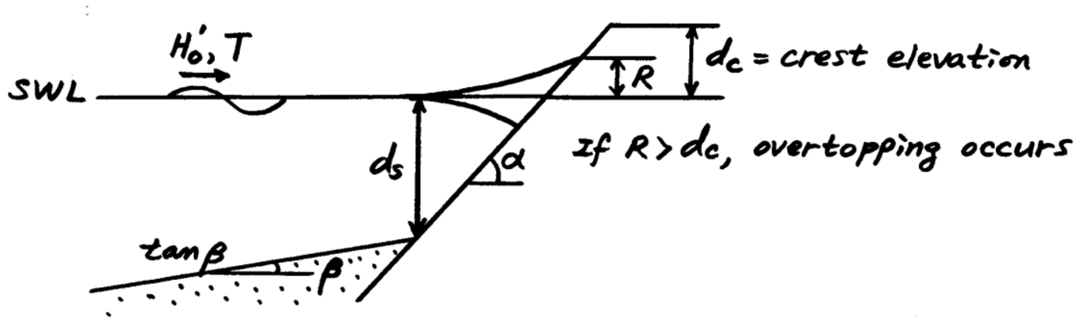
Chap 5. Design of Coastal Dikes and Seawalls

5.1 Random Wave Run-Up on Coastal Dikes and Seawalls

5.1.1 Run-Up Height by Standing Waves at Vertical Wall



For non-breaking regular waves in front of a vertical wall, the crest height $\eta_{\max}$ is estimated by Stokes 4th order standing wave theory:

$$\frac{\eta_{\max}}{H_I} = 2 - \exp\left[-A\left(\frac{H_I}{L_A}\right)^b \right] : A = -0.153 - 2.153\ln(h/L_A),\ b = 1 - 0.06[\ln(h/L_A)]^2 \quad (5.1)$$

which is plotted in Fig. 5.1 as a function of H_I / L_A and h / L_A.

For non-breaking random waves: Rayleigh distribution of wave height → Calculate $\eta_{\max}$ for individual wave heights using Eq. (5.1) → Obtain the distribution of $\eta_{\max}$, which will be wider than the Rayleigh distribution, because $\eta_{\max} / H_I$ is larger for waves of large height than waves of small height due to nonlinear effect (the larger wave height, the more peaked crest; $\eta_{\max} / H_I \simeq 1$ for small H_I, but $\eta_{\max} / H_I \uparrow$ as $H \uparrow$)

5.1.2 Run-Up Height on Smooth Slopes and Coastal Dikes

Representative run-up heights on smooth slopes are given by

$$\frac{R_X}{H_0} = aI_{r,s}^b : 1/30 \le \tan\alpha_s \le 1/5,\ 0.007 \le H_0/L_0 \quad (5.2)$$

where $X = \text{max, 2\%, 1/10, 1/3, or 1}$, a, b = coefficients given in Table 5.1, and

$I_{r,s} = \tan\alpha_s / \sqrt{H_0/L_0}$ = Iribarren number.

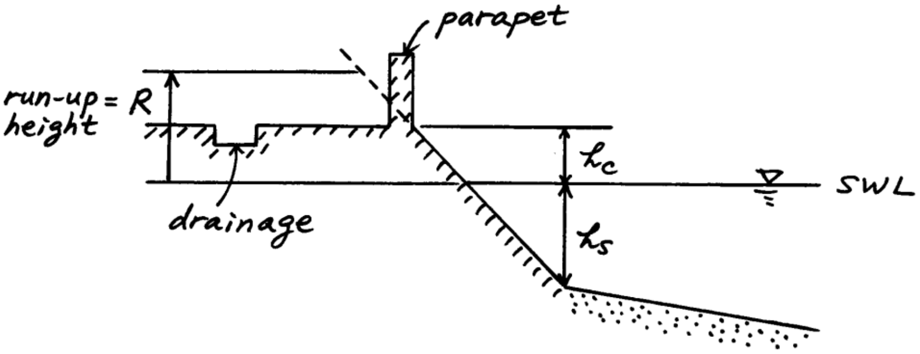
Eq. (5.2) gives a distribution of run-up heights narrower than the Rayleigh distribution, because it was obtained on very gentle slopes in relatively deep water where large waves may lose their energy while propagating on the slope. The run-up height follows the Rayleigh distribution on steeper slopes in shallow water.

For coastal dikes protected with armor layers,

$$\frac{R_{2\%}}{\gamma_f \gamma_\beta H_{1/3}} = \begin{cases} 1.35 I_{r,s,-1} & : I_{r,s,-1} \le 1.74 \\ 4.7 - 4.09 / I_{r,s,-1} & : I_{r,s,-1} > 1.74 \end{cases} \quad (5.3)$$

where $\gamma_f, \gamma_\beta =$ influence factors for slope roughness and oblique wave attack, and $I_{r,s,-1} =$ Iribarren number based on $T_{m-1,0} = m_{-1}/m_0$ and $H_{1/3}$ in front of the dike

5.2 Wave Overtopping Rate of Coastal Dikes and Seawalls



Overtopping occurs if $R > h_c$ (if no parapet)

