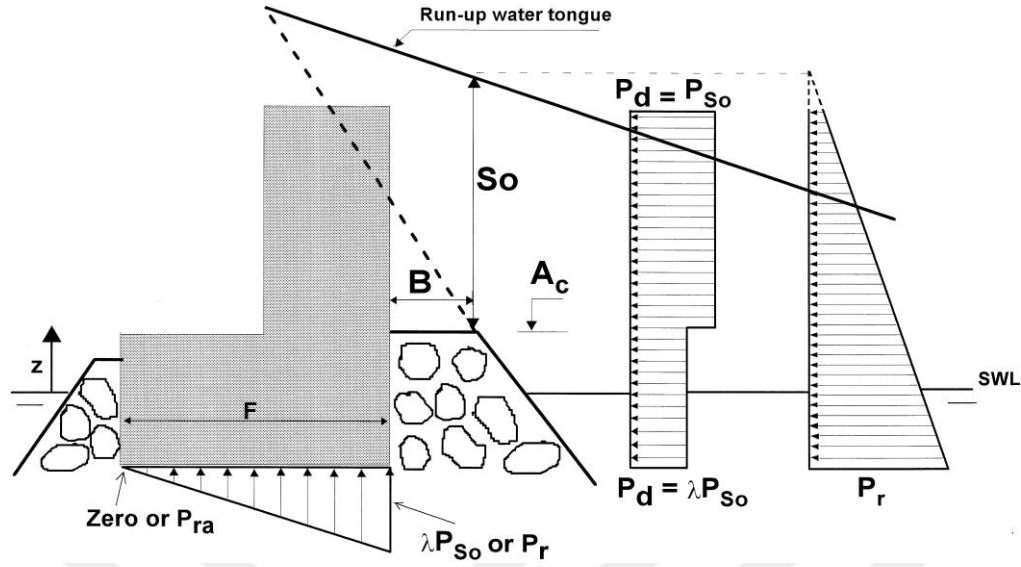


Figure 2.8. Dynamic and reflecting pressure distributions and geometrical parameters of the Martin et al. (1999) study. Adopted from Martin et al. (1999).

The formula for the dynamic pressure, P_{s0} , is given in Eq. (2.26), like the other formulations, greater in the unprotected region and a certain ratio less in the protected region.

$$P_{s0} = \alpha \rho_w g S_0 \quad (2.26)$$

In this equation, α is a non-dimensional parameter as shown in Eq. (2.27), S_0 is a geometrical parameter defined in Eq. (2.28), g is the gravitational acceleration and ρ_w is the water density.

$$\alpha = 2.9 \left(\frac{R_u}{H} \cos \alpha \right)^2 \quad (2.27)$$

$$S_0 = H \left(1 - \frac{A_c}{R_u} \right) \quad (2.28)$$

Dynamic pressure at the vertical face of the crown wall can be calculated as shown in Eq. (2.29).

$$P_d(z) = \begin{cases} P_{s0} & z > A_c \\ \lambda P_{s0} & w_f < z < A_c \end{cases} \quad (2.29)$$

In this equation, z represents the vertical direction where the datum is at the SWL. λ is an empirical parameter which can be found by using Eq. (2.30) and w_f indicates the foundation level of crown wall.

$$\alpha = 0.8e^{-10.9 \frac{B}{L}} \quad (2.30)$$

Martin et al. (1999) stated that S_0 and α are significantly affected by the maximum elevation level. The formulation of Losada and Gimenez-Curto (1981) was used in the formulation of Martin et al. (1999) to determine R_u as shown in Eq. (2.31).

$$\frac{R_u}{H} = A_u(1 - e^{B_u \xi_0}) \quad (2.31)$$

where A_u and B_u represent empirical coefficients depending on armour units. In this study, A_u was selected as 1.2 and B_u was taken as -0.7 for parallelepipedic blocks. In Eq. (2.31), ξ_0 is the surf similarity parameter in deep water which can be calculated as shown in Eq. (2.32).

$$\xi_0 = \frac{\tan \beta}{\sqrt{\frac{H}{L_0}}} \quad (2.32)$$

In this equation, β is the slope angle, H and L_0 are the local wave height and deep-water wavelength, respectively. H_{max} and $T_{H,max}$ were selected as an input for the formulations by Martin et al. (1999).

Reflecting pressure at the vertical face of the crown wall can be calculated as shown in Eq. (2.33).

$$P_r(z) = \mu \rho g (S_0 + A_c - z) \quad \text{for } w_f < z < A_c + S_0 \quad (2.33)$$

In this equation, μ is the dimensionless parameter ($\mu < 1.0$) which was determined from experimental results of the monochromatic waves. Martin et al. (1999) provided several coefficients (a , b , and c) to calculate μ parameter as shown in Eq. (2.34).

$$\mu = ae^{c\left(\frac{H}{L}-b\right)^2} \quad (2.34)$$

As mentioned before, uplift pressure at the foundation of the crown wall varies with dynamic and reflecting pressures. Martin et al. (1999) assumed a triangular distribution under the influence of dynamic pressure, stated that λP_{s0} is the pressure at the seaward edge and zero-base pressure at the landside edge. Under the effect of reflecting pressure, it was stated that there is a trapezoidal distribution, the pressure at the seaward edge is equal to the reflecting pressure P_r at the wave load action, and the pressure at the landward edge is equal to P_{ra} as stated in Losada et al. (1993).

Although the experimental study of Martin et al. (1999) added useful information to the literature and a different interpretation of the design of the crown wall, it could not cover a wide range of different parameters due to the use of only parallelepipedic blocks as armour units with monochromatic wave sets.

2.3.5 Berenguer and Boanza (2006)

Berenguer and Baonza (2006) contributed to the literature with their experimental studies and introduced a new semi empirical formulation. Experimental studies were conducted with generated irregular wave sets to identify the horizontal and vertical wave forces acting on the crown wall. This study analyzed the relationship between the foundation level of the crown wall and the wave loads while considering non-breaking waves (Molines, 2016). In this study, it was found that the maximum horizontal force, $F_{x,max}$, and the maximum vertical force, $F_{y,max}$, occur in different times with a phase difference. However, Berenguer and Baonza (2006) developed formulations for the wave forces assuming that both vertical and horizontal forces

occur at the same time to stay on the conservative side. It is obligatory to determine wave run-up level to use Berenguer and Baonza (2006) formulation like the other methods stated before. Run-up level with 2% exceedance probability, $R_{u2\%}$, was used different than the other methods in this study as shown in Eq. (2.35).

$$R_{u2\%} = 0.86\xi_p^{0.54}H_s\gamma_\beta \quad (2.35)$$

In this equation, $R_{u2\%}$ shows the run-up level exceeded by 2% of incident waves and H_s indicates the significant wave height. ξ_p is the surf similarity parameter (i.e., Iribarren number) corresponding to peak wave period and γ_β is the obliqueness factor which can be determined from EurotOp (2018). Horizontal forces and vertical forces can be calculated as shown in Eq. (2.36) and Eq. (2.37).

$$F_x = \begin{cases} \gamma_w C_h^{0.5} L_p^{1.5} \left(a \frac{R_{u2\%}}{A_c^{2/3} B^{1/3}} + b \right) & \text{if } R_{u2\%} > R_c \\ \gamma_w (R_{u2\%} - W_c)^{0.5} \cdot L_p^{1.5} \left(a \frac{R_{u2\%}}{A_c^{2/3} B^{1/3}} + b \right) & \text{if } R_{u2\%} \leq R_c \end{cases} \quad (2.36)$$

$$F_y = \begin{cases} \gamma_w C_h^{0.5} L_p^{1.5} \left(a \frac{R_{u2\%} - W_c}{A_c^{2/3} B^{1/3}} + b \right) & \text{if } R_{u2\%} > R_c \\ \gamma_w (R_{u2\%} - W_c)^{0.5} \cdot L_p^{1.5} \left(a \frac{R_{u2\%} - W_c}{A_c^{2/3} B^{1/3}} + b \right) & \text{if } R_{u2\%} \leq R_c \end{cases} \quad (2.37)$$

In the force calculation equations presented above, W_c means the bottom foundation level of the crown wall structure in which sea level is taken as datum. R_c is the crest freeboard level and A_c is the armour freeboard level. B identifies the width of the armour crest, C_b shows the length of the base slab of the crown wall and C_h shows the height of the crown wall structure. L_p is the wavelength in terms of peak wave period. Finally, a and b are the empirical coefficients influenced by armour unit type and wave breaking action determined by Berenguer and Baonza (2006) which are shown in detailed tables in Negro et al. (2013). Using these parameters and force calculations, Berenguer and Baonza (2006) provided the moment formulation in both directions as shown in Eq. (2.38) and Eq. (2.39).

$$M_x = \begin{cases} \gamma_w C_h L_p^2 \left(a \frac{F_x}{\gamma_w h_h^{0.5} L_p^{1.5}} + b \right) & \text{if } R_{u2\%} > R_c \\ \gamma_w (R_{u2\%} - W_c) L_p^2 \left(a \frac{F_x}{\gamma_w (R_{u2\%} - W_c)} + b \right) & \text{if } R_{u2\%} \leq R_c \end{cases} \quad (2.38)$$

$$M_y = F_y (C_b - 0.018 L_p) + (0.017 L_p - 0.109 C_b) \dots \dots \left(\frac{0.046 L_p - 0.217 C_b}{0.102 L_p - 0.651 C_b} \right) (C_b - 0.043 L_p)^2 \quad (2.39)$$

Range of application of the above-mentioned formulas are given in Molines (2016) as shown in Table 2.6.

Table 2.6. The range of application in wave load calculations (Berenguer and Baonza, 2006).

Parameters	Range
ξ_p	2.0 – 8.5
H_s/A_c	0.7 – 1.7
R_s/A_c	1.0 – 3.1
A_c/B	0.4 – 1.0
$cot\alpha$	1.5 – 2.0

2.3.6 Molines (2016)

Molines (2016) stated that there are many formulas in the literature for the prediction of crown stability and wave forces; however, the effect of armour unit type is complex and little mentioned. Therefore, Molines (2016) focused on finding new formulations while testing the armour stability, wave overtopping discharge and crown wall stability of the Cubipod-armoured mound breakwaters. As a result of the study, Molines (2016) indicated that the maximum vertical and horizontal forces occur with phase difference as mentioned in Berenguer and Baonza (2006) and completed wave loads formulation by analysing the loadings at the moment of maximum horizontal wave pressure and maximum vertical wave pressure applied by

this force. Molines (2016) estimated the horizontal wave force, $F_{h0.1\%}$, and the vertical wave force, $F_{v(Fh0.1\%)}$, as shown in Eq. (2.40) and Eq. (2.41), respectively.

$$\frac{F_{h0.1\%}}{0.5\rho_w g C_h^2} = \left(-1.29 + 1.80 \frac{\gamma_f R_{u0.1\%}}{R_c} + 0.93 \frac{R_c - A_c}{C_h} + 0.16 \sqrt{\frac{L_m}{G_c}} \right)^2 \quad (2.40)$$

$$\begin{aligned} \frac{F_{v(Fh0.1\%)}}{(0.5\rho_w g C_h C_b)} = & \left(-0.86 + 0.75 \frac{\gamma_f R_{u0.1\%}}{R_c} + 0.41 \frac{R_c - A_c}{C_h} \dots \right. \\ & \left. + 17 \sqrt{\frac{L_m}{G_c}} - 0.9 \frac{F_c}{C_h} \right)^2 \end{aligned} \quad (2.41)$$

$R_{u0.1\%}$ in this formulation is calculated with the same approach used in Pedersen (1996), that is Van der Meer and Stam (1992) wave run-up formulation given in Eq. (2.11). Roughness factor, γ_f , was taken as 0.50 for the randomly placed 2 layers of cube units, 0.46 for the two layers of cubipod and 0.44 for the one layer of cubipod units, respectively. F_c is the foundation level of the crown wall and G_c is the armour crest width of the breakwater. For the mean wave period, Molines (2016) used T_{01} to determine Iribarren number, ζ_{om} , and wavelength in terms of mean period, L_{om} .

2.4 Comparison of the Existing Empirical Wave Load Formulations of Crown Walls in the Literature

There are numerous studies in the literature comparing the above formulas for the design of crown walls based on case studies. The main goal of these studies is to investigate the primary advantages and disadvantages of the empirical wave load formulas to determine in which case the specific formula provides the most accurate result.

The first study that is exemplary among these studies is the comparative study by Camus and Flores (2004), in which the Spanish: Alicante and Ferrol breakwaters and

two different sections of the Coruna breakwater. This study examines the wave load estimates of Günbak and Gökce (1984), Jensen (1984) and Bradbury et al. (1988), Pedersen (1996) and Martin et al. (1999) to sufficiently re-analyse the existing formulations in the widely used comparative study for coastal engineering designers. While calculating the maximum wave forces and moments; H_{max} and T_p values were selected as wave parameter inputs when using the formulations of Günbak and Gökce (1984) and Martin et al. (1999), while $H_{0.1\%}$ parameter was used when evaluating the methods of Jensen (1984) and Bradbury et al. (1988), and Pedersen (1996). Camus and Flores (2004) reported that the maximum horizontal forces, uplift forces and overturning moments were best predicted by Pedersen (1996) formulation and that it did not give poor results even when the range of application was exceeded.

The second of the comparative analyses of wave forces on the surfaces of the crown wall was carried out by Negro et al. (2013). This study analysed different formulations compared to the first study and included the work of Iribarren and Nogales (1954) and Berenguer and Baonza (2006) in their analysis. The studies by Pedersen and Burchart (1992) and Martin et al. (1995) are considered in this paper instead of Pedersen (1996) and Martin et al. (1999), which are the main studies of these authors. Mutriku New Breakwater and Barcelona South Breakwater were investigated experimentally in this study to compare the above-mentioned wave pressure and force formulations. Berenguer and Baonza (2006) formulation gave the greatest force magnitude among the other methods in this study for the Mutriku New Breakwater. However, Bradbury et al. (1988) did the largest force estimation for the Barcelona South Breakwater. Negro et al. (2013) comments on variety of the wave pressure and load formulations on the crown wall design in the literature.

A third comparative study was conducted by Molines (2016) to derive a set of new formulas to fit the experimental data. Molines (2016) completed the root mean square error, *RMSE*, analysis of the various wave load formulations of the crown wall structures to see which accurately estimates the horizontal and vertical forces.

The last important comparative analysis study in the literature is performed by Pereira et al. (2021) for the Eriçeria breakwater by comparing the design formulae of the crown wall to observe the damaging effect of sea level rise due to altering ocean states and climate change. In this study, the design formulae of Berenguer and Baonza (2006) and Norgaard et al. (2013) were considered in addition to the those compared in the study by Camus and Flores (2004). According to Pereira et al. (2021), the methods of Pedersen (1996) and Norgaard et al. (2013) were the most conservative in predicting the horizontal forces, while the formulation of Martin et al. (1999) gave the lowest values. While estimating uplift forces, Martin et al. (1999) again achieved the lowest values, while Günbak and Gökce (1984) did the largest force estimates for some cases and Pedersen (1996) and Norgaard et al. (2013) for some other cases.

2.5 Experimental Studies Investigating the Performance of Antifer Units

There are many experimental studies in the literature investigating the performance of armour layers designed with antifer units. Among these, the most comprehensive study was conducted by Frens (2007), who studied the wave overtopping and stability performance of several different antifer placement methods. In this study, five basic placement methods were studied: irregular, column, filled pyramid, closed pyramid, and double pyramid. The study has comprehensive analyses of the specified placement methods, while it provides important insights for stability and wave overtopping performance.

Yagci and Kapdasli (2003) investigated the performance of four different placement methods. These studies were named as irregular, sloped wall, alternative and regular placement technique respectively. While important information about how these placements were made was mentioned, the stability performances of the placement methods were analysed by comparing their damage ratios.

Chegini and Aghtouman (2006) conducted experimental studies to produce a new formulation for weight calculation of antifer units for different wave characteristics and slopes under random wave conditions.

Freitas et al. (2013) conducted analyses for semi-irregular and regular (closed pyramid according to Frens (2007)) placement methods in the experimental observation of hydraulic stability. This study presented information about K_d coefficient, stability parameter and reflection values.

When the mentioned studies were examined, the study of Frens (2007) was taken as a reference, and the effect of closed pyramid, irregular and double pyramid placement methods, which are widely used and have very good performance, on the wave forces acting on the crown wall is examined in the present study.

2.6 Summary of Literature Review

As explained above, many different experimental studies have been carried out in the literature to make the most accurate crown wall design and the results have been supported by various empirical formulations. Although each study focuses on different wave parameters and design conditions, Pedersen (1996) is the most widely adopted among these studies. The design formula to be used in the physical modelling experiments conducted within the scope of this study was decided by considering the parametric properties of the random wave sets and the experimental condition. In this context, the methodology of Norgaard et al. (2013), which is an extended version of the Pedersen (1996) formulae for shallow water conditions, was selected.

Although different armour units were used in some of the studies, differences in wave run-up height and wave pressure distribution on the crown wall structure that may develop due to the use of various units were not observed. The most detailed study on the effect of armour units was conducted by Molines (2016), in which the effect of cubipod units on wave run-up height and wave pressure was reflected in the

wave forces formulas depending on γ_f . Even though the research objectives and methodology of Molines (2016)'s study overlap considerably with this study, the analyses were completed with Norgaard et al. (2013), since only cubipod-dependent parameter changes are reflected in Molines (2016).





CHAPTER 3

METHODOLOGY

In this chapter, the physical model experiments investigating the effect of different antifer unit placement methods on the wave forces acting on the crown wall are described. The cross-section prepared for the experiments is based on typical breakwater designs that are widely used in the practical coastal applications. The wave channel was prepared to serve the purpose of the experiments. Irregular wave series are generated to symbolise the effects that rubble mound breakwaters will be exposed to under field conditions. Then, the antifer units to be used were selected by deciding which placement methods in the literature would be used. After the location of the crown wall structure and pressure sensors were decided, experimental studies were started to be conducted. At the end of the experiments, the focus was on comparing the measured data with existing studies in the literature, performing dimensional analysis for the formulation process and presenting the results of the study.

3.1 Physical Model Experiments

3.1.1 Experimental Setup

The physical modelling experiments in this study were conducted in a 26.9 m long, 6 m wide and 1 m deep wave channel at Middle East Technical University, Department of Civil Engineering, Coastal and Ocean Engineering Laboratory. In this wave channel, an inner flume with a width of 0.9 metres was built, and the rubble mound breakwater section was constructed in that channel. Side view of the wave flume and experimental setup used for these physical modelling studies are shown in Figure 3.1. There is a wave generator at one end of the wave flume and passive

wave absorbers at the other end to reduce wave reflection. The coordinate system was determined so that the x-direction indicates the horizontal direction of the channel, and the z-direction indicates the vertical direction, and the starting point was placed at the point where the wave generator and the still water level meet (at $x=0$ and $z=0$). With the help of the wave gauges shown in Figure 3.1, surface water elevation (η) and individual wave overtopping measurements are carried out with a total of 14 wave gauges in different parts of the channel.

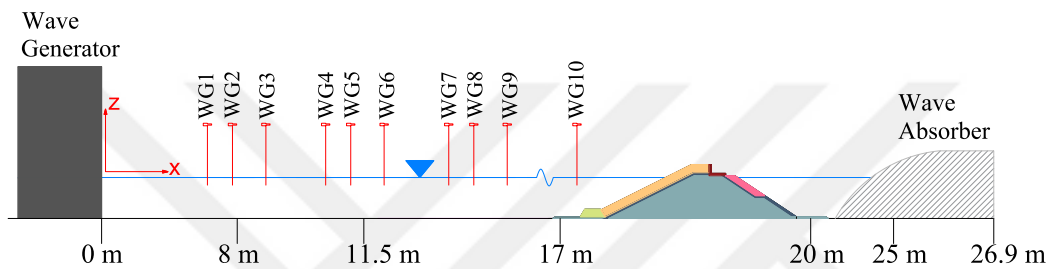


Figure 3.1. Layout of the 2D model test in the wave flume.

The irregular wave series measurements taken from WG1-2-3, WG4-5-6 and WG7-8-9 wave gauges placed at certain intervals in groups of three gauges were analysed using the method proposed by Mansard and Funke (1980) to separate incident and reflected wave spectra. The implications presented by Klopman and van der Meer (1999) were considered when placing the wave probes in front of the breakwater model cross section. Wave gauges WG10-14 shown in Figure 3.1 are used for individual and mean overtopping measurements. In addition to still water level measurements, velocity measurements of water particles are performed using Acoustic Doppler Velocimeter (ADV) at three different points along the channel shown in Figure 3.1. It is noted that the wave overtopping measurements are not presented in the present study, which are the focus of a separate thesis study (Erler, 2023). Furthermore, the velocity measurements are planned to be used in the numerical simulations studies in the future; therefore, these measurements are not discussed in the present study.

During the experiments, a system was installed utilizing LED lamps to synchronise the WG data, pressure data and camera recordings during the analysis process. According to this system, the experiments were started with the lights turned on at the beginning of each different recording, and all recordings were synchronised with the planned colour code during the analysis process. During the experiments, the camera recordings were taken in a way to see the breakwater section from the front view, side view and top view. In addition, for some wave scenarios, the apparatus containing the sensors was recorded directly over the crest. Example of snapshots from the videos during the experiments is given in Figure 3.2.

