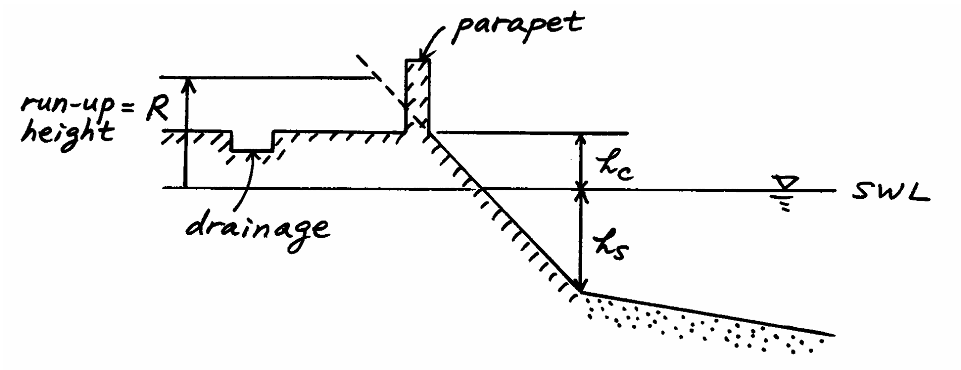
Chap 5. Design of Seawalls

5.1 Wave Overtopping Rate of Seawalls



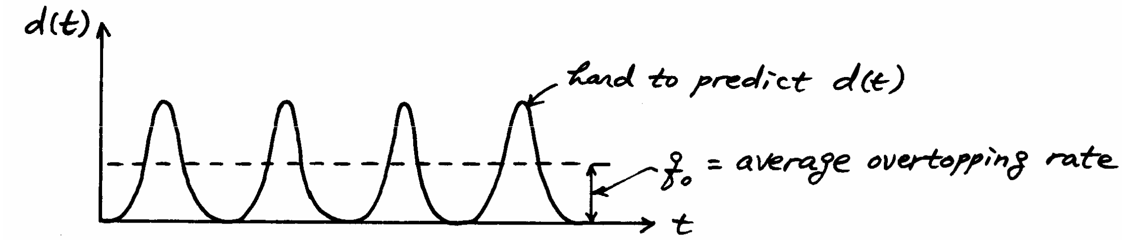
Overtopping occurs if $R > h_c$ (if no parapet)

Overtopping may cause flooding, erosion, wave transmission

↓ ↓ ↓

drainage protection mooring of ships

5.1.0 Overtopping Rate of Regular Waves



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

$q_0 = \frac{1}{T_m}\int_0^{T_m} d(t)dt$; 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 α^* = 1 ~ 2.

