Minikin Forces

Given: $H_i \doteq 20'$ $T = 7s$ $m = 0.05$ $d_s = 11'$

Vertical Wall Fig 7-4 need $\frac{d_s}{gT^2}$, m solve for $K_b = \frac{H_b}{d_s}$
solve for H_b (wave height @ breaking)

$\frac{H_b}{d_s} = 1.1 \rightarrow H_b = 12.1'$

Fig 7-100 (y-axis = non dimensional pressure) solve for p_m (Peak pressure if waves breaking on wall)

Minikin Force = $F_m = \frac{p_m (H_b)}{3} = R_{m_{vertical}}$

Minikin Moment = $M_m = F_m (d_s)$

Check to see if wave exists

If $H_b < H_{o\,max}$ then it can exist

* If given H_o must proove breaking exist using $H_i = K_s H_o$ (From Appx I)

Slope Wall Correction

Solve F_m as if vertical wall

$$F_{m_{sloped}} = F_m \left(\sin^2\theta\right)$$

$$R_m = F_{m_{sloped}} \left(\frac{1}{\cos\left[\frac{(90-\theta)\pi}{180}\right]}\right)$$

make sure in radians

Broken Waves

Plunging Distance = $x_p = (4 - 9.25(m))\,H_b$ (m = slope)

If slope is < 1%, $K_b = 0.78$

get K_b from Fig 7-4, then $H_b = K_b(d_b)$

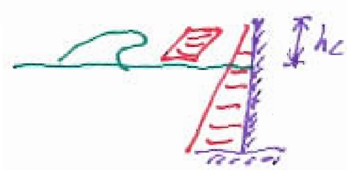
offshore break depth = $d_b = d_s + m\,x_p$ (d_s = inshore break depth)

$$H_b = \frac{d_s}{\frac{1}{K_b} - (4 - 9.25(m))m} \leftarrow \text{(For larger than 1\% slopes)}$$

$d_b = d_s + m\,x_b\,K_b$ Take this d_b & use as d_s in Fig 7-4 and reiterate once more
Find New K_b, H_b (but use original d_s & new K_b)
get new $d_b = d_s + m\,x_b\,K_b$

$$p_m = \frac{\gamma d_b}{2}$$

$$h_c = k_b H_b$$



Wave Force $R_m = p_m(h_c) = F_m$

$$R_s = \frac{1}{2}\gamma(d_s + h_c)^2$$

Total Force = $R_m + R_s$

Component of Wave Force on Sloped Wall

$$F_{m_{slope}} = F_m\left(\sin^2\left(\frac{\theta\pi}{180}\right)\right)$$

$$R_m = F_{m_{slope}}\left(\frac{1}{\cos\left[\frac{(90-\theta)\pi}{180}\right]}\right)$$

make sure in radians

NON-BREAKING Force

K_b (for $m < 1/8$) $= 0.78$ $H_b = 0.78(d_s)$

If $H_i < H_b$ wave should be non-breaking

Fig 7-90. solve for h_o
(calculates shifts "cup of tea" effect on non-break waves)

Deflection Coefficient ≈ 1 for smooth vertical walls

Crest elevation: $y_c = d_s + h_o + \frac{1+\chi}{2} + H_i$

Run up: $R_u = y_c - d_s$

~~Fig 7-90~~ Fig 7-91 solve for F_m via y-axis (just wave force)

Hydrostatic Force: $F = \frac{\gamma d^2}{2}$

Total Force: $F_T = F_m + F$

Fig 7-92 solve for M_m

$M_T = M_m + M$

I made this up

Hydrostatic moment M $= F d_s$
$= F d_s \left(\frac{1}{3}\right)$

use Fig

Forces on Piles

For rough Surface $C_D = 1.2 \to 1.6$ (use 1.6) (Drag Coeff.) (For smooth pile w/ high R_e (10^6) $C_D \simeq 0.7$)

First Calc wave Length $L_A = L_0 \tanh\left(\frac{2\pi d}{L_A}\right)$, $L_0 = \frac{g}{2\pi} T^2$

use $\frac{d}{L_0}$ in Appx I to get L_A, use $\frac{d}{L_A}$ in Appx I $\to K_s = \frac{H}{H_0}$ solve for H_i

Check if Slender pile: $\frac{D}{L_A} \leq 0.05$ (Diameter)

$$U_{max} = \frac{\pi H_i}{T}\left(\frac{L_0}{L_A}\right)$$

Renolds #: $R_e = \frac{U_{max} D}{\nu}$ (Diameter) (ν = Kinematic viscocity) — use $10^{-5} \frac{ft^2}{s}$

Use Fig 7-71, 7-72 to get K_{im}, K_{Dm} (when slope $\simeq \emptyset$ $H_b = 0.78(d)$) — need to use $\frac{H_i\,old}{H_b}$ for ratio H_b line.

$$F_{im} = \frac{1}{4} K_{im} \gamma_w C_m \pi D^2 H$$

$$F_{Dm} = \frac{1}{2} K_{Dm} \gamma_w C_D D H^2$$

$C_m = 2$ (if $R_e < 2.5 \times 10^5$)
$= 2.5 - \frac{R_e}{5(10^5)}$ (if $2.5\times10^5 < R_e < 5\times10^5$)
$= 1.5$ (if $R_e > 5\times10^5$)

Keulegan-Carpenter #: $N_{KC} = \frac{U_{max}(T)}{D}$

Strouhal #: $S_t = f_0 \left(\frac{D}{U}\right)$ (frequency) Fig 3.2 using R_e to get S_t then solve for f_0

$W = \frac{C_m D}{C_D H}$ use 7-78, 7-79, 7-81 to get ϕ_m (may need to interpolate between charts)

$$F_m = \phi_m \gamma_w C_D H^2 D$$

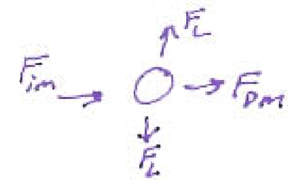
$F_L = \frac{1}{2} \gamma C_L H^2 D K_{Dm}$ (For large N_{KC}, $\frac{C_L}{C_D} \sim 0.7 \to 1$)

w/ Lift ~ Drag $F_{R_{L\text{-}D}} = \sqrt{F_m^2 + F_L^2}$

Force on Piles from Board Review

- Calc wave length
- ✓ if slender
- $U_{max} = \frac{\pi H}{T} \frac{L_0}{L_A}$
- $R_e = \frac{U_{max} D}{\nu}$ find C_D, C_m, S_t, f_0

K_{im} 7-71 K_{Dm} 7-72



$$R_{TD} = \phi C_D H_i^2 D \gamma$$

$$F_L = F_{DM} \frac{C_L}{C_D}$$

$\cot\theta = \frac{x}{y}$

ie slope = 1:2, $\cot\theta = \frac{2}{1} = 2$

$S_{s\,(concrete)} \cong 2.4$

$(rock) \cong 2.65$

$\frac{\rho_a}{\rho_w} \sim \frac{1}{800}$

Seawalls & Breakers

For $m \le 1\%$ $H_b = 0.78\, d_s$ if $> H_s \Rightarrow$ not breaking

calc $L_o = \frac{g T^2}{2\pi}$ → $\frac{d}{L_o}$ in Appx I → get L

Fig 7-13 solve for Run-up (need $\frac{H}{L}$ & embankment slope)

calc. $S_r = S_s / \left(1 + \frac{\text{Salinity (in ppt)}}{1000}\right)$

Get K_D from Table 7-8 (p.109 Lecture 9)

calc $W = \frac{\gamma_{r} H_s^3}{K_D (S_r - 1)^3 \cot\theta}$ (calc twice if Head & Trunk have different K_D values)

$\gamma_r = S_s(\gamma_w)$ (put $\gamma_w = 9.81 \frac{kN}{m^3}$ into → $9810 \frac{N}{m^3}$)

$\gamma_{r_{mass}} = \gamma_r / g$ (units should be $\frac{kg}{m^3}$ if SI)

Design Crest Elevation = SWL + Seasonal + Long term deviation + High Tide + Storm Surge + Barometric Setup + Wind Setup + Seiche + Wave Setup + Wave Run up + Settling/Subsidence

$$Z_{sup} = \frac{\text{Fetch}^{*}(C_f)(U_{ow}^{*})^2(\rho_a)}{2(g)(d)(\rho_w)}$$ (* match units)

↳ d: local depth + MSL + storm surge +

NOT ON CLASS LIST.

Name: Donald Serolleman Due Date January 21 2010

Assignment 2-1. a) Wave periods in Lake Pontchartrain vary from 3 to 6 seconds. Determine the wave lengths and celerities for these periods. Classify the waves as: Deep, Shallow or Transitional (Intermediate). Assume that the lake depth is 4 m.
b) If the 6 second wave has a height of 1.2 m, is the linear wave theory applicable? Based on linear wave theory, estimate the orbital velocity at the lake bed for the 6 second wave.
See Figure II-1-2 and II-1-22 in CEM or see Table in Figure 2.6 SPM.

$L_o = \frac{gT^2}{2\pi} = 46.2 m$

Ans: a)
T= 3 s;
d= 4 m 4.46
c = ~~5.80~~ m/s
L= 13.4 m
Classification: Trans.

T= 6 s;
d= 4 m
c = 5.80 m/s
L= 34.8 m
Classification: Trans.

Ans: b)
T= 6 s;
H = 1.2 m
Linear wave theory applies? No
U_b = =0.799 m/s

St or cn?

Attach your calculations.
$L = L_o \tanh(2\pi d/L)$
$c = L/T$

9/10

Name: Donald Jerolleman Due Date January 21 2010

Assignment 2-2. For a wave period in Lake Pontchartrain of 5 seconds and a wave height of 1 m, estimate the maximum and minimum pressure heads that an object on the bottom of the Lake will experience. Assume that the lake depth is 4 m and the salinity is negligible (~ 3ppt).

Ans: T= 5 s;
H = 1 m
d = 4 m
L= 27.9 m
p/γ max = 4.35 m
p/γ min = -4.35 m

Attach your calculations.

Max p/γ = + H cosh[2π(z + d)/L]/{2cosh(2πd/L)} – z
Min p/γ = - H cosh[2π(z + d)/L]/{2cosh(2πd/L)} - z

3/4

2-1) a) $T = 3s$ $d = 4m$, $L_o = \frac{gT^2}{2\pi} = \underline{14.05m}$,

$\frac{d}{L_o} = \underline{0.285}$, From Table $\frac{d}{L} = \underline{0.2987}$, $C = \frac{L}{T} = \boxed{4.46}$ m/s

↳ $L = \boxed{13.39}$ m

~~$\frac{d}{gT^2} = 0.0453$, $\frac{H}{gT^2} =$~~ $.04 < \frac{d}{L} < 0.5$ ∴ **TRASITONAL**

$T = 6s$ $d = 4m$ $L_o = \underline{56.2m}$, $\frac{d}{L_o} = \underline{0.0712}$,

From table $\frac{d}{L} = \underline{0.1149}$ → $L = \boxed{34.8m}$, $C = \frac{L}{T} = \boxed{5.80 \text{ m/s}}$

$0.04 < \frac{d}{L} < 0.5$ ∴ **TRASITONAL**

(b) $T = 6s$ $d = 4m$ $H = 1.2m$, $L_o = \frac{gT^2}{2\pi} = 56.2m$, $\frac{d}{L} = 0.1149$,

$\frac{d}{L_o} = 0.0712$ $L = 34.8m$ $C = \frac{L}{T} = 5.80$ m/s, Transitional

above given from (a) part II

$\frac{H}{gT^2} = \underline{0.0034}$, $\frac{d}{gT^2} = \underline{0.01133}$,

From Le Mehaute chart: **NON-LINEAR**, NON Breaking, Stokes 3rd order

$U_b = \left(\frac{H}{T}\right) \frac{\pi}{\sinh\left(\frac{2\pi d}{L}\right)}$, using table, $= (0.2) \frac{\pi}{0.7863} = \boxed{0.799 \text{ m/s}}$

record pause stop jump bookmark 0% jump to position 100% playback speed volume

ENCE 4723 S10 H.W. #1 p. 2 of 2 Donald Jerolleman

2-2) $T = 5s$, $H = 1m$, $d = 4m$, assume ~ Fresh water

$L_o = \frac{gT^2}{2\pi} = \underline{39.0}$, $\frac{d}{L_o} = \underline{0.1026}$, from table $\frac{d}{L} = \underline{0.1436}$

→ $L = \boxed{27.9\ m}$, from table $\cosh\left(\frac{2\pi d}{L}\right) = \underline{1.4354}$ $\times 2 = \underline{2.8708}$

from calculator $\cosh\left(\frac{2\pi(z+d)}{L}\right) =$ ~~1.7038~~ ~~1.559~~ $\underline{1}$

where $z =$ ~~[illegible]~~ $-d$

~~$Max\ p/\gamma = H\left(\frac{1.559}{2.8708-0.5}\right) = \boxed{0.6576\ m}$~~

~~$min\ p/\gamma = -H\left(\frac{1.559}{2.8708-0.5}\right) = \boxed{-0.6576\ m}$~~

$Max\ p/\gamma = \left(1\left(\left(\frac{1}{2.8708}\right)\right)^{+4}\right) =$ ~~0.422~~ $= +4.35$

$Min\ p/\gamma = \left(-1\left(\left(\frac{1}{2.8708}\right)\right)^{+4}\right) =$ ~~-0.422~~ $= -4.35$

record pause stop jump bookmark 0% jump to position 100% playback speed volume

ENCE 4723G

Name Donald Jerolleman 2330000

Assignment 3-1 **Due: February 4, 2010**

Graduate Student Problem: Compare the wave shape, c and L for linear and 3rd order theory for the following data: H = 1 m; T = 4.5 s; d = 4 m.

Linear

Lo = ...31.62 m...

L = ...24.39 m...

c = ...5.42 m/s...

Stokes 3 Approximate by USACE

L = ...

c = ...

Stokes 3 Exact Method Eqs 36-39

L = ...

c = ...

a = ...

a_c = ...

a_t = ...

Non-Linear by Figure 3-S-3 From R. Wiegel *Ocean Engineering*

L = ...

c = ...

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Assignment 3-1 **Due: February 4, 2010**

Undergraduate Student Problem: Compare the wave shape, c and L for linear and 3rd order theory for the following data: H = 1 m, T = 4.5 s, d = 4 m.

Linear

Lo = ..31.62 m..

L = ..24.39 m..

c = ..5.42 m/s..

Non-Linear by Figure 3-S-3 From R. Wiegel *Ocean Engineering*

L = ..25.8 m..

c = ..5.73 m/s..

9/8

ENCE 4723

Name Donald Ierollemann

Assignment 3-2 **Due: February 4, 2010**

Undergraduate & Graduate Student Problem: Compare the Linear and Cnoidal wave shape, L and c for the following data:
H = 2 m, d = 4 m and T = 10 s.

Linear

Lo = 154.1 m

L = 196.1 m ✗

c = 19.61 m/s ✗

6.5/8

Cnoidal Theory

L = 58.2 m

c = 5.82 m/s

a = 1 m

a_c = 1.68 m

a_t = 0.32 m

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Name Donald Serolleman

Assignment 3-3 **Due: January 28, 2010**

Undergraduate & Graduate Student Problem:

Given a deep water H_o = 1.5 m and T = 4.5 s approaching a beach with a 2% slope. Estimate H_b and d_b. What type of breaker is expected?

Answers:

H_b =........1.56 m.....

d_b. =........1.934 m

Type of breaker is ....Spilling....

late

4 -
4

3-1) $H = 1m$, $T = 4.5s$, $d = 4m$

Linear $L_0 = \frac{gT^2}{2\pi} = \boxed{31.62}$ $\frac{d}{L_0} = 0.1265 \approx 0.1270$

from table $\frac{d}{L} = 0.1640$ $L = \frac{d}{\frac{d}{L}} = \frac{4}{0.1640} = \boxed{24.39m}$

$c = \frac{L}{T} = \frac{24.39m}{4.5s} = \boxed{5.42\ m/s}$

$\frac{2\pi d}{gT^2} = 0.1265$ $\frac{H}{d} = \frac{1}{4} = 0.25$ by Fig 3.5-3 $\frac{d}{L} = 0.155$

$L = \boxed{25.8}$ $c = \frac{L}{T} = \boxed{5.73\ m/s}$

3-2) $H = 2m$ $d = 4m$ $T = 10s$ $L_0 = \boxed{154.1}$ $\frac{d}{L_0} = 0.0026$ From table $\frac{d}{L} = 0.0204$

$L = \boxed{196.1m}$ $c = \boxed{19.61\ m/s}$

$L = \frac{16d^3}{3H} k K(k)$ $\frac{H}{d} = \frac{2}{4} = \frac{1}{2} = 0.5$

$T\sqrt{\frac{g}{d}} = 15.66$ by Fig 2-11 $k^2 = 1 - 10^{-4}$ $= 0.9999$

(red: -3)

From Fig 2-12 $\frac{L^2 H}{d^3} = 106$ $\rightarrow L = \boxed{58.2}$ ~~351.125~~

$c = \frac{L}{T} = \boxed{5.82\ m/s}$ $a = \frac{H}{2} = 1$

$\frac{y_s - y_t}{H} = 0.21$

$a_t = (0.16)(2) = \boxed{0.32}$ $a_c = H - a_t = \boxed{1.68}$

Name: Donald Jercolleman Due Date: February 4 2010

Assignment 4-1

Find the wave height, wave length and angle at $d_2 = 3\ m$ for the following transitional water wave: $H_1 = 1.5\ m$; $d_1 = 6\ m$; $T = 4.5\ s$ and $\alpha_o = 45^o$. Use Figure C-6

Answers:

H_o = 1.637 m; ? too low

L_o = 31.6 m; ✓

$K_s K_r$ = 0.85 ✓

α_2 = 28 ✓

$H_2 = K_s K_r H_o$ = 1.391 m; ✗

5/7

Assuming that the bed slope is nearly zero, will the wave break before it reaches point 2?

No ✓

4-1 $H = 1.5m$ $d_1 = 6m$ $T = 4.5s$ $\alpha_0 = 45°$ $d_2 = 3m$

$H_1 = K_S H_0$ $L_0 = \frac{gT^2}{2\pi} = 31.6\ m$

$\rightarrow \frac{d}{L_0} = 0.1898 \xrightarrow{\text{Table}} K_S = \frac{H}{H_0} = 0.9161 \rightarrow H_0 = 1.637$ ×

$\frac{d_2}{gT^2} = ~~0.030~~\ 0.0151 \xrightarrow{\text{chart}} \alpha = ~~37.5~~\ 28$ $K_S K_R = ~~0.87~~\ 0.85$

$H_2 = K_S K_R H_0 = ~~1.424~~\ 1.391$

$\frac{H_2}{d_2} = \frac{1.391}{3} = 0.463 < 0.78$ ∴ not breaking

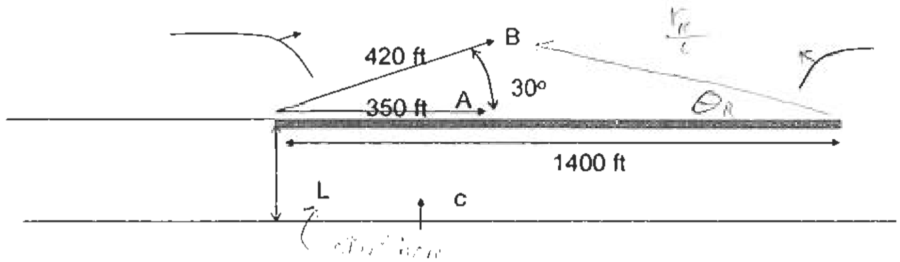
✓

1609.344
3600
0.44704

9 9

Name: Donald Jerolleman Due Date: February 11 2010

Assignment 4-2 Estimate the maximum wave heights and points A and B in the region behind the breakwater in the attached plan. Given: Hi = 5 ft; T = 6s; d =22 ft.



Answers

Maximum H at point A: 1.155

Maximum H at point B: 1.175

6
6

Assignment Problem 4.3: Estimate the reflection coefficient for the stepped seawall along the southshore of Lake Pontchartrain. See details of the seawall on Figure 4.6 (2:1 slope). Assume: Ho ~ 1.0 m and T ~ 5 s. Compare this K_f to a beach with a 5% slope.

Answer:

Seawall K_f = 0.6

Beach K_f = 0.00441

4
4

Assignment Problem 5.1: Find the first and second natural periods for Lake Pontchartrain. Assume a mean depth of 3.6 m and a length of 41 miles and a width of 25 miles.

Answer:

N-S T1 = ~~4.94~~ 3.76 **hours**

E-W T1 = ~~13.79~~ 6.17 **hours**

N-S T2 = ~~2.47~~ 1.88 **hours**

E-W T2 = ~~6.9~~ 3.08 **hours**

4/4

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4-2) $H_1 = 5 \quad T = 6 \quad d = 22 \qquad L_0 = \frac{gT^2}{2\pi} = 184.5$

$\frac{d}{L_0} = 0.1192 \qquad \frac{d}{L} = 0.1573 \rightarrow L = 139.86'$

$r_{LA} = 350' \qquad r_{LA}/L = \frac{350}{139.86} = 2.50 \quad \theta = \emptyset$

From fig 2-33 $\quad K' = k_d = 0.15 \qquad r_{RA} = 1050$

$r_{rA}/L = \frac{1050}{139.86} = 7.50 \qquad \theta = 30$ ✓ $\qquad K' = K_d = 0.087$

$k = K_R + k_L = 0.237 \qquad H = 5(0.237) = 1.185$

$r_{LB} = 420 \qquad r_{LA}/L = \frac{420}{139.86} = 3.003 \qquad K' = 0.14$

$r_{RB} = r_{LB}/L = \frac{1036.76}{139.86} = 7.25 \qquad K' = 0.085$

$H = (5)K_{Tot} = 5(0.14 + 0.085) = 1.175$

angle? for B - Right

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4.3) $H_0 = 1m$ $T = 5s$ $\chi_1 \approx 0.6$

$K_f = \frac{H_r}{H_i} = \chi_1 \chi_2$ $L_0 = \frac{gT^2}{2\pi} = \frac{9.81\ 5^2}{2\pi} = 39.0m$

$\tan^{-1}\left(\frac{1}{2}\right) = 0.46\ rad = \beta$

using $\sqrt{\frac{2\beta}{\pi}}\ \frac{\sin^2\beta}{\pi} = 0.0339 = \left(\frac{H_0}{L_0}\right)_{max}$

$\frac{H_0}{L_0} = \frac{1}{39} = 0.0256 < 0.0339$

$\therefore\ \chi_2 = 1$

$\boxed{K_f = 0.6}$

Beach $H_0 = 1$ $L_0 = 39m$ $\chi_1 = 0.8$

$\tan^{-1}(0.05) = 0.0499 = \beta$

using formula $\frac{H_0}{L_0}max = 0.0001412$

$\frac{H_0}{L_0} = \frac{1}{39} = 0.0256 > 0.0001412$

$\therefore\ \chi_2 = \frac{0.0001412}{0.0256} = 0.00551$

$K_f = 0.00551(0.8) = \boxed{0.00441}$

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5.1

$T_n = 2 L_B / (n(gd)^{1/2})$

T_1 (w/ L_B = 25) = ~~4.94~~ → (13.76)

T_1 (w/ L_B = 41) = ~~13.79~~ → " 6.17

T_2 (w/ L_B = 25) = ~~2.47~~ → " ~~1.88~~ ✓ 1.88

T_2 (w/ L_B = 41) = ~~6.9~~ → " 3.08

25 mi (1609.344 m/mi) = 40233.6 m
41 mi (") = 65983.104 m

@ n_1 $\sqrt{9.81 \frac{m}{s^2} (3600^2)(3.6)} = 21394$ @ n_2 $\times 2 = 42787.6$

7 p.

First Mid-term Test
Fall 2010
b
ENCE 4723
Coastal Engineering

Duration 2 hours

Open-book including calculator, notes and texts.

Attempt all questions

Give your answers on the sheets provided.

Question # 1	9	/10
Question # 2	9	/10
Question # 3	8 ½	/10
Question # 4	10	/10
TOTAL	36 ½	**/40**

Name: Donald Jerolleman PLEASE PRINT!
Last Name

Student No.

2330000

$H=2 \quad T=5.5s \quad d=5 \quad L=? \quad C=? \quad H_o=?$

$C_g=?$

Type - ?

1. Given: a 2-m high wave with a period of 5.5 seconds in a water depth of 5-m.
Circle the closest answer:
Determine:

a) The wave length based on linear wave theory is: [155, 50, 47, 34, 25] m

b) The wave celerity is: [28, 8.6, 6.2, 5.1, ≥30, ≤ 4.5] units ____

c) The group velocity is: [9.1, 7.3, 6.0, 5, ≥10, ≤ 4.5] units______

9

d) The equivalent deep water wave height is:
[2, 2.15, 1.85, 2.10, ≥2.2, ≤ 1.8] units ____

e) Classify the wave. Stokes 3rd order Transitional

Show your calculations here!

$$L_o = \frac{g(5.5)^2}{2\pi} = 47.23m \quad \frac{d}{L_o} = 0.1059 \text{ From Appx I } \frac{d}{L} = 0.1462$$

$$\rightarrow L = 34.2 m \qquad C = \frac{L}{T} = 6.22$$

$$C_g = Cn = 6.22 \left\{ \frac{1 + \left(\frac{4\pi(5)}{34.2}\right)}{\sinh\left(\frac{4\pi d}{L}\right)} \right\}^{1/2} = 6.22\left(\frac{2.837}{3.059}\right)^{1/2} = 5.99$$

$\uparrow 3.059$

K_s

$0.94 \quad \frac{2}{0.94} = 2.12$

$$\frac{d}{gT^2} = 0.0168$$

$$\frac{2}{gT^2} = 0.00674$$

$H = 1.4 m \quad T = 5 s \quad \alpha_0 = 45^\circ$

2. Given: a 1.4-m wave with a period of 5 seconds in deep water with an angle of 45° between the wave crest and the shoreline.

Circle the closest answer:

Determine:

a) The wave height at d = 2 m: [1.43, 1.37, 1.58, 1.25, ≤1.22, ≥1.6] m

b) The wave length at d= 2 m: [39, 34, 21, ≤19, ≥40] m

c) The angle at d = 2 m: [45, 43, 33, 23, ≤20, ≥46] degrees

d) Will the wave break at or before this location? [Yes, No]

Show your calculations here!

9

$L_0 = 39.03 \quad \frac{d}{L_0} = 0.0512$ From Appx

$\frac{d}{L} = 0.09520$ ~~$\frac{H}{H_0} = 1.019$~~ ~~$\frac{H}{H_0} = 1.96$~~ $L = 21.01$

~~$K_s = 0.9444 \quad H = H_0 K_s \rightarrow$~~

$\frac{d}{gT^2} = 0.00815$ using C-6 $K_r K_s = 0.868$

$H = H_0 K_r K_s = 1.215$

$\frac{H}{d} = \frac{1.215}{2} = 0.6075 < 0.78$

$H_i = 2 \quad T = 6 \qquad d = 11m$

3. Given: A 2-m deep water wave with a period of 6 seconds that encounters a 500 m long rock rubble breakwater in a water depth of 11 m. The angle between the wave crest and the breakwater is 0° as shown the sketch below.

Circle the closest answer:

Estimate:

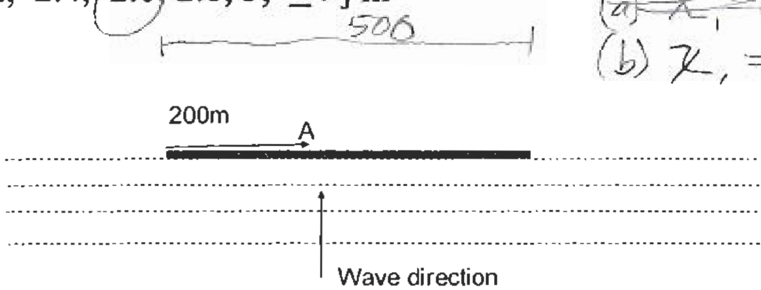
a) The maximum wave height at point A: 4

[0.45, 0.43, 0.40, 0.31, ≤0.25, ≥0.5] m

b) If the incident wave is partially reflected at the breakwater (slope 1H:1V), estimate the maximum wave height on the seaward side of the breakwater. Assume the breakwater is a rubble mound type (rough porous): 4

[≤2, 2.2, 2.4, 2.6, 2.8, 3, ≥4] m

(b) $\chi_1 = 0.3$



Show your calculations here!

$L_o = 56.21 \quad \frac{d}{L_o} = 0.1957 \rightarrow \frac{d}{L} = 0.2218$

$\rightarrow L = 49.59m$

$r_{LA} = 200m \quad \frac{r_{LA}}{L} = 4.03 \quad @\theta = \emptyset$

$K_L = 0.118$

$H_i = H_o K_s$

$r_{RA} = 300m \quad \frac{r_{RA}}{L} = 6.05 \quad @\theta = \emptyset \quad K_R = 0.093$

$K = K_R + K_L = 0.211 \qquad \beta = 45^o \cdot \left(\frac{\pi}{180}\right) = 0.78 \text{ rad}$

$H = H_i K = 0.422 \, m \qquad \left(\frac{H_o}{L_o}\right)_{max} = 0.1109$

$\frac{H_o}{L_o} = \frac{2}{56.21} = 0.0355 < max \quad \therefore \chi_2 = 1$

$\therefore K_r = 0.3 \qquad H_r = H_i K_r = 0.6 \, m + H_i \quad 2.6m$ (for max)

T= (1:20)

4. Given: Ho'= 8.6-m with a period of 25 s. There is a shelf that has a 5% slope.

Circle the closest answer:

Estimate:

a) the breaking wave height: [≤12, 14.5, 16.5, 18, ≥20] m

b) the breaking depth: [≤9, 9.5, 11, 13.5, 14.6, ≥15] m

c) the Breaker type Plunging.

Show your calculations here!

$$\frac{H_o}{L_o} = \frac{H_o}{5.12\,T^2} = \frac{8.6\,m\left(\frac{1\,ft}{0.3048\,m}\right)}{5.12\,(25^2\,s^2)} = 0.0088$$

10

$$\frac{H_b}{H_o'} = 1.7 \qquad H_b = 14.62$$

$$\frac{H_b}{gT^2} = 0.00236 \qquad \frac{d_b}{H_b} = 0.94$$

$$d_b = 13.74$$

Name Nira McDaniel **[illegible]ry 18 2010**

Assignment 6.1: Estimate the [illegible] height and period for TS Isidore. Assume NNE winds at 35 kn[illegible]. Estimate the wind setup for this storm.

Hs = 3.97 ft

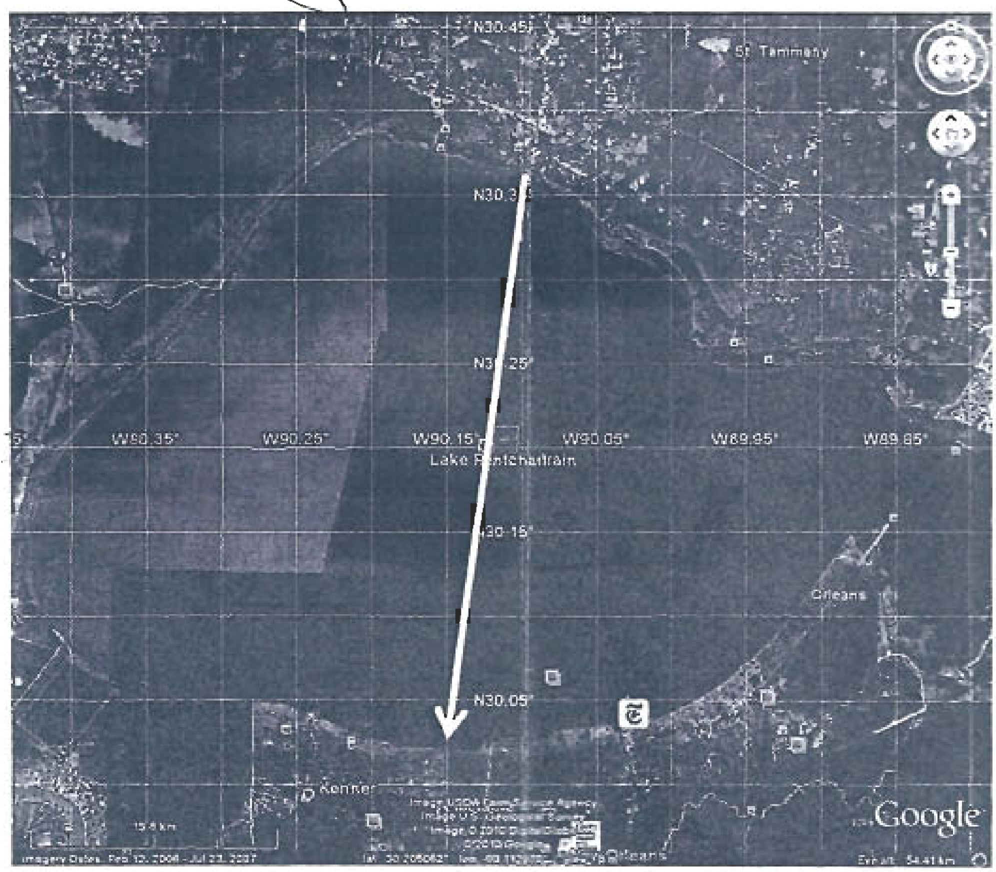
Ts = 4.39 s

9/11

Are the waves: (FETCH LIMITED) or (DURATION LIMITED) ?

Wind setup = ~~56.99.16m~~ Using Equation 6.14

Wind setup = .05 ft Using Equation 6.15



Nina McDaniel

given; winds = 35 knots = 40.3 mph = 58.96 ft/s
duration = 2h = 7200 s = Fa

depth = 14 ft   25 mi = 132000 ft
air temp = 28°C } assuming same temp used in class
T_{sw} = 33°C

U_a ft/s = .539 (speed · $R_L R_T$)$^{1.23}$
= .539 (59 ft/s · 1.02)$^{1.23}$ = 83.2 ft/s

$$\frac{gF_a}{(U_a)^2} = \frac{32.2(132000)}{(83.2)^2} = 614.02$$

$$\frac{gd}{(U_a)^2} = \frac{32.2(14\ ft)}{(83.2)^2} = 0.065$$

graph 3-21 $gH_o/U_a^2 = .018$

$$H_o = \frac{.018(83.2)^2}{32.2} = 3.87\ ft$$

fig 3-22
$$\frac{gT_D}{U_a} = 1.7$$

T_D = 1.7(83.2)/32.2 = 4.392

By: Date: Checked By: Date: Page:
Subject: 2.2 Job No:
Project Title:
EUSTIS
Metairie, Louisiana
Lafayette, Louisiana
Gulfport, Mississippi

$\frac{\rho_a}{\rho_w} \approx \frac{1}{800}$

eqn 6.14

figure 6-8

CD = .0018

$$Z_{avg} = \frac{F\left(\rho_a \; C_D \; U_{avg}^2\right)}{2\,\rho_w\, g\, d}$$

?

$C_f \approx 0.002$

convert to meters

$$\frac{(40233.6\,m)(1.01325\,m/s)\,(.0018)(25.35\,m/s)^2}{2\,(9.8066\,m/s^2)(1.1?)(4.3\,m)} = \frac{m^3/s^2}{m^2/s^2}$$

9.81

$= 5699.6\,m$

$$Z_{avg} = \frac{F V^2}{1400\,d} \qquad \frac{(25\ldots)(40.2\ldots)^2}{1400\,(14\,ft)}$$

$= .051\,ft \times 40.2 \approx 2\,ft$

Name Donald Jerolleman **Due March 4 2010**

Please PRINT

Assignment 6-2

Given a hurricane with the characteristics in the attached table:
R = 25 miles; Forward Speed = 17 mph; $\Delta p \simeq 74.17$ mm Hg with normal pressure at 760 mm. Assume Latitude of 28°. What is the Maximum Wind Speed? Plot the pressure and velocity on the right side of the storm. What are the maximum significant wave height and the corresponding period? Estimate the storm surge.

Data

Δp	74.17	mm Hg	2.92	in Hg
phi	28	degrees latitude		
R	21.7	Kt mi	25	mi
VF	14.8	kt	17	mph
f	0.2450	rad/h		
pn.	760	mm Hg	29.92	in Hg
Umax	105.6 ~~121.4~~	Kt	121.4	mph
UR	98.76	Kt	113.7	mph
α	1.2			
Ho	46.8	ft		
Ts	14.00	sec		

Estimate of Storm Surge (Simple Case)

d ref	35	ft

hss = 3.40 ft

6/6

Donald Jerolleman Assignment | 6-2

$R = 25$ miles $= 21.72$ Nmiles $\Delta p = 74.17$ mm Hg $= 2.92$ " Hg $p_n = 760$ mm Hg $= 29.92$ " Hg

Lat = 28°

Forward Velocity $= V_F = 17$ mph $= 14.8$ Knots

$U_{max} = a\{b\,\Delta p^{1/2} - cRf\}$, $f = 2\omega \sin\phi = 2\left(\frac{2\pi}{24}\right)\sin(28°) = \boxed{0.2458}$

$a = 0.868$ Knots $b = 73$ mi $c = 0.57$ " Hg

$U_{max} = \boxed{105.6 \text{ Knots}} = \boxed{121.4 \text{ mph}}$

$U_R = 0.865(U_{max}) + 0.5(V_F) = \boxed{98.76 \text{ knots}} = \boxed{113.7 \text{ mph}}$

$T_s = A\left(1 + \frac{B\alpha V_F}{U_R^{1/2}}\right)e^{\frac{R\Delta p}{C}}$, $A = 8.6$ $B = 0.104$ $C = 200$ $\alpha = 1.2$

$\boxed{T_s = 14.0 \text{ s}}$

$H_{os} = A_0\left(1 + \frac{B_0 \alpha V_F}{U_R^{1/2}}\right)e^{\frac{R\Delta p}{C_0}}$, $A_0 = 16.5$ $B_0 = 0.208$ $C_0 = 100$ $\alpha = 1.2$

$H_{os} = 46.6$ ft

$h_{ss} \sim K_s K_a\left(\Delta p + \frac{C_R C_F R\, U_R^2}{2 g\, d_{ref}}\left(\frac{\rho_a}{\rho_w}\right)\right)$, $C_F = 0.0025$ $\frac{\rho_a}{\rho_w} \sim \frac{1}{800}$ $d_{ref} = 35'$ $C_R = 1$

$K_s K_a = 1$

$\boxed{h_{ss} = 8.40 \text{ ft}}$

Assignment 7-1

Name Donald Serolleman **Due March 11, 2010**

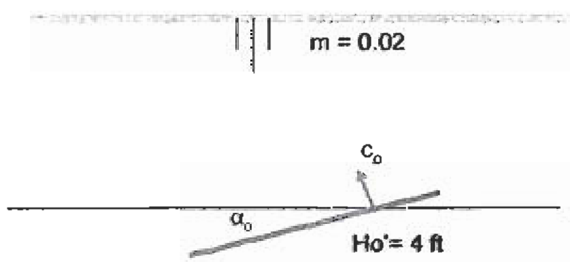
Compute the longshore transport for a deep water equivalent, $H_o' = 3$ ft at an angle (α_o) of 40 degree. The duration of the storm with this wave height is 8 hour.

Q_{ls} = 2 million units yd³/yr.

Total longshore transport for the storm: 1797 units yd³

Calculations

- Given: $H_o' = 3$ ft; $\alpha_o = 40°$; T = 5 s;
- Duration = 8 h
- Find: Q_{ls} =
- Storm Transported Volume =



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Assignment 7-1 | Donald Jerolleman

$H_0' = 3'$ @ $\alpha_0 = 40°$ $t_D = 8 hr.$

Find Q_{ls}, units, Total longshore transport for storm, units

$m = 0.02$

COE easy formula

$$Q_{ls} \approx K_y H_0^{5/2} (\cos \alpha_0)^{1/4} \sin 2\alpha_0 = 1{,}967{,}608 \frac{yd^3}{yr} = 2 \text{ million } \frac{yd^3}{yr}$$

$$K_2 = 1.37(10^5) \text{ for } \frac{yd^3}{yr}$$

$$\forall_{ls} = Q_{ls} \frac{yd^3}{yr} \left(\frac{t_D [hr]}{365 \frac{day}{yr} \left(\frac{24 hr}{day} \right)} \right) = 1796.9 \, yd^3 = 1797 \, yd^3$$

✓

21/11

Name Donald Jerollcman **Due February 18 2010**

Air temp = 28°C water temp = 33°C

Assignment 6.1: Estimate the south shore wave height and period for TS Isidore. Assume NNE winds at 35 knots with 2 hour duration. Estimate the wind setup for this storm.

Hs = ~~6.04 ft~~ 4.128 0/2

Ts = ~~5.69 s~~ 4.39 s 0/2

4 + 2

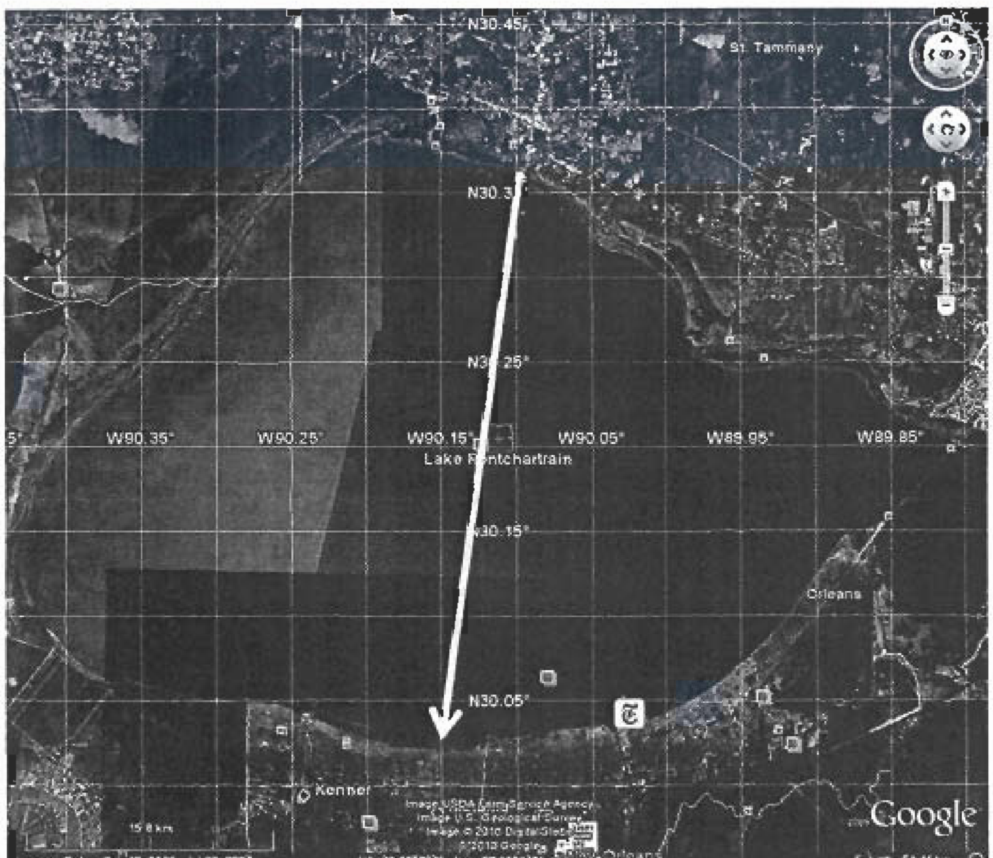
Are the waves: (FETCH LIMITED) or (DURATION LIMITED) ? 1

Wind setup = ~~1.274~~ 0.8713 /2 Using Equation 6.14

Wind setup = ~~3.069~~ 2.0278 Using Equation 6.15

2 + 2

11/14



~~15 km~~ ~~vector = 67.15 Km = 41.7 miles (more than what I expected)~~

1 Km = 0.62137 miles

15.8 km

vector = 39.5 Km = 24.5 miles

Donald Jerolleman H.W. 6.1

U_{ow} = ~~33~~ 35 knots $\left(\frac{1.68781 \text{ fps}}{\text{knots}}\right)$ = ~~55.698~~ 59.073 fps $\left(= \text{~~33~~ 35 knots}\left(\frac{1.15078 \text{ mph}}{\text{knot}}\right) = \text{~~37.98~~ } 40.277 \text{ mph}\right)$ $\left(= \text{~~33~~ 35 knots}\left(\frac{0.5144 \text{ m/s}}{\text{knot}}\right) = \text{~~16.98~~ } 18.00 \text{ m/s}\right)$

From scaling map vector

d = 14 ft F_a = (actual Fetch) = ~~41.7~~ 24.5 miles $\left(\frac{5280 \text{ ft}}{\text{mile}}\right)$ = ~~220308 ft~~ 129360 ft

t_d (duration) = 2 hr $\left(\frac{3600 s}{hr}\right)$ = 7200 s

ΔT = −5 °C

Taken as U_{ow}

$U_A = 0.5\text{~~8~~}3 \text{ 9}(U_{OL_{fps}} R_L R_T)^{1.23}$ From Fig 6.6 R_T = 1.07

R_L = $U_{ow} : U_{OL}$ = 1

U_A = ~~166.7~~ 88.42 ft/s

$\frac{gF_a}{U_a^2} = \frac{32.2(\text{~~220308~~ } 129360)}{\text{~~166.7}^2\text{~~ } 88.42^2}$ = ~~255.3~~ 532.79 (x-axis non-dimensional Fetch) For Fig 3-21

$\frac{gd}{U_a^2} = \frac{32.2(14)}{\text{~~166.7}^2\text{~~ } 88.42^2}$ = ~~0.01622~~ ~~5.098~~ 0.0577 (values on right of chart Fig 3-21)

Fig 3-21 Follow intersection of 2 values above to get y-axis

∴ $\frac{gH}{U_a^2}$ ≈ ~~0.007~~ 0.017

$H_s = \frac{\text{~~0.007~~ } 0.017(\text{~~166.7}^2\text{~~ } 88.42^2)}{32.2}$ = ~~6.04 ft~~ 4.128

Now using $\frac{gF_A}{U_a^2}$ & $\frac{gd}{U_a^2}$ in fig 3.22 → $\frac{gt_d}{U_A}$ = ~~1.1~~ 1.6

$T_s = \frac{\text{~~1.1~~ } 1.6(\text{~~166.7~~ } 88.42)}{32.2}$ = **~~5.693~~ 4.39**

For shallow $\frac{gT_m}{U_a} = \left(\frac{gt_d}{U_a}/537\right)^{3/7} = \left(\left(\frac{32.2(7200)}{\text{~~166.7~~ } 88.42}\right)\frac{1}{537}\right)^{3/7}$ = ~~1.504~~ 1.973

solving for T_m = ~~7.78~~ s 5.418 s

$T_s < T_m$ ∴ T_s Governs ∴ $H = H_s$ = **~~6.04 ft~~ 4.128**

& it is "Fetch" limited not "duration" limited

Formula 6.14 z_{sup} = $\frac{U_{ow}^2(\text{mph}) F_a(\text{mi})}{1400\ d(\text{ft})} = \frac{\text{~~37.98~~ } 40.277^2(\text{~~41.7~~ } 24.5)}{1400(14)}$ = ~~3.069~~ 2.0278

same as 6.15

Formula 6.14 = z_{sup} $\frac{F_a(\rho_a C_f U_{ow}^2)}{2 g \rho_w d} = \frac{\text{~~220308 ft~~ } 129360 (0.07489 \text{ lb/ft}^3)(0.00145)(\text{~~55.698 ft/s~~ } 59.073^2)}{2(32.2 \frac{ft}{s^2})(62.4 \text{ lb/ft}^3)\ 14 \text{ ft}}$ = ~~1.274~~ 0.8713 s

C_f from Fig 6.8

Coastal Jan 21 SIO ①

Fig 2-7 tells: wave type, linear

$L_o = \frac{gT^2}{2\pi}$ | $L = L_o \tanh\left(\frac{2\pi d}{L}\right)$ | $C = \sqrt{gd}$

$\downarrow$ | | $\downarrow$

$C_o = \frac{L_o}{T}$ | | $L = CT$

$E_{TOT} = \Sigma(E_p + E_k)$ $\quad E_p = \frac{1}{16}\gamma H^2 L + \frac{1}{16}\gamma H^2 L = \frac{\gamma H^2 L}{8}\left[\frac{N\cdot m}{m}\right]\left[\frac{lb\cdot ft}{ft}\right]$

(Happen to be the same)

$\bar{E} = \frac{E}{L} = \frac{\gamma H^2}{8}\left[\frac{N\cdot m}{m^2}\right]\left[\frac{lb\cdot ft}{ft^2}\right]$

$\bar{P}$ (Power Transmitted / unit crest width)

$= \bar{E}nc = \left(\frac{E}{L}\right)nc$

$= \bar{E}C_G \quad \left[\frac{\frac{lb\cdot ft}{s}}{ft}\right]$

record pause stop jump bookmark 0% jump to position 100% playback speed volume

Ocean Jan 28 ①

Shoaling changes from deep to shallow, change in wave form (L, H)

$P = nC\bar{E} = C_G \bar{E}$

shoaling coefficient

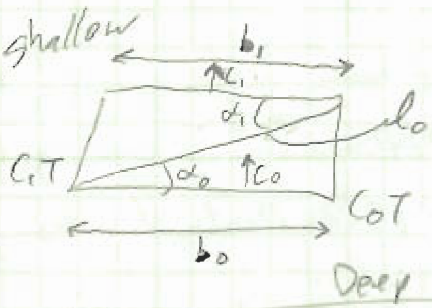
$n_0 C_0 H_0^2 = n_1 C_1 H_1^2 \Rightarrow \frac{H_1}{H_0} = \sqrt{\frac{n_0 C_0}{n_1 C_1}} = \sqrt{\frac{C_{G_0}}{C_{G_1}}} = \sqrt{\frac{n_0 L_0}{n_1 L_1}} = K_s$

K_s is given in table

Example $d_1 = 2.5m$ $H_0 = 1.5m$ $T = 4s$

$L_0 = \frac{g4^2}{2\pi} = \boxed{25m}$ $\frac{d_1}{L_0} = \underline{0.1}$ From table $K_s = 0.9327$

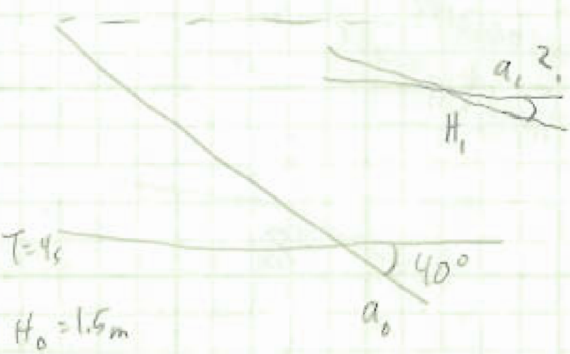
$H = H_0(K_s) = 1.5m(0.9327) = 1.4m$ (check Fig 2-7 or $\frac{H}{d} = \frac{1.4}{2.5} = 0.56$, to see if wave can exist eg: will break. b/c $0.56 < 0.78$ ∴ non-breaking)



Snell's Law $\frac{\sin\alpha_0}{\sin\alpha_1} = \frac{C_0}{C_1}$

$\frac{\cos\alpha_0}{\cos\alpha_1} = \frac{b_0}{b_1}$

Given: $d = 3m$ $H_0 = 1.5m$ $T = 4s$ $a_0 = 40^\circ$



$\frac{d}{gT^2} = 0.019$

using plate C-6 (chart) $K_s K_r \sim 0.85 - 0.87$

$\alpha_1 = 31^\circ$

∴ $H_1 = K_s K_r H_0 = 1.29m$

over

Est. equivelent deep water wave height H_0'

given $H = 2m$ $d = 5m$ $T = 6s$ $\alpha = 30°$

Find L_0, H_0', α_0 use Fig C-6

$L_0 = \frac{gT^2}{2\pi} = \underline{56.2m}$ $\frac{d}{gT^2} = 0.0142$ $\alpha = 30° \rightarrow a_0 = 46°$ From chart

$K_r K_s = 0.83$ From Fig

$H_0' = \frac{H}{K_r K_s} = 2.41m$

deepwater equivelent wave

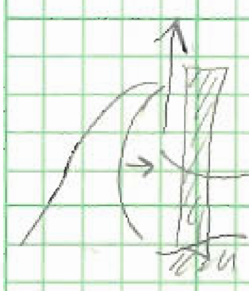
Video $H = 28'$ (significant wave) $N_{wind} = 65$ knots

$T = 25s$

$F = 1000$ Nautical miles

C before breaking $\sim \sqrt{g(d_b + H_b)}$

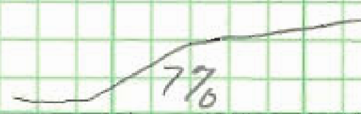
Logcabins' shelf



air pocket

minican force

20-30x pressure from height of wave



Ocean Feb 4

use interchanably

$K_d = K' = \frac{H}{H_i}$ $\qquad K' \propto \sqrt{\frac{1}{r}}$

(H_i: incoming wave)

$\max K_d \approx K_R' + K_L$ (right) (left) $\qquad \min K_d \approx |K_R' - K_L'|$ (abs. value)

$L = 18m \quad H_i = 3m \quad$ pt A inside 450m long Breakw.

– Look @ left $\quad r_{LA} = 72m \rightarrow \frac{r_{LA}}{L} = 4$ (L: wave length)

(r_{LA}: from left side to pt A)

Fig 2-33 $\quad K_R'$ from right $\approx$ 0.12

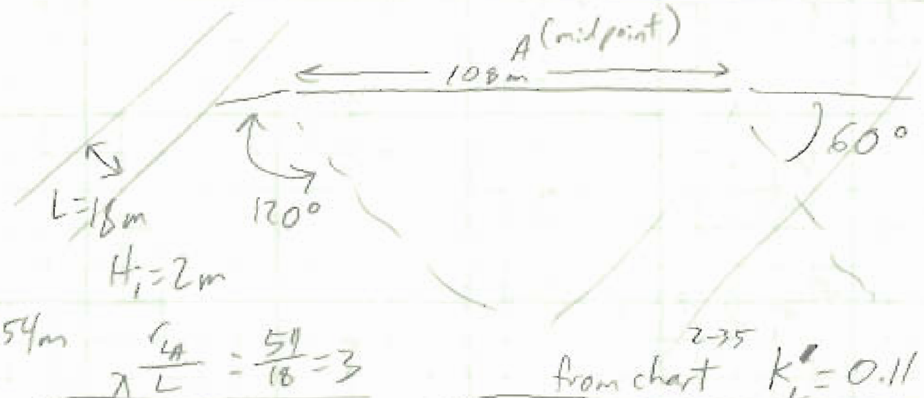
look @ right side $\quad r_{RA} = 450 - 72 = \_\_ \quad \frac{r_{RA}}{L} = 21$

since off chart we take value @ 10 = $K_{d_{10}} = 0.07$

$K_{d_{21}} = K_{d_{10}} \left(\sqrt{\frac{10}{21}}\right) = 0.03$

$K_T' = K_R + K_L = 0.15 \qquad H = H_i K_T' = 3(0.15) = 0.45m$

(H: inside; H_i: insident (incomming))



$r_{LA} = 54m \quad \frac{r_{LA}}{L} = \frac{54}{18} = 3$

same here $\quad \frac{r_{RA}}{L} = 3$

from chart 2-35 $\quad K_L' = 0.11 \quad K_R' = 0.20$

$K_T = 0.31 \qquad H_A = H_i(K_T) \approx 0.54m$ (wave hight @ A)

$$H_{loop} = H_i + H_r = H_i + k_f H_i = (1 + k_f) H_i$$

$$H_{node} = H_i - H_r = H_i - k_f H_i = H_i (1 - k_f)$$

$$H_{loop} + H_{node} = 2 H_i$$

$$H_i = \frac{1}{2}(H_{loop} + H_{node}) \qquad H_r = \frac{1}{2}(H_{loop} - H_{node})$$

$$k_f = \frac{H_r}{H_i}$$

Miche $\quad k_f = X_1 X_2$

Exp. Grass nearly impervious 2H : 1V slope

$H_0 = 1.5m \sim H_i$, $L_0 = 15m$

$\frac{H_0}{L_0} = 0.033$

<u>max steepness</u> $\sqrt{\left(\frac{2\beta}{\pi}\right)} \frac{\sin^2\beta}{\pi} \approx 0.036$

$\frac{H_0}{L_0}$ actual $< \frac{H_0}{L_0}$ max

$$X_2 \sim \frac{0.033}{0.036} = 0.92$$

$$k_f = 0.9 * 0.92 = 0.82$$

(a) T = 3 s d = 4m

2.1) $L_o = \frac{gT^2}{2\pi} = 14.05m$ $C = \frac{L}{T} =$ ____ m/s $\frac{d}{L_o} = \frac{4}{46.2} = 0.0865$

$\frac{d}{L} = 0.1286$ Transitional $L = \frac{4}{0.1286} = 31.1\ m$

T = 6s d = 4m $L_o = \frac{9.81(6)}{2\pi} = 92.5m$ $\frac{d}{L_o} = 0.0433$

$\frac{d}{L} = 0.0866$ Transitional $L = \frac{4}{0.0866} = 46.17m$

$C = \frac{L}{T} = \frac{46.17}{6} = 7.69\ \frac{m}{s}$

(b) T = 6s H = 1.2m

$\frac{H}{gT^2} = \frac{1.2}{9.81(6^2)} = 0.0034$ $\frac{d}{gT^2} = \frac{4}{9.81(6^2)} = 0.01133$

is NON-BREAKING, STOKES 3rd order, Transitional

$U_B = \left(\frac{H}{T}\right) \frac{\pi}{\sinh\left(\frac{2\pi D}{L}\right)} = \left(\frac{H}{T}\right) \frac{\pi}{\quad}$

2-2) T = 5s H = 1m d = 4m

$L_o = \frac{gT^2}{2\pi} = 77.05$ $\frac{d}{L_o} = 0.0517$ $\frac{d}{L} = 0.0952 \rightarrow L = 42.01m$

OCEAN/COAST JAN 14 SIO ①

Coastal engineering manual

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- why water percip on glass & not anything else
- Relativity false?

Fast (slow time) — stationary (time fast)

same speed

same timeing

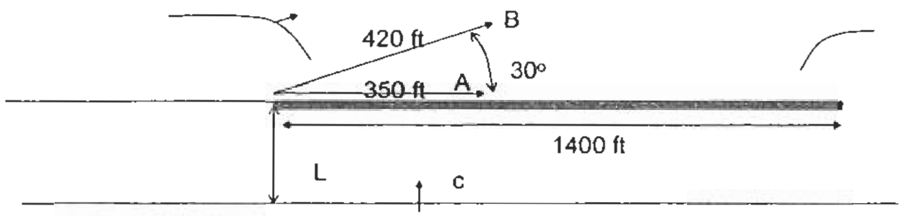
who is to say Im moving fast not you so I dont age

record pause stop jump bookmark 0% jump to position 100% playback speed volume

Name: Donald Serolleman Due Date: February 4 2010

Assignment 4-2

Estimate the maximum wave heights and points A and B in the region behind the breakwater in the attached plan. Given: Hi = 5 ft; T = 6s; d =22 ft.



Answers

Maximum H at point A: ______________

Maximum H at point B: ______________

djerolle

Assignment 4 & 5

Machine Name: w14955

Date: 02/24/2010

Job: 339

Time: 5:09:53 PM

Cost: 0.00

Assignment 4-2

Hi	5 ft	1.52439 m								
T	6 sec	6 s								
d	22 ft	6.707317 m								
Lo	184.4924 ft	56.24769 m								
L	139.6917 ft	42.58895 m								
err	0.000622									
A										
r	320 ft	97.56098 m								
r/L	2.290758	Left								
K'L	0.16									
r	1080 ft	329.2683 m								
r/L	7.731309	Right								
K'R	0.083									
K'T	0.243									
HA max	1.215 ft	0.37 m								
B										
r	420 ft	128.0488 m	angle							
r/L	3.00662	Left		30 o						
K'L	0.16									
r	1057.333 ft	322.3577 m	209.9998		363.7308	1036.27	angle	0.19994	**11.46** o	
r/L	7.569047	Right								
K'R	0.088									
K'T	0.248									
HB max	1.24 ft	0.378 m								

Assign 4-3

Ho	1		
T	5		
Lo	39.03275		
Ho/Lo	0.02562		
Seawall			
cot	2		
Ho/Lo max	0.036		
X2	1		
X1	0.6		
Kf	0.6		
Beach			
cot	20	0.0500	0.050
Ho/Lo max	0.000142		
X2	0.005542		
X1	0.8		
Kf	0.0044		

Assign 5.1

LB	132000 ft	40243.9 m	N-S	25
Mode	1			
T1	13539.04 s	3.761 h		
Mode	2			
T2	6769.519 s	1.88 h	NS	
LB	216480 ft	66000 m	N-S	41
Mode	1			
T1	22204.02 s	6.168 h		
Mode	2			
T2	11102.01 s	3.084 h	NS	

djerolle

Test review 1

Machine Name: w14955

Date: 02/24/2010

Job: 332

Time: 5:06:44 PM

Cost: 0.00

REVIEW 2010

No. 1

1. Given: d = 14 ft; T = 5.2 sec; H = 5.4 ft: Classify

d	14 ft		
T	5.2 sec		
H	5.4 ft		
d/gT2	0.016079		
H/gT2	0.006202		
Fig 2-7	cn/STOKES 3	Trans	NonBreaking

No. 2

2. Given: d = 14 ft; T = 5.2 sec; H = 5.4 ft:
Find:
L =
Ub max =

c =	19.1 ft/sec		
cG=	15.47 ft/sec		
E =	22444 lbs-ft/ft/wave		
Bed p/γ max =	15.90 ft		
Bed p/γ min =	12.10 ft		

d	14 ft		
T	5.2 sec		
H	5.4 ft		
Lo	138.5744 ft		
L	98.67879 ft	Goakseek	98.6783
d/Lo	0.10		0.00049
d/L	0.141 Append I		
L	99.29 ft	approx	
c	19.1 ft/sec		
n	0.81		
cg	15.47 ft/sec		

No. 3

3. Given: d = 14 ft; T = 8.8 sec; H = 6 ft:
Find cnoidal L, c.
What is the best theory for this non-linear wave?

d	14 ft		
T	8.8 sec		
H	5.4 ft		
Lo	396.9 ft		
Tsqrt(g/d)	13.34586		
H/d	0.39		
k2	1-10-2	Fig 2-11	
Ur	75	>26	Cnoidal
L	195.2 ft	Fig 2-12	
c	22.18		

No. 4

4. Given: d = 14 ft; T = 5.2 sec; H = 5.4 ft:
Find:
Ho' =
What is H at 10 ft?

d	14 ft	
T	5.2 sec	
H	5.4 ft	
Lo	138.6 ft	
d/Lo	0.101029	
Ks	0.9327 Appendix I	
Ho'	**5.79** ft	Ho'=H/Ks

No. 5

5. Given: d = 14 ft; T = 5.2 sec; H = 5.4 ft; α = 35o:
Find:
Ho' = 6.75 ft
αo = 52.5 o
At d = 10 ft find:
H = 5.535 ft
α = 29 o

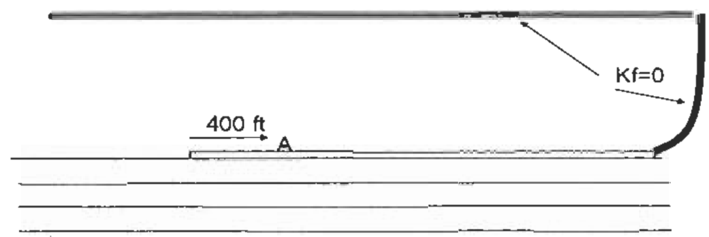
d	14 ft		
T	5.2 sec		
H	5.4 ft		
Lo	138.6 ft		
α	35 o		
d/Lo	0.101		
d/gT2	0.016079		
KsKr	0.8 Fig	C6	d/gT2 intersects with a = 35o
Ho'	6.75 ft	Ho'/KrKs	
αo	52.5 o	C6	

d2	10	ft	
d/gT2	0.011485	Enter Fig C6 withd/gT2 and ao.	
KrKs	0.82		Read C6
α2	29	o	Read C6
H2	5.535	ft	

No. 6

6. Given: d = 14 ft; T = 5.2 sec; H = 5.4ft:
Find: H at point A behind the semi-infinite breakwater.

d	14	ft	
T	5.2	sec	
H	5.4	ft	approx
Lo	138.5744	ft	
L	100	ft	
r	400		
r/L	4		
K'L	0.12	Fig 2.33	
K'R	0		
K'	0.12		
HA	**0.65**	ft	



No. 7
Revised

7. Given: = 14 ft; T = 5.2 sec; Ho = 5.5 ft: Seawall slope Cot = 1.5
Find: Kf if the seawall is rock rubble.

d	14	ft			
T	5.2	sec			
Ho	5.5	ft			
Lo	138.5744	ft			
Ho/Lo	0.03969				
Ho/Lo	0.06	max	> Ho/Lo	X2	1
cot	1.5				
X1	0.3				
Kf	**0.3**				

No. 8
Revised

8. Given: T = 12 sec; Ho = 5.5 ft: Shelf slope 5% Find: Hb and
Revised

T	12	sec	
Ho	5.5	ft	
Lo	737.9703	ft	
Ho/Lo	0.0075		
Hb/Ho'	1.8	Plunging/Surging	Fig 2-65
Hb	9.9	ft	
Hb/gT2	0.002135		
Hb/db	0.94		Fig 2-66
db	9.306	ft	

d	14	ft	
T	5.2	sec	
H	5.4	ft	
Lo	138.5744	ft	
a	35	o	
d/Lo	0.101029		
d/gT2	0.016079		
	24		
KsKr	0.8		C6
Ho'	6.75	ft	
ao	52.5		
d2	10		
d/gT2	0.011485		
KrKs	0.82		
a2	29	o	
H2	5.535	ft	

UNIVERSITY OF NEW ORLEANS

DEPARTMENT OF CIVIL AND ENVIRONMENTAL ENGINEERING

Coastal and Ocean Engineering
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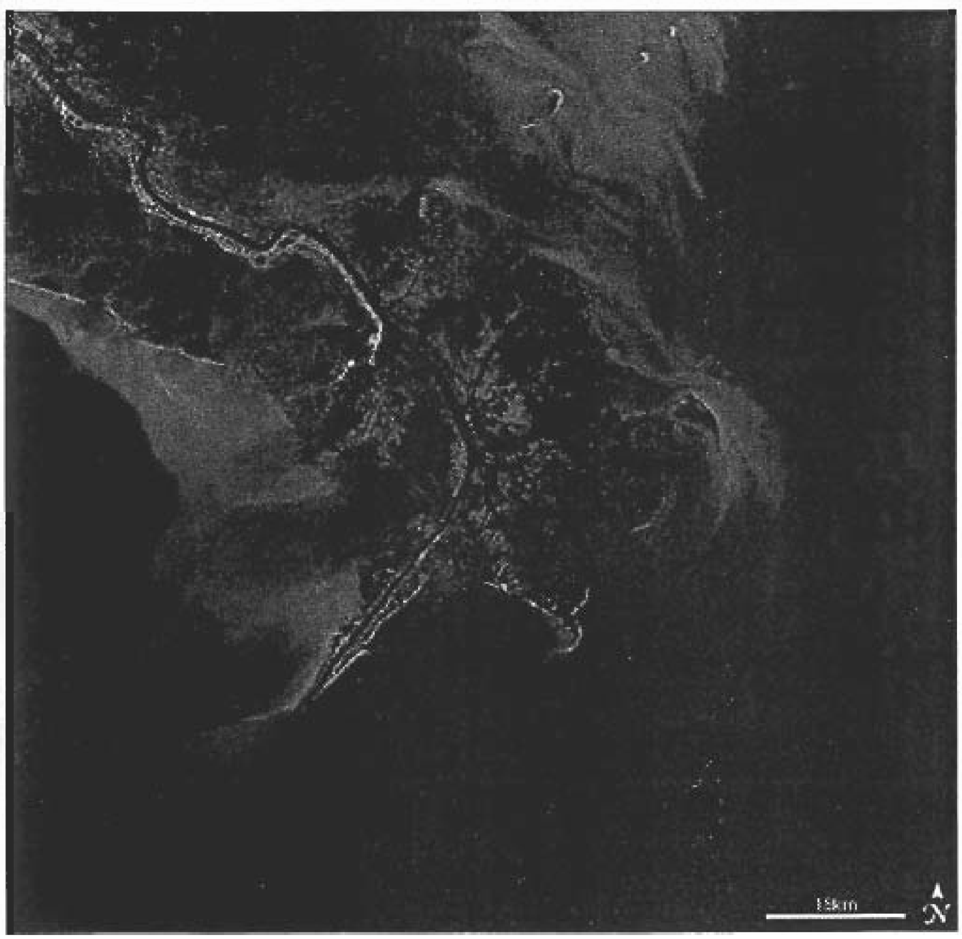
***COURSE OBJECTIVE (ENCE 4723 and 4723G)** : This is an introduction to basic wave theory and its application to Coastal and Ocean Engineering. The emphasis of the course will be on coastal processes; however, deep water theory and applications will be included.*



TEXT: **Coastal Engineering Manual EP 1110-2-1100 (Parts I thru IV)** _______

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Part IV

http://www.usace.army.mil/usace-docs/eng-manuals/em1110-2-1100/PartIV/PartIV.htm

Additional References:

1. US Army Corps of Engineers, 1984 or 1987. "Shore Protection Manual", Coastal Engineering Research Center. [SPM]

2. Sarpkaya, T. and Isaacson, M., 1981. "Mechanics of Wave Forces on Offshore Structures", van Norstrand Reinhold, New York, NY.

3. Silvester, R., 1974. "Coastal Engineering", Vol. I and II, Elsevier Sc. Publ. Co., New York, NY.

4. Per Brune, 1981. "Port Engineering", Gulf Publishing, Houston, TX.

5. LeMehaute, B., 1976. "An Introduction to Hydrodynamics and Water Waves", Springer-Verlag, New York, NY.

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OFFICE HOURS: Thursday 1 pm to 3 pm or by appointment. Please call or arrange for an appointment at the time of the lecture.

GRADING SCHEME:

1. Assignments and project	20%
2. Mid-term tests (2) (open book)	40%.
3. Final examination (open book)	40%.

Grades

A 89.5-100
B 79.5-89.5
C 69.5-79.5
D 59.5-69.5
F Less than 59.5.

NOTICE FOR STUDENTS TAKING THIS COURSE FOR GRADUATE SCHOOL CREDIT: It is the policy of the graduate faculty of the Civil and Environmental Engineering program that a graduate student will be dropped from the Civil and Environmental Engineering program if they make an "F" grade or two "D" grades in their course of study.

Common Symbols

A = Area normal to flow

b = wave crest width

B = Top width [also T]

c = Wave Celerity

c_G = Group velocity

C_c = Contraction coefficient [0.5 for re-entrant case; 0.61 for flush opening]

d = Depth to the bed (for coastal applications d is the depth below the still water level.)

d_b = depth at which waves break

D = Hydraulic mean depth = A/B or A/T

E = Wave Energy

f = frequency

$n = c_G / c$ = the fraction of wave energy being transmitted

F = Force

g = Acceleration due to gravity [use 32.2 ft/sec^2 or 9.81 m/s^2]

h = height

h_z = Elevation

H = Wave Height

H_s = Significant Wave Height

H_o = Deep Water Wave Height

H_b = wave height at breaking

K_s = Shoaling Coefficient

K_r = Refraction Coefficient

K_d = Diffraction Coefficient

K_f = Reflection Coefficient

L = Wave Length

Lo = Deep Water Wave Length

M = Momentum flux or momentum flow

"o" as subscript indicates deep water.

p = Pressure

p = probability

P = wave power

q = Discharge per unit width

Q = Discharge = VA

S_o = Bed slope

S_s = Specific Gravity of a Material

t = Time

T = Wave period

Ts = Significant Wave period

{u,v,w} = Velocity components in {x,y,z} Cartesian coordinates

W = Width

x = Commonly used as direction of flow [measured along the bed]

α = wave crest angle with respect to the shoreline

γ = Specific weight = g ρ (typical 62.4 lbs/ft^3; 9810 N/m^3)

δ = Boundary layer thickness

ε = Roughness height

κ = von Karman universal constant = 0.4

η = elevation with respect to the still water level.

λ = Wave length (in some references)

θ = Bed slope angle

θ_f = Friction angle

μ = Dynamic viscosity

ν = Kinematic viscosity (typical 10^{-5} ft^2/sec; 10^{-6} m^2/s)

σ = Surface tension (typical $5*10^{-3}$ lbs/ft; $7.3*10^{-2}$ N/m)

φ = Side slope angle

ρ = Density (1.94 slugs/ft^3; 1000 kg/m^3)

τ = Shear stress

Lecture 1
Introduction

Coastal Zone:

Ocean Zone:

Types of Waves:

There are two types of water waves that civil engineers encounter:

1. oscillatory (no net transport of fluid - sea waves)
2. translatory (net transport of fluid - e.g. flood waves and waves in the surf zone)

The definitions and characteristics of an oscillatory wave are shown in the attached Figure 1-3 SPM.

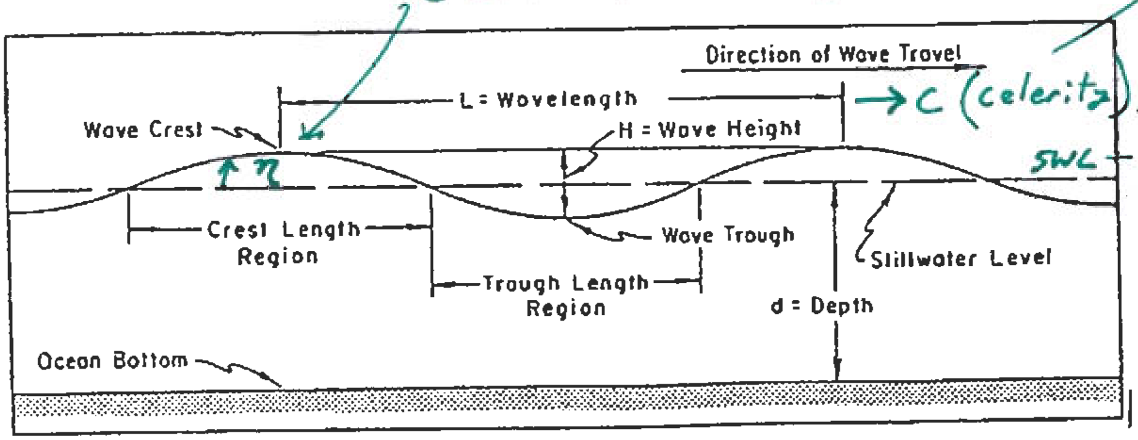


Figure 1-3. Wave characteristics.

(from SPM)

Waves are also classified by the dominant forces that influence their formation or movement; these forces can be restorative or driving forces. Figure 2-1 summarizes the energy spectra for wide range of waves according to driving force and restoration force. Some of these waves are:

tides (driving force gravitational pull or moon/sun)
wind waves (driving force is wind shear)
gravity waves (restorative force is earth's gravity)
capillary waves (restorative force is surface tension)

tsunamis, 'tidal' waves or storm surges (disturbing force barometric pressure and wind or earthquake)
boat waves
internal waves
seiches
elastic waves such as water hammer (restorative force is elastic force of the system)

Diernal tidal? (semi)

Lake Pontchatrain $f = \frac{1}{2} \to \frac{1}{6}$

Gulf $f = 1 \to \frac{1}{30}$

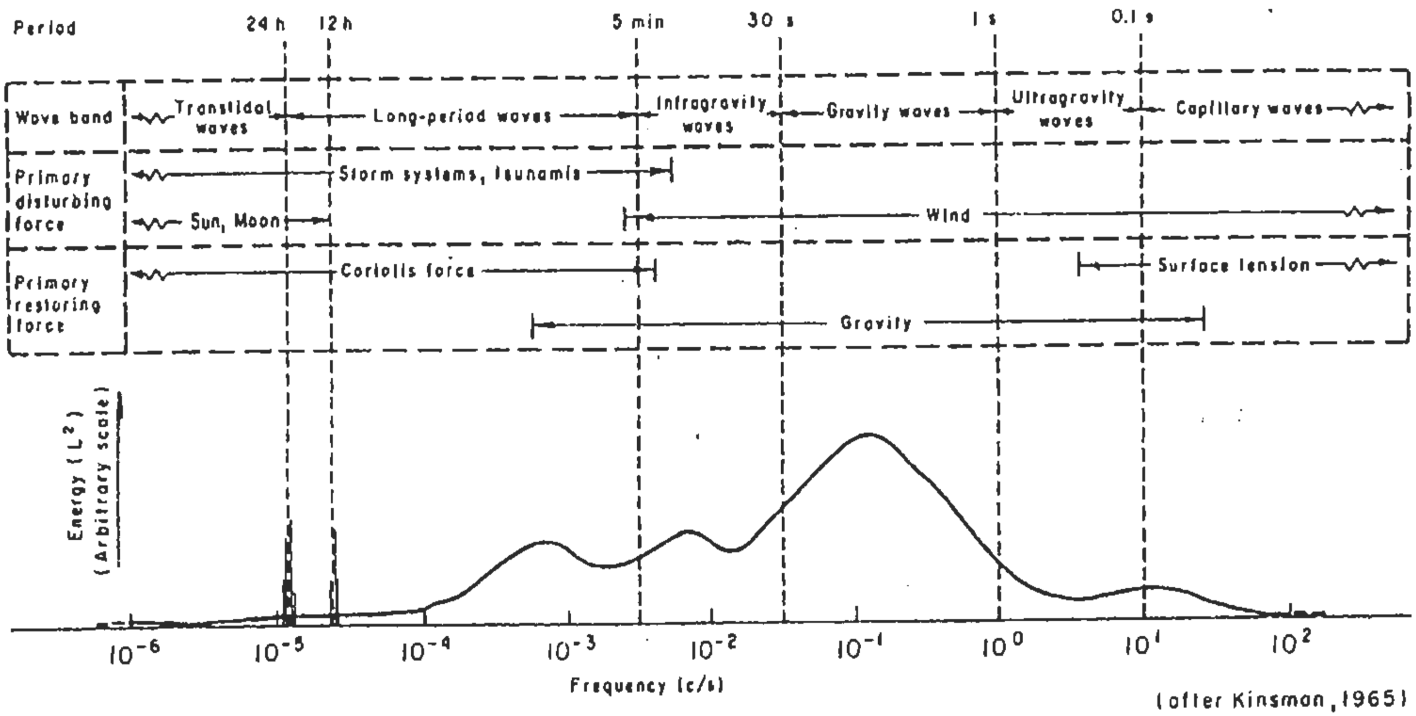
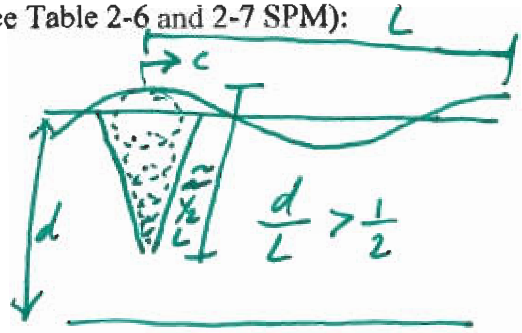


Figure 2-1. Approximate distribution of ocean surface wave energy illustrating the classification of surface waves by wave band, primary disturbing force, and primary restoring force.

The most common gravity waves are generated by wind and are wind-gravity waves. Most wind generate coastal and ocean waves have periods of 3 to 25 seconds.
Gravity waves are further classified as (see Table 2-6 and 2-7 SPM):

Deep water: $d/L > ½$



wave length

$$L_0 = \frac{gT^2}{2\pi}$$

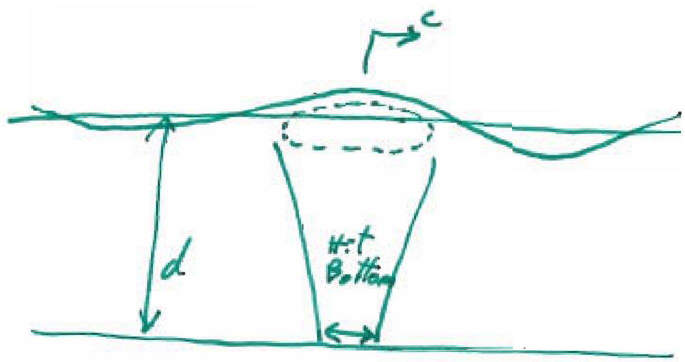
deep water

cusp = limiting height of wave

$$\approx \frac{H}{L} \rightarrow \frac{1}{7}$$

Transitional or intermediate waves: $1/25 < d/L < ½$

effected by wave length & depth



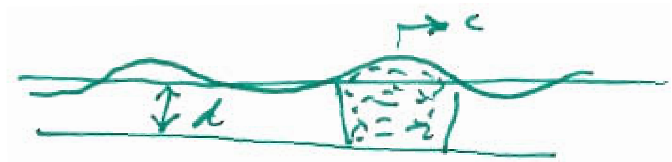
correction factor

$$L = L_0 \tanh\left(\frac{2\pi d}{L}\right)$$ [use tables or goal seak to find L]

$$C = \frac{L}{T}$$

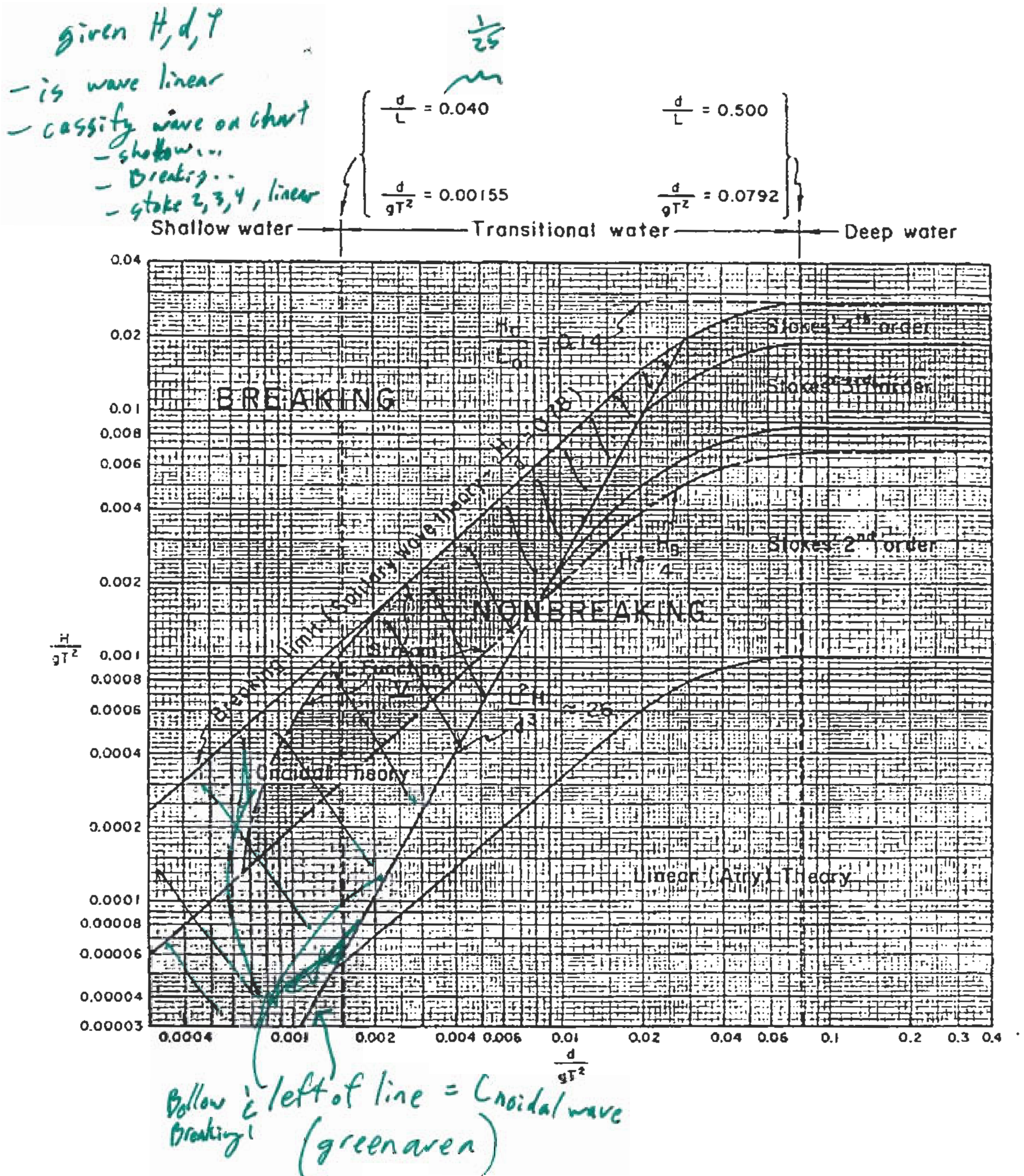
Shallow water: $d/L < 1/25$

speed dependent on depth



$$C = \sqrt{gd} \qquad L = CT$$

Gravity waves can be described by linear (Airy or small amplitude wave, i.e. $H/L << 1/7$ and $H/d << H_b/d_b$) theory or non-linear theory or large amplitude waves. Figure 2-7 SPM shows the limits of linear theory and some examples of non-linear methods. This Figure also distinguishes between breaking and non-breaking waves. Figure 2-6 gives the celerity, and orbital kinematics of linear gravity waves.

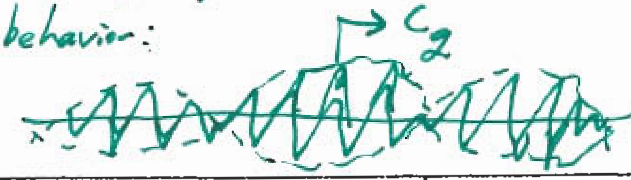


(after Le Mehaute, 1969)

Figure 2-7. Regions of validity for various wave theories.

Group Velocity
$C_G = Cn$
$n = \{1 + (4\pi d/L)/\sinh(4\pi d/L)\}/2$
see appendix I ☆ If linear

@ beach every 7-9 waves is hi-se
group behavior:

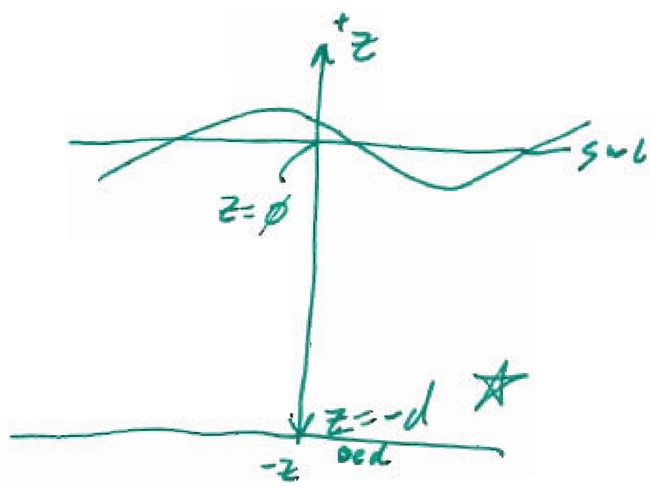


RELATIVE DEPTH	SHALLOW WATER $\frac{d}{L} < \frac{1}{25}$	TRANSITIONAL WATER $\frac{1}{25} < \frac{d}{L} < \frac{1}{2}$	DEEP WATER $\frac{d}{L} > \frac{1}{2}$
1. Wave profile	Same As →	$\eta = \frac{H}{2}\cos\left[\frac{2\pi x}{L} - \frac{2\pi t}{T}\right] = \frac{H}{2}\cos\theta$	← Same As
✱ 2. Wave celerity	$C = \frac{L}{T} = \sqrt{gd}$	$C = \frac{L}{T} = \frac{gT}{2\pi}\tanh\left(\frac{2\pi d}{L}\right)$	$C = C_0 = \frac{L}{T} = \frac{gT}{2\pi}$
✱ 3. Wavelength	$L = T\sqrt{gd} = CT$	✱ $L = \frac{gT^2}{2\pi}\tanh\left(\frac{2\pi d}{L}\right)$	✱ $L = L_0 = \frac{gT^2}{2\pi} = C_0T$
4. Group velocity	$C_g = C = \sqrt{gd}$	$C_g = nC = \frac{1}{2}\left[1 + \frac{4\pi d/L}{\sinh(4\pi d/L)}\right]\cdot C$	$C_g = \frac{1}{2}C = \frac{gT}{4\pi}$
5. Water Particle Velocity (a) Horizontal (max cosθ = 1)	$u = \frac{H}{2}\sqrt{\frac{g}{d}}\cos\theta$	$u = \frac{H}{2}\frac{gT}{L}\frac{\cosh[2\pi(z+d)/L]}{\cosh(2\pi d/L)}\cos\theta$	$u = \frac{\pi H}{T}e^{\frac{2\pi z}{L}}\cos\theta$
(b) Vertical (max sinθ = 1)	$w = \frac{H\pi}{T}\left(1 + \frac{z}{d}\right)\sin\theta$	$w = \frac{H}{2}\frac{gT}{L}\frac{\sinh[2\pi(z+d)/L]}{\cosh(2\pi d/L)}\sin\theta$	$w = \frac{\pi H}{T}e^{\frac{2\pi z}{L}}\sin\theta$
6. Water Particle Accelerations (a) Horizontal	$a_x = \frac{H\pi}{T}\sqrt{\frac{g}{d}}\sin\theta$	$a_x = \frac{g\pi H}{L}\frac{\cosh[2\pi(z+d)/L]}{\cosh(2\pi d/L)}\sin\theta$	$a_x = 2H\left(\frac{\pi}{T}\right)^2 e^{\frac{2\pi z}{L}}\sin\theta$
(b) Vertical	$a_z = -2H\left(\frac{\pi}{T}\right)^2\left(1 + \frac{z}{d}\right)\cos\theta$	$a_z = -\frac{g\pi H}{L}\frac{\sinh[2\pi(z+d)/L]}{\cosh(2\pi d/L)}\cos\theta$	$a_z = -2H\left(\frac{\pi}{T}\right)^2 e^{\frac{2\pi z}{L}}\cos\theta$
7. Water Particle Displacements (a) Horizontal	$\xi = -\frac{HT}{4\pi}\sqrt{\frac{g}{d}}\sin\theta$	$\xi = -\frac{H}{2}\frac{\cosh[2\pi(z+d)/L]}{\sinh(2\pi d/L)}\sin\theta$	$\xi = -\frac{H}{2}e^{\frac{2\pi z}{L}}\sin\theta$
(b) Vertical	$\zeta = \frac{H}{2}\left(1 + \frac{z}{d}\right)\cos\theta$	$\zeta = \frac{H}{2}\frac{\sinh[2\pi(z+d)/L]}{\sinh(2\pi d/L)}\cos\theta$	$\zeta = \frac{H}{2}e^{\frac{2\pi z}{L}}\cos\theta$
8. Subsurface Pressure	$p = \rho g(\eta - z)$	$p = \rho g\eta\frac{\cosh[2\pi(z+d)/L]}{\cosh(2\pi d/L)} - \rho gz$	$p = \rho g\eta e^{\frac{2\pi z}{L}} - \rho gz$

Figure 2-6. Summary of linear (Airy) wave theory--wave characteristics.

Loads on Coastal and Ocean Structures: external forces that act on these hydraulic structure may be static or dynamic. The following are examples of loads that should be considered:

1. Hydrostatic forces, e.g. due to differences in water level,
2. Seepage forces, e.g. due to seepage under a structure,
3. Hydrodynamic force, e.g. due to flow in, around, over or through the structure,
4. Wave forces,
5. Tides and storm surge forces,
5. Forces due to turbulence,
6. Earthquake forces,
7. Silt or soil loads,
8. Ice forces,
9. Debris forces,
10. Elastic forces due to water hammer,
11. Wind forces,
12. Ship forces,



Bottom Vel.
$U_b = \frac{\left(\frac{\pi H}{T}\right)}{\sinh\left(\frac{2\pi d}{L}\right)}$ ☆ Transitional waves B/C deepwater yields ∅

13. Self weight,
15. Forces due to construction activity,
16. Forces due to operational activity.

Assignment 1-1

Homework: Down Load EM 1110-2-1100 Part II Chapter 1.

Define:

Coastal Zone:

Regular waves

Irregular waves

Lecture 2
Linear Wave Theory

Linear wave theory applies to waves that are generally: small in amplitude, sinusoidal in shape, symmetrical about the mean water level, and travel a constant speed. Figure II-1-1 shows the definitions for a linear wave. As Figure 2-7 (SPM) indicates, the theory is valid over a limited range of relative depths and wave steepnesses. The internal velocities and accelerations in a linear wave can be determined by solving the ideal flow equation (Laplace Equation) subject to surface and bottom kinematic boundary conditions and crest and bottom boundary conditions [see CEM II-1-5 or also see Ippen]. The CEM lists the assumptions for linear wave theory. Figure 2-6 SPM gives the celerity, and orbital kinematics of linear gravity waves.

Celerity (c) is the wave speed with respect to the mean water velocity. The wave length, celerity and wave period are related by

$$c = L/T \qquad 2.1\text{ NB}$$

Linear theory gives a shallow water wave celerity of:

$$c = (g\,d)^{1/2} \qquad 2.2\text{ NB}$$

and a deep water wave celerity of

$$c_o = [gL/(2\pi)]^{1/2} \qquad 2.3$$

For transitional or intermediate waves, we have

$$c = [\{gL/(2\pi)\}\tanh(2\pi d/L)]^{1/2} \qquad 2\text{-}4$$

This equation can be solved by Tables (APPENDIX 1) or Figure 2.6 (Attached).

Equation 2.3 can be re-written as

$$L_o = gT^2/(2\pi) \qquad 2.5\text{ NB}$$

and

$$c_o = gT/(2\pi) \qquad 2.6$$

For transitional or intermediate waves, were-write Eq. 2.3 as

$$c = \{gT/(2\pi)\}\tanh(2\pi d/L) \qquad 2.7$$

Or

$L = L_o \tanh(2\pi d/L)$ 2-8 NB

Problem: Given: a period of 4.5 seconds, a water depth of 4 m and a wave height of 0.3 m.
Find: L and c. Assume linear wave theory. Use Figure 2.7 to verify if linear theory is applicable.
Is this: a deep water wave, shallow water wave or a transitional wave?
Estimate the maximum horizontal velocity at the bed?

The sinusoidal weave equation is usually given as

$$\eta = a\cos(kx - \omega t) = (H/2)\cos(2\pi x/L - 2\pi t/T) \quad 2.9$$

$$= (H/2)\cos\{(2\pi/L)(x - c\,t)\}$$

a = wave amplitude = H/2
k = wave number = $2\pi/L$
ω = angular frequency in 1/radians = $2\pi/T$ (rad/s)

$f = \frac{1}{T}\ (\frac{1}{s})$

$L_o = \frac{gT^2}{2\pi} = 31.6\text{ m}$

$\frac{d}{L_o} = \frac{4}{31.6} = 0.126$ go to table

$\frac{d}{L} = 0.1632$

$L = \frac{d}{d/L} = \frac{4}{0.1632} = 24.5\text{ m}$

$C = \frac{L}{T} = \frac{24.5}{4.5} = 5.44\text{ m/s}$

$\frac{d}{gT^2} = \frac{4}{9.81(4.5)^2} = 0.02$

$\frac{H}{gT^2} = \frac{0.3}{9.81(4.5)^2} = 0.0015$

From chart
- Non-linear (type: stokes 2)
- transitional
- non-breaking

$u_b = \left(\frac{H}{T}\right)\frac{\pi}{\sinh(2\pi d/L)} = \frac{0.3}{4.5}\left(\frac{\pi}{1.215}\right) = 0.17\text{ m/s}$ (sinh on Table)

Problem: Find the deep water wave length and celerity for an observed period of 10 s. If the water depth is 100 m, is this a deep water wave?

✓ to see if deep water wave: $L_o = \frac{gT^2}{2\pi} = 156\text{ m}$

$\frac{d}{L_o} = \frac{100}{156} = 0.64 > 0.5$ ∴ Deep

Assignment 2-1. a) Wave periods in Lake Pontchartrain vary from 3 to 6 seconds. Determine the wave lengths and celerities for these periods. Classify the waves as: Deep, Shallow or Transitional (Intermediate). Assume that the lake depth is 4 m.
b) If the 6 second wave has a height of 1.2 m, is the linear wave theory applicable? Based on linear wave theory, estimate the orbital velocity at the lake bed for the 6 second wave.
See Figure II-1-2 and II-1-22 in CEM or see Table in Figure 2.6 SPM.

Figure II-1-3 shows the phase relations for the wave, the orbital velocities (u,v) and the accelerations (a_x, a_y).

The absolute pressure head under a linear wave is given by,

$$p'/\gamma = H \cosh[2\pi(z + d)/L]/\{2\cosh[2\pi d/L]\} - z + p_{atm}/\gamma \qquad 2.10$$

where z is the vertical coordinate (positive upward from the origin at the water surface).

The gauge pressure head is

$$p/\gamma = H \cosh[2\pi(z + d)/L]/\{2\cosh(2\pi d/L)\} - z \qquad 2.11$$

where γ = is the specific weight of water = ρ g [ρ = 1000.0 kg/m^3 3 (1.94 slugs/ft^3) for freshwater and 1025 kg/m^3 3 (2.0 slugs/ft^3) for saltwater].

Assignment 2-2. For a wave period in Lake Pontchartrain of 5 seconds and a wave height of 1 m, estimate the maximum and minimum pressure heads that an object on the bottom of the Lake will experience. Assume that the lake depth is 4 m and the salinity is negligible (~ 3ppt).

Waves tend to travel in groups with a **Group Velocity**, c_G, that is generally equal or less than the speed of individual waves. The ratio of the Group Velocity to the Celerity, c, is the fraction of the wave energy that is transmitted, i.e.

$$n = c_G/c = \text{the fraction of wave energy being transmitted} \qquad 2.12$$

$$c_G = c\,\{1 + (4\pi\, d/L)/\sinh(4\pi\, d/L)\}/2 \qquad 2.13$$

Therefore

$$n = \{1 + (4\pi\, d/L)/\sinh(4\pi\, d/L)\}/2 \qquad 2.14$$

which can be obtained from Figure II-1-5 or II-1-7 in the CEM or handout APPENDIX I..

$$\eta_{envelope} = \pm H \cos\left[\pi\left(\frac{L_2 - L_1}{L_1 L_2}\right)x - \pi\left(\frac{T_2 - T_1}{T_1 T_2}\right)t\right] \qquad \text{(II-1-48)}$$

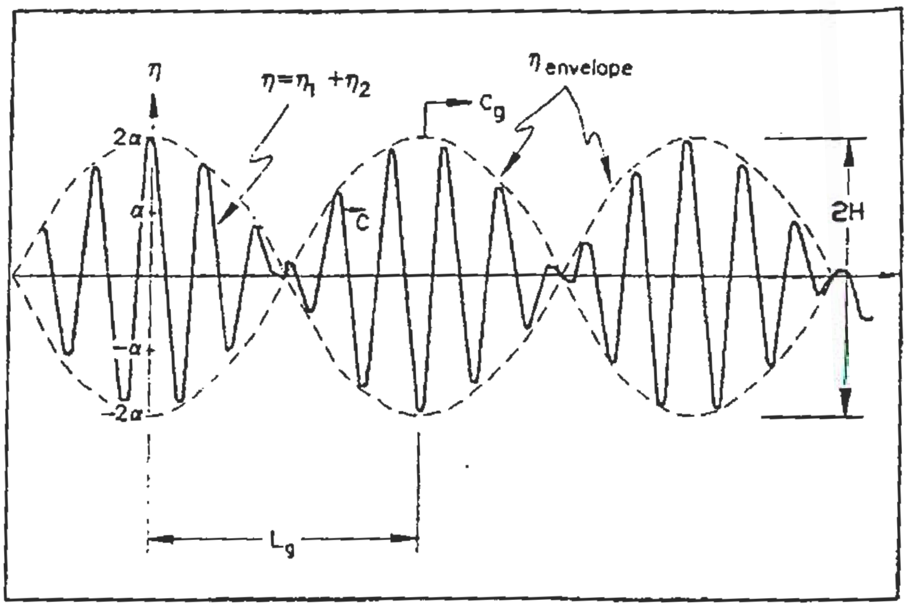


Figure II-1-6. Characteristics of a wave group formed by the addition of sinusoids with different periods

For deep water Eq. 2.13 gives

$$c_G = c_o/2 \qquad 2.15$$

and for Shallow water

$$c_G = c \sim (g\,d)^{1/2} \qquad 2.16$$

Problem: Find the "n" for the wave in for a 0.6 m wave in a water depth of 4 m with a period of 4.5 seconds. $H = 0.6$ m $T = 4.5$ s $d = 4$ m $c = 5.44$ (in table)

$L = 24.4$

$n = \{1 + (4\pi d/L)/\sinh(4\pi d/L)\}/2 = 0.7682$ (in table)

$C_G = c\,n = 5.44 \times 0.7682 = 4.17$ m/s

The **energy** in a single wave per unit width (crest) is the sum of the kinetic energy and the potential energy, i.e.

$$E = E_k + E_p = \rho g H^2 L/8 \qquad 2.17$$

E has units of –m/m or ft-lbs/f. Also $E_k = E_p$.

The energy per unit surface area is

$$\overline{E} = E/L = \rho g H^2/8 \qquad 2.18$$

The **power** transmitted per unit crest width is

$$\overline{P} = \overline{E}\, n\, c = (E/L)\, c_G \qquad 2.19$$

Eq. 2.19 is an important relationship for computing wave transformations.

These equations are all based on linear wave theory. However, they do give us an approximation for the behaviour of non-linear waves.

Definition: ***Wave Steepness = H/L***

Note: Linear wave equations are summarized in Table II-1-9 of the CEM and Figure 2-6.
I will discuss "radiation stresses" after I introduce breaking wave theory (see page II-1-29).

$\gamma_{\text{fresh } H_2O} = 9810\ N/m^3 = 62.4\ lb/ft^3$

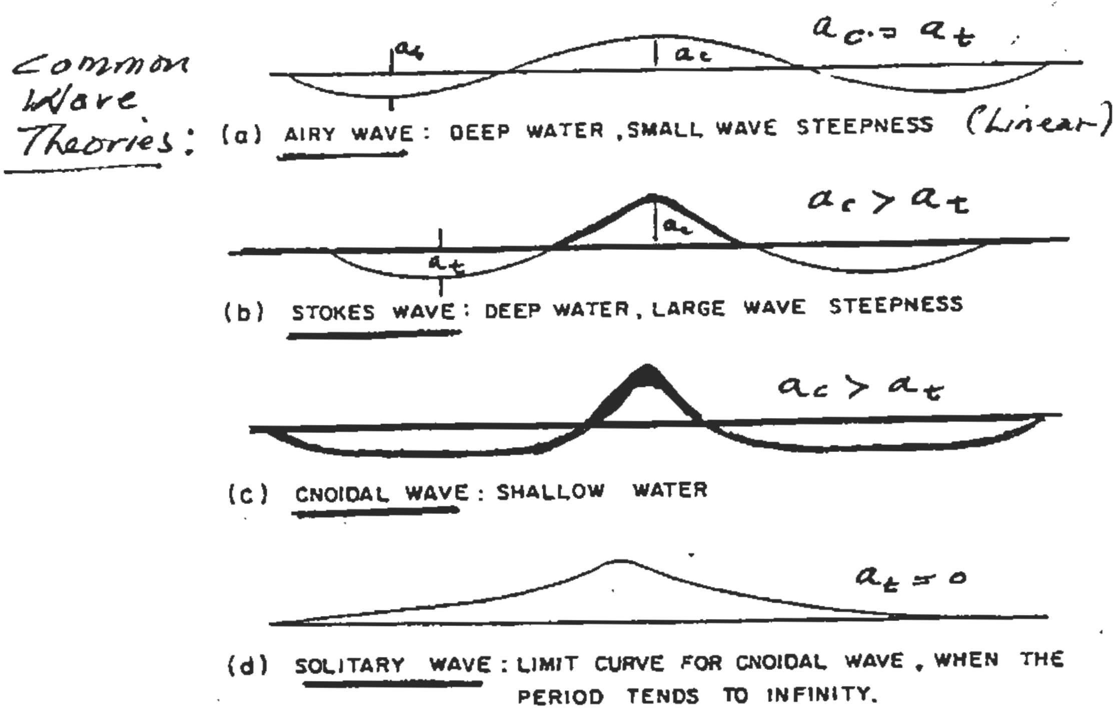
salt = 3% higher

$$\left(\frac{P}{\gamma}\right)_{min} = \left(\frac{-H \cosh\left(\frac{2\pi(z+d)}{L}\right)}{2\cosh\left(\frac{2\pi d}{L}\right)}\right) - z$$

@bed $z = -d$

Lecture 3
Non-Linear Wave Theory

Non-linear waves are waves with amplitudes that are large respect to the depth or wave length. Linear waves are symmetrical about the still water level (SWL); however, as waves become more non-linear the waves tend to be asymmetrical about the SWL. As illustrated below the crest amplitude, a_c become greater than the trough amplitude, a_t.



Rouse (1957) gave some simplified equations for estimating the celerity of non-linear waves. The approximate non-linear shallow water wave celerity is:

$$c = (g\ d)^{½} (1 + 0.75H/d)^{½} \qquad 3.1$$

The upper limit is the wave height at breaking which for a nearly horizontal bed is

$$H/d = H_b/d_b \sim 0.78 \qquad 3.2$$

This gives a shallow water correction factor in the range 1 to 1.26.
The non-linear deep water wave celerity can be approximated by

$$c = \{gL/(2\pi)[1 + (\pi H/L)^2 + (\pi H/L)^4/2 + ...]\}^{½} \qquad 3.3$$

For deep water the breaking wave criteria is,

$$H_b/Lo \leq 1/7, \qquad 3.4$$

therefore $1 \leq c/c_o \leq 1.1$

Mehaute (1969) presented a diagram to determine the applicability of various wave theories depending on dimensionless numbers (Figure 2-7 SPM):

- A non-dimensional depth = $d/(gT^2)$
- A non-dimensional wave height = $H/(gT^2) = 2\pi\{H/Lo\} = 2\pi$(Wave Steepness in Deep Water)

Another non-dimensional depth is ***H/d***. Also in some cases the Ursell number, Ur, is used for wave classification,

$$Ur = L^2H/d^3 \qquad 3.5$$

Stokes Waves theory (Wiegel) approximates non-linear waves by a finite series, e.g.

$$\eta = a\cos(\theta) + (\pi a/L)^2 B_2\cos(2\theta) + (\pi a/L)^3 B_3\cos(3\theta) + \ldots (\pi a/L)^n B_n\cos(n\theta) \qquad \ldots 3.6$$

where n represents the order of the approximation, $\theta = k(x - ct)$; $k = 2\pi/L$ *and for 3^rd^ order,*

$$B_2 = \frac{[2 + \cosh(\frac{4\pi d}{L})]\cosh(\frac{2\pi d}{L})}{2\sinh^3\frac{2\pi d}{L}}$$

$$B_3 = \frac{3[1 + 8\cosh^6(\frac{2\pi d}{L})]}{16\sinh^6\frac{2\pi d}{L}}$$

$$L = L_o\tanh(\frac{2\pi d}{L})\left[1 + (\frac{2\pi a}{L})^2\frac{[14 + 4\cosh^2(\frac{4\pi d}{L})]}{16\sinh^4\frac{2\pi d}{L}}\right] \qquad 3.7$$

$$c = L/T \qquad 3.8$$

$$H = 2a\,[1 + (\pi a/L)^2 B_3]\ a \qquad 3.9$$

Graduate Student Problem: Compare the wave shape, c and L for linear and 3rd order theory for the following data: H = 1 m, T = 5 s, d = 4 m.

The SPM gives an implicit approximation for 3rd Stokes waves:

COE

Approximate Stokes 3 (SPM)

The ...

$$C = \frac{gT}{2\pi} \tanh\left(\frac{2\pi d}{L}\right)\left\{1 + \left(\frac{\pi H}{L}\right)^2 \left[\frac{5 + 2\cosh(4\pi d/L) + 2\cosh^2(4\pi d/L)}{8\sinh^4(2\pi d/L)}\right]\right\} \quad (2\text{-}47)$$

and

$$L = \frac{gT^2}{2\pi} \tanh\left(\frac{2\pi d}{L}\right)\left\{1 + \left(\frac{\pi H}{L}\right)^2 \left[\frac{5 + 2\cosh(4\pi d/L) + 2\cosh^2(4\pi d/L)}{8\sinh^4(2\pi d/L)}\right]\right\} \quad (2\text{-}48)$$

The equation of the free surface for second-order theory is

$$\eta = \frac{H}{2}\cos\left(\frac{2\pi x}{L} - \frac{2\pi t}{T}\right) + \left(\frac{\pi H^2}{8L}\right)\frac{\cosh(2\pi d/L)}{\sinh^3(2\pi d/L)}\left[2 + \cosh(4\pi d/L)\right]\cos\left(\frac{4\pi x}{L} - \frac{4\pi t}{T}\right) \quad (2\text{-}49)$$

For deep water, (d/L > 1/2) equation (2-49) becomes,

$$\eta = \frac{H_o}{2}\cos\left(\frac{2\pi x}{L_o} - \frac{2\pi t}{T}\right) + \frac{\pi H_o^2}{4L_o}\cos\left(\frac{4\pi x}{L_o} - \frac{4\pi t}{T}\right) \quad (2\text{-}50)$$

b. Water Particle Velocities and Displacements. The periodic x and z components of the water particle velocities to the second order are given by

$$u = \frac{HgT}{2L}\frac{\cosh[2\pi(z + d)/L]}{\cosh(2\pi d/L)}\cos\left(\frac{2\pi x}{L} - \frac{2\pi t}{T}\right) + \frac{3}{4}\left(\frac{\pi H}{L}\right)^2 C\,\frac{\cosh[4\pi(z + d)/L]}{\sinh^4(2\pi d/L)}\cos\left(\frac{4\pi x}{L} - \frac{4\pi t}{T}\right) \quad (2\text{-}51)$$

$$w = \frac{\pi H}{L} C\,\frac{\sinh[2\pi(z + d)/L]}{\sinh(2\pi d/L)}\sin\left(\frac{2\pi x}{L} - \frac{2\pi t}{T}\right) + \frac{3}{4}\left(\frac{\pi H}{L}\right)^2 C\,\frac{\sinh[4\pi(z + d)/L]}{\sinh^4(2\pi d/L)}\sin\left(\frac{4\pi x}{L} - \frac{4\pi t}{T}\right) \quad (2\text{-}52)$$

Second-... particle displacements from their mean

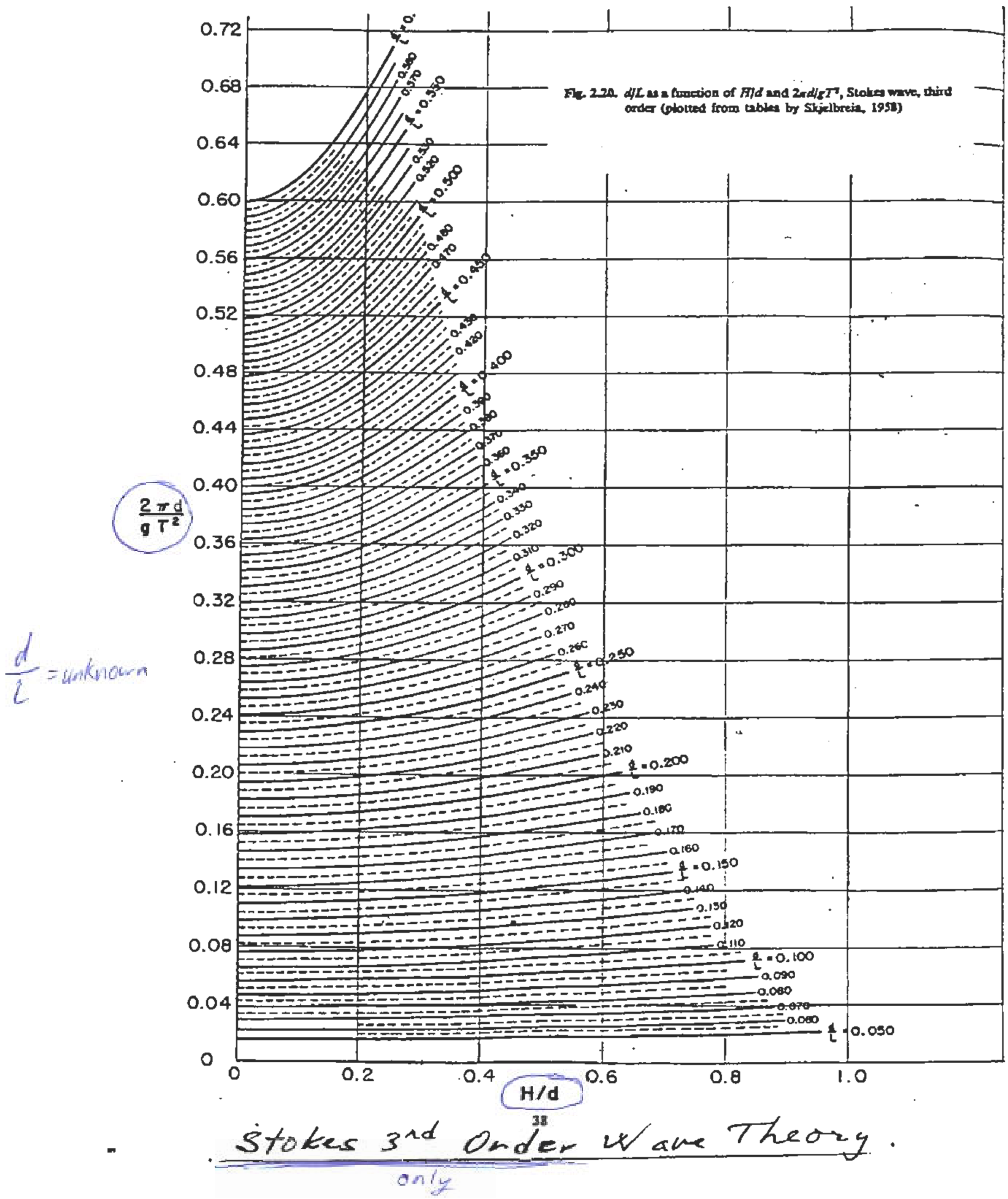


Figure 3-S-3 From R. Weigel *Ocean Engineering*

indicated on the Classification Graph of Mehaute (Figure 2.7) there is a range of shallow and ermediate waves between the linear zone and the breaker limit, that can be describe by **oidal Wave** theory. See CEM p. II-1-38-41. The following equations are from the CEM and attached charts are from the SPM. The position of the free surface above the bed is y_s and is ven by,

$$y_s = y_t + H\, cn^2 \left[2K(k) \left(\frac{x}{L} - \frac{t}{T} \right) , k \right] \qquad 3.10$$

where

y_t = distance from the bottom to the wave trough

H = trough to crest wave height

cn = elliptic cosine function

$K(k)$ = complete elliptic integral of the first kind

k = modulus of the elliptic integrals

$$L = \sqrt{\frac{16d^3}{3H}}\; k\, K(k) \qquad 3.11$$

and wave period by

$$T\sqrt{\frac{g}{d}} = \sqrt{\frac{16 y_t}{3H}\frac{d}{y_t}} \left[\frac{k\, K(k)}{1 + \frac{H}{y_t k^2}\left(\frac{1}{2} - \frac{E(k)}{K(k)} \right)} \right] \qquad 3.12$$

$$p \approx \rho g\, (y_s - y) \qquad 3.13$$

Problem: Compare the Linear and Cnoidal wave shape, L and c for the following data: H = 2 m, d = 4 m and T = 10 s.