Overtopping may cause flooding, erosion, wave transmission

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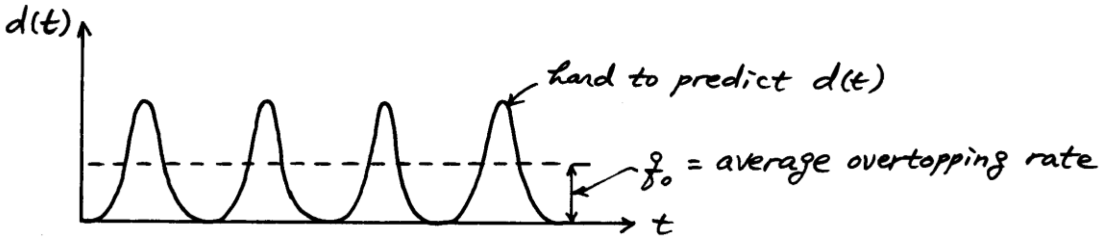
drainage protection mooring of ships

5.2.0 Overtopping Rate of Regular Waves



$d(t)$ = discharge per unit crest length

$q_0 = \dfrac{1}{T_m}\int_0^{T_m} d(t)dt$; T_m = duration of experiment

Volume = $q_0 T_m$

Empirical formula (in SPM):

$$q_0 = \begin{cases} \left(g q_0^* H_0'^3\right)^{1/2}\left(\dfrac{R-h_c}{R+h_c}\right)^{\alpha^*} & \text{for } R > h_c \\ 0 & \text{for } R \le h_c \end{cases}$$

where H_0' = unrefracted deepwater wave height

q_0^*, α^* = empirical constants

$$\alpha^* = 0.1085/\alpha$$

Estimate α, q_0^* using Figs. 7-24 to 7-29 in SPM ($Q_0^* = q_0^*$)

Usually $\alpha^* = 1 \sim 2$.

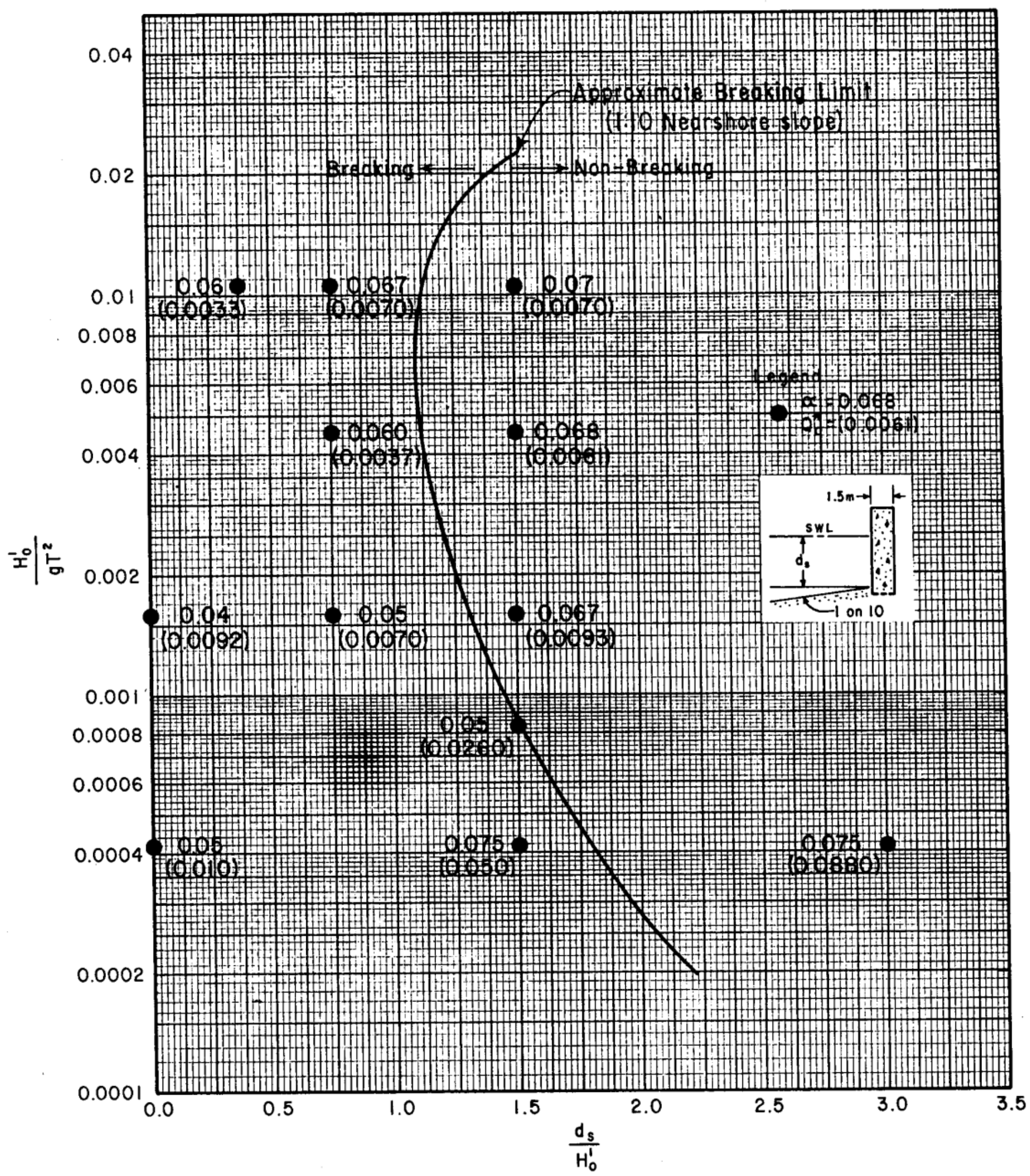


Figure 7-24. Overtopping parameters α and Q_0^* (smooth vertical wall on a 1:10 nearshore slope).