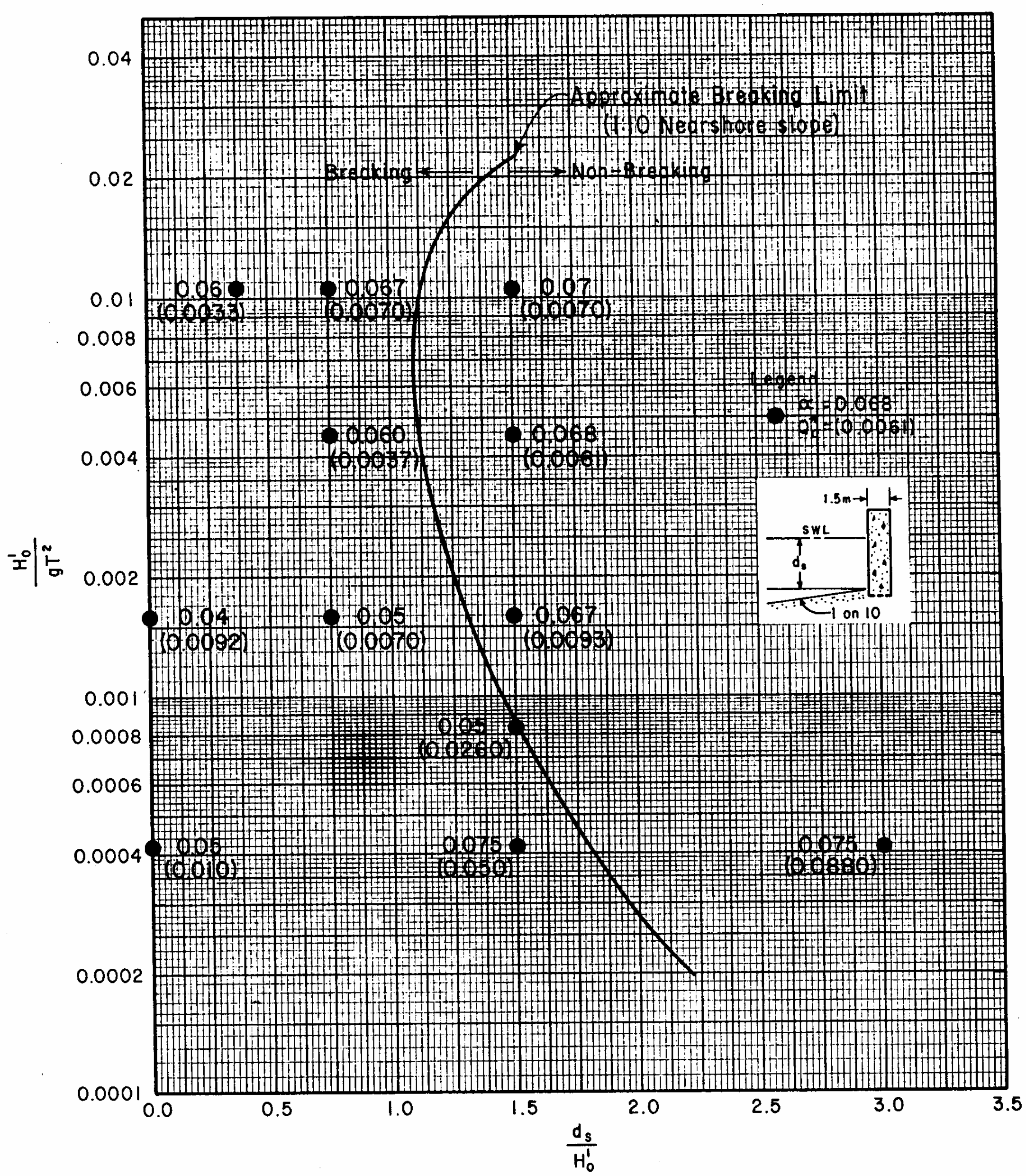


Figure 7-24. Overtopping parameters α and Q_o^* (smooth vertical wall on a 1:10 nearshore slope).

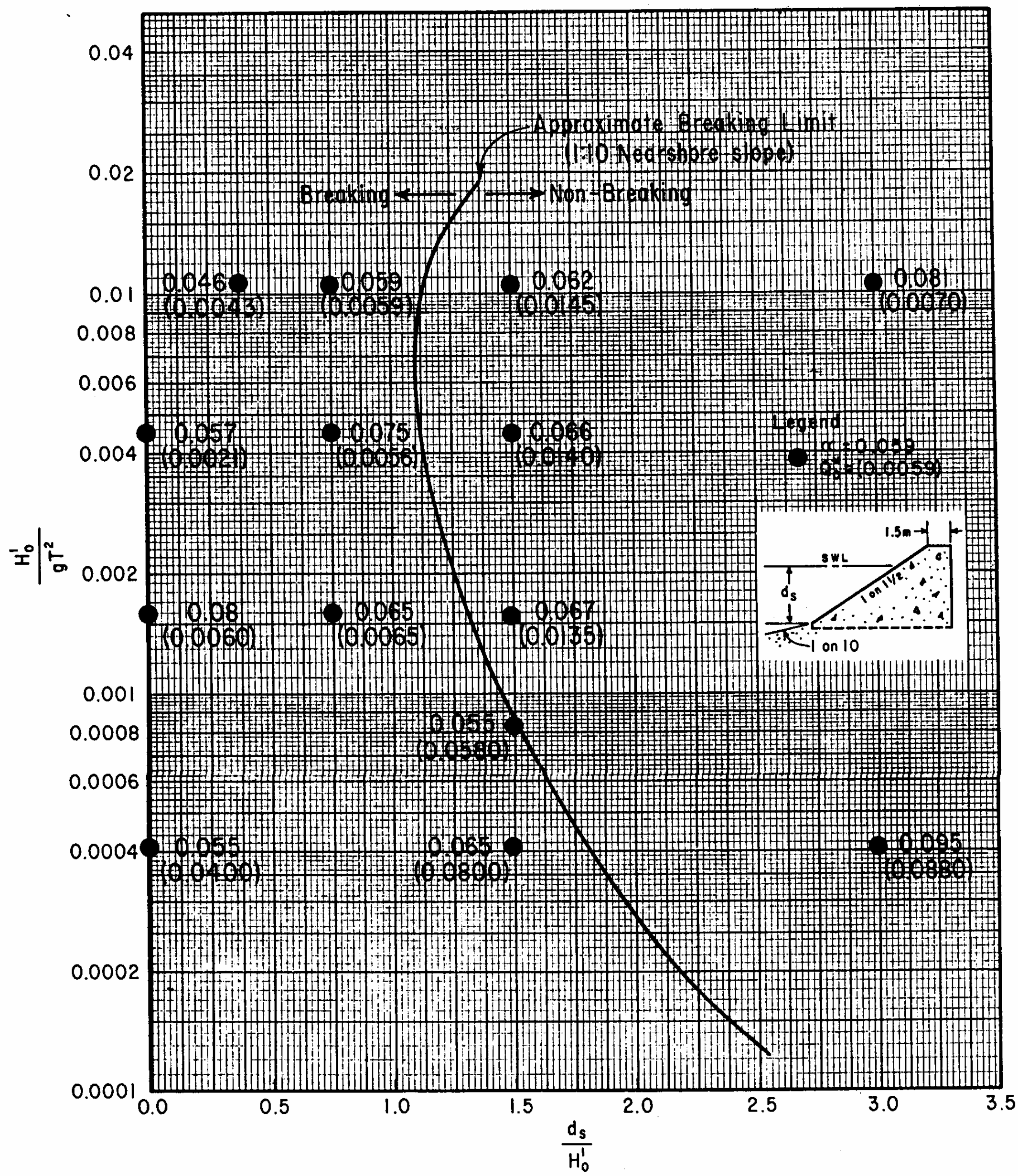


Figure 7-25. Overtopping parameters α and Q_o^* (smooth 1:1.5 structure slope on a 1:10 nearshore slope).