As i
inte
Cn
the
gi

shape parameter

Cnoidal Procedure:

Step 1. Using H/d and $T(g/d)^{1/2}$ and Figure 2.11 (SPM) find k^2

step 1
given H, T, d Find k^2

Lines = H/d

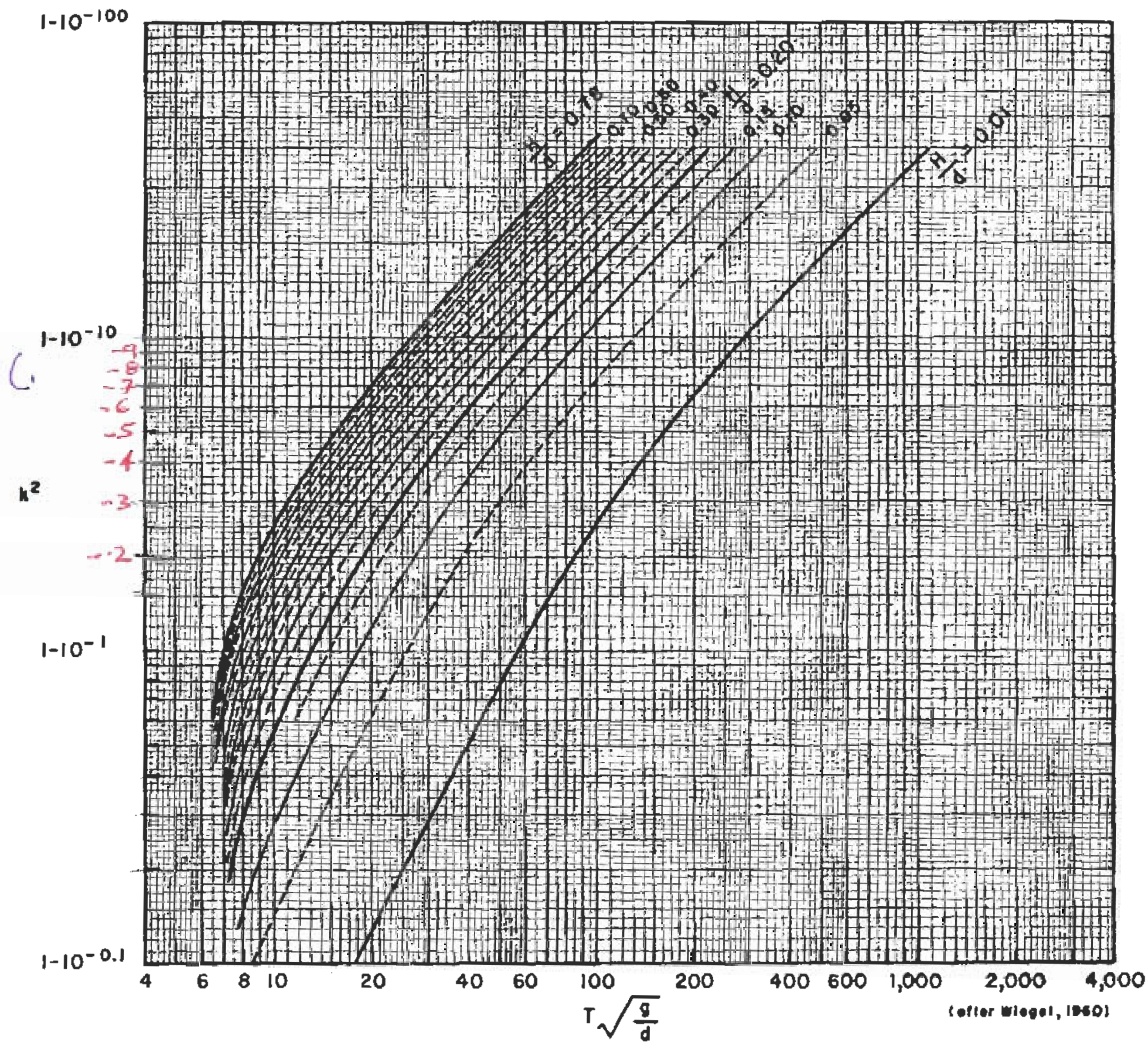


Figure 2-11. Relationship Between k^2, H/d and $T\sqrt{g/d}$

Note: H/d → 0.78 represents the wave breaking limit.

Step 2. From Figure 2-12 and find [3], obtain $L^2 H/d^3$; then compute L.

w/ K^2 find L

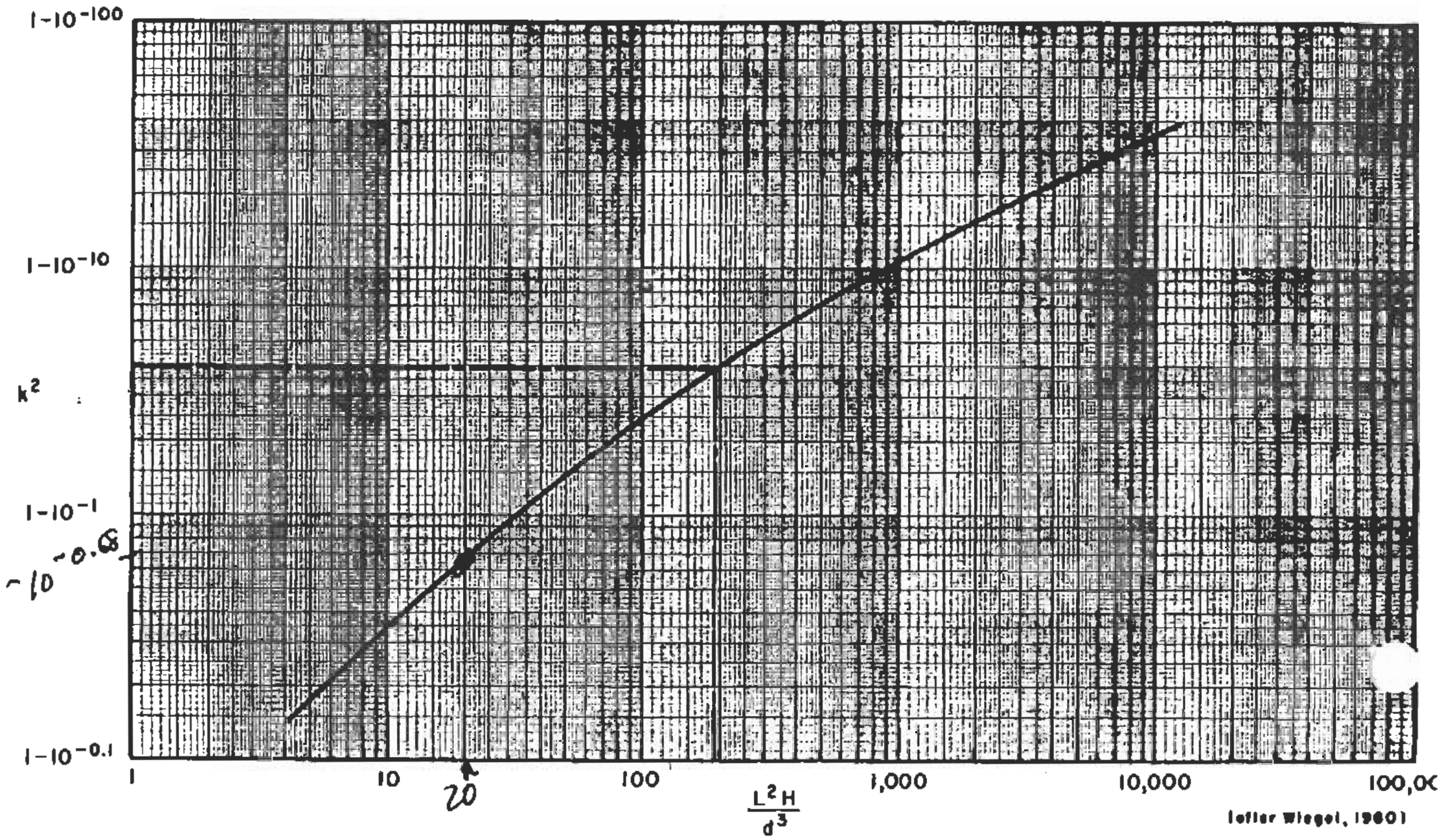


Figure 2-12. Relationships Between k^2 and $L^2 H/d^3$

Step 3: Using Figure 2-9 with k^2 determine the wave shape.

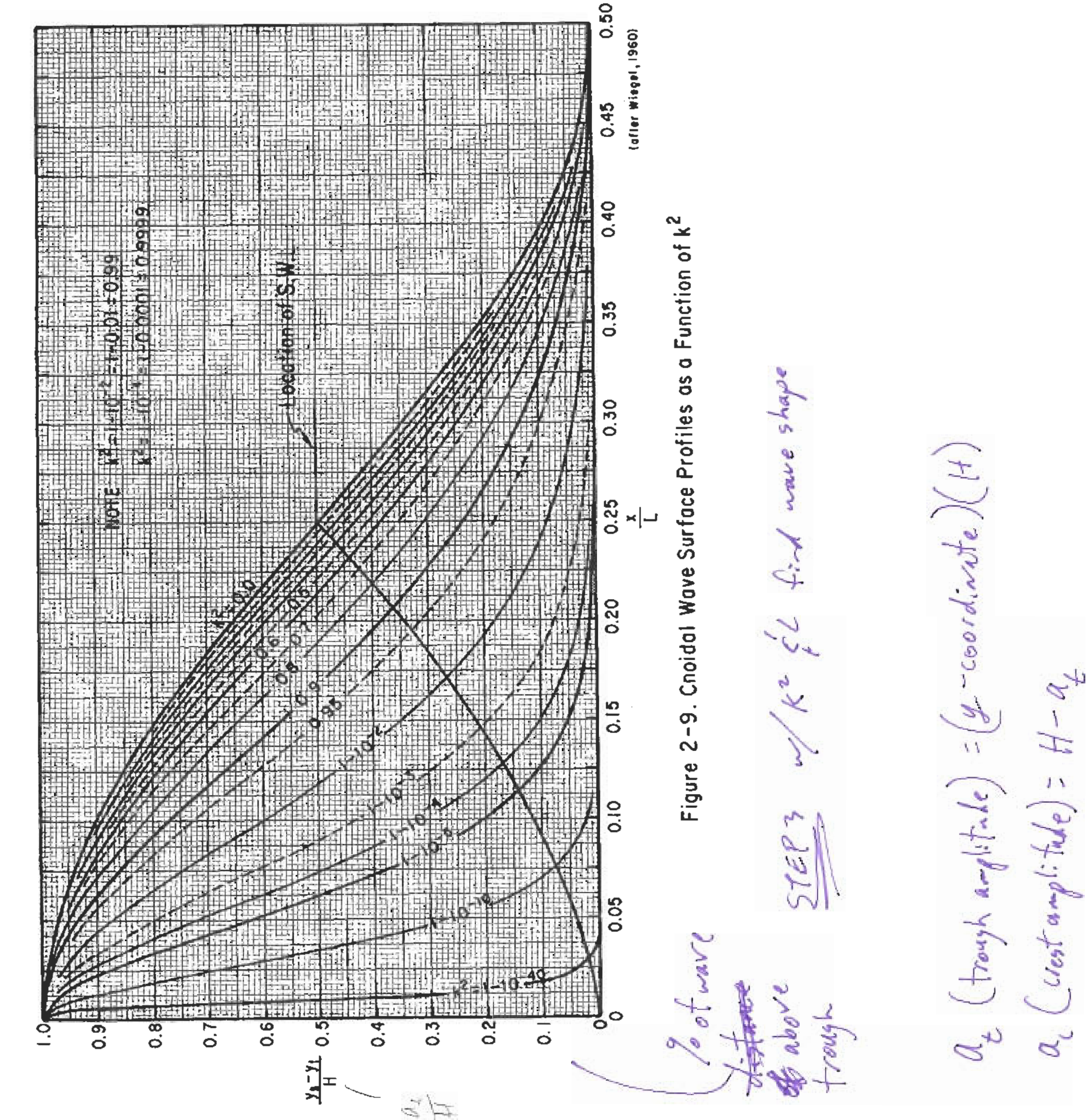


Figure 2-9. Cnoidal Wave Surface Profiles as a Function of k^2

Note: y_s = y to surface and y_t= y to the trough of the wave3. The line starting at 0,0 and ending at (0.25,0.58) is the $(y_s-y_t)/H$ for the SWL.

The **Solitary Wave** theory applies to a single wave which travels on top of the SWL. It occurs in shallow water. It is a translatory wave, i.e. the particle orbits are not close. In fact the motion is entirely in the direction of the wave. The wave length and period are not defined (theoretically they are infinite).

The free surface of the solitary wave is given by,

$$\eta = H \operatorname{sech}^2 \{[3H/(4d^3)]^{1/2} (x - ct)\} \qquad 3.14$$

where sech(-) is the hyperbolic secant = 1/cosh(-), and

$$c \sim \{g(H + d)\}^{1/2} \qquad 3.15$$

The total volume in a solitary wave is

$$Vol = [16d^3H/3]^{1/2} \qquad 3.16$$

The horizontal and vertical velocities (u,v) are given by (Munk, CEM p. II-1-48),

$$u = cN \frac{1 + \cos(\frac{My}{d})\cosh(\frac{Mx}{d})}{[\cos(\frac{My}{d}) + \cosh(\frac{Mx}{d})]^2} \qquad 3.17$$

$$v = cN \frac{1 + \sin(\frac{My}{d})\sinh(\frac{Mx}{d})}{[\cos(\frac{My}{d}) + \cosh(\frac{Mx}{d})]^2} \qquad 3.18$$

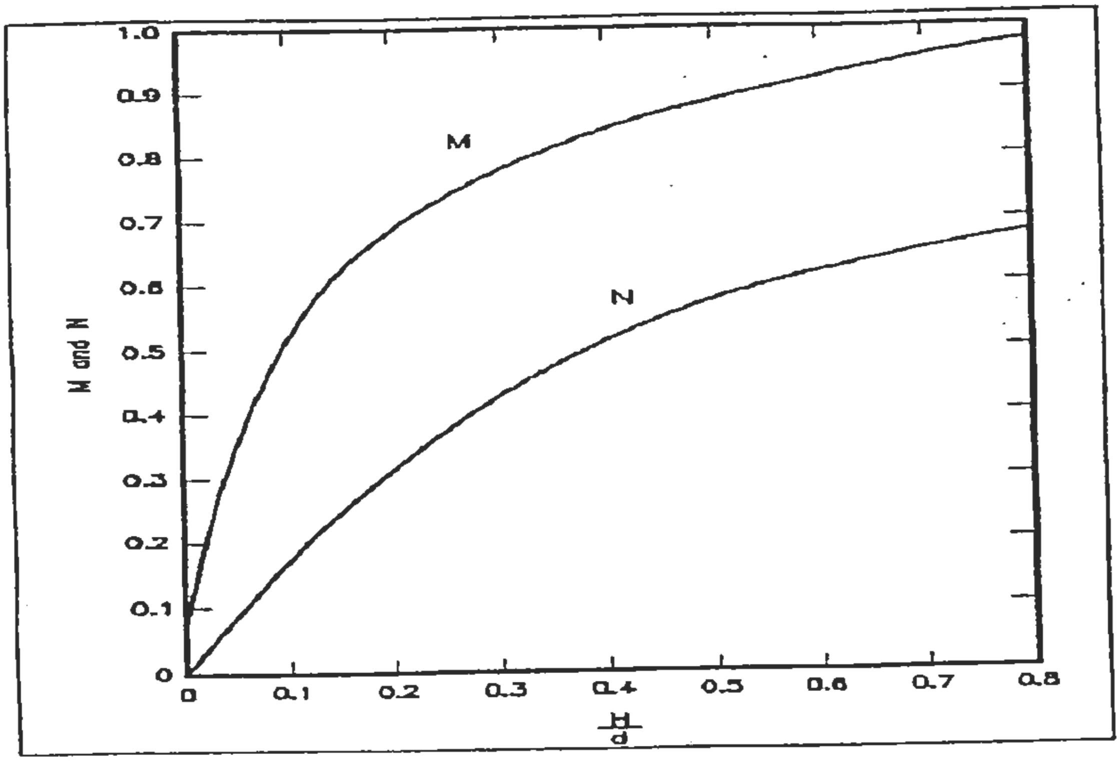
and the maximum horizontal velocity is

$$u_{max} = \frac{cN}{1 + \cos(\frac{My}{d})} \qquad 3.19$$

where M and N are given in Figure II-1-17 and y is measured from the bottom.

The energy for this wave is $E = \frac{8}{3\sqrt{3}} \gamma . H^{3/2} d^{3/2}$

use 4 max velocity $U_{max} = \dfrac{cN}{1 + \cos\left(\frac{My}{d}\right)}$



y is measured from bed

Figure II-1-17. Functions M and N in solitary wave theory (Munk 1949)

Wave Breaking: The solitary wave has a limiting height after which it tends to become unstable and break. Some researchers (SPM) have given this to be,

$$H_b/d_b \sim 0.78 \qquad 3.20$$

However, this has been shown to depend on the bed slope, m, e.g.

$$H_b/d_b = 0.75 + 0.25m - 112m^2 + 3870\,m^3 \qquad 3.21$$

In deep water the depth is no longer important but theoretical studies supported by experiments show that the limiting steepness for deep water waves is

$$H_{ob}/Lo = 0.142 \sim 1/7 \qquad 3.22$$

which corresponds to a crest or cusp angle of 120 °.

For intermediate waves Miche (SPM) gave,

$$(H/L)max = (H_{ob}/Lo)\tanh(2\pi d/L) \qquad 3.23$$

Figure 15-5 shows five types of breaking waves,

- Deep water "white caps"; crest steeping
- Spilling Breaker, due to a long shallow wave on a gentle slope
- plunging breaker
- surging breaker
- tidal bore

Figure 2-65 (SPM) shows the relationship of the breaker height and type to the bed slope (m) and the deep water wave steepness H_o/L_o. Figure 2-66 show how the breaker depth to height changes with bed slope. As the bed slope increases the ratio d_b/H_b decreases.

Assignment: Given a deep water H_o = 1.5 m approaching a beach with a 2% slope. Estimate H_b and d_b. What type of breaker is expected?

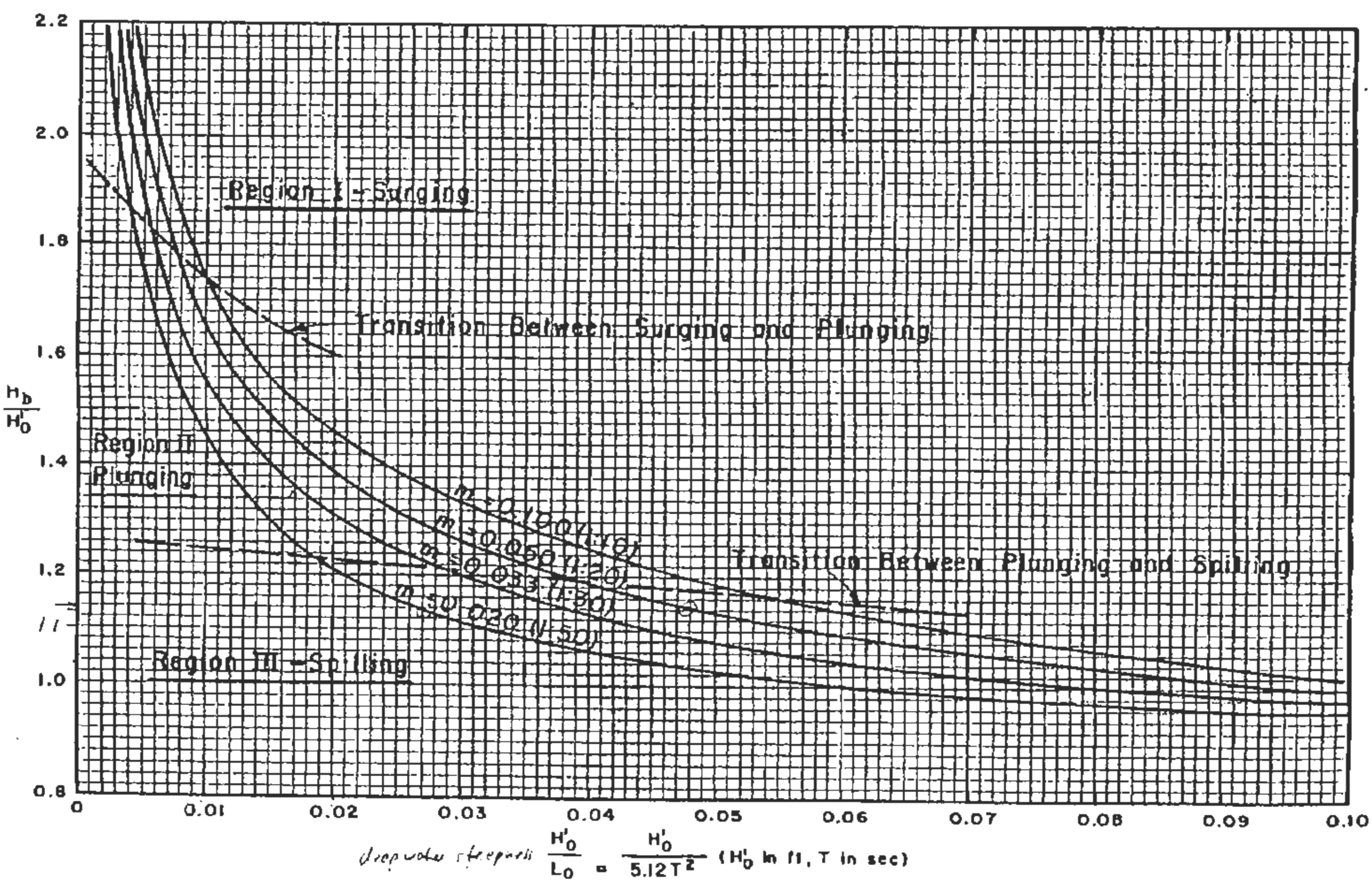


Figure 2-65. Breaker Height Index Versus Deep Water Wave Steepness (SPM)

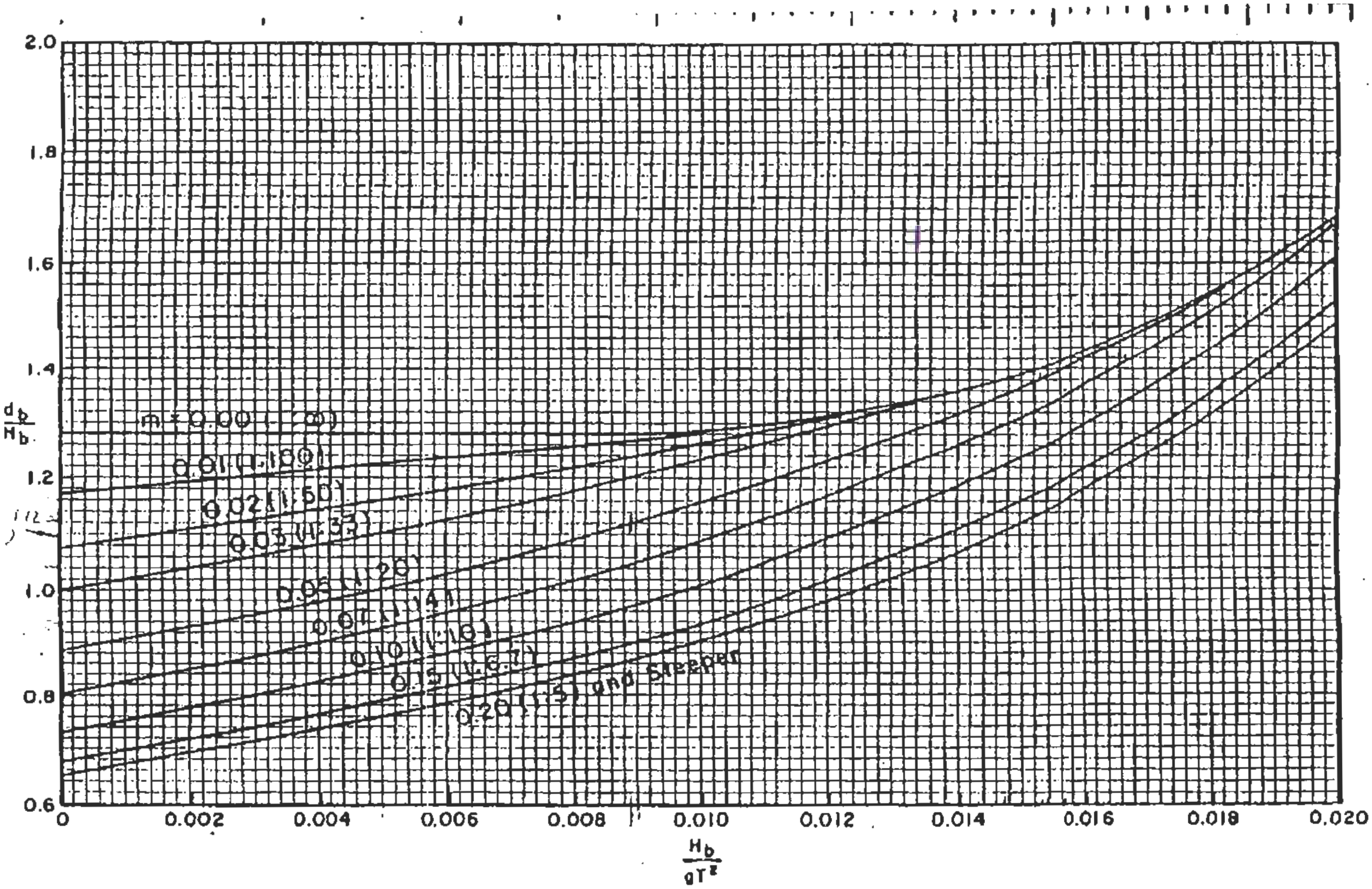


Figure 2-66. Dimensionless Depth at Breaking Versus Breaker Steepness (SPM)

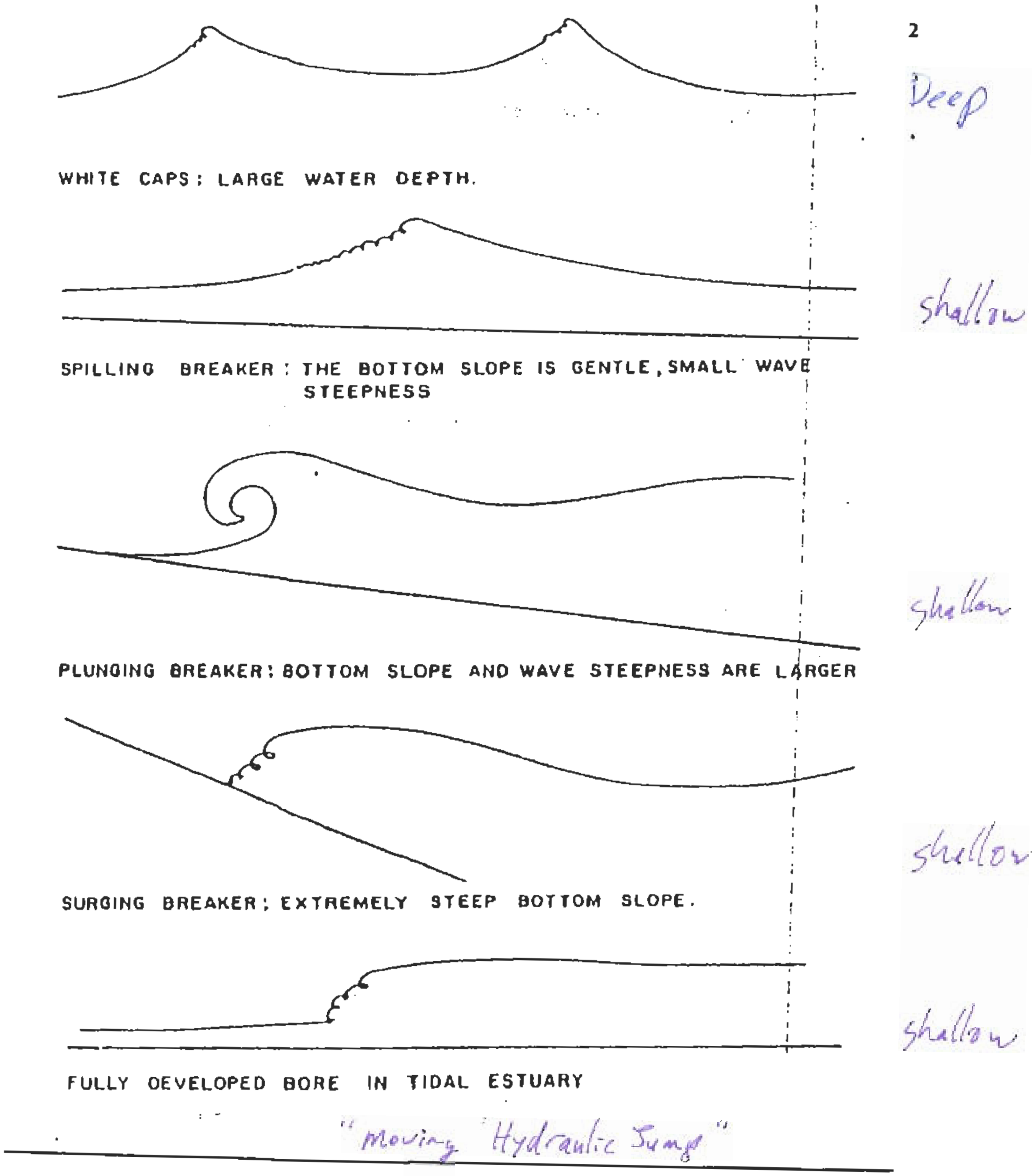


Figure `5-5

Fig 2-65 To get H_b from H_o & T Also type of Breaker

djerolle

Machine Name: w14955

Date: 02/24/2010

Job: 308

Time: 5:00:18 PM

Cost: 0.00

Wave Generation

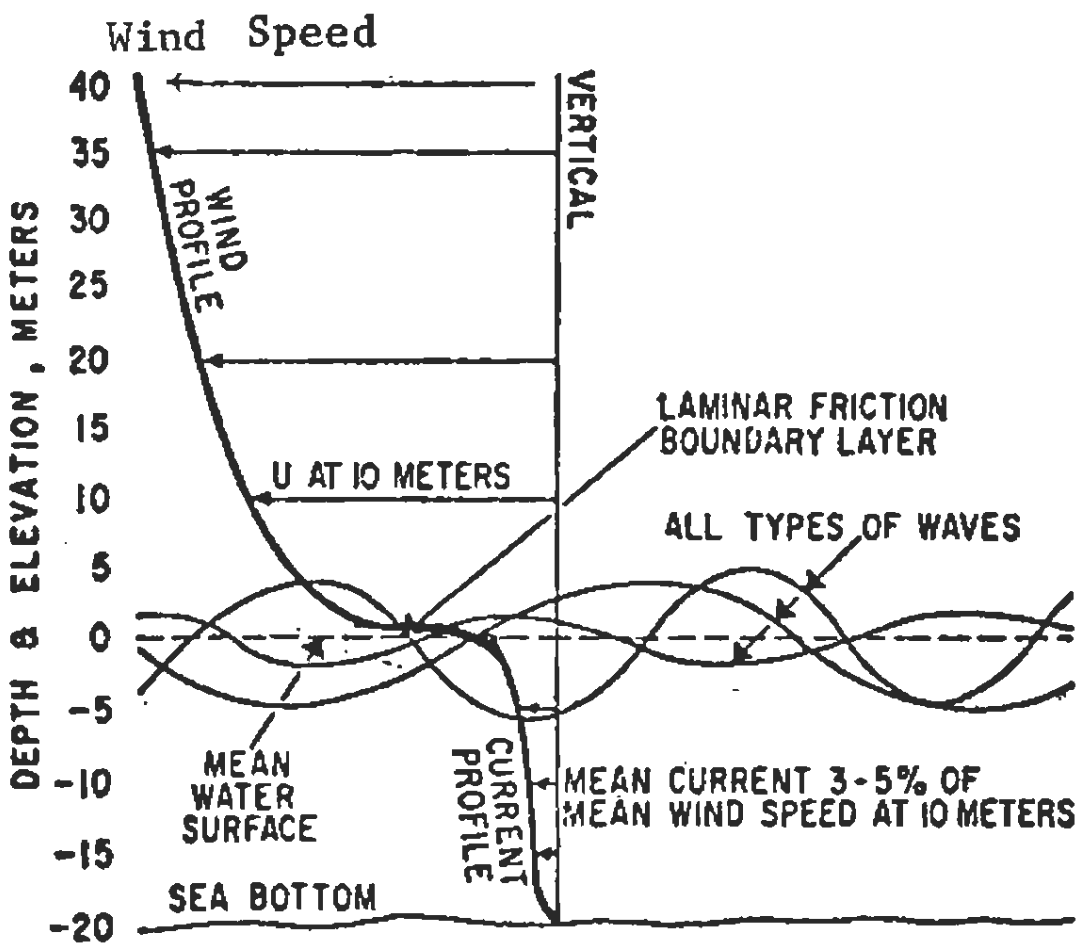
Shore Protection Manual Method

Non-Hurricane Waves

Intermediate & Deep Water

- Factors:
  - Wind speed at 10m over water (Uow)
  - Fetch – clear distance over which wind blows (F)
  - Average water depth over fetch (d)
  - Duration of sustained wind (t_d)
  - Atmospheric stability (air, water temperature) (R_T)
  - Wind shear effect (momentum transfer increases as wave H increases) (U_A)

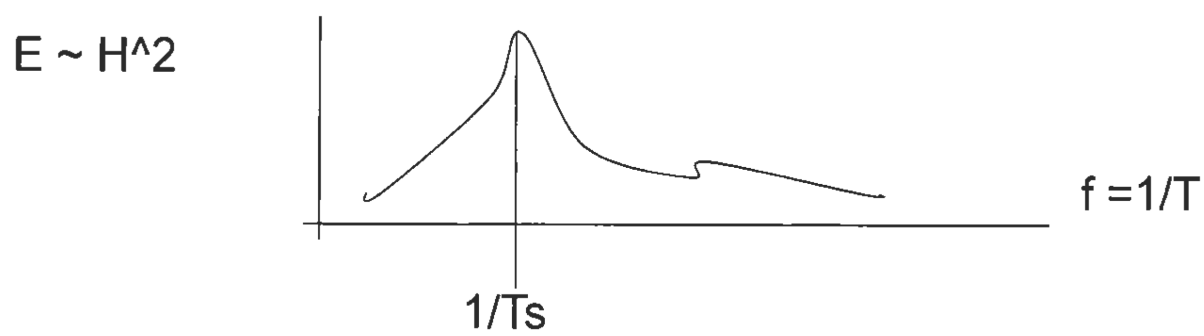
Wind Speed and Water Current



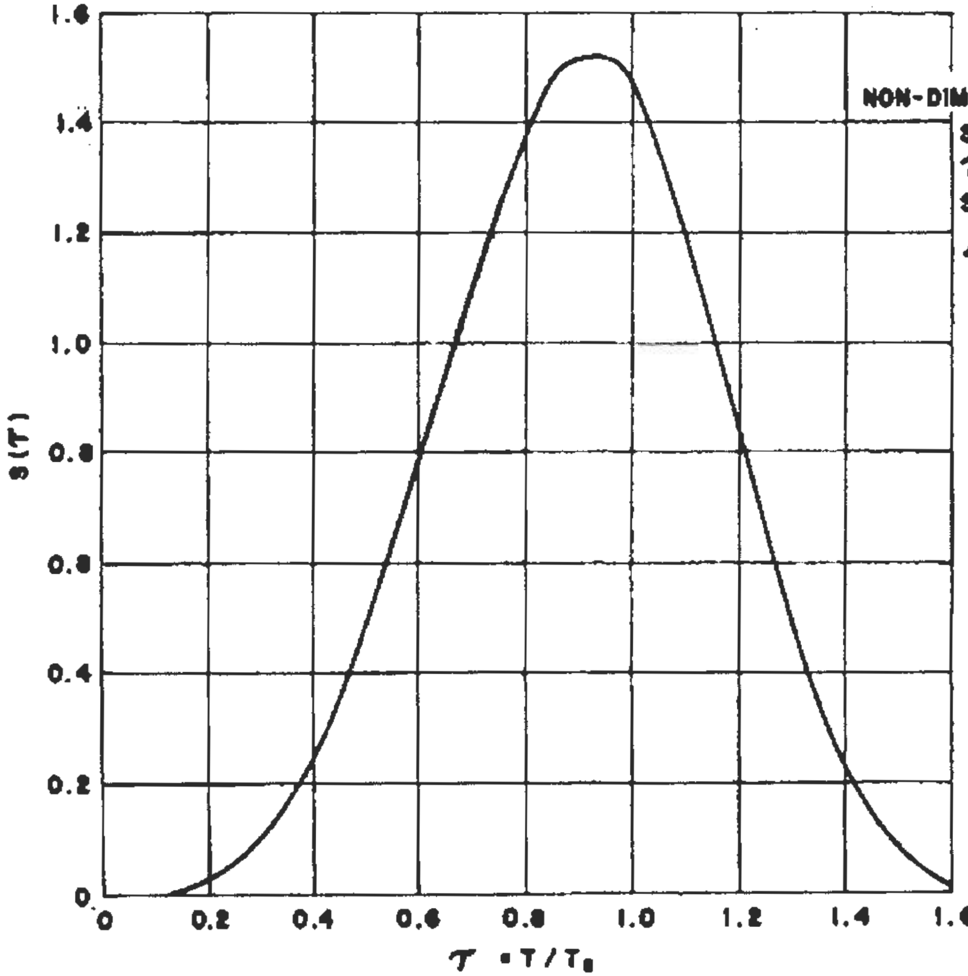
Wave Forecasting Equations

Definitions:

- H_s = significant wave height = average of highest 1/3 of waves in a design storm
- T_s = significant period or period of highest 1/3 of the waves



Wave period spectrum



NON-DIMENSIONAL PERIOD SPECTRUM

$S(\tau) = 4\tau^{-5} e^{-\tau^{-4}}$

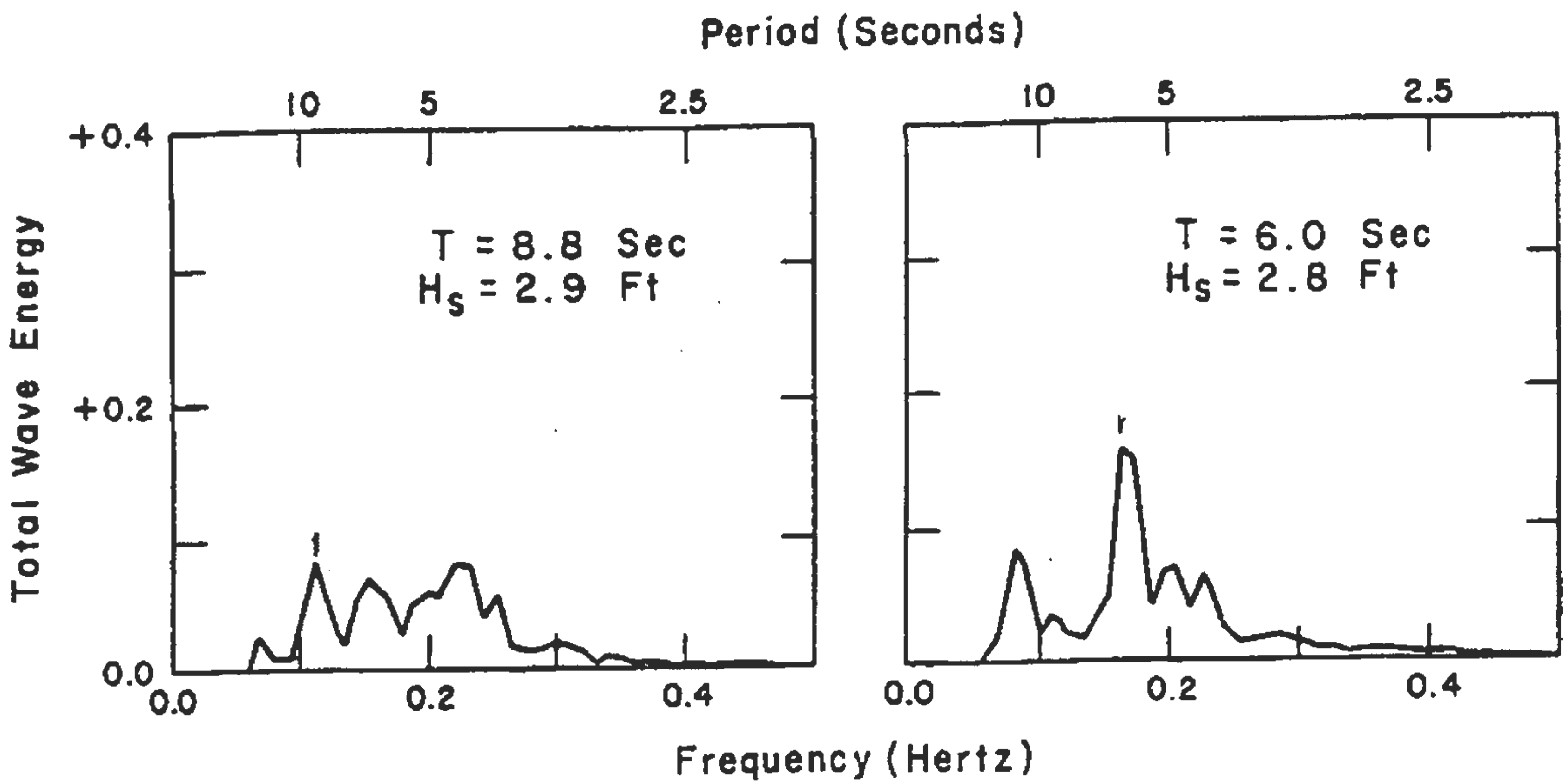
$\tau = T/T_s$; $\int S(\tau)d\tau = 1.0$

$S(T) = H_s^2/T_s\,[S(\tau)]$

$\int S(T)dT = H_s^2$

$\tau = T/T_s$	$S(\tau)$
0.1000	0.003999
0.2000	0.031948
0.3000	0.107128
0.4000	0.249529
0.5000	0.469705
0.6000	0.758977
0.7000	1.079143
0.8000	1.359698
0.9000	1.513027
0.9306	1.522771
1.0000	1.471517
1.1000	1.231369
1.2000	0.869067
1.3000	0.506251
1.4000	0.235541
1.5000	0.085453
1.6000	0.023347
1.7000	0.004636
1.8000	0.000643
1.9000	0.000060
2.0000	0.000003

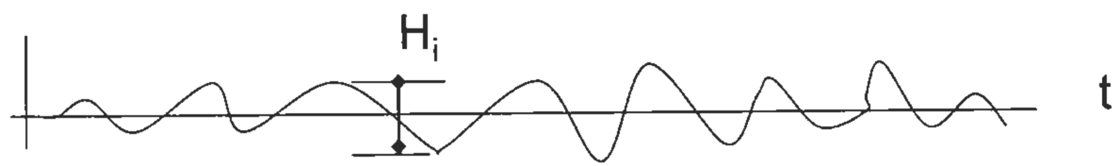
Fig. 2 Non-dimensional period spectrum



Savannah Coast Guard Light Tower

Wave Distribution

- Rayleigh distribution
- $P(H>X) = EXP[-(H/H_{rms})^2]$
- $P_{av}(H_{av}>X)$ = Probability for average of higher waves than P (see Figure 3-3)



- $H_{rms} = \{\Sigma H_i^2/n\}^{1/2}$
- $H_s = 2^{1/2}\, H_{rms}$
- $H_{10\%} = 1.27H_s$
- $H_{1\%} = 1.67H_s$

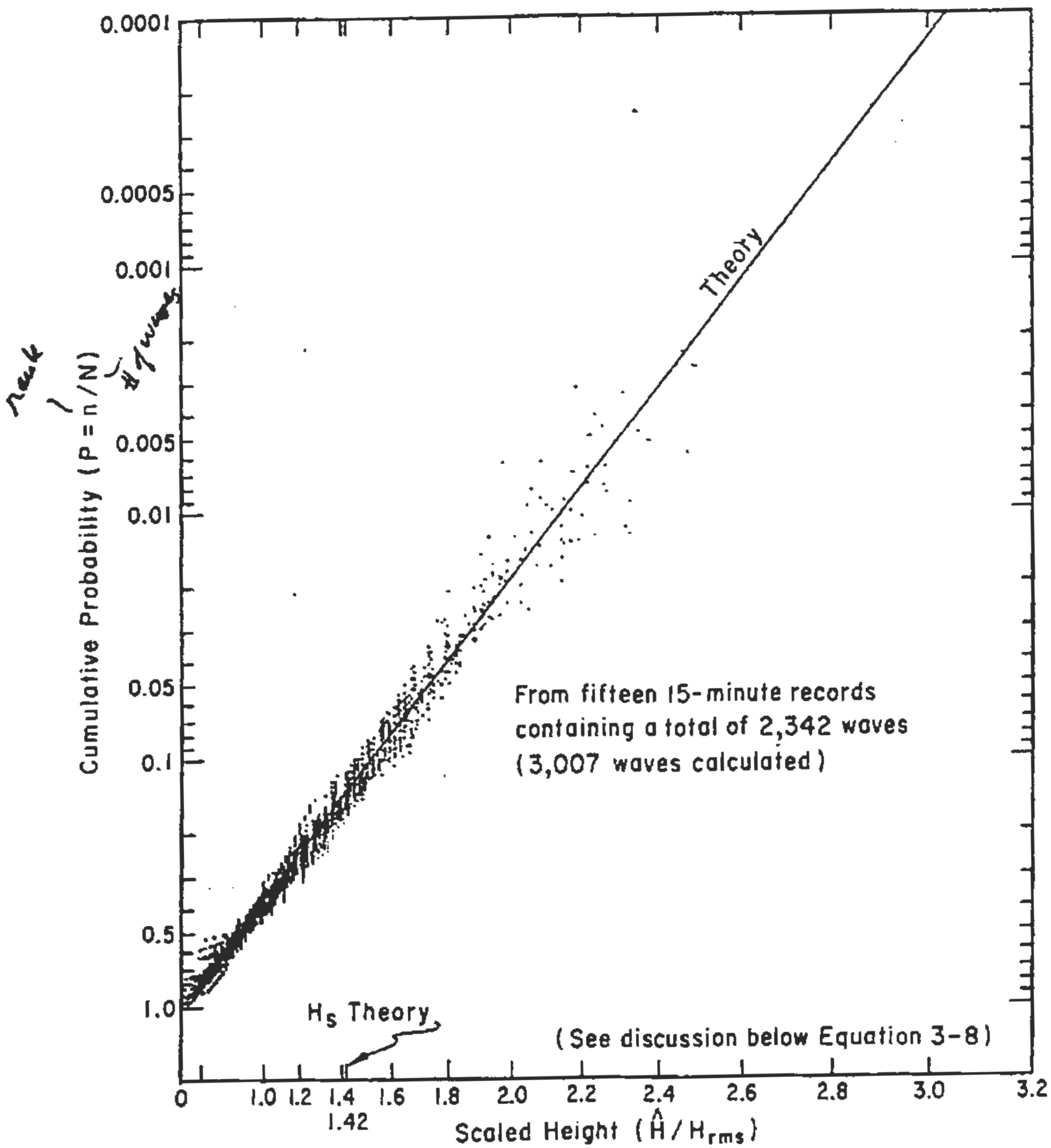


Figure 3-3. Theoretical and Observed Wave-Height Distributions. Observed distributions for 15 individual 15-minute observations from several Atlantic coast wave gages are superimposed on the Rayleigh distribution curve

Rayleigh Distribution

$$P(H > \hat{H}) = e^{-\left(\frac{H}{Hrms}\right)^2}$$

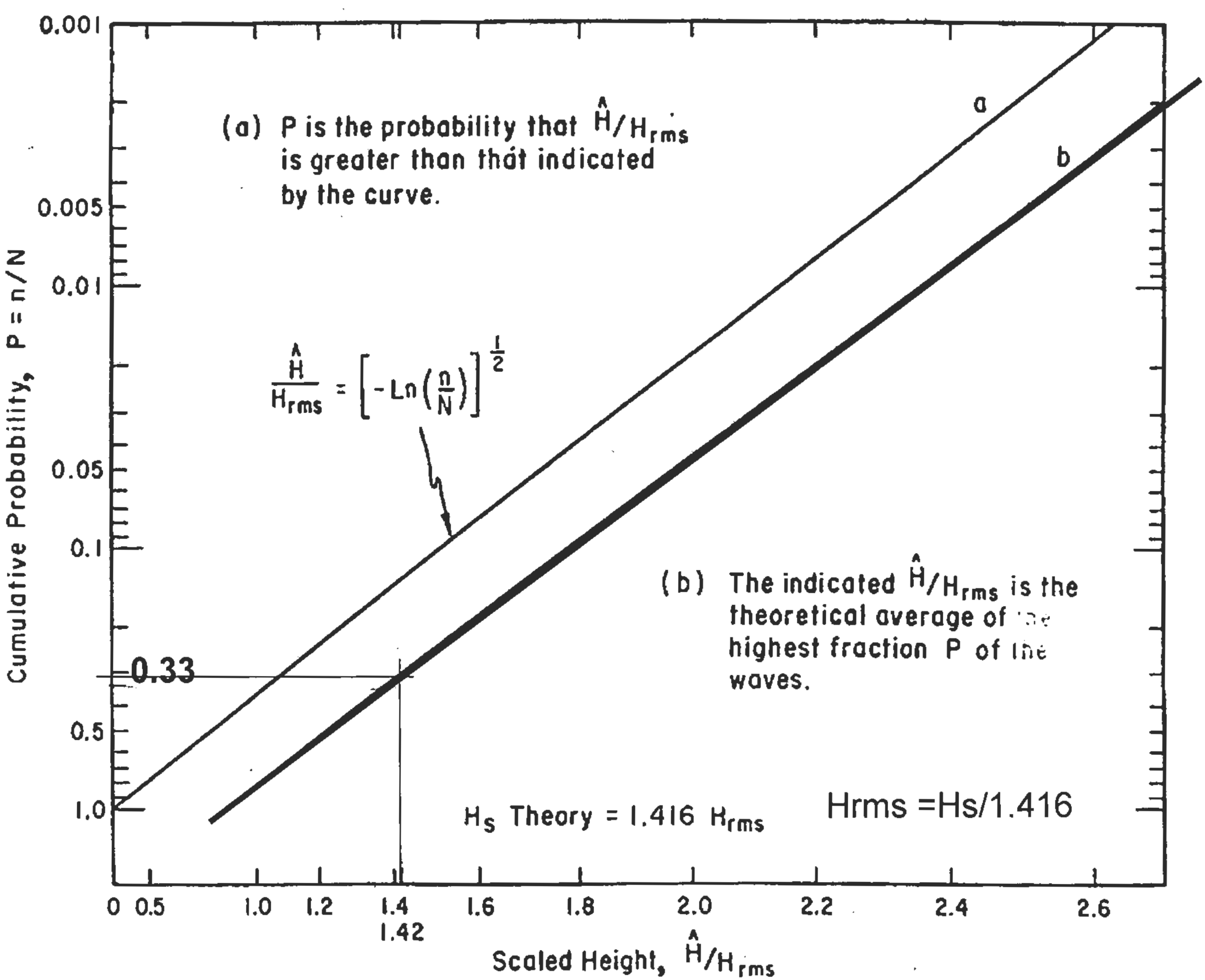


Figure 3-5. Theoretical wave height distributions.

Dimensional Analysis

- **$H_s = f(g, d, U_A, F, t_d)$**
  - : d = average depth over F
  - : U_A = effective wind speed over F
  - : t_d = duration of wind
- **T_s** = ⓁⒼ(g, d, U_A, F, t_d)
- **$gH_s/U_A^2 = fcn(gF'/U_A^2, gd/U_A^2, gt_d/U_A)$**
  - See Figure 3.21
- **$gT_s/U_A = Fcn(gF'/U_A^2, gd/U_A^2, gt_d/U_A)$**
  - See Figure 3.22

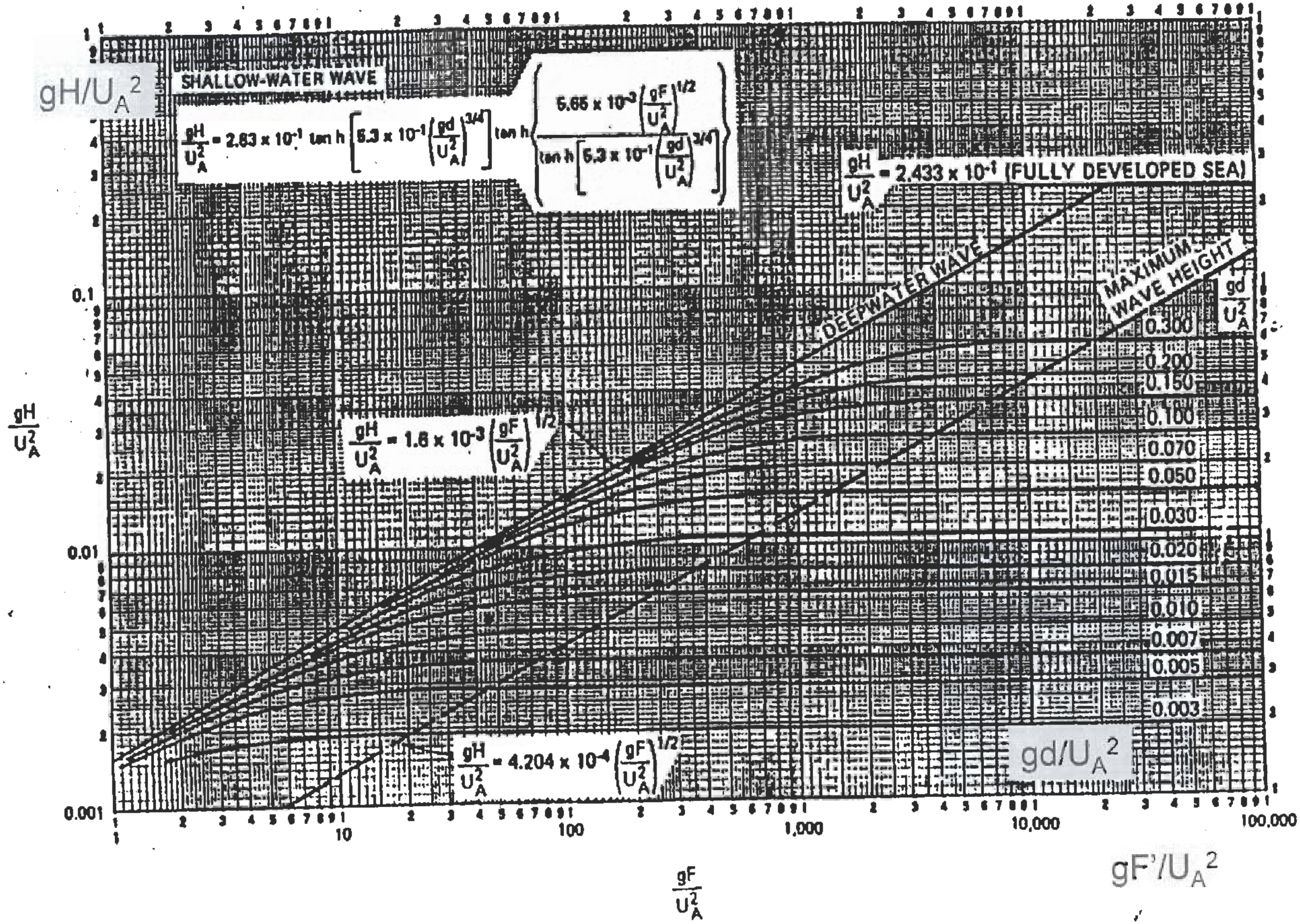


Figure 3-21. Forecasting curves for wave height. Constant water depth.

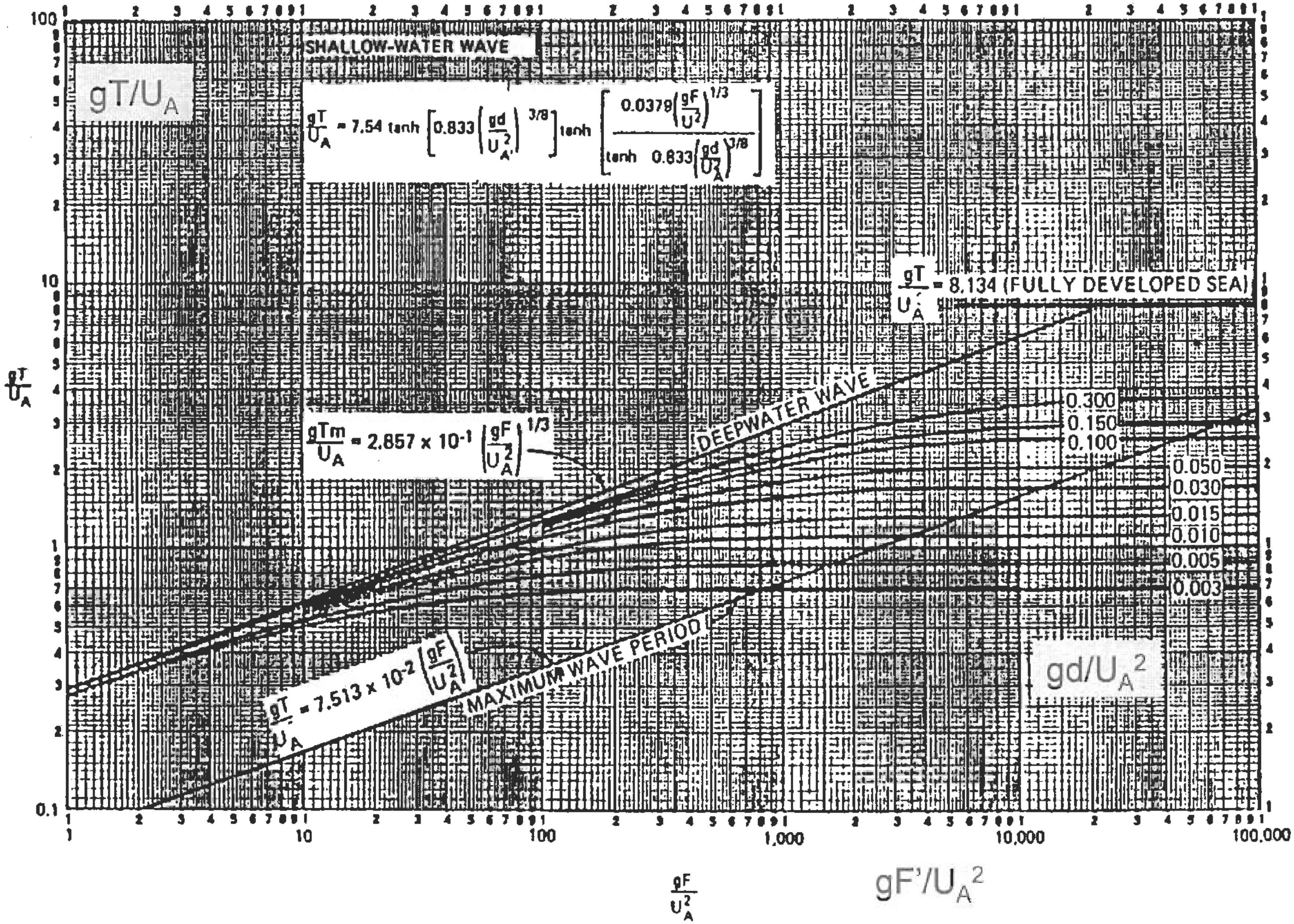
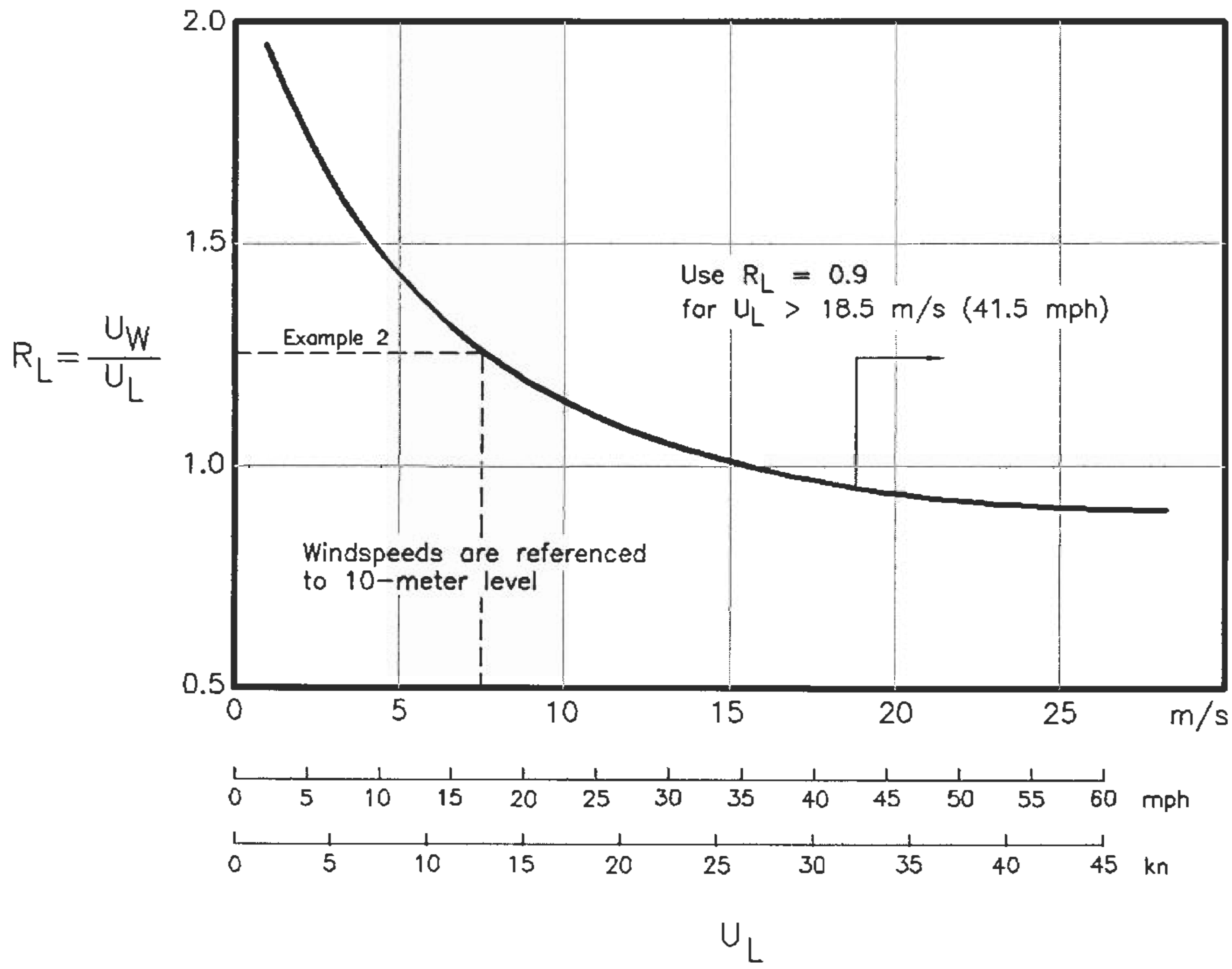


Figure 3-22. Forecasting curves for wave period. Constant water depth.

Shear Corrected Wind U_A

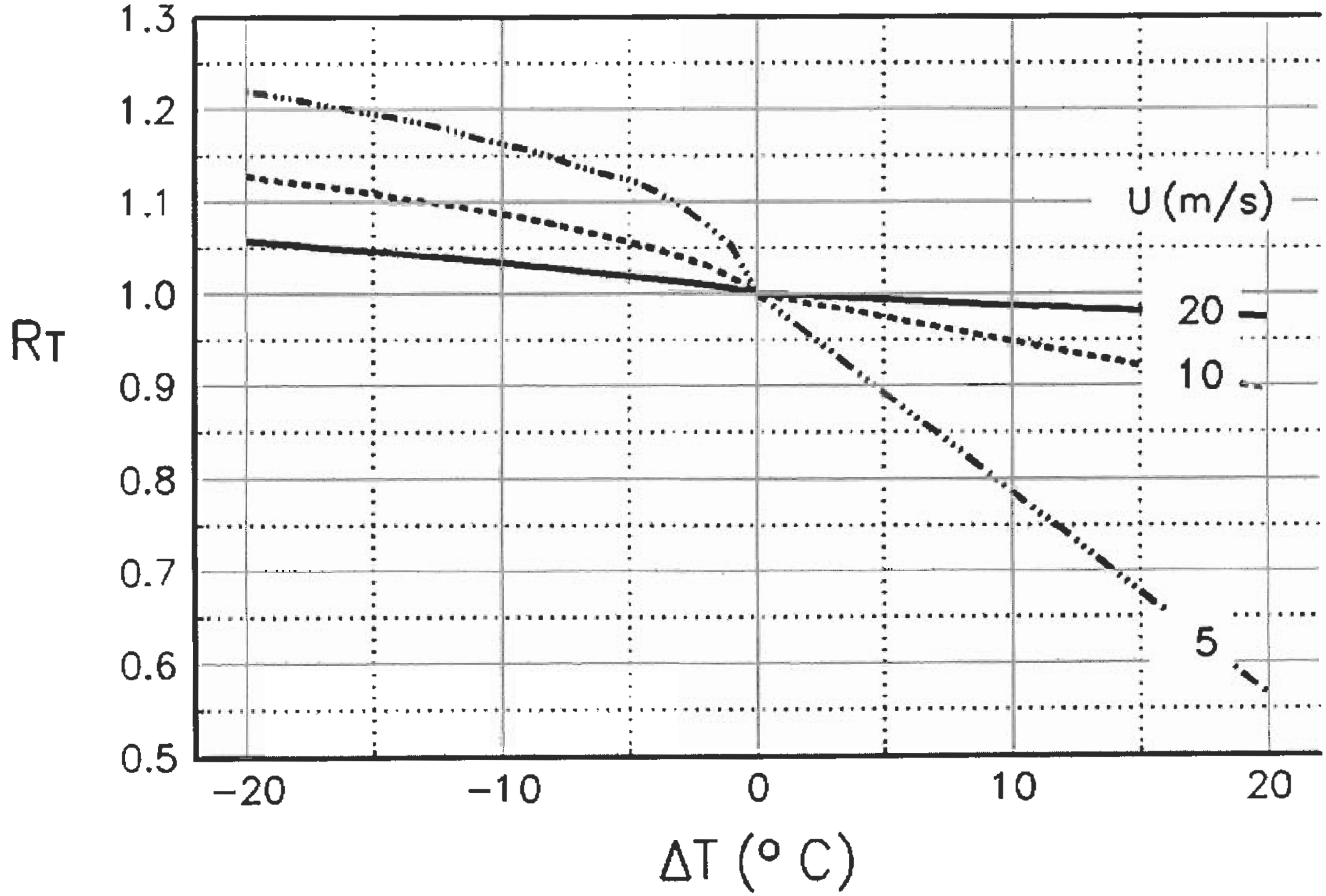
- **$U_A = C_w\{U_{oL} R_L\ R_T,\}^{1.23}$**
  - : $R_L = U_{ow}/U_{oL}$ = over water correction Fig 3.14
  - : R_T= atmospheric stability correction Figure 3.15
  - : C_w= 0.71 for SI
  - : C_w = 0.589 for U in mph
  - : C_w = 0.539 for U in ft/sec

Uow/UoL



$R_L = \frac{U_W}{U_L}$
2.0
1.5
1.0
0.5
0
5
10
15
20
25
m/s
Example 2
Use R_L = 0.9
for U_L > 18.5 m/s (41.5 mph)
Windspeeds are referenced
to 10-meter level
0 5 10 15 20 25 30 35 40 45 50 55 60 mph
0 5 10 15 20 25 30 35 40 45 kn
U_L

Atmospheric Instability Factor



Fetch and Duration

- F' = Effective Fetch which can be limited by physical topography or wind duration or structure of the wind field (e.g. hurricanes)
- $F' = \mathrm{Min}(F_a, F'_{duration\ limited})$
- Deep water case:
- **$gt_d/U_A = 68.8\,(gF'/U_A^2)^{2/3}$**
- Shallow water case:
- **$gt_d/U_A = 537\,(gT_m/U_A)^{7/3}$**

Deep water case

- $gF'/U_A^2 = \{(gt_d/U_A)/68.8\}^{3/2}$
- t_d in seconds
- Enter Figure 3.21 to get H_s

Shallow water case:

$gT_m/U_A = \{(gt_d/U_A)/ 537\}^{3/7}$

- Enter Fig 3.22 to get gF'/U_A^2
- Then use gF'/U_A^2 to get H_s
- $T_s = gU_A \{(gt_d/U_A)/ 537\}^{3/7}$

Wave Generation

- **Water Temperature T_{sw} = 33 ° C**
- **T_a = 28 ° C**
- **U_{ow} = 60 mph**
- **d = 14 ft**
- **F_a = actual fetch = 25 miles**
- **t_d = 1 hour**
- **Find H_s and T_s**

d	14			
Fa	25	mi	132000	ft
Tws	33	oC		
Ta	28	oC		
ΔT	-5	oC		
td	1	hour	3600	sec
UoL				
Uow	60	mph	88	ft/sec

Wave Calculations Part 1.

- Enter Fig 3.21 FF and intersect Fd
- Read $FHs = gH_s/U_A^2$
- $H_s = U_A^2 * FHs/g$
- Enter Fig 3.22 FF and intersect Fd to Read $FTs = gH_s/U_A$
- $T_s = U_A * FTs/g$

Solution

- $\Delta T = T_a - T_{ws} = -5\ ^oC$
- $R_L = 1$ Fig 3.14
- $R_T = 1.02$ Fig 3.15
- $U' = R_L * R_T * UoL = 1.02*60$
- $U_A = 0.589(U'mph)^{1.23} =$
- U_A ft/sec = U_A mph*5280/3600 =
- $Fd = g\ d/U_A^2 =$
- $FF = g\ F/U_A^2 =$
- $Ftd = gt_d/U_A =$

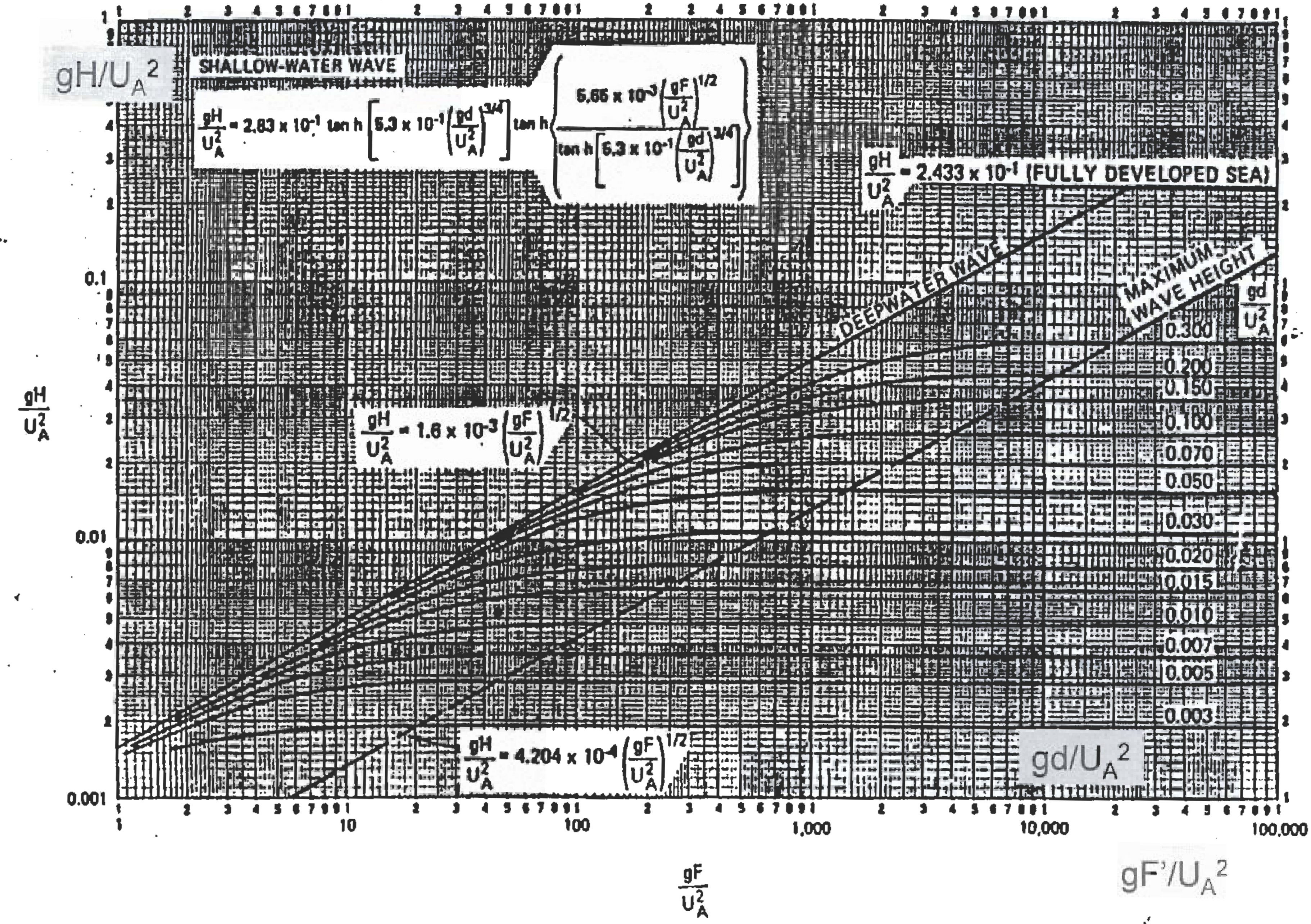


Figure 3-21. Forecasting curves for wave height. Constant water depth.

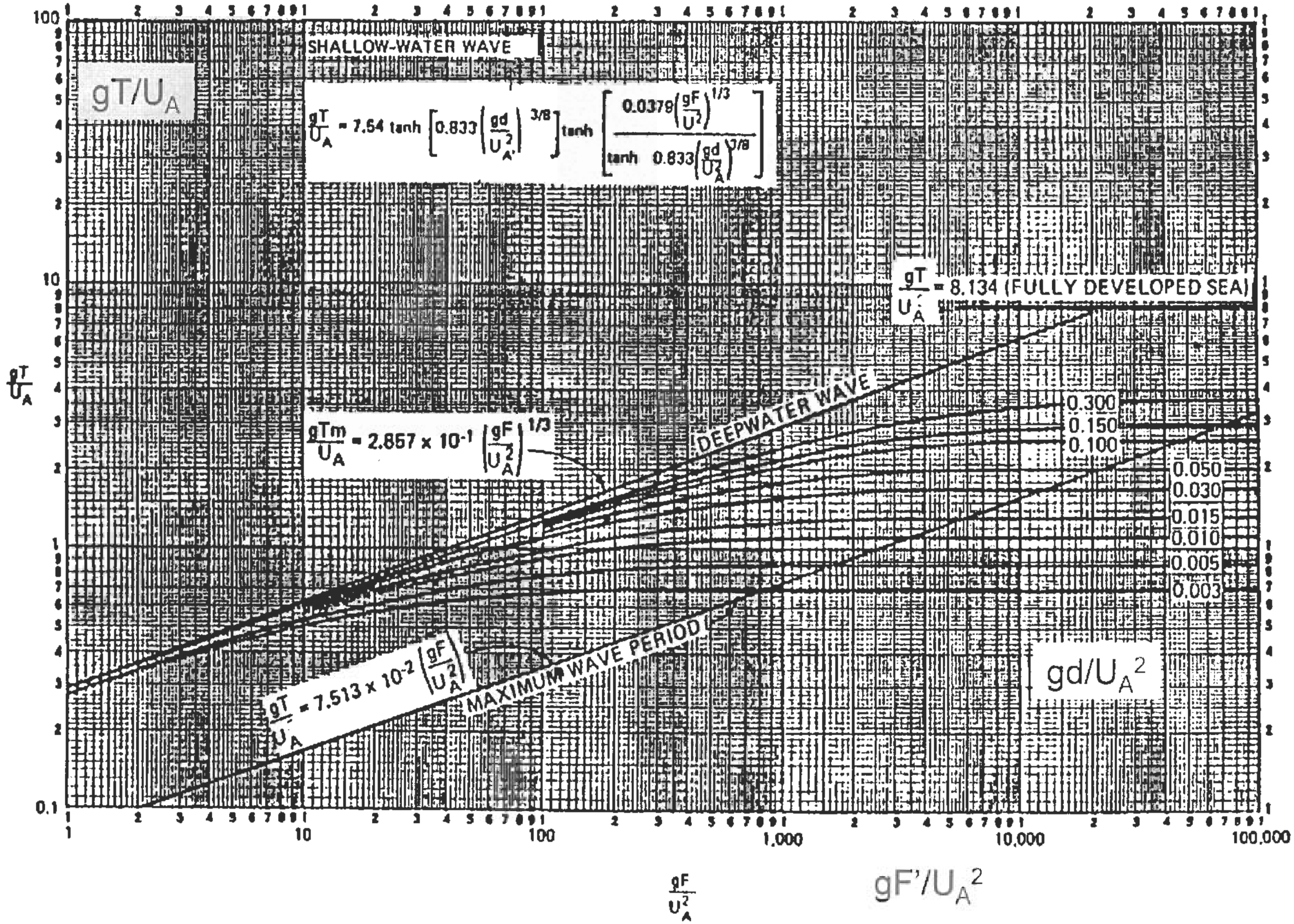


Figure 3-22. Forecasting curves for wave period. Constant water depth.

RL	1				
RT	1.02				
U'	61.2	mph	89.8	ft/sec	
UA	92.9	mph			
UA	136.1	ft/sec			
Fd	0.024				
FF	229				
Ftd	852				
Fig 3.21	gHs/UA2	0.0095	Hs	5.47	ft
Fig 3.22	gTs/UA	1.22	Ts	5.16	sec

Check Duration Part 2

- Assume Shallow if Fig 3.22 indicates
- Cal $(gT_m/U_A) = (Ftd/537)^{3/7}$

If $(Ftd/537)^{3/7} < (gT_s/U_A)$ from part 1 then,

- Enter Fig 3.22 with (gT_m/U_A) and intersect Fd to get FF'
- Use Fig 3.21 with FF' and Fd to Read $FHs' = gH_s/U_A^2$
- $H_s = U_A^2 * FHs'/g$
- $T_s = T_m = U_A * (Ftd/537)^{3/7}/g$
- $F' = U_A^2 FF'/g$

ELSE USE Hs and Hs from Part 1.

Assume Shallow						
gTm/UA		1.22	~	1.22	Therefore use fetch limited for shallow water	
Ts=Tm		5.1507	sec			
Fig 3.22	FF'	229				
Fig 3/21	gHs/UA2	0.0095	Hs	5.47	ft	

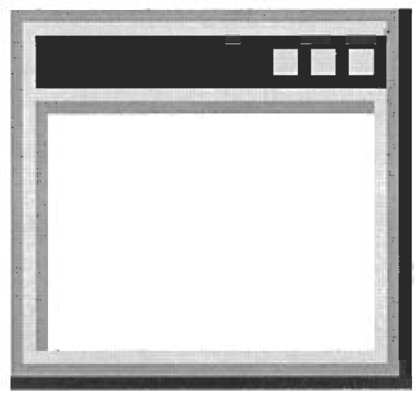
DEEP

gF'/UA2~43.6<<229 deep

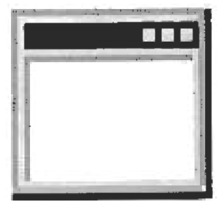
Fig 3.21	gHs/UA2	0.0068	Hs	3.91	ft
Fig 3.22	gTs/UA	0.8	Ts	3.38	sec

d/L ~ 0.14 < 0.5 not deep

UNO WAVE PROGRAM

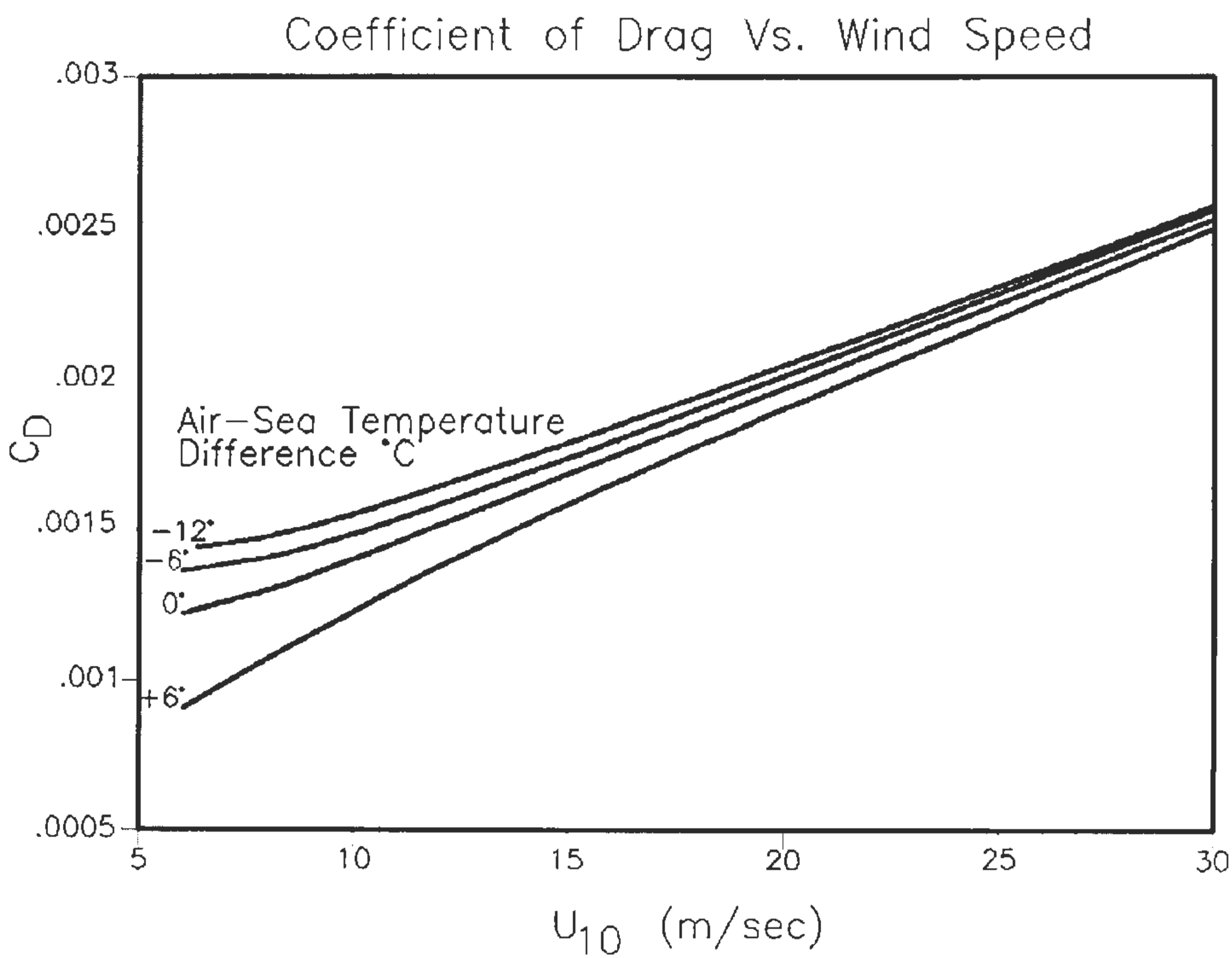


waves4723-08F.exe



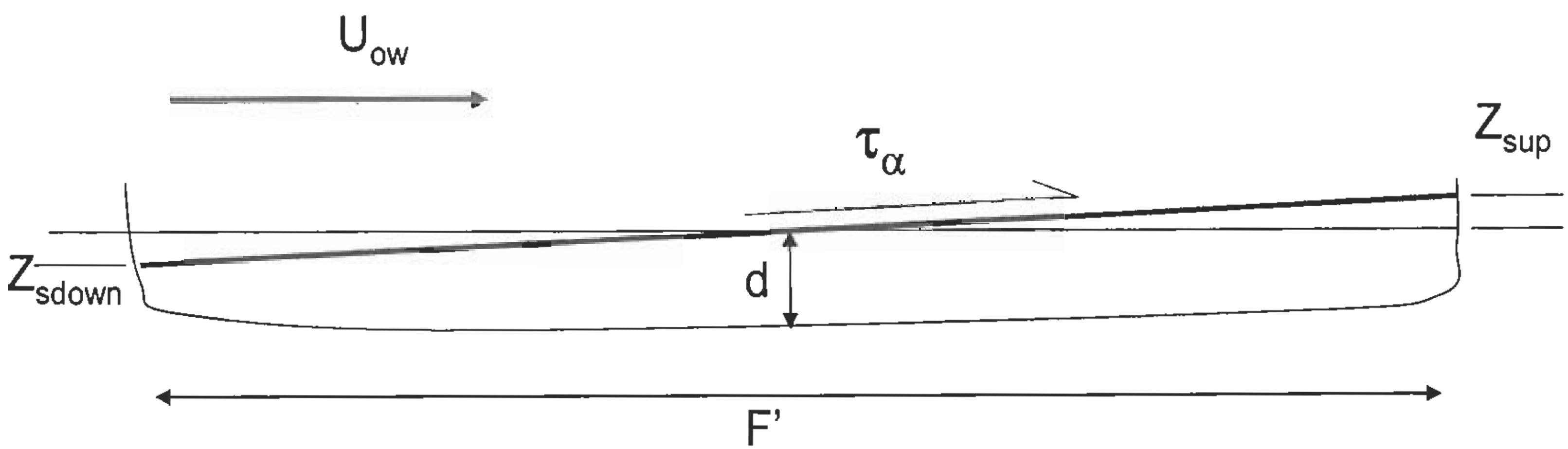
waves4723-08F.exe

Wind Drag



Coefficient of Drag Vs. Wind Speed
.003
.0025
.002
.0015
.001
.0005
C_D
Air–Sea Temperature
Difference °C
−12°
−6°
0°
+6°
5
10
15
20
25
30
U_10 (m/sec)

Setup



Setup

$$F'\tau_a = \gamma_w d\Delta z$$

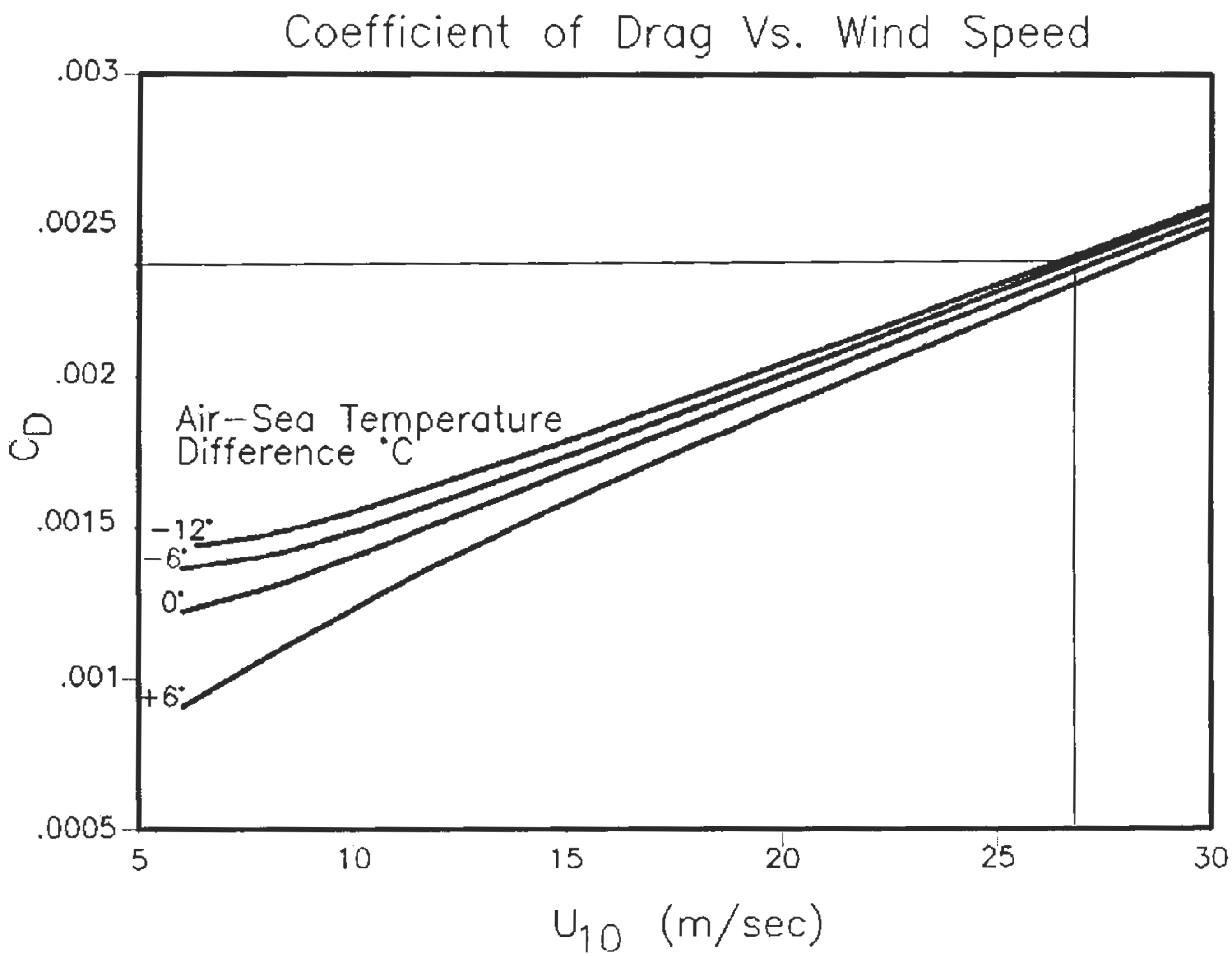
$$\tau_a = \rho_a C_f U_{OW}^{2}$$

$$F'(\rho_a C_f U_{OW}^{2}) = \gamma_w d(2Z_{\sup}) = g\rho_w d(2Z_{\sup})$$

$$Z_{\sup} = \frac{F'(\rho_a C_f U_{OW}^{2})}{2g\rho_w d}$$

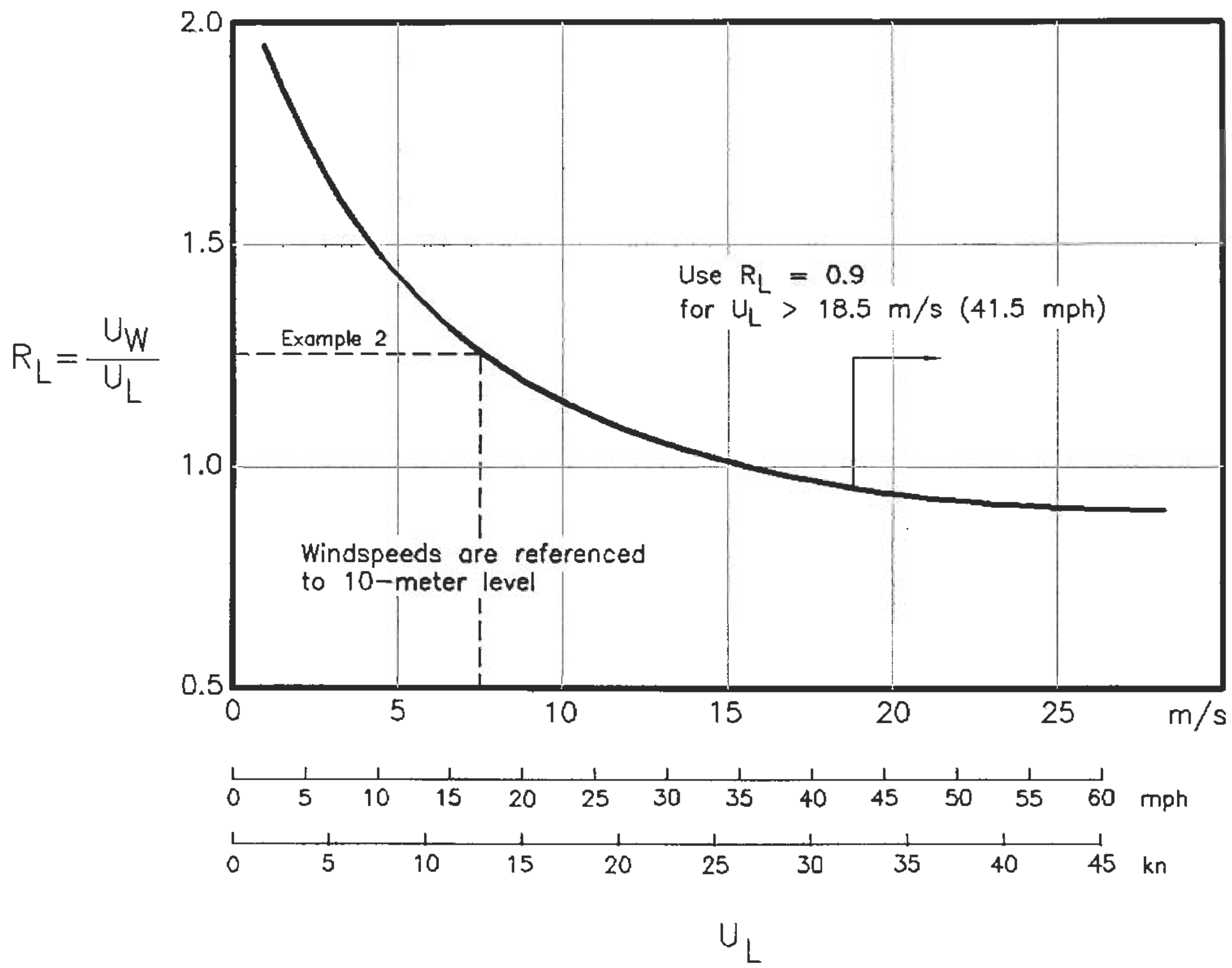
$$C_f \sim 0.0015 - -0.003$$

Wind Drag



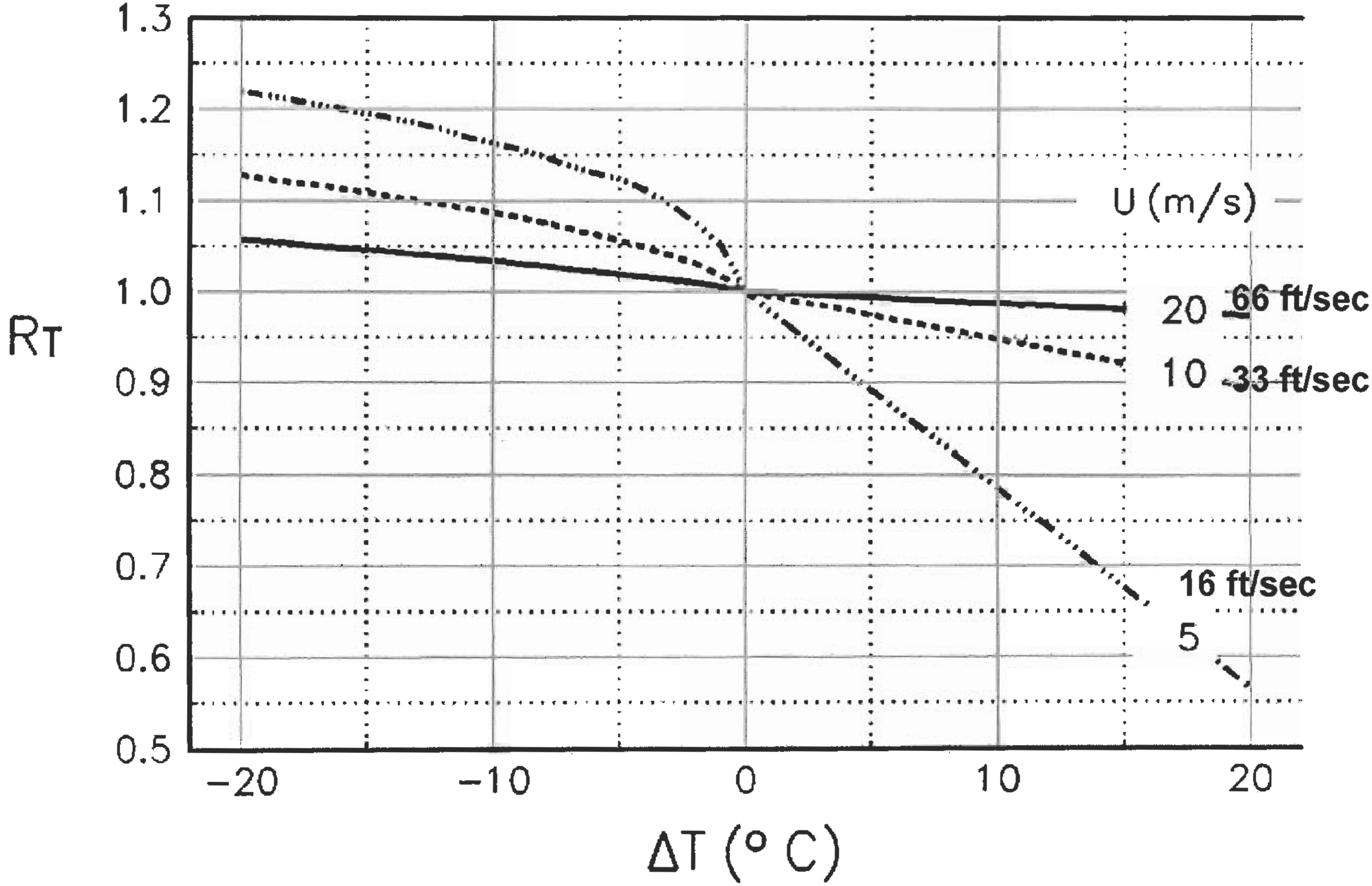
Coefficient of Drag Vs. Wind Speed
.003
.0025
.002
.0015
.001
.0005
C_D
Air–Sea Temperature
Difference °C
−12°
−6°
0°
+6°
5
10
15
20
25
30
U_{10} (m/sec)

Uow/UoL



$R_L = \frac{U_W}{U_L}$
2.0
1.5
1.0
0.5
Example 2
Use R_L = 0.9
for U_L > 18.5 m/s (41.5 mph)
Windspeeds are referenced
to 10-meter level
0 5 10 15 20 25 m/s
0 5 10 15 20 25 30 35 40 45 50 55 60 mph
0 5 10 15 20 25 30 35 40 45 kn
U_L

Atmospheric Instability Factor



Wind Setup

d	14	ft
Uow	89.76	ft/sec
F	132000	ft
Cf	0.0024	
ρw/ρa	800	62.4
Zsup	3.54	ft
Zsup	4.78	ft

Assignment 6.1: Estimate the south shore wave height and period for TS Isidore. Assume NNE winds at 35 knots with 2 hour duration. Estimate the wind setup for this storm.

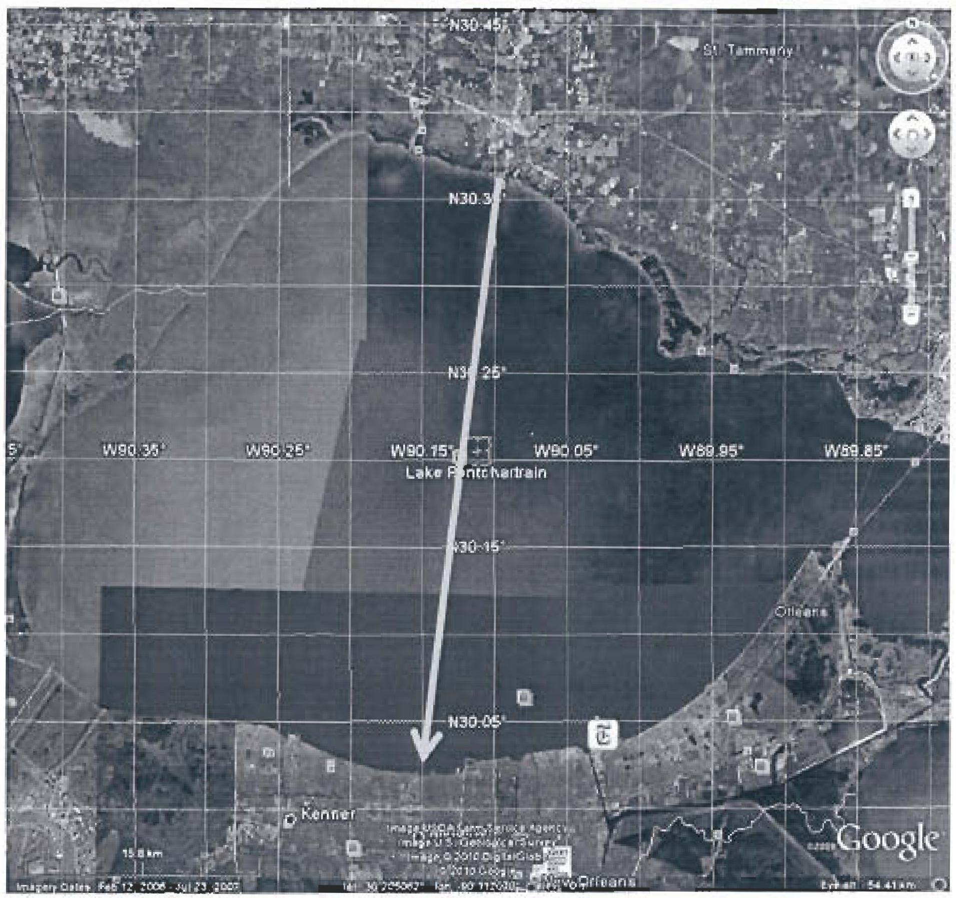
Hs = ____________

Ts = ____________

Are the waves: (FETCH LIMITED) or (DURATION LIMITED) ?

Wind setup = _________________ Using Equation 6.14

Wind setup = _________________ Using Equation 6.15



Shoaling and Refraction Animation.

http://daphne.palomar.edu/lyon/Animations/WaveMotion.swf

(9) Kitaigorodskii (1962) extended the similarity arguments of Phillips to distinct regions throughout the entire spectrum where different mechanisms might be of dominant importance. Pierson and Moskowitz (1964) followed the dimensional arguments of Phillips and supplemented these arguments. with relationships derived from measurements at sea. They extended the form of Phillips spectrum to the classical Pierson-Moskowitz spectrum

$$E(f) = \frac{\alpha\, g^2 f^{-5}}{(2\pi)^4} \exp\left[-0.74\left(\frac{f}{f_u}\right)^{-4}\right] \qquad \text{(II-2-32)}$$

where

f_u = limiting frequency for a fully developed wave spectrum (assumed to be a function only of wind speed)

(12) Hasselmann et al. (1973) collected an extensive data set in the Joint North Sea Wave Project (JONSWAP). Careful analysis of these data confirmed the earlier findings of Mitsuyasu and revealed a clear relationship between Phillips' α and nondimensional fetch (Figure II-2-21). This finding and certain other spectral phenomena, such as the tendency of wave spectra to be more peaked than the Pierson-Moskowitz spectrum during active generation, could not be explained in terms of "first-generation" concepts; however, they could be explained in terms of a nonlinear interaction among wave components. This pointed out the necessity of incorporating wave-wave interactions into wave prediction models, and led to the development of second-generation (2G) wave models. The modified spectral shape which came out of the JONSWAP experiment has come to bear the name of that experiment; hence we now have the JONSWAP spectrum, which can be written as

$$E(f) = \frac{\alpha g^2}{(2\pi)^4 f^5} \exp\left[-1.25\left(\frac{f}{f_p}\right)^{-4}\right] \gamma^{\exp\left[-\frac{\left(\frac{f}{f_p}-1\right)^2}{2\sigma^2}\right]} \qquad \text{(II-2-34)}$$

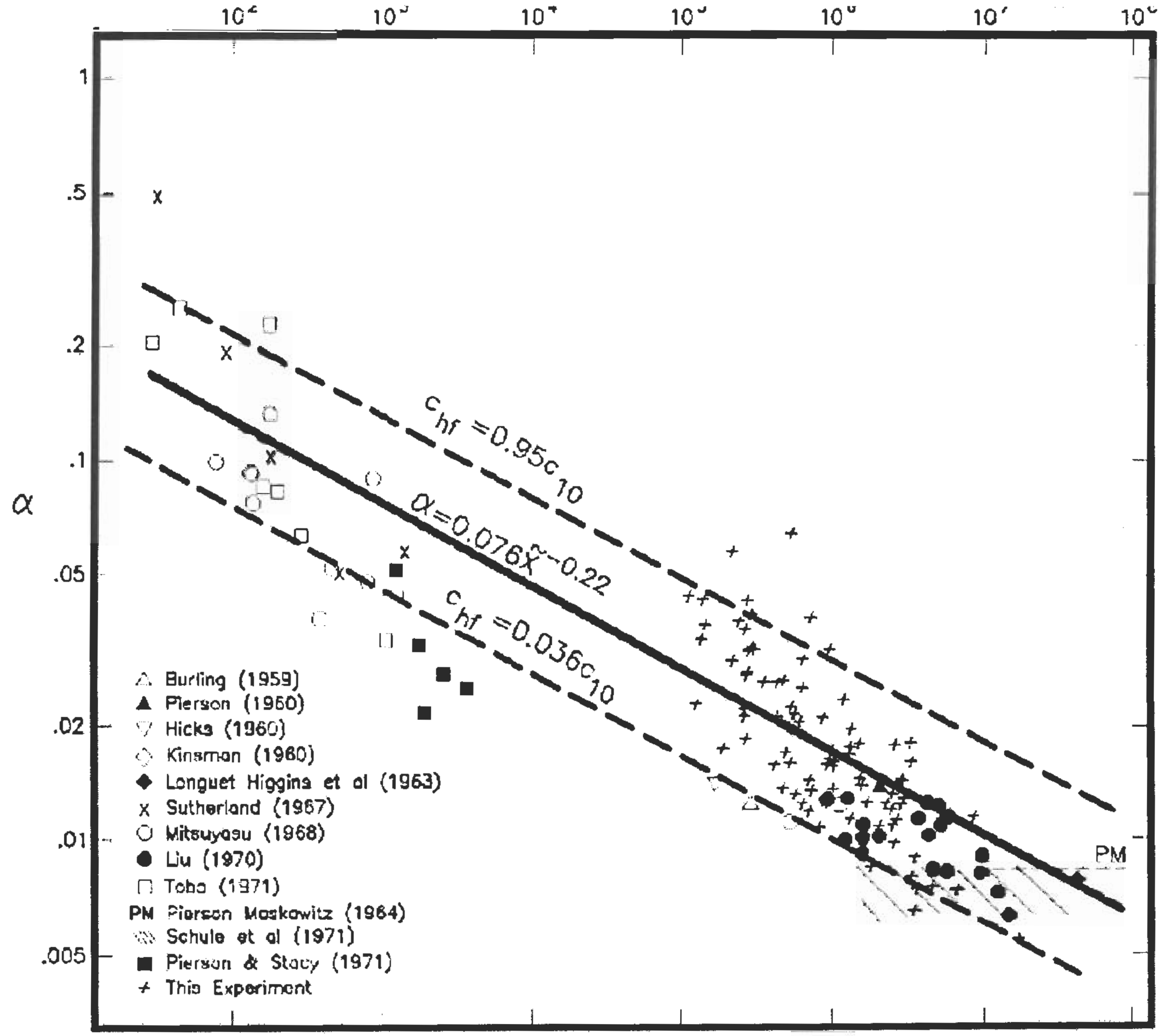
where

α = equilibrium coefficient

σ = dimensionless spectral width parameter, with value σ_a for $f<f_p$ and value σ_b for $f>f_p$

γ = peakedness parameter

The average values of the σ and γ parameters in the JONSWAP data set were found to be $\gamma = 3.3$, $\sigma_a = 0.07$, and $\sigma_b = 0.09$. Figure II-2-22 compares this spectrum to the Pierson-Moskowitz spectrum.



