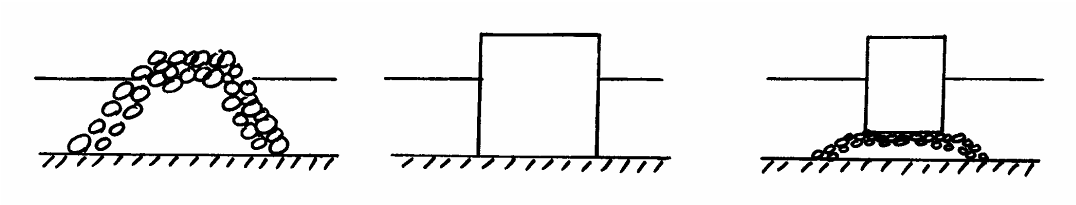
Chap 4. Design of Vertical Breakwaters

4.1 Vertical Breakwaters in Japan



rubble mound　　　　vertical　　　　composite breakwater

↑

"vertical" in Goda's book

Fig. 4.2 X-section of vertical breakwater

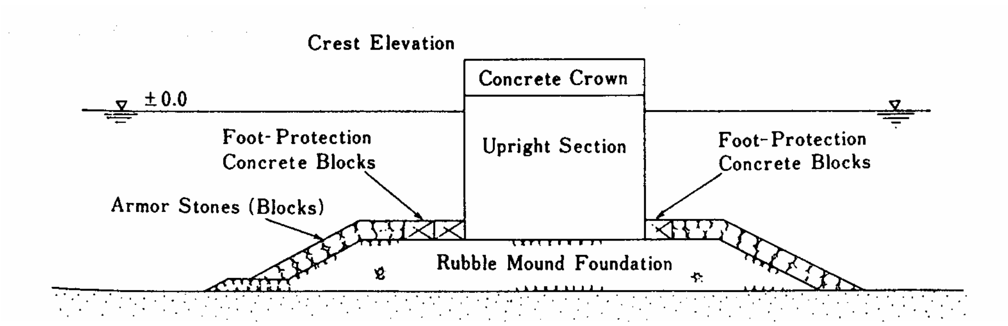
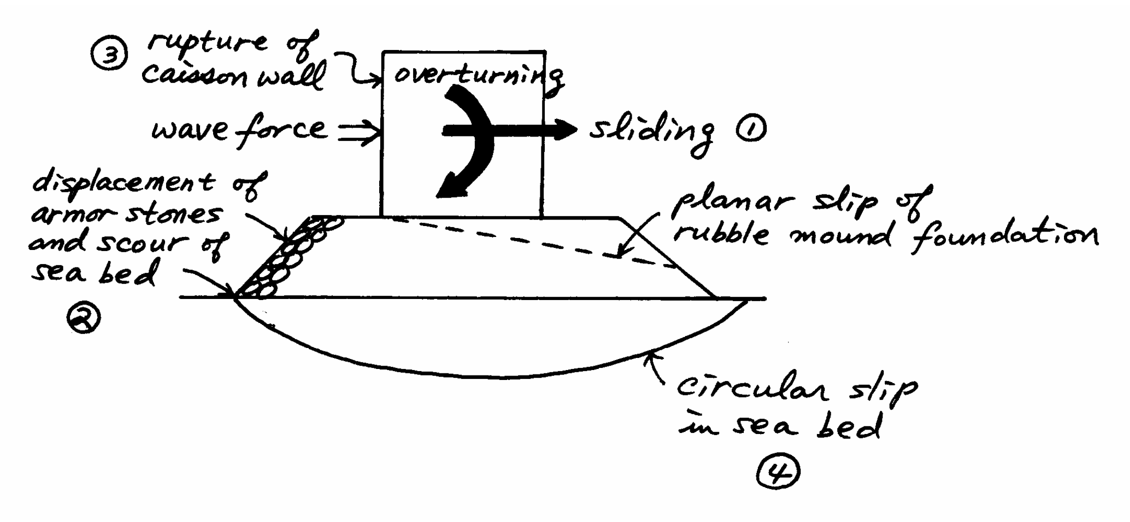