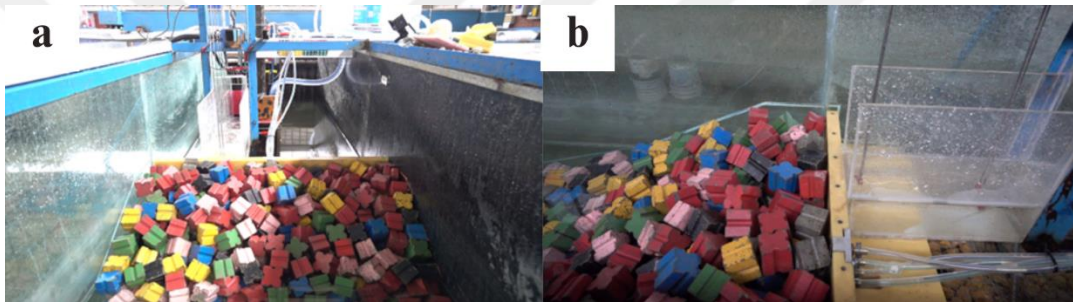


Figure 3.2. Video snapshots from (a) front view, (b) side view.

3.1.2 Design of the Section and Wave Conditions to be Tested in Physical Model Experiments

As will be described in more detail in Section 3.1.4, the experimental programme planned to conduct experiments for eight different armor unit placement methods at a total of three different crown wall crest heights in the armor layer. While designing the breakwater cross section, recommendations about the armor stability given by Frens (2007) on the effect of different placement of armor units on section stability were taken into consideration. A considerable number of repetitions are required to take pressure measurements regularly and effectively on the crown wall model. Accordingly, it is essential that the cross-section of the rubble mound breakwater should be optimally designed, and the experimental setup must be

established in the most efficient way. In this context, the following principles were taken as basis while designing the cross-section prepared for the physical test:

- i. During the application of six different irregular wave sets selected for use in the experiments, the cross-section is not damaged and thus there is no change in pressure measurements influenced by the damage of the cross-section.
- ii. To make an accurate comparison during the experiments, only the units in the armour layer were replaced and no changes were made in other elements (filter layer, slope, and rear side layers) of the cross-section.
- iii. To measure the wave forces acting on crown wall to be obtained because of the wave run-up more accurately, the wave characteristics were selected as large as possible so as not to adversely affect item (i). In addition, to obtain reasonable wave forces, wave sets with small wave height and wave period were avoided. Therefore, before constructing the model section, it was ensured that reasonable wave overtopping values would be obtained for the operation of experiments and to obtain reasonable results.
- iv. Ensuring that the process of changing the location of the pressure sensors required for pressure measurements to be taken on the vertical face and foundation of the crown wall is carried out in the easiest and most effective way without causing any change in the cross-section.

3.1.2.1 Design of the Model Rubble Mound Breakwater Section

By paying attention to listed items stated in Section 3.1.2, breakwater model section design studies were carried out before starting the experiments. The cross-section to be tested in the experiments was designed by considering the principles stated above and the basic design criteria recommended by AYGEM (2016) and CIRIA et al. (2012) for the design of conventional single slope rubble mound breakwater. Accordingly, the design of the armor units to be used in the armor layer was based on the Hudson (SPM, 1984) approach based on the principle of depth-induced wave breaking. The Hudson formula mentioned is shown in Eq. (3.1).

$$M_{50} = \frac{\rho_s H^3}{K_D \left(\frac{\rho_s}{\rho_w} - 1 \right)^3 \cot \alpha} \quad (3.1)$$

In this equation, ρ_s and ρ_w denote mass density of armour unit and water, respectively. H represents characteristic wave height (H_s or $H_{1/10}$), and D_{n50} shows equivalent cube length of armour unit. K_D is stability coefficient and α is slope angle. K_D coefficient was selected as 8.0 for the design and considering $\cot \alpha$ equals to 2.0 for the armour layer slope in non-breaking wave condition.

The filter layers of the breakwater section were designed with a filter thickness of $2D_{n50}$ as specified in AYG (2016) and CIRIA et al. (2012). The layer thickness of the model section armour is designed to place two antifer units as frequently applied in the literature and practical applications.

The dimensions of the antifer unit, crown wall model, the stone weights of the filter, core and toe layers used in the construction of the experimental model section were checked before the experiments to see how real-life conditions were realistically reflected. Various model scales suggested by Wolters et al. (2007) were used during the checking process. In addition, the methodology proposed by Hughes (1993) based on Froude scaling was applied to calculate the prototype dimensions and weights.

As stated in Burcharth et al. (1999), since the Reynolds number in the model scale which is designed with Froude scaling is considerably smaller than the actual Reynolds, inaccurate scaling of surface tension and viscosity leads to very large viscous forces in the underlayer and core layer. Therefore, the core layer and the underlayer of the breakwater was designed according to the calculations proposed by Burcharth et al. (1999) to minimise the viscous effects.

The physical wave mechanism on the cross section and the wave forces acting on the crown wall can be adversely affected, especially by surface tension and viscosity (Burcharth et al., 1999). Since the Reynolds number in Froude scaling is

considerably smaller than the actual Reynolds number, inaccurate scaling of surface tension and viscosity leads to the observation of large viscous forces, especially in the underlayer and core layer. To avoid this negative scale effect in this study, the details mentioned by Burcharth et al. (1999) were taken into consideration during the design process. Weights of the antifer units used in armour layer, filter layer and core used in the section design are given in Table 3.1.

Table 3.1. Unit and material sizes in the breakwater model cross-section.

Antifer units	Core material	Filter material	Toe material
271 gr	D _{n50} = 5 mm	D _{n50} = 20 mm	D _{n50} = 30 mm

To fulfil the reason stated above in item (iii) accurately, the method proposed by EurOtop (2018) was used for wave overtopping estimations based on the wave characteristics, the type of unit to be used in the armour layer of the breakwater and the crest height of the crown wall. The formula used in the EurOtop (2018) is shown in Eq. (3.2).

$$\frac{q}{\sqrt{g \cdot H_{m0}^3}} = 0.09 \cdot \exp \left[- \left(1.5 \frac{R_c}{H_{m0} \cdot \gamma_f \cdot \gamma_\beta} \right)^{1.3} \right] \quad (3.2)$$

In (3.2), q is the mean overtopping discharge per meter structure width (m^3/s per m), g is the gravitational acceleration ($=9.81 \text{ m/s}^2$), H_{m0} is the spectral significant wave height, R_c is the crest freeboard of structure, γ_f is the influence factor for the permeability and roughness of the slope, and γ_β is the influence factor for oblique wave attack. In the procedures using this formula, the value of 0.5 for γ_f , which was determined for antifer units in the CLASH tests conducted by Bruce et al. (2009), was used. γ_β value was taken as 1.0 since the approaching wave angle is 0° at the wave channel.

In the design of the breakwater section, the parameters of the most critical irregular wave series were accepted as the design parameters and the section was kept on the

safe side to avoid any damage and failure. The main purpose here is averting the change in the magnitude of the wave forces on the crown wall due to damage. Considering all these reasons, geometrical parameters of the cross-section of the designed breakwater model are shown in Table 3.2.

Table 3.2. Structural dimensions of the rubble mound breakwater model section.

Tests conditions	R_c (m)	A_c (m)	α [-]	B (m)	R_c/A_c [-]	A_c/B [-]	f_o/A_c [-]
Shallow water	0.15-0.21	0.15	1:2.0	0.17	1.0-1.4	0.9	0-0.4

As stated before, three different R_c values were tested for each antifer placement methods. A_c was set as 0.153 m in all placement method cases, so that the pressure measurements were aimed to be carried out at three different crest freeboard elevation. Thus, pressure measurements can be taken both at the front face of the crown wall protected by the antifer units of the armour layer and at the unprotected surface of the crown wall (with the expectation that the impact pressures would be observed intensively). Pedersen (1996) approach was used for the design of the crown wall. This approach is described in detail in Chapter 2. For the dimensions of the crown wall designed by this method, the parameter R_c , which significantly affects the mean overtopping rate in Eq. (3.2), was also considered to obtain the desired mean wave overtopping rates. Geometrical dimensions and features of the crown wall are described in detail in Section 3.1.3. A close-up view of the breakwater cross-section and the experimental setup is given in Figure 3.3.

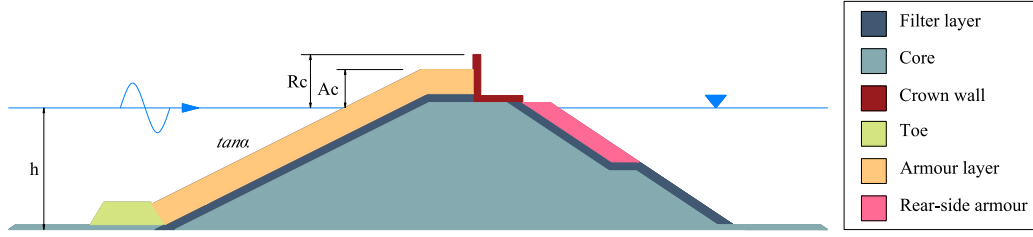


Figure 3.3. Close up view of the experimental setup and cross-section.

3.1.2.2 Wave Conditions

Within the scope of the experiments, a total of six different irregular wave series were generated. The range of target wave parameters and non-dimensional parameters used in the test series are given in Table 3.3 for all different wave conditions. The minimum number of 1000 waves criteria condition was fulfilled for all six irregular wave sets during the experimental study. While generating the waves, JONSWAP spectrum with $\gamma = 3.3$ is used. As can be seen from Table 3.3, with the targeted Iribarren numbers ($\xi_{m-1,0}$). Iribarren number or surf similarity parameter is calculated as shown in Eq. (3.3).

$$\xi_{m-1,0} = \frac{\tan \alpha}{\left(\frac{H_{m0}}{L_{m-1,0}} \right)^{1/2}} \quad (3.3)$$

Surging type wave breaking was expected to be observed in the specified range on the armour layer. As can be observed from the experiment videos, the majority of the waves were subjected to surging type wave breaking ($\xi_{m-1,0} > 2-3$). As can be seen from Table 3.3, when H_{m0}/h values are considered, it is seen that the generated wave series are located in the shallow water to intermediate depth region ($H_{m0}/h > 0.2$).

Table 3.3. The range of target wave parameters used in the test series.

Parameters	Range
Spectral Significant Wave Height, H_{m0} (in m)	0.12 – 0.18
Wave height exceeded by 0.1%, $H_{0.1\%}$ (in m)	0.22 – 0.27
Peak wave period, T_p (in s)	1.75 – 2.05
Spectral mean period, $T_{m-1,0}$ (in s)	1.65 – 1.90
Spectral mean period, T_{01} (in s)	1.50 – 1.70
Spectral mean period, T_{02} (in s)	1.45 – 1.60
Water depth, h (in m)	0.49
Non-Dimensional Parameters	Range
Wave steepness according to spectral mean period, $s_{m-1,0}$	0.02 – 0.04
Iribarren number, $\zeta_{m-1,0N}$ (-)	2.55 – 3.20
H_{m0}/h	0.24 – 0.40
A_c/H_{m0}	0.85 – 1.30
R_c/H_{m0}	0.85 – 1.80

The target parameters selected for the experimental study are similar to pressure measurement studies in the literature (e.g., Pedersen (1996), Martin et al. (1999), Norgaard et al. (2013)). Moreover, below mentioned parameters were chosen to cover the most common structural parameters in prototype coastal structures. In other words, through the selected parameters, the compatibility of the present studies with similar experimental studies in the literature is ensured. Thus, the data from the results of the studies in the literature can be compared with the measured data analysed at the end of the physical experiments of this thesis study. Measured wave parameters of the six different irregular wave series depending on the target ranges are shown in Table 3.4.

Table 3.4. The range of target wave parameters used in the test series.

Wave Sets	H_{m0} (m)	$H_{0.1\%}$ (m)	$T_{m-1,0}$ (s)	T_p (s)	T_{01} (s)	T_{02} (s)	H_{m0}/h
D1	0.160	0.225	1.667	1.759	1.531	1.450	0.33
D2	0.140	0.231	1.734	1.845	1.596	1.506	0.29
D3	0.120	0.238	1.765	1.864	1.633	1.547	0.25
D4	0.168	0.268	1.846	1.906	1.675	1.572	0.34
D5	0.180	0.265	1.887	2.055	1.707	1.597	0.37
D6	0.162	0.246	1.779	1.875	1.613	1.509	0.33

In the formulae presented in the literature for the design of the crown wall, there are several wave parameters calculated with a low exceedance probability. Since maximum wave forces occur when these parameters are encountered, it is very important to have precise information about these parameters. For this reason, the wave height distributions of six different irregular wave series generated for the experiments in accordance with the target range as shown in Table 3.3 were examined and it was checked that they could provide shallow water conditions in the most appropriate way. In addition, these controls were carried out for the accurate estimation of $H_{0.1\%}$ and the $R_{u0.1\%}$ value to be obtained accordingly. Non-dimensionalized histograms of the wave heights were plotted, and the probability density functions of the generated wave series were analysed to see how well they fit the Rayleigh distribution. The analysis of the D-6 irregular wave series is given in Figure 3.4 as an example. Histogram plots for all wave sets can be found in Appendix A. When the histogram plots are analysed, it is seen that the waves have a distribution as desired and have a probability density function very close to the Rayleigh distribution except D4, D5 and D6 irregular wave series.

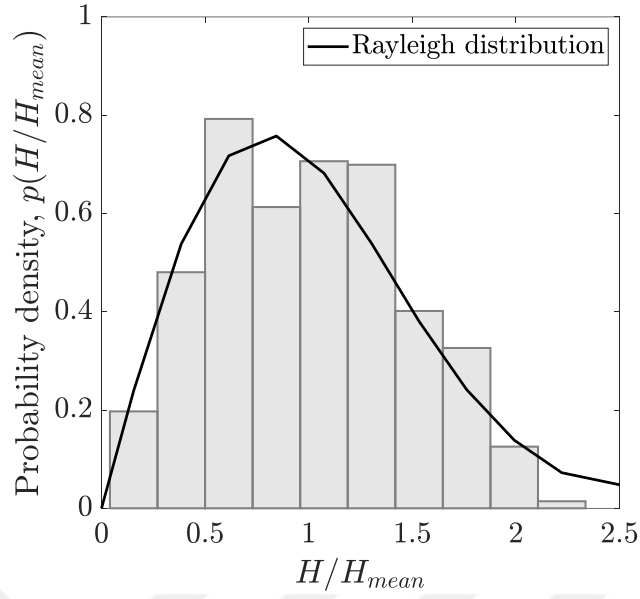


Figure 3.4. Non-dimensionalized histogram of wave heights [D-6 Irregular Wave Condition]

As it is known from the literature, waves in deep water conditions obey the Rayleigh distribution. However, since the scope of this study is considering shallow water conditions, the Rayleigh distribution may overestimate the low exceedance probability wave parameters of the wave series produced. Norgaard et al. (2013) mentioned the overestimation problem of the critical wave height and used the distribution proposed by Battjes and Groenendijk (2000), which gives more realistic results in shallow water condition in the calculation and control of $H_{0.1\%}$. Therefore, Battjes and Groenendijk (2000), Rayleigh and Glukhovskii (1966) distributions were checked during the harmonisation of the generated wave series in this study. Formulation of the Battjes and Groenendijk (2000) distribution is shown in Eq. (3.4).

$$F(H) = \begin{cases} 1 - \exp\left(-\frac{H}{H_1}\right)^{k_1} & \text{for } H \leq H_{tr} \\ 1 - \exp\left(-\frac{H}{H_2}\right)^{k_2} & \text{for } H \geq H_{tr} \end{cases} \quad (3.4)$$

H_1 and H_2 are characteristic normalised wave heights which can be determined from Table 2 in Battjes and Groenendijk (2000). H_{tr} is the transition wave height and can be found by using Eq. (3.5) shown below.

$$H_{tr} = (0.35 + 5.8 \tan \alpha) d \quad (3.5)$$

To use the Table 2, one must determine characteristic normalised transition wave height, $\hat{H}_{tr}$, as shown in Eq. (3.6).

$$\hat{H}_{tr} = H_{tr}/H_{rms} \quad (3.6)$$

Battjes and Groenendijk (2000) suggested calculating root mean squared wave height, H_{rms} , as shown in Eq. (3.7).

$$H_{rms} = (2.69 + 3.24 \sqrt{\frac{m_0}{d}}) \sqrt{m_0} \quad (3.7)$$

In this equation, m_0 defines zeroth moment of the surface elevation frequency spectrum of the wave series. Rayleigh distribution function is given in Eq. (3.8).

$$F(H) = 1 - \exp\left(-2\left(\frac{H}{H_s}\right)^2\right) \quad (3.8)$$

Modified distribution function of Glukhovskii (1966) is shown in Eq. (3.9).

$$F(H) = 1 - \exp\left(-\frac{\pi}{4} \left(\frac{(H/H_m)^{2/(1-\zeta)}}{1 + \frac{\zeta}{\sqrt{2\pi}}} \right)\right) \quad (3.9)$$

In Eq. (3.9), ζ is the ratio of mean wave height, H_m to water depth, d . An example comparison of exceedance probability of the wave height distributions is shown in Figure 3.5 for the D-6 irregular wave series. Wave height distributions for all wave sets can be found in Appendix A. As can be seen from Figure 3.5, it is indeed observed that the Rayleigh distribution may lead to overestimation of the results by predicting greater $H_{0.1\%}$ values, and the waves are more consistent with Battjes and Groenendijk (2000) in accordance with the shallow water condition as stated in

Norgaard et al. (2013). The Glukhovskii distribution (1966), which is an improved version of the Rayleigh distribution considering depth limitation effects, is also compatible with the wave series produced in this experiment.

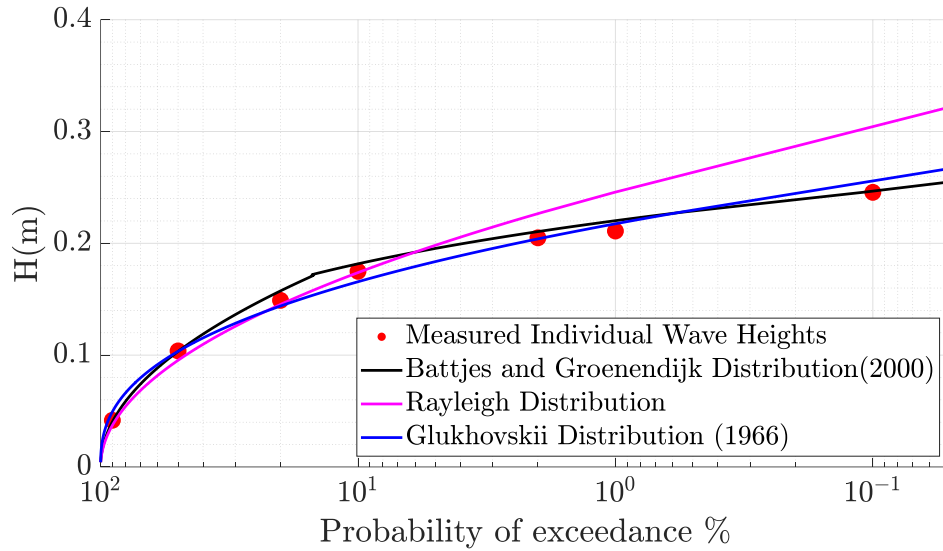


Figure 3.5. An example comparison of three different wave height distributions for the irregular wave series. [D-6 Irregular Wave Condition]

3.1.3 Design of the Crown Wall Model and Location of the Pressure Measurement Sensors

To perform the experiments accurately, it was ensured that there was no damage and failure in the cross-sectional armour layer and the crown wall model. By ensuring this condition, possible changes in wave pressure due to damage and failure were eliminated. When designing the crown wall model, care was taken to make it from a material that is resistant to long wave conditions, will not rust and corrode. Therefore, it is made of kestamite material which is a type of polyamide that is formed by casting and has a tight hardness level. Crown wall model is shown in Figure 3.6.

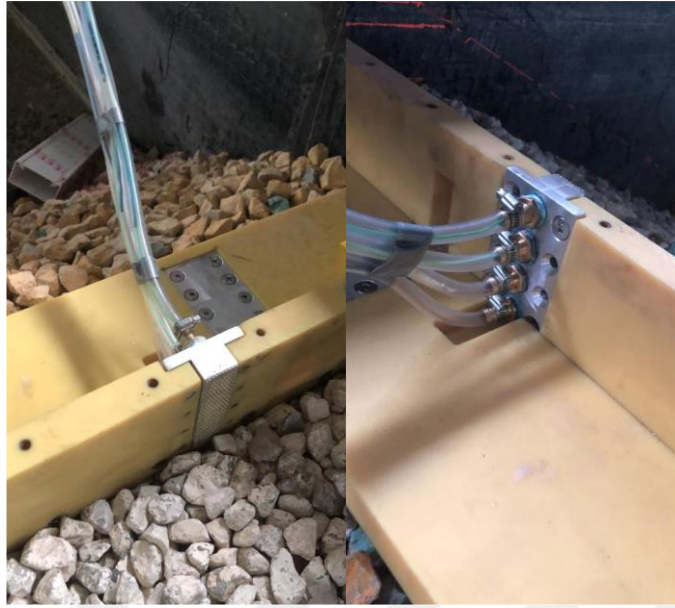


Figure 3.6. Crown wall model designed with the kestamite material.

The crown wall model was fixed to the bottom of the wave flume to prevent any sliding and overturning failure during the experiments. Care was taken to ensure that the apparatus used during fixing did not cause any problems that would change the wave effect on the cross section.

A total of twelve pressure measurements were made on the crown wall model, six at the horizontal slab base and six at the vertical front face. Four KISTLER Type 601 C Piezoelectric Pressure Sensors were used in these experiments to measure wave pressures on the crown wall surface. Since the number of locations to measure wave pressure is more than the number of sensors, the locations of the sensors should be easily changed in repetition experiments. Therefore, these four sensors are mounted on an aluminium apparatus sealed in a watertight manner. This aluminium apparatus is designed to be easily attached to the crown model with screws without disturbing the wave action and pressure development. A photograph of the aluminium apparatus and the location of the sensors on this crown wall model are given in Figure 3.7. As can be seen from Figure 3.7, aluminium apparatus surface and pressure sensors were

cover with a very fine mesh to prevent the contact between antifer units and the sensors.



