Wave forces on cubical armour units on submerged and low-crested breakwaters.

Wasi Tri. Pramono

University of Windsor

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WAVE FORCES ON CUBICAL ARMOUR UNITS
ON SUBMERGED AND LOW-CRESTED BREAKWATERS

by
Wasi Tri Pramono

A Dissertation
Submitted to the Faculty of Graduate Studies Through the
Department of Civil and Environmental Engineering
in Partial Fulfillment of the Requirements
for the Degree of Doctor of Philosophy
The University of Windsor

Windsor, Ontario, Canada
1997
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0-612-30293-8
ABSTRACT

Dynamic forces on a cubical shaped armour unit produced by breaking waves were investigated experimentally in a model of submerged and low-crested breakwaters. Measurements were made along the surface of the breakwater to determine the most critical location of the armour unit. The measurements of the force were conducted using a specially designed dynamometer to enable a simultaneous measurement of the tangential and normal force components acting on a cubical armour unit. Two dynamometers were used to study the simultaneous forces induced by breaking waves on two cubical armour units for selected variables including water depth, the height and period of the wave, the location and orientation of the units, and the slope of the breakwater.

Analysis were made of the transmitted, reflected, and dissipated wave energy. The transmission and reflection coefficients were presented in relation to the submergence of the breakwater. A modified Morison equation was introduced to describe the longitudinal forces on an armour unit. The improved Morison equation was suggested by adding impact force term due to wave breaking to the Morison equation. Envelopes of maximum forces on the armour unit was developed for units placed anywhere along the surface of the breakwater. The envelopes of the forces were presented for the forward, backward, upward, and downward forces on the unit. The stability of the armour unit was analyzed using the dynamic force data. The stability analysis was made for the unit placements at
both sliding and overturning caused by the tangential and normal force components, and the spinning moment about the centre of mass of the cubical armour unit.
ACKNOWLEDGEMENT

The author would like to express his deep gratitude to his Academic Advisor, Dr. J. A. McCorquodale for his invaluable consistent guidance, support, suggestions, and encouragement throughout this study. The author admires his commitment for excellence, and he is grateful under his supervision.

The author would also like to sincerely thanks Dr. K. Hall, his External Examiner, for his constructive suggestions. Special thanks are also given to Dr. G. Rankin, Dr. Northwood, Dr. J. K. Bewtra, and Dr. N. Biswas for their time and effort to make the completion of this study possible.

The author expresses his gratitude to the Natural Sciences and Engineering Research Council (NSERC) of Canada for providing financial support throughout this programme.

Special thanks are also given to all of the Technical Support Staff, University of Windsor, as they carefully built the improved dynamometers for this study.

Finally, the author is grateful to his wonderful and lovely wife and children for their constant prayers, love, and patience.
ABSTRACT

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6.47  Impact, inertia, and drag indices versus (H_{wet}/gT^2); h/B = 1, Slope 1:2 (based on force maxima)

6.48  Impact, inertia, and drag indices versus (H_{wet}/gT^2); h/B = 1, Slope 1:2, averaged (based on force maxima)

6.49  Impact, inertia, and drag indices versus (h/d); h/B = 1, Slope 1:2 (based on force maxima)

6.50  Impact, inertia, and drag indices versus (h/d); h/B = 1, Slope 1:2, averaged (based on force maxima)

6.51  Impact index versus Keulegan Carpenter Number; h/B = 1, slope 1:2 (based on force maxima)

6.52  Impact index versus (H_{wet}/gT^2); h/B = 1, Slope 1:2 (Based on assumption that force maxima are caused by impact)

6.53  Impact index versus (h/d); h/B = 1, Slope 1:2 (Based on assumption that force maxima are caused by impact)

6.54  Inertia index versus Keulegan Carpenter Number; h/B = 1, slope 1:2 (based on force maxima)

6.55  Drag index versus Keulegan Carpenter Number; h/B = 1, slope 1:2 (based on force maxima)

6.56  Predicted and measured maximum force (based on force maxima)
6.58 Comparison based on force maxima:
Measured (experimental) and the predicted (theoretical) forces
\( T = 2.2 \text{ s}, \, H = 179 \text{ mm}, \, d = 375 \text{ mm}, \, h/d = 0.73 \)

6.59 Comparison based on force maxima:
Measured (experimental) and the predicted (theoretical) forces
\( T = 2.2 \text{ s}, \, H = 173 \text{ mm}, \, d = 375 \text{ mm}, \, h/d = 0.73 \)

6.60 Comparison based on force maxima:
Measured (experimental) and the predicted (theoretical) forces
\( T = 1.9 \text{ s}, \, H = 179 \text{ mm}, \, d = 375 \text{ mm}, \, h/d = 0.73 \)

6.61 Comparison based on force maxima:
Measured (experimental) and the predicted (theoretical) forces
\( T = 1.9 \text{ s}, \, H = 195 \text{ mm}, \, d = 375 \text{ mm}, \, h/d = 0.73 \)

6.62 Comparison based on force maxima:
Measured (experimental) and the predicted (theoretical) forces
\( T = 1.9 \text{ s}, \, H = 157 \text{ mm}, \, d = 375 \text{ mm}, \, h/d = 0.73 \)

6.63 Comparison based on force maxima:
Measured (experimental) and the predicted (theoretical) forces
\( T = 1.5 \text{ s}, \, H = 190 \text{ mm}, \, d = 375 \text{ mm}, \, h/d = 0.73 \)

6.64 Comparison based on force maxima:
Measured (experimental) and the predicted (theoretical) forces
\( T = 2.2 \text{ s}, \, H = 184 \text{ mm}, \, d = 350 \text{ mm}, \, h/d = 0.79 \)

6.65 Comparison based on force maxima:
Measured (experimental) and the predicted (theoretical) forces
\( T = 2.2 \text{ s}, \, H = 188 \text{ mm}, \, d = 350 \text{ mm}, \, h/d = 0.79 \)

6.66 Comparison based on force maxima:
Measured (experimental) and the predicted (theoretical) forces
\( T = 2.2 \text{ s}, \, H = 173 \text{ mm}, \, d = 350 \text{ mm}, \, h/d = 0.79 \)

6.67 Comparison based on force maxima:
Measured (experimental) and the predicted (theoretical) forces
\( T = 1.9 \text{ s}, \, H = 189 \text{ mm}, \, d = 350 \text{ mm}, \, h/d = 0.79 \)
6.69 Comparison based on force maxima:
Measured (experimental) and the predicted (theoretical) forces
\( T = 1.9 \text{ s}, \ H = 163 \text{ mm}, \ d = 350 \text{ mm}, \ h/d = 0.79 \)

6.70 Comparison based on force maxima:
Measured (experimental) and the predicted (theoretical) forces
\( T = 1.5 \text{ s}, \ H = 188 \text{ mm}, \ d = 350 \text{ mm}, \ h/d = 0.79 \)

6.71 Comparison based on force maxima:
Measured (experimental) and the predicted (theoretical) forces
\( T = 1.5 \text{ s}, \ H = 172 \text{ mm}, \ d = 350 \text{ mm}, \ h/d = 0.79 \)

6.72 Comparison based on force maxima:
Measured (experimental) and the predicted (theoretical) forces
\( T = 1.5 \text{ s}, \ H = 167 \text{ mm}, \ d = 350 \text{ mm}, \ h/d = 0.79 \)

6.73 Comparison based on force maxima:
Measured (experimental) and the predicted (theoretical) forces
\( T = 2.2 \text{ s}, \ H = 188 \text{ mm}, \ d = 325 \text{ mm}, \ h/d = 0.85 \)

6.74 Comparison based on force maxima:
Measured (experimental) and the predicted (theoretical) forces
\( T = 2.2 \text{ s}, \ H = 174 \text{ mm}, \ d = 325 \text{ mm}, \ h/d = 0.85 \)

6.75 Comparison based on force maxima:
Measured (experimental) and the predicted (theoretical) forces
\( T = 2.2 \text{ s}, \ H = 165 \text{ mm}, \ d = 325 \text{ mm}, \ h/d = 0.85 \)

6.76 Comparison based on force maxima:
Measured (experimental) and the predicted (theoretical) forces
\( T = 1.9 \text{ s}, \ H = 185 \text{ mm}, \ d = 325 \text{ mm}, \ h/d = 0.85 \)

6.77 Comparison based on force maxima:
Measured (experimental) and the predicted (theoretical) forces
\( T = 1.9 \text{ s}, \ H = 158 \text{ mm}, \ d = 325 \text{ mm}, \ h/d = 0.85 \)

6.78 Comparison based on force maxima:
Measured (experimental) and the predicted (theoretical) forces
\( T = 1.9 \text{ s}, \ H = 168 \text{ mm}, \ d = 325 \text{ mm}, \ h/d = 0.85 \)

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6.80 Comparison based on force maxima:
Measured (experimental) and the predicted (theoretical) forces
T = 1.5 s, H = 199 mm, d = 325 mm, h/d = 0.85

6.81 Comparison based on force maxima:
Measured (experimental) and the predicted (theoretical) forces
T = 1.5 s, H = 180 mm, d = 325 mm, h/d = 0.85

6.82 Comparison based on force maxima:
Measured (experimental) and the predicted (theoretical) forces
T = 2.2 s, H = 193 mm, d = 300 mm, h/d = 0.92

6.83 Comparison based on force maxima:
Measured (experimental) and the predicted (theoretical) forces
T = 2.2 s, H = 166 mm, d = 300 mm, h/d = 0.92

6.84 Comparison based on force maxima:
Measured (experimental) and the predicted (theoretical) forces
T = 2.2 s, H = 164 mm, d = 300 mm, h/d = 0.92

6.85 Comparison based on force maxima:
Measured (experimental) and the predicted (theoretical) forces
T = 1.9 s, H = 164 mm, d = 300 mm, h/d = 0.92

6.86 Comparison based on force maxima:
Measured (experimental) and the predicted (theoretical) forces
T = 1.9 s, H = 156 mm, d = 300 mm, h/d = 0.92

6.87 Comparison based on force maxima:
Measured (experimental) and the predicted (theoretical) forces
T = 1.9 s, H = 174 mm, d = 300 mm, h/d = 0.92

6.88 Comparison based on force maxima:
Measured (experimental) and the predicted (theoretical) forces
T = 1.5 s, H = 178 mm, d = 300 mm, h/d = 0.92

6.89 Comparison based on force maxima:
Measured (experimental) and the predicted (theoretical) forces
T = 1.5 s, H = 160 mm, d = 300 mm, h/d = 0.92
6.91 Comparison based on force maxima:
Measured (experimental) and the predicted (theoretical) forces
T = 1.5 s, H = 153 mm, d = 300 mm, h/d = 0.92

6.92 Comparison based on force maxima:
Measured (experimental) and the predicted (theoretical) forces
T = 2.2 s, H = 188 mm, d = 275 mm, h/d = 1

6.93 Comparison based on force maxima:
Measured (experimental) and the predicted (theoretical) forces
T = 2.2 s, H = 172 mm, d = 275 mm, h/d = 1

6.94 Comparison based on force maxima:
Measured (experimental) and the predicted (theoretical) forces
T = 1.9 s, H = 196 mm, d = 275 mm, h/d = 1

6.95 Comparison based on force maxima:
Measured (experimental) and the predicted (theoretical) forces
T = 1.9 s, H = 171 mm, d = 275 mm, h/d = 1

6.96 Comparison based on force maxima:
Measured (experimental) and the predicted (theoretical) forces
T = 1.9 s, H = 164 mm, d = 275 mm, h/d = 1

6.97 Comparison based on force maxima:
Measured (experimental) and the predicted (theoretical) forces
T = 1.5 s, H = 190 mm, d = 275 mm, h/d = 1

6.98 Comparison based on force maxima:
Measured (experimental) and the predicted (theoretical) forces
T = 1.5 s, H = 168 mm, d = 275 mm, h/d = 1

6.99 Comparison based on force maxima:
Measured (experimental) and the predicted (theoretical) forces
T = 1.5 s, H = 163 mm, d = 275 mm, h/d = 1

6.100 Comparison based on force maxima:
Measured (experimental) and the predicted (theoretical) forces
T = 2.2 s, H = 195 mm, d = 250 mm, h/d = 1.1
6.102 Comparison based on force maxima:
Measured (experimental) and the predicted (theoretical) forces
T = 2.2 s, H = 178 mm, d = 250 mm, h/d = 1.1

6.103 Comparison based on force maxima:
Measured (experimental) and the predicted (theoretical) forces
T = 1.9 s, H = 171 mm, d = 250 mm, h/d = 1.1

6.104 Comparison based on force maxima:
Measured (experimental) and the predicted (theoretical) forces
T = 1.9 s, H = 172 mm, d = 250 mm, h/d = 1.1

6.105 Comparison based on force maxima:
Measured (experimental) and the predicted (theoretical) forces
T = 1.9 s, H = 178 mm, d = 250 mm, h/d = 1.1

6.106 Comparison based on force maxima:
Measured (experimental) and the predicted (theoretical) forces
T = 1.5 s, H = 182 mm, d = 250 mm, h/d = 1.1

6.107 Comparison based on force maxima:
Measured (experimental) and the predicted (theoretical) forces
T = 1.5 s, H = 169 mm, d = 250 mm, h/d = 1.1

6.108 Comparison based on force maxima:
Measured (experimental) and the predicted (theoretical) forces
T = 1.5 s, H = 166 mm, d = 250 mm, h/d = 1.1

6.109 Comparison based on force maxima:
Measured (experimental) and the predicted (theoretical) forces
T = 2.2 s, H = 192 mm, d = 225 mm, h/d = 1.22

6.110 Comparison based on force maxima:
Measured (experimental) and the predicted (theoretical) forces
T = 2.2 s, H = 171 mm, d = 225 mm, h/d = 1.22

6.111 Comparison based on force maxima:
Measured (experimental) and the predicted (theoretical) forces
T = 2.2 s, H = 164 mm, d = 225 mm, h/d = 1.22
6.113 Comparison based on force maxima:
Measured (experimental) and the predicted (theoretical) forces
T = 1.9 s, H = 166 mm, d = 225 mm, h/d = 1.22

6.114 Comparison based on force maxima:
Measured (experimental) and the predicted (theoretical) forces
T = 1.9 s, H = 175 mm, d = 225 mm, h/d = 1.22

6.115 Comparison based on force maxima:
Measured (experimental) and the predicted (theoretical) forces
T = 1.5 s, H = 153 mm, d = 225 mm, h/d = 1.22

6.116 Comparison based on force maxima:
Measured (experimental) and the predicted (theoretical) forces
T = 1.5 s, H = 165 mm, d = 225 mm, h/d = 1.22

7.1 Armour unit placements

7.2 Forward force on the cubical unit at point E (leading edge)

7.3 Forward force on the cubical unit at point F

7.4 Forward force on the cubical unit at point G

7.5 Forward force on the cubical unit at point H (seaward toe)

7.6 Forward force on the cubical unit at points E, F, G, H
T = 2.2 s

7.7 Forward force on the cubical unit at points E, F, G, H
T = 1.9 s

7.8 Forward force on the cubical unit at points E, F, G, H
T = 1.5 s

7.9 Forward force on the cubical unit at points E, F, G, H
h/d = 0.73

7.10 Forward force on the cubical unit at points E, F, G, H
h/d = 0.79
7.12 Forward force on the cubical unit at points E, F, G, H
h/d = 0.85

7.13 Forward force on the cubical unit at points E, F, G, H
h/d = 0.92

7.14 Forward force on the cubical unit at points E, F, G, H
h/d = 1.0

7.15 Forward force on the cubical unit at points E, F, G, H
h/d = 1.1

7.16 Forward force on the cubical unit at points E, F, G, H
h/d = 1.22

7.17 Forward force on the cubical unit at point D (landward edge)

7.18 Forward force on the cubical unit at point C

7.19 Forward force on the cubical unit at point B

7.20 Forward force on the cubical unit at point A (landward toe)

7.21 Forward force on the cubical unit at points D, C, B, A
T = 2.2 s

7.22 Forward force on the cubical unit at points D, C, B, A
T = 1.9 s

7.23 Forward force on the cubical unit at points D, C, B, A
T = 1.5 s

7.24 Forward force on the cubical unit at points D, C, B, A
h/d = 0.73

7.25 Forward force on the cubical unit at points D, C, B, A
h/d = 0.79

7.26 Forward force on the cubical unit at points D, C, B, A
h/d = 0.85
7.28 Forward force on the cubical unit at points D, C, B, A  
  h/d = 0.92

7.29 Forward force on the cubical unit at points D, C, B, A  
  h/d = 1.0

7.30 Forward force on the cubical unit at points D, C, B, A  
  h/d = 1.1

7.31 Forward force on the cubical unit at points D, C, B, A  
  h/d = 1.22

7.32 Backward force on the cubical unit at point E

7.33 Forward and backward forces on the cubical unit at point E

7.34 Backward forces at points D and E

7.35 Backward force on the cubical unit at point F

7.36 Backward force on the cubical unit at point G

7.37 Backward force on the cubical unit at point H

7.38 Backward force on the cubical unit at point E, F, G, H  
  T = 2.2 s

7.39 Backward force on the cubical unit at point E, F, G, H  
  T = 1.9 s

7.40 Backward force on the cubical unit at point E, F, G, H  
  T = 1.5 s

7.41 Fx and Fy on the cubical unit at point H

7.42 Backward force on the cubical unit at points E, F, G, H  
  h/d = 0.73

7.43 Backward force on the cubical unit at points E, F, G, H  
  h/d = 0.79
7.45 Backward force on the cubical unit at points E, F, G, H h/d = 0.85
7.46 Backward force on the cubical unit at points E, F, G, H h/d = 0.92
7.47 Backward force on the cubical unit at points E, F, G, H h/d = 1
7.48 Backward force on the cubical unit at points E, F, G, H h/d = 1.1
7.49 Backward force on the cubical unit at points E, F, G, H h/d = 1.22
7.50 Backward force on the cubical unit at point D
7.51 Backward force on the cubical unit at point C
7.52 Backward force on the cubical unit at point B
7.53 Backward force on the cubical unit at point A
7.54 Backward force on the cubical unit at point A, B, C, D T = 2.2 s
7.55 Backward force on the cubical unit at point A, B, C, D T = 1.9 s
7.56 Backward force on the cubical unit at point A, B, C, D T = 1.5 s
7.57 Backward force on the cubical unit at point A, B, C, D h/d = 0.73
7.58 Backward force on the cubical unit at point A, B, C, D h/d = 0.79
7.59 Backward force on the cubical unit at point A, B, C, D h/d = 0.85

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7.61 Backward force on the cubical unit at point A, B, C, D
h/d = 1

7.62 Backward force on the cubical unit at point A, B, C, D
h/d = 1.1

7.63 Backward force on the cubical unit at point A, B, C, D
h/d = 1.22

7.64 Backward force on the cubical unit at point A, B, C, D
h/d = 1.38

7.65 Simultaneous Fx and Fy on the cubical units at points D and E
h/d = 0.85

7.66 Simultaneous Fx and Fy on the cubical units at points D and E
h/d = 1

7.67 Simultaneous Fx and Fy on the cubical units at points D and E
h/d = 1.22

7.68a Simultaneous tangential and normal forces on the cubical unit at point F, h/d = 0.85

7.68b Simultaneous Fx and Fy forces on the cubical unit at point F, h/d = 0.85

7.69a Simultaneous tangential and normal forces on the cubical unit at point F, h/d = 1

7.69b Simultaneous Fx and Fy forces on the cubical unit at point F, h/d = 1

7.70a Simultaneous tangential and normal forces on the cubical unit at point F, h/d = 1.22

7.70b Simultaneous Fx and Fy forces on the cubical unit at point F, h/d = 1.22

7.71a Simultaneous tangential and normal forces on the cubical unit at point G, h/d = 0.85

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7.72a Simultaneous tangential and normal forces on the cubical unit at point G, h/d = 1

7.72b Simultaneous Fx and Fy forces on the cubical unit at point G, h/d = 1

7.73a Simultaneous tangential and normal forces on the cubical unit at point G, h/d = 1.22

7.73b Simultaneous Fx and Fy forces on the cubical unit at point G, h/d = 1.22

7.74 Fx and Fy on the cubical unit at point H (seaward toe), h/d = 0.85

7.75 Fx and Fy on the cubical unit at point H (seaward toe), h/d = 1

7.76 Fx and Fy on the cubical unit at point H (seaward toe), h/d = 1.22

7.77 Upward force on the cubical unit at point E (seaward edge)

7.78 Upward force on the cubical unit at point F

7.79 Upward force on the cubical unit at point G

7.80 Upward force on the cubical unit at point H (seaward toe)

7.81 Upward force on the cubical unit at points E, F, G, H T = 2.2 s

7.82 Upward force on the cubical unit at points E, F, G, H T = 1.9 s

7.83 Upward force on the cubical unit at points E, F, G, H T = 1.5 s

7.84 Upward force on the cubical unit at points E, F, G, H h/d = 0.73
7.86 Upward force on the cubical unit at points E, F, G, H
h/d = 0.79

7.87 Upward force on the cubical unit at points E, F, G, H
h/d = 0.85

7.88 Upward force on the cubical unit at points E, F, G, H
h/d = 0.92

7.89 Upward force on the cubical unit at points E, F, G, H
h/d = 1

7.90 Upward force on the cubical unit at points E, F, G, H
h/d = 1.1

7.91 Upward force on the cubical unit at points E, F, G, H
h/d = 1.22

7.92a Upward force on the cubical unit at point E (seaward edge)

7.92b Upward force on the cubical unit at point F

7.93 Simultaneous Fx and Fy on the cubical unit at point C
h/d = 0.85

7.94 Simultaneous Fx and Fy on the cubical unit at point C
h/d = 1

7.95 Simultaneous Fx and Fy on the cubical unit at point C
h/d = 1.22

7.96 Simultaneous Fx and Fy on the cubical unit at point B
h/d = 0.85

7.97 Simultaneous Fx and Fy on the cubical unit at point B
h/d = 1

7.98 Simultaneous Fx and Fy on the cubical unit at point B
h/d = 1.22
7.100 Simultaneous Fx and Fy on the cubical unit at point A
h/d = 1

7.101 Simultaneous Fx and Fy on the cubical unit at point A
h/d = 1.22

7.102 Upward forces on the cubical unit at point D (Landward edge)

7.103 Upward forces on the cubical unit at point C

7.104 Upward forces on the cubical unit at point B

7.105 Upward forces on the cubical unit at point A (landward toe)

7.106 Upward forces on the cubical unit at points D, C, B, A
T = 2.2 s

7.107 Upward forces on the cubical unit at points D, C, B, A
T = 1.9 s

7.108 Upward forces on the cubical unit at points D, C, B, A
T = 1.5 s

7.109 Upward forces on the cubical unit at points D, C, B, A
h/d = 0.73

7.110 Upward forces on the cubical unit at points D, C, B, A
h/d = 0.79

7.111 Upward forces on the cubical unit at points D, C, B, A
h/d = 0.85

7.112 Upward forces on the cubical unit at points D, C, B, A
h/d = 0.92

7.113 Upward forces on the cubical unit at points D, C, B, A
h/d = 1

7.114 Upward forces on the cubical unit at points D, C, B, A
h/d = 1.1
7.116 Upward forces on the cubical unit at points D, C, B, A

\[ h/d = 1.22 \]

7.117 Forces on the cubical unit at point E (seaward edge)

\[ T = 2.2 \text{ s}, \; H = 196 \text{ mm}, \; d = 200 \text{ mm}, \; h/d = 1.38 \]

7.118 Forces on the cubical unit at point E (seaward edge)

\[ T = 1.9 \text{ s}, \; H = 169 \text{ mm}, \; d = 200 \text{ mm}, \; h/d = 1.38 \]

7.119 Forces on the cubical unit at point E (seaward edge)

\[ T = 1.5 \text{ s}, \; H = 142 \text{ mm}, \; d = 200 \text{ mm}, \; h/d = 1.38 \]

7.120 Downward force on the cubical unit at point E (seaward edge)

7.121 Downward force on the cubical unit at point F

7.122 Downward force on the cubical unit at point G

7.123 Downward force on the cubical unit at point H (seaward toe)

7.124 Downward force on the cubical unit at points E, F, G, H

\[ T = 2.2 \text{ s} \]

7.125 Downward force on the cubical unit at points E, F, G, H

\[ T = 1.9 \text{ s} \]

7.126 Downward force on the cubical unit at points E, F, G, H

\[ T = 1.5 \text{ s} \]

7.127 Downward force on the cubical unit at points E, F, G, H

\[ h/d = 0.73 \]

7.128 Downward force on the cubical unit at points E, F, G, H

\[ h/d = 0.79 \]

7.129 Downward force on the cubical unit at points E, F, G, H

\[ h/d = 0.85 \]

7.130 Downward force on the cubical unit at points E, F, G, H

\[ h/d = 0.92 \]
7.132 Downward force on the cubical unit at points E, F, G, H
h/d = 1

7.133 Downward force on the cubical unit at points E, F, G, H
h/d = 1.1

7.134 Downward force on the cubical unit at points E, F, G, H
h/d = 1.22

7.135 Downward force on the cubical unit at point D (landward edge)

7.136 Downward force on the cubical unit at point C

7.137 Downward force on the cubical unit at point B

7.138 Downward force on the cubical unit at point A (landward toe)

7.139 Downward force on the cubical unit at points D, C, B, A
T = 2.2 s

7.140 Downward force on the cubical unit at points D, C, B, A
T = 1.9 s

7.141 Downward force on the cubical unit at points D, C, B, A
T = 1.5 s

7.142 Downward force on the cubical unit at points D, C, B, A
h/d = 0.73

7.143 Downward force on the cubical unit at points D, C, B, A
h/d = 0.79

7.144 Downward force on the cubical unit at points D, C, B, A
h/d = 0.85

7.145 Downward force on the cubical unit at points D, C, B, A
h/d = 0.92

7.146 Downward force on the cubical unit at points D, C, B, A
h/d = 1

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7.148 Downward force on the cubical unit at points D, C, B, A
h/d = 1.22

7.149 Downward force on the cubical unit at points D, C, B, A
h/d = 1.38

7.150 Horizontal forces on the cubical units arranged in tandem
at the seaward edge on the crest, h/d = 0.92

7.151 Vertical forces on the cubical units arranged in tandem
at the seaward edge on the crest, h/d = 0.92

7.152 Horizontal forces on the cubical units arranged in tandem
at the seaward edge on the crest, h/d = 1

7.153 Vertical forces on the cubical units arranged in tandem
at the seaward edge on the crest, h/d = 1

7.154 Horizontal forces on the cubical units arranged in tandem
at the seaward edge on the crest, h/d = 1.22

7.155 Vertical forces on the cubical units arranged in tandem
at the seaward edge on the crest, h/d = 1.22

7.156 Forward forces on the cubical units arranged in tandem
at the seaward edge on the crest, T = 2.2 s

7.157 Forward forces on the cubical units arranged in tandem
at the seaward edge on the crest, T = 1.9 s

7.158 Forward forces on the cubical units arranged in tandem
at the seaward edge on the crest, T = 1.5 s

7.159 Comparison: Forward forces on the cubical units arranged in tandem
at the seaward edge on the crest, T = 2.2 s, 1.9 s, and 1.5 s

7.160 Upward forces on the cubical units arranged in tandem
at the seaward edge on the crest, T = 2.2 s

7.161 Upward forces on the cubical units arranged in tandem
at the seaward edge on the crest, T = 1.9 s

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7.163 Comparison: Upward forces on the cubical units arranged in tandem at the seaward edge on the crest, $T = 2.2$ s, 1.9 s, and 1.5 s

7.164 Effect of seaward slope on the forward force on the cubical unit at the seaward edge, $T = 2.2$ s

7.165 Effect of seaward slope on the forward force on the cubical unit at the seaward edge, $T = 1.9$ s

7.166 Effect of seaward slope on the forward force on the cubical unit at the seaward edge, $T = 1.5$ s

7.167 Effect of seaward slope on the upward force on the cubical unit at the seaward edge, $T = 2.2$ s

7.168 Effect of seaward slope on the upward force on the cubical unit at the seaward edge, $T = 1.9$ s

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8.1 Forces on the cubical unit
8.3 Total force envelope on the cubical unit at point E (seaward edge)
\[ T = 2.2 \text{ s}, \quad H = 163 \text{ mm}, \quad h/d = 0.79 \]

8.4 Total force envelope on the cubical unit at point E (seaward edge)
\[ T = 1.9 \text{ s}, \quad H = 154 \text{ mm}, \quad h/d = 0.79 \]

8.5 Total force envelope on the cubical unit at point E (seaward edge)
\[ T = 1.5 \text{ s}, \quad H = 155 \text{ mm}, \quad h/d = 0.79 \]

8.6 Total force envelope on the cubical unit at point E (seaward edge)
\[ T = 2.2 \text{ s}, \quad H = 175 \text{ mm}, \quad h/d = 1.22 \]

8.7 Total force envelope on the cubical unit at point E (seaward edge)
\[ T = 1.9 \text{ s}, \quad H = 175 \text{ mm}, \quad h/d = 1.22 \]

8.8 Total force envelope on the cubical unit at point E (seaward edge)
\[ T = 1.5 \text{ s}, \quad H = 172 \text{ mm}, \quad h/d = 1.22 \]

8.9 Dynamic instability on the cubical unit at point E (seaward edge)
\[ T = 2.2 \text{ s}, \quad H = 163 \text{ mm}, \quad h/d = 0.79 \]

8.10 Dynamic instability on the cubical unit at point E (seaward edge)
\[ T = 1.9 \text{ s}, \quad H = 154 \text{ mm}, \quad h/d = 0.79 \]

8.11 Dynamic instability on the cubical unit at point E (seaward edge)
\[ T = 1.5 \text{ s}, \quad H = 155 \text{ mm}, \quad h/d = 0.79 \]

8.12 Dynamic instability on the cubical unit at point E (seaward edge)
\[ T = 2.2 \text{ s}, \quad H = 175 \text{ mm}, \quad h/d = 1.22 \]

8.13 Dynamic instability on the cubical unit at point E (seaward edge)
\[ T = 1.9 \text{ s}, \quad H = 175 \text{ mm}, \quad h/d = 1.22 \]

8.14 Dynamic instability on the cubical unit at point E (seaward edge)
\[ T = 1.5 \text{ s}, \quad H = 172 \text{ mm}, \quad h/d = 1.22 \]

8.15 Envelope of instability number on the cubical unit at point E (seaward edge)
\[ T = 2.2 \text{ s}, \quad H = 163 \text{ mm}, \quad h/d = 0.79 \]

8.16 Envelope of instability number on the cubical unit at point E (seaward edge)
\[ T = 1.9 \text{ s}, \quad H = 154 \text{ mm}, \quad h/d = 0.79 \]
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   $T = 1.5 \text{ s, } H = 155 \text{ mm, } h/d = 0.79$  \hspace{1cm} 567

8.19 Envelope of instability number on the cubical unit at point E (seaward edge)
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8.21 Comparison: Sliding instability number on the cubical units at points
   D (landward edge) and E (seaward edge)  \hspace{1cm} 570

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8.23 Comparison: Sliding and overturning on the cubical units at points
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$A_{\text{face}}$: Front surface area of the cubical unit

$A_x$: Projected area of the armour unit in the vertical plane perpendicular to the incoming waves

$A_y$: Projected area of the armour unit in the horizontal plane

$a$: Side length of the cubical unit

$B$: Crest width of the breakwater

$b$: Distance between the two vertical wire used in the dynamometer

$C_D$: Empirical drag coefficient

$C'_D$: Drag index

$C_{im}$: Impact index

$C_M$: Empirical inertia coefficient

$C'_M$: Inertia index

$C_m$: Added mass coefficient in the inertia force

$c$: Wave celerity

$cn$: Elliptic cosine function in cnoidal wave theory

$D_e$: Equivalent diameter of the armour unit

$D_{ex}$: Equivalent diameter of the projected area of the armour unit to a vertical plane perpendicular to the incoming wave

$D_{ey}$: Equivalent diameter of the projected area of the armour unit to a horizontal plane

$d$: Still water depth at the structure
\( d_{b_{\text{max}}}: \) Maximum breaker depth

\( d_{b_{\text{min}}}: \) Minimum breaker depth

\( d_s: \) Still water depth at the toe in Galvin's travel breaker

\( d_t: \) Vertical distance from bottom to the wave trough

\( E: \) Total wave energy per unit crest width

\( E(k): \) The second complete elliptic integral

\( E_k: \) Kinetic wave energy

\( E_p: \) Potential wave energy

\( F_D: \) Drag force on the armour unit

\( F_f: \) Static friction force on the armour unit

\( F_{\text{imp}}: \) Impact force on the cubical unit

\( F_M: \) Inertia or mass force on the armour unit

\( F_{\text{mi}}: \) Measured horizontal force at time point \(i\)

\( F_T: \) Total hydrodynamic force on the armour unit

\( F_{T*}: \) Dimensionless total hydrodynamic force on the armour unit

\( F_x: \) Dynamic horizontal force on the armour unit due to wave action

\( F_{x*}: \) Dimensionless longitudinal force

\( F_y: \) Dynamic vertical force on the armour unit due to wave action

\( F_{y*}: \) Dimensionless vertical force

\( F_{y1}: \) Vertical force recorded by the load cell at the front side

\( F_{y2}: \) Vertical force recorded by the load cell at the back side
g : Gravity acceleration
H: Wave height
H_b: Breaking wave height
H_i: Incoming wave height
H_r: Apparent reflected wave height
H_s: Significant wave height
H_t: Transmitted wave height
H_{se}: Wave height at the seaward toe of the breakwater
h: Height of breakwater
K_C: Keulegan-Carpenter number
K_{D}: Hudson's stability coefficient of rubble-mound breakwater armour units
K_{ds}: Dissipation coefficient
K(k): The first complete elliptic integral
K_r: Reflection coefficient
K_t: Wave transmission coefficient
K_u: Coefficient for wave transmission over the breakwater
K_n: Coefficient for wave transmission through the breakwater
k: Surface roughness of the cubical units
k: Modulus of elliptic integral in cnoidal wave theory where 0 ≤ k ≤ 1
k_s: Diameter of a block sphere of equivalent weight
k_Δ: Layer coefficient
\( M_r \) : Restricting overturning moment on the cubical unit

\( M_s \) : Spinning moment about the centre of mass of the cubical unit

\( m \) : Nearshore slope

\( m_a \) : Added mass in the inertia force

\( N_s \) : Stability number

\( n \) : Number of rocks armour units comprising the cover layer

\( n_p \) : Porosity of the breakwater

\( R_c \) : Critical crest height

\( Re \) : Reynolds number

\( R_n \) : Reaction of the cubical unit to the other rock in normal direction

\( r \) : The thickness of the armour layer

\( SF \) : Shape factor of the armour unit

\( T \) : Wave period

\( t \) : Instantaneous time

\( U \) : Ursell's or Stokes parameter

\( U \) : Velocity component in x axis

\( u_o \) : Measured maximum velocity

\( V_s \) : Volume of the stone

\( V \) : Velocity component in y axis

\( W \) : Mass of individual armour unit

\( W' \) : Submerged weight of the cubical unit
\( w_r \): Unit weight of armour unit.

\( x_p \): Distance travelled by a plunging breaker

\( \alpha \): The angle between total force and horizontal line

\( \alpha \): Upper limit of breaker travel by Galvin

\( \beta \): Lower limit of breaker travel by Galvin

\( \beta \): Angle of orientation of the cubical armour unit

\( \gamma_r \): Specific weight of the rock

\( \gamma_f \): Specific weight of fluid

\( \Delta t \): Time step

\( \xi \): Surf similarity parameter

\( \eta \): Water surface elevation

\( \Theta \): Slope of breakwater

\( \lambda \): Length ratio of the model

\( \mu \): Fluid viscosity

\( \rho_r \): Mass density of rocks

\( \rho_w \): Mass density of water

\( \sigma \): Surface tension

\( \sigma \): The sum of error in the Least Square Method

\( \tau_p \): Dimensionless plunge distance

\( \phi \): Velocity potential

\( \Phi \): Angle of repose of the rocks

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1.1 **Background**

A rational method for designing stable size of armour units requires a good understanding of forces exerted by breaking waves on the armour units. Unfortunately, a thorough knowledge about that does not exist at the present time. As a result, another approach using statistical analysis has been used for designing armour units. The most widely used method of determining armour unit size is the Hudson formula, U.S. Army Corps of Engineers [Shore Protection Manual, SPM (1984)], which has been used in similar recent developments such as the Koev formula, Koev (1992), and Sherbrook University formula, Ben Belfadhel et al., (1993). Other researchers, using statistical analysis proposed different formulas such as the Bertram formula, Taylor (1973), and the Van Der Meer formula, Van der Meer (1988). Some other studies have also been carried out to investigate the stable size of armour units using the same approach [Gravenson, et al. (1980), and Ahrens (1987)]. Although these studies presented different formulas, they basically use the same approach of introducing a damage criteria, or a damage coefficient which is difficult to determine because of its dependence on many factors.

A more rational concept of determining the stable size of armour units has been initiated by other researchers. They conducted force measurements on armour units in order to understand the dynamic forces induced by the waves. Based on the force records, the stability of the armour units was analyzed. Among those who used this approach are Sigurdsson (1962) who used
layers, Mizutani, et al. (1993) who used spherical armour units, and Pramono (1994) who tested natural armour units. Sigurdsson, and Juhl and Jensen conducted studies on the non-overtopped breakwater type, whereas Mizutani et al., and Pramono conducted studies on the overtopped breakwater type.

Coastal erosion is one of the main problems encountered in coastal areas; this problem requires careful study to prevent it, while at the same time an effort must be made to reduce side effects caused by the coastal erosion control structure. A non-overtopped detached breakwater is the most popular coastal erosion control structure to solve the problem. This type of the structure, when designed and constructed properly, can completely eliminate the direct wave attack to the shoreline. Coastal erosion protection, however, only needs to eliminate the excessive wave energy attack on the shoreline. The critical wave energy that starts to disturb the equilibrium in coastal areas depends on the characteristics of the beach, e.g. slope, and bathymetry, and the characteristics of the bed material in the beach. An overtopped breakwater is a type of breakwater which provides a good solution for coastal erosion protection. Low-crested, and submerged breakwaters reduce the energy by breaking the incoming waves, but still permit flushing to the beach, and thus, are environmentally better than non-overtopped breakwaters.

As it has been mentioned above that coastal erosion protection should eliminate the excessive energy from the incoming waves. Therefore, it is essential to study the characteristics of the beach, prevailing longshore transport and the bed material, to determine the critical wave energy
incoming wave energy is greater than the critical energy. Although beach and bed material characteristics investigations are beyond the scope of this study, the critical wave energy can be related to the transmitted waves by the overtopped breakwater. For that purpose three wave probes were used on the landward side of the breakwater in this research to study the relationship between wave transmission, submergence, and the dimension of the breakwater at the given incoming wave characteristics. The wave energy carried by the transmitted waves must not exceed the critical energy in the local beach.

It has been pointed out by Mizutani, et al (1993), in their study on wave forces on a wide crown submerged breakwater, and Pramono (1994) that the most critical location for armour units on a submerged breakwater is at the leading edge of the crest. Several papers have also noted that wave force is sensitive to the submergence of the breakwater. Therefore, a wide range of submergence from a low-crested to a submerged breakwater is used to identify the most critical condition under which the force acting on the leading edge rock reaches its maximum. Since the orientation of the unit is also suspected to affect the force, three different angles of orientations are selected in this study. Another important aspect of designing armour units is the determination of the most economical size of the units. The common practise for selecting the size of the armour unit has been based on the most critical location. However, such a decision has an economical implication because the unnecessary material cost. Since the dynamic force acting on each individual armour unit varies depending on the location of the rock, the maximum force field along the surface of the breakwater is investigated to provide better information for the
different armour units, simultaneous measurements on pairs of rocks must be conducted, and for this reason two dynamometers were used.

The use of the overtopped breakwater as an erosion control structure also provides a more economical solution in comparison to the use of the non-overtopped type for at least two reasons: smaller magnitude of hydrodynamic force acting on the armour units, and less material used to construct the structure. Williamson and Hall (1994) pointed out that most failures of breakwaters are caused by the movement of armour layer and followed by the damage to the interior layers. When breaking waves intensely strike the armour layer, a strong impact is produced causing a high pore pressure within the breakwater. If this condition is followed by sudden drawdown, the instantaneous combination of very high internal pressure in the core and low external pressure at the surface of the breakwater may cause geotechnical failure. A large size of armour rock is therefore needed to withstand the force induced by this high pressure. This short duration high pressure generally does not exist in the submerged breakwater because the large energy induced by breaking waves can travel over the crest, reducing the possibility of geotechnical failure when the slope of the submerged breakwater is designed properly. Ahren (1989) noted that energy dissipation within a reef (homogeneous low-crested) breakwater is so large that it cannot fail catastrophically. With this consideration, a smaller size of rock can be used for the armour layer. Since the dimensions of low-crested or submerged breakwater are smaller than the dimensions of the non-overtopped type, less materials are needed for the structure. The combination, of smaller size of rock and less materials needed to construct a low-crested or submerged
1.2 Objectives

The objective of this research is to study experimentally the hydrodynamic forces exerted by breaking waves on a cubical shaped armour unit for low-crested and submerged breakwaters. The maximum forces along the surface of the breakwater are investigated, and the critical location and conditions are identified. Using this information, the stable and economical size of the armour unit is investigated. In order to achieve this purpose an experimental study was carried out under various water depths, wave heights and periods, slopes of the breakwater, and orientations of the armour units.

1.3 Definition of the problem

The stable and economical size of armour unit depends on the forces acting on it. The maximum hydrodynamic force field along the surface of submerged or low-crested breakwaters has never been published. In order to establish this force field, a scale model was constructed. Several locations on the slope and crest of the breakwater were chosen to locate the cubical armour unit models attached to dynamometers. Figure 1.1 shows the placements of the two cubical armour units in the dynamometers. Points A, B, C, D, E, F, G, and H represent the selected locations of the two dynamometers. The simultaneous measurements of the force were made at two locations with the following combinations: D-E, C-F, B-G, and A-H. Knowing the force acting on the two
identified. Figure 1.2 shows the definition sketch for the wave, forces and spinning moment acting on the armour unit, the unit location, elevation, and angle of orientation, the geometry of the breakwater, and the water depth.

The dynamics of wave forces and moments on an armour unit under breaking waves condition are very complex depending on such factors as:

(a) location and orientation of the armour unit on the slope or crest of the breakwater,
(b) degree of submergence of the breakwater,
(c) shape, roughness, and sharpness of the armour unit,
(d) degree of interlocking between the armour unit with the other surrounding stones,
(e) permeability of the breakwater,
(f) internal pore pressure of the breakwater,
(g) angle and surface roughness of the slope,
(h) air entrainment that affects the density of the fluid and makes the velocity field uncertain,
(i) wave groupiness effect on the structure.

Direct measurement of the hydraulic forces acting on an armour units, therefore, is an appropriate method of investigating the influence of the various geometric and kinematic variables affecting armour unit stability.
In the experimental studies reported here, the dynamic wave forces and the spinning moments on an armour unit model have been measured by dynamometers for various wave periods and heights, submergences of the structure, location of the armour unit at the crest and on the slope, and three different angle of orientations of the armour units.

Two dynamometers and associated armour units specially designed for this study were placed at the chosen locations to study the simultaneous forces and the force phase acting on the armour units at the selected positions.

By considering the magnitude of the force a better concept of armouring the breakwater can be achieved to produce a more stable and economical design.

About two hundred calibration tests were conducted to determine the positions of the wave paddle to produce three different wave heights at each wave period of $T=1.5\ s$, $T=1.9\ s$, and $T=2.2\ s$, in which the three wave heights are approximately the same for the selected wave periods.

Wave heights were measured using seven wave probes placed along the flume. Three probes were located in the offshore zone, one probe was placed at the seaward toe of the breakwater, and another three probes were placed between the leeward toe and the beach section. The
method, whereas the wave length was computed based on Airy and Third Order Stokes methods, and therefore, the steepness of the waves could be computed. The reflected waves were corrected using Third Order Stokes theory.

Velocity measurements were made using a differential pressure transducer and modified Pitot tube (Mohamed, M.S., 1992). The velocity of the water slightly above the top surface of the armour unit at the leading edge was studied to separate the inertial and the drag force from the total force measured by the load cell.

The data were collected using the OPTIM-TCS3000 data acquisition system and a microcomputer. The measured data were analyzed with the aid of a spread sheet software to study the effect of submergence, location, and the geometry of the breakwater. The results are presented in dimensionless form for practical application purposes.
2.1 Introduction

Coastal erosion is one of the main problems encountered in coastal areas. In Indonesia, Tapak Kuda Island in North Sumatra will disappear within about 85 years. The size of this small island has been reduced from about 1,500 hectares to about 3 hectares. In some places of North Java coastline, the annual erosion rate reaches up to 30 to 50 meters. In 1993, several governmental buildings in Sumatra Island collapsed due to waves attacks. At the present time, the most popular solution against coastal erosion is the construction of non-overtopped detached rubble-mound breakwaters. Rubble-mound breakwaters have been used to protect shorelines from wave action, prevent beach erosion, and supply beach sand by interrupting longshore and wave-generated current. The use of these structures has become popular because of their relatively low cost and effectiveness in wave control. Herbich (1989) noted that in the last two decades the number of offshore breakwaters in Japan increased about ten times while the increase in groin-type structures was about two times. However, the choice of non-overtopped breakwater poses some problems in a coastal amenity. Specifically, it creates an obstruction to water exchange between the two regions separated by the structure, and there is an aesthetics problem.

In 1976, the breakwater at Rossalyn Bay in Australia was severely damaged during Cyclone David (Bremner et al. 1980). The height of the crest was reduced as much as 4 m by the cyclone but still functioned effectively as a submerged breakwater for over 2 years until it was repaired.
findings from the model tests, a low-crested structure was chosen for the breakwater at Townsville Harbour, Australia (Ahren, 1987). Rufin et al. (1994) noted that the installation of submerged breakwaters in Japan in the prevention of coastal erosion and other related coastal disasters is notably increasing. The submerged breakwater is a useful coastal structure that can avoid some of the disadvantages of a conventional breakwaters.

Submerged and low-crested breakwaters are designed to reduce wave energy by breaking the incoming waves, while still allowing flushing of the coastal zone. The difference between submerged and low-crested breakwaters lies on the ratio between the water depth (d) and the height of the crest (h). Submerged breakwaters have h/d < 1, whereas low-crested breakwaters have h/d > 1 but may be overtopped.

The knowledge of wave forces and spinning moments on the armour units induced by breaking waves and near breaking waves is important for the best design of submerged and low-crested structures. Factors affecting the forces on the armour units are complex, but they can be divided into three main groups. The first is the characteristics of the incident waves which includes wave height, wave period, wave steepness, and the still water depth. The second is the characteristics of the breakwaters consisting of the geometry of the structure, and the characteristic of the material used including the rock size, shape, orientation, roughness, and porosity. The third is the interactions between the incoming waves and the breakwaters. This is a very complex interaction which produces shoaling, breaking, reflection, refraction, interference, transmission, and
written on wave forces. Therefore, only literature related closely to the objective of this study are presented.

It is difficult to quantify the magnitude and direction of wave forces on the armour units in the form of simple formula because of the complexity of the problem. This has led researchers to study and quantify them only on the basis on a limited number of factors, while minimizing the effects of the other factors.

In this chapter the available literature related to the objective of this study is presented. The review includes literature on:

(a) hydraulic studies of non-overtopped breakwaters,
(b) hydraulic studies of submerged and low-crested breakwaters, and
(c) scale effects.

2.2 Hydraulics Studies of Non-overtopped Breakwaters

Based on the crest elevation relative to the still water depth, breakwater structures can be divided into non-overtopped and overtopped breakwaters. The former, are referred to conventional breakwaters, whose crests are not be exceeded by wave uprush, whereas the crests in the latter are normally exceeded. By comparison the conventional type is older than the second and has been studied more. Amongst the many papers that have been published on this subject is
which is similar to that previously developed by Iribarren (1938), and Iribarren and Hudson (1951).

The Hudson formula (1958) is widely used because of its simplicity for determining stable size of armour units for conventional breakwaters.

\[ W = \frac{\gamma_r H_{toc}^3}{K_D \left( \frac{\gamma_r}{\gamma_f} - 1 \right)^3 \cot \alpha} \]  

(2.1)

where \( W \) is the weight of the armour unit, \( \gamma_r \) and \( \gamma_f \) the specific weight of rock and fluid, \( H_{toc} \) is the wave height at the location of the proposed structure, \( \alpha \) is angle between slope and horizontal, and \( K_D \) is damage coefficient or Hudson's stability coefficient of rubble-mound breakwater armour units. The stable stone mass is related to relative mass density, wave height, and the given slope angle, whereas other influencing factors including permeability, wave period, random waves, and wave groups are not explicitly taken into account. The use of this formula, therefore, has raised a lot of discussions because of the neglected parameters as well as other reasons such as Hudson's damage coefficient \( K_D \). The U.S. Army Coastal Engineering Research Centre (1984) reported that the coefficient \( K_D \) varies, amongst other factors, with the shape of the armour units, the sharpness of the edges, and the degree of interlocking obtained in placement. The right choice of coefficient \( K_D \) can be difficult. Timco et al (1984) studied the effect of permeability on the stability of the breakwater. It was concluded that the permeability of the core has a very definite effect on the energy dissipation in the breakwater. The greater the
overall stability of the breakwater. Hedar (1986) suggested improved stability formulas for rubble-mound breakwaters. The permeability of underlayers and the core of the breakwater, the angle of repose of the material, and the wave period were considered in the proposed formulas. The weight of the armour unit is given as follows:

\[
W = \frac{\pi}{6} \gamma_s k_s^3
\]

in which \(k_s\) = the diameter of a block sphere of equivalent weight. The diameter \(k_s\) is considered as a function of breaker depth \(d_b\), breaker height \(H_b\), angle of repose \(\Phi\), slope of the breakwater \(\alpha\), the specific weight of rock \(\gamma_s\) and fluid \(\gamma_f\). Hedar proposed four different formulas for \(k_s\) depending on uprush or downrush, and pervious or impervious underlayers. For uprushing wave phase, the diameter \(k_s\) for both pervious and impervious underlayers are respectively:

a. Pervious underlayer

\[
k_s = \frac{0.33 \ (d_b + 0.7H_b)(\tan \phi + 2)}{\left(\frac{\gamma_s}{\gamma_f} - 1\right)(3.6 - \frac{1}{e^{4\tan \beta}}) \cos \alpha \ (\tan \phi + \tan \alpha)}
\]

b. Impervious underlayer

\[
k_s = \frac{0.41 \ (d_b + 0.7H_b)(\tan \phi + 2)}{\left(\frac{\gamma_s}{\gamma_f} - 1\right)(3.3 - \frac{1}{e^{4\tan \beta}}) \cos \alpha \ (\tan \phi + \tan \alpha)}
\]

where \(\beta = \alpha + (\Phi - 48^\circ)\). For the downrushing wave the formulas of \(k_s\) are given below:
\[ k_s = \frac{(d_b + 0.7H_b)(\tan \phi + 2)}{\left(\frac{\gamma_s}{\gamma_f} - 1\right)(e^{4\tan \beta} + 13.7) \cos \alpha (\tan \phi + \tan \alpha)} \] (2.5)

b. Impervious underlayer

\[ k_s = \frac{1.6 \left( d_b + 0.7H_b \right)(\tan \phi + 2)}{\left(\frac{\gamma_s}{\gamma_f} - 1\right)(e^{4\tan \beta} + 16.5) \cos \alpha (\tan \phi + \tan \alpha)} \] (2.6)

where \( \beta = \alpha - (\Phi - 48^\circ) \).

