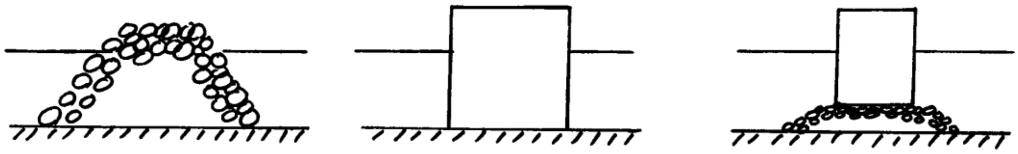
Chap 4. Design of Vertical Breakwaters

4.1 Overviews of Vertical and Composite Breakwaters



rubble mound　　　vertical　　　composite breakwater

↑

"vertical" in Goda's book

Fig. 4.2 X-section of vertical breakwater

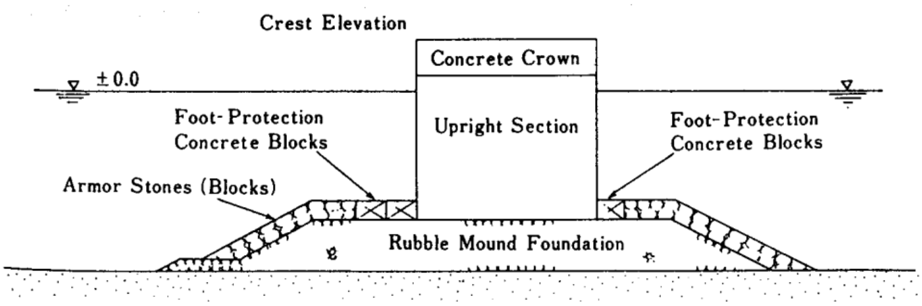
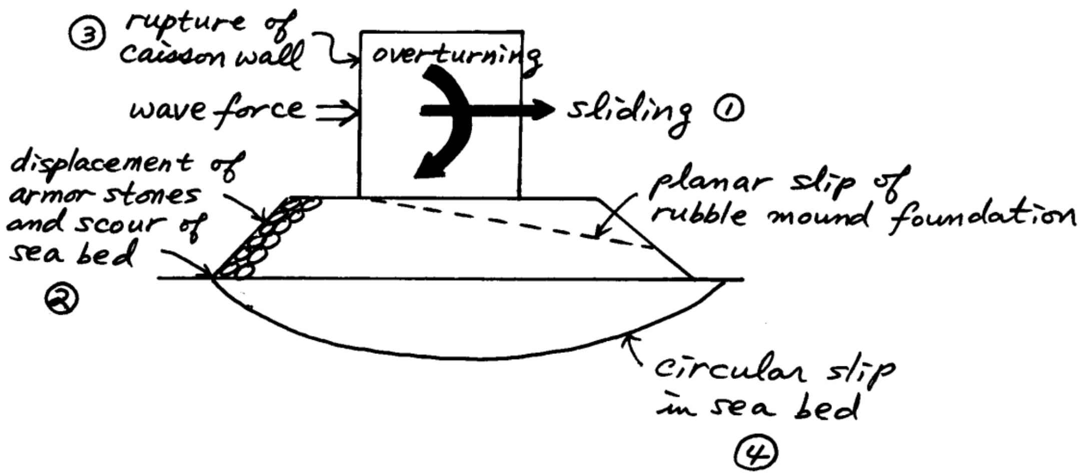


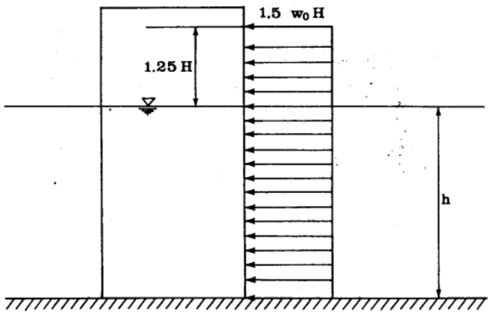
Fig 4.3 Failure modes



4.2 Wave Pressures Exerted on Upright Sections

4.2.1 Overview of Development of Wave Pressure Formulas

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



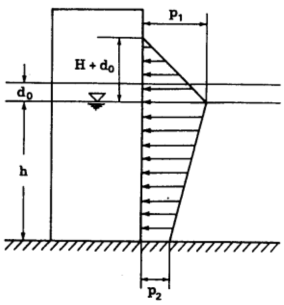
$$p = 1.5\rho g H$$
$$P_{\max} = (h + 1.25H)\,p$$

