Chapter 7. Coastal Structures

7.1 Hydrodynamic Forces in Unsteady Flow

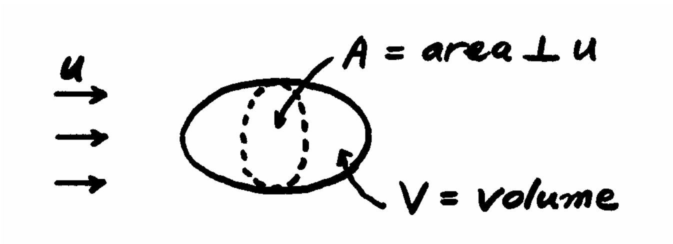
↑

Morison equation

Total force = drag force + inertia force:

$$F = F_d + F_i = \frac{1}{2} C_d \rho A u^2 + C_m \rho V \frac{du}{dt}$$

↑ viscosity ↑ acceleration (unsteady flow)

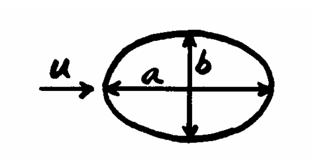


ρ = density of fluid

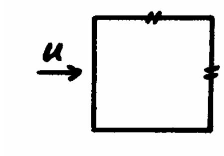
C_d = drag coefficient = $f(\mathbf{R}, \text{roughness})$

C_m = inertia coefficient = $1 + k$; k = added mass coefficient

In potential flow, for elliptic cylinder, $k = \frac{b}{a}$; $k = 1$ for circular cylinder $(a = b)$

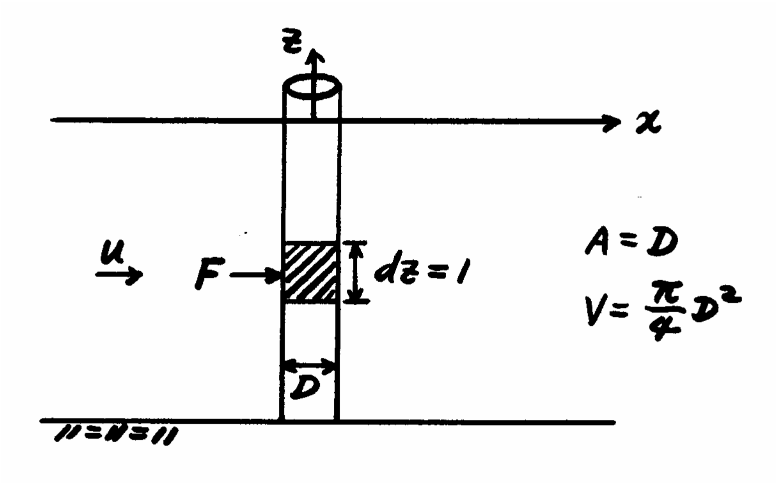


For square cylinder, $k = 1.2$



In a real fluid, $k = f(\text{body's shape, roughness, } \mathbf{R}, \cdots)$

7.2 Piles, Pipelines, Cylinders ← long cylindrical structures



$$F = \frac{C_d}{2}\rho D u^2 + C_m \rho \frac{\pi D^2}{4}\frac{\partial u}{\partial t}$$

Since u and $\partial u / \partial t$ are $90°$ out of phase,

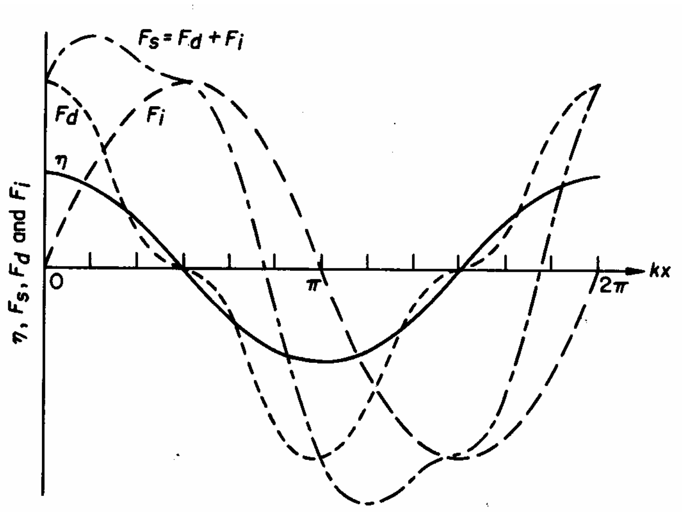


Figure 7.1. **Surface elevation and drag, inertia, and total forces versus phase position—for equal peak drag and inertia components.**

