Wave Loads on Breakwaters, Sea-Walls and other Marine Structures

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- Coastal Research Centre, University Hannover and Technical University Braunschweig, Hannover
Introduction

Main Experimental Facilities and Expertise
Leichtweiss-Institute for Hydraulic Engineering

Technical University Braunschweig

Homepage: http://www.LWI.tu-bs.de

Hydromechanics & Coastal Engineering

(a) Plan view of twin-flume
up to 30cm high solitary waves

(b) Twin-Wave Paddle (Synchron or independent)

Length ≈ 90m

Depth = 1.25m

2m 1m
Coastal Research Centre (FZK) in Hannover

Homepage: http://www.hydrolab.de/

Coastal Research Centre

FZK
Joint Central Institution of the University of Hannover and the Technical University of Braunschweig

Large Wave Flume (GWK)

Dimensions
330m x 5m x 7m

up to 1m high solitary waves
Investigated Coastal Marine Structure (Selected)

- Seadikes and Revetments
- Rubble Mound Breakwaters
- Caisson-Breakwaters
- Wave Absorber as a Sea Wall
- Wave Absorber as Artificial Reef
- Innovative Sea Walls
- Beach Profile Development under Storm Surge Conditions
- Dune Stability and Reinforcement with Geotextile Constructions
- Wave Impact Loading & Scour (in progress)
Sea Wave-Structure-Foundation Interaction

- **CoV ≥ 10%**
  - Incident Waves (Farfield)
  - Waves & Water Levels (Nearfields)

- **CoV ≥ 20%**
  - Waves & Water Levels (at Structure)
  - Structure Load
  - CoV ≥ 20%
  - Structure- and Soil Parameters

- **CoV > 30%**
  - Wave Transmission
  - Wave Overtopping
  - CoV > 50%

- **CoV > 20%**
  - Direct Loading of Foundation Soil
  - Indirect Loading of Foundation Soil

**CoV = Coefficient of Variation**

**TF = Transfer function (Model)**
Research Strategy

Processes observed in nature → Identify most essential and crucial features of processes to be predicted → Processes observed in nature

Conceptual model (original idea/hypothesis)

Numerical modelling to supplement/support small scale model study → Qualitative (and partly quantitative) understanding of physical processes to be investigated → Physical Modelling (small scale model tests for systematic parameter study)

Prototype observations and measurements (model and scale effects) and/or

Large-scale model tests (scale effects)

Improved quantitative understanding of processes to be predicted

Validated sophisticated numerical model used as a research predictive tool → Validated predictive formulae and graphs (empirical/semi-empirical)

Ultimate Scientific Result:

Detailed conceptual model (generalized and validated)

Verification and validation
1. Introduction
2. Wave Loads on Pile Structures
3. Wave Forces on Submerged Bodies
4. Wave Loads on Monolithic Breakwater and Sea Walls
2. Wave Loads on Pile Structures (3D)
2.1 Wave Load Classification
Non Slender and Slender Structures

(a) \( D > 0.05L \)
Non Slender Structures
(Diffraction + Reflection)

(b) \( D < 0.05L \)
Slender Structures
(Hydraulically transparent)

\( \pi \frac{D}{L} = \text{Diffraction parameter} \)
Relative Importance of Drag and Inertia Forces $F_D$ and $F_M$

### MORISON-Formula:

$$f_{gs} = \frac{1}{2} C_D \rho_c D \left| u + c_m \right| \cdot \frac{\pi D^2}{4} \cdot \frac{\partial u}{\partial t}$$

- **Drag component**
  - $D = H \cdot \left( \frac{100}{h \cdot L} \right)$
  - $D = H \cdot \left( \frac{16}{h \cdot L} \right)$

- **Inertia component**
  - $D = H \cdot \left( \frac{1}{h \cdot L} \right)$

### Forces and Water Depth:

- **Deep water** ($h/L \geq 0.5$)
  - $D \leq \left( \frac{H}{32} \right)$
  - $D < \left( \frac{1}{100 \cdot h \cdot L} \right)H$

- **Transition & Shallow water** ($h/L < 0.5$)
  - $D < \left( \frac{H}{32} \right)$
  - $D < \left( \frac{1}{100 \cdot h \cdot L} \right)H$

- **Total Force $F_{Tot}$**
  - $F_{Tot} = F_D$
  - $F_{Tot} = F_M + F_D$
  - $F_{Tot} = F_M$

- **Wave Force Components**
  - $F_{Tot} = \pi \cdot \partial h \cdot \rho \cdot \partial t$
Wave Load at Different Locations

Wave Load Classification

- Seawards of surf zone
  - Non breaking waves
    - Generally quasi-static
    - Enough knowledge available

- Surf zone
  - Breaking waves
    - Generally dynamic

- Shorewards of surf zone
  - Broken waves (bores)
    - Generally dynamic
    - Insufficient knowledge
2.2 Breaking Wave Impact Load on Single Pile in Deeper Water

References:
Extreme Loads Due to Breaking Waves

PROTOTYPE

GWK MODEL
Breaking Wave Forces on Slender Cylinders

Total force \( F_{Tot} \) = inertia/drag force (quasi-static force \( F_M + F_D \)) + impact force \( F_I \)

\( \lambda \) = "Curling Factor"
Slamming Coefficient and Pile-up Effects

$C_S(t=0) = \pi$

von Karman (1929)

$C_S(t=0) = 2\pi$

Wagner (1932)
Theoretical Formulae for Slamming Forces

- analytical solution by flat plate approximation
- experimentally verified by pressure measurement
Theoretical Formulae for Slamming Forces

Line force

\[ f_{\text{max}} = C_S \cdot \rho \cdot R \cdot V^2 \]

with \( C_S = \) Slamming Factor

Impact load duration:

\[ T_D = \frac{13}{32} \cdot \frac{R}{C} \]
Loading Cases for Vertical Cylinder in GWK

Wave breaking behind pile

5.