Figure 3.7. Aluminium apparatus position on the vertical face of the crown wall.

The technical drawing and plan view of the crown wall model are given in Figure 3.8. In this figure, the distances between the sensors, the location of the sensors in the cross-section of the crown wall model, the location of the sensors on the vertical front face and the horizontal base of the crown wall are shown.

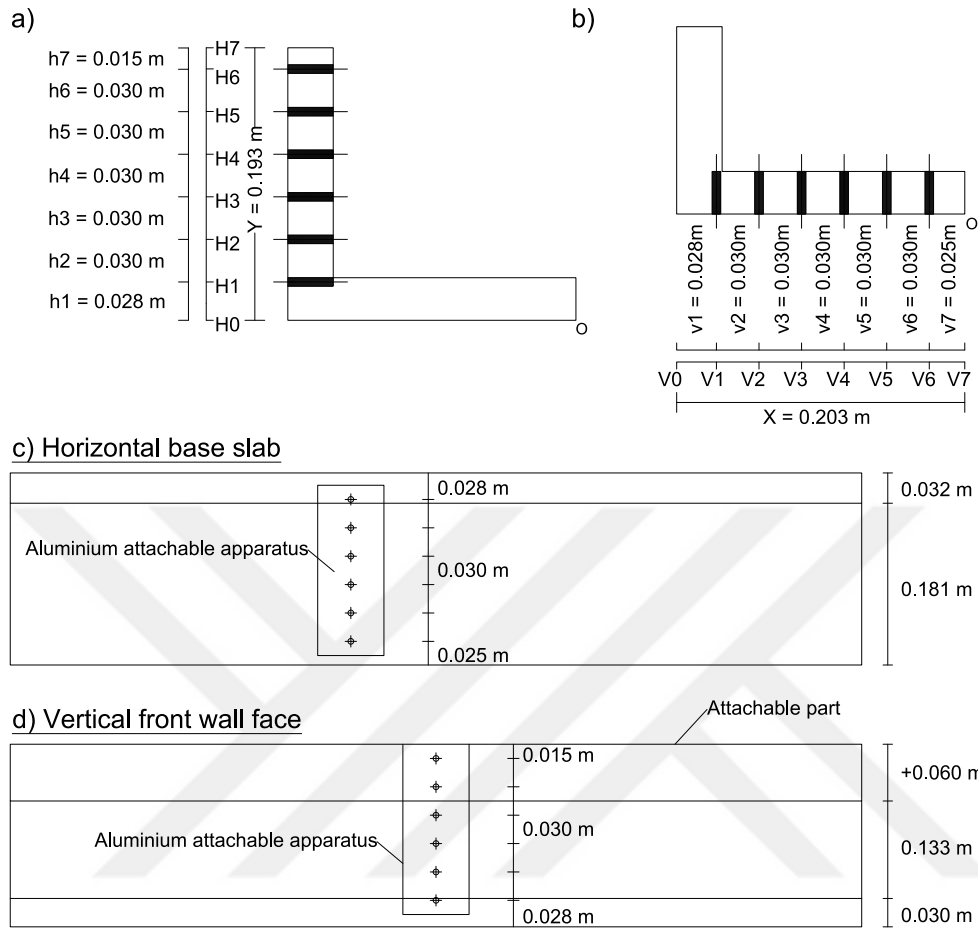


Figure 3.8. Locations of the pressure sensors on the crown wall model. Cross-section view of the positions of the pressure sensors at (a) vertical front wall face, (b) horizontal base slab. Plan view of the positions of the pressure sensors at (c) horizontal base slab, (d) vertical front wall face.

It is noted that, the physical model study of Norgaard et al. (2013) was taken as an inspiration, while designing the crown wall model in the way described above. After the pressure measurements of eight different antifer placement methods were completed for the case where the anterior surface of the crown wall was completely protected by the antifer units (region between H0 and H4 forces as shown in Figure 3.8), it was desired to take measurements of the wave pressures for the unprotected region on the same surface. For this purpose, the crest height of the crown wall was

increased in two different stages to create an unprotected region without changing the armour crest elevation (region consisting of the horizontal forces H5, H6 and H7 in Figure 3.8). In this way, the wave forces generated on the crown wall with a total of three different crest elevations were analysed and the wave pressure distribution in both protected and unprotected region was examined. To make these changes easily on the crown wall model, 3 cm, and 6 cm kestamite attachable apparatus were manufactured. Photograph of the attachable apparatus are shown in Figure 3.9.



Figure 3.9. Attachable apparatus of the 3 cm and 6 cm extension of the crown wall crest elevation.

3.1.4 Experimental Programme

The system designed for pressure measurements is similar to the cross-section and measurement methodology established by Norgaard et al. (2013). In the present study, three different antifer units placement methods inspired from Frens (2007) named as closed pyramid, irregular and double pyramid. The experiment programme applied is presented in Table 3.5.

Table 3.5. Experimental programme for the pressure measurements.

Section	Placement Method	Wave Conditions
0	– [Wave calibration studies]	Six different irregular wave series (Check Table 3.3)
1	CP1: Closed Pyramid, $\gamma_s = 54.8\%$ (Check Table 3.6a)	
2	CP2: Closed Pyramid, $\gamma_s = 49.8\%$ (Check Table 3.6b)	
3	IR1: Irregular Placement, $\gamma_s = 57.1\%$ (Check Table 3.6c)	
4	IR2: Irregular Placement, $\gamma_s = 61.1\%$ (Check Table 3.6d)	
5	DP1: Double Pyramid, $\gamma_s = 54.8\%$ (Check Table 3.6e)	
6	DP2: Double Pyramid, $\gamma_s = 54.8\%$, layers are $\frac{1}{2} D_{n50}$ staggered (Check Table 3.6f)	
7	DP3: Double Pyramid, $\gamma_s = 61.5\%$, layers are $\frac{1}{2} D_{n50}$ staggered (Check Table 3.6g)	
8	DP4: Double Pyramid, $\gamma_s = 61.5\%$ (Check Table 3.6h)	

As can be seen from Table 3.5, a total of eight different types of antifer unit placement methods were tested with six different random wave series conditions. As a result, a total of 144 different experiments were performed to test three different R_c values and eight different antifer placement methods. In the scenario where R_c is equal to A_c , two repetitions were made in the vertical face of the crown wall model. In the other two scenarios where R_c is greater than A_c (i.e. 3 cm and 6 cm increase of the crown wall crest height), a total of four repetition experiments were performed on the front face. The reason for these repetitions is both to check the randomness in the wave parameters during the pressure measurements and to control the changes in the wave pressure distribution according to the position of the antifer unit in front of the sensor. Antifer units arranged in two layers in the armour layer can have two different positions in front of the sensor.

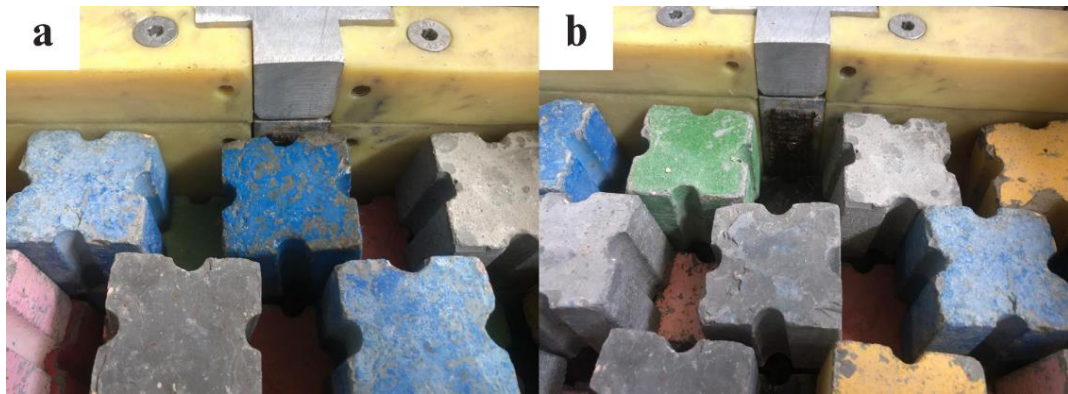


Figure 3.10. (a) DP2A placement method configuration, (b) DP2B placement method configuration as an example for the arrangement in front of the pressure sensors.

As can be seen in Figure 3.10, depending on the placement method, sometimes the antifer units in the second layer and sometimes the antifer units in the first layer can be in front of the sensor block. Antifer units are therefore not able to make a completely smooth distinction between unprotected and protected regions on the front face of the crown wall, as in the case with armour layers built of natural rocks. In order to observe the differences due to the antifer position in front of the sensors, the measurements are repeated. In the experimental program, the tests where the second layer antifer unit was in front of the sensors are referred to as experiments A; on the other hand, the tests where the first layer antifer units were not in front of the sensors are referred to as experiments B. For example, for double pyramid 2 (DP2) case, these two different conditions are denoted as DP2A and DP2B. An example of the different pressure measurements in the sensors caused by this change in the position of the antifer in front of the sensor is shown in Figure 3.11.

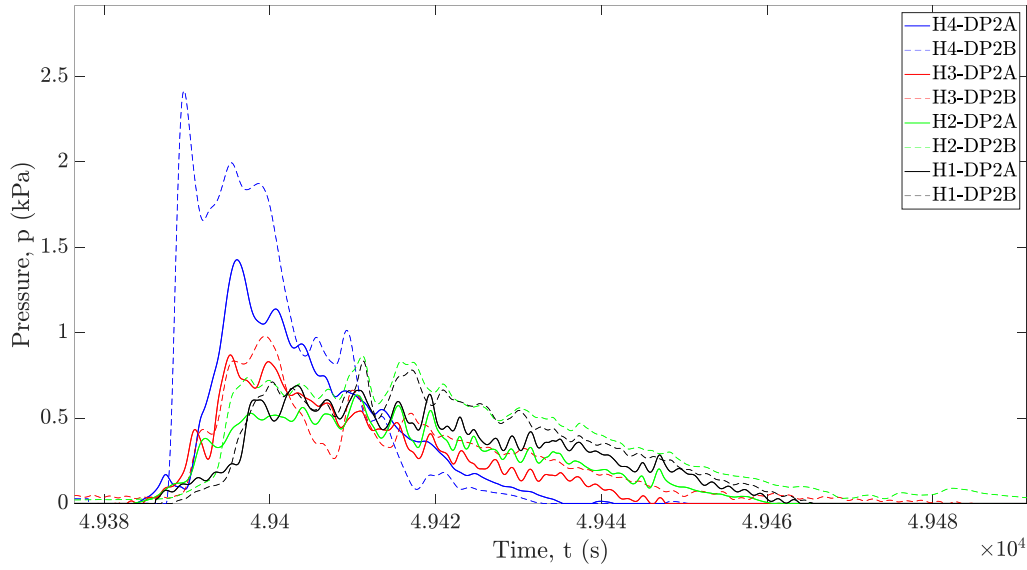


Figure 3.11. Comparison of the maximum wave pressure of the DP2 antifer placement Method for the A and B arrangement type.

As can be seen from this figure, in the DP2A placement method, the recordings at these sensors are smaller due to the second layer antifer unit covers the sensor as can be seen from Figure 3.10a, while in the DP2B placement method, larger pressure recordings were taken at the H3 and H4 sensors due to the first layer antifer unit placed in front of the pressure sensor apparatus as shown in Figure 3.10b.

Six pressure measurements at the base were taken in two repetitions for three different crown wall heights according to the position of the aluminium apparatus with four sensors in the experimental programme. For the reason of the necessity of repetitions, 768 tests were performed on 144 different experiments in total. The four different locations of the aluminium apparatus on the crown wall are shown in Figure 3.12.

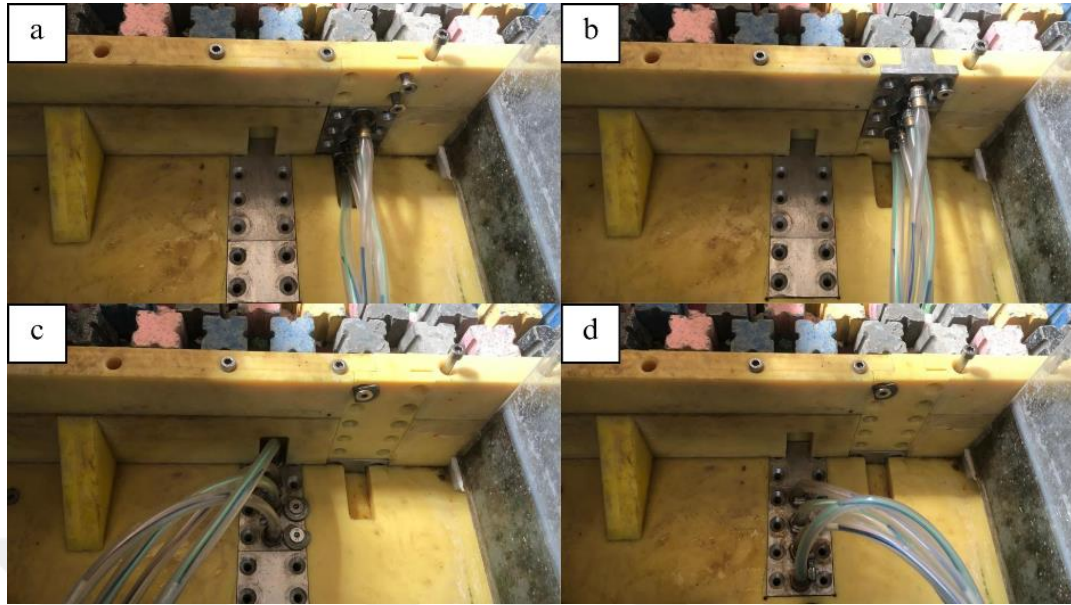


Figure 3.12. Pressure sensor apparatus positions. (a) Vertical1 [V1], (b) Vertical2 [V2], (c) Horizontal1 [H1], (d) Horizontal2 [H2].

As can be seen from Figure 3.12, the code V2 indicates that the upper quadruple pressure was measured on the front face of the crown wall model, while the code V1 indicates that the lower quadruple pressure was measured. The H1 code represents that the front quadruple pressure is measured at the base of the crown wall, while the H2 code indicates that the rear quadruple pressure is measured at the base of the crown wall. Images of the sections placed in the wave flume during the experiments and detailed information about the placement methods are given in Table 3.6 respectively.

Table 3.6. Antifer placement methods

(a) CP1 Placement Method	
	
(a-1) CP1 placement method armour layer	(a-2) CP1 placement method section view
CP1 Placement Method Features $X/D_n = 1.70$, $Y/D_n = 1.14$, Armour layer thickness, $t = 10.2$ cm, $\gamma_s = 54.8\%$	
(b) CP2 Placement Method	
	
(b-1) CP2 placement method armour layer	(b-2) CP2 placement method section view
CP2 Placement Method Features $X/D_n = 1.87$, $Y/D_n = 1.09$, Armour layer thickness, $t = 9.5$ cm, $\gamma_s = 49.8\%$	

<i>Table 3.6 (continued)</i>	
(c) IR1 Placement Method	
	
(c-1) IR1 placement method armor layer	(c-2) IR1 placement method section view
<i>IR1 Placement Method Features</i> Armour layer thickness, $t = 10.8 \text{ cm}$, $\gamma_s = 57.1\%$	
(d) IR2 Placement Method	
	
(d-1) IR2 placement method armor layer	(d-2) IR2 placement method section view
<i>IR2 Placement Method Features</i> Armour layer thickness, $t = 10.8 \text{ cm}$, $\gamma_s = 61.1\%$	

<i>Table 3.6 (continued)</i>	
(e) DP1 Placement Method	
	
(e-1) DP1 placement method armour layer	(e-2) DP1 placement method section view
<i>DP1 Placement Method Features</i> $X/D_n = 1.70$ $Y/D_n = 1.12$, Armour layer thickness, $t = 10.4$ cm, $\gamma_s = 54.8\%$	
(f) DP2 Placement Method	
	
(f-1) DP2 placement method armour layer	(f-2) DP2 placement method section view
<i>DP2 Placement Method Features</i> $X/D_n = 1.70$ $Y/D_n = 1.12$, Armour layer thickness, $t = 10.4$ cm, $\gamma_s = 54.8\%$, $\frac{1}{2}D_{n50}$ arrangement along the slope.	

Table 3.6 (continued)	
(g) DP3 Placement Method	
	
(g-1) DP3 placement method armor layer	(g-2) DP3 placement method section view
<i>DP3 Placement Method Features</i> $X/D_n = 1.40$ $Y/D_n = 1.12$, Armour layer thickness, $t = 10.4$ cm, $\gamma_s = 61.5\%$, $\frac{1}{2}D_{n50}$ staggered arrangement along the slope.	
(h) DP4 Placement Method	
	
(h-1) DP4 placement method armor layer	(h-2) DP4 placement method section view
<i>DP4 Placement Method Features</i> $X/D_n = 1.40$ $Y/D_n = 1.12$, Armour layer thickness, $t = 10.4$ cm, $\gamma_s = 61.5\%$	

3.1.5 Analysis of the Pressure Recordings

3.1.5.1 Processing And Filtering of Pressure Recordings

The analyses of the pressure measurements are performed after applying the test programme given in Table 3.6 in accordance with the position and orientation of the pressure sensors shown in Figure 3.8. To determine the horizontal and vertical wave forces acting on the crown wall model, point pressure measurements were performed on the model. With the calibration range offered by the sensor manufacturer KISTLER, the sensors were calibrated to take pressure measurements with higher sensitivity and within a suitable temperature range, and the measurements were taken with the minimum sampling rate of 12.5 kHz offered by the company.

According to Lamberti et al. (2011), it is recommended to consider to the relationship between the sampling rate of the sensors and the distances between the sensors in experiments where pressure measurements and wave forces are calculated. According to this study, firstly, the raw signals obtained from the sensors are filtered by band-pass filtering for different frequency ranges. These frequency ranges are given as 0-12.5 Hz, 12.5-50 Hz, 50-500 Hz, and 500 Hz and larger. Then, for each sensor, the cross-correlation of the filtered signals between the sensors is analysed. After a certain sampling rate, the cross-correlation coefficients between sensors drop significantly. A similar analysis was performed for the experiments conducted within the scope of this study. The results of the analysis are given in Figure 3.13.

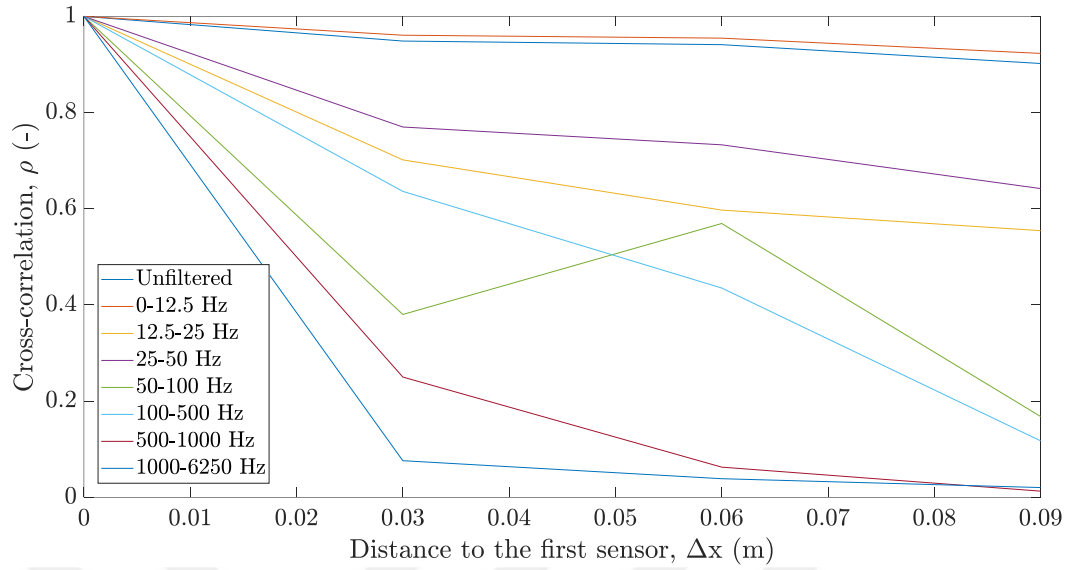


Figure 3.13. Results of the cross-correlation analysis between the pressure sensors.

As can be seen from the figure above, the cross-correlation for the first sensor gives the auto-correlation of the sensor. After 500 Hz levels, it is seen that the correlation between the sensors drops below 0.3. For this reason, the sampling rate of the pressure measurement data taken in the experiments was reduced from 12.5 kHz to 1250 Hz. In addition, looking at the spectral distribution of the raw data, noise was detected at 50 Hz and multiples (100 Hz, 150 Hz, etc.). For this reason, the measurements taken were also subjected to a Butterworth type low-pass filter starting from 50 Hz.

In the DP2A antifer placement method, comparison of the raw pressure data and the processed pressure data of a wave are shown in Figure 3.14 as an example.