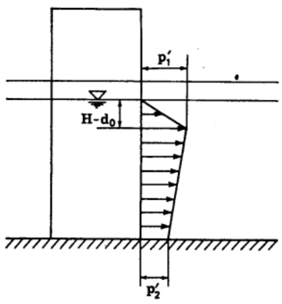
H is usually taken as $H_{1/3}$, which is not much different from $H_{\max}$ in shallow seas

↑

vertically uniform pressure

• 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)

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

Negative force (at wave trough)

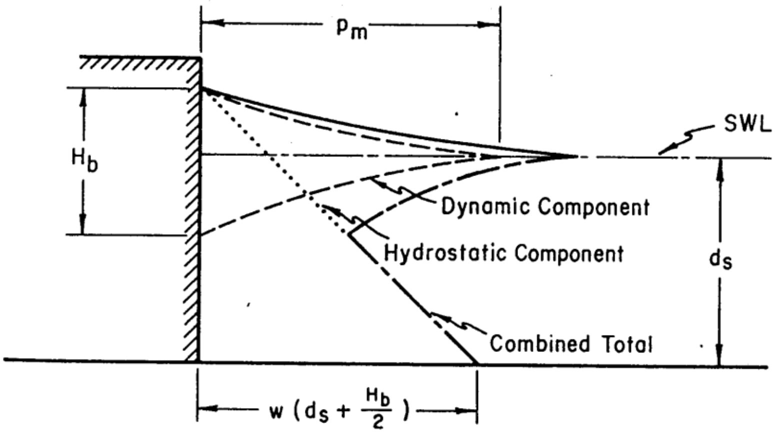
$$\left.\begin{array}{l} p_1' = \rho g (H - d_0) \\ p_2' = p_2 = \dfrac{\rho g H}{\cosh kh} \end{array}\right\} \rightarrow P_{\max} = \frac{p_1'(H - d_0)}{2} + \frac{(p_1' + p_2')(h - H + d_0)}{2}$$

$H = H_{1/3}$, $H_{1/10}$, 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 Goda 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

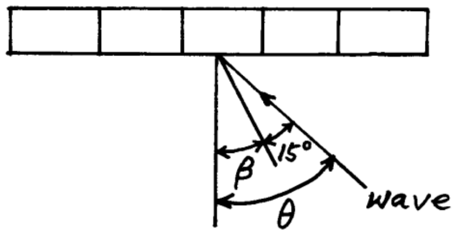
$$\left.\begin{aligned} H_s &= H_{1/3} = \text{significant wave height} \\ T_s &= T_{1/3} = \text{significant wave period} \end{aligned}\right\} \text{at the site of breakwater before construction}$$

(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. (3.48))

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



$\beta = \text{wave angle } (\theta) - 15° \text{ for safety (if } \theta > 15°)$

$\beta = 0° \text{ if } \theta \le 15°$

• Accuracy of the Goda wave pressure formula

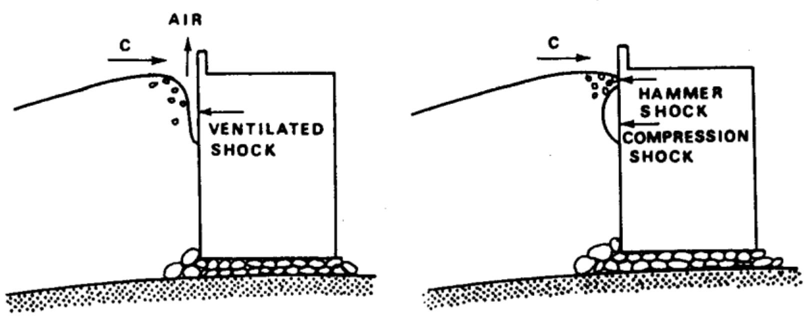
tested against 34 prototype breakwaters under approximately design wave conditions

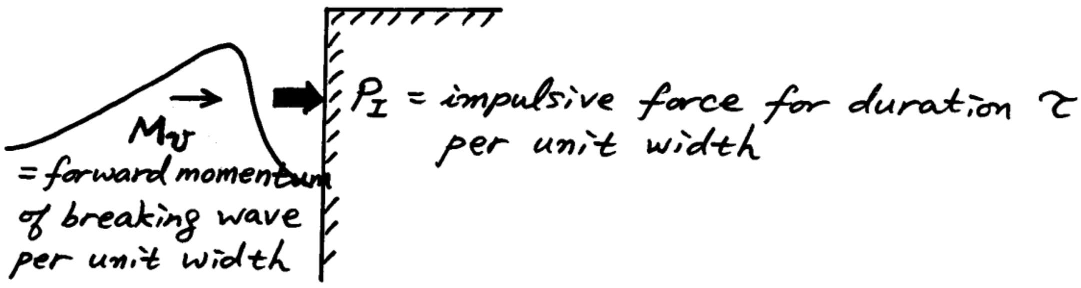
$$\text{safety factor against sliding} = \frac{\text{resistance}}{\text{sliding force}} \quad \begin{matrix} > 1.0 & \text{no sliding} \\ < 1.0 & \text{sliding} \end{matrix}$$