Force vs. Time

quasi-static
Loading Cases for Vertical Cylinder in GWK

- Broken wave at pile
- Wave breaking behind pile
- Wave breaking at pile
- Wave breaking just in front of pile
- Wave breaking in front of pile

- Force vs. Time
- Video

Quasi-static
Loading Cases for Vertical Cylinder in GWK

1. Broken wave at pile
2. Wave breaking in front of pile
3. Wave breaking just in front of pile
4. Wave breaking in front of pile
5. Broken wave at pile

Wave breaking behind pile

1. Force - Time

2. Force - Time

3. Force - Time

4. Force - Time

5. Force - Time

quasi-static
# Loading Cases for Vertical Cylinder in GWK

<table>
<thead>
<tr>
<th>Video</th>
<th>Broken wave at pile</th>
<th>Wave breaking in front of pile</th>
<th>Wave breaking just in front of pile</th>
<th>Wave breaking behind pile</th>
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</thead>
<tbody>
<tr>
<td>Breaker</td>
<td><img src="image1.png" alt="1. Broken wave at pile" /></td>
<td><img src="image2.png" alt="2. Wave breaking in front of pile" /></td>
<td><img src="image3.png" alt="3. Wave breaking just in front of pile" /></td>
<td><img src="image5.png" alt="5. Wave breaking behind pile" /></td>
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<tr>
<td>Force - Time</td>
<td><img src="image4.png" alt="Quasi-static" /></td>
<td><img src="image4.png" alt="Quasi-static" /></td>
<td><img src="image4.png" alt="Quasi-static" /></td>
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## Loading Cases for Vertical Cylinder in GWK

<table>
<thead>
<tr>
<th>Broken wave at pile</th>
<th>Wave breaking in front of pile</th>
<th>Wave breaking just in front of pile</th>
<th>Wave breaking at pile</th>
<th>Wave breaking behind pile</th>
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<tbody>
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<td><img src="image4.png" alt="Image" /></td>
<td><img src="image5.png" alt="Image" /></td>
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</tbody>
</table>

### Breaker

1. Wave breaking behind pile
2. Wave breaking just in front of pile
3. Wave breaking in front of pile
4. Broken wave at pile
5. Wave breaking at pile

### Force - Time

- Quasi-static
<table>
<thead>
<tr>
<th></th>
<th>-45°</th>
<th>-25°</th>
<th>0°</th>
<th>+24.5°</th>
<th>+45°</th>
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<td>1</td>
<td><img src="image1.png" alt="Image" /></td>
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<td><img src="image15.png" alt="Image" /></td>
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<td><img src="image24.png" alt="Image" /></td>
<td><img src="image25.png" alt="Image" /></td>
</tr>
</tbody>
</table>

quasi-static
### Loading Cases Investigated in GWK for Vertical & Inclined Cylinders (2)

<table>
<thead>
<tr>
<th></th>
<th>-45°</th>
<th>-25°</th>
<th>0°</th>
<th>+24.5°</th>
<th>+45°</th>
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</thead>
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<td><img src="image24.png" alt="Image 24" /></td>
<td><img src="image25.png" alt="Image 25" /></td>
</tr>
</tbody>
</table>

{ quasi-static }
### Loading Case 3 for Different Pile Inclination Angles

<table>
<thead>
<tr>
<th>Angle</th>
<th>Description</th>
<th>Image</th>
</tr>
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<tbody>
<tr>
<td>-45°</td>
<td>Inclined against wave direction</td>
<td><img src="image1.png" alt="Image" /></td>
</tr>
<tr>
<td>-25°</td>
<td>Inclined against wave direction</td>
<td><img src="image2.png" alt="Image" /></td>
</tr>
<tr>
<td>0°</td>
<td>Vertical pile</td>
<td><img src="image3.png" alt="Image" /></td>
</tr>
<tr>
<td>+24.5°</td>
<td>Inclined in wave direction</td>
<td><img src="image4.png" alt="Image" /></td>
</tr>
<tr>
<td>+45°</td>
<td>Inclined in wave direction</td>
<td><img src="image5.png" alt="Image" /></td>
</tr>
</tbody>
</table>

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### Loading Case 3
Slamming Forces: Definition Sketch

- \( F_0 \): Total force
- \( F_1 \): Initial force
- \( F_2 \): Final force
- \( T_D \): Duration of the slamming event
- \( t_0 \), \( t_1 \), \( t_2 \): Time intervals

[Diagram showing force and time with labeled variables]
Impact Force for Loading Case 3 Different Pile Inclinations

\[ F_1 = 2 \cdot \pi \cdot (\lambda \cdot \eta') \cdot \rho \cdot R \cdot V^2 \]
$H_B \approx 2.8\text{m just in Front of Cylinder}$

Breaker Heights up to 3.25m generated!
Wave With Vertical Front at Cylinder and Splash Generation

Wave with vertical front at cylinder

Wave crests

\( V(z) \)

\( dz \)

\( R \)
Breaking Wave Impact as a Radiation Process

(a) Side View  
(b) Front View

Wave crest splashes

Breaking wave at cylinder

SWL
Theoretical 3D-Model for Impact Force

\[ F = F_D + F_M + F_I \]

- Morison Force
- Impact Force

For time: \( t = 0 \)

\[ F_I = \rho R v^2 \eta_b \cos^2 \gamma \left( 2\pi - 2 \left( \sqrt{\frac{V \cos \gamma}{R}} t \right) \arctan \left( \sqrt{1 - \frac{1}{4} \frac{V \cos \gamma}{R} t} \right) \right) \]

For time \( t' = \frac{3}{32} \frac{R}{V \cos \gamma} \div \frac{12}{32} \frac{R}{V \cos \gamma} \), with \( t' = t - \frac{1}{32} \frac{R}{V \cos \gamma} \)

\[ F_I = \rho R v^2 \eta_b \cos^2 \gamma \left[ \frac{\pi}{6} \left( \frac{V \cos \gamma}{R} t' \right)^{-1} - \sqrt{\frac{8 V \cos \gamma}{3 R}} t' \arctan h \left( \sqrt{1 - \frac{1}{6} \frac{V \cos \gamma}{R} t' \sqrt{\frac{8 V \cos \gamma}{3 R}} t'} \right) \right] \]