Van der Meer (1988) proposed new stability formulas for rubble mound revetments and breakwaters under random wave attack using deterministic and probabilistic approaches. He incorporated, in his formulas, the influences of wave height and period, damage level, slope angle, permeability, and storm duration. The effect of wave grouping on the stability of armour units was considered to be very small and could be ignored. The proposed formulas have attracted some detailed discussions by Bruun (1990), Hedar (1990), Medina et al (1990), and Losada et al (1990), who have also recommended further investigations for a better design. Contrary to the proposed formula by Van der Meer, Bruun (1985) reviewed the design of rubble mound breakwater and emphasized that the stability of the structure is sensitive to the wave groups, so that wave grouping characteristics should be taken into account to the design of stable armour units. Medina et al (1990) conducted a series of large scale experiments at the O.H. Hindsdale Wave Research Facility at Oregon State University with the goal of resolving the
of wave groups. Two different methodologies which were considered as being representative of the state-of-the-art design methods were used: 1) the S.P.M. method (1984) and 2) the Van der Meer method (1988). The stability of the armour layer in both of these methods is related to a stability number $N_s$, defined as:

$$N_s = \frac{\frac{1}{\rho_r^3} H_s}{\frac{1}{W_{50}^3} \left(\frac{\rho_r}{\rho_w} - 1\right)}$$

(2.7)

where $H_s$ is the significant wave height, $W_{50}$ is the median value of mass distribution of rocks in the armour, and $\rho_r$ and $\rho_w$ are the mass density of rocks and water respectively. There are also secondary factors that affect the damage: slope of the breakwater, thickness of armour layer, permeability of the structure, number of waves in a run, roughness and placement of the armour rocks, mean period, and wave groupiness. Most of those secondary factors were held constant in order to compare the stability number $N_s$ and the wave grouping characteristics of wave runs with the damage or erosion of the armour layer. The characteristics of wave groups in the past have only been correlated with spectral peakedness, with mean run length, or with other related parameters. In their investigation, they considered two fundamental wave grouping characteristics: 1) the spectral shape related to the mean run length, and 2) the energy flux exceedence pattern related to the groupiness factor. It should be noted that the two wave grouping characteristics used above were based on the wave grouping characterization model proposed by Mase and Iwagaki (1986) which are 1) a run length, and 2) wave groupiness factor. The spectral shape and
Results from their research showed that spectral peakedness parameter and the mean run lengths are incomplete parameters for analyzing armour stability. They found that the measured damage functions are dependent on the Envelope Exceedance Coefficient, and the Groupiness Factor. However, they could not give a more precise description of the effects of wave groups on the stability of rubble mound breakwaters because of the statistical variability of the observations, and the limited number of cases analyzed in their experiments.

In the meantime the mathematical modelling approach has also been used to study the dynamic stability of rubble mounds. Hannoura and McCorquodale (1985) developed a generalized conductivity equation for the 2-D confined, unsteady, one- and/or two-phase flow in rubble mounds. The effect of air entrainment was considered in the proposed equation. It was found that air entrainment can significantly reduce the hydraulic conductivity which can affect reflection, transmission, buildup of the internal water surface and seepage forces in the structure. The equation was then used to develop a numerical model of wave motion to simulate the phreatic surface movement and temporal changes in pore pressure in rubble-mounds under a specified wave attack. The model was applied to the Port Sines breakwater in Portugal in order to check the stability of the seaward slope under violent wave attack. Their model showed that the buildup of the pore pressures may have contributed significantly to the failure of this structure.

Hall (1990) conducted an experimental study to further investigate the effects of air entrainment within the voids of rubble mound breakwater on the stability of the armour units. Previous
Entrainment of air in the flow reduces the effective conductivity of the structure. Three sources of air entrainment were observed: 1) direct result of wave breaking onto the outer slope of the breakwater, 2) flow separation resulting from a rapid flow field movement past a solid obstacle e.g. the armour layer unit, filter layer, or core material, and 3) uprushing wave overtaking the internal movement of the internal phreatic surface causing an unsaturated zone to develop. Since hydraulic conductivity has influence on the breakwater armour stability, and the effectiveness of the breakwater in minimizing transmission through and reflection off the structure, several important variables were applied to study the degree of air entrainment in the breakwater. Three different slopes (1:1.5, 1:2, and 1:3); three armour unit types (stones, cubes, and spheres); three sizes of core materials ($D_{50} = 3.5$ mm, 14 mm, 16 mm); and five different thicknesses of armour layer varied from one to five units were used by Hall (1987) in his study. In most of the tests he observed that air entrainment occurred above the SWL, which confirmed what had been noted by Hannoura and McCorquodale (1978). Air bubbles were driven into the breakwater during runup, so that they were moving in the direction of the flow, and then moved vertically as the buoyant force became dominant. He found that air entrainment was the most severe in tests undertaken with spheres in the armour layer. As the slope of the breakwater became steeper, both the air concentration and extent to which air bubbles penetrated the structure increased. Finally, the air entrainment became more severe as the permeability of the core material used increased.

It has been pointed out that internal pore pressure plays an important role to the stability of armour units of conventional breakwaters. Hall (1994) conducted an experimental study on
external pressures were analyzed by studying the external flow at the outer face and the internal flow within porous breakwater caused by wave induced pressure gradient. Wave train, surface roughness, geometry of the breakwater, infiltration of energy into the structure and the subsequent return of energy in the outflow are factors that mainly govern the external flow, and consequently the external pressure as well. The internal pressure depends on the internal flow which is governed by the hydraulic conductivity of the porous medium, which is influenced by material characteristics, air entrainment, and inertial effects. He found that the external pressure response curves were characterized by sharp peaks and rapid rise time, whereas the internal pressure response was smooth and gradual due to damping effects within the armour. He found that the external pressure was not significantly affected by the core material type, number of layers, and shape of the armour units. However, there was a marked influence on the internal pressure field caused by the core material type, the number of layers, and the shape of the materials in the armour and filter layers, and in the core. He also found that the critical parameter governing the stability of conventional breakwater lies on the interaction between the outflow from the structure and the subsequent incoming wave.

Kobayashi and Jacobs (1985) developed a mathematical model to predict the characteristics of flow in the downrush of regular waves and the critical condition for initiation of movement of armour units on the slope of a coastal structure. The effects of permeability, bottom friction, and water depth were neglected in this model. Bruun (1986) indicated that those effects should be considered because they would influence the velocities, and thereby, the lift and drag forces, as
stability occurred when the downrush forces combined with hydrostatic forces from the water pressure inside the structure and suction forces in the toe of the breaking waves. If the permeability is low, or the slope is impermeable, the hydrostatic pressure diminished, and therefore, impermeability could improve the stability of revetments as confirmed by some tests in Holland on coastal protection revetments. The effect of friction was then considered by Kobayashi and Otta (1987) when they developed a numerical stability model to predict the hydraulic stability of armour units on a rough impermeable mild slope under the action of a normally-incident wave train. The lift, drag, and inertia forces are expressed in terms of velocity and acceleration on a rough slope which are predicted independently using a numerical flow model based on a finite difference method. The hydraulic stability analysis was performed for the initiation of armour movement and for sliding motion during downrush and uprush as well. When the slope is relatively steep, the model is not applicable. Chian and Gerritsen (1990) used BEM approach to develop a 2-D flow model to simulate run-up of a non-breaking solitary wave on a mild or steep impermeable smooth slope. The run-up flow model is then combined with an armour stability model to predict the stability of armour units.

Research within European MAST-G6S program has led to the development of the One Dimensional FLOW in Coastal Structures numerical model (ODIFLOCS). The model was developed at the Delft University of Technology by van Gent (1992 and 1993-b), and it can simulate wave motion on and in several types of coastal structures. ODIFLOCS contains several parameters that can be adjusted to calibrate the numerical model with the physical model test
reasonably well with the physical model results. Comparison of ODIFLOCS numerical model to the physical model on submerged rubble mound breakwater conducted by Elkamawy (1995) has also shown a reasonably good agreement.

Wave runup is a subject that has gained a lot of attention on the design of conventional breakwaters. Wave runup is an important factor to be considered in designing conventional breakwaters to determine the elevation of the crest to prevent overtopping, or to evaluate the amount of overtopping. Runup is the vertical height above SWL to which water from incoming waves will rise on the slope of a structure. There are many factors affecting wave runup including incoming wave characteristics, leeward slope, SWL, roughness of the slope, permeability and geometry of the structure. Sollitt and DeBok (1976) noted that wave runup is a Reynolds number dependent physical phenomenon which is affected by the coupling between wave induced motion and real fluid forces. They also noted that a scale effect results due to frictional forces are known to be enhanced at low Reynolds number, causing greater resistance to runup and larger destabilizing forces on armour units. This brings a consequence that a small scale model may lead to structures which are underdesigned relative to runup while overdesigned for armour unit stability. Dretta and Sollit (1994) conducted a large scale (1:10) experimental study on wave runup on a dolos armoured slope. A large scale was chosen to avoid the problem mentioned above. They concluded that 1) steeper waves produced less wave runup than less steep waves; 2) longer waves experience greater runup than shorter waves; and 3) wave steepness is a stronger parameter for controlling wave runup than relative water depth.
and moments acting on the armour unit at a specified location. Many researchers have for many years looked into the question of actual wave forces on breakwater armour units, but only a few have directly studied these forces by experimental measurement on armour units. Sigurdsson (1962) conducted measurements of wave forces on idealised plastic ball armour model units using strain gauge transducers. It was found that considerable impact forces occur when the breaker front strikes the armour unit model. These forces are directed upward and parallel to the slope. It was concluded that these are the strongest forces for flat breakwater slopes. Juhl and Jensen (1990) carried out a hydraulic model test in a wave flume on an idealised breakwater structure with an armour layer consisting of two rows of horizontal pipes. The wave combination covered both non-breaking and breaking waves with the same water depth for all the tests. The force measurements were carried out in nine pipes located at the outer layer using strain gauge transducers for vertical and horizontal components. One pipe was located at the elevation of the still water, four pipes were above it, and the other four were below it on the slope. The forces were measured on the 20 cm wide middle section of the horizontal pipe. The maximum measured forces on an armour unit, both during rush-up and rush-down, were studied as function of wave period, wave height, and its position on the seaward slope. It was found that in the smallest horizontal force occurred on the pipe located two pipes below the one at the still water level (SWL). A significant force increase was, however, recorded from those pipes to the pipe at the SWL. When a greater wave was applied, a higher run-up was produced, and a larger force recorded on the pipe further up, but only slightly larger than that at the SWL.
structure interactions of rubble-mound breakwaters to obtain a more complete understanding of them, most breakwaters have been built based on empirical design methods. Experience has shown that these methods are inadequate in many cases, especially in deep water, and have resulted in destruction of the structure during severe wave attacks. In some cases, breakwaters were repaired by a continuous supply of rocks until they reached their most stable slope. The concept of berm breakwaters, which refers to a composite slope, was developed and first used in 1978 for the Skopun Breakwater Extension, Faroe Island, as mentioned by Jensen, and Sorensen (1988). Kobayashi and Jacobs (1985) conducted riprap and sand bag model tests to investigate the effect of berm width and slope on the stability of armour units under regular waves; they compared their results to those on uniform slopes. It was found that the critical weight of the armour unit decreases as the berm width increases but it increases as the slope increases. Tørum (1994) measured wave-induced parallel and normal forces on an armour unit on a berm breakwater. The armour unit was placed on the flattened part of the reshaped berm breakwater. Simultaneously with the force measurements water-particle velocities in the vicinity of the armour unit have been measured using a laser Doppler anemometer. Drag coefficient $C_D$ and inertia coefficient $C_M$ were obtained using least-square method. He suggested that the average value of $C_D$ was approximately 0.35, and of $C_M$ approximately 0.2. He also concluded that the parallel force is apparently drag-dominated.
The preceding literature review shows that non-overtopped breakwaters have received a lot of attention. While the studies of wave interaction with non-overtopped breakwaters are still in progress to obtain better understanding and design procedures, the use of overtopped structures are gaining popularity for some purposes. In recent years, submerged rubble mound breakwaters have been used for coastline erosion protection, beach restoration projects, pipeline protection in shallow water, and even for harbour protection. The use of submerged detached breakwaters has become attractive not only because of their low cost, and aesthetics, but also their effectiveness as wave control structures with less environmental impacts. Because of wave energy can pass over the crest, the stability of armour units at the seaward slope of submerged breakwaters is higher than that for non-overtopped types, and therefore, smaller wave forces appear to be acting on these units. However, some papers have noted that the armour units on the downstream slope of a low-crested breakwater may suffer more from heavy overtopping than the armour units on the seaward slope, [Lording and Scott (1971), Raichlen (1972), and Lillevang (1977)]. Raichlen discussed the overtopping characteristics and the complexity of the problem. Walker et al (1975) discussed the many factors influencing the stability of heavily overtopped rubble mound breakwaters. Ahrens (1989) studied the stability of reef breakwaters which refers to a low-crested rubble-mound breakwater without a multilayered cross section. Since the size of rocks used for this type of structure is homogeneous, the porosity is high and the potential for internal energy dissipation is high. It was concluded that a reef breakwater does not fail catastrophically because of the combination of its high energy dissipation and the absence of a core. Van der Meer and
the relative height of the breakwater to the water depth is the most sensitive parameter on the stability of the structure. It was concluded that the increase of the submergence significantly increases the rock stability.

In Japan, Mizutani et al (1991), Rufin Jr., et al (1993 and 1994), conducted studies on wave forces acting on a wide-crown submerged breakwater. The width of the crest was approximately ten times of the height of the breakwater. They found that the most critical location of an armour unit was at the vicinity of the leading crown-edge of the submerged wide-crown breakwater, regardless of the shape and size of the armour unit. The wave force profile was classified into five types i.e. 1) single-peak type; 2) double-peak type; 3) pulse type; 4) zig-zag W-shape type; and 5) zig-zag V-shape type.

Pramono (1994), Pramono and McCorquodale (1994) conducted an experimental study of wave forces on a quarry stone armour unit of a submerged breakwater. The irregular shaped armour model unit was located at the crest and subjected to breaking waves. Both sheltered and exposed placements condition were considered to study their effects on the forces acting on the model unit. The components of the forces, vertical and horizontal, were recorded simultaneously to determine the angle of attack of the force. The vertical component was measured at two points at the two ends of the armour unit model to indicate the simultaneous spinning moment induced by breaking waves. He concluded that: 1) the most critical location is when the armour unit model was placed at the leading edge of the crest; 2) forward force is significantly greater than
hydrodynamic force on the armour unit; 4) variation of exposure within the armour layer increases the hydrodynamic forces about 10%, while complete exposure (above the armour layer) results in about 250% increase. He recommended that simultaneous horizontal and vertical force components must be considered for the better design of submerged breakwaters.

Elkamhawy (1995) conducted 13 physical models comprising 2232 individual tests to study wave interaction with a submerged breakwater. Both internal and external wave induced pressure field were recorded simultaneously. Using the pressure information it was found that the leading edge of the breakwater was the most critical location for the stability of the armour unit. This confirmed the conclusion made by Pramono in his study. Then, using the dimensions of the armour unit model used by Pramono (1994), and the pressure information, the hydrodynamic forces were calculated, and compared with the force measurements by Pramono. The results showed that the magnitude of the measured and the estimated forces were in a good agreement.

In the meantime the studies of hydraulic interactions between submerged breakwaters and wave actions were also carried out by several researchers using a mathematical approach. Berkhoff (1972), derived a two-dimensional combined refraction-diffraction wave equation which is applicable for shallow and deep water waves based on the three-dimensional potential equation, the linearised free-surface condition for harmonic waves, and bottom gradient. The diffraction equation for deep water, the linearised shallow water equation, and the mild slope equation were then derived from the equation based on the relative water depth to the corresponding wave
submerged breakwater on wave transformation, reflection, transmission, and the combination of refraction-diffraction. Rojanakamthorn, et al (1990) developed a mathematical model for the wave transformation over a submerged permeable breakwater on the basis of equations for waves over a porous layer which are derived under the mild-slope assumption. Izumiya (1990) derived a wave equation for predicting reflected and transmitted waves for a permeable submerged breakwater. A similar form of the mild slope equation derived by Berkhoff was included in the derived equation. Kobayashi and Wurjanto (1989) developed a numerical model to predict monochromatic wave transmission and reflection over a submerged breakwater. Losada et al (1992) examined the stability of cubical armour units on a submerged layered breakwater using a hybrid method of experimental, numerical, and empirical results. The stability was studied based on the assumption that the minimum stability occurred at the maximum horizontal velocity, which is at the landward edge or slightly landward of a narrow crest. The effect of submergence on velocity over the crest was also studied, and it was found that velocity decreases as the submergence increases, and thus submergence increases the armour stability as well.

Submerged breakwaters can also be used together with another structure by locating it at the seaward side of the other structure to break the incoming waves, and therefore reducing the wave forces. Cox (1991) introduced a combination of conventional breakwater and submerged-reef breakwater operating in tandem. He placed the submerged breakwater at a distance from the toe of the conventional breakwater, and used the gap between the two structures as the energy dissipation zone. Because the performance of the tandem system is contingent on the performance
widths, porosities, and submergences. It was concluded that submergence has more influence on the wave transmission than the crest width.

2.4 Scale Effects

The selection of scale plays an important role in the design of a physical model from the view point of accuracy and economy.

Dai and Kamel (1969) conducted an experimental study to investigate the effect of model scale on breakwater stability. They concluded that there is a critical Reynolds number (Re) above which scale effect (viscous) is negligible. Using this information, Hudson (1975) recommended

\[ \text{Re} > 3 \times 10^4 \]

In this case the Reynolds number is defined as

\[ R_e = \frac{(gH)^{1/2} r}{\nu} \quad (2.8) \]

where \( g \) is the gravity acceleration, \( H \) is the design wave height at the structure in site, \( r \) is the armour layer thickness, and \( \nu \) is the kinematic viscosity of the fluid. The thickness of the armour layer is determined from the following formula:

\[ r = n k_a \left( \frac{W}{w_r} \right)^{1/3} \quad (2.9) \]

where \( n \) is the number of rocks armour units comprising the cover layer, \( k_a \) is the layer coefficient, \( W \) is the weight of individual armour unit, and \( w_r \) is the unit weight of armour unit.
from 20-40 mm. He concluded that the stability did not depend on the Reynolds number for his model.

Jensen and Klinting (1983) also conducted an experimental study, and they concluded that no scale effect will be present if Re > 6 x 10^3 in the outer structure.

In this study, the size of rocks used is 60-90 mm which is greater than the size used by Thompson, and the breakwater is subjected to strong breaking waves for all the tests. The local velocity measured slightly above the top surface of the model unit ranged from 0.7-1.0 m/s. The size of the cubical model unit is 75 mm. Using the kinematic viscosity of the water at 20°, \( \nu = 1.003 \times 10^{-6} \text{ m}^2/\text{s} \), the Reynolds number in this study ranging approximately from 5 x 10^5 to 7 x 10^5, and therefore the scale effect of viscosity is negligible.
3.1 **Objective**

The main purpose of this experimental work was to measure, simultaneously, forces on 1 or 2 cubical armour units under breaking waves conditions in order to provide information for a more rational armour layer design. The locations of the armour units were chosen at specific points in the centre line of the wave flume along the breakwater surface. The two components of the force acting on each of the 'rocks' were also measured simultaneously. The vertical component of the force was measured at two points on the centre line of the upper surface of the unit parallel to the direction of the incoming waves in order to indicate the simultaneous dynamic spinning moment. Based on these measurements, the magnitude, angle of the force, and the simultaneous spinning moment could be identified. Since, in some tests, forces acting on the cubical armour units were recorded simultaneously, the force phase between the two units could be obtained as well. The phase difference of the forces on the two armour units when they are placed end to end (in tandem) explains how a rock is supported, under wave attack, by the adjacent rock behind it. This also explains why armour units have to be arranged properly in order to get the best interlocking between them. Using the magnitude, angle of attack, spinning moment, and force phase information, the concept of designing an armour layer can be improved i.e. from determining a stable size of armour unit, to determining a stable armour layer as a unit. A high speed data acquisition system (Megadac) was used to record simultaneous force and wave height data.
was estimated that the maximum force on the armour unit model would not exceed 5 lbs, and therefore, Mini-beam load cells with a maximum capacity of 5 lbs. were used.

The recorded force was the combined impact, inertial, and drag forces. Therefore, in order to separate these components, velocity measurements were also conducted using a Pitot tube connected to a differential pressure transducer. The velocity of the flow slightly above the upper surface of the unit was computed using the difference between the total pressure and the static pressure. The force and velocity measurements were conducted simultaneously for submerged and low-crested cases.

3.2 Experimental Set-up

The tests were conducted in an aluminum flume with plexiglass side walls. The flume is 50 ft (15.24 m) long, by 2 ft (0.61 m) wide and 3 ft (0.91 m) high. It consisted of a wave generator, a submerged breakwater model, beach zone, and a return flow pipe (Fig.3.1).

3.2.1 Wave Generator Zone

The wave generator zone consisted of a motor and a wave paddle. The wave generator was controlled by a variable speed motor with a speed controller to operate the movement of the wave paddle. The wave paddle was equipped with a manual adjuster to control its submerged
displacement; this was adjusted before the experimental program and kept constant. The combination of the paddle speed, stroke, and its depth produced various wave periods and heights.

3.2.2 Test Zone

The test zone is a zone in the wave flume including the section where the model breakwater was constructed, and the section where the wave probes were placed as shown in Fig. 3.1.

3.2.2.1 Model Breakwater Set-up

The breakwater was built on a plain concrete pad placed above the flume floor. The submerged breakwater model was located at a distance of 7.16 m away from the wave generator. The submerged breakwater model had the following dimensions: 275 mm crest width, by 275 mm high, and three different slopes of 2:1, 3:1, and 4:1 (horizontal : vertical). The model was constructed of rough natural quarry stones (dolomite) with equivalent diameters in the range of 47.2 mm to 88.3 mm. Since the crest and the front slope of the breakwater are more vulnerable to wave attack, the bigger rocks were selected and placed on the crest and front surface of the structure as armour rocks, whereas the smaller rocks were placed in the inner side of the breakwater. The volume and mass of each rock was measured in order to get the total volume of the rocks in the structure; each rock was numbered for future reference. The size distribution
between the pore volume among the rocks (bulk volume of the structure - volume of rock) and the volume of the structure, which was found to be approximately 51%, (see Table 3.1).

### 3.2.2.2 Wave Probes Placements

Seven wave probes were located along the wave flume as shown in Fig.3.1. Three wave probes were located in the deeper water zone between the concrete base pad and the wave paddle; one wave probe at the seaward toe of the breakwater, and the other three wave probes were placed between the breakwater and the beach section. The first three wave probes were used to measure the incoming and reflected wave heights, whereas the second three wave probes were used to measure the transmitted waves. The wave probe at the seaward toe was used to measure the wave height at the toe of the structure, which was related closely to the hydrodynamic force on the armour model unit.

### 3.2.3 Dissipation Beach Zone

The beach zone was constructed of rough irregular rocks with a slope of 10:1 to maximize wave energy dissipation, and therefore minimize the wave reflection caused by the beach. Induce flow through the dissipation beach was returned to the head of the flume by a 6-inch diameter return flow pipe under the wave flume.
The wave generator location, the test section, and the beach dissipation zones were the fixed part in experimental set-up. The variables part of the experimental set-up in this study were the water depths, the wave periods, and the wave heights. The force acting on the armour units were investigated under the combinations of these variables.

3.3.1 Water Depths

There were eight water depths selected to include both submerged and low-crested breakwaters cases in this study. Those depths were: 20 cm, 22.5 cm, 25 cm, 27.5 cm, 30 cm, 32.5 cm, and 37.5 cm.

3.3.2 Wave Periods

Three wave periods were chosen in this study. The selected wave periods were \( T = 2.2 \) s, \( T = 1.9 \) s, and \( T = 1.5 \) s.

3.3.3 Wave Heights

In order to compare the effect of water depths and wave periods on the force acting on the armour unit model, three wave heights were selected, i.e. \( H = 15-16 \) cm, \( H = 16-17 \) cm, and \( H \)
tests of about 200 combinations were conducted, and 72 combinations were chosen to produce the desired wave heights. These were obtained during the calibration tests and were run with the complete breakwater model, dissipation beach and instrumentation in place and functioning.

3.4 Experimental Equipments and Instrumentation

Two cubical model armour units and two dynamometers were specially designed and carefully prepared for this study. A high speed data acquisition system was used to monitor and control the data collection from the wave, force, and pressure sensors to achieve the objective of this research.

3.4.1 Model Armour Units

A cubical armour unit model was selected based on the assumption that its bluff surface would produce the greatest impact when the wave impacted the model unit, and therefore, it would approximate the worst case for wave force on an angular quarry stone. The concrete cubical armour unit models were made using a mould specially designed for this purpose. Each unit was assembled from two halves of the concrete block as shown in Fig.3.2a. The sides of the cubical armour unit were 75 mm long. Two slots were made on the front and back surfaces of the model to locate the pulleys which were part of the dynamometer (see Fig.3.2a). The slot also functioned as a recess to avoid wave obstruction due to the dynamometer pulley at front of the cubical
obtain simultaneous and separate measurements of longitudinal and uplift wave forces, the armour unit model was instrumented with a 25 mm diameter aluminum tube with a ball joint in its centre as shown in Fig.3.2a. The tube was inserted in a prepared hole in the armour unit and fitted tightly to the unit. A 45 mm long and 10 mm diameter aluminum beam was inserted through the ball joint to carry the longitudinal force (see Fig.3.2b). The ball joint was necessary to enable the model to rotate in any direction, and thus largely separate the moment effects due to the uplift force on the longitudinal force record (see Fig. 3.2b for details). Two slots were prepared in the mould to place two vertical aluminum pins (see Fig.3.2a), where two vertical stainless steel wires connected to two load cells were tightened to the pins. The vertical force and the spinning moment acting on the armour unit were obtained based on measured forces at these two points. Two horizontal lateral wires were used to prevent the model from undesirable sideways oscillations when strong waves attacked the model. These wires were adjustable using two turn buckles. The minimum length of the wire was 30 mm, which was twice as long as the distance between the pulley and the aluminum beam in the tube. Also, the side wires were only slightly tensioned to minimize non-linearity in the loop system of the dynamometer. Two cubical armour unit models were made for this study; the weight of the each unit including the instrumentation was 1.1 lbs.

3.4.2 Dynamometers

The dynamometer specially designed for this study was an improved design based on the work
aluminum beam to locate the three load cells, two vertical aluminum bars and two aluminum plate stands to attach the dynamometer to the flume, and a stainless steel frame (mounted to the horizontal bar) to hang the armour unit model as shown in Fig. 3.3. The horizontal aluminum beam was connected to the vertical bars using two adjustable PVC boxes. This simplified the adjustment of the armour unit model to the desired elevation. The vertical aluminum bars were connected to the plates using two hinges, one hinge at each side. The hinge was arranged in such a way that the dynamometer can be tilted to the desired angle of orientation of the armour unit model. With the improved design, the armour unit model and the dynamometer can be moved and located at any location, at different elevations, and with any angle of orientation < 45°, with very little rearrangement of the surrounding rocks.

For testing armour unit forces on the crest, the model armour unit was positioned horizontally, as shown in Fig.3.2a. Two vertical wires were used to connect load cell-2 and load cell-3 to the armour model as shown in Fig.3.2b, and a looped wire system was used to connect load cell-1 to the armour unit model at the ends of the aluminum beam. Each wire had an adjuster to control the position of the armour unit model. Load cell-2 and load cell-3 were used to record the dynamic uplift forces at the locations where the armour unit was hung, whereas load cell-1 was used to record the dynamic longitudinal force. Figure 3.2b shows the looped wire system. It consisted of load cell-1 and a threaded rod, four pulleys, two wire tension adjusters, and the aluminum beam. The system was arranged as a vertical rectangle. The threaded rod (see Fig.3.2b) and load cell-1 were placed at the centre of the upper side of the system, the four pulleys are
on the centre of the lower side. The threaded rod had a smaller diameter than that of the hole on the head of the load cell, but it was longer than the load cell's head. This was necessary so that the threaded rod could function as an adjuster to centre the armour unit model. The threaded rod was then connected to the wires at its ends to form a looped system. Two nuts, one at each side of the load cell head, were used in order to fasten it to load cell-1 after the model unit had been positioned. Two lower pulleys were located close to the upstream and downstream ends of the model in the provided slots. These pulleys transmitted the longitudinal force along the wire from the lower horizontal wire to the vertical wire in the looped system. The other pulleys transmitted the force from the vertical wire in the looped system to the upper horizontal wire, which then transmitted the force to load cell-1. The tension adjusters were used to manually control the tension of the wire. Before the test was run, the two dynamometers were attached and braced to the wave flume using aluminum bars to assure that the dynamometers were not deflected under waves attack.

For the side slope measurement tests, the dynamometer was rotated by the same angle as the slope without changing or adjusting the tension in any of the wires. Then the model armour unit was placed at the desired location on the side slope. With this setting, the weight or submerged weight of the armour unit was carried by the three load cells in the dynamometer. By balancing the data acquisition system, the reading in all of the sensors were adjusted to zero reading. Forces recorded for this case were as normal and tangential components.
uplift forces on the model to the load cells. The use of the small diameter wire was necessary to minimize its effect on the wave around the model and the wave force on the model. The use of stainless steel was important to prevent corrosion, and to assure that the wire could transmit the force without any reduction or non-linearity due to plasticity. To confirm this, a test was conducted in the Material Engineering Laboratory to get the stress-strain diagram of the material. The result is presented in Fig.3.4. Based on this diagram, the horizontal wave force, which was estimated to have a maximum value of 5 lbs (22.2 N), would not cause the wire to exceed its yield stress. The vertical wave force was estimated to be less than 3 lbs., so that the combined dead weight and the vertical wave force would have an approximate maximum value of 8 lbs. Because the vertical force was carried by two wires, each of the wire would carry 4 lbs. This assured that the yield stress in all the wires would not be reached.

There are two kinds of friction in the pulley, axle friction on the contact surface between the stainless steel bearing and the copper pulley, and belt friction between the stainless steel wire and the groove of the pulley. The axle frictional resistance depends upon the speed of rotation, the clearance between axle and bearing, the normal force on the axle, and the viscosity of the lubricant. The belt friction depends on the difference of tensions in the wire i.e. pretension and tension caused by the hydrodynamic force, the kinetic or static friction coefficient, the mass moment of inertia of the pulley, and the angle of contact (radians) between the wire and the pulley. The two types of frictions which exist in the dynamometer introduce errors in the measurement. The result of the two frictions, however, is difficult to determine because of
or brass on stainless steel, and also under submerged condition. The simple way to improve the accuracy of the measurement is by calibrating the loop system; this is presented below.

3.4.3 Data Acquisition System

The data acquisition system hardware used for this study was the Megadac Data Acquisition System Model No. 3008AC, with TCS3000 software, Optim Electronic Corporation, to record the data, and to export the raw data format to ASCII format for further analysis.

The Megadac is a flexible modular data acquisition and control unit. It embraces numerous options for accurate dynamic testing with analog data from a wide range of active and passive transducers. The system has capacity of 30,000 sampling rate per second and a capacity of 128 channels of input. The Megadac contains a battery-powered memory which defines the type of input and output cards it was shipped with which is known as the Megadac Map. This Megadac Map is the basis for all tests using the Megadac system. The Map is set up when the Megadac system is first used to record the data, and it does not need to be changed as long as the system configuration remains the same.

Fourteen channels were used for this experimental study which consisted of six channels for load cells, seven channels for wave probes, and one channel for pressure transducer. In order to create circuit connections between the Megadac and the experimental sensors, two terminal A/D

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terminals representing one SCV-884 module having 8 channels and 8 ground terminals.

The A/D resolutions of wave height and force are computed based on the Full Scale Volts (FSV) and the number of bits i.e. 16. The resolution of waves is \((9\,\text{"'} \times 25.40\,\text{mm}) / 2^{(16-1)} \approx 0.01\,\text{mm},\) and the resolution of force is \((5 \times 4.4\,\text{N}) / 2^{(16-1)} \approx 0.001\,\text{N}.)\)

Elkamhawy (1995) conducted an experimental study on submerged breakwaters at the Hydraulics Laboratory, University of Windsor using the same system with 10 samples per second per sensor. He recommended that a higher sampling rate is needed for a better result. Based on this recommendation, a rate of 20 samples per second per sensor was used.

The TCS3000 software, Optim Electronic Corporation, is designed to run and control the Megadac system through a host computer communication via the IEEE 488, RS232, RS422, or RS485 cards. The IEEE 488 bus was used in this study. The TCS3000 software also provides several format options by which Raw data collected by the Megadac system can be exported. Those options are ASCII, DIF, SDI, Floating Point, and DSP formats. Since the analysis of the data was conducted using QPRO 4.0 DOS-Version spreadsheet, the data were transported from Raw data format to ASCII format.

Figure 3.5 shows a photograph of the Megadac Data Acquisition System.
There were three types sensors used in this study, seven capacitance wave probes, six minibeam load cells, and a P3061 LVDT type pressure transducer.

3.5.1 Calibration of Wave probes

Capacitance wave gauges were used during the wave calibration and wave loading tests. These probes measure the water level to an accuracy approximately of 2 mm. These probes were calibrated frequently during the test by lowering or raising the probes through a certain known distance in still water. Typically the still water level is located at half of the length of the cable. However, the scale factor did not change significantly during the experiments.

3.5.2 Calibration of Load Cells and Dynamometers

There were six load cells for two dynamometers used for this experimental study. The calibration of the load cells were conducted using static loading conditions. Since the maximum approximated force was 5 lbs., each of the load cells was calibrated using known weights ranging from 0.2-5.0 lbs., with the aid of the Megadac data acquisition system, the TCS3000 software, and a high speed computer, to determine that the load cells were in a good condition. The reading shown in the computer had to be the same as the known weight to indicate that the load cells were in good condition. Typically the error was approximately 4% for loading < 0.2 lbs.,
cells used in this study which was not designed for very small loads such as 0.3 lbs or smaller.

The dynamometers were also calibrated using static loading conditions in place after the dynamometers were set-up in the wave flume and ready to use. However, before the dynamometers were placed in the flume, the tension of the wires in each of the dynamometers needed to be adjusted to achieve the highest accuracy of the force measurements. Since the tension of the wires is sensitive to the reading in the load cell, the tension adjustment was monitored using the Megadac data acquisition system, the TCS3000 software, and a microcomputer. Once the tension in all the wires were adjusted properly, each of the dynamometer was ready to be fitted securely in the wave flume for in place calibration, and for hydrodynamic forces measurements.

The procedure of adjusting the wires is presented below:

1. The model unit was suspended in a horizontal position using only the two vertical wires supported by load cell-2, and load cell-3. The position of the model could be adjusted by using the extension adjuster to assure that the longitudinal axis of the model armour unit was horizontal. This could be shown from the computer if the reading in load-cells 2 and 3 was the same. If the reading was not the same, it indicated that the weight of the armour unit was not evenly carried by the two load cells, and it showed that the armour unit was not horizontal. Therefore, the two vertical wires had to be adjusted until they reached the same values. Up to this point, the position of the armour unit was horizontal,
time, the looped wire was kept slack to avoid its interference to the vertical force reading.

2. The looped wire was tighten by using the two adjusters while unfastening the threaded rod to load cell-1 (load cell-1 was not loaded), and maintaining the reading of load cell-2, and load cell-3 at their initial values. Any change in these values indicated that the looped wire was interfering with the other two load cells; this would show that the aluminum beam was not in one line with the two lower pulleys in the looped system (see section 3.6). If the reading in load cell-2 and load cell-3 increased while the looped wire was tighten, this indicated that the aluminum beam within the armour unit was parallel but higher than the line connecting the two lower pulleys. On the other hand, if the reading in load cells 2 and 3 decreased, it showed that the position of the aluminum beam was parallel but lower to the line connecting the two lower pulleys. The two vertical wires were readjusted to return load cells 2 and 3 to the initial values. It should be noted that the tension of the looped wire had to be adjusted in such a way that it had enough tension to avoid slacking of the looped wire in the dynamic measurement mode. This tension, however, should not be so high that the looped wire would not interfere the measurement of the uplift forces. The tension was adjusted manually using the extension adjusters.

3. Load cell-1 was activated by fastening the threaded rod to the load cell-1 using two nuts on both sides of load cell head. At this stage, the sensitivity of the looped system was tested by applying a weak touch to the model unit to create a small magnitude
possible slacking of the looped wire was also tested by applying forward and backward longitudinal forces alternately, within the capacity of the load cell. The at rest reading from the load cell-1 was then recorded as its initial longitudinal value.

4. The side wires were fasten with a much smaller tension than those in the looped system and in the two vertical wires. It should be noticed that the readings of load cell-1, load cell-2 and load cell-3 had to be kept the same as their initial values. Any change in the readings indicated that there was interference with the uplift and horizontal readings. Therefore, the side wire had to be adjusted until its position was horizontal and perpendicular to the side surfaces of the cubical armour unit, and this was indicated by the readings in load cells 1,2, and 3 remained the same.

Up to this point the wires had been adjusted, the load cells had been activated for measurements, and the readings of all the load cells were at the initial values. The final step before using the dynamometers was to calibrate them in the flume. This calibration was necessary to check for nonlinearity of the horizontal force measurement due to the presence of the side wires and frictional error.

A microprocessor based digital force gauge (forcemeter) DFG-10, manufactured by Omega Engineering, with capacity up to 10 lb-f was used to apply a known force on the model armour unit. The Megadac data acquisition was prepared to compare the reading with that shown in the
braced with the aluminum bars. The bracing was needed to avoid or minimize as much as possible deflection on the stainless steel vertical rods in the frame under wave attack; the bracing arrangement is shown in Fig.3.6. An aluminum plate with a slot was prepared as a horizontal base of the forcement, (see Fig.3.7). The slot was made as a guide for the forcement, so that the gauge could be applied horizontally in the longitudinal direction parallel to the direction of the incoming wave. The gauge was equipped with a straight rod and a circular plate at the end of the rod, (see Fig.3.7). A rigid thin plate was also prepared to cover the front surface of the armour unit, (see Fig.3.7). To accurately locate the circular plate head of the gauge to the cover plate, a circle with the same diameter as the circular plate head was drawn at the centre of the cover plate. Then the static calibration was started by pushing the gauge so the circular plate head touched the cover plate of the armour unit at the prepared circle line. Fig.3.7 shows the forcement in operation. The readings were then recorded and compared with the reading in the Megadac data acquisition system, and calibration curved developed as shown in Figures 3.8 and 3.9. Dynamic response was evaluated by manually disturbing the model armour unit while measuring the frequency of the dynamometer response. It was found that the natural frequency of the system was 10 to 12 Hz.

3.5.3 Calibration of Pressure Transducer

A P3061 LVDT differential pressure transducer was used with a Pitot tube to measure local velocity of water slightly above the upper surface of the cubical armour unit model under
The P3061 LVDT sensor is able to measure low pressure, in gauge or differential modes. In this study, the differential mode was used. The P3061 LVDT type has a pressure sensing element inside the equipment which includes an all-welded Ni-span C capsule that offers low hysteresis and constant scale factor with variable temperature. The deflection of the capsule, when pressurized, is measured by an LVDT displacement sensor whose core is directly coupled to the capsule. The LVDT produces an electric output that is directly proportional to core motion which is proportional to the pressure applied to the capsule. The P3061 LVDT has two ports for both atmospheric and hydraulic pressure ports. For gauge pressure measurement, using two plastic tubes of 0.25 inches in diameter, one port is connected to the atmosphere, and the other port is connected to a point in the water where the gauge pressure is measured. For differential pressure measurement between two points, one port is connected to first point, whereas the other is connected to the second point. In order to avoid non-linearity of the pressure measurements, the P3061 LVDT port transmission spaces inside the sensor had to be freed from air. This is done by putting water to each of the port spaces using the plastic tube fitted tightly to the ports. As the water occupies the space inside the sensor, the air is forced by the water to escape outside through the ports. Any air bubble contains in the sensor or in the plastic tubes, reduces the accuracy of the pressure measurements because the bubble is compressible and affects the differential pressure.

The calibration of the pressure transducer was conducted using the Megadac data acquisition system, and a high speed computer. The typical procedure for the calibration of the pressure
1. Connect the pressure transducer with the Megadac data acquisition system, and then activate the Megadac to monitor the measured pressure.

2. Using plastic tubes, fill up the space inside the sensor with water, and remove any air bubbles in the sensor and in the plastic tubes, until the water in the plastic tubes has the same elevation. Record the reading in shown in the computer monitor as the initial reading.

3. Add more water in one plastic tube so that the water elevation in one tube is higher than the other tube. Measure the difference in elevations between the columns of water within the two tubes.

4. Read the reading of the pressure shown in the monitor, and compare the reading with the measurement mention in point 3 above. The values of both the measurement and the reading must be the same. If those values are not the same, the sensitivity factor of the sensor must be adjusted until the two values are the same.

5. Using the same procedure, compare the two reading for different water elevations.

The next step was to calibrate the pressure transducer after being connected with the Pitot tube
the pressure transducer ports was related to the same still water level, the differential pressure shown in the monitor had to be zero with no flow or waves. Using several different still water level, the readings were checked, and had to be zero before the sensor could be used.

3.6 **Experimental Procedure**

The experimental measurements can be conducted after all of the sensors and the dynamometer are calibrated. Considering the water depth, the wave period, and the wave height are variables, for efficiency purposes, the experimental procedure was arranged as follows:

1. (a) set the water depth, e.g. \(d=20\) cm, (b) set the wave period, e.g. at \(T=2.2\) s, (c) adjust the wave paddle to produce a wave height about 15-16 cm, (d) balance the Megadac data acquisition system to make the sensors reading start from 0, and (e) run the first test. Balancing the Megadac system was needed every time the water depth was changed, and had to be done under still water condition. Every test was made with a total length of time about 30-35 seconds. After one test was made, the data were recorded and stored by the TCS3000 software as 'Original Raw Data' with a file size of about 1.1 mega bytes. In order to assure that the record was good and at the same time to reduce the file size, the data were exported in ASCII file format and reviewed. By exporting from 'Original Raw Data' to ASCII format, the 'Original Raw Data' was changed by the TCS300 software into two files i.e. 'Raw Data' and 'ASCII Data' files whose total files were about
each test was made for every sensors to check the quality of the data in each of those sensors. This permitted the repeat of the test under the same set-up before making any changes for the next test, if the record was not acceptable.

2. The next run was to produce a wave height about 16-17 cm at the same water depth \((d=20 \text{ cm})\) and wave period \((T=2.2 \text{ s})\), and therefore, steps (a) and (b) in point (1) were skipped. The wave paddle was adjusted to produce the desired wave height, and then the second test was run.

3. The same procedure as point (2) above was applied to run the third test at the same water depth \((d=20 \text{ cm})\), wave period \((T=2.2 \text{ s})\), but at different wave paddle position to produce the wave height about 17-18 cm.

4. After conducting the first three measurements, the next procedure was to run another three tests at the same water depth \((d=20 \text{ cm})\), and about the same three wave heights, but at the second wave period \((T=1.9 \text{ s})\). The procedure was the same as explained in points (1) to (3) above.

5. Finally, the last three measurements were conducted at the same water depth \((d=20 \text{ cm})\) at the third wave period \((T=1.5 \text{ s})\) using the same step as mentioned above.

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test. Since there were 9 tests for every water depth, the total number of tests was 72 for 8 water depths, for each set-up of the two dynamometers, and also for each slope arrangement of the breakwater. For brevity, the wave paddle had to be adjusted properly for the entire experimental program. In order to meet this requirements, a scale was attached on the paddle in such a way that zero reading was when the paddle exactly touched the bottom of the flume. The scale reading for each calibration run was recorded for future reference.

3.7 Experimental ranges

The experimental studies were divided into three main categories: (1) preliminary tests, (2) selected special tests, and (3) detailed tests.

The purpose of the preliminary tests were to determine the maximum force envelope along the surface of the breakwater consisting of the crest and the slopes. This information was needed to determine the most critical location of the armour unit and also the most critical condition resulting in the worst stability of the armour unit. The two armour units placements were arranged as shown in Fig.3.10, and the combinations of the placements were E-D, F-C, G-B, and H-A. It was assumed that different slopes would produce similar trends, therefore only one slope (2:1) was used in this stage.

The selected special tests were conducted to study the forces at the critical locations and under
and the SWL was top or bottom flush to the armour units. The orientation of the cubical armour unit was $0^\circ$, where the orientation of the cubical armour unit model was defined as the angle between horizontal line and the bottom or upper surface of the unit. At this stage the number of water depths were reduced from 8 to 5 levels which were 22.5 cm, 25.0 cm, 27.5 cm, 30.0 cm, and 32.5 cm, while three slopes of the breakwater were used which were 2:1, 3:1, and 4:1.

The detailed tests were made to study the effect of unit orientations on the force, and the forces acting on the two armour units when they were placed as close as possible in tandem at the most critical location and under the worst depth and wave conditions. Three angles of orientation of the cubical units were used i.e. $0^\circ$, $15^\circ$, and $26.5^\circ$ (slope 2:1). The range of the tests are summarized in Table 3.2.
4.1 Introduction

There are two different approaches that are applied to the concept of similitude in experimental studies. The first one is often called the classical, formal, or theoretical approach and the other one is the engineering approach.

Langhaar (1957), Ledov (1959) and Yalin (1971) discussed the theoretical background of similitude concept. In this approach, similitude is usually presented as a natural consequence of dimensional analysis, which produces the similitude relationships of Froude, Reynolds, Weber, Mach, Cauchy, and Euler. Le Méhaute (1976) pointed out that the formal approach to similitude in its entirety, based on dimensional analysis, has never been achieved by engineers. There are some limitations to the obtaining complete similitude in scale model technology.

Engineering approach, on the other hand, introduces the concept of the scale model in the experimental work. The scale model method is based on similarity between two phenomena at different scales. The engineering approach is a compromise with the theoretical laws of similitude, produced by dimensional analysis, and practical needs to solve engineering problems. The more pragmatic engineering approach allows the engineer to use the tools of a scale model concept beyond the limits allowed by the
however, depends on how well the engineer is acquainted with the laws of similitude. The better the engineer is acquainted with the laws of similitude, the better he or she will be able to estimate the degree of precision from the model.

According to Le Méhauté (1976) a scale model must satisfy the following conditions: (1) it must be exact in the sense that it must reproduce with sufficient exactness the natural phenomenon under investigation; (2) it must be consistent, i.e., it must always give the same results under the same conditions to the required precision; (3) it must be sensitive, i.e., its sensitivity has to be imposed by the needs of the reproduction of the phenomenon under study; and (4) it must be economical, of reasonable size, and completed within reasonable time.

In general, a scale model does not reproduce all the aspects of the phenomenon under study, but models should represent the aspects of interest in the research. The consequence caused by unsatisfactory reproductions of some phenomenon due to the small model scale is an error referred to as a scale effect.

The model scale is chosen as a compromise between economics on the one hand, and the technical requirements for similitude on the other hand. From the economical viewpoint, the cost of scale model experiments increases approximately as \( \lambda^3 \), where \( \lambda = \frac{L_{\text{proto}}}{L_{\text{model}}} \) is the length ratio of the model. If the model is too small, however, the cost of operation
There are two kinds of open channel scale models for which the law of similitude has been proposed, short or large-scale models, and long or small-scale models. In short or large scale models, viscous friction is not as important as the gravity and inertial force, and therefore, the governing law is Froude similitude. In the long or small scale model, friction has a definite influence on the flow pattern, and therefore, a similitude law that induces head loss must be considered. Since the head loss caused by friction depends on Reynolds number, therefore, in a small scale model, the Reynolds similitude must be considered in addition to the Froude and possible other governing laws of similitude. In the experimental work on free surface flows, however, head loss caused by viscous effects which are a function of Reynolds number can never be in perfect similitude. The error is referred to as the viscous scale effect. There are also surface tension scale effects in very small models. Energy dissipation can also occur in small scale cases where the Froude similitude is used, such processes include breaking waves. In such a case the energy loss is caused by turbulent fluctuations, and is not directly caused by laminar viscous effect. While the laminar viscous effects are linearly related to the velocity, $V$, the turbulent fluctuations are proportional to the square of the velocity, $V^2$, which are dimensionally the same as the inertial forces (dimensionally equal to $\rho V^2$). Therefore, this can be related to the gravity force (which is always present in water wave problems) to produce a form of Froude number squared, $V^2/\gamma L$. Hence, the energy loss in turbulent free surface flow is also dependent on Froude number. It can be said that scale effect in
Reynolds number, because the major energy loss is depends on Froude number.

Experimental studies on the stability of breakwaters are always made based on the law of Froude similitude. This requires a relatively large scale model. Le Méhauté (1976) noted that harbour scale models (3-Dimensional model) for wave agitation studies have been done at scales 1/300, 1/150, and 1/100, to find out that 1/150 is typically the most economical. Typical scales used in engineering practise are: (1) Breakwater/rockfill cofferdam, 1/300 - 1/50; (2) wind-wave penetration into harbours, 1/100 - 1/150 where the water depth must be deeper than 2 cm and the wave period has to be greater than 0.5 sec to avoid viscous and surface tension scale effects; (3) spillways, bottom outlet, water power structures typically 1/50 - 1/100; (4) river, estuary-distorted typically 1/100 vertical and 1/800 horizontal; (5) beaches, shoreline processes-distorted typically 1/100 vertical and 1/300 horizontal; and (6) ship dynamic problems typically 1/100.