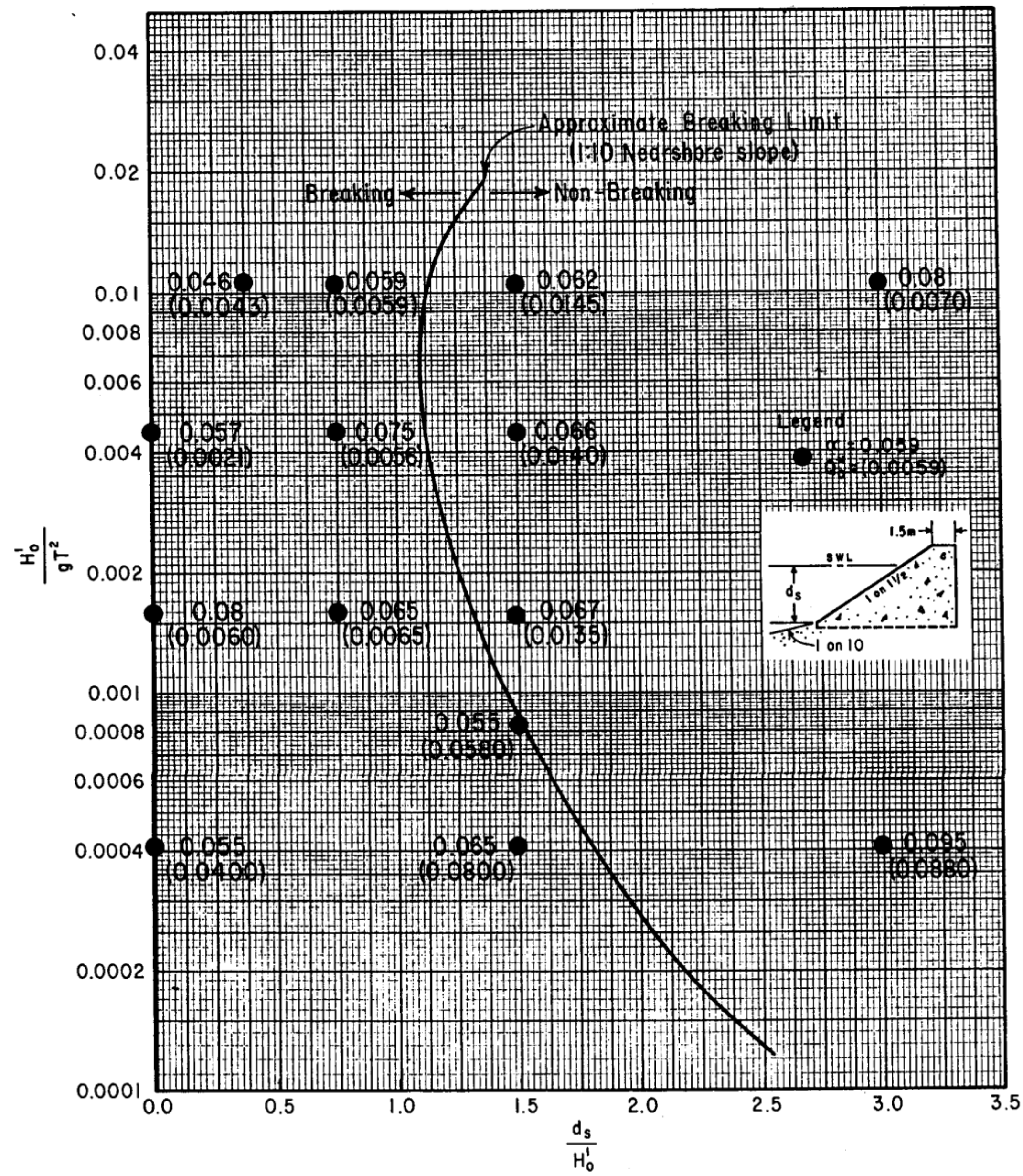


Figure 7-25. Overtopping parameters α and Q_0^* (smooth 1:1.5 structure slope on a 1:10 nearshore slope).