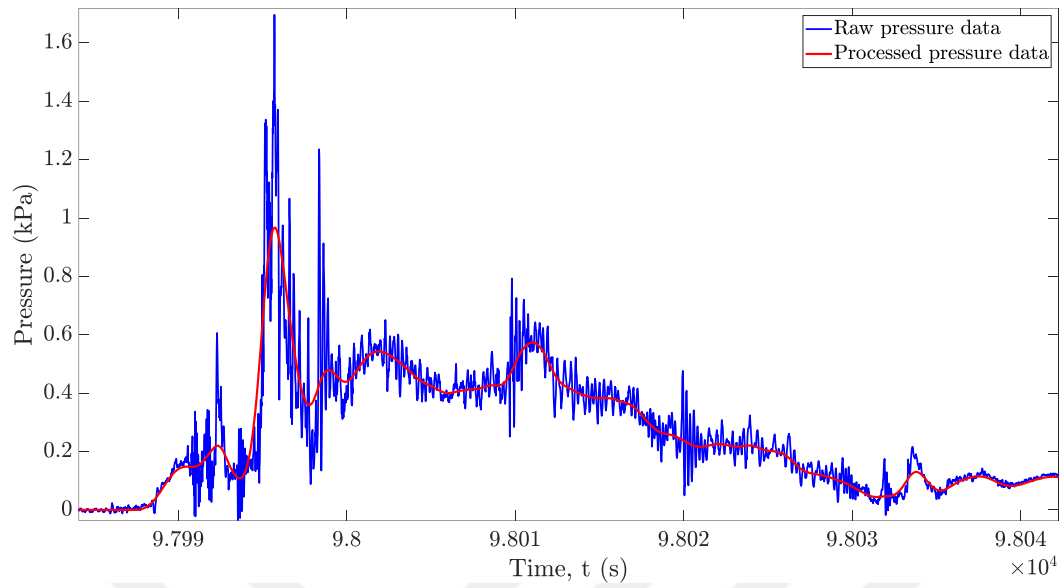


Figure 3.14. Time dependent variation of the pressure obtained from one sensor during a single wave. Raw data is shown by blue line, processed data is shown by red line.

3.1.5.2 Calculation of Wave Forces and Moment from Pressure Recordings

Looking at the formulations presented in the LITERATURE REVIEW, it was decided that the formulation that could be used most appropriately for the scope of the experiments in this study was the formulation of Norgaard et al. (2013), which is a modified version of the Pedersen (1996) method for shallow water. Accordingly, for the application of these formulations valid in shallow water, the run-up formulation based on $H_{0.1\%}$ was used proposed by Norgaard et al. (2013), instead of the wave run-up formulation of Van der Meer and Stam (1992).

The pressure measurements which were processed from the raw data according to this logic were used to calculate the horizontal and vertical wave forces acting on the crown wall and to determine the moment due to these forces. Referring to the force denomination of forces shown in Figure 3.8, the wave forces were estimated by extrapolation, using the measurements obtained at the vertical front of the crown with the lowest H0 and the highest H5 (H6 or H7, depending on the cases where R_c is increased) and at the base of the crown with V0 on the seaward side and V7 on the

landward side. Norgaard et al. (2013) used various approaches to estimate forces from read pressure data. The pressure distribution was assumed to vary linearly between the points where the sensors take measurements. The forces acting on these surfaces and the moments created by these forces at the point O shown in Figure 3.8 were calculated by forming the integral of these pressure distributions. The calculation of the pressures at the tip points of the crown wall by linear extrapolation is shown in Eq. (3.10). The calculation of the horizontal force at each sensor and the total horizontal force acting on the crown wall are given in Eq. (3.11) and Eq. (3.12). The calculation of the overturning moment at each sensor due to the horizontal force and the total moment acting on point O are given in Eq. (3.13) and Eq. (3.14).

$$H_0 = H_1 - \frac{H_2 - H_1}{h_2} h_1 \quad H_5 = H_4 - \frac{H_3 - H_4}{h_4} h_5 \quad (3.10)$$

$$\Delta F_{H,hn} = \frac{1}{2} h_n (H_{n-1} + H_n) \quad (3.11)$$

$$F_H = \sum_{n=1}^{n=5} \Delta F_{H,hn} \quad (3.12)$$

$$\Delta M_{F_{H,hn}} = \frac{1}{2} h_n \left[h_n \left(\frac{1}{3} H_{n-1} + \frac{2}{3} H_n \right) + (H_{n-1} + H_n) \sum_{i=1}^{n-1} h_i \right] \quad (3.13)$$

$$M_{F_{H,hn}}^O = \sum_{n=1}^{n=5} \Delta M_{F_{H,hn}} \quad (3.14)$$

In the equations given above, H_i is the pressure value (in kPa) at the point where the measurement is made (or estimated), h_i is the distance (in meters) between the sensors (H_i), $\Delta F_{h,hn}$ is the force value (in kN/m) per unit width obtained by taking the integral of the pressure distribution between the two sensors, F_H is the total force acting on the surface per unit width, $\Delta M_{F_{H,hn}}$ is the moment (in kN.m/m) of the force between the two sensors with respect to point O per unit width, and $M_{F_{H,hn}}^O$ represents

the moment (in kN.m/m) of the total force acting on the surface with respect to point O for unit width.

Norgaard et al. (2013) determined the wave forces by averaging the sensors at the same elevation level. In this study, the results of the previously mentioned A and B type antifer placements on the vertical front face of the crown wall were used to calculate the wave forces by averaging the sensors at the same level. In this way, the effect of repetition experiments measured with the same sensors is reflected and the effects of different positions of the antifer units in front of the sensors are also considered. The above formulations presented by Norgaard et al. (2013) were used to estimate the vertical forces from the pressure readings in the same way as for the horizontal forces.

In the cases where measurements were taken by shifting the apparatus containing the sensors (H1 and H2, V1 and V2), the pressure magnitudes applied by the waves were checked by comparing the pressure values read by the sensors coinciding in location. Figure 3.15 shows a plot of the pressure sensor readings for the DP2A placement method at a wave interaction time with the crown wall base slab when the maximum base pressure is measured.

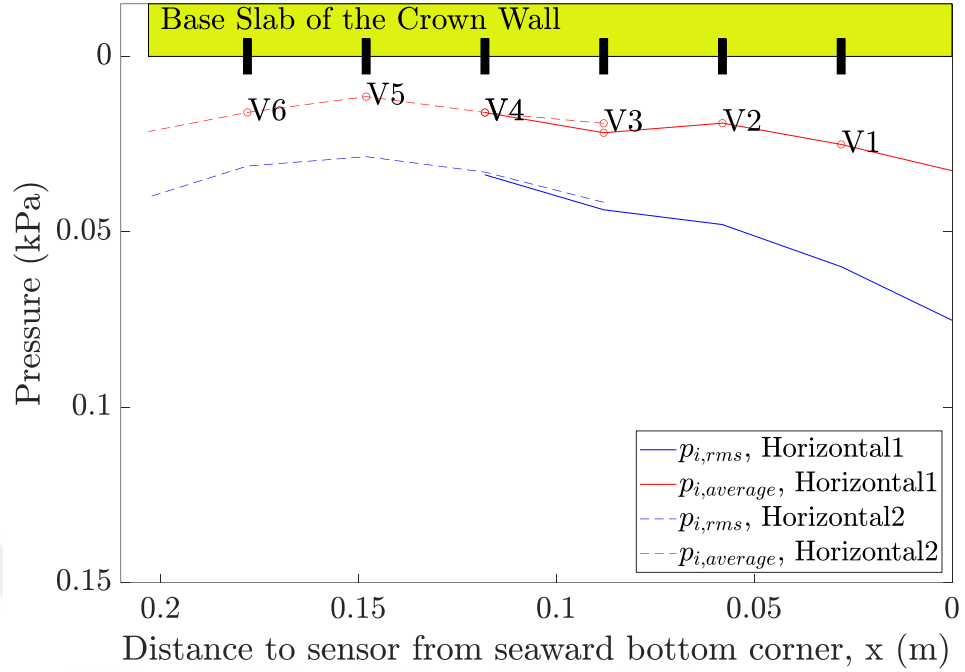


Figure 3.15. An example of average vertical base pressure measurements during the wave loading action at the crown wall base slab for the DP2A placement.

As can be observed in Figure 3.15, which is shown as an example of the case where the changes in the Horizontal1 and Horizontal2 positions of the apparatus are made as shown in Figure 3.12c and Figure 3.12d, the pressure readings V3 and V4 can be taken at twice. In all experiments, as in this example, the differences between the overlapping pressure sensor readings were checked and care was taken to ensure that the pressure difference between the two sensors was maximum 10%. In the repetition experiments when the apparatus was in horizontal and vertical positions, after checking that the difference between the overlapping sensor readings was minimum, their averages were also taken among themselves.

3.1.5.3 Modification of the Empirical Coefficients in Norgaard et al. (2013) Formulation for the Antifer Units

Various statistical analyses were performed while comparing the wave forces calculated from the pressure recording during the experiments with the formulations

of Pedersen (1996) and Norgaard et al. (2013). Linear regression and statistical analyses were performed for the horizontal wave forces calculated using Eq. (2.17), the base pressure calculated using Eq. (2.23) and the tilting moments due to the horizontal wave forces found using Eq. (2.25). The best fit was assigned to the wave pressure data to modify empirical coefficients. The formulations of the applied statistical approaches are given below.

3.1.5.3.1 Root Mean Square Error

To reflect the accuracy of the fit of the formulation to the measured pressure data in the experiments, root mean square error (*RMSE*) method is used. This approach takes the average distance between the measured value and the predicted value of the pressure recordings. This indicates the how far measured values from the regression line. *RMSE* can be calculated as shown in Eq. (3.15).

$$RMSE = \sqrt{\sum_{i=1}^m (w_{measured,i} - w_{prediction,i})^2 / m} \quad (3.15)$$

In Eq. (3.15), *m* shows the number of data, and $w_{measured,i}$ is the *i*th data obtained from the experiments. Calculated *i*th value from the formulations corresponds $w_{prediction,i}$.

3.1.5.3.2 Coefficient of Variation Root Mean Square Error

Coefficient of variation root mean square error, *CVRMSE*, is used to dismiss scale dependencies of the variables while having a wide range of values. Higher *CVRMSE* values means obtained data have less accurate and have more variety. On the other hand, lower *CVRMSE* indicates more accurate and reliable distribution of the data. *CVRMSE* of the pressure recordings is calculated as following Eq. (3.16).

$$CVRMSE = \frac{RMSE}{w_{measured}} \quad (3.16)$$

In Eq. (3.16), $\bar{w}_{measured}$ means the average value of the measurements of the wave pressures.

3.1.5.3.3 Mean Absolute Error

Mean absolute error, MAE , indicates the errors between the measured values and the predicted values. One can also say that MAE is the ratio of the summation of the absolute errors to the sample size. While MAE shows the linear control over the data sample, $RMSE$ is relied on quadratic control over the sample. In other words, all the errors are weighted equally in MAE . However, $RMSE$ brings more weight to the large errors. MAE can be calculated as shown in following Eq. (3.17).

$$MAE = \sum_{i=1}^m \frac{|w_{measured,i} - w_{prediction,i}|}{m} \quad (3.17)$$

3.1.5.3.4 Normalized Mean Absolute Error

Normalized mean absolute error, $NMAE$, eliminates the scale dependencies of MAE of the data sample like what $CVRMSE$ does to $RMSE$. Formulation of the $NMAE$ is given in Eq. (3.18).

$$NMAE = \frac{MAE}{\sigma(w_{measured})} \quad (3.18)$$

3.1.5.3.5 Statistical BIAS

$BIAS$ is the tendency showing over or underestimation of the regression between the predicted and measured values in data analysis. $BIAS$ can be determined as shown in Eq. (3.19).

$$BIAS = \sum_{i=1}^m (w_{measured,i} - w_{prediction,i})/m \quad (3.19)$$

3.1.5.3.6 Statistical Normalized BIAS

Normalized *BIAS* which eliminates the scale dependencies of the *BIAS* can be evaluated as shown in Eq. (3.20).

$$Norm.BIAS = \frac{BIAS}{\sigma(w_{measured})} \quad (3.20)$$

By fitting the regression line to the measured pressure data, statistical assessment is completed using above mentioned approaches to determine the empirical coefficients in Pedersen (1996) and Norgaard et al. (2013) formulations. Lower RMSE (with *CVRMSE*) and MAE (with *NMAE*) values accepted to show better performance than the higher ones.

CHAPTER 4

ANALYSES OF THE EXPERIMENTAL MEASUREMENTS AND DISCUSSION OF RESULTS

In this chapter, the results of the present experimental study in which pressure measurements were performed on the crown wall of a rubble mound breakwater model are given. In the results, the magnitudes of the horizontal and vertical wave pressures are examined, the empirical coefficients in the formulation of Norgaard et al. (2013) are modified for the different placement methods of antifer units. The most important reason for taking Norgaard et al. (2013) as a reference for the present study is that it is a revised version of the Pedersen (1996) method for shallow water condition, which is the most widely used crown wall design formula today. Since present study is based on shallow water conditions, Norgaard et al. (2013) approach for stone armour units is extended for antifer units by modifying related empirical coefficients. With the statistical assessment tools mentioned in Chapter 3, the best fit to the data sample was assigned through analyses between the data measured in the experiments and the values calculated with Norgaard et al. (2013). Coefficient alternatives presented for these fits were shown and plotted.

4.1 Analyses of the $F_{h0.1\%}$ Measurements

4.1.1 Modification of Empirical Coefficients for Antifer Unit Placements

Modification of the empirical coefficients were started from the $F_{h0.1\%}$ formulation because horizontal wave forces also effect $M_{h0.1\%}$ significantly. It is noted that $F_{h0.1\%}$ formulation is for Pedersen (1996) and Norgaard et al. (2013) given in Eq. (2.17). The methodology followed to modify the empirical coefficients is outlined below:

- A cross table for empirical coefficient pairs a and b is prepared (containing 15100 pairs), and the statistical parameters given in Section 3.1.5 are computed using the actual measurements and the corresponding predictions that are calculated using the Norgaard et al. (2013) approach with modified a and b parameters considered in the cross-table.
- The a and b pairs that give the minimum statistical parameters, i.e. giving less errors in the predictions, are selected as the modified empirical coefficients $a_{modified}$ and $b_{modified}$. In other words, these modified empirical coefficients ($a_{modified}$ and $b_{modified}$) give the best fit to the measurements and referred to as present study, hereafter.
- Then, the statistical measures given in Section 3.1.5 are also evaluated for the predicted values using Pedersen (1996) and Norgaard et al. (2013) with original coefficients to show the improvement in the predictions of $F_{h0.1\%}$ using Norgaard et al. (2013) approach with modified coefficients.
- $F_{h0.1\%,measured}$ versus $F_{h0.1\%,calculated}$ plots are also presented to compare the performance of the $a_{modified}$ and $b_{modified}$ with the original coefficients.

In Table 4.1, results of the statistical analysis of the predicted and measured values of horizontal wave forces obtained from the experimental study is shown. Modified a and b empirical coefficients were also given for different antifer placement techniques and for all individual antifer placement units in Table 4.1.

Table 4.1. Statistical assessment of horizontal wave force measurements data to obtain the best fit for the modified empirical coefficients.

F_{H0.1%} Crown Wall Formulation									
<i>Formulation</i>	<i>Fit Type</i>	<i>a</i>	<i>b</i>	<i>RMSE (kN/m)</i>	<i>CVRMSE (-)</i>	<i>MAE (kN/m)</i>	<i>NMAE (-)</i>	<i>BIAS (kN/m)</i>	<i>N-BIAS (-)</i>
Pedersen (1996)	All Placements	0.21	1.60	0.07658	0.44574	0.06573	1.10827	0.06573	1.10826
	Closed Pyramid			0.08016	0.45130	0.07210	1.24981	0.07210	1.24981
	Irregular			0.06092	0.38736	0.00448	0.08351	0.04991	0.93062
	Double Pyramid			0.08347	0.46835	0.07270	1.15706	0.07270	1.15706
Norgaard et al. (2013)	All Placements	0.21	1.00	0.09076	0.52833	0.07925	1.33627	0.07925	1.33627
	Closed Pyramid			0.09433	0.53112	0.08539	1.48020	0.08539	1.48020
	Irregular			0.07503	0.47708	0.00686	0.12795	0.06398	1.19283
	Double Pyramid			0.09790	0.54932	0.08599	1.36857	0.08599	1.36857
Present Study	All Placements	0.36	1.31	0.03839	0.22347	0.02982	0.50290	0.00117	0.01967
	Closed Pyramid	0.39	1.10	0.03239	0.18092	0.02506	0.46144	0.00119	0.02186
	Irregular	0.32	1.39	0.03468	0.22052	0.00181	0.03366	0.00118	0.02198
	Double Pyramid	0.37	1.37	0.03938	0.22095	0.03034	0.48280	0.00129	0.02051

On the one hand, a best fit was assigned to all the different antifer placement methods as a whole; on the other hand, a fit study was performed individually for each different placement method. The results show that the coefficient a determined by Pedersen (1996) for most of the stones and several artificial armour units, and Norgaard et al. (2013) for the use of natural rock units in the armour layer should be increased for antifer units. In addition, for the b coefficient, which was changed from 1.60 to 1.00 for natural rock units in Norgaard et al. (2013), a new b coefficient was suggested for each different antifer placement method to be between these two ranges. Table 4.1 shows that Pedersen (1996) provides more accurate estimates for antifer units than Norgaard et al. (2013) with original coefficients. *RMSE* value obtained by comparing the prediction of Pedersen (1996) with the measured values of the present study is 15.6% lower than Norgaard et al. (2013). Similarly, when Pedersen (1996) and Norgaard et al. (2013) were compared with the measured data of the present study, *MAE* value of Pedersen (1996) was 17.1% lower than Norgaard et al. (2013). Looking at the *BIAS* and normalised *BIAS* values in Table 4.1, it is observed that the horizontal forces calculated with the formulae of Pedersen (1996) and Norgaard et al. (2013) underestimate the measured results.

In Figure 4.1, the horizontal wave forces at the front face of the breakwater measured in the experiments, the horizontal wave forces calculated using the formula of Pedersen (1996), Norgaard et al. (2013) with original coefficients and Norgaard et al. (2013) with the modified coefficients (present study) are compared with the present horizontal wave force measurements for all antifer unit placements.

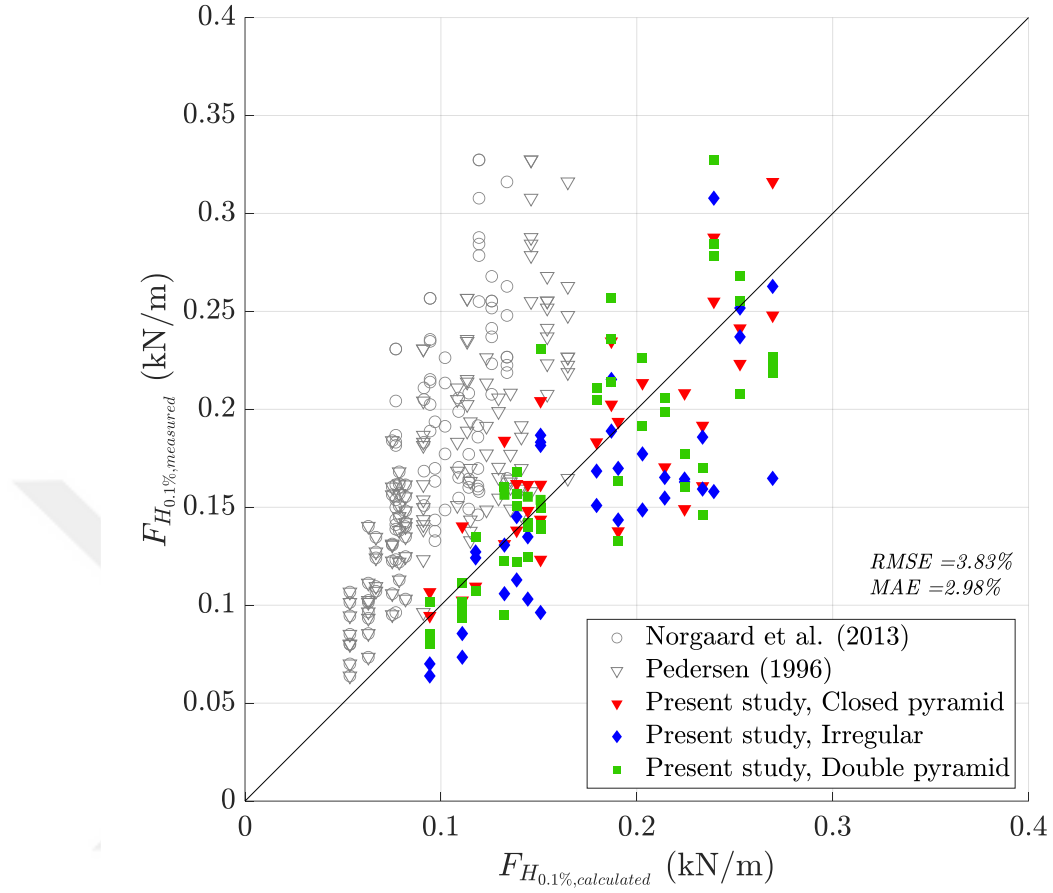


Figure 4.1. Comparison of Norgaard et al. (2013) and Pedersen (1996) formulation with all antifer placement method measurements. (*RMSE* and *MAE* values show the performance of present study with respect to Norgaard et al. (2013) formula with original coefficients.)

Figure 4.1 shows the values corresponding to the measured horizontal wave forces for the CP, IR, and DP placement methods. The *RMSE* and *MAE* values shown in Figure 4.1 are statistical analyses obtained by comparing the *a* and *b* coefficients of Norgaard et al. (2013) and the present study. As mentioned earlier, the coefficients *a* and *b* used for natural rocks in the formulations of Norgaard et al. (2013) and Pedersen (1996) underestimate the scenarios where antifer units are used. As a result of the search for the best fit, $a_{modified}$ and $b_{modified}$ are determined as 0.36 and 1.31 respectively for the calculation of horizontal forces acting on the crown wall for

breakwater sections with armour layer designed with antifer units regardless of the placement method. As can be seen on the plot in Figure 4.1, the use of $a_{modified}$ and $b_{modified}$ gives a better fit when considered together with the results in Table 4.1. This modification clearly improves the performance of the $F_{h0.1\%}$ formula of Pedersen (1996) and Norgaard et al. (2013) for antifer units. When the statistical performance assessment results in Table 4.1 are analysed, the *RMSE* value of the $a_{modified}$ and $b_{modified}$ fit obtained with the modification of the present study is 49.9% lower than Pedersen (1996) and 57.7% lower than Norgaard et al. (2013). In terms of *MAE* values, the present study showed a 54.6% decrease compared to Pedersen (1996) and a 62.4% decrease compared to Norgaard et al. (2013).