4.2 Dimensional Analysis

The governing parameters which influence the longitudinal and uplift wave forces on armour units on a breakwater consist of fluid properties, rock characteristics, geometric and structural conditions (see Fig. 4.1), kinematic and wave characteristics, as follows:
Gravity, \( g \) [LT\(^{-2}\)]

Fluid mass density, \( \rho_t \) [ML\(^{-3}\)]

Viscosity, \( \mu \) [ML\(^{-1}\)T\(^{-1}\)]

Surface tension, \( \sigma \) [MT\(^{-2}\)]

Particle velocities (characteristic velocities), \( U \) [LT\(^{-1}\)], \( V \) [LT\(^{-1}\)]

Rock characteristics:

Surface roughness, \( k \) [L]

Rock mass density, \( \rho_r \) [ML\(^{-3}\)]

Weight, \( W \) [MLT\(^{-2}\)]

Projected areas, \( A_x \) [L\(^2\)], \( A_y \) [L\(^2\)]

Angle of orientation, \( \beta \)

Shape factor, SF

Geometric and structural conditions:

Porosity, \( n_p \)

Slope of breakwater, \( \Theta \)

Height of breakwater, \( h \) [L]

Wave conditions:

Wave period, \( T \) [T]
Water depth, \( d \) [L]

Time, \( t \) [T]

The dynamic longitudinal and uplift forces caused by waves can be expressed in these equations:

\[
F_x = f_{F_x}(F_x g, \rho_f \mu, \sigma, U, k, \rho, W, A_x, \beta, SF, n, \theta, h, T, H, d, t) = 0 \quad (4.1)
\]

\[
F_y = f_{F_y}(F_y g, \rho_f \mu, \sigma, V, k, \rho, W, A_y, \beta, SF, n, \theta, h, T, H, d, t) = 0 \quad (4.2)
\]

By selecting \( \rho_f, U \) (for \( F_x \)) or \( V \) (for \( F_y \)), and \( H \) as the repeating variables, the dimensionless group for \( F_x \) and \( F_y \) are as follows:

\[
f_{F_x}\left( \frac{F_x g H}{\rho_f U^2 H^2}, \frac{\mu}{U^2}, \frac{k}{\rho g H \rho_f U^2 H}, \frac{A_x}{H^2}, \frac{\beta}{\rho_f U^2 H^2}, \frac{SF}{H}, \frac{n}{\rho_f U^2 H^2}, \frac{\theta}{H}, \frac{h}{H}, \frac{d}{H}, \frac{U}{H}, \frac{U}{H} \right) = 0 \quad (4.3)
\]

\[
f_{F_y}\left( \frac{F_y g H}{\rho_f V^2 H^2}, \frac{\mu}{V^2}, \frac{k}{\rho g H \rho_f V^2 H}, \frac{A_x}{H^2}, \frac{\beta}{\rho_f V^2 H^2}, \frac{SF}{H}, \frac{n}{\rho_f V^2 H^2}, \frac{\theta}{H}, \frac{h}{H}, \frac{d}{H}, \frac{V}{H}, \frac{V}{H} \right) = 0 \quad (4.4)
\]

The second term From Eqs 4.3 and 4.4 is the inverse square of the Froude number, the third term is the inverse of the Reynolds number, the fourth term is the inverse of the Weber number, and the seventh term is the Newton Inertial Force number which is related to the armour unit Reynolds number. Using the common form of dimensionless numbers, the magnitudes of longitudinal and vertical forces depend on the Froude number, Reynolds number, and other dimensionless groups, i.e.
\[ F_x = \rho_f U^2 H^2 \phi \left( \frac{V}{\sqrt{g d}} \mu \sigma H \frac{\rho_f}{\rho} \right) \]  
\[ F_y = \rho_f V^2 H^2 \phi \left( \frac{V}{\sqrt{g d}} \mu \sigma H \frac{\rho_f}{\rho} \right) \]

where \( V_r = (U^2 + V^2) \), and \( D_e \) is the equivalent diameter of the armour unit. In all of the tests, the height of the breakwater, and the weight of the armour unit were the same. The fluid density, viscosity, and porosity did not vary significantly in the experiments of this study. Also, surface tension in this study is not important. Therefore, \( n, W, \rho, \rho_0, \mu, \sigma, k, h, \) and \( g \) were constant for this study, whereas the other properties \( (d, T, H, \theta, \beta) \) were the controlled variables.

4.3 Dimensionless Form of the Force Components

From Eqs 4.5 and 4.6 the dimensionless form of the longitudinal and uplift forces were respectively found to be:

\[ \frac{F_x}{\rho_f U^2 H^2} \]

\[ \frac{F_y}{\rho_f V^2 H^2} \]
of $V_T^2$ equals the dimension of $gH$. Furthermore, from the eighth dimensionless group the dimension of $H^2$ are the same as $A_x^\gamma$. Substituting $gH$ for $V_T^2$, and $A_x$ for $H^2$ in Eqs 4.7 and 4.8 respectively, yields

\[
\frac{F_x}{(\rho_f g)HA_x} \quad (4.9)
\]

\[
\frac{F_y}{(\rho_f g)HA_y} \quad (4.10)
\]

The expression of $A_x$ and $A_y$ can be replaced with $D_{ex}^2$ and $D_{ey}^2$. However, since the projected area for both $A_x$ and $A_y$ are the same, then $D_{ex}^2 = D_{ey}^2 = D_e^2$, where $D_{ex}$ and $D_{ey}$ are respectively the equivalent diameter of the projected area of the armour unit to a vertical plane perpendicular to the incoming wave and to a horizontal plane. Letting $\gamma = \rho_f g$, the dimensionless forms of longitudinal and uplift forces can be expressed as follows:

\[
F_{x*} = \frac{F_x}{\gamma D_e^2 H} \quad (4.11)
\]

\[
F_{y*} = \frac{F_y}{\gamma D_e^2 H} \quad (4.12)
\]

where $F_x$ and $F_y$ are longitudinal and uplift forces, $\gamma$ is the specific weight of the fluid, $H$ is the incoming wave height, and $D_e$ is the equivalent diameter of the model unit.
longitudinal-vertical, and horizontal planes are respectively $A_x$ and $A_y$, and they are not equal. Each of these projected areas has an equivalent diameter, $D_{ex}$ and $D_{ey}$ respectively. In this case the dimensionless forms for longitudinal and vertical forces are similar to Eqs. 4.11 and 4.12, except that $D_e$ is replaced by $D_{ex}$ for the longitudinal force, and $D_{ey}$ for the vertical force as shown in the following formulas:

\[
F_{x*} = \frac{F_x}{\gamma D_{ex}^2 H} \quad (4.13)
\]

\[
F_{y*} = \frac{F_y}{\gamma D_{ey}^2 H} \quad (4.14)
\]

The longitudinal force is defined as positive when its direction is the same as the incoming wave and it is negative for the opposite direction, whereas the vertical forces are positive upward and negative downward.

4.3 Dimensionless Form of Total Force

Total force is defined as the vector summation of simultaneous longitudinal and uplift forces. Its direction is defined by the ratio of the uplift force to the longitudinal force. The magnitude of the total force in dimensional form is given in Eq 4.15, and its direction is as in Eq 4.16:
\[ F_T = \sqrt{F_x^2 + F_y^2} \quad (4.15) \]

\[ \tan \alpha = \frac{F_y}{F_x} \quad (4.16) \]

In the dimensionless form, the magnitude of the total force can be expressed in the following formula:

\[ F_{T*} = \frac{1}{\gamma D_e^2 H} \sqrt{(F_x^2 + F_y^2)} \quad (4.17) \]

For irregularly shaped armour unit, the magnitude of the dimensionless total force can be computed by substituting Eqs. 4.13 and 4.14 into Eq 4.17 yielding the following:

\[ F_{T*} = \frac{1}{D_e^2} \sqrt{(F_x, D_x^2)^2 + (F_y, D_y^2)^2} \quad (4.18) \]

4.4 Presentation of Data in Dimensionless Form

The hydrodynamic force acting on the cubical unit placed at the selected locations along the surface of the breakwater consists of the normal and tangential components. For simplicity, however, the hydrodynamic force can also be presented in the horizontal and vertical components. Using this approach, the dimensionless force components can be expressed using Eqs. 4.11 and 4.12.

The dimensionless forms of the maximum horizontal and vertical forces components were used to show the maximum force field along the breakwater surface.
was plotted with the ratio of the height of the breakwater and the water depth (h/d). This ratio is the inverse of the specific submergence defined as the ratio of still water depth and the height of the breakwater (d/h). Eight values of (h/d) were used i.e 0.73, 0.79, 0.85, 0.92, 1.0, 1.1, 1.22, and 1.38.
5.1 Introduction

The hydrodynamic forces, and the spinning moment induced by waves on armour units of breakwaters are the predominant factors in the determination of the stability of armour units (except for seismic sea waves). The magnitude of these forces is dependent to the wave characteristics including the height, the length, and the period of the wave, and sensitive to the still water depth. Consequently, the determination of wave forces on the armour units must be established based on a proper selection of still water level at the site of the structure.

Jackson (1968) noted that the choice of design wave conditions for structural stability should consider whether the structure is subjected to the attack of nonbreaking, breaking, or broken waves. Broken waves occur when the waves break prematurely (relatively far) before reaching the structure, resulting in the smaller waves with the smaller energy attacking the structure. Accordingly, broken waves are weaker than nonbreaking waves for the same incident wave. Among the three conditions, breaking waves have the worst effect to the stability of the structure compared to the nonbreaking and the broken waves. The forces induced by breaking waves are caused by the energy released during breaking, and the impact caused by the fluid moving with a speed greater than the celerity of the wave. Stokes (1880) predicted that breaking occurs when the particles of water move
the breaking point is a point where foam first appears on the crest, where the front face of the wave first becomes vertical, or where the wave crest first begins to curl over the face of the wave.

The determination of the maximum hydrodynamic forces, therefore, requires a proper selection of the design still water depth and the design breaking wave height at the site where the structure is built.

5.2 Method for Determining the Design Breaking Wave Height and Depth

The SPM (1984) presented information from Hedar (1965) who suggested that the breaking process extends over a distance equal to one half of the shallow-water wave length based on the depth at the seaward position. Galvin (1968, 1969) proposed a relationship between the distance travelled by a plunging breaker, \( x_p \), and the breaking wave height, \( H_b \), depends on the nearshore slope, \( m \), as follows:

\[
x_p = \tau_p \, H_b = (4.0 - 9.25 \, m)H_b
\]  

(5.1)

where \( \tau_p = (4.0 - 9.25 \, m) \) is the dimensionless plunge distance. Galvin (1969) discussed that the starting point of the breaking may occur in a region within the range of maximum breaker depth, \( d_{b_{\text{max}}} \), and minimum breaker depth, \( d_{b_{\text{min}}} \), as shown in Fig.5.1. The ratio of the maximum breaker depth to the breaker height, \( H_b \), is called the upper limit, \( \alpha \), which is independent to the nearshore slope \( m \), whereas the ratio between the minimum
to the nearshore slope $m$. The upper and the lower limits are expressed respectively as follows:

$$\alpha = \left(\frac{d_b}{H_b}\right)_{\text{max}}$$  \hspace{1cm} (5.2)

$$\beta = \left(\frac{d_b}{H_b}\right)_{\text{min}}$$  \hspace{1cm} (5.3)

A series of curves showing the relationship between $(d_b / H_b)$ versus $(H_b / gT^2)$ for various nearshore slopes $m$, the upper breaker limit $\alpha$, and the lower breaker limit $\beta$, was provided by Weggel (1972) as shown in Fig.5.2.

It has been noted above that the determination of wave forces on the armour units must be established based on a proper design still water level at the site of the structure. Also, considering that breaking wave condition offers the worst possible case (except for seismic sea waves situation), it is appropriate to determine the maximum breaking wave height to which the structure might be subjected. The lower limit $\beta$, therefore, must be used to obtain plunging on the structure (see Fig.5.1). The design of breaking wave height, consequently, depends on the water depth at the toe of the structure, the bed slope on which the structure is built, the steepness of the incoming wave, and the distance travelled by the waves during breaking, and the embankment slope. The design breaking wave height can be expressed as follows:

65
\[ H_b = \frac{d_s}{\beta - m \tau_p} \]  \hspace{1cm} (5.4)

where \( d_s \) is the still water depth at the toe, \( \beta \) is the breaker lower limit, \( m \) is the nearshore slope, and \( \tau_p = (4.0 - 9.25 \, m) \) is the dimensionless plunge distance. It must be noted, however, that the value of the lower limit \( \beta \) in Eq.5.4 cannot be directly known until the breaking wave height \( H_b \) is evaluated. In order to aid the determination of \( H_b \), Fig.5.3 has been provided to solve the value of breaking height \( H_b \) if the still water depth (design depth) at the toe of the structure \( d_s \), and the incoming wave period \( T \) are known. Figure 5.3 shows the relationship between the dimensionless design breaking wave height \( (H_b/d_s) \) versus relative depth at the structure \( (d_s/gT^2) \). The curves shown in Fig.5.3 were derived from Eqs 5.1 and 5.4 using the lower limit \( \beta \) values in from Fig.5.2. Thus, knowing the magnitude of \( H_b \), the lower limit \( \beta \) can be computed using Eq.5.4, and finally, the breaking depth can be evaluated using Eq.5.3.

Figure 5.3 can be best used for nearshore slope \( 0 \leq m \leq 0.1 \). In this experimental study, the slope of the breakwater is 1 (vertical) : 2 (horizontal), or \( m = 0.5 \gg 0.1 \). The range of the relative depth in this study is \( 0.0042 \leq (d_s/gT^2) \leq 0.0170 \). If the slope \( m = 0.1 \), the corresponding dimensionless design breaking wave height based on Fig. 5.3 is nearly \( 1.75 \geq (H_b/d_s) \geq 1.25 \), and the breaker heights are approximately \( H_b = 35 \, cm \) (at \( d_s = 20 \, cm \)) and \( H_b = 47.875 \, cm \) (at \( d_s = 37.5 \, cm \)). If the slope \( m = 0.5 \) (such as in this study) \( \gg 0.1 \), extrapolation must be used in order to approximate \( (H_b/d_s) \) resulting in \( (H_b/d_s) \gg 1.75 \) or

66
Fig. 5.3). This approximation is over estimated, if applied to this study, and therefore such extrapolation cannot be used. Further study at $m = 0.5$ must be made to obtain data points of $(H_b/d_p)$ at the corresponding $(d_p/gT^2)$. However, Galvin's predicted high magnitude breaker height as shown in Fig. 5.3 is possible since Galvin's structure is vertical non-overtopped impervious wall that produces high reflection. In this study the structure is trapezoidal overtopped pervious breakwater with very low reflection.

5.3 Breaking Wave Height and Depth in the Experimental Studies

The location of the breaking point of the incoming waves varies depending on many factors including the still water depth, the wave characteristics (wave height, length, and period), type of structures e.g. pervious or impervious, and the bed at structure slopes. In this experimental study the still water depths were chosen not only to produce breaking waves that plunge on the structure, but to determine the effect of submergence on the hydrodynamic forces on the armour units. Three wave periods and three wave heights for each wave period were produced by the wave generator for this research. A concrete pad was built with slope of 10 (horizontal) : 1 (vertical) to simulate the nearshore slope. However, it is impossible and also impractical, for all of the tests, to produce breaking waves that plunge at a fixed position such as at the toe of the structure in order to simulate breaking condition as discussed in section 5.2. Therefore a compromise was made in this study by producing incoming breaking waves that plunge on the structure.
wave heights.

Rufin Jr., et al (1993 and 1994), Pramono (1994), Pramono and McCorquodale (1994) conducted experimental studies on wave forces on submerged breakwaters, and they found that the leading edge is the most critical location for the armour units. The breaker design discussed in section 5.2 predicts the plunging at the toe of the structure. Therefore, the breaker design used in this study produces worse conditions than the breaker design mentioned in section 5.2 because it gives the greater hydrodynamic forces on the armour units at the leading edge.

The maximum hydrodynamic forces are best related to the breaking wave height because the energy released during breaking is used to accelerate the fluid to produce plunging, surging, or collapsing waves that attack the armour units. Laboratory observations, however, indicated that the location of the breaking point was not always at the same place. As a result, it is difficult to measure the breaking heights in order to relate the breaking heights to the corresponding wave forces. The easier approach was to measure the incoming wave height at the toe of the breakwater instead of measuring the breaking heights. Since the breaking heights can be related to the wave heights at the toe, it is sufficient to establish a relationship between the wave heights at the structure toe and the corresponding wave forces. This approach was also used by Hudson (1958) as shown in Eq.2.1 for determining the stable size of armour units for conventional breakwaters.
in this study. Galvin's prediction of \((H_h d)\) for this study is \(1.75 \geq (H_h d) \geq 1.25\) for the range of \(0.0042 \leq (d_g T^2) \leq 0.0170\) in this study. The ratio of \((H_{tot} d)\) in this study varies depending on the wave characteristics and submergence of the structure. Typically \((H_{tot} d) = 0.60 - 0.67\) for \((h d) = 1.22\), \((H_{tot} d) = 0.45 - 0.55\) for \((h d) = 1.00\), \((H_{tot} d) = 0.38 - 0.50\) for \((h d) = 0.85\).

5.4 Wave Theory Selection - Linear Wave Theory

The actual water wave motions in general are very complex phenomena because of their apparent random behaviour, 3-D characteristics, and non-linearities. Therefore, it is very difficult to exactly describe the wave motions mathematically. Simple models must be used in order to apply the theories of water particle motions and other phenomena to the waves. Hence, the following general assumptions are made: the water is to be considered non-viscous and incompressible; the water depth is constant, or changes very slowly; the wave is part of a train that has a constant period and height (mono-chromatic); the wave height is considered to be very small compared to the wave length; the wave motion is to be considered irrotational: the wave form is sinusoidal with crest and trough of equal amplitude from the mean water level; and the wave is long-crested and is propagating in a straight line.

Theory which incorporates the first term of the series and neglect the higher orders is
assumptions are made in the Airy wave theory:

The flow is irrotational.

The fluid is incompressible and non-stratified.

The fluid has a uniform finite depth.

The bottom is smooth and impermeable.

The waves are non-breaking type, and have a permanent form i.e. length, height, and period.

The main concerns of the coastal engineers with respect to wave theories are: wave energy, water surface profiles, the motions of water particles including velocities, accelerations, and amplitudes, and pressure fluctuations.

Using this theory, a simple solution for wave energy, water surface profile, horizontal and vertical particles velocities, particle acceleration, pressure variation induced by wave motion can be provided. The formulas for these are given in the SPM (1984)

Le Méhauté (1976) and Dean (1970) presented the range and limits of applicability of various wave theories as shown in Figures 5.4 and 5.5.

When the wave height becomes relatively large, it is impossible to neglect the non-linear terms which are ignored in the small amplitude wave theory. In such cases a finite
solved with sufficient accuracy by the linear wave theory.

The incident, transmitted, and reflected waves in this study were computed using the linear wave theory.

5.5 Waves Theory Selection - Finite Amplitude Wave Theory

In this section, several water wave theories will be discussed including trochoidal wave theory, Stokes wave theory, cnoidal wave theory, and solitary wave theory.

5.5.1 Trochoidal Wave Theory

Among finite amplitude wave theories, the trochoidal wave theory is the oldest. The trochoidal wave theory was originally derived by Gerstner (1802) to provide a solution for deep water waves or approximate solution for shallow water waves of rotational motion. The surface form of this theory is a trochoid, referring to the path of a point on a disc whose circumference rotates along a straight line. The paths or orbits of water particles are circles. The diameters of these orbits are maximum at the surface, and decrease exponentially with increasing depth below the surface. This theory, however, is not recommended by the SPM (1984) for application, since the water particle motion predicted is not as observed in nature.
Stokes (1847), in contrast to Gerstner's trochoidal wave theory, developed an asymptotic solution for deep water irrotational waves with a permanent wave profile. Stokes' solution is more satisfactory than the trochoidal wave theory.

The Stokes' first order theory provides solution which is the same as the Linear (Airy) wave theory. The form of the wave is symmetrical about the s.w.l., and a vertical line through crest. The water particles move in closed orbits.

The Stokes' second order approximation predict that the form of the wave is unsymmetrical about the s.w.l., but still symmetrical about a vertical through crest. The formulas for wave celerity and the wave length are identical to the expression in the linear wave theory. The water particle orbits are open (shifted) ellipses.

The Stokes' third order approximation shows that the velocity of propagation depends on the wave height. The correction factors for both the wave celerity and the wave length are the same. The water particles orbits are open ellipses.

Shallow water waves have been analyzed by many researchers using higher order theory. Among those are Skjelbreia (1959), and Skjelbreia and Hendrickson (1962) who prepared wave tables in order to reduce the possibility of error using equations.
water particle velocity component $u$ at the crest equal or larger than the wave celerity $c$. At this point waves steepness reaches the maximum value, and the wave crest angle is $120^\circ$. At this point the waves will break to dissipate a part of their energy. Mitchell (1893) calculated using the criterion that at breaking $c = u_{crest}$, and showed that the maximum steepness of deep water waves is given by:

$$\left(\frac{H}{L}\right)_{\text{max}} = 0.142 \approx \frac{1}{7} \quad (5.5)$$

McCowan (1894) showed that the wave crest critical angle in shallow water is also $120^\circ$. Miche (1944), gives the limiting steepness for waves travelling in transitional and shallow water:

$$\left(\frac{H}{L}\right)_{\text{max}} = 0.142 \tanh \frac{2\pi h}{L} \quad (5.6)$$

Hamada (1951), and Danel (1952) conducted independent laboratory measurements and showed that Eq.(5.12) is in good agreement with an envelope curve of laboratory observations.

5.5.3 Cnoidal Wave Theory

Long waves with finite amplitude and permanent form propagating in shallow water are often best described by cnoidal wave theory. This type of waves was first recognized by
In the asymptotic expansion of Stokes wave theory, the wave steepness \((H/L)\) is assumed to be small compared to unity, while for shallow water waves, the relative water depth \((d/L)\) has an important influence on wave motion. Therefore, the quantities of \((H/L)\) and \((d/L)\) should be considered in the finite amplitude wave theory. In order to represent the relative magnitude between the above two values, a parameter called Ursell's or Stokes parameter is introduced:

\[
U = \frac{H L^2}{d^3} = \frac{(H/d)^3}{(H/L)^2}
\]  
(5.7)

The region \(U \ll 1\) corresponds to that where the Stokes wave theory is applicable. In the opposite extreme, no permanent wave form can exist in the region \(U \gg 1\). A wave in this region deforms as it propagates. Cnoidal waves exist in the intermediate region where \(U\approx 1\). The term *cnoidal* derives from the wave profile which is given by the Jacobian elliptical cosine function expressed by \(cn\).

From Eq.(5.13) it can be seen as wave length \(L\) becomes long and approaches infinity, the Ursell parameter \(U \gg 1\), cnoidal wave theory reduces to the solitary wave theory which will be described in section 5.5.4. Also, when \((H/d)\) becomes small (infinitesimal wave height), the wave form approaches the sinusoidal profile predicted by the linear (Airy) wave theory.
celerity are given respectively by:

\[ \eta = H \, cn^2 \left[ 2K(k) \left( \frac{x}{L} - \frac{t}{T} \right), k \right] = H \, cn^2 ( ) \quad (5.8) \]

\[ L = k \, K(k) \sqrt{\frac{16 \, d_t^3}{3H}} \quad (5.9) \]

\[ C = \sqrt{gd_t} \left[ 1 + \frac{H}{d_t \, k^2} \left( \frac{1}{2} - \frac{E(k)}{K(k)} \right) \right] \quad (5.10) \]

and the second-order approximations for the surface elevation, the wave length, and the wave celerity are:

\[ \eta = H \, cn^2 ( ) - \left[ \frac{3}{4} \, \frac{H^2}{d_t} \, cn^2 ( ) \right] \left[ 1 - cn^2 ( ) \right] \quad (5.11) \]

\[ L = k \, K(k) \sqrt{\frac{16 \, d_t^3}{3H}} \left[ 1 + \frac{H}{h_t} \, \frac{7k^2 - 2}{8k^2} \right] \quad (5.12) \]

\[ C = \sqrt{gd_t} \left[ 1 + \frac{H}{d_t \, k^2} \left( \frac{1}{2} - \frac{E(k)}{K(k)} \right) \right] + \]
\[
\sqrt{gd_r} \left[ \left( \frac{H}{h_r} \right)^2 \frac{1}{k^4} \frac{E(k)}{K(k)} \left( \frac{E(k)}{K(k)} - \frac{k^4 + 16k^2 + 31}{40} \right) \right]
\]

(5.13)

where

\[ \eta \] = water surface elevation, \( \eta_{max} = H \)

\[ cn \] = elliptic cosine function

\[ k \] = modulus of elliptic integral, where \( 0 \leq k \leq 1 \)

\( (\ ) \) = argument of \( cn^2 \)

\[ d_i \] = vertical distance from bottom to the wave trough

\[ K(k) \] = the first complete elliptic integral, where

\[
K(k) = \int_0^{\pi/2} \frac{dx}{\sqrt{1 - k \sin^2 x}}
\]

(5.14)

\[ E(k) \] = the second complete elliptic integral, where

\[
E(k) = \int_0^{\pi/2} \sqrt{1 - k \sin^2 x} \ dx
\]

(5.15)

Shoaling water wave computations are ideally, best be performed using cnoidal wave theory because this theory best describes wave motion in relatively shallow or shoaling water. Simple and completely satisfactory numerical solutions for cnoidal wave theory are not available. However, the SPM provides charts showing the principal properties of cnoidal waves for given values of modulus of elliptic integral \( k \).

Although this theory is best describing shallow or shoaling wave motions, cnoidal wave
5.5.4 Solitary Wave Theory

A solitary wave is theoretically a single long wave. It is a progressive wave, whose motion is not affected by the preceding or following crests such as the effect of swell running up a gradually sloping beach with the distant crests, or a tsunami generating by seismic disturbance. In the solitary wave theory, the surface profile is completely above the s.w.l., therefore it is not periodic and does not have definite wave length. This type of wave was first observed by Russel (1838), and reported by him in 1944. Since then many researchers have treated the solitary wave theoretically, and they have obtained important results on solitary wave characteristics.

The first theoretical derivation of the solitary wave in shallow water was made by Boussinesq (1872), then followed by Rayleigh (1876), McCowan (1891), Keulegan and Patterson (1940), Keulegan (1948), Munk (1949), and Iwasa (1955).

The first order approximation for water surface elevation and the wave celerity for solitary wave theory are given by:

\[ \eta = H \text{sech}^2 \left[ \sqrt{\frac{4H}{3d^3}} (x - Ct) \right] \]  

(5.16)
where the origin of \( x \) is at the wave crest. The total energy per unit crest width \( (li) \) of a solitary wave is expressed:

\[
E = \rho g d_t^3 \left( \frac{4H}{3d_t} \right)^{1.5}
\]

(5.18)

and the total volume of water per unit crest width above the s.w.l is:

\[
V = \int_{-\infty}^{\infty} \eta \, dx = 4d_t^2 \sqrt{\frac{H}{3d_t}}
\]

(5.19)

From Eq.(5.24) take the value of \( H d_t = 0.5 \), the wave energy contained in the region of \( x = \pm 1.6 \, d_t \) and \( \pm 1.6 \, d_t \) are 90% and 98% from the total energy respectively. Also, from Eq.(5.25) using the same value \( H d_t = 0.5 \), the volume of water in the region of \( x = \pm 2.4 \, d_t \) and \( \pm 3.8 \, d_t \) are 90% and 98% from the total volume above the s.w.l. These explain that although solitary waves have infinite wave length, the energy and the volume (or mass) above the S.W.L. are concentrated in the limited region close to the wave crest.

Unlike cnoidal wave theory, the solitary wave theory is simple, and easy to use because it reduces to functions that can be evaluated without special tables.

A solitary wave becomes unstable and breaks as it moves into shcaling water. McCowan (1891) assumed that a solitary wave starts to break when the water particle velocity at the
\[
\left( \frac{H}{d} \right)_{\text{max}} = 0.78
\]

5.6 Wave Theory Selection for the Investigation

The use of a proper wave theory for solving wave problems is determined by the wave steepness and the relative depth. As the wave steepness becomes greater, the non-linearity terms in the equation becomes greater. There is a maximum value of steepness which a wave can attain without breaking. Many research have been conducted for determining the maximum wave steepness, and all correct solutions are approximately \(1/7\) for deep water waves. As the relative water depth \((d/L)\) decreases, the wave steepness also decreases.

The region where a wave theory is valid should be identified. Since investigators differ on the limiting conditions for several theories, some overlap must be permitted in defining the regions.

Le Meauté (1969) and Dean (1970) presented Figures 5.4 and 5.5 respectively to illustrate the approximate limits of the validity for several wave theories. In this study, the waves are in the range of Stokes' third order as shown in Fig.5.4 or Stokes' Fifth in Fig. 5.5. Both classification diagrams shows that Stream Function V can be used for the
Reasonable approximation to regular waves (especially in deep water waves) is given by the first order theory, which approximates the wave form as sinusoidal. This linear approximation forms the basis for much of the wave force work in the ocean engineering.

There are two advantages to using the first order approximation approach. The first and the most important one is the fact that the first order theory is linear. This assumption permits to superposition of any number of waves, with any height, length, and direction of propagation, to represent a realistic sea condition.

The second advantage is that the mathematical formulation for the linearized problems and their solutions are relatively simple and easier to understand compared to the non-linear approaches. Consequently, many important aspects of the linearized theory has been extensively developed and put into common use.

Summary:
The waves offshore of the structure in this study can be described by cnoidal wave theory, Stokes third order, Stokes fourth or fifth order, and Stream Function V.

5.7. Wave Transmission, Reflection, and Dissipation
The purpose of the submerged and low-crested breakwaters is mainly to reduce the wave energy by breaking the incoming wave height at the structures. The transmitted wave energy, which depends on the transmitted wave, must be limited to an amount that is acceptable to the area behind the breakwaters. Consequently, the design of transmitted waves must be considered in the design of submerged and low-crested breakwaters.

One of the most important features that increase the interest of using low-crested and submerged breakwaters for coastal erosion protection is the ability of these structures to dissipate wave energy. The amount of energy that can be dissipated depends on the characteristics of the waves and the degree of submergence of the structures. Energy dissipation for longer waves is generally smaller than the energy dissipation for the shorter waves. It is also obvious that the higher the crest elevation causes the higher energy dissipation. The relation between the transmitted and the dissipated wave energy is inherently antagonistic. The more the wave is dissipated, the less wave energy will be transmitted over the breakwater, and vice versa. As a result, the determination of the dissipated wave energy is already included in the design of the transmitted wave energy, or wave height.

Wave reflection also plays an important role in the design of coastal structures, especially for structures related to harbour development. In such case, a build up of energy may
fluctuation of the water surface can cause excessive motion of the moored floating objects in the harbour resulting in great strain in the mooring lines. However, the reflected waves caused by submerged and low-crested breakwaters, are relatively small compared to the reflected waves caused by conventional breakwaters or other structures such as vertical walls and seawalls.

5.7.2 Transmitted Wave

Wave transmission for permeable breakwaters consists of transmission by overtopping and transmission through the breakwaters. Generally, the wave transmission is expressed as a coefficient showing the ratio between the transmitted wave height and the incoming wave height as given by:

\[ K_t = \frac{H_t}{H_i} \]  \hspace{1cm} (5.21)

where \( H_t \) and \( H_i \) are the transmitted and incoming waves respectively. The coefficient of wave transmission for permeable submerged and low crested breakwaters is given by:

\[ K_t = \sqrt{K_{ro}^2 + K_n^2} \]  \hspace{1cm} (5.22)

where \( K_{ro} \) and \( K_n \) are the coefficients for wave transmission over and through the breakwater respectively. The wave transmission through the structure is complex, and it depends on the characteristics of the wave and the structure. Astronomical tides with normally very low wave steepness, may transmit totally through the breakwater while
breakwaters also totally caused by energy transmission through the structure. For low-crested breakwaters, and especially submerged breakwaters, however, wave transmission is mainly caused by overtopping. In this study, the two components of wave transmission will not be separated.

Davidson (1969) conducted an experimental study on monochromatic wave transmission of breakwaters with trobar armour units. He found that $K_r$, initially declines, then rapidly increases as transmission by overtopping begins (see Fig.5.6). When wave transmission by overtopping occurs, the transmission coefficient increases as the incident wave height increases, all other factors being fixed.

5.7.2.1 **Effect of Crest Width on the Transmitted Wave**

Van der Meer noted that wave transmission is more sensitive to submergence than to the crest width. From the economical point of view, the increase of crest width will significantly increase the use of material for the construction of the breakwater. Therefore, in this experimental study the effect of crest width on the transmitted wave was not emphasized. Only one crest width was used throughout the tests where the crest width was chosen the same as the height of the structure.

Many studies have been conducted to find the effect of crest width of submerged
relationship between $K_i$ and $(b/h)$ for both permeable and impermeable low crested breakwaters with $(h/d) = 1.033, 1.133, \text{ and } 1.33$, with the range $0.875 \leq b/h \leq 3.25$, and with various wave steepness (see Figures 5.7 to 5.10).

Seelig (1980) modified the work previously conducted by Cross and Sollitt (1971) to show that the transmission by overtopping can be estimated from:

$$K_{to} = C \left(1.0 - \frac{F}{R}\right) \tag{5.23}$$

where

$$C = 0.51 - 0.11 \left(\frac{B}{h}\right) ; \quad 0 < \frac{B}{h} < 3.2 \tag{5.24}$$

$$\frac{F}{R} = \frac{(F / H)}{(R / H)} < 1.0 \tag{5.25}$$

where $C$ is an empirical overtopping coefficient, $F$ is crest elevation above S.W.L., and $R$ is the runup that would occur if there is no overtopping. The ratio of runup $R$ to the incident wave height $H$ was given by Ahrens (1981) as a function of wave steepness $(H/gT^2)$

$$\frac{R}{H} = 1.38 + 318 \left(\frac{H}{gT^2}\right) - 19700 \left(\frac{H}{gT^2}\right)^2 \tag{5.26}$$

Take a special case where the crest of the breakwater is at the s.w.l. $(F=0)$, and the width
$K_i = 0.40$ is in a good agreement with the chart provided by Saville (1963) for
impermeable structures, [see Fig.(5.7)]. However, Saville's chart gives a wider range of
0.33 to 0.42 for $H/gT^2 = 0.00248$ and $H/gT^2 = 0.00384$ respectively compared to the
solution given by Seelig. Saville's solution for permeable structures is $0.42 < K_i < 0.52$
for $H/gT^2 = 0.00269$ and 0.00248 respectively. The porosity in the Saville's experiments
was not mentioned.

The transmission coefficient $K_i$ in this experimental study were found to be $K_i = 0.61$
($T=2.2$ s), $K_i = 0.57$ ($T=1.9$ s), and $K_i = 0.52$ ($T=1.5$ s). The non-dimensional parameter
$(H/gT^2)$ for wave heights at $T=2.2$ s, $T=1.9$ s, and $T=1.5$ s were computed by averaging
the values of $(H/gT^2)$ for each wave period, and were found to be:

\[
\begin{align*}
H/gT^2 &= 0.0037 \text{ for } T=2.2\text{ s, standard deviation STD } = 0.000257 \\
H/gT^2 &= 0.0048 \text{ for } T=1.9\text{ s, standard deviation STD } = 0.000467 \\
H/gT^2 &= 0.0075 \text{ for } T=1.5\text{ s, standard deviation STD } = 0.000952
\end{align*}
\]

The higher transmission coefficient, $K_i$, in this study compared to the value computed
by Saville was probably caused by permeability, submergence, and wave steepness. The
porosity of the structure in this study was 48%, whereas Saville did not mention the
porosity he used in his research. The ratio of the crest height and the water depth $(h/d)$
in Saville's was 1.033, whereas the ratio used in this study (for comparison purpose) is
1. Finally, the range of $H/gT^2$ provided in Saville's charts is $0.00248 < H/gT^2 < 0.00384$,
whereas in this study the range is $0.0038 < H/gT^2 < 0.0075$. The prediction of

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prediction given by Saville (permeable case).

The comparison of \( K_t \) for submerged case between this study and Saville's prediction, with \( (h/d) \) approximately 0.9, shows in a good agreement. The range of transmission coefficient in this study is \( 0.55 < K_t < 0.63 \), whereas the predicted \( K_t \) Saville gave is \( 0.585 < K_t < 0.66 \) (see Fig. 5.9 submerged case).

5.7.2.2 Effect of Submergence on the Transmitted Wave

Submergence, compared to the crest width, has a very sensitive effect to the transmitted waves. This can be shown in Saville's wave transmission prediction given in Figures 5.9 and 5.10.

Select an initial case with \( (h/d) = 1.033 \), \( (H/gT^2 = 0.00441) \), and \( (b/h = 1) \). The predicted transmission coefficient in that condition is \( K_t = 0.62 \), see Fig. 5.9. If the crest height is increased up to \( (h/d) = 1.133 \) while maintaining \( (b/h = 1) \), Saville predicts that the transmission coefficient decreases to \( K_t = 0.268 \) at \( (H/gT^2 = 0.00403 \), the closest value to the initial \( H/gT^2 = 0.00441 \) available in Saville's chart), see Fig 5.10.

The transmission coefficient can also be reduced by increasing the crest width, namely \( (b/h = 3) \), while maintaining the crest height at \( (h/d) = 1.033 \). Using the same line at
From the above discussion it can be seen that as the crest height increases by 9.7% (from \( h = 1.033 \, d_c \) to \( h = 1.133 \, d_c \)) the transmission coefficient \( K_t \) decreases by 56.77% (from \( K_t = 0.62 \) to \( K_t = 0.268 \)). On the other hand, the increase of crest width by 200% (from \( b = h \) to \( b = 3h \)) decreases the transmission coefficient \( K_t \) only by 28.23% (from \( K_t = 0.62 \) to \( K_t = 0.445 \)).

Elkamhawy (1994) compared between his research and Saville's experimental work as presented in Fig.5.11. Elkamhawy's prediction shows that it requires 400% increase of crest width (from \( b = h \) to \( b = 5h \)) to reduce transmission coefficient \( K_t \) by 24.2% (from \( K_t = 0.62 \) to \( K_t = 0.47 \)) as shown in Fig. 5.11.

It can be concluded that it is not economical to reduce the transmitted wave by increasing the width of the crest, because a large amount of materials will be needed.

The relationship between the transmission coefficient and the submergence is presented in Fig.5.12. The submergence defined in this study is the ratio between the crest height to the still water depth at the structure. The range of the submergence is \( 0.73 < h/d < 1.38 \) to cover both submerged and low-crested structures.

Fig.5.12 shows both the averaged data and the linear fit of transmission coefficient \( K_t \) for
the three wave periods and wave heights were presented in section 5.7.2.1.

The approximate transmission coefficient linear regression equations consist of three curves for T=2.2 s (or H/gT^2 = 0.0037), T=1.9 s (or H/gT^2 = 0.0048), and T=1.5 s (or H/gT^2 = 0.0075). Each of these curves is divided in three regions, i.e. 0.73 ≤ h/d ≤ 0.85, 0.85 ≤ h/d ≤ 1.22, and 1.22 ≤ h/d ≤ 1.38 based on the distribution of the data, [see Fig.5.12]. The linear equation is given by:

\[ K_t = \frac{H_t}{H_i} = a \left( \frac{h}{d} \right) + b \]  \hspace{1cm} (5.27)

where \( a \) and \( b \) are constants, and the values in each of the region are given in Fig.5.12, \( h \) is the crest height, and \( d \) is the still water level at the structure.

From Fig.5.12 it can be seen that wave transmission for longer wave periods are higher than the transmission of the shorter wave periods.

The relationship between transmission coefficient and the relative crest height \( (R_c / H_i) \) was presented by Van der Meer (1990), see Fig.5.13, where \( R_c \) and \( H_i \) are crest height and incident wave height respectively. The maximum transmission coefficient is suggested to be 0.8 for the range of \( (R_c / H_i) \leq -1.15 \), and the minimum value is 0.1 for \( (R_c / H_i) \geq 1.15 \). A linear relationship from \( K_t = 0.8 \) to 0.2 is suggested within \(-1.15 \leq (R_c / H_i) \leq 1.15 \).
Figure 5.14 shows the comparison between the transmission coefficient obtained by Van der Meer (1990-b), Elkamhawy (1994), and this study. Generally, Van der Meer's data are found to be lower than the data in this experimental study, but higher than Elkamhawy's data. This may be due to the effect of porosity of the structure in this study is higher than that of Van der Meer's model. Some of Van der Meer's data show in fairly a good agreement with this study for wave period \( T = 1.5 \) s within the range of \(-0.25 \leq R_c / H_t \leq 0\), see Fig.5.14. Van der Meer and Elkamhawy, however, did not include the influence of wave period in their data.

5.7.3 Reflected Wave

The reflected wave height is expressed in term of reflection coefficient \( K_r \) which is the ratio between the reflected and the incoming wave heights given by

\[
K_r = \frac{H_r}{H_i}
\]

where \( H_t \) is the apparent reflected wave height and \( H_i \) is the apparent incident wave height given by:

\[
H_i = \frac{H_{(\text{max})1} + H_{(\text{min})1}}{2} ; \quad H_r = \frac{H_{(\text{max})1} - H_{(\text{min})1}}{2}
\]

where \( H_{(\text{max})1} \) = average of measured positive amplitude at the upstream of the model.
Fig. 5.15 shows the relationship between the coefficient of reflection $K_r$ and $(h/d)$ for wave periods $T=2.2$ s, $T=1.9$ s, and $T=1.5$ s.

Elkamhawy (1994) suggested that the reflected coefficient can be expressed as:

$$K_r = \frac{h}{d} \left( A_r + B_r \frac{H_i}{L_o} \right)$$  \hspace{1cm} (5.30)

where $A_r$ and $B_r$ are constants obtained by using least squares method depending on $(b/h)$, and based on his data the two constants are given as follows:

$$A_r = 8.16 - 8.0 \left( \frac{b}{h} \right)^{-0.003}$$  \hspace{1cm} (5.31)

$$B_r = 117.5 - 115.5 \left( \frac{b}{h} \right)^{-0.002}$$  \hspace{1cm} (5.32)

Elkamhawy (1994) concluded that reflection coefficient for submerged breakwaters is not sensitive to $(b/h)$, and $A_r = 0.16$ and $B_r = 2.0$ can practically be used for $h/d \leq 0.5$, and therefore, Eq. 5.37 can be simplified to:

$$K_r = \frac{h}{d} \left( 0.16 + 2.0 \frac{H_i}{L_o} \right)$$  \hspace{1cm} (5.33)

In this experimental study, $b/h = l$, resulting $A_r = 0.16$ and $B_r = 2.0$, and therefore
comparison between Elkamhawy's prediction and the available data in this study. It can be seen that Elkamhawy's prediction expressed in Eq.5.40 is higher than the data. It should be noted, however, that a valid comparison can be made only for \( h/d \leq 1 \) (submerged structures). The reflected wave, according to Elkamhawy's prediction is \( H_r = 0.26 \, H_i \) for \( T=2.2 \, s \); \( H_r = 0.28 \, H_i \) for \( T=1.9 \, s \); and \( H_r = 0.31 \, H_i \) for \( T=1.5 \, s \); whereas the data shows that \( H_r = 0.13 \, H_i \) for \( T=2.2 \, s \); \( H_r = 0.09 \, H_i \) for \( T=2.2 \, s \); and \( H_r = 0.02 \, H_i \) for \( T=1.5 \, s \), see Fig.5.16. It should be noted that Elkamhawy used two wave probes to determine the \( H_i \) and \( H_r \), whereas in this study three wave probes were used.

Figure 5.17 shows the data point and the linearized relationship between the reflection coefficient and \( h/d \). The trend shows that wave reflection increases as \( h/d \) increases. It can also be seen in Fig.5.17 that the slope of the longer wave is higher than the slope of the shorter wave, indicating that the longer waves have the faster increase in wave reflection than the shorter waves.

**Correction Factor**

The apparent reflection coefficient \( K_r \) expressed in Eq.5.35 is computed based on Airy wave theory. A correction factor should be applied to the apparent reflection coefficient to obtain the correct reflection coefficient. For this purpose Carry (1953) derived correction factor from the second order Stokes theory. Similarly Goda and Abe (1968)
on Stokes, the third order theory. The correct reflection coefficient $K_r'$ can be obtained by evaluating $K_r, d/L$ operating at the upstream toe of the structure, and the wave steepness based on the Airy wave theory (see Fig.5.18). From Fig.5.18 it can be seen that for $K_r \leq 0.3$, the correction factor is approximately equal to unity ($K_r = K_r'$) for $d/L \geq 0.12$ whereas for $d/L \leq 0.12$ the corrected reflection coefficient can be obtained from Fig.5.18, [see Silvester (1974)].

5.7.3 Dissipated Wave

When incident waves interact with submerged or low crested structures, some of the energy is transmitted to the form of transmitted waves, some is reflected, and some of the energy is dissipated. The energy contained in a single wave of unit crest width consists of kinetic energy $E_k$ and potential energy $E_p$, and based on Airy theory the magnitude of the total is given by:

$$E = E_k + E_p = \frac{\rho g H^2 L}{16} + \frac{\rho g H^2 L}{16} = \frac{\rho g H^2 L}{8} \quad (5.34)$$

Generally, the amount of energy contained in a single wave is expressed in term of specific energy or energy density which is defined as the amount of energy per unit wave length given by
\[ E = \frac{\text{Pa} \cdot \text{m}}{8} \]  

Equation 5.42 shows that the energy density of a wave is sensitive to the wave height, while it also depends on the specific density of the fluid, and the local acceleration of gravity. The energy density of the transmitted or reflected waves, can be expressed as in Eq.5.42 by exchanging the term \( H \) with \( H_i \) or \( H_r \). Accordingly, the dissipated energy can also be expressed in term of dissipated wave height, where the equivalent dissipated wave height equals to

\[ H_{ds}^2 = H_i^2 - H_t^2 - H_r^2 \]  

(5.36)

Figure 5.19 shows the relationship between the dissipation coefficient and \( h/d \), where the dissipation coefficient \( K_{ds} \) is defined as the ratio of the equivalent dissipated wave height and the incident wave height

\[ K_{ds} = \sqrt{\frac{(H_i^2 - H_t^2 - H_r^2)}{H_i^2}} \]  

(5.37)

It can be seen in Fig.5.19 that the dissipated height of the shorter waves is greater than the dissipated height of the longer waves. The trend of the curves shows that dissipated height increases with the increase of \( h/d \).
6.1 Introduction

As the use of rubble-mound breakwaters becomes more popular for coastal erosion protection or wave control structures, the stability of armour units of breakwaters has become an important subject for research. Many suggestions have been made as a result of their studies which include empirical formulas for armour units stability, numerical modelling, and theoretical relationships between the stability of armour units and the related wave, structure, and rock properties. In spite of the many suggestions, investigators agree that the stability of armour units depends strongly on the hydrodynamic forces, specifically, hydrodynamic forces caused by breaking waves. Therefore, accurate knowledge of the hydrodynamic forces on the individual armour unit is essential for a more rational design.

Hydrodynamic forces acting on an armour unit are very complex and are still under intense investigation. The forces on different shaped blocks have been expressed in a Morison force formulation (Van Gent, 1994). However, it is not precisely known which velocities, drag, mass, and lift coefficients, should be applied in the Morison equation since the armour blocks are embedded among each other, and the flow is between as well as under and over the armour blocks. This has led many researchers to use different approaches to obtain reliable velocity data that can be related to the force data.
using a micropropeller. Sarawangi et al. (1982) measured particle velocities on the slope of a breakwater by filming particles made of sponge with the same specific mass as water introduced in the water. The point-to-point sponge movement were recorded on film taken by a high speed film camera (50 frames per second). They found that the non-dimensional maximum velocity was a function of the surf similarity parameter and the ratio between the wave height and the water depth \( u_{\text{max}}/(gH)^{1/2} = F(\xi, H/h_0) \), where \( u_{\text{max}} \) is the maximum measured velocity in runup or rundown; \( \xi = \tan \alpha (H/L_0)^{1/2} \) is the surf similarity parameter; \( L_0 \) deep water wave length; \( h_0 \) is the still water depth in front of the breakwater; \( H \) is wave height; and \( \alpha \) is the slope angle. Iwata et al. (1985) carried out an experimental study on the wave forces on a single armour unit on a two-layer rubble-mound slope with uniform slopes 1 (vertical) : 2 (horizontal) and 1:3. The measurements also included water-particle velocities just above the force sensing rubble unit. Kobayashi et al. (1986, 1987) and Kobayashi and Wurjanto (1989) developed a numerical model for computation of water-particle velocities on an impervious rubble-mound slope based on the finite-amplitude shallow water wave equation. Then, using their model they calculated the vertically averaged horizontal velocities as well as runup and rundown. Kobayashi and Wurjanto (1989) also calculated the forces on single armour stone using some assumptions about the drag, mass, and lift coefficients. Losada et al. (1988) measured the forces under solitary waves on a cubic block close to a flat bottom. The block was a single block and not surrounded by other blocks. They analyzed the forces in view of the Morison equation based on wave particle velocities and accelerations obtained from the theoretical solitary wave theory.

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unit having relatively vertical front surface subject to a breaking wave. This impact component cannot be included in the inertia nor drag forces because it acts only in a very short time when the breaking wave starts to slam the front face of armour unit. Impact can be the dominant component producing the maximum force magnitude especially for non-overtopped breakwaters where the breaking waves can plunge directly on the armour units. For completely submerged rubble-mound breakwaters wave impact may be greatly damped by the fluid. Wave impact is also insignificant for broken or non-breaking waves.

In this experimental study the hydrodynamic forces were measured simultaneously with the dynamic pressure which was measured using a Pitot tube to obtain the water particle velocity slightly above the top surface of the cubical armour unit.

6.2 The Morison Equation

One of the formulas which has been widely used for estimating wave forces on objects is the semi empirical formula for hydrodynamic force known as the Morison (1950) equation. Morison force modelling was first applied for waves on vertical cylinders. The form of the Morison force formulation is given in many references, (see SPM 1987). Basically, the Morison equation consists of two parts i.e. the inertia force, and the drag force. The inertia force is in-phase with the fluid acceleration, while the drag force is in-phase with fluid velocity. It is assumed that the two forces are independent, and that their effect can be added to obtain the total force on the
the wave length, so that the scattering effect due to the presence of the armour unit are
negligible. Therefore the armour unit has a negligible effect on the waves so that the applicable
velocities and accelerations are those calculated as if the armour unit was absent; this may not
be true of breaking waves which can have a very steep advancing front.

6.2.1 Inertia Force

The inertia force, which is due to change of kinetic energy in the flow field, develops from the
fluid accelerating around the body. The acceleration is due to a pressure gradient in the fluid
which acts on the volume \( V_s \) displaced by the armour unit. The inertia force consists of two
components. The first force component, also called the Froude-Krylov force, is due to the
undisturbed pressure field which would exist on the boundary in the absence of the body.
Therefore, this force component has a similar form to the buoyancy force acting on a submerged
body. The magnitude is given by the mass of fluid (known as the virtual mass) displaced by the
armour unit times the acceleration, \( \rho V_s (\partial u/\partial t) \). However, the fluid in the wave must also move
around the armour unit, and consequently additional accelerations are involved. This imposes a
force caused by an added mass \( m_a \) times the acceleration: \( m_a (\partial u/\partial t) \), where the added mass
depends on the form of the wake in the separated flow. The total inertia force on the armour unit
becomes
\[ F_M = (\rho \ V_s + m_a) \left( \frac{\partial u}{\partial t} \right) \quad (6.1) \]

where, \( F_M \) = inertia or mass force, \( V_s \) = volume of the stone, \( m_a \) = added mass, and \( \left( \frac{\partial u}{\partial t} \right) \) = water particle acceleration. Introducing an empirical inertia coefficient \( C_M \) defined as

\[ C_M = 1 + \frac{m_a}{\rho \ V_s} = 1 + C_n \quad (6.2) \]

where \( C_n \) = added mass coefficient, and the 1 indicates the Froude-Krylov force. The total inertia force becomes

\[ F_M = C_M \ \rho \ V_s \left( \frac{\partial u}{\partial t} \right) \quad (6.3) \]

### 6.2.2 Drag Force

The drag force on the armour unit is primarily due to skin friction and pressure difference across the armour unit resulting from the fluid momentum flux. This pressure difference is mainly caused by separation of flow from the body of the armour unit which creates a low pressure region behind the armour unit. The position at which the separation occurs, the way in which it occurs, and the resultant wake width, all influence the pressure difference, and consequently the drag force.