α
1
.5
.2
.1
.05
.02
.01
.005
10^4
10^5
10^6
10^7
10^8
$c_{hf} = 0.95 c_{10}$
$\alpha = 0.076 \tilde{X}^{-0.22}$
$c_{hf} = 0.036 c_{10}$
PM
Burling (1959)
Pierson (1960)
Hicks (1960)
Kinsman (1960)
Longuet Higgins et al (1963)
Sutherland (1967)
Mitsuyasu (1968)
Liu (1970)
Toba (1971)
PM Pierson Moskowitz (1964)
Schule et al (1971)
Pierson & Stacy (1971)
This Experiment

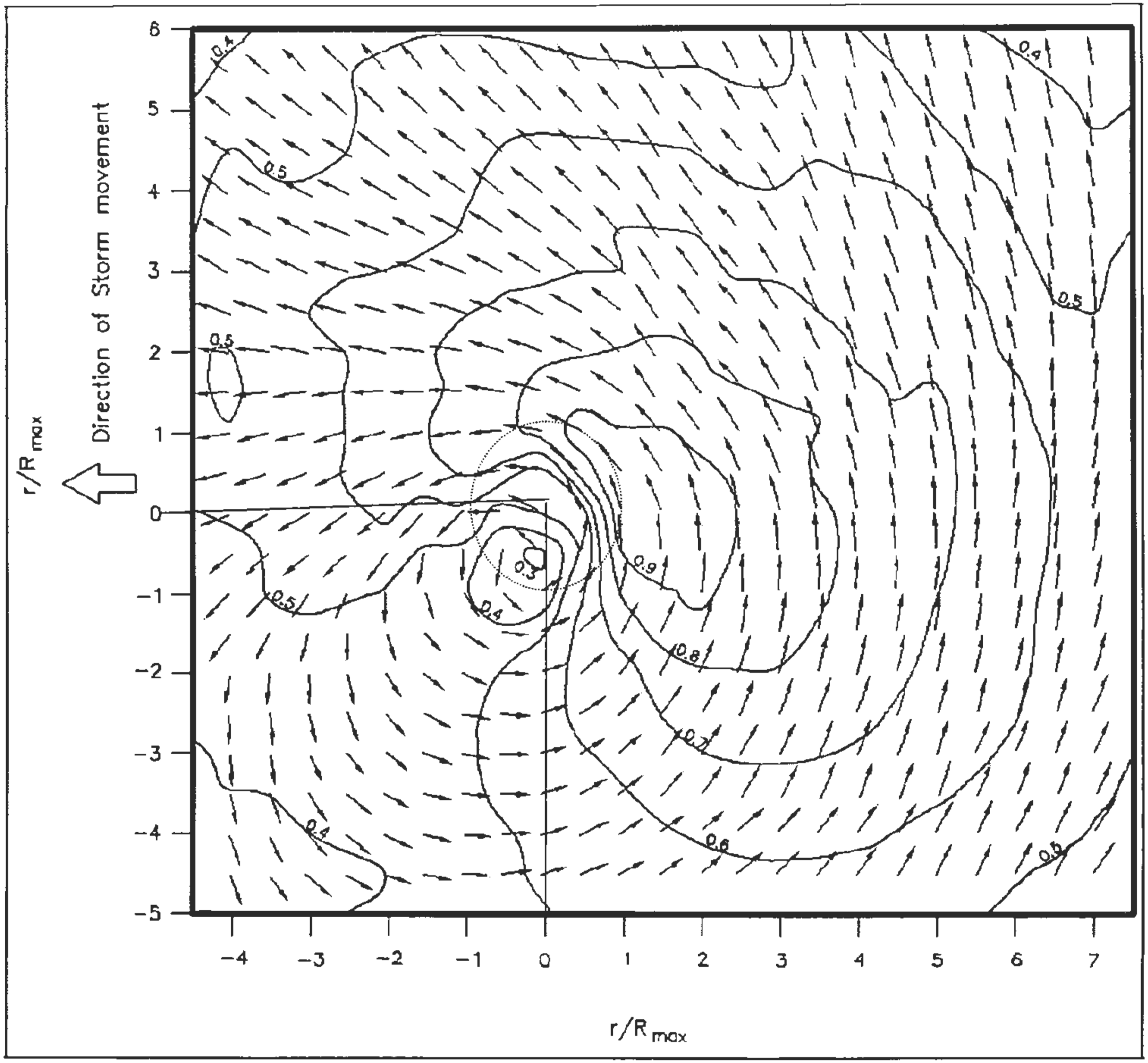


Figure II-2-28. Values of $H_{mo}/H_{mo\,max}$ plotted relative to center of hurricane (0,0)

djerolle

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Lecture 6
Wave Prediction

Wave Statistics

An irregular train of waves is illustrated in Figure 6.1. The waves in this train have various heights and periods. One way of describing this set of random waves is the Root Mean Square of the heights of all the waves, i.e.

$$H_{rms} = \{\Sigma H_i^2/n\}^{1/2} \qquad 6.1$$

Each wave is separated by successive zero crossings. The number of waves in the train is n. We can also conduct a probability analysis on the wave heights, i.e. we can plot the probability, P of exceeding a certain wave height H, e.g.

$$P = m/(1 + n) \qquad 6.2$$

where m = the rank of a wave in descending order. Weibull and Rayleigh distributions have been used to fit the wave distributions of storm waves. Figure 3-3 (SPM 73) shows a normalized Rayleigh distribution.

The wave periods also can be considered by a probability distribution. Alternately a wave spectrum analysis can be preformed. The wave spectrum is the plot of wave energy versus the wave frequency (f = 1/T). A Fourier Analysis can be completed to construct the wave spectrum. Commercial software packages such as Statistica can be used if the wave form is digitized. Figure 3-6 (SPM 73) shows a typical wave spectrum. A significant wave can be defined by the dominant energy in the wave train. Theoretical spectra are available for waves in a few coastal areas. For example the SPM gives the Pierson-Moskowitz Spectra as,

$$\begin{aligned}
&E(\omega)d\omega = \frac{\alpha g^2}{\omega^5} e^{-\beta(\omega_o/\omega)^4} \\
&\alpha = 0.0081 \\
&\beta = 0.74 \qquad\qquad 6.3 \\
&\omega = angular_frequency = 2\pi f = 2\pi / T \\
&\omega_o = g / U_{OW} \\
&U_{OW} = overwater.windspeed,(ship.report)
\end{aligned}$$

There are also spectra that involve frequency and direction.

Wave period spectrum

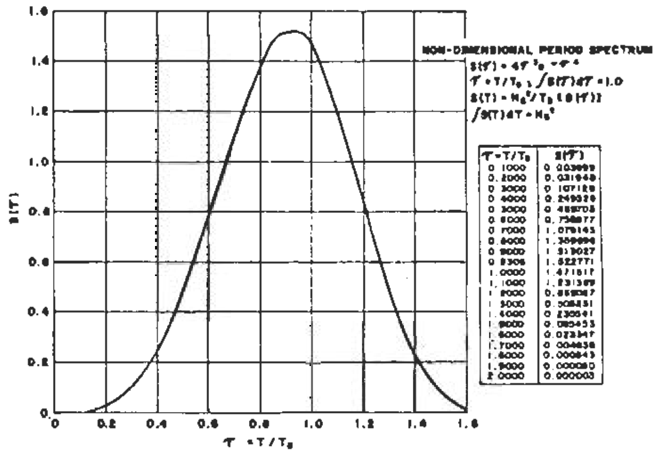


Fig. 2 Non-dimensional period spectrum

Fig 6.1 Period Spectrum

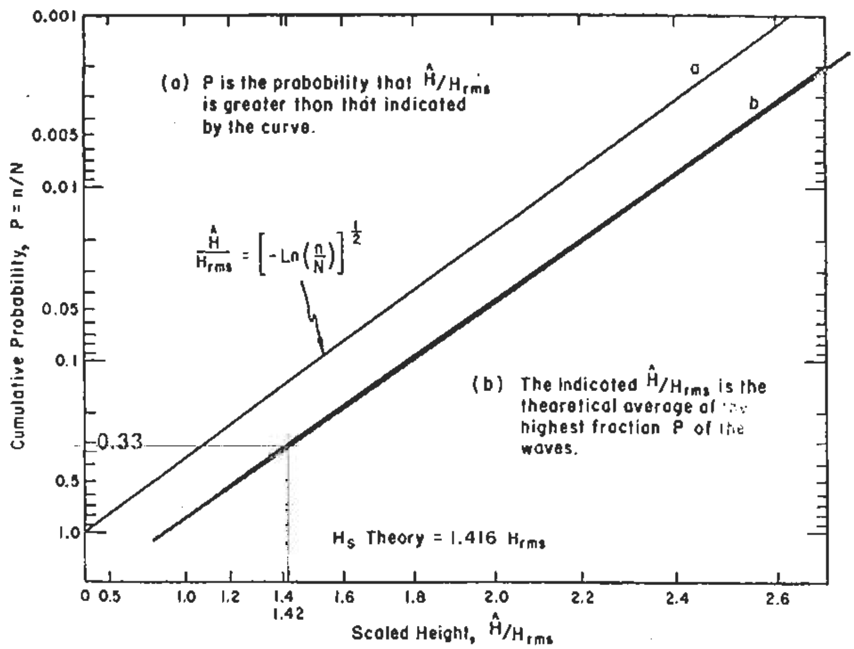


Figure 3-5. Theoretical wave height distributions.

Fig 6.2 Rayleigh Distribution for a) all waves and b) averaged higher than waves.

Wave generated by a wind depend on a number of factors, such as: effective wind speed V_a, wind direction, available fetch (distance over which wind blows) F, water depth d, atmospheric stability R_T, and wind duration t_d. A given wind condition generates a spectrum of waves, with the dominant wave energy at a particular wave height H_s and period T_m. Often it is assumed that the dominant wave height is approximately the average of the highest 1/3 of all the waves; the SPM uses the spectral significant wave height, H_s with the T_m, being the significant wave period for the maximum spectral energy. As shown in Figure 6.2, waves have been found to fit the Rayleigh Distribution:

$$P(H > \hat{H}) = e^{-\left(\frac{H}{H_{rms}}\right)^2} \qquad 6.4$$

where P is the probability of exceeding and certain x-value.

Based on a Rayleigh frequency distribution the relationship of H_s to some other wave height is

$$H_s = 2 \cdot H_{rms} \qquad 6.5a$$

$$H_{10} = 1.27 H_s \qquad 6.5b$$

$$H_1 = 1.67 H_s \qquad 6.5c$$

The subscripts 1 and 10 refer to the average the highest 1 % and 10% of the waves in a storm. The SPM provides a comprehensive wave prediction procedure for H_s and T_m based on dimensionless charts as shown in Figures 3-21 and 3-22. The charts show the functions that are used; these are in the forms:

$$g H_s / U_A^2 = \text{fcn}\,(F_d, F_{\blacktriangle F}) \qquad 6.6$$

and $$g T_s / U_A = \text{Fcn}\,(F_d, F_{\blacktriangle F}) \qquad 6.7$$

where $$F_d = \textit{depth number} = g\, d / U_A^2 \qquad 6.8$$

$$F_{\blacktriangle F} = \textit{fetch number} = g\, F_{\blacktriangle} / U_A^2 \qquad 6.9$$

$$F_{\blacktriangle} = \textit{effective fetch} = \textit{minimum}\,(F_a\,,\ F_{\blacktriangle d = \text{duration limited}}) \qquad 6.10$$

The relation between t_d *and* $F_{\blacktriangle}$ for deep water conditions is

$$g\, t_d / U_A = 68.8\,(g F_{\blacktriangle} / U_A^2)^{2/3} \qquad 6.11$$

and for shallow water we have,

$$g\, t_d / U_A = 537\,(g T_m / U_A)^{7/3} < 1.28\,(g F_{\blacktriangle} / U_A^2)^{7/3} \qquad 6.12$$

Note: Figures 3-21 and 3-22 (SPM) show the prediction equations that are used for Eq 6.6 and 6.7.

The wind speed that is used is the corrected for water:land effects (R_L, Fig. 6.5), atmospheric stability (R_T, Fig 3-14 SPM, also Figure 6.6) and for the effective wind shear; this is given as:

$U_A = 0.589\,(U_{oLmph} R_L R_T)^{1.23}$ in mph or $U_A = 0.539\,(U_{oLfps} R_L R_T)^{1.23}$ in ft/sec

$U_A = 0.77\,(U_{oL} R_L R_T)^{1.23}$ in m/s 6.13

where $U_{oL} R_{wL}$ = over water velocity at 10 m above surface.

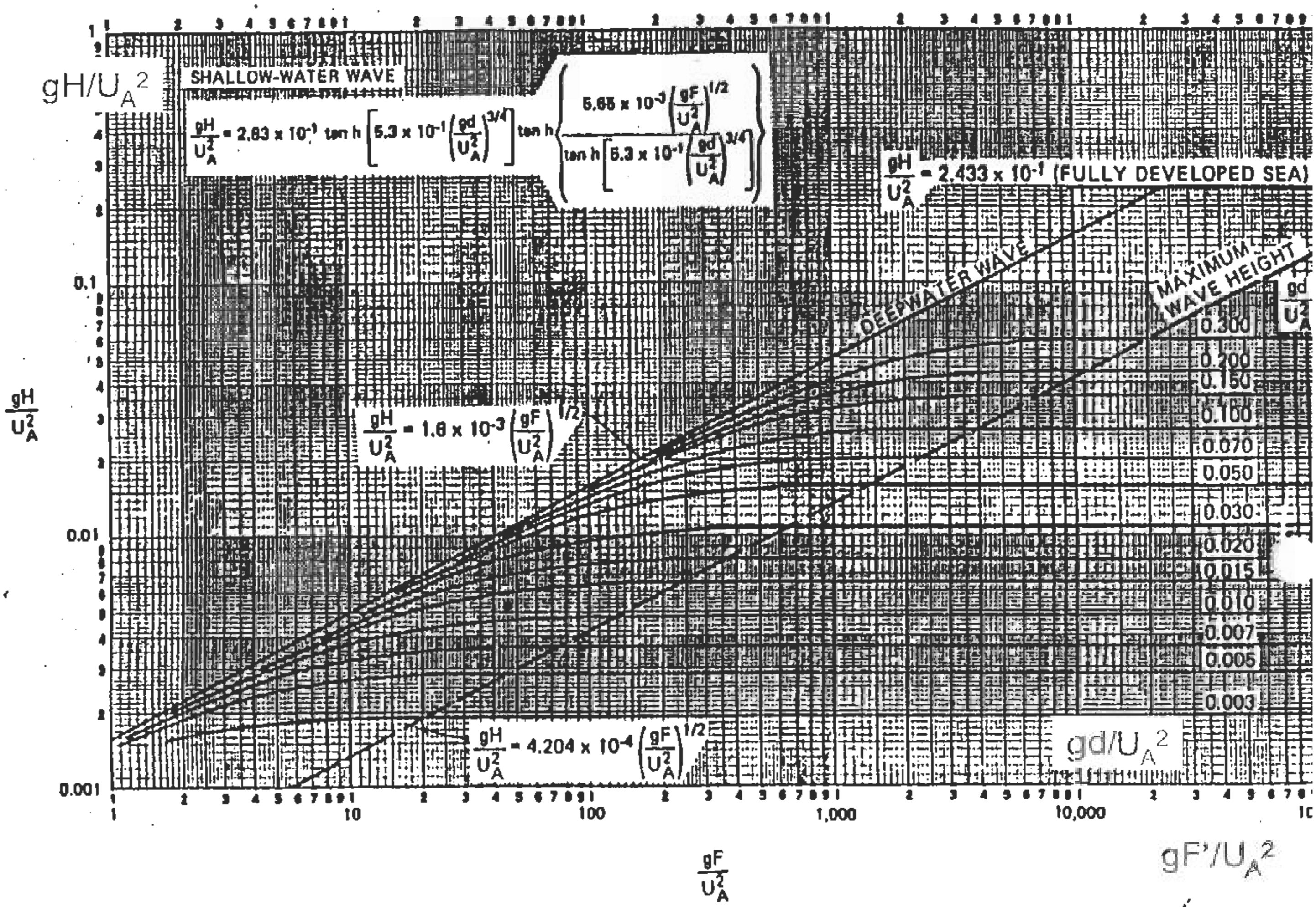


Figure 3-21. Forecasting curves for wave height. Constant water depth.

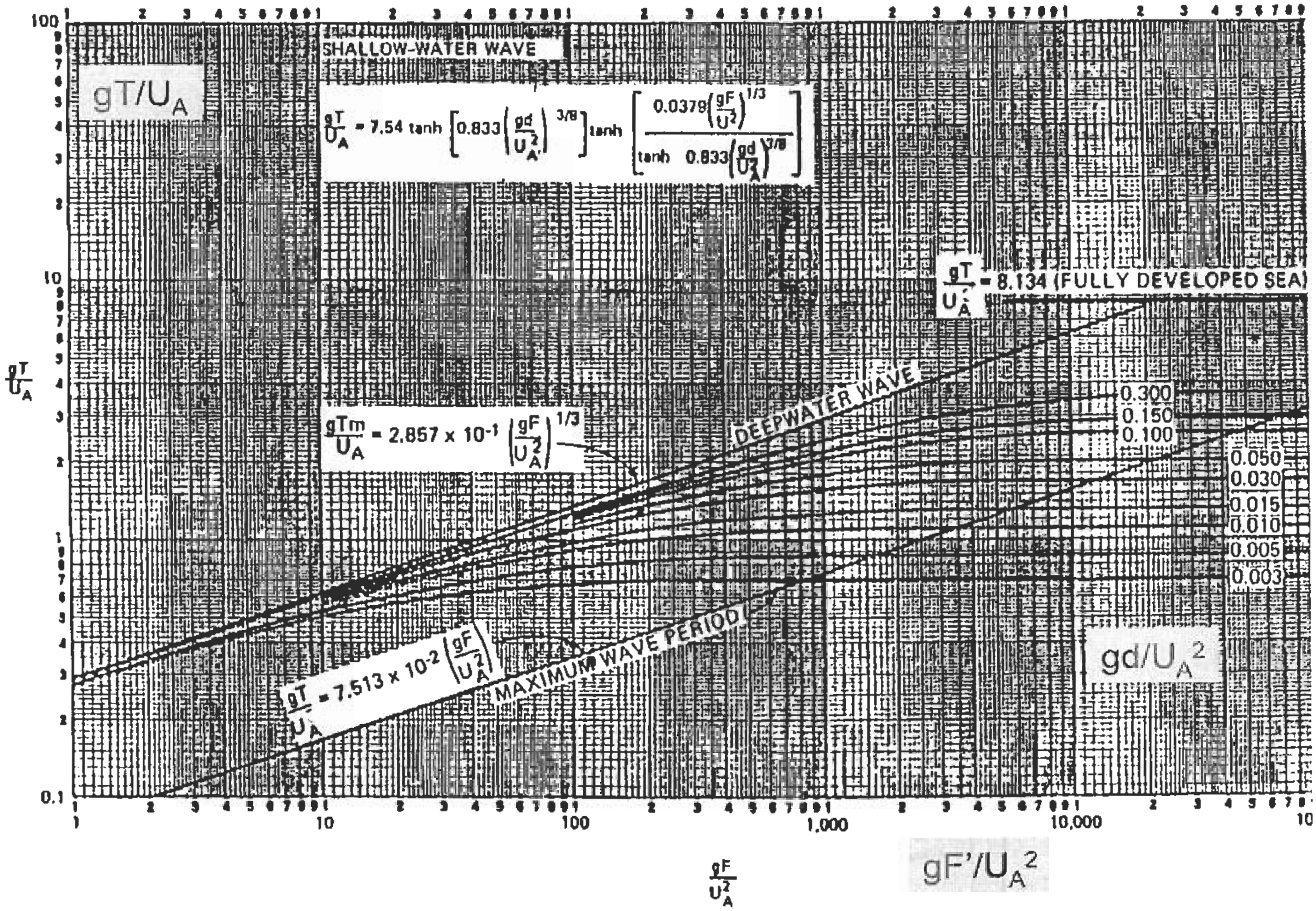


Figure 3-22. Forecasting curves for wave period. Constant water depth.

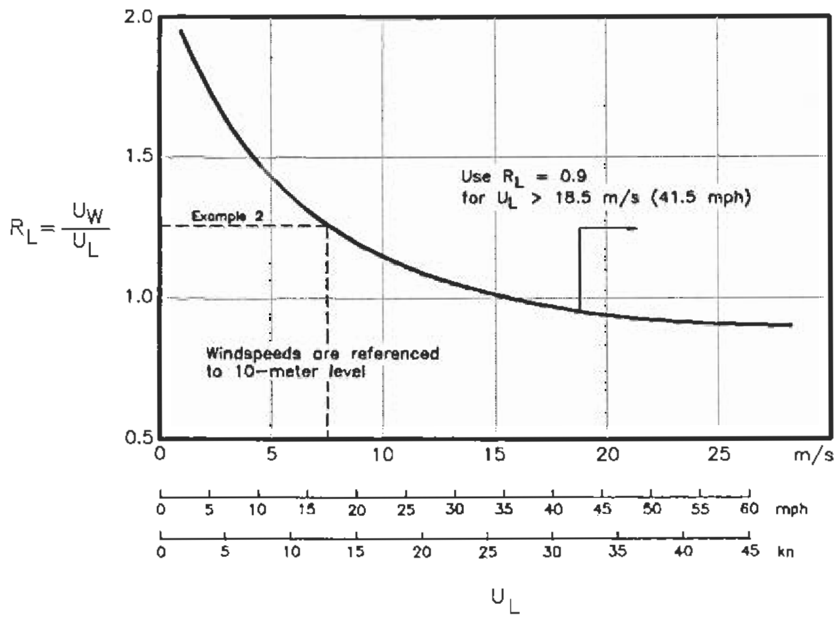


Fig 6.5 R_L = Uow:UoL Ratio (CEM and SPM)

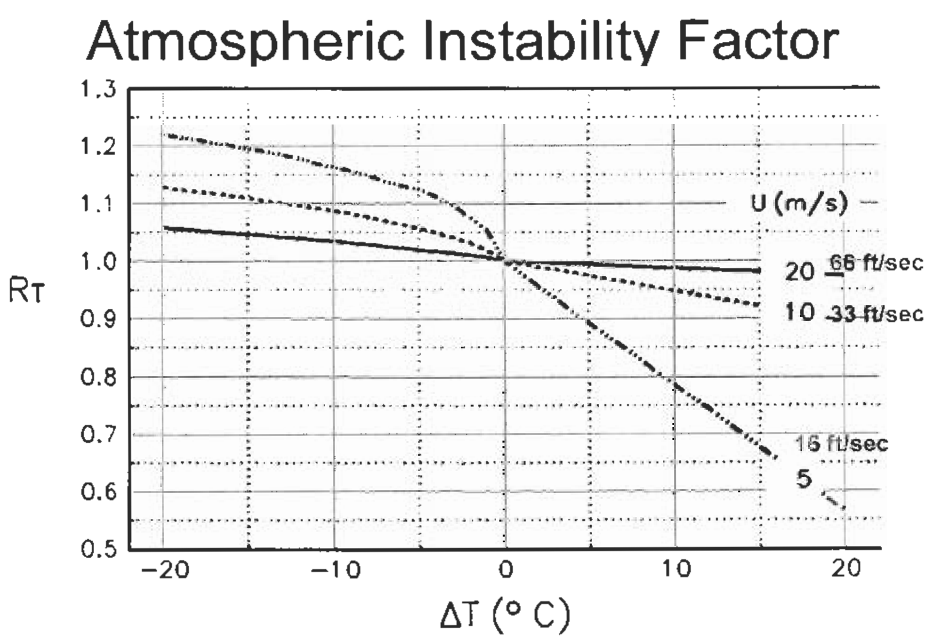


Fig 6.6 R_T (CEM)

Setup:- increase in water depth (downwind) due to wind shear. The form of this equation can be demonstrated by the following analysis. Assume a unit wide strip with a length equal to the effective fetch, F'; the wind shear force is $\tau_a F'$ = the hydrostatic resistance ~ $\gamma d \Delta z$

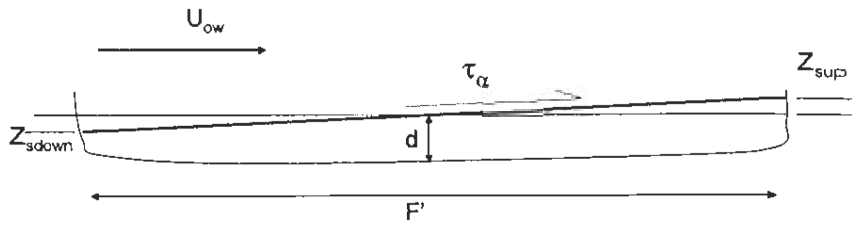


Fig 6.7 Wind Shear and Setup Diagram

$\mathbf{\Delta z = Z_{sup} + Z_{sdown}}$

$$F'\tau_a = \gamma_w d \Delta z$$

$$\tau_a = \rho_a C_f U_{OW}^2$$

$$F'(\rho_a C_f U_{OW}^2) = \gamma_w d(2Z_{sup}) = g\rho_w d(2Z_{sup}) \qquad 6.14$$

$$Z_{sup} = \frac{F'(\rho_a C_f U_{OW}^2)}{2g\rho_w d}$$

$$C_f \sim 0.001 \rightarrow 0.003$$

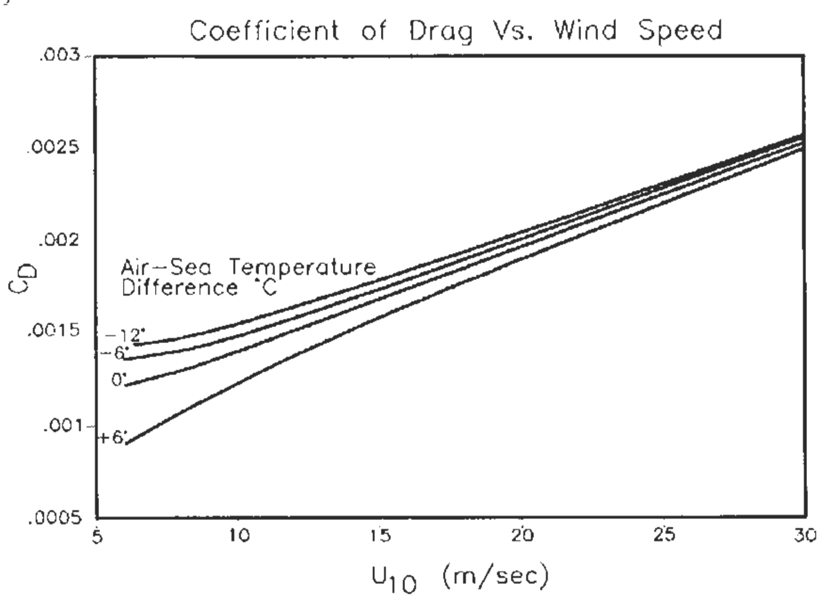


Figure 6.8 Wind Friction Coefficient (C_f)

An empirical formula for $\mathbf{Z_{sup}}$ is,

$$z_{sup}\,(ft) = F\,(miles)\ V(mph)^2\ /[1400\ d\ (ft)] \qquad 6.15$$

Class Example Problem:
Determine the significant wave period and height and the setup near the south shore of Lake Pontchartrain for a 2- hour wind out of the north over the Lake.
Assume:

- a 25 mile fetch with an average depth of 3.8 m before the storm surge,
- a storm surge of 2 m,
- a local depth of 3 m without the storm surge or setup,
- a water temperature of 80°F and air temperature of 70°F.
- the over-water velocity is 80 mph.

Assignment 6.1: Estimate the south shore wave height and period for TS Isidore. Assume NNE winds at 35 knots with 2 hour duration. Estimate the wind setup for this storm.

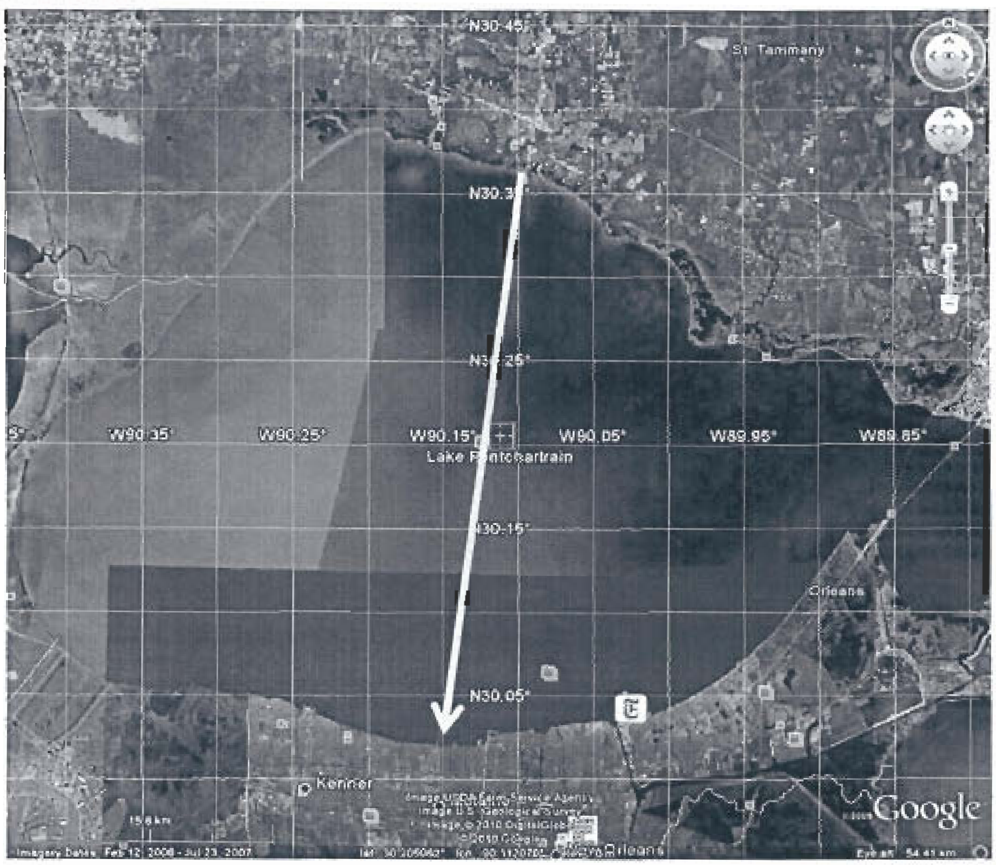
Hs = ____________

Ts = ____________

Are the waves: (FETCH LIMITED) or (DURATION LIMITED) ?

Wind setup = _________________ Using Equation 6.14

Wind setup = _________________ Using Equation 6.15



djerolle

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Lecture 6 (Continued)

Hurricane Generated Waves and Storm Surges

The wave climate associated with a hurricane depends on the radius of the storm, its sustained maximum wind speed and the forward speed to the storm. The SPM provides a typical distribution of wave heights with the storm (1984 SPM Figure 3-43).

The simplest model for the wind speed distribution in a hurricane is the Rankin Vortex,

$$U = K\,r \qquad \text{for} \quad r < R \qquad 6.16a$$

and $$U = K\,R^2/r \qquad \text{for} \quad r > R \qquad 6.16b$$

where K = constant = U_{max}/R; R = radius at the maximum speed; r is the radius to a point from the storm centre. This simple model gives a idea of how winds vary as we go away from the center but it omits many effects such as the boundary layer, the Coriolis effect, the forward speed of the storm and the radial flow at the base.

The sea surface pressure is fairly well described by the Myers equation,

$$p = p_o + \Delta p\, e^{-\{R/r\}} \qquad 6.17$$

where p_o = central pressure and $\Delta p = p - p_o$

Figure 9 shows a typical velocity and pressure variation as a function of the distance from the center of the hurricane.

The associated Gradient Wind Speed, U_{gr} for a nearly stationary storm is

$$U_{gr}^2/r + f\,U_{gr} \simeq \{R\,\Delta p/(\rho_a\,r^2)\}e^{-\{R/r\}} \qquad 6.18$$

where f = Coriolis parameter = $2\omega\sin(\phi)$; ρ_a =air density.

The vector to be added to U_{gr} to correct for the forward speed V_f is given by

$$\underline{U_{SM}} = \underline{V}_f R\,r/(R^2 + r^2) \qquad 6.19$$

Note: Resultant velocity is found from the vector sum of

$$\underline{U_T} = \underline{U_{SM}} + \underline{U}_{gr} \qquad 6.20$$

Thus in the northern hemisphere the speeds on the right of the center are higher than on the left.

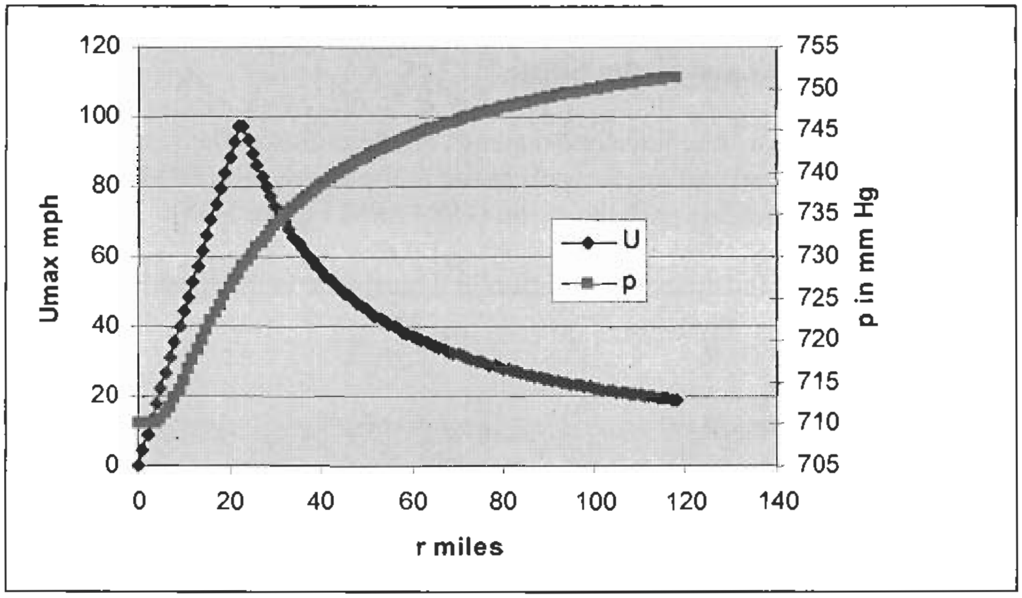


Figure 6.9: Pressure and Velocity Distribution in a Hurricane (R=22.5 miles; Δp= 50 mm)

Hurricane Waves

The SPM gives the following equations for the significant deep water wave height H_{os} and period T_s at the location of the maximum wind speed:

$$T_s = A\left[1+\frac{B\alpha V_F}{\sqrt{U_R}}\right]e^{\frac{R\Delta p}{C}} \qquad 6.20$$

in which A = 8..6 for US and SI units; B =200 for US and 9400 for SI units and C= 0.104 for US and 0.145 for SI units.

Also the deep water significant wave height is:

6.21

$$H_{os} = A_o\left[1+\frac{B_o\alpha V_F}{\sqrt{U_R}}\right]e^{\frac{R\Delta p}{C_o}}$$

in which A_o = 16.5 (5.02); B_o =100 (4700) and C_o= 0.208 (0.29) for US (SI) units. The spatial distribution of the significant wave heights are shown in Figures 10 and 11.

Inputs:

R = radius of maximum wind (***Nautical Miles*** or ***km***)

Δp= central pressure depression (in ***inches*** of ***mm*** of Hg)

V_F = forward speed in ***knots*** or ***km/h***

$$U_{max} = a\{b\Delta p^{1/2} - cRf\}$$
$$U_R = 0.865U_{max} + 0.5V_F$$

$$f = 2\omega_e \sin\phi = Coriolis(rad/h)$$
$\alpha \sim 1$ to 1.2
where a = 0.868 (0.447); b=73 (14.5) and c= 0.57 (0.31) for US (SI) units

Note: Normal atmospheric pressure is 29.92 inches Hg = 760 mm Hg.
The earth's angular velocity is $2\pi/24$ rad/h;
1 nautical miles = 1852 m = 6076 ft

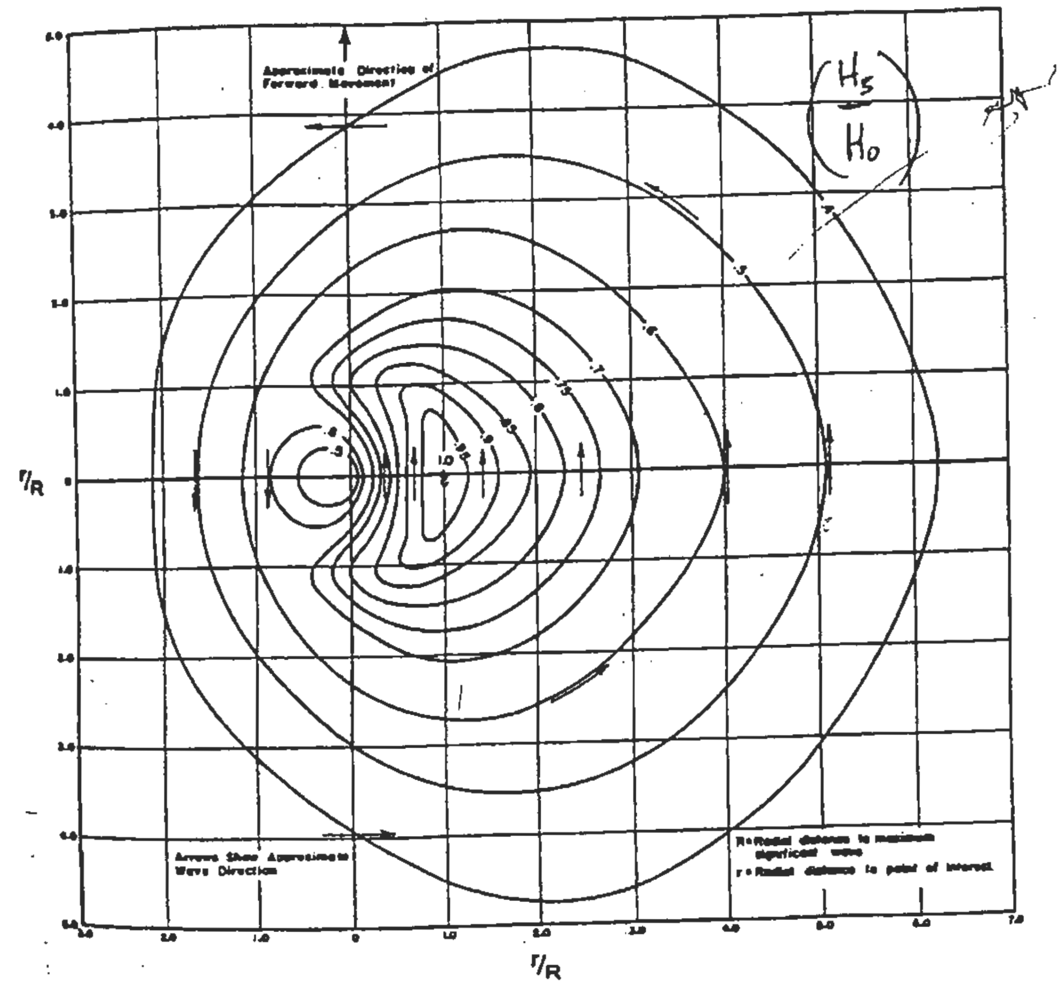


Figure 3-43. Isolines of relative significant wave height for slow-moving hurricane.

Fig 6.10 Distribution of wave heights (SPM)

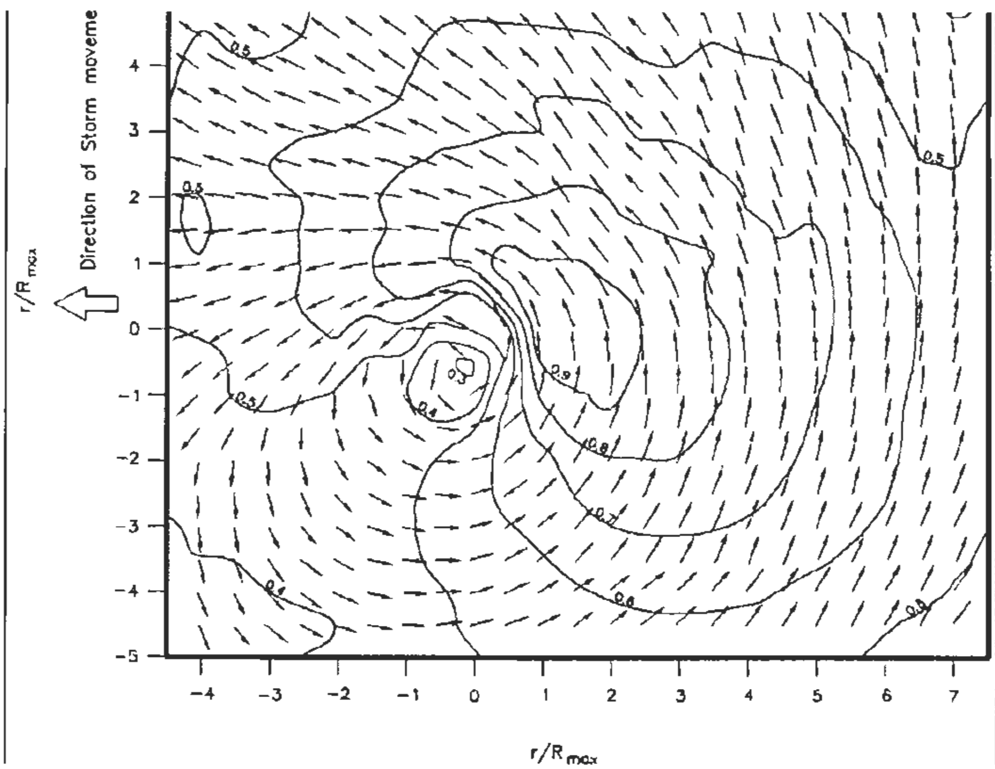


Fig 6.11 Distribution of wave heights (CEM)

The storm surge h_{ss} consists of two parts: central depression Δp and wind setup:

$$h_{ss} \approx K_s K_r \left[\Delta p + \frac{C_R C_f R U_R^{\,2}}{2 g d_{ref}} \frac{\rho_a}{\rho_w} \right]$$

$$C_R \approx 1 \to 2$$

$$\frac{\rho_a}{\rho_w} \approx 1/800$$

$$C_f \approx 0.0025$$

$$d_{ref} \sim 10 \to 20m$$

$$TimeScale \approx \sqrt{\frac{gR}{2\pi}}$$

Example: **A h**urricane has the following approximate characteristics:
R = 22.5 miles; Maximum Wind Speed ~ Umax; Forward Speed = 16 mph; Δp $\simeq$ 50 mm Hg with normal pressure at 760 mm. Assume Latitude of 28°. Plot the pressure and velocity on the right side of the storm. What are the maximum significant wave height and the corresponding period? Assume: d_{ref} ~ 10 m; C_R ~ 1; KrKs ~1; estimate the surge height.

djerolle

Machine Name: w14955

Date: 02/24/2010

Job: 317

Time: 5:01:36 PM

Cost: 0.00

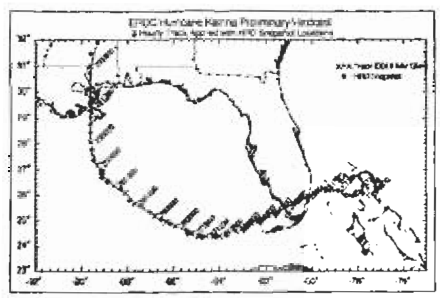
Wave Generation

Shore Protection Manual
Method
Hurricane Waves

Katrina Aug 28 05

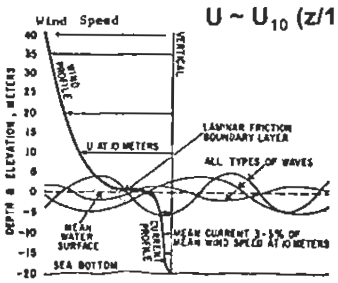
Katrina Aug 28 05

STORM TRACK



Wind Speed and Water Current

$U \sim U_{10} (z/10)^{1/7}$



U_{max} & U_R

U_R is maximum sustained wind at 10 m

Umax is maximum gradient wind

V_F = forward speed of the hurricane

Estimation of U_{max} & U_R

$$U_{max} = a\{b\Delta p^{1/2} - cRf\}$$

$$U_R = 0.865U_{max} + 0.5V_F$$

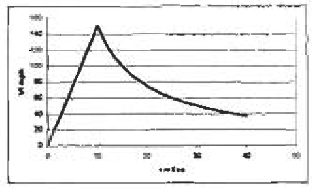
$$f = 2\omega_e \sin\phi = Coriolis(rad/h)$$

a = 0.888 ; b=73 and c= 0.57 for US units Knots, Nmiles; inches Hg;

a =0.447; b=14.5 and c= 0.31 for SI units kph, km; mm Hg;

U_R is maximum sustained wind at 10 m
U_{max} is maximum gradient wind

Rankin Vortex,
$U = K\,r$ for $r < R$
and $U = C/r$ for $r > R$
where K = constant = Umax/R
C= Umax*R;
R = radius at the maximum speed;
r is the radius to a point from the storm centre.



Pressure in a Hurricane

The sea surface pressure by the Myers equation.

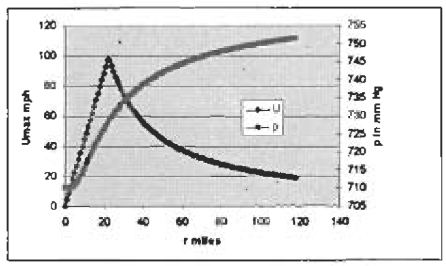
$$p = p_o + \Delta p\, e^{-R/r}$$

where p_o = central pressure

$\Delta p = p_n - p_o$

p_n = Atmospheric pressure

Velocity and Pressure



Hurricane (R=22.5 miles; Δp= 50 mm)

Waves

H_{os} = Reference Significant Wave Height

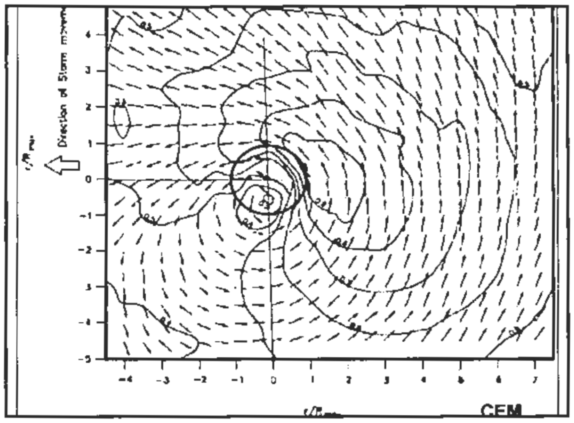
T_s = Reference Significant Wave Period

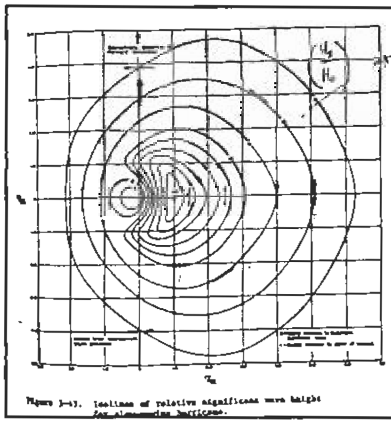
$$H_{os} = A_o\left[1 + \frac{B_o \alpha V_F}{\sqrt{U_R}}\right] e^{\frac{R\Delta p}{C_o}}$$

Ao = 16.5 (5.02);
Bo =100 (4700) and
Co= 0.208 (0.29)
$\alpha \sim 1 - 1.2$
for US (SI) units

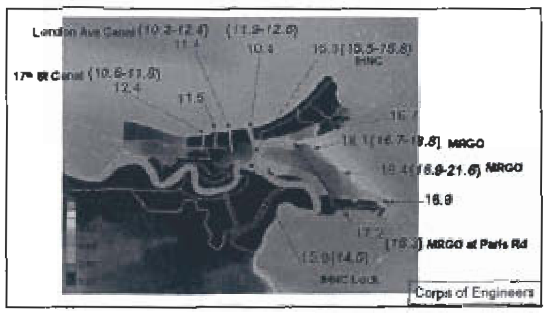
$$T_s = A\left[1 + \frac{B\alpha V_F}{\sqrt{U_R}}\right]e^{\frac{R\Delta p}{C}}$$

A = 16.5 (5.02);
B =100 (4700) and
C= 0.208 (0.29)
$\alpha \sim 1 - 1.2$
for US (SI) units

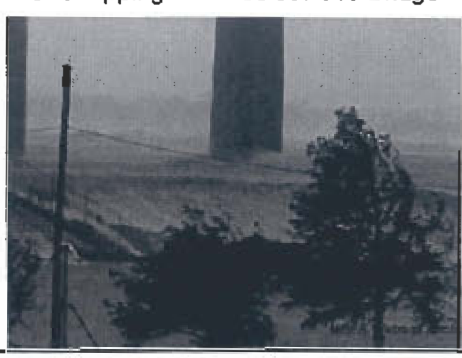




Storm Surges



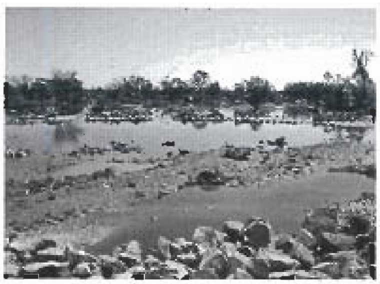
Overtopping of Levee at I-510 Bridge



Breach in London Canal

Residential Area near Breach in London Canal

LOWER 9th Breach



Barge on BV Control Structure

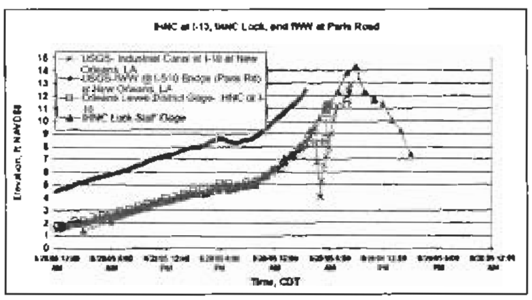
Scour Hole at BV Control Structure

Levee Breach at Bienvenue Flood Gate

100 year old West End Lighthouse

UNO 2 weeks later…

STORM SURGE EAST N.O.



STORM SURGE

hss ~ KrKs (Surge due to Δp + Wind Shear Effect)

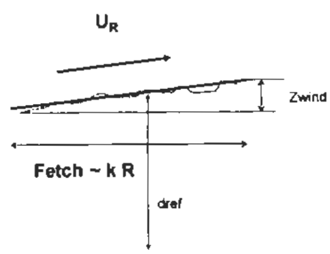
Depression Surge

$h_{\delta p} \sim (10\ m)\ (\Delta p\ mm)/760$ in m

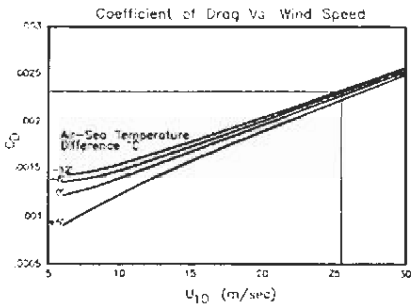
Or

$h_{\delta p} \sim (34m)\ (\Delta p\ mm)/760$ in ft

Setup



Wind Drag



Wind Shear

$$F'\tau_a = \gamma_w d\Delta z$$

$$\tau_a = \rho_a C_f U_{OW}^2$$

$$F'(\rho_a C_f U_{OW}^2) = \gamma_w d(2Z_{sup}) = g\rho_w d(2Z_{sup})$$

$$F' = kR$$

$$Uow \approx U_R$$

$$Z_{wind} = \frac{C_R R(\rho_a C_f U_R^2)}{2g\rho_w d_{ref}}$$

Surge Estimation

h_{ss} ~ KrKs (Surge due to Δp + Wind Shear Effect)

$$h_{ss} \approx K_s K_r \left[\Delta p + \frac{C_R C_f R U_R^2}{2gd_{ref}} \frac{\rho_a}{\rho_w}\right]$$

$$C_R \approx 1 \rightarrow 2$$

$$\frac{\rho_a}{\rho_w} \approx 1/800$$

$$C_f \approx 0.0025$$

$$d_{ref} \sim 10 \rightarrow 20m$$

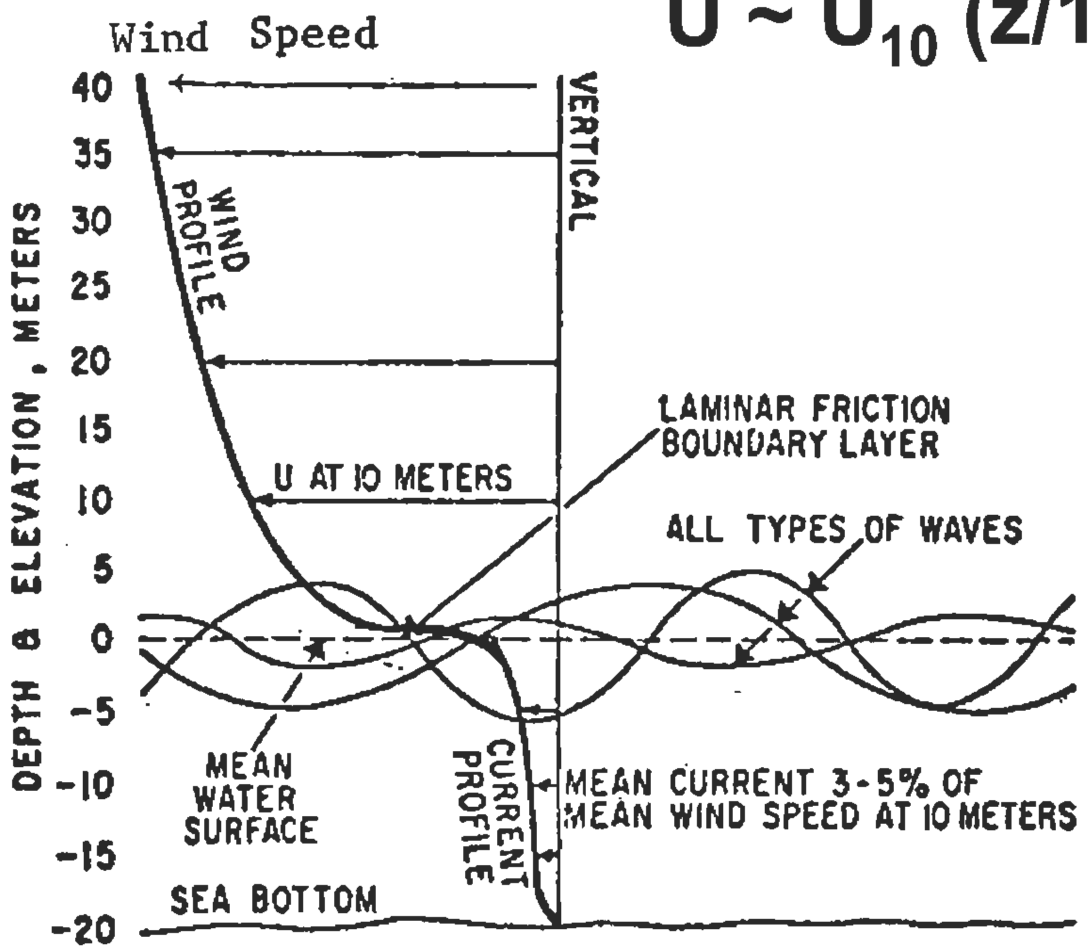
Example:

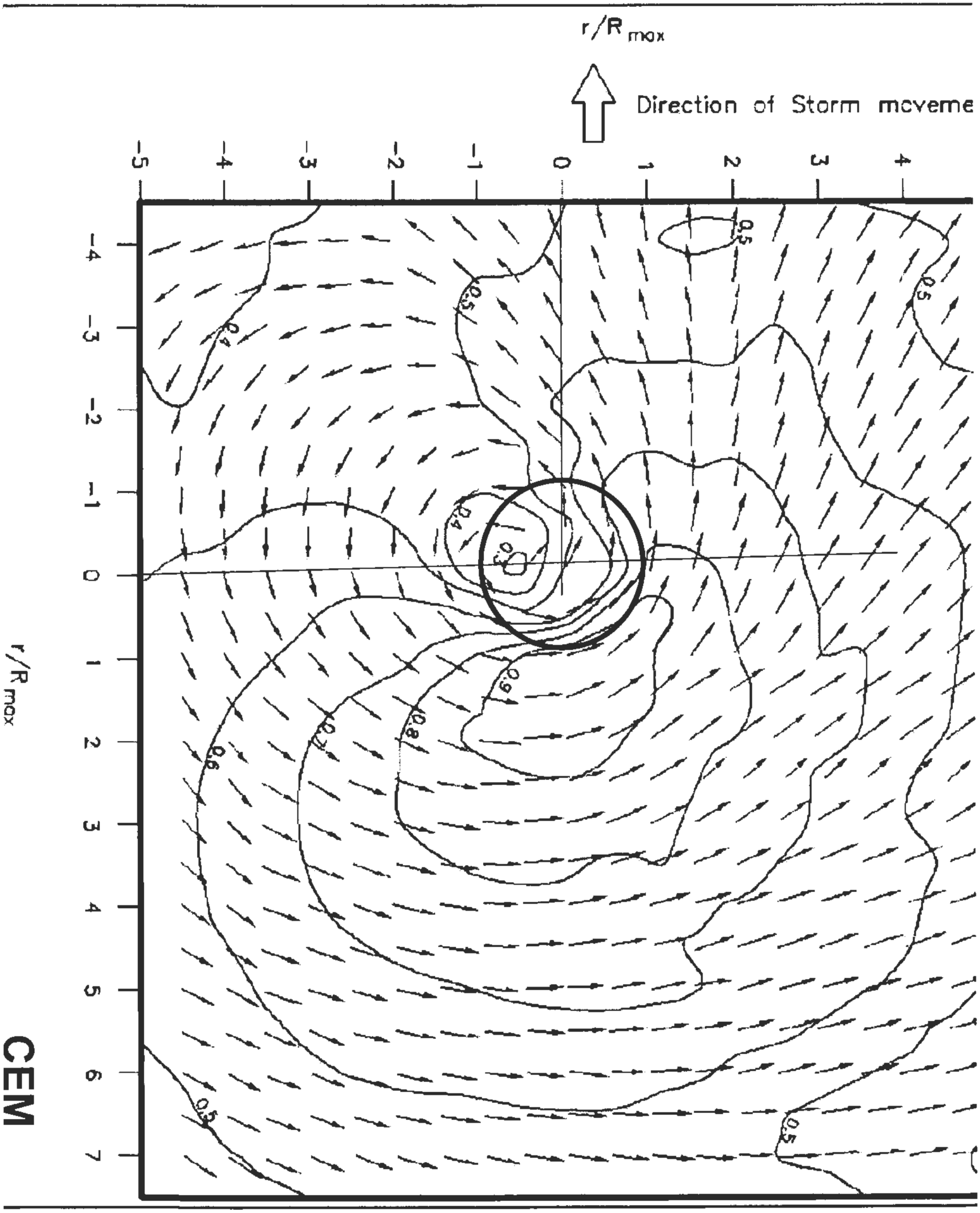
A Hurricane has the following approximate characteristics:
R = 22.5 miles;
Maximum Wind Speed ~ Umax;
Forward Speed = 16 mph;
$\Delta p \simeq$ 50 mm Hg with normal pressure at 760 mm.
Assume Latitude of 28°.
Plot the pressure and velocity on the right side of the storm.
What are the maximum significant wave height and the corresponding period?
Assume: d_{ref} ~ 10 m;
CR ~ 1;
Use $U=U_R$
KrKs ~1;
Estimate the surge height.

Δp	50	mm Hg	1.968	in Hg	po	710	mm
phi	28	degrees latitude					
R	19.54	Kt mi	22.5	mi			
VF	13.89	kt	16	mph			
f	0.25	rad/h	0.25				
pn	760	mm Hg	29.92	in Hg			
Umax	86.50	kt	99.61	mph			
UR	81.77	kt	94.16	mph	138.1	ft/sec	
α	1.2						
Ho	33.5	ft	10.22	m			
Ts	12.3	sec					
Estimate of Storm Surge (Simple Case)							
d	32.8	ft	cR	1			
hss	5.6	ft	2.726	m			

Wind Speed and Water Current

$$U \sim U_{10}\,(z/10)^{1/7}$$




r/R max
Direction of Storm moveme
r/R max
CEM
0.5
0.4
0.3
0.6
0.7
0.8
0.9

SPM

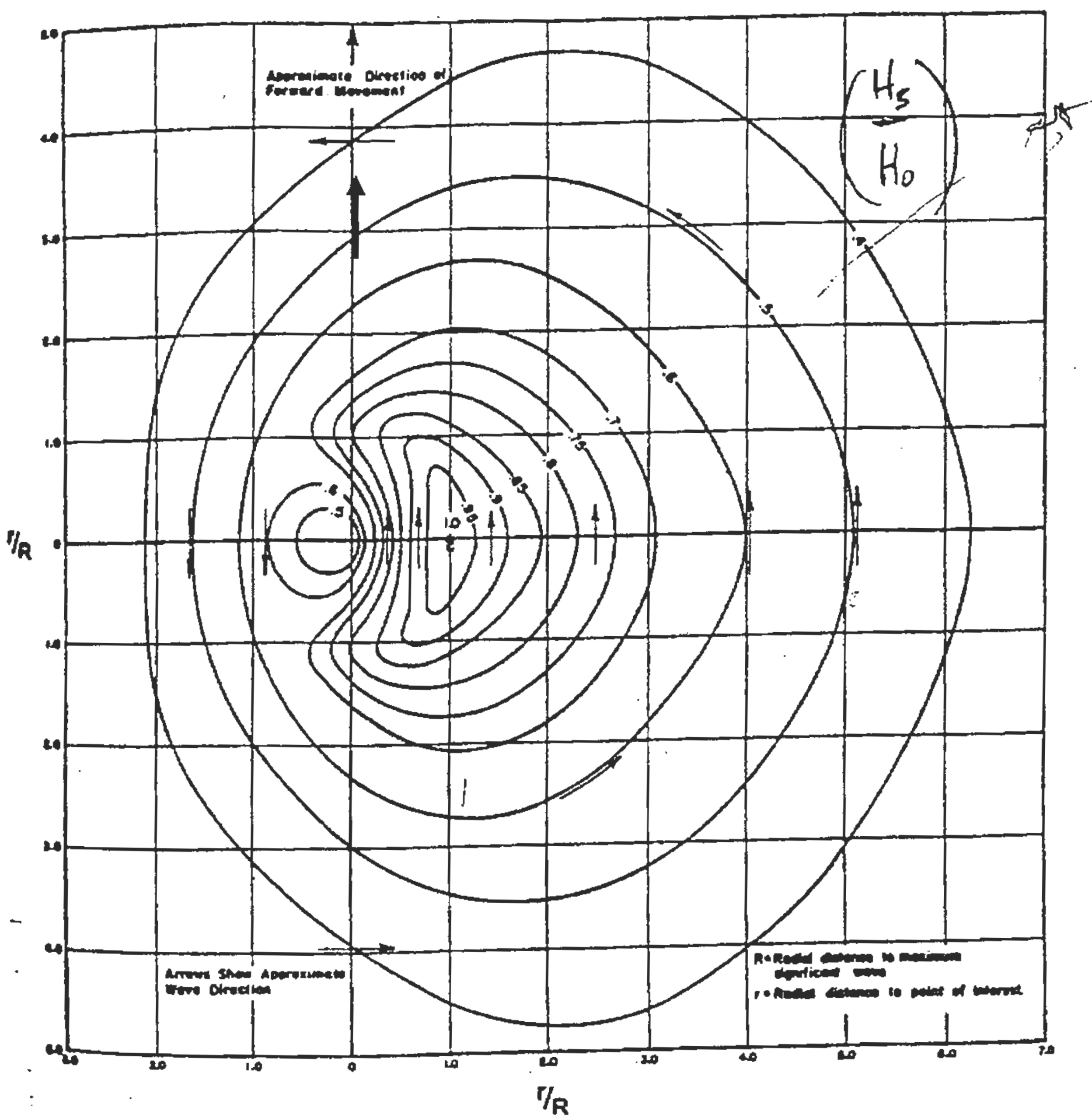
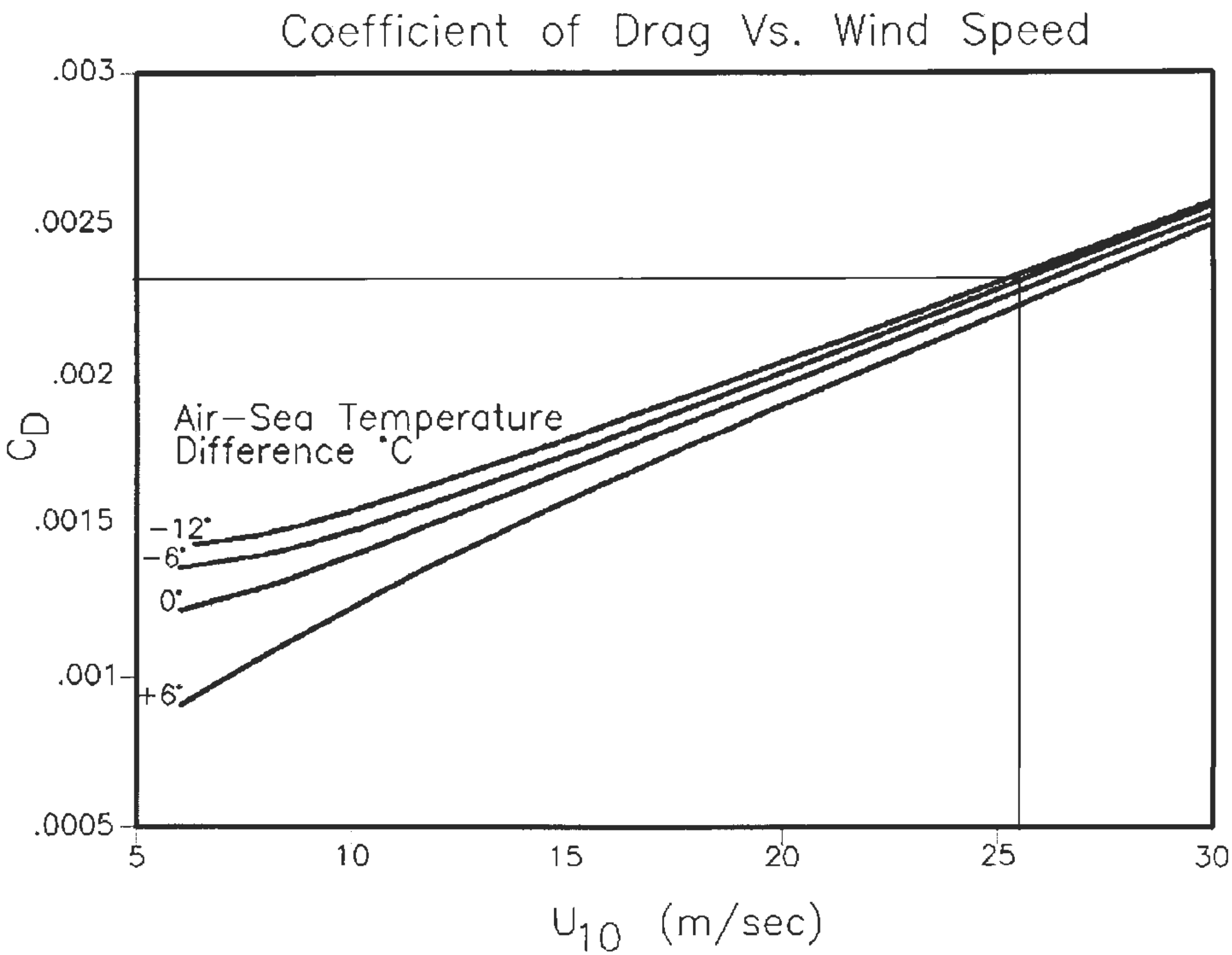


Figure 3-43. Isolines of relative significant wave height for slow-moving hurricane.

Wind Drag



Coefficient of Drag Vs. Wind Speed
.003
.0025
.002
.0015
.001
.0005
C_D
Air–Sea Temperature
Difference °C
−12°
−6°
0°
+6°
5
10
15
20
25
30
U_10 (m/sec)

Review

2nd Test

Topics

1. Wind wave generation
   - Fetch limited
   - Duration limited
2. Wind setup
3. Hurricane wave estimation
4. Longshore currents
5. Wave setup and set down
6. Longshore transport
7. Beach profile stability
8. On-offshore movement of sediment
9. Closure depth
10. Wave drift
11. Wind driven currents

Carry over from 1st part

- Lo, L,
- c, c_G
- Orbital velocity P.104 lecture 8
- Wave breaking, e.g. $H_b \sim 0.78 d_b$
- **APPENDIX I**
- **Figure C6** KsKr
- Find H_o' from H, d, T and α.

P.68 Lecture 6

Wave Generation

- Given: F= 10 miles; Ta=60°C;Ts=65°C

Uow=42 mph; td = 1 h; d = 40 ft

Find: Hs, Ts, Z sup

1) R_L (if U_L given use fig 6.5) if not $R_L = 1$ — OR 3-15
2) R_T (Fig 6.6)
3) U_A (P.66 Lecture 6)
4) Fig 3-21 use lines $\left(\frac{gd}{U_A^2}\right)$ & x-axis to find y-axis then calc. H_s
5) Fig 3-22 do same w/ Fig 3-22 solve for T_s
6) assume shallow water use $\frac{gT_m}{U_A} = \left\{\left(\frac{g t_d}{U_A}\right)\frac{1}{537}\right\}^{3/7}$ Solve for T_m (t_d in sec.)

$\Delta T = a - s$ (0.002)

P.64 Lecture 6 Fig 6.8

$$Z_{sup} = \frac{1}{800}\frac{F(C_f)(U_{ow}^2)}{2g\,d}$$

If $T_m > T_s$ $H = H_s$

If not put T_m into Fig 3-22 solve for $\frac{gF}{U_A^2}$

Plug into Fig 3-21 & solve for $\frac{gH}{U_A^2} \rightarrow H$ (New H_s)

1 mile = 0.86897 N. miles
1 N. mile = 1.1508
1 mph = 0.869 Knots = 1.467 ft/s
1 mm = 0.394 inches
1 inch = 25.4 mm

$\frac{\rho_a}{\rho_w} \sim \frac{1}{800}$

Hurricane

- Δp = 70 mm Hg (need in inches)
- VF = 15 mph (put in knots)
- R = 20 miles (put in N. miles)
- Latitude 30° (phi)
- α = 1.2

Find: Hos, Tos

1) $U_{max} = a\{b\,\Delta p^{1/2} - c R f\}$ (Δp in inches, R in N. miles); $f = 2\left(\frac{2\pi}{24}\right)\sin\phi$ (in degrees)

$a = 0.868$ knots, $b = 73$ miles, $c = 0.57''$ Hg — give in Knots — P. 73 lecture 6 cont.

2) $U_R = 0.865\,(U_{max}) + 0.5\,(VF)$ (knots) — given in knots

3) $T_s = A\left(1 + \frac{B\alpha V_F}{U_R^{1/2}}\right)e^{\frac{R\Delta p}{C}}$ — given in sec. $A = 8.6$, $B = 0.104$, $C = 200$ P. 72

4) $H_{os} = A_o\left(1 + \frac{B_o \alpha V_F}{U_R^{1/2}}\right)e^{\frac{R\Delta p}{C_o}}$ — given in feet. $A_o = 16.5$, $B_o = 0.208$, $C_o = 100$

5) $h_{ss} \sim K_s K_\alpha \cdot \left(\Delta p + \frac{C_R C_F R U_R^2}{2g\, d_{ref}}\left(\frac{\rho_a}{\rho_w}\right)\right)$

$C_F = 0.0025$, $\frac{\rho_a}{\rho_w} \sim \frac{1}{800}$, $C_R = 1$ p. 74

hss given in ft

Lecture 7 p. 76

V_b

- Given: H_b = 8 ft; T = 6s; α_b = 22°; beach slope 5%
- Find v_b max and v_b with friction.

$U_{bmax} = M_1\, m\,(g H_b) - \sin(2\alpha_b)$ (m = slope)

- $v_{bmax} = 20.7\, m\,(gH_b)^{1/2}\sin(2\,\alpha_b)$

Use Fig 7.1b

mul. V_{bmax} by 0.582 to get V_{bmax} w/ friction

Longshore transport

- Given: H_b= 8 ft; T= 6s; α_b = 22°; beach slope 5%; td = 12 h.
- Find: Qls

Excel

- Given: fij Table and Hoi, αoj
- Find: Annual net and gross Qls

Re-organize

NOTE:
W and E are split
Into two 22.5° BINS; all other BINS are 45°.

Excel

		Wind	West	SW	S	SE	East	
		WWNW	WWSW	SW	S	SE	EESE	EENE
H ft	α_o	>90	78.75	45	0	-45	-78.75	<-90
0.5	1		2	5	7	8.5	1.5	
1	2		1	3	4	**8**	1.25	
2	3		0.5	1	1.5	2	0.5	
3	4		0.25	0.5	1	1.5	0.5	
4	5		0.1	0.2	0.4	0.6	0.125	
5	6		0.025	0.04	0.04	0.2	0.02	

Estimating Annual Transport

Excel

$$Q_{ls\ net} \approx K_y \sum_i \sum_j f_{ij} \{H_{oi}^{5/2} (\cos \alpha_{oj})^{1/4} \sin 2\alpha_{oj}\}$$

Apply this equation to each bin to find the contribution to Qls from that BIN.

Example: SW BIN (-45o) at Ho 1 to 2 ft

ΔQls = 137000*(8/100)*(1.5^2.5)*[cos(-45°)]^0.25*sin(2*(-45°))

= -28000 yd3/year

P.98 Lecture 8

Beach profile stability

- Given: D50 = 0.26 mm; Temperature of water=30°C. Exposed
- What is the stable beach slope?

Fig 7.2

$1\ \frac{ft}{sec} = 0.3048\ \frac{m}{s}$

1.180 lecture 8

On-offshore movement of Sediment

- Given: H_o= 8 ft; T= 6s; D50 = 0.26 mm; Temperature of water=30°C. Exposed.
- Determine if this storm will result in Berm or Bar formation.
- What is the closure depth for this wave?

Fig 8.14
Fig 4-29
Fig 4-30

$L_o = \frac{gT^2}{2\pi}$

Fig 4-31 To Find w (Fall velocity) put into Ft/s

Plug into $F_o = \frac{H_o}{V_s T}$ as V_s solve F_o

$F_o > 1$ offshore deposition

$F_o < 1$ Onshore deposition

Fig 8.8 gives closure depth (long period waves 15s)

Fig 4-22 critical bottom depth Find critical veloc. (y-axis)

orbital velocity = critical veloc.

$\cosh(2\pi(d+z)/L) \to 1$

appendix

$L_o \to \frac{d}{L_o} \to \frac{d}{L}$

change d

Review

2nd Test

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   - Duration limited
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7. Beach profile stability
8. On-offshore movement of sediment
9. Closure depth
10. Wave drift
11. Wind driven currents

Carry over from 1st part

- Lo, L,
- c, c_G
- Orbital velocity
- Wave breaking, e.g. $H_b \sim 0.78 d_b$
- **APPENDIX I**
- **Figure C6** KsKr
- Find H_o' from H, d, T and α.

Wave Generation

- Given: F= 10 miles; Ta=60°C;Ts=65°C

Uow=42 mph; td = 1 h; d = 40 ft

Find: Hs, Ts, Z_{sup}

Hs= 4.6 ft
Ts=4 s

Zsup=0.32 ft by FU^2/(1400d)
=0.002*(42*5280/3600/3.28)^2*10*5280/(800*2*32.2*40)
= 0.20 ft

Hurricane

- Δp = 70 mm Hg
- VF= 15 mph
- R = 20 miles
- Latitude 30°
- α = 1.2

Find: Hos=35.5 ft, Tos=12.7 sec

v_b

- Given: H_b= 8 ft; T= 6s; α_b = 22°; beach slope 5%
- Find v_b max and v_b with friction.
- v_{bmax} = 20.7 m $(gH_b)^{1/2}\sin(2\alpha_b)$=11.5 ft/sec
- V with friction ~ 0.5*11.5=5.8 ft/sec

Longshore transport

- Given: H_b= 8 ft; T= 6s; α_b = 22°; beach slope 5%; td = 12 h.
- Find: Qls~32000 yd3

$$P_{ls} = \frac{1}{16}\rho g H_b^{5/2}\sqrt{\frac{g}{K_b}}\sin 2\alpha_b$$

$$Q_{ls} = K_Q P_{ls}$$

- Given: fij Table and Hoi, αoj
- Find: Annual net and gross Qls

Re-organize

NOTE:
W and E are split
Into two 22.5° BINS; all other BINS are 45°.

		Wind	West	SW	S	SE	East	
		WWNW	WWSW	SW	S	SE	EESE	EENE
H ft	α_o	>90	78.75	45	0	-45	-78.75	<-90
0.5	1		2	5	7	8.5	1.5	
1	2		1	3	4	**8**	1.25	
2	3		0.5	1	1.5	2	0.5	
3	4		0.25	0.5	1	1.5	0.5	
4	5		0.1	0.2	0.4	0.6	0.125	
5	6		0.025	0.04	0.04	0.2	0.02	

Estimating Annual Transport

$$Q_{ls\ net} \approx K_y \sum_i \sum_j f_{ij} \{H_{oi}{}^{5/2} (\cos \alpha_{oj})^{1/4} \sin 2\alpha_{oj}\}$$

Apply this equation to each bin to find the contribution to Qls from that BIN.

Example: SW BIN (-45o) at Ho 1 to 2 ft

ΔQls = 137000*(8/100)*(1.5^2.5)*[cos(-45°)]^0.25*sin(2*(-45°))

= -28000 yd3/year

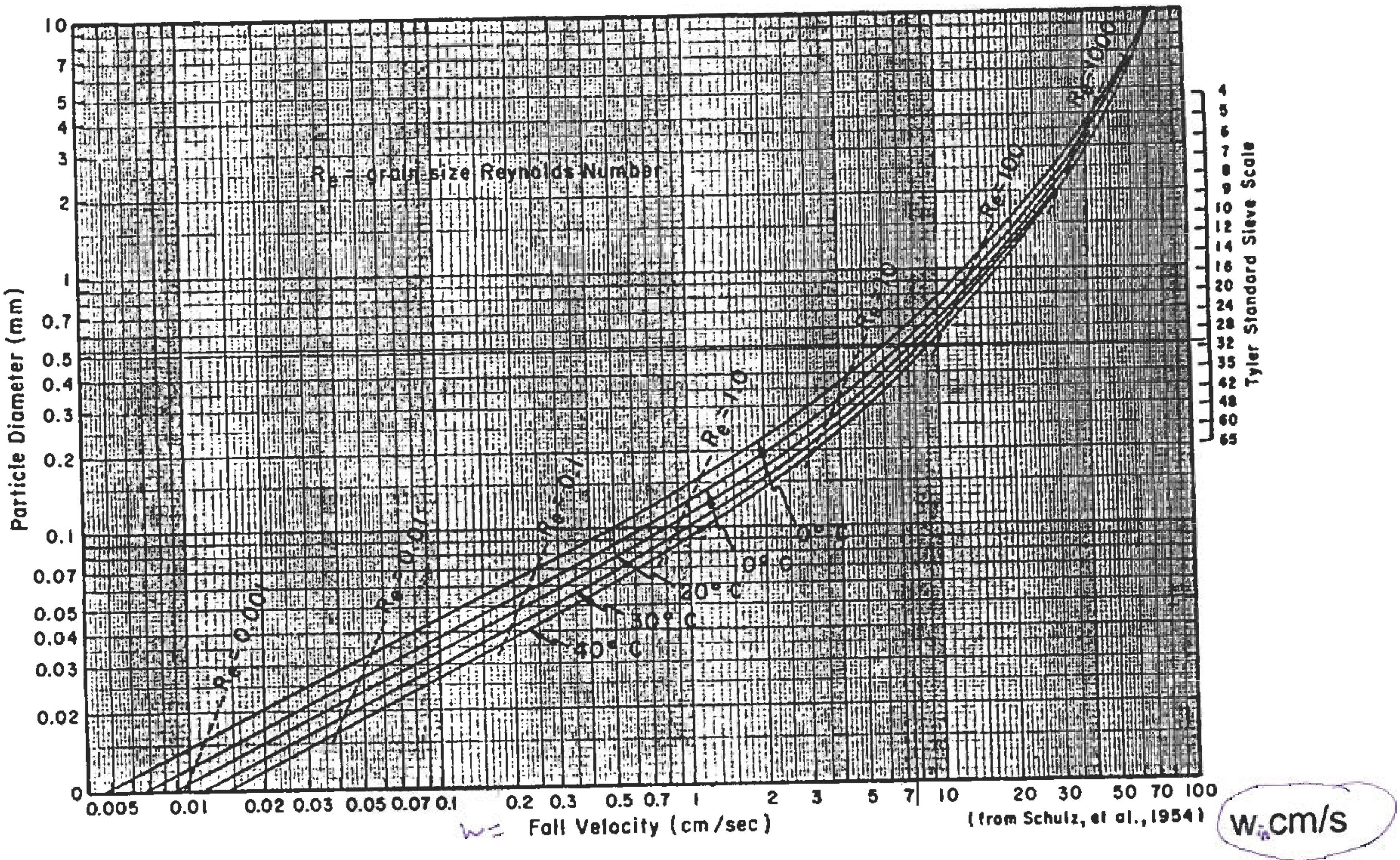
Beach profile stability

- Given: D50 = 0.26 mm; Temperature of water=30°C. Exposed
- What is the stable beach slope?
- m ~ 0.02

On-offshore movement of Sediment

- Given: H_o= 8 ft; T= 6s; D50 = 0.26 mm; Temperature of water=30°C. Exposed.
- Determine if this storm will result in Berm or Bar formation.
- W~3.5 cm/s; Lo=184.5 ft; Ho/Lo=0.043
- Pi w/gT=0.0019 Bar formation
- What is the closure depth for this wave?

H	8	ft		
T	6	s		
D50	0.26	mm	0.0009	ft
Temp	20	oC		
m	0.02			
w=Vf	3.5	cm/s	0.1148	ft/sec
Lo	184.5	ft		
L	183.2	ft	0.000281805	
dclosure	82	ft	0.004757581	
Ub	0.505	ft/sec		
Ub crit	0.5	ft/sec	Fig 4-22	



$Re = \frac{V_s D}{\nu g}$ Figure 4-31. Fall Velocity of Quartz Spheres in Water as a Function of Diameter and Temperature

$L_0 = \frac{gT^2}{2\pi}$ $H/L_0 =$ $\frac{\pi w}{gT}$ Fig 8.14

H	30	Wind	West	SW	S	SE	East	
		WNW	WWSW	SW	S	SE	EESE	ENE
		>90	78.75	45	0	-45	-78.75	<90
0.5	1		2	5	7	8.5	1.5	
1	2		1	3	4	8	1.25	
2	3		0.5	1	1.5	2	0.5	
3	4		0.25	0.5	1	1.5	0.5	
4	5		0.1	0.2	0.4	0.6	0.125	
5	6		0.025	0.04	0.04	0.2	0.02	

GIVEN:						
H ft	W	SW	S	SE	E Wind Direction	Σ
	Percent	of Year				
0.5-1	4	5	7	8.5	3	27.5
1 to 2	2	3	4	8	2.5	19.5
2 to 3	1	1	1.5	2	1	6.5
3 to 4	0.5	0.5	1	1.5	1	4.5
4 to 5	0.2	0.2	0.4	0.6	0.25	1.65
5 to 6	0.05	0.04	0.04	0.2	0.04	59.65

Qls	137000	radians	1.37445	0.785398	0	-0.785398	-1.3744
	H ft						
	0.75		339	3060	0	-5202	-255
	1.5		960	10386	0	-27696	-1200
	2.5		1722	12415	0	-24830	-1722
	3.5		1996	14396	0	-43187	-3993
	4.5		1497	10793	0	-32380	-1871
	5.5		618	3565	0	-17825	-494
		Sum	7132	54615	0	-151119	-9534

365354.4672 29228.36

18.26667 32.52525

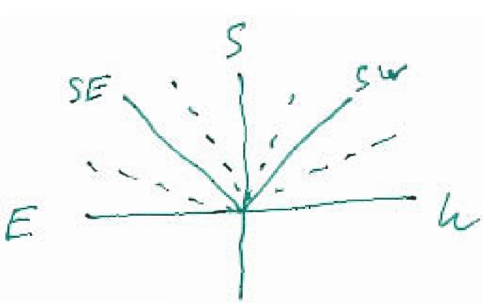
W-E 61747 yd3/year

E-W 160653 yd3/year

NetQls -98906 or 98906 yd3/year E-W

Gross 222400 yd3/year

$$Q_{ls\,net} = K_y H_{oi}^{5/2} (\cos \alpha_{oj})^{1/4} \sin 2\alpha_{oj} \left(\frac{\%}{100}\right)$$



Potential Longshore Qls

The SPM gives the Potential Longshore transport approximation:

$$Q_{ls} \approx K_y H_o^{5/2} (\cos \alpha_o)^{1/4} \sin 2\alpha_o$$

Where Ky ~ 137000 for Ho in ft and Qls in yd3/year

Estimating Annual Transport

$$Q_{ls\ net} \approx K_y \sum_i \sum_j f_{ij} \{H_{oi}^{5/2} (\cos \alpha_{oj})^{1/4} \sin 2\alpha_{oj}\}$$

Apply this equation to each bin to find the contribution to Qls from that BIN.

Example: SW BIN (-45o) at Ho 1 to 2 ft

ΔQls = 137000*(8/100)*(1.5^2.5)*[cos(-45º)]^0.25*sin(2*(-45º))

= -23000 yd3/year

Lecturc 8
Sediment Transport in the Littoral Zone
Part II

Computation of Net and Gross Annual Longshore Transport

The SPM gives an approximation that makes Eq. 7.9 easier to use, i.e.

$$Q_{ls} \sim K_y H_o^{5/2} (\cos \alpha_o)^{1/4} \sin 2\alpha_o \qquad 8.1$$

where Ky ~ 1.37×10^5 is the calibration constant for Qls expressed in cubic yards per year.
The SPM formula (Eq. 8.1) estimates the annual rate of transport as if the Ho value persisted for a whole year from a specified direction αo. The actual wave distribution is based on the wind speeds and directions. Thus it is common to use a wind rose to describe the wind speed and directional distribution by a wind rose (see attached Figure 8-1 for midLake Pontchartrain; the details of the frequencies of the wind speed in the various classes is given in Table 8-1). A wind rose separates the wind into classes or bins of direction and speed with an assigned probability of occurrence associated with each class (see attached table). The distribution is expressed as the fraction of the time (e.g. fraction of a year) that the wind follows into a certain speed and direction class.The ***net*** annual transport is then found from

$$Q_{ls\,net} = K_y \Sigma_i \Sigma_j \{H_{oj}^{5/2} (\cos \alpha_{oi})^{1/4} \sin 2\alpha_{oi}\} f_{ij} \qquad 8.2$$

where (α_{oi}) = +ve if movement of littoral material to the right,
(α_{oi}) = -1ve if movement of littoral material to the left,
f_{ij} = probability of occurrence for the class i,j (i.e. fraction of the year that a wind in this class exists).

The **gross** transport is similar to Eq. 8.2 except that absolute movements to the right and left are added, e.g.

$$Q_{ls\,g} = K_y \Sigma_i \Sigma_j \mathbf{Abs}[\{H_{oj}^{5/2} (\cos \alpha_{oi})^{1/4} \sin 2\alpha_{oi}\} f_{ij}] \qquad 8.3$$

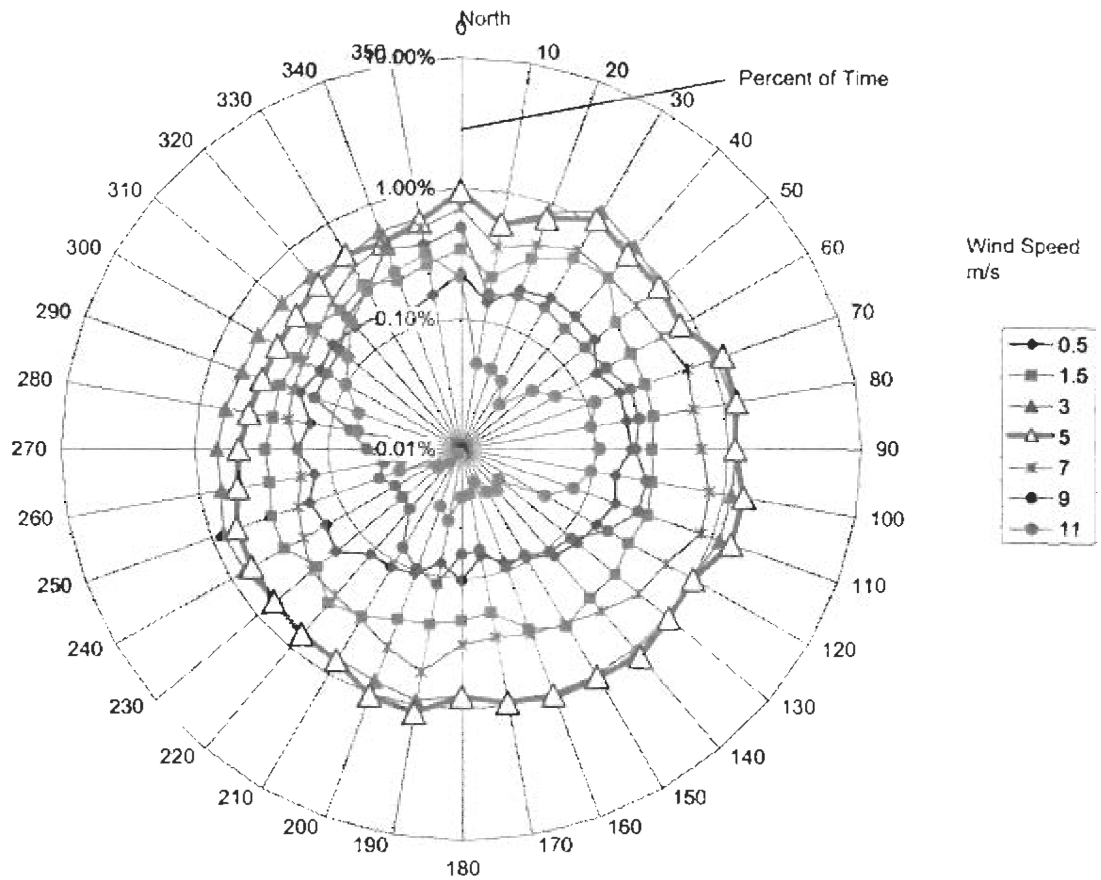


Figure 8-1 Annual Windrose for Lake Pontchartrain (MidLake) NOAH wind data

Percent of time in each Wind Class

DIR.	calm< 1m/s	2.7	4.5	7.2	10.1	13.0	16.2	19.7	23.9	27.6	31.9	Sum
N	0.717	1.092	1.483	1.792	0.792	0.225	0.000	0.000	0.000	0.000	0.000	6.100
NNE	1.100	1.525	2.058	1.875	0.375	0.033	0.000	0.000	0.000	0.000	0.000	6.967
NE	1.042	1.575	2.158	2.033	0.308	0.025	0.000	0.000	0.000	0.000	0.000	7.142
ENE	0.792	1.517	2.358	1.992	0.400	0.100	0.017	0.000	0.000	0.000	0.000	7.175
E	0.842	1.783	3.075	2.475	0.442	0.117	0.042	0.000	0.000	0.000	0.000	8.775
ESE	0.808	1.675	2.958	2.333	0.425	0.083	0.000	0.000	0.000	0.000	0.000	8.283
SE	0.875	1.633	2.767	1.717	0.175	0.017	0.000	0.000	0.000	0.000	0.000	7.183
SSE	0.817	1.742	2.658	1.242	0.150	0.008	0.000	0.000	0.000	0.000	0.000	6.617
S	0.642	1.367	2.492	1.308	0.167	0.017	0.000	0.000	0.000	0.000	0.000	5.992
SSW	0.833	1.250	2.392	1.483	0.142	0.008	0.000	0.000	0.000	0.000	0.000	6.108
SW	0.958	1.317	1.900	1.192	0.100	0.000	0.000	0.000	0.000	0.000	0.000	5.467
WSW	0.917	1.308	1.608	0.808	0.100	0.008	0.000	0.000	0.000	0.000	0.000	4.750
W	0.950	1.250	1.333	0.717	0.125	0.033	0.000	0.000	0.000	0.000	0.000	4.408
WNW	0.833	1.083	1.050	0.850	0.292	0.083	0.017	0.000	0.000	0.000	0.000	4.208
NW	0.825	0.967	1.192	1.058	0.467	0.175	0.042	0.017	0.000	0.000	0.000	4.742
NNW	0.742	0.933	1.242	1.667	0.842	0.300	0.025	0.017	0.000	0.000	0.000	5.767
VAR												
CLM												
ALL	13.750	22.000	32.717	24.600	5.333	1.367	0.192	0.050	0.000	0.000	0.000	

Table 8-1

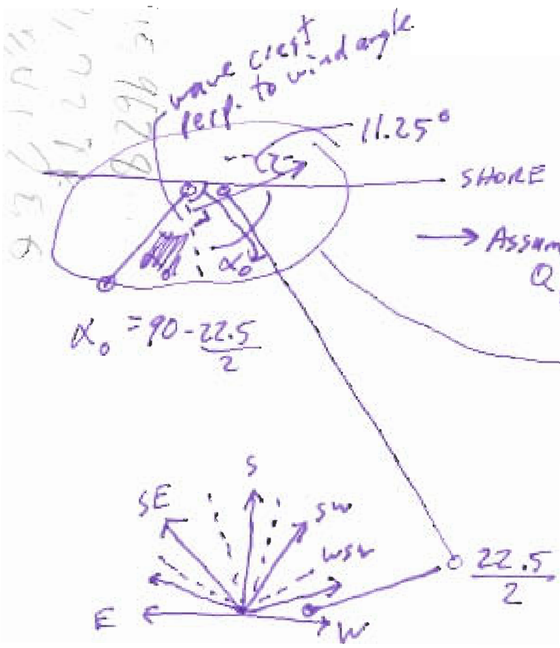
$$Q_{ls\ net} \sim K_y \Sigma_i \Sigma_j f_{ij} \left(H_{0i}^{5/2} (\cos \alpha_{0j})^{1/4} (\sin 2\alpha_{0j}) \right)$$

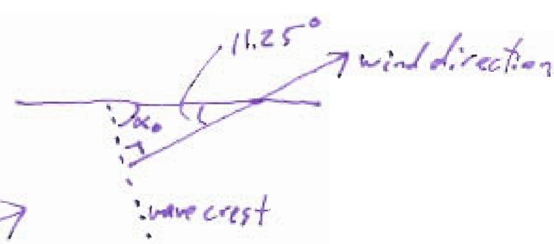
Assignment 8.1. For the wave rose represented in the following table, estimate:

1. The longshore transport to the right,
2. The longshore transport to the left,
3. The net longshore transport,
4. The gross longshore transport.

H ft	W	SW	S	SE	E Wind Direction
0.5-1	2	6	6	8	2
1-1.5	1	6	7	7	5
1.5-2	1	2	3	3.5	2
2-3	0.5	1	1	1	0.7

The shoreline runs due east-west. The beach slope is 1%.





f_{ij} = fraction that wave exist in particular direction @ a particular height

useing $Q_{ls} = K_y H_0^{5/2} (\cos \alpha_0)^{1/4} \sin 2\alpha_0$

an East wind = ∅ not True so need to fix

using 11.25 wind angle yield transport (using increments i.e 45° may change)

so east & west b/c parallel we assume uses ½ toward shore & ½ to offshore

Onshore-Offshore Transport

The onshore or offshore movement of littoral material depends on the storm waves/swell waves, storm stage, beach grain size, angle of repose and the beach slope. Figures 8-2a and 8-2b show the forces on a grain on a beach. The attached Figure 8-3 (SPM 7.20) shows the stable beach slope as a function of grain size. The figure also shows the effect of protection on beaches. Protection refers to offshore limitations on the wave energy that can reach the beach, e.g. offshore reefs.

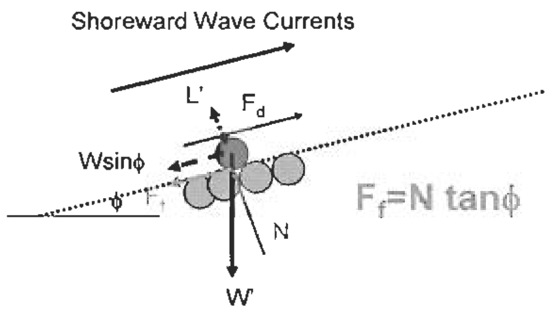


Figure 8-2a

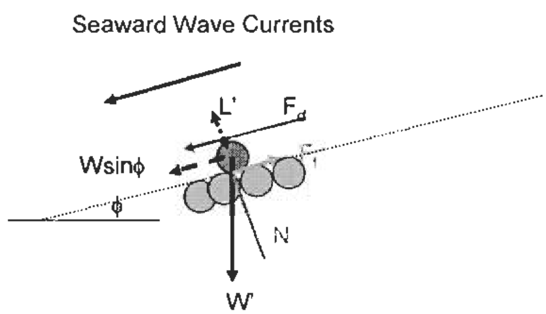


Figure 8-2b Forces on a grain on a beach.

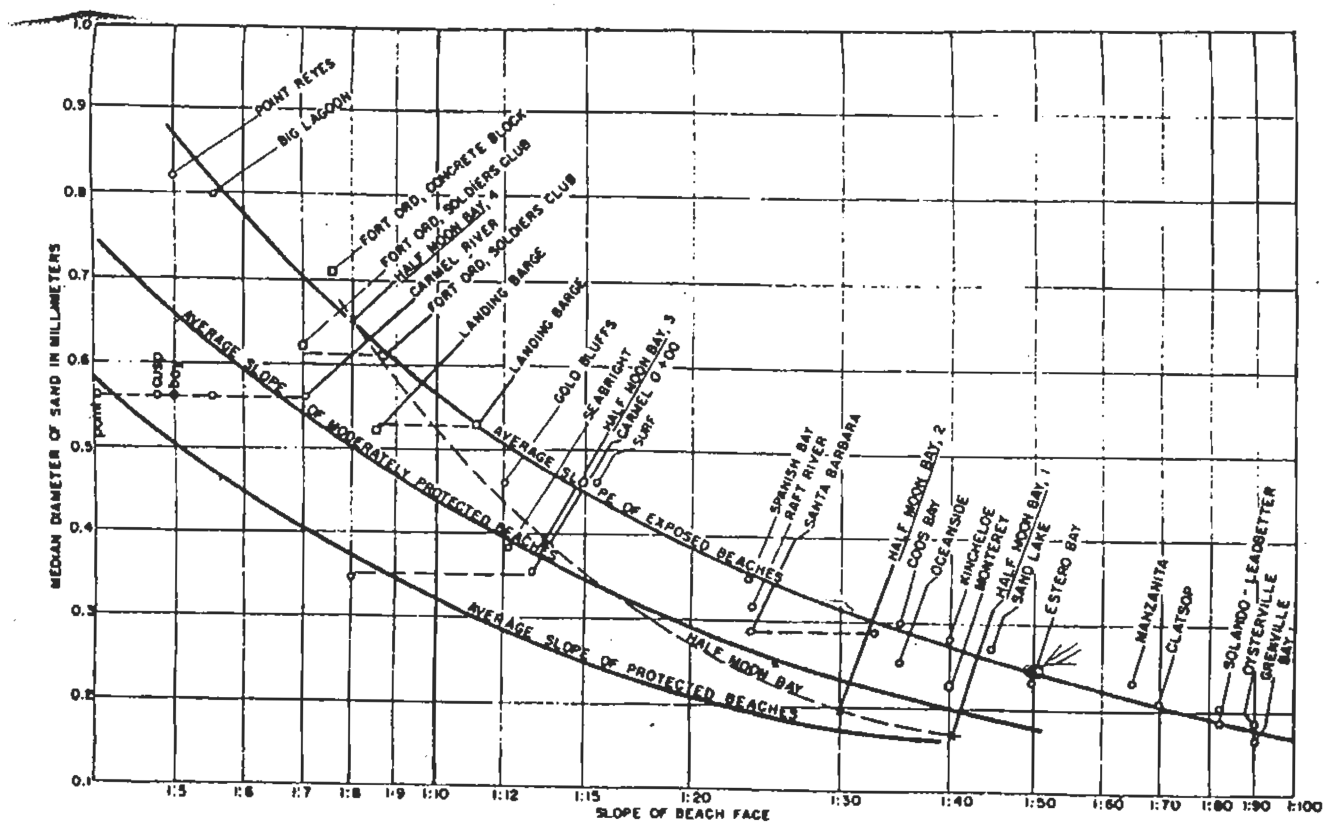


Figure 7.20. Relationship between beach slope and sand side at the mid-tide level for Pacific coast beaches (3).

Figure 8-3 Stable beach slopes from SPM

Figure 8-4 (1-7) shows the shoreline response to seas and swells. In general storm waves occur at high water levels and result in shoreline erosion and net offshore transport of sediment. On the other hand long swells and smaller wave heights tend to move sediment onshore and rebuild the beach. If the storm waves erode the dunes, the mechanism for restoring these landforms is inter-storm onshore movement of sand by waves followed by wind driven transport to the dunes.

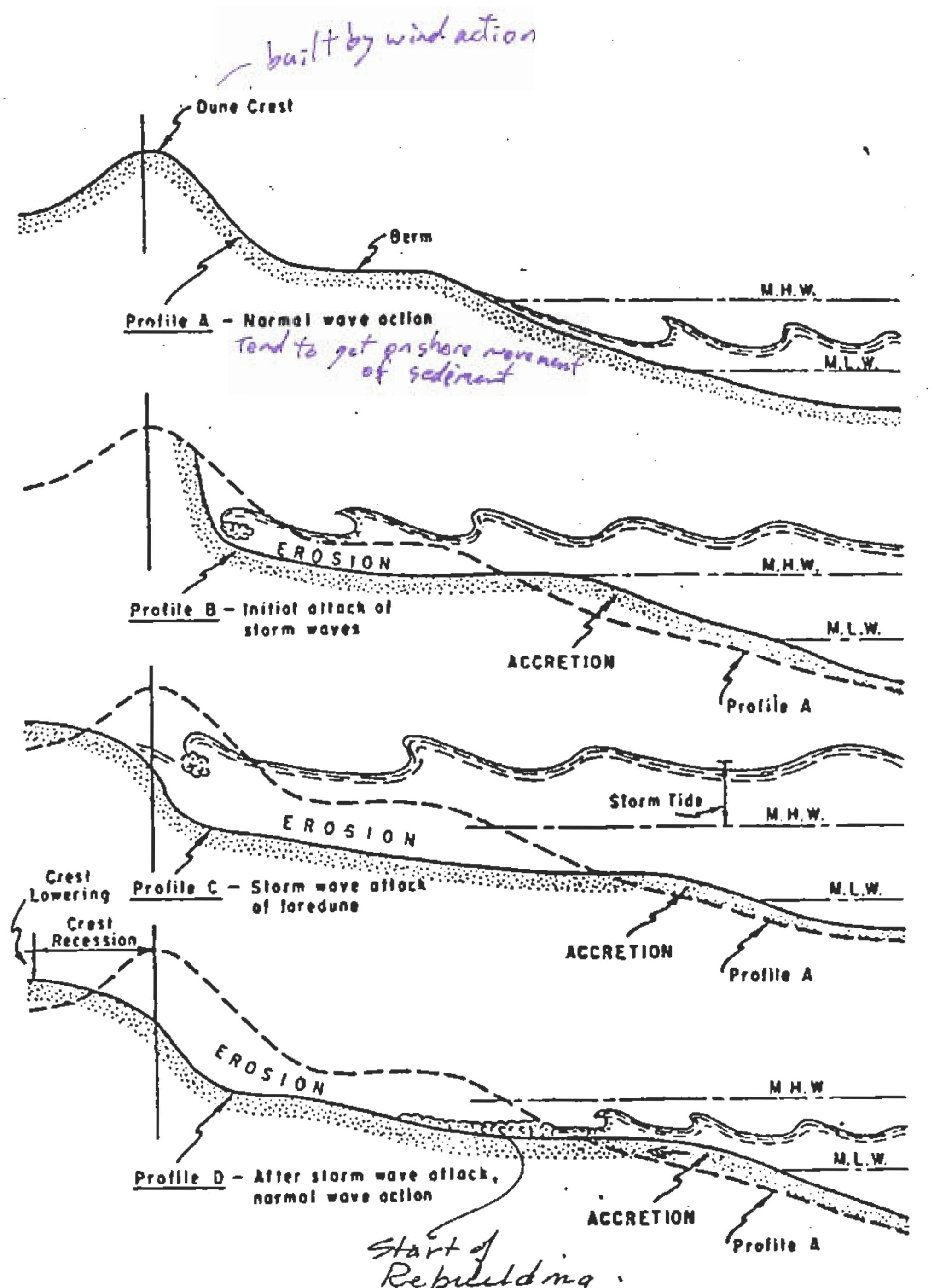


Figure 1-7. Schematic Diagram of Storm Wave Attack on Beach and Dune

Figure 8-4 Coastline responses to Storm and Inter-storm waves

Fig 4-31 useful

In Figure 8-5, Dean and Dalrymple have presented a chart to estimate when wave will tend to move sand onshore or offshore.

$w = V_f = V_s$ = Fall velocity depends on book

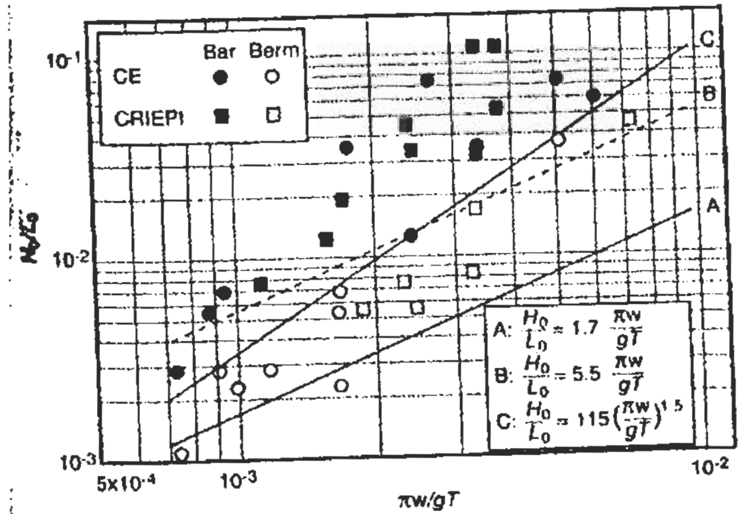


Figure 8.14 Criteria for the generation of storm or normal profiles (from Kraus and Larson 1988).

Figure 8-5 Criterion for Onshore (Berm) and Offshore (Bar) Movement of Sand ***[w is the fall velocity; T = wave period; Ho = deep water equivalent wave height; Lo = deep water wave length]***

The USACE has developed two charts that indicate the tendency of littoral material to move off or onshore. Their approach is to present the onshore/offshore observations on a regime chart that plots:

a) Dimensionless Fall Time versus Deep water wave steepness {Figure 8-6 (4-29 SPM▲73)}

a) Dimensionless Fall Time versus Wave Height/Grain Size {Figure 8-7 (4-30 SPM▲73)}

distance partical would settle in one wave period

COE approach: F_0 = Dimensionless Fall Time = Ho/(Vs T)

In the 1984 SPM, the criteria is simplified to

$F_0 > 1$ Offshore deposition

$F_0 < 1$ Onshore deposition

Example: Given a median grain size of 0.25 mm and deep wave height of 2 ft with a period of 3 seconds. The beach slope is 2%.
a)Will the net motion be onshore or offshore? Use Figures 4-29 and 4-30.
b) Is the beach slope likely to be stable or unstable? Assume that the beach is moderately protected.