In Figure 4.2, the horizontal wave forces at the front face of the breakwater measured in the experiments, the horizontal wave forces calculated using the formula of Pedersen (1996), Norgaard et al. (2013) with original coefficients and present study are compared with the present horizontal wave force measurements for CP placement method.

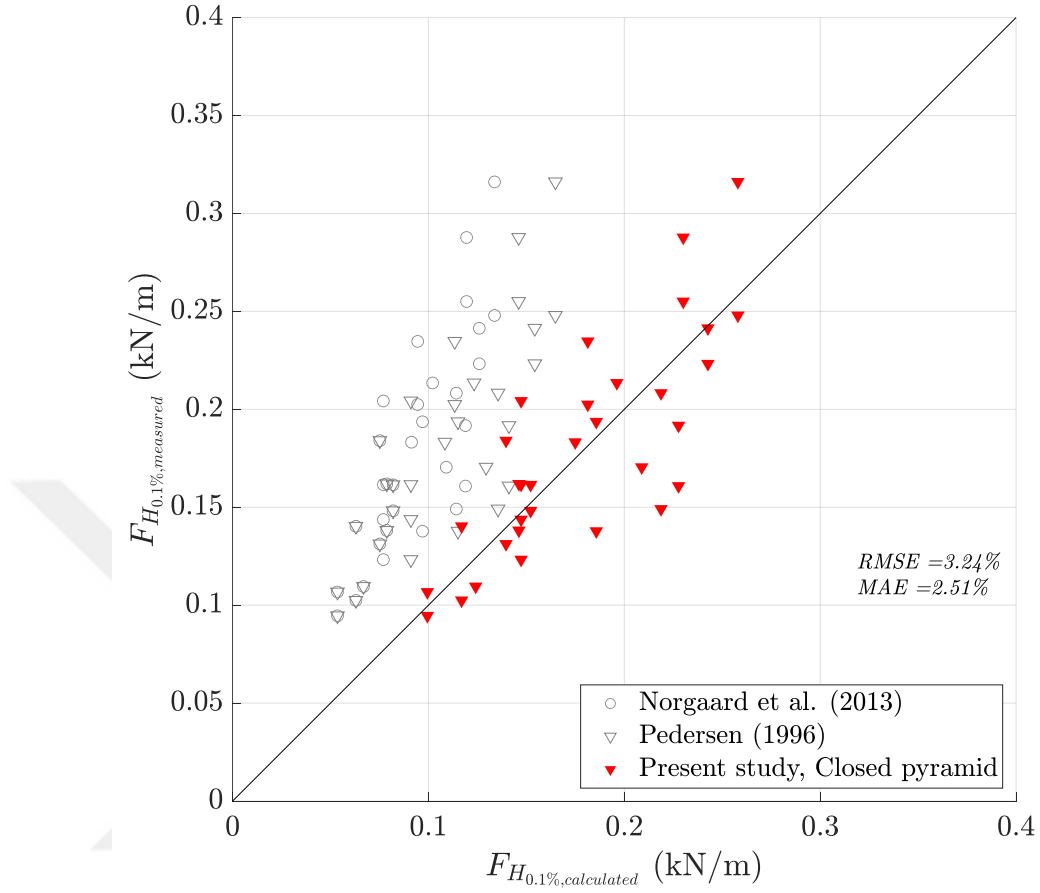


Figure 4.2. Comparison of Norgaard et al. (2013) and Pedersen (1996) formulation with CP placement method measurements. ($RMSE$ and MAE values show the performance of present study with respect to Norgaard et al. (2013) formula with original coefficients.)

Figure 4.2 shows the values corresponding to the measured horizontal wave forces only for the CP placement method. The $RMSE$ and MAE values shown in Figure 4.2 are statistical analyses obtained by comparing the a and b coefficients of Norgaard et al. (2013) and the present study. As a result of the fit obtained by comparing the measured horizontal wave force values obtained from CP1 and CP2 experiments with the predicted values from Norgaard et al. (2013) and Pedersen (1996) formulae, $a_{modified,CP}$ and $b_{modified,CP}$ values were obtained as 0.39 and 1.10, respectively. As can be seen on the plot in Figure 4.2, the use of $a_{modified,CP}$ and $b_{modified,CP}$ for the CP

placement method gives a better fit when considering together with the results in Table 4.1. This modification clearly improves the performance of the $F_{h0.1\%}$ formula of Pedersen (1996) and Norgaard et al. (2013) for CP placement method. When the statistical performance assessment results in Table 4.1 are analysed, the *RMSE* value of the $a_{modified,CP}$ and $b_{modified,CP}$ fit obtained with the modification of the present study is 59.6% lower than Pedersen (1996) and 65.7% lower than Norgaard et al. (2013). In terms of *MAE* values, the present study showed a 65.2% decrease compared to Pedersen (1996) and a 70.6% decrease compared to Norgaard et al. (2013).

In Figure 4.3, the horizontal forces at the front face of the breakwater measured in the experiments and the horizontal forces calculated using the formula of Pedersen (1996), Norgaard et al. (2013) with original coefficients and present study are compared with the present horizontal wave force measurements for IR placement method.

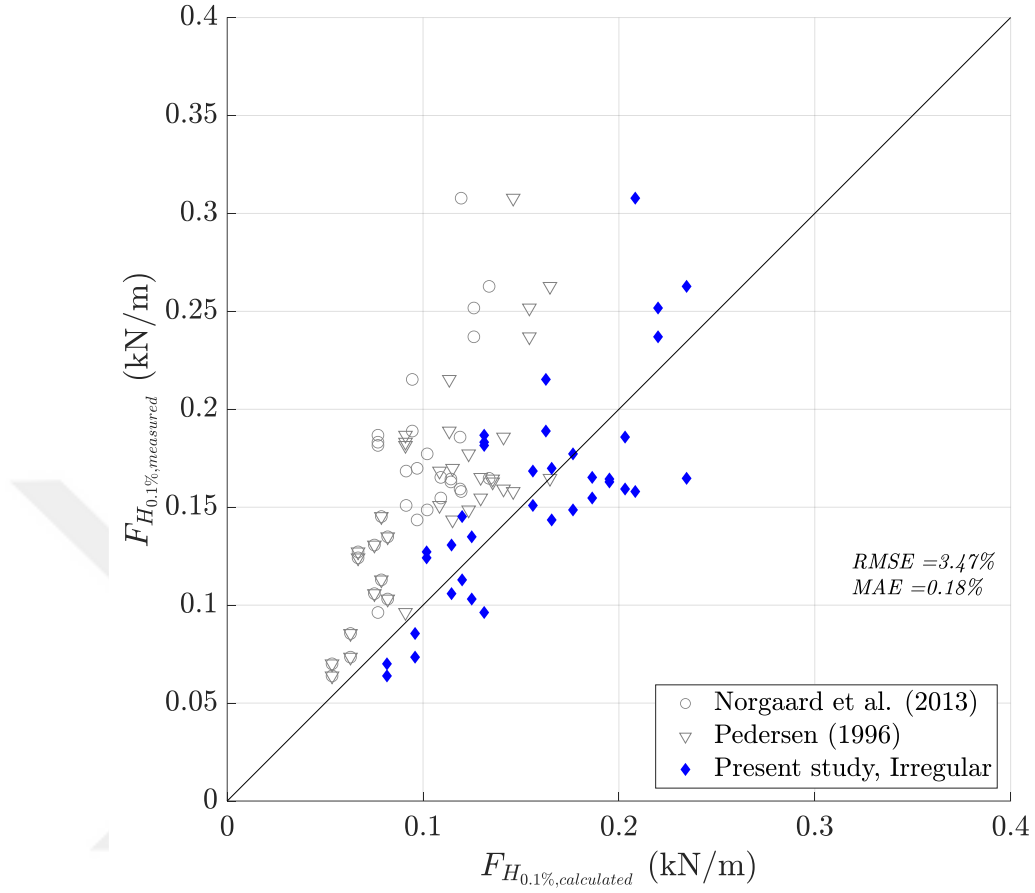


Figure 4.3. Comparison of Norgaard et al. (2013) and Pedersen (1996) formulation with IR placement method measurements. ($RMSE$ and MAE values show the performance of present study with respect to Norgaard et al. (2013) formula with original coefficients.)

Figure 4.3 shows the values corresponding to the measured horizontal wave forces only for the IR placement method. The $RMSE$ and MAE values shown in Figure 4.3 are statistical analyses obtained by comparing the a and b coefficients of Norgaard et al. (2013) and the present study. As a result of the fit obtained by comparing the measured horizontal wave force values obtained from IR1 and IR2 experiments with the predicted values from Norgaard et al. (2013) and Pedersen (1996) formulae, $a_{modified,IR}$ and $b_{modified,IR}$ values were obtained as 0.32 and 1.39, respectively. As can be seen on the plot in Figure 4.3, the use of $a_{modified,IR}$ and $b_{modified,IR}$ for the IR

placement method gives a better fit when considering together with the results in Table 4.1. This modification clearly improves the performance of the $F_{h0.1\%}$ formula of Pedersen (1996) and Norgaard et al. (2013) for IR placement method. When the statistical performance assessment results in Table 4.1 are analysed, the *RMSE* value of the $a_{modified,CP}$ and $b_{modified,CP}$ fit obtained with the modification of the present study is 59.6% lower than Pedersen (1996) and 65.7% lower than Norgaard et al. (2013). In terms of *MAE* values, the present study showed a 65.2% decrease compared to Pedersen (1996) and a 70.6% decrease compared to Norgaard et al. (2013).

In Figure 4.4, the horizontal forces at the front face of the breakwater measured in the experiments and the horizontal forces calculated using the formula of Pedersen (1996), Norgaard et al. (2013) with original coefficients and present study are compared with the present horizontal wave force measurements for DP placement method.

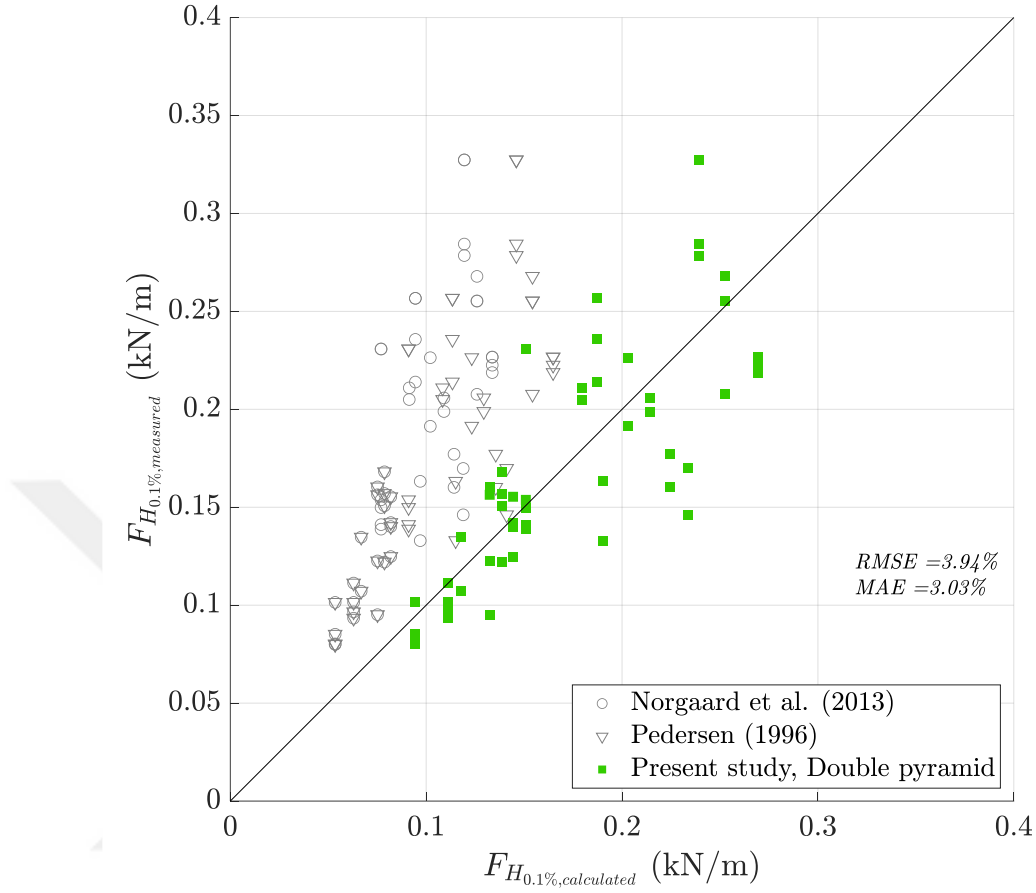


Figure 4.4. Comparison of Norgaard et al. (2013) and Pedersen (1996) formulation with DP placement method measurements. ($RMSE$ and MAE values show the performance of present study with respect to Norgaard et al. (2013) formula with original coefficients.)

Figure 4.4 shows the values corresponding to the measured horizontal wave forces only for the DP placement method. The $RMSE$ and MAE values shown in Figure 4.4 are statistical analyses obtained by comparing the a and b coefficients of Norgaard et al. (2013) and the present study. As a result of the best fit obtained by comparing the measured horizontal wave force values obtained from DP1, DP2, DP3 and DP4 experiments with the predicted values from Norgaard et al. (2013) and Pedersen (1996) formulae, $a_{modified,DP}$ and $b_{modified,DP}$ values were obtained as 0.37 and 1.37, respectively. As can be seen on the plot in Figure 4.4, the use of $a_{modified,DP}$ and

$b_{modified,DP}$ for the DP placement method gives a better fit when considering together with the results in Table 4.1. This modification clearly improves the performance of the $F_{h0.1\%}$ formula of Pedersen (1996) and Norgaard et al. (2013) for DP placement method. When the statistical performance assessment results in Table 4.1 are analysed, the *RMSE* value of the $a_{modified,DP}$ and $b_{modified,DP}$ fit obtained with the modification of the present study is 52.8% lower than Pedersen (1996) and 59.8% lower than Norgaard et al. (2013). In terms of *MAE* values, the present study showed a 58.3% decrease compared to Pedersen (1996) and a 64.7% decrease compared to Norgaard et al. (2013).

4.1.2 Effects of the Packing Density

In this section, an analysis is made to see whether there is a change in the packing density induced wave forces in each different placement method of the antifer units. To achieve this, the maximum horizontal wave force measurements obtained from CP1 and CP2, IR1 and IR2, DP1-DP2 and DP3-DP4 are compared respectively.

In CP placement method, packing density values were determined as 54.8% for CP1 and 49.8% for CP2. Looking at the $F_{h0.1\%}$ measurements, differences up to 30% (15% on average) are observed. IR placement method has two different packing densities such as 57.1% for the IR1 and 61.1% for the IR2. $F_{h0.1\%}$ results indicate that differences between these packing densities are up to 38% (11.3% on average). For the DP placement method, packing density values were assigned as 54.8% for DP1 and DP2, and 61.5% for DP3 and DP4. Comparison of the $F_{h0.1\%}$ measurements shows that differences between packing densities of DP placement methods are up to 35% (3% to 8.5% on average).

The abovementioned differences are sometimes in favour of CP1 (similarly for IR1, DP1, DP2) and sometimes in favour of CP2 (similarly for IR2, DP3, DP4). No patterns can be identified considering the wave series and geometrical changes in the cross-sections within the scope of the present measurements. In other words, there is

no meaningful distinction in the maximum horizontal wave forces regarding the change in packing density. Therefore, no curve-fitting analysis and statistical assessments are carried out for the effect of packing density.

An increase in packing density decreases the up-rushing wave reaching the front face of the crown wall for the protected layer and reduces the wave pressure, whereas an increase in packing density as it is a poor factor in dissipating the energy of the up-rushing wave in scenarios where R_c is greater than A_c may increase the impact pressure.

4.1.3 Effects of Staggered Placement

In this section, it is observed whether the staggered array has any effect on the wave forces when the packing density is constant. To examine this, DP1 and DP2, DP3 and DP4, which have the same packing density but different properties in terms of staggered array, are compared among themselves.

Comparing the results of DP1 with a normal arrangement and DP2 with a staggered array, $F_{h0.1\%}$ measurement differences of around 6.3% on average are observed. However, DP1 and DP2 did not have any superiority over each other under the effect of staggered array in the measurement of horizontal wave forces. The largest difference observed was around 22.4% in the scenario where R_c was 6 cm more than A_c . Nevertheless, since higher pressure readings were taken in some cases in DP1 and in some cases in DP2, the effect of staggered arrangement was not observed in this placement method. When the results of DP4 with normal array and DP3 with staggered array are compared, $F_{h0.1\%}$ measurement differences are observed around 11.8% on average. In the same way as in the comparison of DP1 and DP2, DP3 and DP4 do not have any advantage over each other under the effect of staggered array in the measurement of horizontal wave forces. The largest difference observed was around 23.6% in the scenario where R_c was 6 cm more than A_c . Nevertheless, since

higher pressure readings were taken in DP3 in some cases and in DP4 in some cases, the effect of staggered arrangement was not observed in this placement method.

The abovementioned differences are sometimes in favour of DP1 and DP4 (placement methods without staggered arrangement) and sometimes in favour of DP2 and DP3 (placement methods with staggered arrangement). No patterns can be identified considering the wave series and staggered arrangement in the cross-sections within the scope of the present measurements. In other words, there is no meaningful distinction in the maximum horizontal wave forces regarding the change in staggered placement of the antifer units. Therefore, no curve-fitting analysis and statistical assessments are carried out for the effect of packing density.

4.1.4 General Observation on $F_{h0.1\%}$ for Antifer Units

The results showed that the IR placement method generally requires designing with the lowest measured wave forces acting on the crown wall. In cases where R_c and A_c were equal, the measured wave force values were very close for all placement methods, but with increasing R_c , the wave forces increased for DP and CP arrangements. It can be seen from Figure 4.1 that as R_c gradually increases, the intervals between the scatters start to increase depending on the randomness of the impact pressure. When all the different antifer placement methods were analysed, it was observed that the crown wall was subjected to more wave forces in CP and DP placements than IR in the order of 15-30% on average. CP placement method is generally subjected to larger wave forces than the IR placement method. Looking at the $F_{h0.1\%}$ measurements of IR and CP placement methods, differences up to 52.7% (27.7% on average) are observed. In addition, an increase in horizontal wave forces was also observed in the DP method compared to the IR placement method, although not as much as the CP. When $F_{h0.1\%}$ measurements of IR and DP methods are compared, differences in horizontal wave forces up to 26.6% (18.4% on average) are observed. When CP and DP placement methods are compared, it is observed that the difference of $F_{h0.1\%}$ measurements of CP method is at most 27% (15.2% on average).

The general trend that can be deduced from these results is that the largest forces for the crown wall subjected to horizontal wave forces are observed with the CP, then DP and IR placement methods, respectively.

In conclusion, the empirical coefficients assessed for the formulae of Pedersen (1996) and Norgaard et al. (2013) underestimate the wave forces acting on the crown wall for the antifer placement method, mainly due to forming a surface having different roughnesses compared to the previous studies. Pedersen (1996) gives relatively better predictions for antifers than Norgaard et al. (2013), as can be observed in Table 4.1 and Figure 4.1.

4.2 Analyses of the $M_{h0.1\%}$ Measurements

4.2.1 Modification of Empirical Coefficients for Antifer Unit Placements

After the coefficients a and b in the Norgaard et al. (2013) formulae used to determine the $F_{h0.1\%}$ for the antifer placement methods were modified according to the experimental results, $M_{h0.1\%}$ predictions and measurements are compared. The methodology followed is outlined below:

- The overturning moments ($M_{h0.1\%}$) are calculated using the formula Eq. (2.25) given by Norgaard et al. (2013). In these calculations, $F_{h0.1\%}$ is computed with the modified coefficients $a_{modified}$ and $b_{modified}$.
- The statistical parameters given in Section 3.1.5 are evaluated for $M_{h0.1\%}$. These parameters are also calculated comparing the predictions using Pedersen (1996) and Norgaard et al. (2013) with original coefficients of $F_{h0.1\%}$ to show the improvements by the present approach.

It is noted that the formula for $M_{h0.1\%}$ is directly used, without any modifications, as it is already argued by Norgaard et al. (2013) that the relationship between the maximum horizontal force and the overturning moments are linear. To confirm this statement, a similar analysis performed for $F_{h0.1\%}$, i.e. searching for alternative

coefficients e_1 and e_2 in the overturning moment formula Eq. (2.25), is repeated, and it is seen that the coefficients given by Norgaard et al. (2013) are also applicable for the present study.

Table 4.2 shows the statistical performance assessment results of the overturning moments calculated using the formula of Pedersen (1996), Norgaard et al. (2013) with original coefficients and Norgaard et al. (2013) with the modified coefficients of $a_{modified}$ and $b_{modified}$ (referred to as present study) are compared with the present overturning moment measurements for all antifer unit placements.



Table 4.2. Statistical assessment of the moment measurements data to obtain the best fit for the modified empirical coefficients.