The magnitude of the drag force is taken to be proportional to square of the instantaneous
theories. The form of the drag force is given by:

$$F_D = C_D \frac{1}{2} \rho \ u |u| \ A_x$$  \hspace{1cm} (6.4)$$

where $F_D$ = drag force; $C_D$ = drag coefficient; $\rho$ = fluid density; $u$ = local velocity (x-direction); and $A_x$ = projected area of the rock in x-axis.

6.2.3 Total Force of the Morison Equation

The total force according to Morison et. al., is obtained by combining Eqs. 6.3 and 6.4. So the Morison equation becomes

$$F = F_M + F_D$$

$$F = C_M \rho \ V_s \left( \frac{\partial u}{\partial t} \right) + C_D \frac{1}{2} \rho \ A_x \ u |u|$$  \hspace{1cm} (6.5)$$

6.3 Breaking Wave Impact

6.3.1 Introduction

Generally, breaking wave impact is considered only for vertical walls. Studies on this subject have been made in the laboratory by Bagnold (1939), Denny (1951), Carr (1954), Ross (1955), Leendertse (1961), Nagai (1961), Kamel (1968), Weggel (1968), and Weggel and Maxwell
Bagnold (1939) found that breaking wave impacts occur at the instant that the vertical front face of a breaking wave slams the wall and only when a plunging wave entraps a cushion of air against the wall. Since high impact pressure is dependant on wave geometry, it does not occur frequently against prototype structures. He suggested, however, that the possible occurrence of impact pressure has to be recognized and considered in design.

Minikin (1955, 1963) developed a procedure for designing vertical walls subjected to breaking waves. His formula was developed based on observations of full-scale breakwaters and results from Bagnold's study. Minikin's assumption was that the maximum pressure occurs at the S.W.L. The SPM (1987), however, made a comment that Minikin's method should be used with caution because it can give wave forces that are extremely high, as much as 15 to 18 times those computed for non-breaking waves, (see the SPM for more information).

6.3.2 Effect of Armour Unit Orientation on the Impact Force

In this experimental study, the effect of armour unit orientation on the hydrodynamic forces were investigated. Figure 6.1 shows the ratio of the projected area $A_x$ (on a vertical plane perpendicular to the incoming waves) for the cubical armour unit when it was rotated for different angle of orientation. As the cubical armour unit is rotated to $\beta = 15^\circ$ and $26.57^\circ$, $A_x$ increases by 22% and
(vertical) forces on the same cubical armour unit, at the same location (leading edge of the breakwater), at the same wave period \((T = 1.9 \text{ s})\) and water depth \((d = 22.5 \text{ cm})\), but at different orientations \(\beta = 0^\circ, 15^\circ, \text{ and } 26.57^\circ\). The orientation of the armour unit is defined as the angle \(\beta\) between the bottom surface of the unit and horizontal line. It can be seen from Figures 6.2 to 6.4 that as the angle of orientation \(\beta\) increases, the maximum horizontal force decreases. The maximum horizontal forces on the unit at \(\beta = 0^\circ, 15^\circ, \text{ and } 26.57^\circ\) are respectively 15.37 N (for \(H = 174.1 \text{ mm}\)), 12.88 N (for \(H = 174.2 \text{ mm}\)), and 7.017 N (for \(H = 179.1 \text{ mm}\)). The corresponding averaged maximum horizontal forces are 13.37 N (for \(\beta = 0^\circ\)), 9.91 N (for \(\beta = 15^\circ\)), and 6.033 N (for \(\beta = 26.57^\circ\)), (see Figures 6.2 to 6.4).

The magnitude of the horizontal force on the armour unit is proportional to the projected area \(A_z\), and also dependent on the degree of exposure to wave attack. As a result, as the armour unit is rotated by angle \(\beta \leq 45^\circ\), the projected area \(A_z\) as well as the exposure to wave attack increase accordingly (see Fig.6.1). The projected area \(A_z\) and the degree of exposure reach their maximum when \(\beta = 45^\circ\). On the contrary, Figures 6.2 to 6.4 show the opposite trend in the forces. The horizontal force decreases as \(\beta\) increases. This shows that orientation of the armour unit affects the hydrodynamic forces. When a strong breaking wave plunges against a vertical surface (such as in the cubical armour unit when it is placed at the normal orientation) impact force caused by shock pressure contributes considerably to the increase in the maximum horizontal force. In this case, the impact force must be considered in addition to the inertia and the drag forces components. A non-vertical face experiences less impact force than a vertical face.
Figures 6.5a to 6.5r show the wave attack on the breakwater for the low crested case where d = 225 mm, T = 1.9 s, h/d = 1.22, and H = 178.3 mm. The wave movement was recorded by video at a rate of 30 frames per second. Figures 6.5a to 6.5r are typical of images that were made with the aid of frame by frame analysis, and the Corel Draw Software.

Figures 6.5a to 6.5h show the movement of the first wave from still water condition. It should be noted that term '0.0 sec' shown in Fig. 6.5a does not refer to the time when test # 498 was started. It only indicates the time when the first wave started to arrive in the breakwater zone. The hydrodynamic forces recorded within that period of time were not the maximum.

Figures 6.5i to 6.5r show the movement of wave 3.9 to 4.83 seconds after the first wave arrived. By comparing the water surface profiles between the first wave and the wave captured in Figures 6.5i to 6.5r it can be seen that this wave cycle induces the greater forces compared to those produced by the first wave. The wave front shown in Fig. 6.5k and followed by Fig. 6.5l (within only 0.03 s) show the rapid pressure gradient change which produces a shock pressure or impact on the armour unit located at the leading edge of the crest.

Figures 6.6 and 6.7 show the hydrodynamic forces profiles on the cubical armour units in Test # 498 in which the two armour units were arranged in tandem. Fig. 6.6 shows the forces on the front (at the leading edge) armour unit. Observe that the maximum horizontal force on the 1 kg
located immediately behind the first unit. The maximum horizontal force on the second cubical armour unit (also of 1 kg mass) was only 4.427 N. Figure 6.8 shows the comparison between the two simultaneous horizontal forces acting on the two armour units.

The smaller force experienced by the second armour unit was not only caused by the smaller velocity developed in front of the unit due to sheltering effect, but also the absence of impact force such as experienced by the front armour unit.

Minikin related the maximum dynamic pressure on the vertical wall to breaker height as expressed in the following empirical equation, (see SPM 1987):

\[ p_m = 101 \gamma \frac{H_b}{L_b} \frac{d_s}{D} (D + d_s) \]  \hspace{1cm} (6.6)

where \( \gamma \) = unit weight of fluid, \( H_b \) = breaker height, \( d_s \) = depth at the toe of the vertical wall, \( D \) = depth at location as far as wave length \( L_o \) from the vertical wall. The corresponding force \( (R_m) \) was given by

\[ R_m = \frac{p_m H_b}{3} \]  \hspace{1cm} (6.7)

In this study the impact force component was related to the wave height at the leading toe of the breakwater. Ideally, this force must be related to the breaker height. From laboratory observations the location of the breaker was not at a fixed point for different variables. Therefore, it was
is to relate the impact force to the wave height at the toe \(H_{\text{toe}}\), (see also discussion in Section 5.3). Since a wave probe was located at the toe, then the horizontal force can always be related to the corresponding \(H_{\text{toe}}\). Hudson (1958) also used \(H_{\text{toe}}\) for determining the stable weight of armour units for conventional breakwaters, (see Eq.2.1).

6.3.4 Formulation of Impact Force

The formulation of impact force in this study is given by

\[
F_{\text{imp}} = C_{\text{im}} A_{\text{face}} \gamma H_{\text{toe}}
\]  \(\text{(6.8)}\)

where \(C_{\text{im}}\) = impact index, \(A_{\text{face}}\) = front surface area, \(\gamma\) = fluid specific weight, and \(H_{\text{toe}}\) = wave height at the toe.

6.4 Formulation of Improved Wave Force

Considering the impact force discussed in Section 6.3, the improved equation of force acting on the cubical armour unit can be constructed by adding the impact force (Eq.6.8) to the Morison Equation (Eq.6.5)

\[
F = C_{\text{im}} A_{\text{face}} \gamma H_{\text{toe}} + C_M \rho V_s \left( \frac{\partial u}{\partial t} \right) + C_D \frac{1}{2} \rho A_x u |u| \]  \(\text{(6.9)}\)

Since the armour unit was placed at the leading edge and partially embedded among the surrounding rocks, it cannot be expected to achieve the normal inertia and drag coefficients as
and \( C_D \) are used in this study to replace the inertia and the drag coefficients \( C_M \) and \( C_D \).

Equation 6.9, therefore, can be rewritten as

\[
F = C_{im} A_{face} \gamma H_{loc} + C'_M \rho V_s \left( \frac{\partial u}{\partial t} \right) + C'_D \frac{1}{2} \rho A_x u |u| 
\]  

(6.10)

6.5 **Application of the Improved Force Equation**

As mentioned in Section 6.1 that the water particle velocity was measured at a point in an elevation slightly above the top line of the armour unit front surface as shown in Fig. 6.9. Therefore, the dynamic pressure on the Pitot tube started to exist when the water particle started to reach that point. The hydrodynamic forces, on the other hand, were experienced by the unit when the wave started to touch the armour unit. Consequently, the hydrodynamic forces started to be recorded while the the dynamic pressure at the measurement point was still zero. Figure 6.9 illustrates how the simultaneous forces and the dynamic pressure measurements were made. Figure 6.9 represents the cubical armour unit under low-crested breakwater case.

As the wave moves forward, its surface moves upward, (see Fig.6.9). The armour unit starts to experience hydrodynamic forces when the wave starts to touch the bottom line represented by point A of the unit (see phase 1 in Fig. 6.9). In the mean time the dynamic pressure at a point slightly above point B is still zero. At a very short time later the surface water moves from phase 1 to phase 2 where now some part of the unit body experiences uplift force and longitudinal
surfaces. The dynamic pressure at the measurement point slightly above point B is still zero. The hydrodynamic forces on the armour unit increase as the water surface increases rapidly to build the higher pressure gradient as illustrated in phase 3 of Fig. 6.9. At this moment the whole unit front surface is now experiencing hydraulic pressure causing significant increase on the horizontal force, see also Figs. (6.5k and 6.5l). At this moment the dynamic pressure at the measuring point starts to exist and the dynamic pressure is recorded by the pressure transducer. While the pressure gradient may still be built up (up to the breaking point), it is assumed that the maximum impact force occurs when the whole front surface of the unit first experiences the hydraulic pressure as shown in phase 3 in Fig. 6.9.

As explained above, there is time lag between forces and the dynamic pressure measurements because of the location of the load cells and the pressure transducer set-up. Figures 6.10 to 6.12 show the time lag between the horizontal force and the computed velocity for different wave periods. In reality, however, the hydrodynamic forces cannot exist without the presence of a dynamic pressure. Consequently, an adjustment has to be made in order that the inertia and drag force components in Eq. 6.9 can be applied. The adjustment is made by shifting the dynamic pressure data (which starts in phase 3) to start at phase 1 as illustrated in Fig. 6.9.

The amount of time needed for the water surface to change from phase 1 to phase 3 is very small depending also on the wave periods, and it may only on the order of hundredths of a second (see also Figs. 6.5k and 6.5l). Therefore, an assumption is made in this study that the impact force
Figures 6.13 to 6.18 show the typical measured $H_{10}$ and the corresponding measured $F_x$, and the computed (from the measured dynamic and static pressures) velocity ($u$) and acceleration ($du/dt$) for wave periods ($T$) = 2.2 s, 1.9 s, and 1.5 s (at approximately the same wave height ($H$); and ($h/d$) = 1.22, and 1). Figures 6.13 to 6.15 show the low-crested case at ($h/d$) = 1.22, and wave periods $T$ = 2.2 s, 1.9 s, and 1.5 s respectively. Observe that there is a sharp spike in the beginning of the horizontal force ($F_x$) in each of the figures. It can also be seen that the sharper spike occurs in forces at the shorter wave period, (see Figs. 6.13 to 6.15). When the water depth increases, e.g. ($h/d = 1$), the spike in the curve becomes less sharp as shown in Figs. 6.17 and 6.18 (for $T$ = 1.9 s and $T$ = 1.5 s respectively), and even disappears as indicated in Fig. 6.16 (for $T$ = 2.2 s). The presence of the sharp spike in the beginning of the horizontal force plot indicates the occurrence of impact forces on the armour unit.

Figure 6.19 shows the schematic diagram of the horizontal force on the armour unit. The shaded area in Fig. 6.19 shows the influence of impact (shock pressure) on the force which depends on the wave periods, submergence, and also the exposure to waves attack.

In applying Eq. 6.9 to the wave, hydrodynamic forces, and dynamic pressure data, it is necessary to make the following notes:

1. The horizontal water particle velocity was computed based on the dynamic pressure data.
\[ u = \sqrt{2g \left( p_{tot} - p_s \right)} \]  

(6.11)

where \( p_{tot} \) = total pressure head, and \( p_s \) = static pressure head, and \( g \) = gravity acceleration.

2. The acceleration of the fluid was computed based on the water particle velocity:

\[
\frac{\partial u}{\partial t} \approx \frac{u_i - u_{i-1}}{\Delta t} = \frac{\Delta u}{\Delta t}
\]  

(6.12)

where \( u_i \) = velocity at time point \( t_i \); and \( u_{i-1} \) = velocity at time point \( t_{i-1} \), where \( \Delta t = 0.05 \) s.

3. The impact force starts at the same time with the inertia and drag forces, but stops at the force peak, while the inertia and drag forces continue to exist throughout the wave cycles.

4. The \( H_{toe} \) used in the impact force component is the actual (not averaged) wave height at the front toe of the breakwater which produced the corresponding hydrodynamic forces on the armour unit.

5. The least squares method was used in the analysis for determining the best impact, inertia, and drag indices. This method involves minimizing the sum of the squares of the errors to the measured forces using the improved Morison equation with the measured dynamic pressure for computing the velocity, and the derived accelerations. The sum is given by
\sigma^2 = \frac{1}{I} \sum_{i=1}^{I} \left[ F_{\text{ml}} - F_{\text{imp}} - F_M - F_D \right]^2 \tag{6.13}

where $F_{\text{ml}}$ = measured horizontal force at time point $i$; and $I$ = total number of data points; $F_{\text{imp}}$ = impact force; $F_M$ = inertia force; and $F_D$ = drag force.

The procedure for the determination of $C_{\text{im}}$, $C'_M$ and $C'_D$ is explained below, and typical results are shown in Figs. 6.20 to 6.21 as an example.

1. Select one cycle of force data and the corresponding $H_{\text{tot}}$, velocity, and acceleration as shown in Fig. 6.20. Figure 6.21 shows the lag time between the measured force data and the computed velocity and acceleration. The lag time is $t = (17.45 \text{ s} - 17.30 \text{ s}) = 0.15 \text{ s}$ (see Fig. 6.20).

2. Shift the velocity and acceleration data to eliminate the lag time of 0.15 s, i.e. velocity and acceleration from $t = 17.45 \text{ s}$ to $t = 17.30 \text{ s}$ (mentioned in point 1), so that they are in phase with the force data. When shifting the velocity and acceleration data, an assumption is made that the velocity and acceleration at the measuring point are the same as if they act at the middle of front surface of the armour unit. Fig. 6.21 shows the condition after the velocity and acceleration data are shifted.

3. Choose any initial values of $C_{\text{im}} \neq 0$, $C'_M \neq 0$, and $C'_D \neq 0$ to be applied to Eq. 6.10 to
applied to Eq. 6.10 from the beginning of the selected cycle to the peak of the force (see Fig. 6.21), whereas drag index $C'_D$ and inertia index $C'_M$ are applied to Eq. 6.10 from the beginning to the end of the cycle.

4. Using Eq. 6.13 minimize the sum of the squares of the errors to the measured forces for the determination of best impact index $C_{im}$, inertia index $C'_M$, and drag index $C'_D$. Computation was carried out using the optimization method in the Quattro Pro 4.0 software to solve $C_{im}$, $C'_M$, $C'_D$ simultaneously.

6.6 **Comparison of the Measured and the Theoretical Forces**

The length of measurement time in each test in this study is approximately 30 seconds. Since three wave periods of $T = 2.2$ s, 1.9 s, and 1.5 s were chosen in this study, there are 13 to 20 waves as well as force cycles in every test.

Comparisons of the measured and the predicted forces are made in two ways. The first is by comparing the measured and theoretical forces of a randomly selected cycle of force in one test. Eighteen tests were selected for this purpose representing various submergence and wave periods. The second is by comparing the maximum measured and theoretical forces in each test, for all the tests. Sixty tests were analyzed for this purpose. The impact, inertia, and drag indices in both comparisons will also be analyzed to determine the values of the indices that can be suggested
6.6.1 Comparison of the Measured and the Theoretical Forces: Randomly selected cycle in each test

Eighteen tests were chosen in this section to compare between the experimental and the predicted forces. One force cycle was selected randomly (any one cycle) in each test for comparison purpose.

Figures 6.22 to 6.39 show the comparison between the measured and the theoretical forces computed using Eq. 6.10, for the conditions given below:

- Figs. 6.22 to 6.24: \( d = 225 \text{ mm} \) \( h/d = 1.22 \) \( T = 2.2 \text{ s}, 1.9 \text{ s}, \text{ and } 1.5 \text{ s} \)
- Figs. 6.25 to 6.27: \( d = 250 \text{ mm} \) \( h/d = 1.10 \) \( T = 2.2 \text{ s}, 1.9 \text{ s}, \text{ and } 1.5 \text{ s} \)
- Figs. 6.28 to 6.30: \( d = 275 \text{ mm} \) \( h/d = 1.00 \) \( T = 2.2 \text{ s}, 1.9 \text{ s}, \text{ and } 1.5 \text{ s} \)
- Figs. 6.31 to 6.33: \( d = 300 \text{ mm} \) \( h/d = 0.92 \) \( T = 2.2 \text{ s}, 1.9 \text{ s}, \text{ and } 1.5 \text{ s} \)
- Figs. 6.34 to 6.36: \( d = 325 \text{ mm} \) \( h/d = 0.85 \) \( T = 2.2 \text{ s}, 1.9 \text{ s}, \text{ and } 1.5 \text{ s} \)
- Figs. 6.37 to 6.39: \( d = 350 \text{ mm} \) \( h/d = 0.79 \) \( T = 2.2 \text{ s}, 1.9 \text{ s}, \text{ and } 1.5 \text{ s} \)

The values of the impact, inertia, and drag indices, the maximum measured and predicted forces in the randomly selected cycle in each case, and the corresponding figure are summarized in Table 6.1.

The ranges of the impact index are \(-0.0058 \leq C_{im} \leq 0.1032\) (for \( T = 2.2 \text{ s} \)), \(-0.1497 \leq C_{im} \leq 0.1822\) (for \( T = 2.2 \text{ s} \)), and \(-0.0432 \leq C_{im} \leq 0.0849\) (for \( T = 2.2 \text{ s} \)).
index obtained in this study is difficult to explain; however, this is probably a defect in the
method of fitting the modified Morison Equation to the data. The average of the impact index
is $C_{im} = 0.0661$ at $STD = 0.1226$, (see Table 6.1).

The ranges of the inertia index are $0.0433 \leq C'_M \leq 0.1551$ (for $T = 2.2$ s), $0.0867 \leq C'_M \leq 0.1730$
(for $T = 1.9$ s), and $0.0655 \leq C'_M \leq 0.2275$ (for $T = 1.5$ s). The average of all of the inertia index
in this comparison is $C'_M = 0.1056$ at $STD = 0.0484$, (see Table 6.1).

The range of the drag index is $0.0108 \leq C'_D \leq 0.0632$ (for $T = 2.2$ s), $0.0014 \leq C'_D \leq 0.0497$ (for
$T = 1.9$ s), and $-0.0091 \leq C_D \leq 0.0443$ (for $T = 1.5$ s). As in the case of negative impact index,
the negative drag index (which is irrational and difficult to explain) were also found in several
computations in this study, (see Table 6.1). The average of the drag index is $C_D = 0.0223$ at $STD$
$= 0.0210$, (see Table 6.1).

The comparison of the measured and predicted maximum forces in each of the selected cycle are
shown in Table 6.1. The comparison is expressed in terms of the ratio (in %) between the
predicted (theoretical) maximum force and the measured (experimental) maximum force. The
average ratio is $101.8551 \%$ at $STD = 31.5777$, (see Table 6.1). The complete comparison of the
measured and the computed force patterns in each cycle is given in the corresponding figures as
listed in column 'Reference' in Table 6.1.
mentioned in Figs. 6.22 to 6.39 and the steepness \((H_{\text{toe}}/gT^2)\). The plots of the impact, inertia, and drag total average are presented in Fig. 6.41, and also Table 6.1. There does not appear to be a relationship between the various indices and steepness.

The relationship between the impact, inertia, and drag indices and the submergence \((h/d)\) is given in Figures 6.42 and 6.43. Figure 6.42 shows the average values of the impact, inertia, and drag indices, whereas Fig. 6.43 shows the average of those indices. The impact index appears to increase with \((h/d)\).

Tørum (1993) conducted an experimental study of wave-induced forces on an armour unit on a berm breakwater. The armour unit was placed on the sloping berm. The force and the velocity parallel to the slope were measured simultaneously. The drag and inertia coefficients were presented in form of a relationships between those coefficients and the Keulegan-Carpenter (KC) number for different wave periods. Three wave periods were used in his study i.e. \(T = 2.1, 1.8, \text{ and } 1.5\) seconds. The Keulegan-Carpenter (KC) number was defined as

\[
KC = \frac{u_o T}{D}
\]

(6.14)

where \(u_o\) = measured maximum velocity; \(T\) = wave period; \(D\) = diameter of the stone which was equals to \((\text{stone mass/}\rho_s)^{1/3}\), where \(\rho_s\) is the specific density of the stone.

Figure 6.44 shows the plot of impact index and the Keulegan-Carpenter number in this study.
were found. These impact indices were produced by waves at $T = 1.9$ s and $1.5$ s correspondingly at $(h/d) = 1.22$ as shown in Fig. 6.43.

Figure 6.45 shows the plot of inertia index (in this study), Tørum's inertia coefficient, and the KC number. It can be seen in Fig. 6.45 that the inertia indices of this study are similar but generally slightly smaller than the inertia coefficient of Tørum's for $T = 1.5$ s, and $T = 1.9$ s and $T = 1.8$ s, except for $T = 2.2$ s and $T = 2.1$ s in which Tørum's data are scattered even with negative inertia values. However, in both this study and Tørum's investigation, the values of $C'_M$ or $C_M$ are generally small. Since $C_M$ is often written as $C_M = 1 + C_m$ (see Eq. 6.2), where $C_m$ is the added mass and $1$ is the Froude-Krylov force, the value of $C_M < 1$ shows that $C_m$ is negative which is not realistic. This probably indicates that a Morison force formulation based on the flow at the measurement point may not give a reasonable description of the forces. There are also some other logical possibilities e.g. velocity scale may be off; separation of impact and other forces may be poor; and the optimization process for curve fitting may be insensitive to impact, inertia, and drag.

Figure 6.46 shows the comparison between the drag index in this study and the drag coefficients of Tørum's. In both cases, the values of drag index and drag coefficient are generally small, and they are not sensitive to the wave period. In Tørum's case, the force on the armour unit at the slope is drag dominated, and in this study where the cubical armour unit was located at the leading edge, the force is inertia or impact dominated, (see Figures 6.41 to 6.43). For the
Tørum's placed the armour unit (diameter $D = 3.84$ cm) at the slope where the distance of the armour's highest point to the S.W.L. was 10.3 cm and the water depth $d = 70$ cm. With that set-up, the armour unit was always submerged and free from plunging resulting in no or very small impact (pressure shock). Since Tørum's does not separate the impact component from the rest of force equation, the drag and inertia components in this study are consequently greater than the drag and inertia.

6.6.2 Comparison of the Measured and the Theoretical Forces: At the maximum force in each test

The same procedure of analysis as mentioned in Section 6.6.1 is applied to the force maxima in 80 tests representing different submergence, wave periods, and wave heights. The results of the analysis are summarized in Table 6.2 including the impact, inertia, and drag indices; the measured and predicted maximum forces in each cycle; and reference of the corresponding plots showing the comparison between the experimental and the theoretical forces.

From Table 6.2 it can be seen that negative values of the impact, inertia, and drag indices were found which are not realistic and difficult to explain. The average values of the impact, inertia, and drag indices are 0.0809 (STD = 0.1220), 0.0972 (STD = 0.0581), and 0.0281 (STD = 0.0338) respectively, (see Table 6.2).
forces cases are greater than those of the randomly selected force cycles. This can be shown in
the summary below:

Randomly selected cycle:

<table>
<thead>
<tr>
<th>Average</th>
<th>C_{im} = 0.0661</th>
<th>STD = 0.1266</th>
</tr>
</thead>
<tbody>
<tr>
<td>Average</td>
<td>C'_{M} = 0.1056</td>
<td>STD = 0.0484</td>
</tr>
<tr>
<td>Average</td>
<td>C'_{D} = 0.0223</td>
<td>STD = 0.0210</td>
</tr>
</tbody>
</table>

Cycles at the force maxima:

<table>
<thead>
<tr>
<th>Average</th>
<th>C_{im} = 0.0809</th>
<th>STD = 0.1220</th>
</tr>
</thead>
<tbody>
<tr>
<td>Average</td>
<td>C'_{M} = 0.0972</td>
<td>STD = 0.0581</td>
</tr>
<tr>
<td>Average</td>
<td>C'_{D} = 0.0281</td>
<td>STD = 0.0338</td>
</tr>
</tbody>
</table>

Figures 6.47 and 6.48 show the relationship between the values of the impact, mass (inertia), and
drag indices and the non dimensional \((H_{toe}/gT^2)\).

The relationship between the impact, inertia, and drag indices versus \((h/d)\) is also presented in
Figures 6.49 and 6.50. In general the impact and inertia indices are greater than the drag index
at \((h/d) \leq 1.1\) as shown in Fig. 6.49, whereas at \((h/d) = 1.22\) it is obvious that the impact is
strong.

Figures 6.51 to 6.53 respectively show the distribution of the impact, inertia, and drag versus the
Keulegan-Carpenter Number.
Table 6.2) and at \((h \, d) = 1.22\), (see Fig. 6.49). The second highest \(C_{im} = 0.3039\) was caused by wave \(H_{toe} = 14.63 \, \text{cm}\) at \(T = 1.9 \, \text{s}\), (see Table 6.2) and at \((h \, d) = 1.22\), (see Fig. 6.49). Comparing Fig. 6.51 to Fig. 6.44, the maximum \(C_{im} = 0.3600\) in Fig. 6.44 was produced by wave \(H_{toe} = 13.10 \, \text{cm}\) at \(T = 1.5 \, \text{s}\) (see Table 6.1) is greater than the maximum \(C_{im} = 0.3463\) in Fig. 6.51 produced by wave \(H_{toe} = 16.58 \, \text{cm}\) at \(T = 2.2 \, \text{s}\), (see Table 6.2). If the highest peak forces produced by strong breaking waves are mainly caused by impact or shock pressure (\(F_{imp} > F_M\) and \(F_{imp} > F_D\)), so that both \(F_M\) and \(F_D\) can be neglected at the peak force), the impact index \(C_{im}\) is given in Figures 6.52 and 6.53. The maximum impact index is \(C_{im\, max} = 1.37\) occurs at \((h \, d) = 1.22\) at the value of \((H_{ve}/gT^2) = 0.041\). From Figures 6.51 to 6.53 it can be concluded that, higher wave heights produce a greater impact index on the unit; shorter the wave periods produce a greater impact index on the unit; and the greater the value of \((h \, d)\) the greater is the impact index.

Figure 6.54 shows the distribution of the values of the inertia index with respect to the Keulegan-Carpenter Number. The maximum \(C'_M = 0.3086\) was produced by wave \(H_{ve} = 14.09 \, \text{cm}\) at \(T = 1.9 \, \text{s}\) (see Table 4.2), and at \((h'd) = 1.1\) (see Fig. 6.49). Tørum's data mentioned in Fig. 5.54 are the same as those mentioned in Fig. 6.45. Tørum estimated in his experimental investigation that the maximum \(C'_M = 0.3700\) occurred at \(T = 1.8 \, \text{s}\) and \(KC = 29\), whereas in this study the maximum \(C'_M = 0.3086\) occurred at \(T = 1.9 \, \text{s}\) and at \(KC = 22.99\). Comparing Fig. 6.54 to Fig. 6.45 it can be seen that the maximum inertia index in Fig. 6.54 is greater than that of Fig. 6.45 for wave periods of \(T = 2.2 \, \text{s}, 1.9 \, \text{s}, \text{and } 1.5 \, \text{s}\).
$C'_D = 0.0989$ was produced by wave $H_{toe} = 14.09$ cm at $T = 1.9$ s, (see Table 6.2). For the randomly selected cycle discussed in Section 6.6.1 the maximum $C'_D = 0.0683$ was produced by wave $H_{toe} = 15.03$ cm at $T = 2.2$ s, (see Table 6.1 and Fig. 6.16). The maximum drag index for the force maxima is greater than that of the randomly selected cycle case. Tørum’s drag and inertia coefficients are greater than the values in this study because Tørum does not separate the impact component from the force equation, whereas in this study the impact force is separated as shown in Eq 6.10 (see also discussion on Section 6.6.1).

Figure 6.56 shows the comparison between the predicted and the measured maximum forces. The percent ratio of the theoretical and the experimental forces is indicated at the left ordinate, whereas the magnitudes of the forces are shown at the right ordinate. The range of the ratio is approximately $60\% < (F_{th} / F_{exp}) < 140\%$, whereas the values of the ratio are given in Table 6.2. The average of the ratio between the theoretical and the measured maximum forces is $90.97\%$ at STD = 18.45, (see Table 6.2).

The detailed comparisons of the theoretical and the experimental forces for all of the force maxima in each test are presented in Figures 6.57 to 6.116. The corresponding water depth, wave period, wave height at the toe ($H_{toe}$), the computed impact, inertia, and drag indices, the measured and predicted maximum forces are given in Table 6.2.

From Figures 6.57 to 6.116 it can be observed that the predicted force in some cases show a
6.76, 6.77, 6.78, 6.85, 6.87, 6.88, 6.89, 6.92, 6.100, 6.104, 6.105, 6.106, 6.107, and 6.114. On the other hand, in some cases the predicted plots show unsatisfactory results, for example Figures 6.59, 6.60, 6.61, 6.62, 6.63, 6.64, 6.67, 6.68, 6.71, 6.80, and 6.86. In some other cases, the predicted force shows a fair agreement, for example Figures 6.73, 6.75, 6.84, and 6.95.

From the above discussion it can be concluded that the proposed improved force equation as given in Eq. 6.10 does not always accurately predict the actual force. Although the improved force equation may predict the maximum force magnitude in a good agreement with the measured maximum force such as shown in Fig 6.67 (F-th/F-exp = 104.53 %), Fig. 6.71 (F-th/F-exp = 98.32 %), and Fig. 6.73 (F-th/F-exp = 101.66 %), the prediction of the force does not reflect the force variation with time. Several reasons can be given for this: first, the location of the flow measurement point may not give a reasonable description of the forces; second, the technique for separation of impact and other forces may be poor; and third, the optimization may be insensitive to or indiscriminant of the force components (impact, inertia, and drag).

The correct location of velocity measurement is yet to be investigated to obtain accurate force prediction. Since the impact force produced by shock pressure occurs on the units subjected to strong breaking waves, this must be separated from the rest of the force components. As a result, impact force component must be considered in the complete force equation. However, since hydrodynamic pressure on the unit's front surface varies from point to point, more than one point velocity measurement is needed for better accuracy.
7.1 Introduction

The common practice in designing armour units for breakwaters is to determine the stable weight of the unit at the most critical location, and under a selected or design hydrodynamic loading. The stable weight of a unit determined at this location is then used as the stable weight of the armour units for the whole design. This involves the determination of design waves that can be expected to occur in the site, the dimension of the breakwater, and the materials to be used to construct the breakwater. The design parameters include: the design of maximum and minimum still water depths, significant wave height, breaking waves, set-up, run-up and rush-down, wave transmission, reflection, diffraction and refraction, and other wave characteristics such as groupiness. The geometry of the breakwater consists of the seaward and landward slopes, and the height and width of the crest. The characteristics of material to be used to construct breakwaters include: the specific weight, shape, surface roughness and angularity, and the unit weight of armour units. Although the above approach is practical and provides a conservative stability for the breakwater, it may not be the most cost effective design.

A more rational approach is the determination of the stable weight of an armour unit at a certain location by a relation to the hydrodynamic forces at that location. This requires the determination of the maximum hydrodynamic force field along the surface of the breakwater. The determination of the maximum force field involves several variables that exist at the site such as wave periods,
on the structure must be analyzed based on the maximum hydrodynamic forces at that location.

In this chapter the maximum force field along the surface of the breakwater will be discussed. Eight points A, B, C, D, E, F, G, and H as shown in Fig. 7.1 were selected for the experimental study of the forces on the cubical armour unit located at those points. Figure 7.1 also shows the orientations of the armour unit located at those points. The maximum force field includes the horizontal (forward and backward), vertical (upward and downward), or the tangential and normal components of the forces on the armour units. The discussion of the stability of the armour units due to sliding or overturning will be presented in Chapter VIII. The overturning moment in this discussion includes that caused by the tangential force and the spinning moment.

Interlocking is another important factor that increases the stability of the armour units. How the interlocking and sheltering stabilizes the armour units will also be discussed by looking at the correlation of simultaneous forces acting on two armour units placed in tandem. This knowledge will also be useful to improve the present concept of designing armour units, i.e. determination of a stable individual armour unit size, to a new concept namely of designing an integrated armour system in which several armour units are inter-connected to carry the hydrodynamic loadings.
The sampling rate of each sensor in this study is 20 data per second, and the record time length of each test is approximately 30-35 seconds. This produces about 13-16, 15-18, and 20-23 cycles for waves at $T = 2.2$ s, 1.9 s, and 1.5 s respectively. The wave profile in each cycle, however, is not always exactly the same mainly due to the interaction between the waves and the breakwater.

Wave variation recorded in this study increases with $(h \cdot d)$. This may be affected by wave reflection i.e. $K$, increases as $(h \cdot d)$ increases. As the reflected waves reach the paddle, they are reflected back by the paddle which produce secondary reflection that moves to the breakwater. The secondary reflection coefficient off the paddle may be significant due to its vertical plain of the paddle. The primary and secondary reflections interact with the incoming waves that influence variability of the incoming waves. Since the hydrodynamic forces depend on the wave pattern, the hydrodynamic forces on the armour units in each cycle are not identical either. Some random variation in waves may also be produced within one test run.

It should be noted that all of the tests conducted in this study were under breaking wave conditions; this breaking condition may amplify slight variations in the wave characteristics. As a result, the ratio of maximum to the minimum peaks of the force within one test run varies from one test to another.
1.5 s. Wave breaking produced by waves at $T = 1.9$ and 1.5 seconds are stronger than those produced by wave at $T = 2.2$ s. In order to better show the trend of the maximum force field, the averaged maximum values are used.

The variation of hydrodynamic forces on the cubical units can be seen from their Standard Deviation (STD). The value of STD of each test was computed as a percent of the averaged maxima, and the ranges of the value are presented as follows:

<table>
<thead>
<tr>
<th></th>
<th>Seaward placement</th>
<th>Landward placement</th>
</tr>
</thead>
<tbody>
<tr>
<td>of the armour unit</td>
<td></td>
<td>of the armour unit</td>
</tr>
<tr>
<td>Forward Force:</td>
<td>STD = (0.75 - 28.69) %</td>
<td>STD = (1.09 - 57.33) %</td>
</tr>
<tr>
<td>Backward Force:</td>
<td>STD = (0.70 - 52.74) %</td>
<td>STD = (0.40 - 65.29) %</td>
</tr>
<tr>
<td>Upward Force:</td>
<td>STD = (1.64 - 36.53) %</td>
<td>STD = (1.75 - 58.67) %</td>
</tr>
<tr>
<td>Downward Force:</td>
<td>STD = (0.87 - 47.48) %</td>
<td>STD = (1.79 - 51.61) %</td>
</tr>
</tbody>
</table>

(The highest STDs occurred at the highest $h/d$ values).

Wave steepness is expressed as $H gT^2$, and the relative depth is presented in the form of $d gT^2$. Since two dynamometers were used to measure wave forces on the two armour units simultaneously, the value of wave steepness is the same for each of the combination of the units placement, (see Section 3.7). The values of $H gT^2$ are presented in Tables 7.1 to 7.4, and the values of $d gT^2$ are given in Table 7.5:
### Tabel 7.1

$H/gT^2$ for the hydrodynamic forces on the cubical armour units located simultaneously at points A and H (see Fig. 7.1).

<table>
<thead>
<tr>
<th>h/d</th>
<th>T=2.2 s</th>
<th>T=1.9 s</th>
<th>T=1.5 s</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.73</td>
<td>0.00370</td>
<td>0.00507</td>
<td>0.00846</td>
</tr>
<tr>
<td>0.79</td>
<td>0.00390</td>
<td>0.00500</td>
<td>0.00786</td>
</tr>
<tr>
<td>0.85</td>
<td>0.00386</td>
<td>0.00507</td>
<td>0.00849</td>
</tr>
<tr>
<td>0.92</td>
<td>0.00387</td>
<td>0.00476</td>
<td>0.00761</td>
</tr>
<tr>
<td>1.00</td>
<td>0.00379</td>
<td>0.00512</td>
<td>0.00813</td>
</tr>
<tr>
<td>1.10</td>
<td>0.00395</td>
<td>0.00524</td>
<td>0.00810</td>
</tr>
<tr>
<td>1.22</td>
<td>0.00391</td>
<td>0.00501</td>
<td>0.00768</td>
</tr>
<tr>
<td>1.38</td>
<td>0.00402</td>
<td>0.00468</td>
<td>0.00702</td>
</tr>
</tbody>
</table>

### Tabel 7.2

$H/gT^2$ for the hydrodynamic forces on the cubical armour units located simultaneously at points B and G (see Fig. 7.1).

<table>
<thead>
<tr>
<th>h/d</th>
<th>T=2.2 s</th>
<th>T=1.9 s</th>
<th>T=1.5 s</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.73</td>
<td>0.00375</td>
<td>0.00504</td>
<td>0.00844</td>
</tr>
<tr>
<td>0.79</td>
<td>0.00388</td>
<td>0.00511</td>
<td>0.00812</td>
</tr>
<tr>
<td>0.85</td>
<td>0.00388</td>
<td>0.00495</td>
<td>0.00838</td>
</tr>
<tr>
<td>0.92</td>
<td>0.00389</td>
<td>0.00483</td>
<td>0.00774</td>
</tr>
<tr>
<td>1.00</td>
<td>0.00379</td>
<td>0.00508</td>
<td>0.00811</td>
</tr>
<tr>
<td>1.10</td>
<td>0.00393</td>
<td>0.00517</td>
<td>0.00798</td>
</tr>
<tr>
<td>1.22</td>
<td>0.00387</td>
<td>0.00503</td>
<td>0.00742</td>
</tr>
<tr>
<td>1.38</td>
<td>0.00393</td>
<td>0.00469</td>
<td>0.00627</td>
</tr>
</tbody>
</table>
\[
\frac{H}{gT^2}
\]

<table>
<thead>
<tr>
<th>h/d</th>
<th>T=2.2 s</th>
<th>T=1.9 s</th>
<th>T=1.5 s</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.73</td>
<td>0.00374</td>
<td>0.00445</td>
<td>0.00746</td>
</tr>
<tr>
<td>0.79</td>
<td>0.00386</td>
<td>0.00427</td>
<td>0.00678</td>
</tr>
<tr>
<td>0.85</td>
<td>0.00387</td>
<td>0.00498</td>
<td>0.00825</td>
</tr>
<tr>
<td>0.92</td>
<td>0.00379</td>
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<td>0.00758</td>
</tr>
<tr>
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<td>0.00405</td>
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<tr>
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<tr>
<td>1.22</td>
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<td>0.00493</td>
<td>0.00728</td>
</tr>
<tr>
<td>1.38</td>
<td>0.00379</td>
<td>0.00456</td>
<td>0.00639</td>
</tr>
</tbody>
</table>

**Table 7.4:** \(H/gT^2\) for the hydrodynamic forces on the cubical armour units located simultaneously at points C and F (see Fig. 7.1).

\[
\frac{H}{gT^2}
\]

<table>
<thead>
<tr>
<th>h/d</th>
<th>T=2.2 s</th>
<th>T=1.9 s</th>
<th>T=1.5 s</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.73</td>
<td>0.00364</td>
<td>0.00494</td>
<td>0.00848</td>
</tr>
<tr>
<td>0.79</td>
<td>0.00372</td>
<td>0.00467</td>
<td>0.00738</td>
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<tr>
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<td>0.00376</td>
<td>0.00461</td>
<td>0.00755</td>
</tr>
<tr>
<td>0.92</td>
<td>0.00376</td>
<td>0.00482</td>
<td>0.00745</td>
</tr>
<tr>
<td>1.00</td>
<td>0.00370</td>
<td>0.00505</td>
<td>0.00791</td>
</tr>
<tr>
<td>1.10</td>
<td>0.00392</td>
<td>0.00508</td>
<td>0.00797</td>
</tr>
<tr>
<td>1.22</td>
<td>0.00383</td>
<td>0.00497</td>
<td>0.00743</td>
</tr>
<tr>
<td>1.38</td>
<td>0.00382</td>
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<td>0.00649</td>
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</table>

125
\[
\frac{d}{gT^2}
\]

<table>
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<th>T=1.9 s</th>
<th>T=1.5 s</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.73</td>
<td>0.0079</td>
<td>0.0106</td>
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</tr>
<tr>
<td>0.79</td>
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<td>0.0099</td>
<td>0.0159</td>
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<tr>
<td>0.85</td>
<td>0.0068</td>
<td>0.0092</td>
<td>0.0147</td>
</tr>
<tr>
<td>0.92</td>
<td>0.0063</td>
<td>0.0085</td>
<td>0.0136</td>
</tr>
<tr>
<td>1.00</td>
<td>0.0058</td>
<td>0.0078</td>
<td>0.0125</td>
</tr>
<tr>
<td>1.10</td>
<td>0.0053</td>
<td>0.0071</td>
<td>0.0113</td>
</tr>
<tr>
<td>1.22</td>
<td>0.0047</td>
<td>0.0064</td>
<td>0.0102</td>
</tr>
<tr>
<td>1.38</td>
<td>0.0042</td>
<td>0.0056</td>
<td>0.0091</td>
</tr>
</tbody>
</table>

7.3 Horizontal Forward Force on the Seaward Slope

The maximum horizontal forward force field along the surface of the breakwater will be presented to show the effect of armour unit location on the horizontal force. The force is written in dimensionless form, and presented as a function of submergence (h/d) and wave period (T).

Figures 7.2 to 7.5 show the horizontal forward force on the armour units located at points E (leading edge), F, G, and H (leading toe) respectively (see also Fig. 7.1). The dimensionless forces at those points are given in Tables 7.6 to 7.10.
Dimensionless Force, \( \frac{F_s}{\gamma D_e^2 H} \)

<table>
<thead>
<tr>
<th>h/d</th>
<th>T=2.2 s</th>
<th>T=1.9 s</th>
<th>T=1.5 s</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.73</td>
<td>0.1263</td>
<td>0.1193</td>
<td>0.1377</td>
</tr>
<tr>
<td>0.79</td>
<td>0.1394</td>
<td>0.1623</td>
<td>0.1652</td>
</tr>
<tr>
<td>0.85</td>
<td>0.1531</td>
<td>0.2012</td>
<td>0.2143</td>
</tr>
<tr>
<td>0.92</td>
<td>0.1714</td>
<td>0.2319</td>
<td>0.2541</td>
</tr>
<tr>
<td>1.00</td>
<td>0.1802</td>
<td>0.2674</td>
<td>0.3145</td>
</tr>
<tr>
<td>1.10</td>
<td>0.2532</td>
<td>0.3163</td>
<td>0.3624</td>
</tr>
<tr>
<td>1.22</td>
<td>0.3446</td>
<td>0.5584</td>
<td>0.9336</td>
</tr>
<tr>
<td>1.38</td>
<td>0.5922</td>
<td>0.8374</td>
<td>0.7087</td>
</tr>
</tbody>
</table>

Table 7.7: Dimensionless force at point F

Dimensionless Force, \( \frac{F_s}{\gamma D_e^2 H} \)

<table>
<thead>
<tr>
<th>h/d</th>
<th>T=2.2 s</th>
<th>T=1.9 s</th>
<th>T=1.5 s</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.73</td>
<td>0.0821</td>
<td>0.0956</td>
<td>0.1042</td>
</tr>
<tr>
<td>0.79</td>
<td>0.0887</td>
<td>0.1243</td>
<td>0.1256</td>
</tr>
<tr>
<td>0.85</td>
<td>0.0905</td>
<td>0.1281</td>
<td>0.1305</td>
</tr>
<tr>
<td>0.92</td>
<td>0.0980</td>
<td>0.1392</td>
<td>0.1678</td>
</tr>
<tr>
<td>1.00</td>
<td>0.1267</td>
<td>0.1785</td>
<td>0.1826</td>
</tr>
<tr>
<td>1.10</td>
<td>0.1616</td>
<td>0.2316</td>
<td>0.2527</td>
</tr>
<tr>
<td>1.22</td>
<td>0.2441</td>
<td>0.3157</td>
<td>0.3098</td>
</tr>
<tr>
<td>1.38</td>
<td>0.2867</td>
<td>0.4059</td>
<td>0.4588</td>
</tr>
</tbody>
</table>
Table 7.9:  Dimensionless force at point H (leading toe)

<table>
<thead>
<tr>
<th>h/d</th>
<th>T=2.2 s</th>
<th>T=1.9 s</th>
<th>T=1.5 s</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.73</td>
<td>0.0671</td>
<td>0.0626</td>
<td>0.0740</td>
</tr>
<tr>
<td>0.79</td>
<td>0.0807</td>
<td>0.0831</td>
<td>0.0874</td>
</tr>
<tr>
<td>0.85</td>
<td>0.0710</td>
<td>0.1091</td>
<td>0.1092</td>
</tr>
<tr>
<td>0.92</td>
<td>0.0878</td>
<td>0.1254</td>
<td>0.1272</td>
</tr>
<tr>
<td>1.00</td>
<td>0.0908</td>
<td>0.1375</td>
<td>0.1426</td>
</tr>
<tr>
<td>1.10</td>
<td>0.1217</td>
<td>0.1490</td>
<td>0.1579</td>
</tr>
<tr>
<td>1.22</td>
<td>0.1288</td>
<td>0.1850</td>
<td>0.1969</td>
</tr>
<tr>
<td>1.38</td>
<td>0.1631</td>
<td>0.2517</td>
<td>0.2776</td>
</tr>
</tbody>
</table>
force on the armour unit located at the seaward slope produced by waves at $T = 1.5$ s is greater than the averaged maximum force produced by wave at $T = 1.9$ s. Waves at $T = 2.2$ s produced the smallest force in this study. Figure 7.2, however, shows an exception where at $h/d = 1.38$ the averaged maximum force of waves at $T = 1.9$ s ($F_z/\gamma D_s^2H = 0.8374$) is greater than that of waves at $T = 1.5$ s ($F_z/\gamma D_s^2H = 0.7087$), which is shown in Table 7.6. This probably caused by the breaker location of waves at $T = 1.5$ s which is further seaward than for the breaker at $T = 1.9$ s for $h/d = 1.38$. When the water depth is deeper, the breaker location moves closer to the crest resulting in the higher force, (see Fig. 7.2 an Table 7.6 at $h/d = 1.22$).

As the armour unit was located at the deeper locations, namely at points F, G, and H, the magnitude of the force was significantly reduced, (see Figures 7.3, 7.4, and 7.5, and compare Tables 7.6 to 7.9). This trend can also clearly be seen in Figures 7.6 to 7.8 in which the average maximum force at points E, F, G, and H (see definitions in Fig. 7.1) are compared. Figures 7.6 7.7 and 7.8 show the comparisons at $T = 2.2$ s, at $T = 1.9$ s, and at $T = 1.5$ s respectively. A sharp increase on the force occurs at the leading edge placement as shown in the first series in Figures 7.6 to 7.8. This sharp increase starts at $h/d = 1.22$ for waves at $T = 2.2$ s, and at $h/d = 1.1$ for $T = 1.9$ s and $T = 1.5$ s indicating that breaking for waves at $T = 1.9$ s and 1.5 s are more sensitive to submergence than wave at $T = 2.2$ s.

Figures 7.9 to 7.16 also show how the averaged maximum force decreased as the armour unit was placed on the seaward slope closer to the leading toe. The ordinate ($z/h$) in those figures
the toe to the location of the armour unit, and \( h \) is the height of the crest. The relationships of \( (F_x/\gamma D_e H) \) and \( (z/h) \) for \( (h/d) = 0.73, 0.79, 0.85, 0.92, 1.00, 1.10, 1.22, \) and 1.38 are presented in Figures 7.9 to 7.16 respectively. Figures 7.9 to 7.16, compared to Figures 7.6 to 7.8, can also be used to better interpolate the forces on the armour units located on the seaward slope.

The aim of the discussion in this chapter is to provide a more rational approach of the determination of stable weight of armour units, and to improve the economy of the design. However, the positive aspects in the present method of designing armour units must be maintained in the new approach. The positive aspects in the present method are practicality of the design, and safety factor for the stability of the armour units. A reasonable technique to achieve the aim of this study while still maintaining the positive values in the present method is by dividing the design into two parts. Firstly, the armour units placed from the midslope and above are designed based on the maximum hydrodynamic forces acting on the leading edge, and secondly, the armour units placed below the midslope are designed based on the hydrodynamic forces acting on the armour unit at the midslope.

Since the 2-D hydrodynamic force consists of horizontal and vertical components, the ratio of averaged maximum forces acting on the armour unit located at points G (seaward midslope) and E (leading edge) are needed for both horizontal and vertical components. These ratio are important to approximate the limit to which the design can be improved.
at point G (seaward midslope) vary are approximately from 20% to 60% of the averaged maximum forces on the armour unit located at E (leading edge) depending on the submergence, and the wave period. All of the percentages are given in Table 7.10; the overall average ratio is approximately 46%.

Table 7.10: Ratio of averaged maximum forces on the cubical armour unit placed at G (seaward midslope) to E (leading edge), in percent.