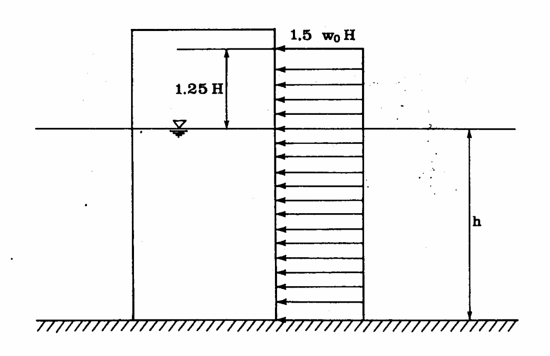
Fig 4.3 Failure modes



4.2 Pressure Formulas for Upright Sections

4.2.1 Overview of Development of Wave Pressure Formulas

- Hiroi (1919): based on field measurements, breaking waves in relatively shallow seas



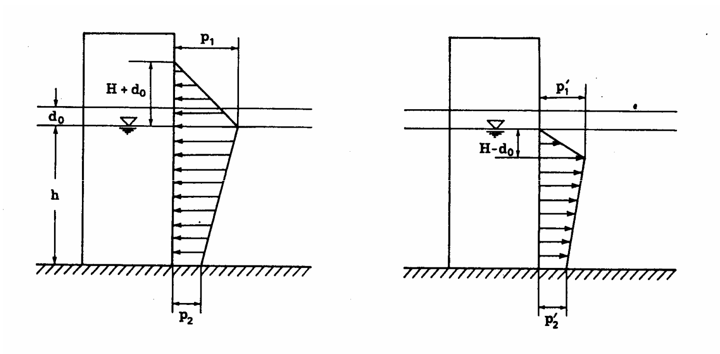
↑

vertically uniform pressure

$$p = 1.5\rho g H$$
$$P_{\max} = (h + 1.25H)\,p$$

H is usually taken as $H_{1/3}$, which is not much different from $H_{\max}$ in shallow seas

- Sainflou (1928): standing (non-breaking) wave force based on trochoidal wave theory



$$d_0 = \frac{kH^2}{2}\coth kh$$

Positive force (at wave crest)

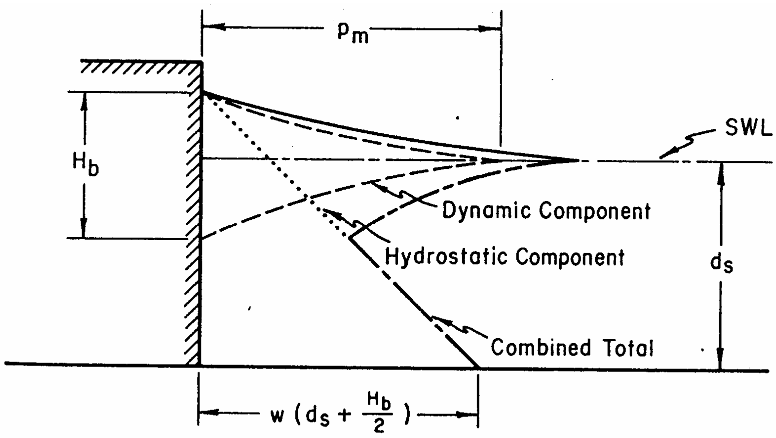
$$\left.\begin{aligned} p_1 &= (p_2 + \rho g h)\frac{H + d_0}{H + h + d_0} \\ p_2 &= \frac{\rho g H}{\cosh kh} \end{aligned}\right\} \rightarrow P_{\max} = \frac{p_1(H + d_0)}{2} + \frac{(p_1 + p_2)h}{2}$$

Negative force (at wave trough)

$$\left.\begin{aligned} p_1' &= \rho g (H - d_0) \\ p_2' &= p_2 = \frac{\rho g H}{\cosh kh} \end{aligned}\right\} \rightarrow P_{\max} = \frac{p_1'(H - d_0)}{2} + \frac{(p_1' + p_2')(h - H + d_0)}{2}$$

$H = H_{1/3}$, $H_{1/10}$, or $H_{\max}$ depending on the importance of the breakwater.

• Minikin (1950): based on Bagnold's laboratory data,
breaking wave pressure including impulsive pressure,
yields excessive wave forces (too conservative?)



$$p_m = 101\rho g \frac{H_b}{L_h}\frac{d_s}{h}(h + d_s) = \text{max.dynamic pressure (at SWL)}$$

H_b = breaker height

d_s = water depth at the toe of the wall

h = water depth at one wave length in front of the wall

L_h = wave length at depth h

$$P_{\max} = \frac{p_m H_b}{3}$$

Note: For a composite breakwater, d_s = water depth on the rubble mound, h = water depth at the toe of rubble mound.