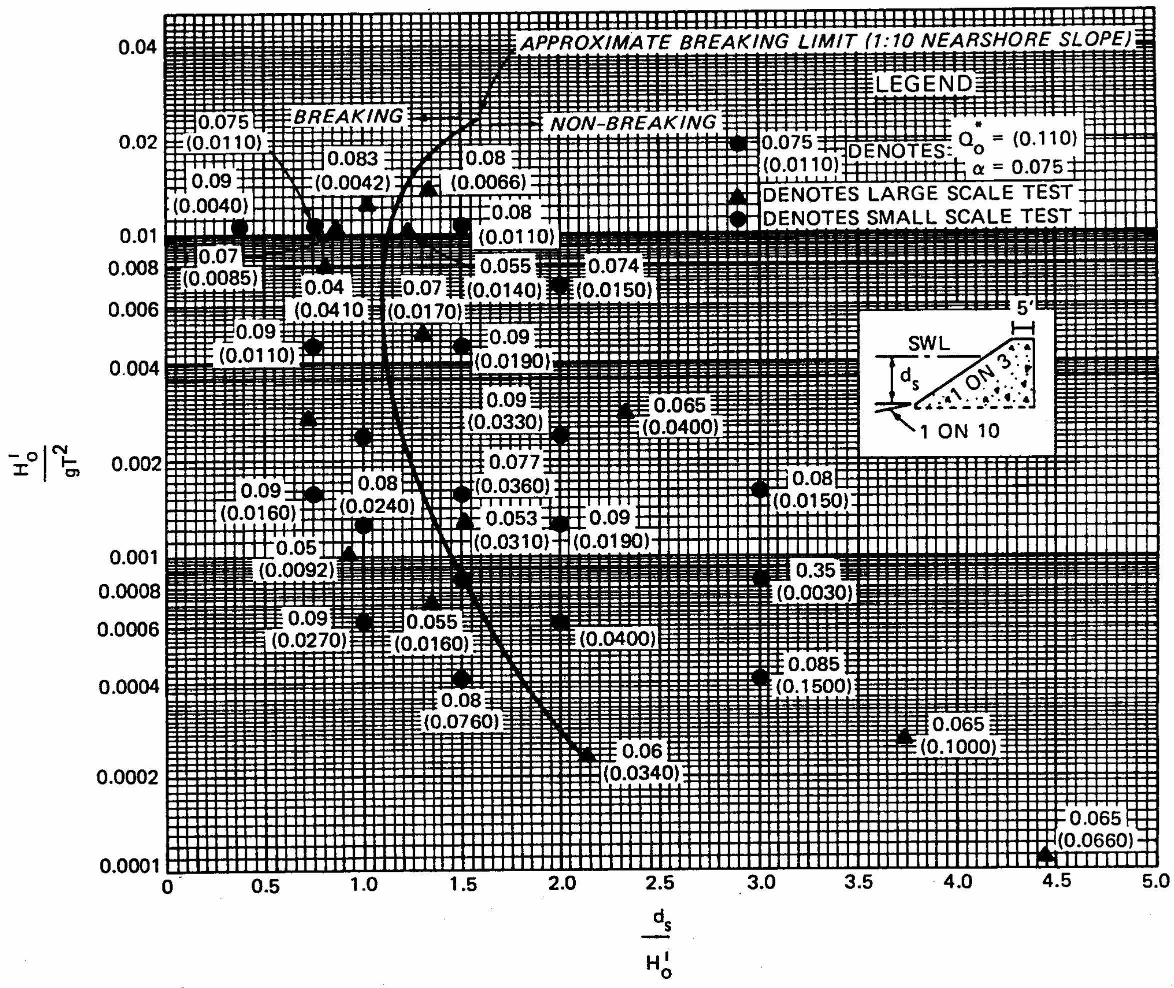


Figure 7-26. Overtopping parameters α and Q_o^* (smooth 1:3 structure slope on a 1:10 nearshore slope).