<table>
<thead>
<tr>
<th>h/d</th>
<th>T=2.2 s</th>
<th>T=1.9 s</th>
<th>T=1.5 s</th>
</tr>
</thead>
<tbody>
<tr>
<td>Submerged:</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>0.73</td>
<td>53.13</td>
<td>52.47</td>
<td>53.75</td>
</tr>
<tr>
<td>0.79</td>
<td>57.88</td>
<td>51.23</td>
<td>52.93</td>
</tr>
<tr>
<td>0.85</td>
<td>46.38</td>
<td>54.20</td>
<td>50.93</td>
</tr>
<tr>
<td>0.92</td>
<td>51.24</td>
<td>54.08</td>
<td>50.06</td>
</tr>
<tr>
<td>Average:</td>
<td>52.16</td>
<td>53.00</td>
<td>51.92</td>
</tr>
<tr>
<td>Low-crested:</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1.00</td>
<td>50.38</td>
<td>51.44</td>
<td>45.33</td>
</tr>
<tr>
<td>1.10</td>
<td>48.09</td>
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<td>43.57</td>
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<tr>
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<td>21.09</td>
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<tr>
<td>1.38</td>
<td>27.54</td>
<td>30.05</td>
<td>39.17</td>
</tr>
<tr>
<td>Average:</td>
<td>40.85</td>
<td>40.44</td>
<td>37.29</td>
</tr>
<tr>
<td>Overall:</td>
<td>Average:</td>
<td>46.50</td>
<td>46.72</td>
</tr>
</tbody>
</table>

Figures 7.6 to 7.16 show that the armour unit should be designed based on the greatest wave force which is produced by waves having T = 1.5 s in this model. If a breakwater is designed as a submerged structure, the size of the armour units located below the seaward midslope are
in a significantly more efficient design. For a low-crested structure, the saving can even be greater because the averaged maximum force at the seaward midslope is now approximately 37% of the horizontal component at the leading edge, (see Table 7.10).

7.4 **Forward Horizontal Force on the Landward Slope**

In this section the forward horizontal force on the armour unit located at points A, B, C, and D (see Fig. 7.1) will be evaluated.

Figures 7.17 to 7.20 show the averaged maximum forward force on the cubical armour units located at points D (landward edge), C, B, and A (landward toe) respectively, whereas the dimensionless forces are presented in Tables 7.11 to 7.14.

Figure 7.17 shows the comparison of the averaged maximum force on the armour unit at point D (landward edge). While the forces at the leading edge clearly vary with wave periods for \( h/d \) \( \leq 0.92 \) (see Fig. 7.2 and Table 7.6), the forces at the landward edge caused by waves of \( T = 2.2 \) s, \( T = 1.9 \) s, and \( T = 1.5 \) s are relatively similar for the submerged case where \( h/d \) \( \leq 0.92 \), (see Fig. 7.17 and Table 7.11). As the water depth decreases, beginning at \( h/d = 0.92 \) there is a rapid increase of force starting with the wave at \( T = 1.5 \) s, then followed by waves at \( T = 1.9 \) s, and \( T = 2.2 \) s, (see Fig. 7.17).
Dimensionless Force, \( \frac{F_x}{\gamma D_e^2 H} \)

<table>
<thead>
<tr>
<th>h/d</th>
<th>T=2.2 s</th>
<th>T=1.9 s</th>
<th>T=1.5 s</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.73</td>
<td>0.1156</td>
<td>0.1043</td>
<td>0.1145</td>
</tr>
<tr>
<td>0.79</td>
<td>0.1321</td>
<td>0.1108</td>
<td>0.1242</td>
</tr>
<tr>
<td>0.85</td>
<td>0.1442</td>
<td>0.1204</td>
<td>0.1226</td>
</tr>
<tr>
<td>0.92</td>
<td>0.1367</td>
<td>0.1263</td>
<td>0.1484</td>
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<td>0.2782</td>
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<tr>
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<td>0.3868</td>
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<tr>
<td>1.22</td>
<td>0.3778</td>
<td>0.4543</td>
<td>0.3840</td>
</tr>
<tr>
<td>1.38</td>
<td>0.3718</td>
<td>0.2978</td>
<td>0.2635</td>
</tr>
</tbody>
</table>

Table 7.12: Dimensionless forward horizontal force at point C

Dimensionless Force, \( \frac{F_x}{\gamma D_e^2 H} \)

<table>
<thead>
<tr>
<th>h/d</th>
<th>T=2.2 s</th>
<th>T=1.9 s</th>
<th>T=1.5 s</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.73</td>
<td>0.0579</td>
<td>0.0534</td>
<td>0.0606</td>
</tr>
<tr>
<td>0.79</td>
<td>0.0650</td>
<td>0.0469</td>
<td>0.0548</td>
</tr>
<tr>
<td>0.85</td>
<td>0.0626</td>
<td>0.0530</td>
<td>0.0524</td>
</tr>
<tr>
<td>0.92</td>
<td>0.0649</td>
<td>0.0500</td>
<td>0.0498</td>
</tr>
<tr>
<td>1.00</td>
<td>0.0669</td>
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<td>0.0699</td>
</tr>
<tr>
<td>1.10</td>
<td>0.0749</td>
<td>0.0864</td>
<td>0.0877</td>
</tr>
<tr>
<td>1.22</td>
<td>0.1015</td>
<td>0.1434</td>
<td>0.1271</td>
</tr>
<tr>
<td>1.38</td>
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<td>0.1687</td>
<td>0.1638</td>
</tr>
</tbody>
</table>
Dimensionless Force, \( \frac{F_x}{\gamma D_e^2 H} \)

<table>
<thead>
<tr>
<th>h/d</th>
<th>( T=2.2 ) s</th>
<th>( T=1.9 ) s</th>
<th>( T=1.5 ) s</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.73</td>
<td>0.0414</td>
<td>0.0441</td>
<td>0.0397</td>
</tr>
<tr>
<td>0.79</td>
<td>0.0449</td>
<td>0.0450</td>
<td>0.0416</td>
</tr>
<tr>
<td>0.85</td>
<td>0.0425</td>
<td>0.0423</td>
<td>0.0368</td>
</tr>
<tr>
<td>0.92</td>
<td>0.0381</td>
<td>0.0339</td>
<td>0.0294</td>
</tr>
<tr>
<td>1.00</td>
<td>0.0401</td>
<td>0.0336</td>
<td>0.0295</td>
</tr>
<tr>
<td>1.10</td>
<td>0.0448</td>
<td>0.0357</td>
<td>0.0304</td>
</tr>
<tr>
<td>1.22</td>
<td>0.0495</td>
<td>0.0507</td>
<td>0.0385</td>
</tr>
<tr>
<td>1.38</td>
<td>0.0489</td>
<td>0.0453</td>
<td>0.0398</td>
</tr>
</tbody>
</table>

Table 7.14: Dimensionless forward force at point A (landward toe)

Dimensionless Force, \( \frac{F_x}{\gamma D_e^2 H} \)

<table>
<thead>
<tr>
<th>h/d</th>
<th>( T=2.2 ) s</th>
<th>( T=1.9 ) s</th>
<th>( T=1.5 ) s</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.73</td>
<td>0.0330</td>
<td>0.0369</td>
<td>0.0288</td>
</tr>
<tr>
<td>0.79</td>
<td>0.0333</td>
<td>0.0400</td>
<td>0.0318</td>
</tr>
<tr>
<td>0.85</td>
<td>0.0332</td>
<td>0.0357</td>
<td>0.0297</td>
</tr>
<tr>
<td>0.92</td>
<td>0.0329</td>
<td>0.0371</td>
<td>0.0276</td>
</tr>
<tr>
<td>1.00</td>
<td>0.0329</td>
<td>0.0323</td>
<td>0.0251</td>
</tr>
<tr>
<td>1.10</td>
<td>0.0325</td>
<td>0.0266</td>
<td>0.0209</td>
</tr>
<tr>
<td>1.22</td>
<td>0.0336</td>
<td>0.0221</td>
<td>0.0198</td>
</tr>
<tr>
<td>1.38</td>
<td>0.0236</td>
<td>0.0194</td>
<td>0.0159</td>
</tr>
</tbody>
</table>

134
affected by the location of plunging. The location of plunging for the submerged case where \((h/d)\) \(\leq 0.92\) in this study is behind the landward edge which results in small forces. As the water depth decreases, the location of breaking moves upward (seaward) resulting in the location of plunging becoming closer to the landward edge, which increases the force on the armour unit at the landward edge. Forces due to waves with shorter length respond more sensitively to the water depth than those due to the longer waves as indicated in Fig. 7.17. As the water depth decreases further, the location of plunging moves further seaward of the leading edge resulting in the smaller force on the landward edge armour units; at the same time, the force on the leading edge armour unit starts to increase rapidly (compare Figures 7.2 and 7.17). It can be seen from Figures 7.2 and 7.17 that the force on the landward armour unit starts to increase relatively more rapidly at the lower \((h/d)\) than the force at the leading edge armour unit.

Generally, especially for submerged structures, the magnitude of the averaged maximum force on the leading edge armour unit is greater than the force on the landward edge armour unit, (see Figures 7.2 and 7.17). Although the location of plunging for submerged structures is closer to the landward edge than to the leading edge, it is behind the landward edge armour unit. Moreover, the pressure gradient at the leading edge armour unit is higher than that at the landward edge unit. This results in a greater force at leading edge than the force at the landward edge.

For low-crested structures, the breaking wave may plunge on the landward edge or leading edge armour unit depending on the water depth and wave characteristics. However, the pressure
leading edge armour units are exposed to wave attack, the landward edge armour units are
sheltered by the units located in front of them. As a result, the force at the landward edge armour
units may be greater than the force on the leading edge armour units only if the breaking wave
plunges directly on the landward armour units. In general, the force at the leading edge armour
unit is greater than the force at the landward edge unit due to its vulnerability to plunging, high
pressure gradient, and also hydrodynamic impact. Table 7.15 shows the comparison of forces on
armour units placed at both landward and leading edges.

Table 7.15: Ratio of averaged maximum forces on the cubical armour unit placed at D
(landward edge) to E (seaward edge), in percent.

<table>
<thead>
<tr>
<th>h/d</th>
<th>T=2.2 s</th>
<th>T=1.9 s</th>
<th>T=1.5 s</th>
</tr>
</thead>
<tbody>
<tr>
<td>Submerged:</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>0.73</td>
<td>91.49</td>
<td>87.41</td>
<td>83.13</td>
</tr>
<tr>
<td>0.79</td>
<td>94.76</td>
<td>68.27</td>
<td>75.18</td>
</tr>
<tr>
<td>0.85</td>
<td>94.22</td>
<td>59.87</td>
<td>57.21</td>
</tr>
<tr>
<td>0.92</td>
<td>79.78</td>
<td>54.49</td>
<td>58.39</td>
</tr>
<tr>
<td>Average:</td>
<td>90.06</td>
<td>67.51</td>
<td>68.48</td>
</tr>
<tr>
<td>Low-crested:</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1.00</td>
<td>86.49</td>
<td>66.93</td>
<td>88.45</td>
</tr>
<tr>
<td>1.10</td>
<td>85.03</td>
<td>107.39</td>
<td>106.74</td>
</tr>
<tr>
<td>1.22</td>
<td>109.63</td>
<td>81.36</td>
<td>41.13</td>
</tr>
<tr>
<td>1.38</td>
<td>62.78</td>
<td>35.57</td>
<td>37.18</td>
</tr>
<tr>
<td>Average:</td>
<td>85.98</td>
<td>72.81</td>
<td>68.38</td>
</tr>
<tr>
<td>Overall:</td>
<td>Average:</td>
<td>88.02</td>
<td>70.16</td>
</tr>
</tbody>
</table>
the force on the leading armour units at \((h/d) = 1.1\) for waves at \(T = 1.9\) s and \(T = 1.5\) s, and at \((h/d) = 1.22\) for waves at \(T = 2.2\) s. This is due to the direct impact of the plunging wave on the landward units.

As the cubical armour unit was located at the lower position on the landward slope, the force acting on it was reduced significantly. Figure 7.18 shows the averaged maximum force on the armour unit placed at point C (see Fig. 7.1). The force at this point is small for \((h/d) \leq 1\), (see also Table 7.12). As the water depth decreases, the force at this point increases as shown in Fig. 7.18 and Table 7.12). This trend can also be seen in Figures 7.21 to 7.23.

Figures 7.19 and 7.20 show the force on the armour unit at points B and A respectively (see Figure 7.1). Forces at those points are relatively very small compared to positions C and D as shown in Tables 7.13 and 7.9.

Figures 7.21 to 7.23 show the comparison of the averaged maximum forward horizontal forces for the armour units located at points D (landward edge), C, B, and A (landward toe) as defined in Fig. 7.1. It is shown in Figures 7.21 to 7.23 that the forces at points A and B are negligible compared to the forces on the armour units placed at points C and D.

Figures 7.24 to 7.31 show the envelope of averaged maximum forward horizontal force along the landward slope for \((h/d) = 0.73, 0.79, 0.85, 0.92, 1.00, 1.10, 1.22,\) and 1.38 respectively.
of forces at other locations with the force acting on the leading edge armour units is useful for the designer. Table 7.16 below shows the ratio between the force acting on the landward midslope armour units with the force acting on the leading edge armour units. It should be noted, however, that the ratios in Tables 7.15, 7.16, and also 7.10, are based on the force at the leading edge at the corresponding \((h/d)\) and \(T\).

Table 7.16: Ratio of averaged maximum forces on the cubical armour unit placed at B (landward midslope) to E (seaward edge), in percent.

<table>
<thead>
<tr>
<th>h/d</th>
<th>(T=2.2) s</th>
<th>(T=1.9) s</th>
<th>(T=1.5) s</th>
</tr>
</thead>
<tbody>
<tr>
<td>Submerged:</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>0.73</td>
<td>32.76</td>
<td>36.93</td>
<td>28.82</td>
</tr>
<tr>
<td>0.79</td>
<td>32.25</td>
<td>27.75</td>
<td>25.19</td>
</tr>
<tr>
<td>0.85</td>
<td>27.74</td>
<td>21.00</td>
<td>17.17</td>
</tr>
<tr>
<td>0.92</td>
<td>22.24</td>
<td>14.61</td>
<td>11.56</td>
</tr>
<tr>
<td>Average:</td>
<td>28.75</td>
<td>25.07</td>
<td>20.69</td>
</tr>
<tr>
<td>Low-crested:</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1.00</td>
<td>22.26</td>
<td>12.56</td>
<td>9.38</td>
</tr>
<tr>
<td>1.10</td>
<td>17.70</td>
<td>11.28</td>
<td>8.40</td>
</tr>
<tr>
<td>1.22</td>
<td>14.36</td>
<td>9.07</td>
<td>4.12</td>
</tr>
<tr>
<td>1.38</td>
<td>8.26</td>
<td>5.41</td>
<td>5.62</td>
</tr>
<tr>
<td>Average:</td>
<td>15.65</td>
<td>9.58</td>
<td>6.88</td>
</tr>
<tr>
<td>Overall:</td>
<td>Average: 22.20</td>
<td>17.33</td>
<td>13.78</td>
</tr>
</tbody>
</table>

Table 7.16 shows that the forward horizontal force on the landward midslope armour units are at a maximum about 37% for a submerged structure, and 23% for low-crested structure of the
7.5 Backward Horizontal Force on the Seaward Slope

The stability of armour units on the seaward slope must be evaluated against failure during wave downrushing. The most critical location of the armour unit during downrushing will be evaluated in this section.

Figure 7.32 shows the averaged maximum backward force on the leading edge armour unit (point E in Fig. 7.1). The maximum backward force is produced by the wave at $T = 2.2$ s for the submerged case where $(h'd) = 0.85$. The trend of the forward force produced by the wave at $T = 2.2$ s increases with $(h'd)$, but the trend of the backward force, relatively, starts to decrease for $(h'd)$ at 0.85 as shown in Fig. 7.33. The rapid increasing trend of the backward force at $(h'd) \leq 0.85$ shown in Fig. 7.32 is caused by combination of the flow during backwash and the elevation of the armour unit relative to the still water level. If the still water level is relatively high compared to the crest of the breakwater, most of the energy can pass over the crest. As the water depth decreases, the armour units on the crest produce more obstruction to the flow of the energy. During backwashing, the velocity of the fluid around the leading edge is higher than the velocity around the landward edge due to the rapid drawdown effect at the leading edge. This results in a greater backward force at the leading edge armour units compared to the force at the landward edge armour units. Figure 7.34 shows the correlation between the backward forces at the two edges. As the water level decreases, or $(h/d)$ increases, the energy blockage caused by
smaller force on armour units located at both edges. The backward force on the leading edge armour unit, however, is reduced more rapidly than the backward force on the landward edge because more energy loss occurs as the fluid travels along the crest. When the water level decreases even further, the fluid backflow velocity becomes smaller resulting in the smaller backward force on the leading edge armour unit than the force on the landward armour units, (see Fig. 7.34). This backflow may also occur in the flume because of the ponding effect behind the breakwater. The backward force at the landward edge starts to decrease rapidly when the water level behind the breakwater becomes shallower. The more the water depth decreases, the more the leading edge armour units experience plunging which causes a strong forward force, (see Fig. 7.33), but a smaller amount of fluid is now able to flow over the crest. The forward flow also occurs through the pore between rocks of the breakwater. The increase of the backward force at \((h/d) \geq 1.22\) may also caused by the reduction of resistance to fluid motion due to decrease in the water depth.

The backward force envelopes on the leading (seaward) edge armour units are also influenced by wave period as shown in Fig. 7.32. Figures 7.35 to 7.37 show the averaged maximum force on the cubical armour units located at points F, G, and H (leading toe) respectively, (see also Fig. 7.1). From Figures 7.35 to 7.37 it can be seen that the armour unit located at point F experienced nearly the same magnitude of backward force as the leading edge armour unit, (compare Figures 7.32 and 7.35). The nondimensional backward forces on the armour units at points E (seaward or leading edge), F, G, and H (seaward toe) are presented in Tables 7.17 to 7.20.
Dimensionless Force, \( \frac{F_x}{\gamma D_e^2 H} \)

<table>
<thead>
<tr>
<th>h/d</th>
<th>T=2.2 s</th>
<th>T=1.9 s</th>
<th>T=1.5 s</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.73</td>
<td>0.1035</td>
<td>0.0807</td>
<td>0.0670</td>
</tr>
<tr>
<td>0.79</td>
<td>0.1317</td>
<td>0.0991</td>
<td>0.0806</td>
</tr>
<tr>
<td>0.85</td>
<td>0.1398</td>
<td>0.0935</td>
<td>0.0815</td>
</tr>
<tr>
<td>0.92</td>
<td>0.1188</td>
<td>0.0743</td>
<td>0.0654</td>
</tr>
<tr>
<td>1.00</td>
<td>0.0714</td>
<td>0.0511</td>
<td>0.0582</td>
</tr>
<tr>
<td>1.10</td>
<td>0.0316</td>
<td>0.0370</td>
<td>0.0627</td>
</tr>
<tr>
<td>1.22</td>
<td>0.0321</td>
<td>0.0270</td>
<td>0.0684</td>
</tr>
<tr>
<td>1.38</td>
<td>0.0502</td>
<td>0.0977</td>
<td>0.0252</td>
</tr>
</tbody>
</table>

Table 7.18: Dimensionless backward horizontal force at point F

Dimensionless Force, \( \frac{F_x}{\gamma D_e^2 H} \)

<table>
<thead>
<tr>
<th>h/d</th>
<th>T=2.2 s</th>
<th>T=1.9 s</th>
<th>T=1.5 s</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.73</td>
<td>0.0596</td>
<td>0.0469</td>
<td>0.0558</td>
</tr>
<tr>
<td>0.79</td>
<td>0.0824</td>
<td>0.0723</td>
<td>0.0673</td>
</tr>
<tr>
<td>0.85</td>
<td>0.0821</td>
<td>0.0725</td>
<td>0.0806</td>
</tr>
<tr>
<td>0.92</td>
<td>0.0968</td>
<td>0.0850</td>
<td>0.0950</td>
</tr>
<tr>
<td>1.00</td>
<td>0.0527</td>
<td>0.0807</td>
<td>0.0884</td>
</tr>
<tr>
<td>1.10</td>
<td>0.0508</td>
<td>0.0602</td>
<td>0.0692</td>
</tr>
<tr>
<td>1.22</td>
<td>0.0642</td>
<td>0.0627</td>
<td>0.0871</td>
</tr>
<tr>
<td>1.38</td>
<td>0.0657</td>
<td>0.1263</td>
<td>0.1404</td>
</tr>
</tbody>
</table>
Dimensionless Force, $\frac{F_x}{\gamma D_e^2 H}$

<table>
<thead>
<tr>
<th>h/d</th>
<th>T=2.2 s</th>
<th>T=1.9 s</th>
<th>T=1.5 s</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.73</td>
<td>0.0451</td>
<td>0.0356</td>
<td>0.0515</td>
</tr>
<tr>
<td>0.79</td>
<td>0.0422</td>
<td>0.0370</td>
<td>0.0529</td>
</tr>
<tr>
<td>0.85</td>
<td>0.0442</td>
<td>0.0460</td>
<td>0.0583</td>
</tr>
<tr>
<td>0.92</td>
<td>0.0419</td>
<td>0.0527</td>
<td>0.0640</td>
</tr>
<tr>
<td>1.00</td>
<td>0.0477</td>
<td>0.0534</td>
<td>0.0611</td>
</tr>
<tr>
<td>1.10</td>
<td>0.0497</td>
<td>0.0520</td>
<td>0.0599</td>
</tr>
<tr>
<td>1.22</td>
<td>0.0557</td>
<td>0.0646</td>
<td>0.0683</td>
</tr>
<tr>
<td>1.38</td>
<td>0.0600</td>
<td>0.0620</td>
<td>0.0851</td>
</tr>
</tbody>
</table>

Table 7.20: Dimensionless backward horizontal force at point H (leading toe)

Dimensionless Force, $\frac{F_x}{\gamma D_e^2 H}$

<table>
<thead>
<tr>
<th>h/d</th>
<th>T=2.2 s</th>
<th>T=1.9 s</th>
<th>T=1.5 s</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.73</td>
<td>0.0509</td>
<td>0.0387</td>
<td>0.0628</td>
</tr>
<tr>
<td>0.79</td>
<td>0.0536</td>
<td>0.0493</td>
<td>0.0641</td>
</tr>
<tr>
<td>0.85</td>
<td>0.0566</td>
<td>0.0605</td>
<td>0.0666</td>
</tr>
<tr>
<td>0.92</td>
<td>0.0631</td>
<td>0.0613</td>
<td>0.0640</td>
</tr>
<tr>
<td>1.00</td>
<td>0.0614</td>
<td>0.0577</td>
<td>0.0614</td>
</tr>
<tr>
<td>1.10</td>
<td>0.0606</td>
<td>0.0603</td>
<td>0.0652</td>
</tr>
<tr>
<td>1.22</td>
<td>0.0652</td>
<td>0.0626</td>
<td>0.0704</td>
</tr>
<tr>
<td>1.38</td>
<td>0.0650</td>
<td>0.0668</td>
<td>0.0802</td>
</tr>
</tbody>
</table>
0.1404) occur at point F caused by waves of \( T = 1.9 \) s and \( T = 1.5 \) s respectively at \((h/d) = 1.38\). At this stage the water depth is \( d = 200 \) mm, or the still water level is slightly below the centroid of the armour unit at point F which is at 206 mm above the flume bottom. With that condition there is very little fluid pressure developed at the front surface of the armour unit during backwashing. In the mean time a pressure gradient is developed at the back surface of the armour unit tending to push the unit seaward on the slope, resulting in a relatively strong backward (seaward) force as shown in Fig. 7.35.

Figure 7.35 shows the backward horizontal forces on the armour unit at point F caused by waves of \( T = 2.2 \) s, \( T = 1.9 \) s, and \( T = 1.5 \) s. The armour unit at point F experienced about the same magnitude backward horizontal force as that at the leading edge, (see Figures 7.32 and 7.35). The dimensionless force on the armour unit at point F is 0.1404, whereas the force at point E (leading edge) is 0.1398. However, the averaged maximum backward horizontal force at the leading edge occurred at \((h/d) = 0.85\) (submerged case) and \( T = 2.2 \) s, while the averaged maximum force at point F took place at \((h/d) = 1.38\) (low-crested case) and \( T = 1.5 \) s. Also, the orientation of the armour unit at the leading edge was \( \beta = 0^\circ \) while at F it was \( \beta = 26.57^\circ \).

Figures 7.36 and 7.37 show the averaged backward forces on the armour units located at point G (seaward midslope) and H (seaward toe). The magnitude of backward forces at points G and H are very small as shown in Figures 7.36 and 7.37, and Tables 7.19 and 7.20.
on the armour units placed at points E (leading edge), F, G (seaward midslope), and H (seaward toe) for $T = 2.2$ s, $T = 1.9$ s, and $T = 1.5$ s respectively. From Figures 7.38 to 7.40 it can be seen that the backward horizontal force on the armour unit at the toe is greater than the force at the middle of the seaward toe, even at high submergence $h/d = 0.73$, (see also Tables 7.19 and 7.20). This phenomenon is counter intuitive and difficult to explain. It should be noted that the structure in this study is very permeable which would result in relatively high internal flow. One possible reason is the orientation of the armour unit at the toe which was set at $\beta = 0^\circ$ and may have contribute some additional backward force on the seaward toe armour unit compared to an angle of $\beta = 26.57^\circ$ which was the orientation of the armour unit at the middle of the seaward slope. Although the averaged maximum backward horizontal forces at those points are relatively very small and should not cause instability of the armour units, backward movement of sediments at the seaward toe may take place if the backward force induced during backwashing is significant. Nevertheless, net sediment transport around the toe should tend to move forward since the maximum forward force which is greater than the backward force as shown in Fig. 7.41. Observe also that the simultaneous upward-forward forces are greater than the simultaneous downward-backward forces, indicating that forward movement of sediments is probably stronger than the backward movement, thus preventing erosion, which could harm to the stability of the breakwater. It should be noted that the erosion and deposition mechanics are very complex and the forces on the unit are only indicators of possible effect.

Figures 7.42 to 7.49 show the envelopes of the averaged maximum backward forces on the
and 1.38 respectively. It is observed that the location of maximum backward horizontal force changes from point E or the seaward edge (Figures 7.42, 7.43, 7.44, and 7.45) to point F (Figures 7.46, 7.47, 7.48, and 7.49) depending on the submergence and the wave period. As the water depth decreases, the location of maximum force gradually changes, from points E to F, first for $T = 1.5$ s (see Fig. 7.44 at $h/d = 0.85$), followed by $T = 1.9$ s (see Fig. 7.45 at $h/d = 0.92$), and then by $T = 2.2$ s (see Fig. 7.47 at $h/d = 1.1$).

In order to see the critical location of the armour unit on the seaward slope due to the averaged maximum backward horizontal force, several maximum forces (dimensionless) are ranked and tabulated as follows:

<table>
<thead>
<tr>
<th>Table 7.21</th>
<th>Backward Force Rank on the Seaward Slope</th>
</tr>
</thead>
<tbody>
<tr>
<td>$h/d$</td>
<td>Force</td>
</tr>
<tr>
<td>1.38</td>
<td>0.1404</td>
</tr>
<tr>
<td>0.85</td>
<td>0.1398</td>
</tr>
<tr>
<td>0.79</td>
<td>0.1317</td>
</tr>
<tr>
<td>1.38</td>
<td>0.1263</td>
</tr>
<tr>
<td>0.92</td>
<td>0.1188</td>
</tr>
<tr>
<td>0.73</td>
<td>0.1035</td>
</tr>
<tr>
<td>0.79</td>
<td>0.0991</td>
</tr>
<tr>
<td>1.38</td>
<td>0.0977</td>
</tr>
<tr>
<td>0.92</td>
<td>0.0968</td>
</tr>
<tr>
<td>0.92</td>
<td>0.0950</td>
</tr>
</tbody>
</table>

It can be seen that the armour unit at point E (leading edge) must be evaluated against a critical backward force for the submerged case ($h/d = 0.85 < 1$), and at point F (see Fig. 7.1) it must be evaluated against a critical backward force for the low-crested case ($h/d = 1.38 > 1$).
The magnitude of the averaged maximum backward horizontal forces on the armour unit located on the landward slope are generally small compared to the backward forces on the seaward slope. Due to their small magnitudes and direction, i.e. tending to push the armour units into the body of the breakwater, these forces do not cause instability to the armour units. The only location, on the landward slope for a narrow-crested structure, where armour units must be evaluated against backward force, is at the landward edge. A wide-crested structure provides a stronger resistance against crest failure due to wave forces than a narrow-crested structure.

Figures 7.50 to 7.53 show the backward horizontal force on the armour units located at points D (landward edge), C, B (landward midslope), and A (landward toe) respectively for various submergence and wave periods of $T = 2.2$ s, 1.9 s, and 1.5 s. From Fig. 7.50 it can be seen that the maximum dimensionless backward horizontal force is 0.0752 at $h/d = 1$ and $T = 1.5$ s. This magnitude is about 54% of the maximum backward horizontal force on the seaward tip which is caused by a wave of $T = 2.2$ s and $h/d = 0.85$. Having the smaller backward horizontal force and the stronger support than the armour unit at the seaward tip, the landward tip armour unit is more stable against a backward force than the leading edge unit.

Tables 7.22 to 7.25 show the magnitude of dimensionless backward force on the armour units located on the landward slope.
Dimensionless Force, $\frac{F_x}{\gamma D_e^2 H}$

<table>
<thead>
<tr>
<th>h/d</th>
<th>T=2.2 s</th>
<th>T=1.9 s</th>
<th>T=1.5 s</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.73</td>
<td>0.0631</td>
<td>0.0571</td>
<td>0.0589</td>
</tr>
<tr>
<td>0.79</td>
<td>0.0662</td>
<td>0.0555</td>
<td>0.0509</td>
</tr>
<tr>
<td>0.85</td>
<td>0.0608</td>
<td>0.0601</td>
<td>0.0526</td>
</tr>
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<td>0.0516</td>
</tr>
<tr>
<td>1.00</td>
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<td>0.0512</td>
<td>0.0752</td>
</tr>
<tr>
<td>1.10</td>
<td>0.0485</td>
<td>0.0519</td>
<td>0.0469</td>
</tr>
<tr>
<td>1.22</td>
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<td>0.0285</td>
<td>0.0164</td>
</tr>
<tr>
<td>1.38</td>
<td>0.0159</td>
<td>0.0230</td>
<td>0.0104</td>
</tr>
</tbody>
</table>

Table 7.23: Dimensionless backward horizontal force at point C

Dimensionless Force, $\frac{F_x}{\gamma D_e^2 H}$

<table>
<thead>
<tr>
<th>h/d</th>
<th>T=2.2 s</th>
<th>T=1.9 s</th>
<th>T=1.5 s</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.73</td>
<td>0.0352</td>
<td>0.0168</td>
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</tr>
<tr>
<td>0.79</td>
<td>0.0361</td>
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</tr>
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<td>0.85</td>
<td>0.0390</td>
<td>0.0233</td>
<td>0.0237</td>
</tr>
<tr>
<td>0.92</td>
<td>0.0400</td>
<td>0.0265</td>
<td>0.0231</td>
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<tr>
<td>1.38</td>
<td>0.0253</td>
<td>0.0226</td>
<td>0.0299</td>
</tr>
</tbody>
</table>

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Dimensionless Force, \( \frac{F_x}{\gamma D_e^2 H} \)

<table>
<thead>
<tr>
<th>h/d</th>
<th>T=2.2 s</th>
<th>T=1.9 s</th>
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<tbody>
<tr>
<td>0.73</td>
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<td>0.0261</td>
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<td>0.0211</td>
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<tr>
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<td>0.0314</td>
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<td>0.0238</td>
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<tr>
<td>1.22</td>
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<td>0.0279</td>
<td>0.0276</td>
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<tr>
<td>1.38</td>
<td>0.0195</td>
<td>0.0238</td>
<td>0.0194</td>
</tr>
</tbody>
</table>

Table 7.25: Dimensionless backward horizontal force at point A (landward toe)

Dimensionless Force, \( \frac{F_x}{\gamma D_e^2 H} \)

<table>
<thead>
<tr>
<th>h/d</th>
<th>T=2.2 s</th>
<th>T=1.9 s</th>
<th>T=1.5 s</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.73</td>
<td>0.0160</td>
<td>0.0245</td>
<td>0.0256</td>
</tr>
<tr>
<td>0.79</td>
<td>0.0136</td>
<td>0.0249</td>
<td>0.0270</td>
</tr>
<tr>
<td>0.85</td>
<td>0.0147</td>
<td>0.0188</td>
<td>0.0225</td>
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<tr>
<td>0.92</td>
<td>0.0169</td>
<td>0.0159</td>
<td>0.0179</td>
</tr>
<tr>
<td>1.00</td>
<td>0.0178</td>
<td>0.0149</td>
<td>0.0156</td>
</tr>
<tr>
<td>1.10</td>
<td>0.0163</td>
<td>0.0108</td>
<td>0.0158</td>
</tr>
<tr>
<td>1.22</td>
<td>0.0142</td>
<td>0.0136</td>
<td>0.0072</td>
</tr>
<tr>
<td>1.38</td>
<td>0.0101</td>
<td>0.0112</td>
<td>0.0075</td>
</tr>
</tbody>
</table>
edge) at \((h'd) \leq 1.1\) for \(T = 1.9\) s and \(T = 1.5\) s, and at \((h'd) \leq 1.22\) for \(T = 2.2\) s. As the water depth decreases, the backward horizontal force on the landward edge decreases as well. This trend is shown in Figures 7.54 to 7.56.

Figures 7.57 to 7.64 show the envelope of averaged maximum backward horizontal forces on the armour units located on the landward slope for \((h'd) = 0.73, 0.79, 0.85, 0.92, 1.00, 1.10, 1.22,\) and 1.38 respectively. It is observed that up to \((h'd) = 1.1\) the maximum backward horizontal force is at \((h'd) = 1.22\) (landward edge), and then at \((h'd) = 1.38\) the maximum backward horizontal force changes from point D to point C (see Fig. 7.1).

### 7.7 Upward Force on the Seaward Slope

Upward force affects the stability of armour units by counteracting the submerged weight of the armour units, so that the normal force at the contact with other units is reduced. The upward component of the vertical force tends to lift the units from the body of the breakwater, and is always important for armour unit stability regardless of the location of the armour units.

The peak of upward force on the unit does not always act simultaneously with the peak of forward horizontal force. However, the upward force peak becomes closer to the forward force peak as the water depth decreases or \((h'd)\) increases. This trend appears on the armour unit located at points E (leading edge), F, and G (seaward midslope), whereas at the seaward toe the
In order to show the trend mentioned above, simultaneous upward and forward forces on the armour unit located at each of points E, F, G, and H are plotted at three different submergences of \((h/d) = 0.85\) (submerged), 1.00 (transition), and 1.22 (low-crested).

Figures 7.65 to 7.67 show the simultaneous forces on the armour units located on the two edges of the crest at \((h/d) = 0.85\) (submerged), 1.00 (transition), and 1.22 (low-crested) respectively. Figure 7.65 shows that there is a time lag between the upward and forward force peaks. It can be observed that there are seven stages of upward force related to the simultaneous horizontal force experienced by the armour unit at the upstream edge at \((h/d) = 0.85\):

1. The armour unit starts to experience upward force while it is still experiencing backward horizontal force.
2. The upward force continues to increase, whereas at the same time the backward force is diminishing.
3. The upward force increases further towards its peak, and simultaneously the armour unit starts to experience forward horizontal force.
4. The upward force reaches its peak, while the forward force is still increasing.
5. The upward force starts to decrease, while the forward force continues to increase, until the two forces have the same magnitudes.
6. The upward force further decreases but the forward force continues increasing until it reaches its peak.

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As the water depth decreases, from \( h/d = 0.85 \), the peak of the upward force on the leading edge armour unit tends to act simultaneously with the peak of forward horizontal force as shown in Figures 7.66 \( h/d = 1 \) and 7.67 \( h/d = 1.22 \).

Figure 7.68 to 6.73 show the forces on the armour unit located at points F and G, (see Fig. 7.1). Since the armour unit orientation at points F and G is \( \beta = 26.57^\circ \) (i.e. parallel to the slope), two plots are presented in each of Figures 7.68 to 7.73 to show the tangential and normal forces, and the horizontal and vertical forces.

Figures 7.68a and 7.68b show how the peak of normal and upward forces on the armour unit at point F act slightly ahead of the peaks of the corresponding tangential and forward horizontal forces at \( h/d = 0.85 \). As the water depth decreases \( h/d = 1.00 \) and 1.22, the peak of normal and upward forces become closer to the corresponding tangential and forward forces, (see Figures 7.69 and 7.70). This trend also appears for forces on the armour unit located at point G, (see Figures 7.71 to 7.73).

Figures 7.74 to 7.76 show that the peaks of the upward and forward forces on the armour unit located at the seaward toe act nearly simultaneously at \( h/d = 0.85, 1.00, \) and 1.22.

It can be concluded that generally the peak of the upward force is nearly coincident with the peak
submerged case, the upward peak acts ahead of the forward peak. Since the maximum forward force occurs at the leading edge as discussed in Section 7.3, the low-crested case gives the worst possible instability to the armour unit compared to the submerged cases.

The magnitude of maximum upward force on the armour unit on the seaward slope depends on the submergence of the breakwater, unit location on the seaward slope, and wave period. Figures 7.77 to 7.80 show the upward forces on the armour units placed at points E (leading edge), F, G, and H (seaward toe) respectively, whereas Tables 7.26 to 7.29 show the magnitude of the averaged maximum dimensionless upward forces at the corresponding points.

It can be seen from Figures 7.77 to 7.80 that the upward force generally increases as the water depth decreases or \((h/d)\) increases. Also, generally the magnitude of the upward forces on the armour unit increases with the elevation of the unit on the seaward slope. This trend is better shown in Figures 7.81 to 7.83 for wave periods \(T = 2.2\) s, \(T = 1.9\) s, and \(T = 1.5\) s respectively. The maximum non-dimensional upward force (0.3223) occurred at the leading edge armour unit, and this force was induced by waves at \(T = 1.5\) s, and at \((h/d) = 1.22\), (see Fig. 7.83 and Table 7.26).

Figures 7.84 to 7.91 show the averaged maximum envelopes of upward force located on the seaward slope at \((h/d) = 0.73, 0.79, 0.85, 0.92, 1.00, 1.22, \) and 1.38 respectively. Each of the figures consists of the upward force envelope caused by waves of \(T = 2.2\) s, \(1.9\) s, and \(1.5\) s.
Table 7.27: Dimensionless upward force at point F

<table>
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<tr>
<th>h/d</th>
<th>T=2.2 s</th>
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<th>T=1.5 s</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.73</td>
<td>0.0606</td>
<td>0.0728</td>
<td>0.0820</td>
</tr>
<tr>
<td>0.79</td>
<td>0.0763</td>
<td>0.1095</td>
<td>0.1170</td>
</tr>
<tr>
<td>0.85</td>
<td>0.1102</td>
<td>0.1809</td>
<td>0.1725</td>
</tr>
<tr>
<td>0.92</td>
<td>0.1554</td>
<td>0.1920</td>
<td>0.2022</td>
</tr>
<tr>
<td>1.00</td>
<td>0.1418</td>
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<tr>
<td>1.10</td>
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<td>0.2476</td>
<td>0.2528</td>
</tr>
<tr>
<td>1.22</td>
<td>0.2331</td>
<td>0.3024</td>
<td>0.3223</td>
</tr>
<tr>
<td>1.38</td>
<td>0.2563</td>
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</table>
### Dimensionless Force, $\frac{F_y}{\gamma D_e^2 H}$

<table>
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<tr>
<th>h/d</th>
<th>T=2.2 s</th>
<th>T=1.9 s</th>
<th>T=1.5 s</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.73</td>
<td>0.0371</td>
<td>0.0380</td>
<td>0.0472</td>
</tr>
<tr>
<td>0.79</td>
<td>0.0579</td>
<td>0.0593</td>
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<tr>
<td>0.85</td>
<td>0.0565</td>
<td>0.0911</td>
<td>0.0899</td>
</tr>
<tr>
<td>0.92</td>
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<td>0.1109</td>
<td>0.1041</td>
</tr>
<tr>
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<td>0.0795</td>
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<td>0.1170</td>
</tr>
<tr>
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<td>0.1293</td>
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<td>1.38</td>
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</table>

Table 7.29: Dimensionless upward force at point H (seaward toe)

### Dimensionless Force, $\frac{F_y}{\gamma D_e^2 H}$

<table>
<thead>
<tr>
<th>h/d</th>
<th>T=2.2 s</th>
<th>T=1.9 s</th>
<th>T=1.5 s</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.73</td>
<td>0.0105</td>
<td>0.0123</td>
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</tr>
<tr>
<td>0.79</td>
<td>0.0133</td>
<td>0.0137</td>
<td>0.0150</td>
</tr>
<tr>
<td>0.85</td>
<td>0.0160</td>
<td>0.0181</td>
<td>0.0156</td>
</tr>
<tr>
<td>0.92</td>
<td>0.0199</td>
<td>0.0210</td>
<td>0.0163</td>
</tr>
<tr>
<td>1.00</td>
<td>0.0225</td>
<td>0.0240</td>
<td>0.0172</td>
</tr>
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<td>1.10</td>
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<td>0.0283</td>
<td>0.0209</td>
</tr>
<tr>
<td>1.22</td>
<td>0.0305</td>
<td>0.0344</td>
<td>0.0241</td>
</tr>
<tr>
<td>1.38</td>
<td>0.0303</td>
<td>0.0394</td>
<td>0.0311</td>
</tr>
</tbody>
</table>
as the unit was located at a higher elevation on the slope from point H (z/h = 0) up to point F (at z/h = 0.75).

Figure 7.84 shows that at \((h/d) = 0.73\) the upward force envelope on the unit at points E and F is practically the same for forces induced by waves of the same wave period, (compare also Tables 7.26 and 7.27 at \(h/d = 0.73\)). As the water depth decreases to \((h/d) = 0.79\), the upward forces on the unit located at points E and F increase, (compare Figs. 7.84 and 7.85, also Tables 7.26 and 7.27). However, the upward force increase on the unit at point F for \(T = 2.2\) s and \(T = 1.9\) s is greater than that at point E, resulting in the greater upward force on the unit at point F than that at point E. For upward force induced by wave at \(T = 1.5\) s, the increase in the force at point E is greater than that at point F causing the force on the unit at point E to be greater than that at point F, (compare Figs. 7.84 and 7.85, also Tables 7.26 and 7.27). As the water depth further decreases to \((h/d) = 0.85\), the upward force on the unit at point E becomes greater than that at point F, (see Fig. 7.86).

Figure 7.87 shows the averaged maximum upward force envelopes on the units on the seaward slope at \((h/d) = 0.92\). As \((h/d)\) increases from 0.85 to 0.92, the upward forces on the unit at points E and F increase with the force on the leading edge armour unit being greater than that at point F (compare Figs. 7.86 and 7.87, also see Tables 7.26 and 7.27).

When \((h/d)\) increases to 1.0, however, the upward force on the unit at point F increases, while
Figures 7.81 and 7.83), causing the upward force on the unit at point F to be approximately the same as that at point E except for the forces induced by wave at $T = 1.9$ s. This phenomenon is difficult to explain because as $(h/d)$ increases further from 1.0 to 1.1, 1.22, and 1.38, the upward force on the leading edge unit (at point E) is greater than that at point F, (see Figures 7.89 to 7.91). Consider also that at $(h/d) = 1$, the upward forces induced by waves at $T = 2.2$ and 1.5 seconds show the same trend, whereas at $T = 1.9$ s which is in between the first two wave periods, shows a different trend as indicated in Fig. 7.88. Examination of the maximum upward force envelope (as compared to the averaged maximum upward force) indicates that it has the same trend at $0.92 \leq h/d \leq 1$ as shown in Fig. 7.89, (compare Figures 7.77 with 7.92a, and also Figures 7.78 with 7.92b). It can be seen in both Figures 7.77 and 7.92a that the upward force on the leading edge armour unit at $T = 2.2$ s and $T = 1.5$ s decreases as $(h/d)$ increases within the range $0.92 \leq h/d \leq 1$, whereas at $T = 1.9$ s it increases. The upward force on the unit at point F within $0.92 \leq h/d \leq 1$ increases at $T = 2.2$ s, $1.9$ s, and $1.5$ s as shown in Figures 7.78 and 7.92b.

7.8 **Upward Force on the Landward Slope**

The action of upward force on the armour unit located on the landward slope depends on the location of the unit, and the location of plunging which is influenced by the degrees of submergence. In order to show the action of upward force, simultaneous horizontal and vertical forces on the unit located at each of points D (landward edge), C, B (landward midslope), and
The simultaneous forces on the unit at point D (landward edge) are shown in Fig. 7.65 to 7.67 for \( (h/d) = 0.85 \) (submerged case), 1.00 (transition), and 1.22 (low-crested) respectively. Figure 7.65 shows that the upward force occurs in phase with the backward force, and then its magnitude starts to decrease as the horizontal force starts to change from backward to forward on the unit. It is shown in Fig. 7.65 that the upward force peak occurs approximately when there is only pure vertical force (at times where the horizontal force is zero). As the value of \( (h/d) \) increases, however, the upward peak is in phase with the forward force peak, (see Figures 7.66 and 7.67).

The simultaneous horizontal and vertical forces at point C (see Fig. 7.1) are shown in Figures 7.93 to 7.95 for \( (h/d) = 0.85, 1.00, \) and 1.22 respectively. Upward force on the unit at point C indicates that it is in phase with backward force as shown in Figures 7.93 and 7.94 for \( (h/d) = 0.85 \) and 1.00 respectively. The maximum upward force occurs as the horizontal force starts to change from backward to forward just as in the case where the unit is at the landward edge. Upward force on the unit at \( (h/d) = 1.22 \), however, is somewhat random as shown in Fig. 7.95, although in general the upward force is in phase with the backward force.

The simultaneous vertical and horizontal forces on the unit at point B (landward midslope) are shown in Figures 7.96 to 7.98 for \( (h/d) = 0.85, 1.00, \) and 1.22 respectively. Upward force on the unit at point B occurs during backflow, and in general it reaches its peak shortly after the
rapidly to zero, and it switches to a downward force thereafter as shown in Figures 7.96 to 7.98.

Figures 7.99 to 7.101 show the typical simultaneous horizontal and vertical forces on the unit at point A (landward toe) at \( (h/d) = 0.85, 1.00, \) and \( 1.22 \) respectively. Generally upward force occurs during the backflow force as in the case of the upward force on the unit at points D, C, and B. However, unlike at points D, C, and B, upward force on the unit at point A has a relatively long duration (flat) peak. The magnitude of the upward force increases slowly to the peak, then starts to decrease slowly as the backward force also starts to decrease, and finally it diminishes shortly after the backward force on the unit changes to be forward force, (see Figures 7.99 to 7.101).

The magnitude of the averaged maximum upward force on the armour unit located on the landward slope is smaller than that at the same elevation on the seaward slope. In general, upward force increases in phase with \( (h/d) \) except for the force on the unit located at the landward toe.

Figures 7.102 to 7.105 show the envelopes of averaged maximum upward dimensionless forces on the armour unit located at point D (landward edge), C, B, and A (landward toe) respectively, while the corresponding dimensionless magnitudes are presented in Tables 7.30 to 7.33. The maximum upward force is experienced by the unit at point D (landward edge), and then it decreases as the armour unit is located at lower elevations on the slope.
Dimensionless Force, \( \frac{F_y}{\gamma D_e^2 H} \)

<table>
<thead>
<tr>
<th>h/d</th>
<th>T=2.2 s</th>
<th>T=1.9 s</th>
<th>T=1.5 s</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.73</td>
<td>0.0474</td>
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<td>1.38</td>
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</tr>
</tbody>
</table>

Table 7.31: Dimensionless upward force at point C

Dimensionless Force, \( \frac{F_y}{\gamma D_e^2 H} \)

<table>
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<tr>
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<th>T=2.2 s</th>
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</tr>
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<td>0.85</td>
<td>0.0286</td>
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</tr>
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<td>0.0356</td>
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</tr>
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<tr>
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159
Dimensionless Force, \( \frac{F_y}{\gamma D_e^2 H} \)

<table>
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<th>T=1.5 s</th>
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</table>

Table 7.33: Dimensionless upward force at point A (landward toe)

Dimensionless Force, \( \frac{F_y}{\gamma D_e^2 H} \)

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<tr>
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<td>0.0052</td>
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<tr>
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<tr>
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<td>0.0042</td>
<td>0.0048</td>
<td>0.0037</td>
</tr>
</tbody>
</table>
landward slope is not sensitive to wave period.

The behaviour of averaged maximum upward force on the unit at point D, in general, can be divided into three regions. The first region is $0.73 \leq h/d \leq 0.92$ marked by an approximately horizontal line (constant force) indicating that the upward force is not sensitive to submergence, (see Fig. 7.102). The second region is $0.92 \leq h/d \leq 1.22$ marked by a rapid increase in the averaged maximum upward force, indicating that the force is sensitive to the submergence. The third region is $1.2 \leq h/d \leq 1.38$. The trend of the averaged maximum upward force in this range is sensitive to the wave period. As $(h/d)$ increases, the averaged maximum upward force decreases at $T = 2.2$ s, but it increases at $T = 1.9$ s, and it remains approximately the same at $T = 1.5$ s, (see Fig. 7.102).

This phenomenon, i.e. the maximum upward force on the unit induced by wave at $T = 1.9$ s increasing with $(h/d)$ while decreasing or constant at $T = 2.2$ s and $T = 1.5$ s, is similar to the phenomenon of upward force on the unit at the leading edge at $(h/d) = 1$, (see Fig. 7.88 and the discussion in Section 7.7). The difference between the two phenomena is that the upward force at the leading edge is in phase with the forward force, whereas at the landward edge it is in phase with the backward force. A closer investigation of the upward force, influenced by wave period around $T = 1.9$ s, is probably needed to find the maximum possible upward force. Consider also that the maximum upward force on the armour unit at the leading edge at $(h/d) = 1.38$ is wave force induced by wave at $T = 1.9$ s, (see Fig. 7.91).
that upward force increases as \((h/d)\) increases. It can be seen that upward force at point C is not sensitive to the wave period, except at \(T = 1.9\) s within the region \((h/d) \geq 1.1\). This also indicates that wave at \(T = 1.9\) s shows a different response than waves at \(T = 2.2\) s and \(T = 1.5\) s. However, the maximum upward force in point C is approximately 50% of the maximum upward force at the landward edge, and therefore it is not as critical for the unit stability at this point.

The upward force envelope on the unit at point B (landward midslope) is given in Fig. 7.104. Upward force at this point increases with \((h/d)\). Although the upward force at this point is very small, the wave at \(T = 1.9\) s \((h/d > 1.1)\) also shows an increase just as was noted at points D and C.

Figure 7.105 shows the upward force envelope on the unit at point A (landward toe). It is shown that the magnitude of upward force located at the landward toe is very small. Unlike the trend of upward force on the unit at points B, C, and D (landward edge), the upward force at point A decreases as \((h/d)\) increases.