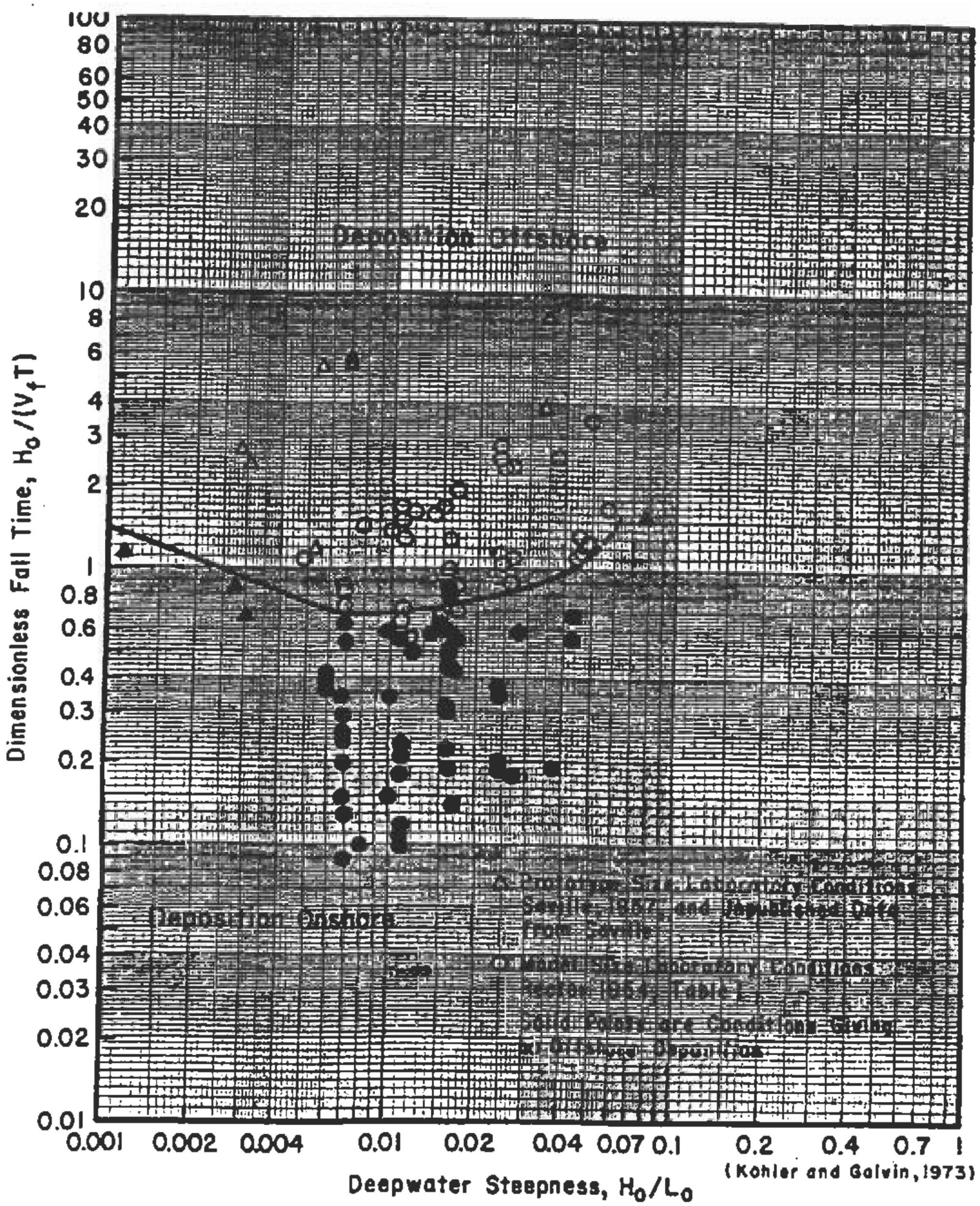


Figure 4-29. Berm-Bar Criterion Based on Dimensionless Fall Time and Deep Water Steepness

4-82

Figure 8-6. Onshore/Offshore Criteria

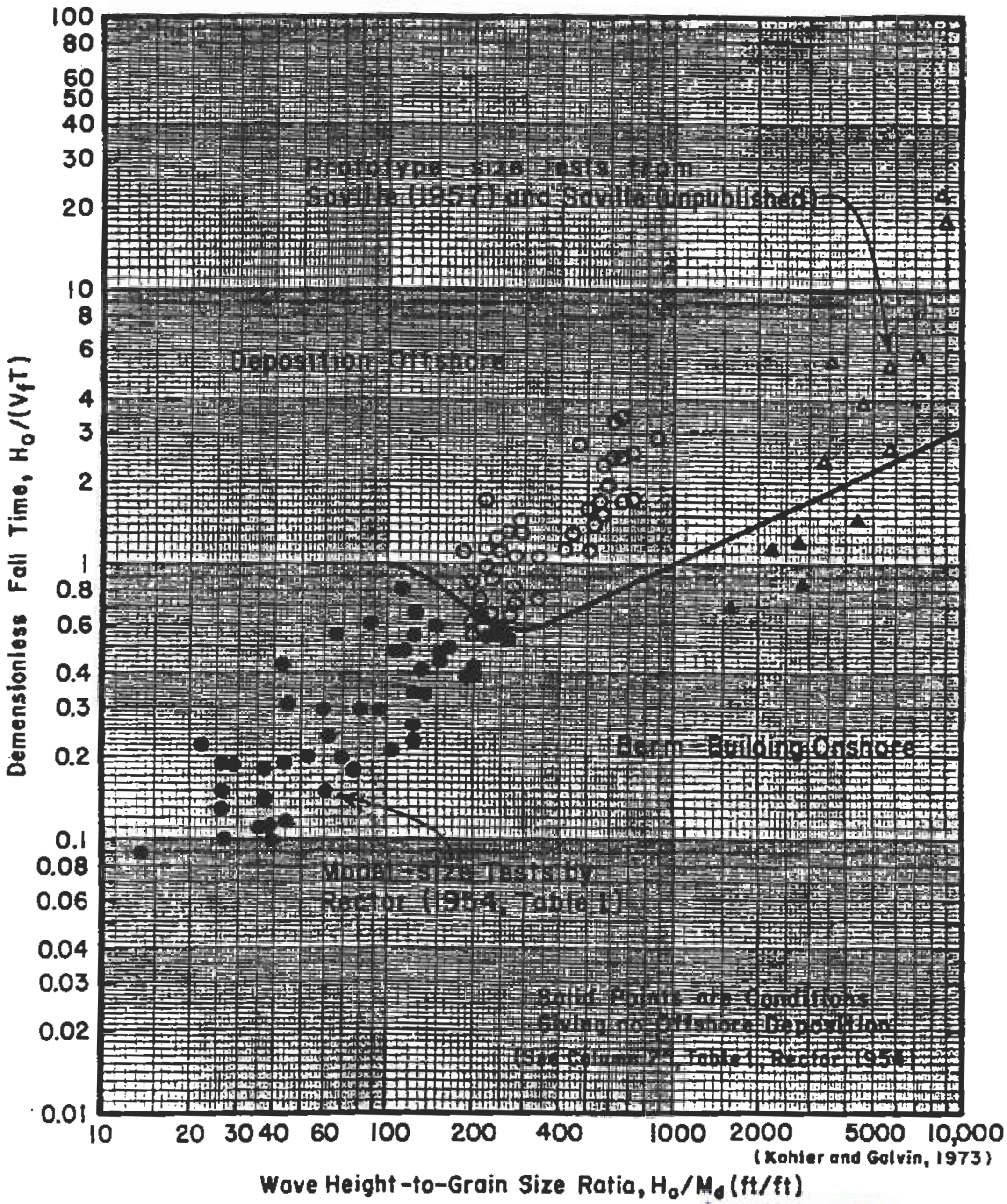


Figure 4-30. Berm-Bar Criterion Based on Dimensionless Fall Time and Height-to-Grain Size Ratio

median grain size D_{50}

4-83

Figure 8-7. Onshore/Offshore Criteria

Dean and Dalrymple have defined the 'Closure depth' as the depth at which the bed velocity (shear) will not move the grains. Figure 8-8 can be used to estimate this depth for T = 15s. Figure 8-9 can be used along with orbital currents to extend this curve.

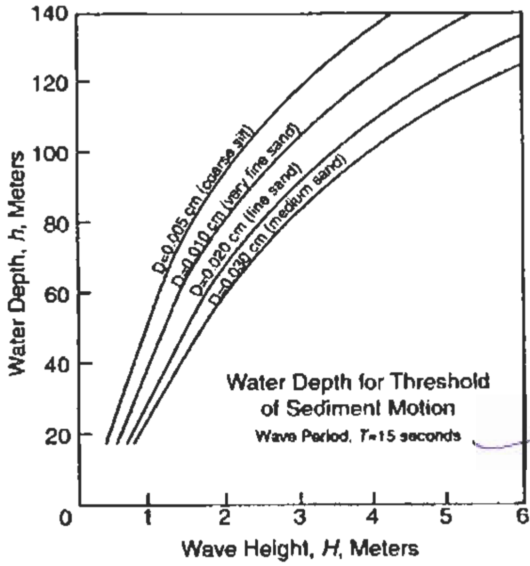
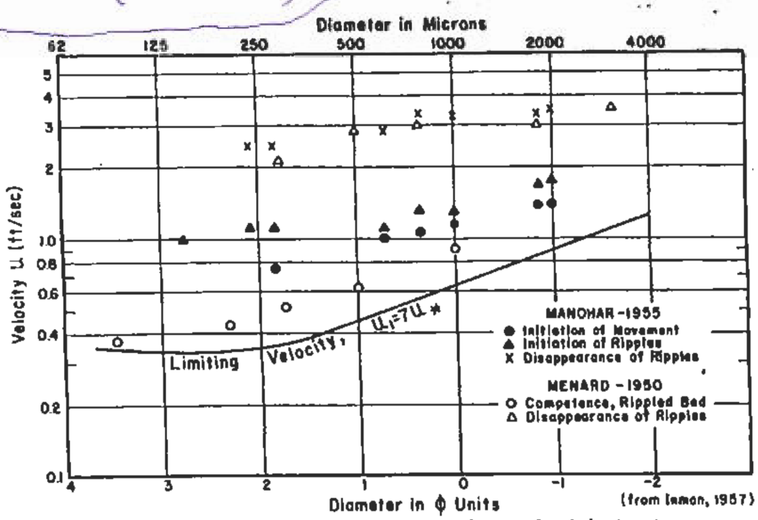


Figure 8-8. Closure Depth as a Function of Wave Height and Sand size for T =15 s.

(no activity)

use to find closure depth for short period waves



Limiting or Minimum Velocity for the Initiation of Motion of Sand of a Given Size.

Limiting velocity, arbitrarily defined as equal to $7u_{*c}$ where u_* is the threshold or critical friction velocity. (Inman, 1949.) For unidirectional flow this relation would give a limiting velocity equivalent, for example, to the mean velocity measured 1 foot above a bottom which has a roughness length of 2 cm. Field observations near the surf zone indicate that planation and disappearance of ripples does not occur unless the maximum velocity associated with the wave crest somewhat exceeds that listed by Menard and Manohar.

Figure 4-22. Initiation of Ripple Motion

Figure 8-9 Initial Motion

The orbital velocity can be estimated from

Horizontal max {+ or -}

$U = (H/2)*(gT/L)*\cosh(2\pi(d+z)/L)/\cosh(2\pi d/L)$

or at $z = -d$

$U_b = (H/2)*(gT/L)/\cosh(2\pi d/L)$

bottom velocity

Example. Determine the closure depth for the following wave:
H = 6 ft;
T = 7 sec.
Grain size = 0.5 mm
Solution:

Offshore Currents

Offshore currents can be in the form of underflow, overflow or rip currents. Under storm wave conditions it is possible to have onshore currents near the surface and underflow currents near the bottom; however, storm waves also cause rip currents. Rip currents are concentrated offshore jets that extend from the shoreline to outside of the surf zone {see http://www.ripcurrents.noaa.gov/overview.shtml }. These currents can exceed 1 m/s and can transport large amounts of material offshore. Figure 8-10 shows schematically the nature of rip currents. Figure 8-11 shows a plan view photograph of a rip currents. The relationship of sand bars and rip currents is illustrated in Figure 8-12. We will see later that jetties and headlands can cause rip currents under certain wave conditions.

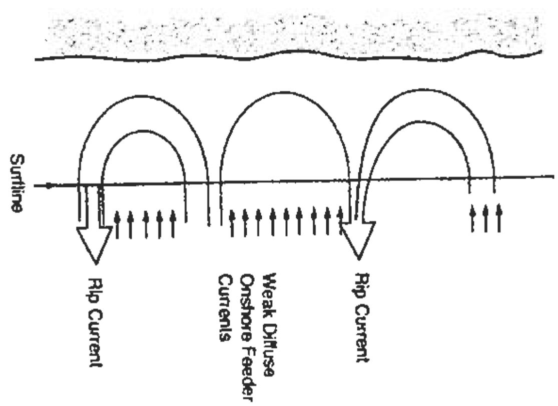


Figure 8-10: Rip Currents (After Dean and Dalrymple)

Figure 8-11 Photograph of a Rip Current

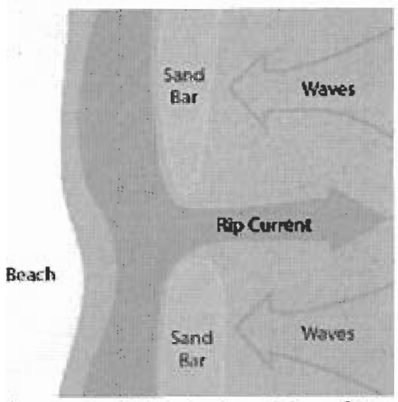


Figure 8-12 Relationship of Bars to Rip Currents

Ex $H_o = 4'$ $T = 6s$ $D50 = 0.5mm$ Temp $= 20°C$ $m = 0.02$

Determine movement

$w = 7.4 \text{ cm/s} = 0.243 \text{ ft/s}$

$L_o = 184.5 \text{ ft}$ $L_o = \frac{g}{2\pi} T^2$

$\frac{H_o}{L_o} = 0.022$

$\frac{\pi w}{gT} = 0.004$

Fig 8.14 Slightly favoring Berm formation

Determine Beach Stability

Fig 7.20 we only need 1.5 diameter for our 2% slope

djerolle

Machine Name: w14947

Date: 03/11/2010

Job: 380

Time: 3:33:40 PM

Cost: 0.00

Assignment 8.1.

Name:____

For the wave rose represented in the following table, estimate:

1. The longshore transport in each BIN,
2. The net longshore transport,
3. The gross longshore transport.
4. If the beach D50 is 0.4 mm and it is exposed, will the beach be stable?

H ft	W	SW	S	SE	E Wind Direction
0.5-1	2	6	6	8	2
1-1.5	1	6	7	7	5
1.5-2	1	2	3	3.5	2
2-3	0.5	1	1	1	0.7

The shoreline runs due east-west. The beach slope is 1%.
Enter the ΔQls values in the Table below in yd^3/year

H ft	WSW	SW	S	SE	ESE Wind Direction
0.5-1					
1-1.5					
1.5-2					
2-3					
Sub TOTALS					

*Net Qls =*___________________________

*Gross Qls=*___________________________

Beach: STABLE or UNSTABLE

Attach sample calculations.


1478 m
Image © 2010 TerraMetrics
Image NOAA
© 2010 Cnes/Spot Image
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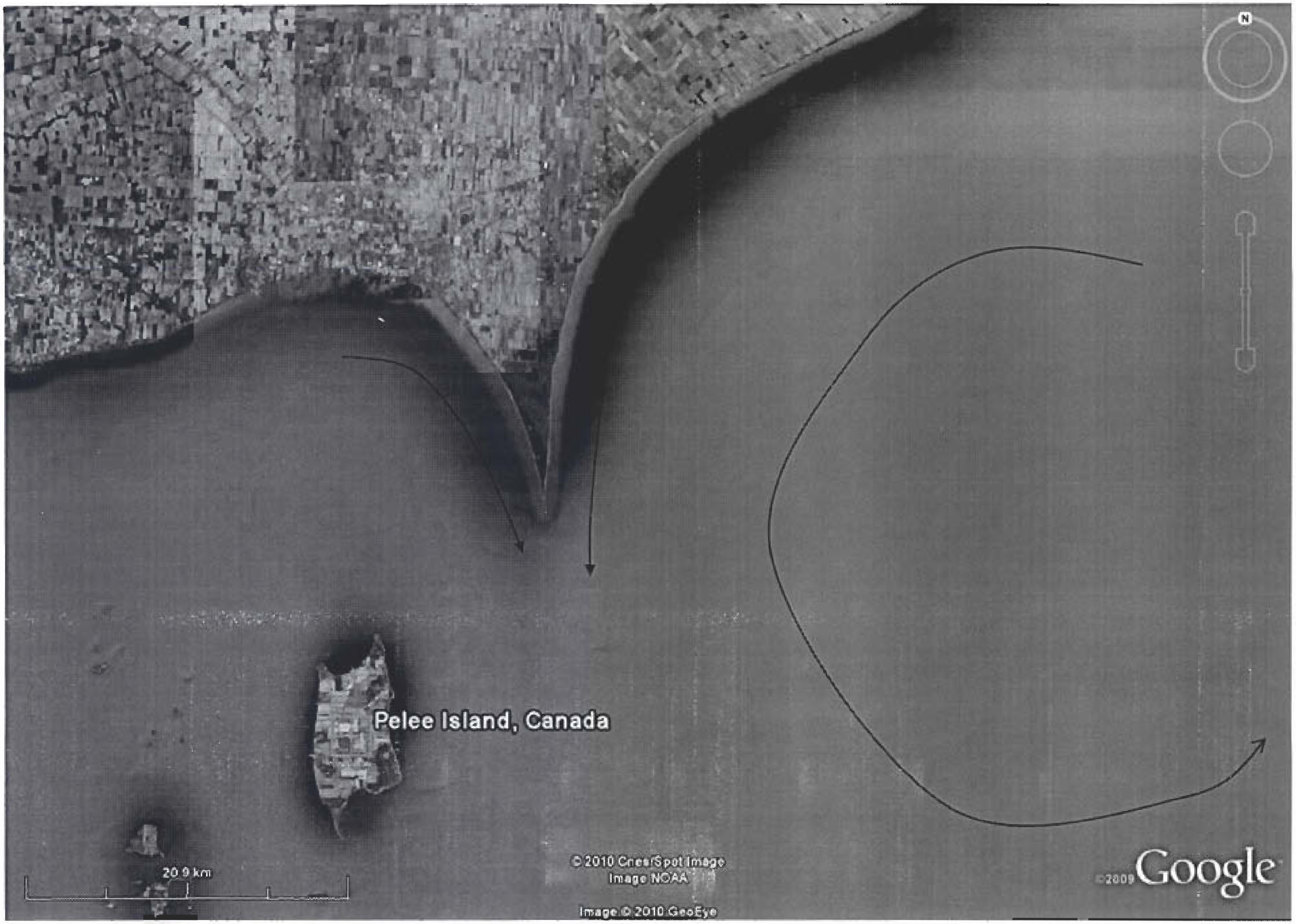
N
Pelee Island, Canada
20.9 km
© 2010 Cnes/Spot Image
Image NOAA
Image © 2010 GeoEye
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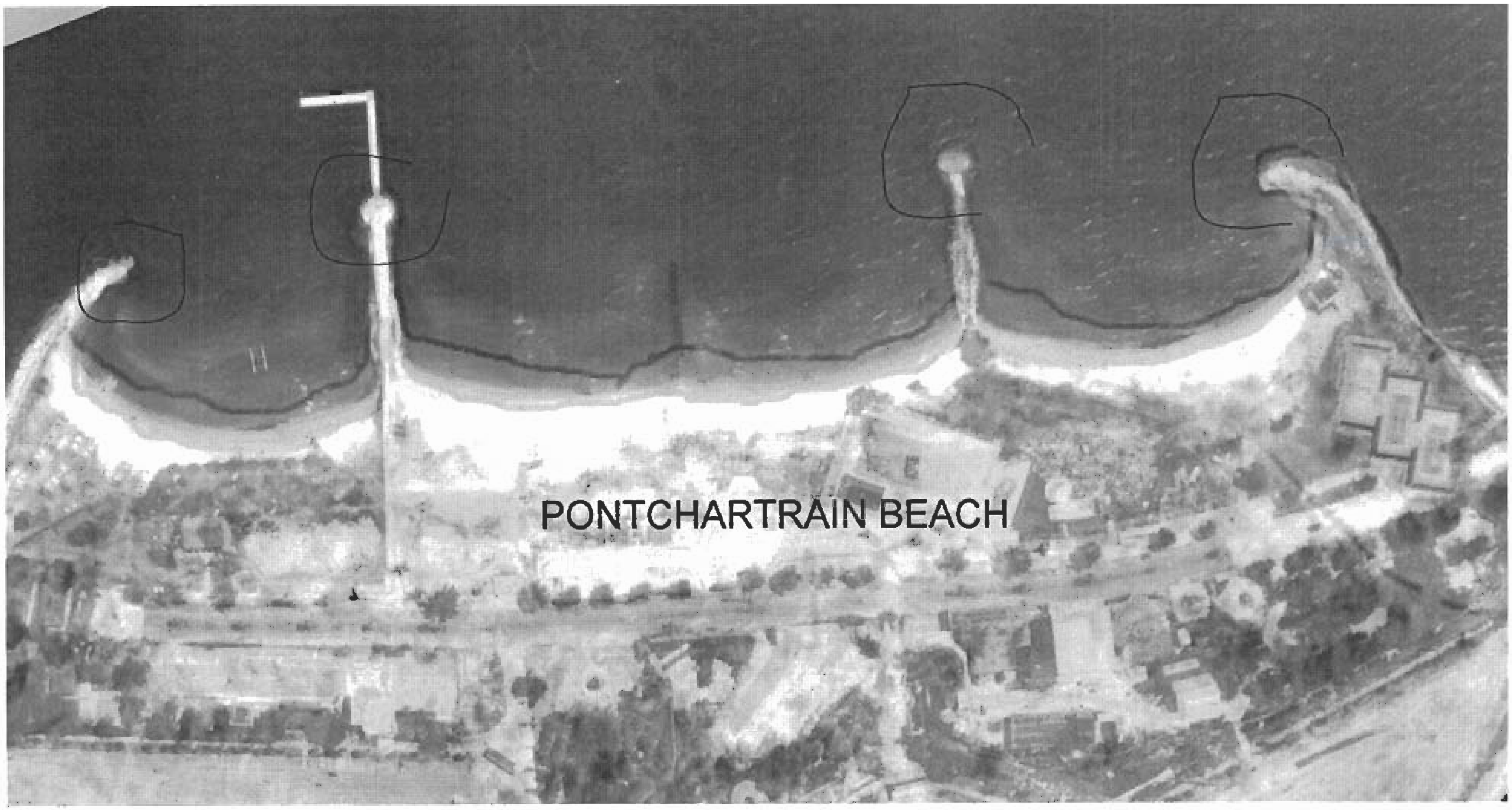
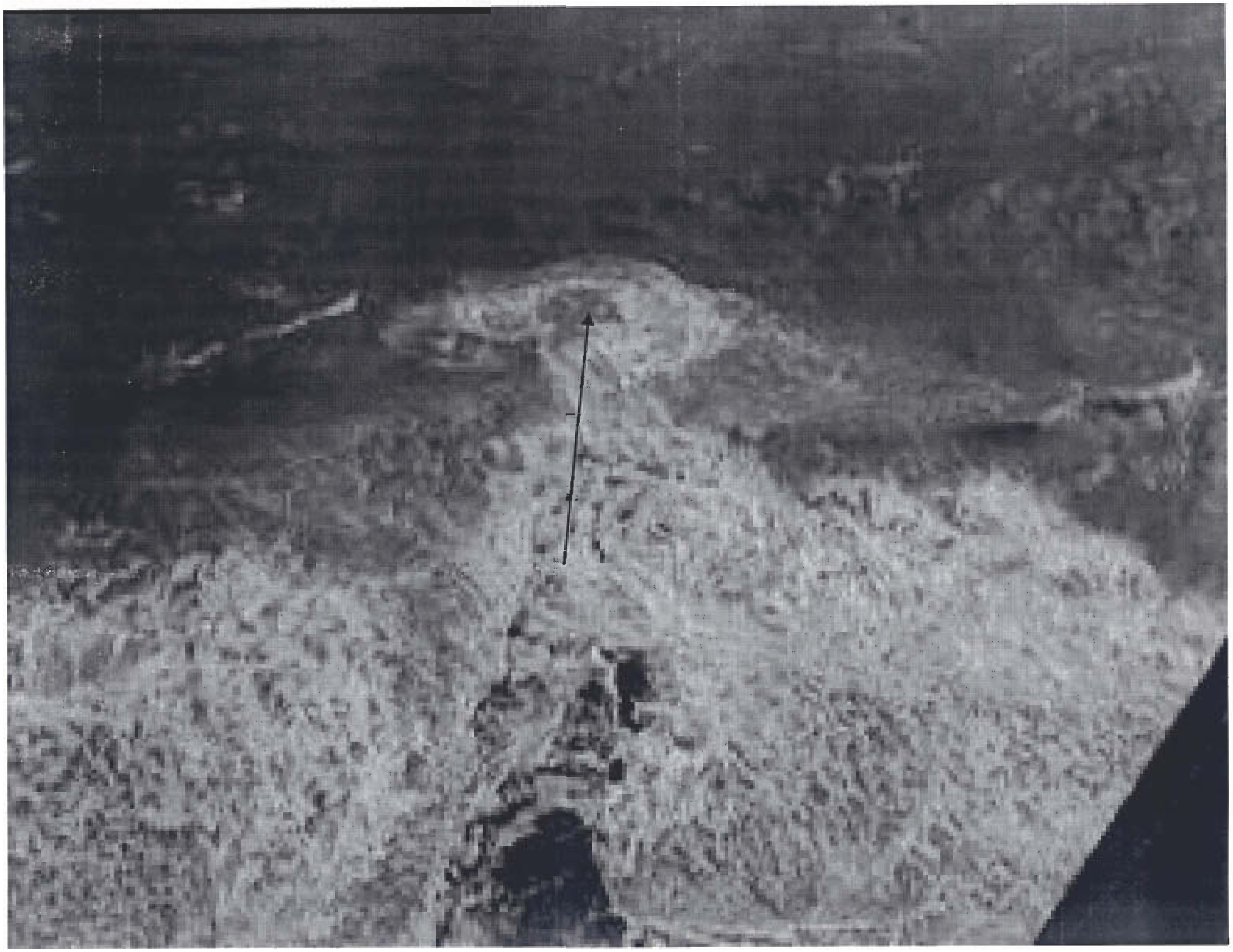
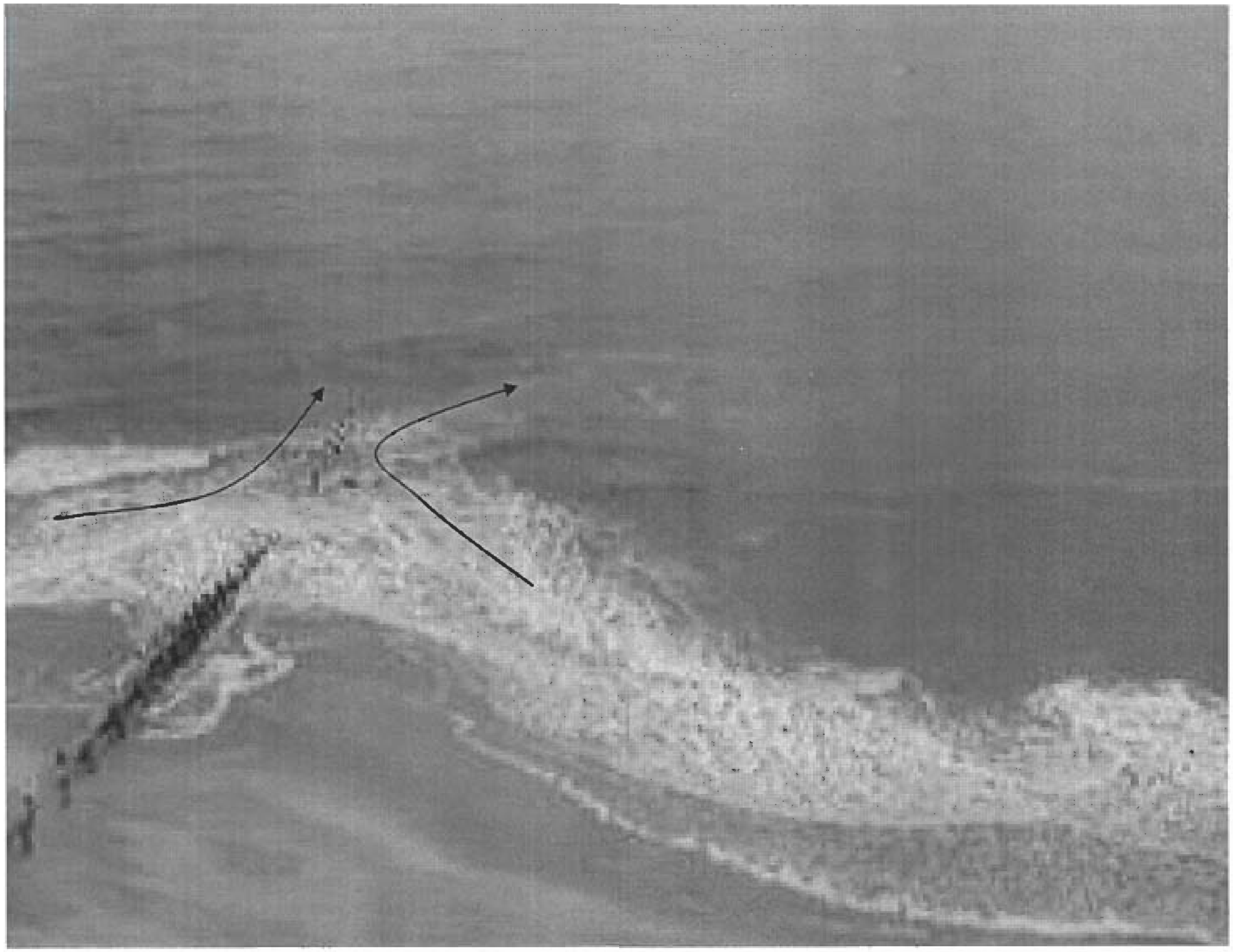
Potential Rip Current Sites



Rip Current at a Headland (URI)



Rip Current at a Jetty (URI)



Basic Rip Current Mechanics

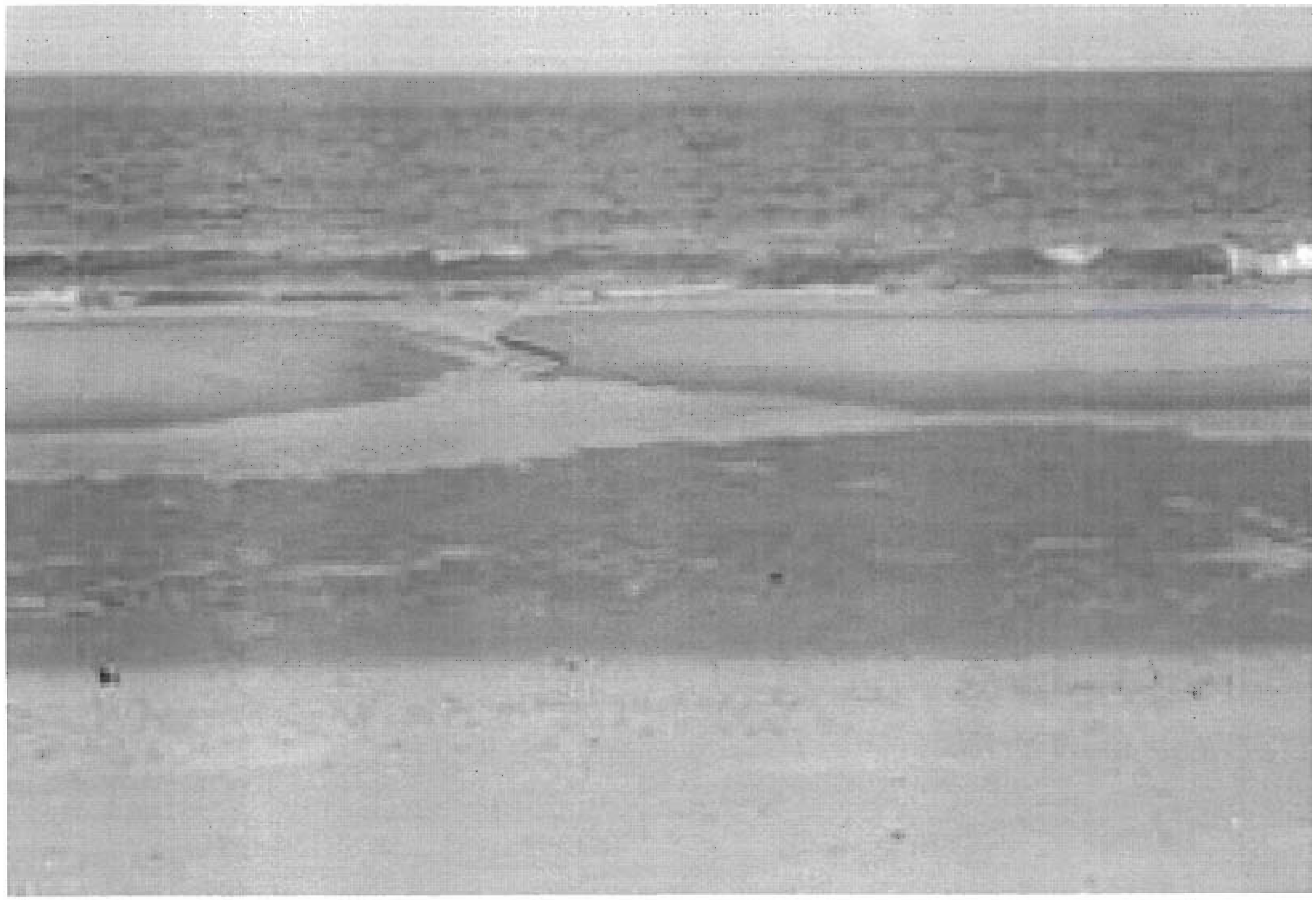
- *Speeds are typically 1-2 ft/sec but can pulse up to 8 ft/sec.*

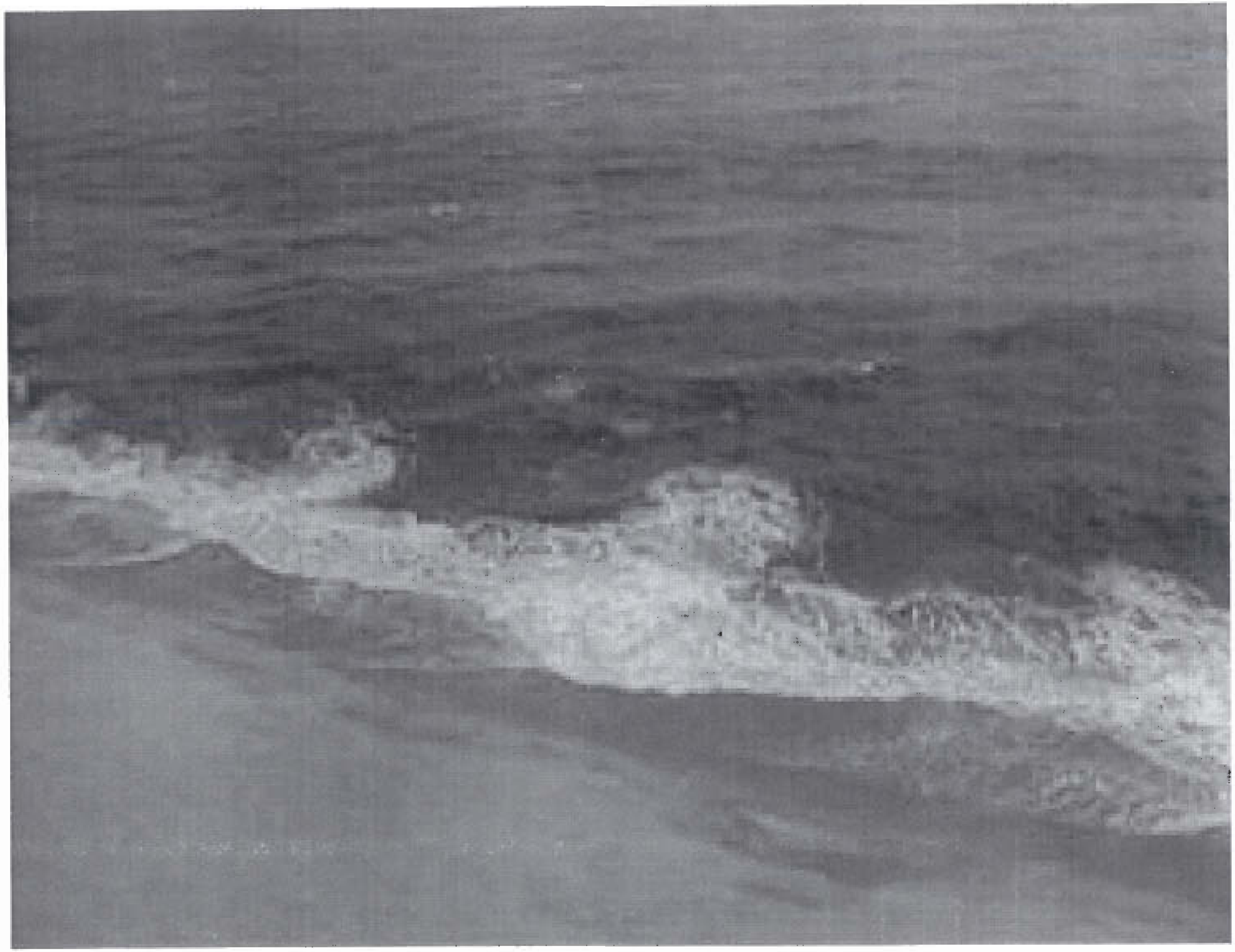
- *Speed increases with Wave breaker height.*

- *Cross shore structures like jetties and redirect longshore currents in the offshore direction causing a rip current to develop.*

- *Headland points that often separate littoral cells, can cause rip currents.*

View from the ground and taken at low tide; this channel has formed at a break in the sand bar. *Photo courtesy University of Delaware Sea Grant College Program*

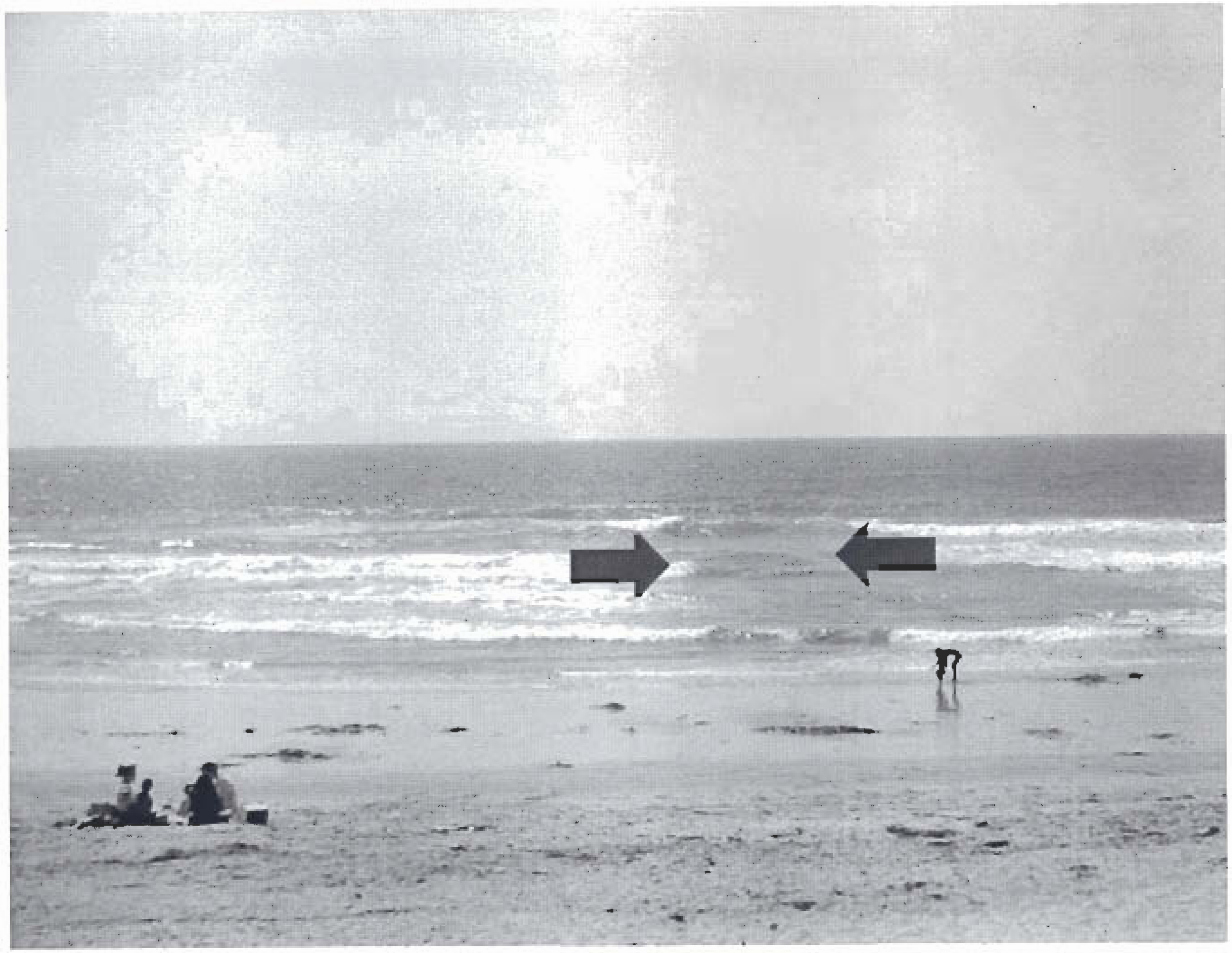




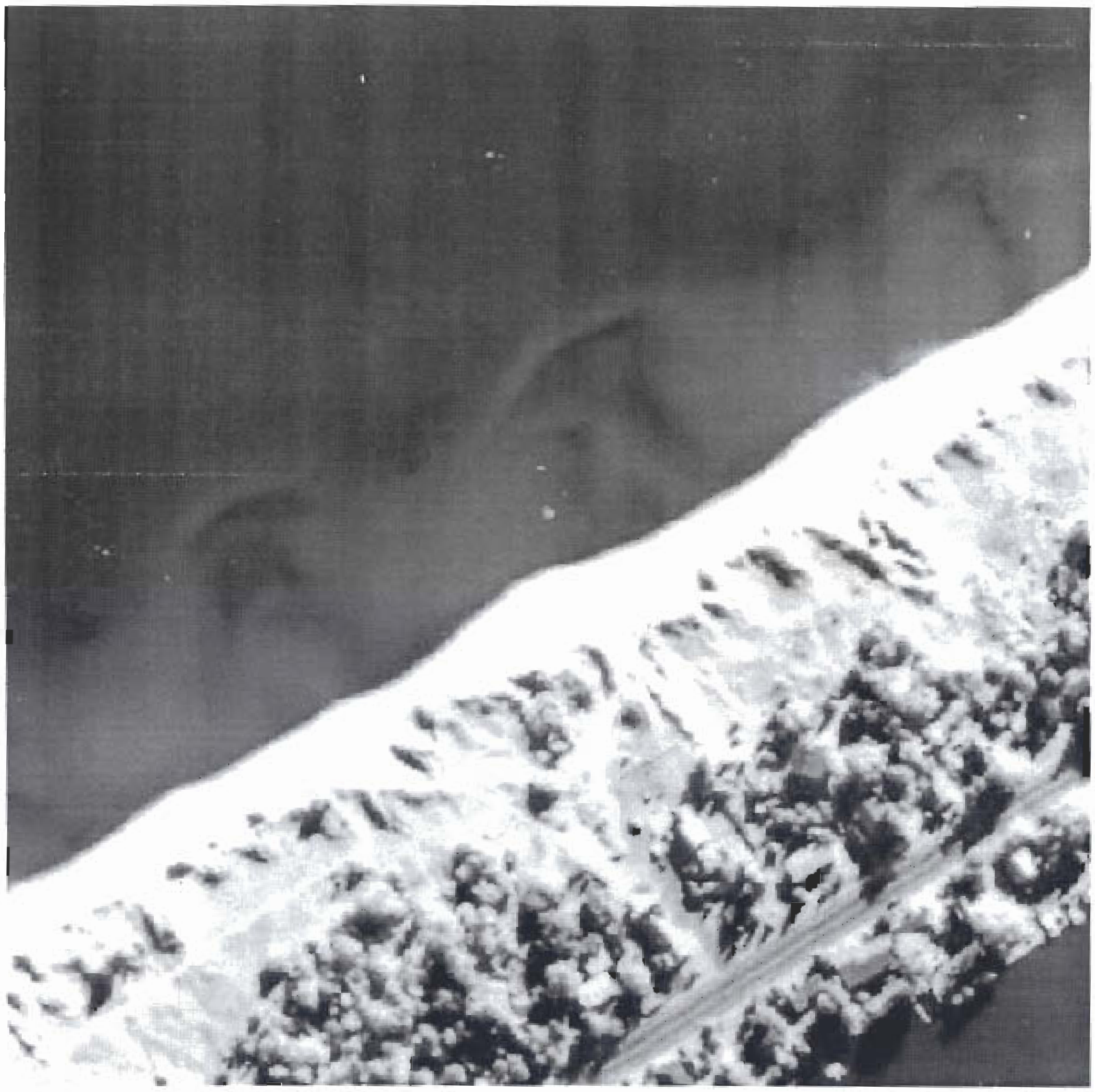
Viewed from the air, rip currents can be seen flowing past
the line of breaking waves.
Rip current spacing along an open coast
may be dependent on many factors, including the shape of the nearshore bottom.
Photo courtesy University of Delaware Sea Grant College Program .

Photo of Rip Current

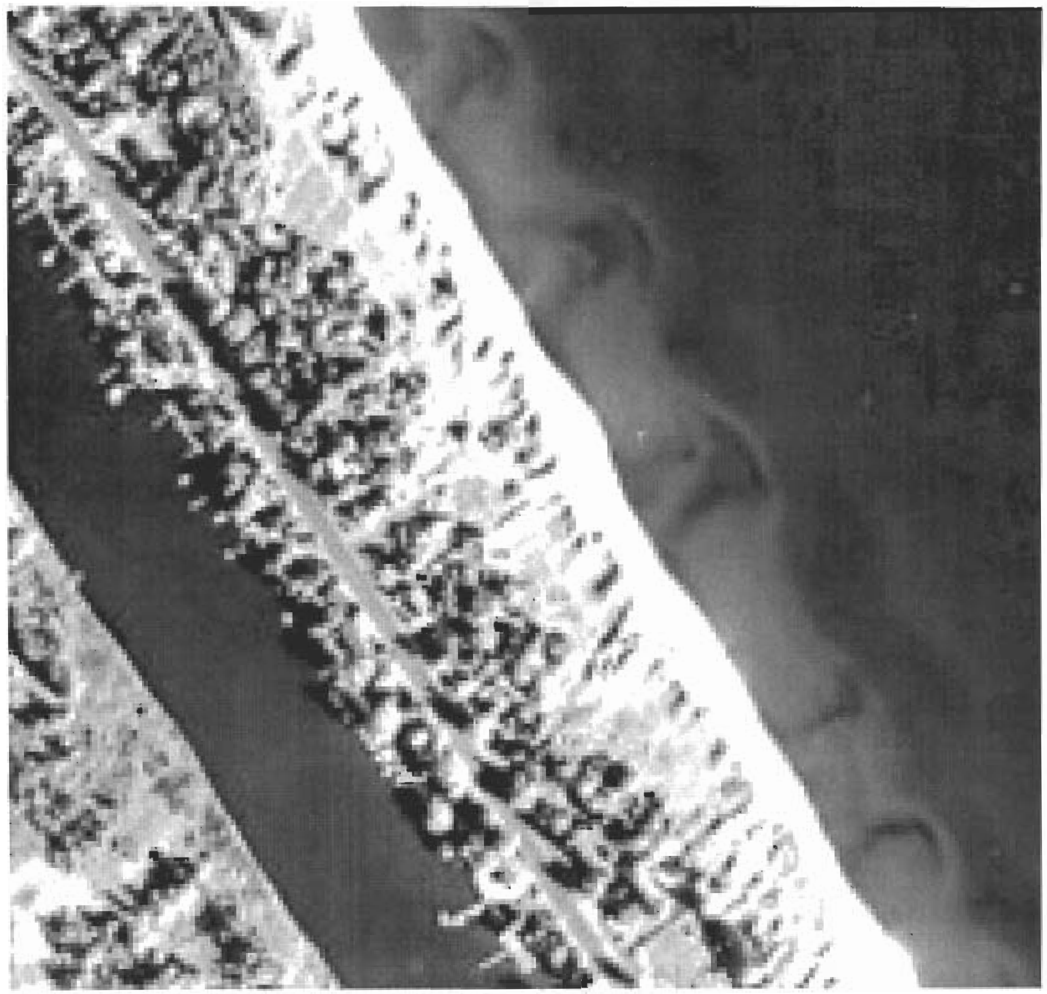
Photo of Rip Current



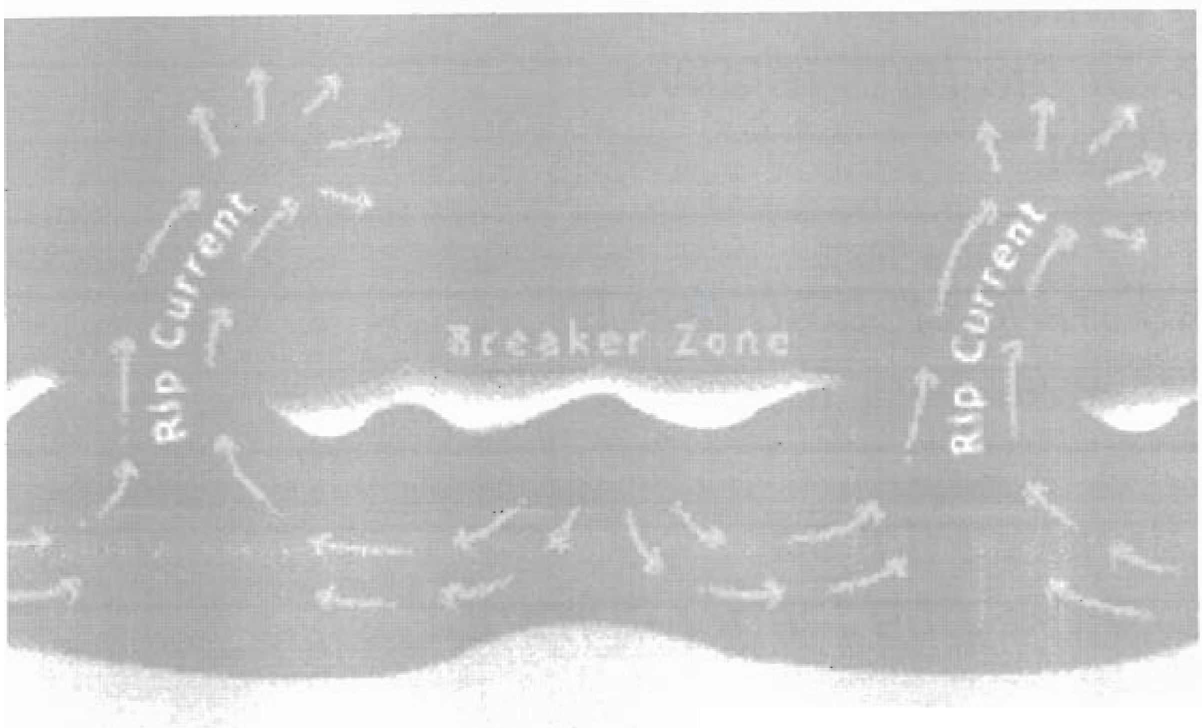
Rip Current



Rip Current in Relation to Bars



Where Rip Currents occur



Basic Rip Current Mechanics

- *Waves break on the sand bars before they break in the channel area.*

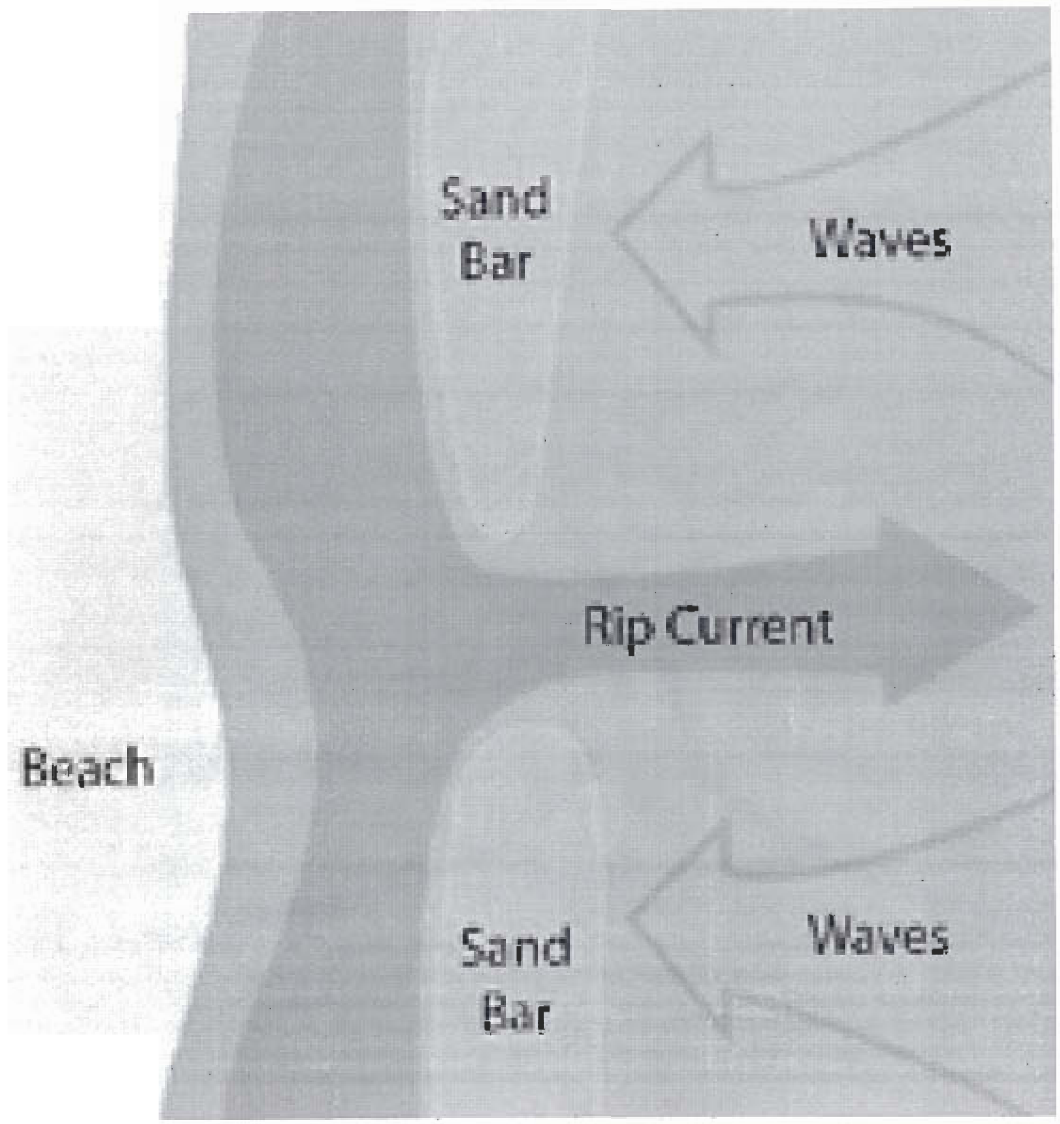
- *Wave breaking causes an increase in water level over the bars relative to the channel level.*

- *A pressure gradient is created due to the higher water level over the bars.*

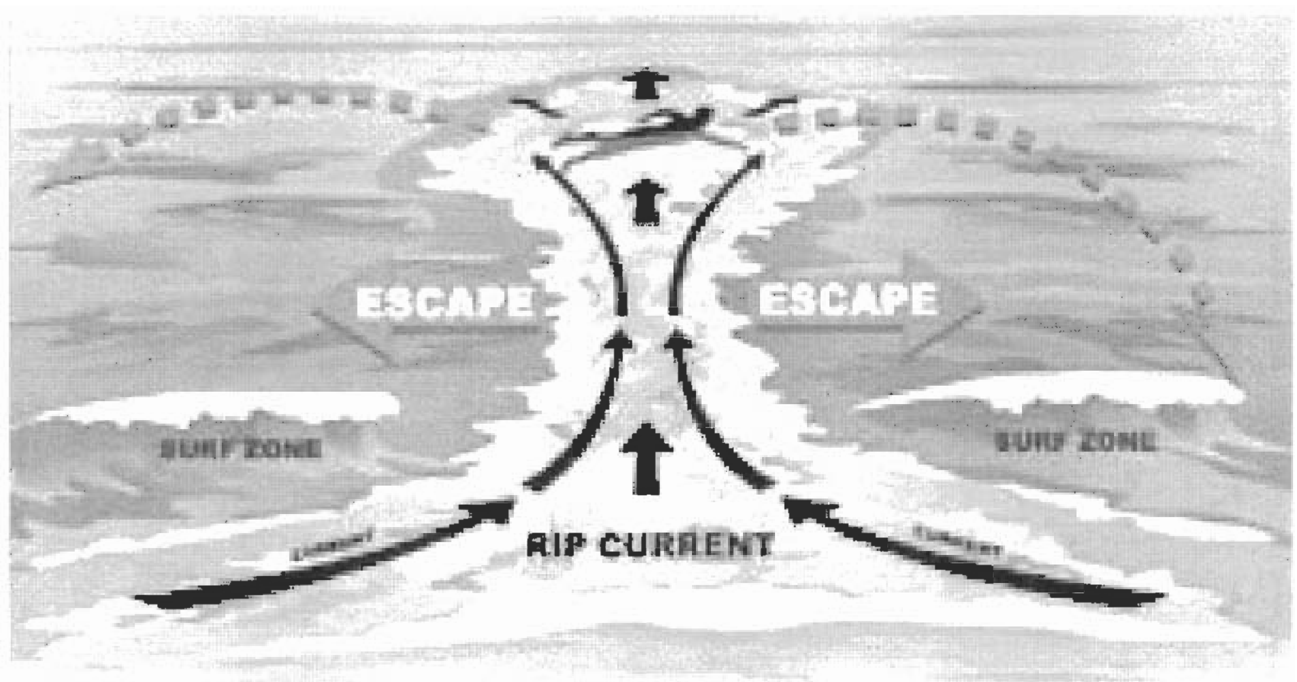
- *This pressure gradient drives a current alongshore (the feeder current).*

- *The longshore currents converge and turn seaward, flowing through the low area or channel between the sand bars.*

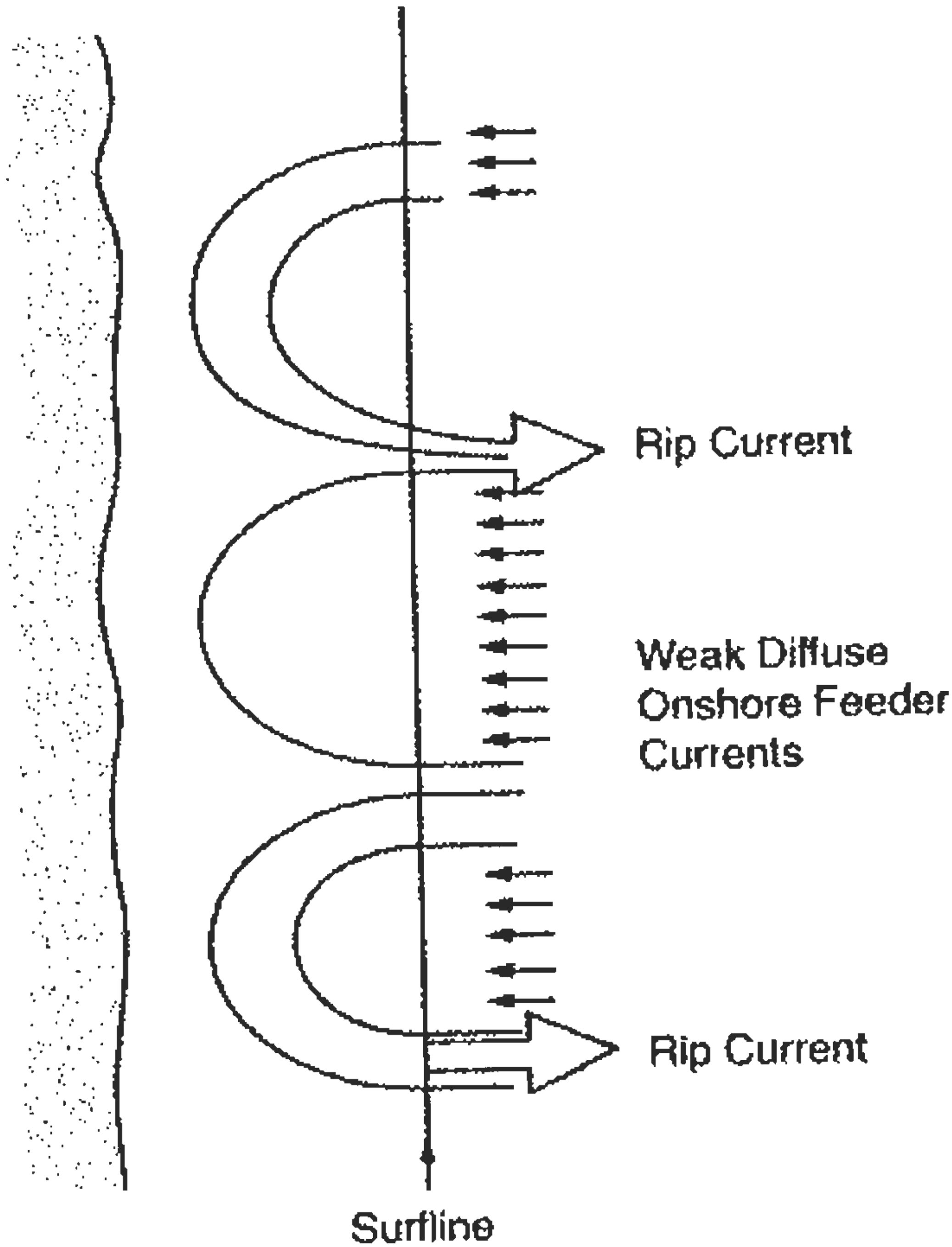
Relationship Of Rip Currents to Bars



Schematic of Rip Current



Rip Currents



NOTE: Closure depth is where there is almost no bed sediment movement.

$U_b = (H/2)*(gT/L)/\cosh(2\pi d/L) \rightarrow U_b$ critical (Figure 4-22)

Both *d* and *L* are to be determined.

H	4	ft		
T	6	s		
D50	0.5	mm	0.0016	ft
Temp	20	oC		
m	0.02			
w=Vf=Vs	7.4	cm/s	0.2428	ft/sec
Lo	184.5	ft		
L	176.1	ft	-0.000517103	
$d_{closure}$	52.61	ft	0.006146841	
U_b	0.656	ft/sec		
U_b crit	0.65	ft/sec	Fig 4-22	

The maximum orbital velocity can be estimated from Horizontal max {+ or -}

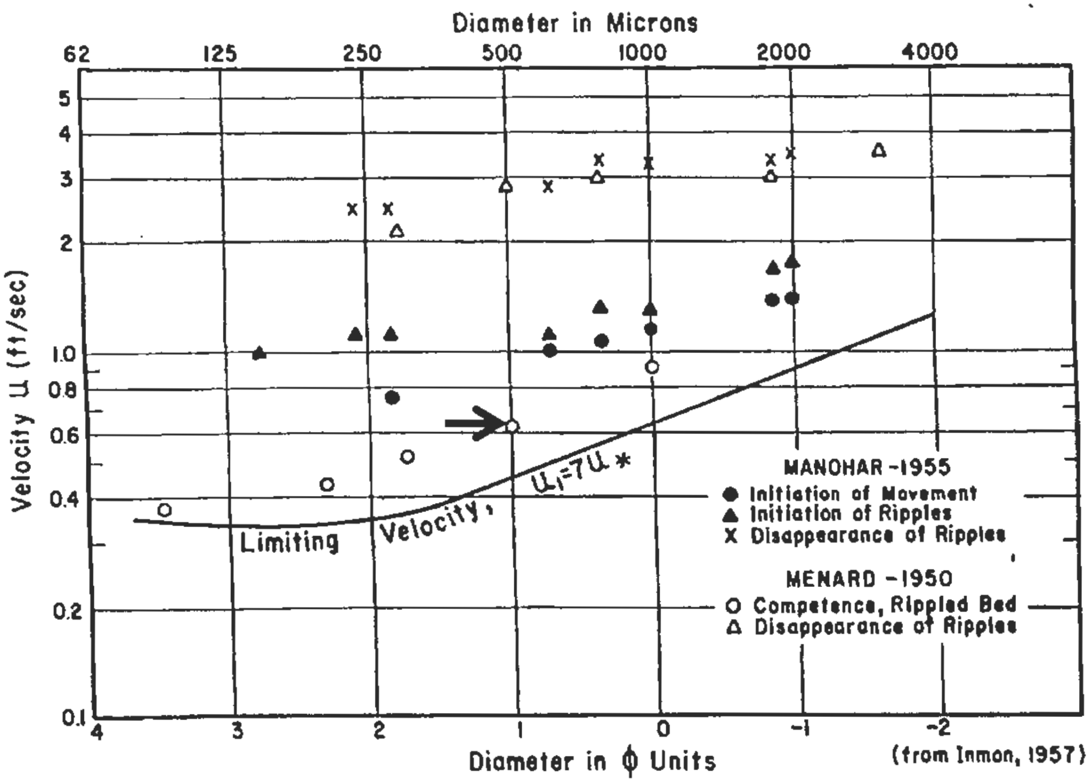
$$U = (H/2)*(gT/L)*\cosh(2\pi(d+z)/L)/\cosh(2\pi d/L)$$

or at $z = -d$

$$U_b = (H/2)*(gT/L)/\cosh(2\pi d/L)$$

Example. Determine the closure depth for the following wave:
H = 4 ft;
T = 6 sec.
Grain size = 0.5 mm
Solution:

Initial Motion

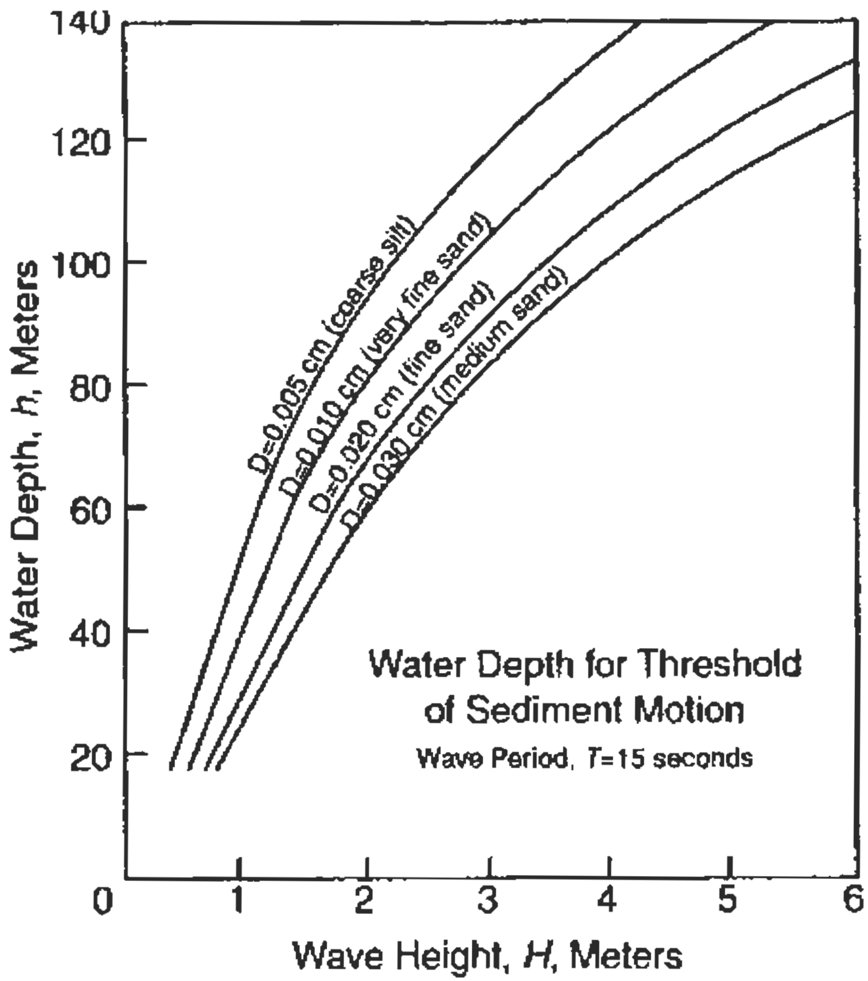


Limiting or Minimum Velocity for the Initiation of Motion of Sand of a Given Size.

Limiting velocity, arbitrarily defined as equal to $7u_{*c}$ where u_* is the threshold or critical friction velocity. (Inman, 1949.) For unidirectional flow this relation would give a limiting velocity equivalent, for example, to the mean velocity measured 1 foot above a bottom which has a roughness length of 2 cm. Field observations near the surf zone indicate that planation and disappearance of ripples does not occur unless the maximum velocity associated with the wave crest somewhat exceeds that listed by Menard and Manohar.

Figure 4-22. Initiation of Ripple Motion

Closure depth



Ho	4	ft		
T	6	s		
D50	0.5	mm	0	ft
Temp	20	oC		
m	0.02			
w=Vf	7.4	cm/s	0.24	ft/sec
Lo	184.5	ft		
Ho/Lo	0.022			
Ho/(wT)	2.746			
Fig 4.29	Favoring offshore Bar formation			
Ho/Md	2440			
Fig 4.30	Favoring offshore Bar formation			
Fig 7-20	Stable Beach			

Example

- Given: Ho = 4 ft; T=6 sec; m=2%; D50=0.5mm
- Determine: for this wave is movement on or offshore?
- Is the beach stable? Assume "exposed" beach.

Beach Stability

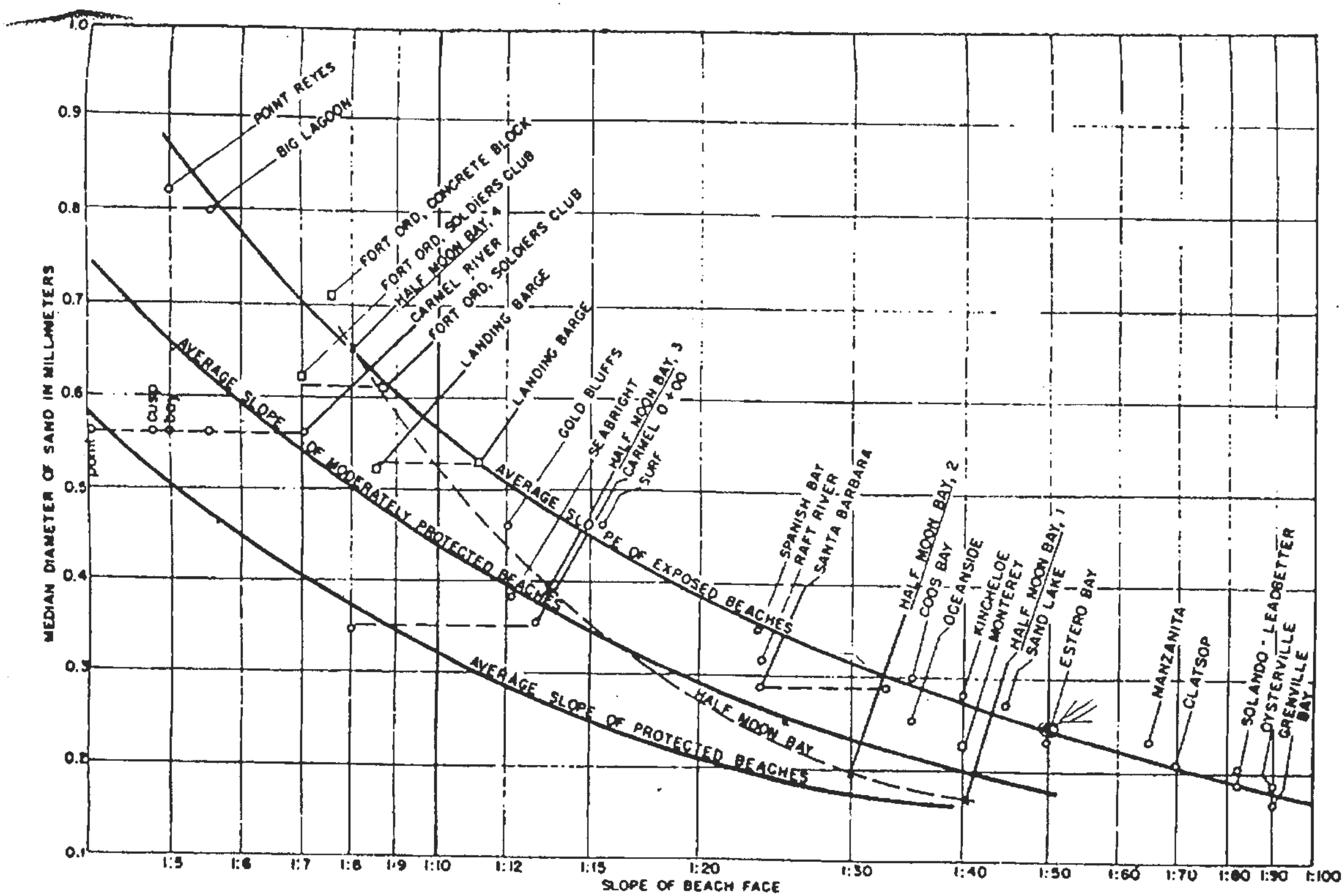


Figure 7.20. Relationship between beach slope and sand side at the mid-tide level for Pacific coast beaches (3).

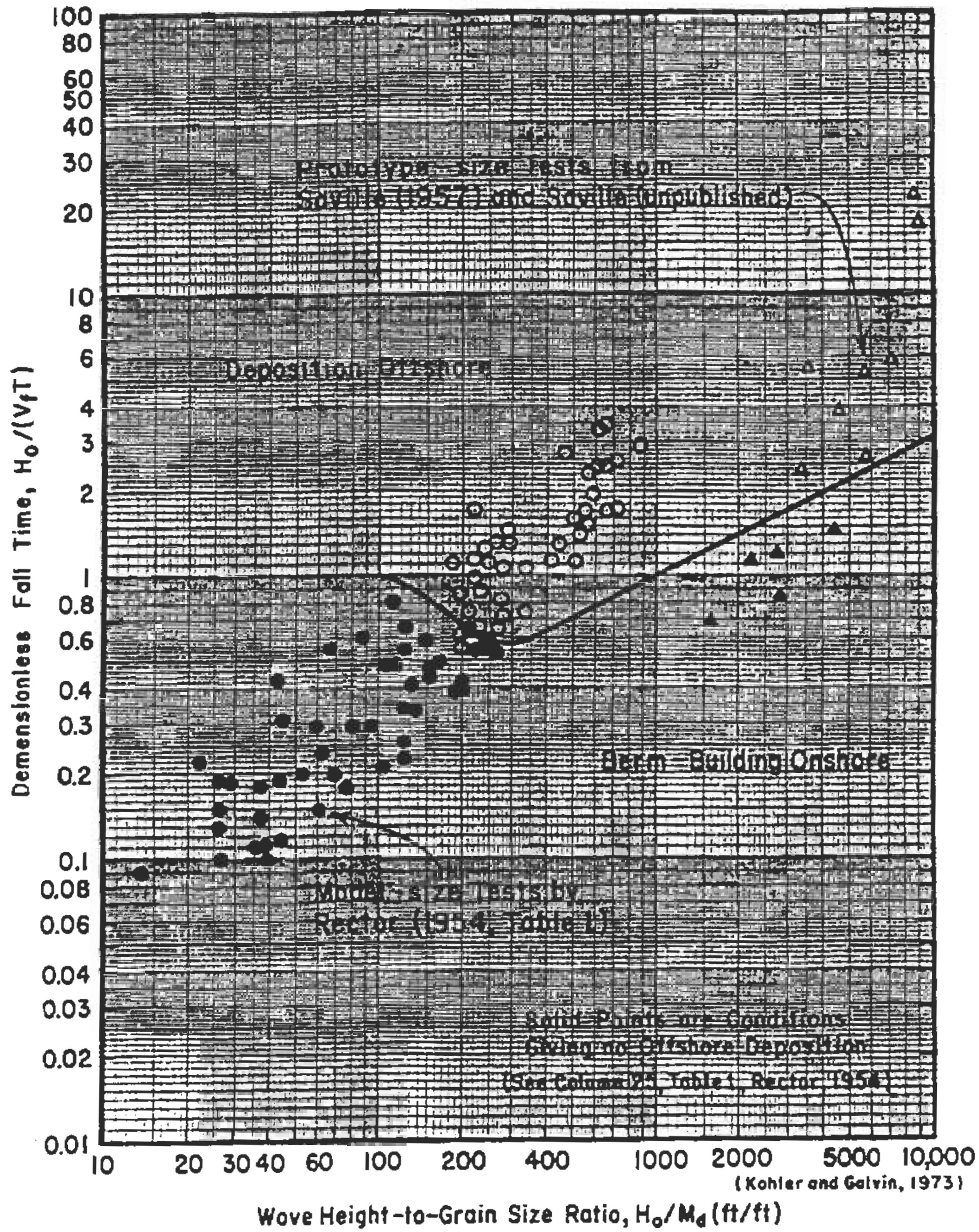


Figure 4-30. Berm-Bar Criterion Based on Dimensionless Fall Time and Height-to-Grain Size Ratio

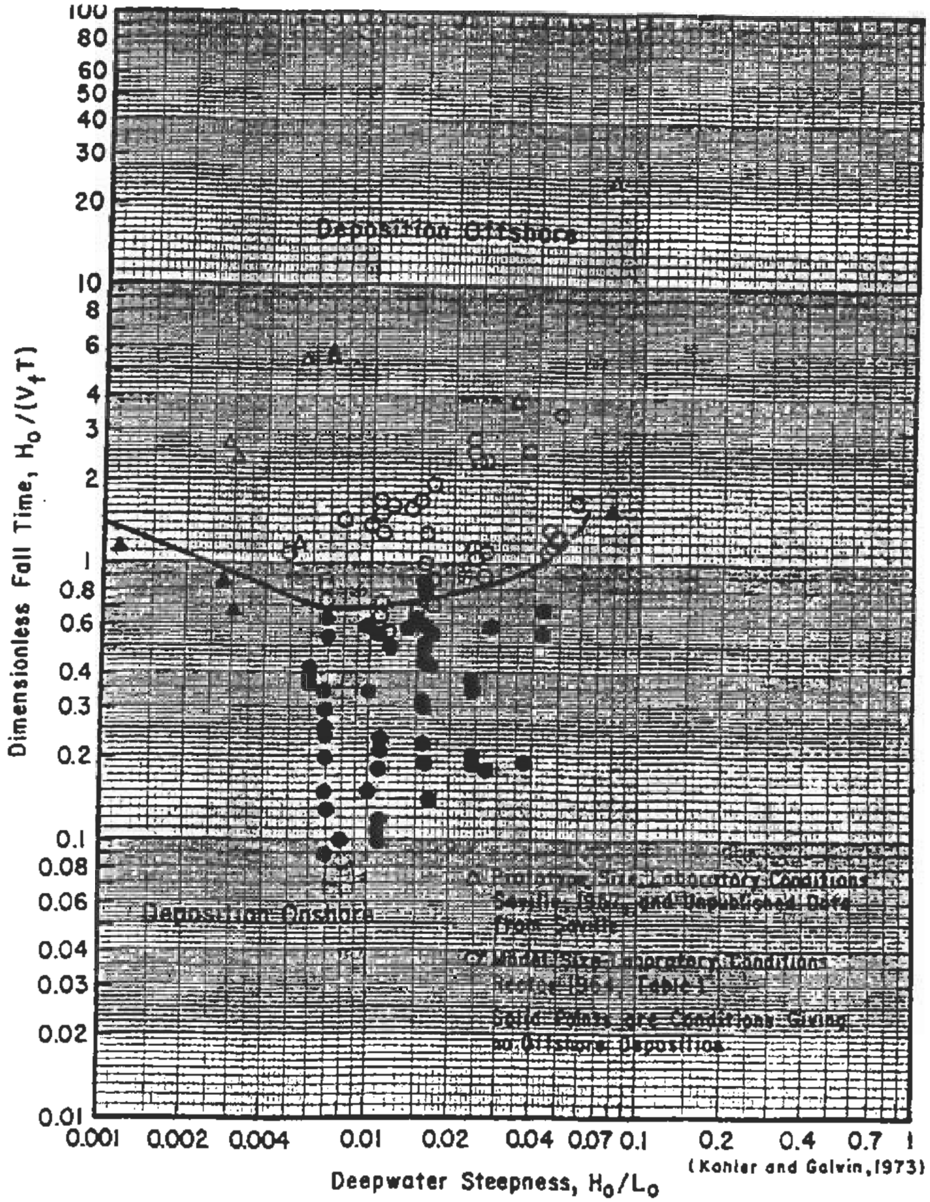


Figure 4-29. Berm-Bar Criterion Based on Dimensionless Fall Time and Deep Water Steepness

F_o = Dimensionless Fall Time = $Ho/(V_f\,T)$

$F_o > 1$ Offshore deposition

$F_o < 1$ Onshore deposition

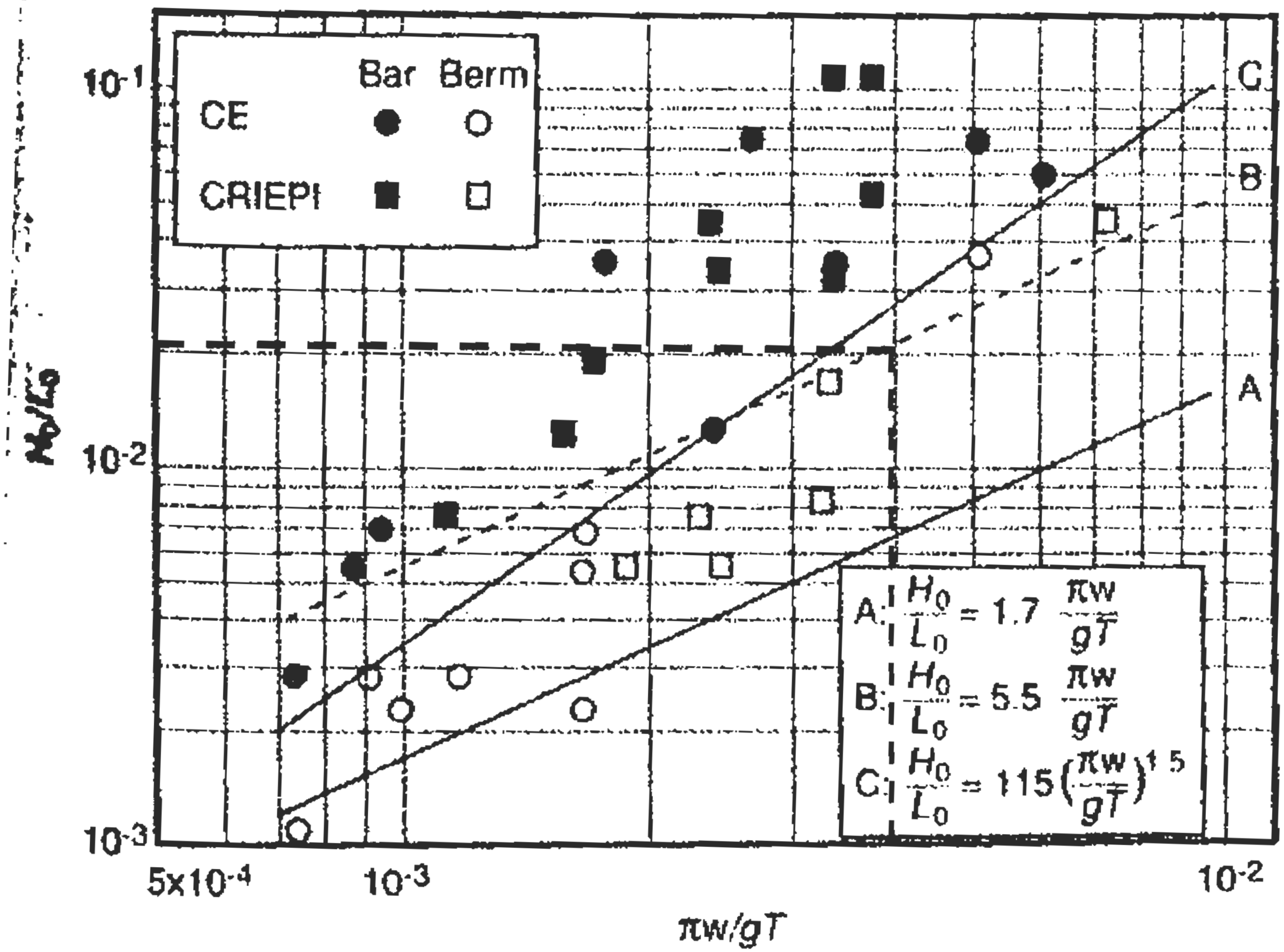
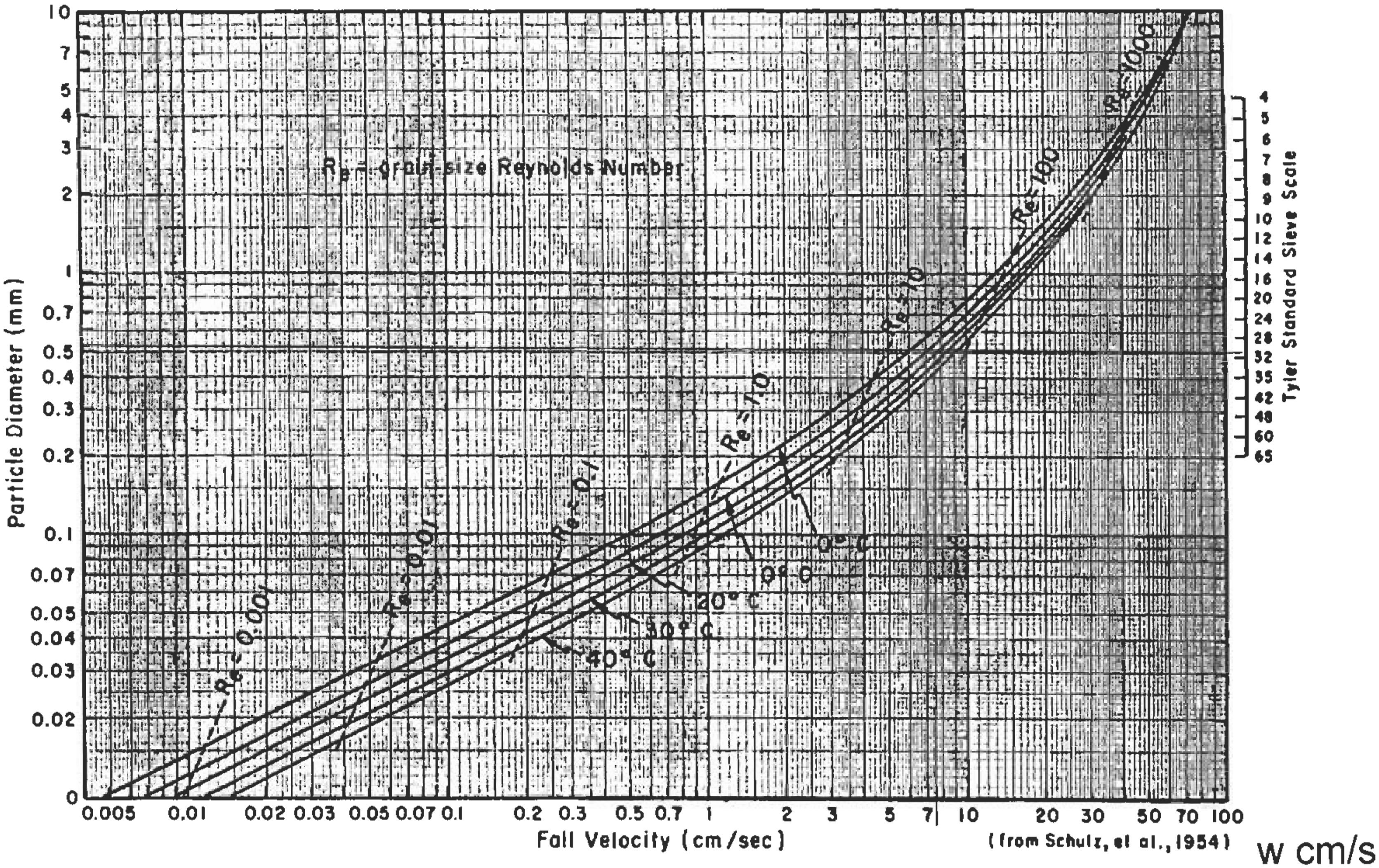


Figure 8.14 Criteria for the generation of storm or normal profiles (from Kraus and Larson 1988).

Ho	4	ft		
T	6	s		
D50	0.5mm			
Temp	20	oC		
m	0.02			
w	7.4	cm/s	0.243	ft/sec
Lo	184.5	ft		
Ho/Lo	0.022			
pi w/gT	0.004			
Fig 8.14	Slightly favoring Berm formation			

Example

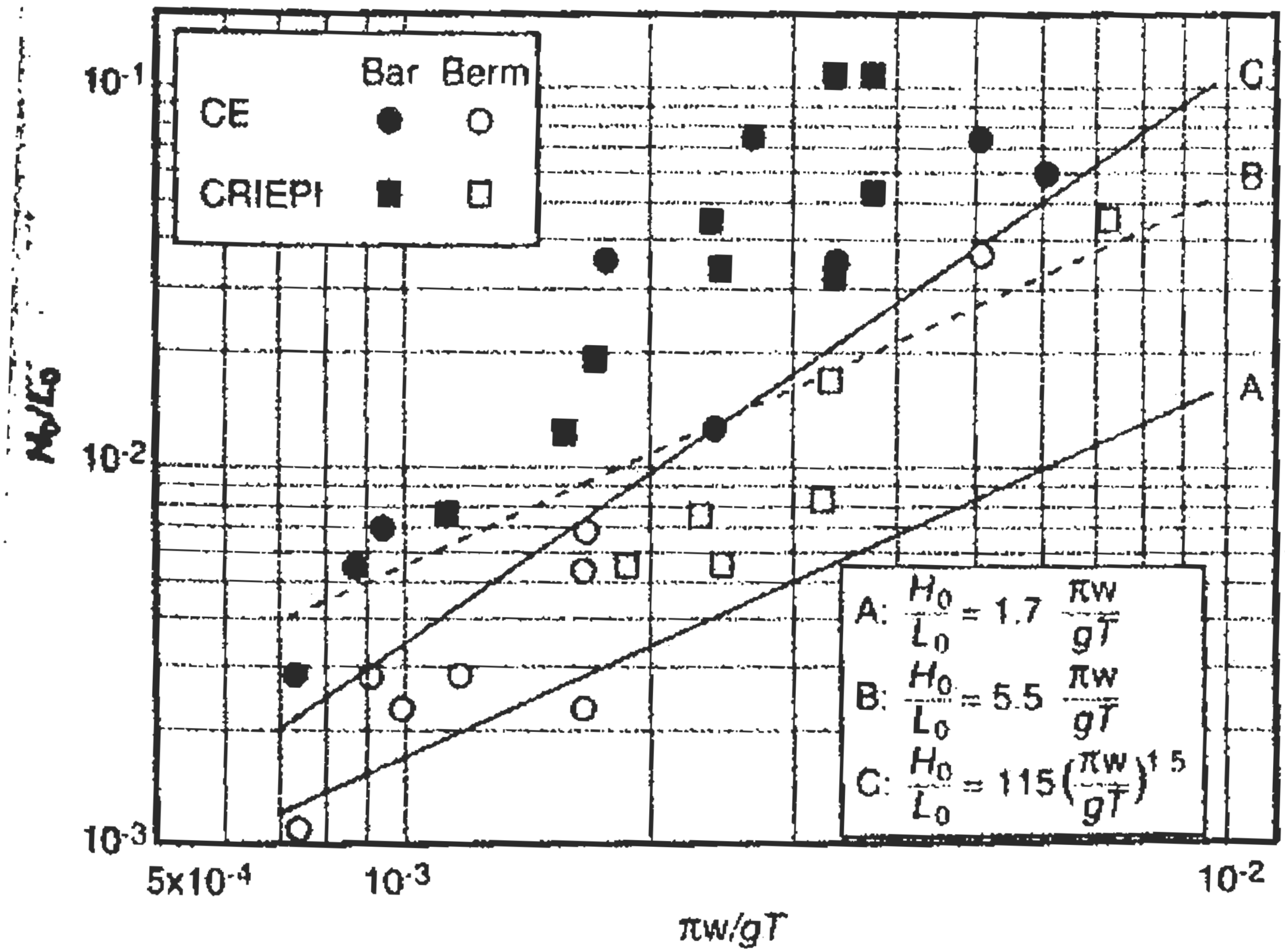
- Given: Ho = 4 ft; T = 6 sec; D50 = 0.5 mm; Temp ~ 20oC; m~2%
- Determine if the tendency will be to form bars or berm.
- Solution:
  - Find w
  - Find Lo = $gT^2/(2\pi)$
  - Find Ho/Lo =
  - Find $\pi w/(gT)$=



$Re = \frac{V_s D}{\upsilon}$

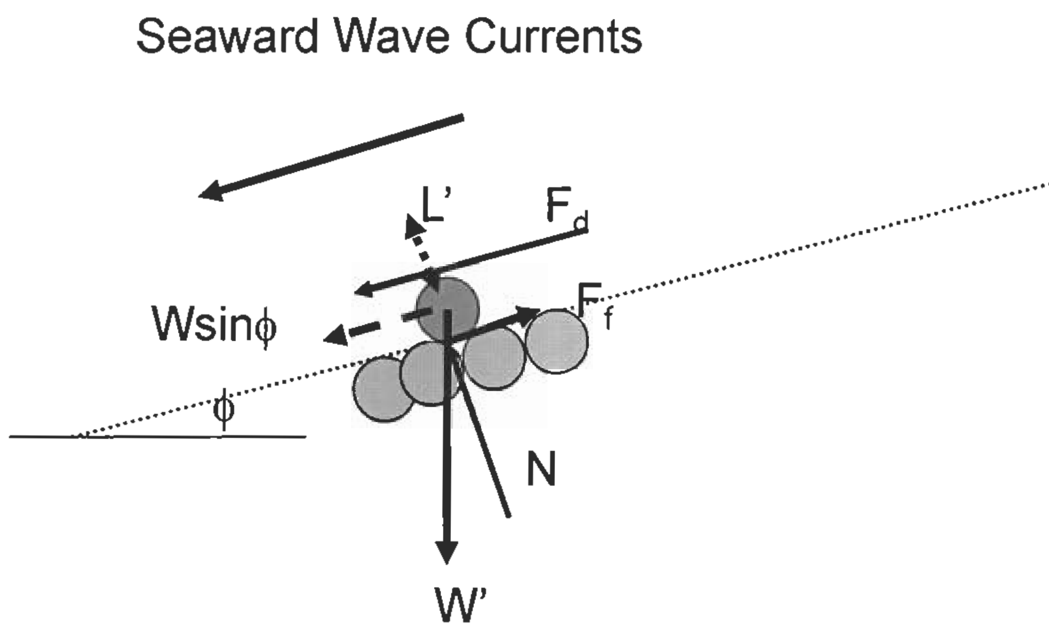
Figure 4-31. Fall Velocity of Quartz Spheres in Water as a Function of Diameter and Temperature

From Dean and Dalrymple



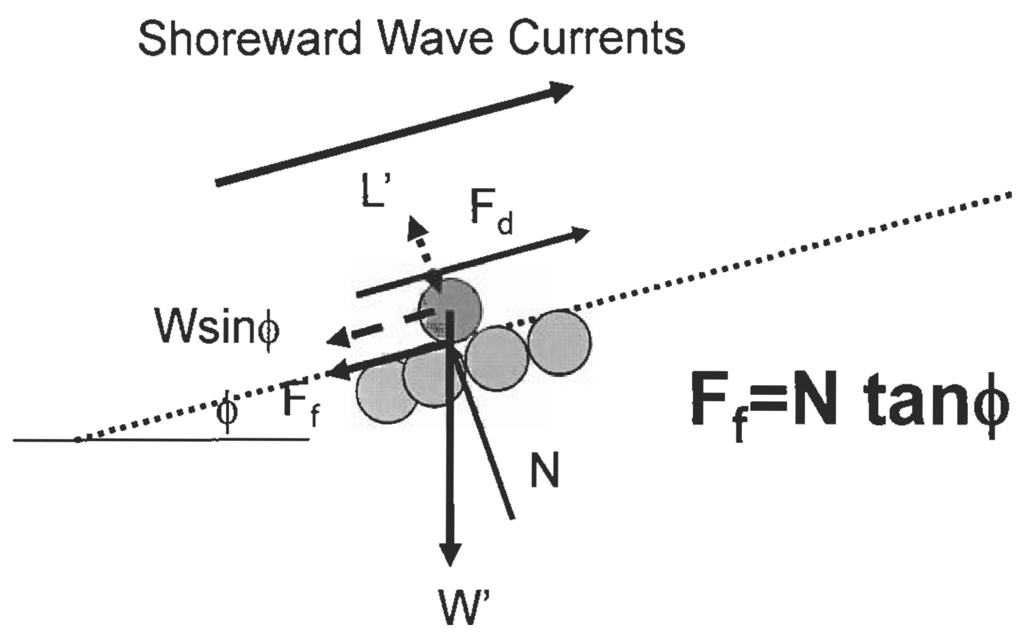
w=fall velocity

Figure 8.14 Criteria for the generation of storm or normal profiles (from Kraus and Larson 1988).

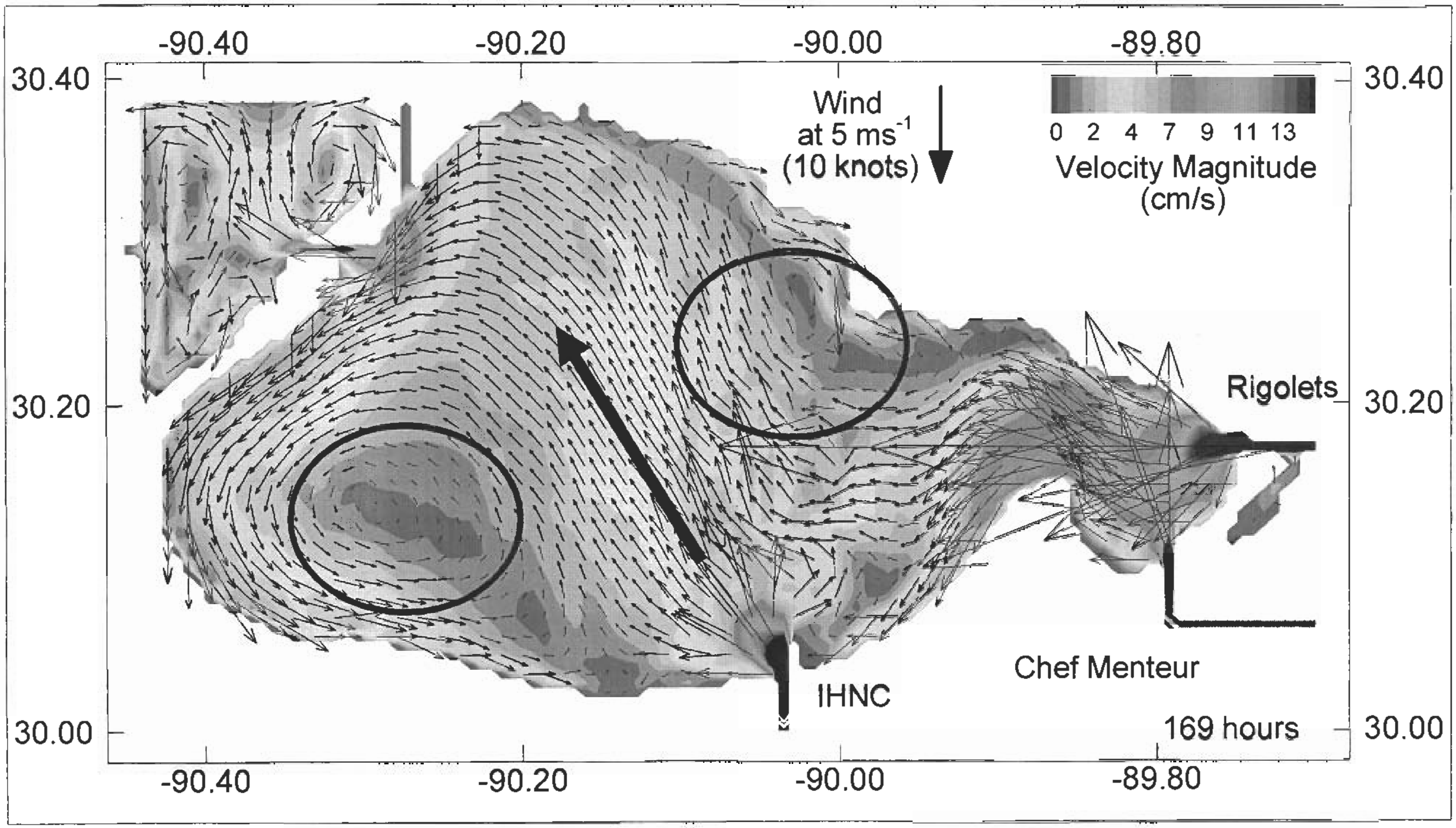


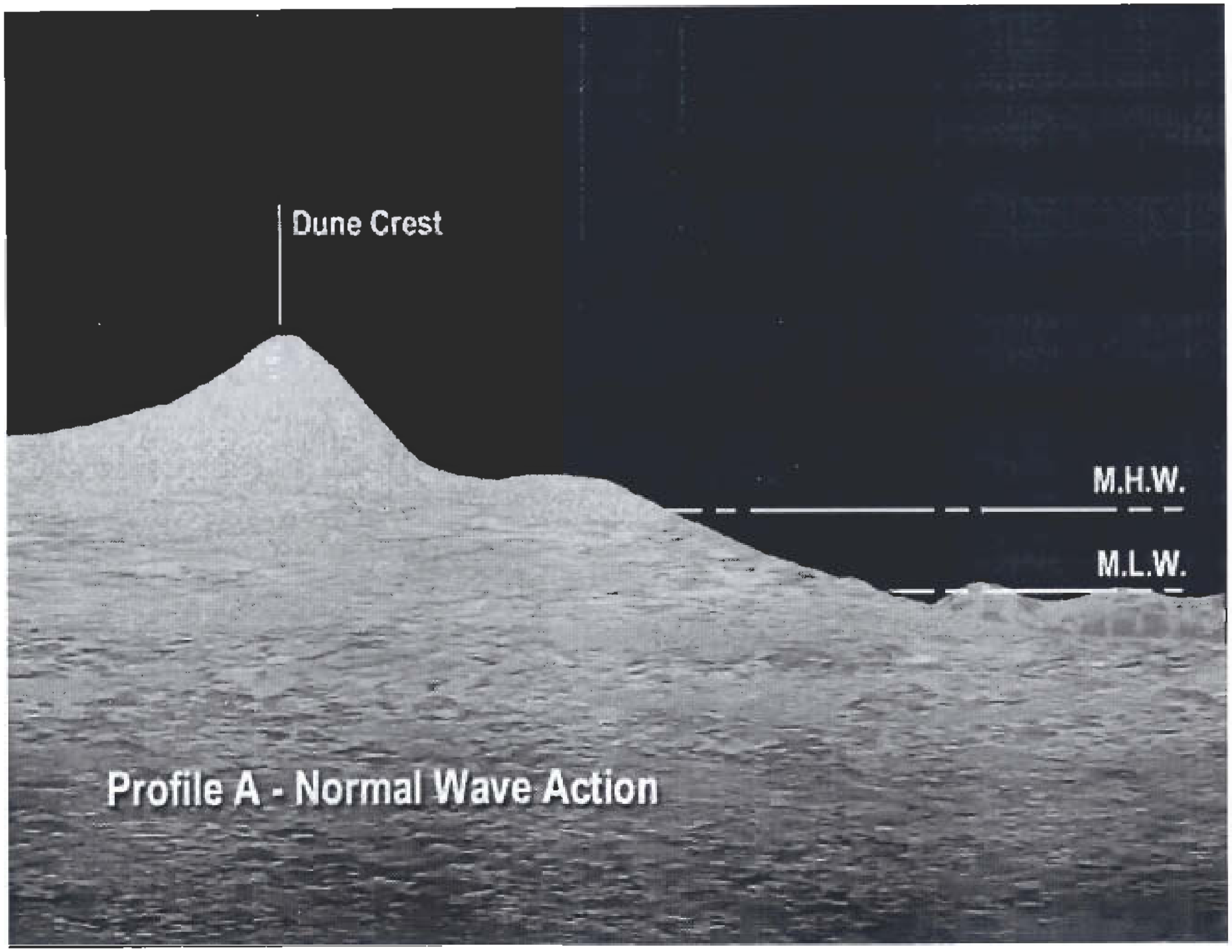
Seaward Wave Currents
L’
F_d
F_f
Wsinϕ
ϕ
N
W’

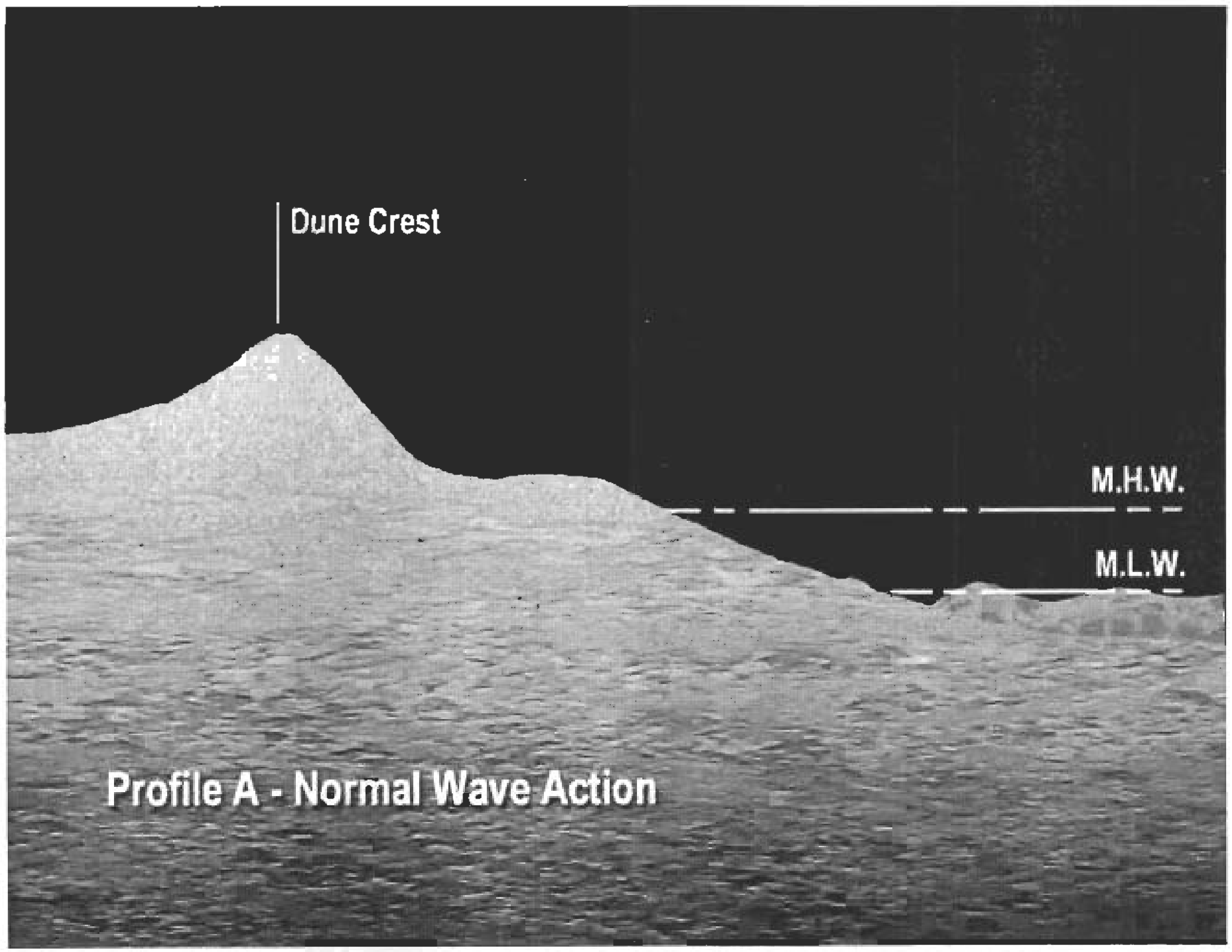
Onshore Transport -- Beach


Shoreward Wave Currents
L'
F_d
Wsinϕ
ϕ
F_f
N
W'
F_f=N tanϕ

Wind induced circulation




Dune Crest
M.H.W.
M.L.W.
Profile A - Normal Wave Action


Dune Crest
M.H.W.
M.L.W.
Profile A - Normal Wave Action

Cross-shore Movement

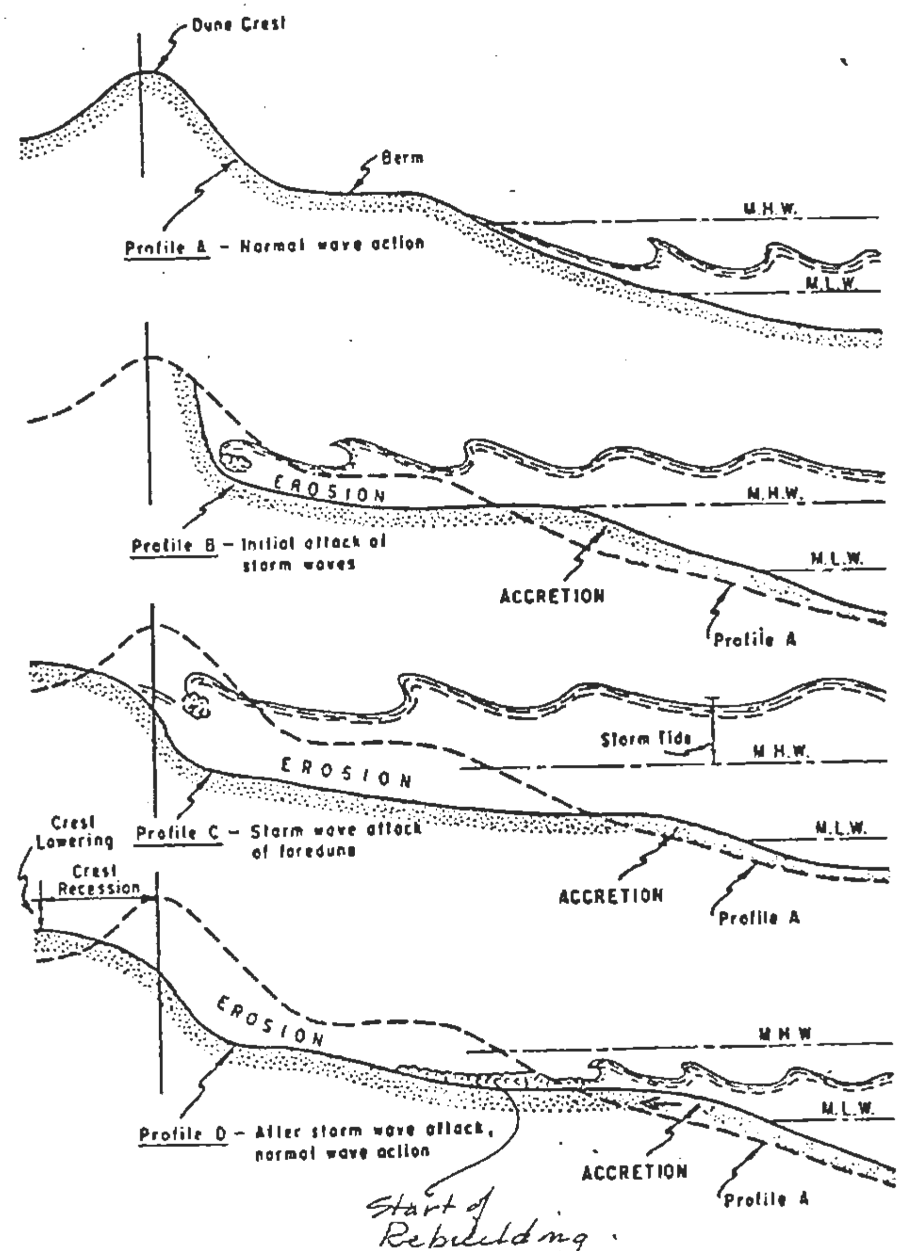


Figure 1-7. Schematic Diagram of Storm Wave Attack on Beach and Dune

Cross Shore Processes

- Dune and berm erosion
- Beach erosion
- Beach and berm rebuilding
- Dune rebulding
- Beach stability
- Bar formation
- Rip currents

Summary

W-E	61747			
E-W	160653			
NetQls	-98906	or	98906	**yd3/year E-W**
Gross	222400	yd3/year		

Complete set of BIN ΔQIs

QIs	137000	radians	1.374	0.785	0	-0.785	-1.374
	H ft						
	0.75		339	3060	0	-5202	-255
	1.5		960	10386	0	-27696	-1200
	2.5		1722	12415	0	-24830	-1722
	3.5		1996	14396	0	-43187	-3993
	4.5		1497	10793	0	-32380	-1871
	5.5		618	3565	0	-17825	-494
		Sum	7132	54615	0	-151119	-9534

Estimating Annual Transport

$$Q_{ls\ net} \approx K_y \sum_i \sum_j f_{ij} \{ H_{oi}^{5/2} (\cos \alpha_{oj})^{1/4} \sin 2\alpha_{oj} \}$$

Apply this equation to each bin to find the contribution to Qls from that BIN.