• Goda (1973): extend the formula of Ito (1966),
a single formula for both breaking and non-breaking waves

• Tanimoto et al. (1976) included the effect of oblique incidence.

4.2.2 Formulas of Wave Pressure under Wave Crests

See Fig. 4.4 and Eqs. (4.2) to (4.15).
Calculates uplift force as well as horizontal force.
Design wave = highest wave in design sea state

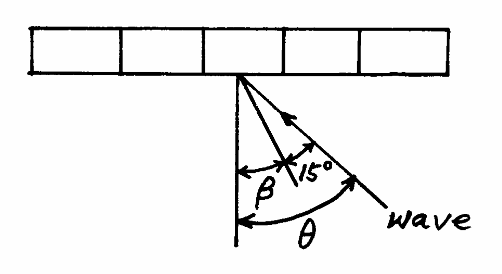
$$\left.\begin{array}{l} H_s = H_{1/3} = \text{significant wave height} \\ T_s = T_{1/3} = \text{significant wave period} \end{array}\right\} \text{at the site of breakwater before construction}$$

(no reflected wave yet)

Outside surf zone: $H_{\max} = 1.8 H_{1/3}$ and $T_{\max} = T_{1/3}$

Within surf zone: $H_{\max}$ = max. height of random breaking waves at $5H_{1/3}$ seaward of breakwater (calculated by Eq. (3.26))

$T_{\max} = T_{1/3}$



β = wave angle (θ) $-15°$ for safety (if $\theta > 15°$)
$\beta = 0°$ if $\theta \le 15°$

4.2.3 Pressure under a Wave Trough

↓

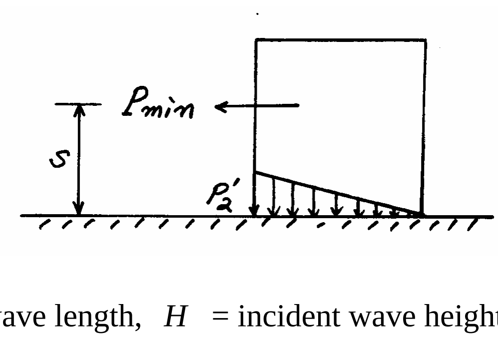
negative dynamic pressure → seaward movement of caisson

Negative pressure for breaking waves has not been examined in detail. Goda and Kakizaki (1966) used finite-amplitude (2nd order) standing wave theory.

Fig. 4.9: $\dfrac{P_{\min}}{w_0 H h}$ versus $\dfrac{H}{L}$ and $\dfrac{h}{L}$

Fig. 4.10: $\dfrac{s}{h}$ versus $\dfrac{H}{L}$ and $\dfrac{h}{L}$

Fig. 4.11: $\dfrac{p_2'}{w_0 H}$ versus $\dfrac{H}{L}$ and $\dfrac{h}{L}$



where $w_0 = \rho g$, h = water depth, L = wave length, H = incident wave height

4.2.4 Accuracy of Wave Pressure Formula

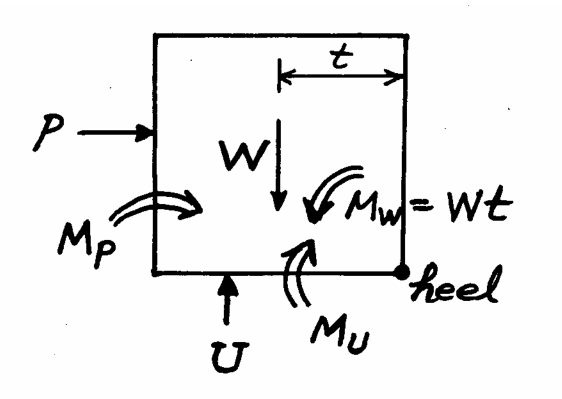
tested against 34 prototype breakwaters under approximately design wave conditions

$$\text{safety factor against sliding} = \frac{\text{resistance}}{\text{sliding force}} \quad \begin{matrix} > 1.0 & \text{no sliding} \\ < 1.0 & \text{sliding} \end{matrix}$$

See Fig. 4.12 (a) conventional formulas $\rightarrow$ poor
(b) Goda formula

4.3 Design of Upright Sections

4.3.1 Stability Condition for an Upright Section



Sliding

Frictional force between mound and caisson = $\mu(W-U)$, where μ = friction factor $\cong 0.6$.