The magnitude of the upward force on the unit at point A depends on the vertical distance between point A to the fluid surface, and the backflow. The maximum horizontal velocity of the fluid occurs close to the surface of the fluid. The greater upward force will be experienced by a unit if the maximum velocity occurs just above the unit's top surface. Since the unit at the landward toe is far from the maximum horizontal velocity, the upward force on the unit at the
upward force decreases if the backflow decreases as well. Under the submerged case \((h/d) < 1\), the backflow is greater than that in the low-crested case \((h/d) > 1\). This causes the upward force on the landward toe unit to decrease as \((h/d)\) increases.

The upward force on the unit located on the landward tip, on the other hand shows different response. The upward force is affected by flow curvature, and also flow contraction above the crest. As the water depth decreases, the armour unit exposure against the greater backward horizontal velocity increases, and coupled with vertical flow component from the landward slope, results in a greater force at a greater \((h/d)\).

Figures 7.106 to 7.108 show the comparison of upward force envelope on the unit located at points D, C, B, and A for \(T = 2.2\) s, \(T = 1.9\) s, and \(T = 1.5\) s respectively. It can be seen in Figures 7.106 to 7.108 that at \((h/d) \leq 0.92\) the upward force on the unit at any location on the landward slope is small. Figures 7.106 to 7.108 also show that the armour unit experiences a greater upward force at a higher elevation on the landward slope.

Figures 7.109 to 7.116 show the averaged maximum envelope of the upward force on the unit at points D \((z/h = 1)\), C \((z/h = 0.75)\), B \((z/h = 0.5)\), and A \((z/h = 0)\) at \((h/d) = 0.73, 0.79, 0.85, 0.92, 1.00, 1.10, 1.22,\) and \(1.38\) respectively. In general it can be seen that upward force is not sensitive to the wave period especially from point C \((z/h = 0.75)\) and below. Relatively, the greatest upward forces occurred on the unit at point D (landward edge) at \((h/d) = 1.10, 1.22,\) and
In general, since upward force on the landward slope is in phase with the backward force, whereas at the seaward slope it is in phase with the forward force, the upward force on the unit at the seaward slope is more critical for unit stability than that on the landward slope.

7.9 **Downward Force on the Seaward Slope**

Unlike the upward force which reduces the unit stability by counteracting the unit's submerged weight, the downward force increases the stability of the unit by adding downward vector to the submerged weight of the unit. Therefore, downward force is generally useful for the stability of armour unit. However, the downward force may also brings a detrimental effect to the breakwater by pushing it down which may cause settlement. This can be minimized by using flat slopes to provide the larger base area of the breakwater.

While the upward force on the seaward slope acts in phase with the forward horizontal force, the downward force in general acts in phase with the backward horizontal force during backwashing. This can be shown in Figures 7.65 to 7.67 for armour unit at the leading edge, Figures 7.68b, 7.69b, and 7.70b for the unit at point F, Figures 7.71b, 7.72b, and 7.73b for the unit at point G, and Figures 7.74 to 7.76 for the unit at point H (seaward toe). At \( (h/d) = 1.38 \), however, it is possible that plunging occurs directly on the leading edge armour unit. In this case, the
simultaneous horizontal and vertical forces on the leading edge armour unit at \((h/d) = 1.38\) will be investigated.

Figures 7.117 to 7.119 show the simultaneous forces on the unit at the leading edge at submergence \((h/d) = 1.38\) induced by waves at periods \(T = 2.2\) s, \(T = 1.9\) s, and \(T = 1.5\) seconds respectively. Figures 7.117 and 7.118 show that the downward force is in phase with the backward force. Figure 7.119, however, shows that the wave at \(T = 1.5\) s may produce simultaneous forward horizontal and downward forces. This indicates that plunging occurs directly on the leading edge armour unit.

The magnitude of the downward force on the unit caused by backflow is influenced by the flow of the fluid, the location of the unit in the flow, and damping effect by the fluid if the unit is submerged. The fluid backflow is greater at a smaller \((h/d)\), but the location of the unit in the flow is relatively far from the surface of the fluid. At a greater \((h/d)\), on the other hand, the flow becomes smaller, but the unit location in the flow is now relatively closer to the surface of the fluid. The closer the location of the unit to the fluid surface, the greater force will be experienced by the unit. The fluid at the sea side provides damping to reduce the downward force acting on the unit. This damping effect exists if the unit is submerged (partially or fully) in the seaward pond, and it vanishes if the unit is not submerged in the sea side. The more the unit is submerged, the greater the damping effect on the force experienced by the unit.
F, G (seaward midslope), and H (seaward toe) respectively. Figures 7.120 to 7.122 show that the downward force is very small at small \((h/d)\). A steep increase of downward force on the unit starts at approximately \((h/d) = 0.8\) for unit placement at point E, \((h/d) = 0.85\) for unit placement at point F, and \((h/d) = 0.92\) for unit placement at point G. The downward force on the unit at the higher elevation on the seaward slope responses more to an increase of \((h/d)\) than that at the lower elevation. The downward force on the unit at point H (seaward toe) is very small and it is not sensitive to \((h/d)\). It can also be observed from Figures 7.120 and 7.121 that the maximum downward force on the unit occurs if the SWL is approximately at the top point of the unit. It can be shown that the maximum downward force occurs at \((h/d) = 1.00\) for the unit placement at the leading edge, and at \((h/d) = 1.22\) for unit placement at point F.

Figure 7.120 shows the downward force on the leading edge unit. It can be seen that downward force increases with \((h/d)\) for the submerged case. As the water depth decreases the dynamic effect of the fluid on the unit becomes stronger causing a greater dynamic downward force on the unit. The maximum downward force on the leading edge unit occurs at \((h/d) = 1.00\).

As the water depth continues to decrease at \((h/d) < 1\), the backflow becomes smaller, resulting in the smaller downward force to the unit. Therefore, for the low-crested case, the magnitude of the downward force decreases as \((h/d)\) increases. It is shown in Fig. 7.120 that downward force produced by wave at \(T = 1.5\) s decreases from \(1 \leq (h/d) \leq 1.22\), and then it increases again at \((h/d) \geq 1.22\). As discussed previously, the downward force at \((h/d) \leq 1.22\) is in phase with backflow, whereas at \((h/d) = 1.38\) the downward force is caused by plunging, (see Fig. 7.119).
The elevation of point F is at 206.25 mm above the bottom of the flume, whereas the highest \((h/d)\) used in this study is 1.38 which is a water depth \(d = 200\) mm. The cubical unit at point F is exposed 23 mm above the SWL and 77.6 mm submerged under the SWL at \((h/d) = 1.38\). During rapid draw down due to wave rush down, however, the unit is more exposed to the air resulting in smaller damping by the fluid. At \((h/d) = 1.22\) the water depth is \(d = 225\) mm. Under this condition the armour unit is completely submerged under the SWL, and then during backwashing the unit is partially exposed to the air. At \((h/d) \leq 1.10\) the cubical armour unit at point F is always submerged. Figure 7.121 shows that the downward force on the unit at F is not sensitive to submergence at \((h/d) \leq 0.85\) and at \((h/d) \geq 1.22\), whereas at \(0.85 < (h/d) < 1.22\) the downward in sensitive to submergence.

Figure 1.122 shows downward force on the cubical unit at point G (seaward midslope). The unit at this location is always submerged at all \((h/d)\) selected in this study. The trend shows that downward force increases with \((h/d)\). Figure 1.123 shows the downward force on the unit at G at the seaward toe. The magnitude of maximum downward force on the unit at this point is practically negligible.

The averaged maximum nondimensional downward force on the cubical armour unit located at points E (leading edge), F, G (midslope), and H (toe) on the seaward slope is presented in Tables
is experienced by the unit located at points E and F, (compare Tables 7.34 to 7.37)

**Table 7.34:** Dimensionless downward force at point E (leading edge)

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**Table 7.35:** Dimensionless downward force at point F

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Table 7.37: Dimensionless downward force at point H (seaward toe)
unit located at points E (leading edge), F, G (seaward midslope), and H (seaward toe) produced by wave at $T = 2.2$ s, $T = 1.9$ s, and $T = 1.5$ s respectively. The absolute maximum downward force caused by backflow is experienced by the unit located at the leading edge at $(h/d) = 1$, (see Figures 7.124 and 7.125). However, the unit at point F also experiences only a slightly smaller downward force at $(h/d) = 1.22$ (see Fig. 7.124) or at $(h/d) = 1.38$ (see Fig. 7.125) than the maximum downward force on the unit at the leading edge at the corresponding wave period. For maximum downward force on the unit induced by backwashing wave at $T = 1.5$ s, the unit at point F experiences slightly greater magnitude than that at the leading edge, (see Fig. 7.126). At $(h/d) = 1.38$, however, the leading edge armour unit experienced plunging resulting in the greater downward force than that on the unit at point F caused by backwashing. Overall, the leading edge armour unit located on the seaward slope experiences the greatest downward force caused by plunging at $(h/d) = 1.38$, and at $T = 1.5$ s, as shown in Figures 7.120 and 7.126, and Table 7.34.

Figures 7.127 to 7.134 show the downward force on the unit located at points E, F, G, and H at $(h/d) = 0.73, 0.79, 0.85, 0.92, 1.00, 1.10, 1.22,$ and 1.38 respectively. Each figure comprises three plots for $T = 2.2$ s, $T = 1.9$ s, and $T = 1.5$ s. The downward force is sensitive to wave period at the smaller $(h/d)$ as shown in Figures 7.127 and 7.128, but as the water depth increases, it becomes insensitive to wave period, (see Figures 7.129 to 7.134).
In general, the maximum downward force on the unit located at the landward slope is in phase with the forward horizontal force. It is shown in Figures 7.65 to 7.66 for unit placement at the landward edge, and Figures 7.93 to 7.101 for unit placement at points C, B (landward midslope), and A (landward toe).

Figures 7.135 to 7.138 show the averaged maximum downward forces on the unit located at points D (landward edge), C, B (landward midslope), and A (landward toe); the nondimensional magnitudes are presented in Tables 7.38 to 7.41. Significant downward force is experienced by the model unit located at point D at \( (h/d) = 1.00 \) and \( T = 1.5 \) s, (see Fig. 7.135). Downward force on the unit at points C, B, and A is practically very small as shown in Figures 7.136 to 7.138.

The downward force on the unit at point D starts to decline at \( (h'd) \geq 1.00 \). This is caused by the smaller flow passes over the crest under at low-crested conditions compared to the submerged cases, (see Fig. 7.135).

Figures 7.139 to 7.141 compare the downward force envelopes on the unit located at points D, C, B, and A. It can be seen that relatively significant downward force occurs on the unit at point D and at \( (h'd) = 1.00 \). The absolute maximum downward force decreases as the unit is located at the lower elevation on the landward slope.
### Table 7.39: Dimensionless downward force at point C

<table>
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</tr>
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</table>

Dimensionless Force, \( \frac{F_y}{\gamma D_e^2 H} \)
### Table 7.41: Dimensionless downward force at point A (landward toe)

<table>
<thead>
<tr>
<th>h/d</th>
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<th>T=1.5 s</th>
</tr>
</thead>
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</table>
C, B (landward midslope), and A (landward toe) at $(h/d) = 0.73, 0.79, 0.85, 0.92, 1.00, 1.10, 1.22, \text{ and } 1.38 \text{ respectively. It can be seen in Figures 7.142 to 7.149 that downward force on the landward edge experiences the absolute maximum magnitude compared to the downward forces on the unit at points C, B, and A, (see Fig. 7.146 and Table 7.38). The magnitude of the downward force beginning from point C decreases rapidly as the unit is located at a lower elevation on the landward slope.}

**Summary:**

From discussions presented in sections 7.1 to 7.10 it was found that the leading edge is the most critical location of the cubical armour unit. The envelope of forces on the seaward slope were checked by selected intermediate reading close to point E between points E and F with the same orientation as the unit at point F. It was found that the forces on the leading edge unit for $0.73 \leq (h/d) \leq 1.38$ is greater than the forces on the intermediate location.

7.11 **Forces on the Two Cubical Units Arranged in Tandem**

Interlocking between armour units is an important consideration for the armour rock stability. Hudson (1958) introduced Stability Coefficient $K_D$ for determining the stable weight of an individual quarry stone armour unit. A comprehensive study was conducted by Hudson to provide the best selection of a Stability Coefficient using a statistical approach. One of the many variables affecting the Damage Coefficient $K_D$ is the interlocking between armour units which is discussed
Interlocking among armour units is important for the stability of cover layer because it provides a force distribution mechanism among the armour units. Hydrodynamic forces acting on an armour unit can be distributed to the rocks around the unit if there is interlocking among those rocks. Such interlocking is complex due to the irregular shapes which results in random contact points at the side surfaces between one rock and surrounding rocks. The hydrodynamic forces are distributed to the surrounding rocks through these contact points. Because of its complexity, it is difficult to quantify objectively force distribution due to interlocking. An objective study can only be done if the hydrodynamic forces acting on a group of rocks can be measured simultaneously without any actual contact among the rocks. Such a study, however, is not economical.

The potential interlocking benefit can be quantified by studying the simultaneous forces acting on two armour units arranged in tandem. This arrangement may represent the simplest form of interlocking between two armour units. Simultaneous hydrodynamic forces acting on the two units can be measured by arranging the two units as close as possible in tandem without actual contact between the two units. Beneficial force distribution between these two units occurs if the magnitude of force acting on the front unit is greater than the simultaneous force on the other unit during wave forward motion, and visa versa during wave backward motion.

In the previous sections in this chapter the critical location of the cubical unit at the leading edge
example of how force distribution exists among armour units. One cubical unit was placed at the leading edge, and the other cubical unit was positioned at about 1 cm behind the first unit.

Figure 7.150 shows the simultaneous horizontal forces acting on the leading edge unit and on the other unit behind it for the submerged case ($h/d > 1$). Fig. 7.150 shows that the dynamic force on the leading edge unit is greater than the simultaneous dynamic force on the second unit. It can also be observed that the peaks of the two forces do not occur simultaneously. The peak force on the leading edge unit occurs slightly ahead of the peak force on the other unit. This explains that if there is a contact point between the two units, force distribution occurs between the two units.

Figure 7.150 also shows that the backward peak force on the tandem unit is greater than the backward peak force on the leading edge unit. This also indicate that force distribution occurs between the unit during backwashing.

Figure 7.151 shows the simultaneous corresponding vertical forces on both armour units. Figure 7.151 indicates that the vertical peak force on the leading edge unit is greater than the vertical peak force on the other tandem unit. Since the 2-D movement of any particle in the wave action consists of horizontal and vertical motions, and horizontal force is stronger than the vertical force, vertical force is not as important as the horizontal force in force distribution.
unit behind it for the case \((h \cdot d \leq l)\). It is shown that the forward peak force on the leading edge unit occurs when the tandem unit is experiencing backward force. In this case the tandem unit is providing support to the leading edge unit, i.e. the backward force on the tandem unit is counteracting the forward force acting on the leading edge unit. During backwashing, however, there is no force distribution from the tandem unit to the leading edge unit, since as shown in Fig. 7.152 the backward force peak on the leading edge unit is greater than the backward force peak on the tandem unit. In this case, the leading edge unit tends to separate from the tandem unit during backwashing. Fortunately, the magnitude of the peak backward force on the tandem unit is very small as shown in Fig. 7.152, and therefore the tandem unit remains stable toward backward movement. The magnitude of the peak backward force on the leading edge unit is smaller (about 15\%) than the forward force. If interlocking is present between the leading edge unit and the other rock at the seaward slope, backward force is less harmful than the forward force to the stability of the leading edge unit. Figure 7.153 shows the simultaneous corresponding vertical force on the two cubical armour units for \((h/d) = 1\).

Figure 7.154 shows the simultaneous horizontal forces acting on the leading edge unit and on the tandem unit behind it for low-crested case \((h \cdot d < l)\). This case is similar to the transition case discussed above. However, the magnitude of the maximum forward force is significantly amplified even though the wave height indicated in Fig. 7.154 \((H=178 \text{ mm})\) is smaller than that in Fig. 7.152 where \(H=190 \text{ mm}\), (compare Fig. 7.152 and Fig. 7.154). In this situation, force distribution becomes very important for the stability of the leading edge unit. During
corresponding vertical forces on both cubical armour units for $(h/d) = 1.22$.

Figures 7.156 to 7.158 show the dimensionless forward forces on both armour units for various $(h/d)$ at wave periods $T = 2.2$ s, $T = 1.9$ s, and $T = 1.5$ s respectively. It can be seen in Figures 7.156 to 7.158 that the maximum forward force on the leading edge is always greater than the maximum forward force on the tandem unit indicating that force distribution occurs between the two units.

Figure 7.159 shows the comparison of Figures 7.156 to 7.158. It can be concluded in this comparison that force distribution becomes greater for the stronger forward force induced by the shorter waves. Figures 7.160 to 7.162 show the dimensionless upward forces on both armour units for various $(h/d)$ at wave periods $T = 2.2$ s, $T = 1.9$ s, and $T = 1.5$ s respectively; Fig. 7.163 shows the comparison of Figures 7.160 to 7.162. The upward force on the leading edge unit is greater than the upward force on the tandem unit for $(h/d) \leq 1.10$.

7.12 **Effect of Seaward Slope on the Force**

Effect of the slope of a breakwater on the force acting on an armour unit was also investigated in this study. Since forward and upward forces are relatively more important than the backward and downward forces for the stability of the armour unit, only the effect of slope on forward and upward forces will be presented in this section. Although the magnitude of downward force
submerged weight, and therefore increases the stability of the unit. On the other hand, the horizontal component of the plunging force is harmful to the stability of the unit.

The leading and landward edges were selected to locate the cubical units investigating the effect of slope on the forward and upward forces acting on both cubical units.

7.12.1 **Effect of Seaward Slope on the Forward Force on the Leading Edge Cubical Unit**

As discussed previously, the stability of a wave is sensitive to the water depth. The wave starts to be unstable and breaks if its depth approaches the breaker depth. If a wave starts to break, the fluid particle movement changes from elliptical orbits to be translatory. Breaking has been classified as spilling, plunging, and surging (Patrick and Wiegel, 1955). Galvin proposed another subdivision of breaker type referred to as collapsing (Galvin, 1968). In this experimental study, the type of breaker was plunging. A plunging breaker curls over at the crest with a plunging forward of the mass of water at the crest (SPM, 1984). There is a distance between the locations of the breaker and the plunging; this distance is called *the breaker travel distance* which was discussed in Section 5.2. The maximum magnitude of forward hydrodynamic force on the leading edge armour unit is affected by the location of plunging.

There are two interrelated factors affecting the locations of breaker and plunging. The first factor is water depth as mentioned in the above paragraph. The second factor is wave steepness (H/L).
the steepness for transitional and shallow water waves is given by Eq. 5.12. Once the wave steepness approaches its limit, the wave becomes unstable and starts to break. The two factors, however, are modified by the slope of the breakwater. As the wave propagates on the slope of the breakwater, the depth decreases gradually causing shoaling, and in the shoaling process the steepness of the wave increases. Thus the slope is a critical factor affecting the locations of breaker and plunging.

The effect of slope on the location of the breaker of low-crested breakwaters is different than that of the submerged breakwaters. If an incoming wave moves on the slope at a horizontal distance of $dx$ from the seaward toe, the depth decreases by $dy$. In the low-crested case, the effective depth at that point decreases from $d_1 < h$ to be $(d_1 - dy)$. In the submerged case, the depth also decreases namely from $d_2 > h$ to be $(d_2 - dy)$. Since $d_1 < d_2$ therefore $(d_1 - dy)/d_1 > (d_2 - dy)/d_2$ or $dy/d_1 > dy/d_2$. This shows that the change of depth in the low-crested condition is more sensitive to the breaker than that in the submerged case. Consequently, the force on the leading edge unit of a low-crested breakwater is more affected by the slope than that of the submerged breakwaters.

As discussed above, slope plays an important role affecting the magnitude of the force acting on the leading edge armour unit by forcing a wave travelling on it to break. A steep slope produces a more abrupt change of water depth above the slope than a flat slope. However, a steep slope has a shorter horizontal distance between the leading edge and the toe than that of the flat slope.
because of shoaling effect after its height reaches the breaker height $H_b$ at a certain breaker depth $d_b$. The location of the breaker if the wave propagates on a steep slope will be closer to the leading edge than that if the wave travels on a less steep slope. The breaker travel distance, however, will be shorter if the wave moves on the steep slope than that if the wave moves on a less steep slope, (see Eq. 5.1 in Section 5.2). The closer the location of plunging is to the leading edge the greater will be the forward force on the leading edge unit.

Wave force on the leading edge armour unit, based on the location of plunging, can be classified into three, i.e. plunging, breaking wave, and broken wave forces. Plunging force occurs when the plunging location is at the armour unit. A breaking wave affects the unit if the location of the plunging is close to the armour unit. A broken wave affects the unit if the location of the plunging is far in front of the armour unit. Among the three classifications, plunging force is the greatest, and broken wave force is the smallest. Therefore, if a wave breaks in front of the leading edge and it plunges at a distance behind the edge, the leading edge unit experiences a breaking wave force. On the other hand, if the same wave travels on a flatter slope, the wave will break at a point farther in front of the leading edge. If the location of plunging is relatively far in front of the leading edge, the leading edge unit will experience a broken wave.

For the submerged breakwaters with the front slope 1:2, the wave starts to break closer to the leading edge than those with 1:3 or 1:4 slopes; however, the 1:2 slope causes the location of plunging to occur at a farther point behind the leading edge than with 1:3 or 1:4 slopes. As a
For the low-crested case, the 1:2 slope causes the wave to break farther from the leading edge than that of the submerged case with the same slope. However, the location of the plunging becomes closer to the leading edge than that of the submerged case resulting in a greater forward force on the leading edge armour unit. The 1:3 or 1:4 slopes (low-crested case) cause the wave to break farther in front of the leading edge than that of 1:2 slope. The locations of the plunging caused by the 1:3 or 1:4 slopes (low-crested case) are also farther in front of the leading edge than that of the 1:2 slope (low-crested case). In this case, the 1:3 or 1:4 slopes of the low-crested case may produce broken wave forces which result in a smaller force than that produced by the 1:2 slope (low-crested case).

Figures 7.164 to 7.166 show the effect of slope on the forward force on the leading edge unit at wave periods of $T = 2.2\ s$, $T = 1.9\ s$, and $T = 1.5\ s$ respectively. Figures 7.164 to 7.166 show that forward forces on the leading edge unit at the breakwaters with slopes of 1:3 and 1:4 are greater than that at a slope of 1:2 for $(h/d) \leq 1.10$. This indicates that at $(h/d) \leq 1.10$ the breakwater slopes 1:3 and 1:4 cause the incoming wave to plunge at a point closer to the leading edge armour unit than if the wave travels on slope of 1:2, (see Figures 7.164 to 7.166). A trend can be observed in Figures 7.164 to 7.166, that the difference of force magnitude induced by waves at $T = 2.2\ s$ on the leading edge unit at slope 1:2 and slopes 1:3 or 1:4 (see Fig. 7.164) is greater than those at $T = 1.9\ s$ (see Fig. 7.165) and $T = 1.5\ s$ (see Fig.7.166). In general, this force magnitude difference becomes smaller as the wave period becomes shorter i.e as the
At submergence of \((h/d) > 1.10\), the 1:3 and 1:4 slopes cause the incoming waves to break and plunge relatively far from the leading edge unit resulting in smaller wave forces on the leading edge unit. Figure 7.164 shows that the maximum forward force on the unit at slopes 1:3 and 1:4 occurs at \((h/d) = 1.10\). This indicates that wave plunging at the leading edge unit on slopes of 1:3 or 1:4 produced by waves at \(T = 2.2\) s occurs at approximately \((h/d) = 1.10\); for this \((h/d)\), the 1:2 slope does not produce a wave plunging at the leading edge unit, (see Fig. 7.164). As \((h/d)\) increases, however, the 1:2 slope produces plunging location closer to the leading edge. This is indicated by the increase of the force magnitude for the slope of 1:2 (at \(h/d > 1.10\)) in Fig. 7.164.

On the other hand, slopes 1:3 and 1:4 at \((h/d) > 1.10\) cause the wave to plunge at locations farther in front of the leading edge than that at \((h/d) = 1.10\). It can be observed in Fig. 7.164 that the force decreases at \((h/d) > 1.10\) at slopes of 1:3 and 1:4. Figure 7.165 indicates a similar trend of forward force induced by waves at \(T = 1.9\) s, where slopes 1:3 and 1:4 produce plunging at the leading edge unit at approximately \((h/d) = 1.10\). At \((h/d) = 1.22\), however, the 1:2 slope causes the wave to induce a greater force on the leading edge unit than that produced by waves breaking on slopes of 1:3 and 1:4. Figure 7.166 shows the abrupt increase of forward force caused by waves at \(T = 1.5\) s breaking on a slope of 1:2 at \((h/d) = 1.22\) which indicates that the plunging occurs at the leading edge unit at approximately \((h/d) = 1.22\); for the same case, the 1:3 and 1:4 slopes may cause the wave to plunge relatively far in front of the leading edge unit.
7.12.2 Effect of Seaward Slope on the Upward Force on the Leading Edge Cubical Unit

Upward force on the unit is caused by pore pressure changes and lift effects of the fluid particles moving over the unit. The uplift force results from the difference between the upward pore pressure force on the bottom and the pressure force on the upper surface. The pressure on the upper surface is the combination of the gravity force due to the depth flow and the force due to curvilinear flow (lift). If the submerged weight of unit is smaller than the upward force, the unit may be lifted upward.

Figures 7.167 to 7.169 show the effect of slope on the upward force on the leading edge unit at wave periods of $T = 2.2$ s, $T = 1.9$ s, and $T = 1.5$ s respectively. It is shown in Figs 7.167 to 7.169 that the 1:2 slope affects waves moving above the slope to produce a greater upward force on the leading edge unit than that produced by waves travelling on slopes of 1:3 and 1:4. There is no consistent trend of the upward force affected by slopes of 1:3 and 1:4 as shown in Figs 7.167 to 7.169.

7.12.3 Effect of Seaward Slope on the Forward Force on the Landward Edge Cubical Unit

The effect of slope on the forward force acting on the landward edge unit depends on the wave
7.170 to 7.172; however, there is no consistent trend of forward force in its relation with the slope of the breakwater. At wave period of $T = 2.2$ s, the forward force on the landward edge unit increases with $(h'd)$ at slopes 1:2, 1:3, and 1:4, (see Fig. 7.170). At wave period $T = 1.9$ s, the magnitude of forward force reaches its peak at $(h'd) = 1.1$ at slopes 1:3 and 1:4, whereas at slope 1:2, the forward force continues to increase with $(h'd)$, (see Fig. 7.171). Finally, a wave period, $T = 1.5$ s, the force peak occurs at $(h'd) = 1.1$ at slope 1:2, 1:3, and 1:4, and at higher $(h'd)$, the forward force is nearly constant or reduced as shown in Fig. 7.172. Although a trend showing the effect of slope on the forward force on the landward edge unit cannot be generalized from the data in this study, the maximum forward force on the landward edge unit occurs at a slope of 1:2 as shown in Fig. 7.171, followed by the forces produced by waves travelling on the slopes of 1:3 (see Fig. 7.170), and 1:4 (see Figures 7.171 and 7.172). These maxima occur at $(h'd) = 1.22$.

7.12.4 Effect of Seaward Slope on the Upward Force on the Landward Edge Cubical Unit

Figures 7.173 to 7.175 show the effect of slope on the upward force on the landward edge unit. It can be seen that there is no significant difference in the upward force, due to different breakwater slopes. The maximum force on the landward edge unit occurs at the 1:2 seaward slope.
8.1 Stability Analysis of Cubical Armour Unit

When waves are applied to the cubical armour unit within the dynamometer, the unit experiences hydrodynamic forces and a spinning moment about its centroid. The components of the hydrodynamic force are generally expressed as tangential and normal forces. Tangential force refers to component of force on the unit which acts parallel to the surface where the unit is placed, whereas the normal force refers to component whose direction is perpendicular to the surface where the unit is located.

The fluid particle movement due to wave energy consists of horizontal and vertical motions. If the cubical unit is located with angle of orientation $\beta = 0^\circ$, the horizontal movement of the fluid particles contribute skin friction and form drag forces to the tangential force component, in addition, there is a fluid pressure difference between the two front and back surfaces, induced by the wave from which causes fluid acceleration and an associated inertial force. The load cell in the loop system in the dynamometer was used to record the total dynamic tangential component of the force. The force is due to drag and fluid acceleration around the unit (inertia) as given in the Morison Equation.

The magnitude of the skin friction forces is influenced by surface roughness, surface area, and local hydrodynamic velocity. If the model unit is placed on the breakwater with actual contact
the friction force, is applied to the unit. It is assumed, that skin friction force between the unit and the fluid is very small compared to the friction force between the unit and the other rocks, and therefore, it can be included in the drag force.

Vertical fluid particle motion (velocity and acceleration) contributes to the normal force component caused by primarily fluid pressure difference between the top and bottom surfaces of the unit, and wave induced shear force (parallel to the normal force) on the other four surfaces of the unit. Skin friction forces on the four surfaces of the unit are produced due to their contact with the moving fluid particles. It is also assumed that these skin friction is very small compared to that due to pressure difference between the top and bottom surface due to other effects, and therefore, is negligible.

The local velocity of the fluid at the top surface of the unit has a much greater magnitude than that at the bottom surface. This produces higher wave induced shear and drag forces on the top surface of the unit than on the bottom surface. This force gradient causes the unit to tend to spin or to rotate around its lateral axis. Figure 8.1 shows the forces on the unit. The two load cells used in the dynamometer were arranged in such a way that they could restrict such movement, and simultaneously they were able to record the total normal force component on the unit, (see discussion on the dynamometer presented in section 3.4.2). The spinning moment causes a difference in the normal force recorded at the two normally positioned load cells in the dynamometer.
addition to the dynamic force and the spinning moment. The magnitude of buoyant force reaches its maximum if the unit is totally submerged. This magnitude decreases if the unit is partially submerged, and it vanishes if the unit is exposed completely to the air. The buoyant force counteracts the weight of the unit causing the unit to respond as if it were lighter. The two components of wave force and the spinning moment tend to move the unit, but the friction force on the bottom surface of the unit and the reaction forces at contact points with other units along with submerged weight of the unit counteract such movement.

Figure 8.2a illustrates the free body diagram of the cubical unit showing hydrodynamic forces $F_x$ and $F_y$, spinning moment $M_s$, submerged weight $W'$, and reactions at the bottom due to friction $F_f$ and at the contact points with the other rocks $R_n$ around the unit. If interlocking with other rocks around the unit is not present because of poor placement, reactions $R_n$ are not present, and this represents the most unstable condition for overturning and sliding. The angle of the total force $F_T$ is shown at the right side of Fig. 8.2a. The horizontal force $F_x$ consists of the resultant of pressure on the front and back surfaces of the unit, and wave induced shear force on the other four surfaces. Figure 8.2b illustrates the modified free body diagram of the unit and the forces acting on it. Vertical force $F_y$ and spinning moment $M_s$ are replaced by two vertical forces $F_{y1}$ and $F_{y2}$ at points where the two load cells in the dynamometer were set up.

The movement of the unit due to the hydrodynamic forces and the spinning moment can be classified as sliding or overturning. The initiation of sliding occurs if the tangential force
overturning moment about the toe of the unit exceeds the resisting moment due to the submerged weight of the unit.

8.2 Sliding

Sliding motion occurs if the tangential force component is greater than the friction force. Based on its direction, sliding can be classified as either forward and backward sliding. Forward sliding is caused by forward tangential force, whereas backward sliding is due to backward tangential force. The magnitude of the friction force depends on the resultant of normal force and friction coefficient between the model unit and the other rocks. This friction coefficient depends on the angle of repose of the rocks used to built the breakwater. In order to determine this angle of repose, ten tests were conducted by using the same rocks used to construct the model breakwater. The rocks were randomly piled, and the slope of the pile was measured to obtain the angle of repose $\Phi$. Friction coefficient is expressed as follows:

$$ f = \tan \phi $$

(8.1)

where $\Phi$ is the angle of repose of the rocks. The limiting magnitude of the static friction force can be expressed by:

$$ F_f = f (W' - F') $$

(8.2)

Combining Eqs. 8.1 and 8.2 the limiting static friction force can be rewritten as:
Assuming there is no interlocking effect (no reaction with other units), the critical condition due to sliding occurs when the magnitude of the tangential force is the same as the friction force. In such a state the ratio between the tangential and friction forces becomes unity as expressed below:

$$\frac{F_x}{F_f} = 1$$

(8.4)

The expression \((F_x / F_f)\) can also be called as sliding instability number of the unit. Sliding may occur if \((F_x / F_f) > 1\), (see Fig. 8.2a). The dynamic stability of the unit can be examined by evaluating the dynamic instability number \((F_x / F_f)\).

8.3 Overturning

Overturning movement of the unit is caused by tangential and normal hydrodynamic forces, and spinning moment acting on the unit. Based on its direction, overturning moment can be classified into forward overturning and backward overturning. Considering the worst case where there is no interlocking between the cubical unit and the other surrounding rocks, forward overturning is evaluated about the back toe of the unit, whereas the backward overturning is examined about the front toe of the cubical unit model, (see Fig. 8.2a). The magnitude of the hydrodynamic overturning moment (worst case) is given by:
\[ M = F_x \left( \frac{a}{2} \right) + F_y \left( \frac{a}{2} \right) + M_s \]  

(8.5)

\[ F_y = F_{y1} + F_{y2} \]  

(8.6)

\[ M_s = b \left( F_{y1} - F_{y2} \right) \]  

(8.7)

where

- \( a \) : side length of the cubical unit
- \( b \) : distance between the two vertical wire used in the dynamometer, (see Fig. 8.2b)
- \( M_s \) : spinning moment
- \( F_x \) : Horizontal hydrodynamic force component, (see Fig. 8.2a)
- \( F_y \) : Vertical hydrodynamic force component, (see Fig. 8.2a)
- \( F_{y1} \) : Vertical force recorded by the load cell at the front side, (see Fig. 8.2b)
- \( F_{y2} \) : Vertical force recorded by the load cell at the back side, (see Fig. 8.2b)

The hydrodynamic overturning moment is countered by the resistance moment about the toe of the unit which is caused by the submerged weight of the unit. The magnitude of the restricting moment, \( M_r \), is given by:

\[ M_r = W' \left( \frac{a}{2} \right) \]  

(8.8)

Overturning motion may occur if \( M > M_r \), and critical condition occur if the ratio of the overturning moment and the restricting moment is unity as expressed below:
\[
\frac{M}{M_r} = \frac{F_x \left( \frac{a}{2} \right) + F_y \left( \frac{a}{2} \right) + M_y}{W' \left( \frac{a}{2} \right)} = 1
\] (8.9)

The expression \((M/M_r)\) is the overturning instability number of the unit. Overturning may occur if \((M/M_r) > 1\).

8.4 Simultaneous Dynamic Sliding and Overturning Stability

As discussed in the previous sections in this chapter, the critical location of the unit is at the leading edge. Therefore, the stability analysis of the unit at the leading edge will be discussed in this section. The simultaneous stability of the unit at the landward edge caused by the same wave will be presented to compare the unit stability at the two edges.

The hydrodynamic force and spinning moment acting on the model unit may produce instability due to sliding or overturning of the unit. Sliding motion occurs if the sliding instability number as expressed in Eq. 8.4 exceeds unity before the overturning instability number given in Eq. 8.9 and vice versa.

In order to show the simultaneous dynamic sliding and overturning instability of the unit, two conditions are selected as examples. The first example is the submerged case at \((h'd) = 0.79\) and the second one is the low-crested case at \((h'd) = 1.22\).
caused by waves of $T = 2.2$ s, $T = 1.9$ s, and $T = 1.5$ s respectively, or $H/gT^2 = 0.00372$, $0.00467$, and $0.00738$, (see Table 7.4). Figures 8.6 to 8.8 show the corresponding total forces at $(h/d) = 1.22$. The abscissa of the plot shows the angle of attack of the dynamic force, whereas the ordinate shows the magnitude of the force. The angle of force attack ($\alpha$) is defined in Fig. 8.2a. Pure forward and backward forces occur at $\alpha = 0^\circ$ and $180^\circ$ respectively, whereas pure upward and downward forces occur at $\alpha = 90^\circ$ and $270^\circ$.

The corresponding dynamic simultaneous instabilities of the unit caused by waves presented in Figures 8.3 to 8.5 are shown in Figures 8.9 to 8.11, whereas the corresponding unit dynamic instabilities caused by waves shown in Figures 8.6 to 8.8 are shown in Figures 8.12 to 8.14.

It can be seen in Figure 8.9 to 8.11 that the unit at the leading edge is stable against sliding and overturning. Waves of $T = 1.9$ s and $T = 1.5$ s produce greater sliding and overturning instability numbers compared to those produced by a wave of $T = 2.2$ s. For the low-crested case, however, the unit is not stable in either sliding or overturning as indicated in Figures 8.12 to 8.14 which show that the maximum instability number for both sliding and overturning is greater than unity.

Tables 8.1 to 8.3 show the maximum values of the forward sliding and overturning instability numbers caused by waves of $T = 2.2$ s, $T = 1.9$ s, and $T = 1.5$ s respectively. Each table consists of the wave height, submergence and the instability numbers of the unit at points E and D.

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unit located at the leading edge (pt. E) and at the landward edge (pt. D) at wave period $T = 2.2$ s. The corresponding $H/gT^2$ values are given in Table 7.4.

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<th>Overturning</th>
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194
The table below summarizes the experimental data obtained from a unit located at the leading edge (pt. E) and at the landward edge (pt. D) at wave period $T = 1.9$ s. The corresponding $H/gT^2$ values are given in Table 7.4.

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195
unit located at the leading edge (pt. E) and at the landward edge (pt. D) at wave period $T = 1.5$ s. The corresponding $H/gT^2$ values are given in Table 7.4.

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<th>Overturning</th>
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unit is stable against sliding and overturning at points E and D. The cubical units start to be unstable at \((h/d) > 1.00\) (low-crested).

For the shorter waves, namely at \(T = 1.9\) s and \(T = 1.5\) s, the leading edge cubical unit starts to be unstable at a lower \((h/d) \geq 0.85\) as shown in Tables 8.2 and 8.3. The landward edge unit starts to be unstable at \((h/d) \geq 1.10\).

The angle of the hydrodynamic force plays an important role of contributing the instability of the model unit. Figures 8.15 to 8.17 are chosen as examples to show the envelope of sliding and overturning instability numbers of the leading edge unit for the submerged case and their relationship with the angle of attack of the hydrodynamic force. Figure 8.15 shows that the maximum forward sliding and overturning instability numbers occur at approximately \(\alpha = 15^\circ\) whereas the maximum backward sliding and overturning occur at approximately \(\alpha = 180^\circ\) at \(T = 2.2\) s. The values of the forward sliding and overturning instability numbers are slightly smaller than the numbers for backward motions. This can also be seen in Fig. 8.9 where the dynamic instability numbers are shown. Figure 8.16 shows the envelope of instability numbers of the leading edge cubical unit at \(T = 1.9\) s. The forward sliding and overturning instability numbers increase as the wave period becomes shorter, whereas the backward instability numbers decrease. This trend continues as the wave period becomes \(T = 1.5\) s as shown in Fig. 8.17. Observe also that the angle of hydrodynamic force at the maximum instability numbers shifts from around \(\alpha = 15^\circ\) at \(T = 2.2\) s to about \(\alpha = 30^\circ\) at \(T = 1.9\) s and \(T = 1.5\) s.

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low-crested case for $T = 2.2 \text{ s}$, $T = 1.9 \text{ s}$, and $T = 1.5 \text{ s}$ respectively. The corresponding dynamic instability numbers are previously shown in Figures 8.12 to 8.13. Under these conditions, the leading edge unit is instable for both sliding and overturning. However, initiation of sliding occurs if the sliding instability number exceeds unity before the overturning or vice versa. Figures 8.18 to 8.20 show that the maximum sliding and overturning moment increases as the waves become shorter. The angle which produces the maximum instability numbers continue to shift from about $\alpha = 30^\circ$ or smaller under the submerged case to about $\alpha = 45^\circ$ under the low-crested case.

Tables 8.4 to 8.6 show the instability numbers for backward motions for wave period at $T = 2.2 \text{ s}$, $T = 1.9 \text{ s}$, and $T = 1.5 \text{ s}$ respectively. The maximum instability numbers for backward motions in this study are smaller and in all cases smaller than unity indicating that the cubical unit is stable against backward motions. This can be shown by comparing Tables 8.4 to 8.6 to the corresponding Tables 8.1 to 8.3.

The negative sign of the instability numbers in Tables 8.4 to 8.6 is used to indicate that the instability numbers are caused by backward motion of the fluid as also indicated in Figures 8.9 to 8.20.
number for the cubical unit located at the leading edge (pt. E) and at the landward edge (pt. D) at wave period $T = 2.2$ s. The corresponding $H/gT^2$ values are given in Table 7.4.

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<th>Overturning</th>
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Table 8.5: Simultaneous maximum backward sliding and backward overturning instability number for the cubical unit located at the leading edge (pt. E) and at the landward edge (pt. D) at wave period $T = 1.9$ s. The corresponding $H/gT^2$ values are given in Table 7.4.

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Table 8.6: Simultaneous maximum backward sliding and backward overturning instability number for the cubical unit located at the leading edge (pt. E) and at the landward edge (pt. D) at wave period $T = 1.5$ s. The corresponding $H/gT^2$ values are given in Table 7.4.

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Figures 8.21a to 8.21c show the relationship between forward sliding instability number of the leading edge and landward edge units at $T = 2.2$ s, $T = 1.9$ s, and $T = 1.5$ s respectively, whereas Figures 8.22a to 8.22c show the corresponding overturning instability numbers.

Figures 8.21a and 8.22a indicate that at $T = 2.2$ s, the leading edge unit starts to be unstable due to sliding or overturning at $(h/d) = 1.10$, while the landward edge unit is near the critical condition.

Figures 8.21b and 8.21b show that at $T = 1.9$ s, the leading edge unit starts to be unstable due to sliding and overturning at $(h/d) = 0.92$, while the landward edge unit experiences instability at $(h/d) = 1.10$. Similar indications are also experienced by the two units at both crest tips at wave period $T = 1.5$ s as shown in Figures 8.21c and 8.22c.

Sliding and overturning instability numbers are obviously dependent on wave height. Therefore it is also important to show the relationship between wave height $(H)$, wave period $(T)$, submergence represented by $(h/d)$, and the instability numbers as expressed in Eq. 8.4 and 8.9 for sliding and overturning motions respectively. A dimensionless parameter is introduced in this study to show that relationship. This parameter is formed by dividing the submergence $(d/h)$ by the dimensionless wave steepness $(H/gT^2)$ resulting in the following form: 

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Using the expression above, a dimensionless relationship between instability number and wave period, submergence, and wave height can be made as shown in Figures 8.23a and 8.23b for sliding and overturning respectively.

Figures 8.23a and 8.23b show that the leading edge cubical unit is unstable at approximately 
\[ gT^2/(h'd(H)) \leq 250 \] due to sliding or overturning, whereas the landward edge cubical unit starts to be unstable at 
\[ gT^2/(h'd(H)) \leq 230. \]

Table 8.7 show the values of 
\[ gT^2/(h'd(H)) \] and the corresponding sliding and overturning instability numbers for the cubical units at the leading edge and the landward edge.

Table 8.7: Simultaneous maximum sliding and overturning instability number for the cubical unit located at the leading edge (pt. E) and at the landward edge (pt. D).

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9.1. **Conclusions**:

Several conclusions are made from this study regarding wave transmission and reflection, wave forces on the armour units, and the stability of an armour unit.

**Wave transmission and reflection**

1. Transmission coefficient $K_t$ decreases as $(h/d)$ increases for the range of $1.5 \leq T \leq 2.2$ s; $0.73 \leq (h/d) \leq 1.38$; $0.00370 \leq (H/gT^2) \leq 0.00848$; and $0.0042 \leq (d/gT^2) \leq 0.0170$ in this study. The maximum $K_t$ in this study was approximately 0.80 at $(h/d) = 0.73$ (submerged case), whereas the minimum $K_t$ was approximately 0.27 at $(h/d) = 1.38$ (low-crested case), which shows that a low-crested breakwater is a good wave energy dissipator while still allowing flushing to the coastal zone. The transmission coefficient transmission $K_t$ produced by a longer wave is greater than that of a shorter wave at the same $(h/d)$.

2. The reflection coefficient $K_r$ increases as $(h/d)$ increases. At $(h/d) = 0.73$ (submerged case) the reflected coefficient $K_r$ was approximately 0.02 to 0.06, whereas at $(h/d) = 1.38$ (low-crested case) $K_r$ was approximately 0.11 to 0.13. A wave of $T = 2.2$ s produces a greater coefficient $K_r$ than a wave of $T = 1.5$ s in this study.
3. A wave breaking on the seaward slope of a breakwater produces an impact (shock pressure), inertia, and drag forces on the armour unit.

4. Orientation of the armour unit influences the magnitude of maximum force acting on the armour unit. A vertical plane front surface of an armour unit perpendicular to the wave travel produces the greatest force due to impact on the unit.

5. The maximum forward, backward, upward, and downward hydrodynamic forces on the armour unit is experienced by the unit located at the leading edge (seaward tip). A wave of $T = 1.5$ s produces the greatest forward, upward, and downward forces at the leading edge unit, whereas a wave of $T = 2.2$ s causes the maximum backward force on the unit. The magnitude of the maximum forward force on the leading edge unit at $(h'/d) > 1$ is greater than the maximum magnitude at $(h'/d) < 1$, whereas the maximum backward force on the unit at $(h'/d) = 0.85$ (see Fig. 7.38) is greater than the backward force at $(h'/d) > 1$. This indicates that the stability of the leading edge unit against forward movements (overturning or sliding) must be evaluated under a low-crested case, whereas the instability due to backward motions (overturning or sliding) must be evaluated under submerged case.

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the seaward slope increases with \((z/h)\) and \((h'/d)\). However, unit placement at point F \((z/h = 0.75)\) may experience the greater backward force than the leading edge unit at \((h'/d) = 1.38\), (see Figs. 7.39 and 7.40). This indicates that the stability of the unit at point F should be evaluated against backwashing due to the rapid draw down.

7. The maximum forward, backward, upward, and downward forces on the landward slope increases with \((z/h)\) and \((h'/d)\) for the range of \(0.73 \leq (h'/d) \leq 1.38\), and therefore, the landward edge is the most critical location for the unit at the landward slope.

8. The 1:3 or 1:4 front slope of a breakwater produces a greater forward force on the leading edge unit than that produced by 1:2 slope. On the contrary, the 1:3 or 1:4 front slope of a breakwater produces a smaller upward force on the unit than that produced by 1:2 slope.

9. The stability of the armour unit due to sliding or overturning at the leading edge (seaward tip) is worse than the stability of the armour unit at the landward tip. The instability number increases as the \((h'/d)\) increases for \(0.73 \leq (h'/d) \leq 1.38\).

9.2. **Recommendations** :

The following recommendations are made to improve the experimental study of submerged and low-crested breakwaters.
missing peak force values in several tests. The missing force peaks may also due to response time of the sensors. Therefore, it is recommended to use a higher rate of samples per second, and also a faster response time of sensors for a better accuracy of an experimental study.

2. Time step \( \Delta t \) used for the determination of impact index \( C_{im} \), inertia index \( C_M \), and drag index \( C_D \) in this study was 0.05 s. It is recommended to use different time steps to test the sensitivity of \( \Delta t \) on the computation.

3. Wave reflection may cause wave height variation in an experimental study. In order to minimize wave reflection, it is recommended to use a long flume where the breakwater can be constructed at a distance greater than three maximum wave length from the wave paddle, and a computer controlled absorbing wave reflection machine. It is also recommended to do replications of tests to study reflection coefficient \( K_r \).

4. The correct location of wave velocity measurement still needs to be investigated. Based on this study, it is recommended to use two or more measurement points representing the fluid velocity field at the front surface of the armour unit. It is also recommended to use two Pitot tubes arranged in two directions for the determination of forward and backward fluid velocities.
recommended to study simultaneously the forces acting on two or more armour units in tandem in order to quantify interlocking which exists among the armour units.

6. The research of this thesis should be extended to a wider range of wave period (T) and height (H), breakwater geometry such as height (h), width (B), and type of structure such as non-overtopped, or berm breakwater.

7. The effect of response time and natural frequency of the dynamometer should be investigated.

Design Considerations:

8. The simultaneous tangential and normal force must be considered for a more rational armour unit design.

9. Impact force produced by breaking waves must be considered as a force component besides inertia and drag forces for submerged and low-crested breakwaters.

10. Since a breakwater requires a large number of large rocks, a more innovative armour unit design is needed so that the size of each individual armour unit can be minimized for a more economical design of breakwaters.

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Appendix A
<table>
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<th>Range (ml)</th>
<th>Median (ml)</th>
<th>Diameter (cm)</th>
<th>Volume (ml)</th>
<th>% Total</th>
<th>Note: Volume of breakwater : 142764 ml</th>
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<td>&lt; 5.76</td>
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<tr>
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<td>125</td>
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<td>18157</td>
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<tr>
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<td>175</td>
<td>5.59 - 7.24</td>
<td>28120</td>
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</tr>
<tr>
<td>200 - 249</td>
<td>225</td>
<td>7.26 - 7.81</td>
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<td>20.44</td>
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<tr>
<td>250 - 299</td>
<td>275</td>
<td>7.82 - 8.30</td>
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<tr>
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<td>&gt; 8.31</td>
<td>2820</td>
<td>4.07</td>
<td></td>
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</table>

Total : 69337 100

Total volume of rocks to construct the breakwater in the flume = 69337 ml.
Porosity of the structure = (142764 - 69337) / 142764 = 51.43 %

**Table 3.2: Preliminary Tests**

Selected Test Variables of each Water Depth

(d = 20.0 cm, 22.5 cm, 25.0 cm, 27.5 cm, 30.0 cm, 32.5 cm and 35.0 cm, and 37.5 cm)

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<tr>
<th>Units Placement</th>
<th>T(s)</th>
<th>(H)</th>
<th>Units Placement</th>
<th>T(s)</th>
<th>(H)</th>
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<td>H1</td>
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<td>H1</td>
<td>B - G</td>
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<td>H1</td>
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<tr>
<td>D - E</td>
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<td>H1</td>
<td>B - G</td>
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<tr>
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<td>H2</td>
<td>B - G</td>
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<td>H2</td>
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<td>H2</td>
<td>B - G</td>
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<td>D - E</td>
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<td>H2</td>
<td>B - G</td>
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<td>H2</td>
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<th>(H)</th>
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<td>A - H</td>
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<td>A - H</td>
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<td>A - H</td>
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</table>

Note:
In the Selected and Special Tests, the units placement combination was D-E; the water depths were d = 22.5 cm, 25.0 cm, 27.5 cm, 30.0 cm, and 32.5 cm; and the slopes were 2:1, 3:1, and 4:1 (horizontal : vertical)
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<thead>
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<th>D(cm)</th>
<th>T(s)</th>
<th>Htoe (cm)</th>
<th>Ci</th>
<th>C'M</th>
<th>C'D</th>
<th>F-exp(N) max</th>
<th>F-th(N) max</th>
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AVERAGE : 0.0661 0.1056 0.0223 101.8551
STD : 0.1266 0.0484 0.0210 31.5777
### Table 6.2

**Comparison of the Measured Force and the Predicted Force**

*(Based on Force Maxima)*

<table>
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<tr>
<th>D(cm)</th>
<th>T(s)</th>
<th>Htoe (cm)</th>
<th>Ci</th>
<th>C'M</th>
<th>C'D</th>
<th>F-exp(N) max</th>
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Average: 0.0809 0.0972 0.0281 90.97  
STD: 0.1220 0.0581 0.0338 18.45
Appendix B
Figure 1.1
Placements of the Cubical Units on the Breakwater

Landward Slope Placements:

A : Landward toe
B : Middle of the landward slope
C : 0.75 h above the bottom of the breakwater
D : Landward edge

Seaward Slope Placements:

E : Seaward edge
F : 0.75 h above the bottom of the breakwater
G : Middle of the seaward slope
H : Seaward toe
Figure 1.2
Definition Sketch of the Hydrodynamic Forces and Spinning Moment on the Cubical Unit

F_y

F_x

F_f

M

W'

F_T

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INSTRUMENTATION IN THE ARMOUR UNIT MODEL

**Figure 3.2a**

- Stainless steel wires connected to load cells LC-2 and LC-3.
- Stainless steel wire connected to load cell LC-1 in the loop system.
- Tension adjuster.
- Load cell head.
- Fastener (nut).
- Threaded rod.
- Pulley.
- Aluminum tube diameter 25 mm.
- Stainless steel rod.