M_{H0.1%} Crown Wall Formulation									
<i>Formulation</i>	<i>Fit Type</i>	<i>e₁</i>	<i>e₂</i>	<i>RMSE (kNm/m)</i>	<i>CVRMSE (-)</i>	<i>MAE (kNm/m)</i>	<i>NMAE (-)</i>	<i>BIAS (kNm/m)</i>	<i>N-BIAS (-)</i>
Pedersen (1996)	All Placements	0.55		0.03709	2.32213	0.13750	18.82246	0.13739	18.80784
	Closed Pyramid			0.02123	1.22141	0.04249	5.85393	0.04249	5.85393
	Irregular			0.01590	1.09374	0.00069	0.10503	0.03072	4.68479
	Double Pyramid			0.02593	1.61122	0.06418	8.30269	0.06418	8.30269
Norgaard et al. (2013)	All Placements	0.95	0.40	0.04482	2.80571	0.17270	23.64067	0.17270	23.64067
	Closed Pyramid			0.02530	1.45531	0.05170	7.12195	0.05170	7.12195
	Irregular			0.02002	1.37711	0.00115	0.17561	0.04106	6.26223
	Double Pyramid			0.03111	1.93325	0.07994	10.34097	0.07994	10.34097
Present Study	All Placements	0.95	0.40	0.01980	1.23943	0.06672	9.13312	0.02516	3.44386
	Closed Pyramid			0.01096	0.63028	0.01979	2.72621	0.01315	1.81219
	Irregular			0.00882	0.60671	0.00069	0.10579	-0.00235	-0.35794
	Double Pyramid			0.01394	0.86585	0.03037	3.92938	0.01435	1.85645

Table 4.2 shows that Pedersen (1996) provides more accurate estimates for antifer units than Norgaard et al. (2013). *RMSE* value obtained by comparing the prediction of Pedersen (1996) with the measured values of the present study is 17.2% lower than Norgaard et al. (2013). Similarly, when Pedersen (1996) and Norgaard et al. (2013) were compared with the measured data of the present study, *MAE* value of Pedersen (1996) was 20.4% lower than Norgaard et al. (2013). Looking at the *BIAS* values, it is seen that Pedersen (1996) and Norgaard et al. (2013) greatly underestimate the present study $M_{h0.1\%}$ measurements, as expected from the $F_{h0.1\%}$ values.

In Figure 4.5, the overturning moments at the front face of the breakwater measured in the experiments, the overturning moments calculated using the formula of Pedersen (1996), Norgaard et al. (2013) with original coefficients and present study are compared with the overturning moment measurements for all antifer unit placements.

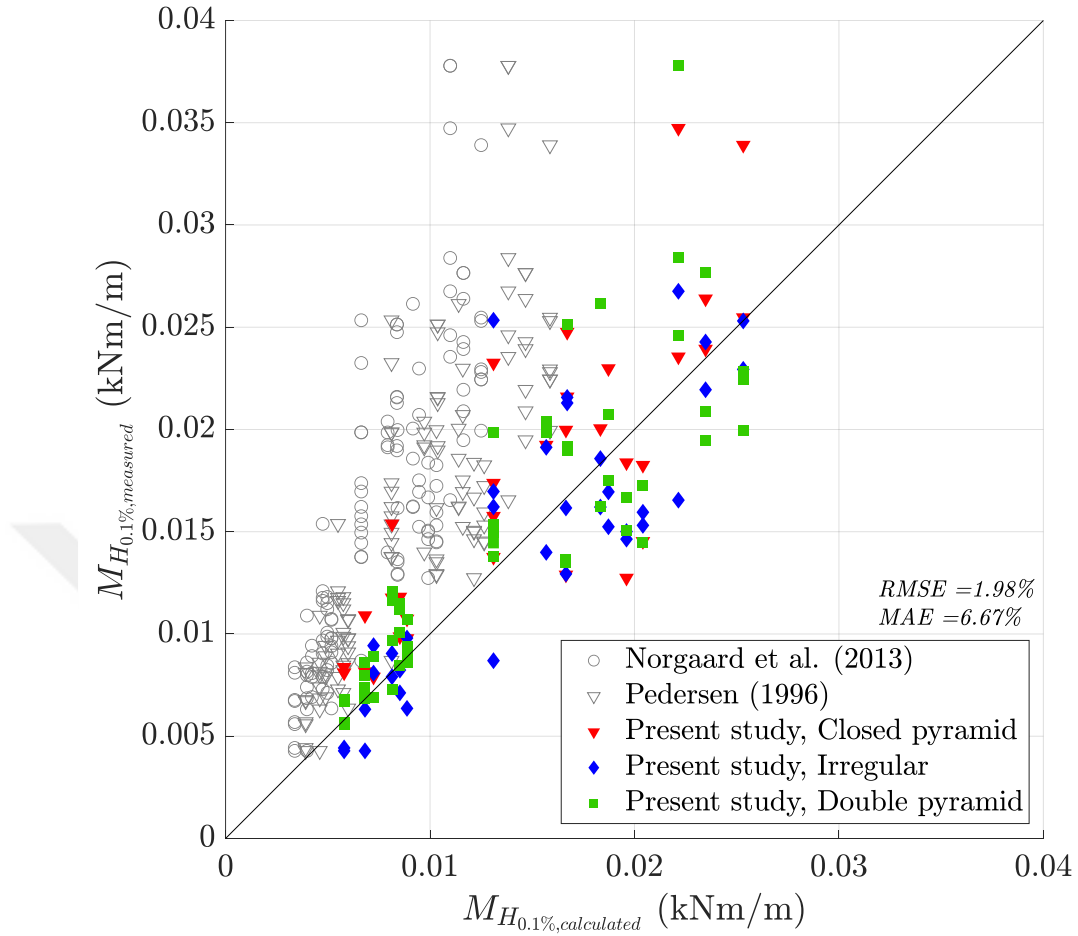


Figure 4.5. Comparison of $M_{h0.1\%}$ values for Pedersen (1996), Norgaard et al. (2013) and present study with the modified coefficients for all antifer placement method measurements. ($RMSE$ and MAE values show the performance of present study with respect to Norgaard et al. (2013) formula with original coefficients.)

Figure 4.5 shows the values corresponding to the measured overturning moments due to the horizontal wave forces for the CP, IR, and DP placement methods. The $RMSE$ and MAE values shown in Figure 4.5 are statistical analyses obtained by comparing the e_1 and e_2 coefficients of Norgaard et al. (2013) and the present study. As can be seen on the plot in Figure 4.5, the use of $a_{modified}$ and $b_{modified}$ for the all placement methods gives a better fit when considering together with the results in Table 4.2. This modification clearly improves the performance of the $M_{h0.1\%}$ formula of Norgaard et al. (2013) for the armour layer designed with antifer units regardless

of the placement method. When the statistical performance assessment results in Table 4.2 are analysed, the $RMSE$ value of the $a_{modified}$ and $b_{modified}$ fit obtained with the modification of the present study is 68.7% lower than Pedersen (1996) and 55.8% lower than Norgaard et al. (2013) overturning moment equations. In terms of MAE values, the present study showed an 83.2% decrease compared to Pedersen (1996) and a 61.4% decrease compared to Norgaard et al. (2013).

4.2.2 General Observation on $M_{h0.1\%}$ for Antifer Units

When the results are analysed, it is observed that the linear relationship between $F_{h0.1\%}$ and $M_{h0.1\%}$ is well provided by the overturning moment formula modified by Norgaard et al. (2013). After the prediction calculations made with the $a_{modified}$ and $b_{modified}$ values obtained after the fit assignment for the $F_{h0.1\%}$ expression, it was observed that the performance of the $M_{h0.1\%}$ formula increased in the same way, hence there is no need for major changes for the e_1 and e_2 coefficients. When Figure 4.5 is closely investigated, it is observed that the moment scatter is quite low in the scenarios where R_c is equal to A_c while an increase in moment scatter due to impact pressure is observed at gradually increasing R_c values. This was due to the more random development of forces in the unprotected layer. This feature observed for $M_{h0.1\%}$ is also parallel with $F_{h0.1\%}$. For $M_{h0.1\%}$, which is generally affected by $F_{h0.1\%}$, the comments made for $F_{h0.1\%}$ are also valid.

4.3 Analyses of the $P_{b0.1\%}$ Measurements

4.3.1 Modification of Empirical Coefficient for Antifer Unit Placement

Finally, statistical analyses were completed for the empirical coefficient d by examining the performance of the $P_{b0.1\%}$ formulation by comparing the base pressure measurements and expected base pressure values.

Since there are no parameters associated with $F_{h0.1\%}$ and $M_{h0.1\%}$ in the $P_{b0.1\%}$ expression presented by Norgaard et al. (2013) and Pedersen (1996) as shown in Eq. (2.23), the modification of this formula will be distinct from other changes in the formulas and coefficients. Since Norgaard et al. (2013) has made practically no changes to the Pedersen (1996) base pressure formula, the results in the plots to be presented below are only compared with Norgaard et al. (2013). It is noted that the only difference between Pedersen (1996) and Norgaard et al. (2013) regarding the estimation of the base pressures is that the calculation of $R_{u0.1\%}$. As stated in Chapter 2, Norgaard et al. (2013) uses $H_{0.1\%}$ rather than H_s in the calculation of $R_{u0.1\%}$. In Table 4.3, results of the statistical analysis of the predicted and measured values of base pressures obtained from the experimental study is shown. Modification of the d empirical coefficient in Eq. (2.23) was performed for all different antifer placement methods.

Table 4.3. Statistical assessment of base pressure measurements data to obtain the best fit for the modified empirical coefficient.

P_{0.1%} Crown Wall Formulation								
<i>Formulation</i>	<i>Fit Type</i>	<i>d</i>	<i>RMSE (kPa)</i>	<i>CVRMSE (-)</i>	<i>MAE (kPa)</i>	<i>NMAE (-)</i>	<i>BIAS (kPa)</i>	<i>N-BIAS (-)</i>
Norgaard et al. (2013)	All Placements	1.00	0.24421	0.21331	0.19412	0.73453	-0.08510	-0.32199
	Closed Pyramid		0.24643	0.21057	0.18147	0.65140	-0.06139	-0.22036
	Irregular		0.24494	0.21860	0.06663	0.25519	-0.10800	-0.41362
	Double Pyramid		0.24260	0.21135	0.19891	0.75787	-0.08217	-0.31308
Present Study	All Placements	0.93	0.22820	0.19932	0.18040	0.68260	0.00100	0.00379
	Closed Pyramid	0.95	0.23820	0.20354	0.18089	0.64932	0.00019	0.00070
	Irregular	0.91	0.21943	0.19584	0.05385	0.20625	0.00256	0.00981
	Double Pyramid	0.93	0.22730	0.19801	0.18405	0.70123	0.00393	0.01499

As can be seen from the Table 4.3, the empirical coefficient with the smallest value was obtained for the IR placement method as expected based on the above mentioned $F_{h0.1\%}$ and $M_{h01\%}$ statistical assessment results. However, when all antifer placement methods are considered, it can be said that the variations for different placement methods are quite small, and the base pressure is generally independent of the placement method; however, the change of the armour unit cause a slight change in the d parameter. It is also seen that the order of the statistical results presented in Table 4.3 does not change by changing the d coefficient.

In Figure 4.6, base pressures on the crown wall model measured in the present study experiments and the base pressures calculated using the formula of Norgaard et al. (2013) is compared for all antifer placement methods.

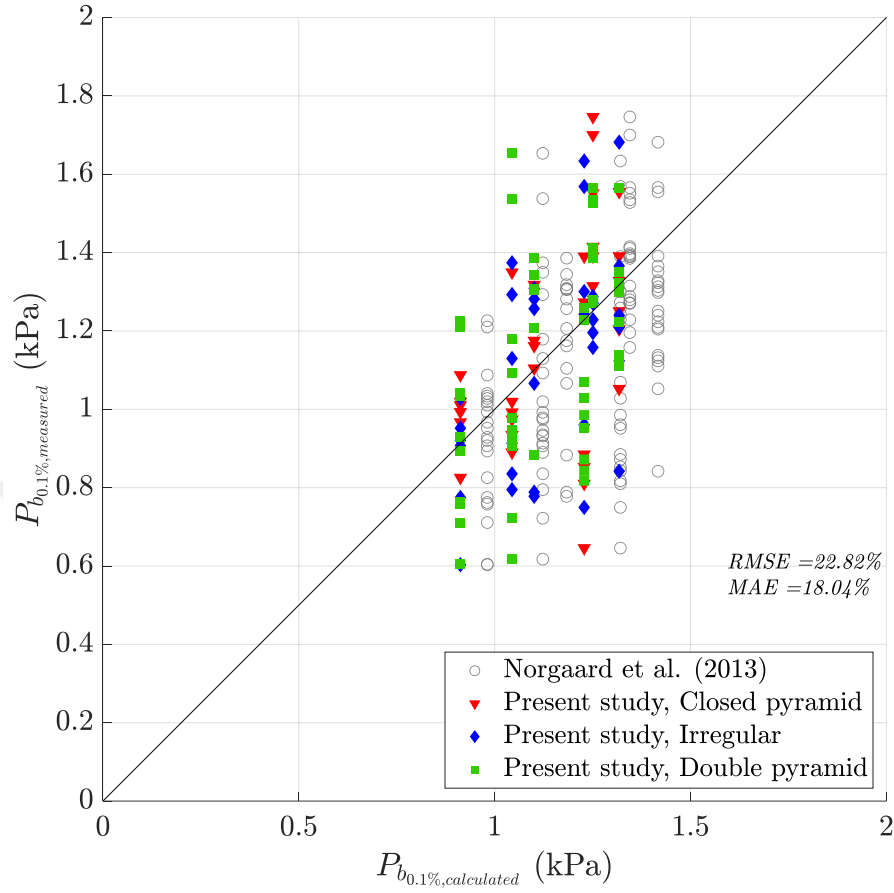


Figure 4.6. Comparison of $P_{b0.1\%}$ Norgaard et al. (2013) formulation with all antifer placement method measurements. ($RMSE$ and MAE values show the performance of present study with respect to Norgaard et al. (2013) formula with original coefficients.)

The $RMSE$ and MAE values shown in Figure 4.6 are statistical analyses obtained by comparing d coefficient of Norgaard et al. (2013) and the present study. As a result of the fit obtained by comparing the measured horizontal wave force values obtained from all antifer placement methods experiments with the predicted values from Norgaard et al. (2013) formulae, d value was obtained as 0.93. As can be seen on the plot in Figure 4.6, the use of d_{modified} for the regardless of the placement method gives a better fit when considering together with the results in Table 4.3. This modification slightly improves the performance of the $P_{b0.1\%}$ formula of Pedersen (1996) and

Norgaard et al. (2013) for all antifer placement methods. When the statistical performance assessment results in Table 4.3 are analysed, the *RMSE* value of the $d_{modified}$ fit obtained with the modification of the present study is 6.6% lower than Pedersen (1996) and Norgaard et al. (2013). In terms of *MAE* values, the present study showed a 7.1% decrease compared to Pedersen (1996) and Norgaard et al. (2013).

In Figure 4.7, base pressures on the crown wall model measured in the experiments and the base pressures calculated using the formula of Norgaard et al. (2013) is compared for CP placement method.



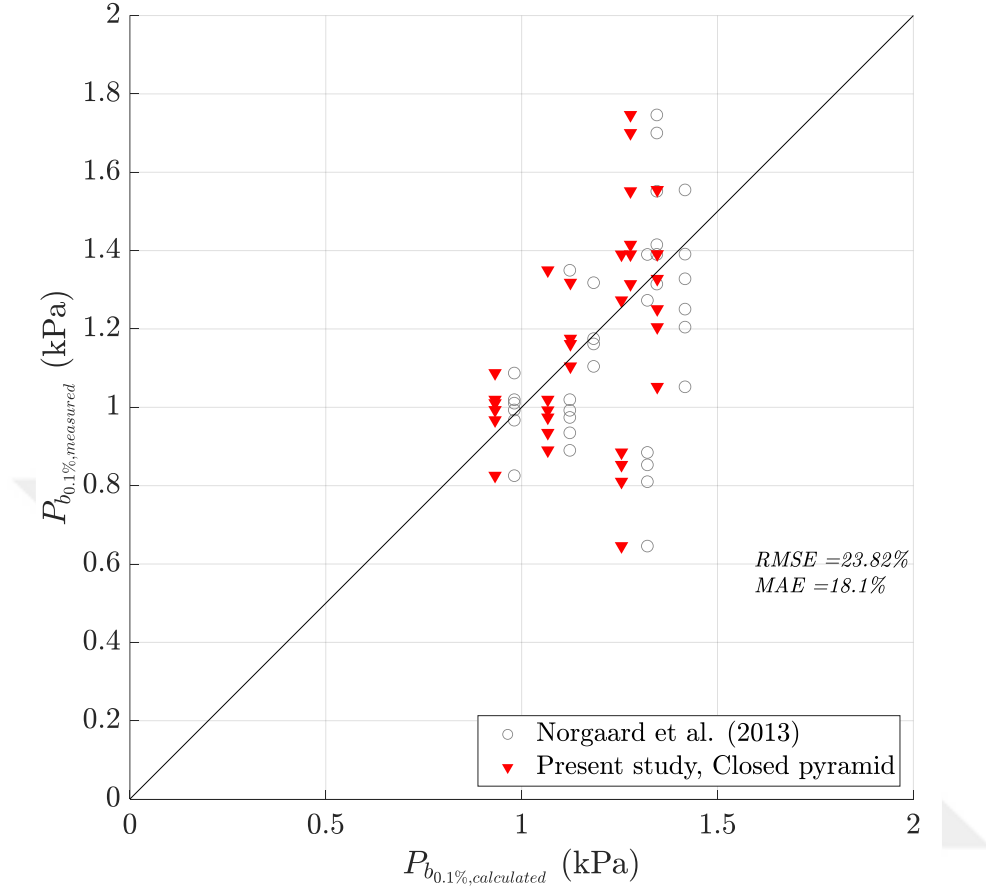


Figure 4.7. Comparison of Norgaard et al. (2013) and Pedersen (1996) formulation with CP placement method measurements. (*RMSE* and *MAE* values show the performance of present study with respect to Norgaard et al. (2013) formula with original coefficients.)

The *RMSE* and *MAE* values shown in Figure 4.7 are statistical analyses obtained by comparing *d* coefficient of Norgaard et al. (2013) and the present study. As a result of the fit obtained by comparing the measured horizontal wave force values obtained from CP placement methods experiments with the predicted values from Norgaard et al. (2013) formulae, *d* value was obtained as 0.95. As can be seen on the plot in Figure 4.7, the use of $d_{modified}$ for the regardless of the placement method gives a better fit when considering together with the results in Table 4.3. This modification slightly improves the performance of the $P_{b0.1\%}$ formula of Pedersen (1996) and

Norgaard et al. (2013) for all antifer placement methods. When the statistical performance assessment results in Table 4.3 are analysed, the *RMSE* value of the $d_{modified}$ fit obtained with the modification of the present study is 3.3% lower than Pedersen (1996) and Norgaard et al. (2013). In terms of *MAE* values, the present study showed a 0.03% decrease compared to Pedersen (1996) and Norgaard et al. (2013).

In Figure 4.8, base pressures on the crown wall model measured in the experiments and the base pressures calculated using the formula of Norgaard et al. (2013) is compared for IR placement method.



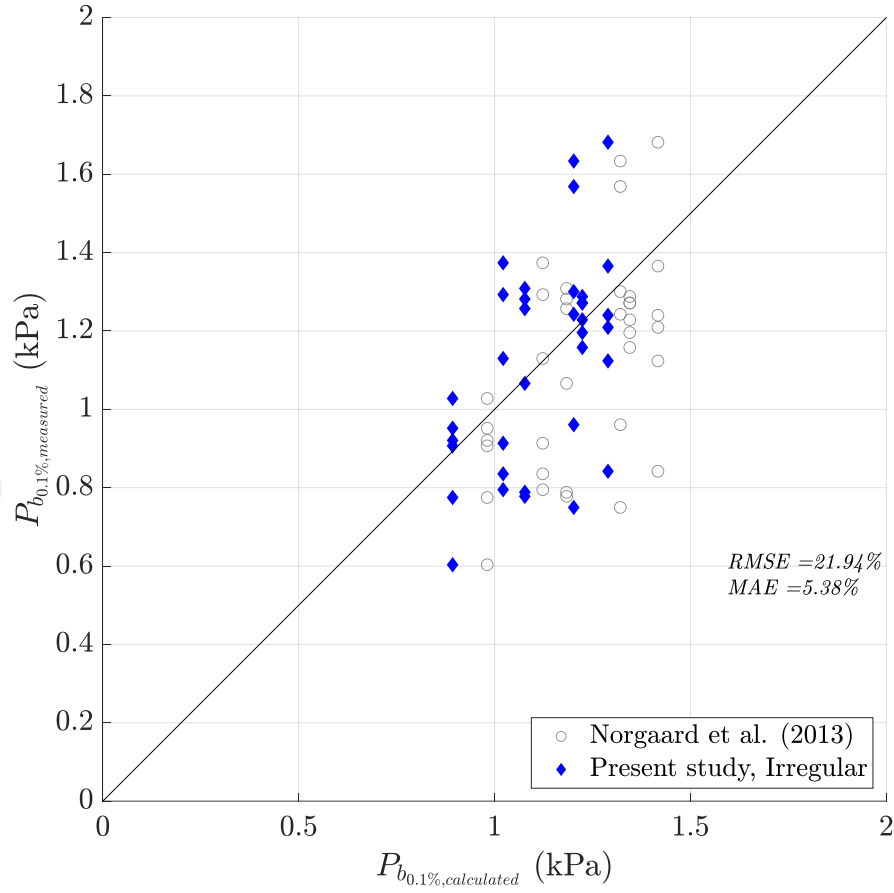


Figure 4.8. Comparison of Norgaard et al. (2013) and Pedersen (1996) formulation with IR placement method measurements. ($RMSE$ and MAE values show the performance of present study with respect to Norgaard et al. (2013) formula with original coefficients.)