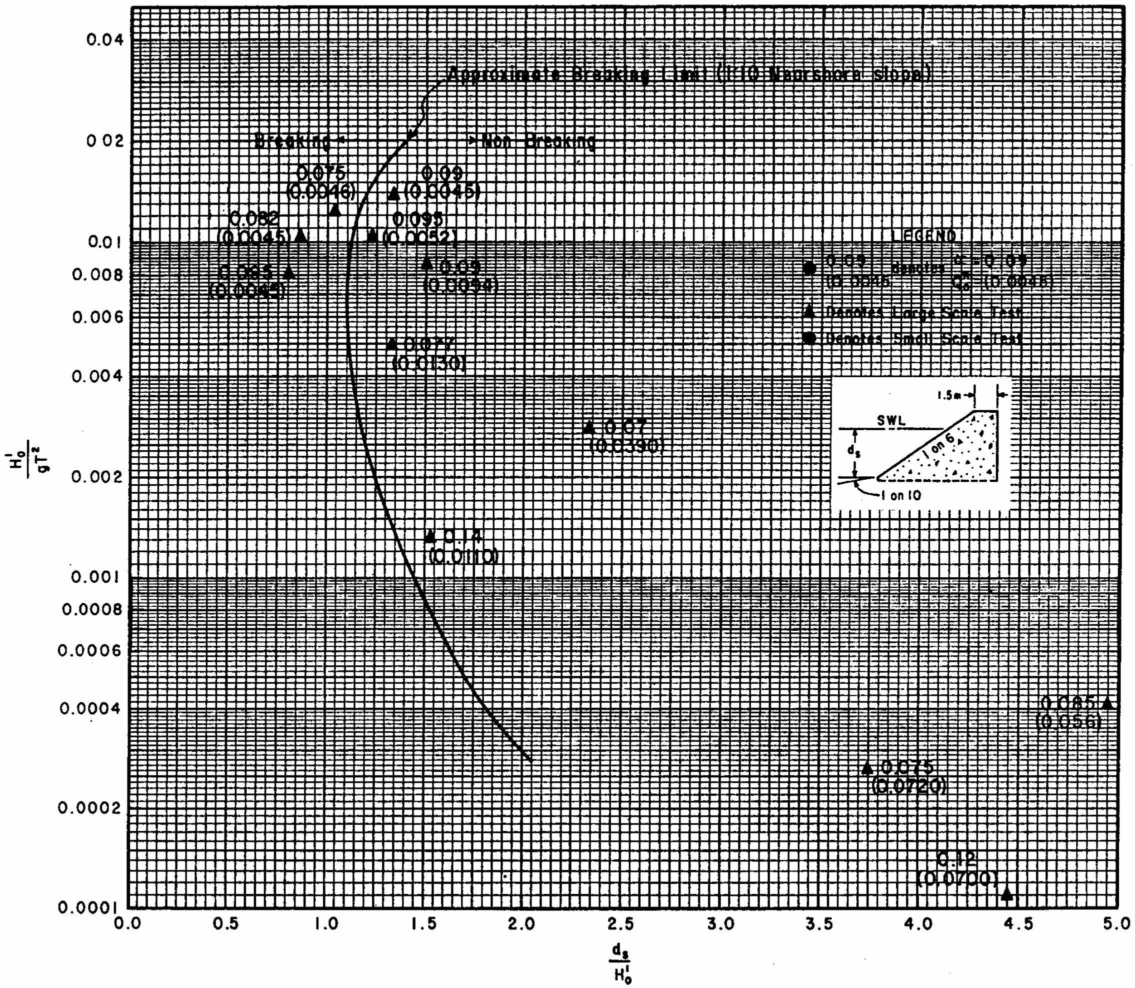


Figure 7-27. Overtopping parameters α and Q_0^* (smooth 1:6 structure slope on a 1:10 nearshore slope).