Example: SW BIN (-45o) at Ho 1 to 2 ft

ΔQls = 137000*(8/100)*(1.5^2.5)*[cos(-45°)]^0.25*sin(2*(-45°))

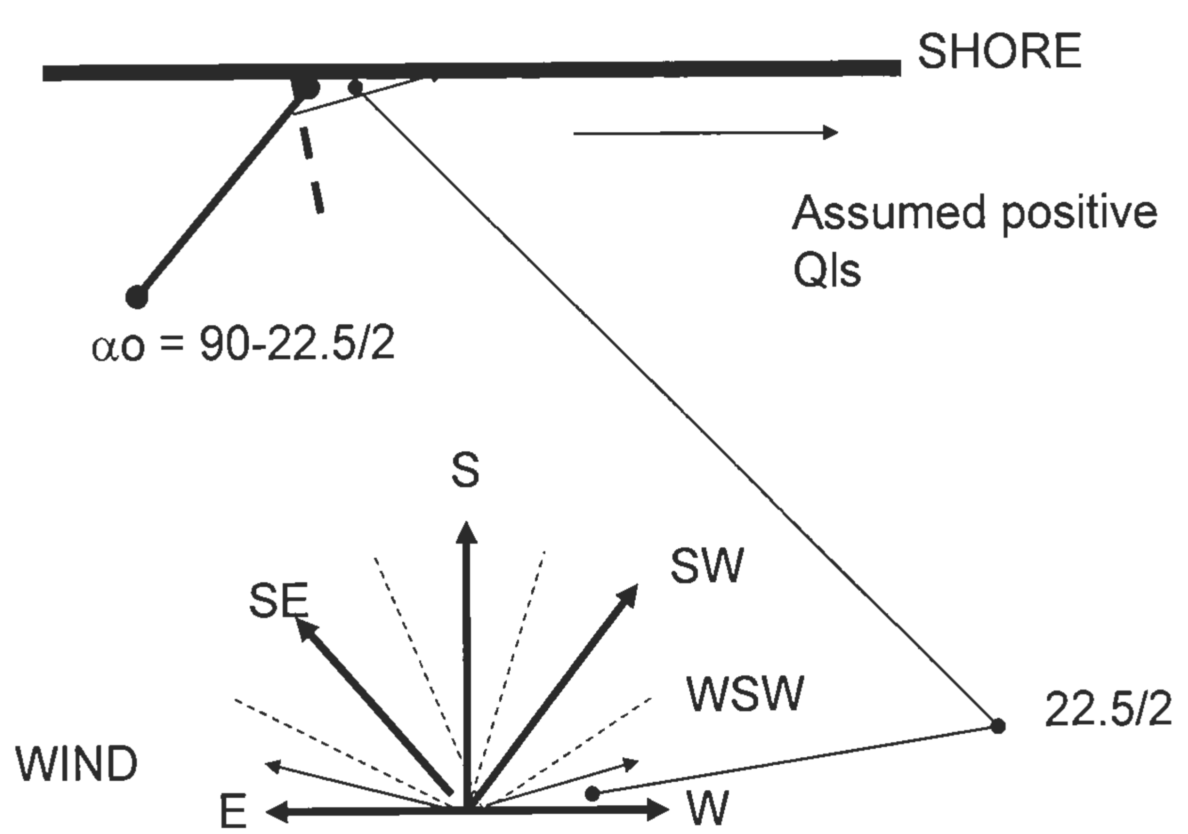
= -28000 yd3/year

Re-organize

NOTE:
W and E are split
Into two 22.5° BINS; all other BINS are 45°.

		Wind	West	SW	S	SE	East	
		WWNW	WWSW	SW	S	SE	EESE	EENE
H ft	α_o	>90	78.75	45	0	-45	-78.75	<-90
0.5	**1**		2	5	7	8.5	1.5	
1	**2**		1	3	4	**8**	1.25	
2	**3**		0.5	1	1.5	2	0.5	
3	**4**		0.25	0.5	1	1.5	0.5	
4	**5**		0.1	0.2	0.4	0.6	0.125	
5	**6**		0.025	0.04	0.04	0.2	0.02	

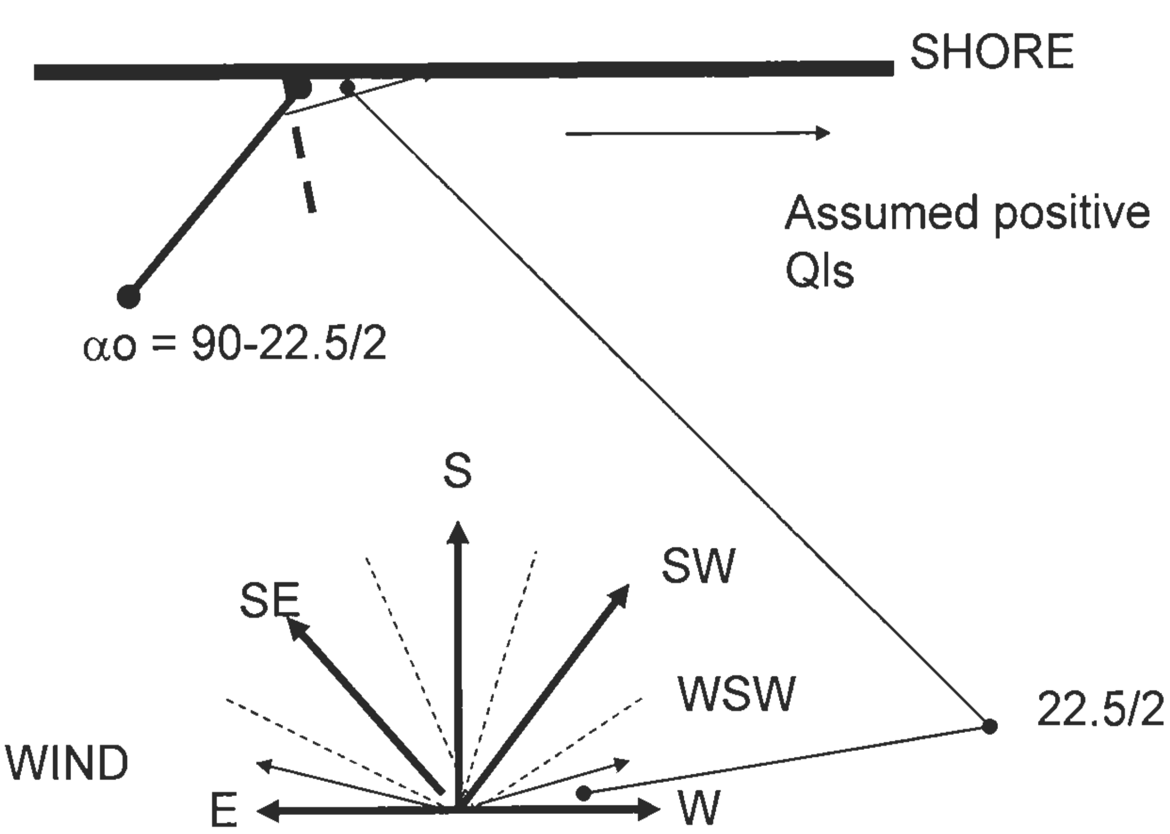
BINS for winds along the shore



Simplified Example of Wave Bins

Ho ft	W	SW	S	SE	E Wind Direction
	Percent	of Year			
0.5-1	4	5	7	8.5	3
1 to 2	2	3	4	8	2.5
2 to 3	1	1	1.5	2	1
3 to 4	0.5	0.5	1	1.5	1
4 to 5	0.2	0.2	0.4	0.6	0.25
5 to 6	0.05	0.04	0.04	0.2	0.04

BINS for winds along the shore

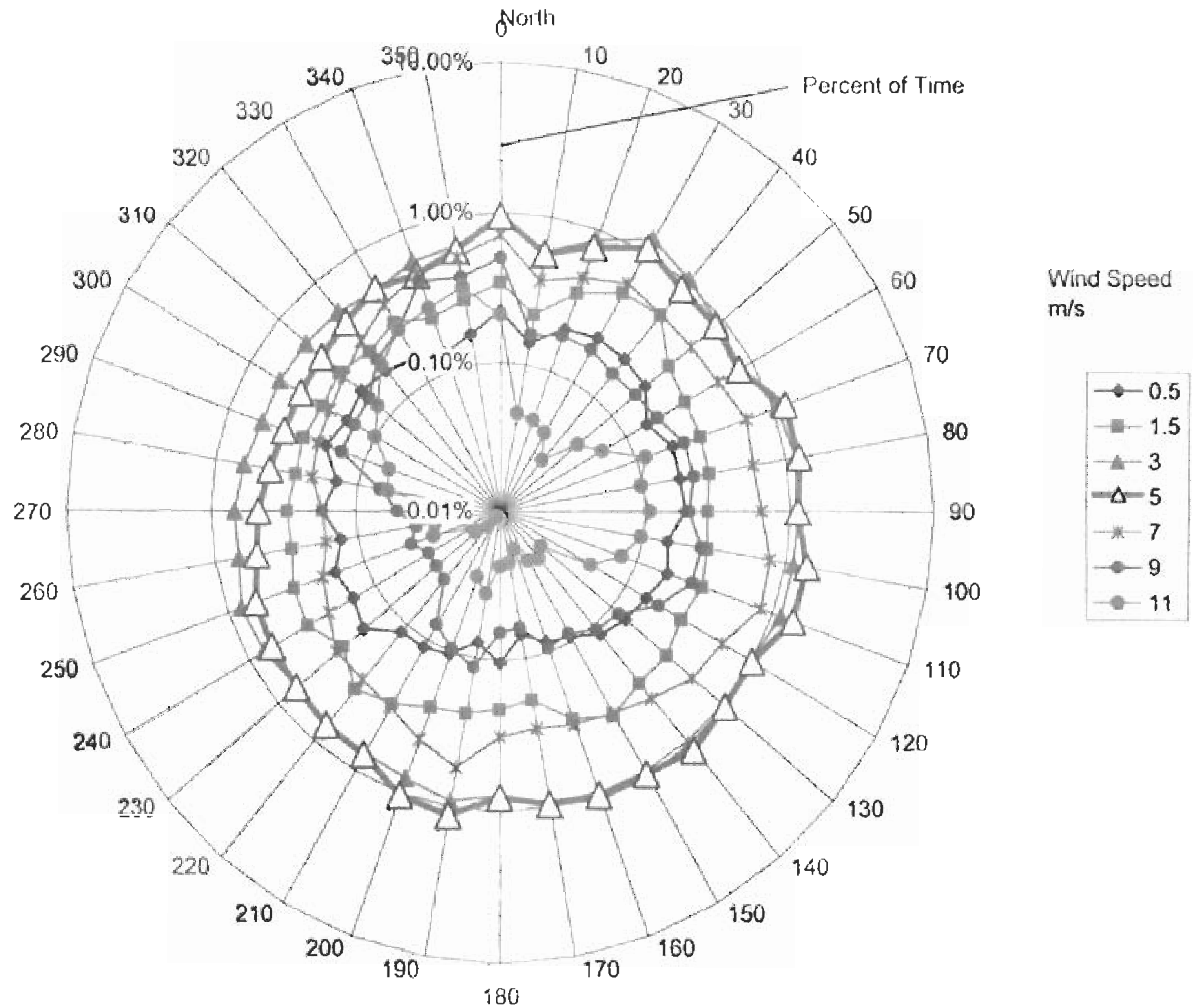


Bins for 10 year record on LP

Percent of time in each Wind Class

DIR.	calm< 1m/s	2.7	4.5	7.2	10.1	13.0	16.2	19.7	23.9	27.6	31.9	Sum
N	0.717	1.092	1.483	1.792	0.792	0.225	0.000	0.000	0.000	0.000	0.000	6.100
NNE	1.100	1.525	2.058	1.875	0.375	0.033	0.000	0.000	0.000	0.000	0.000	6.967
NE	1.042	1.575	2.158	2.033	0.308	0.025	0.000	0.000	0.000	0.000	0.000	7.142
ENE	0.792	1.517	2.358	1.992	0.400	0.100	0.017	0.000	0.000	0.000	0.000	7.175
E	0.842	1.783	3.075	2.475	0.442	0.117	0.042	0.000	0.000	0.000	0.000	8.775
ESE	0.808	1.675	2.958	2.333	0.425	0.083	0.000	0.000	0.000	0.000	0.000	8.283
SE	0.875	1.633	2.767	1.717	0.175	0.017	0.000	0.000	0.000	0.000	0.000	7.183
SSE	0.817	1.742	2.658	1.242	0.150	0.008	0.000	0.000	0.000	0.000	0.000	6.617
S	0.642	1.367	2.492	1.308	0.167	0.017	0.000	0.000	0.000	0.000	0.000	5.992
SSW	0.833	1.250	2.392	1.483	0.142	0.008	0.000	0.000	0.000	0.000	0.000	6.108
SW	0.958	1.317	1.900	1.192	0.100	0.000	0.000	0.000	0.000	0.000	0.000	5.467
WSW	0.917	1.308	1.608	0.808	0.100	0.008	0.000	0.000	0.000	0.000	0.000	4.750
W	0.950	1.250	1.333	0.717	0.125	0.033	0.000	0.000	0.000	0.000	0.000	4.408
WNW	0.833	1.083	1.050	0.850	0.292	0.083	0.017	0.000	0.000	0.000	0.000	4.208
NW	0.825	0.967	1.192	1.058	0.467	0.175	0.042	0.017	0.000	0.000	0.000	4.742
NNW	0.742	0.933	1.242	1.667	0.842	0.300	0.025	0.017	0.000	0.000	0.000	5.767
VAR												
CLM												
ALL	13.750	22.000	32.717	24.600	5.333	1.367	0.192	0.050	0.000	0.000	0.000	

L Pontchartrain Windrose



Potential Longshore Qls

The SPM gives the Potential Longshore transport approximation:

$$Q_{ls} \approx K_y H_o^{5/2} (\cos \alpha_o)^{1/4} \sin 2\alpha_o$$

Where Ky ~ 137000 for Ho in ft and Qls in yd3/year

Shore Processes

- Shoaling
- Refraction
- Diffraction
- Reflection
- Wind Setup
- Drift Current
- Wave Set down
- Wave Setup
- Runup
- Rush down or run down
- Longshore currents
- Longshore transport
- Cross shore transport
- Surf Beat
- Edge Waves
- Shear waves
- Rip currents
- Swash

Beach mechanics I & Assignment 8.1

Littoral Zone II

Assignment 7-1

Name ______________________________ **Due March 11, 2010**

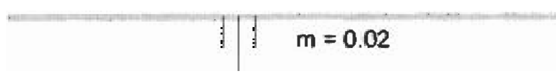
Compute the longshore transport for a deep water equivalent, H_o' = 3 ft at an angle (α_o) of 40 degree. The duration of the storm with this wave height is 8 hour.

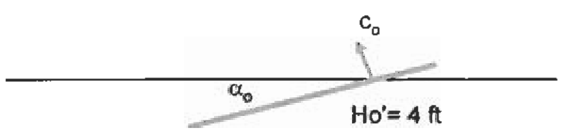
Q_{ls} = ______________________ units __________

Total longshore transport for the storm: ______________ units __________

Calculations

- Given: H_o'= 3 ft; α_o = 40°; T = 5 s;
- Duration = 8 h
- Find: Q_{ls} =
- Storm Transported Volume =





TERMS

http://www.csc.noaa.gov/beachnourishment/html/geo/shorelin.htm

- **Littoral cell** - A reach of the coast that is isolated sedimentologically from adjacent coastal reaches and that features its own sources and sinks. Isolation is typically caused by protruding headlands, submarine canyons, inlets, and some river mouths that prevent littoral sediment from one cell from passing into the next.
- **Littoral zone** - In beach terminology, an indefinite zone extending seaward from the shoreline to just beyond the breaker zone.
- **Longshore bar** - A sand bar that extends roughly parallel to the shoreline.
- **Longshore direction** - Parallel to and near the shoreline, alongshore.
- **Longshore sand bars** - A sand ridge or ridges, running roughly parallel to the shoreline and extending along the shore outside the trough, that may be exposed at low tide or may occur below the water level in the offshore.
- **Longshore transport** - A wave- and/or tide-generated movement of shallow-water coastal sediments parallel to the shoreline.
- **Low energy environments** - Coastlines where wave and tidal forces are typically relatively small due to the climate, the location of the site and / or due to nearshore submerged features that function to reduce incoming wave energy.

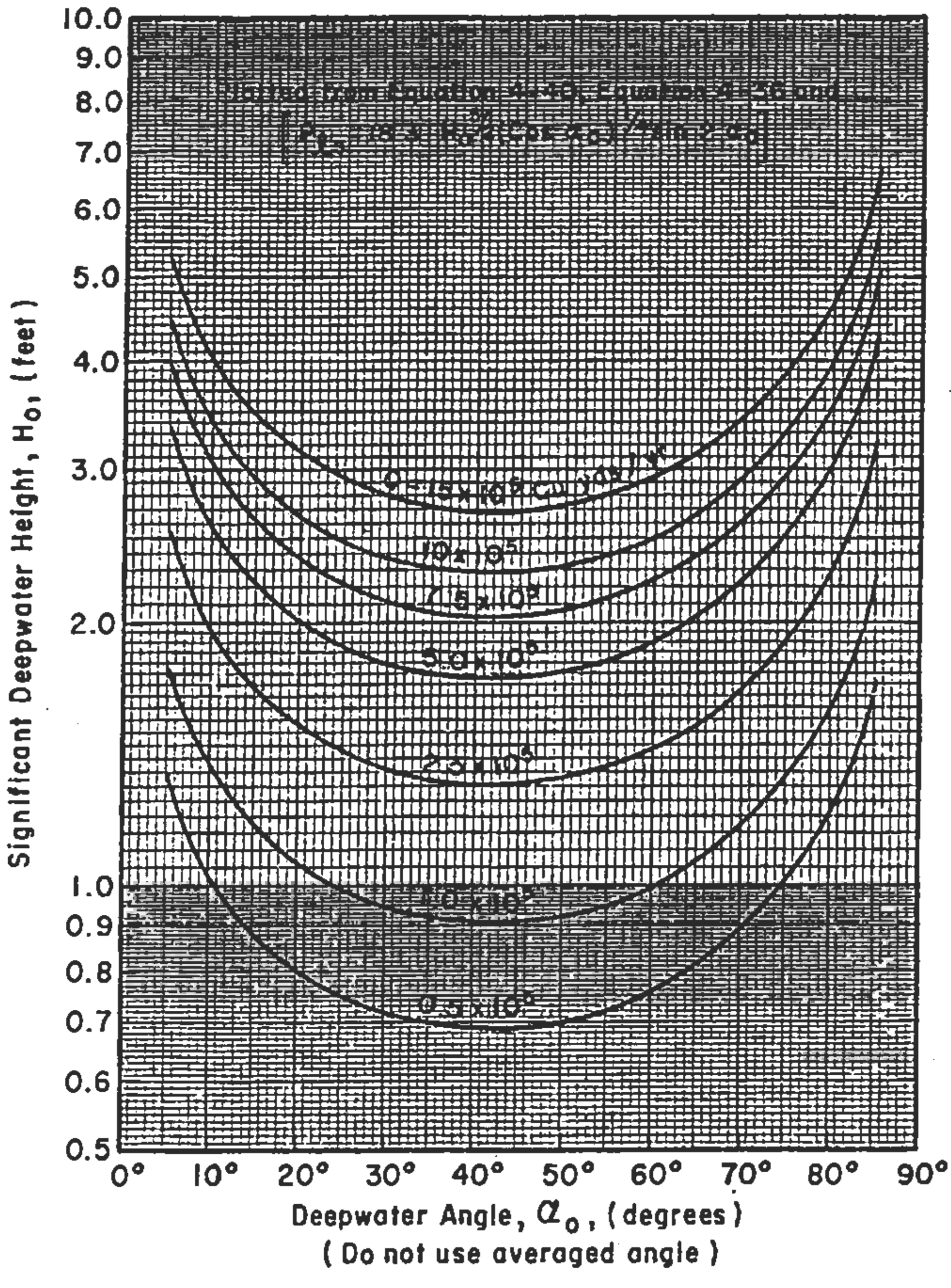


Figure 4-39. Longshore Transport Rate as a Function of Deepwater Height and Deepwater Angle

Figure 7.13 Q_{ls} prediction from deep water wave data

For other examples of formulae and procedures for estimating K see the CEM. Figure 7.12 (SPM) show the volume flux and Figure 7.13 can be used to obtain an estimate of Q_{ls} in terms of deep water data. In this approximation, the Corps of Engineers gives

$$Q_{ls} \approx K_y H_o^{5/2} [\cos\alpha_o]^{1/4} \sin 2\alpha_o$$
$$K_y = 1.37 * 10^5 \, for.yd^3 / year \qquad 7.10$$

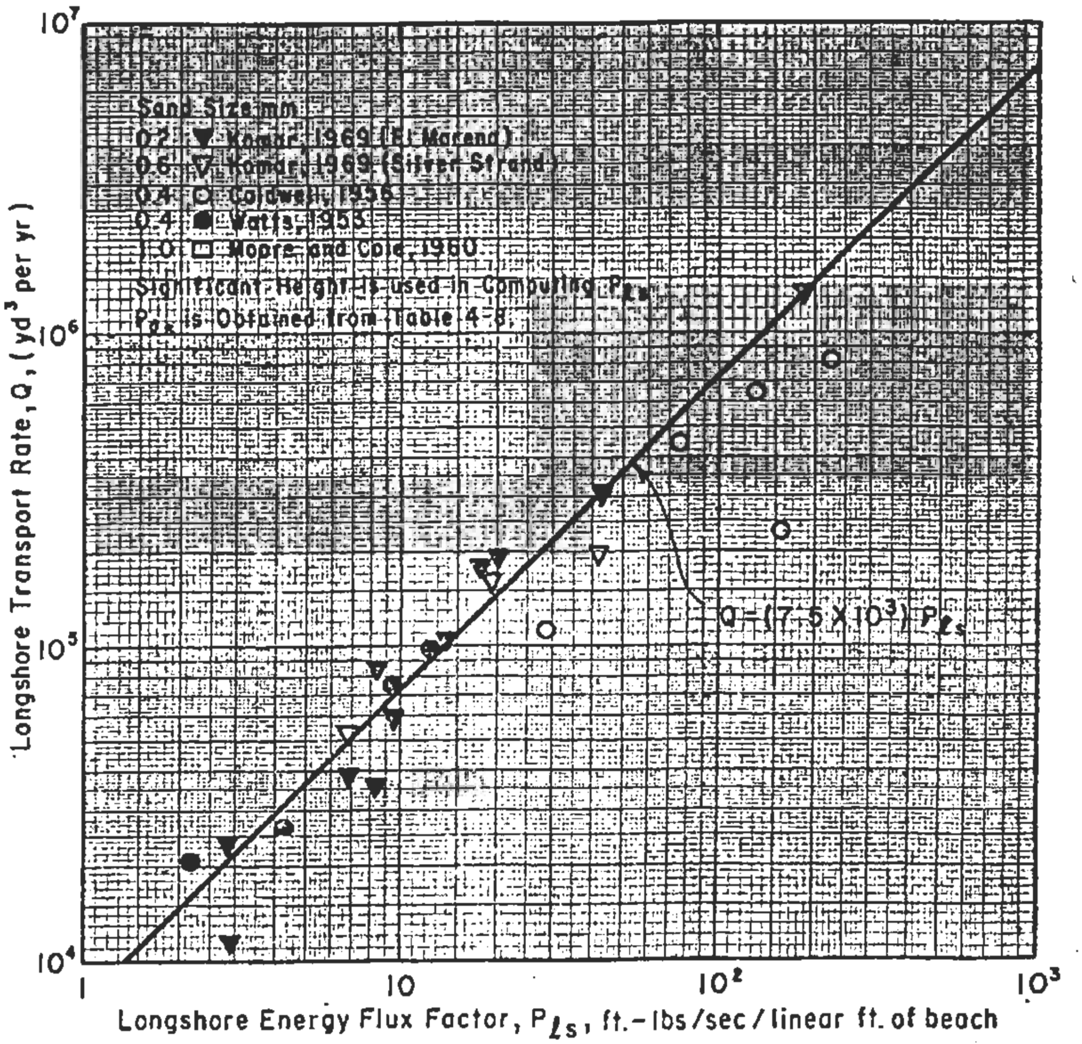


Figure 4-37. Design Curve for Longshore Transport Rate Versus Energy Flux Factor. Only field data are included.

Figure 7.12 Q_{ls} versus P_{ls}

This leads to

$$P_{ls} = \{\rho g H_b^{5/2}/16\}\{g/K_b\} \cdot \sin 2\alpha_b$$

It is assumed (SPM Fig. 7.11) that the longshore weight flux (I_{ls}) is proportional to the longshore energy flux (P_{ls}):

$$I_{ls} = K P_{ls} = K\{\rho g H_b^{5/2}/16\}\{g/K_b\} \cdot \sin 2\alpha_b \qquad 7.8$$

and the volume flux is

$$Q_{ls} = K P_{ls} / \{(\gamma_s - \gamma)(1 - \text{porosity})\}$$

$$= K\{\rho g H_b^{5/2}/16\}\{g/K_b\} \cdot \sin 2\alpha \,/\, [(\gamma_s - \gamma)(1 - \text{porosity})] \qquad 7.9$$

where K is a field determined Coefficient. (See Figure II-2-4 CEM)

The SPM Chart 4-39 is useful for easy rough estimation of the longshore transport.

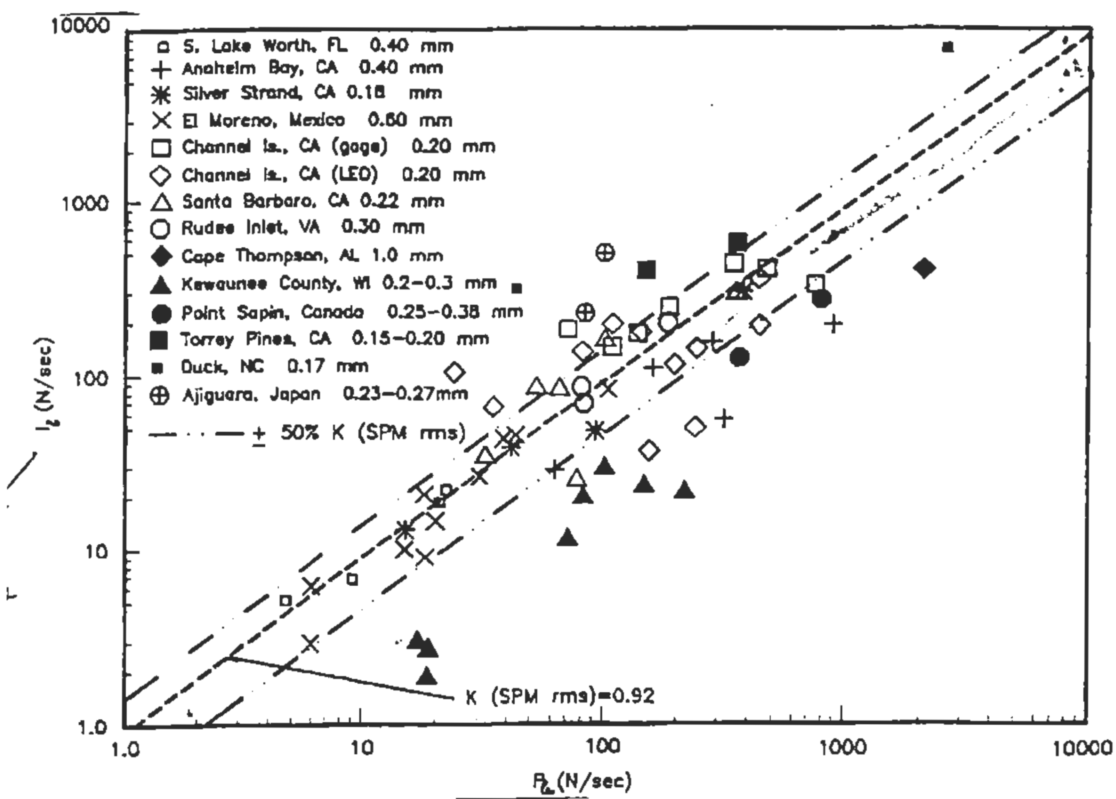


Figure 7-11: Weight Flux (I_{ls}) proportional to the longshore energy flux (P_{ls}):

Longshore Transport Prediction

Longshore transport may be expressed in terms of submersed weight per unit time (I_{ls}) or bulk volume per unit time (Q_{ls}). The conversion is

$$I_{ls} = Q_{ls}(\gamma_s - \gamma)(1 - \text{porosity}) \qquad 7.5$$

Where the sand porosity ~ 0.4.

A traditional method of estimating the longshore transport is to assume that it is proportional to the longshore component of the wave energy flux or power (P_{ls}) which is taken at the surf zone.

Referring to Figure 7.10 below, we can state that the wave energy flux per unit length of the wave crest is

$$\overline{E_b} = \frac{1}{8}\rho g H_b^2$$

$$P_{ls} = \frac{1}{16}\rho g H_b^2 \sqrt{g d_b} \qquad 7.6$$

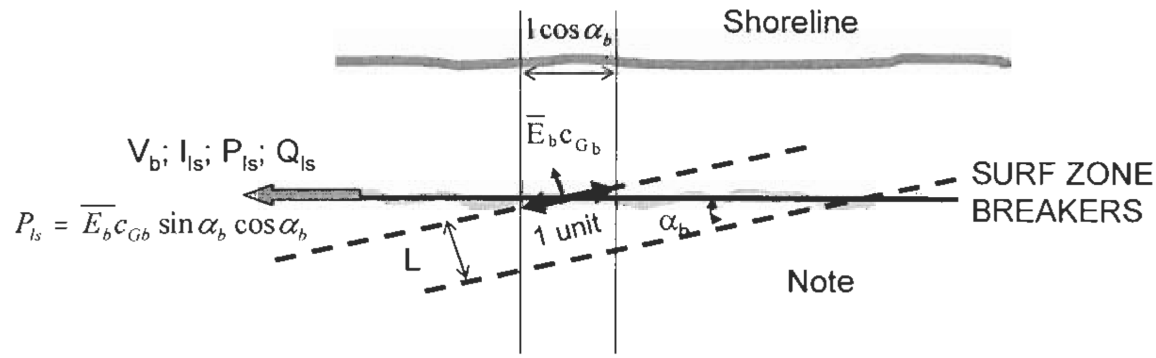


Figure 7.10 Longshore energy flux per unit length of shoreline

Based on the projected beach length (1 x cos α_b) and the longshore component we get

$$P_{ls} = \overline{E_b} c_{Gb} \sin\alpha_b \cos\alpha_b \qquad 7.7$$

$$= \{\rho g H_b^2/8\} \{g d_b\}^{\cdot} \cos\alpha_b \sin\alpha_b$$

$$= \{\rho g H_b^2/8\} \{g H_b/K_b\}^{\cdot} \cos\alpha_b \sin\alpha_b$$

where $K_b = \{H_b/d_b\} \sim 0.78$ for flat beach.

Table 4-14. Sand Budget of the Littoral Zone

Sources	
Rivers and streams	The major source in the limited areas where rivers carry sand to the littoral zone. In affected areas notable floods may contribute several times Q_g.
Cliff, dune and backshore erosion	Generally the major source where rivers are absent. 1 to 4 cu.yd./yr./ft.
Transport from offshore	Quantity uncertain.
Wind transport	Not generally important as a source.
$CaCO_3$ production	Significant in tropical climate. The value of 0.25 cu.yd./yr./ft. seems reasonable upper limit on temperate beach.
Beach replenishment	Varies from 0 to greater than Q_g.
Sinks	
Inlets and lagoons	May remove from 5 to 25 percent of Q_g per inlet. Depends on number of inlets, inlet size, tidal flow characteristics, and inlet age.
Overwash	Less than 1 cu.yd./yr./ft. at most, and limited to low barrier islands.
Beach storage	Temporary, but possibly large, depending on beach condition when budget is made. (See Table 4-5, pages 4-72, 4-73.)
Offshore slopes	Uncertain quantity. May receive much fine material, some coarse material.
Submarine canyons	Where present, may intercepr up to 80 percent of Q_g.
Deflation	Usually less than 2 cu.yd./yr./ft. of beach front, but may range up to 10 cu.yd./yr./ft.
$CaCO_3$ loss	Not known to be important.
Mining and dredging	May equal or exceed Q_g in some localities.
Convective Processes	
Longshore transport (waves)	May result in accretion of Q_g, erosion of Q_n, or no change depending on conditions of equilibrium.
Tidal Currents	May be important at mouth of inlet and vicinity, and on irregular coasts with high tidal range.
Winds	Longshore winds are probably not important, except in limited regions.

Figure 7.9 Quantification of Processes

Table 1-1 (SPM) classifies the various processes as natural or man-induced. Table 4-14 (SPM) provides some rough guidelines of the magnitudes for several of the sinks and sources.

Table 1-1. Causes of coastal erosion.

Natural	Man-induced
a. Sea level rise + natural subsidence	a. Land subsidence from removal of subsurface resources
b. Variability in sediment supply to the littoral zone	b. Interruption of material in transport
c. Storm waves	c. Reduction of sediment supply to the littoral zone
d. Wave and surge overwash	d. Concentration of wave energy on beaches
e. Deflation	e. Increase water level variation
f. Longshore sediment transport	f. Change natural coastal protection
g. Sorting of beach sediment	g. Removal of material from the beach

Figure 7.8 Nature and Man-made Causes of Coastal Erosion.

Sediment Budget in the Littoral Zone

Figure 4-49 SPM (Fig 7.7) shows a schematic of the processes that contribute to the sediment budget in the littoral zone. Although it may not be possible to completely quantify all of the fluxes, it is important to recognize the role of each component in any coastal restoration project.

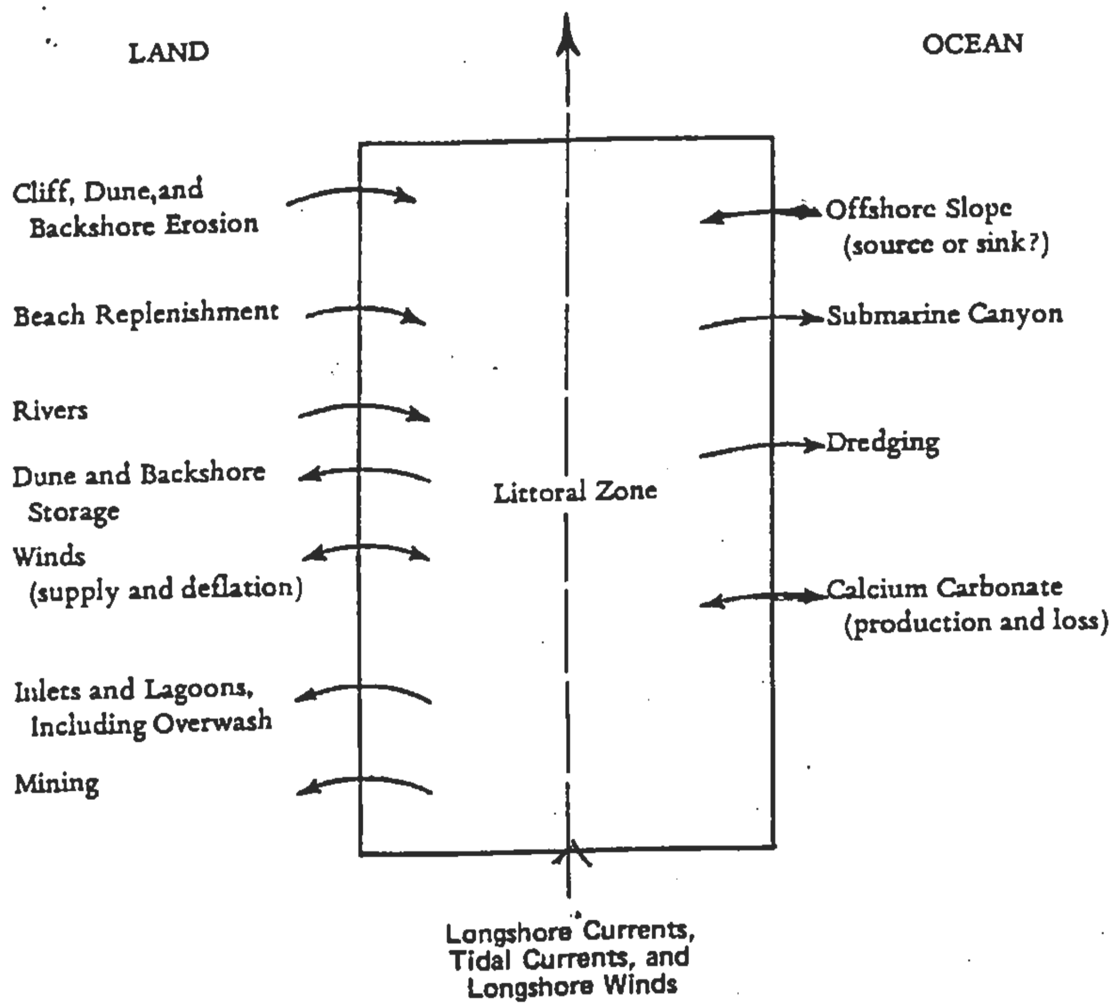


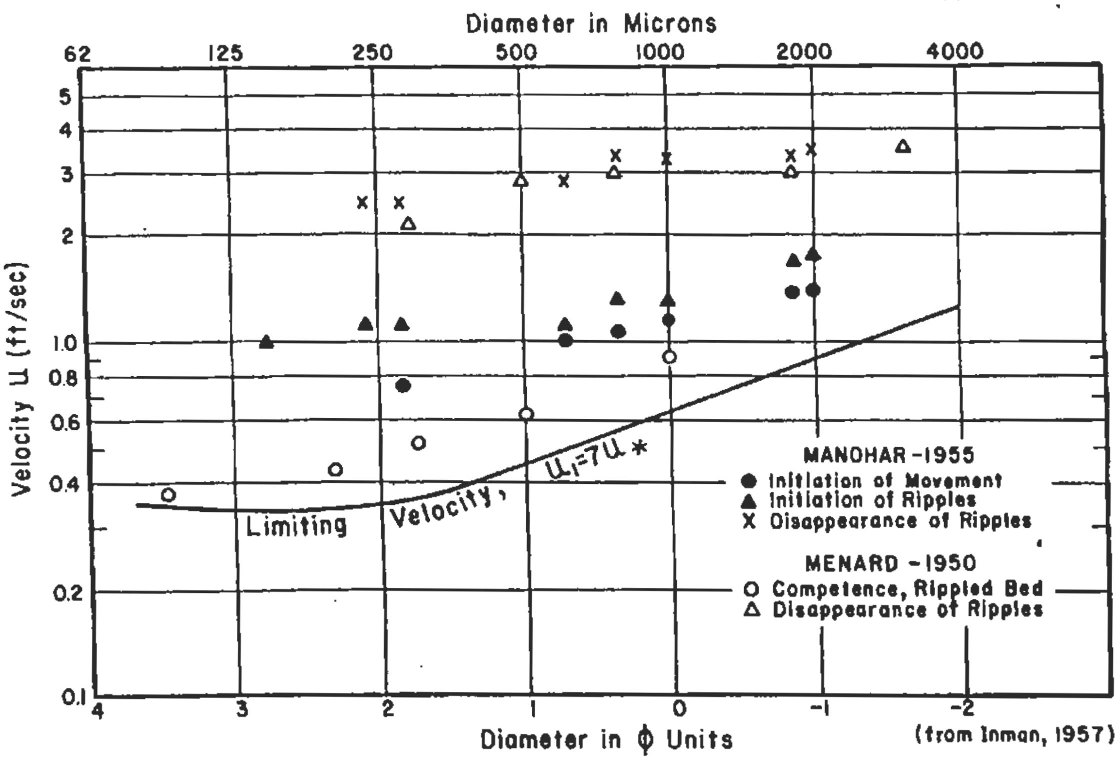
Figure 4-49. Materials budget for the littoral zone.

Figure 7.7 Sediment Budget for a littoral cell (SPM)

Table III-1-2
Sediment Particle Sizes

ASTM (Unified) Classification[1]	U.S. Std. Sieve[2]	Size in mm	Phi Size	Wentworth Classification[3]
Boulder		4096.	-12.0	
		1024.	-10.0	Boulder
	12 in. (300 mm)	256.	-8.0	
Cobble		128.	-7.0	Large Cobble
		107.64	-6.75	Small Cobble
		90.51	-6.5	
	3 in. (75 mm)	76.11	-6.25	
		64.00	-6.0	Very Large Pebble
		53.82	-5.75	
		45.26	-5.5	
Coarse Gravel		38.05	-5.25	Large Pebble
		32.00	-5.0	
		26.91	-4.75	
		22.63	-4.5	Medium Pebble
	3/4 in. (19 mm)	19.03	-4.25	
		16.00	-4.0	
		13.45	-3.75	Small Pebble
		11.31	-3.5	
		9.51	-3.25	
Fine Gravel	2.5	8.00	-3.0	Granule
	3	6.73	-2.75	
	3.5	5.66	-2.5	
	4 (4.75 mm)	4.76	-2.25	Very Coarse Sand
	5	4.00	-2.0	
Coarse Sand	6	3.36	-1.75	
	7	2.83	-1.5	Coarse Sand
	8	2.38	-1.25	
	10 (2.0 mm)	2.00	-1.0	
	12	1.68	-0.75	Medium Sand
	14	1.41	-0.5	
	16	1.19	-0.25	
	18	1.00	0.0	Fine Sand
Medium Sand	20	0.84	0.25	
	25	0.71	0.5	
	30	0.59	0.75	Very Fine Sand
	35	0.50	1.0	
	40 (0.425 mm)	0.420	1.25	
	45	0.354	1.5	Coarse Silt
	50	0.297	1.75	
	60	0.250	2.0	
	70	0.210	2.25	Medium Silt
Fine Sand	80	0.177	2.5	Fine Silt
	100	0.149	2.75	Very Fine Silt
	120	0.125	3.0	Coarse Clay
	140	0.105	3.25	Medium Clay
	170	0.088	3.5	Fine Clay
	200 (0.075 mm)	0.074	3.75	
Fine-grained Soil:	230	0.0625	4.0	
	270	0.0526	4.25	
Clay if PI ≥ 4 and plot of PI vs. LL is on or above "A" line and the presence of organic matter does not influence LL.	325	0.0442	4.5	
	400	0.0372	4.75	
		0.0312	5.0	
		0.0156	6.0	
		0.0078	7.0	
Silt if PI < 4 and plot of PI vs. LL is below "A" line and the presence of organic matter does not influence LL.		0.0039	8.0	
		0.00195	9.0	
		0.00098	10.0	
		0.00049	11.0	
(PI = plasticity limit; LL = liquid limit)		0.00024	12.0	
		0.00012	13.0	
		0.000061	14.0	

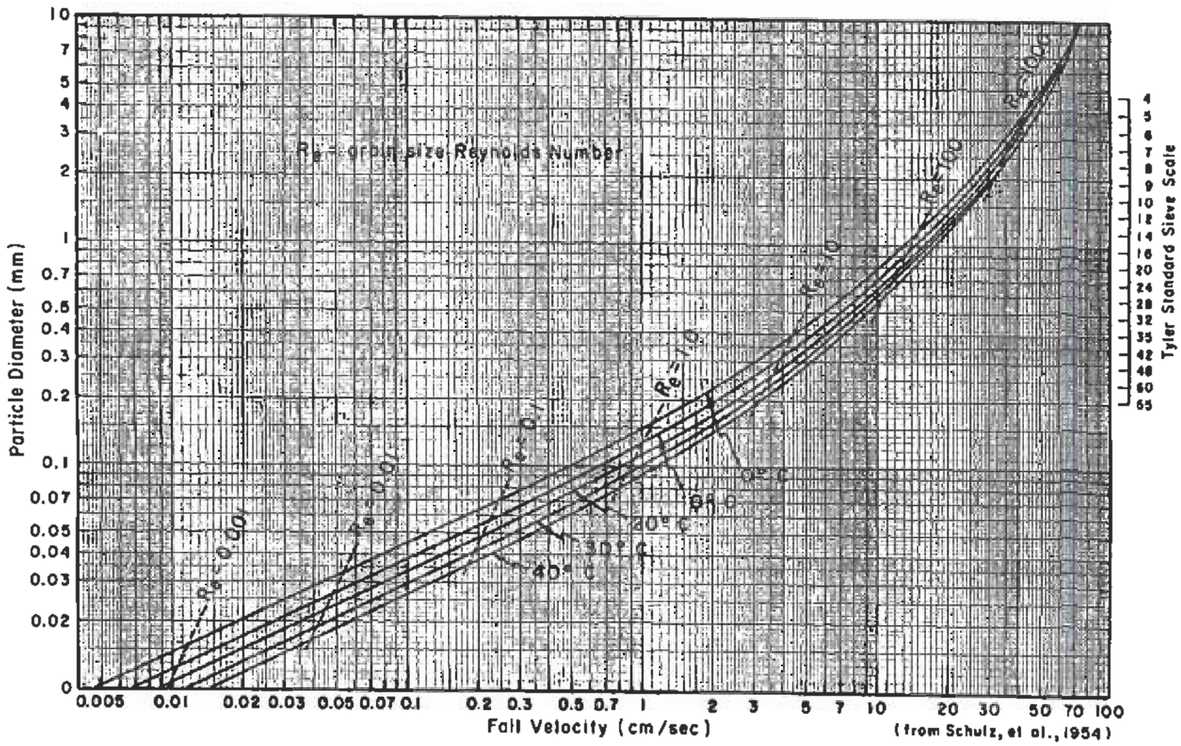
[1] ASTM Standard D 2487-92. This is the ASTM version of the Unified Soil Classification System. Both systems are similar (from ASTM (1994)).
[2] Note that British Standard, French, and German DIN mesh sizes and classifications are different.



Limiting or Minimum Velocity for the Initiation of Motion of Sand of a Given Size.
Limiting velocity, arbitrarily defined as equal to $7u_{*t}$ where u_* is the threshold or critical friction velocity. (Inman, 1949.) For unidirectional flow this relation would give a limiting velocity equivalent, for example, to the mean velocity measured 1 foot above a bottom which has a roughness length of 2 cm. Field observations near the surf zone indicate that planation and disappearance of ripples does not occur unless the maximum velocity associated with the wave crest somewhat exceeds that listed by Menard and Manohar.

Figure 4-22. Initiation of Ripple Motion

Figure 7.6 Velocity to initiate movement of sand grains.



$R_e = \frac{V_s D}{\nu g}$ Figure 4-31. Fall Velocity of Quartz Spheres in Water as a Function of Diameter and Temperature

Figure 7.5 Fall Velocity Chart

Porosity is the ratio of pore volume to total volume. Sands have porosities i the range 0.25 to 0.5 with a typical value of 0.4. Cohesive sediments have a much greater range of porosities from less than 0.2 to more than 0.8.

Entrainment Velocity is the bed velocity at which sand transport is initiated. The critical entrainment velocity is related to the sediment type, grain size, density, shape, water temperature and porosity. For sands the entrainment velocity varies in a similar way to the settling or fall velocity. For cohesive sediment the major factor is the porosity, i.e. the entrainment velocity decreases with increasing porosity.

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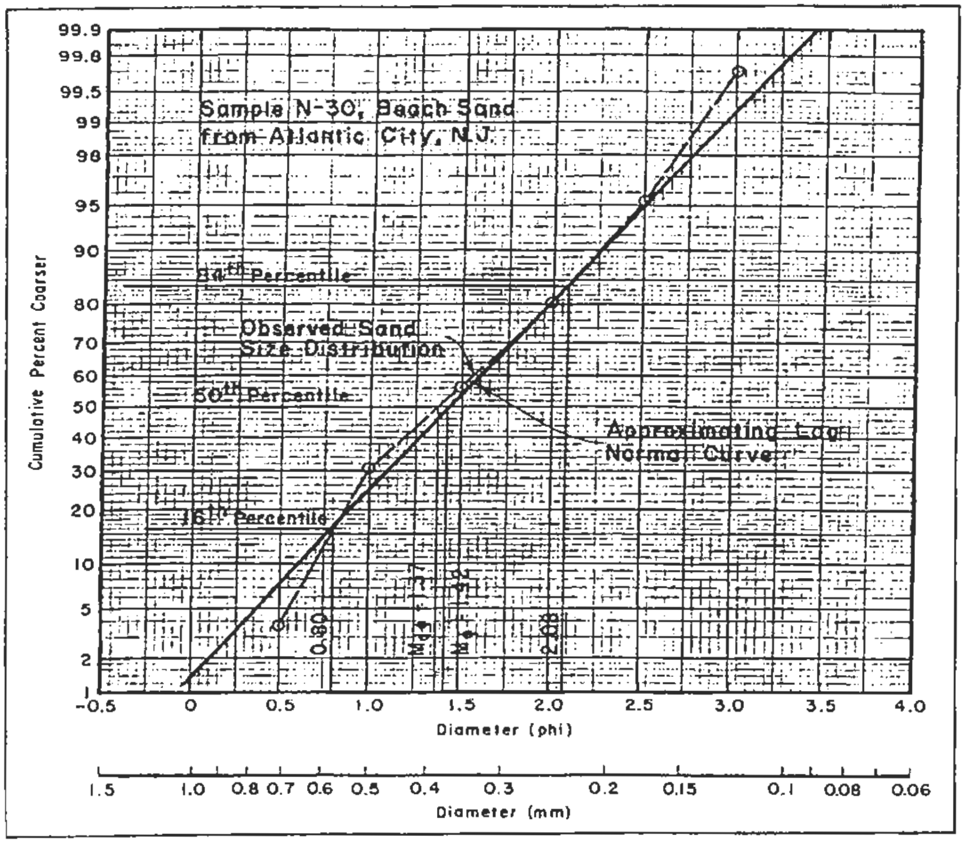


Figure III-1-2. Example of sediment distribution using log-normal paper

Figure 7-4 Example of grain size distribution for beach sand

Fall velocity is another useful description of sediment. The fall velocity depends on the grain size, density, and shape as well fluid properties such as viscosity and density. Figure III-1-6 CEM shows the settling velocity of quartz spheres. (See attached Figure4-31 from SPM) The settling velocity is related density and drag coefficient by,

$$0.5\, \rho\, C_D\, D_e^{\,2}\, V_s^{\,2} = C_{sh}\, \pi\, D_e^{\,3}\, (\gamma_s - \gamma)/6 \qquad 7.4$$

where C_D = drag coefficient = fcn(R_e) and C_{sh} = shape factor; the shape factor is 1 for spheres and less than 1 for all other shapes. D_e = equivalent spherical diameter. Plate-like particles can have much lower settling rates and higher drag coefficients than spheres; they are less resistant to entrainment..

Littoral Material Properties

Chapter 1-III of the CEM provides a detailed description of coastal sediments. The important properties include:

- Grain size {boulders, cobbles, gravel, sand, silt and clay}
- Grain size distribution
- Specific gravity {Ss = 2.65 for quartz sand}
- Shape
- Composition {resistance to abrasion}
- Bulk properties {porosity, permeability, bulk density, bearing strength, friction angle}

Grain Size may be express as:

Sieve number is compared to other measures in Table III-1-2 CEM.

The grain diameter can be given in mm or phi (ϕ) units where

$$\phi = -\log_2 (D \text{ mm}). \qquad 7.2$$

or

$$D = 2^{-(\phi)}$$

Grain size distribution is sometimes presented as a grain size versus cumulative probability (see Fig II-1-2 CEM) or in terms of distribution coefficients in terms of ϕ . For example, the standard deviation expressed in ϕ units is,

$$\sigma_\phi = (\phi 84 - \phi 16)/4 + (\phi 95 - \phi 5)/6 \qquad \text{(CEM revision)} \quad 7.3$$

$$\sigma_\phi = (\phi 84 - \phi 16)/2 \qquad \text{(SPM 1984)}$$

where 84, 16 etc refer to the cumulative % coarser in ϕ units. *Note: geotechnical reports often use cumulative % finer than.*

Surf Zone

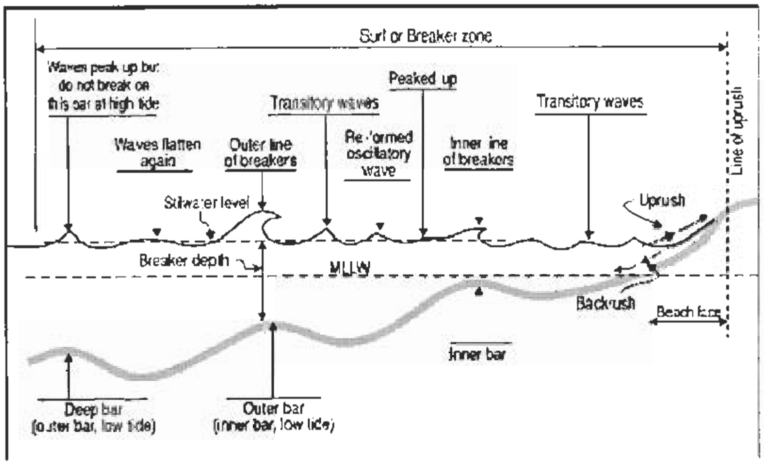
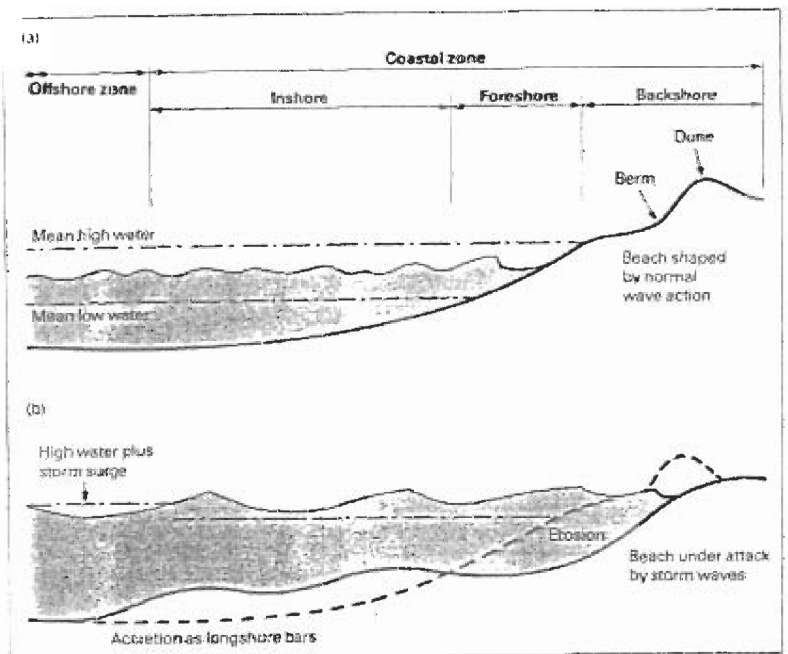


Figure 7.3b Coastal zone response to normal and storm waves (Terms)



Main zones and features of a typical coastline, showing the effect that waves have when combined with a storm surge as in (b).

Figure 7.3c Coastal zone response to normal and storm waves.

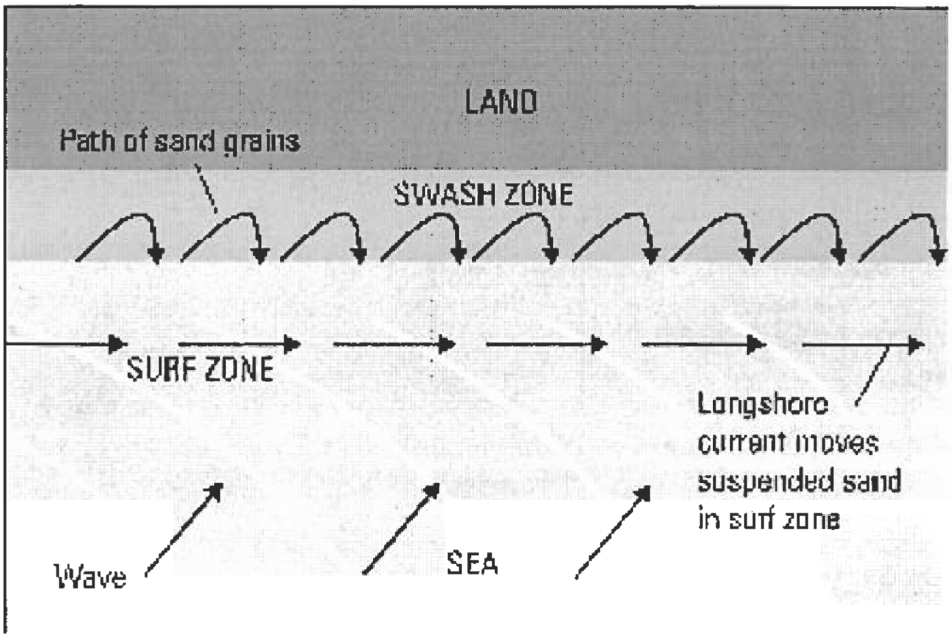


Figure 7.2 Plan view of wave attach on a beach.

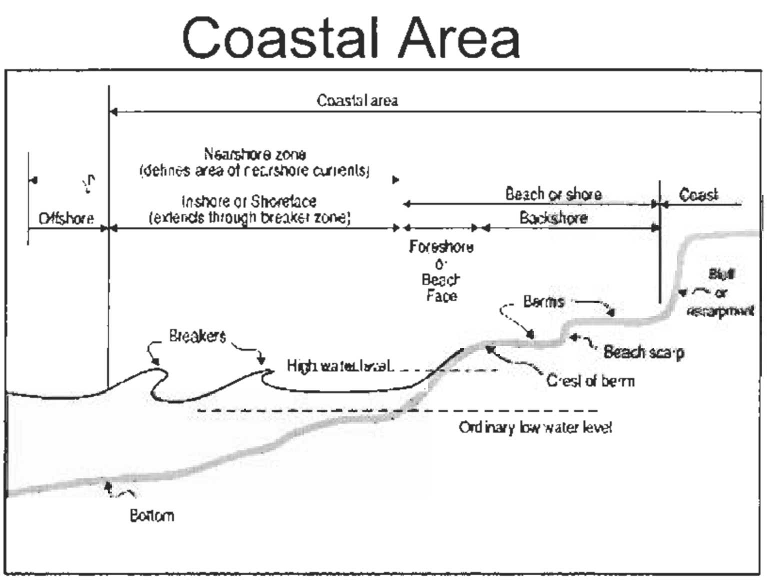


Figure 7.3a Coastal zone response to normal and storm waves (Terms)

Equation 7.1 gives the theoretical maximum current at the breaker line. Friction effect will result in reduction in this maximum and re-distribution of the momentum both onshore and offshore of the breaker line. A typical distribution is illustrated in Figure 7.1b (Dean and Dalrymple 2002).

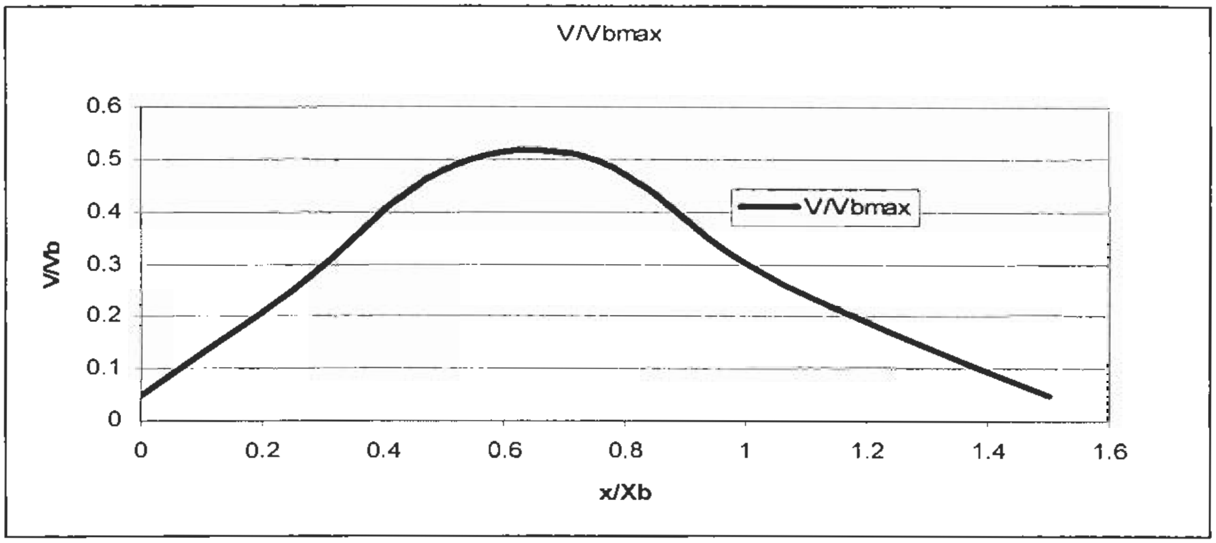


Figure 7.1b Effect of Friction on Distribution of Longshore Current (Longuet-Higgins from Dean and Dalrymple, 2002).

The onshore component results in a "wave setup" on the beach and provides the force that causes "rip-currents" to develop. Shoaling waves as they approach the breaker limit tend to have mainly translatory onshore motion (all water moving in the direction of the wave); the rip-current balances the flow by returning the flow to the offshore region. These currents may transport sediment offshore. In addition, bottom current seaward of the breaker zone may also result in offshore movement of the resuspended sediment. Figure 7.2 is a schematic plan view of the effect of refracted waves on a beach where the surf zone is where the waves break and the swash zone is where the broken waves runup the shoreline of the beach. Figure 7.3 define the parts of the littoral zone and shows the onshore-offshore responses due to storm waves/storm surge (b) and swells (inter-storm) events (a). The storm waves may erode the stored sand on the foreshore or even the backshore and deposit this in a bar or multiple bars in the littoral zone. Waves tend to break on these bars thus providing a natural reduction in the wave energy that reaches the beach.

Lecture 7
Sediment Transport in the Littoral Zone
Part I

Littoral Transport

This refers to transport of sand in the ***littoral zone*** which extends from the beach to the edge of the region where wave energy is sufficient to entrain and transport the bed material. The transport in this zone may be ***onshore-offshore*** and/or ***longshore***. Longshore refers to transport or currents that are parallel to the shoreline. A ***littoral cell*** is a self-contained littoral zone. A list of terms (NOAA) related to the littoral zone is attached to this lecture.

The energy (some references use momentum) to resuspend the bed sediment is attributed to the currents and turbulent energy that is released when the waves break. The longshore current to transport the resuspended sediment is mainly caused by the excess momentum in the waves at the time of breaking plus the general lake or sea circulation. The total excess momentum can be resolved into onshore and longshore components. Figure 7.1 shows a shoaling-refracted wave and the resulting longshore transport. Note that transport can be due to diffraction as well as refraction of waves. Figure 7.3 shows the effect of a harbour jetty on longshore transport. A commonly used formula for the strength of the longshore current is due to Longuet-Higgins (SPM 1973),

$$v_b = M_1 \, m \, (gH_b)^{-} \sin(2\,\alpha_b) \qquad 7.1$$

where $M_1 = 0.694\, \Lambda / \{ f_f \, (2\,\beta)^{\cdot\cdot} \} \simeq 20.7$; m = beach slope; f_f = friction coefficient ~ 0.01 for sand beaches; $\beta = d_b / H_b \sim 1.2$; Λ = mixing coefficient ~ 0.2 to 0.5. Field calibration is in good agreement with $M_1 = 20.7$.

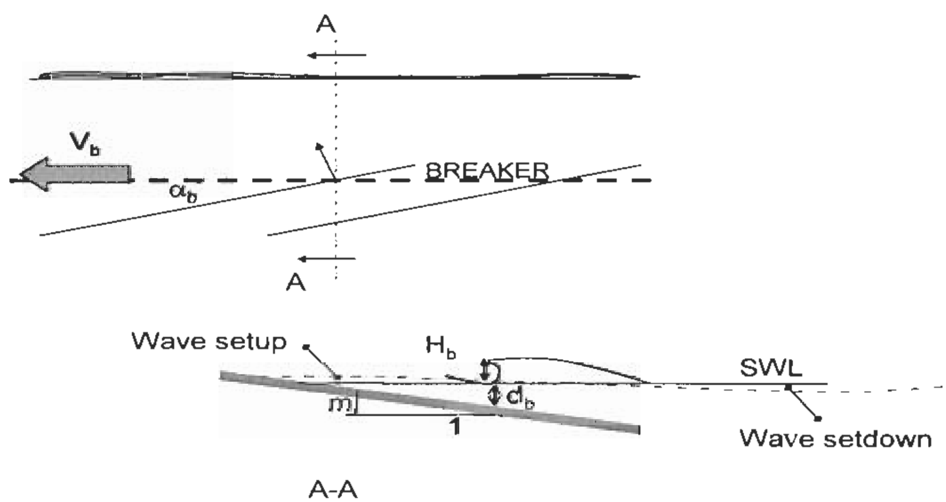
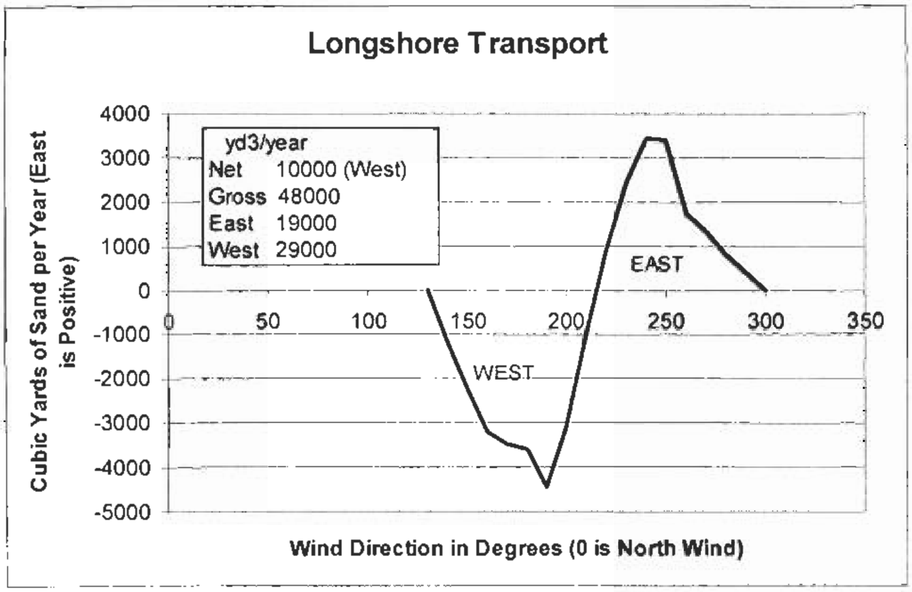
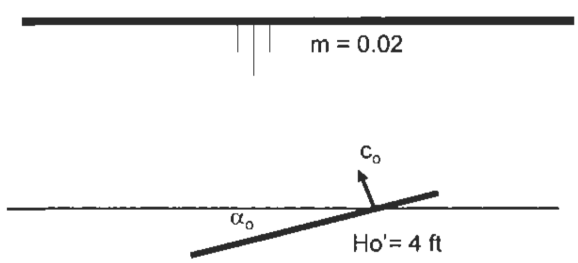


Figure 7.1a Longshore Currents

Longshore Transport at Fontainebleau Beach

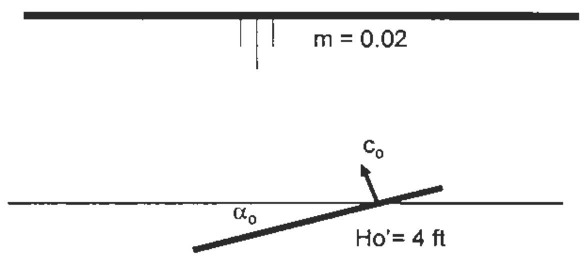


- Given: H_o'= 3 ft; α_o = 40°; T = 5 s;
- Duration = 8 h
- Find: Q_{ls} =
- Storm Transported Volume =



Sample Problem

- Given: H_o'= 4 ft; α_o = 30°; T = 5 s;
- Duration = 4 h
- Find: Q_{ls} =
- Storm Transported Volume =



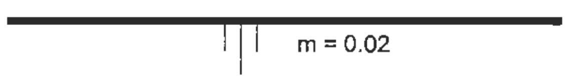
Sample Problem

$$Q_{ls} \approx K_y H_o^{5/2}[\cos\alpha_o]^{1/4}\sin 2\alpha_o$$

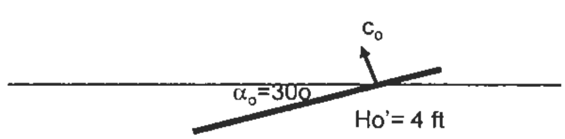
$$K_y = 1.37 * 10^5 \; for.\, yd^3 / year$$

Q_{ls} = 3.7 million yd3/year

Volume *ls* = 1670 yd3



- Duration = 4 h



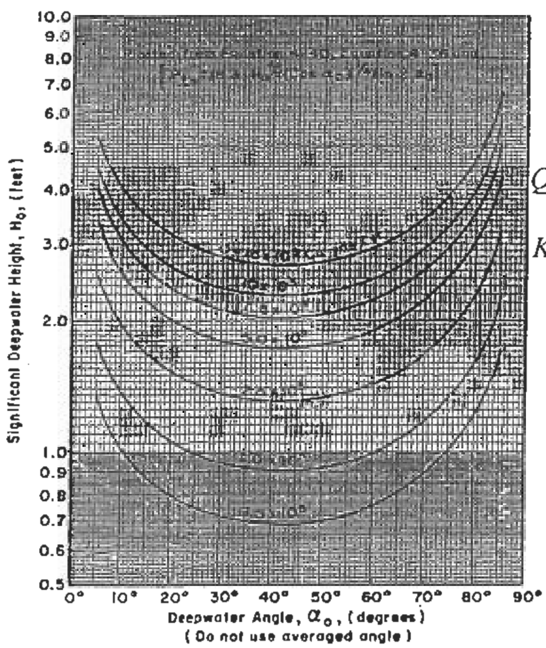
Longshore Transport Rate Qls

- Given:
  - H_b = 4 ft
  - T = 5 sec
  - □ α_b = 20°
  - m = 2%

KQ = 7500

Find: $$Q_{ls} = K_Q P_{ls}$$

3.9 million yd3/(year)



$$Q_{ls} \approx K_y H_o^{5/2} [\cos\alpha_o]^{1/4} \sin 2\alpha_o$$

$$K_y = 1.37 * 10^5 \; for.\, yd^3 \,/\, year$$

Figure 4-39. Longshore Transport Rate as a Function of Deepwater Height and Deepwater Angle

Longshore Transport Rate Ils

- Given:
  - H_b = 4 ft
  - T = 5 sec
  - α_b = 20°
  - m = 2%

K = 0.92

Find: $$I_{ls} = KP_{ls}$$

474 lbs/(sec)

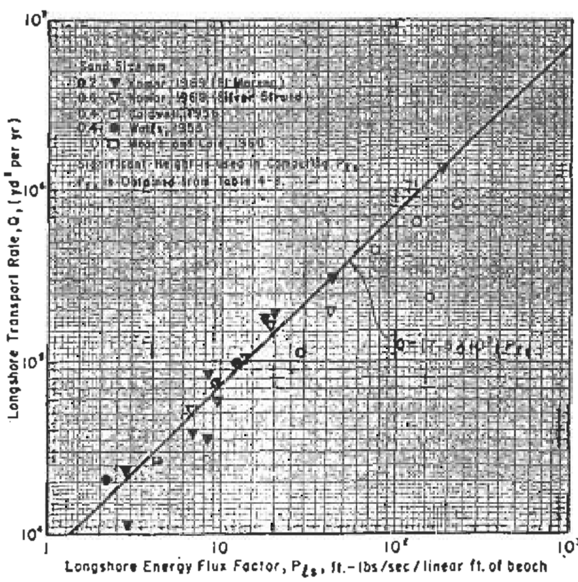


Figure 4-37. Design Curve for Longshore Transport Rate Versus Energy Flux Factor. Only field data are included.

$Q_{ls} = K_Q P_{ls}$

Longshore Energy Flux

- Given:
  - – $H_b = 4$ ft
  - – $T = 5$ sec
  - □ $\alpha_b = 20^o$
  - – $m = 2\%$

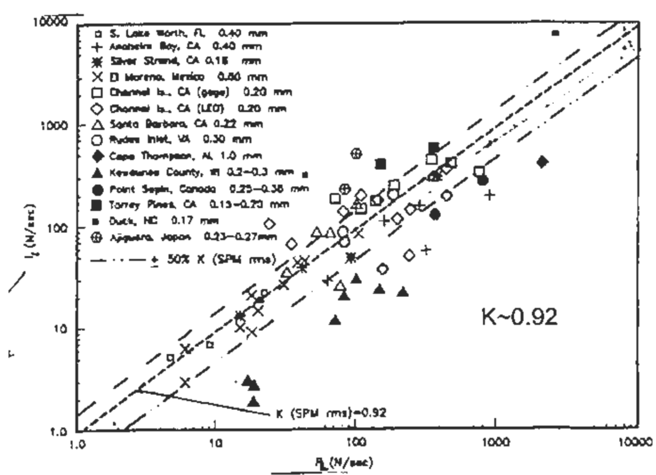
Find:

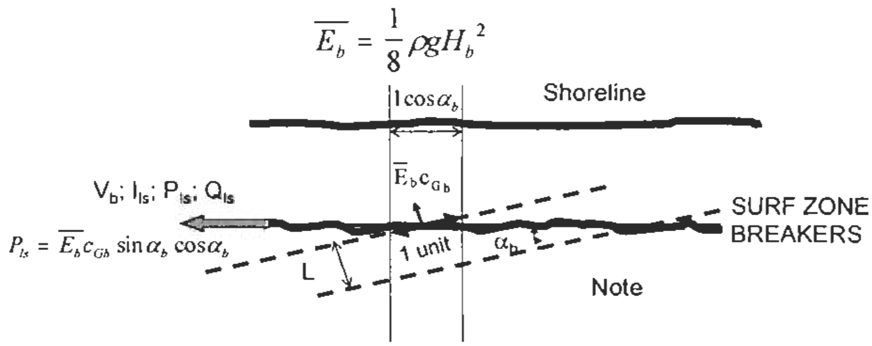
$$P_{ls} = \frac{1}{16}\rho g H_b^{5/2}\sqrt{\frac{g}{K_b}}\sin 2\alpha_b$$

515 ft-lbs/(sec.ft)

Weight vs longshore energy flux

$$I_{ls} = K\,P_{ls}$$





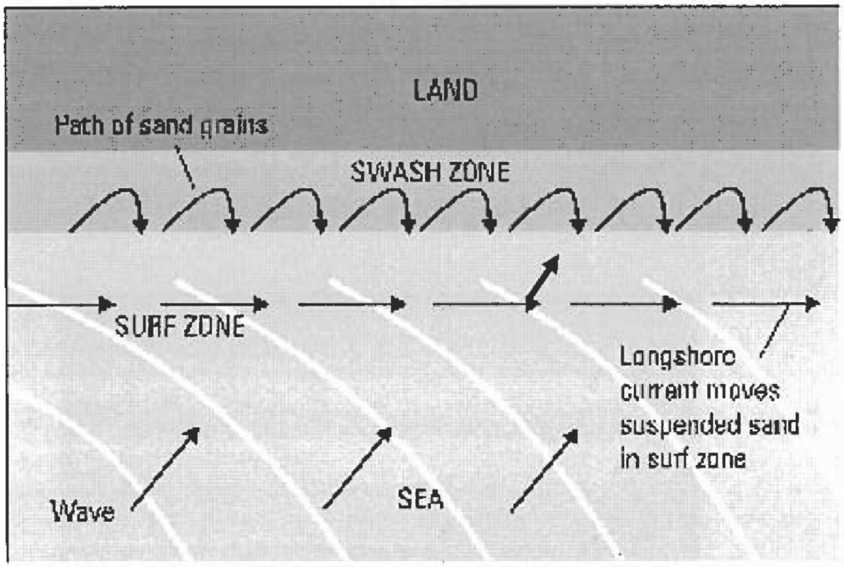
Longshore energy flux

$$P_{ls} = \overline{E_b} c_{Gb} \sin\alpha_b \cos\alpha_b$$

$$P_{ls} = \frac{1}{16}\rho g H_b^2 \sqrt{g d_b} \sin 2\alpha_b$$

$K_b = \{H_b/d_b\}$~ 0.78 for flat beach.

$$P_{ls} = \frac{1}{16}\rho g H_b^{5/2} \sqrt{\frac{g}{K_b}} \sin 2\alpha_b$$



Distribution of Longshore Energy Flux

$I_{ls} = Q_{ls}(\gamma_s - \gamma)(1 - \text{porosity})$

I_{ls} = longshore Submerged weight transport rate

Q_{ls} = longshore volume transport rate

P_{ls} = longshore energy flux

γ_s = specific weight of sediment

γ = specific weight of water

- 30,000 m^3/yr trapped since 1983/84

Table 4-14. Sand Budget of the Littoral Zone

Sources	
Rivers and streams	The major source in the limited areas where rivers carry sand to the littoral zone. In affected areas notable floods may contribute several times Q_g.
Cliff, dune and backshore erosion	Generally the major source where rivers are absent. 1 to 4 cu.yd./yr./ft.
Transport from offshore	Quantity uncertain.
Wind transport	Not generally important as a source.
$CaCO_3$ production	Significant in tropical climates. The value of 0.25 cu.yd./yr./ft. seems reasonable upper limit on temperate beach.
Beach replenishment	Varies from 0 to greater than Q_g.
Sinks	
Inlets and lagoons	May remove from 5 to 25 percent of Q_g per inlet. Depends on number of inlets, inlet size, tidal flow characteristics, and inlet age.
Overwash	Less than 1 cu.yd./yr./ft. at most, and limited to low barrier islands.
Beach storage	Temporary, but possibly large, depending on beach condition when budget is made. (See Table 4-5, pages 4-71, 4-72.)
Offshore slopes	Uncertain quantity. May receive much fine material, some coarse material.
Submarine canyons	Where present, may intercept up to 80 percent of Q_g.
Deflation	Usually less than 1 cu.yd./yr./ft. of beach front, but may range up to 10 cu.yd./yr./ft.
$CaCO_3$ loss	Not known to be important.
Mining and dredging	May equal or exceed Q_g in some localities.
Convective Processes	
Longshore transport (waves)	May result in accretion of Q_g, erosion of Q_n, or no change depending on conditions of equilibrium.
Tidal Currents	May be important at mouth of inlet and vicinity, and on irregular coasts with high tidal range.
Winds	Longshore winds are probably not important, except in limited regions.

Table 1-1. Causes of coastal erosion.

Natural	Man-induced
a. Sea level rise + natural subsidence	a. Land subsidence from removal of subsurface resources
b. Variability in sediment supply to the littoral zone	b. Interruption of material in transport
c. Storm waves	c. Reduction of sediment supply to the littoral zone
d. Wave and surge overwash	d. Concentration of wave energy on beaches
e. Deflation	e. Increase water level variation
f. Longshore sediment transport	f. Change natural coastal protection
g. Sorting of beach sediment	g. Removal of material from the beach

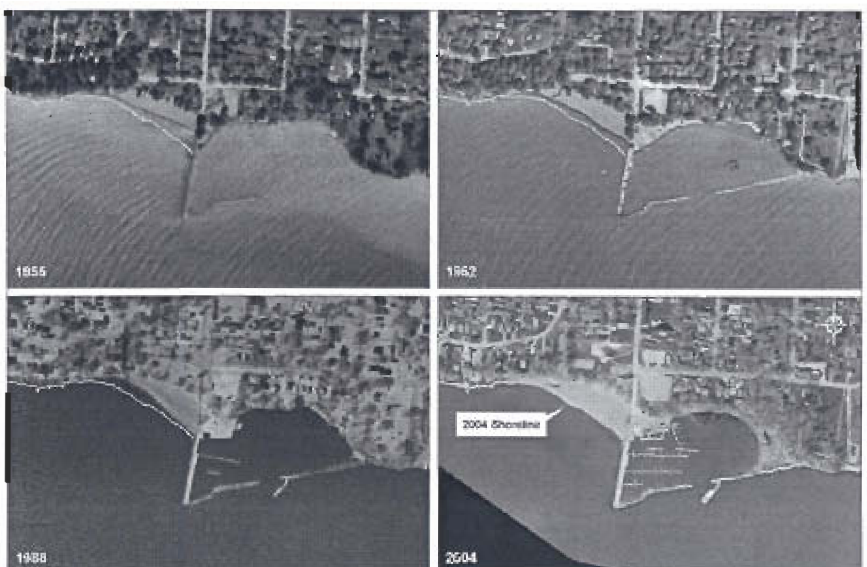


Figure 3.11 Breakwater and Beach Changes at Colchester Harbour

Baird

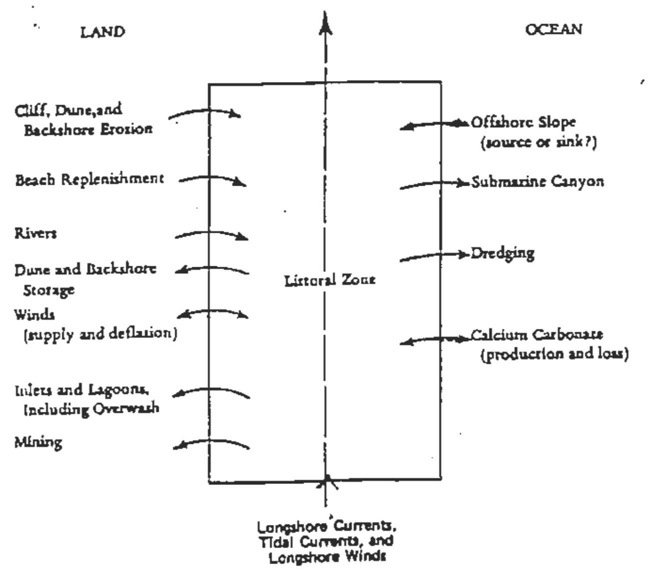
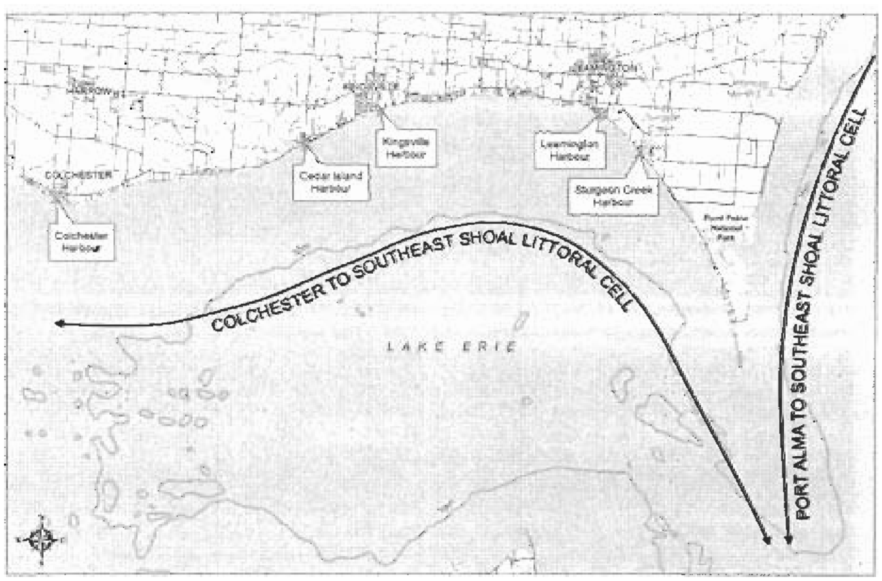
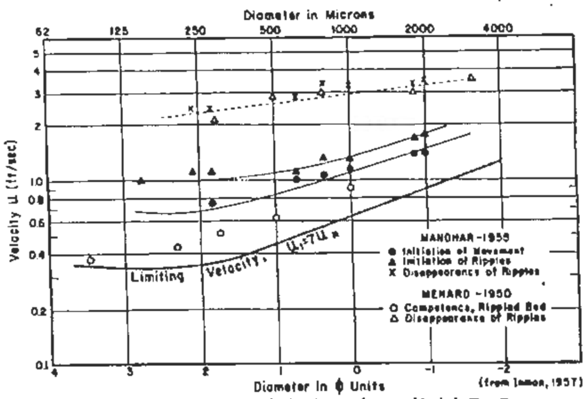


Figure 4-49. Materials budget for the littoral zone.



Initial Motion



Limiting or Minimum Velocity for the Initiation of Motion of Sand of a Given Size.
Limiting velocity, arbitrarily defined as equal to $7u_*$, where u_* is the threshold or critical friction velocity. (Inman, 1949.) For unidirectional flow this relation would give a limiting velocity equivalent, for example, to the mean velocity measured 1 foot above a bottom which has a roughness length of 2 cm. Field observations near the surf zone indicate that planation and disappearance of ripples does not occur unless the maximum velocity associated with the wave crest somewhat exceeds that listed by Menard and Manohar.

Figure 4-22. Initiation of Ripple Motion

Initial Motion Velocity for Pontchartrain Beach Sand

- D50 = 0.52 mm

T = 20°C

Figure 4.22

Uc = 0.45 to 0.85 ft/sec

ϕ for Pontchartrain Beach Sand

- D50 = 0.52 mm

□ $\phi = -\log_2 (Dmm)$
$= -\{\log_{10}(Dmm)\}/\log_{10}(2) = 0.94$

Fall velocity for Pontchartrain Beach Sand

- D50 = 0.52 mm

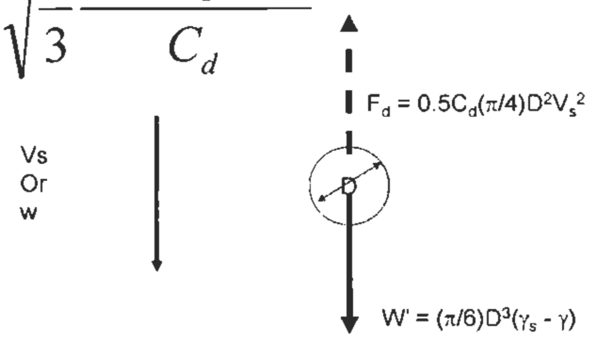
T = 20°C

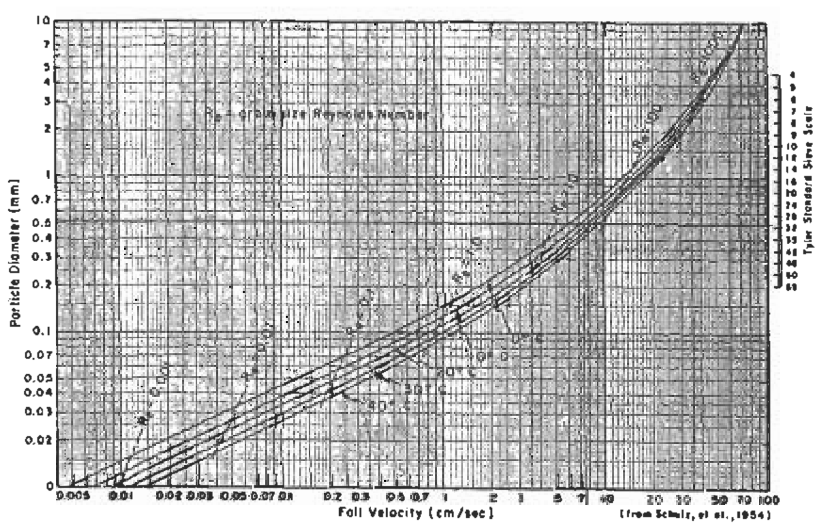
Figure 4.31

w = 7.5 cm/s

Settling Velocity

$$V_s = \sqrt{\frac{4}{3}\frac{gD(S_s - 1)}{C_d}}$$





$R_e = \frac{V_s D}{\nu}$ Figure 4-31. Fall Velocity of Quartz Spheres in Water as a Function of Diameter and Temperature

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		45.26	-5.5	
Coarse Gravel		38.05	-5.25	Large Pebble
		32.00	-5.0	
		26.91	-4.75	
		22.63	-4.5	Medium Pebble
	¾ in. (19 mm)	19.03	-4.25	
		16.00	-4.0	
		13.45	-3.75	Small Pebble
		11.31	-3.5	
		9.51	-3.25	
Fine Gravel	2.5	8.00	-3.0	Granule
	3	6.73	-2.75	
	3.5	5.66	-2.5	
	4 (4.75 mm)	4.76	-2.25	Very Coarse Sand
	5	4.00	-2.0	
Coarse Sand	6	3.36	-1.75	
	7	2.83	-1.5	Coarse Sand
	8	2.38	-1.25	
	10 (2.0 mm)	2.00	-1.0	
	12	1.68	-0.75	Medium Sand
	14	1.41	-0.5	
	16	1.19	-0.25	
	18	1.00	0.0	Fine Sand
Medium Sand	20	0.84	0.25	
	25	0.71	0.5	
	30	0.59	0.75	Very Fine Sand
	35	0.50	1.0	
	40 (0.425 mm)	0.420	1.25	
	45	0.354	1.5	Coarse Silt
	50	0.297	1.75	
	60	0.250	2.0	
	70	0.210	2.25	Medium Silt
Fine Sand	80	0.177	2.5	Fine Silt
	100	0.149	2.75	Very Fine Silt
	120	0.125	3.0	Coarse Clay
	140	0.105	3.25	Medium Clay
	170	0.088	3.5	Fine Clay
	200 (0.075 mm)	0.074	3.75	
Fine-grained Soil	230	0.0625	4.0	
	270	0.0526	4.25	
Clay if PI ≥ 4 and plot of PI vs. LL is on or above "A" line and the presence of organic matter does not influence LL.	325	0.0442	4.5	
	400	0.0372	4.75	
		0.0312	5.0	
		0.0156	6.0	
Silt if PI < 4 and plot of PI vs. LL is below "A" line and the presence of organic matter does not influence LL.		0.0078	7.0	
		0.0039	8.0	
		0.00195	9.0	
(PI = plasticity limit; LL = liquid limit)		0.00098	10.0	
		0.00049	11.0	
		0.00024	12.0	
		0.00012	13.0	
		0.000061	14.0	

1 ASTM Standard D 2487-92. This is the ASTM version of the Unified Soil Classification System. Both systems are similar [illegible]

Beach Grain Size Distribution

EM 1110-2-1100 (Part III)
30 Apr 02

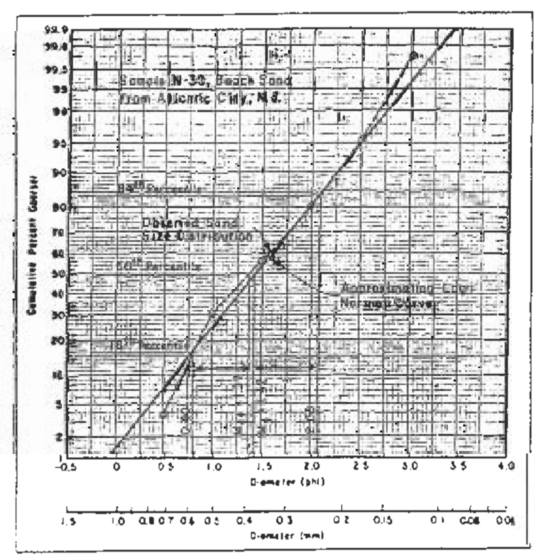
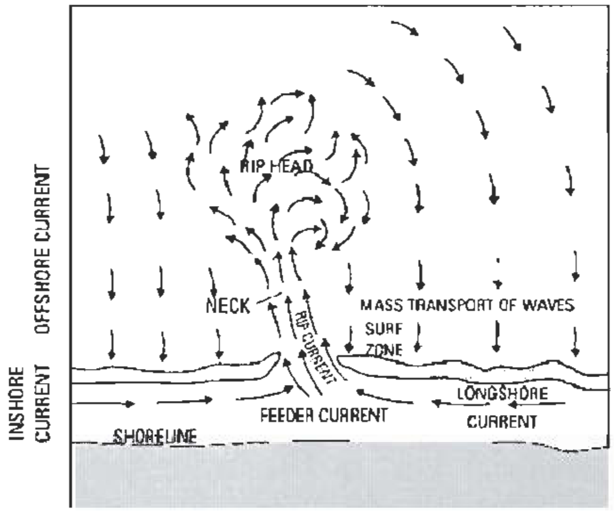


Figure III-1-2. Example of sediment distribution using log-normal paper

$\phi = -\log_2 (D \text{ mm})$.

$D = 2^{-(\phi)}$

Offshore Currents



Onshore Transport -- Beach

Evidence of Longshore Transport

Wave Setup

$$S_w = \Delta S - S_b$$

$$\Delta S \approx 0.15 d_b$$

Sw as function of beach slope m

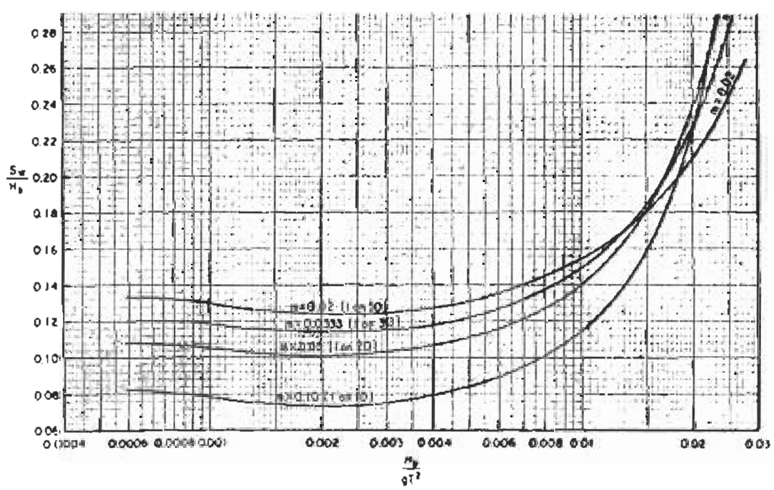


Figure 3-50. S_w/H_b versus H_b/gT^2.

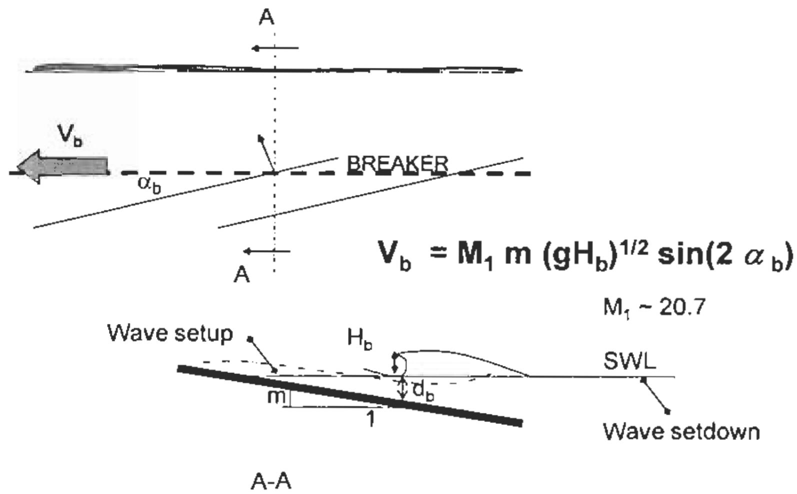
Longshore current

- Given:
  - H_b = 4 ft
  - T = 5 sec
  - α_b = 20°
  - m = 2%

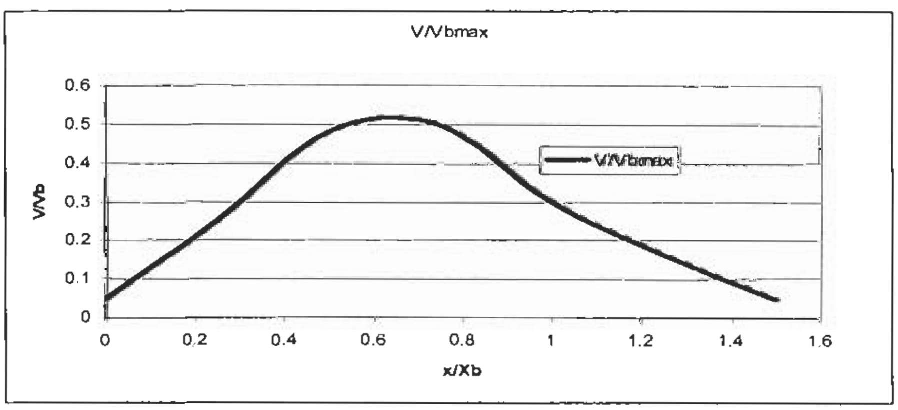
Find:
  - Vb max **= 20.7 m $(gH_b)^{1/2}$ sin(2 α_b)**
  - 3 ft/sec due to friction this could be 1.5 ft/sec.

Wave Setdown

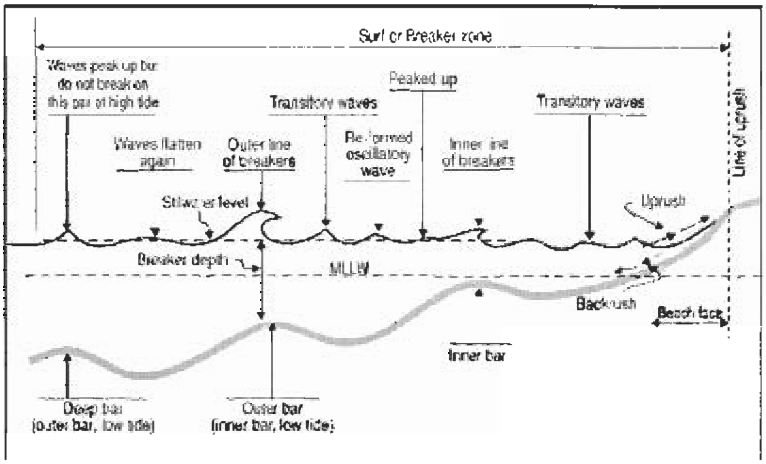
$$S_b = -\frac{\sqrt{g}H_o^{\,2}T}{64\pi d_b^{\,3/2}}$$

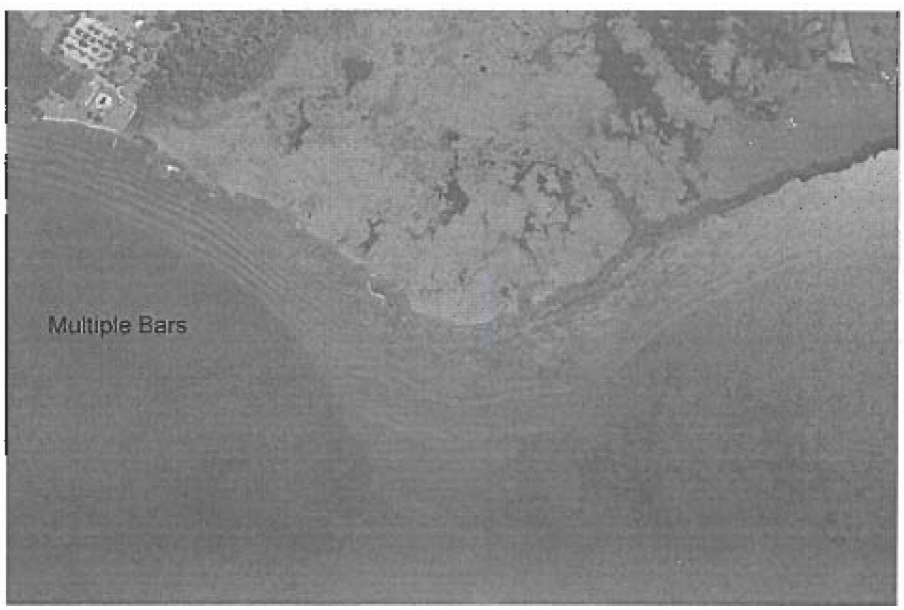


Distribution of Longshore Current

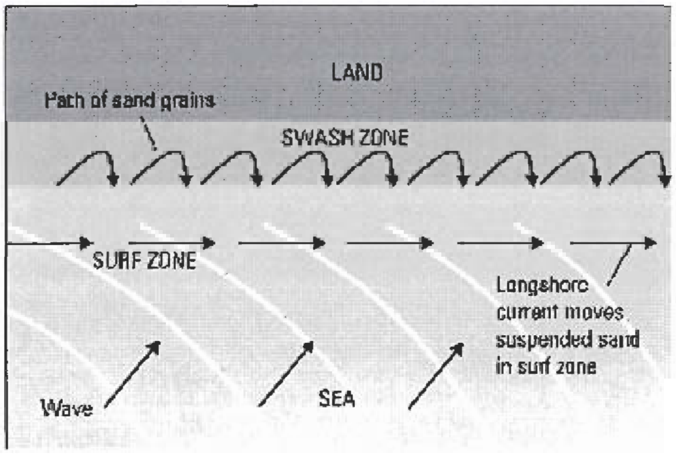


Surf Zone

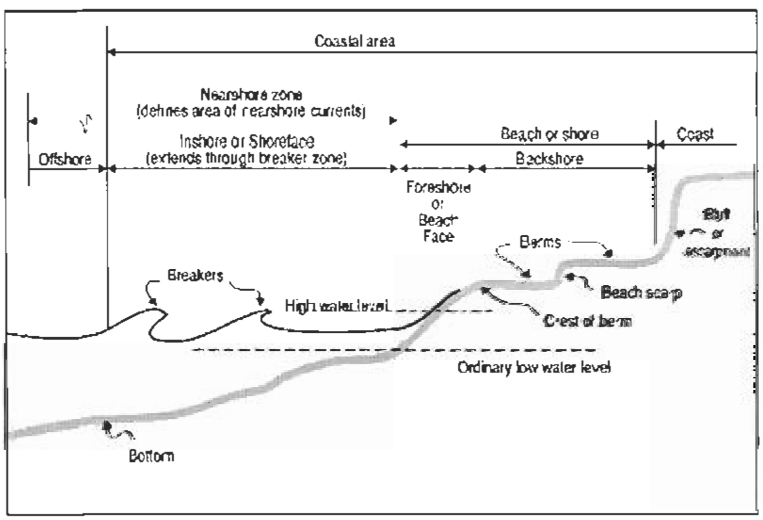

Surf or Breaker zone
Peaked up
Transitory waves
Transitory waves
Waves flatten again
Outer line of breakers
Inner line of breakers
wave
Stillwater level
Breaker depth
MLLW
Uprush
Backrush
Beach face
Line of Uprush
Inner bar
Outer bar
(inner bar, low tide)


Multiple Bars

Refraction and Swash Zone

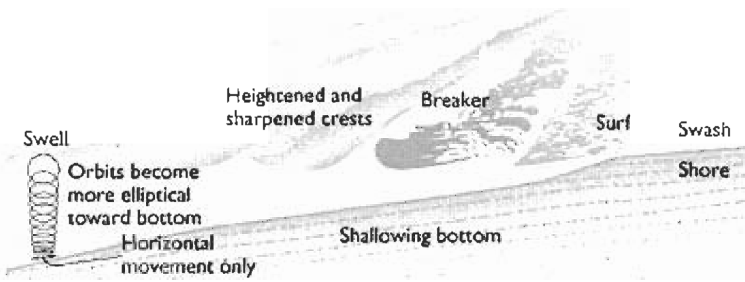


Coastal Area



Coastal Zones

Note: non-linear waves result in a 'drift' or net transport in the direction of the wave propagation.



Drift

$$U_o \approx \frac{(\pi H)^2}{TL}\left[\frac{\cosh(\frac{4\pi d}{L})}{2\sinh^2(\frac{2\pi d}{L})}\right]$$

$$U_b \approx \frac{(2\pi H)^2}{TL}\left[\frac{5}{16\sinh^2(\frac{2\pi d}{L})}\right]$$

Drift current

- Given:
  - H = 4 ft
  - d=14 ft
  - T = 5 sec

Find:

- Uo ~ 0.48 ft/sec
- Ub ~ 0.36 ft/sec

$$U_o \approx \frac{(\pi H)^2}{TL}\left[\frac{\cosh(\frac{4\pi d}{L})}{2\sinh^2(\frac{2\pi d}{L})}\right]$$

H	4	ft	
d	14	ft	
T	5	s	
Lo	128	ft	
L	94	ft	-0
Uo	0.48	ft/sec	

TERMS

http://www.csc.noaa.gov/beachnourishment/html/geo/shorelin.htm

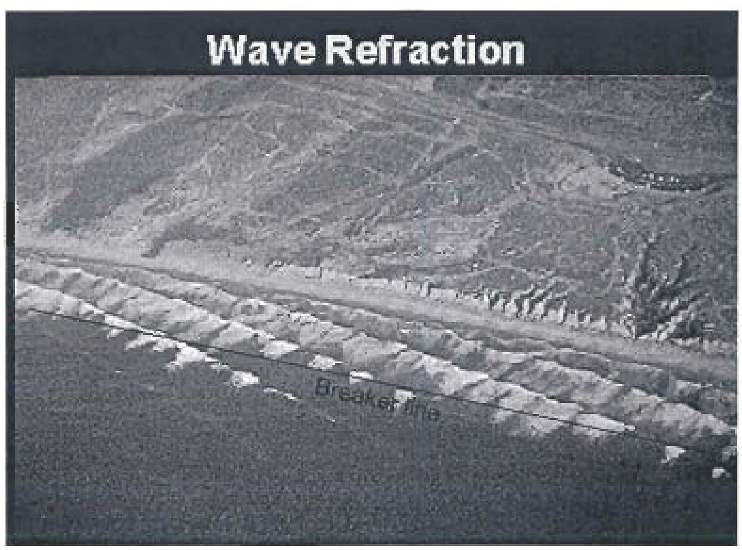
- **Littoral cell** - A reach of the coast that is isolated sedimentologically from adjacent coastal reaches and that features its own sources and sinks. Isolation is typically caused by protruding headlands, submarine canyons, inlets, and some river mouths that prevent littoral sediment from one cell from passing into the next.
- **Littoral zone** - In beach terminology, an indefinite zone extending seaward from the shoreline to just beyond the breaker zone.
- **Longshore bar** - A sand bar that extends roughly parallel to the shoreline.
- **Longshore direction** - Parallel to and near the shoreline, alongshore.
- **Longshore sand bars** - A sand ridge or ridges, running roughly parallel to the shoreline and extending along the shore outside the trough, that may be exposed at low tide or may occur below the water level in the offshore.
- **Longshore transport** - A wave- and/or tide-generated movement of shallow-water coastal sediments parallel to the shoreline.
- **Low energy environments** - Coastlines where wave and tidal forces are typically relatively small due to the climate, the location of the site and / or due to nearshore submerged features that function to reduce incoming wave energy.

Shore Processes

- Shoaling
- Refraction
- Diffraction
- Reflection
- Wind Setup
- Drift Current
- Wave Set down
- Wave Setup
- Runup
- Rush down or run down
- Longshore currents
- Longshore transport
- Surf Beat
- Edge Waves
- Shear waves
- Rip currents
- Swash

Coastal Processes

Currents and Sediment Transport



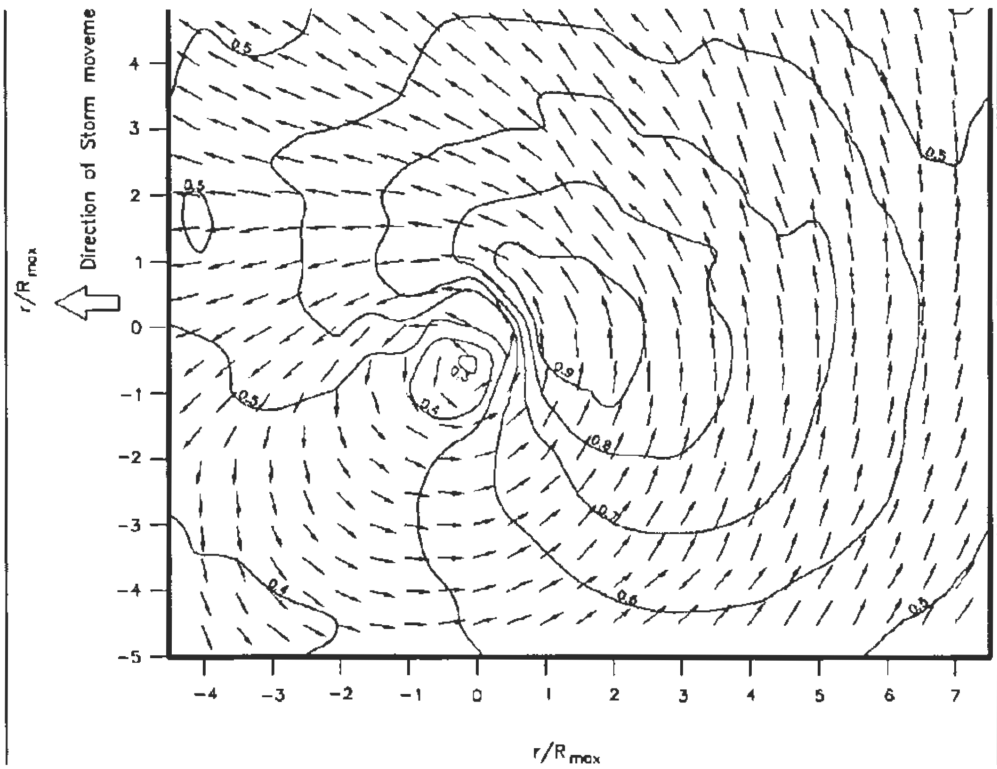


Fig 6.11 Distribution of wave heights (CEM)

The storm surge h_{ss} consists of two parts: central depression Δp and wind setup:

$$h_{ss} \approx K_s K_r \left[\Delta p + \frac{C_R C_f R U_R^2}{2 g d_{ref}} \frac{\rho_a}{\rho_w} \right]$$

$F' = KR$

$$C_R \approx 1 \to 2$$

$$\frac{\rho_a}{\rho_w} \approx 1/800$$

$$C_f \approx 0.0025$$

$$d_{ref} \sim 10 \to 20m$$

$$TimeScale \approx \sqrt{\frac{gR}{2\pi}}$$

Example: A hurricane has the following approximate characteristics:
R = 22.5 miles; Maximum Wind Speed ~ Umax; Forward Speed = 16 mph; Δp ≃ 50 mm Hg with normal pressure at 760 mm. Assume Latitude of 28°. Plot the pressure and velocity on the right side of the storm. What are the maximum significant wave height and the corresponding period? Assume: d_{ref} ~ 10 m; C_R ~ 1; KrKs ~1; estimate the surge height.

$$U_{max} = a\{b\Delta p^{1/2} - cRf\}$$
$$U_R = 0.865U_{max} + 0.5V_F$$

$$f = 2\omega_e \sin\phi = Coriolis(rad / h)$$

$\alpha \sim 1$ to 1.2

where a = 0.868 (0.447); b=73 (14.5) and c= 0.57 (0.31) for US (SI) units

Note: Normal atmospheric pressure is 29.92 inches Hg = 760 mm Hg.
The earth's angular velocity is $2\pi/24$ rad/h;
1 nautical miles = 1852 m = 6076 ft

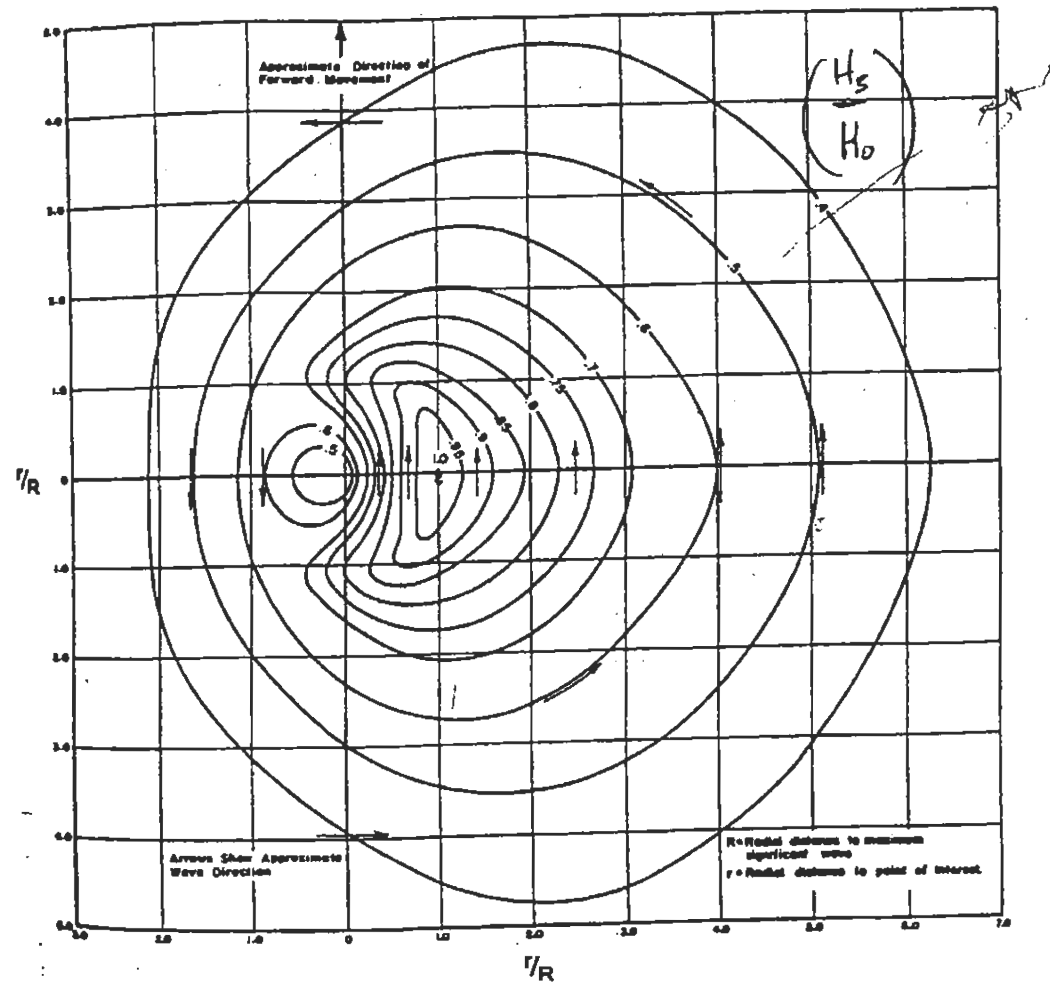


Figure 3-43. Isolines of relative significant wave height for slow-moving hurricane.