$F_{\max}$ occurs somewhere between $kx = 0$ and $kx = \pi/2$. And also $F_d = 0$ at $(F_i)_{\max}$, and $F_i = 0$ at $(F_d)_{\max}$. Using $\partial F / \partial(kx) = 0$ at $F_{\max}$,

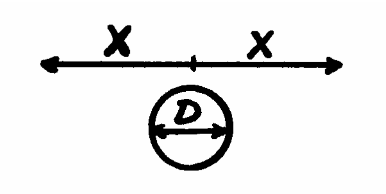
$$\sin(kx)_m = \frac{2C_m V \sinh kd}{C_d AH \cosh k(z+d)} \rightarrow \text{indicates relative magnitudes of } F_d \text{ and } F_i$$

Since $V \propto D^2$ and $A \propto D$,

$$\sin(kx)_m \propto \frac{D}{H}$$

Therefore, large D/H $\rightarrow$ inertia-dominant
small D/H $\rightarrow$ drag-dominant

<u>Keulegan-Carpenter number</u> (KC)



$$u = u_m \cos\sigma t\,; \quad X = \int_0^{T/4} u_m \cos\sigma t dt = \frac{u_m T}{2\pi}$$

$$\frac{X}{D} = \frac{1}{2\pi}\left(\frac{u_m T}{D}\right)$$

Defining $u_m T/D$ as Keulegan-Carpenter number (KC), since $u_m \propto \pi H/T$,

$$KC \propto \frac{H}{D}$$

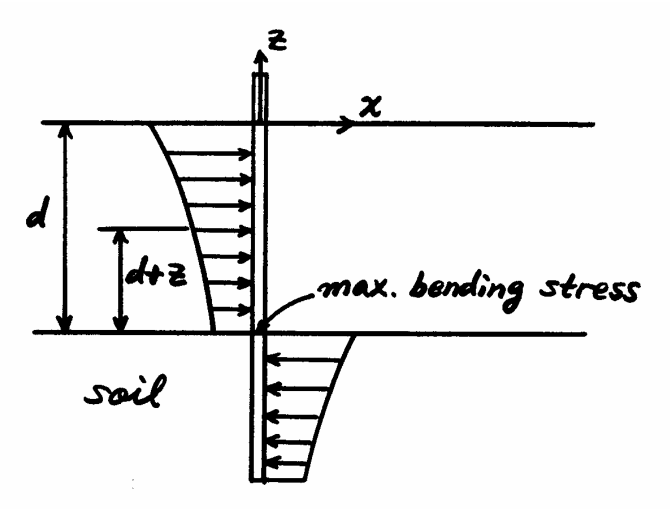
Therefore, large KC $\rightarrow$ drag-dominant
small KC $\rightarrow$ inertia-dominant

Vertical pile

Total horizontal force acting on the pile is

$$
\begin{aligned}
F &= \int_{-d}^{\eta} \left(F_d + F_i\right) dz \\
&\cong \int_{-d}^{0} \left(\frac{C_d}{2} \rho D u^2 + C_m \rho \frac{\pi D^2}{4} \frac{\partial u}{\partial t} \right) dz \\
&= \frac{C_d}{8} \rho g D H^2 n \cos^2(\sigma t) + C_m \rho g \frac{\pi D^2}{8} H \tanh kd \sin(-\sigma t)
\end{aligned}
$$

Total moment about mudline is



$$M = \int_{-d}^{\eta} \left(F_d + F_i\right)(d+z)\,dz \quad = \text{Eq. (7.8)}$$

Maximum force and moment?

$$\frac{\partial F}{\partial(\sigma t)} = 0 \ \rightarrow \text{find} \ F_{\max}; \qquad \frac{\partial M}{\partial(\sigma t)} = 0 \ \rightarrow \text{find} \ M_{\max}$$

Determination of C_d and C_m (Read text p. 195-198)

$$C_d, C_m = f(\mathbf{R})$$

$C_d \uparrow$ as roughness of cylinder $\uparrow$

7.3 Large Submerged Structures

Use potential flow theory

$$\phi = \phi_i + \phi_s$$

where ϕ_i = incident wave potential (known), and ϕ_s = scattered wave potential (unknown).