If $P > \mu(W-U)$, sliding occurs.

S.F. against sliding = $\dfrac{\mu(W-U)}{P}$

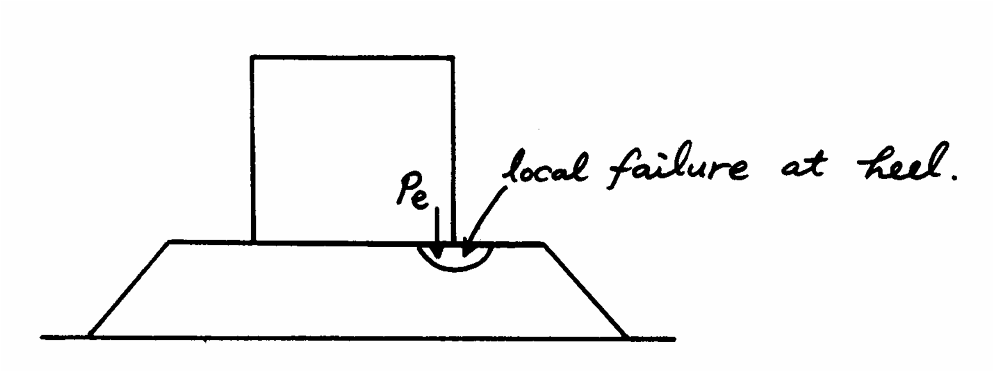
Overturning

If $M_P > M_W - M_U$, overturning occurs.

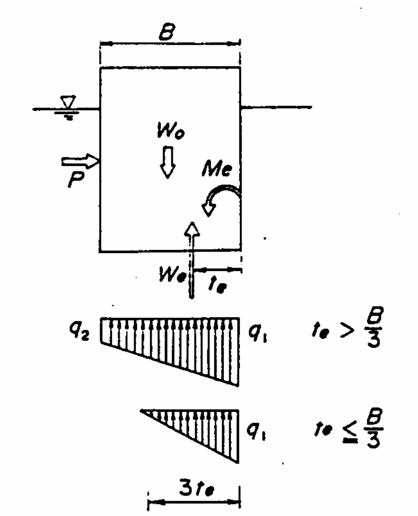
S.F. against overturning = $\dfrac{M_W - M_U}{M_P} = \dfrac{Wt - M_U}{M_P}$

In general, if the caisson is stable against sliding, it is stable against overturning as well.

Bearing capacity



The bearing pressure at the heel, p_e , should be less than a certain value: $p_e \leq 400 \sim 600$ kPa/m^2.



• A trapezoidal or triangular distribution of bearing pressure is assumed depending on t_e.

• Net weight, $W - U$, is supported by the normal stress between stones and bottom slab ($W_e = W - U > 0$).

• Net moment (ccw) due to W , P , and U about heel is

$$M_e = Wt - M_P - M_U$$

which must be balanced by the moment (cw) due to W_e .

$$M_e = W_e t_e \quad \rightarrow \quad t_e = \frac{M_e}{W_e}$$

If $t_e > \frac{B}{3}$, $q_1 = \frac{2W_e}{B}\left(2 - 3\frac{t_e}{B}\right)$, $q_2 = \frac{2W_e}{B}\left(\frac{3t_e}{B} - 1\right)$

If $t_e \le \frac{B}{3}$, $q_1 = \frac{2W_e}{3t_e}$, $q_2 = 0$

- $q_1 = p_e$ must be less than 400~500 kPa/m^2 usually. Recently the limit is increased to 600 kPa/m^2 due to increasing weight of caisson in deeper water.