Duration \( T_D \) of Impact Force \( F_I \)

\[ T_D = \frac{13}{32} \frac{R}{V \cos \gamma} \]  

Adopted in ISO/CD 21650 “Actions from wave and currents” (in print)
Adopted in New German Lloyd Guidelines (2005)

„Curling Factor“ for Vertical and Inclined Cylinder

\[ \lambda = 0.46 \text{ für } \alpha = 0 \]

\[ \lambda = \cos(\alpha - \beta) \quad \text{für } \alpha = -45^\circ \text{ bis } +45^\circ \quad \text{und } \beta \approx -45^\circ \]
Time Dependent Slamming Coefficient (Wienke and Oumeraci, 2005)

\[ C_s = \frac{f_s}{\rho R V^2} \]

- Fabula (1957)
- Cointe (1989)
- Goda et al. (1966)
- Wienke (2001)
- Campbell et al. (1977)
- Campbell & Weynberg (1979)
- Von Karman (1929)
- Wagner (1932)
2.3 Breaking Wave Impact Load on Single Pile in Shallow Water

References:
Breaking Wave Loads (Slamming Coefficient Approach)

\( F_{\text{Tot}} = \frac{1}{2} C_{D}' \rho w (D \Delta z) \left| u \right| u \)

For shallow water:
\[ u \approx \sqrt{gh_b} \approx \sqrt{gH_b} \]

\[ \left| u \right| u = u^2 = gH_b \]

\( \Delta z = \text{Impact height of } f_D = f(\text{Breaktype}) : \)
\[ \Delta z \approx H_b \]

\( F_{\text{Tot}} \approx 0.88 \rho_w \cdot g \cdot D \cdot H_b^2 \)

Problem: Response characteristics of structure not considered.
Quasi-Static Force and Impact Forces

Non-breaking wave:
- quasi-static force

MORISON equation
\[ F = F_D + F_M \]

Breaking wave:
- quasi-static + impact force

\[ F = (F_D + F_M) + F_I \]

\[ F_I = \rho \cdot R \cdot V^2 \cdot C_s \cdot \lambda \cdot \eta_b \cdot \cos^2 \alpha \]

- \( C_s \): slamming coefficient
- \( \lambda \): curling factor
Curling Factor for Depth Limited Wave Breaking

Non-breaking wave:
- quasi-static force

MORISON equation
\[ F = F_D + F_M \]

Breaking wave:
- quasi-static + impact force

\[ F = (F_D + F_M) + F_I \]

\[ F_I = \rho \cdot R \cdot V^2 \cdot C_s \cdot \lambda \cdot \eta_b \cdot \cos^2 \alpha \]

- \( C_s \): slamming coefficient
- \( \lambda \): curling factor

\[ \Rightarrow \] Estimation of curling factor \( \lambda \) for depth limited breaking waves

PhD-Thesis of Mr. IRSCHIK to be completed in Summer 2007
Model Set-up in GWK (1)

5 inclinations of test cylinder
\[ a = -45^\circ/-22.5^\circ/0^\circ/22.5^\circ/45^\circ \]
Flume wall:
- current meters (8)
  - propeller probes (4)
  - Acoustic Doppler Velocimeters (ADV) (4)
- wave gauges (20)

Test cylinder:
- five inclinations
  - $\alpha = -45^\circ/-22.5^\circ/0^\circ/22.5^\circ/45^\circ$
- force transducers -strain gauges- (8)
  - top and bottom bearing
  - inline and transverse
- pressure transducers (16)
  - front-line
- wave gauges (4)
Wave Conditions Tested in GWK

GWK-Data
x = 201m

GWK-Data
x = 201m
**Example 1:** $T = 4s$ (Plunging Breaker)

- $T = 4s$
- $\eta_{\text{max}}$
- $t = -0.4s$
- $t = -0.2s$
- $t = 0$
- $t = 0.08s$
- $t = 0.16s$
- $H = 1.55m$, $d = 4.12m$, $2091201.056$
- SWL
- trough

**Example 2:** $T = 8s$ (Collapsing Breaker)

- $T = 8s$
- $\eta_{\text{max}}$
- $t = -0.4s$
- $t = -0.2s$
- $t = 0$
- $t = 0.08s$
- $t = 0.16s$
- $H = 1.50m$, $d = 3.96m$, $2091906.024$
- SWL
- trough
plunging breaker
breaker tongue at top
very small air gap
**breaker tilt angle 30-45°**

collapsing breaker
breaker tongue at SWL
**breaker tilt angle >45°**
**Breaker Tilt Angles**

- **plunging breaker**
  - breaker tongue at top
  - air gap
  - **breaker tilt angle 30-45°**

- **collapsing breaker**
  - breaker tongue at SWL
  - very small air gap
  - **breaker tilt angle >45°**

Galvin (1968)
Breaking Wave Loads in Shallow Water
Normalized Max Total Force

\[ F_{\text{tot}} / \rho g D H_{\text{cyl}}^2 \]

-8 -6 -4 -2 0 2 4

distance \( x_b - x_{\text{cyl}} \) [m]

Time Dependent impact force (highly variable)

3D model of Wienke (2001)

quasi-static load (constant)

Morison equation

\( x = x_b = \) Breaking Point Location:
Curling factor

\[ \lambda = \frac{F_{\text{dyn}}(t)}{\rho \cdot C_s(t) \cdot R \cdot C_b^2 \cdot \eta_b} \]

- The time history of the impact can be estimated using the same curling factors for both plunging and collapsing breakers.