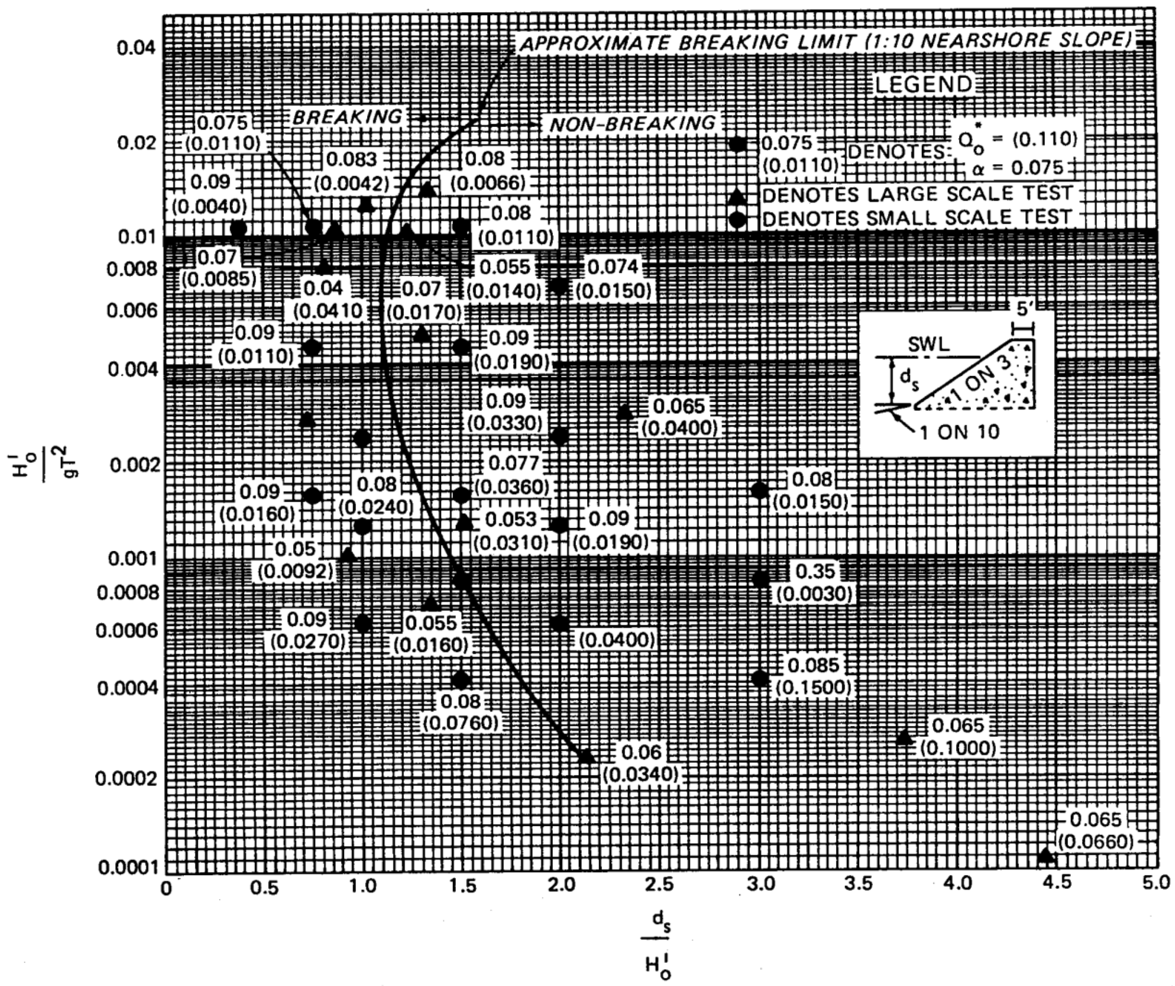


Figure 7-26. Overtopping parameters α and Q^*_o (smooth 1:3 structure slope on a 1:10 nearshore slope).

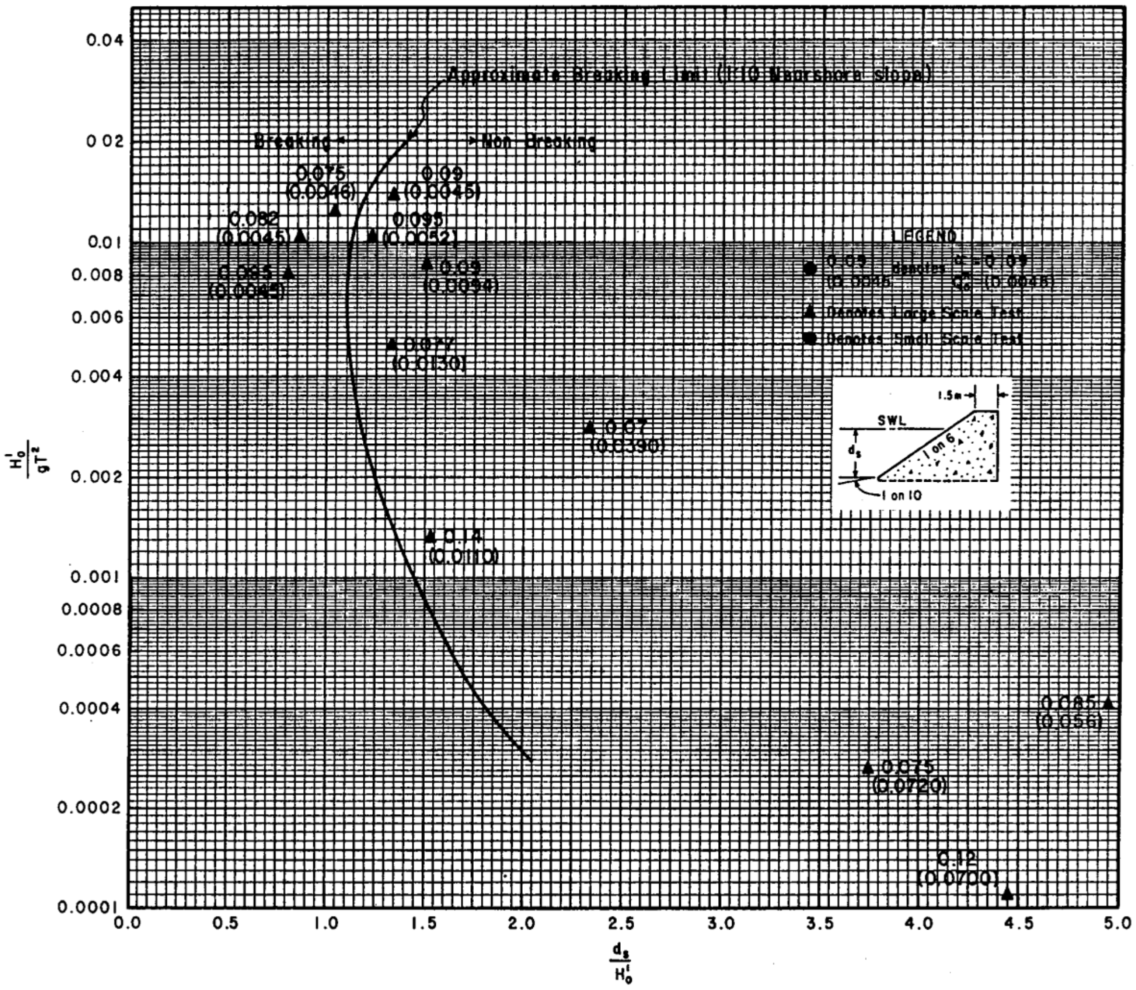


Figure 7-27. Overtopping parameters α and Q_0^* (smooth 1:6 structure slope on a 1:10 nearshore slope).