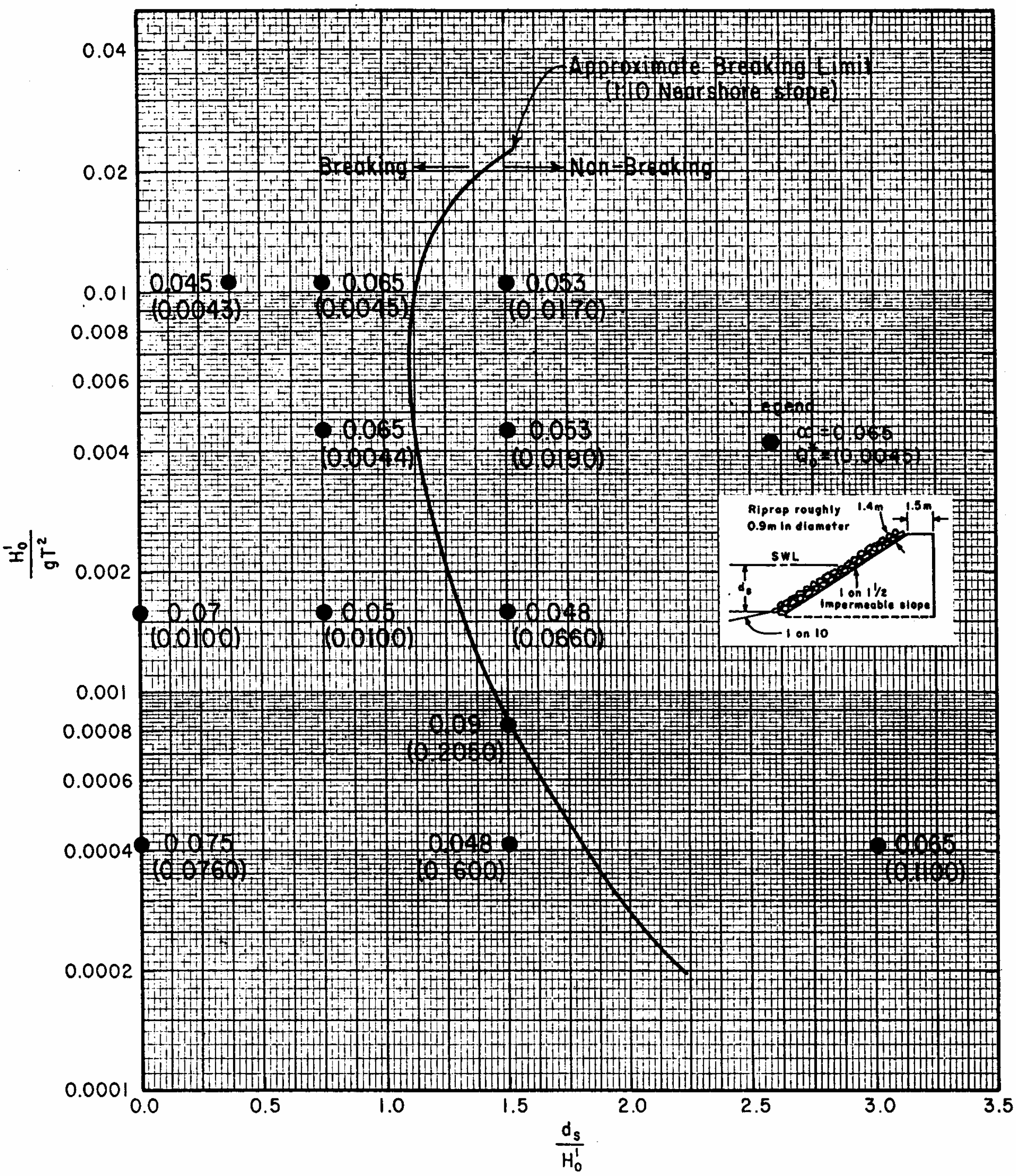


Figure 7-28. Overtopping parameters α and Q_o^* (riprapped 1:1.5 structure slope on a 1:10 nearshore slope).