Wiegel (1982)
Estimation of curling factor $\lambda$ for maximum loading case

![Graph showing curling factor $\lambda$ vs. angle $\alpha$ for different angles. The graph includes data points for 10% highest values and mean value, marked with symbols and lines.](image-url)
Comparison with Goda et al. (1966), Wiegel (1982) and Wienke (2001)

Wienke (2001): transient wave packets on horizontal bottom – “freak waves”

Wienke, J (2001): Impact loading of slender pile structures induced by breaking waves in deeper water, PhD-Thesis, Leichtweiss-Institute, TU Braunschweig (in German)

2.4 Effect of Neighbouring Piles on Wave Loading of Slender Pile

References:
Measuring Cylinder in GWK

Model set-up in the Large Wave Channel

Cross-section:
- Support structure
- Measuring cylinder: $D = 0.324m$
- Strain gauges
- Wave gauges
- Current meters
- SWL: +4.26m
- +2.40m
- 0.00 m
Measuring Cylinders and Locations of Neighbouring Cylinders

Model set-up in the Large Wave Channel

Wave gauges
- 107.29 m
- 106.64 m
- 105.99 m
- 105.34 m
- 104.69 m
- 104.04 m
- 103.39 m
- 102.74 m
- 102.09 m

Location in front of the wavemaker

Current meters

Measuring cylinder

2.50 m

Video control

PLAN VIEW
Cylinder Group Configurations Tested

1. Measuring cylinder
2. Neighbouring cylinder

D = Cylinder diameter
\( \vec{c} \) = Incident wave direction

15 Investigated cylinder group configurations

see paper Sparboom, U.; Hildebrandt, A.; Oumeraci, H in Proc. ICCE ’06
### Loading Cases for the Selected Cylinder Group Configurations (1)

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<thead>
<tr>
<th>Cylinder configurations</th>
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### Tested Wave Conditions

**Water Depth**

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<thead>
<tr>
<th>Wave Height (H) [m]</th>
<th>0.80</th>
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<td>6</td>
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<td>x</td>
<td>x</td>
</tr>
<tr>
<td>7</td>
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</tr>
<tr>
<td>8</td>
<td>x</td>
<td>x</td>
<td>x</td>
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</tbody>
</table>

**Water Depth** \(d = 4.26\) m

**Peak Period** \(T_p\) [s]

<table>
<thead>
<tr>
<th>Significant Wave Height (H_s) [m]</th>
<th>0.80</th>
<th>1.00</th>
<th>1.00</th>
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<tbody>
<tr>
<td>4</td>
<td>x</td>
<td>-</td>
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<td>6</td>
<td>-</td>
<td>x</td>
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</tr>
<tr>
<td>8</td>
<td>-</td>
<td>-</td>
<td>x</td>
</tr>
</tbody>
</table>

(a) Regular Non-Breaking Waves

(b) Irregular Non-Breaking Waves (JONSWAP)

(c) Breaking Waves (Gaussian Wave packets)
Surface elevation and wave height development in the near field of a single isolated cylinder for loading cases 1-5

(a) Locations of wave gauges and cylinder

(b) Surface elevation above SWL

(c) Wave development in front and behind the cylinder
Bending moments and surface elevation for isolated single cylinder

Regular Waves: \( H = 1.4 \text{m}; \, T = 4 \text{s} \)

- \( \eta(t) \): surface elevation at cylinder location
- \( M_x(t) \): Moment in longitudinal direction
- \( M_y(t) \): Moment in transverse direction
- \( M_r(t) \): Resulting Moment
- \( \Delta t = \) Phase lag related to \( \eta_{\text{max}} \)

- \( M_x = \) Moment in longitudinal direction
- \( M_y = \) Moment in transverse direction
- \( M_r = \) Resulting Moment
- \( \Delta t = \) Phase lag related to \( \eta_{\text{max}} \)
Measured wave kinematics with calculated accelerations

Regular Waves: $H = 1.4m; T = 4s$

- $u =$ horizontal Particle velocity
- $v =$ vertical Particle velocity

![Graph showing wave kinematics and calculated accelerations](image)
Bending moments and surface elevation for side-by-side arrangement

Regular Waves: H = 1.4m; T = 4s

\[ M_x = \text{Moment in longitudinal direction} \]
\[ M_y = \text{Moment in transverse direction} \]
\[ M_r = \text{Resulting Moment} \]
\[ \Delta t = \text{Phase lag related to } \eta_{\text{max}} \]

\[ \eta(t) : \text{surface elevation at cylinder location} \]
Wave kinematics for side-by-side arrangement

Regular Waves: $H = 1.4m; T = 4s$

$u$ = horizontal Particle velocity
$v$ = vertical Particle velocity

$\eta(t)$
$u(t)$
$v(t)$

$\frac{du}{dt}$
$\frac{dv}{dt}$

Surface elevation $\eta [m]$
Particle acceleration $[m/s^2]$
Particle velocity $[m/s]$

$u =$ horizontal Particle velocity
$v =$ vertical Particle velocity

Time [s]
## Reduction Amplification Factor for Loading Cases 1-5

\[
K = \frac{M_{\text{group}}}{M_{\text{single}}} \quad [-]
\]

<table>
<thead>
<tr>
<th>Configuration no.</th>
<th>Reduction Amplification Factor</th>
<th>Waves (D)</th>
<th>Waves (3D)</th>
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<td>LC 1</td>
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<td>1.04</td>
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<td>LC 3</td>
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<td>1.05</td>
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<td>LC 4</td>
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<td>LC 5</td>
<td>0.78</td>
<td>1.17</td>
<td>0.92</td>
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</table>
3. Wave Forces on Submerged Bodies

References:

Sliding and Overturning Stability

### Sliding Stability
- **Mobilising force:** Drag + Inertia force
  \[ F_D + F_M = 0.5 \rho_w u^2 C_D A_s + C_{D\mu} \frac{\partial u}{\partial t} \]

- **Resisting Force:**
  \[ F_{\text{Resisting}} = \mu \left( F_{GSC} - F_{\text{lift}} \right) \]

**Horizontal Sliding**
- **Mobilising force:** Drag + Inertia force
- **Resisting force**

\[ l_c(\text{sliding}) \geq u^2 \left[ 0.5C_D + 2.5C_L \mu \right] \]
\[ \frac{\mu \Delta g - C_M}{C_L} \frac{\alpha u}{\alpha t} \]

### Overturning Stability
- **Mobilising Moment:**
  \[ F_{GSC} \frac{l}{2} \geq F_D \frac{l}{10} + F_M \frac{l}{10} + F_{\text{lift}} \frac{l}{2} \]