The $RMSE$ and MAE values shown in Figure 4.8 are statistical analyses obtained by comparing d coefficient of Norgaard et al. (2013) and the present study. As a result of the best fit obtained by comparing the measured horizontal wave force values obtained from IR placement methods experiments with the predicted values from Norgaard et al. (2013) formulae, d value was obtained as 0.91. Figure 4.8, the use of $d_{modified}$ for the regardless of the placement method gives a better fit when considering together with the results in Table 4.3. This modification slightly improves the performance of the $P_{b0.1\%}$ formula of Pedersen (1996) and Norgaard et al. (2013) for

all antifer placement methods. When the statistical performance assessment results in Table 4.3 are analysed, the $RMSE$ value of the $d_{modified}$ fit obtained with the modification of the present study is 10.4% lower than Pedersen (1996) and Norgaard et al. (2013). In terms of MAE values, the present study showed a 1.9% decrease compared to Pedersen (1996) and Norgaard et al. (2013).

In Figure 4.8, base pressures on the crown wall model measured in the experiments and the base pressures calculated using the formula of Norgaard et al. (2013) is compared for IR placement method.

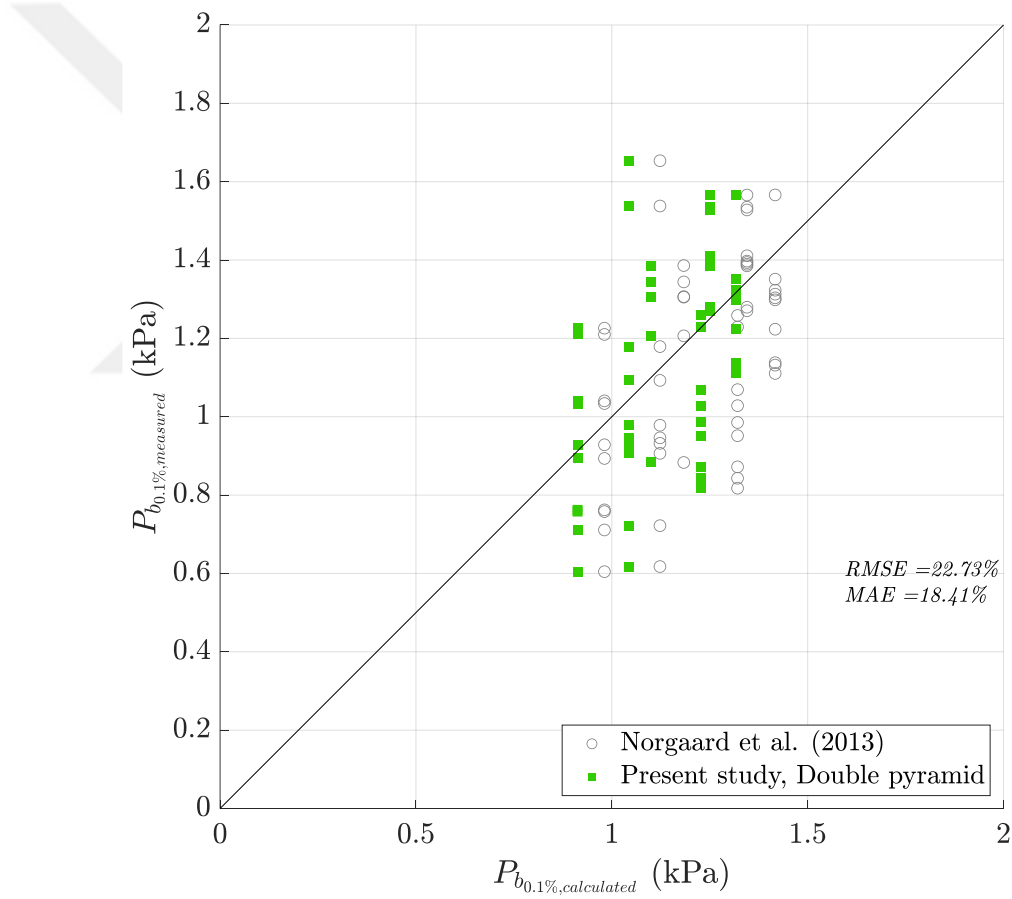


Figure 4.9. Comparison of Norgaard et al. (2013) and Pedersen (1996) formulation with DP placement method measurements. ($RMSE$ and MAE values show the performance of present study with respect to Norgaard et al. (2013) formula with original coefficients.)

The *RMSE* and *MAE* values shown in Figure 4.9 are statistical analyses obtained by comparing d coefficient of Norgaard et al. (2013) and the present study. As a result of the best fit obtained by comparing the measured horizontal wave force values obtained from IR placement methods experiments with the predicted values from Norgaard et al. (2013) formulae, d value was obtained as 0.93. Figure 4.9, the use of $d_{modified}$ for the regardless of the placement method gives a better fit when considering together with the results in Table 4.3. This modification slightly improves the performance of the $P_{b0.1\%}$ formula of Pedersen (1996) and Norgaard et al. (2013) for all antifer placement methods. When the statistical performance assessment results in Table 4.3 are analysed, the *RMSE* value of the $d_{modified}$ fit obtained with the modification of the present study is 6.3% lower than Pedersen (1996) and Norgaard et al. (2013). In terms of *MAE* values, the present study showed a 7.5% decrease compared to Pedersen (1996) and Norgaard et al. (2013).

4.3.2 General Observation on $P_{b0.1\%}$ for Antifer Units

When the results of the base pressures are analysed, it is observed that there is no major difference between the antifer placement methods affecting the base pressure. Therefore, $d_{modified}$ can be chosen as 0.93 for all antifer units with reference to the best fit. However, Pedersen (1996) stated that $P_{b0.1\%}$ has a low correlation with $F_{h0.1\%}$ and $M_{h0.1\%}$, which may lead to unsafe estimation. Based on this argument, it is suggested that instead of choosing the $d_{modified}$ value of 0.93 for the antifer units, base pressure of the crown wall structures of the rubble mound breakwaters designed with the antifer armour layer should be calculated with the actual coefficient d value of 1.00 used by Pedersen (1996) and Norgaard et al. (2013). Nevertheless, even though this modified result is not a bad estimate as it is quite close to Pedersen (1996) and Norgaard et al. (2013), it was decided not to change the actual d empirical coefficient to stay on the safe side based on the statement of Pedersen (1996).

4.4 Case Study for the Conventional Breakwater Design

A case study was carried out at the end of the study to compare the significance of the results obtained as a result of the present study and its effects on conventional breakwater with other methods. For the case study, a study called Project E. was carried out to design a 1:2 armour slope breakwater with an antifer units and crown wall design using wave characteristics at HWL for 100 years. The wave characteristics are shown in Table 4.4. Conventional breakwater design is shown in Figure 4.10.

Table 4.4. The range of application in wave load calculations (Berenguer and Baonza, 2006).

H_{m0} (m)	$H_{01\%}$ (m)	T_p (s)	$T_{m-1,0}$ (s)	T_{01} (s)	T_{02} (s)	H_{m0}/h (-)
8.60	15.40	12.35	11.25	10.50	1.450	0.45

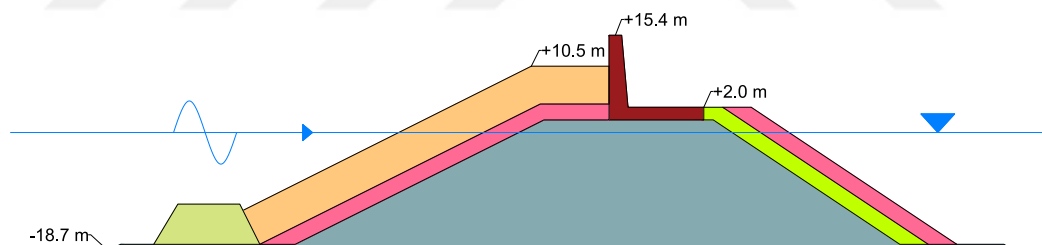


Figure 4.10. Conventional breakwater design for the Project E.

While the design parameters to be used in the formulations of Pedersen (1996), Norgaard et al. (2013) and the present study are given in Table 4.4, the characteristics of the breakwater section required for these formulations are given in Figure 4.10. As a result of the calculations for three different formulations, the results in Table 4.5 were obtained.

Table 4.5. Wave pressure calculations of the case study according to the three different design formulations.

Pedersen (1996)	a	b	c		$F_{h0.1\%}$ (kN/m)	$M_{h0.1\%}$ (kNm/m)	Factor of Safety against Sliding
	0.21	1.60	0.55		391.7	2304.7	1.05
Norgaard et al. (2013)	a	b	e_1	e_2	$F_{h0.1\%}$ (kN/m)	$M_{h0.1\%}$ (kNm/m)	Factor of Safety against Sliding
	0.21	1.00	0.95	0.40	289.0	1618.6	1.50
Present Study	a	b	e_1	e_2	$F_{h0.1\%}$ (kN/m)	$M_{h0.1\%}$ (kNm/m)	Factor of Safety against Sliding
	0.36	1.31	0.95	0.40	544.3	3214.1	0.80

When looking at Table 4.5, it is obvious that there are significant differences between the methods calculated with three different methods, especially when looking at the factor of safety against sliding. Pedersen (1996) found 28% smaller wave forces than in the present study, while Norgaard et al. (2013) estimated 49.9% smaller horizontal wave forces. When calculations are made to satisfy the factor of safety against sliding condition for Pedersen (1996) and the present study for 1.20, 9.32 m³/m extra concrete is needed for present study. On the other hand, 2.52 m³/m extra concrete is needed for Pedersen (1996).

CHAPTER 5

CONCLUSIONS

In this study, the effect of different placement methods of antifer units on the wave forces acting on the crown wall, a superstructure frequently used in the crest of breakwaters, was investigated in shallow water condition. Within the scope of this study, horizontal and vertical forces were measured on the crown wall model with pressure sensors, overturning moment values related to these forces were calculated and the results were compared with the formulation of Pedersen (1996), which is the most widely used in the literature, and Norgaard et al. (2013), which is a modified version of this method for shallow water condition. The main findings of this study are as follows:

- As a result of the study, it was observed that Pedersen (1996) and Norgaard et al. (2013) formulations underestimate the forces and overturning moments when calculating the wave forces acting on the crown walls of rubble mound breakwaters designed using antifer units in the armour layer. In other words, Pedersen (1996) formulations derived by designing the armour layer with quarry rocks and several artificial armour units and modified version of these formulations, Norgaard et al. (2013), with the experiments using quarry rocks in the armour layer are not accurate enough for estimating wave forces for antifer armour units.
- As a result of the comparison of the measured data obtained from the experimental studies carried out in the present study with the $F_{h0.1\%}$ formula of Norgaard et al. (2013), the actual empirical coefficients of $a=0.21$ and $b=1.00$ determined by Norgaard et al. (2013) were suggested as $a_{modified}=0.36$ and $b_{modified}=1.31$ for antifer units. When the placement methods are considered individually, it is suggested to use the empirical coefficients a and

b as 0.39 and 1.11 for CP placement method, 0.32 and 1.39 for IR placement method and 0.37 and 1.37 for DP placement method, respectively.

- IR placement causes decreasing wave pressures on the crown wall compared to DP and CP placement methods. In other words, up rushing wave is disrupted better by IR placement method which reduces the horizontal wave forces on the crown wall. The order of magnitude of the horizontal force difference between IR placement method and CP placement methods is at most 52.7% with an average of 26.7%. When IR and DP placement methods were compared, it was observed that the DP method was subjected to more horizontal wave force than IR method, with differences up to 26.6% (18.4% on average) in horizontal force measurements. As in the case of IR method, the DP method generally outperformed the CP, with the horizontal wave force difference up to 27% (15.2% on average). When the general trend of the performance of the placement methods is investigated, the highest wave forces were measured for the CP method most of the time, while the lowest wave forces were observed for the IR method.
- When comparing alternatives with different packing densities for the same placement methods, there was no significant change in the magnitude of horizontal wave forces due to the packing density. In cases where the packing density changed, measurement differences were observed at an average of about 15% for average CP placement, about 11.3% for IR placement and about 3-8.5% for average DP placement. When these differences investigated, it was observed that this feature did not have any significant effect, since there was no superiority between scenarios where the packing density was less or more.
- While the effect of staggered arrangement on the horizontal wave forces in DP placement methods was investigated, it was not observed that the magnitude of forces was significantly affected by the staggered antifer units. Measurement differences between staggered and normal arrangement for DP

placement methods were observed at an average of approximately 6.3-11.8%.

- When the performances of $M_{h0.1\%}$ formulas are analysed, Norgaard et al. (2013) moment formula, which is a modified version of Pedersen's (1996) overturning moment formula, establishes the linear relationship with $F_{h0.1\%}$ well for shallow water condition. The actual e_1 and e_2 coefficients in the moment formula of Norgaard et al. (2013) have already shown good performance, especially when using the $a_{modified}$ and $b_{modified}$ determined by the present study. Therefore, no modifications are proposed for these coefficients.
- As a result of the regression and statistical performance assessment calculations for the $P_{b0.1\%}$, the values predicted by Pedersen (1996) and Norgaard et al. (2013) base pressure formula was very close to the measured values in this study. Despite the observed slight overestimation, no modification is made to the actual d empirical coefficient to stay on the safe side based on the arguments of Pedersen (1996). Nevertheless, the use of the presented $d_{modified}$ parameters was left to the designer's choice, provided that the arguments specified by Pedersen (1996) were considered.

These experiments have shown that the different antifer unit placement methods caused by the change in the wave forces acting on the crown wall. Although the available experimental data is investigated carefully, there are remains regarding the use of the antifer units and data assessment process that are left as future studies. These include the following suggestions:

- Considering that Norgaard et al. (2013) only investigates armour layers with natural stones, the actual empirical coefficients in these formulations bring an underestimation for the use of antifer units. Based on this, it is obvious that there will be differences in other units such as Xbloc, tetrapod, Accropode to be used in the armour layer. Therefore, with the methodology used in this experiment, the data set should be expanded in the future by

conducting experiments with different units and new empirical coefficients should be found.

- It would be useful to add the γ_f parameter, which is used to reflect the roughness of the armour units when deciding the amount of wave overtopping, to the wave pressure equations to represent the wave pressures that will be affected using different armour units in the same way.
- The concept of protected and unprotected layers as presented by Pedersen (1996) needs to be revised for artificial armor units. Depending on the placement methods, especially in the antifers, in the protected layer below the armor crest elevation, gaps are observed between the antifer units due to packing density. These gaps behave like an unprotected layer which causes the wave pressure to increase. In other words, there is no smooth transition between protected and unprotected layer for antifer units, just like in quarry rocks. While it is quite obvious that this different behavior will be observed in units such as Tetrapod, Xbloc and Accropode, this behavior needs to be observed in more detail.
- To increase the coverage of the experiment, this study can be expanded with physical modelling experiments using different berm width, armour slope and crest heights.
- To better analyse the impact pressure mechanism and the associated randomness, more random wave sets which may lead to stronger impact pressure and an expansion with more repetitions can be used to clarify the uncertainty in this part of the calculation of wave pressures.

REFERENCES

- AYGM, (2016). *Kıyı Yapıları Planlama ve Tasarım Teknik Esasları*, T.C. Ulaştırma, Denizcilik ve Haberleşme Bakanlığı, Altyapı Yatırımları Genel Müdürlüğü, 2016. (In Turkish).
- Battjes, J. A., & Groenendijk, H. W. (2000). Wave height distributions on shallow foreshores. *Coastal Engineering*, 40(3), 161-182.
- Berenguer, J. M., & Baonza, A. (2006, May). On the design of crown walls of rubble mound breakwaters. In *Proceedings of the 31st PIANC Congress, Estoril, Portugal* (pp. 14-18).
- Bradbury, A.P., Allsop, N.W.H., Stephens, R.V., (1988). *Hydraulic performance of breakwater crown walls*. (Report SR146). H.R. Wallingford.
- Braña, P. C., & Guillén, J. F. (2005). Wave forces on crown walls: Evaluation of existing empirical formulations. In *Coastal Engineering 2004: (In 4 Volumes)* (pp. 4087-4099).
- British Standards Institution (1991). *British Standard: Maritime structures - Part 7: Guide to the design and construction of breakwaters*. In: COMMITTEE, C. E. A. B. S. P. & CSB/17, T. C. (eds.) BS 6349, Maritime Structures. British Standards Institution.
- Bruce, T., Van der Meer, J. W., Franco, L., & Pearson, J. M. (2009). Overtopping performance of different armour units for rubble mound breakwaters. *Coastal Engineering*, 56(2), 166-179.
- Burcharth, H. F. (1993). *The design of breakwaters*. Internal report; Aalborg University.

- Burcharth, H. F., Liu, Z., & Troch, P. (1999). Scaling of core material in rubble mound breakwater model tests. In *Fifth International Conference on Coastal and Port Engineering in Developing Countries: Proceedings of the COPEDEC V: Cape Town, South Africa, 19-23 April 1999* (pp. 1518-1528). COPEDEC.
- Chegini, V., & Aghtouman, P. (2006). An investigation on the stability of rubble mound breakwaters with armour layers of anti-fer cubes. *Journal Of Marine Engineering*, 2(1), 86-93.
- CIRIA, CUR, CETMEF (2007). The Rock Manual. *The use of rock in hydraulic engineering*.
- Coastal Engineering Research Center (US). (1984). *Shore protection manual* (Vol. 1). Department of the Army, Waterways Experiment Station, Corps of Engineers, Coastal Engineering Research Center.
- Contestabile, P., Di Lauro, E., Buccino, M., & Vicinanza, D. (2016). Economic Assessment of Overtopping Breakwater for Energy Conversion (OBREC): a case study in Western Australia. *Sustainability*, 9(1), 51.
- Creel, L. (2003). *Ripple effects: population and coastal regions* (pp. 1-7). Washington, DC: Population reference bureau.
- EurOtop, (2018). *Manual on wave overtopping of sea defences and related structures. An overtopping manual largely based on European research, but for worldwide application*. Van der Meer, J.W., Allsop, N.W.H., Bruce, T., De Rouck, J., Kortenhaus, A., Pullen, T., Schüttrumpf, H., Troch, P. and Zanuttigh, B., www.overtopping-manual.com.
- Freitas, P., Trigo-Teixeira, A., & Araújo, M. A. V. C. (2013). Hydraulic stability of anti-fer block armour layers. In *Proceedings of 6th International Short Course/Conference on Applied Coastal Research, Lisbon, Portugal*.

- Frens, A.B., 2007. *The impact of placement method on Antifer-block stability*. Master of Science thesis, Delft University of Technology, Netherlands.
- Glukhovskiy, B. K. (1966, February). Investigation of sea wind waves. In *Proceedings of the Sea Climatology Conference, Paris, France* (pp. 51-71).
- Gunbak, A.R., Ergin, A., (1983). Damage and repair of Antalya harbor breakwater. Proc. Conf. on Coastal Structures, Alexandria, Egypt.
- Gunbak, A.R., Gokce, T., (1984). Wave screen stability of rubble mound breakwaters. *International Symposium of Maritime Structures in the Mediterranean Sea*. Athens, Greece, pp. 2.99–2.112.
- Haggi, A. A. (2010). Report on Underwater Excavation at the Phoenician Harbour, Atlit, Israel. *International Journal of Nautical Archaeology*, 39(2). 278-285.
- Hughes, S. A. (1993). *Physical models and laboratory techniques in coastal engineering* (Vol. 7). World Scientific.
- Iuppa, C., Contestabile, P., Cavallaro, L., Foti, E., & Vicinanza, D. (2016). Hydraulic performance of an innovative breakwater for overtopping wave energy conversion. *Sustainability*, 8(12), 1226.
- Jensen, O. J. (1984). *A monograph on rubble mound breakwaters*. Hørsholm, Denmark: Danish Hydraulic Institute.
- Kisacik, D., Tarakcioglu, G. O., & Baykal, C. (2019). Stilling wave basins for overtopping reduction at an urban vertical seawall–The Kordon seawall at Izmir. *Ocean Engineering*, 185, 82-99.
- Klopman, G., & Van der Meer, J. W. (1999). Random wave measurements in front of reflective structures. *Journal of Waterway, Port, Coastal, and Ocean engineering*, 125(1), 39-45.

- Kralli, V., Theodossiou, N., & Karambas, T. V. (2019). Optimal design of overtopping breakwater for energy conversion (OBREC) systems using the Harmony Search algorithm. *Frontiers in Energy Research*, 7.
- Koosheh, A., Etemad-Shahidi, A., Cartwright, N., Tomlinson, R., & van Gent, M. R. (2022). Experimental study of wave overtopping at rubble mound seawalls. *Coastal Engineering*, 172, 104062.
- Lamberti, A., Martinelli, L., Gabriella Gaeta, M., Tirindelli, M., & Alderson, J. (2011). Experimental spatial correlation of wave loads on front decks. *Journal of Hydraulic Research*, 49(1), 81-90.
- Losada, M.A., Gimenez-Curto, L.A. (1981). Flow characteristics on rough, permeable slopes under wave action. *Coastal Engineering*, 4(0), 187–206.
- Losada, I. J., Dalrymple, R. A., & Losada, M. A. (1993). Water waves on crown breakwaters. *Journal of waterway, port, coastal, and ocean engineering*, 119(4), 367-380.
- Mansard, E. P., & Funke, E. R. (1980). The measurement of incident and reflected spectra using a least squares method. In *Coastal Engineering 1980* (pp. 154-172).
- Martin, F. L., Losada, M. A., & Medina, R. (1999). Wave loads on rubble mound breakwater crown walls. *Coastal Engineering*, 37(2), 149-174.
- Molines Llodra, J. (2016). *Wave overtopping and crown wall stability of cube and Cubipod-armored mound breakwaters* (Doctoral dissertation, Universitat Politècnica de València).
- Molines, J., Bayón, A., Gómez-Martín, M. E., & Medina, J. R. (2019). Influence of Parapets on Wave Overtopping on Mound Breakwaters with Crown Walls. *Sustainability*, 11(24), 7109.