4.3.2 Width of Upright Section

Required $B = \text{function}(H, T, i, D, \beta, h, d)$
See Figs. 4.13~4.18:

$B\uparrow$ as $H\uparrow$

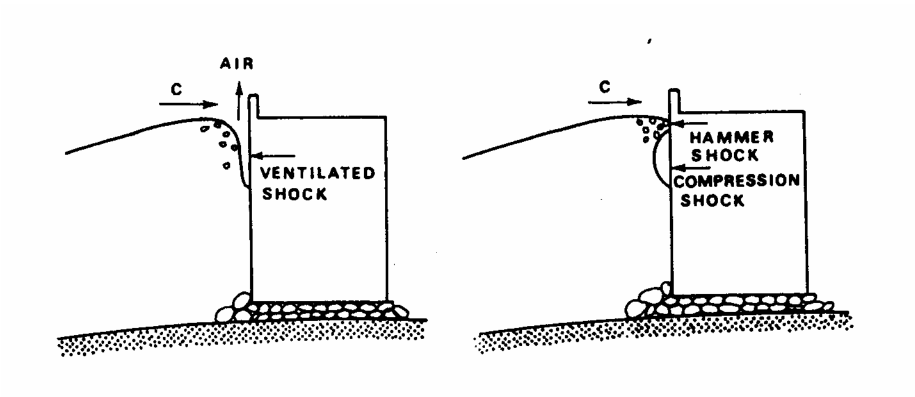
$T\uparrow$

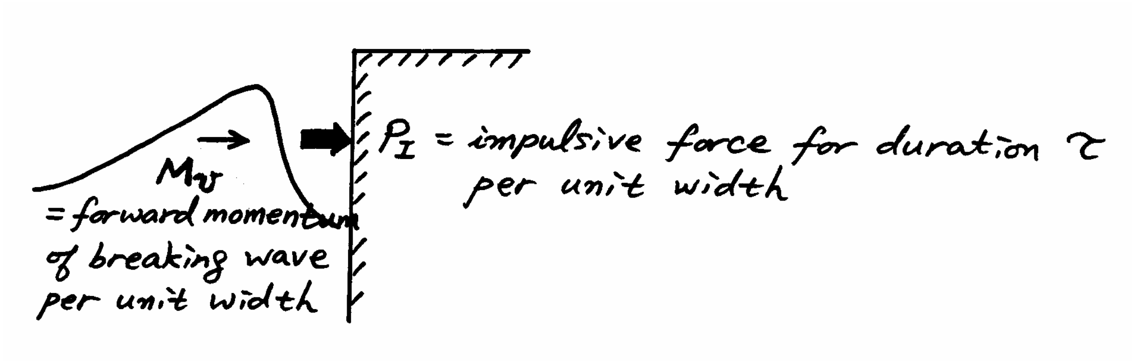
$\left.\begin{array}{l} i\uparrow \text{ in shallow water} \\ D\uparrow \text{ in shallow water} \end{array}\right\}$ breaking wave force acting on the caisson

$\beta\downarrow$

Due to many uncertainties, B is usually determined by hydraulic model tests. However, because it is difficult to change B, sliding test is usually made by changing W instead of B.

4.3.3 Precautions against Impulsive Breaking Wave Pressure





Newton's 2nd law: $F = ma = m\dfrac{du}{dt}$

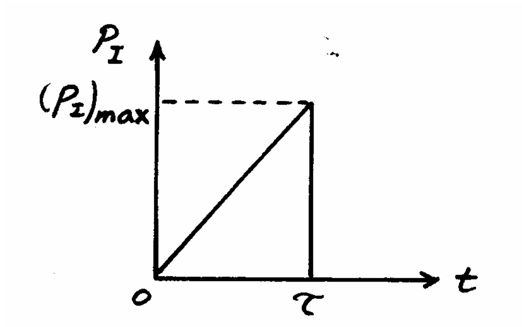
$$M = mu \;\rightarrow\; dM = mdu = Fdt$$

$$\int_0^\tau dM = \left[M\right]_0^\tau = \int_0^\tau Fdt$$

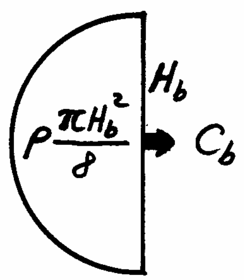
$$M\Big|_{t=0} = M_v\,,\;\; M\Big|_{t=\tau} = 0\,,\;\; F = -P_I$$

$$\int_0^\tau P_I dt = M_v$$

Order of magnitude of $\left(P_I\right)_{\max}$?