- **Resisting Moment**
  \[ l_c(\text{over}) \geq u^2 \left[ 0.05C_D + 1.25C_L \right] \]
  \[ 0.5 \Delta g - 0.1C_M \frac{\alpha u}{\alpha t} \]

**Empirical Coefficients (C_D, C_M and C_L) needed**
Tested Configurations (1)

Configuration 0: Force Transducer without GSC-container

Configuration 1: Single GSG-container on sea bed, z=0.03m
Tested Configurations (2)

**CONFIGURATION 2: Single GSC-container 9cm above sea bed**
- **FRONTAL VIEW**
  - Container fixed to force transducer
  - Distance from bottom to GSC is 9 cm
  - False bottom
  - Sand filling
- **CROSS SECTION**
  - Container fixed to force transducer
  - Distance from bottom to GSC is 9 cm
  - Sand filling
- **LAY OUT**
  - Container fixed to force transducer

**CONFIGURATION 3: Single GSC-container 13cm from sea bed**
- **FRONTAL VIEW**
  - Container fixed to force transducer
  - Distance from bottom to GSC is 13 cm
  - False bottom
  - Sand filling
- **CROSS SECTION**
  - Container fixed to force transducer
  - Distance from bottom to GSC is 13 cm
  - Sand filling
- **LAY OUT**
  - Container fixed to force transducer

**CONFIGURATION 4: Group of GSC-containers, instrumented container 13cm from sea bed**
- **FRONTAL VIEW**
  - Container fixed to force transducer
  - Filling of the rigid containers to the flume to avoid measurement disturbances
  - Sand filling
- **CROSS SECTION**
  - Container fixed to force transducer
  - Distance from bottom to GSC is 13 cm
  - Normal container
- **LAY OUT**
  - Container fixed to force transducer
Tested Configurations (4)

CONFIGURATION 7: Group of GSC-containers, instrumented container 3 cm from sea bed

FRONTAL VIEW
- Containers fixed to the "bridge"
- Wave direction
- Sand filling
- Not to Scale

CROSS SECTION
- Container fixed to force transducer
- Force transducer
- Not to Scale

LAY OUT
- Containers fixed to force transducer
- Normal containers

CONFIGURATION 8: GSC-structure, instrumented container 3 cm from sea bed

FRONTAL VIEW
- Containers fixed to the "bridge"
- Wave direction
- Sand filling
- Not to Scale

CROSS SECTION
- Container fixed to force transducer
- Force transducer
- Not to Scale

LAY OUT
- Containers fixed to force transducer
- Normal containers

CONFIGURATION 9: GSC-structure, instrumented container 23 cm from sea bed

FRONTAL VIEW
- Fixing of the rigid containers to the frame to avoid measurement disturbances
- Sand filling
- Not to Scale

CROSS SECTION
- Container fixed to force transducer
- Force transducer
- Not to Scale

LAY OUT
- Container fixed to force transducer
- Normal containers
## Reynolds Numbers and KC Numbers for Tested Conditions

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<th>Depth (m)</th>
<th>Wave Height</th>
<th>Wave Period (seconds)</th>
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<th>2.5</th>
<th>3</th>
<th>3.5</th>
<th>4</th>
<th>4.5</th>
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<td></td>
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<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
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<tr>
<td>0.70</td>
<td>0.12</td>
<td>Re=5.4x10^4</td>
<td></td>
<td></td>
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<td>Re=6.8x10^4</td>
<td>Re=7.28x10^4</td>
<td>Re=7.2x10^4</td>
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<td></td>
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<td></td>
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<td></td>
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<td>KC=3.8</td>
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<tr>
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<td>0.24</td>
<td>Re=9.4x10^4</td>
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<td>KC=4.95</td>
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<tr>
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<td></td>
<td>KC=2.6</td>
<td></td>
<td></td>
<td>KC=3.7</td>
<td>KC=5.3</td>
<td>KC=7.0</td>
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<td>Re=7.28x10^4</td>
<td>Re=7.2x10^4</td>
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<td>KC=4.7</td>
<td>KC=7.0</td>
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<td></td>
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<td>Re=1.2x10^6</td>
<td>Re=9.4x10^4</td>
<td>Re=1.4x10^5</td>
<td>Re=9.9x10^4</td>
<td>Re=7.28x10^4</td>
<td>Re=7.2x10^4</td>
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</tr>
</tbody>
</table>
Comparison Aceleration Data-Linear Theory

Differences within 17%

Acceleration derived from data

Acceleration from linear theory

H=0.12m T=2s d=0.70m

Time [s]
Comparison with COBRAS-Computations

Horizontal Forces

<table>
<thead>
<tr>
<th>Forces [N]</th>
<th>Time [s]</th>
</tr>
</thead>
<tbody>
<tr>
<td>z = 0.13m</td>
<td></td>
</tr>
</tbody>
</table>

Calc. Cobras vs Measured

Difference within 12%

Forces [N]:
- Landward
- Seaward

Time [s]:
- 0
- 10
- 20
- 30
Drag Dominance

Horizontal Velocity

Horizontal Force

Dyanmometer Fx-Component
Maximum Solitary Wave Generated in LWI-Twin Wave Flume
Comparison of Solitary Wave Shape Measured with Cobras

(a) Free Surface experiment

(b) Free Surface Cobras

Time [s]
4. Wave Loads on Monolithic Breakwaters and Sea Walls
4.1 Wave Loads Classification
Parameter Map for Classification of Loading Cases

Composite breakwater

Vertical breakwater
$h_b^* < 0.3$

Low mound
$0.3 < h_b^* < 0.6$

Moderate mound
$0.6 < h_b^* < 0.9$

High mound
$0.9 < h_b^* < 1.0$

Crown wall of rubble-mound breakwater
$h_b^* > 1.0$

1. Quasi-standing wave

2. Slightly breaking wave

3. Impact loads

4. Broken waves

with

$$h_b^* = \frac{h_b}{h_s}; \quad h_s^* = \frac{h_s}{h_s}; \quad B^* = \frac{B_{eq}}{h_s^2}; \quad F_h^* = \frac{F_h}{\rho g h_b^2} \quad \text{and} \quad B_{eq} = B + \frac{h_b}{2\tan\alpha}$$