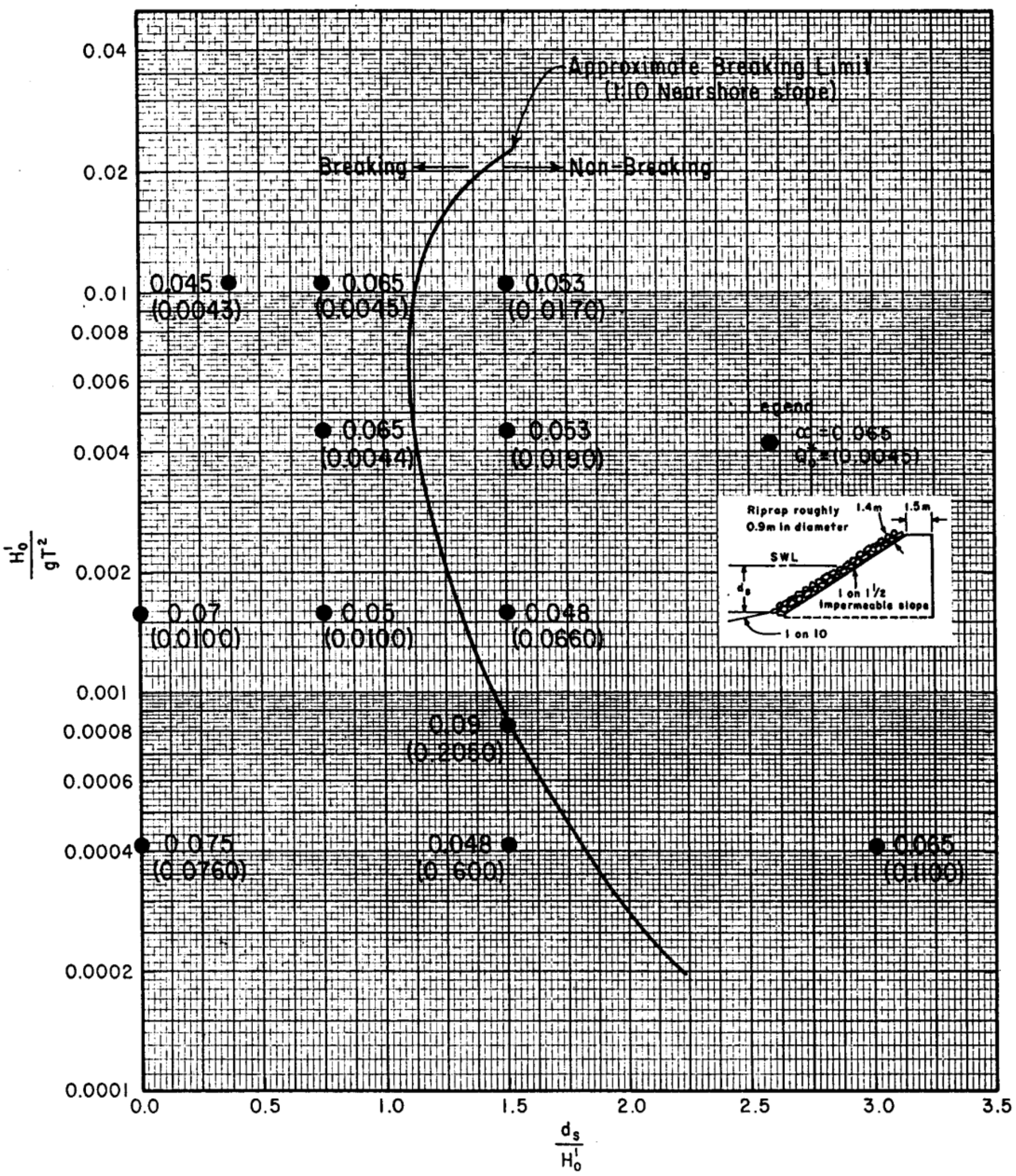


Figure 7-28. Overtopping parameters α and Q_o^* (riprapped 1:1.5 structure slope on a 1:10 nearshore slope).