Fig 6.10 Distribution of wave heights (SPM)

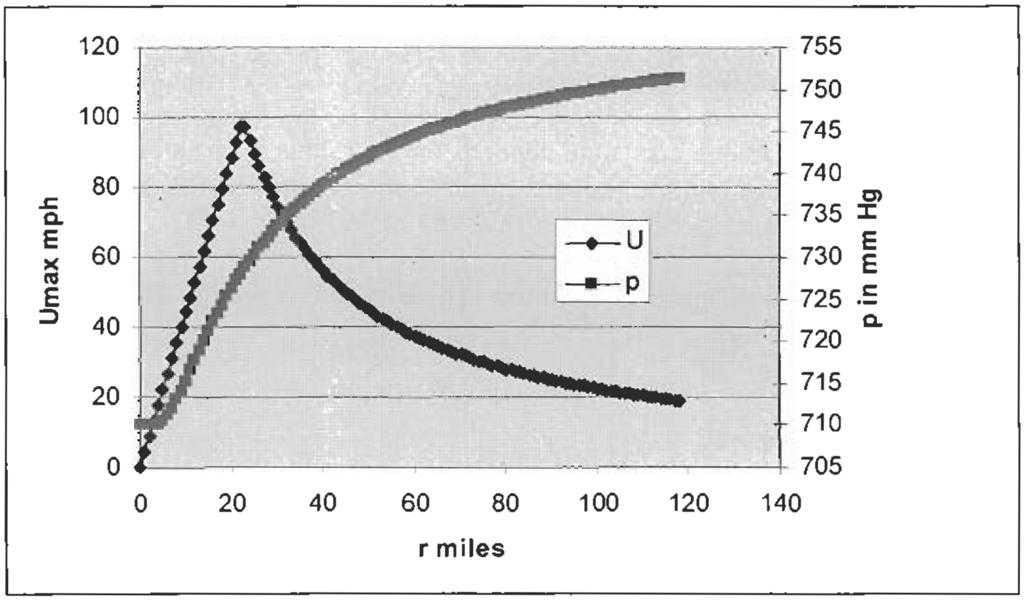


Figure 6.9: Pressure and Velocity Distribution in a Hurricane (R=22.5 miles; Δp= 50 mm)

Hurricane Waves

The SPM gives the following equations for the significant deep water wave height H_{os} and period T_s at the location of the maximum wind speed:

$$T_s = A\left[1 + \frac{B\alpha V_F}{\sqrt{U_R}}\right]e^{\frac{R\Delta p}{C}} \qquad 6.20$$

in which A = 8.6 for US and SI units; B= 0.104 for US and 0.145 for SI units.
and C =200 for US and 9400 for SI units.
Also the deep water significant wave height is:

$$H_{os} = A_o\left[1 + \frac{B_o\alpha V_F}{\sqrt{U_R}}\right]e^{\frac{R\Delta p}{C_o}} \qquad 6.21$$

in which A_o = 16.5 (5.02); B_o= 0.208 (0.29) and C_o =100 (4700) for US (SI) units.
The spatial distribution of the significant wave heights are shown in Figures 10 and 11.

Inputs:
R = radius of maximum wind (***Nautical Miles*** or ***km***)
Δp= central pressure depression (in ***inches*** of ***mm*** of Hg)
V_F = forward speed in ***knots*** or ***km/h***

max wind gradient $\rightarrow U_{max} = a\{b\,\Delta p^{1/2} - cRf\}$ $f = 2\omega_e \sin\phi$ = coriolis $\frac{rad}{h}$

$U_R = 0.865\,U_{max} + 0.5\,V_f$ (max sustained wind @ 10m)

Lecture 6 (Continued)

Hurricane Generated Waves and Storm Surges

The wave climate associated with a hurricane depends on the radius of the storm, its sustained maximum wind speed and the forward speed to the storm. The SPM provides a typical distribution of wave heights with the storm (1984 SPM Figure 3-43).

The simplest model for the wind speed distribution in a hurricane is the Rankin Vortex,

$$U = K\,r \qquad \text{for} \quad r < R \tag{6.16a}$$

and

$$U = K\,R^2/r \qquad \text{for} \quad r > R \tag{6.16b}$$

where K = constant = U_{max}/R; R = radius at the maximum speed; r is the radius to a point from the storm centre. This simple model gives a idea of how winds vary as we go away from the center but it omits many effects such as the boundary layer, the Coriolis effect, the forward speed of the storm and the radial flow at the base.

The sea surface pressure is fairly well described by the Myers equation,

$$p = p_o + \Delta p\, e^{-\{R/r\}} \tag{6.17}$$

where p_o = central pressure and $\Delta p = p - p_o$

Figure 9 shows a typical velocity and pressure variation as a function of the distance from the center of the hurricane.

The associated Gradient Wind Speed, U_{gr} for a nearly stationary storm is

$$U_{gr}^2/r + f\,U_{gr} \simeq \{R\,\Delta p/(\rho_a r^2)\}\,e^{-\{R/r\}} \tag{6.18}$$

where f = Coriolis parameter = $2\omega \sin(\phi)$; ρ_a =air density.

The vector to be added to U_{gr} to correct for the forward speed V_f is given by

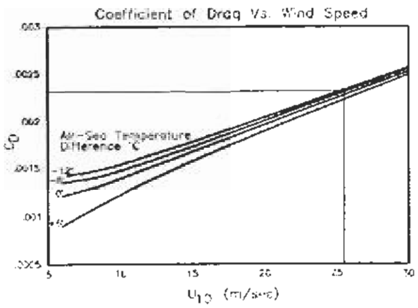
$$\underline{\mathbf{U}_{SM}} = \underline{\mathbf{V}}_f R\, r/(R^2 + r^2) \tag{6.19}$$

Note: Resultant velocity is found from the vector sum of

$$\underline{\mathbf{U}_T} = \underline{\mathbf{U}_{SM}} + \underline{\mathbf{U}_{gr}} \tag{6.20}$$

Thus in the northern hemisphere the speeds on the right of the center are higher than on the left.

Wind Drag



Wind Shear

$$F'\tau_a = \gamma_w d\Delta z$$

$$\tau_a = \rho_a C_f U_{OW}^2$$

$$F'(\rho_a C_f U_{OW}^2) = \gamma_w d(2Z_{\sup}) = g\rho_w d(2Z_{\sup})$$

$$F' = kR$$

$$Uow \approx U_R$$

$$Z_{wind} = \frac{C_R R(\rho_a C_f U_R^2)}{2g\rho_w d_{ref}}$$

Surge Estimation

h_{ss} ~ KrKs (Surge due to Δp + Wind Shear Effect)

$$h_{ss} \approx K_s K_r \left[\Delta p + \frac{C_R C_f R U_R^2}{2g d_{ref}} \frac{\rho_a}{\rho_w} \right]$$

$$C_R \approx 1 \rightarrow 2$$

$$\frac{\rho_a}{\rho_w} \approx 1/800$$

$$C_f \approx 0.0025$$

$$d_{ref} \sim 10 \rightarrow 20m$$

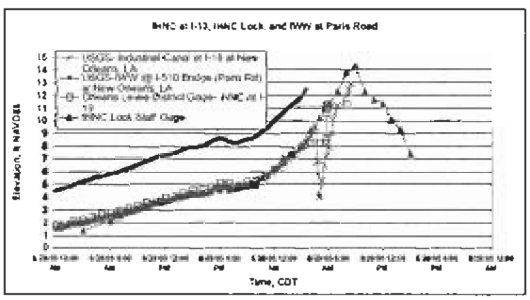
Example:

A Hurricane has the following approximate characteristics:
R = 22.5 miles;
Maximum Wind Speed ~ Umax;
Forward Speed = 16 mph;
Δp ~ 50 mm Hg with normal pressure at 760 mm.
Assume Latitude of 28°.
Plot the pressure and velocity on the right side of the storm.
What are the maximum significant wave height and the corresponding period?
Assume: d_{ref} ~ 10 m;
CR ~ 1;
Use $U=U_R$
KrKs ~1;
Estimate the surge height.

Near Lake Pontchartrain

UNO 2 weeks later…

STORM SURGE EAST N.O.



STORM SURGE

hss ~ KrKs (Surge due to Δp + Wind Shear Effect)

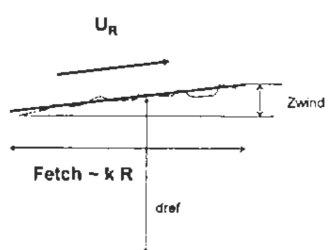
Depression Surge

$h_{\delta p}$ ~ (10 m) (Δp mm)/760 in m

Or

$h_{\delta p}$ ~ (34m) (Δp mm)/760 in ft

Setup



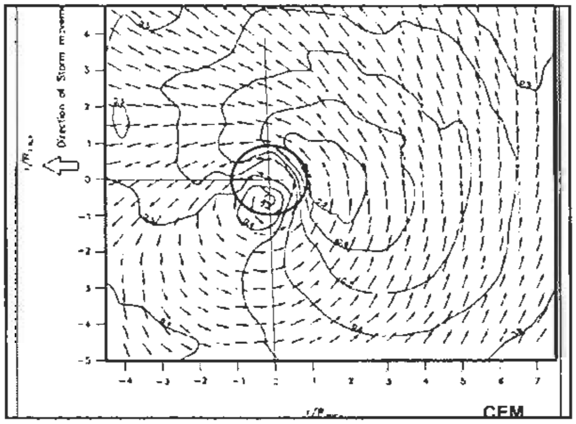
LOWER 9th Breach

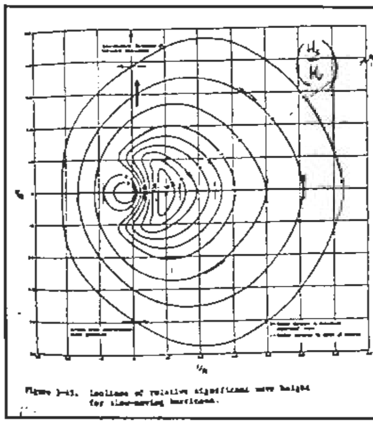
Levee Breach at Bienvenue Flood Gate

Scour Hole at BV Control Structure

Barge on BV Control Structure

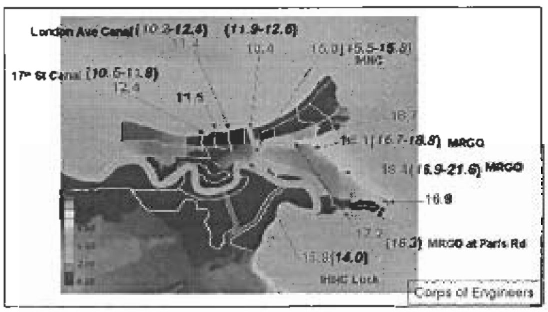
100 year old West End Lighthouse





SPM

Storm Surges

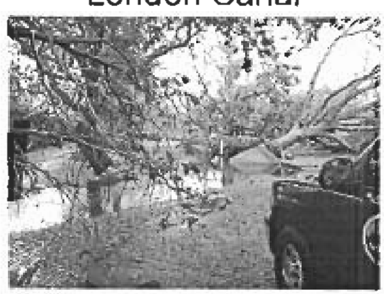


Overtopping of Levee at I-510 Bridge

Breach in London Canal

NOLA website

Residential Area near Breach in London Canal



Rankin Vortex,

$U = K r$ for $r < R$

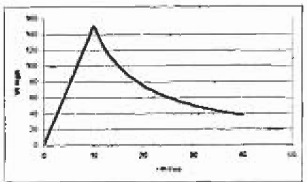
and $U = C/r$ for $r > R$

where K = constant = Umax/R

C= Umax*R;

R = radius at the maximum speed;

r is the radius to a point from the storm centre.



Pressure in a Hurricane

The sea surface pressure by the Myers equation,

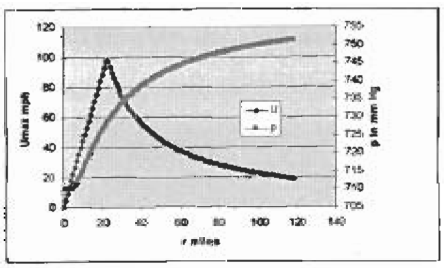
$$p = p_o + \Delta p\, e^{-R/r}$$

where p_o = central pressure

$\Delta p = p_n - p_o$

p_n = Atmospheric pressure

Velocity and Pressure



Hurricane (R=22.5 miles; Δp= 50 mm)

Waves

H_{os} = Reference Significant Wave Height

T_s = Reference Significant Wave Period

$$H_{os} = A_o\left[1 + \frac{B_o \alpha V_F}{\sqrt{U_R}}\right] e^{\frac{R\Delta p}{C_o}}$$

Ao = 16.5 (5.02);
Bo =0.208 (0.29)
Co= 100 (4700)
$\alpha \sim 1 - 1.2$
for US (SI) units

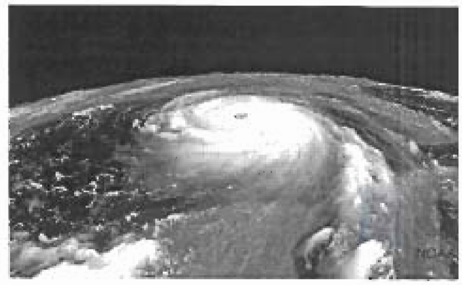
$$T_s = A\left[1 + \frac{B \alpha V_F}{\sqrt{U_R}}\right] e^{\frac{R\Delta p}{C}}$$

A = 8.6 (8.6);
B = 0.104 (0.145) and
C= 200 (9400)
$\alpha \sim 1 - 1.2$
for US (SI) units

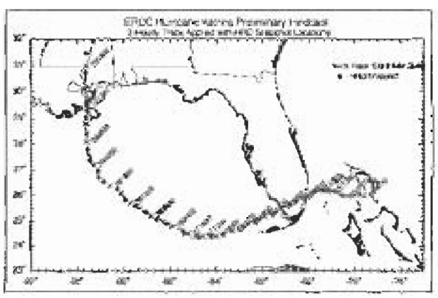
Wave Generation

Shore Protection Manual
Method
Hurricane Waves

Katrina Aug 28 05

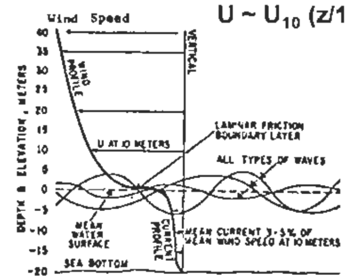


STORM TRACK



Wind Speed and Water Current

$U \sim U_{10}\,(z/10)^{1/7}$



U_{max} & U_R

U_R is maximum sustained wind at 10 m

Umax is maximum gradient wind

V_F = forward speed of the hurricane

Estimation of U_{max} & U_R

$$U_{max} = a\{b\Delta p^{1/2} - cRf\}$$

$$U_R = 0.865U_{max} + 0.5V_F$$

$$f = 2\omega_e \sin\phi = Coriolis(rad/h)$$

a = 0.868 ; b=73 and c= 0.57 for US units
Knots, Nmiles; Inches Hg;

a =0.447; b=14.5 and c= 0.31 for SI units
kmph, km; mm Hg;

U_R is maximum sustained wind at 10 m
U_{max} is maximum gradient wind

Assignment 6.1: Estimate the south shore wave height and period for TS Isidore. Assume NNE winds at 35 knots with 2 hour duration. Estimate the wind setup for this storm.

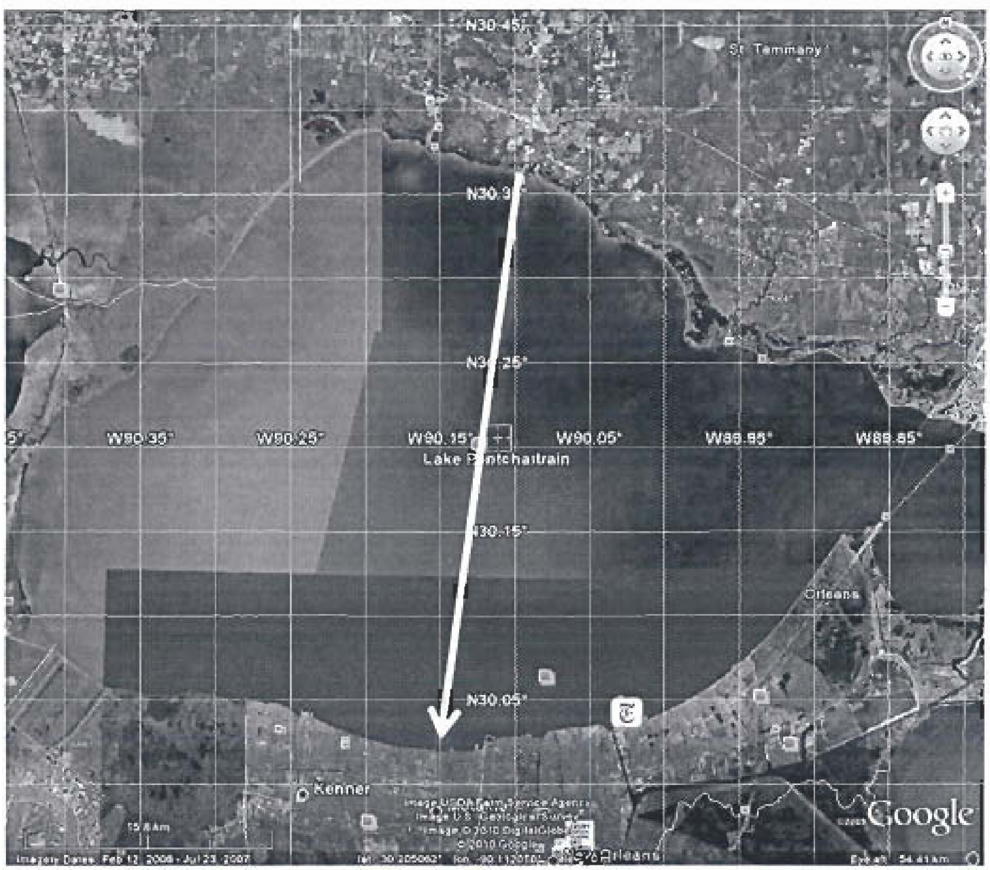
Hs = ____________

Ts = ____________

Are the waves: (FETCH LIMITED) or (DURATION LIMITED) ?

Wind setup = __________________ Using Equation 6.14

Wind setup = __________________ Using Equation 6.15



An empirical formula for $\mathbf{Z_{sup}}$ is,

$$z_{sup}\,(ft) = F(miles)\;V(mph)^2\,/[1400\;d\;(ft)] \qquad 6.15$$

Class Example Problem:
Determine the significant wave period and height and the setup near the south shore of Lake Pontchartrain for a 2- hour wind out of the north over the Lake.
Assume:

- a 25 mile fetch with an average depth of 3.8 m before the storm surge,
- a storm surge of 2 m,
- a local depth of 3 m without the storm surge or setup,
- a water temperature of 80°F and air temperature of 70°F.
- the over-water velocity is 80 mph.

Setup:- increase in water depth (downwind) due to wind shear. The form of this equation can be demonstrated by the following analysis. Assume a unit wide strip with a length equal to the effective fetch, F'; the wind shear force is $\tau_a F'$ = the hydrostatic resistance ~ $\gamma d\Delta z$

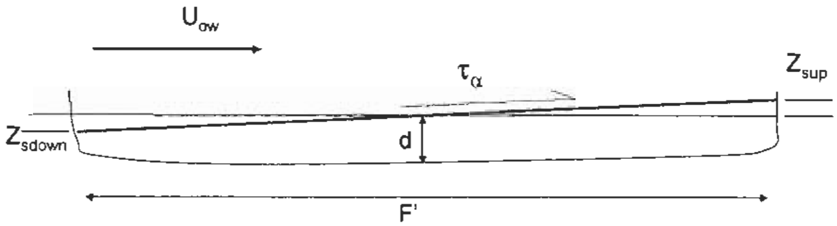


Fig 6.7 Wind Shear and Setup Diagram

$\mathbf{\Delta z = Z_{sup} + Z_{sdown}}$

$$F'\tau_a = \gamma_w d\Delta z$$

$$\tau_a = \rho_a C_f U_{OW}^2$$

$$F'(\rho_a C_f U_{OW}^2) = \gamma_w d(2Z_{sup}) = g\rho_w d(2Z_{sup}) \qquad 6.14$$

$$Z_{sup} = \frac{F'(\rho_a C_f U_{OW}^2)}{2g\rho_w d}$$

$$C_f \sim 0.001 \rightarrow 0.003$$

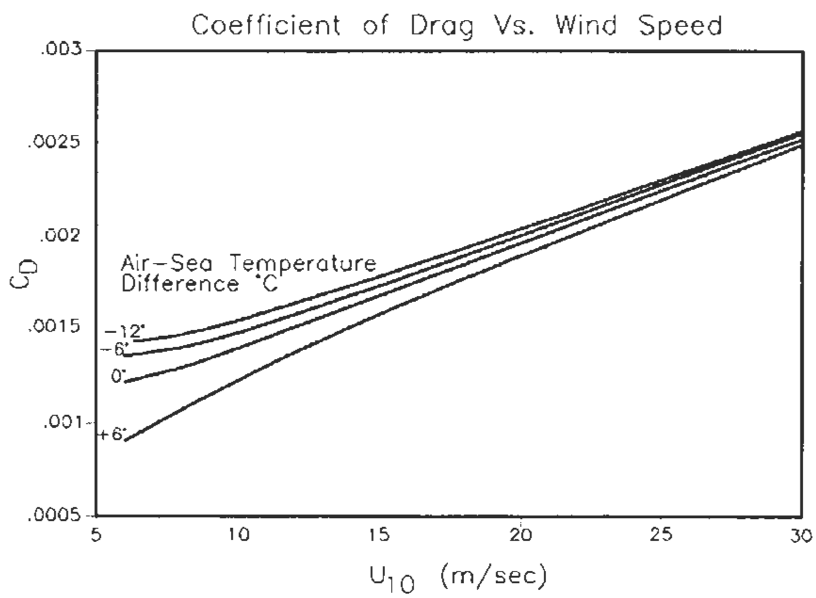


Figure 6.8 Wind Friction Coefficient (C_f)

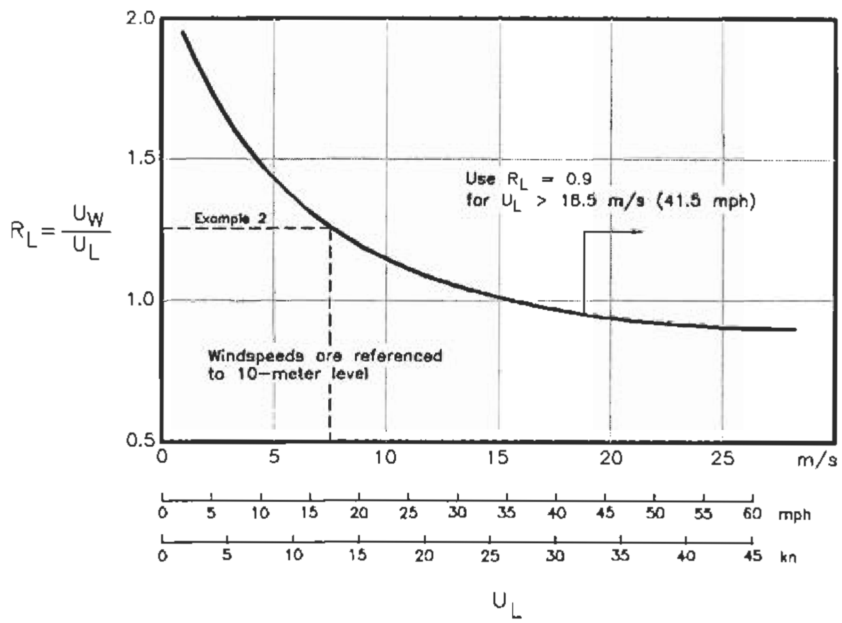
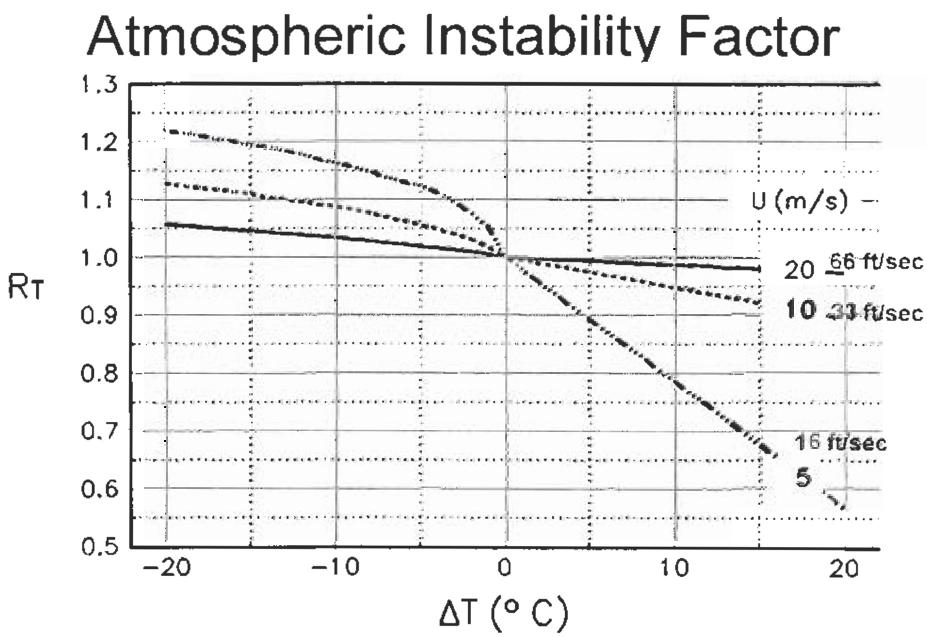


Fig 6.5 R_L = Uow:UoL Ratio (CEM and SPM)



Δt = a - s

Fig 6.6 R_T (CEM)

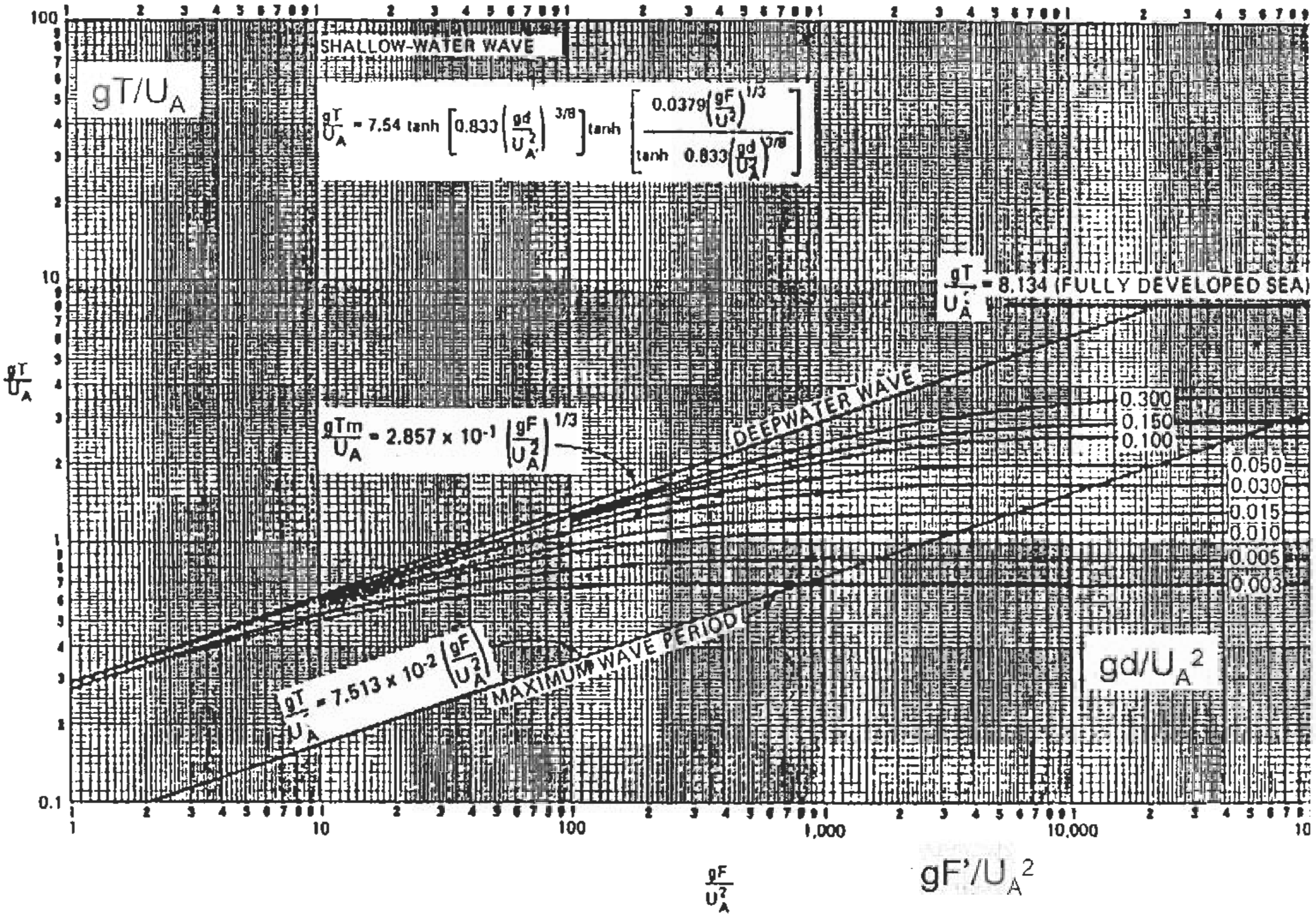


Figure 3-22. Forecasting curves for wave period. Constant water depth.

atmospheric stability (R_T , Fig 3-14 SPM, also Figure 6.6) and for the effective wind shear; this is given as:

$U_A = 0.589\ (U_{oLmph}\ R_L\ R_T)^{1.23}$ in mph or $U_A = 0.539\ (U_{oLfps}\ R_L\ R_T)^{1.23}$ in ft/sec

$U_A = 0.77\ (U_{oL}\ R_L\ R_T)^{1.23}$ in m/s 6.13

where $U_{oL}\ R_{wL}$ = over water velocity at 10 m above surface.

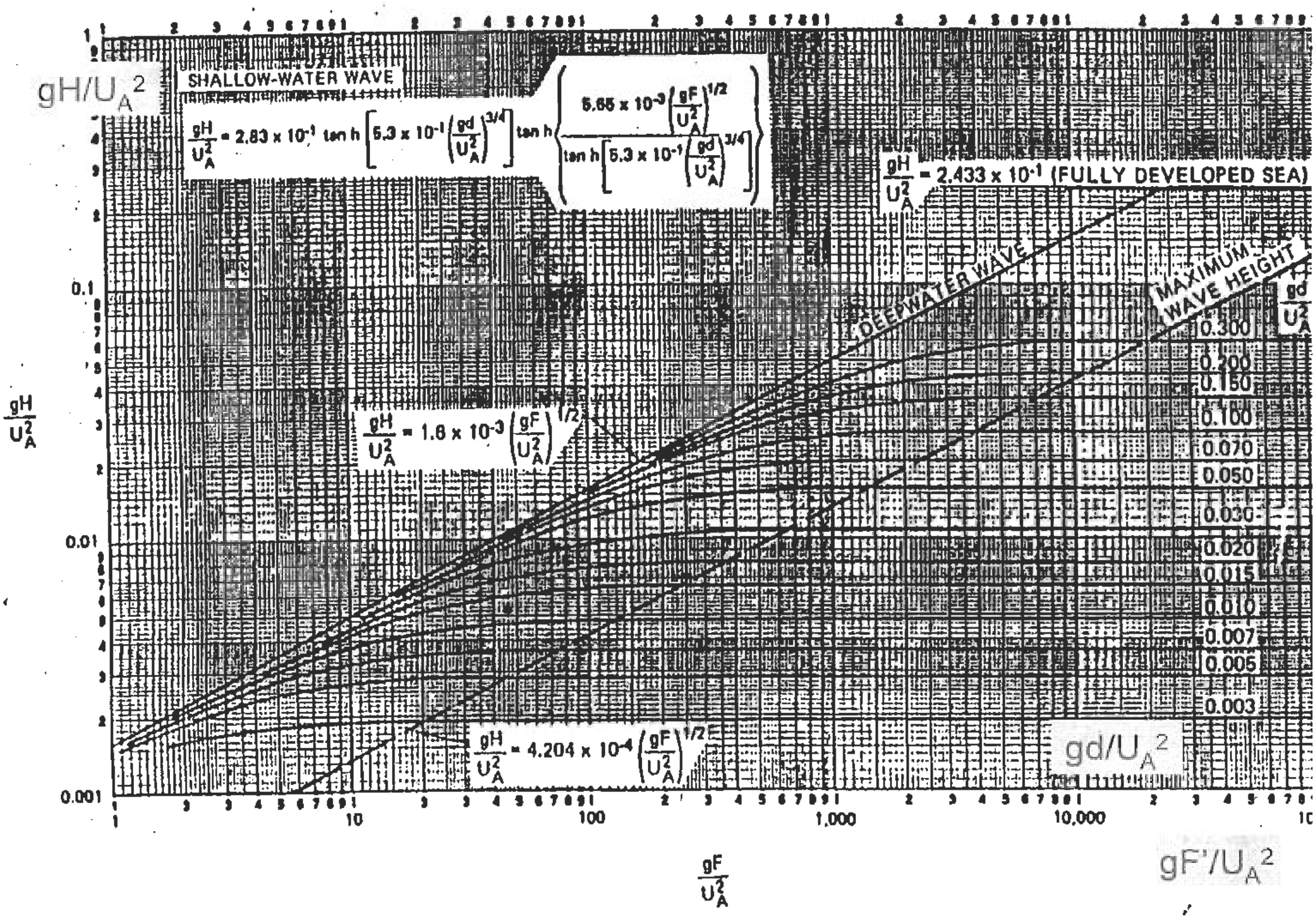


Figure 3-21. Forecasting curves for wave height. Constant water depth.

period T_m. Often it is assumed that the dominant wave height is approximately the average of the highest 1/3 of all the waves; the SPM uses the spectral significant wave height, H_s with the T_m, being the significant wave period for the maximum spectral energy. As shown in Figure 6.2, waves have been found to fit the Rayleigh Distribution:

$$P(H > \hat{H}) = e^{-\left(\frac{H}{H_{rms}}\right)^2} \quad 6.4$$

where P is the probability of exceeding and certain x-value.

Based on a Rayleigh frequency distribution the relationship of H_s to some other wave height is

$$H_s = 2 \cdot H_{rms} \quad 6.5a$$

$$H_{10} = 1.27 H_s \quad 6.5b$$

$$H_1 = 1.67 H_s \quad 6.5c$$

The subscripts 1 and 10 refer to the average the highest 1 % and 10% of the waves in a storm. The SPM provides a comprehensive wave prediction procedure for H_s and T_m based on dimensionless charts as shown in Figures 3-21 and 3-22. The charts show the functions that are used; these are in the forms:

$$g H_s / U_A^2 = \text{fcn}\,(F_d, F_F) \quad 6.6$$

and $$gT_s / U_A = \text{Fcn}\,(F_d, F_F) \quad 6.7$$

where F_d = *depth number* = $g\,d / U_A^2$ **6.8**

F_F = *fetch number* = $g\,F / U_A^2$ **6.9**

F = *effective fetch* = *minimum* $(F_a\ ,\ F_{d = \text{duration limited}})$ **6.10**

The relation between t_d ***and*** F for deep water conditions is

$$g\,t_d / U_A = 68.8\,(gF/U_A^2)^{2/3} \quad 6.11$$

and for shallow water we have,

$$g\,t_d / U_A = 537\,(gT_m/U_A)^{7/3} < 1.28\,(gF/U_A^2)^{7/3} \quad 6.12$$

Note: Figures 3-21 and 3-22 (SPM) show the prediction equations that are used for Eq 6.6 and 6.7.

The wind speed that is used is the corrected for water:land effects (R_L , Fig. 6.5), Wave period spectrum

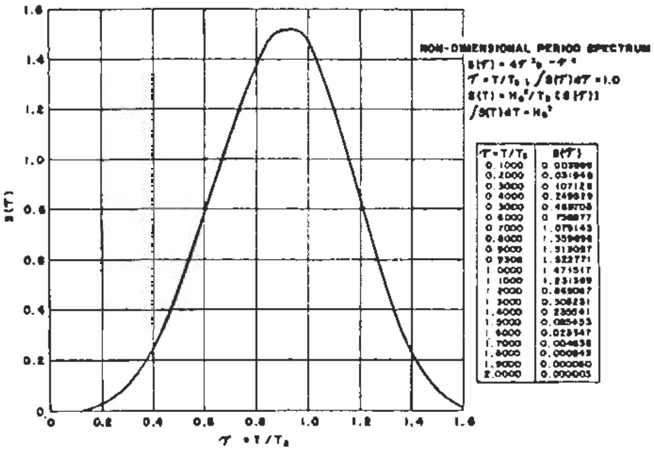


Fig. 2 Non-dimensional period spectrum

Fig 6.1 Period Spectrum

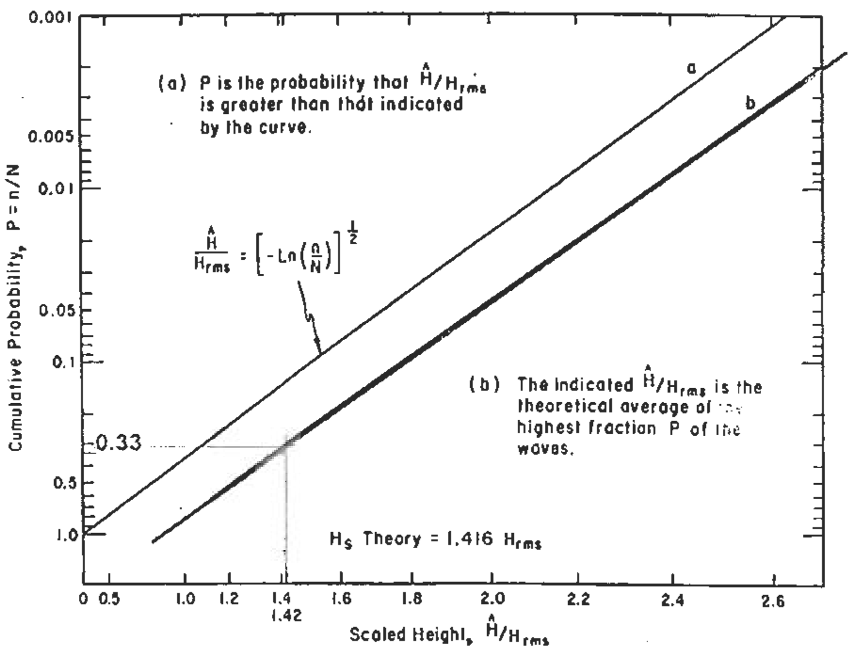


Figure 3-5. Theoretical wave height distributions.

Fig 6.2 Rayleigh Distribution for a) all waves and b) averaged higher than waves.

Wave generated by a wind depend on a number of factors, such as: effective wind speed V_a, wind direction, available fetch (distance over which wind blows) F, water depth d, atmospheric stability R_T, and wind duration t_d. A given wind condition generates a spectrum of waves, with the dominant wave energy at a particular wave height H_s and

Lecture 6
Wave Prediction

Wave Statistics

An irregular train of waves is illustrated in Figure 6.1. The waves in this train have various heights and periods. One way of describing this set of random waves is the Root Mean Square of the heights of all the waves, i.e.

$$H_{rms} = \{\Sigma H_i^2/n\}^{1/2} \qquad 6.1$$

Each wave is separated by successive zero crossings. The number of waves in the train is n. We can also conduct a probability analysis on the wave heights, i.e. we can plot the probability, P of exceeding a certain wave height H, e.g.

$$P = m/(1 + n) \qquad 6.2$$

where m = the rank of a wave in descending order. Weibull and Rayleigh distributions have been used to fit the wave distributions of storm waves. Figure 3-3 (SPM 73) shows a normalized Rayleigh distribution.

The wave periods also can be considered by a probability distribution. Alternately a wave spectrum analysis can be preformed. The wave spectrum is the plot of wave energy versus the wave frequency (f = 1/T). A Fourier Analysis can be completed to construct the wave spectrum. Commercial software packages such as Statistica can be used if the wave form is digitized. Figure 3-6 (SPM 73) shows a typical wave spectrum. A significant wave can be defined by the dominant energy in the wave train. Theoretical spectra are available for waves in a few coastal areas. For example the SPM gives the Pierson-Moskowitz Spectra as,

$$E(\omega)d\omega = \frac{\alpha g^2}{\omega^5} e^{-\beta(\omega_o/\omega)^4}$$
$$\alpha = 0.0081$$
$$\beta = 0.74 \qquad 6.3$$
$$\omega = angular\_frequency = 2\pi f = 2\pi / T$$
$$\omega_o = g / U_{OW}$$
$$U_{OW} = overwater.windspeed,(ship.report)$$

There are also spectra that involve frequency and direction.

Assignment 6.1: Estimate the south shore wave height and period for TS Isidore. Assume NNE winds at 35 knots with 2 hour duration. Estimate the wind setup for this storm.

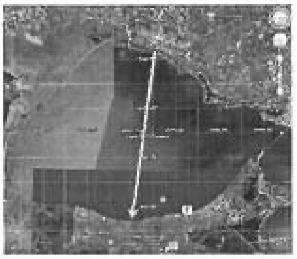
Hs = ____________

Ts = ____________

Are the waves: (FETCH LIMITED) or (DURATION LIMITED) ?

Wind setup = ________________ Using Equation 6.14

Wind setup = ________________ Using Equation 6.15



Shoaling and Refraction Animation.

http://daphne.palomar.edu/lyon/Animations/WaveMotion.swf

(9) Kitaigorodskii (1962) extended the similarity arguments of Phillips to distinct regions throughout the entire spectrum where different mechanisms might be of dominant importance. Pierson and Moskowitz (1964) followed the dimensional arguments of Phillips and supplemented these arguments with relationships derived from measurements at sea. They extended the form of Phillips spectrum to the classical Pierson-Moskowitz spectrum

$$E(f) = \frac{\alpha g^2 f^{-5}}{(2\pi)^4} \exp\left[-0.74\left(\frac{f}{f_u}\right)^{-4}\right] \quad \text{(II-2-32)}$$

where

f_u = limiting frequency for a fully developed wave spectrum (assumed to be a function only of wind speed)

(12) Hasselmann et al. (1973) collected an extensive data set in the Joint North Sea Wave Project (JONSWAP). Careful analysis of these data confirmed the earlier findings of Mitsuyasu and revealed a clear relationship between Phillips' α and nondimensional fetch (Figure II-2-21). This finding and certain other spectral phenomena, such as the tendency of wave spectra to be more peaked than the Pierson-Moskowitz spectrum during active generation, could not be explained in terms of "first-generation" concepts; however, they could be explained in terms of a nonlinear interaction among wave components. This pointed out the necessity of incorporating wave-wave interactions into wave prediction models and led to the development of second-generation (2G) wave models. The modified spectral shape which came out of the JONSWAP experiment has come to bear the name of that experiment; hence we now have the JONSWAP spectrum which can be written as

$$E(f) = \frac{\alpha g^2}{(2\pi)^4 f^5} \exp\left[-1.25\left(\frac{f}{f_p}\right)^{-4}\right] \gamma^{\exp\left[-\frac{\left(\frac{f}{f_p}-1\right)^2}{2\sigma^2}\right]} \quad \text{(II-2-34)}$$

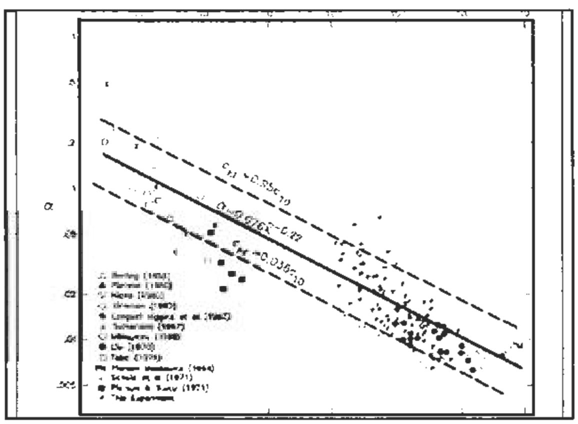
where

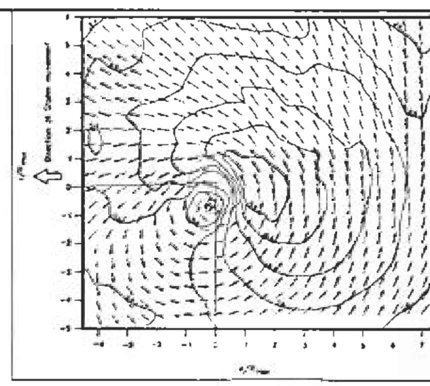
α = equilibrium coefficient

σ = dimensionless spectral width parameter, with value σ_a for $f \le f_p$ and value σ_b for $f > f_p$

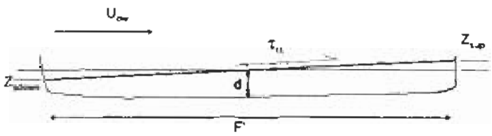
γ = peakedness parameter

The average values of the σ and γ parameters in the JONSWAP data set were found to be $\gamma = 3.3$, $\sigma_a = 0.07$, and $\sigma_b = 0.09$. Figure II-2-22 compares this spectrum to the Pierson-Moskowitz spectrum.





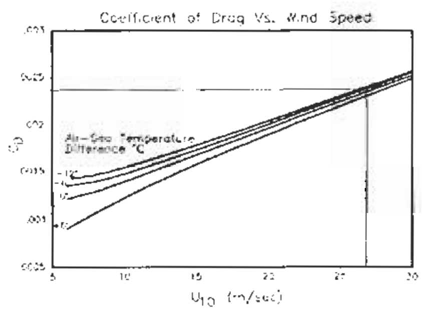
Setup



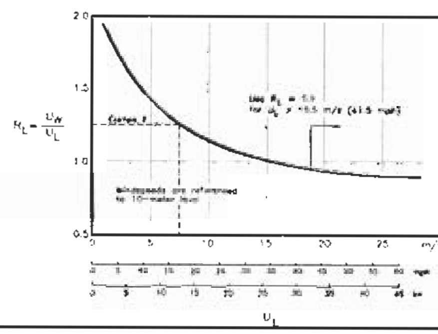
Setup

$$F'\tau_a = \gamma_w d\Delta z$$
$$\tau_a = \rho_a C_f U_{OW}^2$$
$$F'(\rho_a C_f U_{OW}^2) = \gamma_w d(2Z_{sup}) = g\rho_w d(2Z_{sup})$$
$$Z_{sup} = \frac{F'(\rho_a C_f U_{OW}^2)}{2g\rho_w d}$$
$$C_f \sim 0.0015 - -0.003$$

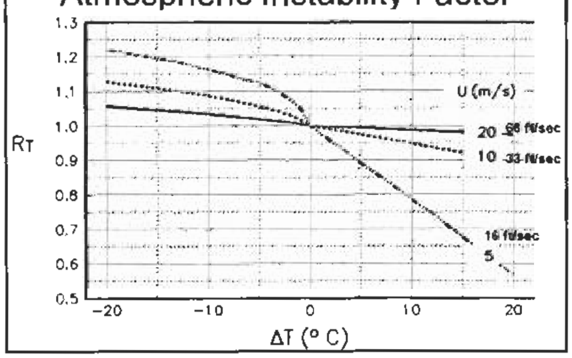
Wind Drag



Uow/UoL



Atmospheric Instability Factor



Wind Setup

d	14	ft
Uow	89.76	ft/sec
F	132000	ft
Cf	0.0024	
pw/pa	800	62.4
Zsup	3.54	ft
Zsup	4.78	ft

RL	1				
RT	1.02				
U'	61.2	mph	89.8	ft/sec	
UA	92.9	mph			
UA	136.1	ft/sec			
Fd	0.024				
FF	229				
Ftd	382				
Fig 3.21	gHs/UA2	0.0095	Hs	5.47	ft
Fig 3.22	gTs/UA	1.22	Ts	5.16	sec

Check Duration Part 2

- Assume Shallow if Fig 3.22 indicates
- Cal $(gT_m/U_A) = (Ftd/537)^{3/7}$

If $(Ftd/537)^{3/7} < (gT_s/U_A)$ from part 1 then,

- Enter Fig 3.22 with (gT_m/U_A) and intersect Fd to get FF'
- Use Fig 3.21 with FF' and Fd to Read $FHs' = gH_s/U_A^2$
- $H_s = U_A^2 * FHs'/g$
- $T_s = T_m = U_A * (Ftd/537)^{3/7}/g$
- $F' = U_A^2 FF'/g$

ELSE USE Hs and Hs from Part 1

Assume Shallow						
gTm/UA		1.22	-	1.22	Therefore use fetch limited for shallow water	
Ts=Tm		5.1507	sec			
Fig 3.22	FF'	229				
Fig 3/21	gHs/UA2	0.0095	Hs	5.47	ft	

DEEP

gF'/UA2~43.6<<229 deep

Fig 3.21	gHs/UA2	[illegible]	Hs	[illegible]	ft
Fig 3.22	gTs/UA	[illegible]	Ts	[illegible]	sec

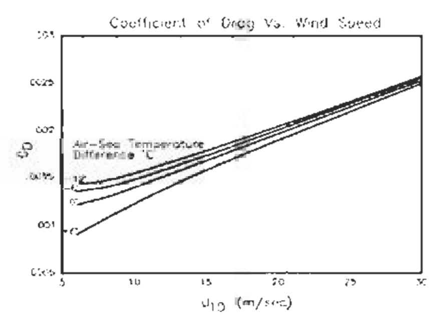
d/L ~ 0.14 < 0.5 not deep

UNO WAVE PROGRAM

waves4723-08F.exe

waves4723-08F.exe

Wind Drag



Wave Generation

- Water Temperature T_{sw} = 33 ° C
- T_a = 28 ° C
- U_{ow} = 60 mph
- d = 14 ft
- F_a = actual fetch = 25 miles
- t_d = 1 hour
- Find H_s and T_s

d	14			
Fa	25	mi	132000	ft
Tws	33	oC		
Ta	28	oC		
ΔT	-5	oC		
td	1	hour	3600	sec
UoL				
Uow	60	mph	88	ft/sec

Wave Calculations Part 1.

- Enter Fig 3.21 FF and intersect Fd
- Read $FHs = gH_s/U_A^2$
- $H_s = U_A^2 * FHs/g$
- Enter Fig 3.22 FF and intersect Fd to Read $FTs = gH_s/U_A$
- $T_s = U_A * FTs/g$

Solution

- $\Delta T = T_a - T_{ws} = -5$ °C
- R_L = 1 Fig 3.14
- R_T = 1.02 Fig 3.15
- $U' = R_L * R_T * UoL = 1.02*60$
- $U_A = 0.589(U'mph)^{1.23} =$
- U_A ft/sec = U_A mph*5280/3600 =
- $Fd = g\, d/U_A^2 =$
- $FF = g\, F/U_A^2 =$
- $Ftd = gt_d/U_A =$

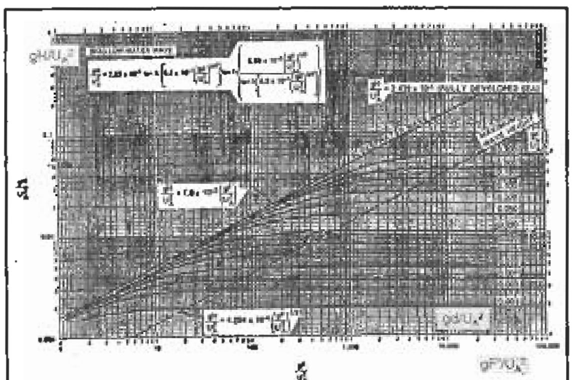


Figure 3-21. Forecasting curves for wave height. Constant water depth.

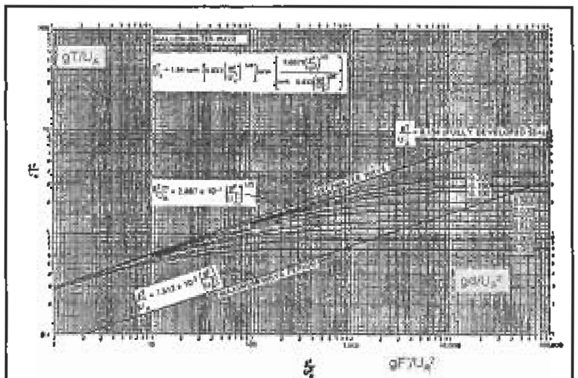
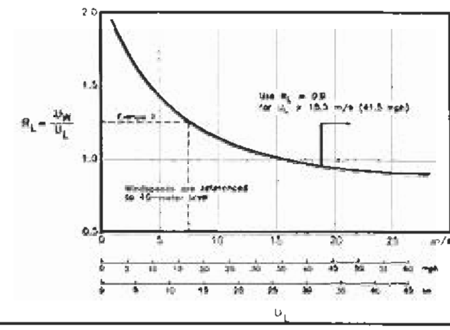


Figure 3-22. Forecasting curves for wave period. Constant water depth.

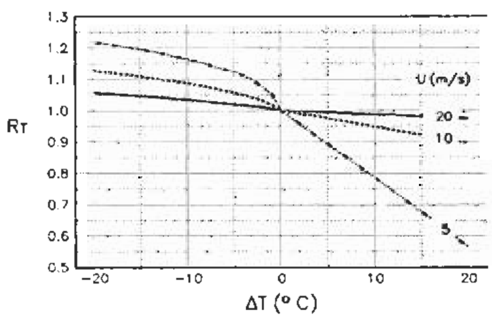
Shear Corrected Wind U_A

- $U_A = C_w\{U_{oL} R_L\ R_{T},\}^{1.23}$
  - : $R_L = U_{ow}/U_{oL}$ = over water correction Fig 3.14
  - : R_T= atmospheric stability correction Figure 3.15
  - : C_w= 0.71 for SI
  - : C_w = 0.589 for U in mph
  - : C_w = 0.539 for U in ft/sec

Uow/UoL



Atmospheric Instability Factor



Fetch and Duration

- F' = Effective Fetch which can be limited by physical topography or wind duration or structure of the wind field (e.g. hurricanes)
- F' = Min(F_a, $F'_{duration\ limited}$)
- Deep water case:
- **gt_d/U_A= 68.8 $(gF'/U_A^2)^{2/3}$**
- Shallow water case:
- **gt_d/U_A= 537 $(gT_m/U_A)^{7/3}$**

Deep water case

- **$gF'/U_A^2 = \{(gt_d/U_A)/68.8\}^{3/2}$**
- **t_d in seconds**
- **Enter Figure 3.21 to get H_s**

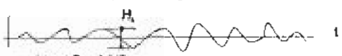
Shallow water case:

$gT_m/U_A = \{(gt_d/U_A)/ 537\}^{3/7}$

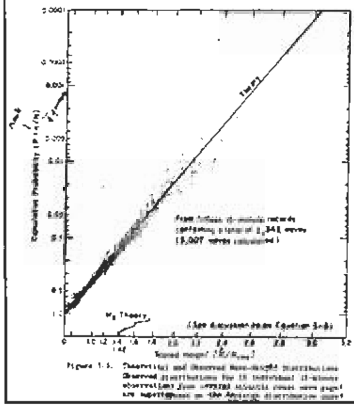
- **Enter Fig 3.22 to get gF'/U_A^2**
- **Then use gF'/U_A^2 to get H_s**
- **$T_s = gU_A\ \{(gt_d/U_A)/ 537\}^{3/7}$**

Wave Distribution

- Rayleigh distribution
- $P(H>X) = EXP[-(H/H_{rms})^2]$
- $P_{av}(H_{av}>X)$ = Probability for average of higher waves than P (see Figure 3-3)



- $H_{rms} = \{\Sigma H_i^2/n\}^{1/2}$
- $H_s = 2^{1/2} H_{rms}$
- $H_{10\%} = 1.27H_s$
- $H_{1\%} = 1.67H_s$



Rayleigh Distribution

$$P(H > \hat{H}) = e^{-\left(\frac{H}{Hrms}\right)^2}$$

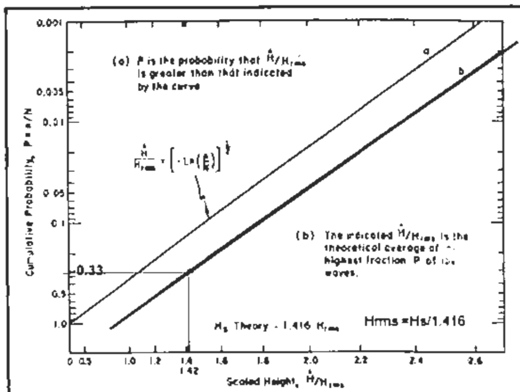


Figure 3-5. Theoretical wave height distributions.

Dimensional Analysis

- **$H_s = f(g, d, U_A, F, t_d)$**
  - : d = average depth over F
  - : U_A = effective wind speed over F
  - : t_d = duration of wind
- **$T_s = \Theta(g, d, U_A, F, t_d)$**
- **$gH_s/U_A^2 = fcn(gF'/U_A^2, gd/U_A^2, gt_d/U_A)$**
  - See Figure 3.21
- **$gT_s/U_A = Fcn(gF'/U_A^2, gd/U_A^2, gt_d/U_A)$**
  - See Figure 3.22

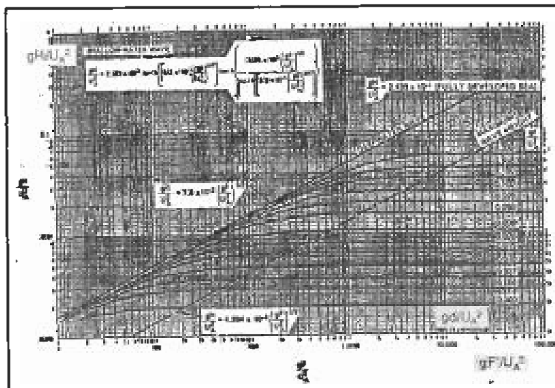


Figure 3-21. Forecasting curves for wave height. Constant water depth.

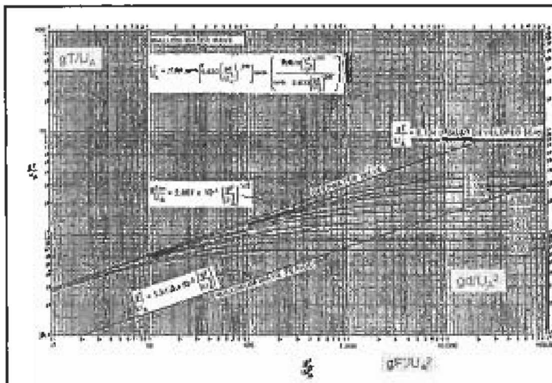


Figure 3-22. Forecasting curves for wave period. Constant water depth.

Lectweb-1 ppt
Session 5 wave Generation

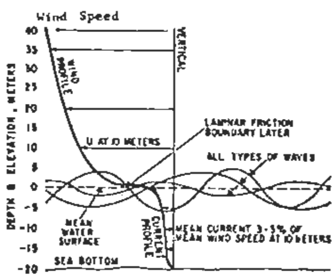
Wave Generation

Shore Protection Manual
Method
Non-Hurricane Waves

Intermediate & Deep Water

- Factors:
  - Wind speed at 10m over water (Uow)
  - Fetch – clear distance over which wind blows (F)
  - Average water depth over fetch (d)
  - Duration of sustained wind (t_d)
  - Atmospheric stability (air, water temperature) (R_T)
  - Wind shear effect (momentum transfer increases as wave H increases) (U_A)

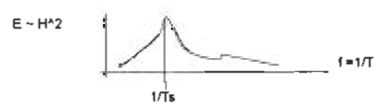
Wind Speed and Water Current



Wave Forecasting Equations

Definitions:

- H_s = significant wave height = average of highest 1/3 of waves in a design storm
- T_s = significant period or period of highest 1/3 of the waves



Wave period spectrum

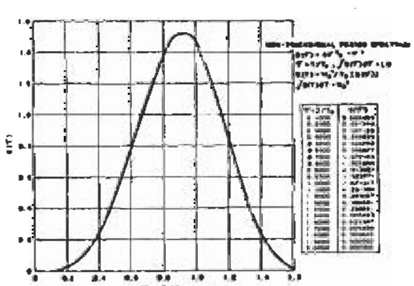
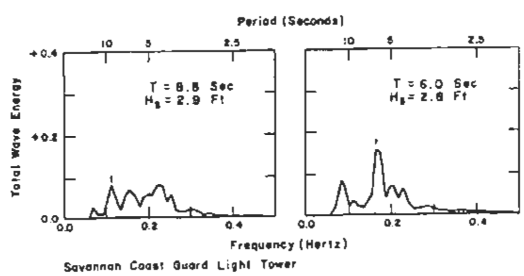


Fig. 2 Non-dimensional period spectrum



Major Assignment Problem 9.1:

Name: Donald Jerolleman Due April 9, 2010

22/25

Design a rubble mound breakwater for the significant wave and setup near the south shore of Lake Pontchartrain for a 1 hour wind out of the north over the Lake.

Given:

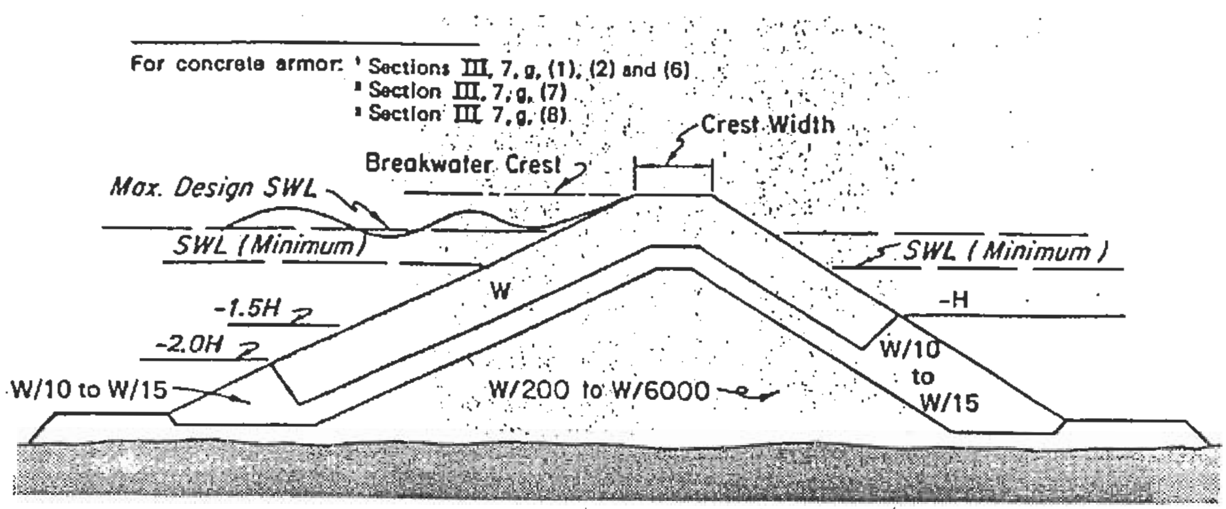
1. a 25 mile fetch
2. an average depth of 3.8 m before the storm surge,
3. a storm surge of 2.2 m,
4. a local depth of 2.8 m at the breakwater without the storm surge or setup,
5. a water temperature of 80°F and air temperature of 70°F.
6. an over-water velocity is 80 mph.
7. spring tidal range 0.16 m.
8. Gulf seasonal amplitude 0.1 m (positive for hurricane season).

Design Period 50 years.
Design the crest elevation for no overtopping at the design Hs.

Assume: Ss =2 .63;
slope 1:2;
individually placed rough angular rock armour stone; 2 layers
the mean SWL in Lake Pontchartrain is 0.2 m MSL;
the minimum SWL is -0.3 m;
RSLR including subsidence rate 0.9 cm/year;
settlement = 0.3 m.

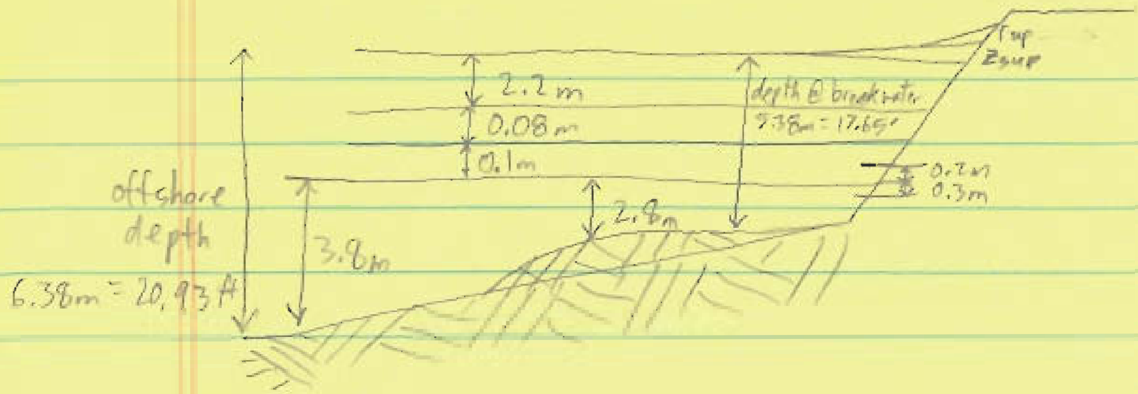
Sal = 6 ft

Use the Standard Section shown below.



Recommended Three-layer Section

```
REPORT
US Units
Density=   1.940000
Offshore Depth = ft or m    20.93180
Fetch = ft or m    132000.0
Speed = ft/sec or m/s    117.3333
Duration = hr    2.000000
Rt stability factor =    1.000000
RL sea/land factor =
Setup in feet =    5.459908
Setup in m =    1.664606
Stress Ua =    189.3422
Non-dimensional depth Fd  1.8800424E-02
Non-dimensional fetch FF   118.5591
Non-dimensional duration Ftd   1224.450
Fetch Limited
Non-dimensional Wave Height FHs =  7.4579213E-03
Non-dimensional Period FTs =   1.067452
Significant Wave height Hs (ft or m) =    8.303414
Significant Wave Period Ts (s) =    6.276821
Deep wave length and celerity (Lo,Co)    201.9089       32.16739
Wave length and celerity (L1,C1)   145.1952       23.13196
Local depth d near surf zone =    17.65080
Offshore Angle Alfa =    13.00000
 EFFECTIVE DEEP WATER WAVE ANGLE =     18.22912
 DEEP WATER WAVE HEIGHT =    9.044266
EFFECTIVE DEEP WATER WAVE HEIGHT =    9.044266
Local angle =    12.15290
Local wave length and height    135.8814       8.425593
Local wave height and celerity    8.425593       21.64812
Group Ratio, Ks and Kr (Local)   0.8317500      0.9451187      0.9856912
Breaking Wave height    13.76762
-     Hrms,            Hs,            H10,           H1
Offshore   5.871401       8.303414       10.54534       13.86670
Local    5.957794       8.425593       10.70050       13.76762
```



offshore depth = 3.8 + 0.2 + 0.1 + 0.8 + 2.2 = 6.38 m = 20.93 ft

breakwater depth = 2.8 + 0.2 + 0.08 + 2.2 = 5.38 m = 17.65 ft

From UNO wave program

H_s = 8.43 ft (local) = 8.30 ft (offshore)

$$Z_{sup} = \frac{F C_f U_m^2 \rho_a}{2 g d \rho_w} = \frac{25(5280)(0.003)(117.3)^2(1)}{2(32.2)(17.65)(800)} = \underline{6.00 \text{ ft}}$$

$$H_I = H_s \quad \frac{H_i}{L} = \frac{8.43 \text{ ft}}{135.88 \text{ ft}} = 0.062$$

relative runup = $0.73 = \frac{r_{up}}{H_I} \rightarrow \underline{r_{up} = 6.15 \text{ ft}}$
p.117 Fig 7-13

Total Local Depth = $17.65' + \frac{8.30'}{2} + 6.00' + 6.15' = \underline{33.9'}$ elevation of crest ?

settlement = 0.3 m = 0.99'

subsidance = $0.9 \frac{cm}{yr} \left(\frac{1 in}{2.54 cm}\right)(50 yr)$ = 17.7 in = 1.48 ft

crest elevation = 33.21' + 0.99' + 1.48' = $\underline{35.68'}$ ×

$S_s = 2.63$, salinity = 6 ppt

$$S_r = S_s\left(\frac{1}{1+\frac{6}{1000}}\right) = 2.61 \qquad H = H_s(local) = 2.57 m$$

$\cot\theta = 1.8 \approx 2$

this is a footnote not exponent

Rough angular – Special[6] per Table 7-8

Head — $K_D = 6.4$

Trunk — $K_D^{(2)} = 7.0 \rightarrow K_D = 2.65$

$S.G. = \frac{\gamma_r}{\gamma_w}$

$\gamma_r = 2.63\ \gamma_w = 2.63\ \frac{9.81\ kN}{m^3}\left(\frac{1000\ N}{kN}\right) = 25800\ \frac{N}{m^3}$

$\gamma_{r_{mass}} = \gamma_r\left(\frac{1}{9.81\ \frac{m}{s^2}}\right) = 2630\ \frac{kg}{m^3}$

$H = H_1 = 13.76' = 4.19m$

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

use Hs

Head

$$W = \frac{2630\frac{kg}{m^3}(4.19m)^3}{6.4(2.61-1)^3(2)}$$

$$= \frac{193462.96}{53.42}$$

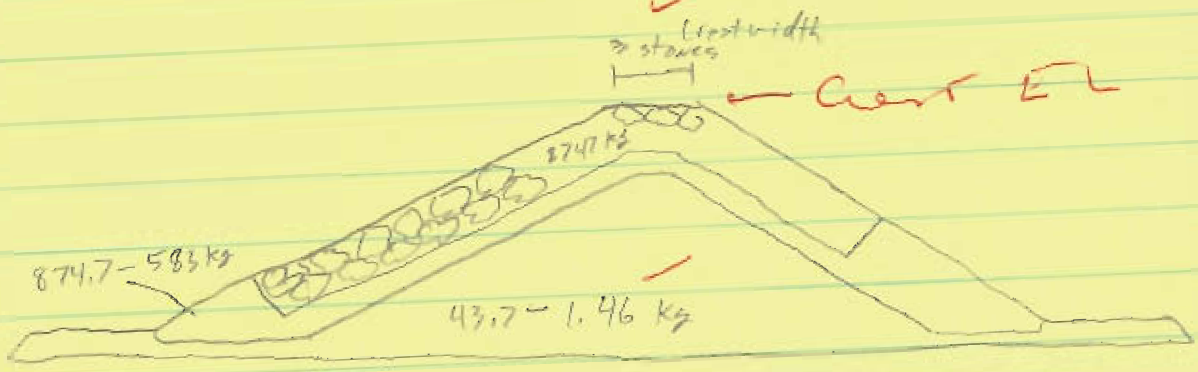
$$= 3622\ kg$$

Trunk

$$W = \frac{2630(4.19m)^3}{2.65(2.61-1)^3(2)}$$

$$= \frac{193462.96}{22.12}$$

$$= 8747\ kg$$



Donald Jecolleman 10.2

Does minikin theory apply: ~~No~~ Yes 8

ok

H_b = 17.36'

Base on your H_b

minikin p_m = 15707 lb/ft

minikin R_m = 90893 lb/ft

minikin M_m = 1272503 lb·ft/ft

R_T landward = 106942 lb/ft

M_T landward = 1394152 lb-ft/ft

$H_o < 14'$ $T = 7.6s$ $d_s = 14'$ wall h = 27' $m = 0.05$ $\gamma_w = 62.4$ lb/ft³

$\frac{d_s}{gT^2} = 0.007527 \rightarrow$ Fig 7-4 $\frac{H_b}{d_s} \approx 1.24 \rightarrow H_b = 17.86$ ft

Fig 7-100 $\frac{P_m}{\gamma_w H_b} = 14.3 \rightarrow P_m = 15707$ lb/ft

$R_m = \frac{P_m H_b}{3} = 90893$ lb/ft M_m (b/c Theory does not apply) $= \frac{P_m H_b d_s}{3} = 1272503 \frac{lb\text{-}ft}{ft}$

$R_s = \frac{1}{2}\gamma_w\left(d_s + \frac{H_b}{2}\right)^2 = 16049$ lb/ft

$R_T = R_s + R_m = 106942$ lb/ft

$M_s = \frac{\gamma\left(d_s + \frac{H_b}{2}\right)^3}{6} = 121,650$ lb-ft/ft

$M_T = M_s + M_m = 1,394,152$ lb-ft/ft

Donald Jerolleman 10.3

NO

Does broken theory apply : Yes

H_b @ X_p = 22.2 ft

d_b @ X_p = 17.93 ft

Broken wave p_m = 559 lb/ft

" " R_m = 15428 lb/ft

" " M_m = 215993 ft-lb/ft

" " R_T landward = 35114 lb/ft

" " M_T landward = 380817 lb-ft/ft

method ok

9.6

8.0

$H_o \le 14 ft$ $T = 7.6 s$ $d_s = 14 ft$ wall h = 27 ft $m = 0.05$

use highest H_b @ X_p based on $H_o \le 14 ft$

$\frac{d_s}{gT^2} = 0.007527$ Fig 7-4 $\frac{H_b}{d_s} = 1.24 \rightarrow K_B = 1.24$

?

1.06

$H_b = \frac{d_s}{\frac{1}{K_b} - m(4-9.25m} = 22.2 ft$ $X_p = (4-9.25m) H_b = 78.7 ft$

$d_b = d_s + m X_p = 17.93 ft$ $H_{o\,max} = \frac{H_b}{K_s} = 22.2 ft$

$P_m = \frac{\gamma(c)^2}{2g} = \frac{\gamma d_b}{2} = 559 \, lb/ft^2$ $R_m = \frac{\gamma d_b h_c}{2} = \frac{\gamma d_b (K_b H_b)}{2}$

$= 15428 \, lb/ft$

$M_m = \frac{P_m H_b d_s}{2} = R_m d_s = 215993 \frac{ft-lb}{ft}$

$R_s = \frac{\gamma(d_s + \frac{H_b}{2})^2}{2} = 19686 \, lb-ft$ $R_T = R_s + R_m = 35114 \, lb/ft$

$M_s = \frac{\gamma(d_s + \frac{H_b}{2})^3}{6} = 164824$ $M_T = M_s + M_m = 380817 \frac{lb-ft}{ft}$

Forces

15

- Hi = 6 ft; T=8 sec; d=18 ft; D=2 ft; m~0
- C_M= 1.5; C_D=0.7
- →d/gT2= 0.0087 ;H/Hb= 0.43
- Maximum inertia Force?
- Kim = 0.35 Figure 7-71
- Fim = 618 lb
- Maximum drag Force?
- K_{Dm} = 0.43 Figure 7-72
- F_{Dm} = 67.6 lb
- NAME Donald [illegible]

Resultant Forces

- Hi = 6 ft; T=8 sec; d=18 ft; D=2 ft; m~0
- C_M= 1.5; C_D=0.7
- →d/gT2= 0.0087 ;H/gT2=0.0029
- W=0.71
- Resultant of inertia and drag Forces?

☐ ϕm = 0.24 Figure 7-78 W = 0.5

☐ ϕm = 0.34 Figure 7-79 W=1

☐ ϕm = 0.29 Figure 7-79 W=0.71

- FRm = 900 lbs
- NAME Donald Serolleman

Resultant Forces

- Hi = 6 ft; T=8 sec; d=18 ft; D=2 ft; m~0
- C_M= 1.5; C_D=0.7;
- →d/gT2= 0.0087 ;H/gT2=0.0029
- Find: Resultant of lift and drag Forces? fo?
- Lift = 676 lb
- Drag = 676 lb
- Resultant = 956 lb ✓
- fo = 0.75 - 0.96 Hz ✓
- NAME Donald Serallema

Second Term Test
Spring 2010
B
ENCE 4723
Coastal Engineering

Duration 2 hours

Open-book including calculator, notes and texts.

Attempt all questions

Give your answers on the sheets provided.

Question # 1 ______ 11 /12

Question # 2 ______ 7½ /8

Question # 3 ______ 7½ /~~12~~ 8

Question # 4 ______ 10 /~~8~~ 12

TOTAL 36 /40

Name: DONALD JEROLLEMAN ______ PLEASE PRINT!
Last Name

Student No.
2330000

FPS(0.3048) = mps FPS(0.6818) = mph

1m = 3.281 ft
1 ft = 0.3048 m

1. Given: A lake that is 45 miles long and 40 ft deep. The overwater wind speed at 7-m above the surface is 76 mph. Assume: The air and water temperatures are nearly the same. Td = 1.25 hours.

Circle the closest answer:

Determine:

a) Hs=: [≤8, 11, 12, 13, 15, ≥20] ft 2/3

b) Ts= : [≤2, 3, 4, 5, 7, ≥15] sec 3/3

c) Using a friction coefficient of 0.0025, the wind setup is:
[≤1.5, 2, 3, 4, 5, 6, ≥7] ft 2/2

d) The waves are: [Fetch Limited, Duration Limited] 1/1

e) H_{rms} = [≤5, 7.5, 8.5, 10, 12, ≤13] ft 1

Show your calculations here!

F = 45 miles = 237600 ft = 72420 m d = 40 ft U_{ow} = 76 mph = 111.47 fps = 33.98 mps Td = 1.25 hr = 4500 s

$R_L = 1$

$R_T = 1$

$U_A = 0.589(76 mph)^{1.23} =$ ~~44.764 mph~~ 121.2
$0.539(111.47)^{1.23} = 177.66$ fps

$\frac{gd}{U_A^2} =$ ~~0.6418~~ 0.0408

$\frac{gF}{U_A^2} =$ ~~0.723~~ (feet) = 242.4

$\frac{gH}{U_A^2} = 0.011 \rightarrow H_s = 10.78$ ft

$\frac{gT}{U_A} = 1.3 \rightarrow T_s = 7.17$ s

$\frac{gT_m}{U_A} = \left\{\left(\frac{g t_d}{U_A}\right)\frac{1}{537}\right\}^{3/7}$ $T_m = 6.6$

$T_m < T_s$

3

$\frac{gT_m}{U_A} = 1.19$ $\frac{gd}{U_A^2}$ .022 → $\frac{gH}{U_A^2} = 0.0085 \rightarrow H_m = 8.33$ ft

$Z_{sup} = \frac{1}{900}\,\frac{F(C_f)U_{ow}^2}{2gd} = 3.59$

2 Given: A hurricane in the northern Gulf of Mexico with:

- $\Delta p = 60$ mm = 2.36"
- R = 18 Nmiles
- $V_F = 11$ mph = 9.559 knots
- $\alpha = 1.2$
- Reference depth 44 ft.
- Latitude 28 degrees

Circle the closest answer:

Determine:

a) H_{os} is: [<20, 30, 35, 38, >40] ft

b) T_{os} is: [<12, 13, 14.5, 15, >15] sec

.

c) If CR=2 and Cf=0.0026, the surge height is approximately:
[<1, 2, 4, 6, 8, >10] ft

2

2

1/2 / 1

Show your calculations here!

$U_{max} = 0.868 \text{ knots} \{73 \text{ miles} (2.36 \text{ in})^{1/2} - 0.575 \text{ inch} (18 \text{ miles})(\frac{4\pi}{24}) \sin 28° \}$

$= 0.868 (112.145 - 2.52) = 95.15$ knots

$U_R = 0.865 (95.15) + 0.5 (9.559) = 87.08$ knots

$T_s = 8.6 \left(1 + \frac{0.104 (1.2)(9.559)}{\sqrt{87.08}}\right) e^{\frac{18(2.36)}{200}} = 11.99$ s

$H_{os} = 16.5 \left(1 + \frac{0.208 (1.2)(9.559)}{\sqrt{87.08}}\right) e^{\frac{18(2.36)}{100}} = 31.68$ ft

3

ft/sec

$h_{ss} = k_s K_a \left(2.36 + \frac{2(0.0026)(87.08^2)}{2(32.2)(44)(800)}\right) \approx k_s K_a (2.36)$

1mm = 0.003281 ft   1 cm/s = 0.03281 ft/s

3. Given: A beach with a D50 = 0.45 mm. Assume water temperature approximately 20 degrees C.

Circle the closest answer:

a) The fall velocity is: [<0.03, 0.15, 0.2, 0.25, 0.3, >0.4] ft/sec   1/1

b) The stable beach slope is: [< 1%, 1.5%, 2%, 3%, >4%] "Exposed"   1/1

c) For a deep water wave Ho = 5 ft and T = 10 sec, the beach profile tends to: [Berm; Offshore bar(s), Neither] State method 1984 SPM $F_o = \frac{H_o}{V_f T}$ 2/2   Fig 8.14   >1 → offshore, <1 → onshore

d) The closure depth for the wave in (b) is:
[<50, 80, 100, 120, 130, >150] ft   1½/1

Show your calculations here!

D_{50} = 0.45 mm Exposed $H_o = 5'$ T = 10 s

= 0.0014765 ft

$\frac{H_o}{D_{50} \text{ in ft}} = 3386.5$ 2/2

1:16 = 0.0625 = 6.25%

Dimensionless fall time = $\frac{H_o}{V_f T}$

$L_o = \frac{gT^2}{2\pi} = 512$ ft

w = 6.5 cm/s = 0.213 ft/s

$F_o = \frac{5}{(.213)(10)} = 2.35$

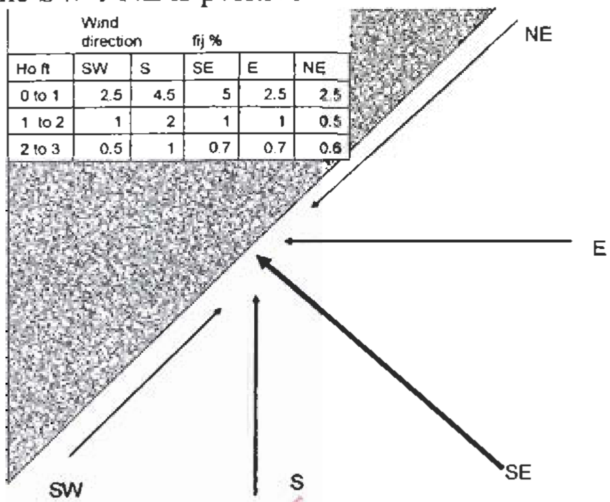
$\frac{\pi(0.213)}{32.2(10)} = 0.0213$

$\frac{H_o}{L_o} = \frac{5}{512} = 0.00976$

$u = 0.5$ ft/s $= \frac{5}{2}\left(\frac{32.2(10)}{L}\right)\frac{1}{\cosh(2\pi d/L)}$

D_{50} = 450 micron

4. Given: Given: the wave frequency table below. The beach slope is 1.25%.
Assume SW→NE is positive



Circle the closest answer:
Estimate:

a) the net longshore transport is:
[≤6500, 7500, 10000, 15000, 30000, ≥360000] yd3/yr

b) the gross longshore transport is:
[≤7000, 7500, 14000, 22000, 36000, ≥50000] yd3/yr

c) the maximum breaker induced current is (use 3 ft wave):
[≤ 0.5, 1, 1.25, 1.5, 1.75, 2, ≥4] ft/sec

Show your calculations here and in table above!

Wind direction fij %

Ho ft	SW	S	SE	E	NE
0 to 1	1.25 2.5	4.5	5	2.5	1.25 2.5
1 to 2	0.5 1	2	1	1	0.25 0.5
2 to 3	0.25 0.5	1	0.7	0.7	0.3 0.6
Hoi rad	1.37445	0.7854	0	-0.7854	-1.3744
0.5	774.53	29.9 1000	0	-16.855	-14.574
1.5	48090.5	2076924	0	-10 53462	-45240
2.5	162.3 861	12415	0	-2 63 8690	-194.8 -1633
	267.3	608	0	-380.1	-254.6
	1418	20339		-12707	-1390

$137000 \left(H_{oi}^{5/2}\right)\left(\cos \alpha_{oj}\right)^{1/4} \sin(2(\alpha_{oj}))\left(\frac{\%}{100}\right)$

SW – NE = ~~475.3~~ 21757 yd3/yr

NE – SW = ~~634.7~~ 14057 yd3/yr

Net Q_L = ~~240.6~~ 7700 SW – NE yd3/yr

Gross = 1510 yd3/yr 35814

Assignment 8.1.

Name: Donald Serolleman

9.5/10

For the wave rose represented in the following table, estimate:

1. The longshore transport in each BIN,
2. The net longshore transport,
3. The gross longshore transport.
4. If the beach D50 is 0.4 mm and it is exposed, will the beach be stable?

H ft	W	SW	S	SE	E Wind Direction
0.5-1	2	6	6	8	2
1-1.5	1	6	7	7	5
1.5-2	1	2	3	3.5	2
2-3	0.5	1	1	1	0.7

The shoreline runs due east-west. The beach slope is 1%.

Enter the ΔQ_{ls} values in the Table below in yd^3/year

H ft	WSW	SW	S	SE	ESE Wind Direction
0.5-1	170	3672	0	-4896	-170
1-1.5	304	13168	0	-15363	-1522
1.5-2	706	10179	0	-17814	-1412
2-3	861	12415	0	-12415	-1205
Sub TOTALS	2041	39434	0	-50497	-4308

Net Qls = -13321 → 13321 E→W

Gross Qls= 96270

Beach: ***STABLE*** *or UNSTABLE* very

units

Attach sample calculations.

Donald Jerolleman

Grain-size to slope Fig 7.20 For exposed beaches.

$$Q_{ls} = K_y H_o^{5/2} (\cos \alpha_o)^{1/4} \sin 2\alpha_o$$

$$Q_{ls_{net}} = K_y \Sigma_i \Sigma_j \{ H_{oj}^{5/2} (\cos \alpha_{oj})^{1/4} \sin 2\alpha_o \} f_{ij}$$

$$Q_{ls_g} = K_y \Sigma_i \Sigma_j \left| \{ H_{oj}^{5/2} (\cos \alpha_{oj})^{1/4} \sin 2\alpha_{oj} \} f_{ij} \right|$$

Example For WSW

$$a_o = 90 - \frac{22.5}{2} = 78.75 \qquad H_o = \frac{1+0.5}{2} = 0.75$$

$$Q_{ls_{net}} = 7132 + 54615 + 0 - 151119 - 9534 = -98906$$

$$Q_{ls_{gross}} = 61747 + 160653 = 222400$$

units

ENCE ENME NAME 4723 and 4723G

Final Examination 2010

Thursday May 6, 2010
Time 8 pm – 10 pm
Regular Class Room
Open Book

Topics
Design of breakwaters:

- Determination of Design Still Water Level (tide, annual fluctuations, storm surge, wind setup, wave setup).
- Selection of Design wave
- Determination of armour unit weight and zone details.
- Determination of Crest ELEVATION (Design SWL , runup, RSLR and settlement).

Design of seawalls:

- Determination of Design Still Water Level (tide, annual fluctuations, storm surge, wind setup, wave setup).
- Selection of Design wave
- Determination of armour unit weight and zone details.
- Determination of Crest ELEVATION (Design SWL , runup, rushdown, RSLR and settlement).
- Overtopping flow.

Forces on seawalls:

- Vertical walls:
  - Non –breaking waves
  - Breaking waves (Minkin)
  - Broken waves
- Sloping walls.
- Oblique wave attack on walls.

Forces on piles:

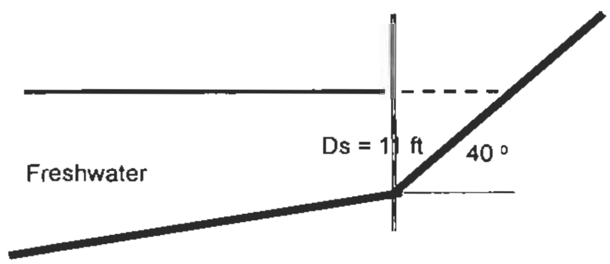
- Slender piles:
  - Inertia
  - Drag
  - Lift
  - Frequency of forces.
- Large diameter piles.

Review for Final Exam

EN 4723

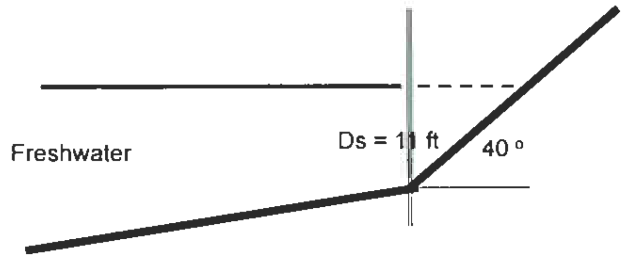
1. Minikin Forces

- Given: Hi≤20 ft; T = 7 sec; m = 5%; ds=11 ft
- A) Find Minikin Force on a vertical wall
- B) Find Horizontal component of the non-breaking Minikin Force on a 40 degree sloped concrete wall



2. Broken Wave Forces

- Given: Hi≤20 ft; T = 7 sec; m = 5%; ds=11 ft
- A) Find Broken Wave Force on a vertical wall
- B) Find horizontal component of the non-breaking Broken Wave Force on a 40 degree sloped concrete wall

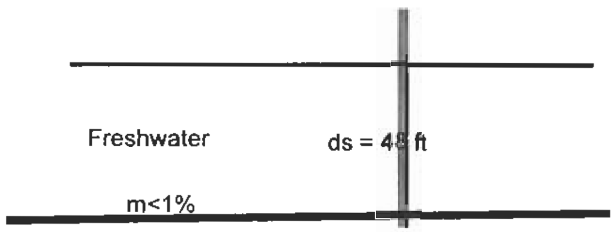


3. Non-Breaking Wave Forces

- Given: Hi=6 ft; T = 7 sec; m < 1%; ds=11 ft
- A) Find non-breaking Wave Force on a vertical wall
- B) Find Horizontal component of the non-breaking Force on a 40 degree sloped concrete wall

4. Forces on Piles

- Given: Hi=16 ft; T = 7 sec; m ~0; ds=48 ft; D=6 ft
- Assume:CL=0.7 CD
- Find the wave forces on a rough circular pile.
- i.e Fim, FDm, FL and the resultants of these
- Find St and fo.

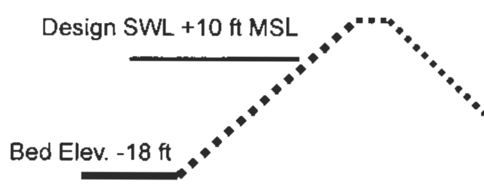


5. Seawall & Breakwaters

- Design the trunk of a Breakwater:
- Find Armour layer W and Crest Elevation.
- Given:
  - Local Design SWL Elev. 10 ft MSL (including, storm surge, spring tide, seasonal fluctuation & wind setup.
  - Local Bed Elev. -18 ft MSL
  - Hs = 8 ft; Ts= 5.5 sec
  - Use Tetrapods in two layers
  - Slope 2H:1 V
  - Salinity 34 ppt
  - Ss= 2.4
  - m=<1%

  RSLR = 1.5 ft

  Settlement =1 ft



PROB 1

Hi	<=	20	ft	
T		7	s	
ds		11		
ds/gT2		0.0070		
m		0.05		
Fig 7-4				
Kb=Hb/Hs		1.1		
Hb		12.1	ft	<20	Exists

Fig 7-100			Part
pm/gam*Hb	13.5		A
pm	10193	lbs/ft2	A
Fm	41552	lbs/ft2	A
Mm	452231	ft-lbs/ft	A
40 o Slope			
Fm at 40o	16866	lbs/ft2	B
Rm	29420	lbs/ft2	B

Problem 3

Hi	<=	20	ft	
Use Hi		6	ft	
T		7	s	
ds		11		
Hb		7.81	ft	>6	nonBreaking
ds/gT2		0.0070		
Hi/gT2		0.0038		
Hi/ds		0.55		
m		0		
Fig 7-90				
ho/Hi		0.71		
ho		4.26	ft	A
F/gam*d2		0.75		A	Figure 7-91
Fm		5663	lbs/ft	A
FT		9438	lbs/ft	A
Fm at 40o		2340	lbs/ft2	B
Rm at 40o		3640	lbs/ft2	B

Prob 4

Use Hi	16	ft		CHECK
T	7	s		Lo 251.1 ft
ds	45			L 220.5 ft
D	6	ft		D/L 0.0272 m
m	0			
Kb	0.78			
Hb	37.44	ft	>16	nonBreaking
ds/gT2	0.030			
Hi/gT2	0.010			
nu	0.00001	ft2/sec		
H/Hb	(A)			

Fig 7-71	Kim	0.44	
Fig 7-72	KDm	0.28	
Umax	8.2	ft/sec	
Re	5.E+06		
CM	1.5	High Re	
CD	1.65	Rough	
CL	1.12	0.7*CD	
Fim	18631	lbs	
FDm	21470	lbs	
FL	15029	lbs	
RDL	26207	lbs	Drag+lift
W	0.352		
Wo	0.100	Wo	0.5
phi	0.150	phi	0.215
phi	0.190	at W=0.35	
R,Q	29137	lbs	Morison and Drag

St 0.290

fo 0.40 hz

Breakwater		
Ss	2.4	
Sa	34	
Sr	2.3211	
Hi	8	ft
T	5.5	
Design SWL	10	ft
Bed	-18	ft
ds	28	ft
m	0	
Kb	0.78	
Hb	21.84	ft
Nonbreaking	8.0	ft
Lo	154.9	ft
L	134.0	ft
Hs/L	0.059710364	0.06
Fig 7-13	Ru/Hs	0.72
Ru	5.76	
Crest Elev	18.26	ft MSL
KD	8	Tetrapods
cotθ	2	
W	2079	lbs

RSLR 1.5 | Settlement 1

SPM 1984

PROB 2

Hi	<=	20	ft	
T		7	s	
ds		11		
ds/gT2		0.0070		
m		0.05		
Fig 7-4				
Kb=Hb/Hs		1.1		
Hb		15.0	ft	<20	Exists
Xp		53.1	ft	A	Ok
dc		14	lbs/ft2	A
hc		17	lbs/ft2	A
Am		426		
Fm		7541	lbs/ft	A

Fm at 40o	2809	lbs/ft2	B
Rm	4526	lbs/ft2	B

ds/gT2	0.0087		
Hb/db	1.0400		
Hb	14.0	ft	
Xp	49.6	ft	
db	13.5	ft	
hc	14.8	ft	
pm	421	Lbs/ft2	
Fm	5696	lbs/ft	A

40o Slope

Fm at 4	2436	lbs/ft2	B
Rm	3790	lbs/ft2	B

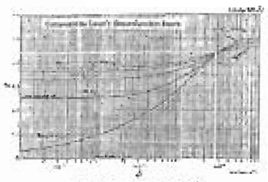
ds/gT2	0.0085		
Hb/db	1.0400	ok	STOP HERE
Hb	14.0	ft	
Xp	49.6	ft	
db	13.5	ft	
hc	14.8	ft	
pm	421	Lbs/ft2	
Fm	5696	lbs/ft	A

40o Slope

Fm at 4	2436	lbs/ft2	B
Rm	3790	lbs/ft2	B

Definitions

$$u_{max} \cong \frac{\pi H L_o}{T L_A}$$

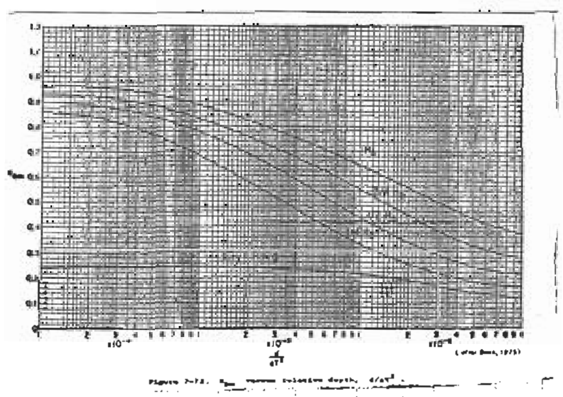


$$F_{im} = C_M \rho g \frac{\pi D^2}{4} H K_{im} \quad (7-37)$$

$$F_{Dm} = C_D \frac{1}{2} \rho g D H^2 K_{Dm} \quad (7-38)$$

$$M_{im} = F_{im} d S_i \quad (7-39)$$

$$M_{Dm} = F_{Dm} d S_D \quad (7-40)$$



HUDSON EQUATION

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

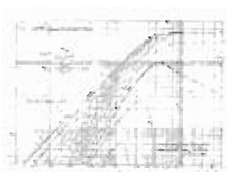
γ_r = Dry_Specific_Weight_of_Armor_Unit

K_D = Stability_Coefficient(Damage_Coefficient)

θ = Slope_of_face

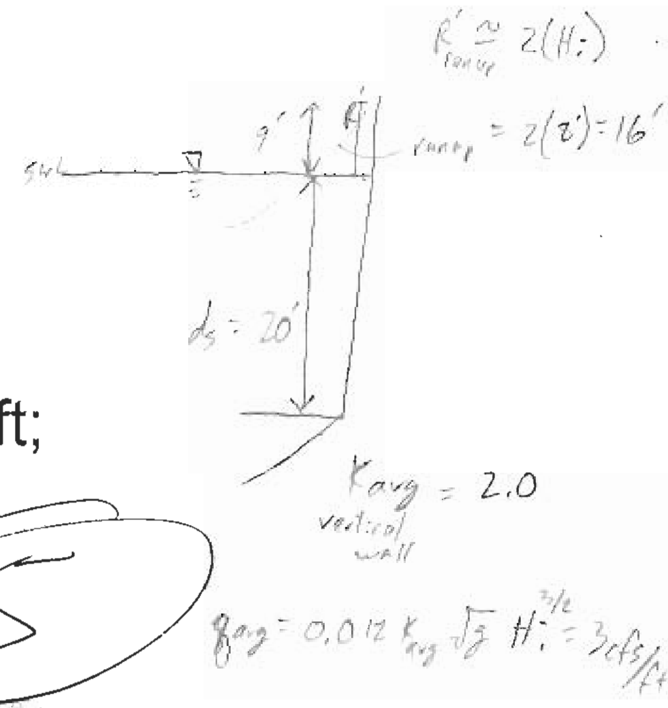
$S_r = \frac{Specific\_gravity\_Armor\_Unit}{Local\_Specific\_Gravity\_of\_Water}$

H = Design_wave_height

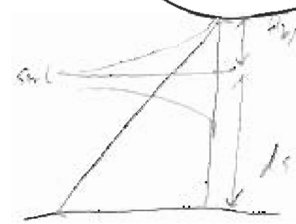


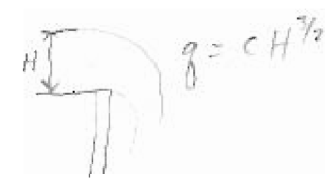
Name Donald Jerolleman

Overtopping Example



- Given: d_s = 20 ft; H_i=8 ft; T= 7sec; h = 9 ft;
- ~3H:1V slope
- Find: qave overtopping
- Solution: Ru = R' = 2*H_i=16 ft
- h/R'=9/16 = 0.56
- Kave= 2 (From graph)
- qave= 3.08 cfs/ft
- --------------------





- *For the same wave and a vertical wall with R'=11.6 ft*
- *qave=(0.25), (0.5), (1), (2), (3) Units* cfs

$H_b = 7.6'$, $d_s = 10'$, $T = 7s$, $m = 3\%$ find p_m, R_m, M_m, R_t, M_t

$\frac{d_s}{gT^2} = 0.0063$, H_b (based on d_s not H_i) worse case breaking wave Fig 7.4 → $\frac{H_b}{d_s} = 1$ → $H_b = 10'$, now calc max pressure Fig 7-100

moment = $M_m = R_m(d_s) = 291{,}000\ \frac{ft\text{-}lb}{ft}$

$\frac{p_m}{\gamma H_b} = 14$ → $p_m = 8730\ lb/ft^2$, $R_m = \frac{p_m H_b}{3} = 29{,}100\ lb/ft$

Hydrostatic Force $R_s = \frac{1}{2}\gamma(d_s + \frac{H_b}{2})^2 = 7020\ lb/ft$

con't: $R_T = R_s + R_m = 36120\ lb/ft$

$M_s = \frac{1}{6}\gamma(d_s + \frac{H_b}{2})^3$

$M_T = M_s + M_m$

P_m	351	lb/ft²
h_c	11.25	ft
R_m	3948	lb/ft
M_m	61700	ft-lbs/ft

① Est. H_b @ X_p assume K_b Fig 7.4

ⓐ need → $\frac{d_s}{gT^2} = 0.0063$, $m = 0.03$

↳ $\frac{H_b}{d_b} = K_B \simeq 1.0$

ⓑ → $H_b = \frac{d_s}{\frac{1}{K_p} - m(4 - 9.25(0.03))} = 11.25'$

ⓒ → $X_p = (4 - 9.25(0.03))11.25 = 41.9' \sim 42'$

ⓓ → $d_b = d_s + X_p m = 11.25'$

---

skip ② & will come back to it (see if wave is possible)

---

③ ⓐ $p_m = \frac{62.4(11.25)}{2} = 351\ lb/ft^2$ (γ → 62.4, d_b ← 11.25)

ⓑ → $h_c = K_p H_b = 11.25\ ft$

ⓒ → $R_m = \frac{\gamma(h_c)(d_b)}{2} = p_m(h_c) = 3948\ lb/ft$

ⓓ → $M_m = R_m\left(d_s + \frac{h_c}{2}\right) = 61700\ \frac{ft\cdot lb}{ft}$

② $H_{o\,max} \simeq 11'$ ↙ produce → $H_b = 11.25$

$H_o' = \frac{H_b}{K_s} = \frac{11.25}{1.06} = \underline{10.6}$ ∴ wave could occur and every type of wave bellow it.

(K_s shoaling coefficient) can get from Appx I

Ho	6 ft	Lo	160.5632		
Hi	5.634 ft				
T	5.6 s				
ds	15	ds/Lo	0.093421		
m	0.05	Ks	0.939		
Hi/ds	0.3756				
Hi/gT2	0.005579	ds/gT2	0.014855		
ho	0.41	2.30994 ft			
yc	22.94394 ft	<27	ft		no OT
yt	11.67594 ft				

Name ____________________

Assignment 10.1 Due April 15, 2010

Given: Smooth vertical seawall; H_o = 6 ft; T=5.6 sec; d_s = 15 ft; height of wall = 27 ft above the bed; m =0.05

Find: the following: y_c, y_t, Runup above SWL; Wave induced landward force; Total landward force; Wave induced seaward force; Total seaward force (assume SWL behind the wall).

Answers:

Does non-breaking theory apply? Yes No YES Hb 13.5 ft

y_c = ________ 22.94 ft fig 7-4 >Hi Nonbreaking

y_t = ________ 11.68 ft

Runup =_____ 7.944 ft

$F_{m\ landward}$ = _________ 0.39 5476 lbs

$F_{T\ landward}$ = _________ 7020 12496 lbs

$F_{m\ seaward}$ = ___ 3229 0.235 -3299.4 lbs

$F_{T\ seaward}$ = _____________ 3720.6 lbs

Show Solution:

4) Forces on Piles

$H_i = 16'$ $T = 7s$ $m \sim 0$ $d_s = 48'$ $D = 6'$ (diameter of pile) assume $C_L = 0.7 C_D$ (C_L: Lift coefficient; C_D: Drag coefficient)

rough circular pile

rough surface → $C_D = 1.2 \rightarrow 1.6$ (use 1.6 here)

First Calc. wave Length
then check if have slender pile $\frac{D}{L_A} \leq 0.05$

$$U_{max} = \frac{\pi H}{T} \frac{L_o}{L_A}$$

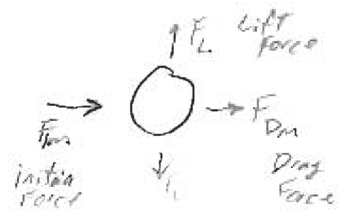
$$R_e = \frac{U_{max} D}{\nu}$$ (ν = Kinematic viscosity) → use $10^{-5} \frac{ft^2}{s}$

→ C_D (0.7 for smooth pile w/ high R_n)

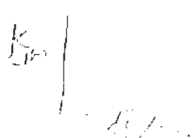
→ $S_t = \left(\frac{f_L D}{U_{max}}\right)$ (non dimensional frequency)

Fig 3.2

→ C_m



K_{im} 7-71



K_{Dm} 7-72

Fig 7-78 (allow to add K_{im}, K_{Dm})

$$R_{iD} = \phi C_D H_i^2 D \gamma$$

$$F_L = F_{Dm} \frac{C_L}{C_D}$$

If Pile is rough (eg. has barnicles) C increases ~1.3-1.5 C_m ~ 1.2

Alternative way to solve

$U_{max} \approx \frac{\pi H L_0}{T L_p} \approx 4.26$ ft/s

Reynolds Number $R_e = \frac{U_m D}{\nu}$ ← dynamic viscocity

(non dimensional) unitless

$\nu_{70°} = 10^{-5}$ ft²/s

$Re = 8.5 (10^5)$

Keulegan-Carpenter Number $N_{KC} = \frac{U_{max} T}{D} \approx 17$ (oscillation)

Strouhal Number $S_t = f_0 \frac{D}{U}$ (frequency)

Table 2.3

need to interpolate between Fig 7-79 & 7-76

Fig 7-76 (not right w) w = 0.05 curves are

$F_m = \phi_m \gamma_w C_D H^2 D$

each w has a different set of curves

$W = \frac{C_m D}{C_D H} \approx 0.71$

$w = 0.5 \rightarrow \phi_m = 0.24$
$w = 1 \rightarrow \phi_m = 0.34$

> Interpolating $w = 0.71 \rightarrow \phi_m = 0.285$

$F_m = 900$ lbs (Force in direction of wave)

→ O eddie? > alternating eddies Vortex shedding

Lift $F_L = F_{LM} \cos 2\theta = C_L \frac{\rho g}{2} D H^2 K_{Dm} \cos 2\theta$

$C_L = C_D$ max Drag 676 ∴ max Lift = 676

Lift ≈ Drag b/c

$\sqrt{676^2 + 676^2}$

$F_{RL-D} = \sqrt{Lift^2 + Drag^2}$

Fig 3.2 Re vs. S_t chart Fig 3.1a Flow regimes

$S_t = 0.35 - 0.45$ $f_0 = \frac{S_t (U_{max})}{D \sim 2}$ 4.26 = 0.75 – 0.96 Hz

Ocean & Coastal Structures III Forces on Piles

slender piles - support structure

Large Diameter ksons?

Traditionally

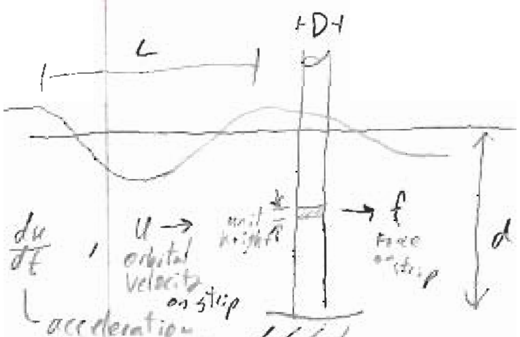
$$\text{Drag} = \frac{1}{2} \rho C_D A_\perp u|u|$$

(u: orbital velocity; $A_\perp$: area perpendicular to velocity)

slender

$\frac{D}{L} \leq 0.5 \rightarrow$ pile is slender

> 0.5 Large Diameter



$\frac{du}{dt}$ — acceleration on that strip

sum fs over whole d

each is max when the other is ϕ

inertia force (associated to acceleration)

Drag force (associated w/ velocity)

inertia or virtual mass Force

Drag Coeff.

D not $A_\perp$ b/c Area = D(1) (unit area)

Force on unit strip: $f = f_i + f_D = C_M \rho \frac{\pi D^2}{4} \frac{du}{dt} + C_D \frac{1}{2} \rho D u|u|$

Force on whole column: $F_m = F_i + F_D$

can use Fig 7-68 to get L_A but Appx. 1 is better

Fig 7-69 allow to correct η

Fig 7-70 use to get non-linear wave length

$H_o' = 6'$ $T = 8s$ $d = 18'$ $D = 2'$ $C_m = 1.5$ $C_D = 0.7$

$L_A = L_o \tanh\left(\frac{2\pi d}{L_A}\right)$ $L_o = \frac{g}{2\pi} T^2 = 328'$

'A' just means Airy

$\frac{d}{L_o} = 0.055 \rightarrow$ Appx I $\rightarrow$ $L_A = 181.5$

$\frac{D}{L_A} = 0.01$ ∴ slender pile theory applys

use again $\frac{d}{L_o} \rightarrow$ Appx I $K_s = \frac{H}{H_o'} = 1.006 \rightarrow H_i = 6.04 \text{ ft} \simeq 6 \text{ ft}$ (local wave height)

Fig 7-71, 7-72 are corrections for using non-linear wave theory

when slope = 0 $H_b = 0.78(d) = 14.04 \text{ ft}$

$H_i = \frac{\text{old } H_i \text{ (or } H_o'\text{)?}}{H_b} = 0.43$

$\frac{d}{gT^2} = \frac{d}{gT^2} = 0.0087 \rightarrow$ Fig 7-71 $\rightarrow K_{im} = 0.35$

$F_{im} = \frac{1}{4} K_{im} \gamma_w C_M \pi \pi D^2 H = \underline{618 \text{ lbs}}$

Fig 7-72 $K_{Dm} = 0.43$

$F_{Dm} = \frac{1}{2} K_{Dm} \gamma_w D H^2 C_D = \underline{676 \text{ lbs}}$

Assuming Salinity = 34 ppt

Hudson Eqn

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

SWL = 0.2 m (mean), −0.3 m (min)

Individually placed, 2 layers

S_s = 2.63 slope = 1:2

RSLR 0.9 cm/yr, settlement = 0.3 m

$\cot\theta = 1.8 \approx 2$

$S_s = 2.65$

$S_r = 2.65\left(\frac{1}{1+\frac{34}{1000}}\right) = 2.563$

$\gamma_r = S_s(1000) = 2650 \frac{kg}{m^3}$

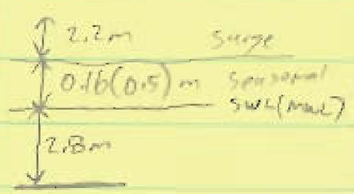
Rough Angular Quarrystone (Special[6]) KD = 5.3 Head, KD[2] = 5.8 Trunk

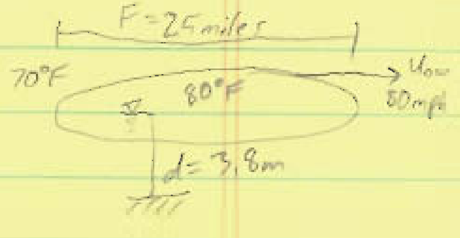
$H_s = \sqrt{2}\, H_{rms}$

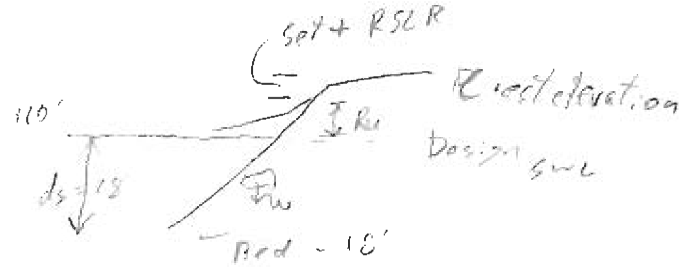
$H_{10} = 1.27\, H_s$

$H_1 = 1.67\, H_s$

Crest Elevation = MWL (2.8 m) + Seasonal (0.16 m (0.5)) + long term deviation from MWL + High Tide + Surge + Barometric Setup + Wind setup + Seiche + Wave Setup + Wave rundown + Settling / subsidence







5) Sea... & Breakers

$H_s = 8'$ $d_s = 28'$

$m \approx 0$ ∴ $H_b = 0.78(28) > 8'$ ∴ non breaking (see if breaking)

calc. $L_0 = \frac{32.2\,T^2}{2\pi}$ → L from Appx I

Fig. 7-13 req. steepness & slope of structure

calc R_u (run up)

use hudson equ. for $W = \frac{(S_s\,\gamma)\,H_s^3}{K_D\,(S_r - 1)^3 \cot\theta}$

(rock; fresh water (62.4 lb/ft³); K_D from 1984 chart)

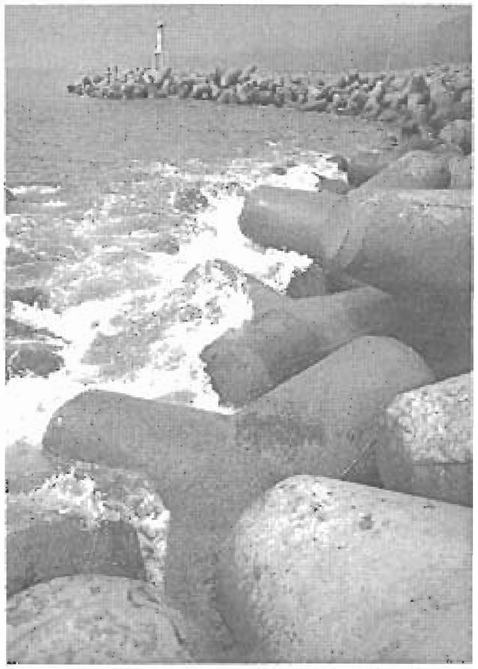
$S_{s\,concrete} = 2.4$ $S_{s\,rock} \approx 2.65$

S_r has to be corrected for local salinity

$S_r = \frac{S_s}{\left(1 + \frac{34}{1000}\right)}$

Design swl = NSWL (swl) + surge + tide + seasonal + wind z...