Solve $\nabla^2 \phi = 0$ for ϕ using the boundary condition $\partial\phi / \partial n = 0$ on body surface. Then

$$p = -\rho g z - \rho \frac{\partial \phi}{\partial t}$$

$$F = \int_S p dS$$

7.4 Floating Breakwaters

Advantages: Read text
Disadvantage: effective only for short period waves ($T \leq 3$ s)

7.5 Rubble Mound Structures

Advantages:

1) gradual (not sudden) damage during storms
2) smaller wave reflection
3) easy water exchange between ocean and harbor

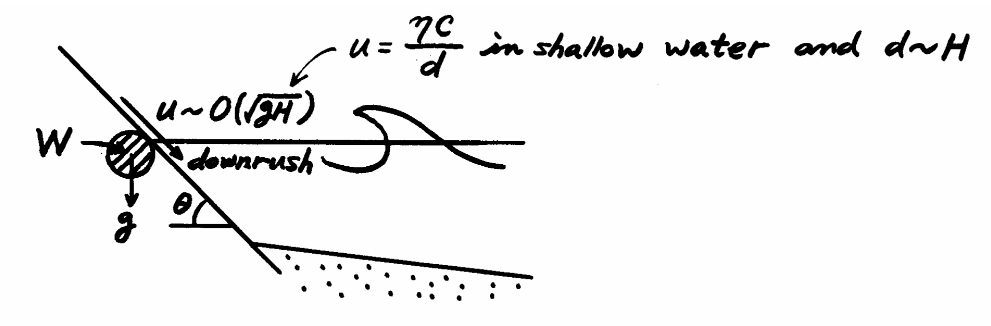
⋮

See figure 7.4 for typical cross-section

1) Fore-slope $\leq 1:1.5$
2) Width of cap concrete $\geq 3\times$ A-stone diameter
3) Weight of A-stone = W

   B-stone = $\left(\frac{1}{10} \sim \frac{1}{15}\right)W$, C-stone $\cong \frac{1}{200}W$
4) Outer layer (A-stone) usually consists of two layers of armor units

How to estimate W ?



Let l = characteristic length of armor unit, so that

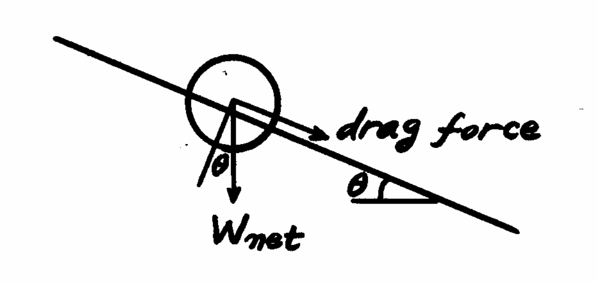
$$W \propto \rho_s g l^3 = \gamma_s l^3$$

where γ_s = unit weight of armor unit. Assuming the armor unit is fully submerged (worst case),

$$W_{net} \propto (\gamma_s - \gamma_w) l^3$$

Express drag force $\propto \frac{1}{2}\rho_w C_d u^2 l^2$.

Force balance at initiation of movement:



$$\text{drag force} + W_{net}\sin\theta = \mu W_{net}\cos\theta$$

where μ = friction coefficient between armor units. Now

$$\frac{1}{2}\rho_w C_d u^2 l^2 \propto W_{net}(\mu\cos\theta - \sin\theta)$$

Using $u \sim O(\sqrt{gH})$,

$$\frac{1}{2}\rho_w g C_d H l^2 \propto (\gamma_s - \gamma_w) l^3 (\mu\cos\theta - \sin\theta)$$

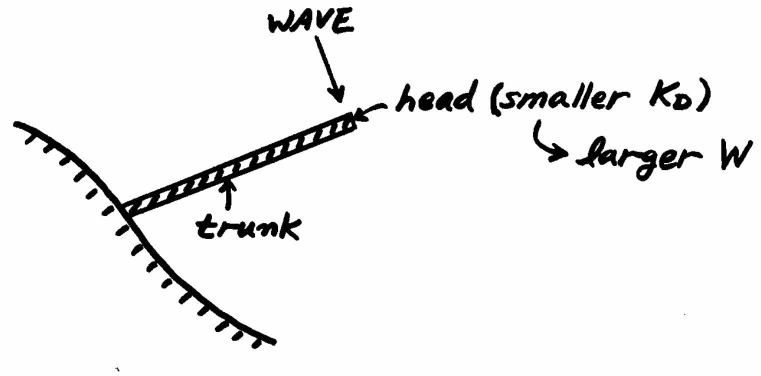
$$l \propto \frac{H}{(\gamma_s - \gamma_w)(\mu\cos\theta - \sin\theta)}$$

$$l \propto \frac{H}{(S-1)(\mu\cos\theta - \sin\theta)}; \quad S = \frac{\gamma_s}{\gamma_w}$$

Since $W \propto \gamma_s l^3$,

$$W = \frac{\gamma_s H^3}{K_D (S-1)^3 \cot\theta}$$

This is the Hudson formula, where $\cot\theta$ is used instead of $(\mu\cos\theta - \sin\theta)^3$, and K_D = stability coefficient, which includes everything not accounted for (see Table 7.1).