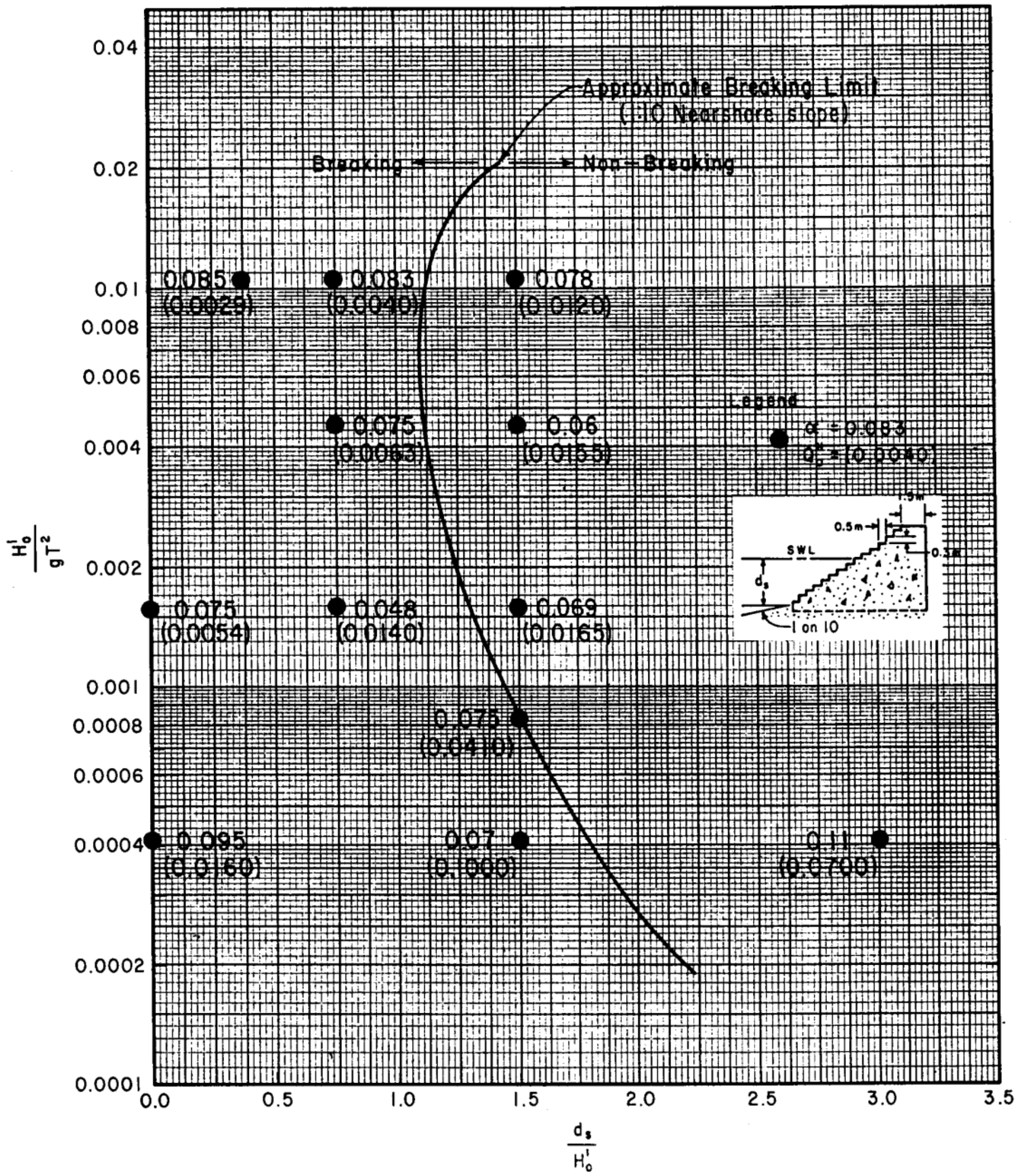


Figure 7-29. Overtopping parameters α and Q_o^* (stepped 1:1.5 structure slope on a 1:10 nearshore slope).

5.7 越波量

越波量은 그림 5.7.1에 도시된 바와 같은 장치를 이용하여 측정하였다. 이 장치는 방파제 배후에 위치한 內徑 50 cm의 원통형 물받이 통과 방파제 천단에서 이 용기까지를 연결시켜주는 도수로로 구성되어 있다. 도수로는 다시 3 개의 수로로 나누어져 있고 중앙수로를 제외한 좌우측 수로는 수문을 설치하여 월파량의 크기에 따라 수문을 열고 닫음으로써 원통형 용기에 도달하는 유량을 조절하도록 하였다. 이 導水路의 방파제 윗 부분은 수평이나, 이후 물받이 통까지는 경사지게 하여 일단 방파제 배후로 월파된 물은 되돌아오지 않도록 하였다. 용기에 담겨진 물은 펌핑하여 무게를 달아 월파량을 환산하였다. 월파량은 실험실에서 420초(설계유의파 주기 200 개의 시간) 동안 관측된 값으로부터 원형에서의 單位길이당 單位시간당 부피로 환산하여 제시하였다.

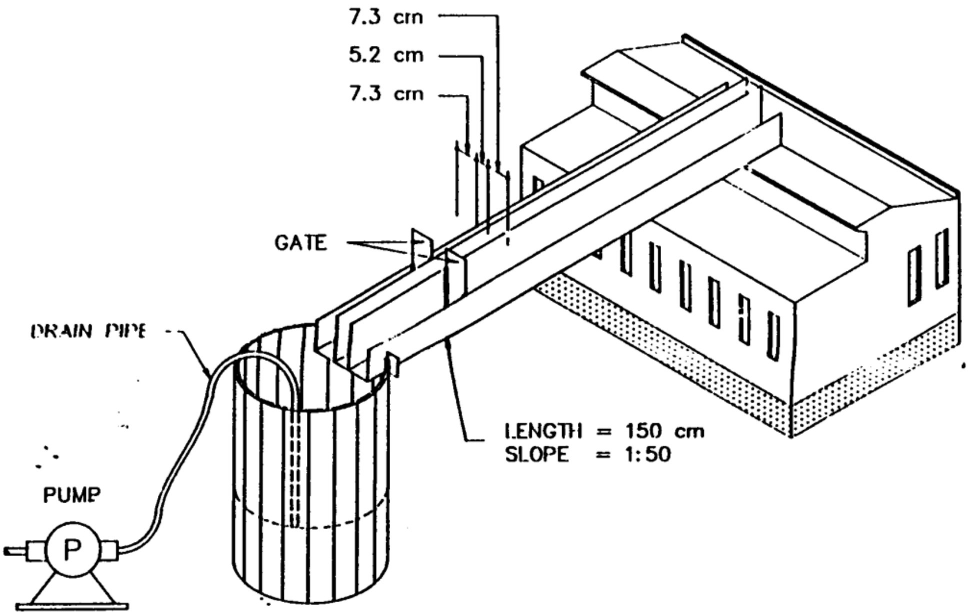


그림 5.7.1 월파량 측정장치.