Balkema, Amsterdam 316 p.
Characteristics of Pulsating and Impact Wave Loads

a) quasi-static load (pulsating)

\[ F_h = \frac{F_h}{\rho g H^2} \]

\[ F_{h,\text{max}} < 2.5 \]

\[ \bar{t}_d \approx 0.5 \]

\[ t_d > T_N \]

\[ T_N = \text{natural period of structure oscillation} \]

b) Impact load

\[ F_h = \frac{F_h}{\rho g H_b^2} \]

\[ F_{h,\text{max}} = 2.5 \text{ bis } 15 \]

\[ \bar{t}_d \approx 0.01 \text{ bis } 0.001 \]

\[ t_d < T_N \]

\[ T = \text{Wave period} \]

\( (*) \) Figures represent only order of magnitude
4.2 Breaking Wave Loads
Breaking Wave Loading of Caisson Breakwaters in GWK
Breaking Wave Loads and Splash in GWK (Video)
### Breaking Wave Loads

#### Slightly breaking waves

(pulsating load!)

<table>
<thead>
<tr>
<th>$F_h / \rho g H_b^2$</th>
<th>$H_b$</th>
<th>$F_h$</th>
</tr>
</thead>
<tbody>
<tr>
<td>$F_{h,\text{max}}$</td>
<td></td>
<td></td>
</tr>
<tr>
<td>$F_{h,q}$</td>
<td>$T$ = Wave period</td>
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</tr>
</tbody>
</table>

#### Strongly breaking waves

(impact load)

<table>
<thead>
<tr>
<th>$F_h / \rho g H_b^2$</th>
<th>$F_{h,\text{max}}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>$F_{h,q}$</td>
<td></td>
</tr>
</tbody>
</table>

#### GODA-Formulae

(static stability analysis)

\[
\frac{F_{h,\text{max}}}{F_{h,q}} = 1,0 \div 2,5
\]

#### PROVERBS-Approach

(dynamic stability analysis)

\[
\frac{F_{h,\text{max}}}{F_{h,q}} > 2,5
\]
Simplified Force Time History

Actual load

Idealised load

$F_h(t)$

$F_{h,\text{max}}$

$I_{rFh}$

$I_{dFh}$

$t_{rFh}$

$t_{dFh}$

$t_r$

$t_d$
Simplified Impact Pressure Distribution (Parameterization)

\[ F_h(t) \]

SWL = Static Water Level

\[ R_c = \text{freeboard} \]

\[ h^* = 0.8H_b \]

\[ p_3 = 0.45 \, p_1 \]

\[ \text{SWL} \]

\[ d \]

\[ d_c \]

\[ t_r \]

\[ t_d \]

\[ t \]

\[ t \]

\[ F_{h,\text{max}} \]

\[ F_h(t) \]

\[ F_{h,\text{max}} \]
Calculation of Static Equivalent Wave Loads (1)

\[
(F_h)_{stat} = \nu_D \cdot F_{h,max}
\]  
(1) with \( \nu_D = \) dynamic load factor

Calculation of dynamic load factor \( \nu_D \) by assuming a triangular load-time function:

• Maximum Impact force \( F_{h,max}[kN/m] \):

\[
F_{h,max} \approx 10 \rho_w \cdot g \cdot H_b^2
\]  
(2) \( H_b = \) Breaker height

• Rise time of Impact force \( t_r \):

\[
t_A = \frac{\pi^2}{g} \cdot \frac{h}{h + g \cdot H_b^2}
\]  
(3) \( h = \) Water depth directly at the wall

• Total load duration \( t_d \):

\[
t_d = t_A \cdot \left[ 2.0 + 8 \cdot \exp \left( -18 \frac{t_A}{T_p} \right) \right]
\]  
(4) \( T_p = \) Peak Period

or approximated by:

\[
t_d \approx 2.5 \cdot t_A
\]  
(5)

\[
\nu_D = 1.4 \left[ \tanh \left( \frac{t_d}{T_N} \right)^{0.55} \right] + 0.25 \left[ 2\pi \left( \frac{t_d}{T_N} \right)^c \right]
\]  
(6)

with \( c = 0.55 \left( \frac{t_d}{t_A} \right)^{-0.63} \)

Dynamic load factor $\nu_D$

$$\nu_D = 1.4 \left[ \tanh \left( \frac{t_d}{T_N} \right)^{0.55} \right] + 0.25 \left[ 2\pi \left( \frac{t_d}{T_N} \right) c \right] \quad \text{with} \quad c = 0.55 \left( \frac{t_d}{t_A} \right)^{-0.63}$$

(6)
Effect of Successive Breaking Wave Loads

<table>
<thead>
<tr>
<th>horiz. Force $F_h$ [kN/m]</th>
<th>Total horiz. Displacement of Structure [cm]</th>
</tr>
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<tbody>
<tr>
<td>Time [s]</td>
<td>Time [s]</td>
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<td>20.0</td>
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<td>44.0</td>
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<tr>
<td>47.0</td>
<td>47.0</td>
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</tbody>
</table>

(a) Horizontal breaking wave force

(b) Accumulation of residual displacement

(c) Caisson displacements
Storm surge (1973/74):
- $H_{\text{max}}$: $\approx 10.0$ m ($t_D=0.6s$)
- $T_p$: 13.0s

Caisson dimensions:
- Width: $B=17.0$ m
- Height: $h_c=11.63$ m
- Length: $L=20.95$ m

Static Standard Approach:
- $\eta_S=1.2$

Dynamic calculation ($t_D=0.6s$):
- $\eta_S=0.50$

$\eta_S = \text{Safety coefficient against sliding}$
Storm surge im Winter 73/74 ⇒ $H_{\text{max}} = 10 \text{m, } T_p = 13\text{s}$