Stability number, $N_s = \left(K_D \cot\theta\right)^{1/3} = \dfrac{H}{(S-1)(W/\gamma_s)^{1/3}}$

• van der Meer (1988, 1995) included the effects of wave period, breaker type, storm duration, damage level, permeability.

• Berm breakwater

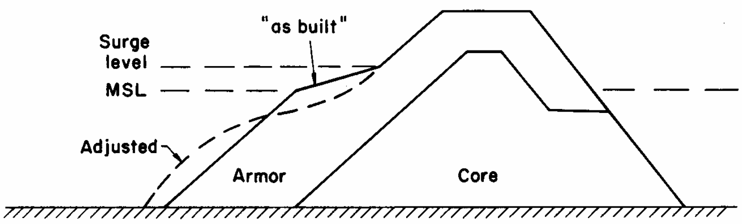
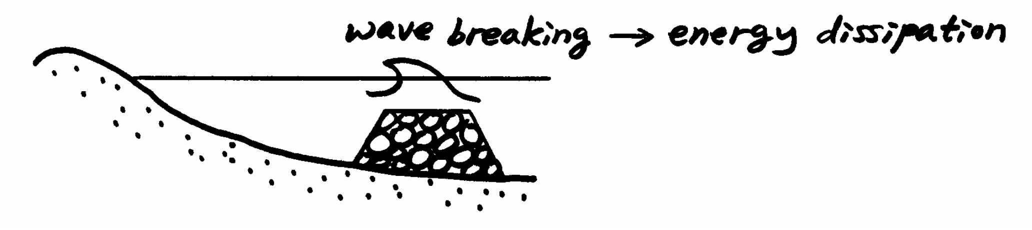


Figure 7.7. Berm breakwater cross-section.

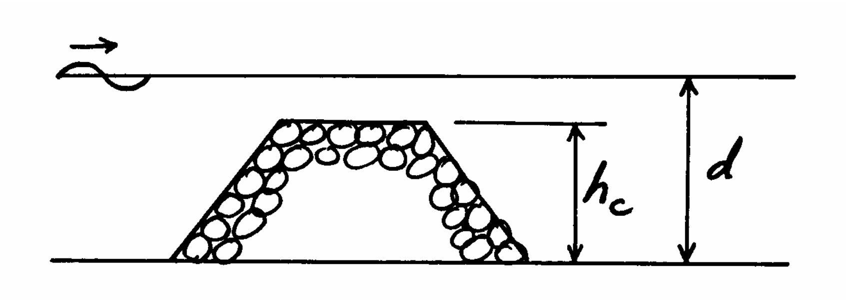
- Rubble mound forms an S-shaped profile to stabilize itself against wave action.
- Construct a rubble mound being close to its equilibrium profile from the beginning.
- Smaller size and wider size range of armor stones

• Low-crested (or submerged) breakwater

- mostly for shore protection
- good for seascape



- Failure usually occurs at the top of the breakwater

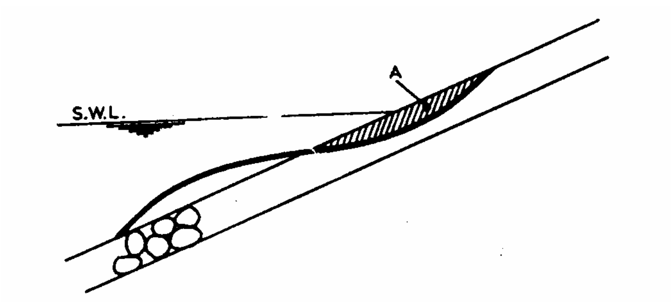


$$\frac{h_c}{d} = (2.1 + 0.1S_d)e^{-0.14N_S^*}$$

$$N_S^* = \frac{H^{2/3}L^{1/3}}{(S-1)(W/\gamma_s)^{1/3}}$$

S_d = damage level = $\dfrac{A}{D_{50}^2}$; $\quad D_{50} = \left(\dfrac{W}{\gamma_s}\right)^{1/3}$

S_d = 2.0 for onset of damage, and 8.0 for failure.