$$\int_0^\tau P_I dt \cong \frac{\tau}{2}\left(P_I\right)_{\max}$$



$$M_v = \rho\frac{\pi H_b^2}{8}C_b\,; \quad C_b \cong \sqrt{gH_b}$$

$$\left(P_I\right)_{\max} \cong \frac{\rho\pi H_b^2 C_b}{4\tau} = \frac{\rho g\pi H_b^2 C_b}{4g\tau} \qquad (4.23)$$

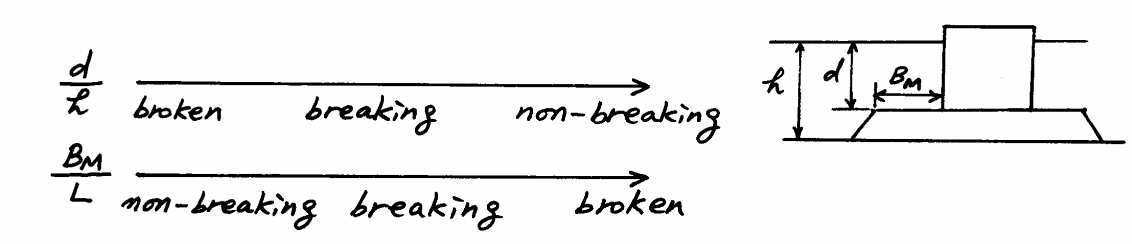
• Use Table 4.1 to check the danger of impulsive breaking waves.
• Impulsive breaking wave pressure may occur
as wave angle ↓ (20°)
bottom slope ↑ (1/50)

H_0'/L_0 ↓ (0.03)

h_c ↑ ($0.3H$)

↑
threshold values

• Also mound height and mound berm width can give favorable conditions for waves to break just in front of the caisson (See Fig. 4.20)



Takahashi et al. (1994) proposed Eqs. (4.24)~(4.31) for the coefficient α_I for impulsive breaking wave pressure.

• It is recommended to design the breakwater not to withstand the impulsive pressure but to avoid the favorable condition for the impulsive breaking wave to occur.

• Countermeasures: Perforated-wall caisson, horizontally composite breakwater

4.3.4 Comments on Design of Concrete Caissons (read text)

4.4 Design of Rubble Mound Foundation

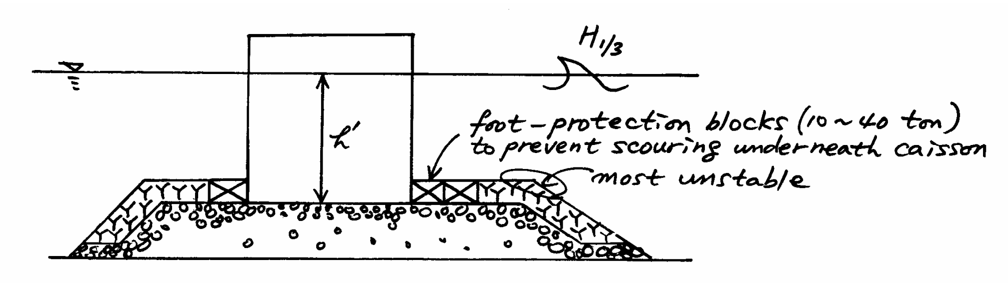
4.4.1 Dimension of Rubble Mound

• Height of mound: The lower, the better. But needs a minimum height (≥1.5 m) to spread the weight of the caisson and wave force over a wide area of seabed and to provide workability of a diver.

• Berm width = 5 ~ 10 m. Wide berm is desirable to protect scouring of seabed, but cost and danger of impulsive pressure on the caisson should be considered.

• Mound slope = 1:2 ~ 1:3 for seaward side, 1:1.5 ~ 1:2 for harbor side.

4.4.2 Foot Protection Blocks and Armor Units



Required mass of armor units

$$M = \frac{\rho_r}{N_s^3 (S_r - 1)^3} H_{1/3}^3$$

ρ_r = density of armor units

($\cong$ 2650 kg/m^3 for quarry stones, 2300 kg/m^3 for concrete blocks)

$S_r = \frac{\rho_r}{\rho_w}$; ρ_w = density of sea water ($\cong$ 1030 kg/m^3)

N_s = stability number given by Eqs. (4.33) ~ (4.38).

4.4.3 Protection against Scouring of the Seabed in Front of a Breakwater