5.2.1 Overtopping Rate by Random Sea Waves

Overtopping rate averaged over the duration of storm waves

$$q = \frac{1}{t_0} \sum_{i=1}^{N_0} Q(H_i, T_i)$$

where $t_0 = \sum_{i=1}^{N_0} T_i$ = duration of storm waves

N_0 = total number of waves

$Q(H_i, T_i)$ = amount of overtopped water by i -th individual wave with height H_i and period T_i

Overtopping rate of random waves, q, can be approximated by using q_0 for regular waves:

$$q \cong \frac{1}{t_0} \sum_{i=1}^{N_0} T_i q_0 (H_i, T_i)$$

which neglects 1) random process of wave breaking, 2) presence of surf beat, 3) interference by preceding waves.

Assuming $T_i = T_{1/3}$ for all i (= 1 to N_0), further approximation can be made to give

$$q \cong q_{\exp} = \int_0^{\infty} q_0 (H \mid T_{1/3}) p(H) dH$$

where

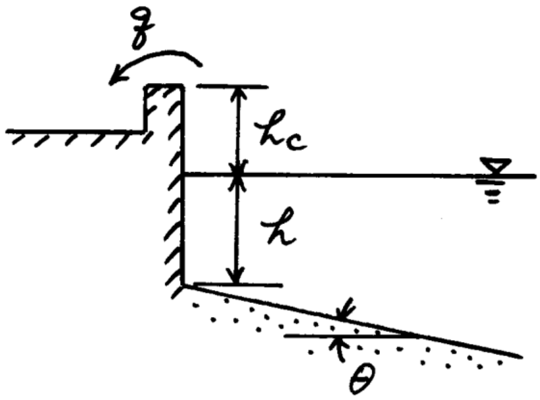
$q_0 (H \mid T_{1/3})$ = overtopping rate by regular waves with height H and period $T_{1/3}$

$p(H)$ = probability density function of wave height (e.g. Rayleigh distribution)

Note: Estimation of overtopping rate by assuming the random waves as a regular wave with $H_{1/3}$ and $T_{1/3}$ may give underestimated results because the estimation ignores the existence of individual waves higher than the significant wave.

5.2.2 Mean Rate of Wave Overtopping at Vertical and Block-Mound Seawalls

Vertical revetment



Dimensional analysis indicates

$$\frac{q}{\sqrt{2g(H_0')^3}} = \mathrm{function}\left(\theta, \frac{H_0'}{L_0}, \frac{h}{H_0'}, \frac{h_c}{H_0'}\right)$$

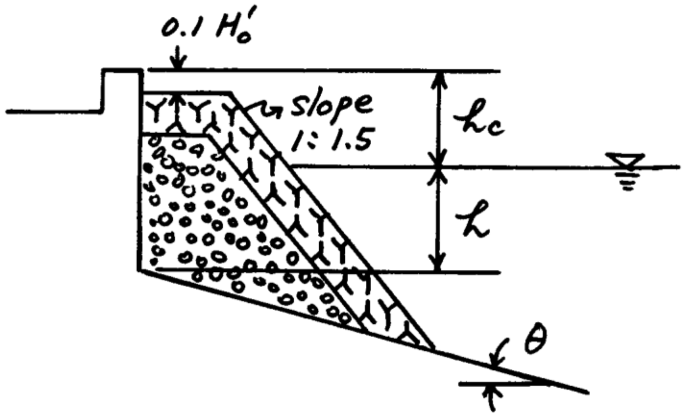
H_0' = equivalent deepwater wave height corresponding to $H_{1/3}$

Use Figs. 5.3 and 5.4 in the text.

For given $\theta, \frac{H_0'}{L_0}, \frac{h}{H_0'}, \frac{h_c}{H_0'}$ $\rightarrow$ obtain $q/\sqrt{2g(H_0')^3}$ using the left part of the figures

$\rightarrow$ calculate q directly for given H_0' (or use the right part of the figures)

Block mound seawalls



$$\frac{q}{\sqrt{2g(H_0')^3}} = \text{function}\left(\theta, \frac{H_0'}{L_0}, \frac{h}{H_0'}, \frac{h_c}{H_0'}\right)$$

Use Figs. 5.6 and 5.7 in the text.

Note:

Given θ, H_0', L_0, h, h_c $\rightarrow$ q: large variation (cf. Table 5.2 in text)

But,

Given θ, H_0', L_0, h, q $\rightarrow$ h_c: small variation ($\pm 20\%$)

Figs. 5.3, 5.4, 5.6, and 5.7 give only order of magnitude of overtopping. Hydraulic model test is desirable for more accurate results.

5.2.3 Mean Rate of Wave Overtopping at Coastal Dikes of Plane Slope

$$\frac{q}{\sqrt{2g\left(H_0'\right)^3}} = \text{function}\left(\alpha_s, \theta, \frac{H_0'}{L_0}, \frac{h}{H_0'}, \frac{h_c}{H_0'}\right)$$

Use Figs. 5.8 to 5.10 in the text.