**Figure 3.2b**

- Load cell LC-1 is used for the tangential force measurement.
- Load cells LC-2 and LC-3 are used for the normal force measurement, and the determination of spinning moment.
- Loop arrangement for tangential force measurement.
Figure 3.3
Dynamometers
Figure 3.4
Force - Displacement Test of the Stainless Steel Wire

Note:
The length of the stainless steel wire for this test was 100 mm.
Figure 3.5
Megadac Data Acquisition System
Figure 3.6
Bracing of the two dynamometers
Figure 3.7
Microprocessor Based Digital Force Meter DFG-10 in Operation for Calibration of Load Cell -1 and Load Cell-A
CALIBRATION OF Fx MEASUREMENT
(Load Cell-1 in the Dynamometer)
Figure 3.9

CALIBRATION OF Fx MEASUREMENT
_Load Cell-A in the Dynamometer_

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Figure 3.10
Placement Combinations of the Two Cubical Armour Units

ARMOUR UNIT PLACEMENTS

Combinations:
D-E
C-F
B-G
A-H
Figure 4.1
Geometric and Structural Conditions of the Breakwater

BREAKWATER

CUBICAL ARMOUR UNIT

Wave Period, T

Porosity, np

θ

θ

d

h

Angle of Orientation of the Armour Unit

Surface Roughness, k

Projected Areas, Aw and Ay
Figure 5.1
Definition of Breaker Geometry (after SPM, 1984)

Proposed Structure (Effect of Structure on Breaking has not been Considered)

Wave Profile at Start of Breaking

Wave Profile when Breaking is Nearly Complete

Region where Breaking Starts

Distance = \gamma H_b

(d_b)_{min} = \beta H_b

(d_b)_{max} = \sigma H_b

X_p = Breaker Travel

H_b

m

l

m

l

\beta H_b

\sigma H_b
Figure 5.2
\(\alpha\) and \(\beta\) versus \(H/gT^2\) (after Weggel, 1972)
The range of \(H/gT^2\) in this study is \(0.00370 \leq H/gT^2 \leq 0.00848\)
Dimensionless Design Breaker Height ($H_b/d_0$) versus Relative Depth ($d_0/gT^2$) at Structure (after SPM, 1984). The range of $d_0/gT^2$ is $0.0042 \leq d_0/gT^2 \leq 0.0170$. 
Figure 5.4
Ranges of suitability for various wave theory as suggested by Le Méhaute (1976)

Figure 5.5
Ranges of wave theories giving the best fit to the dynamic free surface boundary condition as suggested by Dean (1976)
Figure 5.6

$K_T$ versus $H/gT^2$ (laboratory data from Davidson, 1969)
The range of $H/gT^2$ in this study is $0.00370 \leq H/gT^2 \leq 0.00848$
Monochromatic wave transmission, impermeable rubble-mound breakwater, where $h/d_s = 1.033$. In this study $H_t/H_i = 0.5752$ (STD = 0.0412) at $h/d = 1.00$ and $b/h = 1$.
Monochromatic wave transmission, impermeable rubble-mound breakwater, where $h/d_s = 1.133$. In this study $H_r/H_i = 0.5297$ (STD = 0.0426) at $h/d = 1.10$ and $b/h = 1$. 

(after Soille, 1963)
Monochromatic wave transmission, permeable rubble-mound breakwater, where $h/d = 0.899$ and 1.033. In this study $H_i/H_o = 0.5804$ (STD = 0.0376) at $h/d = 0.92$, and $H_i/H_o = 0.5752$ (STD = 0.0412) at $h/d = 1.00$; $b/h = 1$. 

(after Saville, 1963)
Monochromatic wave transmission, permeable rubble-mound breakwater, where $h/d = 1.133$. In this study $H_t/H_i = 0.5297$ (STD = 0.0426) at $h/d = 1.10$ and $b/h = 1$. 

(figure showing a graph with various data points and annotations)
Elkamhawy's Design Equation, Saville's Experimental Data, and This Study

STD = 0.1197

Elkamhawy  Saville (1963)  This study
Figure 5.12

TRANSMISSION COEFFICIENT vs SUBMERGENCE

$h/B = 1$, Slope = 1:2

$K_t = \frac{H_t}{H_i}$

- $a = -1.252; b = 1.711; H/gT^2 = 0.0037$
- $a = -0.975; b = 1.439; H/gT^2 = 0.0048$
- $a = -0.995; b = 1.424; H/gT^2 = 0.0075$

- $a = -0.279; b = 0.887$
- $a = -0.302; b = 0.869$
- $a = -0.413; b = 0.932$

Submergence, $h/d$

$T = 2.2 \text{ s}, \text{Data}$  $T = 1.9 \text{ s}, \text{Data}$  $T = 1.5 \text{ s}, \text{Data}$  $T = 2.2 \text{ s}, \text{Linear Fit}$  $T = 1.9 \text{ s}, \text{Linear Fit}$  $T = 1.5 \text{ s}, \text{Linear Fit}$
Figure 5.13
Transmission Coefficient $K_t$ versus Relative Crest Height $R_c/H_i$
by various researchers
Figure 5.14

$K_t$ versus $R_c/H_i$
$h/B=1$, Slope=1:2

$K_t = H_t/H_i$

Relative Crest Height $R_c/H_i$

\[\begin{array}{c}
\Delta \ T=2.2 \text{ s} \\
\square \ T=1.9 \text{ s} \\
\times \ T=1.5 \text{ s} \\
\blacksquare \ \text{Van der Meer} \\
\rightarrow \ 	ext{Elkamhawy}
\end{array}\]
Figure 5.15

REFLECTION COEFFICIENT vs SUBMERGENCE
h/B = 1, Slope = 1:2

Kr = Hr/Hi

Submergence, h/d

T = 2.2 s  T = 1.9 s  T = 1.5 s
Figure 5.16

REFLECTION COEFFICIENT vs SUBMERGENCE
h/B=1, Slope=1:2

Elkamhawy's prediction: \( Kr = \frac{h}{d}(0.16 + 2.0 \frac{Hi}{Lo}) \)

Data of this study

- ▲ T=2.2 s, Data
- □ T=1.9 s, Data
- ★ T=1.5 s, Data
- ▲ T=2.2 s, Elkamhawy
- □ T=1.9 s, Elkamhawy
- ★ T=1.5 s, Elkamhawy
Figure 5.17

REFLECTION COEFFICIENT vs SUBMERGENCE
h/B=1, Slope=1:2

\[ Kr = \frac{H_r}{H_l} \]

Submergence, h/d

STD = 0.0307
STD = 0.0162
STD = 0.0058

\[ \Delta \] T=2.2 s, data
\[ \square \] T=1.9 s, data
\[ \star \] T=1.5 s, data

\[ \text{---} \] T=2.2 s, linearized
\[ \text{---} \] T=1.9 s, linearized
\[ \text{-----} \] T=1.5 s, linearized
Figure 5.18
Corrected Reflection Coefficient $K_r$ from Apparent Coefficient $K_r$ (after Silvester, 1974)
Figure 5.19

DISSIPATION COEFFICIENT vs SUBMERGENCE
h/B=1, Slope=1:2

\[ K_{ds} = \frac{H_i^2 - H_r^2 - H_i^2}{H_i^2} \]

Submergence, h/d

\[ \Delta \text{ T=2.2 s} \quad \square \text{ T=1.9 s} \quad * \text{ T=1.5 s} \]
Figure 6.2

LONGITUDINAL AND UPLIFT FORCES
T=1.9 s, H=174 mm, d=225 mm, h/d=1.22

Armour unit orientation = 0 deg

---

Fx  ---  Fy

---

E

SLOPES 1:2
Figure 6.3

LONGITUDINAL AND UPLIFT FORCES
$T=1.9\text{ s}, H=174\text{ mm}, d=225\text{ mm}, h/d=1.22$

Armour unit orientation = 15 deg

Force (N)

Time (s)

--- Fx --- Fy

SLOPES 1:2

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LONGITUDINAL AND UPLIFT FORCES
T=1.9 s, H=179 mm, d=225 mm, h/d=1.22

Armour unit orientation = 26.57 deg

---

260
LONGITUDINAAL AND UPLIFT FORCES
T=1.9 s, H=178 mm, d=225 mm, h/d=1.22

Time (s)

Force (N)

--- Fx --- Fy

SLOPES 1:2
LONGITUDINAL AND UPLIFT FORCES
T=1.9 s, H=178 mm, d=225 mm, h/d=1.22

---

266
Figure 6.8

FORCES ON FRONT AND BACK ARMOUR UNITS
T=1.9 s, H=178 mm, d=225 mm, h/d=1.22

---

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SIDE VIEW

PRESSURE MEASUREMENTS
C : TOTAL PRESSURE
D : STATIC PRESSURE

FRONT VIEW

Figure 6.9
VELOCITY AND Fx, LEADING EDGE UNIT

T = 2.2 s, H = 164 mm, d = 225 mm, h/d = 1.22

V (m/s) and Fx (N)

Time (s)

Velocity — Fx

SLOPES 1:2
Figure 6.11

VELOCITY AND Fx, LEADING EDGE UNIT
T=1.9 s, H=166 mm, d=225 mm, h/d=1.22

![Graph showing velocity and force over time](image)

---

B

H

SLOPES 1:2
Figure 6.12

VELOCITY AND Fx, LEADING EDGE UNIT
T=1.5 s, H=165 mm, d=225 mm, h/d=1.22

![Diagram of velocity and force measurements with annotations](image)

**SLOPES 1:2**
Figure 6.13

WAVE, VELOCITY, ACCELERATION, AND FORCE

$T = 2.2 \text{ s, } H = 164 \text{ mm, } d = 225 \text{ mm, } h/d = 1.22$

- ▲ Wave(toe)-cm
- \(\times\) \(\frac{dV}{dT}(\text{m/s}^2)\)
- □ Velocity(\text{m/s})
- \(\overline{\text{x}}\) Fx (N)

Time (s)
Figure 6.14

WAVE, VELOCITY, ACCELERATION, AND FORCE
T = 1.9 s, H = 166 mm, d = 225 mm, h/d = 1.22

Time (s)

-10 -5 0 5 10 15

wave(toe)-cm  \(\frac{dV}{dT}(m/s^2)\)  velocity(m/s)  Fx(N)
Figure 6.15

WAVE, VELOCITY, ACCELERATION, AND FORCE
T=1.5 s, H=165 mm, d=225 mm, h/d=1.22

Time (s)

Wave(toe)-cm  dV/dT(m/s²)  Velocity(m/s)  Fx(N)
Figure 6.16

WAVE, VELOCITY, ACCELERATION, AND FORCE
T=2.2 s, H=152 mm, d=275 mm, h/d=1

Time (s)

- ▲ Wave(toe)-cm
- ▪ dV/dT(m/s²)
- □ Velocity(m/s)
- • Fx(N)
WAVE, VELOCITY, ACCELERATION, AND FORCE
T=1.9 s, H=164 mm, d=275 mm, h/d=1

Time (s)

- Wave(toe)-cm
- dV/dT(m/s²)
- Velocity(m/s)
- Fx(N)
Figure 6.18

WAVE, VELOCITY, ACCELERATION, AND FORCE
$T=1.5\text{ s}, H=163\text{ mm}, d=275\text{ mm}, h/d=1$

- ▲ Wave(toe)-cm
- ▶ $dV/dT(\text{m/s2})$
- □ Velocity(\text{m/s})
- ✗ $F_x(\text{N})$

Time (s)
Horizontal Force Profile
Figure 6.20

WAVE, VELOCITY, ACCELERATION, AND FORCE
T=2.2 s, H=164 mm, d=225 mm, h/d=1.22

Time (s)

Wave(toe)-cm  dV/dT(m/s2)  Velocity(m/s)  Fx (N)
Figure 6.21

EXPERIMENTAL AND THEORETICAL FORCES
T=2.2 s, H=164 mm, d=225 mm, h/d=1.22

Time (s)

Wave(toe)-cm  dV/dT(m/s²)  Velocity(m/s)  Fx-exp(N)
Figure 6.22

EXPERIMENTAL AND THEORETICAL FORCES
$T=2.2 \text{ s}, H=164 \text{ mm}, d=225 \text{ mm}, h/d=1.22$

- $\Delta$ Wave(toe)-cm
- $\times$ Fx-exp(N)
- $\blacksquare$ Fx-th(N)
EXPERIMENTAL AND THEORETICAL FORCES
T=1.9 s, H=166 mm, d=225 mm, h/d=1.22

- Wave(toe)-cm - Fx-exp(N) - Fx-th(N)
Figure 6.24

EXPERIMENTAL AND THEORETICAL FORCES
T=1.5 s, H=165 mm, d=225 mm, h/d=1.22

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EXPERIMENTAL AND THEORETICAL FORCES
T=2.2 s, H=171 mm, d=250 mm, h/d=1.1
Figure 6.26

EXPERIMENTAL AND THEORETICAL FORCES
T=1.9 s, H=172 mm, d=250 mm, h/d=1.1

![](image)

- Wave(toe)-cm
- Fx-exp(N)
- Fx-th(N)
Figure 6.27

EXPERIMENTAL AND THEORETICAL FORCES
T=1.5 s, H=169 mm, d=250 mm, h/d=1.1

Time (s)

Wave(toe)-cm  Fx-exp(N)  Fx-th(N)
EXPERIMENTAL AND THEORETICAL FORCES

$T=2.2 \text{ s}, \ H=152 \text{ mm}, \ d=275 \text{ mm}, \ h/d=1$

![Graph showing experimental and theoretical forces with time (s) on the x-axis and force on the y-axis. The graph includes symbols for Wave(toe)-cm, Fx-exp(N), and Fx-th(N).]
EXPERIMENTAL AND THEORETICAL FORCES
T=1.9 s, H=164 mm, d=275 mm, h/d=1

- Wave(toe)-cm
- Fx-exp(N)
- Fx-th(N)
EXPRESSMENT AND THEORETICAL FORCES
$T=1.5 \text{ s, } H=163 \text{ mm, } d=275 \text{ mm, } h/d=1$
EXPERIMENTAL AND THEORETICAL FORCES
T=2.2 s, H=166 mm, d=300 mm, h/d=0.92

Time (s)

Wave(toe)-cm  Fx-exp(N)  Fx-th(N)
Figure 6.32

EXPERIMENTAL AND THEORETICAL FORCES
T=1.9 s, H=164 mm, d=300 mm, h/d=0.92

- Δ Wave(toe)-cm
- X Fx-exp(N)
- ■ Fx-th(N)

Time (s)
EXPERIMENTAL AND THEORETICAL FORCES
T=1.5 s, H=160 mm, d=300 mm, h/d=0.92

Wave(toe)-cm  Fx-exp(N)  Fx-th(N)
Figure 6.34

EXPERIMENTAL AND THEORETICAL FORCES
T=2.2 s, H=165 mm, d=325 mm, h/d=0.85

Time (s)

Wave(toe)-cm  Fx-exp(N)  Fx-th(N)
Figure 6.35

EXPERIMENTAL AND THEORETICAL FORCES
T=1.9 s, H=158 mm, d=325 mm, h/d=0.85
Figure 6.36

EXPERIMENTAL AND THEORETICAL FORCES
T=1.5 s, H=157 mm, d=325 mm, h/d=0.85

Time (s)

Wave(toe)-cm Fx-exp(N) Fx-th(N)
EXPERIMENTAL AND THEORETICAL FORCES
T=2.2 s, H=173 mm, d=350 mm, h/d=0.79

- Wave(toe)-cm
- Fx-exp(N)
- Fx-th(N)
Figure 6.38

EXPERIMENTAL AND THEORETICAL FORCES
T=1.9 s, H=173 mm, d=350 mm, h/d=0.79

- Wave(toe)-cm
- Fx-exp(N)
- Fx-th(N)
EXPERIMENTAL AND THEORETICAL FORCES
$T=1.5\ s$, $H=172\ mm$, $d=350\ mm$, $h/d=0.79$

- $\Delta$ Wave(toe)-cm
- $\times$ Fx-exp(N)
- $\blacksquare$ Fx-th(N)
Figure 6.40

IMPACT, INERTIA, AND DRAG INDICES
h/B=1, Slope 1:2, Cubical Unit

![Graph showing the relationship between various impact, inertia, and drag indices. The graph has a scatter plot with data points marked by different symbols indicating Cim, C'M, and C'D. The x-axis represents \((H-toe)/gT^2\), and the y-axis represents Cim, C'M, and C'D.](image)
Figure 6.41

IMPACT, INERTIA, AND DRAG INDICES
h/B=1, Slope 1:2, Cubical Unit

STD = 0.1266
STD = 0.0484
STD = 0.0210

Cim, C'M, C'D

(H-toe)/gT^2

Cim (avg)  C'M (avg)  C'D (avg)
Figure 6.42

IMPACT, INERTIA, AND DRAG INDICES
h/B=1, Slope 1:2, Cubical Unit

h/d

Cim, C'M, C'D

Cim (avg) — C'M (avg) — C'D (avg)
Figure 6.43

IMPACT, INERTIA, AND DRAG INDICES
h/B=1, Slope 1:2, Cubical Unit

STD = 0.0484
STD = 0.0210
STD = 0.1266

Cim, C'M, C'D

<table>
<thead>
<tr>
<th></th>
<th>Cim</th>
<th>C'M</th>
<th>C'D</th>
</tr>
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<tbody>
<tr>
<td></td>
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</tbody>
</table>

Cim (avg) C'M (avg) C'D (avg)
Figure 6.44

IMPACT INDEX
h/B=1, Slope 1:2, Cubical Unit

Keulegan-Carpenter Number, KC

Cim

□ T=2.2 s ▲ T=1.9 s ★ T=1.5 s
Figure 6.45

INERTIA INDEX
h/B=1, Slope 1:2, Cubical Unit

Keulegan-Carpenter Number, KC

C'M

-0.3 -0.2 -0.1 0 0.1 0.2 0.3 0.4 0.5 0.6

T=2.2 s, this study ▲ T=1.9 s, this study * T=1.5 s, this study
☑ Torum (T=2.1 s) ☑ Torum (T=1.8 s) ☑ Torum (T=1.5 s)
Figure 6.46

DRAG INDEX
h/B=1, Slope 1:2, Cubical Unit

Keulegan-Carpenter Number, KC

C'D

☐ T=2.2 s, this study  ▲ T=1.9 s, this study  ★ T=1.5 s, this study
⊗ Torum (T=2.1 s)  □ Torum (T=1.8 s)  + Torum (T=1.5 s)
Figure 6.47

IMPACT, INERTIA, AND DRAG INDICES
\( h/B=1 \), Slope 1:2, Cubical Unit

Based on forces maxima

\[ \frac{Cim, C'M, C'D}{(H-toe)/gT^2} \]

\( \square \) Cim  \( \blacktriangle \) C'M  \( \blackstar \) C'D
Figure 6.48

IMPACT, INERTIA, AND DRAG INDICES
h/B=1, Slope 1:2, Cubical Unit

STD = 0.0581
STD = 0.0338
STD = 0.1220

Based on forces maxima

\( \frac{(H\text{-toe})}{gT^2} \)

\( C_{im}, C'M, C'D \)

- Cim
- C'M
- C'D

Cim (avg)  C'M (avg)  C'D (avg)
Figure 6.49

IMPACT, INERTIA, AND DRAG INDICES
h/B=1, Slope 1:2, Cubical Unit

Based on force maxima

Cim, C'M, C'D

h/d

Cim (avg) C'M (avg) C'D (avg)
Figure 6.50

IMPACT, INERTIA, AND DRAG INDICES
h/B=1, Slope 1:2, Cubical Unit

Based on force maxima

Cim, C'M, C'D

STD = 0.0581
STD = 0.0338
STD = 0.1220

h/d

309
Figure 6.51

IMPACT INDEX

$h/B=1$, Slope 1:2, Cubical Unit

Based on force maxima

Keulegan-Carpenter Number, KC

- $T=2.2$ s
- $T=1.9$ s
- $T=1.5$ s
Based on assumption that force maxima are caused by impact
Figure 6.53

IMPACT INDEX
h/B=1, Slope 1:2, Cubical Unit

Based on assumption that force maxima are caused by impact.
INERTIA INDEX
h/B=1, Slope 1:2, Cubical Unit

Based on force maxima

Keulegan-Carpenter Number, KC

-0.3 -0.2 -0.1 0 0.1 0.2 0.3 0.4 0.5 0.6

$T=2.2 \text{ s, this study}$  $T=1.9 \text{ s, this study}$  $T=1.5 \text{ s, this study}$
$\otimes \text{ Torum (}T=2.1 \text{ s)}$  $\times \text{ Torum (}T=1.8 \text{ s)}$  $\ast \text{ Torum (}T=1.5 \text{ s)}$
Figure 6.55

DRAG INDEX
h/B=1, Slope 1:2, Cubical Unit

Based on force maxima

Keulegan-Carpenter Number, KC

CD

□ T=2.2 s, this study ▲ T=1.9 s, this study ★ T=1.5 s, this study
⊗ Torum (T=2.1 s) ⊗ Torum (T=1.8 s) + Torum (T=1.5 s)
PREDICTED AND MEASURED MAXIMUM FORCES  
$h/B=1$, Slope 1:2, Leading Edge Unit

![Graph showing predicted and measured maximum forces.](image)

Based on force maxima

- **□** (F-th/F-exp)
- **▲** F-exp
- **★** F-th

315
EXPERIMENTAL AND THEORETICAL FORCES

$T = 2.2 \text{ s}, \ H = 189 \ \text{mm}, \ d = 375 \ \text{mm}, \ h/d = 0.73$

Time (s)
EXPERIMENTAL AND THEORETICAL FORCES
T=2.2 s, H=179 mm, d=375 mm, h/d=0.73

Time (s)

- Δ Wave(toe)-cm  - F-exp(N)  - Fx-th(N)
EXPERIMENTAL AND THEORETICAL FORCES
$T=2.2 \text{ s}, H=173 \text{ mm}, d=375 \text{ mm}, h/d=0.73$
Figure 6.60

EXPERIMENTAL AND THEORETICAL FORCES
T=1.9 s, H=179 mm, d=375 mm, h/d=0.73

Time (s)
EXPERIMENTAL AND THEORETICAL FORCES
T=1.9 s, H=195 mm, d=375 mm, h/d=0.73

![Graph showing experimental and theoretical forces over time with labels for each data set: Wave(toe)-cm, F-exp(N), and Fx-th(N).]
Figure 6.62

EXPERIMENTAL AND THEORETICAL FORCES
T=1.9 s, H=157 mm, d=375 mm, h/d=0.73

- Wave(toe)-cm
- F-exp(N)
- Fx-th(N)
EXPERIMENTAL AND THEORETICAL FORCES
$T=1.5 \text{ s}, \ H=190 \text{ mm}, \ d=375 \text{ mm}, \ h/d=0.73$
Figure 6.64

EXPERIMENTAL AND THEORETICAL FORCES
T=2.2 s, H=184 mm, d=350 mm, h/d=0.79

Time (s)
EXPERIMENTAL AND THEORETICAL FORCES
T=2.2 s, H=188 mm, d=350 mm, h/d=0.79
EXPERIMENTAL AND THEORETICAL FORCES

$T=2.2 \text{ s}, H=173 \text{ mm}, d=350 \text{ mm}, h/d=0.79$
Figure 6.67

EXPERIMENTAL AND THEORETICAL FORCES
T=1.9 s, H=189 mm, d=350 mm, h/d=0.79

[Graph showing experimental and theoretical forces over time]

- Wave(toe)-cm
- F-exp(N)
- Fx-th(N)
EXPERIMENTAL AND THEORETICAL FORCES

$T=1.9 \text{ s, } H=163 \text{ mm, } d=350 \text{ mm, } h/d=0.79$

---

**Legend:**

- ▲ Wave(toe)-cm
- ☐ F-exp(N)
- ☐ Fx-th(N)
EXPERIMENTAL AND THEORETICAL FORCES
T=1.9 s, H=173 mm, d=350 mm, h/d=0.79

Time (s)
Figure 6.70

EXPERIMENTAL AND THEORETICAL FORCES
\( T=1.5 \text{ s}, \ H=188 \text{ mm}, \ d=350 \text{ mm}, \ h/d=0.79 \)

![Graph showing experimental and theoretical forces over time with legend: Wave(toe)-cm, F-exp(N), Fx-th(N)]
EXPERIMENTAL AND THEORETICAL FORCES

$T = 1.5\, s,\, H = 172\, mm,\, d = 350\, mm,\, h/d = 0.79$

Time (s)

Wave(toe)-cm, F-exp(N), Fx-th(N)
Figure 6.72

EXPERIMENTAL AND THEORETICAL FORCES
T=1.5 s, H=167 mm, d=350 mm, h/d=0.79

- ▲ Wave(toe)-cm
- - F-exp(N)
- □ Fx-th(N)
EXPERIMENTAL AND THEORETICAL FORCES
T=2.2 s, H=188 mm, d=325 mm, h/d=0.85

Time (s)
Figure 6.74

EXPERIMENTAL AND THEORETICAL FORCES
\[ T = 2.2 \text{ s}, \; H = 174 \text{ mm}, \; d = 325 \text{ mm}, \; h/d = 0.85 \]
EXPERIMENTAL AND THEORETICAL FORCES
T=2.2 s, H=165 mm, d=325 mm, h/d=0.85

Time (s)

Wave(toe)-cm  F-exp(N)  Fx-th(N)
EXPERIMENTAL AND THEORETICAL FORCES

T = 1.9 s, H = 185 mm, d = 325 mm, h/d = 0.85

- ▲ Wave(toe)-cm  - ◊ F-exp(N)  - ■ Fx-th(N)

Time (s)

\[18 \quad 18.2 \quad 18.4 \quad 18.6 \quad 18.8 \quad 19 \quad 19.2 \quad 19.4 \quad 19.6 \quad 19.8 \quad 20\]
Figure 6.77

EXPERIMENTAL AND THEORETICAL FORCES
T=1.9 s, H=158 mm, d=325 mm, h/d=0.85

Time (s)

-6 -4 -2 0 2 4 6 8 10

- - Wave(toe)-cm - F-exp(N) - Fx-th(N)
Figure 6.78

EXPERIMENTAL AND THEORETICAL FORCES
T=1.9 s, H=168 mm, d=325 mm, h/d=0.85

- Wave(tce)-cm
- F-exp(N)
- Fx-th(N)
Figure 6.79

EXPERIMENTAL AND THEORETICAL FORCES
T=1.5 s, H=199 mm, d=325 mm, h/d=0.85

[Graph showing experimental and theoretical forces over time with legend: Wave(toe)-cm, F-exp(N), Fx-th(N)]
EXPERIMENTAL AND THEORETICAL FORCES
T=1.5 s, H=180 mm, d=325 mm, h/d=0.85

Time (s)

Wave(toe)-cm | F-exp(N) | Fx-th(N)
Figure 6.81

EXPERIMENTAL AND THEORETICAL FORCES
T=1.5 s, H=157 mm, d=325 mm, h/d=0.85

Time (s)

Wave(toe)-cm  F-exp(N)  Fx-th(N)
Figure 6.82

EXPERIMENTAL AND THEORETICAL FORCES
T=2.2 s, H=172 mm, d=300 mm, h/d=0.92

- ▲ Wave(toe)-cm - × F-exp(N) - ■ Fx-th(N)
Figure 6.83

EXPERIMENTAL AND THEORETICAL FORCES
T=2.2 s, H=193 mm, d=300 mm, h/d=0.92

Time (s)

- Wave(toe)-cm
- F-exp(N)
- Fx-th(N)
Figure 6.84

EXPERIMENTAL AND THEORETICAL FORCES
T=2.2 s, H=166 mm, d=300 mm, h/d=0.92
Figure 6.85

EXPERIMENTAL AND THEORETICAL FORCES
T=1.9 s, H=164 mm, d=300 mm, h/d=0.92

![Graph showing experimental and theoretical forces over time with markers indicating different forces.]

- ▲ Wave(toe)-cm
- □ F-exp(N)
- ■ Fx-th(N)
Figure 6.86

EXPERIMENTAL AND THEORETICAL FORCES
T=1.9 s, H=156 mm, d=300 mm, h/d=0.92

Time (s)

Wave(toe)-cm  F-exp(N)  Fx-th(N)
EXPERIMENTAL AND THEORETICAL FORCES

$T = 1.9 \text{ s, } H = 174 \text{ mm, } d = 300 \text{ mm, } h/d = 0.92$

![Graph showing experimental and theoretical forces over time.](image)
Figure 6.88

EXPERIMENTAL AND THEORETICAL FORCES
T=1.5 s, H=178 mm, d=300 mm, h/d=0.92

Time (s)

- Wave(toe)-cm - F-exp(N) - Fx-th(N)
EXPERIMENTAL AND THEORETICAL FORCES

$T=1.5 \text{ s}, H=160 \text{ mm}, d=300 \text{ mm}, h/d=0.92$
EXPERIMENTAL AND THEORETICAL FORCES
T=1.5 s, H=153 mm, d=300 mm, h/d=0.92

Figure 6.90
Figure 6.91

EXPERIMENTAL AND THEORETICAL FORCES
T = 2.2 s, H = 188 mm, d = 275 mm, h/d = 1

- Δ Wave(toe)-cm
- ✗ Fx-exp(N)
- ■ Fx-th(N)

Time (s)
Figure 6.92

EXPERIMENTAL AND THEORETICAL FORCES
T=2.2 s, H=172 mm, d=275 mm, h/d=1

- Wave(toe)-cm
- F-exp(N)
- Fx-th(N)
Figure 6.93

EXPERIMENTAL AND THEORETICAL FORCES
T=2.2 s, H=152 mm, d=275 mm, h/d=1

Time (s)

△ Wave(toe)-cm  — F-exp(N)  ■ Fx-th(N)
EXPERIMENTAL AND THEORETICAL FORCES

\[ T = 1.9 \text{ s}, \ H = 196 \text{ mm}, \ d = 275 \text{ mm}, \ h/d = 1 \]
Figure 6.95

EXPERIMENTAL AND THEORETICAL FORCES
T=1.9 s, H=171 mm, d=275 mm, h/d=1

\[ \text{Time (s)} \]

- ▲ Wave(toe)-cm
- ━ F-exp(N)
- ■ Fx-th(N)
EXPERIMENTAL AND THEORETICAL FORCES
T=1.9 s, H=164 mm, d=275 mm, h/d=1
EXPERIMENTAL AND THEORETICAL FORCES
T=1.5 s, H=190 mm, d=275 mm, h/d=1
EXPERIMENTAL AND THEORETICAL FORCES

$T = 1.5 \text{ s, } H = 168 \text{ mm, } d = 275 \text{ mm, } h/d = 1$
EXPERIMENTAL AND THEORETICAL FORCES
T=1.5 s, H=163 mm, d=275 mm, h/d=1
Figure 6.100

EXPERIMENTAL AND THEORETICAL FORCES
$T=2.2 \text{ s}, H=195 \text{ mm}, d=250 \text{ mm}, h/d=1.1$

Time (s)

Wave(toe)-cm  F-exp(N)  Fx-th(N)
Figure 6.101

EXPERIMENTAL AND THEORETICAL FORCES

$T=2.2\ s$, $H=178\ mm$, $d=250\ mm$, $h/d=1.1$

![Graph showing experimental and theoretical forces over time with specific time values and force levels.](image-url)
Figure 6.102

EXPERIMENTAL AND THEORETICAL FORCES
T=2.2 s, H=171 mm, d=250 mm, h/d=1.1

Time (s)

Wave(toe)-cm  F-exp(N)  Fx-th(N)
Figure 6.103

EXPERIMENTAL AND THEORETICAL FORCES
T=1.9 s, H=184 mm, d=250 mm, h/d=1.1

- Wave(toe)-cm
- F-exp(N)
- Fx-th(N)
EXPERIMENTAL AND THEORETICAL FORCES
T=1.9 s, H=172 mm, d=250 mm, h/d=1.1

Time (s)

- Wave(toe)-cm
- F-exp(N)
- Fx-th(N)
EXPERIMENTAL AND THEORETICAL FORCES
$T=1.9 \text{ s, } H=178 \text{ mm, } d=250 \text{ mm, } h/d=1.1$
EXPERIMENTAL AND THEORETICAL FORCES

$T=1.5 \text{ s}, H=182 \text{ mm}, d=250 \text{ mm}, h/d=1.1$

![Graph showing experimental and theoretical forces over time with specified parameters.]
EXPERIMENTAL AND THEORETICAL FORCES
T=1.5 s, H=169 mm, d=250 mm, h/d=1.1

Time (s)

Wave(toe)-cm  F-exp(N)  Fx-th(N)
Figure 6.108

EXPERIMENTAL AND THEORETICAL FORCES
T = 1.5 s, H = 166 mm, d = 250 mm, h/d = 1.1

- Wave(toe)-cm
- F-exp(N)
- Fx-th(N)
EXPERIMENTAL AND THEORETICAL FORCES
T=2.2 s, H=192 mm, d=225 mm, h/d=1.22

Time (s)
Figure 6.110

EXPERIMENTAL AND THEORETICAL FORCES
T=2.2 s, H=171 mm, d=225 mm, h/d=1.22

Time (s)

Wave(toe)-cm  F-exp(N)  Fx-th(N)
EXPERIMENTAL AND THEORETICAL FORCES
T=2.2 s, H=164 mm, d=225 mm, h/d=1.22

Time (s)

Wave(toe)-cm  F-exp(N)  Fx-th(N)
EXPERIMENTAL AND THEORETICAL FORCES
T=1.9 s, H=166 mm, d=225 mm, h/d=1.22

- Wave(toe)-cm
- F-exp(N)
- Fx-th(N)

371
Figure 6.113

EXPERIMENTAL AND THEORETICAL FORCES
T=1.9 s, H=182 mm, d=225 mm, h/d=1.22

Wave(toe)-cm  F-exp(N)  Fx-th(N)
EXPERIMENTAL AND THEORETICAL FORCES

$T=1.9 \text{ s}, H=175 \text{ mm}, d=225 \text{ mm}, h/d=1.22$

---

Wave(toe)-cm  F-exp(N)  Fx-th(N)
EXPERIMENTAL AND THEORETICAL FORCES

$T=1.5 \text{ s}, \ H=153 \text{ mm}, \ d=225 \text{ mm}, \ h/d=1.22$

Time (s)

-6 -4 -2 0 2 4 6 8 10 12

- Wave(toe)-cm  - F-exp(N)  - Fx-th(N)

Figure 6.115
EXPERIMENTAL AND THEORETICAL FORCES
T=1.5 s, H=165 mm, d=225 mm, h/d=1.22

- Wave(toe)-cm
- F-exp(N)
- Fx-th(N)
ARMOUR UNIT PLACEMENTS

ANGLE OF ORIENTATION OF THE ARMOUR UNIT

CUBICAL ARMOUR UNIT MODEL

Figure 7.1
Figure 7.2

FORWARD FORCE ON A CUBICAL ARMOUR UNIT
h/B=1, Slope 1:2, Upstream Tip

\[
\frac{F_x}{\gamma D^2 H}
\]

Submergence, h/d

- ▲ T=2.2 s  - □ T=1.9 s  - --- T=1.5 s

---

B

H

d
h

SLOPES 1:2

377
FORWARD FORCE ON A CUBICAL ARMOUR UNIT

$h/B=1$, Slope 1:2, Placement F

$\frac{F_x}{\gamma D^2 H}$

Submergence, $h/d$

- $T=2.2$ s
- $T=1.9$ s
- $T=1.5$ s

SLOPES 1:2
Figure 7.4

FORWARD FORCE ON A CUBICAL ARMOUR UNIT
h/B=1, Slope 1:2, Placement G

\[ \frac{F_x}{\gamma D^2 H} \]

Submergence, h/d

\[ T = 2.2 \text{ s} \quad \square \quad T = 1.9 \text{ s} \quad \bullet \quad T = 1.5 \text{ s} \]

379
Figure 7.5

FORWARD FORCE ON A CUBICAL ARMOUR UNIT
h/B=1, Slope 1:2, Upstream Toe

\[ \frac{F_x}{\gamma D_e^2 H} \]

Submergence, h/d

- ▲ T=2.2 s
- □ T=1.9 s
- - - - T=1.5 s

SLOPES 1:2

380
FORWARD FORCE ON A CUBICAL ARMOUR UNIT
T=2.2 s, h/B=1, Slope 1:2

$F_x \over \gamma D^2 H$

Submergence, h/d

- ▲ Upstream Tip
- ○ Placement F
- ▼ Placement G
- X Upstream Toe

SLOPES 1:2

381
Figure 7.7

FORWARD FORCE ON A CUBICAL ARMOUR UNIT
T=1.9 s, h/B=1, Slope 1:2

\[ \frac{F_y}{\gamma d^2 H} \]

Submergence, \( h/d \)

- ▲ Upstream Tip
- ○ Placement F
- - Placement G
- - - - Upstream Toe

SLOPES 1:2

382
Figure 7.8

FORWARD FORCE ON A CUBICAL ARMOUR UNIT
T=1.5 s, h/B=1, Slope 1:2

\[ \frac{F_x}{\gamma D^2 H} \]

Submergence, h/d

- ▲ Upstream Tip
- ○ Placement F
- ● Placement G
- × Upstream Toe

SLOPES 1:2

383
FORWARD FORCE ON A CUBICAL ARMOUR UNIT

\[ \frac{F_x}{\gamma D_c^2H} \]

\( h/d = 0.73, \ h/B = 1, \ \text{Slope 1:2} \)

\( z/h, \ \text{seaward slope} \)

\[ 0.05 \ 0.06 \ 0.07 \ 0.08 \ 0.09 \ 0.1 \ 0.11 \ 0.12 \ 0.13 \ 0.14 \]

\[ 0 \ 0.1 \ 0.2 \ 0.3 \ 0.4 \ 0.5 \ 0.6 \ 0.7 \ 0.8 \ 0.9 \ 1 \]

\[ \text{T}=2.2 \ \text{s} \quad \text{T}=1.9 \ \text{s} \quad \text{T}=1.5 \ \text{s} \]
Figure 7.10

FORWARD FORCE ON A CUBICAL ARMOUR UNIT
h/d=0.79, h/B=1, Slope 1:2

\[ \frac{F_x}{\gamma D^2 H} \]

\[ \begin{array}{c}
\text{T=2.2 s} \\
\text{T=1.9 s} \\
\text{T=1.5 s}
\end{array} \]

---

385
Figure 7.11

FORWARD FORCE ON A CUBICAL ARMOUR UNIT
h/d = 0.85, h/B = 1, Slope 1:2

\[
\frac{F_x}{\gamma D_e^2 H}
\]

- ▲ T = 2.2 s
- □ T = 1.9 s
- ... T = 1.5 s

SLOPES 1:2

386
FORWARD FORCE ON A CUBICAL ARMOUR UNIT
h/d = 0.92, h/B = 1, Slope 1:2

\[ \frac{F_x}{\gamma D_e^2 H} \]

\[ \square T = 2.2 \text{ s} \quad \triangle T = 1.9 \text{ s} \quad \ldots T = 1.5 \text{ s} \]

---

SLOPES 1:2
FORWARD FORCE ON A CUBICAL ARMOUR UNIT
h/d=1, h/B=1, Slope 1:2

\[ \frac{F_x}{\gamma D_c^2 H} \]

- ▲ T=2.2 s  - - - T=1.9 s  - - - - - - T=1.5 s

SLOPES 1:2
FORWARD FORCE ON A CUBICAL ARMOUR UNIT
h/d=1.1, h/B=1, Slope 1:2

\[ \frac{F_x}{\gamma D_e^2 H} \]

- \( T=2.2 \text{ s} \)
- \( T=1.9 \text{ s} \)
- \( T=1.5 \text{ s} \)
Figure 7.15

FORWARD FORCE ON A CUBICAL ARMOUR UNIT
h/d=1.22, h/B=1, Slope 1:2

\[ \frac{F_x}{\gamma D^2 H} \]

- ▲ T=2.2 s
- □ T=1.9 s
- ⋯ T=1.5 s

SLOPES 1:2
Figure 7.16

FORWARD FORCE ON A CUBICAL ARMOUR UNIT
h/d=1.38, h/B=1, Slope 1:2

$\frac{F_x}{\gamma D_e^2 h}$

- $T=2.2$ s
- $T=1.9$ s
- $T=1.5$ s

SLOPES 1:2
FORWARD FORCE ON A CUBICAL ARMOUR UNIT
h/B=1, Slope 1:2, D.Stream Tip

\[ \frac{F_x}{\gamma DL} \]

Submergence, h/d

\[ \begin{align*}
\Delta T=2.2 \text{ s} & - T=1.9 \text{ s} & - T=1.5 \text{ s} \\
\end{align*} \]

SLOPES 1:2
FORWARD FORCE ON A CUBICAL ARMOUR UNIT
h/B=1, Slope 1:2, Placement C

\[
\frac{F_x}{\gamma D^2 H}
\]

Submergence, h/d

- ▲ T=2.2 s
- ■ T=1.9 s
- □ T=1.5 s

SLOPES 1:2
Figure 7.19

FORWARD FORCE ON A CUBICAL ARMOUR UNIT
h/B=1, Slope 1:2, Placement B

\[ \frac{F_x}{\gamma D^2 H} \]

Submergence, h/d

- \( T=2.2 \) s
- \( T=1.9 \) s
- \( T=1.5 \) s

SLOPES 1:2

394
FORWARD FORCE ON A CUBICAL ARMOUR UNIT
h/B=1, Slope 1:2, D.Stream Toe

\[ \frac{F_x}{YD^2H} \]

Submergence, h/d

- ▲ T=2.2 s
- □ T=1.9 s
- ▼ T=1.5 s

SLOPES 1:2

395
Figure 7.21

FORWARD FORCE ON A CUBICAL ARMOUR UNIT
T=2.2 s, h/B=1, Slope 1:2

\[
\frac{F_x}{\gamma D^2 H}
\]

Submergence, h/d

- ▲ D.Stream Tip
- □ Placement C
- --- Placement B
- -- Placement Toe

SLOPES 1:2
Figure 7.22

FORWARD FORCE ON A CUBICAL ARMOUR UNIT
T=1.9 s, h/B=1, Slope 1:2

\[ \frac{F_x}{\gamma D^2 H} \]

Submergence, h/d

- ▲ D.Stream Tip
- □ Placement C
- -. Placement B
- -- D.Stream Toe

SLOPES 1:2

Fx

Fx
Figure 7.23

FORWARD FORCE ON A CUBICAL ARMOUR UNIT
T=1.5 s, h/B=1, Slope 1:2

\[ \frac{F_x}{\gamma D_z H} \]

Submergence, h/d

D.Stream Tip  Placement C  Placement B  D.Stream Toe

SLOPES 1:2
Figure 7.24

FORWARD FORCE ON A CUBICAL ARMOUR UNIT
h/d = 0.73, h/B = 1, Slope 1:2

\[ \frac{F_x}{\gamma D_c^2 H} \]

\[ \text{\textbullet\, T=2.2 s - - - - - - - - - T=1.9 s - - - - - - - T=1.5 s} \]

\[ \text{SLOPES 1:2} \]
Figure 7.25

FORWARD FORCE ON A CUBICAL ARMOUR UNIT
h/d=0.79, h/B=1, Slope 1:2

\[ \frac{F_x}{\gamma D_c^2 H} \]

\begin{itemize}
  \item \( \triangle T=2.2\ s \)
  \item \( \square T=1.9\ s \)
  \item \( \Box T=1.5\ s \)
\end{itemize}
Figure 7.26

FORWARD FORCE ON A CUBICAL ARMOUR UNIT
h/d = 0.85, h/B = 1, Slope 1:2

\[ \frac{F_x}{\gamma D_e^2 H} \]

- \[ \Delta \] T = 2.2 s
- \[ \square \] T = 1.9 s
- \[ \cdots \] T = 1.5 s

SLOPES 1:2
Figure 7.27

FORWARD FORCE ON A CUBICAL ARMOUR UNIT
h/d = 0.92, h/B = 1, Slope 1:2

\[ \frac{F_z}{\gamma D_c^2 H} \]

- \( T = 2.2 \text{ s} \)
- \( T = 1.9 \text{ s} \)
- \( T = 1.5 \text{ s} \)

\[ d \quad F_x \quad A \quad B \quad C \quad D \quad H \]

SLOPES 1:2
Figure 7.28

FORWARD FORCE ON A CUBICAL ARMOUR UNIT
h/d=1, h/B=1, Slope 1:2

\[ \frac{F_x}{\gamma D_c^2 H} \]

- \( T = 2.2 \) s
- \( T = 1.9 \) s
- \( T = 1.5 \) s

SLOPES 1:2
Figure 7.29

FORWARD FORCE ON A CUBICAL ARMOUR UNIT
h/d = 1.1, h/B = 1, Slope 1:2

\[
\frac{F_x}{\gamma D^2 H}
\]

\[\text{T = 2.2 s} \quad \text{\textbullet} \quad \text{T = 1.9 s} \quad \text{\textsquare} \quad \text{T = 1.5 s} \quad \text{\textldots}
\]

SLOPES 1:2

404
Figure 7.30

FORWARD FORCE ON A CUBICAL ARMOUR UNIT
h/d = 1.22, h/B = 1, Slope 1:2

\[ \frac{F_x}{\gamma D_e^2 H} \]

- T = 2.2 s
- T = 1.9 s
- T = 1.5 s
FORWARD FORCE ON A CUBICAL ARMOUR UNIT
h/d = 1.38, h/B = 1, Slope 1:2

\[
\frac{F_x}{\gamma D_e^2 H}
\]

- ▲ T=2.2 s
- □ T=1.9 s
- ☐ T=1.5 s

\[ \text{SLOPES 1:2} \]
Figure 7.32

BACKWARD FORCE ON A CUBICAL ARMOUR UNIT
h/B=1, Slope 1:2, Upstream Tip

\[ \frac{F_x}{\gamma D^2 H} \]

Submergence, h/d

\[ \Delta \text{ T=2.2 s} \quad \text{-} \text{ T}=1.9 \text{ s} \quad \text{-} \text{ T}=1.5 \text{ s} \]

---

SLOPES 1:2

407
Figure 7.33

FORWARD AND BACKWARD FORCES
h/B=1, Slope 1:2, Seaward Tip

\[ \frac{F_t}{\gamma D^2 H} \]

Submergence, h/d

- ▲ Forward, T=2.2 s
- □ Backward, T=2.2 s

408
Figure 7.34

BACKWARD FORCES
\( h/B = 1, \text{ Slope } 1:2, \text{ Units at points D & E } \)

\[ \frac{F_z}{\gamma D_e H} \]

Submergence, \( h/d \)

\( \cdots \) Point D, \( T = 2.2 \text{ s} \)  \( \cdots \) Point E, \( T = 2.2 \text{ s} \)

SLOPES 1:2

409
Figure 7.35

BACKWARD FORCE ON A CUBICAL ARMOUR UNIT
h/B=1, Slope 1:2, Placement F

\[ \frac{F_x}{gD^2H} \]

Submergence, h/d

\[ \begin{align*}
\text{T=2.2 s} & \quad \text{T=1.9 s} & \quad \text{T=1.5 s}
\end{align*} \]

410
Figure 7.36

BACKWARD FORCE ON A CUBICAL ARMOUR UNIT
h/B=1, Slope 1:2, Placement G

\[
\frac{F_x}{\gamma D^2 H}
\]

Submergence, h/d

- ▲ T=2.2 s  
- □ T=1.9 s  
- ○ T=1.5 s

H

SLOPES 1:2

411
Figure 7.37

BACKWARD FORCE ON A CUBICAL ARMOUR UNIT
h/B=1, Slope 1:2, Upstream Toe

\[
\frac{F_x}{\gamma D^2 H}
\]

Submergence, h/d

\[\Delta \text{ } T=2.2 \text{ s } \quad \square \text{ } T=1.9 \text{ s } \quad \bullet \text{ } T=1.5 \text{ s}\]

---

SLOPES 1:2

---

412

---

H
BACKWARD FORCE ON A CUBICAL ARMOUR UNIT
T=2.2 s, h/B=1, Slope 1:2

\[
\frac{F_x}{\sqrt{\frac{g}{H^2}}}
\]

Submergence, h/d

- ▲ Upstream Tip
- □ Placement F
- ● Placement G
- × Upstream Toe

SLOPES 1:2
Figure 7.39

BACKWARD FORCE ON A CUBICAL ARMOUR UNIT
T=1.9 s, h/B=1, Slope 1:2

\[ \frac{F_x}{\gamma D^2 H} \]

Submergence, h/d

- ▲ Upstream Tip
- □ Placement F
- ● Placement G
- ≪≫ Upstream Toe
Figure 7.40

BACKWARD FORCE ON A CUBICAL ARMOUR UNIT
T=1.5 s, h/B=1, Slope 1:2

![Graph showing force distribution with submergence](image)

- ▲ Upstream Tip
- □ Placement F
- ◯ Placement G
- ⬤ Upstream Toe

![Diagram showing force directions](image)
LONGITUDINAL AND UPLIFT FORCES
$T=1.5\text{ s, } H=199\text{ mm, } h/B=1, h/d=1$

Armour unit placement at the seaward toe (point H)

$F_x \quad F_y$

$d = \text{water depth}$

$H = \text{crest height}$

SLOPES 1:2
BACKWARD FORCE ON A CUBICAL ARMOUR UNIT
h/d = 0.73, h/B = 1, Slope 1:2

\[ \frac{F_x}{\gamma D^2 H} \]

\[ \Delta \ T = 2.2 \text{ s} \quad \square \ T = 1.9 \text{ s} \quad \ldots \quad \square \ T = 1.5 \text{ s} \]
Figure 7.43