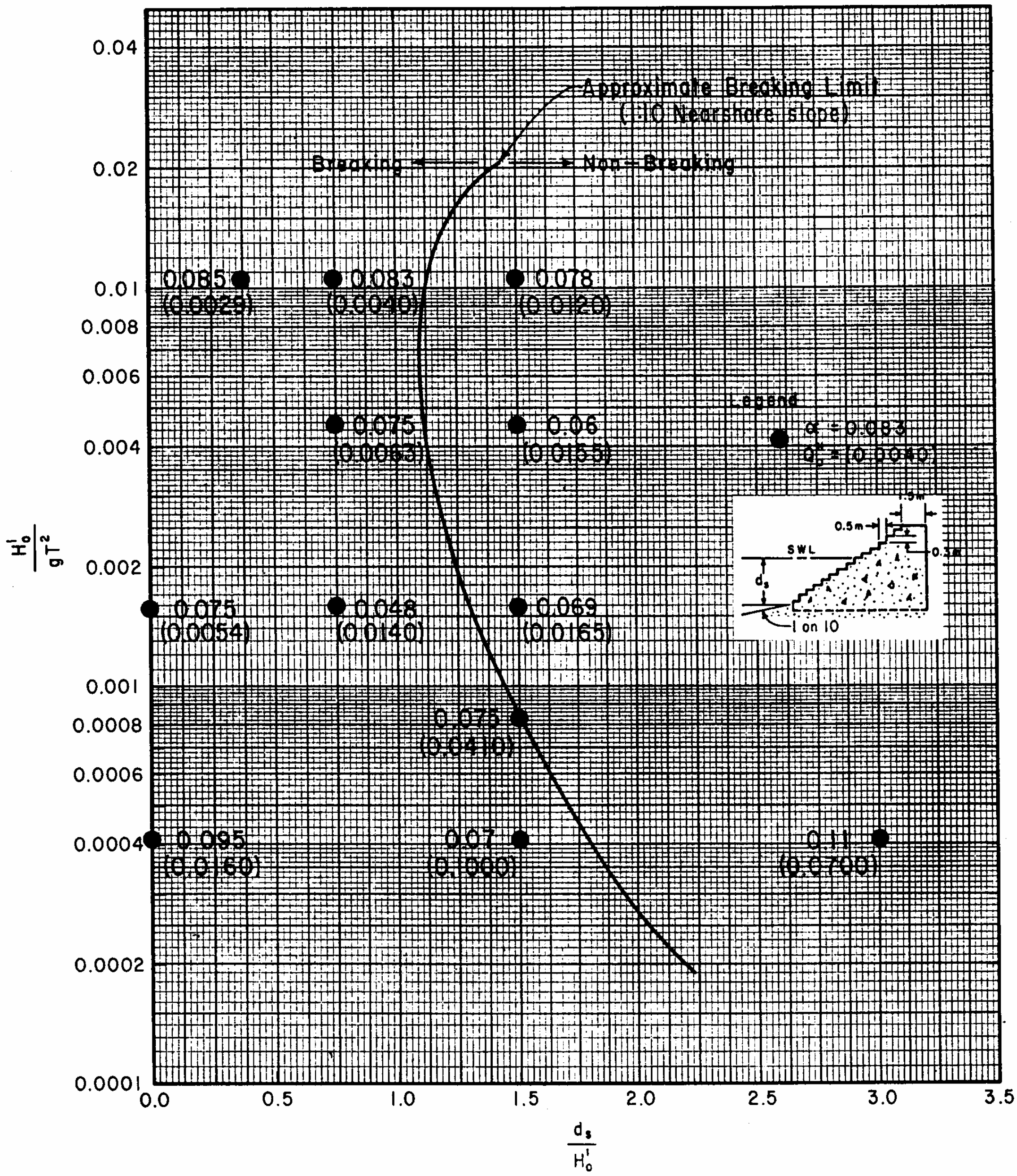


Figure 7-29. Overtopping parameters α and Q_o^* (stepped 1:1.5 structure slope on a 1:10 nearshore slope).

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.

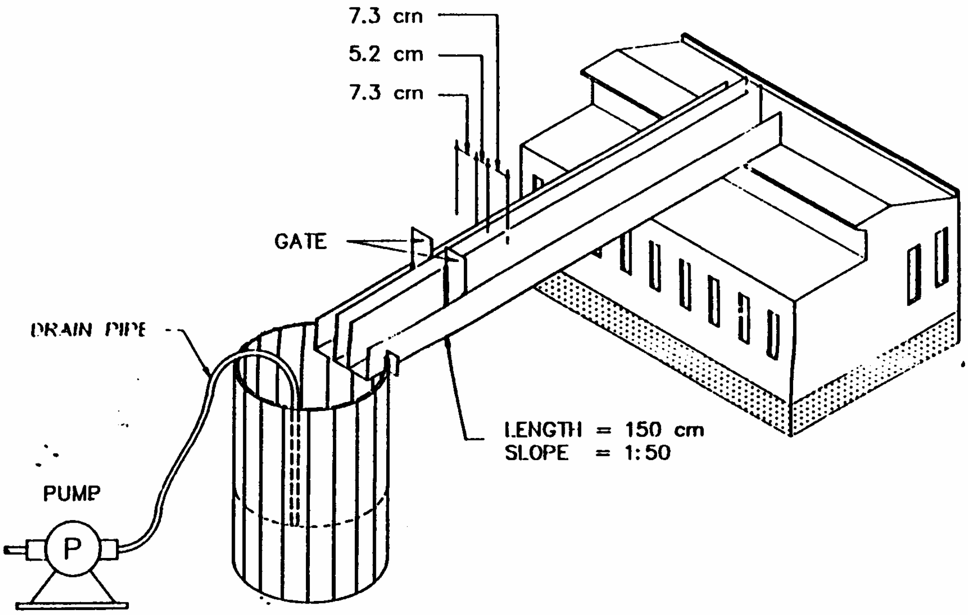


그림 5.7.1 월파량 측정장치.

5.1.1 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

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

where

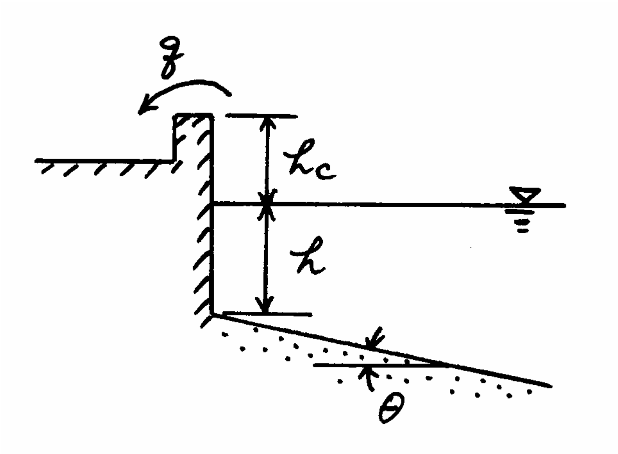
$q_0(H, T_{1/3})$ = overtopping rate by regular waves with height H and period $T_{1/3}$

$p(H)$ = 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.

5.1.2 Wave Overtopping Rate of Vertical Revetments and Block Mound Seawalls

<u>Vertical revetment</u>



Dimensional analysis indicates

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

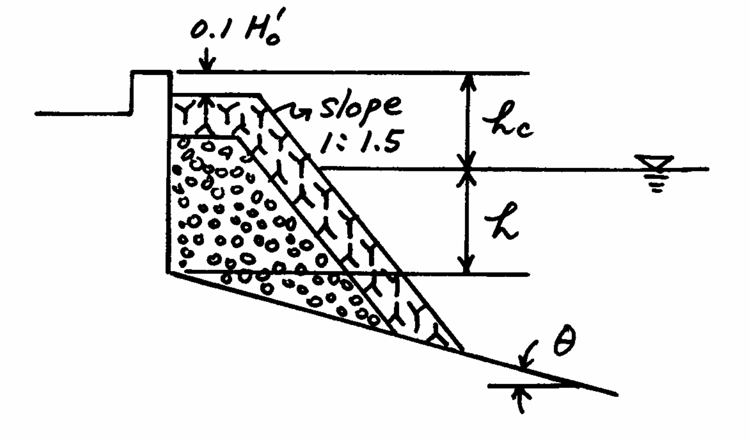
H_0' = equivalent deepwater wave height corresponding to $H_{1/3}$

Use Figs. 5.1 and 5.2 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. 5.4 and 5.5 in the text.