Note: S is used as both specific gravity and damage level in the textbook.

7.6 Rigid Vertical-Faced Structures

Non-breaking wave forces

↑

standing wave pressure + wave setup

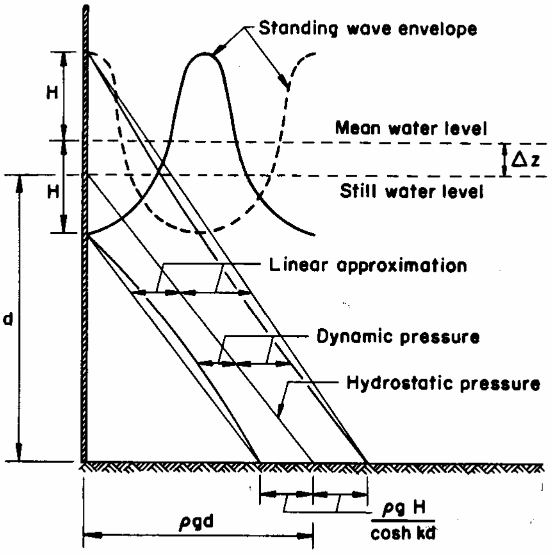


Figure 7.8. **Standing wave pressure distributions on a vertical wall. (U.S. Army Coastal Engineering Research Center, 1984.)**

$$p_d(z) = \rho g H \frac{\cosh k(d+z)}{\cosh kd} \cos \sigma t$$

$$p_d(z=-d) = \frac{\rho g H}{\cosh kd} \cos \sigma t; \qquad H = \frac{1+C_r}{2} H_i$$

$$\Delta z(\text{wave setup}) = \frac{\pi H^2}{L} \coth kd$$

Assuming standing wave pressure varies linearly (see Figure 7.8),

Crest: from 0 (at $z = \Delta z + H$) to $\rho g d + \dfrac{\rho g H}{\cosh kd}$ (at $z=-d$)

Trough: from 0 (at $z = \Delta z - H$) to $\rho g d - \dfrac{\rho g H}{\cosh kd}$ (at $z=-d$)

Breaking wave forces

Goda formula (1974)

- Applicable for both non-breaking and breaking wave forces
- Horizontal force (on front of caisson) + uplift force (on bottom of caisson)

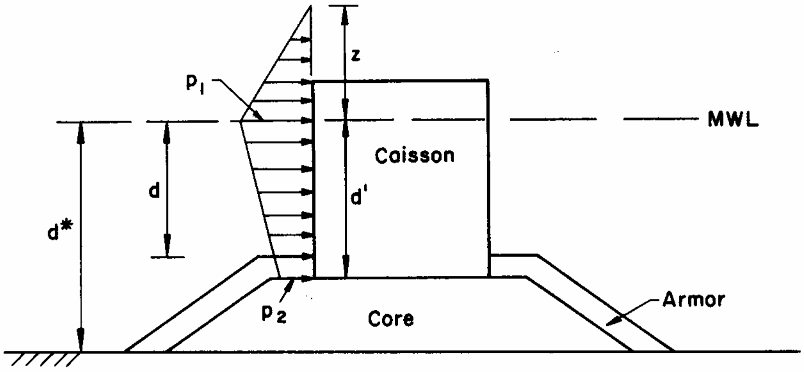


Figure 7.9. **Broken wave pressure distribution on a caisson. (From Goda, 1985.).**

$$z = 0.75(1+\cos\beta)H_{max}$$

$$\beta = \begin{cases} \theta - 15^\circ & \text{if } \theta > 15^\circ \\ 0^\circ & \text{if } \theta \le 15^\circ \end{cases} \quad \text{for safety}$$

where θ = wave angle from normal to the breakwater

$$p_1 = 0.5(1+\cos\beta)(\alpha_1 + \alpha_2 \cos^2\beta)\gamma H_{max}$$

$$p_2 = \alpha_3 p_1$$

$$p_3 = \frac{p_1}{\cosh kd^*}$$

$$p_4 = \begin{cases} p_1(1-d_c/z) & \text{for } z > d_c \\ 0 & \text{for } z \le d_c \end{cases}$$

$$p_u = 0.5(1+\cos\beta)\alpha_1\alpha_3\gamma H_{max}$$

where

$$\alpha_1 = 0.6 + 0.5\left(\frac{2kd^*}{\sinh 2kd^*}\right)^2$$

$$\alpha_2 = \min\left\{\frac{d_b - d}{3d_b}\left(\frac{H_{\max}}{d}\right)^2, \frac{2d}{H_{\max}}\right\}$$

$$\alpha_3 = 1 - \frac{d'}{d^*}\left(1 - \frac{1}{\cosh kd^*}\right)$$

d_b = water depth at $5H_s$ from the breakwater

7.7 Other Loadings on Coastal Structures

current, wind, ice, earthquakes (read text)