BACKWARD FORCE ON A CUBICAL ARMOUR UNIT
h/d=0.79, h/B=1, Slope 1:2

\[
\frac{F_x}{\gamma D_e^2 H}
\]

\[\text{Δ } T=2.2 \text{ s } \quad \square \ T=1.9 \text{ s } \quad \cdots \cdots \ T=1.5 \text{ s}\]
BACKWARD FORCE ON A CUBICAL ARMOUR UNIT
\( h/d = 0.85, h/B = 1, \text{Slope 1:2} \)

\[ \frac{F_x}{\gamma D^2 H} \]

\( T = 2.2 \text{ s} \) \( \square \) \( T = 1.9 \text{ s} \) \( \cdots \cdots \) \( T = 1.5 \text{ s} \)
Figure 7.45

BACKWARD FORCE ON A CUBICAL ARMOUR UNIT
h/d = 0.92, h/B = 1, Slope 1:2

\[
\frac{F_x}{\gamma D_e^2 H}
\]

- ▲ T = 2.2 s
- ■ T = 1.9 s
- ⋄ T = 1.5 s

SLOPES 1:2
Figure 7.46

BACKWARD FORCE ON A CUBICAL ARMOUR UNIT
h/d=1, h/B=1, Slope 1:2

\[ \frac{F_x}{\gamma D_e^2 H} \]

- ▲ T=2.2 s  □ T=1.9 s  □ T=1.5 s

SLOPES 1:2
BACKWARD FORCE ON A CUBICAL ARMOUR UNIT
h/d=1.1, h/B=1, Slope 1:2

\[
\frac{F_z}{\gamma D_s^2 H}
\]

- \( T=2.2 \) s
- \( T=1.9 \) s
- \( T=1.5 \) s

SLOPES 1:2
BACKWARD FORCE ON A CUBICAL ARMOUR UNIT
$h/d=1.22$, $h/B=1$, Slope 1:2

$\frac{F_x}{\gamma D_2^2 H}$

$\blacktriangle$ $T=2.2$ s $\square$ $T=1.9$ s $\cdots$ $T=1.5$ s

SLOPES 1:2
Figure 7.49

BACKWARD FORCE ON A CUBICAL ARMOUR UNIT
h/d=1.38, h/B=1, Slope 1:2

\[ \frac{F_x}{\gamma D c^2 H} \]

\[ \text{\textbullet\quad} T=2.2 \text{ s} \quad \text{- - -} \text{\quad} T=1.9 \text{ s} \quad \text{- - - - - - -} \text{\quad} T=1.5 \text{ s} \]

SLOPES 1:2
Figure 7.50

BACKWARD FORCE ON A CUBICAL ARMOUR UNIT
h/B=1, Slope 1:2, D.Stream Tip

\[ \frac{F_x}{\gamma D^2 H} \]

Submergence, h/d

\[ \Delta T=2.2 \, \text{s} \quad \square T=1.9 \, \text{s} \quad \cdots T=1.5 \, \text{s} \]

425
Figure 7.51

BACKWARD FORCE ON A CUBICAL ARMOUR UNIT
h/B=1, Slope 1:2, Placement C

\[ \frac{F_x}{\gamma D^2 H} \]

Submergence, h/d

- ▲: T=2.2 s
- ○: T=1.9 s
- ⬤: T=1.5 s

SLOPES 1:2
Figure 7.52

BACKWARD FORCE ON A CUBICAL ARMOUR UNIT
h/B=1, Slope 1:2, Placement B

\[ \frac{F_x}{\gamma D^2 H} \]

Submergence, h/d

- ▲ T=2.2 s
- □ T=1.9 s
- ■ T=1.5 s

SLOPES 1:2

427
Figure 7.53

BACKWARD FORCE ON A CUBICAL ARMOUR UNIT
h/B=1, Slope 1:2, D.Stream Toe

\[ \frac{F_x}{\gamma D^2 H} \]

Submergence, h/d

\[ \begin{align*}
\blacksquare & \ T=2.2\ s \\
\square & \ T=1.9\ s \\
\circ & \ T=1.5\ s \\
\end{align*} \]

SLOPES 1:2

428
Figure 7.54

BACKWARD FORCE ON A CUBICAL ARMOUR UNIT
T=2.2 s, h/B=1, Slope 1:2

\[ \frac{F_x}{\gamma D^2 H} \]

Submergence, h/d

- ▲ D.Stream Tip
- □ Placement C
- ● Placement B
- ❋ D.Stream Toe

429
Figure 7.55

BACKWARD FORCE ON A CUBICAL ARMOUR UNIT
T=1.9 s, h/B=1, Slope 1:2

---

$\frac{F_x}{gD^2H}$ vs Submergence, h/d

- ▲ D.Stream Tip
- ◇ Placement C
- ● Placement B
- X D.Stream Toe

---

Diagram showing forces Fx at different positions A, B, C, and D on a slope with a height H and submergence h.
Figure 7.56

BACKWARD FORCE ON A CUBICAL ARMOUR UNIT
T=1.5 s, h/B=1, Slope 1:2

\[ \frac{F_x}{\gamma D^2 H} \]

Submergence, h/d

- ▲ D.Stream Tip
- □ Placement C
- ▣ Placement B
- △ D.Stream Toe

SLOPES 1:2

431
Figure 7.57

BACKWARD FORCE ON A CUBICAL ARMOUR UNIT
h/d=0.73, h/B=1, Slope 1:2

\[
\frac{F_x}{\gamma D^2 H}
\]

- ▲ T=2.2 s
- □ T=1.9 s
- ⋯ T=1.5 s

Diagram showing forces and slopes with labels A, B, C, D and annotations.
Figure 7.58

BACKWARD FORCE ON A CUBICAL ARMOUR UNIT
h/d = 0.79, h/B = 1, Slope 1:2

\[ \frac{F_x}{\gamma D_s^2 H} \]

\[ T = 2.2 \text{ s} \quad T = 1.9 \text{ s} \quad T = 1.5 \text{ s} \]

[Diagram showing forces and positions labeled A, B, C, D, with slopes 1:2]
BACKWARD FORCE ON A CUBICAL ARMOUR UNIT
h/d=0.85, h/B=1, Slope 1:2

\[ \frac{F_x}{\gamma D^2 H} \]

\[ \rightarrow T=2.2 \text{ s} \quad \square T=1.9 \text{ s} \quad \cdots \cdots T=1.5 \text{ s} \]
BACKWARD FORCE ON A CUBICAL ARMOUR UNIT
h/d=0.92, h/B=1, Slope 1:2

\[ \frac{F_x}{\gamma D_c^2 H} \]

\[ \text{T}=2.2 \text{ s} \quad \text{T}=1.9 \text{ s} \quad \text{T}=1.5 \text{ s} \]
Figure 7.61

BACKWARD FORCE ON A CUBICAL ARMOUR UNIT
h/d=1, h/B=1, Slope 1:2

\[
\frac{F_x}{\gamma D^2 H}
\]

\[z/h, \text{ leeward slope}\]

\[0.01 \quad 0.02 \quad 0.03 \quad 0.04 \quad 0.05 \quad 0.06 \quad 0.07 \quad 0.08\]

- \[T=2.2 \text{ s}\]
- \[T=1.9 \text{ s}\]
- \[T=1.5 \text{ s}\]

\[\text{SLOPES 1:2}\]
BACKWARD FORCE ON A CUBICAL ARMOUR UNIT
h/d=1.1, h/B=1, Slope 1:2

\[ \frac{F_x}{\gamma D_c^2 H} \]

z/h, leeward slope

- T=2.2 s
- T=1.9 s
- T=1.5 s

SLOPES 1:2
Figure 7.63

BACKWARD FORCE ON A CUBICAL ARMOUR UNIT

\( h/d = 1.22, \ h/B = 1, \) Slope 1:2

\[ \frac{F_x}{\gamma D^2 H} \]

\( \rightarrow T = 2.2 \ s \quad \square \ T = 1.9 \ s \quad \circ \ T = 1.5 \ s \)

438
Figure 7.64

BACKWARD FORCE ON A CUBICAL ARMOUR UNIT
h/d=1.38, h/B=1, Slope 1:2

\[
\frac{F_x}{\gamma D_e^2 H}
\]

- ▲ T=2.2 s
- □ T=1.9 s
- --- T=1.5 s

--- SLOPES 1:2 ---

439
Figure 7.65

FORCES ON BOTH ARMOUR UNITS
T=1.9 s, H=153 mm, d=325 mm, h/d=0.85

Armour units at the seaward and landward crest tips

Fx-Upstream  Fx-Downstream  Fy-Upstream  Fy-Downstream

SLOPES 1:2

440
FORCES ON BOTH ARMOUR UNITS
T=1.9 s, H=174 mm, d=275 mm, h/d=1

![Graph showing forces over time](image)

- **Fx-Upstream**
- **Fx-Downstream**
- **Fy-Upstream**
- **Fy-Downstream**

441
Figure 7.67

FORCES ON BOTH ARMOUR UNITS
T=1.5 s, H=156 mm, d=225 mm, h/d=1.22

---

---

Fx-Upstream   Fx-Downstream   Fy-Upstream   Fy-Downstream

---

← B →

H

D

E

SLOPES 1:2

442
Figure 7.68a

FORCES ON THE CUBICAL UNIT AT POINT F
T = 1.9 s, H = 188 mm, d = 325 mm, h/d = 0.85

Figure 7.68b

FORCES ON THE CUBICAL UNIT AT POINT F
T = 1.9 s, H = 188 mm, d = 325 mm, h/d = 0.85
Figure 7.69a

FORCES ON THE CUBICAL UNIT AT POINT F
T = 1.9 s, H = 204 mm, d = 275 mm, h/d = 1

Figure 7.69b

FORCES ON THE CUBICAL UNIT AT POINT F
T = 1.9 s, H = 204 mm, d = 275 mm, h/d = 1

444
FORCES ON THE CUBICAL UNIT AT POINT F
T = 1.9 s, H = 185 mm, d = 225 mm, h/d = 1.22

Figure 7.70a

Figure 7.70b

FORCES ON THE CUBICAL UNIT AT POINT F
T = 1.9 s, H = 185 mm, d = 225 mm, h/d = 1.22
FORCES ON THE CUBICAL UNIT AT POINT G
T=1.9 s, H=189 mm, d=325 mm, h/d=0.85

Figure 7.71a

FORCES ON THE CUBICAL UNIT AT POINT G
T=1.9 s, H=189 mm, d=325 mm, h/d=0.85

Figure 7.71b
Figure 7.72a

FORCES ON THE CUBICAL UNIT AT POINT G
T = 1.9 s, H = 174 mm, d = 275 mm, h/d = 1

Figure 7.72b

FORCES ON THE CUBICAL UNIT AT POINT G
T = 1.9 s, H = 203 mm, d = 275 mm, h/d = 1
Figure 7.73a

FORCES ON THE CUBICAL UNIT AT POINT G

T = 1.9 s, H = 188 mm, d = 225 mm, h/d = 1.22

Figure 7.73b

FORCES ON THE CUBICAL UNIT AT POINT G

T = 1.9 s, H = 188 mm, d = 225 mm, h/d = 1.22
FORCES ON THE CUBICAL UNIT AT POINT H
T=1.9 s, H=176 mm, d=325 mm, h/d=0.85
FORCE ON THE CUBICAL UNIT AT POINT H
T = 1.9 s, H = 198 mm, d = 275 mm, h/d = 1

---

$F_x$, $F_y$
Figure 7.76

FORCES ON THE CUBICAL UNIT AT POINT H
T=1.9 s, H=187 mm, d=225 mm, h/d=1.22

![Graph showing force vs time with labels Fx and Fy]

SLOPES 1:2

451
Figure 7.77

UPWARD FORCE ON A CUBICAL ARMOUR UNIT
h/B=1, Slope 1:2, Upstream Tip

\[ \frac{F_y}{\gamma D^2 H} \]

Submergence, h/d

- \( T=2.2 \) s
- \( T=1.9 \) s
- \( T=1.5 \) s

SLOPES 1:2

452
Figure 7.78

UPWARD FORCE ON A CUBICAL ARMOUR UNIT
h/B=1, Slope 1:2, Placement F

\[ \frac{F_y}{\gamma D^2 H} \]

Submergence, h/d

\[ \Delta \quad T=2.2 \text{ s} \quad \square \quad T=1.9 \text{ s} \quad \ldots \quad T=1.5 \text{ s} \]

453
Figure 7.79

UPWARD FORCE ON A CUBICAL ARMOUR UNIT
h/B=1, Slope 1:2, Placement G

\[
\frac{F_y}{\gamma D^2 H}
\]

Submergence, h/d

- ▲ T=2.2 s
- □ T=1.9 s
- ▬▬▬ T=1.5 s

SLOPES 1:2

454
Figure 7.80

UPWARD FORCE ON A CUBICAL ARMOUR UNIT
h/B=1, Slope 1:2, Upstream Toe

\[
\frac{F_y}{\gamma D_e H}
\]

Submergence, h/d

- \( T=2.2 \) s
- \( T=1.9 \) s
- \( T=1.5 \) s

455
Figure 7.81

UPWARD FORCE ON A CUBICAL ARMOUR UNIT
T=2.2 s, h/B=1, Slope 1:2

\[ \frac{F_y}{\gamma D^2 H} \]

Submergence, h/d

- ▲ Upstream Tip
- □ Placement F
- ● Placement G
- ● Upstream Toe

H

SLOPES 1:2

456
Figure 7.82

UPWARD FORCE ON A CUBICAL ARMOUR UNIT

T = 1.9 s, h/B = 1, Slope 1:2

\[ \frac{F_y}{\gamma D^2 H} \]

Submergence, h/d

- ▲ Upstream Tip
- □ Placement F
- --- Placement G
- --- Upstream Toe

SLOPES 1:2

457
Figure 7.83

UPWARD FORCE ON A CUBICAL ARMOUR UNIT
T=1.5 s, h/B=1, Slope 1:2

\[ \frac{F_y}{\gamma D^2 H} \]

Submergence, h/d

- ▲ Upstream Tip
- ◻ Placement F
- ◼ Placement G
- ⌊ Upstream Toe

SLOPES 1:2

458
UPWARD FORCE ON A CUBICAL ARMOUR UNIT

$\frac{F_y}{\gamma D^2 H}$

$h/d = 0.73$, $h/B = 1$, Slope 1:2

\begin{align*}
\text{z/h, seaward slope} & \\
0.1 & \\
0.2 & \\
0.3 & \\
0.4 & \\
0.5 & \\
0.6 & \\
0.7 & \\
0.8 & \\
0.9 & \\
1 & \\
\end{align*}

$\gamma D^2 H$ 

\begin{align*}
T = 2.2 \text{ s} & \\
& \\
T = 1.9 \text{ s} & \\
& \\
T = 1.5 \text{ s} & \\
& \\
\end{align*}
Figure 7.85

UPWARD FORCE ON A CUBICAL ARMOUR UNIT
h/d=0.79, h/B=1, Slope 1:2

\[ \frac{F_y}{\gamma D^2 H} \]

z/h, seaward slope

- ▲ T=2.2 s
- □ T=1.9 s
- ☐ T=1.5 s

SLOPES 1:2
Figure 7.86

UPWARD FORCE ON A CUBICAL ARMOUR UNIT
h/d=0.85, h/B=1, Slope 1:2

\[ \frac{F_y}{\gamma D^2 H} \]

\( z/h, \text{ seaward slope} \)

\( 0.1 \quad 0.2 \quad 0.3 \quad 0.4 \quad 0.5 \quad 0.6 \quad 0.7 \quad 0.8 \quad 0.9 \quad 1 \)

\( 0 \quad 0.02 \quad 0.04 \quad 0.06 \quad 0.08 \quad 0.1 \quad 0.12 \quad 0.14 \quad 0.16 \quad 0.18 \quad 0.2 \)

\( \triangle \ T=2.2 \text{ s} \quad \square \ T=1.9 \text{ s} \quad \cdots \ T=1.5 \text{ s} \)
UPWARD FORCE ON A CUBICAL ARMOUR UNIT
\( h/d = 0.92, \ h/B = 1, \ \text{Slope 1:2} \)

\[
\frac{F_y}{\gamma D_c^2 H}
\]

\( z/h, \ \text{seaward slope} \)

- \( T = 2.2 \ s \)
- \( T = 1.9 \ s \)
- \( T = 1.5 \ s \)
UPWARD FORCE ON A CUBICAL ARMOUR UNIT
h/d=1, h/B=1, Slope 1:2

\[ \frac{F_y}{\gamma D^2 H} \]

- \( T = 2.2 \text{ s} \)
- \( T = 1.9 \text{ s} \)
- \( T = 1.5 \text{ s} \)
Figure 7.89

UPWARD FORCE ON A CUBICAL ARMOUR UNIT

\( h/d = 1.1, \ h/B = 1, \ \text{Slope 1:2} \)

\[
\frac{F_y}{\gamma D_e^2 H}
\]

\( z/h, \ \text{seaward slope} \)

\[
\begin{align*}
\triangle & \ T = 2.2 \ s \\
\square & \ T = 1.9 \ s \\
\cdots & \ T = 1.5 \ s
\end{align*}
\]
Figure 7.90

UPWARD FORCE ON A CUBICAL ARMOUR UNIT
h/d=1.22, h/B=1, Slope 1:2

\[ \frac{F_y}{\gamma D_e^2 H} \]

\( \Delta \ T=2.2 \text{ s} \quad \square \ T=1.9 \text{ s} \quad \cdots \cdots \ T=1.5 \text{ s} \)
UPWARD FORCE ON A CUBICAL ARMOUR UNIT

$\frac{F_y}{\gamma D_e^2 H}$

$h/d = 1.38, h/B = 1$, Slope 1:2

$z/h$, seaward slope

$T = 2.2 \text{ s} \quad T = 1.9 \text{ s} \quad T = 1.5 \text{ s}$
Figure 7.92a

MAXIMUM UPWARD FORCE ON THE UNIT
h/B=1, Slope 1:2, Seaward Tip

Figure 7.92b

MAXIMUM UPWARD FORCE ON THE UNIT
h/B=1, Slope 1:2, Point F
Figure 7.93

FORCES ON THE CUBICAL UNIT AT POINT C
T=1.9 s, H=188 mm, d=325 mm, h/d=0.85

---

468
Figure 7.94

FORCES ON THE CUBICAL UNIT AT POINT C
T = 1.9 s, H = 204 mm, d = 275 mm, h/d = 1

Time (s)

Force (N)

--- Fx --- Fy

\[ d = \text{water depth} \]
\[ h = \text{crest height} \]

SLOPES 1:2

469
Figure 7.95

FORCES ON THE CUBICAL UNIT AT POINT C
T=1.9 s, H=185 mm, d=225 mm, h/d=1.22

![Graph showing forces over time]

Fx — Fy

![Diagram showing slopes and points]

SLOPES 1:2

470
FORCES ON THE CUBICAL UNIT AT POINT B
T=1.9 s, H=189 mm, d=325 mm, h/d=0.85
Figure 7.97

FORCES ON THE CUBICAL UNIT AT POINT B
T=1.9 s, H=203 mm, d=275 mm, h/d=1

---

Force (N)

Time (s)

---

F_x  F_y

---

d = water depth

B SLOPES 1:2

h = crest height

---

472
FORCES ON THE CUBICAL UNIT AT POINT B
$T=1.9\,\text{s},\, H=188\,\text{mm},\, d=225\,\text{mm},\, h/d=1.22$
Figure 7.99

FORCES ON THE CUBICAL UNIT AT POINT A
T = 1.9 s, H = 194 mm, d = 325 mm, h/d = 0.85

Time (s)

Force (N)

Fx  Fy

SLOPES 1:2

474
Figure 7.100

FORCES ON THE CUBICAL UNIT AT POINT A
T=1.9 s, H=198 mm, d=275 mm, h/d=1

![Graph showing forces over time with labels Fx and Fy]
Figure 7.101

FORCES ON THE CUBICAL UNIT AT POINT A
T=1.9 s, H=187 mm, d=225 mm, h/d=1.22

---

F_x  F_y

---

SLOPES 1:2

---

476
Figure 7.102

UPWARD FORCE ON A CUBICAL ARMOUR UNIT
h/B=1, Slope 1:2, D.Stream Tip

\[ \frac{F_y}{\gamma D^2 H} \]

Submergence, h/d

- \( T=2.2 \) s
- \( T=1.9 \) s
- \( T=1.5 \) s

SLOPES 1:2
Figure 7.103

UPWARD FORCE ON A CUBICAL ARMOUR UNIT
h/B=1, Slope 1:2, Placement C

\[ \frac{F_y}{yD^2H} \]

Submergence, h/d

- ▲ T=2.2 s
- □ T=1.9 s
- --- T=1.5 s

\[ F_y \]

\[ B \]

\[ d \]

\[ h \]

SLOPES 1:2
UPWARD FORCE ON A CUBICAL ARMOUR UNIT
h/B=1, Slope 1:2, Placement B

\[
\frac{F_y}{\gamma D_e^2 H}
\]

Submergence, h/d

- \( T = 2.2 \text{ s} \)
- \( T = 1.9 \text{ s} \)
- \( T = 1.5 \text{ s} \)

SLOPES 1:2
Figure 7.105

UPWARD FORCE ON A CUBICAL ARMOUR UNIT
h/B=1, Slope 1:2, D.Stream Toe

$\frac{F_y}{\gamma d^2 H}$

Submergence, h/d

- $T=2.2$ s
- $T=1.9$ s
- $T=1.5$ s

SLOPES 1:2
Figure 7.106

UPWARD FORCE ON A CUBICAL ARMOUR UNIT
T=2.2 s, h/B=1, Slope 1:2

\[ \frac{F_y}{\gamma D^2 H} \]

Submergence, h/d

- ▲ D.Stream Tip
- □ Placement C
- □ Placement B
- — — D.Stream Toe

Diagram showing forces and submergence relationship.
UPWARD FORCE ON A CUBICAL ARMOUR UNIT

$T = 1.9 \text{ s, } h/B = 1, \text{ Slope 1:2}$

Submergence, $h/d$

$\frac{F_y}{\gamma D^2 H}$
Figure 7.108

UPWARD FORCE ON A CUBICAL ARMOUR UNIT
T = 1.5 s, h/B = 1, Slope 1:2

\[
\frac{F_y}{\gamma_d^2 H}
\]

Submergence, h/d

D.Stream Tip
Placement C
Placement B
D.Stream Toe

\[
\begin{align*}
A & \quad \text{SLOPES 1:2} \\
B & \\
C & \\
D & \\
F_y & \\
F_y & \\
F_y & \\
F_y &
\end{align*}
\]
UPWARD FORCE ON A CUBICAL ARMOUR UNIT
h/d=0.73, h/B=1, Slope 1:2

\[ \frac{F_y}{\gamma D_c^2 H} \]

\begin{itemize}
  \item \( \bullet \) T=2.2 s
  \item \( \square \) T=1.9 s
  \item \( \cdots \) T=1.5 s
\end{itemize}
Figure 7.110

UPWARD FORCE ON A CUBICAL ARMOUR UNIT
h/d=0.79, h/B=1, Slope 1:2

\[ \frac{F_y}{\gamma D_i^2 H} \]

\[ T=2.2 \text{ s} \quad \square \quad T=1.9 \text{ s} \quad \cdots \cdots \quad T=1.5 \text{ s} \]
Figure 7.111

UPWARD FORCE ON A CUBICAL ARMOUR UNIT
h/d=0.85, h/B=1, Slope 1:2

\[ \frac{F_y}{\gamma D_e^2 H} \]

T=2.2 s  T=1.9 s  T=1.5 s
UPWARD FORCE ON A CUBICAL ARMOUR UNIT
h/d=0.92, h/B=1, Slope 1:2

\[
\frac{F_y}{\gamma D_e^2 H}
\]

\[z/h, \text{ leeward slope}\]

\\[\text{T}=2.2 \text{ s} \quad \text{T}=1.9 \text{ s} \quad \text{T}=1.5 \text{ s}\]

SLOPES 1:2
Figure 7.113

UPWARD FORCE ON A CUBICAL ARMOUR UNIT
h/d=1, h/B=1, Slope 1:2

\[ \frac{F_y}{\gamma D^2 H} \]

\[ z/h, \text{ leeward slope} \]

- \( T=2.2 \text{ s} \)
- \( T=1.9 \text{ s} \)
- \( T=1.5 \text{ s} \)

SLOPES 1:2
UPWARD FORCE ON A CUBICAL ARMOUR UNIT

\[
\frac{F_y}{\gamma D^2_e H}
\]

\(z/h, \text{ leeward slope}\)

\(0 \quad 0.02 \quad 0.04 \quad 0.06 \quad 0.08 \quad 0.1 \quad 0.12 \quad 0.14 \quad 0.16\)

\[T=2.2 \ s \quad T=1.9 \ s \quad T=1.5 \ s\]
Figure 7.115

UPWARD FORCE ON A CUBICAL ARMOUR UNIT
h/d=1.22, h/B=1, Slope 1:2

\[ \frac{F_y}{\gamma D_r^2 H} \]

\[
\begin{align*}
\text{z/h, leeward slope} \\
0 & \quad 0.1 & \quad 0.2 & \quad 0.3 & \quad 0.4 & \quad 0.5 & \quad 0.6 & \quad 0.7 & \quad 0.8 & \quad 0.9 & \quad 1 \\
0 & \quad 0.05 & \quad 0.1 & \quad 0.15 & \quad 0.2 & \quad 0.25
\end{align*}
\]

- \( \text{T}=2.2 \text{ s} \)
- \( \text{T}=1.9 \text{ s} \)
- \( \text{T}=1.5 \text{ s} \)

SLOPES 1:2

490
Figure 7.116

UPWARD FORCE ON A CUBICAL ARMOUR UNIT
h/d=1.38, h/B=1, Slope 1:2

\[ \frac{F_y}{\gamma D_c^2 H} \]

z/h, leeward slope

\[ T=2.2 \text{ s} \quad \square \ T=1.9 \text{ s} \quad \cdots \quad \cdot \cdot \cdot \ T=1.5 \text{ s} \]
Figure 7.117

FORCES ON THE LEADING EDGE ARMOUR UNIT
T=2.2 s, H=196 mm, d=200 mm, h/d=1.38

<table>
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<tr>
<th>Force (N)</th>
<th>Time (s)</th>
</tr>
</thead>
<tbody>
<tr>
<td>14</td>
<td>10</td>
</tr>
<tr>
<td>12</td>
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<td>26</td>
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<tr>
<td>-2</td>
<td>28</td>
</tr>
<tr>
<td>-2</td>
<td>30</td>
</tr>
</tbody>
</table>

---

F_x  F_y

SLOPES 1:2

h d
Figure 7.118

FORCES ON THE LEADING EDGE ARMOUR UNIT
T = 1.9 s, H = 169 mm, d = 200 mm, h/d = 1.38

Time (s)

Force (N)

--- Fx --- Fy

SLOPES 1:2
FORCES ON THE LEADING EDGE ARMOUR UNIT
T=1.5 s, H=142 mm, d=200 mm, h/d=1.38

\[\text{Force (N)}\]

\[\text{Time (s)}\]

---

\[\text{Fx} \quad \text{Fy}\]

\[\text{SLOPES 1:2}\]
DOWNWARD FORCE ON A CUBICAL ARMOUR UNIT

$h/B = 1$, Slope $1:2$, Upstream Tip

\[ \frac{F_y}{\gamma D^2 H} \]

Submergence, $h/d$

\[ \rightarrow T = 2.2 \text{ s} \quad \square T = 1.9 \text{ s} \quad \cdots T = 1.5 \text{ s} \]

SLOPES $1:2$

495
Figure 7.121

DOWNWARD FORCE ON A CUBICAL ARMOUR UNIT
h/B=1, Slope 1:2, Placement F

\[
\frac{F_y}{\rho g^2 H^2}
\]

Submergence, \( h/d \)

\[\begin{align*}
&\text{\textbullet T=2.2 s} & \text{\square T=1.9 s} & \text{\dash T=1.5 s}
\end{align*}\]

SLOPES 1:2
Figure 7.122

DOWNWARD FORCE ON A CUBICAL ARMOUR UNIT
h/B=1, Slope 1:2, Placement G

\[ \frac{F_y}{\gamma D^2 H} \]

Submergence, h/d

\[ \Delta T=2.2 \text{ s} \quad \square T=1.9 \text{ s} \quad \cdots \cdots T=1.5 \text{ s} \]

SLOPES 1:2

497
DOWNWARD FORCE ON A CUBICAL ARMOUR UNIT
h/B=1, Slope 1:2, Upstream Toe

\( \frac{F_y}{\gamma d^2 H} \)

Submergence, h/d

- ▲ T=2.2 s - □ T=1.9 s - ----- T=1.5 s

498
Figure 7.124

DOWNWARD FORCE ON A CUBICAL ARMOUR UNIT
T=2.2 s, h/B=1, Slope 1:2

\[
\frac{F_y}{γD^2H}
\]

Submergence, h/d

- ▲ Upstream Tip
- ○ Placement F
- --- Placement G
- --- Upstream Toe

SLOPES 1:2

499
Figure 7.125

DOWNWARD FORCE ON A CUBICAL ARMOUR UNIT
T=1.9 s, h/B=1, Slope 1:2

\[
\frac{F_y}{\gamma D^2 H}
\]

Submergence, h/d

- ▲ Upstream Tip
- □ Placement F
- ⬇️ Placement G
- ➡️ Upstream Toe

Diagram showing force distribution and submergence.
Figure 7.126

DOWNWARD FORCE ON A CUBICAL ARMOUR UNIT
T=1.5 s, h/B=1, Slope 1:2

$\frac{F_y}{\gamma D^2 H}$

Submergence, h/d

- ▲ Upstream Tip
- □ Placement F
- ◯ Placement G
- × Upstream Toe

SLOPES 1:2
Figure 7.127

DOWNWARD FORCE ON A CUBICAL ARMOUR UNIT
h/d = 0.73, h/B = 1, Slope 1:2

\[ \frac{F_y}{\gamma D_e^2 H} \]

\[ \begin{align*}
\triangle & T = 2.2 \text{ s} \\
\square & T = 1.9 \text{ s} \\
\cdots & T = 1.5 \text{ s}
\end{align*} \]
Figure 7.128

DOWNWARD FORCE ON A CUBICAL ARMOUR UNIT

$h/d=0.79$, $h/B=1$, Slope 1:2

$\frac{F_y}{\gamma D_d^2 H}$

$\blacktriangle T=2.2 \text{ s}$  $\square T=1.9 \text{ s}$  $\cdots\cdots\cdots\cdots\cdots T=1.5 \text{ s}$
Figure 7.129

DOWNWARD FORCE ON A CUBICAL ARMOUR UNIT

$h/d=0.85$, $h/B=1$, Slope 1:2

$\frac{F_y}{\gamma D_s^2 H}$

$\uparrow$ $T=2.2 \, s$  $\square$ $T=1.9 \, s$  $\blacksquare$ $T=1.5 \, s$

Diagram showing force components and locations labeled E, F, G, H.
Figure 7.130

DOWNWARD FORCE ON A CUBICAL ARMOUR UNIT
h/d=0.92, h/B=1, Slope 1:2

\[ \frac{F_y}{\gamma D_e^2 H} \]

\( \rightarrow \) T=2.2 s \( \rightarrow \) T=1.9 s \( \rightarrow \) T=1.5 s
DOWNWARD FORCE ON A CUBICAL ARMOUR UNIT

$h/d = 1$, $h/B = 1$, Slope 1:2

$\frac{F_y}{\gamma D_e^2 H}$

$- \blacktriangle T = 2.2 \text{ s} \quad \square T = 1.9 \text{ s} \quad \cdots T = 1.5 \text{ s}$
DOWNWARD FORCE ON A CUBICAL ARMOUR UNIT
h/d = 1.1, h/B = 1, Slope 1:2

\[ \frac{F_y}{\gamma D_e^2 H} \]

- \( \triangle T = 2.2 \text{ s} \)
- \( \square T = 1.9 \text{ s} \)
- \( \bullet T = 1.5 \text{ s} \)
Figure 7.133

DOWNWARD FORCE ON A CUBICAL ARMOUR UNIT
h/d = 1.22, h/B = 1, Slope 1:2

\[ \frac{F_y}{\gamma D_e^2 H} \]

- \( \Delta \) T = 2.2 s
- \( \square \) T = 1.9 s
- \( \cdots \) T = 1.5 s

SLOPES 1:2

508
Figure 7.134

DOWNWARD FORCE ON A CUBICAL ARMOUR UNIT
h/d=1.38, h/B=1, Slope 1:2

\[
\frac{F_y}{\gamma D^2 H}
\]

- ▲ T=2.2 s  - □ T=1.9 s  - G T=1.5 s

SLOPES 1:2
DOWNWARD FORCE ON A CUBICAL ARMOUR UNIT
h/B=1, Slope 1:2, D.Stream Tip

\[ \frac{F_y}{yD_e^2 H} \]

Submergence, h/d

- \( T=2.2 \text{ s} \)
- \( T=1.9 \text{ s} \)
- \( T=1.5 \text{ s} \)

SLOPES 1:2
Figure 7.136

DOWNWARD FORCE ON A CUBICAL ARMOUR UNIT
h/B=1, Slope 1:2, Placement C

\[ \frac{F_y}{\gamma D^2 H} \]

Submergence, h/d

- \( T=2.2 \) s
- \( T=1.9 \) s
- \( T=1.5 \) s

SLOPES 1:2
Figure 7.137

DOWNWARD FORCE ON A CUBICAL ARMOUR UNIT
h/B=1, Slope 1:2, Placement B

\[ \frac{F_y}{\gamma D_i H} \]

Submergence, h/d

- \( T=2.2 \) s
- \( T=1.9 \) s
- \( T=1.5 \) s

Diagram showing the force distribution and submergence relationship for different time periods.

SLOPES 1:2
Figure 7.138

DOWNWARD FORCE ON A CUBICAL ARMOUR UNIT
h/B=1, Slope 1:2, D.Stream Toe

\[ \frac{F_y}{\gamma D_e^2 H} \]

Submergence, h/d

- ▲ T=2.2 s
- ■ T=1.9 s
- ● T=1.5 s

---

SLOPES 1:2

513
Figure 7.139

DOWNWARD FORCE ON A CUBICAL ARMOUR UNIT
T = 2.2 s, h/B = 1, Slope 1:2

\[ \frac{F_y}{\gamma D^2 H} \]

Submergence, h/d

- ▲ D.Stream Tip
- Placement C
- Placement B
- ⋄ D.Stream Toe

SLOPES 1:2
Figure 7.140

DOWNWARD FORCE ON A CUBICAL ARMOUR UNIT
T=1.9 s, h/B=1, Slope 1:2

\[ \frac{F_y}{\gamma D_e^2 H} \]

Submergence, h/d

- ▲ D.Stream Tip
- □ Placement C
- ● Placement B
- -- D.Stream Toe

SLOPE 1:2

515
Figure 7.141

DOWNWARD FORCE ON A CUBICAL ARMOUR UNIT
T=1.5 s, h/B=1, Slope 1:2

\[ \frac{F_y}{\gamma D_e^2 H} \]

Submergence, \( h/d \)

- ▲ D.Stream Tip
- ◇ Placement C
- ● Placement B
- ● D.Stream Toe

Diagram showing forces and submergence with labels A, B, C, D, and SLOPES 1:2.
Figure 7.142

DOWNWARD FORCE ON A CUBICAL ARMOUR UNIT
h/d=0.73, h/B=1, Slope 1:2

\[ \frac{F_y}{\gamma D_c^2 H} \]

\[ \rightarrow T=2.2 \text{ s} \quad \square T=1.9 \text{ s} \quad \cdots T=1.5 \text{ s} \]
Figure 7.143

DOWNWARD FORCE ON A CUBICAL ARMOUR UNIT
h/d=0.79, h/B=1, Slope 1:2

\[ \frac{F_y}{\gamma D^2 H} \]

\[ \text{z/h, leeward slope} \]

\[ 0 \rightarrow 0.005 \rightarrow 0.01 \rightarrow 0.015 \rightarrow 0.02 \rightarrow 0.025 \rightarrow 0.03 \rightarrow 0.035 \rightarrow 0.04 \rightarrow 0.045 \rightarrow 0.05 \rightarrow 0.055 \]

- \( T=2.2 \text{ s} \)
- \( T=1.9 \text{ s} \)
- \( T=1.5 \text{ s} \)
DOWNWARD FORCE ON A CUBICAL ARMOUR UNIT
h/d=0.85, h/B=1, Slope 1:2

\[ \frac{F_y}{\gamma D^2 H} \]

- \( T=2.2 \text{ s} \)
- \( T=1.9 \text{ s} \)
- \( T=1.5 \text{ s} \)
Figure 7.145

DOWNWARD FORCE ON A CUBICAL ARMOUR UNIT
h/d=0.92, h/B=1, Slope 1:2

\[ \frac{F_y}{\gamma D^2 H} \]

- ▲ T=2.2 s
- □ T=1.9 s
- ◯ T=1.5 s

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DOWNWARD FORCE ON A CUBICAL ARMOUR UNIT
h/d=1, h/B=1, Slope 1:2

\[ \frac{F_y}{\gamma D_c^2 H} \]

- \( T = 2.2 \text{ s} \)
- \( T = 1.9 \text{ s} \)
- \( T = 1.5 \text{ s} \)
Figure 7.147

DOWNWARD FORCE ON A CUBICAL ARMOUR UNIT
\( h/d = 1.1, \ h/B = 1, \) Slope 1:2

\[ \frac{F_y}{\gamma D^2 H} \]

\( z/h, \) leeward slope

- \( \Delta \) T = 2.2 s
- \( \square \) T = 1.9 s
- \( \cdots \) T = 1.5 s

SLOPES 1:2

522
Figure 7.148

DOWNWARD FORCE ON A CUBICAL ARMOUR UNIT
h/d=1.22, h/B=1, Slope 1:2

\[ \frac{F_y}{\gamma D_i^2 H} \]

- \( T=2.2 \text{ s} \)
- \( T=1.9 \text{ s} \)
- \( T=1.5 \text{ s} \)
DOWNWARD FORCE ON A CUBICAL ARMOUR UNIT
h/d=1.38, h/B=1, Slope 1:2

\[
\frac{F_y}{\gamma D^2 H}
\]

- ▲ T=2.2 s
- ○ T=1.9 s
- ◯ T=1.5 s

524
Figure 7.150

HORIZONTAL FORCES ON BOTH CUBICAL UNITS
T=1.9 s, H=181 mm, d=300 mm, h/d=0.92

![Graph showing horizontal forces over time]

- Leading edge
- Behind leading edge

SLOPES 1:2

525
Figure 7.151

VERTICAL FORCES ON BOTH CUBICAL UNITS

$T=1.9 \text{ s}, \ H=181 \text{ mm}, \ d=300 \text{ mm}, \ h/d=0.92$

- **Force (N)**
- **Time (s)**

Graph showing forces over time with labels for leading and behind leading edges.

Diagram illustrating slopes 1:2.
HORIZONTAL FORCES ON BOTH CUBICAL UNITS
T=1.9 s, H=190 mm, d=275 mm, h/d=1
Figure 7.153

VERTICAL FORCES ON BOTH CUBICAL UNITS
T=1.9 s, H=190 mm, d=275 mm, h/d=1

---

Leading edge  Behind leading edge

---

d = water depth  SLOPES 1:2  h = crest height

---

528
HORIZONTAL FORCES ON BOTH CUBICAL UNITS
T=1.9 s, H=178 mm, d=225 mm, h/d=1.22

Force (N)

Time (s)

--- Leading edge ---
--- Behind leading edge ---

SLOPES 1:2

529
Figure 7.155

VERTICAL FORCES ON BOTH CUBICAL UNITS
T=1.9 s, H=178 mm, d=225 mm, h/d=1.22

![Graph showing vertical forces over time](image)

- Leading edge
- Behind leading edge

---

530
Figure 7.156

FORWARD FORCES ON THE CUBICAL UNITS
h/B=1, Slope 1:2, T=2.2 s, Tandem

\[ \frac{F_x}{\gamma D^2 H} \]

Submergence, h/d

- ▲ Leading edge
- □ Behind leading edge

SLOPES 1:2

531
Figure 7.157

FORWARD FORCES ON THE CUBICAL UNITS
h/B=1, Slope 1:2, T=1.9 s, Tandem

\[ \frac{F_x}{\gamma d^2 H} \]

Submergence, h/d

- ▲ Leading edge
- □ Behind leading edge

SLOPES 1:2

532
Figure 7.158

FORWARD FORCES ON THE CUBICAL UNITS
h/B=1, Slope 1:2, T=1.5 s, Tandem

\[ \frac{F_x}{\gamma D^2 H} \]

Submergence, h/d

- ▲ Leading edge
- ■ Behind leading edge

SLOPES 1:2
FORWARD FORCES ON THE CUBICAL UNITS
h/B=1, Slope 1:2, T=2.2 s, Tandem

---

FORWARD FORCES ON THE CUBICAL UNITS
h/B=1, Slope 1:2, T=1.9 s, Tandem

---

FORWARD FORCES ON THE CUBICAL UNITS
h/B=1, Slope 1:2, T=1.5 s, Tandem
UPWARD FORCES ON THE CUBICAL UNITS
h/B=1, Slope 1:2, T=2.2 s, Tandem

Submergence, h/d

$\frac{F_y}{\gamma D_e^2 H}$

Leading edge

Behind leading edge

SLOPES 1:2

535
UPWARD FORCES ON THE CUBICAL UNITS
$h/B=1$, Slope 1:2, $T=1.9$ s, Tandem

\[ \frac{F_L}{\gamma D^2 H} \]

Submergence, $h/d$

- $\blacktriangle$ Leading edge
- $\square$ Behind leading edge

SLOPES 1:2
UPWARD FORCES ON THE CUBICAL UNITS
h/B=1, Slope 1:2, T=1.5 s, Tandem
Figure 7.164

EFFECT OF SLOPE ON THE FORWARD FORCE
h/B=1, T=2.2 s, Leading Edge Unit

\[ \frac{F_x}{H^2 d^4} \]

Submergence, h/d

- ▲ Slope 1:2
- □ Slope 1:3
- ■ Slope 1:4

539
Figure 7.165

EFFECT OF SLOPE ON THE FORWARD FORCE
h/B = 1, T = 1.9 s, Leading Edge Unit

\[ \frac{F}{\gamma D^2 H} \]

Submergence, h/d

- Slope 1:2
- Slope 1:3
- Slope 1:4
Figure 7.166

EFFECT OF SLOPE ON THE FORWARD FORCE
h/B=1, T=1.5 s, Leading Edge Unit

\[
\frac{F_x}{\gamma D^2 H}
\]

Submergence, h/d

- ▲ Slope 1:2
- □ Slope 1:3
- ● Slope 1:4

\[d \quad h \quad E \quad B \quad H\]
EFFECT OF SLOPE ON THE UPWARD FORCE
h/B=1, T=2.2 s, Leading Edge Unit

Submergence, h/d

\[ \frac{F_y}{\gamma D^2 H} \]

- ▲ Slope 1:2
- □ Slope 1:3
- ● Slope 1:4

542
Figure 7.168

EFFECT OF SLOPE ON THE UPWARD FORCE
\( h/B = 1, \ T = 1.9 \text{ s}, \) Leading Edge Unit

\[ \frac{F_y}{\gamma D^2 h} \]

Submergence, \( h/d \)

- ▲ ▲ ▲ ▲ ▲ Slope 1:2
- ◯ ◯ ◯ ◯ ◯ Slope 1:3
- □ □ □ □ □ Slope 1:4

543
Figure 7.169

EFFECT OF SLOPE ON THE UPWARD FORCE
h/B=1, T=1.5 s, Leading Edge Unit

Submergence, h/d

\[ \frac{F_y}{y_d^2 H} \]

- ▲ Slope 1:2
- □ Slope 1:3
- ■ Slope 1:4

544
Figure 7.170

EFFECT OF SLOPE ON THE FORWARD FORCE

$h/B=1$, $T=2.2$ s, Landward Edge Unit

$\frac{F_x}{D^2H}$ against $h/d$

- ▲ Slope 1:2
- ○ Slope 1:3
- □ Slope 1:4

545
EFFECT OF SLOPE ON THE FORWARD FORCE
\(h/B=1, T=1.9\text{ s},\) Landward Edge Unit

\[
\frac{F_x}{YD^2H} \text{ vs. Submergence, } h/d
\]

- ▲ Slope 1:2
- □ Slope 1:3
- ■ Slope 1:4
Figure 7.172

EFFECT OF SLOPE ON THE FORWARD FORCE
h/B=1, T=1.5 s, Landward Edge Unit

\[ \frac{F_d}{\gamma D_e H} \]

Submergence, h/d

- ▲ Slope 1:2
- □ Slope 1:3
- ◼ Slope 1:4

547
Figure 7.173

EFFECT OF SLOPE ON THE UPWARD FORCE
h/B=1, T=2.2 s, Landward Edge Unit

\[ \frac{F_y}{YD^2H} \]

Submergence, h/d

- ▲ Slope 1:2
- □ Slope 1:3
- ● Slope 1:4

548
EFFECT OF SLOPE ON THE UPWARD FORCE
h/B=1, T=1.9 s, Landward Edge Unit

\[
\frac{F_y}{\gamma D^2 H}
\]

Submergence, h/d

- ▲ Slope 1:2
- □ Slope 1:3
- ■ Slope 1:4

\[\text{Figure 7.174}\]

549
Figure 7.175

EFFECT OF SLOPE ON THE UPWARD FORCE
h/B = 1, T = 1.5 s, Landward Edge Unit

\[ \frac{F_z}{\gamma D^2 H} \]

Submergence, h/d

0.04 0.06 0.08 0.1 0.12 0.14 0.16 0.18 0.2

0.75 0.8 0.85 0.9 0.95 1 1.05 1.1 1.15 1.2 1.25

△ Slope 1:2 □ Slope 1:3 ■ Slope 1:4

\[ \rightarrow B \rightarrow \]

\[ \text{SLOPE 1:2 SLOPE 1:3 SLOPE 1:4} \]

550
Fig. 8.1
Figure 8.3

TOTAL FORCE ENVELOPE
T=2.2 s, H=163 mm, d=350 mm, h/d=0.79
Figure 8.4

TOTAL FORCE ENVELOPE

\[ T = 1.9 \text{ s, } H = 154 \text{ mm, } d = 350 \text{ mm, } h/d = 0.79 \]
Figure 8.5

TOTAL FORCE ENVELOPE
T=1.5 s, H=155 mm, d=350 mm, h/d=0.79
Figure 8.6

TOTAL FORCE ENVELOPE
T=2.2 s, H=175 mm, d=225 mm, h/d=1.22
Figure 8.7

TOTAL FORCE ENVELOPE
T=1.9 s, H=175 mm, d=225 mm, h/d=1.22
Figure 8.8

TOTAL FORCE ENVELOPE
T=1.5 s, H=172 mm, d=225 mm, h/d=1.22

Force (N)

Angle (Degree)
Figure 8.9

STABILITY OF THE CUBICAL UNIT
T = 2.2 s, H = 163 mm, d = 350 mm, h/d = 0.79

![Graph showing stability of a cubical unit with time and instability number](image)

- Sliding
- Overturning
Figure 8.10

STABILITY OF THE CUBICAL UNIT
T = 1.9 s, H = 154 mm, d = 350 mm, h/d = 0.79

Instability Number

Time (s)

-0.3 -0.2 -0.1 0 0.1 0.2 0.3 0.4 0.5

- - Sliding ▲ Overturning
Figure 8.11

STABILITY OF THE CUBICAL UNIT
T = 1.5 s, H = 155 mm, d = 350 mm, h/d = 0.79

Instability Number

Time (s)

-0.2 -0.1 0 0.1 0.2 0.3 0.4 0.5

- Sliding  Overturning
Figure 8.12

STABILITY OF THE CUBICAL UNIT
T = 2.2 s, H = 175 mm, d = 225 mm, h/d = 1.22

Instability Number

Time (s)

- Sliding
- Overturning

562
Figure 8.13

STABILITY OF THE CUBICAL UNIT
T=1.9 s, H=175 mm, d=225 mm, h/d=1.22

Instability Number

Time (s)

- Sliding
- Overturning

563
Figure 8.14

STABILITY OF THE CUBICAL UNIT
T=1.5 s, H=172 mm, d=225 mm, h/d=1.22

![Graph showing instability number over time]

- Sliding
- Overturning
Figure 8.15

STABILITY OF CUBICAL ARMOUR UNIT MODEL
T=2.2 s, H=163 mm, d=350 mm, h/d=0.79

![Graph showing Instability Number vs Angle (Degree) with symbols for Sliding and Overturning.]

- Sliding
- Overturning
STABILITY OF CUBICAL ARMOUR UNIT MODEL
T=1.9 s, H=154 mm, d=350 mm, h/d=0.79

Figure 8.16

- Sliding
- Overturning
Figure 8.17

STABILITY OF CUBICAL ARMOUR UNIT MODEL
$T=1.5\text{ s}, H=155\text{ mm}, d=350\text{ mm}, h/d=0.79$

![Graph showing instability number vs. angle (degree) for sliding and overturning events.](image-url)

- □ Sliding
- ▲ Overturning
STABILITY OF CUBICAL ARMOUR UNIT MODEL
T=2.2 s, H=175 mm, d=225 mm, h/d=1.22
Figure 8.19

STABILITY OF CUBICAL ARMOUR UNIT MODEL
T=1.9 s, H=175 mm, d=225 mm, h/d=1.22

- Instability Number
- Angle (Degree)

□ Sliding ▲ Overturning
Figure 8.20

STABILITY OF CUBICAL ARMOUR UNIT MODEL
T=1.5 s, H=172 mm, d=225 mm, h/d=1.22

![Graph showing stability of cubical armour unit model with data points indicating sliding and overturning.]

- Sliding
- Overturning
STABILITY OF THE CUBICAL UNIT
h/B=1, Slope 1:2, Sliding

STABILITY OF THE CUBICAL UNIT
h/B=1, Slope 1:2, Overturning

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The author was born on September 29, 1954, in Surakarta, Indonesia. In 1973, he completed his secondary school, and in 1974 he joined the Department of Civil Engineering, Gajah Mada University, Yogyakarta, Indonesia. In 1979, he completed his Bachelor of Civil Engineering, and in 1981 he graduated from Gajah Mada University with a degree of Civil Engineer. After graduation, the author spent 3 years working in The Citanduy River Basin Development Project, West Java, Indonesia, as a supervisor of the construction of The Manganti Movable Weir and The Manganti Bridge connecting Central-West Java. In 1983 he joined Immanuel Christian University, Yogyakarta, to establish the Department of Civil Engineering.

In September 1991, he received a CIDA grant for Master of Applied Science Degree Programme at Civil and Environmental Engineering Department, University of Windsor, Windsor, Ontario, Canada. During this programme he invented a dynamometer system permitting horizontal and vertical hydrodynamic forces to be measured simultaneously for his research. He completed his M.A.Sc. degree in March 1994.

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