$7.63 \frac{N}{m^3} = 7.63 \frac{kg \cdot m}{m^3 s^2} \left(\frac{1}{9.81 \frac{m}{s^2}}\right) = \frac{kg}{m^3}$



Coastal and Ocean Structures I

SPM Classification

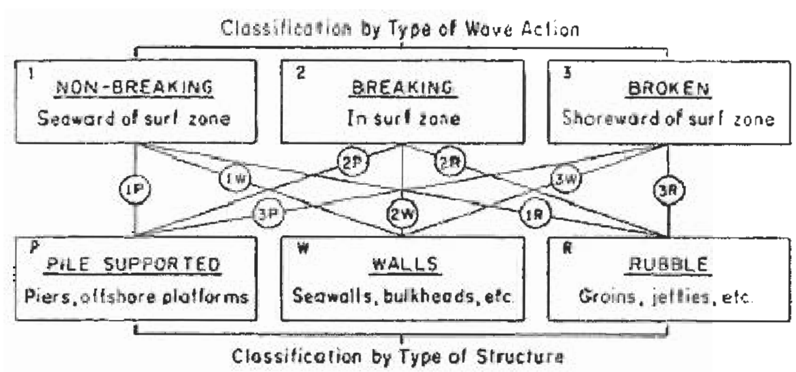
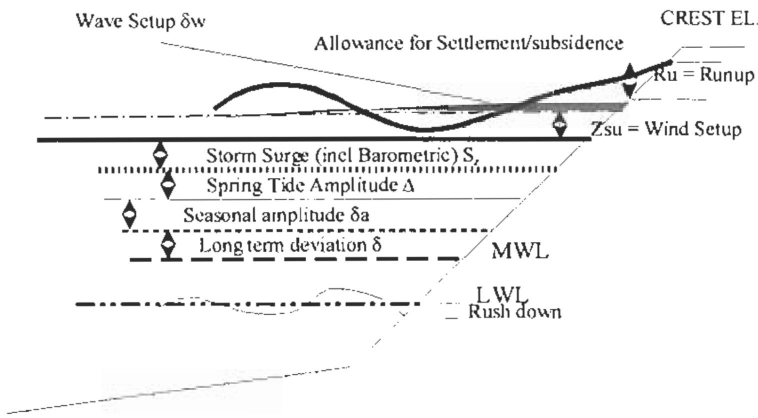


Figure 7-66. Classification of wave force problems by type of wave action and by structure type.

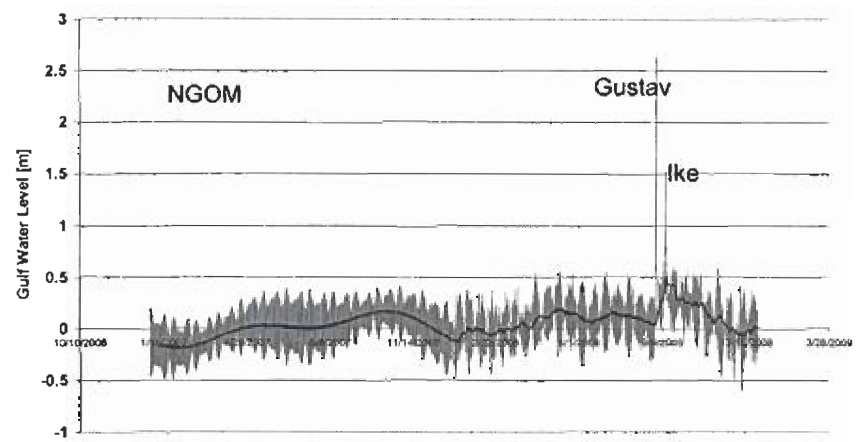
Shoreline Water Level



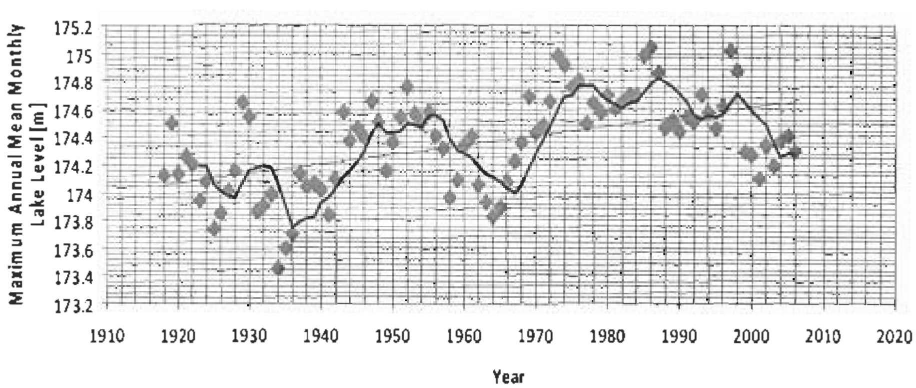
Inputs to Water Level Determination

- Long term mean
- Seasonal variation
- Long term change
- Tide
- Storm Surge
- Wind setup
- Wave setup
- Runup
- Rush down
- Minimum water level
- RSLR

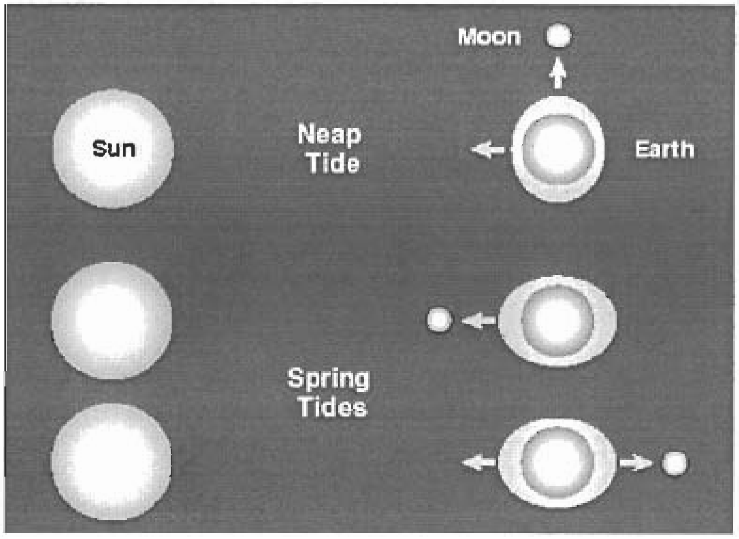
Short & Seasonal WLs



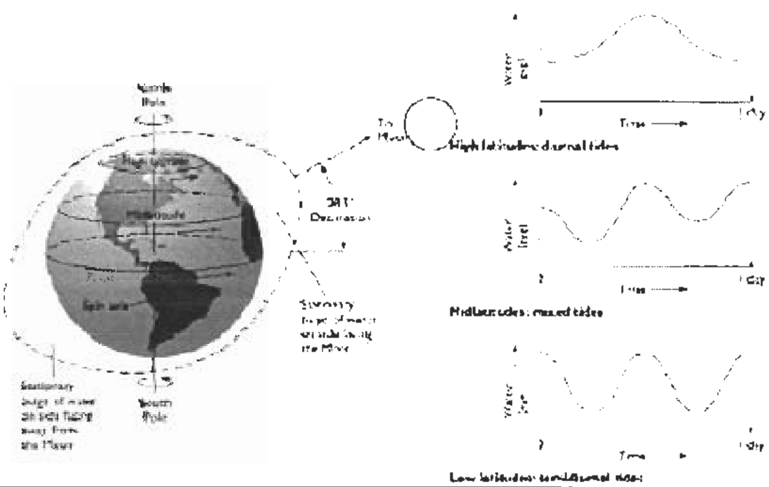
Long Term Trends



Tides



Cause: The tilt of the earth's axis relative to the orbital plane of the moon



56 / THE SEA

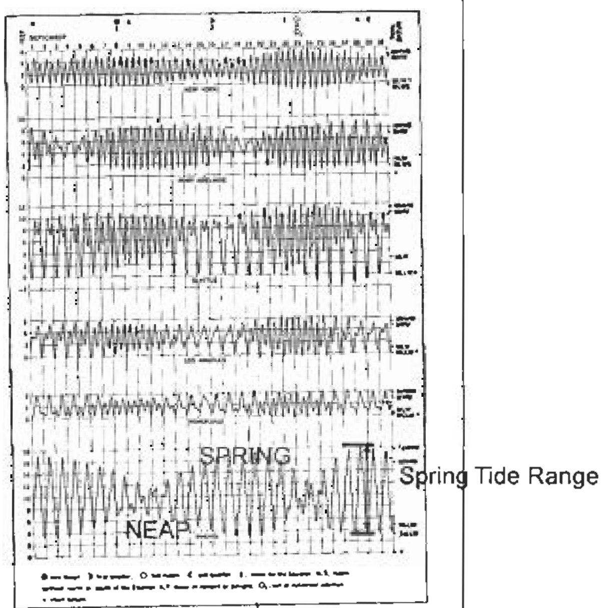


Figure 5–6 *Tidal variations at various places during a month.*

54 / THE SEA

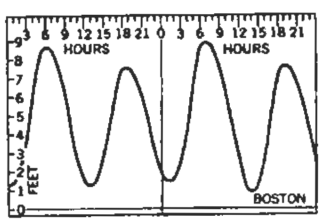


Figure 5–3 *Semidiurnal type of tide.*

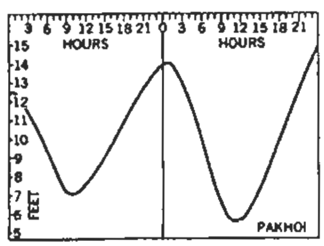


Figure 5–4 *Diurnal type of tide.*

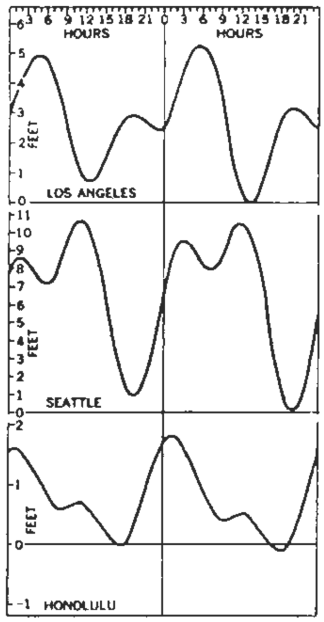


Figure 5–5 *Mixed types of tide.*

Storm Surge

$$h_{ss} \approx K_s K_r \left[\Delta p + \frac{C_R C_f R U_R^{\,2}}{2 g d_{ref}} \frac{\rho_a}{\rho_w} \right]$$

$$C_R \approx 1 \rightarrow 2$$

$$\frac{\rho_a}{\rho_w} \approx 1/800$$

$$C_f \approx 0.0025$$

$$d_{ref} \sim 10 \rightarrow 20m$$

Wind Setup

$$Z_{\sup} = \frac{F'(\rho_a C_f U_{OW}^{\ 2})}{2g\rho_w d}$$

$$C_f \sim 0.001 \rightarrow\ 0.003$$

$$Z_{\sup} = 0.0325\{U_{ow\ ,kph}\}^{\ 2}\{F_{km}\}/d_m$$

$$Setup\_m$$

$$Z_{\sup} = \{U_{ow\ ,mph}\}^2\{F_{,miles}\}/(1400 d_{ft})$$

$$Setup\_ft$$

Wave Setdown

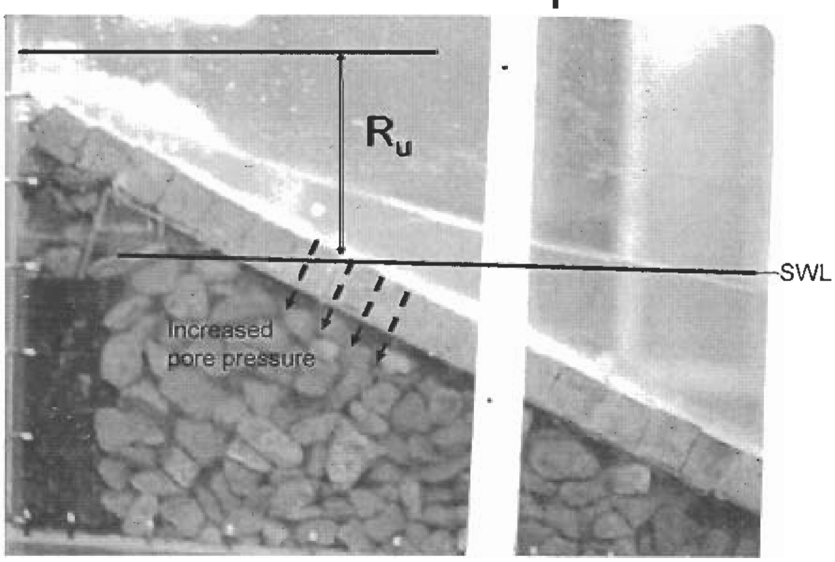
$$S_d = -\frac{\sqrt{g}H_o^{\ 2}T}{64\pi d_b^{\ 3/2}}$$

Wave Setup

- $S_w = 0.19\ H_b\left[1 - 2.82\sqrt{\dfrac{H_b}{gT^2}}\right]$

Only needed if the waves break

Wave Runup



Estimation of Runup

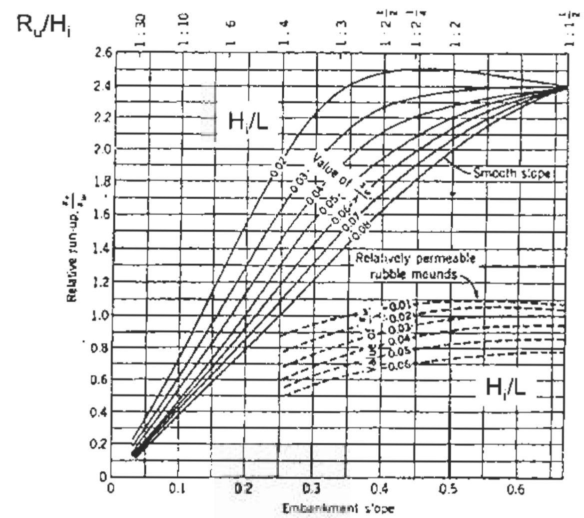
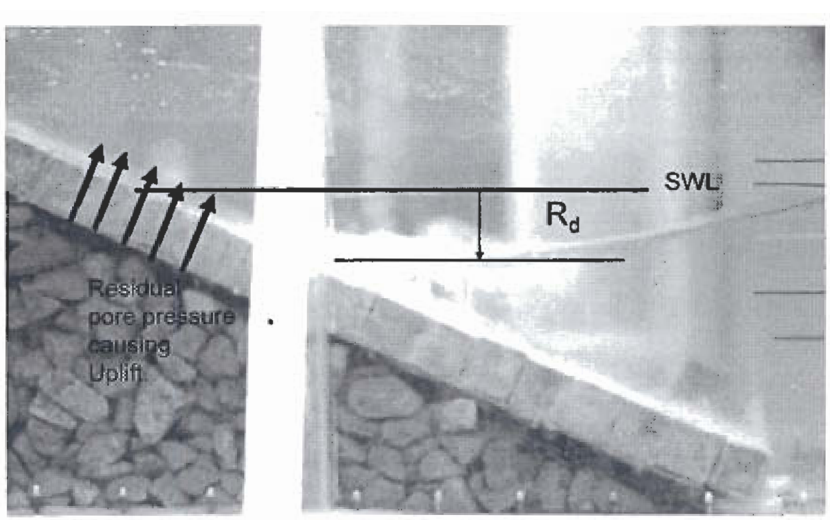
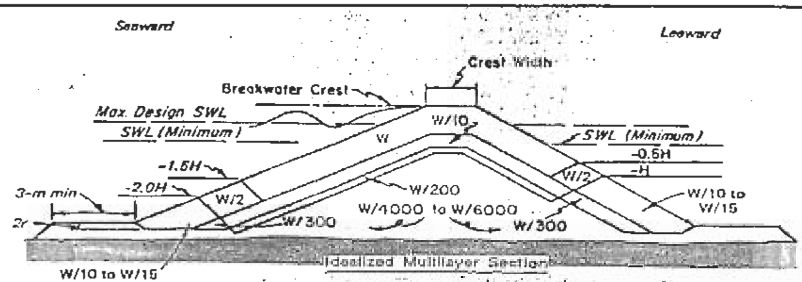


FIG. 7-13 Wave run-up ratios vs. wave steepness and embankment slopes. (From Saville, McClendon, and Cochran)

Wave Rush Down

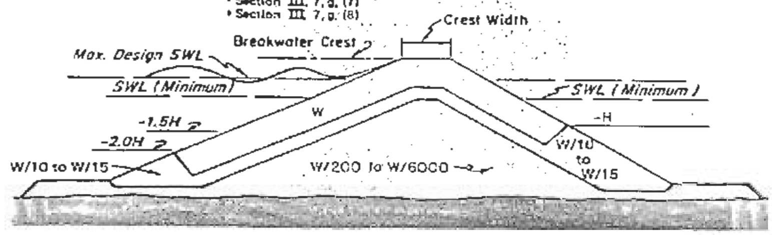




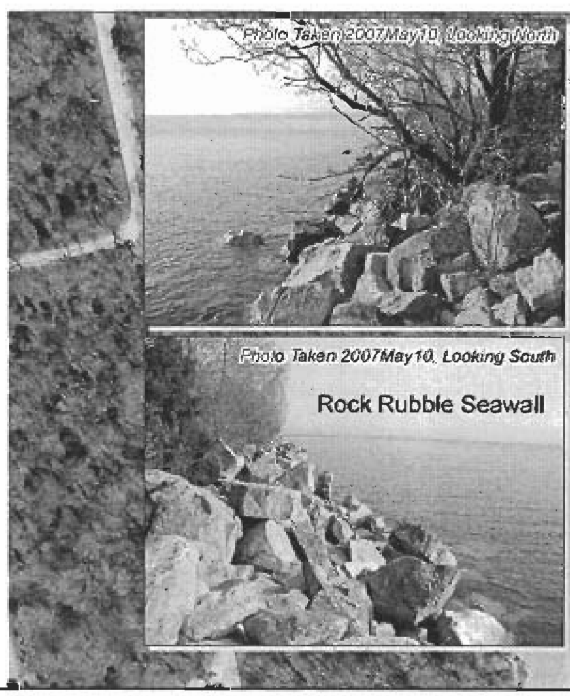
Rock Size	Layer	Rock Size Gradation (%)
W	Primary Cover Layer[1]	125 to 75
W/2 and W/15	Secondary Cover Layer[2]	125 to 75
W/10 and W/300	First Underlayer[3]	130 to 70
W/200	Second Underlayer	150 to 50
W/4000-W/6000	Core and Bedding Layer	170 to 30

H = Wave Height
W = Weight of individual Armor Unit
r = Average Layer Thickness

For concrete armor: [1] Sections III, 7, g, (1), (2) and (6)
[2] Section III, 7, g, (7)
[3] Section III, 7, g, (8)

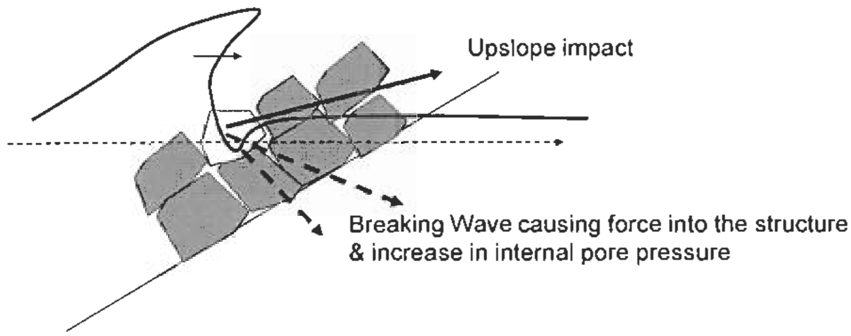


Recommended Three-layer Section



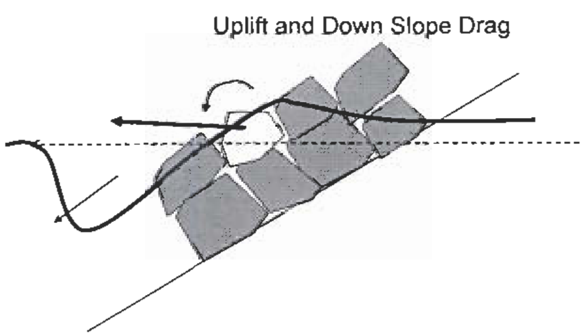
FORCES ON ARMOUR UNITS

WAVE ATTACH and RUNUP PHASE



FORCES ON ARMOUR UNITS

WAVE RECESSION and RUSH DOWN PHASE



HUDSON EQUATION

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

$$\gamma_r = Dry\_Specific\_Weight\_of\_Armour\_Unit$$

$$K_D = Stability\_Coefficient(Damage\_Coefficient)$$

$$\theta = Slope\_of\_face$$

$$S_r = \frac{Specific\_gravity\_Armour\_Unit}{Local\_Specific\_Gravity\_of\_Water}$$

$$H = Design\_wave\_height$$

Design H

Design wave height is usually takes as the

significant wave Height, Hs

This leads to some damage 0-5% for the design storm since waves can exceed Hs

$$H_s = \sqrt{2} H_{rms}$$

$$H_{10} = 1.27 H_s$$

$$H_1 = 1.67 H_s$$

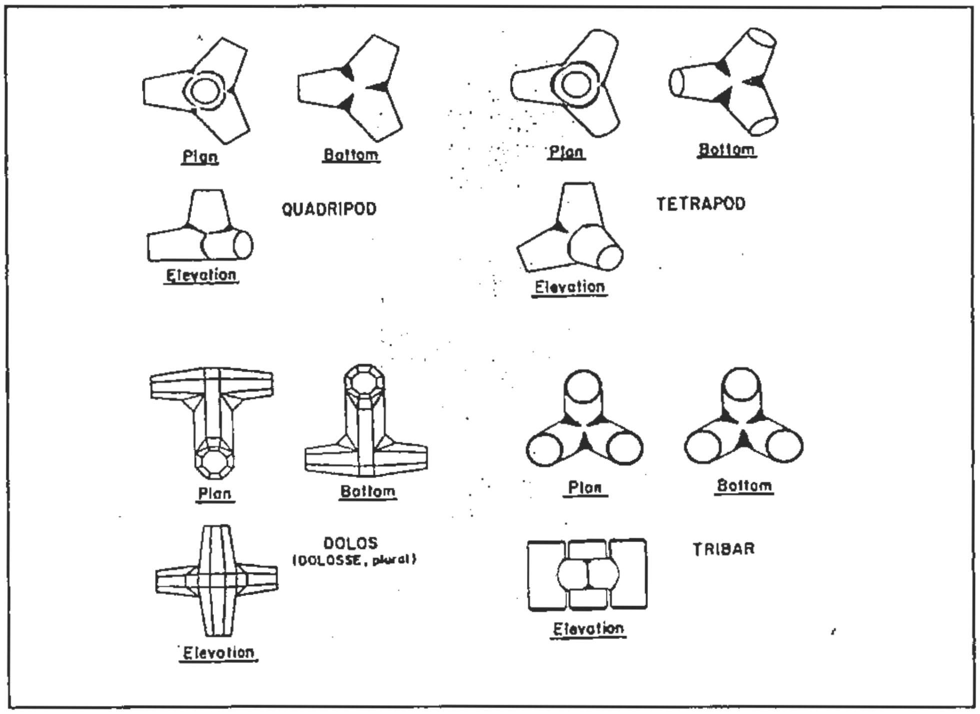


Table 7-8. Suggested K_D Values for use in determining armor unit weight[1].

Armor Units	n[3]	Placement	Structure Trunk K_D[2] Breaking Wave	Structure Trunk K_D[2] Nonbreaking Wave	Structure Head K_D Breaking Wave	Structure Head K_D Nonbreaking Wave	Structure Head Slope Cot θ
No-Damage Criteria and Minor Overtopping							
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 1.6 1.3	3.2 2.8 2.3	1.5 2.0 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 and Quadripod	2	Random	7.0	8.0	5.0 4.5 3.5	6.0 5.5 4.0	1.5 2.0 3.0
Tribar	2	Random	9.0	10.0	8.3 7.8 6.0	9.0 8.5 6.5	1.5 2.0 3.0
Dolos	2	Random	15.8[8]	31.8[8]	8.0 7.0	16.0 14.0	2.0[9] 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.

SPM 1984

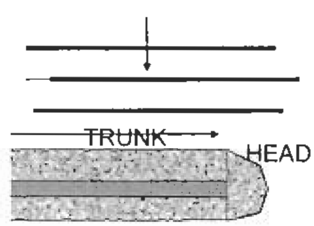


Table 7-8. Suggested K_D Values for use in determining armor unit weight[1].

SPM (1984)

No-Damage Criteria and Minor Overtopping

Armor Units	n[3]	Placement	Structure Trunk K_D[2] Breaking Wave	Structure Trunk K_D[2] Nonbreaking Wave	Structure Head K_D Breaking Wave	Structure Head K_D Nonbreaking Wave	Slope Cot θ
Quarrystone							
Smooth rounded	2	Random	1.2	2.4	1.1	1.9	1.5 to 3.0[5]
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]
Smooth–rough round		sp.		3.7?			
Rough angular	2	Random	2.0	4.0	1.9 1.6 1.3	3.2 2.8 2.3	1.5 2.0 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 and Quadripod	2	Random	7.0	8.0	5.0 4.5 3.5	6.0 5.5 4.0	1.5 2.0 3.0
Tribar	2	Random	9.0	10.0	8.3 7.8 6.0	9.0 8.5 6.5	1.5 2.0 3.0
Dolos	2	Random	15.8[8]	31.8[8]	8.0 7.0	16.0 14.0	2.0[9] 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	—	—	

Table 7-6. Suggested K_D Values for Use in Determining Armor Unit Weight

SPM(1973)

No-Damage Criteria and Minor Overtopping

Armor Units	n *	Placement	Structure Trunk K_D § Breaking wave	Structure Trunk K_D § Nonbreaking wave	Structure Head K_D Breaking wave	Structure Head K_D Nonbreaking wave	Slope cot θ
Quarrystone							
Smooth rounded	2	random	2.1	2.4	1.7	1.9	1.5 to 3.0
Smooth rounded	>3	random	2.8	3.2	2.1	2.3	‖
Rough angular	1	random †	†	2.9	†	2.3	‖
Rough angular	2	random	3.5	4.0	2.9 2.5 2.0	3.2 2.8 2.3	1.5 2.0 3.0
Rough angular	>3	random	3.9	4.5	3.7	4.2	‖
Rough angular	2	special ‡	4.8	5.5	3.5	4.5	‖
Tetrapod and Quadripod	2	random	7.2	8.3	5.9 5.5 4.0	6.6 6.1 4.4	1.5 2.0 3.0
Tribar	2	random	9.0	10.4	8.3 7.8 7.0	9.0 8.5 7.7	1.5 2.0 3.0
Dolos	2	random	22.0 ¶	25.0 ¶	15.0 13.5	16.5 15.0	2.0 £ 3.0
Modified Cube	2	random	6.8	7.8	——	5.0	‖
Hexapod	2	random	8.2	9.5	5.0	7.0	‖
Tribar	1	uniform	12.0	15.0	7.5	9.5	‖
Quarrystone (K_{RR}) Graded angular	–	random	2.2	2.5			

Example Problem 1

- Given: Hs = 2 m; T= 7 s
- Slope: 1:2
- Armour Rock (Rough Angular)
- 2 layers; random
- Ss= 2.65
- Salinity = 6 ppt
- Breaking waves
- Find the W for the *HEAD* and *TRUNK for a rubble mound breakwater.*

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

SPM (1984)

No-Damage Criteria and Minor Overtopping

Armor Units	n[3]	Placement	Structure Trunk K_D[2] Breaking Wave	Structure Trunk K_D[2] Nonbreaking Wave	Structure Head K_D Breaking Wave	Structure Head K_D Nonbreaking Wave	Slope 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
smooth-rough rounds		sp.		3.75			
Rough angular	2	Random	2.0	4.0	1.9 1.6 1.3	3.2 2.8 2.3	1.5 2.0 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 and Quadripod	2	Random	7.0	8.0	5.0 4.5 3.5	6.0 5.5 4.0	1.5 2.0 3.0
Tribar	2	Random	9.0	10.0	8.3 7.8 6.0	9.0 8.5 6.5	1.5 2.0 3.0
Dolos	2	Random	15.8[8]	31.8[8]	8.0 7.0	16.0 14.0	2.0[9] 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

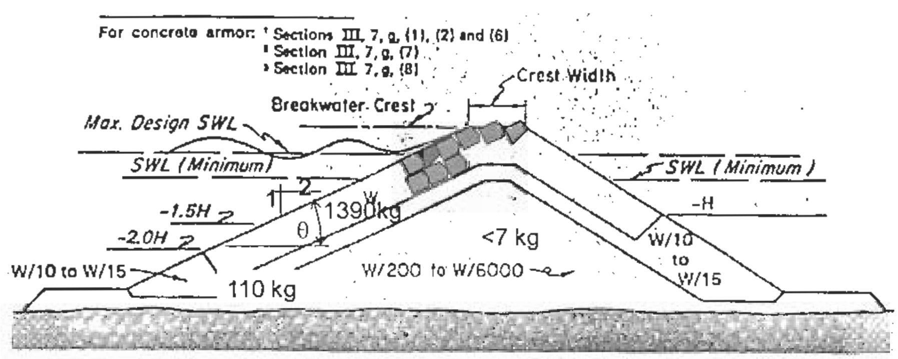
Solution

- Ss = 2.65*/[1+Sal/1000)]=2.4/1.006
- Sr=2.634
- γ_r=Ss*1000*9.81=9.81*2650 N/m3
- $\gamma_{r\,mass}$ =2650 kg/m3
- Hs = 2 m
- Cotθ = 2
- Trunk, Breaking K_D = 2 SPM(1984)
- W = 1390 kg
- Head, Breaking K_D = 1.8 SPM(1984)
- W=1541 kg

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

Typical X-section

SPM 1984



Recommended Three-layer Section

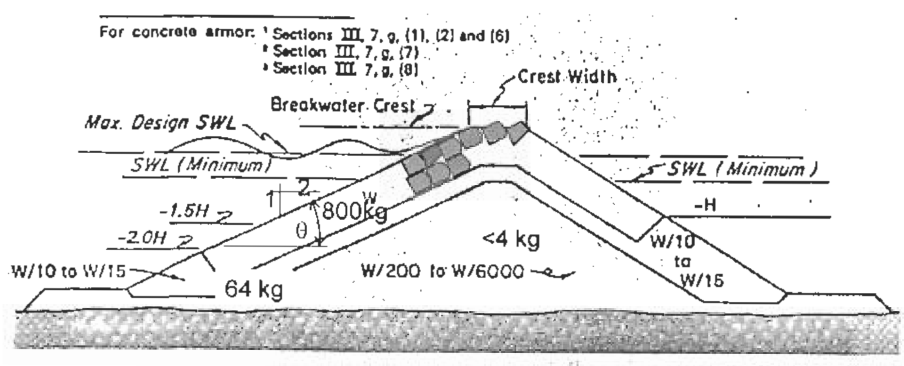
Solution

- Ss = 2.65*/[1+Sal/1000)]=2.4/1.006
- Sr=2.634
- □ γ_r=Ss*1000*9.81=9.81*2650 N/m3
- □ $\gamma_{r\ mass}$ =2650 kg/m3
- Hs = 2 m
- Cotθ = 2
- Trunk, Breaking K_D = 2.5 SPM(1973)
- W = 793 kg ~ 800 kg
- Head, Breaking K_D = 2.5 SPM(1973)
- W=1110 kg

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

Typical X-section

SPM 1973



Recommended Three-layer Section

Example Problem 2

- Check the design of the Port Sines Breakwater
- Hs = 11 m; T= 13.5 s
- Slope: 2:3→ cotθ=1.5
- 42-t Dolos Armour Units
- Seawater Salinity ~34 ppt
- d_s ~35 to 50 m
- Freshwater Ss= 2.4
- Check Non-Breaking & Breaking waves
- Find the W for the *TRUNK for a rubble mound breakwater.*

Note: the Port Sines Breakwater Failed in 1978. To check the original design we are using the SPM 1973 KD values.

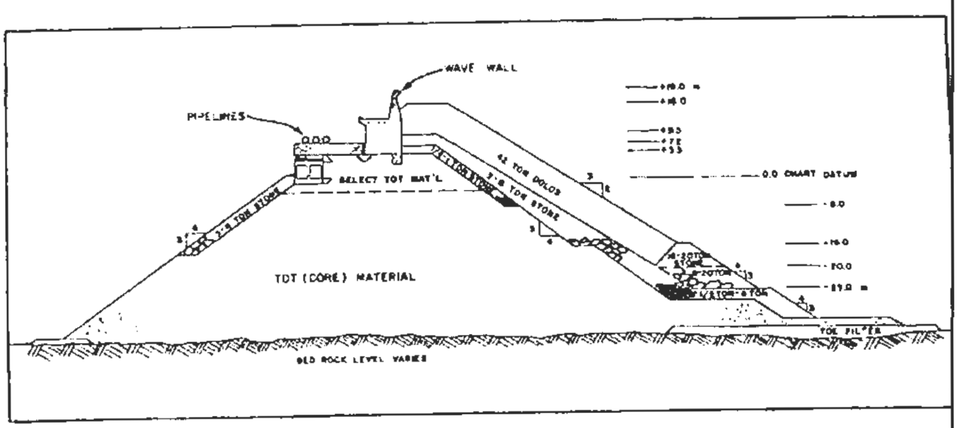


Table 7-6. Suggested K_D Values for Use in Determining Armor Unit Weight

SPM(1973) No-Damage Criteria and Minor Overtopping

Armor Units	n *	Placement	Structure Trunk K_D § Breaking wave	Structure Trunk K_D § Nonbreaking wave	Structure Head K_D Breaking wave	Structure Head K_D Nonbreaking wave	Structure Head Slope cot θ
Quarrystone							
Smooth rounded	2	random	2.1	2.4	1.7	1.9	1.5 to 3.0
Smooth rounded	>3	random	2.8	3.2	2.1	2.3	‖
Rough angular	1	random †	†	2.9	†	2.3	‖
Rough angular	2	random	3.5	4.0	2.9	3.2	1.5
					2.5	2.8	2.0
					2.0	2.3	3.0
Rough angular	>3	random	3.9	4.5	3.7	4.2	‖
Rough angular	2	special ‡	4.8	5.5	3.5	4.5	‖
Tetrapod and Quadripod	2	random	7.2	8.3	5.9	6.6	1.5
					5.5	6.1	2.0
					4.0	4.4	3.0
Tribar	2	random	9.0	10.4	8.3	9.0	1.5
					7.8	8.5	2.0
					7.0	7.7	3.0
Dolos	2	random	22.0 ¶	25.0 ¶	15.0	16.5	2.0 £

Solution

- Ss@34ppt = Ss*/[1+Sal/1000)]=2.4/1.034
- Sr=2.325
- $\gamma_{r\,mass}$=Ss*1000=2400 kg/m3
- Hs = 11 m
- Cotθ = 3/2=1.5
- Breaking K_D = 22 *Note the KD is from SPM 1973 since this what was available when the design was completed. SPM 1973*
- W = 42-t (as shown in the as-built drawing)
- Non-Breaking K_D = 25 *SPM 1973*
- W=37-t *SPM 1973*
- SPM 1984 Breaking K_D = 15.8 → W~59-t
- SPM 1984 non-Breaking K_D = 31.8 → W~29-t

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

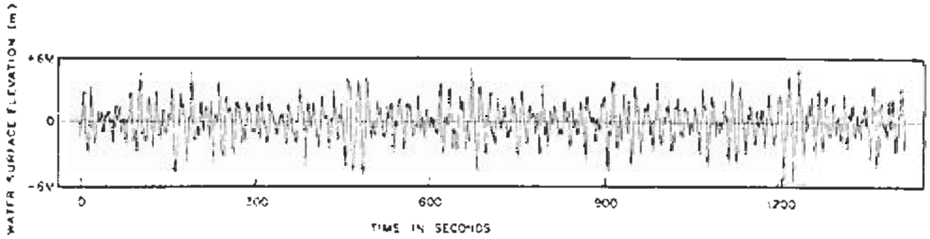


FIGURE 6.3 SAMPLE OF WAVE RECORD FROM CABO DA ROCA ON FEBRUARY 26, 1978

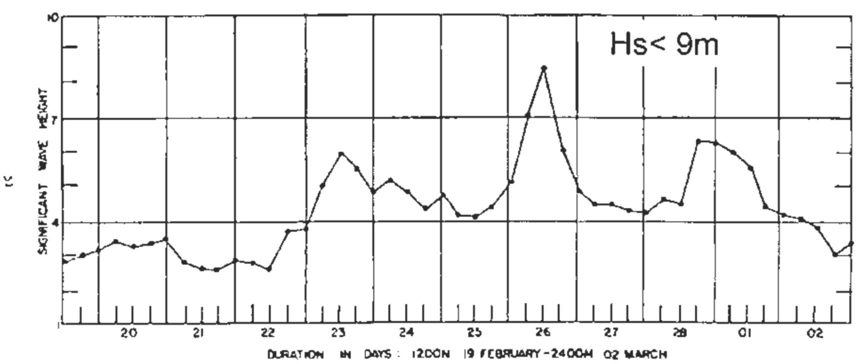


FIGURE 6.2 SYNTHESIS OF WAVE RECORD AT SINES FROM RECORDED DATA AT CABO DA ROCA (MANSARD AND PLOEG, 1978)

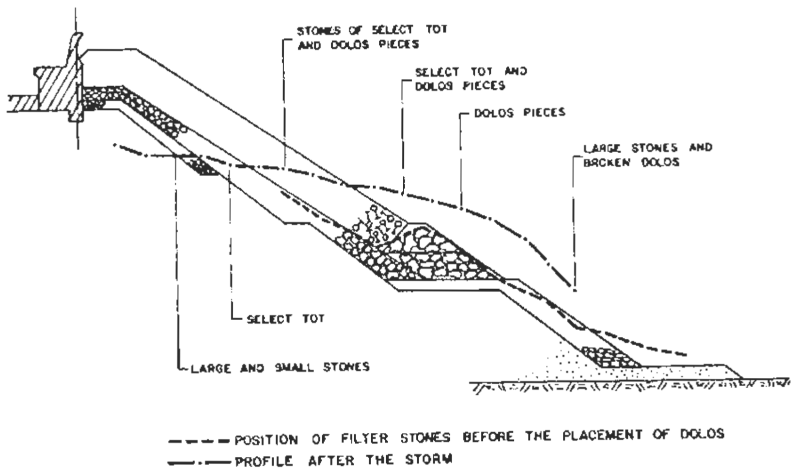
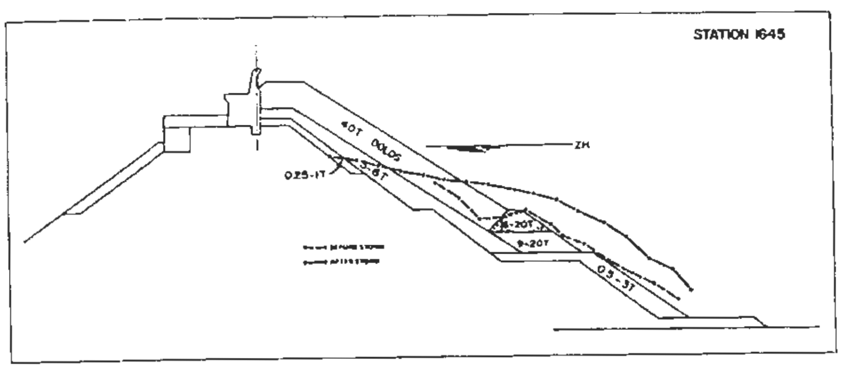
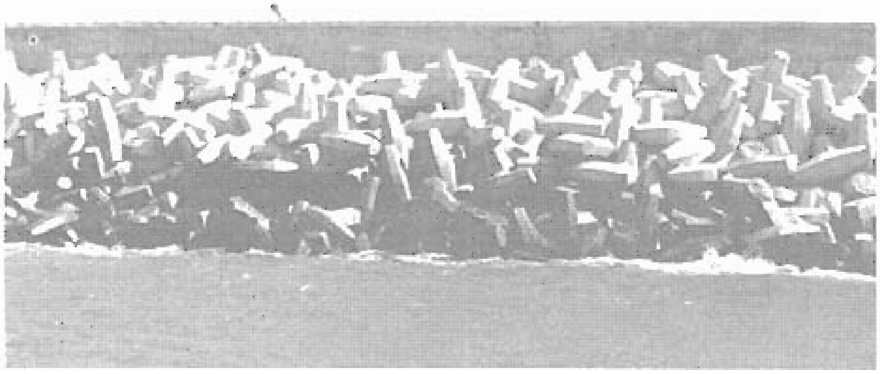


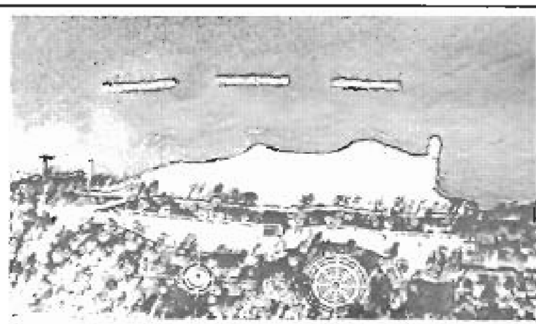
FIGURE 7.1 A TYPICAL CROSS-SECTION AS OBSERVED BY DIVERS

Failure Profile

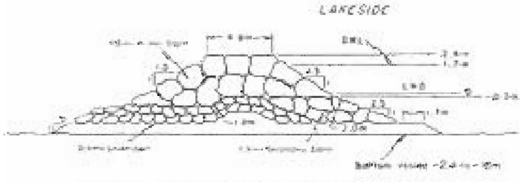


Failure – Why?

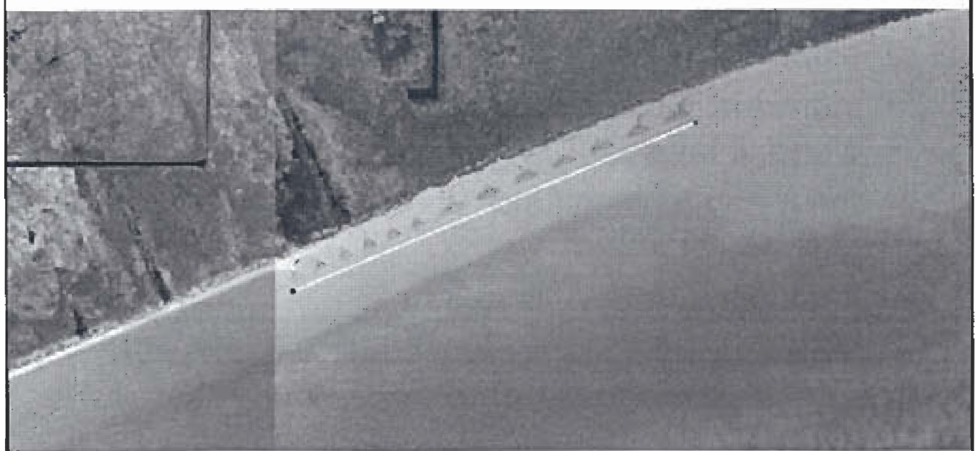




SEGMENTED BREAKWATER



SEGMENTED BREAKWATER



Assignment Problem 9.2:
Design a rubble mound breakwater for the significant wave and setup near the south shore of Lake Pontchartrain for a 1 hour wind out of the north over the Lake.

Design Period 50 years. RSLR including subsidence rate 0.9 cm/year.

Design for no overtopping at the design Hs.

Assume: salinity ~ 6 ppt; Ss =2 .65; slope 1:2
individually placed rough angular rock armour stone.

Coastal Structures II

Seawalls
Overtopping,
Minikin Force

Overtopped Seawall

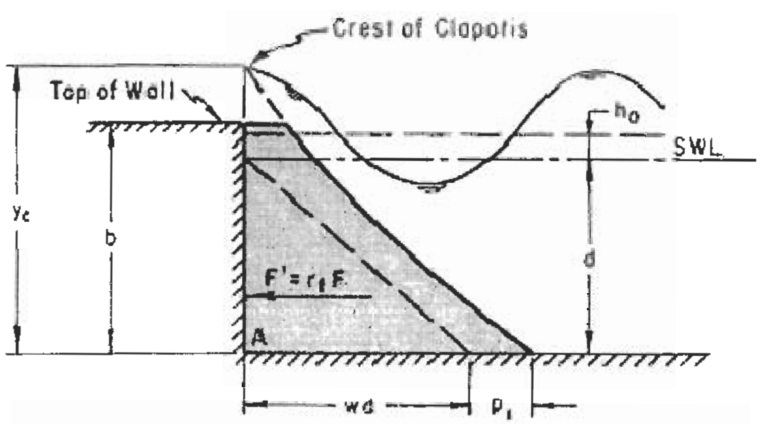
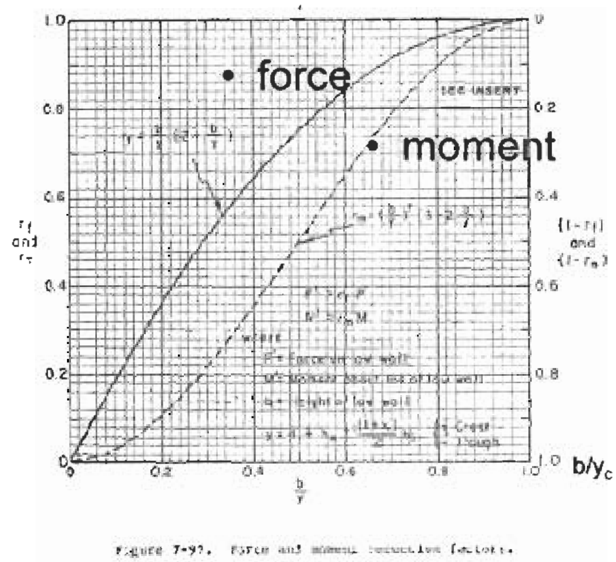


Figure 7-96. Pressure distribution on wall of low height.

Effect of Overtopping

$F'_{wave} = F_{wave} f_f$



Overtopping Flow

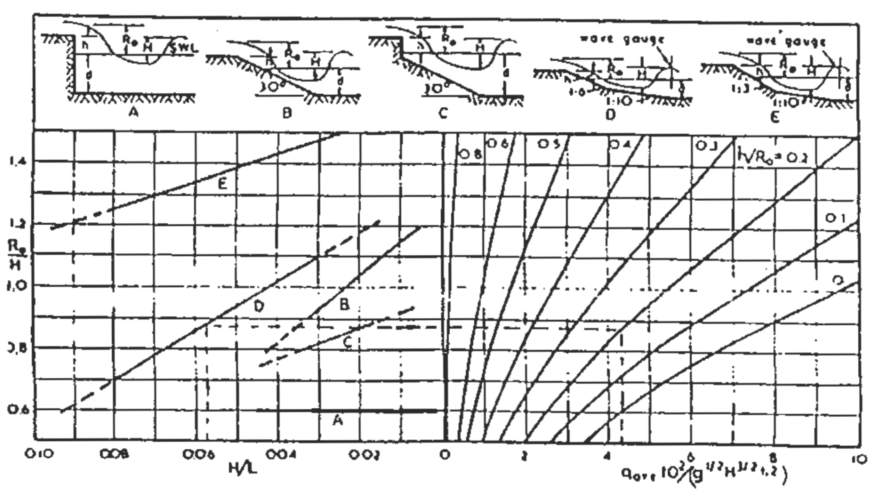
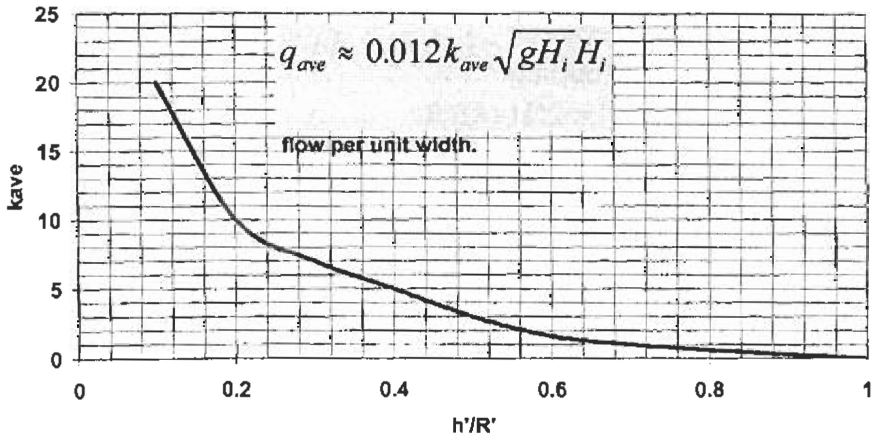


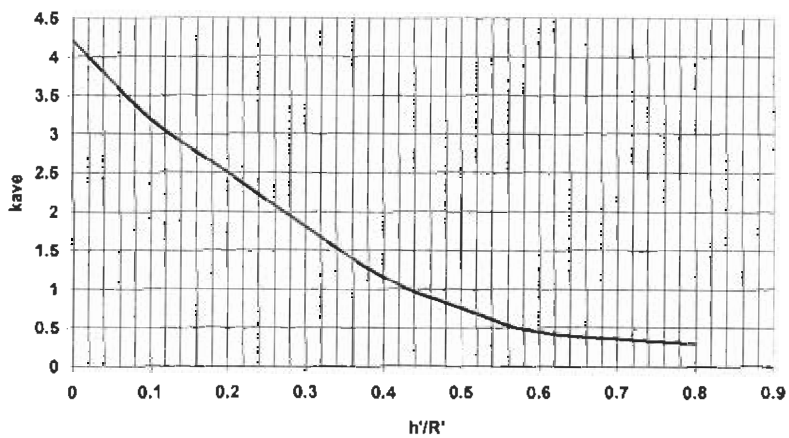
Fig. 7-24. Average overtopping discharge q_{ave} per unit length of walls illustrated.

Figure 10-1 kave for z=3
From Silvester
h' = Crest-SWL
R' = Runup for no overtopping



$$q_{ave} \approx 0.012 k_{ave} \sqrt{gH_i} H_i$$

Vertical seawall

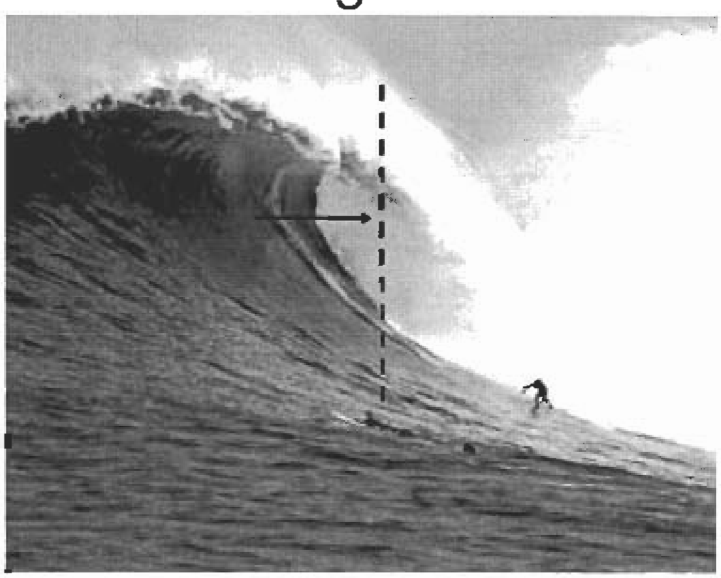


Name________________________________

Overtopping Example

- Given: d_s = 20 ft; H_i=8 ft; T= 7sec; h = 9 ft;
- ~3H:1V slope
- Find: qave overtopping
- Solution: Ru = R' = 2*H_i=16 ft
- h/R'=9/16 = 0.56
- Kave=2.
- qave=2.0*0.012*sqrt(32.2*8)*8~3 cfs/ft
- --------------------
- *For the same wave and a vertical wall with R'=11.6 ft*
- *qave=(0.25), (0.5), (1), (2), (3) Units____________*

Breaking Waves



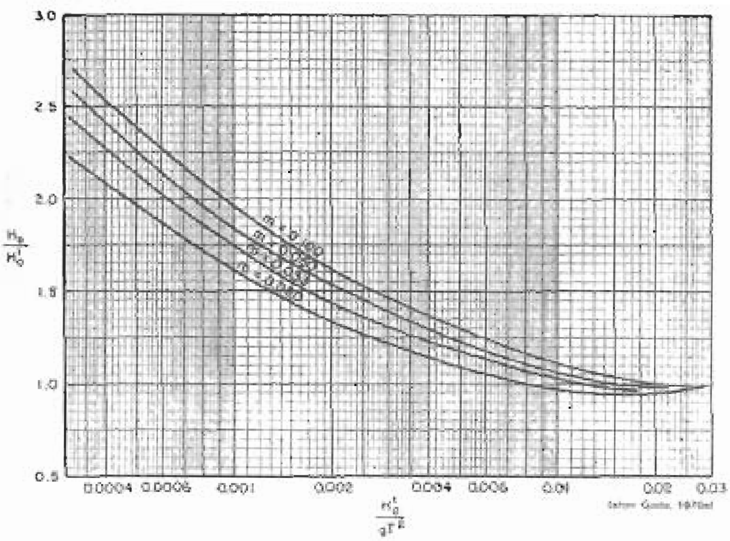


Figure 7-3. Breaker height index H_b/H_o' versus deepwater wave steepness H_o'/gT^2.

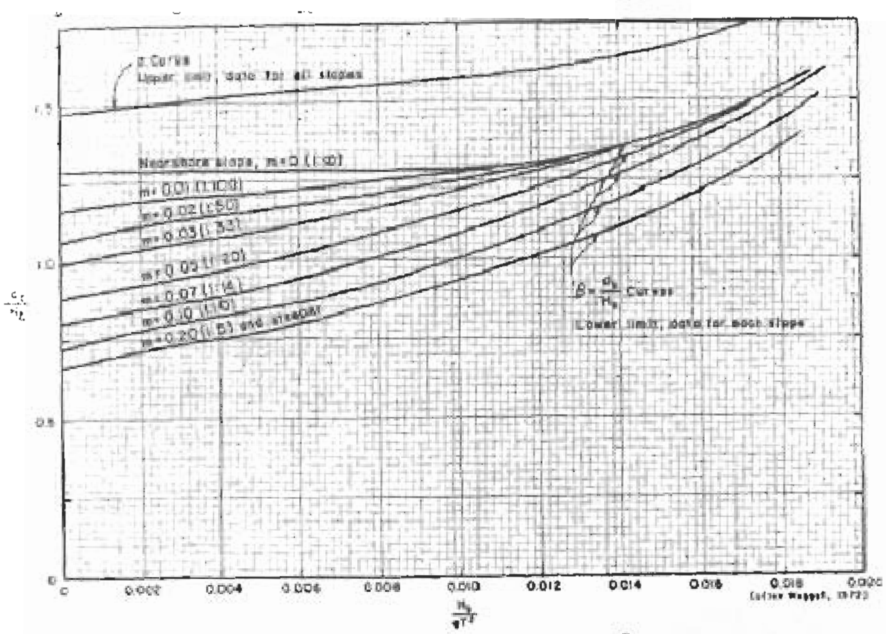


Figure 7-2. α and β versus H_b/gT^2.

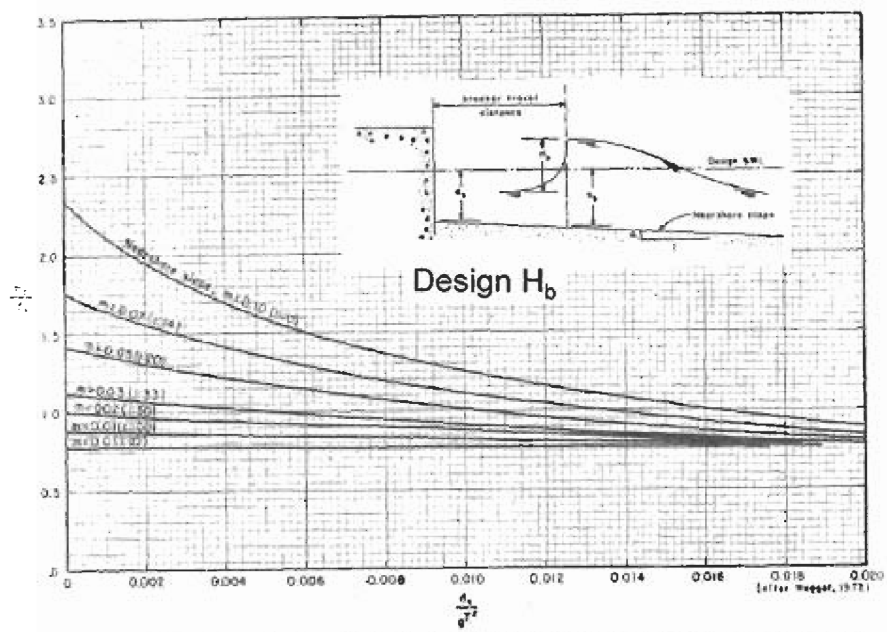


Figure 7-4. Dimensionless design breaker height versus relative depth at structure.

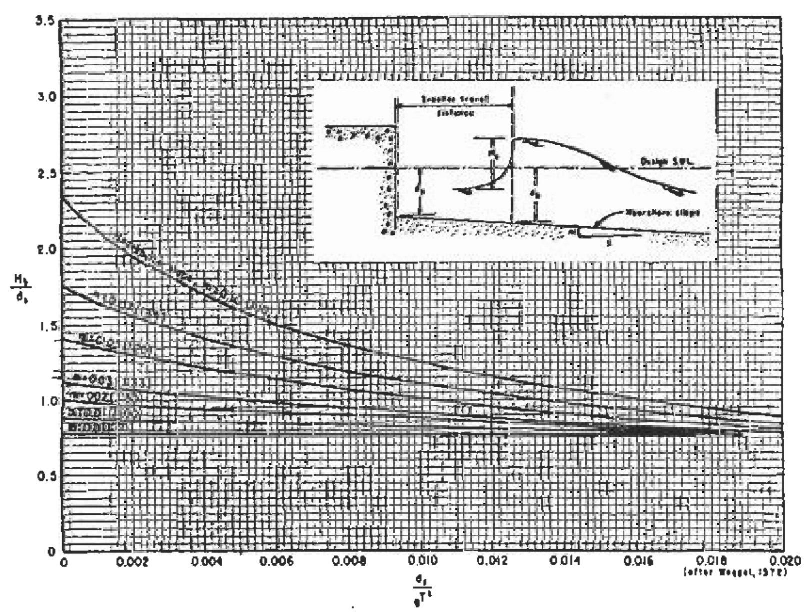


Figure 7-4. Dimensionless design breaker height versus relative depth at structure.

Minikin Force

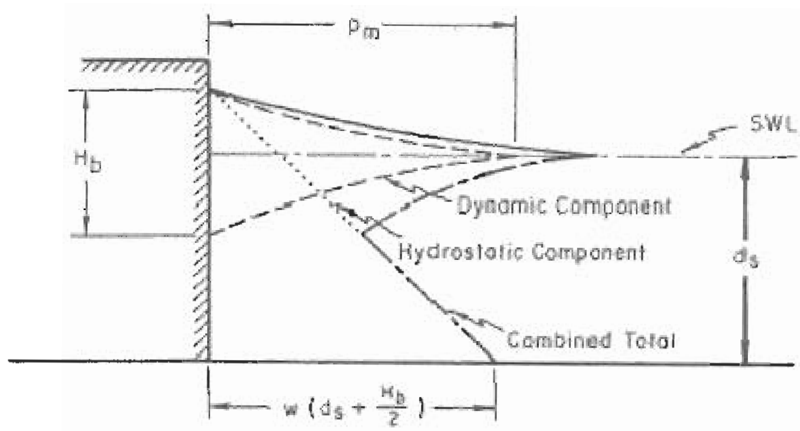


Figure 7-99. Minikin wave pressure diagram.

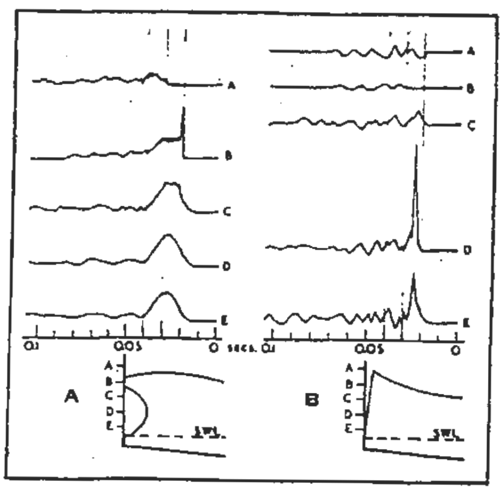


Fig. 8-11. Concurrent recordings of pressure at various heights of a vertical wall with breaking waves [18].

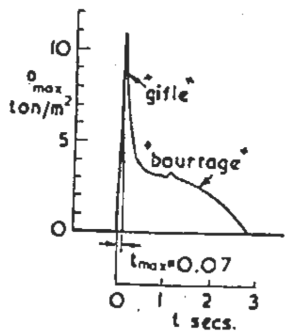
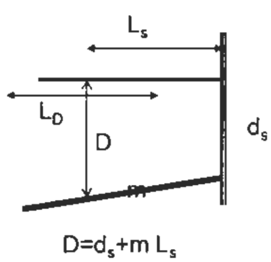


Fig. 8-10. Most typical shock-type wave measured at Hoboro harbour [16].

Minikin Pressure, Force and Moment

$$p_m = 101\gamma \frac{H_b d_s}{L_D D}[d_s + D]$$



$$R_m = \frac{p_m H_b}{3}$$

$$M_m = \frac{p_m H_b d_s}{3}$$

Minikin Pressure
& Force

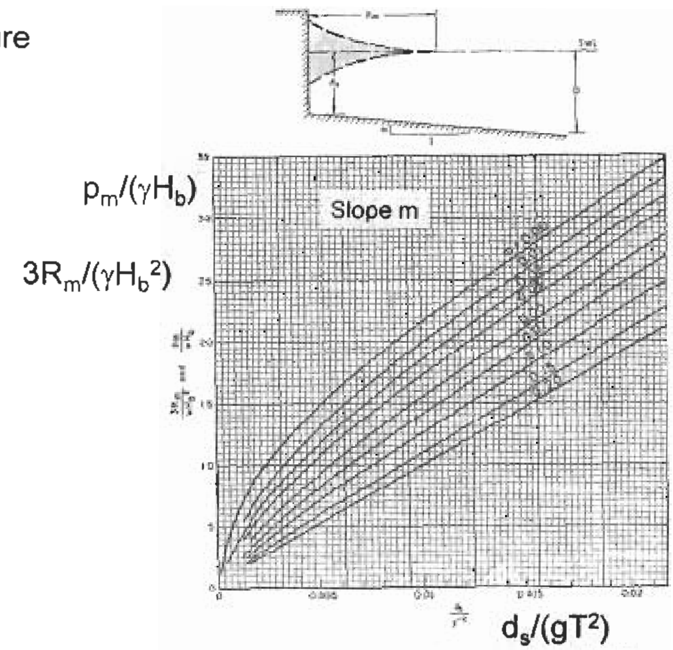


Figure 7-100. Dimensionless Minikin wave pressure and force.

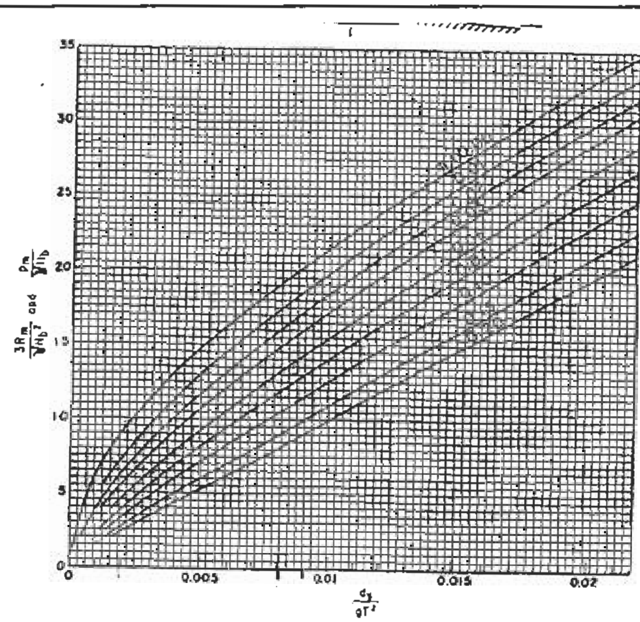


Figure 7-100. Dimensionless Minikin wave pressure and force.

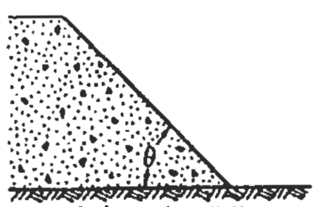
Total Force from Breaking Waves

$$R_t = R_m + \gamma \frac{(d_s + \frac{1}{2} H_b)^2}{2}$$

$$M_t = M_m + \gamma \frac{(d_s + \frac{1}{2} H_b)^2}{6}$$

Slope Reduction

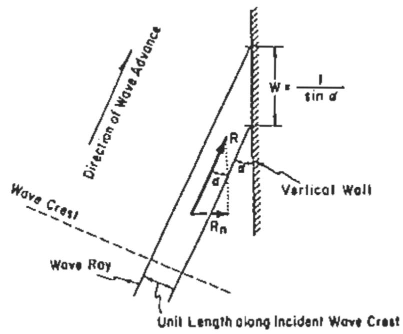
$$R_m = \frac{p_m H_b}{3} \sin^2 \theta$$



(a) Sloping Wall

Oblique Attack

$$R_m = \frac{p_m H_b}{3} \sin^2 \alpha$$



Example

- Given: Ho' <= 8.6 ft; d_s=10 ft; m=0.03; T = 7 sec
- Find: p_m ; R_m ; M_m ; R_t ; M_t

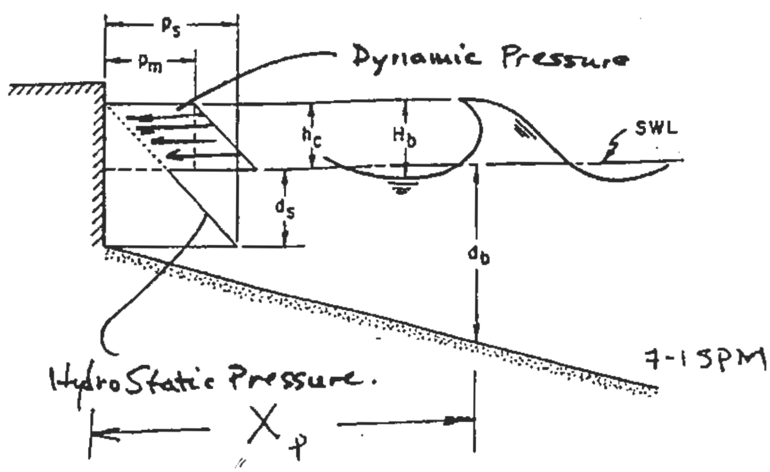
Calculations:

Ho'	**<=**	**8.6 ft**			
ds		**10 ft**			
T		**7 sec**			
m		**0.03**			
Step 1					
Find	Hb for given ds				
ds/gT2	0.0063				
m	0.03				
Fig 7-4					
Hb/ds	0.96				
Hb	9.6 ft				
Step 2					
Find Hb for given Ho' max					
Fig 7-3					
Ho'/gT2	0.0055				
Hb/Ho'	1.13				
Hb	9.718 ft	>9.6	ft	Minikin exists	
Therefore use Hb		9.6 ft			
Step 3: Force Cal					
pm					
ds/gT2	0.0083 Fig 7-100				
pm/Hb	12.5				
pm	**120 ft**	7488 lbs/ft2	52 psi		
Rm	**23962 lbs/ft**				
Mm	**239616 ft-lbs/ft**				
Rt	**=Rm+R**	**30796 lbs/ft**			
Mt	**=Mm+M**	**273331 ft-lbs/ft**			

Coastal Structures II
Broken waves

Seawalls

Broken Waves



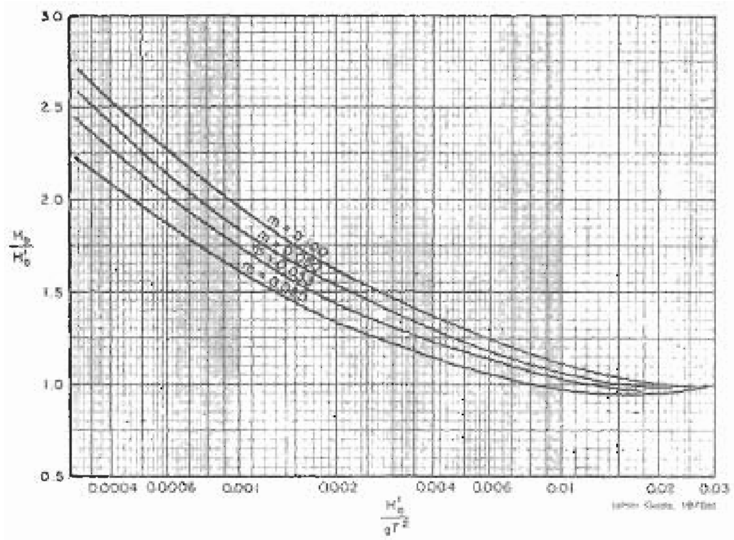


Figure 7-3. Breaker height index H_b/H_o' versus deepwater wave steepness H_o'/gT^2.

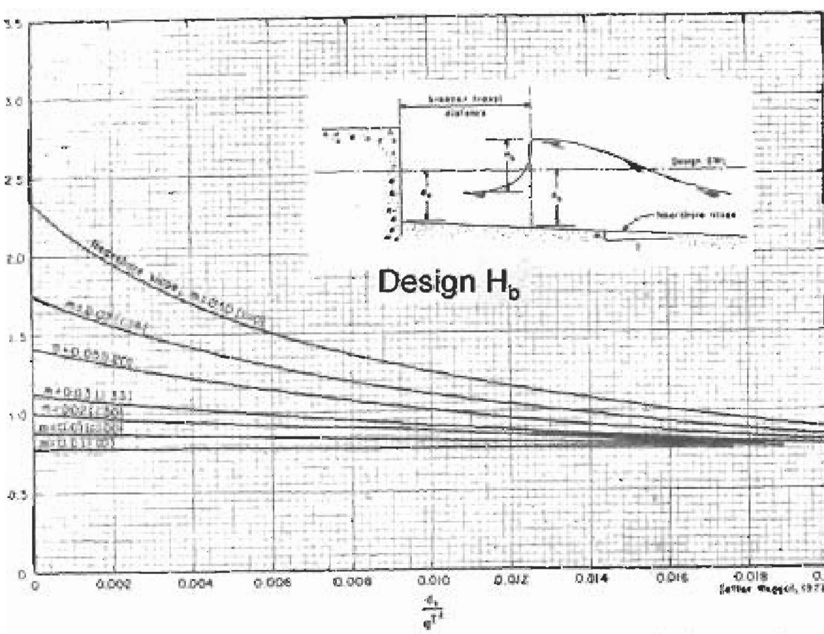


Figure 7-4. Dimensionless design breaker height versus relative depth at structure.

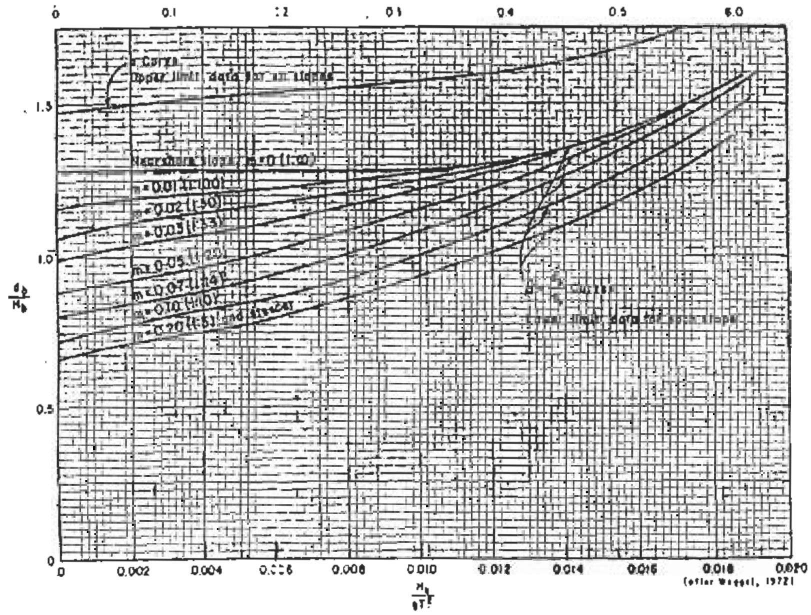


Figure 7-2. α and β versus H/gT^2.

Estimation of H_b at X_p

$$H_b = K_\beta (d_s + m X_p) \le H_{b\,max}$$

X_p = Plunge distance = $(4 - 9.25\,m)\,H_b$

$K_\beta = H_b / d_b \sim 0.78$ *for small m.*

$$H_b = \frac{d_s}{[1/K_\beta - m(4 - 9.25\,m)]}$$

$$X_p = (4 - 9.25\,m)\,H_b$$

$$d_b = d_s + m X_p$$

Wave Pressure, Force and Moment due to impact of a broken wave

$$p_m \sim \gamma c_b^{\,2} / 2g = \gamma d_b / 2$$

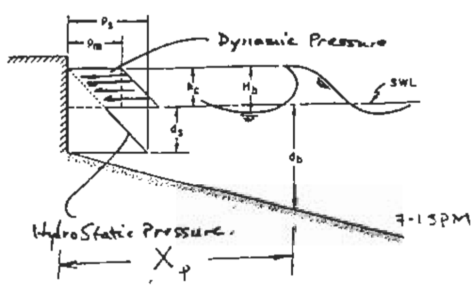
$$h_c = K_\beta H_b \; \textit{above SWL}$$

$$R_m = \gamma d_b h_c / 2$$

$$M_m = \gamma d_b h_c (d_s + h_c / 2) / 2$$

Total Force and Moment

- Add hydrostatic component
- $R_T = R_m + R_s$
- $\boldsymbol{R_s = \gamma (d_s + h_c)^2/2}$
- $M_T = M_m + M_s$
- $M_s = R_s \boldsymbol{(d_s + h_c)/3}$



Example

- Given: Ho'max < 10.94 ft; d_s=10 ft; m=0.03;

T = 7 sec

- Find: Broken:Hb, Xp, db, p_m ; R_m ; M_m ; R_t ; M_t
- Step 1. Est. Hb at Xp assume Kb based on Fig 7-4
- Step 2. Est. Xp
- Step 3. Cal. db
- Step 4. Est. Req'd Ho' max for Hb, T and m Fig 7-3; Ho'max = Hb/Ksshould be < Ho' actual
- Step 5. Cal. p_m ; R_m ; M_m ; R_t ; M_t

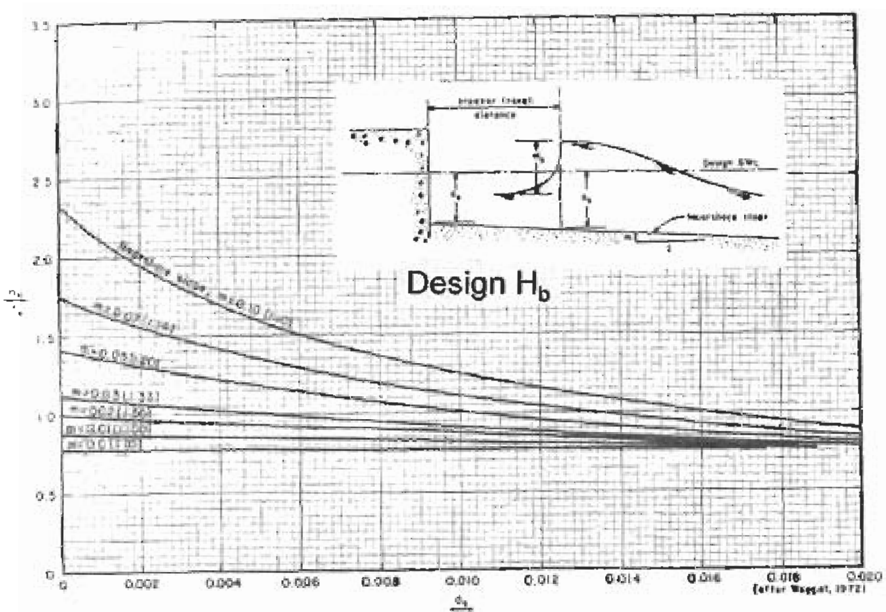


Figure 7-4. Dimensionless design breaker height versus relative depth at structure.

Solution

m	0.03			
Ho max	10.94	ft	Lo=251	32.2*7^2/(2*3.1415)
T	7	s		
ds	10			
ds/gT2	0.0064	Hbminikin= 0.95*10=9.5	Kb ~	0.95
Hb=10/[1/0.95-.03(4-9.25*0.03)]	10.6	ft	at Xp	
Xp=(4-9.25m)Hb	39.6	ft		
db=ds+Xp*m	11.19			
db/gT2	0.0071	ok	Kb~	0.95
Appendix I	ds/Lo	10/251=0.04	Ks	1.064
Ho max given	10.94	ft	Req'd	+10.6/1.064=9.96
Ho max/gT2	0.0061			ok

Forces and Moments

p_m	349	lbs/ft2
h_c	10.6	ft
R_m	3700	lbs/ft
M_m	56600	ft lbs/ft

Ocean and Coastal Structures III

Forces on Piles

Slender

Large diameter

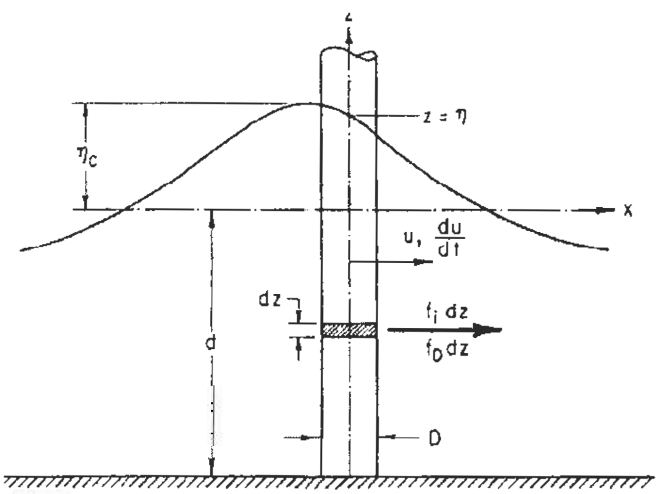


Figure 7-67. Definition sketch of wave forces on a vertical cylinder.

Slenderness Criterion $\frac{D}{L_A} < 0.05$

Morrison Equation

$$f = f_i + f_D = C_M \rho \frac{\pi D^2}{4} \frac{du}{dt} + C_D \frac{1}{2} \rho D u|u|$$

- f_i: INERTIA
- f_D: DRAG
- C_M: INERTIA Or VIRTUAL MASS COEF
- C_D: DRAG COEF

f_i = inertial force per unit length of pile

f_D = drag force per unit length of pile

ρ = density of fluid (1025 kilograms per cubic meter for sea water)

D = diameter of pile

u = horizontal water particle velocity at the axis of the pile (calculated as if the pile were not there)

$\frac{du}{dt}$ = total horizontal water particle acceleration at the axis of the pile, (calculated as if the pile were not there)

C_D = hydrodynamic force coefficient, the "drag" coefficient

C_M = hydrodynamic force coefficient, the "inertia" or "mass" coefficient

L_A=Linear Wave Length

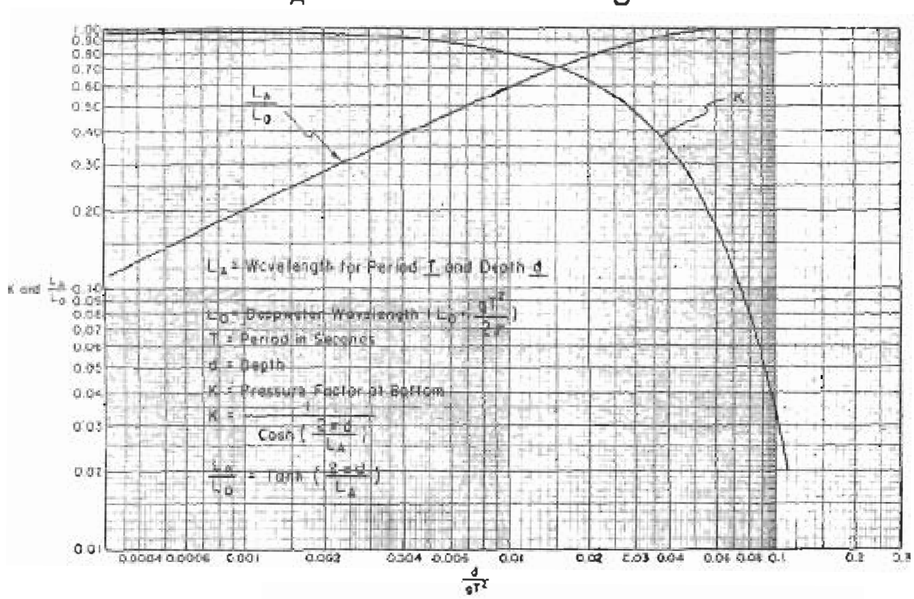


Figure 7-68. Relative wavelength and pressure factor versus d/gT^2 (Airy wave theory).

Crest Amplitude

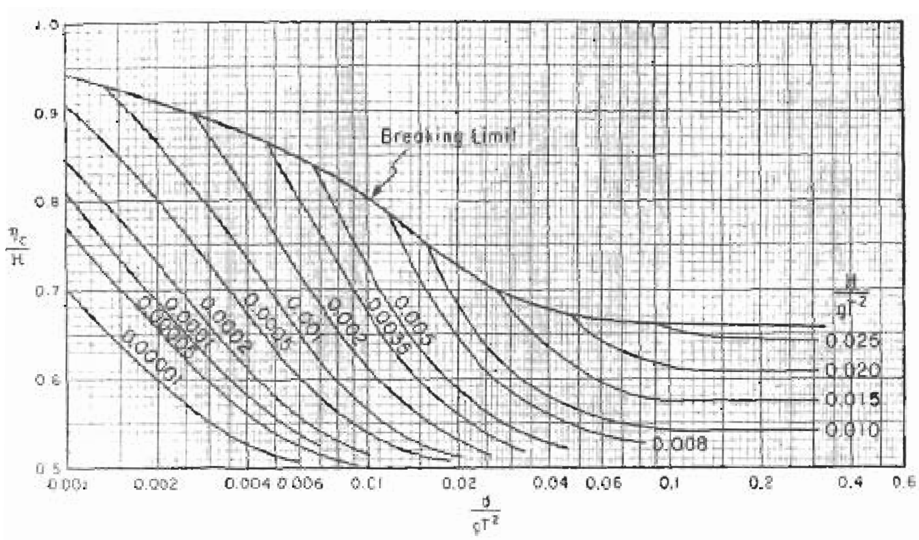


Figure 7-69. Ratio of crest elevation above still-water level to wave height.

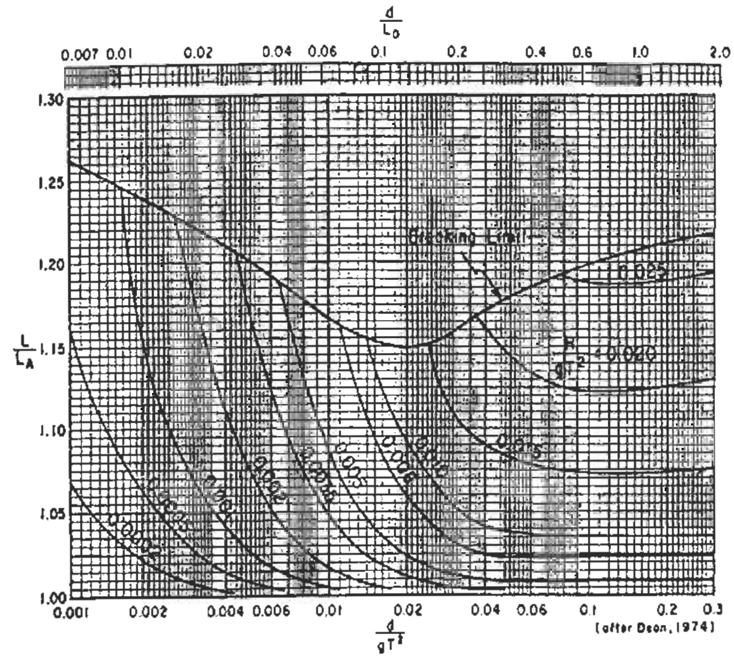


Figure 7-70. Wavelength correction factor for finite amplitude effects.

Non-linear L

Check on Slenderless

- Ho' = 6 ft; T=8 sec; d=18 ft; D=2 ft
- Does this classify as a slender pile?
- Lo = _328_______
- L_A = Lo tanh($2\pi d/L_A$) = 181.5 ft______
- D/ L_A = 2/181.5=0.011
- Slender x Yes No
- NAME ____________________

Horizontal orbital velocity, acceleration and forces per unit length—Airy theory

$$u = \frac{H}{2}\frac{gT}{L}\frac{\cosh[2\pi(z+d)/L]}{\cosh[2\pi d/L]}\cos\left(\frac{2\pi t}{T}\right) \quad (7\text{-}23)$$

$$\frac{du}{dt} = \frac{\partial u}{\partial t} = \frac{g\pi H}{L}\frac{\cosh[2\pi(z+d)/L]}{\cosh[2\pi d/L]}\sin\left(-\frac{2\pi t}{T}\right) \quad (7\text{-}24)$$

Introducing these expressions into equation (7-20) gives

$$f_i = C_M\,\rho g\,\frac{\pi D^2}{4}\,H\left\{\frac{\pi}{L}\frac{\cosh[2\pi(z+d)/L]}{\cosh[2\pi d/L]}\right\}\sin\left(-\frac{2\pi t}{T}\right) \quad (7\text{-}25)$$

$$f_D = C_D\,\frac{1}{2}\,\rho g\,D\,H^2\left\{\frac{gT^2}{4L^2}\left\{\frac{\cosh[2\pi(z+d)/L]}{\cosh[2\pi d/L]}\right\}^2\right\}\cos\left(\frac{2\pi t}{T}\right)\left|\cos\left(\frac{2\pi t}{T}\right)\right| \quad (7\text{-}26)$$

Integration from bed to water surface

$$F = \int_{-d}^{\eta} f_i\,dz + \int_{-d}^{\eta} f_D\,dz = F_i + F_D \quad (7\text{-}27)$$

$$M = \int_{-d}^{\eta}(z+d)\,f_i\,dz + \int_{-d}^{\eta}(z+d)\,f_D\,dz = M_i + M_D \quad (7\text{-}28)$$

Forces and Moments (Airy)

$$F_i = C_M \rho g \frac{\pi D^2}{4} H K_i \qquad (7\text{-}29)$$

$$F_D = C_D \frac{1}{2} \rho g D H^2 K_D \qquad (7\text{-}30)$$

$$M_i = C_M \rho g \frac{\pi D^2}{4} H K_i d S_i = F_i d S_i \qquad (7\text{-}31)$$

$$M_D = C_D \frac{1}{2} \rho g D H^2 K_D d S_D = F_D d S_D \qquad (7\text{-}32)$$

Forces and Moments Integration Coefficients (Airy)

$$K_i = \frac{1}{2} \tanh\left(\frac{2\pi d}{L}\right) \sin\left(-\frac{2\pi t}{T}\right) \qquad (7\text{-}33)$$

$$K_D = \frac{1}{8}\left(1 + \frac{4\pi d/L}{\sinh[4\pi d/L]}\right) \left|\cos\left(\frac{2\pi t}{T}\right)\right| \cos\left(\frac{2\pi t}{T}\right) \qquad (7\text{-}34)$$

$$= \frac{1}{8} n \left|\cos\left(\frac{2\pi t}{T}\right)\right| \cos\left(\frac{2\pi t}{T}\right)$$

$$S_i = 1 + \frac{1 - \cosh[2\pi d/L]}{(2\pi d/L)\sinh[2\pi d/L]} \qquad (7\text{-}35)$$

$$S_D = \frac{1}{2} + \frac{1}{2n}\left(\frac{1}{2} + \frac{1 - \cosh[4\pi d/L]}{(4\pi d/L)\sinh[4\pi d/L]}\right) \qquad (7\text{-}36)$$

Forces and Moments Integration Coefficients (non-Linear Fig 7-71 & 7-72)

$$F_{im} = C_M \rho g \frac{\pi D^2}{4} H K_{im} \qquad (7\text{-}37)$$

$$F_{Dm} = C_D \frac{1}{2} \rho g D H^2 K_{Dm} \qquad (7\text{-}38)$$

$$M_{im} = F_{im} d S_i \qquad (7\text{-}39)$$

$$M_{D_m} = F_{Dm} d S_D \qquad (7\text{-}40)$$

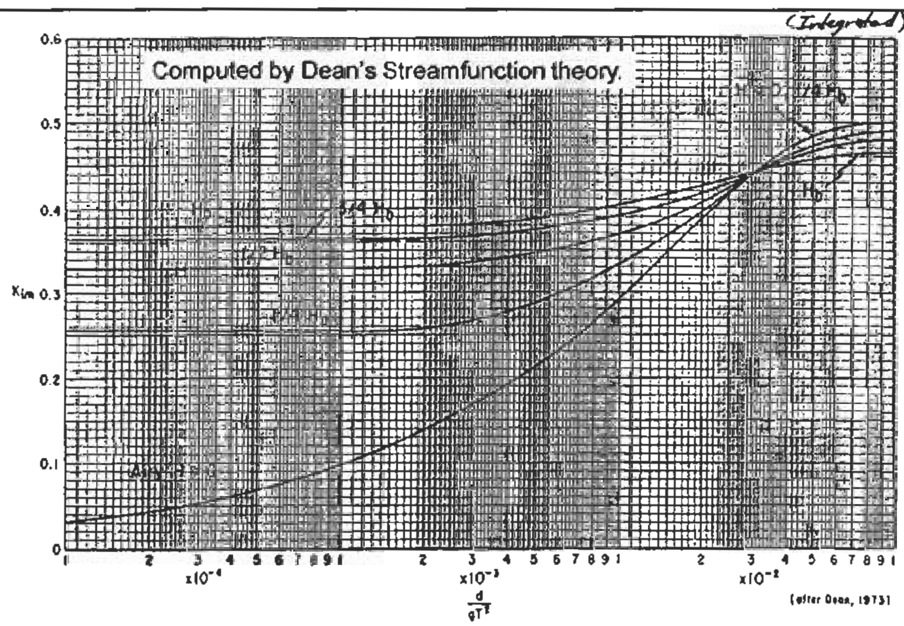


Figure 7-71. K_{im} versus relative depth, d/gT^2.

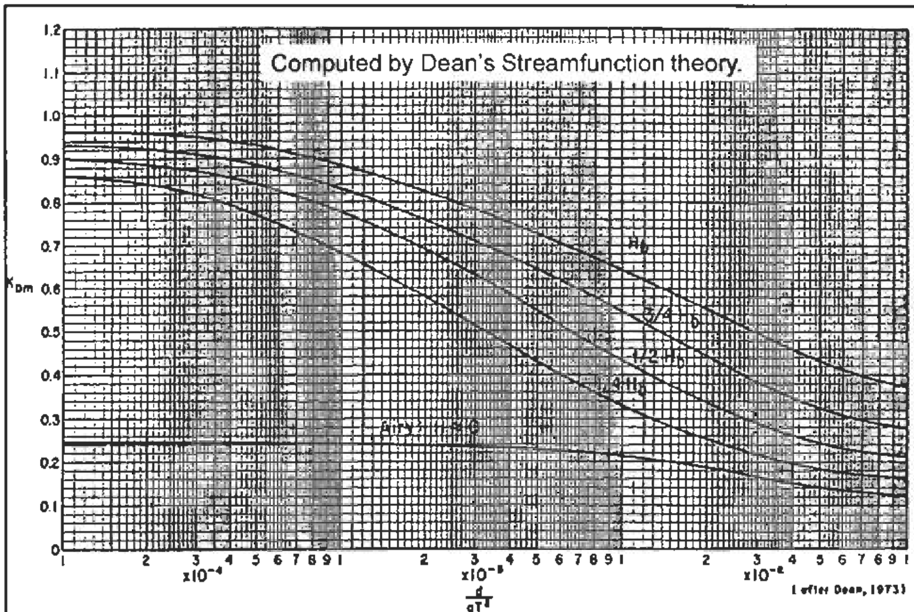


Figure 7-72. K_{Dm} versus relative depth, d/gT^2.

Definitions

- Reynolds Number=
  Re = UD/ν
- Keulegan-Carpenter Number N_{KC}=UT/D
- Strouhal Number
  $S_t = f_o D/U$
- U = maximum horizontal orbital velocity =umax
- f_o = shedding frequency= S_t U/D

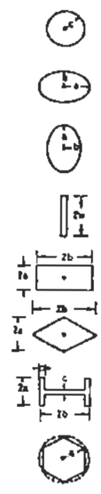
$$u_{\max} \cong \frac{\pi H L_o}{T L_A}$$

D=2ft; T=8s; Ho=6ft; d=18ft
Hi=KsHo=1.007*6 ft~6.04ft

- $\nu=10^{-5}$ ft2/sec
- Re=UmaxD/ν
- Umax~4.26 ft/sec
- Re=4.26*2/ν
- ~850,000
- K_{kc}=4.26*8/2~17

d/gT2	0.008734	
H/gT2	0.002911	
W	0.714286	
0.25	0.34	0.285
FR	833.4144	lbs
Lo	328.1523	181.4971
LA	181.4975	0.000347
umax	4.257918	

Sarpkaya / Isaacson



REVIEW OF THE FUNDAMENTAL EQUATIONS AND CONCEPTS 47

APPENDIX A

Table 2.3 Added Masses of Various Bodies

SHAPE	ADDED MASS PER UNIT LENGTH ← MOTION				
CIRCLE	$\rho\pi c^2$				
ELLIPSE	$\rho\pi b^2$				
ELLIPSE	$\rho\pi a^2$				
PLATE	$\rho\pi a^2$				
RECTANGLE	a/b		a/b		
	∞	1.00 $\rho\pi a^2$	1	1.51 $\rho\pi a^2$	
	10	1.14 "	0.5	1.70 "	
	5	1.21 "	0.2	1.98 "	
	2	1.36 "	0.1	2.23 "	(Wendel 1950)
DIAMOND	2	0.85 "			
	1	0.76 "			
	0.5	0.67 "			
	0.2	0.61 "	(Wendel 1950)		
I-BEAM a/c = 2.0 b/c = 3.4		2.11 $\rho\pi a^2$	(Patton 1965)		
REGULAR POLYGON	n = 3	0.654 $\rho\pi a^2$			
	4	0.787 "			
	5	0.823 "			
	6	0.867 "			
OR	∞	1.000 "	(Wendel 1950)		

n SIDED

C_m =Added Mass/(Displaced Mass)

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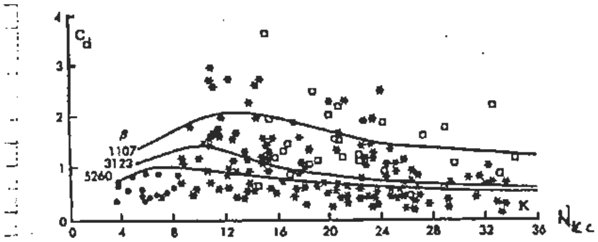


Fig. 5.2. Drag coefficients from field data, as compiled by Wiegel et al. (1957). Solid lines from Sarpkaya (1976a).

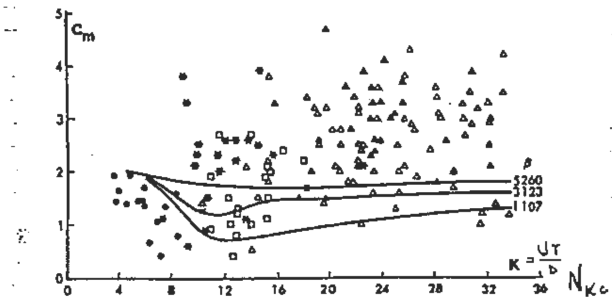


Fig. 5.3. Inertia coefficients from field data, as compiled by Wiegel et al. (1957). Solid lines from Sarpkaya (1976a).

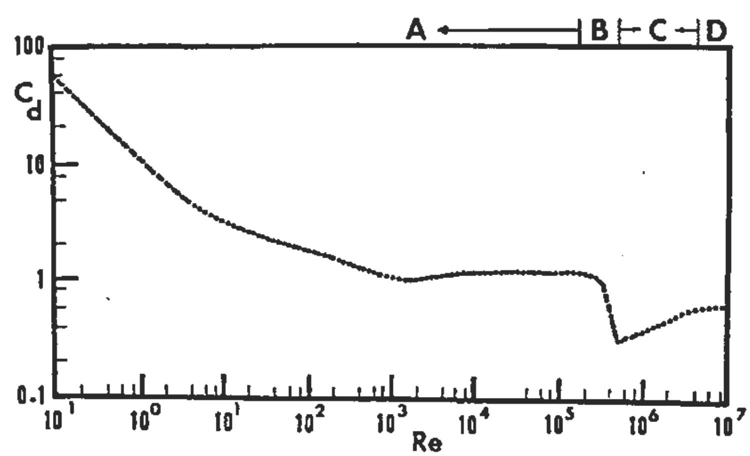


Fig. 3.1b. Drag coefficient for circular cylinders as a function of Reynolds number (Schlichting 1968).

$$S_N = \frac{f_o D}{V} \quad ; \quad R_e = \frac{DV}{\nu} \sim \frac{DH}{T\nu}$$

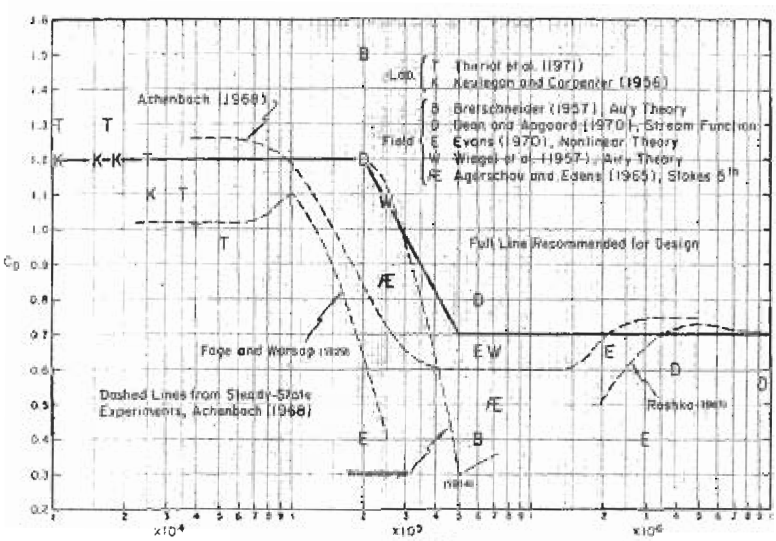


Figure 7-85. Variation of drag coefficient C_D with Reynolds number R_e.

Default C_M

$$\left.\begin{aligned} C_M &= 2.0 \quad \text{when} \quad R_e < 2.5 \times 10^5 \\ C_M &= 2.5 - \frac{R_e}{5 \times 10^5} \quad \text{when} \quad 2.5 \times 10^5 < R_e < 5 \times 10^5 \\ C_M &= 1.5 \quad \text{when} \quad R_e > 5 \times 10^5 \end{aligned}\right\} \qquad (7\text{-}53)$$

$$\frac{F_{Im}}{F_{Dm}} \approx \begin{cases} 1.25 \frac{D}{H} & \text{(shallow-water waves)} \\ 5.35 \frac{D}{H} & \text{(deepwater waves)} \end{cases}$$

Relative Importance of Drag and Inertia

$$\frac{F_{im}}{F_{Dm}} = \frac{\pi}{2} \frac{C_M}{C_D} \frac{D}{H} \frac{K_{im}}{K_{Dm}}$$

$$\frac{F_{im}}{F_{Dm}} \approx \begin{cases} 1.25 \frac{D}{H} & \text{(shallow-water waves)} \\ 5.35 \frac{D}{H} & \text{(deepwater waves)} \end{cases}$$

Forces

- Hi = 6 ft; T=8 sec; d=18 ft; D=2 ft; m~0
- C_M= 1.5; C_D=0.7
- →d/gT2= ;H/Hb=
- Maximum inertia Force?
- Kim = ________ Figure 7-71
- Fim = ______________0.25 $K_{im}C_M\gamma\pi HD^2$
- Maximum drag Force?
- K_{Dm} = ________ Figure 7-72
- F_{Dm} = ______________ $0.5K_{Dm}C_D\gamma DH^2$
- NAME ____________________

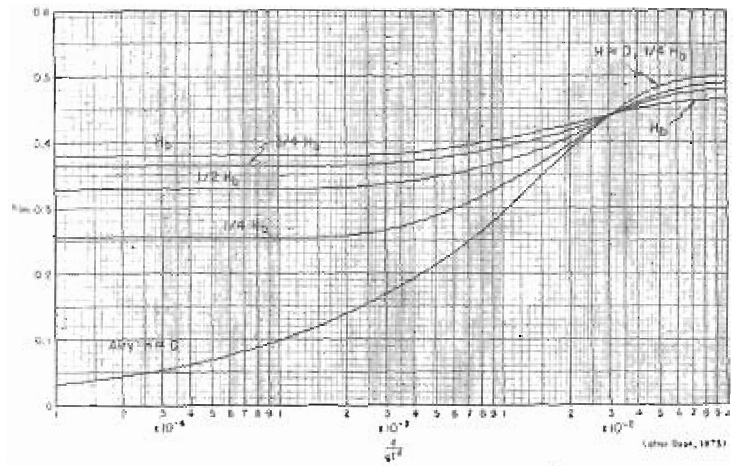


Figure 7-71. K_{im} versus relative depth, d/gT^2.

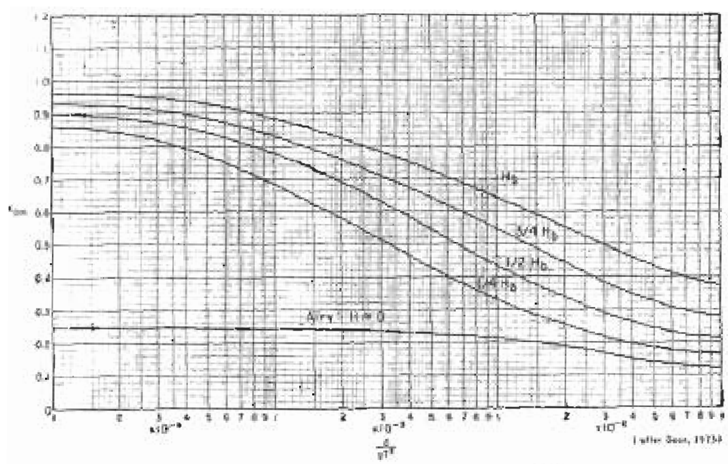


Figure 7-72. K_{Dm} versus relative depth, d/gT^2.

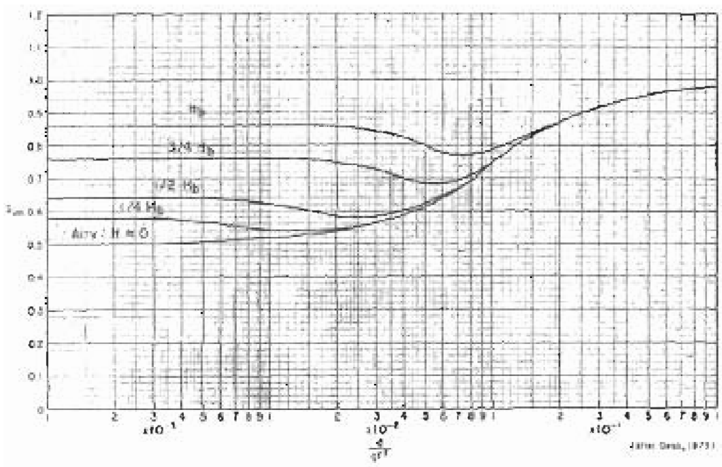


Figure 7-73. Inertia force moment arm, S_{im}, versus relative depth, d/gT^2.

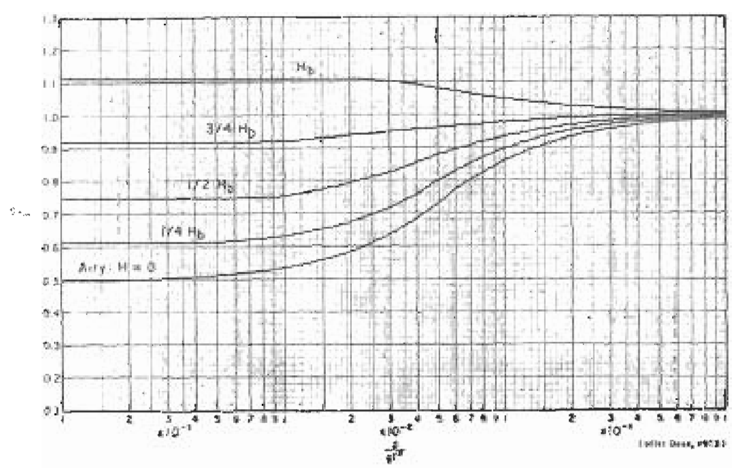


Figure 7-74. Drag force moment arm, S_{Dm}, versus relative depth, d/gT^2.

$$W = \frac{C_M}{C_D}\frac{D}{H} \qquad F_m = \phi_m \, w C_D H^2 D$$

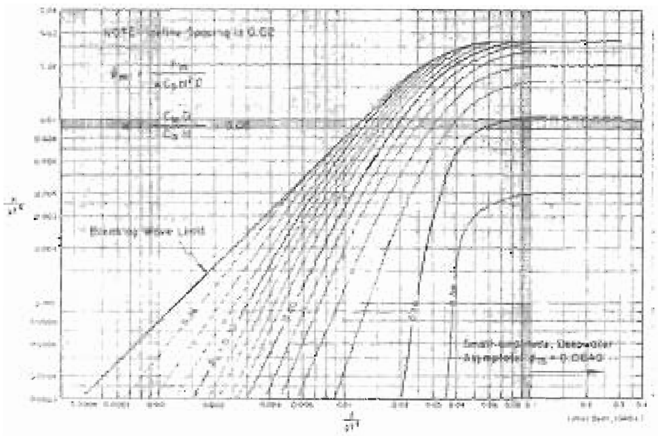


Figure 7-76. Isolines of ϕ_m versus H/gT^2 and d/gT^2 ... (W = 0.05).

W~0.05

$$W = \frac{C_M}{C_D}\frac{D}{H} \qquad F_m = \phi_m \, w C_D H^2 D$$

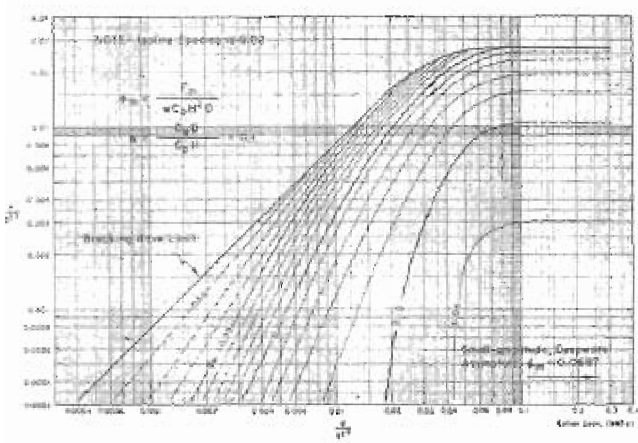


Figure 7-77. Isolines of ϕ_m versus H/gT^2 and d/gT^2 ... (W = 0.1).

W~0.1

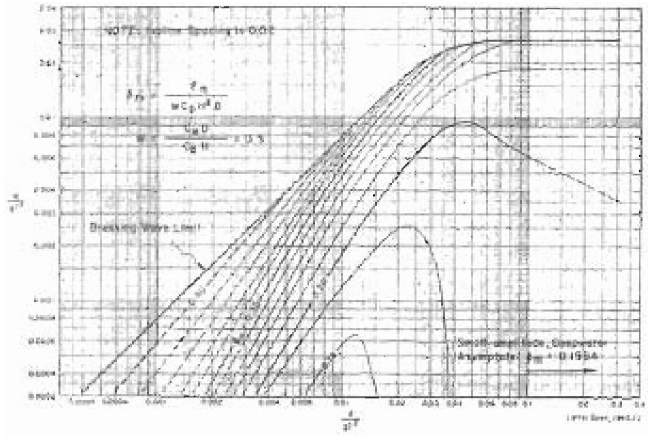


Figure 7-76. Isolines of ϕ_m versus H/gT^2 and d/gT^2 ... (W = 0.5) .