Note:

Given θ, H_0', L_0, h, h_c $\rightarrow$ q: large variation (cf. Table 5.1 in text)

But,

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

Figs. 5.1, 5.2, 5.4, and 5.5 give only order of magnitude of overtopping. Hydraulic model test is desirable for more accurate results.

5.2 Crest Elevation

5.2.1 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:

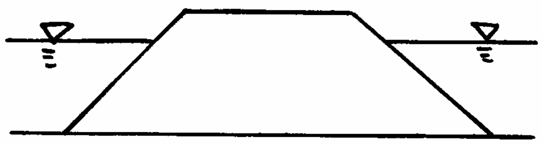
- Allow some extent of overtopping
- Consider drainage system and countermeasure for erosion against overtopping
- Tolerable limit of overtopping?

5.2.2 Tolerable Rate of Wave Overtopping

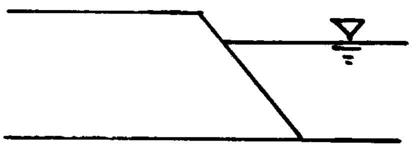
Viewpoint of structural safety:

- Erosion of soil should be considered
- Paved structure → higher tolerable limit of overtopping (See Table 5.2)

Coastal dyke



Revetment



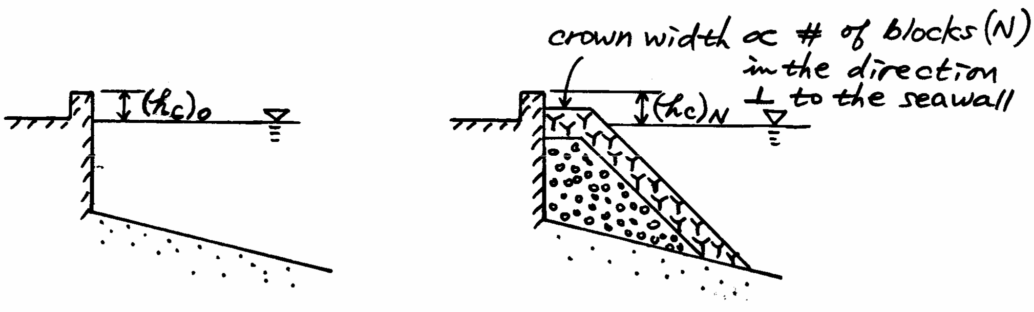
Viewpoint of utilization of land:

- Densely populated area: $q \le 0.01\,\mathrm{m^3/m\cdot s}$
- Coastal highway: $q \le 10^{-4}\,\mathrm{m^3/m\cdot s}$

5.2.3 Determination of Crest Elevation of a Seawall

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. 5.6 and 5.7

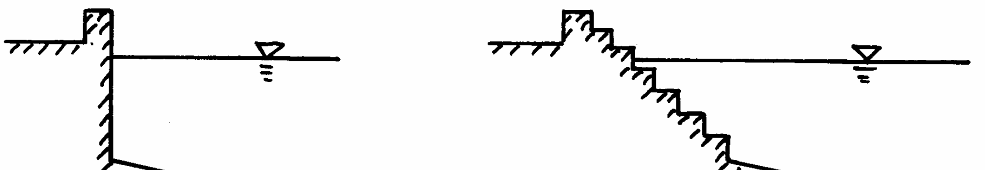
- In shallow water, $h_c \uparrow$ as $\theta \uparrow$, but, in deep water, the effect of bottom slope is negligible.
- $h_c \uparrow$ as $H_0'/L_0 \downarrow$, but, for the same H_0'/L_0, $h_c \uparrow$ as $H_0' \uparrow$ and $L_0 \uparrow$.
- Vertical revetment versus block mound seawall



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

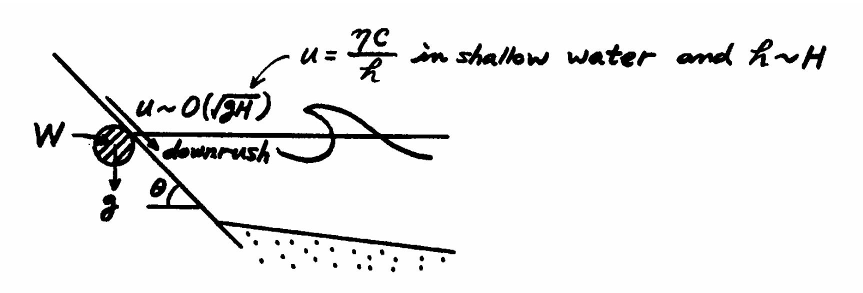


More overtopping than vertical revetment

$\downarrow$

Need higher crest elevation

5.3 Required Weight of Concrete Blocks



Let l = characteristic length of armor unit, so that

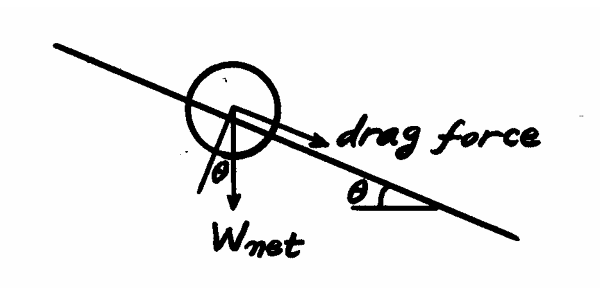
$$W \propto \rho_s g l^3 = \gamma_s l^3$$