For $\tan\alpha_s = 1/3$ and 1/5, q (plane slope) > q (vertical wall)
For $\tan\alpha_s = 1/7$, q (plane slope) < q (vertical wall)
Lower wave run-up on a milder slope (See Eq. (5.3) and Fig. 5.2)

5.2.4 Unified Formulas for Wave Overtopping Rate of Vertical and Inclined Seawalls

For smooth vertical or inclined seawalls,

$$\frac{q}{\sqrt{g\left(H_{1/3}\right)_{\text{toe}}^3}} \equiv q^* = \exp\left\{-\left[A + B\frac{h_c}{(H_{1/3})_{\text{toe}}}\right]\right\}; \quad 0 \le h/(H_{1/3})_{\text{toe}} \le 6,\ 0 \le \cot\alpha_s \le 7 \qquad (5.9)$$

where $A, B =$ constants calculated by Eqs. (5.10) – (5.13).
For a vertical seawall, $\cot\alpha_s = 0$.

Accuracy of Eq. (5.9) is shown in Fig. 5.11 (vertical wall) and Fig. 5.12 (inclined wall).
Slight tendency of overestimation is observed in both cases ($q_{\text{meas}}/q_{\text{est}}<1.0$).
Less accurate for smaller overtopping rates (∵ Small overtopping occurs by few large waves when the crest height is high → Occurrence probability of large waves (and thus overtopping rate) is varied from test to test)

5.3 Influence of Various Factors on Wave Overtopping Rate

Read textbook.

5.4 Tolerable Rate of Wave Overtopping and Determination of Crest Elevation

5.4.1 Design Principles for Determination of Crest Elevation

Run-up based design:

- Set the crest level higher than run-up height so that no overtopping will occur
- No countermeasure for the case of overtopping
- Impossible to build a high seawall to allow no overtopping
- Possible damage in case of extraordinary storm

Overtopping-based design:

- Allow some extent of overtopping
- Consider drainage system and countermeasure for erosion against overtopping
- Tolerable limit of overtopping?

5.4.2 Tolerable Rate of Wave Overtopping

Viewpoint of structural safety:

- Erosion of soil should be considered
- Paved structure $\rightarrow$ higher tolerable limit of overtopping (See Table 5.3)

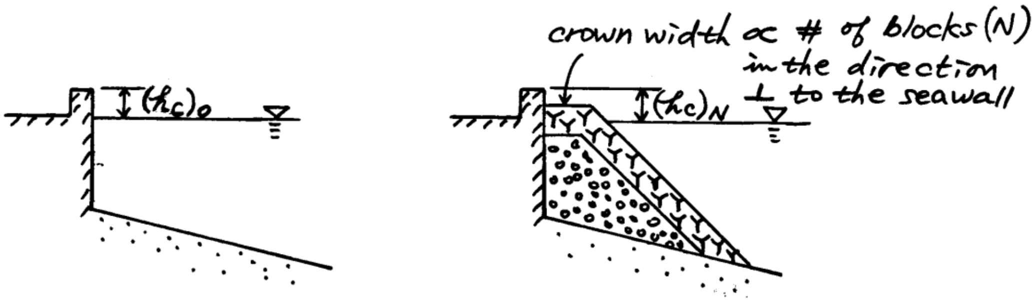
Viewpoint of utilization of land:

- Densely populated area: $q \leq 0.01\,\mathrm{m^3/m \cdot s}$
- Vehicles, pedestrians, and buildings: $q \leq 10^{-4} \sim 10^{-6}\ \mathrm{m^3/m \cdot s}$

5.4.3 Examples of Determining Crest Elevation of Seawalls

For given q, $\dfrac{h_c}{H_0'} = \text{function}\left(\dfrac{h}{H_0'}, \dfrac{H_0'}{L_0}, \theta, H_0'\right)$: Figs. 5.13 and 5.14

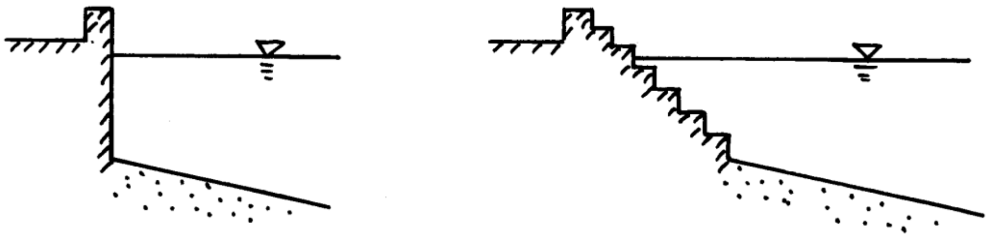
- In shallow water, $h_c \uparrow$ as $\theta \uparrow$, but, in deep water, the effect of bottom slope is negligible.
- $h_c \uparrow$ as $H_0'/L_0 \downarrow$, but, for the same H_0'/L_0, $h_c \uparrow$ as $H_0' \uparrow$ and $L_0 \uparrow$.
- Vertical revetment versus block mound seawall



In general, $(h_c)_0 > (h_c)_N$, but $(h_c)_N \uparrow$ as $H_0'/L_0 \downarrow$ (long period waves)

$(h_c)_N \downarrow$ as $N \uparrow$ (wide crown width)

- Vertical revetment versus sloping seawall



More overtopping than vertical revetment

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Need higher crest elevation

5.5 Additional Design Problems Related to Seawalls

Read textbook.