Credit: Dr. Takahashi, PARI
B=17m; h=11-12m; L≈21m  \Rightarrow  H_s≈7m; T=13s

Although safety coefficient against sliding $\eta_s=1.2$ and against tilting $\eta_{ov}>5$ according to static (Standard) approach.
Dynamic Stability Calculation of SAKATA Breakwater

Impact load for $H_{\text{max}}=10\text{m}$, $T_p=13\text{s}$ and $d=3.43\text{m}$

Adhesion and friction force

Horizontal displacement of Caisson: ca. 20mm/Impact event
Seaward Impact Loading Induced by Wave Overtopping

Research Performed in PROVERBS for Wave Impact of Vertical Structures

Parameter Map for classification of loading case

- Berm height and width
- Water depth & wave conditions at structure

**IMPACT LOADING**

**Horizontal Loads (shoreward)**
- New prediction methods with following steps:
  - * statistical distribution of max. force ($F_{h,1}$)
  - * force history $F_{h,1} = f$ (duration)
  - * correction of $F_{h,1}$ for aeration
  - * pressure distribution

**Vertical Loads (uplift)**
- New prediction methods with following steps:
  - * statistical distribution of max. force ($F_{u,1}$)
  - * force history $F_{u,1} = f$ (duration)
  - * correction of $F_{u,1}$ for aeration
  - * pressure distribution

**Effect of wave overtopping**
- Seaward impact loads $F_{h,2}$ (see Section 2.5.2)
- Slamming on top slab $F_{v,2}$ (outside Proverbs)

**Constructional Measures to Reduce Impact Forces**
- * Perforated Structures (Section 2.8.1)
- * Rubble as Damping Layer (Section 2.8.2)
- * Unconventional Alternatives Related to Front Geometry (MCS-Project) and foundation (Section 3)

(Oumeraci et al, 2001)
4.3 Scale Effects Associated with Breaking Wave Impacts
Physical Processes Involved in the Wave Load History and Associated scaling Problems

- Compression of Air Pocket
- Impact of Breaker Tongue
- Oscillations of Air Pocket
- Escape of Air
- Maximal Run-up

\[ F_{h, \text{nature}} = N_l^{\alpha_F} F_{h, \text{model}} \]
\[ t_{\text{nature}} = N_l^{\alpha_t} t_{\text{model}} \]

- FROUDE: \( \alpha_F = 3 \) and \( \alpha_t = 0.5 \)
- MACH-CAUCHY: \( \alpha_F = 2 \) and \( \alpha_t = 1.0 \)
- ALTERNATIVE Scaling depending on air entrainment:
  - \( \alpha_F = 2 \) to 3
  - \( \alpha_t = 0.5 \) to 1.0

(Oumeraci & Hewson, 1997)
Suggested Procedure for Scaling the Various Components of the Wave Load History

Scaling of total force history:

\[ F_{\text{tot}}(t) = \left( F_{\text{dyn}} + F_{\text{osc}} \right)_{\text{MACH-Cauchy}} + F_{\text{Froude}} \]

- Impact component governed by compressibility: \( F_{\text{dyn}}(\text{MACH-Cauchy}) \)
- Oscillatory component governed by compressibility: \( F_{\text{osc}}(\text{MACH-Cauchy}) \)
- Quasi-static component governed by gravity: \( F_{q}(\text{Froude}) \)

\( t_r, t_{di}, t_d, t_q \) represent time durations.

(Oumeraci et al, 2001)
**Scale Effects in Modelling Breaking Wave Loading and Response of Sea Dikes**

<table>
<thead>
<tr>
<th>Force</th>
<th>Model Law</th>
<th>Scale</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>1:1</td>
</tr>
<tr>
<td>Gravity</td>
<td>FROUDE</td>
<td>1</td>
</tr>
<tr>
<td>Friction</td>
<td>REYNOLDS</td>
<td>1</td>
</tr>
<tr>
<td>Elasticity</td>
<td>CAUCHY</td>
<td>1</td>
</tr>
<tr>
<td>Surface Tension</td>
<td>WEBER</td>
<td>1</td>
</tr>
</tbody>
</table>

Breaker and Impact: WEBER, REYNOLDS, CAUCHY

Run-up and down: REYNOLDS (WEBER)

Surface: CAUCHY (REYNOLDS)

Core material: CAUCHY

Bottom friction: REYNOLDS

C

(after Führböter, 1986)
4.4 Broken Wave Loads on Sea Walls

(a) Seawall front **landward** of shoreline

(b) Seawall **seaward** of shoreline

- Front of seawall
- Seawall
- Beach slope
- Broken wave
- Virtual max. run-up
- SWL
- Shoreline
Assumption According to CEM (2003)

1. Bore height
   \[ H_b' = 0.78 \cdot H_b \]  
   \[ H_{w1} = (0.2 + 0.58 \cdot \frac{h_w}{h_b}) \cdot H_b \]  
   \[ H_{SWL} = 0.2 \cdot H_b \]  
   \[ H_{RWS} = 0.2 \cdot H_b \left(1 - \frac{x_2}{x_A}\right) \]

2. Bore velocity
   \[ v_b = c_b = \sqrt{g \cdot h_b} \]  
   \[ v_{w1} = c_b = \sqrt{g \cdot h_b} \]  
   \[ v_{RWS} = c_b = \sqrt{g \cdot h_b} \]  
   \[ v_{w2} = c_b = \sqrt{g \cdot h_b} \left(1 - \frac{x_2}{x_A}\right) \]

---

**Breaker height**

- Linear decrease of bore height
- Linear decrease of bore velocity from \( v_{SWL} = \sqrt{g \cdot h_b} \) to \( v_A = 0 \) at max. run-up

**Seawall**

- Seawall front seawards of shoreline
- Seawall front landwards of shoreline

**Beach slope**

- \( x_A = (z_A)_{max} / \tan \alpha \)
Seawall Front Seawards of Shoreline: Wave Load Formulae