See Fig. 4.9 (a) conventional formulas → poor

(b) Goda formula

4.2.3 Impulsive Breaking Wave Pressure and Its Estimation





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

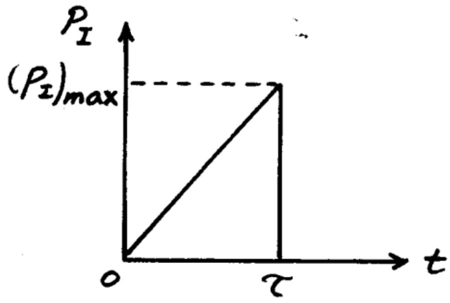
$$M = mu \quad \to \quad dM = mdu = Fdt$$

$$\int_0^\tau dM = [M]_0^\tau = \int_0^\tau F\,d$$

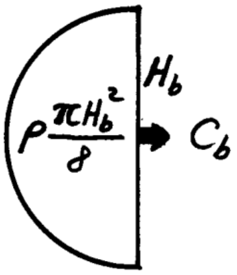
$$M\big|_{t=0} = M_v\,,\quad M\big|_{t=\tau} = 0\,,\quad F = -P_I$$

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

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



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



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

$$(P_I)_{\max} \cong \frac{\rho\pi H_b^2 C_b}{4\tau} = \frac{\rho g\pi H_b^2 C_b}{4g\tau} \qquad (4.18)$$

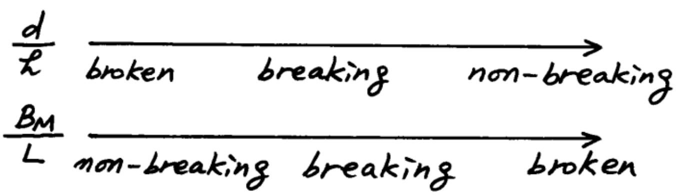
• Impulsive breaking wave pressure may occur
as wave angle ↓ (20°)
bottom slope ↑ (1/50)

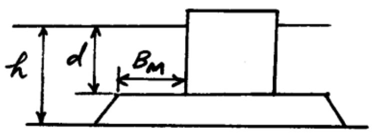
H_0'/L_0 ↓ (0.03)

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





Takahashi et al. (1994) proposed Eqs. (4.19)~(4.26) 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

4.2.4 Sliding of Upright Section by Single Wave Action

In the performance-based design method (Chapter 6), the expected sliding distance during the lifetime of a breakwater is examined, which is the ensemble average of the total sliding distance during the lifetime. The total sliding distance is calculated by accumulating the sliding distances by single individual waves.

Assume the horizontal wave force is the larger between the standing wave force (sinusoidal form) and impulsive wave force (triangular pulse):

$$P(t) = \max\{P_1(t), P_2(t)\}$$

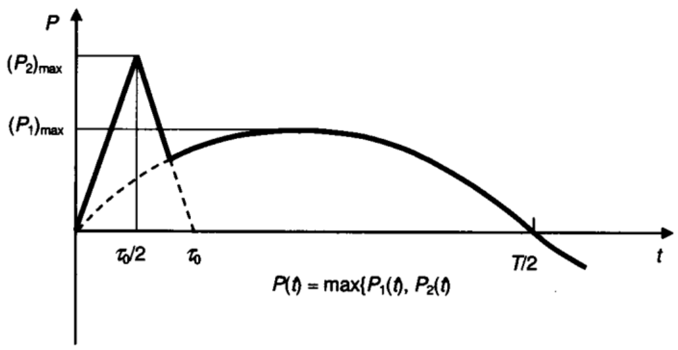


Fig. 4.11 A model of temporal variation of horizontal wave force (after Tanimoto *et al.*[27]).