7.8 Wave-Structure Interactions

Reflection coefficient:

$$C_r = \frac{aI_r^2}{b + I_r^2}$$

where a, b = empirical constants,

$I_r = \dfrac{m}{\sqrt{H / L_0}}$ = Iribarren number

Wave runup:

Regular wave on smooth impermeable slope: Figure 2.15 $\rightarrow$ R_i

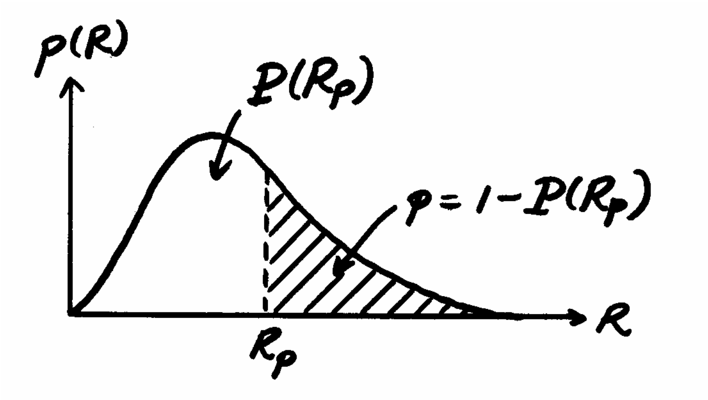
Regular wave on rough permeable slope: $R_r = rR_i$; r is given in Table 2.1

For irregular waves,

$$R_p = R_s\left(\frac{\ln(1/p)}{2}\right)^{1/2}$$

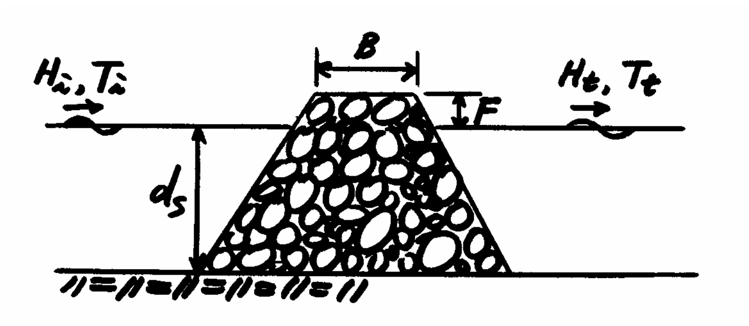
where $R_s = R$ for H_s (assuming regular wave)

R_p = run-up for $1 - P(R_p) = p$



If $R >$ crest elevation, overtopping occurs. Wave overtopping is a very complex phenomenon, so usually hydraulic model test is used.

Wave transmission:



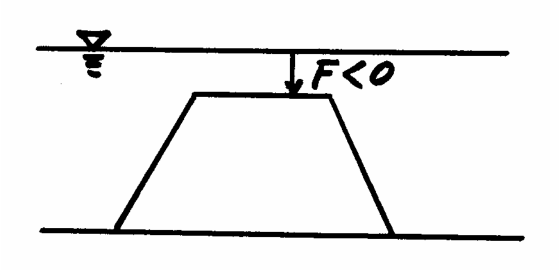
Wave transmission mainly due to overtopping: $T_t < T_i$ in general

$$C_t = \frac{H_t}{H_i} = C\left(1 - \frac{F}{R}\right); \quad C = 0.51 - \frac{0.11B}{d_s + F}$$

For low-crested breakwaters,

$C_t = f(F / H_i)$ ← see Figure 7.10

Note that F can be negative.



7.9 Selection of Design Waves

Wave measurements or hindcasting (using meteorological data)

↓ extreme wave analysis

Deepwater wave of particular return period

↓ wave transformation model (shoaling, refraction, diffraction)

Waves at the location of structure

↓ Rayleigh distribution

$H_{1/3}$ or $H_{1/10}$ for rubble mound structures

$H_{1/100}$ or $H_{\max}$ for rigid structures (e.g. vertical caisson)