Force Components:

\[ F_{\text{wo}} = \frac{1}{2} \cdot \rho \cdot g \cdot H_{\text{w}}^2; \]
\[ F_{\text{stau}} = \frac{1}{2} \cdot \rho \cdot g \cdot h_b \cdot H_{\text{w}}^2; \]
\[ F_{\text{wu}} = \rho \cdot g \cdot H_{\text{w}} \cdot h_{\text{w}}; \]

\[ F_{\text{stat}} = \frac{1}{2} \cdot \rho \cdot g \cdot H_{\text{w}}^2 \]

\[ p_{\text{stau}} = \rho \cdot \frac{v^2}{2} \]

\[ v = \sqrt{g \cdot h_b} \]

\[ p_w = \rho \cdot g \cdot H_{\text{w}} \]

\[ h_{\text{w}} = \text{Breaking depth} \]

\[ H_b = \text{Breaker height for water depth } h_b \]

\[ SWL = 0,2 \cdot H_b \]

\[ H_{\text{SWL}} = 0,2 \cdot H_b \]
Seawall front landwards of shoreline: Wave Loads Formulae

\[ v_{w2} = \sqrt{g \cdot h_b \cdot \left(1 - \frac{x_2}{x_A}\right)} \]

\[ \text{SWL} = \text{Bore height at shoreline} \]
\[ H_{w2} = \text{Bore height at wall} \]

\[ p_w = \rho \cdot g \cdot H_{w2} \]
\[ p_{stau} = \rho \cdot \frac{v_{w2}^2}{2} \]

\[ z_A = \frac{z_A}{\tan \alpha} \]

\[ F_{wo} = \frac{1}{2} \rho \cdot g \cdot H_{w2}^2 \]
\[ F_{stau} = \frac{1}{2} \rho \cdot g \cdot h_b \cdot H_{w2} \cdot \left(1 - \frac{x_2}{x_A}\right) \]

See also PhD-Thesis RAMSDEN (Caltech).

\[ z_A = \text{max. wave run-up} \]
4.5  Wave Force Reduction by Armouring (Tests Performed in Large Wave Flume)
Wave Force Reduction by Armouring

1 Definition of parameters:
- DWL = design water level
- B_crown = crest width (larger than two armour blocks)
- m = cot α = 1.35 to 1.50
- R_c = freeboard
- L_hs = wave length for depth h_s at toe
- b_0 = width of dissipating mound at height of DWL

\[ b_0 = B_{\text{crown}} + h_s \left( \frac{B_{\text{bottom}} + B_{\text{crown}}}{h_s + R_c} \right) \]
Damping of Pulsating Wave Loads by Armouring

Without dissipating mound:

\( \text{(F}_{h}\text{)}_{\text{max}} = 1.2 \)
\( \text{(F}_{h}\text{)}_{\text{max}} = 0.6 \)

\( (\mu_{D,h})_{\text{max}} \approx 50\% \)

With dissipating mound:

\( (\mu_{D,u})_{\text{max}} \approx 40\% \)

Relative horizontal force \( F_h = F_h / \rho g H_o^2 \)

Relative uplift force \( F_u = F_u / \rho g H_o^2 \)

\( \mu_{D,h} = \frac{F_h - F_{h,D}}{F_h} \)

\( \mu_{D,u} = \frac{F_u - F_{u,D}}{F_u} \)

(Oumeraci, 2004)
Damping of Pulsating Wave Loads by Armouring

\[ \mu_{D,h} = \frac{F_h - F_{h,D}}{F_h} \]

\[ \mu_{D,u} = \frac{F_u - F_{u,D}}{F_u} \]

\[ (F_{h,max}) = 4.1 \text{ with dissipating mound} \]

\[ (F_{u,max}) = 2.6 \text{ with dissipating mound} \]

\[ (\mu_{D,h})_{max} \approx 80\% \]

\[ (\mu_{D,u})_{max} \approx 60\% \]

(Oumeraci, 2004)
4.6 Stability of Structure Foundation Under Extreme Wave Loads
Main Modes of Vertical Breakwater Failure

OVERALL FAILURE MODES

a) Sliding
b) Overturning
c) Settlement followed by slip failure and seaward tilt
d) Settlement followed by slip failure and shoreward tilt

LOCAL FAILURE MODES

e) Erosion beneath seaward and shoreward edges
f) Punching failure at seaward and shoreward edges
g) Seabed scour and toe erosion
Wave Induced Dynamic Process in Foundation of Coastal Structures

- **Measuring Devices:**
  1. Pressure
  2. Pore Water Pressure
  3. Pore Water Pressure + Total Stress (isotropic)

- **Wave Gauges**
- **Inductive Displacement Meters**
- **Caisson with Sandfill**
- **Sand Berm**
  - $D_{50} = 0.35$ mm
- **Bedding layer**
  - 0.2 m
- **Sand layer**
  - 2.45 m
- **Sand Berm**
  - 4.05 m SWL
- **Sand D50 = 0.21 mm**
- **Impermeable Sheets**
  - $D_{50} = 0.35$ mm
- **Separation Wall**
  - (Sandbags)
- **Electrical Conductivity Measurement**
- **Pressure**
- **Pore Water Pressure**
- **Pore Water Pressure + Total Stress (isotropic)**
Caisson Breakwater Construction in GWK
Processes Leading to Partial Soil Liquefaction

Regular waves: $H=0.9m$, $T=6.5s$, $h_s=1.6m$

- (a) Total moment around caisson heel
- (b) Transient motions of the shoreward caisson edge
- (c) Transient pore pressure (P36)
- (d) Residual pore pressure (P36)
- (e) Residual deformation (vertical)

Mean Value $M_{\text{max}} \approx 210 \text{kNm/m}$

Number of wave load cycles [-]

$S \approx 373 \text{ cycles}$
Wave-Induced Pore Pressure and Soil Deformation Beneath Structures (Soil Liquefaction)

Regular waves with $H=0.7\text{m}$, $T=6.5\text{s}$, $h_s=1.6\text{m}$
Residual Pore Pressure vs. Caisson Motions

Impact Load  Pulsating Load

Pulsating Load

Residual Pore Pressure $u_r$ [kPa] after 53 loading cycles

Downward motion amplitude $d_{v,b}$ [mm]

$(d_{v,b})_{crit} = -0.3mm$
Thank you for your attention!