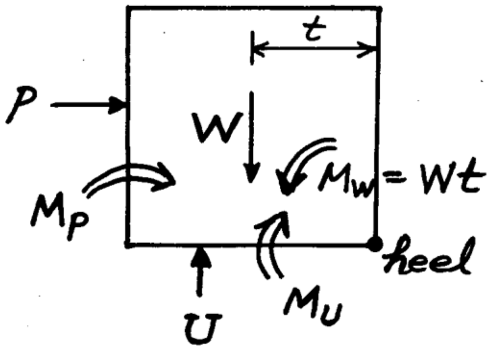
Solve the equation of motion describing caisson sliding:

$$\left(\frac{W_a}{g} + M_a\right)\frac{d^2 x_G}{dt^2} = P - F_R \qquad (4.34)$$

The sliding distance x_G can be calculated by numerically integrating the above equation twice w.r.t. time.

4.3 Preliminary Design of Upright Sections

4.3.1 Stability Condition for an Upright Section



<u>Sliding</u>

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 = $\dfrac{\mu(W-U)}{P}$

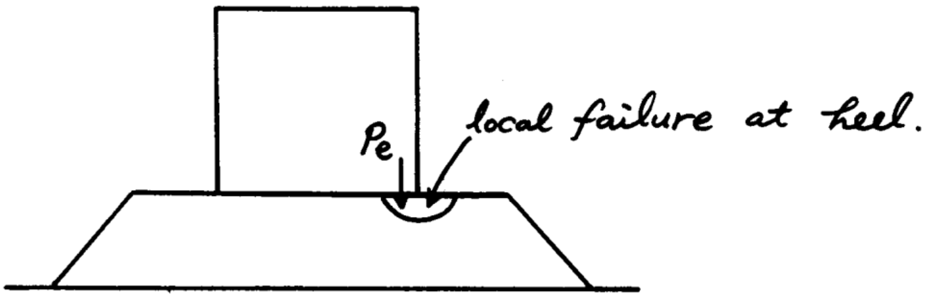
Overturning

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

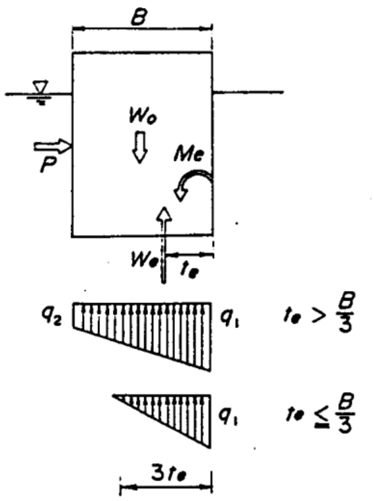
S.F. against overturning = $\dfrac{M_W - M_U}{M_P} = \dfrac{Wt - M_U}{M_P}$

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: $p_e \le 400 \sim 600$ 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 $t_e > \dfrac{B}{3}$, $q_1 = \dfrac{2W_e}{B}\left(2 - 3\dfrac{t_e}{B}\right)$, $q_2 = \dfrac{2W_e}{B}\left(\dfrac{3t_e}{B} - 1\right)$

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

- $q_1 = p_e$ 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 Stable Width of Upright Section

Required $B = \text{function}(H, T, i, D, \beta, h, d)$

See Figs. 4.12~4.16:

$$
\begin{aligned}
B\uparrow \quad \text{as} \quad & H\uparrow \\
& T\uparrow \\
& \left.\begin{aligned} & i\uparrow \text{ in shallow water} \\ & D\uparrow \text{ in shallow water} \end{aligned}\right\} \text{breaking wave force acting on the caisson}
\end{aligned}
$$

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.

4.4 Several Design Aspects of Composite Breakwaters

4.4.1 Wave Pressure under a Wave Trough

$\downarrow$

negative dynamic pressure $\rightarrow$ 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.