- Negro Valdecantos, V., López Gutiérrez, J. S., & Polvorinos Flors, J. I. (2013, March). Comparative study of breakwater crown wall–calculation methods. In *Proceedings of the Institution of Civil Engineers-Maritime Engineering* (Vol. 166, No. 1, pp. 25-41). Thomas Telford Ltd.
- Nørgaard, J. Q. H., Andersen, T. L., & Burcharth, H. F. (2013). Wave loads on rubble mound breakwater crown walls in deep and shallow water wave conditions. *Coastal Engineering*, 80, 137-147.
- Pedersen, J., & Burcharth, H. F. (1993). Wave forces on crown walls. In *Coastal Engineering 1992* (pp. 1489-1502).
- Pedersen, J., 1996. *Wave Forces and Overtopping on Crown Walls of Rubble Mound Breakwaters*. Ph.D. thesis Series Paper 12Hydraulics & Coastal Engineering Lab., Dept. of Civil Engineering, Aalborg University, Denmark (ISBN 0909–4296).
- Pereira, F. L., Neves, M. G. P. M. S., López-Gutiérrez, J., Cuesta, A., & Negro, V. (2021). Comparison of existing equations for the design of crown walls: application to the case study of Ericeira Breakwater (Portugal). *Journal of Marine Science and Engineering*, 9(3), 285.
- U.S. Army Corps of Engineers (2006). *Coastal Engineering Manual*. (2006). EM-1110-2-1100, U.S. Army Engineers Waterways Experiment Station, Washington, D.C.: U.S. Government Printing Office. 2838 p.
- Van der Meer, J. W., & Stam, C. J. M. (1992). Wave runup on smooth and rock slopes of coastal structures. *Journal of Waterway, Port, Coastal, and Ocean Engineering*, 118(5), 534-550.
- Vicinanza, D., Contestabile, P., Nørgaard, J. H., & Andersen, T. L. (2014). Innovative rubble mound breakwaters for overtopping wave energy conversion. *Coastal Engineering*, 88, 154–170.

- Vos-Rovers, I., Reedijk, B., & Ekelmans, S. (2009). Crown-Wall with Extended Base Slab. In *Coastal Engineering 2008: (In 5 Volumes)* (pp. 3830-3841).
- Yagci, O., & Kapdasli, S. (2003). Alternative placement technique for antifer blocks used on breakwaters. *Ocean Engineering*, 30(11), 1433-1451.
- Wolters, G., van Gent, M., Allsop, W., Hamm, L., & Mühlestein, D. (2007). Guidelines for Physical Model Testing of Breakwaters, Rubble Mound Breakwaters. *Report prepared for Hydralab III, May*.



APPENDICES

A. Wave Height Distributions and Energy Spectrums

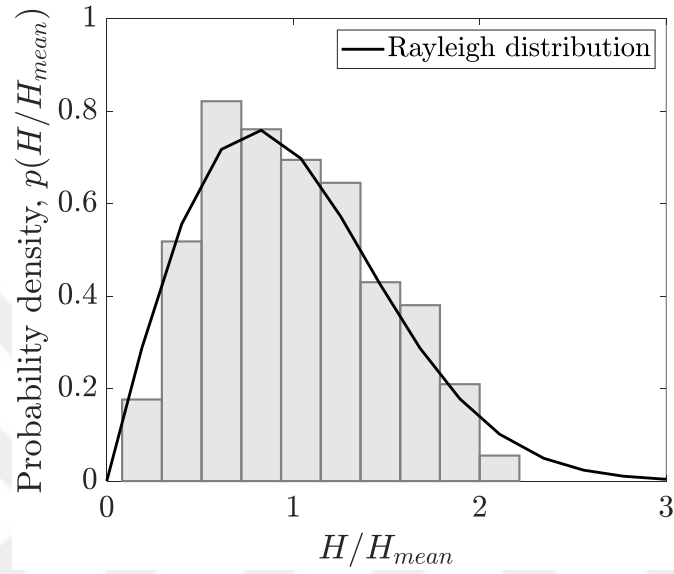


Figure A.1. Non-dimensionalized histogram of wave heights [D-1 Irregular Wave Condition]

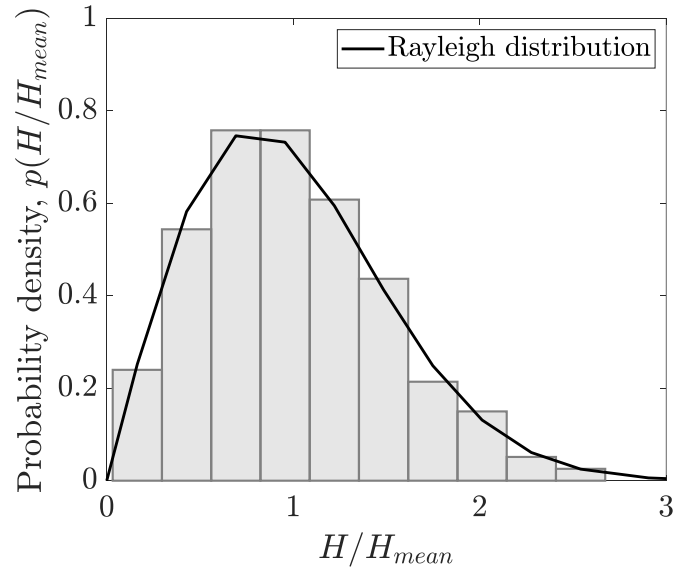


Figure A.2. Non-dimensionalized histogram of wave heights [D-2 Irregular Wave Condition]

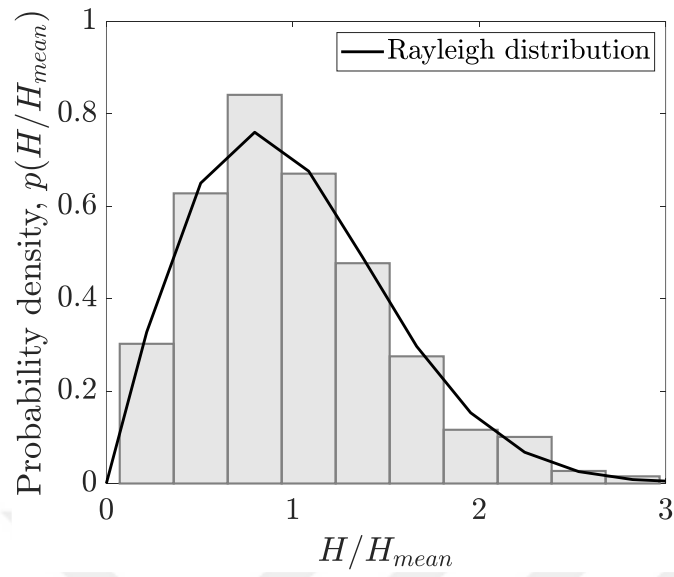


Figure A.3. Non-dimensionalized histogram of wave heights [D-3 Irregular Wave Condition]

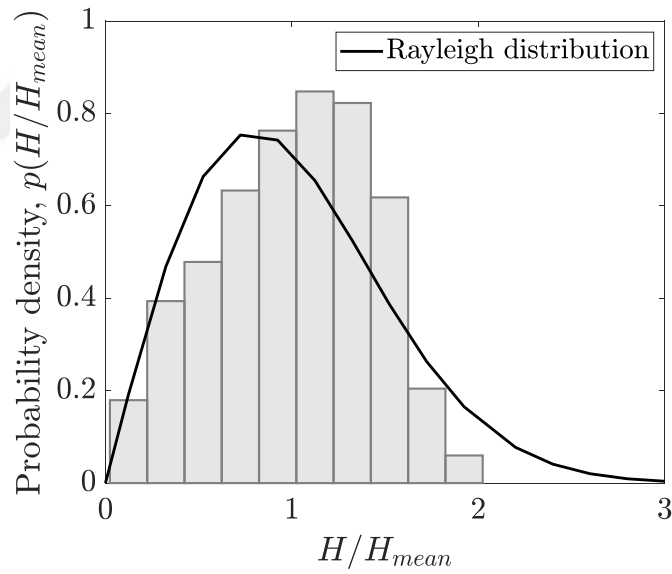


Figure A.4. Non-dimensionalized histogram of wave heights [D-4 Irregular Wave Condition]

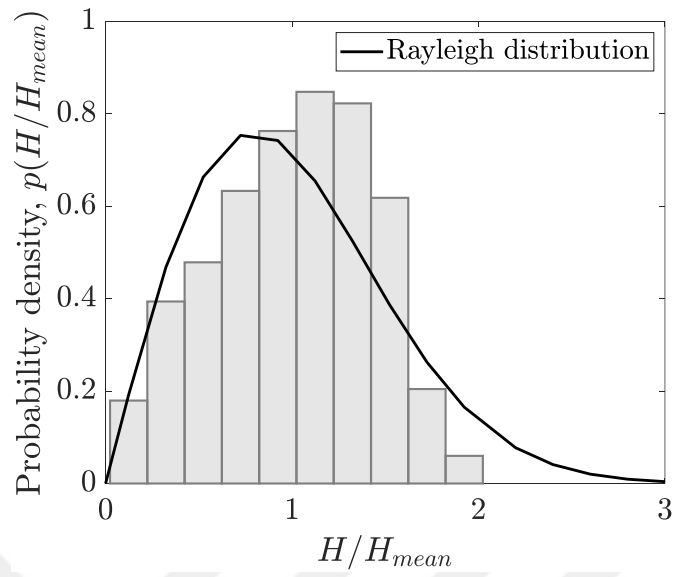


Figure A.5. Non-dimensionalized histogram of wave heights [D-5 Irregular Wave Condition]

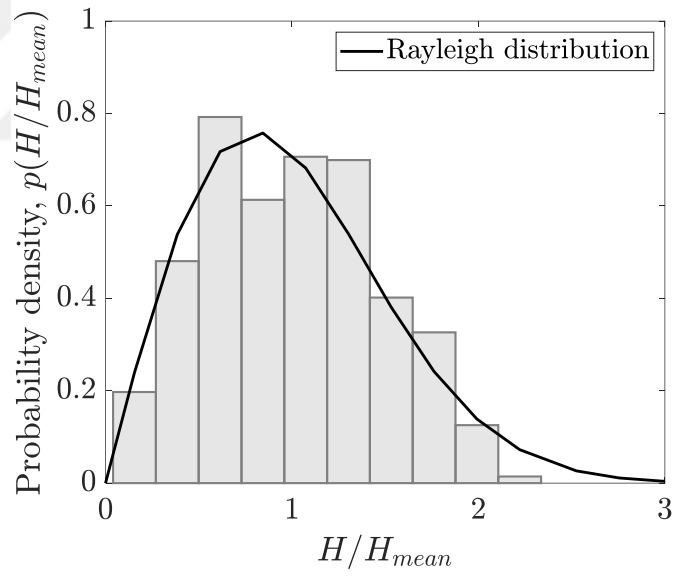


Figure A.6. Non-dimensionalized histogram of wave heights [D-6 Irregular Wave Condition]

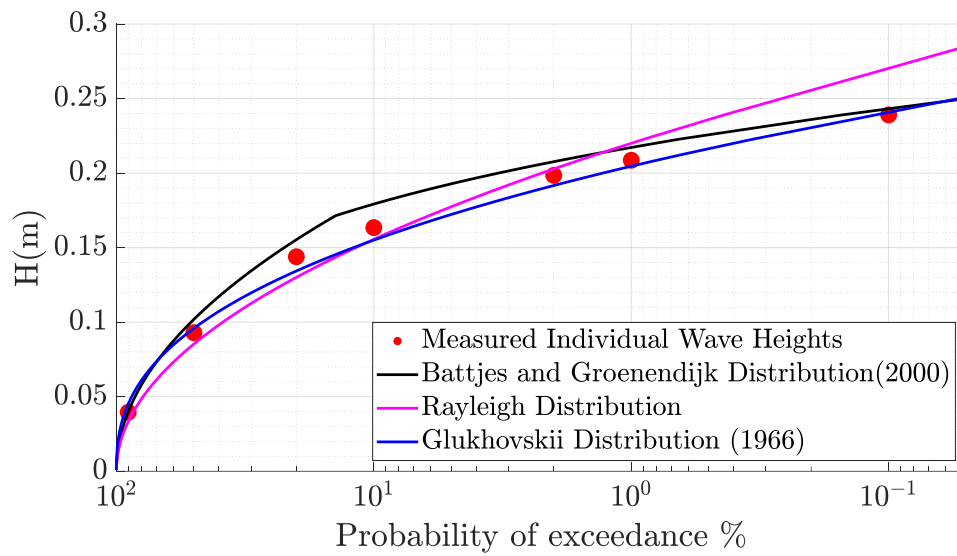


Figure A.7. Wave height distribution comparison with Battjes and Groenendijk (2000), Rayleigh and Glukhovskii (1966) methods. [D-1 Irregular Wave Condition]

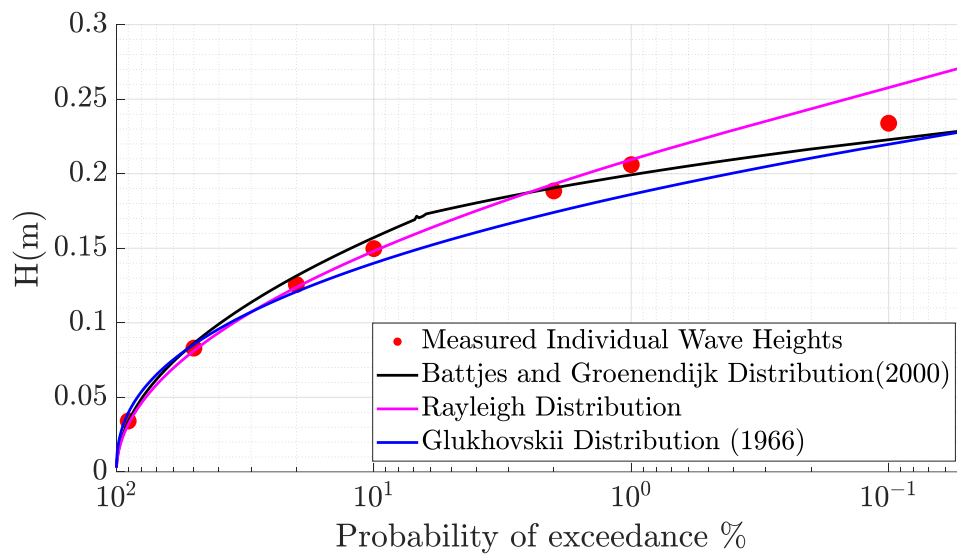


Figure A.8. Wave height distribution comparison with Battjes and Groenendijk (2000), Rayleigh and Glukhovskii (1966) methods. [D-2 Irregular Wave Condition]

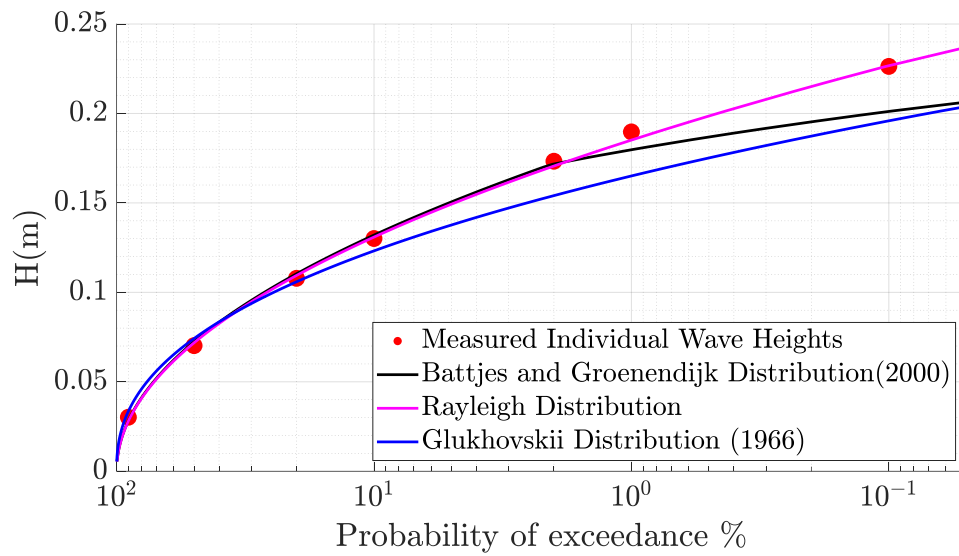


Figure A.9. Wave height distribution comparison with Battjes and Groenendijk (2000), Rayleigh and Glukhovskii (1966) methods. [D-3 Irregular Wave Condition]

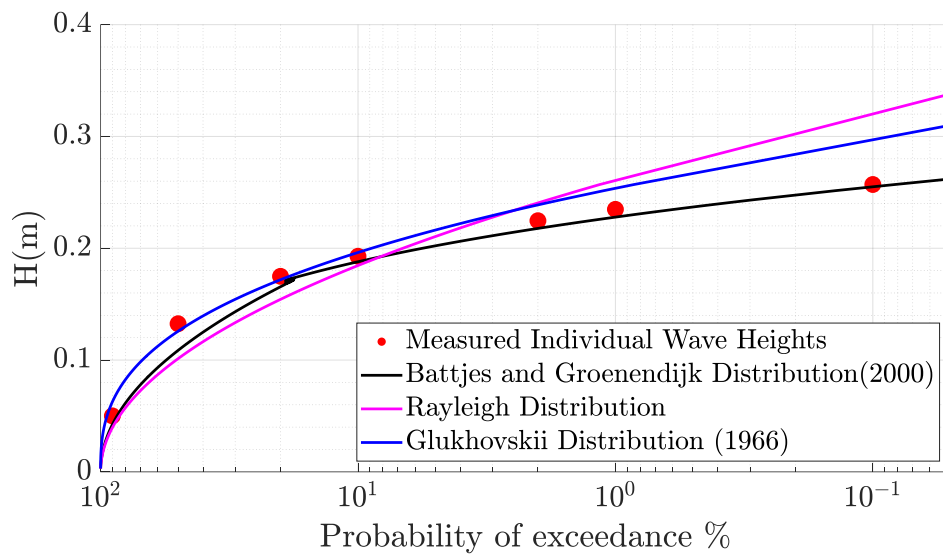


Figure A.5.1. Wave height distribution comparison with Battjes and Groenendijk (2000), Rayleigh and Glukhovskii (1966) methods. [D-4 Irregular Wave Condition]

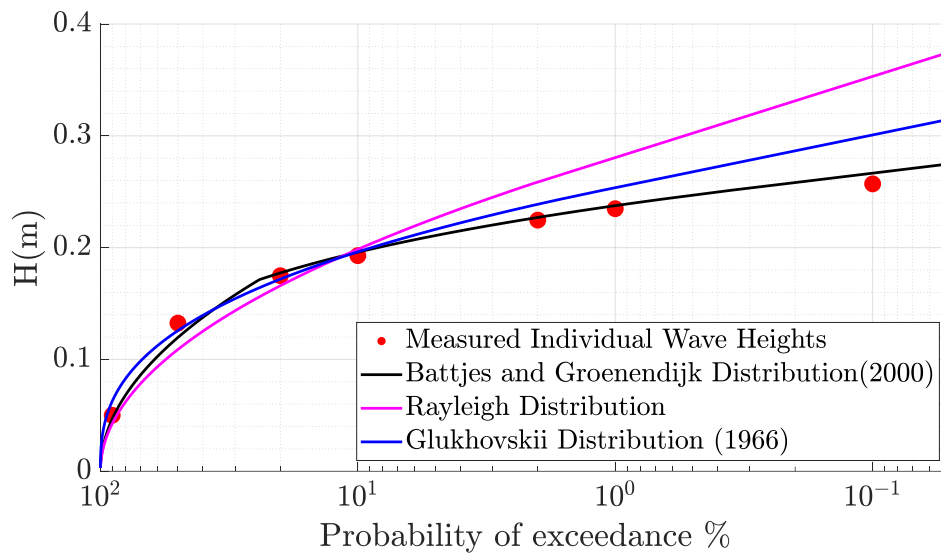


Figure A.5.2. Wave height distribution comparison with Battjes and Groenendijk (2000), Rayleigh and Glukhovskii (1966) methods. [D-5 Irregular Wave Condition]

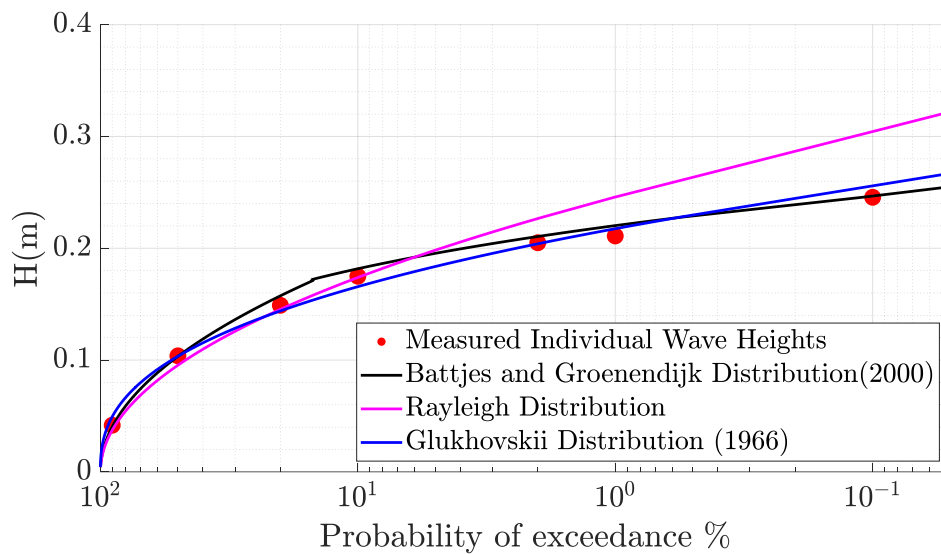


Figure A.5.3. Wave height distribution comparison with Battjes and Groenendijk (2000), Rayleigh and Glukhovskii (1966) methods. [D-6 Irregular Wave Condition]