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

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

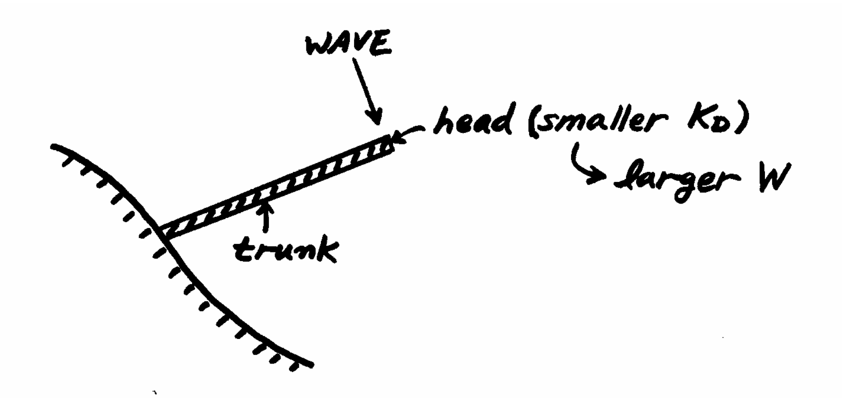
$$l \propto \frac{H}{(\gamma_s - \gamma_w)(\mu\cos\theta - \sin\theta)}$$

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

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

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

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 7.8, SPM).



$K_D \uparrow$ as number of layers $\uparrow$ and for special placement (vs random placement).

Table 7-8. Suggested K_D Values for use in determining armor unit weight[1].

No-Damage Criteria and Minor Overtopping							
Armor Units	n[3]	Placement	Structure Trunk K_D[2]		Structure Head K_D		Slope
			Breaking Wave	Nonbreaking Wave	Breaking Wave	Nonbreaking Wave	Cot θ
Quarrystone							
Smooth rounded	2	Random	*1.2*	2.4	*1.1*	1.9	1.5 to 3.0
Smooth rounded	>3	Random	*1.6*	*3.2*	*1.4*	*2.3*	[5]
Rough angular	1	Random [4]	[4]	*2.9*	[4]	*2.3*	[5]
Rough angular	2	Random	2.0	4.0	*1.9*	3.2	1.5
					1.6	2.8	2.0
					1.3	2.3	3.0
Rough angular	>3	Random	*2.2*	*4.5*	*2.1*	*4.2*	[5]
Rough angular	2	Special [6]	5.8	7.0	*5.3*	6.4	[5]
Parallelepiped[7]	2	Special [1]	7.0 - 20.0	*8.5 - 24.0*	--	--	
Tetrapod					*5.0*	6.0	1.5
and	2	Random	7.0	8.0	*4.5*	5.5	2.0
Quadripod					*3.5*	4.0	3.0
Tribar					*8.3*	9.0	1.5
	2	Random	*9.0*	10.0	*7.8*	8.5	2.0
					6.0	6.5	3.0
Dolos	2	Random	15.8[8]	31.8[8]	*8.0*	*16.0*	2.0[9]
					7.0	*14.0*	3.0
Modified cube	2	Random	*6.5*	7.5	-----	*5.0*	[5]
Hexapod	2	Random	*8.0*	9.5	*5.0*	*7.0*	[5]
Toskane	2	Random	*11.0*	22.0	--	--	[5]
Tribar	1	Uniform	12.0	15.0	*7.5*	*9.5*	[5]
Quarrystone (K_{RR})							
Graded angular	-	Random	2.2	2.5	--	--	

[1] CAUTION: Those K_D values shown in *italics* are unsupported by test results and are only provided for preliminary design purposes.

[2] Applicable to slopes ranging from 1 on 1.5 to 1 on 5.

[3] n is the number of units comprising the thickness of the armor layer.

[4] The use of single layer of quarrystone armor units is not recommended for structures subject to breaking waves, and only under special conditions for structures subject to nonbreaking waves. When it is used, the stone should be carefully placed.

[5] Until more information is available on the variation of K_D value with slope, the use of K_D should be limited to slopes ranging from 1 on 1.5 to 1 on 3. Some armor units tested on a structure head indicate a K_D-slope dependence.

[6] Special placement with long axis of stone placed perpendicular to structure face.

[7] Parallelepiped-shaped stone: long slab-like stone with the long dimension about 3 times the shortest dimension (Markle and Davidson, 1979).

[8] Refers to no-damage criteria (<5 percent displacement, rocking, etc.); if no rocking (<2 percent) is desired, reduce K_D 50 percent (Zwamborn and Van Niekerk, 1982).

[9] Stability of dolosse on slopes steeper than 1 on 2 should be substantiated by site-specific model tests.