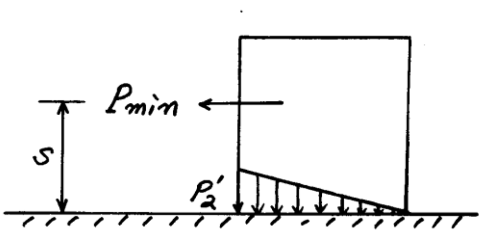


Fig. 4.17: $\dfrac{P_{\min}}{w_0 Hh}$ versus $\dfrac{H}{L}$ and $\dfrac{h}{L}$

Fig. 4.18: $\dfrac{s}{h}$ versus $\dfrac{H}{L}$ and $\dfrac{h}{L}$

Fig. 4.19: $\dfrac{p_2'}{w_0 H}$ versus $\dfrac{H}{L}$ and $\dfrac{h}{L}$

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

4.4.2 Uplift on a Large Footing

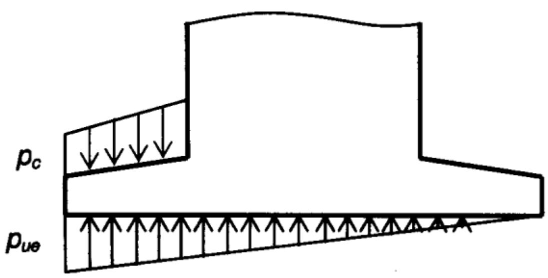
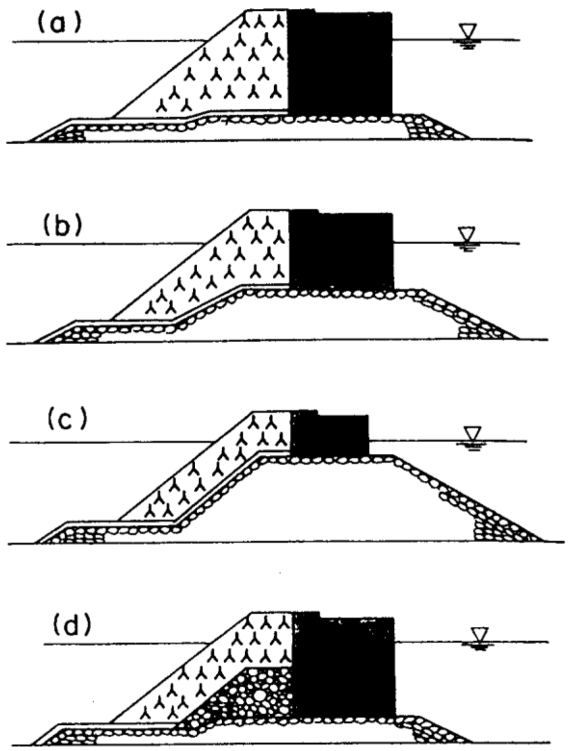


Fig. 4.20 Distributions of downward wave pressure and uplift exerted on a large footing and caisson.

Mostly, the width of footing is 0.5~1.0 m $\leftarrow$ neglected in calculation of uplift force

Large footing is used to reduce the heel pressure or for other purposes. Bending moment and shear stress are important for design of large footing $\leftarrow$ Use Eqs. (4.40) and (4.41) to calculate downward wave pressure p_c and uplift pressure p_{ue}

4.4.3 Wave Pressure on Horizontally-Composite Breakwaters



Pressure correction factors λ_1, λ_2, and λ_3 in Goda's formula for horizontally-composite breakwaters are given by Eqs. (4.42) and (4.43): No impulsive breaking wave force $\rightarrow$ $\lambda_2 = 0$; Standing wave pressure is reduced to 80% of vertical caisson without concrete blocks when $H_{\max} / h > 0.6$, but no reduction when $H_{\max} / h \le 0.3$.

4.4.4 Comments on Design of Concrete Caissons (read text)

4.5 Design of Rubble Mound Foundation

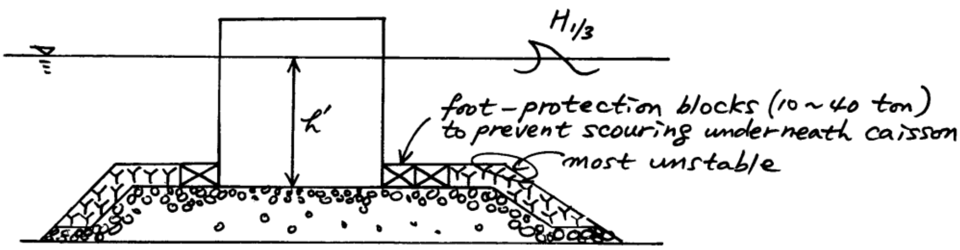
4.5.1 Dimension of Rubble Mound

• 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.

4.5.2 Foot Protection Blocks and Armor Units



Height of foot protection blocks

$$t = AH_{1/3}\left(\frac{h'}{h}\right)^{-0.787} : \quad 0.4 \leq \frac{h'}{h} \leq 1.0, \quad A = 0.21\,(\text{head}),\ 0.18\,(\text{trunk})$$

Required mass of armor units

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

$$D_n = \frac{H_{1/3}}{N_s \Delta} : \quad \Delta = S_r - 1$$

ρ_r = density of armor units

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

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

N_s = stability number given by Eqs. (4.47) ~ (4.55).

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