W~0.5

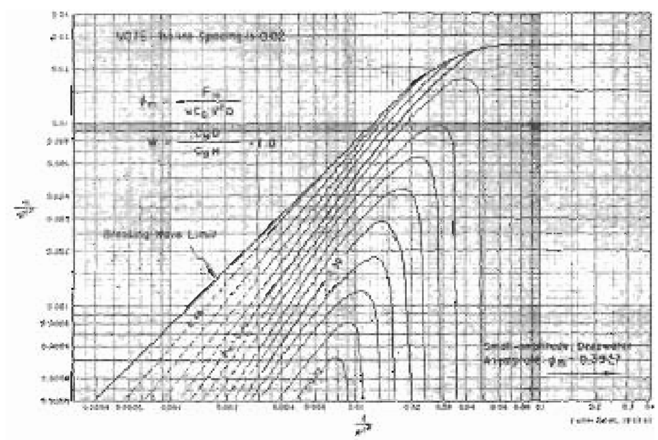


Figure 7-74. Isolines of ϕ_m versus H/gT^2 and d/gT^2 ... (W = 1.0) .

W~1

Resultant Forces

- Hi = 6 ft; T=8 sec; d=18 ft; D=2 ft; m~0
- C_M= 1.5; C_D=0.7
- →d/gT2= 0.0087 ;H/gT2=0.0029
- W=0.71
- Resultant of inertia and drag Forces?

□ ϕm = _________ Figure 7-78 W = 0.5

□ ϕm = _________ Figure 7-79 W=1

□ ϕm = _________ Figure 7-79 W=0.71

- FRm = _____________ $\phi_m C_D \gamma D H^2$
- NAME ______________________

Resultant Forces

- Hi = 6 ft; T=8 sec; d=18 ft; D=2 ft; m~0
- C_M= 1.5; C_D=0.7
- →d/gT2= 0.0087 ;H/gT2=0.0029
- W=0.71
- Resultant of inertia and drag Forces?

□ ϕm = __0.25______ Figure 7-78 W = 0.5

□ ϕm = ___0.34_____ Figure 7-79 W=1

□ ϕm = ___0.285_____ Figure 7-79 W=0.71

- FRm = 896 lbs_______________
- NAME ______________________

$$W = \frac{C_M}{C_D}\frac{D}{H}$$

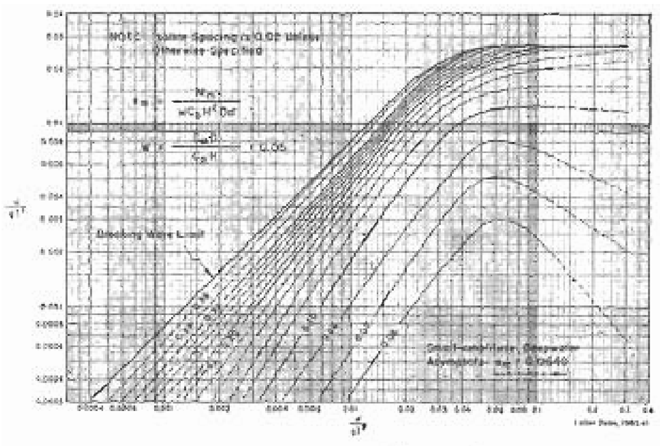


Figure 7-80. Isolines of ϕ_m versus H/gT^2 and d/gT^2 ... (W = 0.05).

W~0.05

$$W = \frac{C_M}{C_D}\frac{D}{H}$$

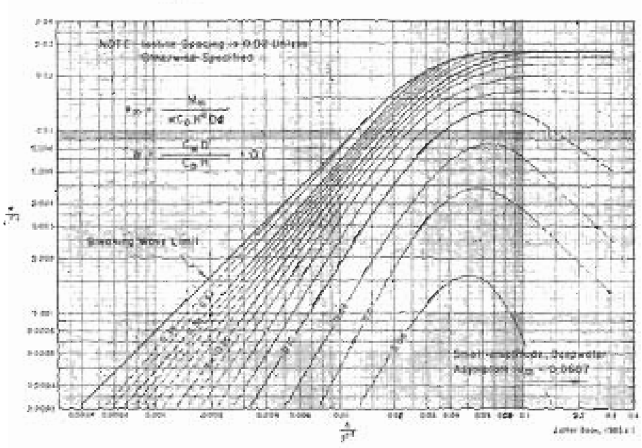


Figure 7-81. Isolines of ϕ_m versus H/gT^2 and d/gT^2 ... (W = 0.1).

W~0.1

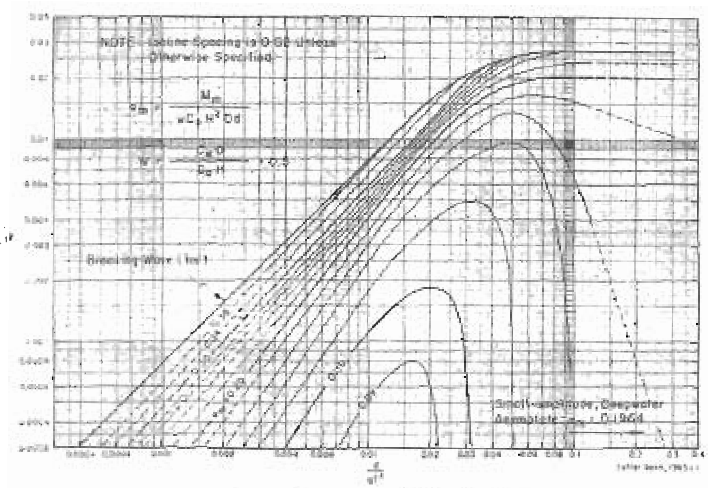


Figure 7-82. Isolines of α_m versus H/gT^2 and d/gT^2 ... (W = 0.5).

W~0.5

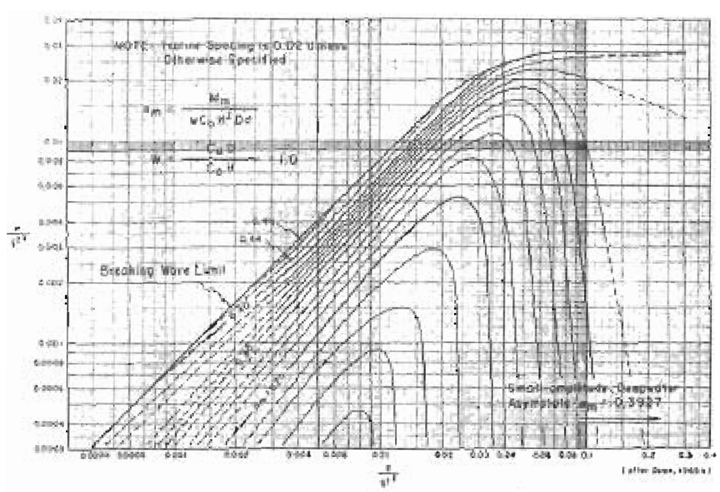


Figure 7-83. Isolines of α_m versus H/gT^2 and d/gT^2 ... (W = 1.0).

W~1

$$F_L = F_{Lm} \cos 2\theta = C_L \frac{\rho g}{2} D H^2 K_{Dm} \cos 2\theta$$

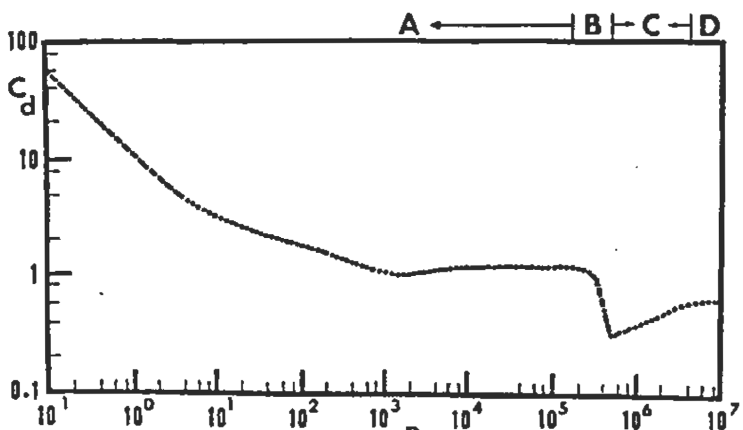
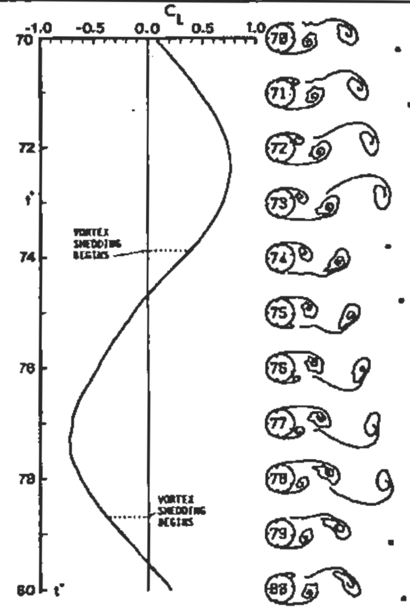


Fig. 3.1b. Drag coefficient for circular cylinders as a function of Reynolds number (Schlichting 1968).

$$S_N = \frac{f_o D}{V} \quad ; \quad R_e = \frac{DV}{\nu} \sim \frac{DH}{T\nu}$$



Sarpyaka/ Isaacson

Fig. 3.9. Evolution of lift and wake in the later stages of impulsively-started flow (Sarpkaya and Shoaff 1979a).

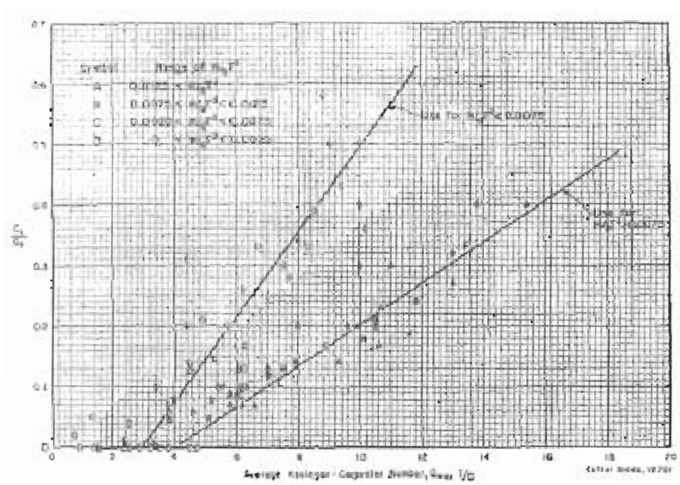


Figure 7-84. Variation of C_L/C_D with Keulegan-Carpenter number and H/gT^2.

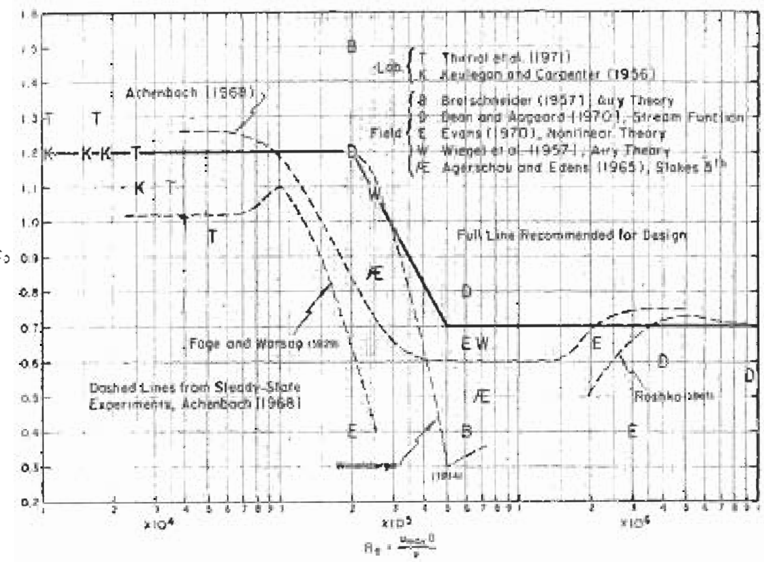


Figure 7-85. Variation of drag coefficient C_D with Reynolds number R_e.

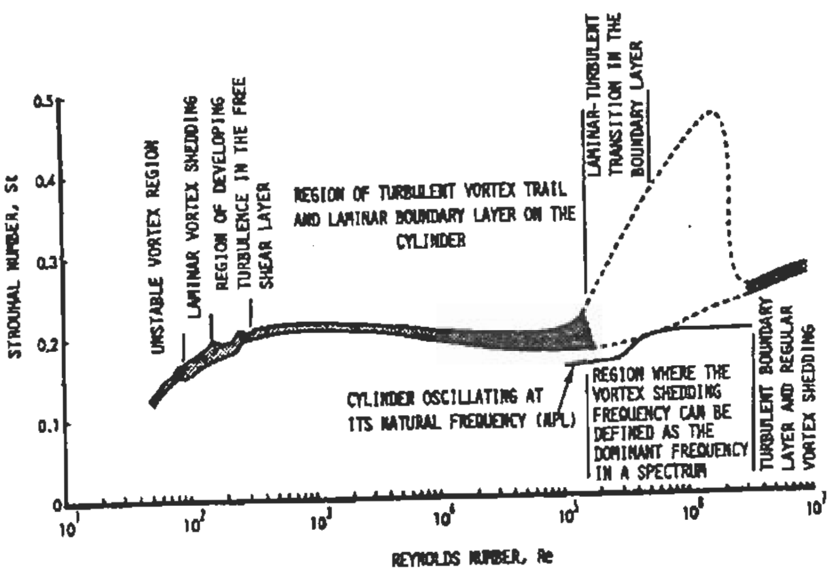


Fig. 3.2. The Strouhal-Reynolds number relationship for circular cylinders (Lienhard 1966).

Sarpkaya/Isaacson

58 MECHANICS OF WAVE FORCES

	A Subcritical	B Critical	C Supercritical	D Post-supercritical
Boundary layer	laminar	transition	turbulent	turbulent
Separation	about 82 deg.	transition	120 - 130 deg.	about 120 deg.
Shear layer near separation	laminar		laminar separation, bubble; turbulent reattachment	turbulent
Strouhal number	$S = 0.212 - \frac{2.7}{Re}$	transition	0.35 - 0.45	about 0.29
Wake	Re<60 laminar; 60<Re<5000 vortex street Re > 5000 turbulent	not periodic		
Approximate Re range	$< 2x10^5$	$2x10^5$ to $5x10^5$	$5x10^5 - 3x10^6$	$> 3x10^6$

Fig. 3.1a. Incompressible flow regimes and their consequences.

Resultant Forces

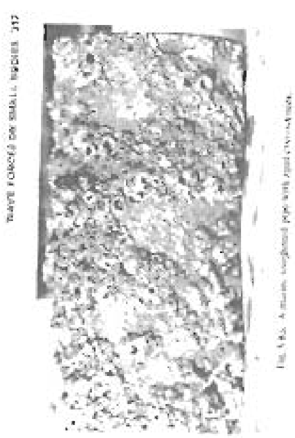
- Hi = 6 ft; T=8 sec; d=18 ft; D=2 ft; m~0
- C_M= 1.5; C_D=0.7;
- →d/gT2= 0.0087 ;H/gT2=0.0029
- Find: Resultant of lift and drag Forces? fo?
- Lift = ________
- Drag = ________
- Resultant = ________
- fo = ______________
- NAME ____________________

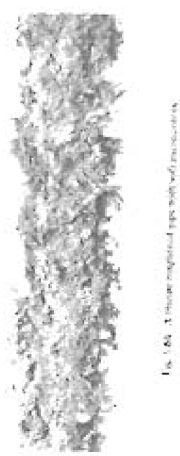
Note of Caution!!

- If the pile is rough e.g. coated with barnacles, the C_D increases ~ 1.3-1.5
- The C_M ~ 1.2-1.6 for circular piles with Nkc up to 100

Roughness

HARD





SOFT

WAVE FORCES ON LARGE BODIES 385

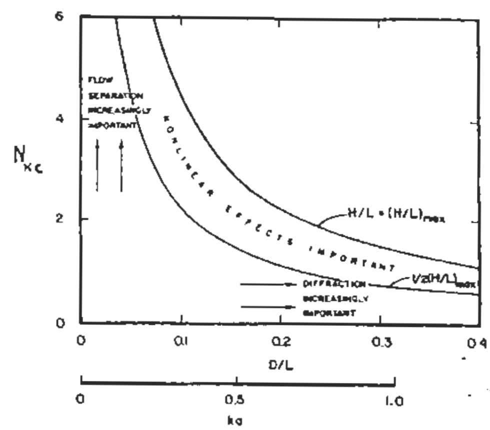


Fig. 6.1. Wave force regimes (Isaacson 1979a).

Large Diameter Piles

- D/L>0.05
- Diffraction becomes more important

$$k = wave.number = 2\pi / L$$

$$a = D / 2$$

$$F_{\max} = K_{dF}\gamma Had\frac{\tanh(kd)}{kd}$$

$$= K_{dF}\gamma HaL\frac{\tanh(kd)}{2\pi}$$

K_{dF}

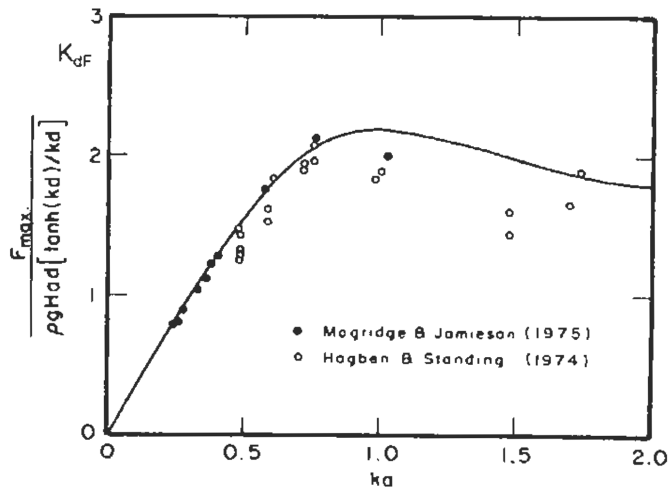


Fig. 6.6. Comparison of the closed-form solution for the wave force on a vertical circular cylinder with the experimental results of Hogben and Standing (1974) and Mogridge and Jamieson (1975).

WAVE FORCES ON SMALL BODIES 329

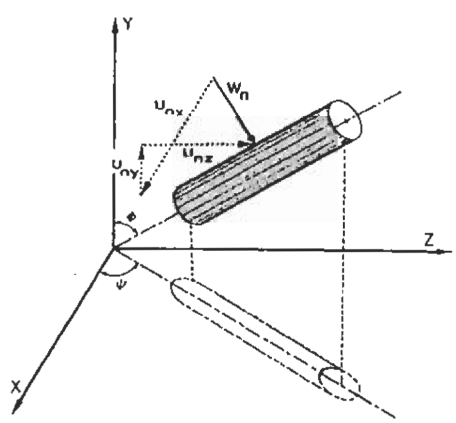


Fig. 5.9. Definition sketch for an inclined cylinder.

$$Re = |W_n| D/\nu, \quad K = |W_n| T/D \tag{5.29}$$

$$\begin{Bmatrix} F_x \\ F_y \\ F_z \end{Bmatrix} = 0.5\, \rho C_d D |W_n| \begin{Bmatrix} u_{nx} \\ u_{ny} \\ u_{nz} \end{Bmatrix} + 0.25\, \pi \rho C_m D^2 \begin{Bmatrix} \dot{u}_{nx} \\ \dot{u}_{ny} \\ \dot{u}_{nz} \end{Bmatrix} \tag{5.23}$$

where C_d and C_m are assumed to be known.

Velocity of breaking wave

'a' = speed of sound in water ≈ 4700'

g = 32.2

$$\frac{p}{\gamma} = \frac{V_b \ a}{g}$$

Broken Waves

(Breaker velocity) breaker depth (away from wall) offshore

$$V_b \simeq \sqrt{g b_d}$$

$$p_m = \gamma \left(\frac{V_b^2}{2g}\right) = \frac{\gamma d_b}{2}$$

Coastal Structures II

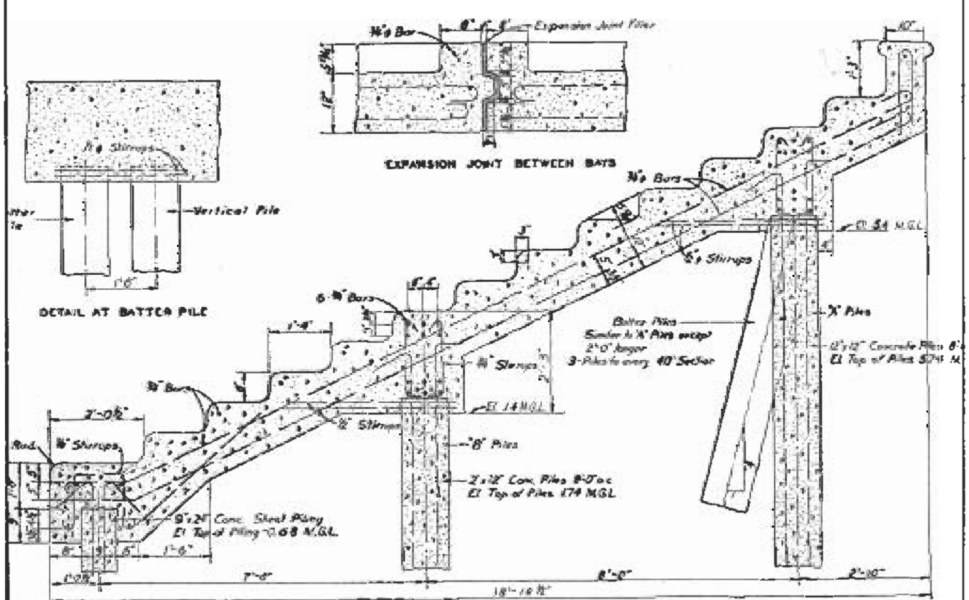
Seawalls
Non-breaking waves

Issues

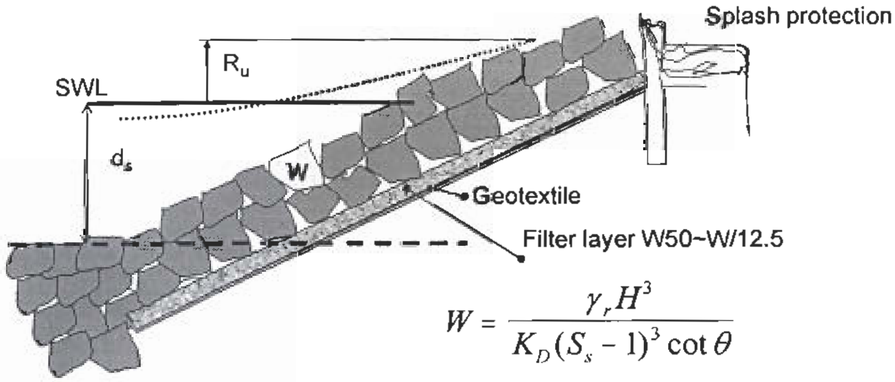
- Stability
- Overtopping
- Forces
- Runup
- Rush down
- Toe scour
- Loss of subgrade
- Scour behind the wall
- Loss of material to littoral zone

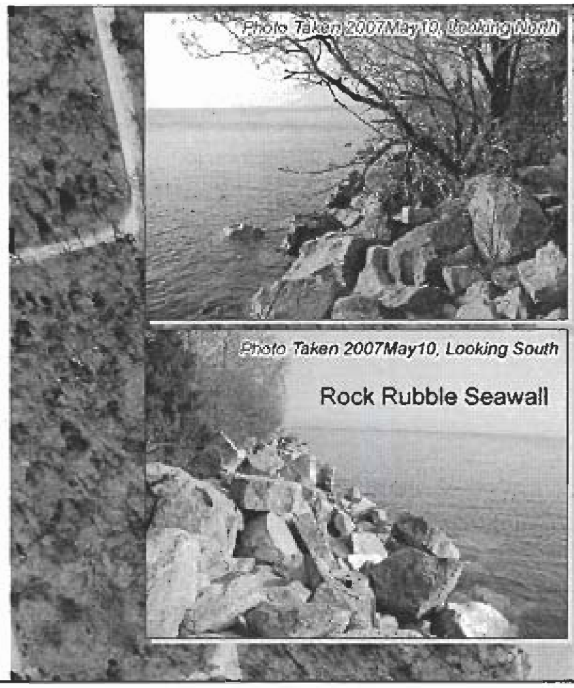
Types of Seawalls

- Sloped
  - Stepped concrete
  - Articulated mats
  - Rubble Mound
  - Grouted riprap
  - Gabions
  - Vegetated
- Vertical
  - Sheetpile
  - Gabion
  - Wooden crib type
  - Caisson
  - Concrete or masonry

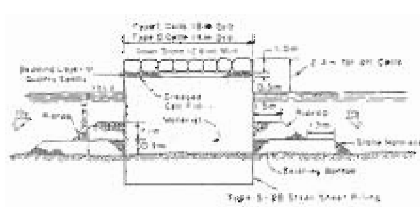


TYPICAL SECTION


Splash protection
Ru
SWL
ds
W
Geotextile
Filter layer W50~W/12.5
$W = \frac{\gamma_r H^3}{K_D (S_s - 1)^3 \cot\theta}$



Caisson Type



HUDSON EQUATION

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

$$\gamma_r = Dry\_Specific\_Weight\_of\_Armour\_Unit$$

$$K_D = Stability\_Coefficient(Damage\_Coefficient)$$

$$\theta = Slope\_of\_face$$

$$S_r = \frac{Specific\_gravity\_Armour\_Unit}{Local\_Specific\_Gravity\_of\_Water}$$

$$H = Design\_wave\_height$$

Design H

Design wave height is usually taken as the

significant wave Height, Hs for finding W

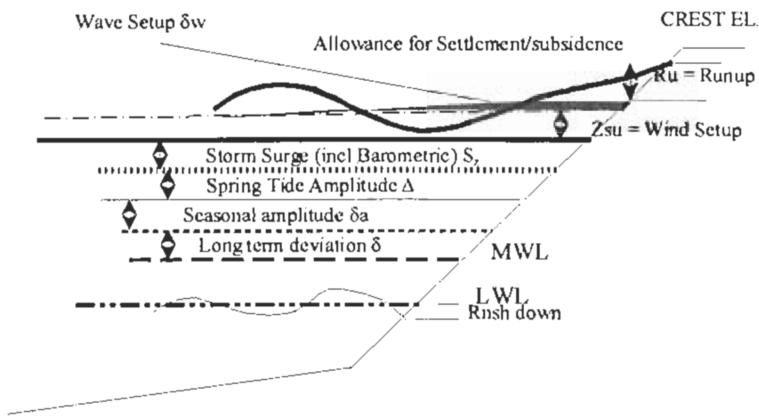
This leads to some damage 0-5% for the design storm since waves can exceed Hs

$$H_s = \sqrt{2} H_{rms}$$

$$H_{10} = 1.27 H_s$$

$$H_1 = 1.67 H_s$$

Shoreline Water Level



Design H for no overtopping of rubble mound seawalls

Design wave height is usually higher than Hs;
For example H10 will result in 5% of waves overtopping the seawall since for the design storm 5% of the waves will be > H10

$$H_s = \sqrt{2}H_{rms}$$

$$H_{10} = 1.27H_s$$

$$H_1 = 1.67H_s$$

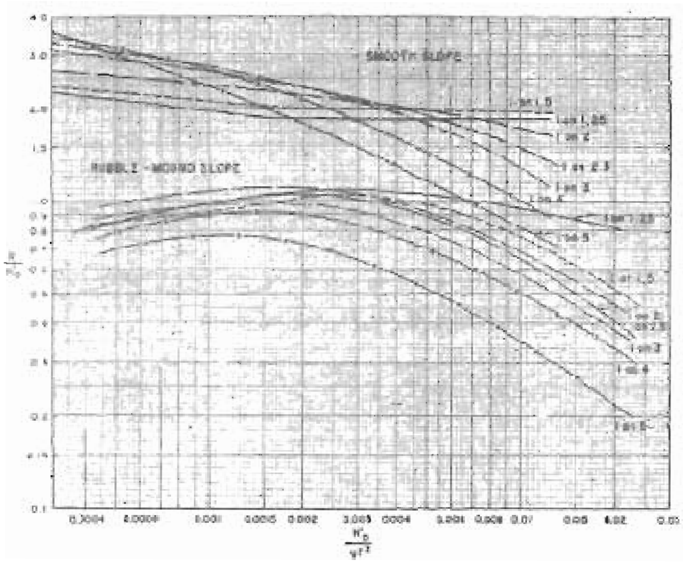
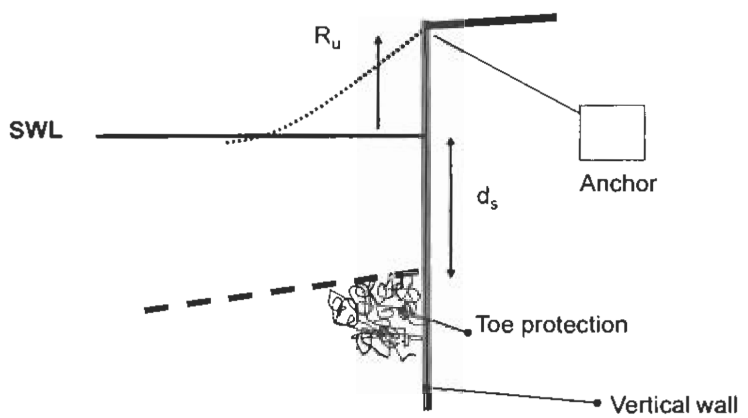


Figure 7-20. Comparison of wave runup on smooth slopes with runup on permeable rubble slopes (data for $d_s/H'_0 > 3.0$).



Design H

Design wave height for vertical seawalls that can be severely is usually by a few waves should be designed for Hi~ H1; this applies to rigid structures.

$$H_s = \sqrt{2} H_{rms}$$

$$H_{10} = 1.27 H_s$$

$$H_1 = 1.67 H_s$$

Non-breaking waves

- $H_i < H_b$ considering waves at least up to H_1. The breaking condition must be checked for all vertical seawalls. The highest force is due to waves that have crests parallel to the wall.
- Note: forces are greatly increased for waves that break against the wall.

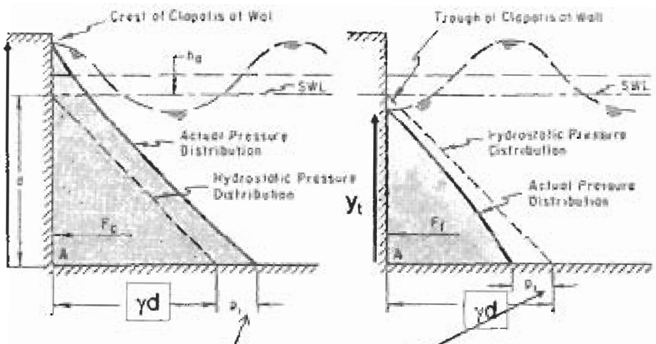


Figure 7-89. Pressure distributions for nonbreaking waves.

$$p_1 = \frac{(1+\chi)\gamma H_i}{2\cosh(\frac{2\pi d}{L})}$$

Rundgren

pressure at the wall is given by

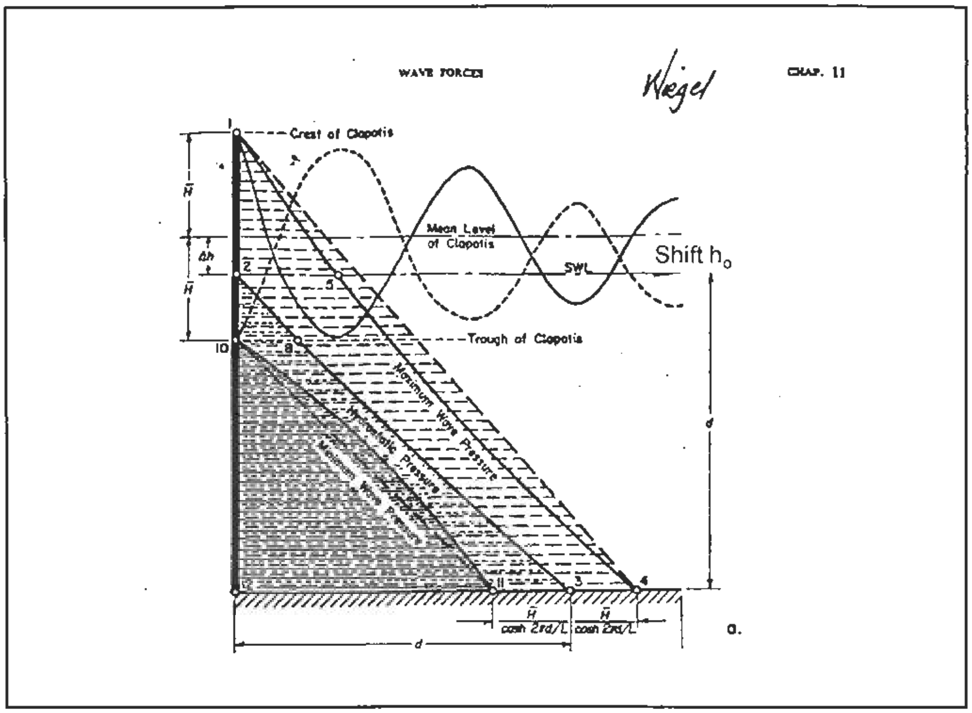
$$\frac{p_w}{\gamma} = h - s + H\frac{\cosh ks}{\cosh kh} + \pi\frac{H^2}{L}\tanh kh\left[\frac{3}{4}\frac{\cosh 2ks}{\sinh^4 kh} - 1\right] \quad (8.34)$$

the maximum amplitude of the wave at the wall is

$$a_w = H + \frac{\pi H^2}{L} n \coth kh$$

where

$$n = 1 + \frac{3}{4\sinh^2 kh} - \frac{1}{4\cosh^2 kh} \quad (8.35)$$



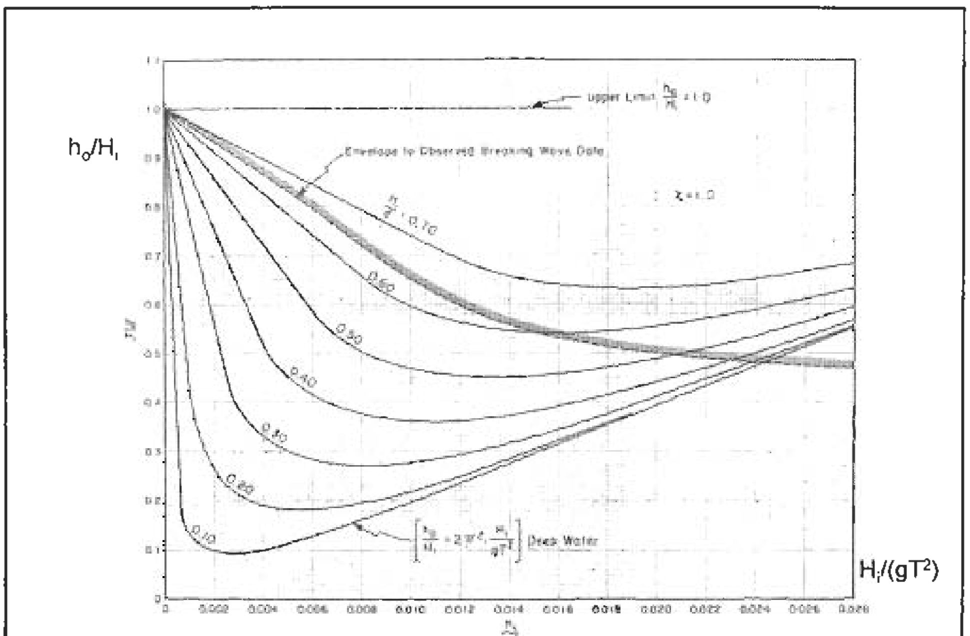


Figure 7-90. Nonbreaking waves; χ = 1.0 .

$$y_c = h_o + d + \frac{(1+\chi)H_i}{2}$$

$$y_t = h_o + d - \frac{(1+\chi)H_i}{2}$$

$\chi \sim 1$

Maximum and Minimum Depths at Vertical Seawall

- Given: d_s = 20 ft; H_i=8 ft; T= 7sec
- Solution:
- Find: Hi/ds = 8/20=0.4 & Hi/(gT^2)=0.0051
- ho= 0.45*8 = 3.6 ft Fig 7-90
- Max: yc=d+ho+0.5(1+1.0)*8=20+3.6+8=31.6ft
- Min: yt =20+3.6-8=15.6 ft

Forces and Moments on vertical seawalls

NON-BREAKING WAVES
Theory due to Miche-Rundgren and Sainflou
SPM Figure 7-91 and 7-92
For reflection coefficient $\chi = 1$.
$s=z$; $k=2\pi/L$; $h=d$

$$\frac{p_w}{\gamma} = h - s + H\frac{\cosh ks}{\cosh kh} + \pi\frac{H^2}{L}\tanh kh\left[\frac{3}{4}\frac{\cosh 2ks}{\sinh^4 kh} - 1\right] \quad (8.34)$$

the maximum amplitude of the wave at the wall is

$$a_w = H + \frac{\pi H^2}{L} n \coth kh$$

where

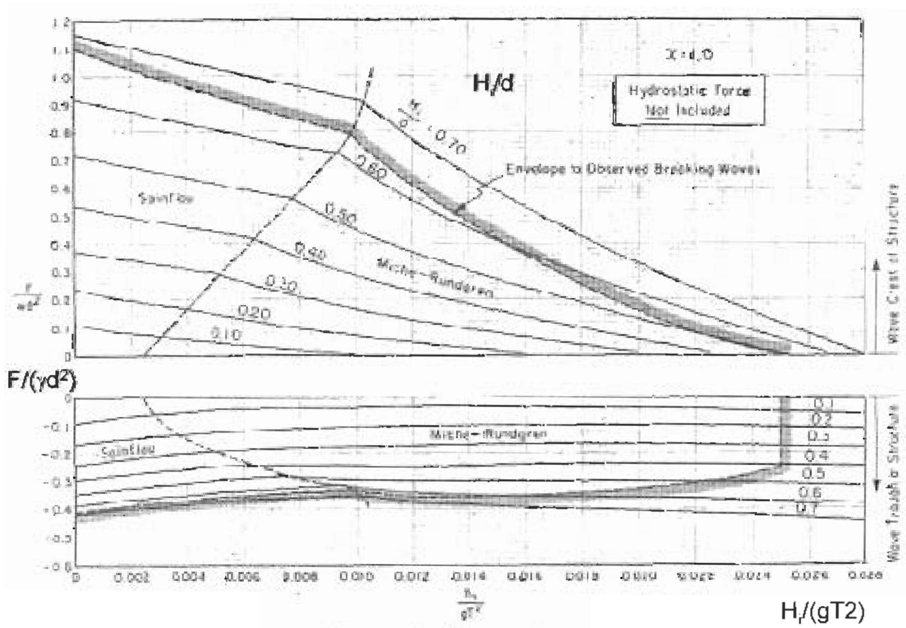
$$n = 1 + \frac{3}{4\sinh^2 kh} - \frac{1}{4\cosh^2 kh} \quad (8.35)$$

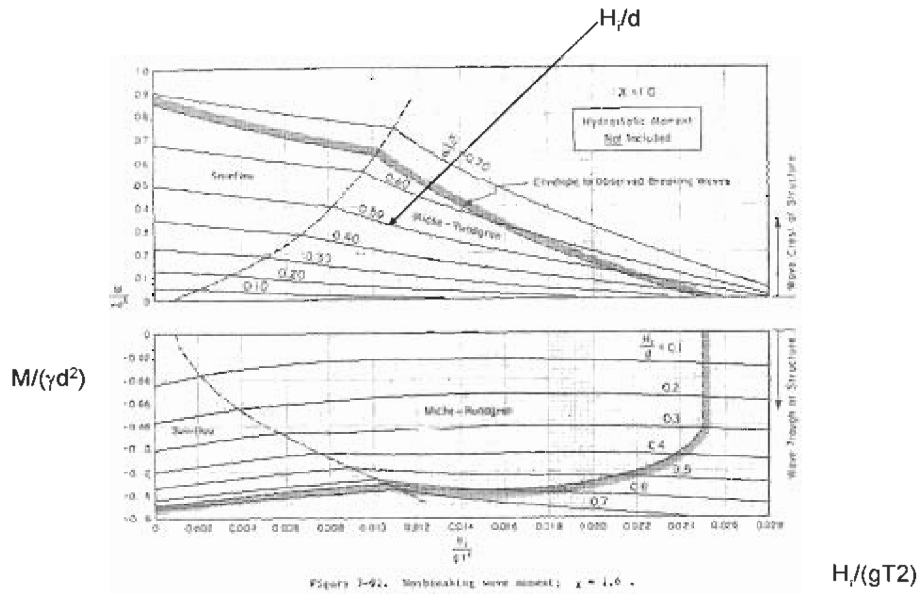
Fig 7-91

- The upper part of this figure shows the landward (+ve) added force due to the non-breaking wave; the force is rendered non-dimensional be dividing by γd^2.
- The lower part of this figure shows the seaward (-ve) reduction force due to the non-breaking wave; the force is rendered non-dimensional be dividing by γd^2.
- Note: wave breaking limit is indicated.

Fig 7-92

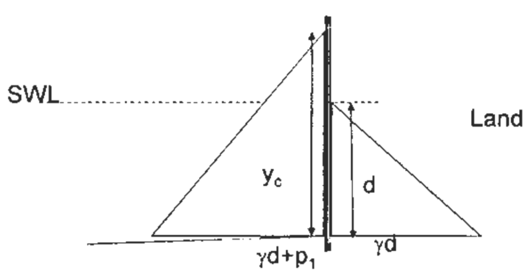
- The upper part of this figure shows the landward (+ve) added moment due to the non-breaking wave; the moment is rendered non-dimensional be dividing by γd^3.
- The lower part of this figure shows the seaward (-ve) reduction moment due to the non-breaking wave; the moment is rendered non-dimensional be dividing by γd^3.
- Note: wave breaking limit is indicated.





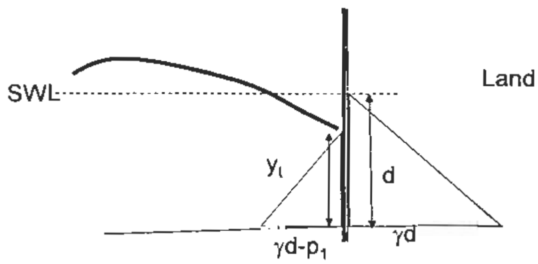
$$F_{Total,seaward} = F_{wave(+ve)} + \gamma d^2 / 2$$

$$M_{Total,seaward} = M_{wave(+ve)} + \gamma d^3 / 6$$



$$F_{Total,seaward} = F_{wave(-ve)} + \gamma d^2 / 2$$

$$M_{Total,seaward} = M_{wave(-ve)} + \gamma d^3 / 6$$



Runup at Vertical Wall

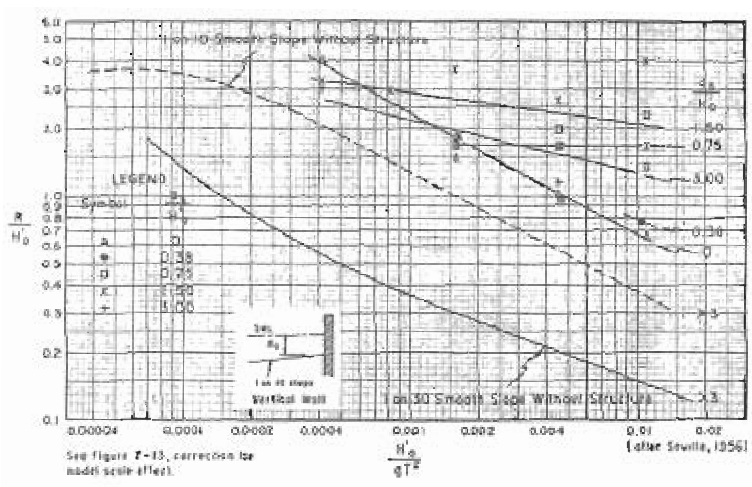


Figure 3-14. Wave runup on impermeable, vertical wall versus H'_0/gT^2.

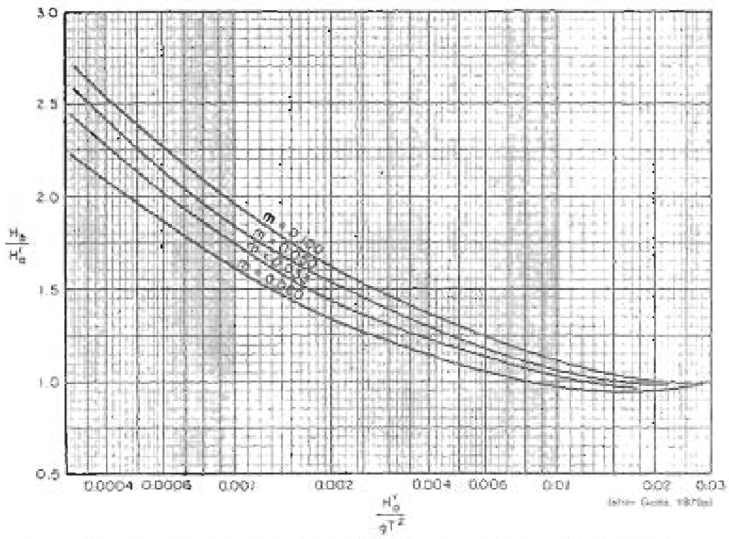


Figure 7-3. Breaker height index H_b/H_o' versus deepwater wave steepness H_o'/gT^2 .

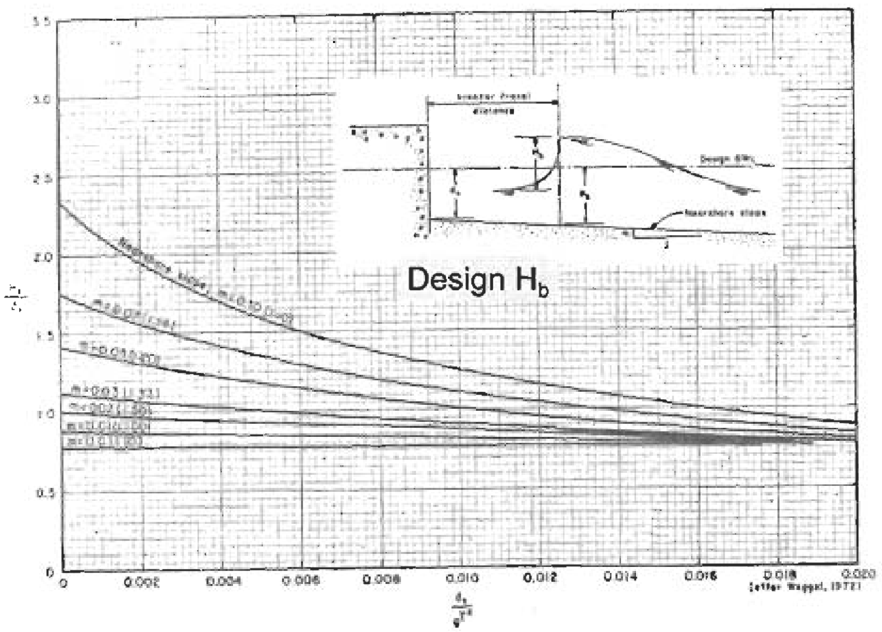


Figure 7-4. Dimensionless design breaker height versus relative depth at structure.

Landward Wave Forces and Moments on a Seawall

- Given: d_s = 20 ft; H_i=8 ft; T= 7sec; m=0.02
- Solution:
- Find: Fwave +ve & Mwave +ve
- Fwave+ = 11,232 lbs/ft
- Mwave+= 150,000 ft-lbs/ft

F+	11232	lbs/ft	0.45	Fig 7-91
M+	149760	ft-lbs/ft	0.3	Fig 7-92

Seaward Wave Forces and Moments on a Seawall

- Given: d_s = 20 ft; H_i=8 ft; T= 7sec
- Solution:
- Find: Fwave -ve & Mwave -ve
- Fwave- = 6,500 lbs/ft
- Mwave-= 60,000 ft-lbs/ft

F-	6490	lbs/ft	0.26	Fig 7-91
M-	57907	ft-lbs/ft	0.116	Fig 7-92

Total Forces and Moments

FT+ 23712 lbs/ft
MT+ 232960 ft-lbs/ft

FT- 18970 lbs/ft
MT- 141107 ft-lbs/ft

Lecture 9
Coastal Structures
Part I

Wave action on coastal structures can be classified by the type of wave and the type of structure as shown below. This classification is useful for selecting the best formulae for estimating the forces on various structural elements.

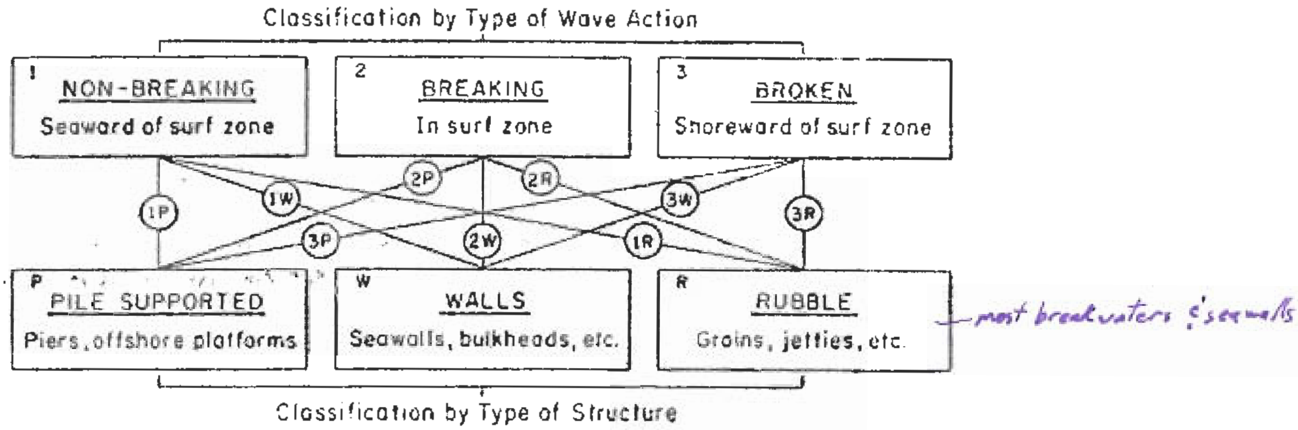


Figure 7-66. Classification of wave force problems by type of wave action and by structure type.

Figure 9-1. Wave Interaction with Structures

Breakwater Design:

The most common type of breakwater is the rubble mound. This structure consists of a pile of individual units of rock or concrete generally placed with the smaller units at the core and the heavier units in the wave impact zone as shown in the typical section in Figure 7-116 SPM. These heavy units on the surface layer are referred to as *Armour Units*.

The *W* in Figure 7-116 SPM represents the rock size need to resist the design wave, i.e. Hs, H10 or H1 depending on the margin of safety required. The Hudson formula is generally accepted as the most reasonable one for rubble mound design:

$$W = \gamma S_s H^3 / \{K_D (S_s - 1)^3 \cot \theta\} \qquad 9.1$$

where K_D = stability or damage coefficient as given by flume tests (see Table 7-8 SPM). S_s = specific gravity based on ambient fluid. As indicated by Table 7-8 SPM the damage coefficient depends on the unit shape, number of layers, method of placement, location along the breakwater and the slope.

Once the armour layer W is determined, the unit sizes for the internal zones of the breakwater are defined as a fraction of W. This fraction is set so as to ensure good energy dissipation, and to prevent loss of the internal units through the pore spaces in the armour layer. Figure 9-2 shows a typical breakwater cross-section.

A design for a segmented breakwater is shown in Figure 9-3. An installation has been constructed near the Rockerfeller Wildlife Refuge (Figure 9-4).

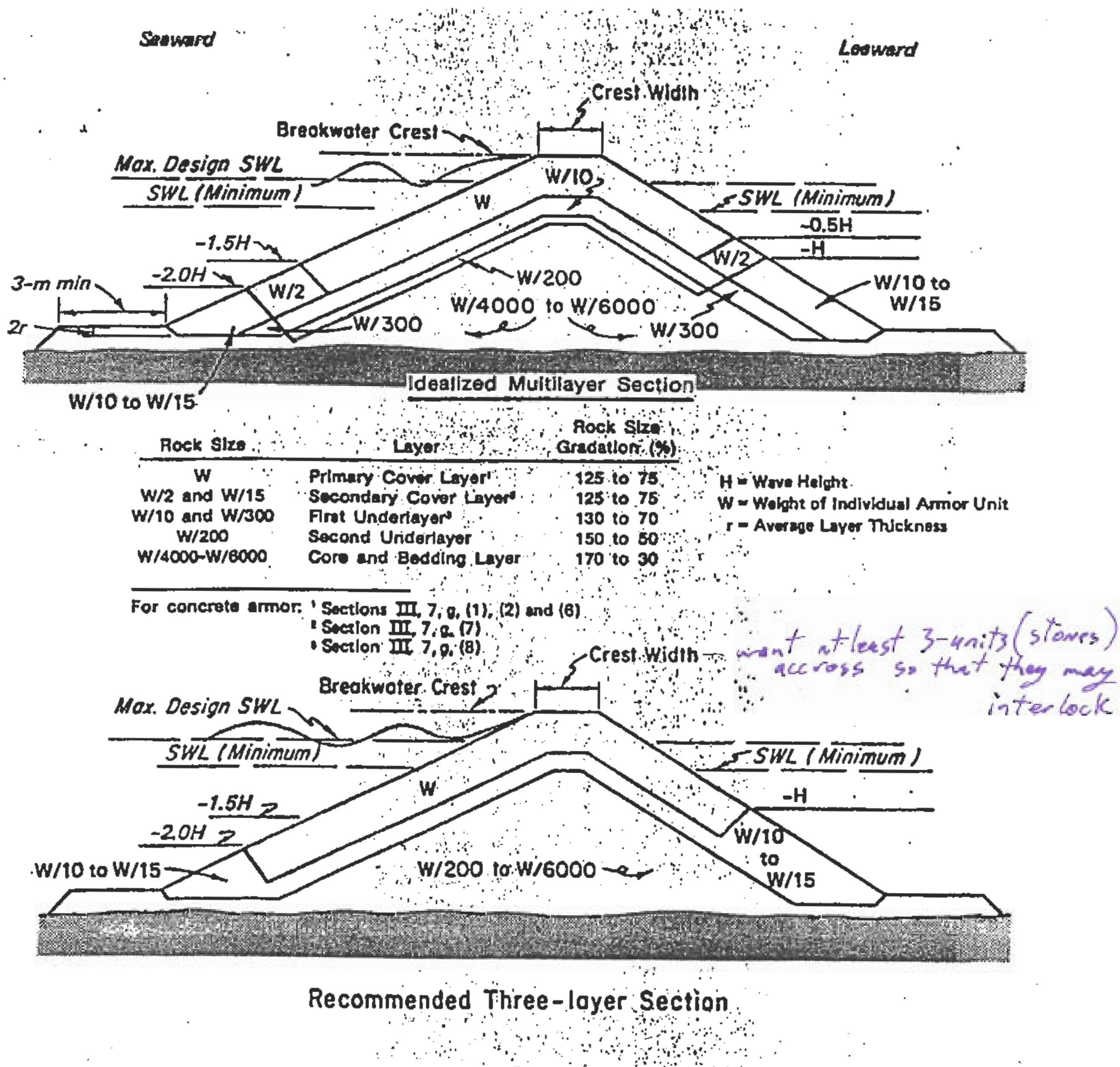


Figure 7-116. Rubble-mound section for seaward wave exposure with zero-to-moderate overtopping conditions.

Figure 9-2: Typical Rubble Mound Breakwater Cross-sections (SPM)

Hudson Eqn

W = weight for rock to be used

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

γ_r = Dry spec. weight of Armor Unit

H = design wave height (H_i)

K_D = stability Coefficient (Damage Coefficient)

θ = slope of face

$$S_r = \frac{\text{Specific gravity of Armor Unit}}{\text{Local } S_G \text{ of water (salt? fresh? etc.)}}$$

Design H

Significant wave height (H_s) the damage is 0–5% for an design event (storm) b/c waves can exceed H_s

$H_s = \sqrt{2}\ H_{rms}$

$H_{10} = 1.27\ H_s$

$H_1 = 1.67\ H_s$

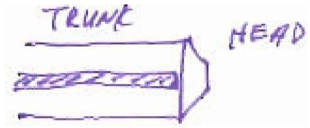


Table 9-1: K_D Values

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

No-Damage Criteria and Minor Overtopping							
			Structure Trunk		Structure Head		
			K_D[2]		K_D		Slope
Armor Units	n[3]	Placement	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 and Quadripod	2	Random	7.0	8.0	5.0	6.0	1.5
					4.5	5.5	2.0
					3.5	4.0	3.0
Tribar	2	Random	9.0	10.0	8.3	9.0	1.5
					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.

$H_s = 2m$ $T = 7s$ slope $= 1:2$

Armor Rock (rough angular) – 2 layers

$S_s = 2.65$

$S_s = 2.65 * / (1 + (Sal / 1000)) = 2.4 / 1.006$

$S_r = 2.634$

$\gamma_r = S_s (1000)(9.81) = 9.81(2650)$ N/m³ or γ_r mass $= 2650$ kg/m³

$$W_{kg} = \frac{2650 (2^3)}{3.5 (2.634 - 1)^3 2} = 800 \text{ kg}$$

C_{kg}

$H_s = 2m$ $\cot\theta = 2$

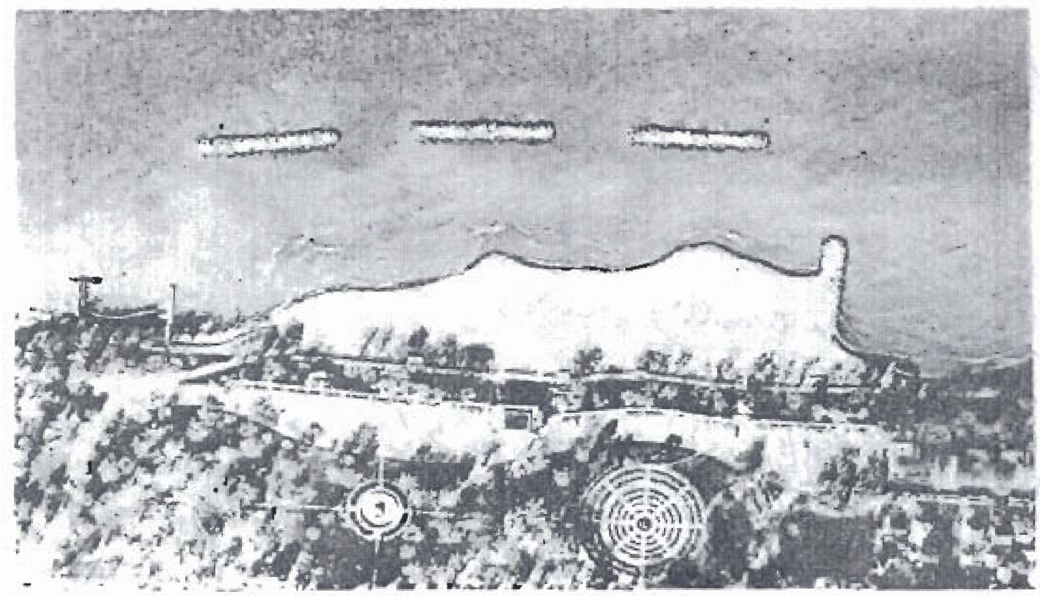
– Trunk, Breaking $K_D = 3.5$

$W = 793$ kg ~ 800 kg

– Head, Breaking $K_D = 2.5$

$W = 1110$ kg

$$W_{kg} = \frac{2650 (2^3)}{2.5 (2.634 - 1)^3 (2)} = 1110 \text{ kg}$$



Lakeview Park, Lorain, Ohio (Apr. 1981)

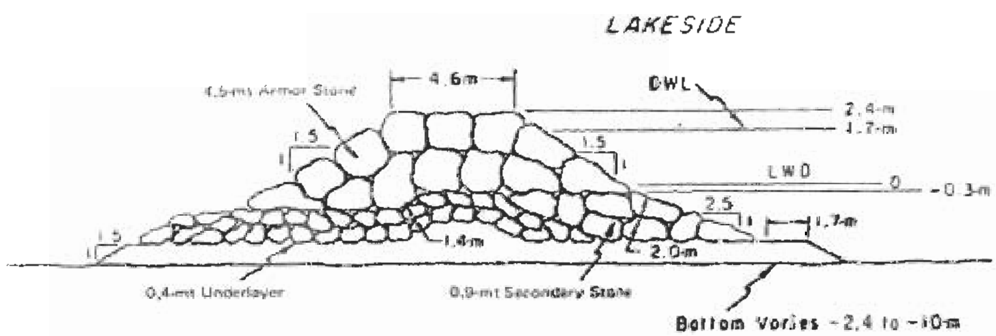


Figure 6-66. Segmented rubble-mound offshore breakwaters.

Figure 9-3: Segmented Breakwater

✓ design of B.W.

$H_s = 11m$ $T = 13.5s$ slope $= 2:3$ $\cot\theta = 1.5$

42-t Dolos Armor Units Seawater Salinity = 34 ppt

$d_s \sim 35-50$ m Freshwater $S_s = 2.4$

check for Breaking & non-Breaking waves

S_s @ 34ppt = $S_s^* / (1 + (Sal/1000)) = 2.4/1.034$

$S_r = 2.325$

$\gamma_{r\,mass} = S_s(1000) = 2400\ kg/m^3$ $H_s = 11m$ $\cot\theta = 3/2 = 1.5$

Breaking $K_D = 22$ $W = 42$-t (42000 kg) = 42 metric tons

Non-Breaking $K_D = 25$ $w = 37$-t

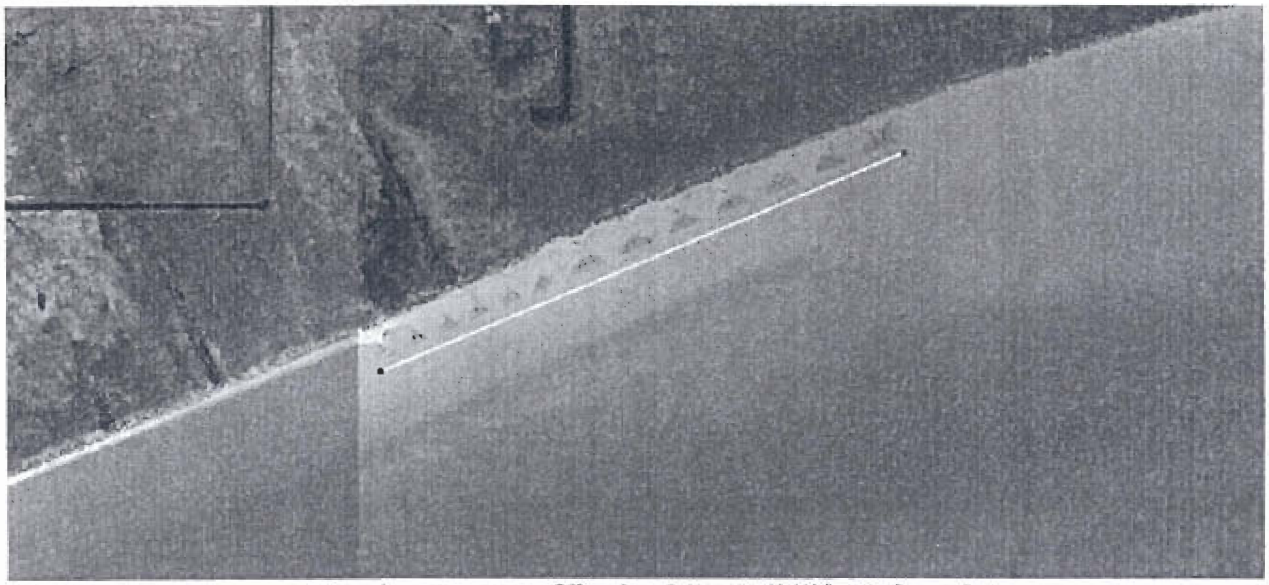


Figure 9-4: Segmented Breakwater east of Rockerfeller Wildlife Refuge, LA

Crest Level:-

There are three types of breakwaters: non-overtopped (infrequently overtopped), low crested (frequently overtopped) and submerged (also called reef breakwaters and are normally under water). We will concentrate on the non-overtopped type.

The following equation summarizes the factors that must be considered in setting the crest elevation:

CREST ELEV = *Mean Water Level +*
Seasonal Amplitude +
Long term deviation from MWL +
Amplitude of High Tide +
Storm Surge + Barometric Setup +
Wind Shear Setup +
Seiche +
Wave Setup +
Wave Runup +
Allowance for Settling and/or Subsidence 9.2

SWL = Still Water Level = Water Level Before Runup or Rushdown is added.

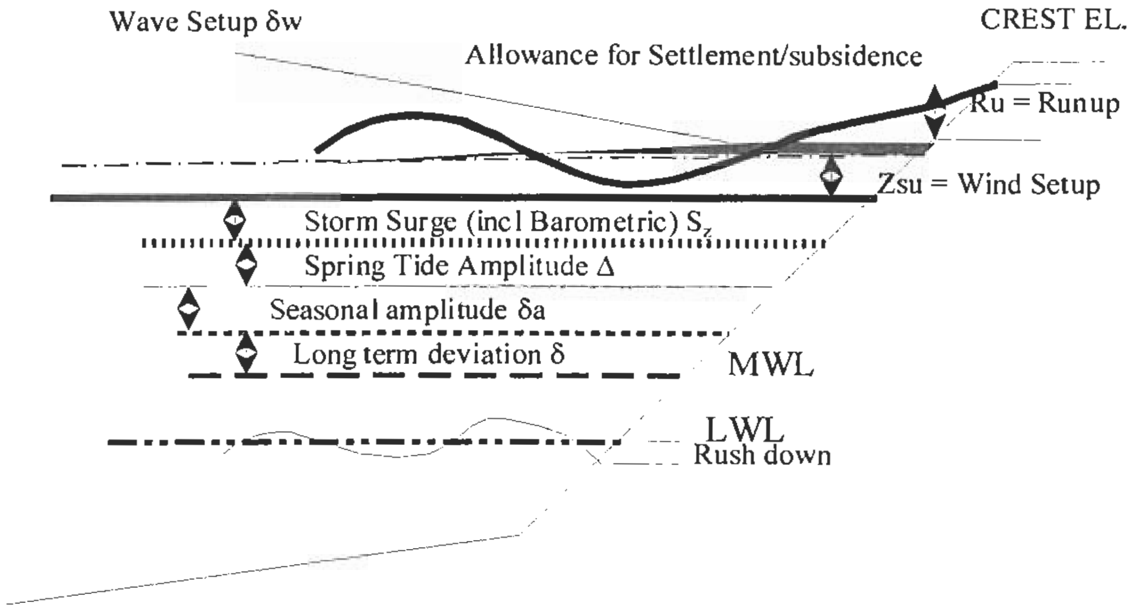


Figure 9-5: Factors Affecting the Crest Elevation of a Seawall or Breakwater.

Tides:-

Astronomical tides are generated by gravitational fields of the moon and the sun on the sea and earth and centrifugal forces of the rotating system. The moon and earth form a rotating system with the axis of rotation within the earth. Tide generally have a two maximum ranges (High tide) and two minimum ranges in each lunar month (28 days). The tidal period theoretical should be about 12 hours (11.7 hours) with one higher range and one lower range during the 24 hour day ; however, in some locations, the lower range is negligible and effectively the tidal period is close to 24 hours (23.4 hours). The tide in Lake Borgne has a period of about 23.4 hours.

http://csep10.phys.utk.edu/astr161/lect/time/tides.html

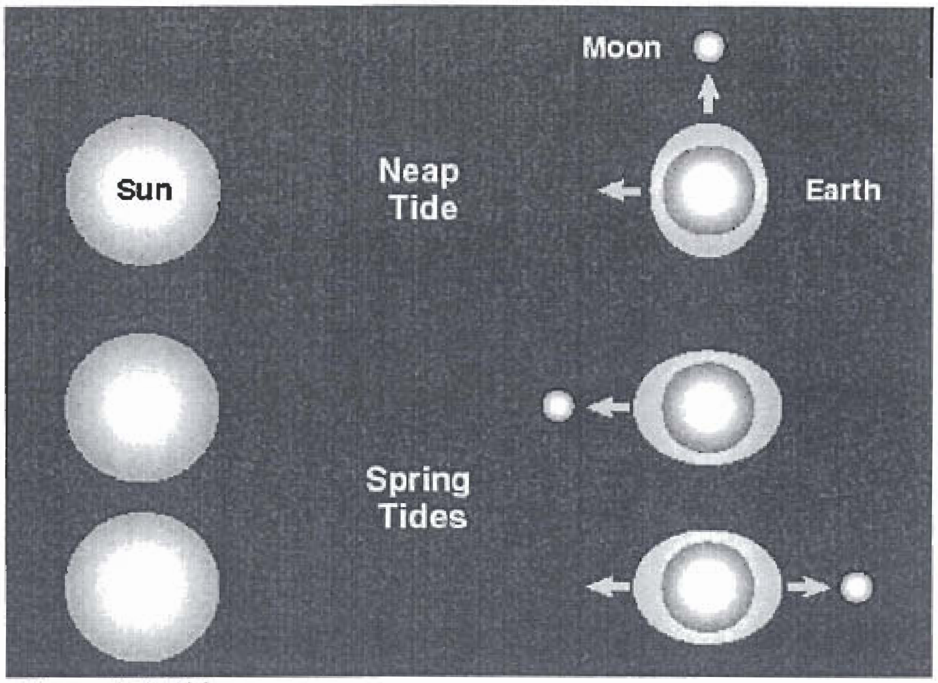


Figure 9-5 Tides

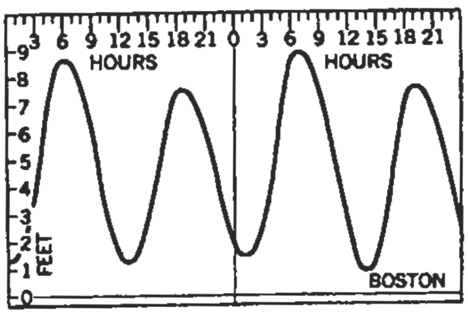


Figure 5–3 ***Semidiurnal type of tide.***

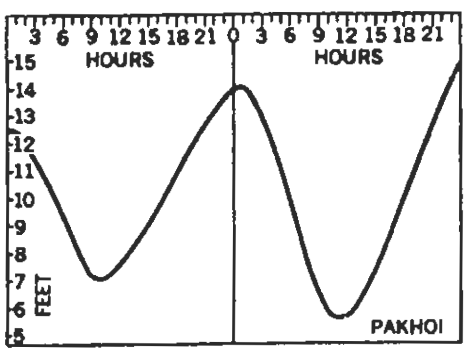


Figure 5–4 ***Diurnal type of tide.***

Figure 9-6: Types of Tides

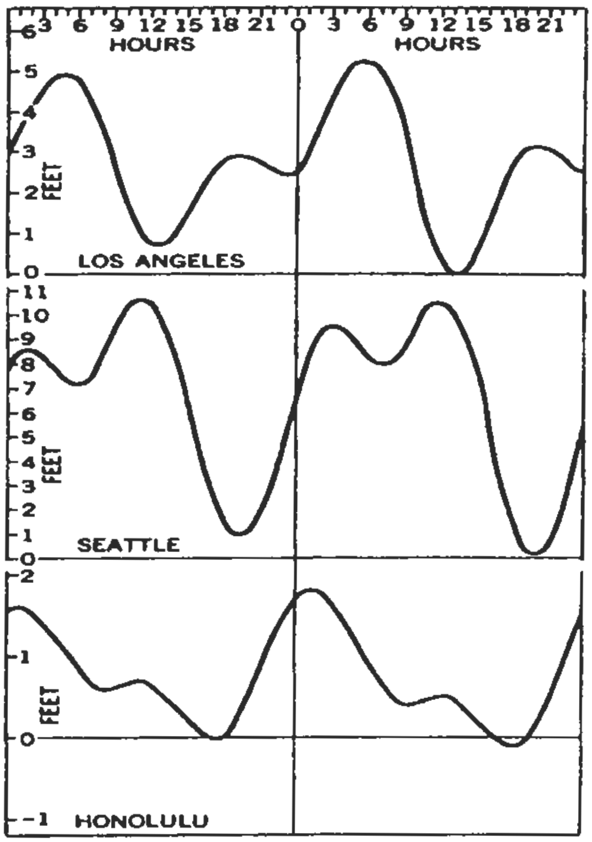


Figure 5–5 ***Mixed types of tide.***

Figure 9-7: Mixed Tides

The alignment of the Sun, Moon and Earth produces a high tidal range referred to as a Spring Tide. A low tidal range occurs when the Sun, Earth and Moon form a right angle as shown in Figure 9-5.

56 / *THE SEA*

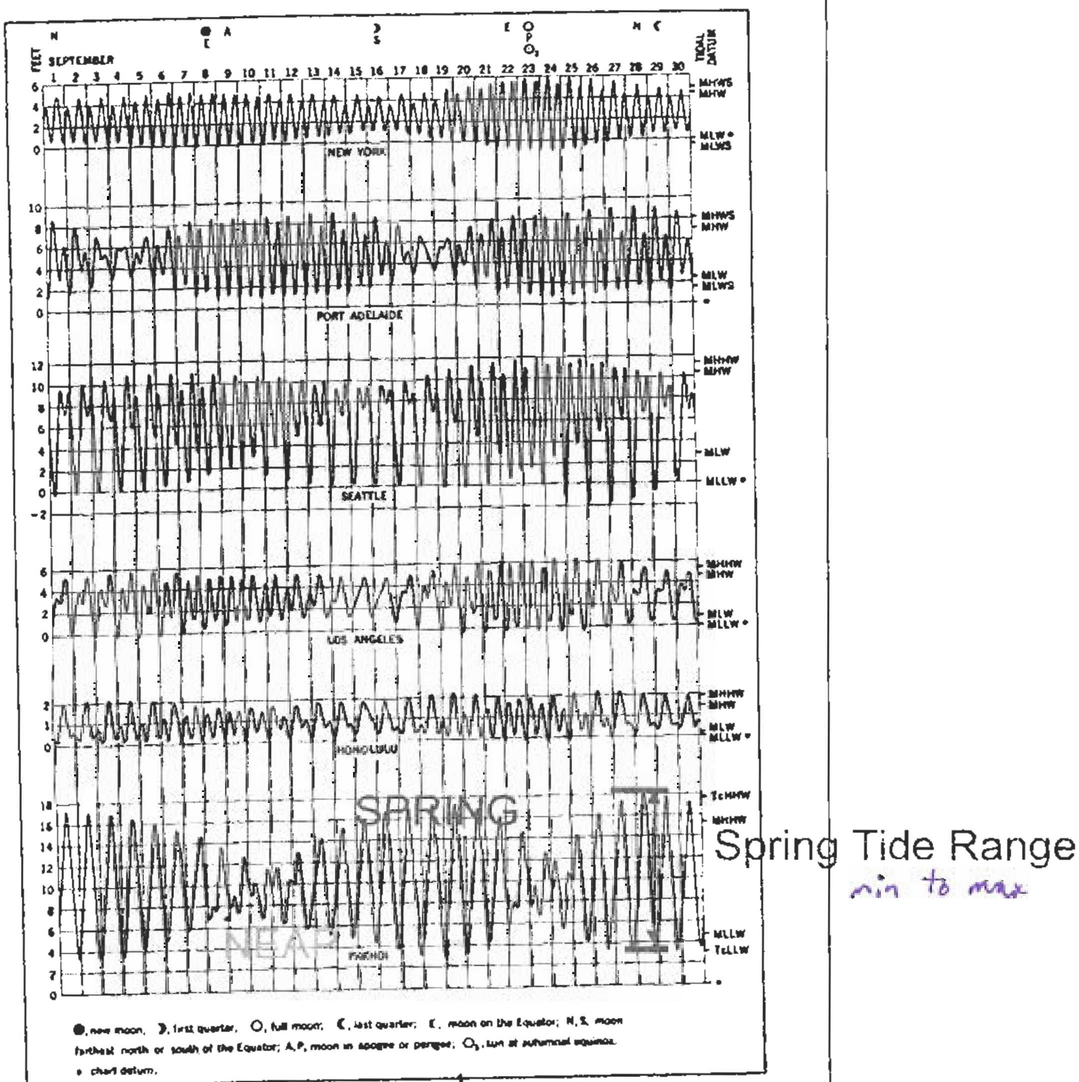


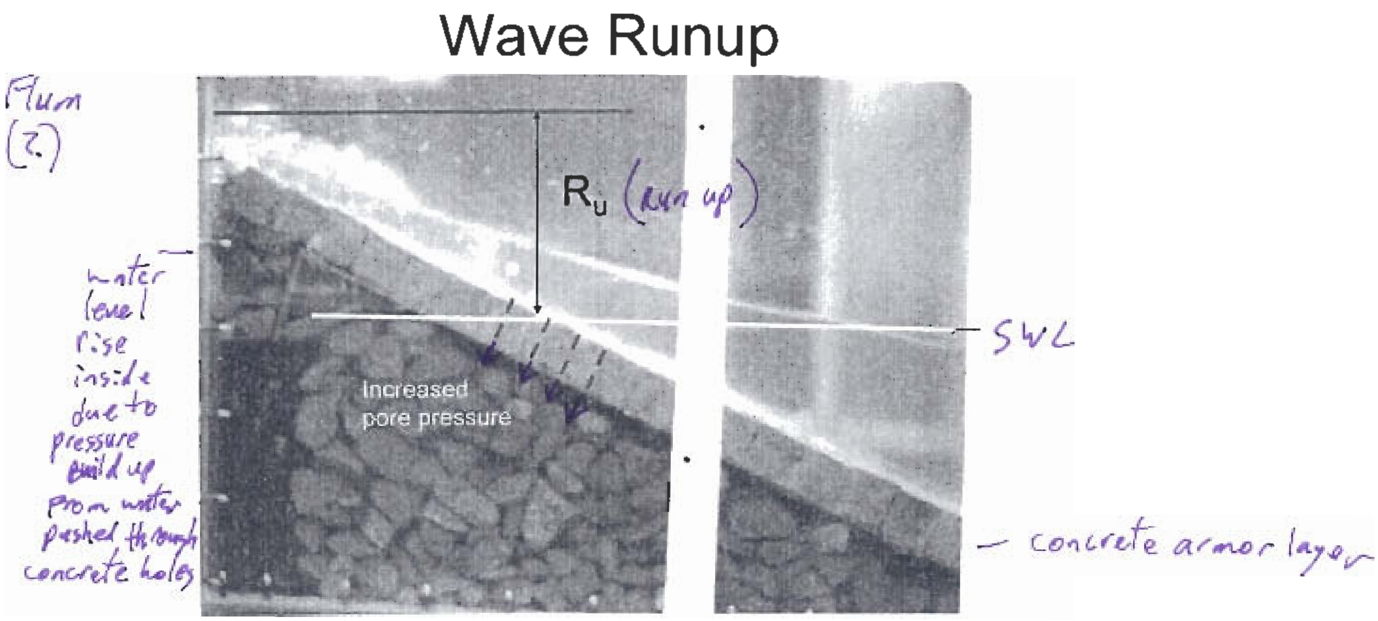
Figure 5–6 *Tidal variations at various places during a month.*

Figure 9-8: Lunar Cycles

The tidal range is the vertical range between the trough of the tide and the crest of the tide. The tidal amplitude is ½ of the tidal range. For design purposes the maximum tidal range or spring (see Figure 9-8) tide is usually assumed.

Runup and Rush Down:-

The wave runup is the vertical rise of the wave impact water level above SWL as illustrated in Figue 9-9. For rubble mound this is approximately one wave height (see Figure 7-13 Saville et al in *Water Resources Engg* - Lindsley and Franzini). The runup on a smooth impervious slope can be approximately twice the wave height. Figure 9-10 gives the Runup as a function of seawall slope, wave steepness and type of surface.



` Figure 9-9: Wave Runup on a Seawall

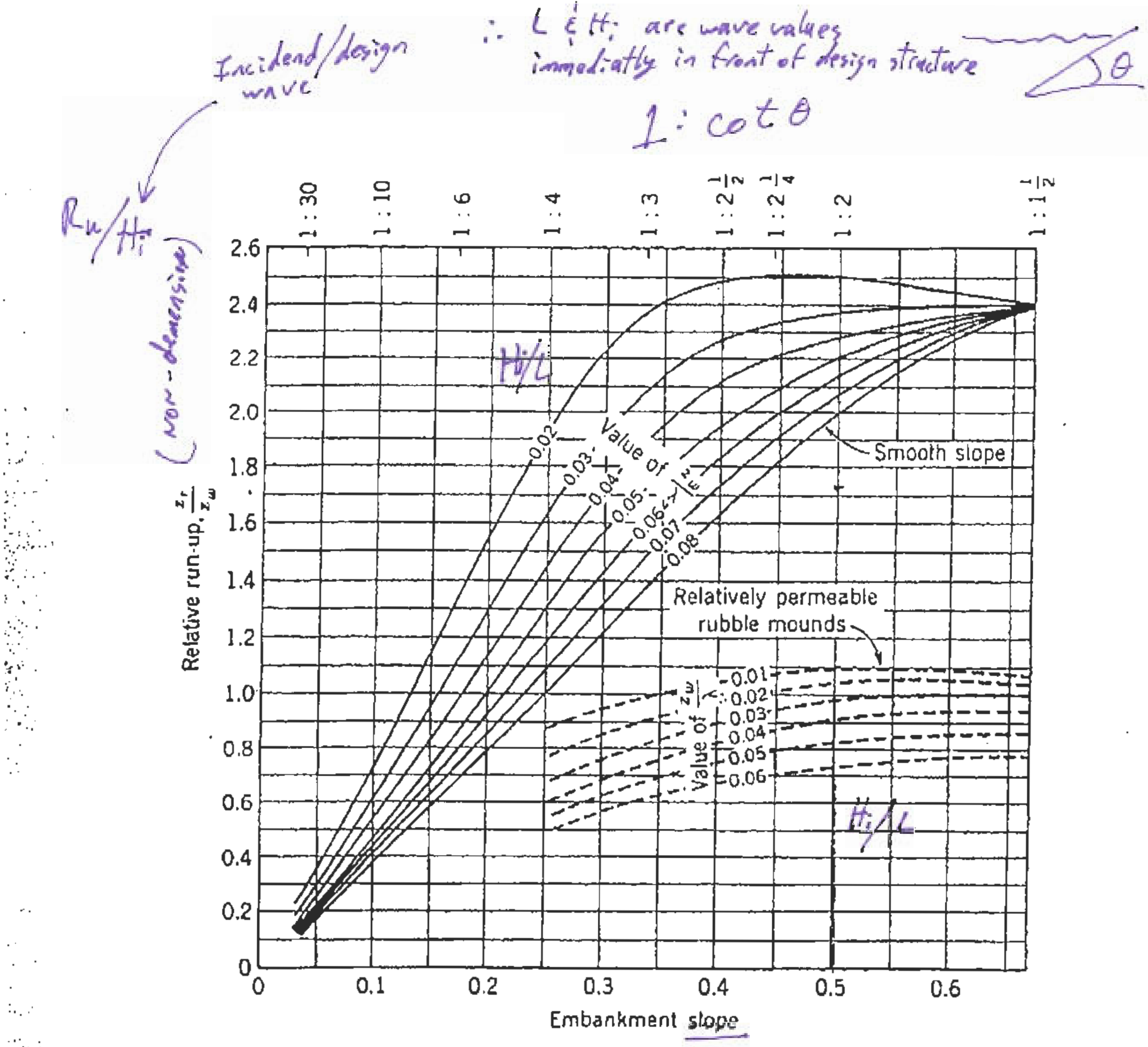


FIG. 7-13 Wave run-up ratios vs. wave steepness and embankment slopes. (From Saville, McClendon, and Cochran)

Figure 9-10: Non-dimensional Runup (Ru/Hi). Note: z_w/λ= Hi/L

The retreating wave on a slope results in what is called a wave rush down as illustrated in Figure 9-11. Failure of breakwaters can occur on the down rushing wave when surface units are displaced due to the uplift from unbalanced internal pore pressure and the drag of the outflowing and down rushing water.

Wave Rush Down

effective weight of particles reduced b/c of uplift

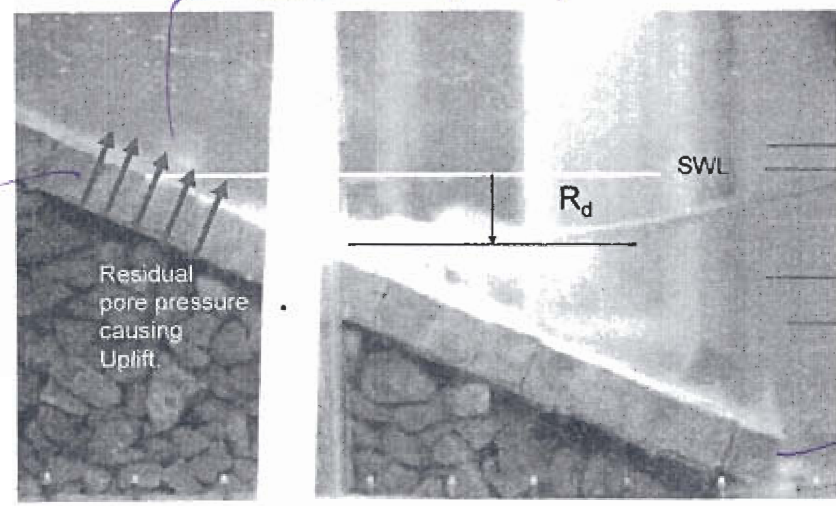


Figure 9-11. Wave Rush Down — typically not as much as R_u (≈ 60%)

Wave Setup:- Wave setup is due to the momentum of the solitary wave at the time of breaking.

The SPM •73 gives

$$\text{Wave Setup} = S_w = 0.19\,H_b\left[1 - 2.82\sqrt{\frac{H_b}{gT^2}}\right] \quad \text{..............................9.3}$$

Example: Given H_b = 1.2 m and T = 5 s. Estimate Wave Setup.

Hi-Tech → Adserp (?) model } models used to
older method → Slosh model

Storm Surge:

Storm surge may be due to tropical storms or extra-tropical systems including barometric gradients. A rough approximation for a storm surge is given by: (use if do not have data)

$$h_{ss} \approx K_s K_r \left[\Delta p + \frac{C_R C_f R U_R^2}{2 g d_{ref}} \frac{\rho_a}{\rho_w} \right]$$

$$C_R \approx 1 \rightarrow 2$$

$$\frac{\rho_a}{\rho_w} \approx 1/800$$

$$C_f \approx 0.0025$$

$$d_{ref} \sim 10 \rightarrow 20m$$

$$S_d = \frac{\sqrt{g}\, H_0^2 T}{64 \pi d}$$ (setdown)

$$S_w = 0.19 H_b \left[1 - 2.82 \sqrt{\frac{H_b}{g T^2}} \right]$$

Wind Setup:

Wind setup can be estimated from equations such as,

$$Z_{sup} = \frac{F'(\rho_a C_f U_{OW}^2)}{2 g \rho_w d}$$

$$C_f \sim 0.001 \rightarrow 0.003$$

Long-term and Seasonal Water Levels:

Large water bodies like the Gulf of Mexico or the Great Lakes are subject to seasonal water level fluctuations. Figure 9-12 illustrates the seasonal changes in the water level in the Northern Gulf of Mexico. In addition, there can be a long-term Relative Sealevel Rise (RSLR) of about 1 cm/year. In the Great Lakes there is a 10-20 year cyclical amplitude of about 1 m.

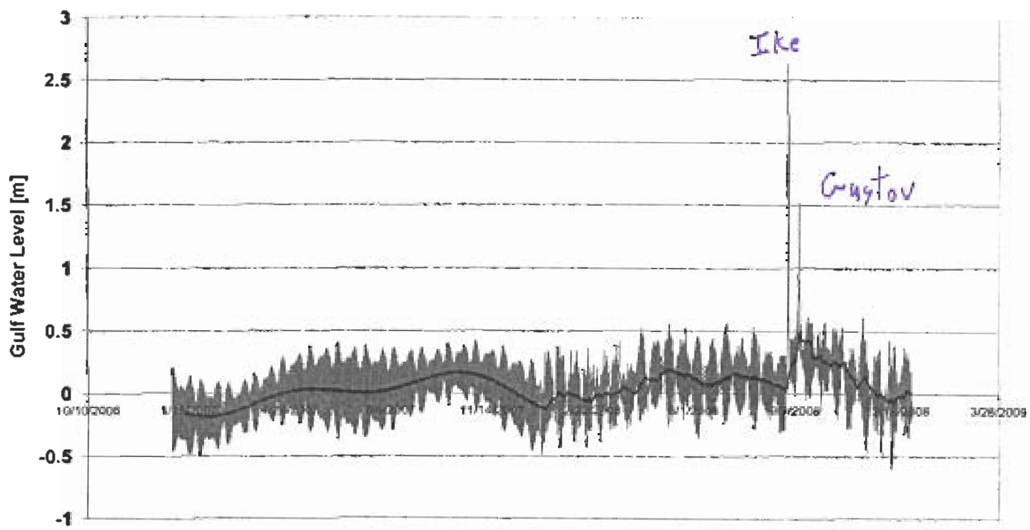


Figure 9-12 Typical Northern Gulf of Mexico Water Levels

Major Assignment Problem 9.1:

Name:__________________________________ Due April 9, 2010

Design a rubble mound breakwater for the significant wave and setup near the south shore of Lake Pontchartrain for a 1 hour wind out of the north over the Lake.

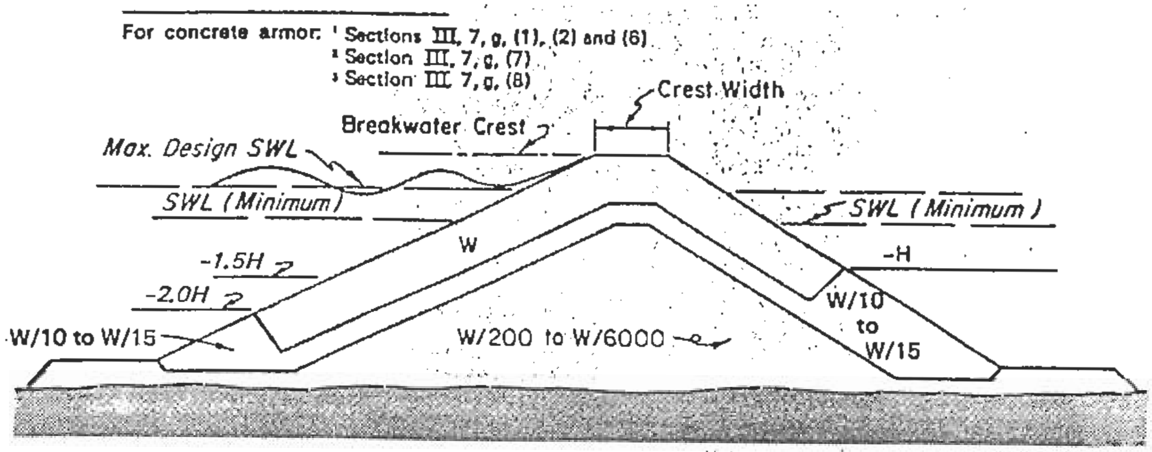
Given:

1. a 25 mile fetch
2. an average depth of 3.8 m before the storm surge,
3. a storm surge of 2.2 m,
4. a local depth of 2.8 m at the breakwater without the storm surge or setup,
5. a water temperature of 80oF and air temperature of 70oF.
6. an over-water velocity is 80 mph.
7. spring tidal range 0.16 m.
8. Gulf seasonal amplitude 0.1 m (positive for hurricane season).

Design Period 50 years.
Design the crest elevation for no overtopping at the design Hs.

Assume: Ss =2 .63;
slope 1:2;
individually placed rough angular rock armour stone; 2 layers
the mean SWL in Lake Pontchartrain is 0.2 m MSL;
the minimum SWL is -0.3 m;
RSLR including subsidence rate 0.9 cm/year;
settlement = 0.3 m.

Use the Standard Section shown below.



Recommended Three-layer Section

APPENDIX 1. TABLE OF FUNCTIONS OF d/L_o

d/L_o	d/L	$2\pi d/L$	tanh $2\pi d/L$	sinh $2\pi d/L$	cosh $2\pi d/L$	K	$4\pi d/L$	sinh $4\pi d/L$	cosh $4\pi d/L$	n	C_G/C_o	H/H'_o	M
0	0	0	0	0	1	1	0	0	1	1	0		
.0001000	.003990	.02507	.02506	.02507	1.0003	.9997	.05014	.05016	1.001	.9998	.02506	4.467	7,855
.0002000	.005643	.03546	.03544	.03547	1.0006	.9994	.07091	.07097	1.003	.9996	.03543	3.757	3,928
.0003000	.006912	.04343	.04340	.04344	1.0009	.9991	.08686	.08697	1.004	.9994	.04338	3.395	2,620
.0004000	.007982	.05015	.05011	.05018	1.0013	.9987	.1003	.1005	1.005	.9992	.05007	3.160	1,965
.0005000	.008925	.05608	.05602	.05611	1.0016	.9984	.1122	.1124	1.006	.9990	.05596	2.989	1,572
.0006000	.009778	.06144	.06136	.06148	1.0019	.9981	.1229	.1232	1.008	.9988	.06128	2.856	1,311
.0007000	.01056	.06637	.06627	.06642	1.0022	.9978	.1327	.1331	1.009	.9985	.06617	2.749	1,124
.0008000	.01129	.07096	.07084	.07102	1.0025	.9975	.1419	.1424	1.010	.9983	.07072	2.659	983.5
.0009000	.01198	.07527	.07513	.07534	1.0028	.9972	.1505	.1511	1.011	.9981	.07499	2.582	874.3
.001000	.01263	.07935	.07918	.07943	1.0032	.9969	.1587	.1594	1.013	.9979	.07902	2.515	787.0
.001100	.01325	.08323	.08304	.08333	1.0035	.9966	.1665	.1672	1.014	.9977	.08285	2.456	715.6
.001200	.01384	.08694	.08672	.08705	1.0038	.9962	.1739	.1748	1.015	.9975	.08651	2.404	656.1
.001300	.01440	.09050	.09026	.09063	1.0041	.9959	.1810	.1820	1.016	.9973	.09001	2.357	605.8
.001400	.01495	.09393	.09365	.09407	1.0044	.9956	.1879	.1890	1.018	.9971	.09338	2.314	562.6
.001500	.01548	.09723	.09693	.09739	1.0047	.9953	.1945	.1957	1.019	.9969	.09663	2.275	525
.001600	.01598	.1004	.1001	.1006	1.0051	.9949	.2009	.2022	1.020	.9967	.09977	2.239	493
.001700	.01648	.1035	.1032	.1037	1.0054	.9946	.2071	.2086	1.022	.9965	.1028	2.205	463
.001800	.01696	.1066	.1062	.1068	1.0057	.9943	.2131	.2147	1.023	.9962	.1058	2.174	438
.001900	.01743	.1095	.1091	.1097	1.0060	.9940	.2190	.2207	1.024	.9960	.1087	2.145	415
.002000	.01788	.1123	.1119	.1125	1.0063	.9937	.2247	.2266	1.025	.9958	.1114	2.119	394
.002100	.01832	.1151	.1146	.1154	1.0066	.9934	.2303	.2323	1.027	.9956	.1141	2.094	376
.002200	.01876	.1178	.1173	.1181	1.0069	.9931	.2357	.2379	1.028	.9954	.1161	2.070	359
.002300	.01918	.1205	.1199	.1208	1.0073	.9928	.2410	.2433	1.029	.9952	.1193	2.047	343
.002400	.01959	.1231	.1225	.1234	1.0076	.9925	.2462	.2487	1.031	.9950	.1219	2.025	329
.002500	.02000	.1257	.1250	.1260	1.0079	.9922	.2513	.2540	1.032	.9948	.1243	2.005	316
.002600	.02040	.1282	.1275	.1285	1.0082	.9919	.2563	.2592	1.033	.9946	.1268	1.986	304
.002700	.02079	.1306	.1299	.1310	1.0085	.9916	.2612	.2642	1.034	.9944	.1292	1.967	292
.002800	.02117	.1330	.1323	.1334	1.0089	.9912	.2661	.2692	1.036	.9942	.1315	1.950	282
.002900	.02155	.1354	.1346	.1358	1.0092	.9909	.2708	.2741	1.037	.9939	.1338	1.933	272
.003000	.02192	.1377	.1369	.1382	1.0095	.9906	.2755	.2790	1.038	.9937	.1360	1.917	263
.003100	.02228	.1400	.1391	.1405	1.0098	.9903	.2800	.2837	1.040	.9935	.1382	1.902	255
.003200	.02264	.1423	.1413	.1427	1.0101	.9900	.2845	.2884	1.041	.9933	.1404	1.887	247
.003300	.02300	.1445	.1435	.1449	1.0104	.9897	.2890	.2930	1.042	.9931	.1425	1.873	240
.003400	.02335	.1467	.1456	.1472	1.0108	.9893	.2934	.2976	1.043	.9929	.1446	1.860	233
.003500	.02369	.1488	.1477	.1494	1.0111	.9890	.2977	.3021	1.045	.9927	.1466	1.847	226
.003600	.02403	.1510	.1498	.1515	1.0114	.9887	.3020	.3065	1.046	.9925	.1487	1.834	220
.003700	.02436	.1531	.1519	.1537	1.0117	.9884	.3061	.3109	1.047	.9923	.1507	1.822	214
.003800	.02469	.1551	.1539	.1558	1.0121	.9881	.3103	.3153	1.049	.9921	.1527	1.810	208
.003900	.02502	.1572	.1559	.1579	1.0124	.9878	.3144	.3196	1.050	.9919	.1546	1.799	203
.004000	.02534	.1592	.1579	.1599	1.0127	.9875	.3184	.3238	1.051	.9917	.1565	1.788	198
.004100	.02566	.1612	.1598	.1619	1.0130	.9872	.3224	.3280	1.052	.9915	.1584	1.777	193
.004200	.02597	.1632	.1617	.1639	1.0133	.9869	.3263	.3322	1.054	.9912	.1602	1.767	189
.004300	.02628	.1651	.1636	.1659	1.0137	.9865	.3302	.3362	1.055	.9910	.1621	1.756	184
.004400	.02659	.1671	.1655	.1678	1.0140	.9862	.3341	.3403	1.056	.9908	.1640	1.746	180
.004500	.02689	.1690	.1674	.1698	1.0143	.9859	.3380	.3444	1.058	.9906	.1658	1.737	176
.004600	.02719	.1708	.1692	.1717	1.0146	.9856	.3417	.3483	1.059	.9904	.1676	1.727	172
.004700	.02749	.1727	.1710	.1736	1.0149	.9853	.3454	.3523	1.060	.9902	.1693	1.718	169
.004800	.02778	.1745	.1728	.1754	1.0153	.9849	.3491	.3562	1.062	.9900	.1711	1.709	165
.004900	.02807	.1764	.1746	.1773	1.0156	.9846	.3527	.3601	1.063	.9898	.1728	1.701	162
.005000	.02836	.1782	.1764	.1791	1.0159	.9843	.3564	.3640	1.064	.9896	.1746	1.692	159
.005100	.02864	.1800	.1781	.1809	1.0162	.9840	.3599	.3678	1.066	.9894	.1762	1.684	156
.005200	.02893	.1818	.1798	.1827	1.0166	.9837	.3635	.3715	1.067	.9892	.1779	1.676	153
.005300	.02921	.1835	.1815	.1845	1.0169	.9834	.3670	.3753	1.068	.9889	.1795	1.669	150
.005400	.02948	.1852	.1832	.1863	1.0172	.9831	.3705	.3790	1.069	.9887	.1811	1.662	147
.005500	.02976	.1870	.1848	.1880	1.0175	.9828	.3739	.3827	1.071	.9885	.1827	1.654	145
.005600	.03003	.1887	.1865	.1898	1.0178	.9825	.3774	.3864	1.072	.9883	.1843	1.647	142
.005700	.03030	.1904	.1881	.1915	1.0182	.9822	.3808	.3900	1.073	.9881	.1859	1.640	140
.005800	.03057	.1921	.1897	.1932	1.0185	.9818	.3841	.3937	1.075	.9879	.1874	1.633	137
.005900	.03083	.1937	.1913	.1949	1.0188	.9815	.3875	.3972	1.076	.9877	.1890	1.626	135
.006000	.03110	.1954	.1929	.1967	1.0192	.9812	.3908	.4008	1.077	.9875	.1905	1.620	133
.006100	.03136	.1970	.1945	.1983	1.0195	.9809	.3941	.4044	1.079	.9873	.1920	1.614	130
.006200	.03162	.1987	.1961	.2000	1.0198	.9806	.3973	.4079	1.080	.9871	.1935	1.607	128
.006300	.03188	.2003	.1976	.2016	1.0201	.9803	.4006	.4114	1.081	.9869	.1950	1.601	126
.006400	.03213	.2019	.1992	.2033	1.0205	.9799	.4038	.4148	1.083	.9867	.1965	1.595	124
.006500	.03238	.2035	.2007	.2049	1.0208	.9796	.4070	.4183	1.084	.9865	.1980	1.589	123
.006600	.03264	.2051	.2022	.2065	1.0211	.9793	.4101	.4217	1.085	.9863	.1994	1.583	121
.006700	.03289	.2066	.2037	.2081	1.0214	.9790	.4133	.4251	1.087	.9860	.2009	1.578	119
.006800	.03313	.2082	.2052	.2097	1.0217	.9787	.4164	.4285	1.088	.9858	.2023	1.572	117
.006900	.03338	.2097	.2067	.2113	1.0221	.9784	.4195	.4319	1.089	.9856	.2037	1.567	116

d/L_o	d/L	$2\pi d/L$	tanh $2\pi d/L$	sinh $2\pi d/L$	cosh $2\pi d/L$	K	$4\pi d/L$	sinh $4\pi d/L$	cosh $4\pi d/L$	n	C_G/C_o	H/H_o'	M
.05000	.09416	.5916	.5310	.6267	1.1802	.8473	1.183	1.479	1.786	.8999	.4779	1.023	17.50
.05100	.09520	.5981	.5357	.6344	1.1843	.8444	1.196	1.503	1.805	.8980	.4811	1.019	17.19
.05200	.09623	.6046	.5403	.6421	1.1884	.8415	1.209	1.526	1.825	.8961	.4842	1.016	16.90
.05300	.09726	.6111	.5449	.6499	1.1926	.8385	1.222	1.550	1.845	.8943	.4873	1.013	16.62
.05400	.09829	.6176	.5494	.6575	1.1968	.8356	1.235	1.574	1.865	.8924	.4903	1.010	16.35
.05500	.09930	.6239	.5538	.6652	1.2011	.8326	1.248	1.598	1.885	.8905	.4932	1.007	16.09
.05600	.1003	.6303	.5582	.6729	1.2053	.8297	1.261	1.622	1.906	.8886	.4960	1.004	15.84
.05700	.1013	.6366	.5626	.6805	1.2096	.8267	1.273	1.646	1.926	.8867	.4988	1.001	15.60
.05800	.1023	.6428	.5668	.6880	1.2138	.8239	1.286	1.670	1.947	.8849	.5015	.9985	15.36
.05900	.1033	.6491	.5711	.6956	1.2181	.8209	1.298	1.695	1.968	.8830	.5042	.9958	15.13
.06000	.1043	.6553	.5753	.7033	1.2225	.8180	1.311	1.719	1.989	.8811	.5068	.9932	14.91
.06100	.1053	.6616	.5794	.7110	1.2270	.8150	1.3231	1.744	2.011	.8792	.5094	.9907	14.70
.06200	.1063	.6678	.5834	.7187	1.2315	.8121	1.336	1.770	2.033	.8773	.5119	.9883	14.50
.06300	.1073	.6739	.5874	.7256	1.2355	.8093	1.348	1.795	2.055	.8755	.5143	.9860	14.30
.06400	.1082	.6799	.5914	.7335	1.2402	.8063	1.360	1.819	2.076	.8737	.5167	.9837	14.11
.06500	.1092	.6860	.5954	.7411	1.2447	.8035	1.372	1.845	2.098	.8719	.5191	.9815	13.92
.06600	.1101	.6920	.5993	.7486	1.2492	.8005	1.384	1.870	2.121	.8700	.5214	.9793	13.74
.06700	.1111	.6981	.6031	.7561	1.2537	.7977	1.396	1.896	2.144	.8682	.5236	.9772	13.57
.06800	.1120	.7037	.6069	.7633	1.2580	.7948	1.408	1.921	2.166	.8664	.5258	.9752	13.40
.06900	.1130	.7099	.6106	.7711	1.2628	.7919	1.420	1.948	2.189	.8646	.5279	.9732	13.24
.07000	.1139	.7157	.6144	.7783	1.2672	.7890	1.432	1.974	2.213	.8627	.5300	.9713	13.08
.07100	.1149	.7219	.6181	.7863	1.2721	.7861	1.444	2.000	2.236	.8609	.5321	.9694	12.92
.07200	.1158	.7277	.6217	.7937	1.2767	.7833	1.455	2.026	2.260	.8591	.5341	.9676	12.77
.07300	.1168	.7336	.6252	.8011	1.2813	.7804	1.467	2.053	2.284	.8572	.5360	.9658	12.62
.07400	.1177	.7395	.6289	.8088	1.2861	.7775	1.479	2.080	2.308	.8554	.5380	.9641	12.48
.07500	.1186	.7453	.6324	.8162	1.2908	.7747	1.490	2.107	2.332	.8537	.5399	.9624	12.34
.07600	.1195	.7511	.6359	.8237	1.2956	.7719	1.502	2.135	2.357	.8519	.5417	.9607	12.21
.07700	.1205	.7569	.6392	.8312	1.3004	.7690	1.514	2.162	2.382	.8501	.5435	.9591	12.08
.07800	.1214	.7625	.6427	.8386	1.3051	.7662	1.525	2.189	2.407	.8483	.5452	.9576	11.95
.07900	.1223	.7683	.6460	.8462	1.3100	.7634	1.537	2.217	2.432	.8465	.5469	.9562	11.83
.08000	.1232	.7741	.6493	.8538	1.3149	.7605	1.548	2.245	2.458	.8448	.5485	.9548	11.71
.08100	.1241	.7799	.6526	.8614	1.3198	.7577	1.560	2.274	2.484	.8430	.5501	.9534	11.59
.08200	.1251	.7854	.6558	.8687	1.3246	.7549	1.571	2.303	2.511	.8413	.5517	.9520	11.47
.08300	.1259	.7911	.6590	.8762	1.3295	.7522	1.583	2.331	2.537	.8395	.5533	.9506	11.36
.08400	.1268	.7967	.6622	.8837	1.3345	.7494	1.594	2.360	2.563	.8378	.5548	.9493	11.25
.08500	.1277	.8026	.6655	.8915	1.3397	.7464	1.605	2.389	2.590	.8360	.5563	.9481	11.14
.08600	.1286	.8080	.6685	.8989	1.3446	.7437	1.616	2.418	2.617	.8342	.5577	.9469	11.04
.08700	.1295	.8137	.6716	.9064	1.3497	.7409	1.628	2.448	2.644	.8325	.5591	.9457	10.94
.08800	.1304	.8193	.6747	.9141	1.3548	.7381	1.639	2.478	2.672	.8308	.5605	.9445	10.84
.08900	.1313	.8250	.6778	.9218	1.3600	.7353	1.650	2.508	2.700	.8290	.5619	.9433	10.74
.09000	.1322	.8306	.6808	.9295	1.3653	.7324	1.661	2.538	2.728	.8273	.5632	.9422	10.65
.09100	.1331	.8363	.6838	.9372	1.3706	.7296	1.672	2.568	2.756	.8255	.5645	.9411	10.55
.09200	.1340	.8420	.6868	.9450	1.3759	.7268	1.684	2.599	2.785	.8238	.5658	.9401	10.46
.09300	.1349	.8474	.6897	.9525	1.3810	.7241	1.695	2.630	2.814	.8221	.5670	.9391	10.37
.09400	.1357	.8528	.6925	.9600	1.3862	.7214	1.706	2.662	2.843	.8204	.5682	.9381	10.29
.09500	.1366	.8583	.6953	.9677	1.3917	.7186	1.717	2.693	2.873	.8187	.5693	.9371	10.21
.09600	.1375	.8639	.6982	.9755	1.3970	.7158	1.728	2.726	2.903	.8170	.5704	.9362	10.12
.09700	.1384	.8694	.7011	.9832	1.4023	.7131	1.739	2.757	2.933	.8153	.5716	.9353	10.04
.09800	.1392	.8749	.7039	.9908	1.4077	.7104	1.750	2.790	2.963	.8136	.5727	.9344	9.962
.09900	.1401	.8803	.7066	.9985	1.4131	.7076	1.761	2.822	2.994	.8120	.5737	.9335	9.884
.1000	.1410	.8858	.7093	1.006	1.4187	.7049	1.772	2.855	3.025	.8103	.5747	.9327	9.808
.1010	.1419	.8913	.7120	1.014	1.4242	.7022	1.783	2.888	3.057	.8086	.5757	.9319	9.734
.1020	.1427	.8967	.7147	1.022	1.4297	.6994	1.793	2.922	3.088	.8069	.5766	.9311	9.661
.1030	.1436	.9023	.7173	1.030	1.4354	.6967	1.805	2.956	3.121	.8052	.5776	.9304	9.590
.1040	.1445	.9076	.7200	1.037	1.4410	.6940	1.815	2.990	3.153	.8036	.5785	.9297	9.519
.1050	.1453	.9130	.7226	1.045	1.4465	.6913	1.826	3.024	3.185	.8019	.5794	.9290	9.451
.1060	.1462	.9184	.7252	1.053	1.4523	.6886	1.837	3.059	3.218	.8003	.5803	.9282	9.384
.1070	.1470	.9239	.7277	1.061	1.4580	.6859	1.848	3.094	3.251	.7986	.5812	.9276	9.318
.1080	.1479	.9293	.7303	1.069	1.4638	.6833	1.858	3.128	3.284	.7970	.5820	.9269	9.254
.1090	.1488	.9343	.7327	1.076	1.4692	.6806	1.869	3.164	3.319	.7954	.5828	.9263	9.191
.1100	.1496	.9400	.7352	1.085	1.4752	.6779	1.880	3.201	3.353	.7937	.5836	.9257	9.129
.1110	.1505	.9456	.7377	1.093	1.4814	.6752	1.891	3.237	3.388	.7920	.5843	.9251	9.068
.1120	.1513	.9508	.7402	1.101	1.4871	.6725	1.902	3.274	3.423	.7904	.5850	.9245	9.009
.1130	.1522	.9563	.7426	1.109	1.4932	.6697	1.913	3.312	3.459	.7888	.5857	.9239	8.950
.1140	.1530	.9616	.7450	1.117	1.4990	.6671	1.923	3.348	3.494	.7872	.5864	.9234	8.891
.1150	.1539	.9670	.7474	1.125	1.5051	.6645	1.934	3.385	3.530	.7856	.5871	.9228	8.835
.1160	.1547	.9720	.7497	1.133	1.5108	.6619	1.944	3.423	3.566	.7840	.5878	.9223	8.780
.1170	.1556	.9775	.7520	1.141	1.5171	.6592	1.955	3.462	3.603	.7824	.5884	.9218	8.726
.1180	.1564	.9827	.7543	1.149	1.5230	.6566	1.966	3.501	3.641	.7808	.5890	.9214	8.673
.1190	.1573	.9882	.7566	1.157	1.5293	.6539	1.977	3.540	3.678	.7792	.5896	.9209	8.621

APPENDIX 1. (Continued)

d/L_o	d/L	$2\pi d/L$	tanh $2\pi d/L$	sinh $2\pi d/L$	cosh $2\pi d/L$	K	$4\pi d/L$	sinh $4\pi d/L$	cosh $4\pi d/L$	n	C_G/C_o	H/H_o'	M
	.1581	.9936	.7589	1.165	1.5356	.6512	1.987	3.579	3.716	.7776	.5902	.9204	8.569
[illegible]	.1590	.9989	.7612	1.174	1.5418	.6486	1.998	3.620	3.755	.7760	.5907	.9200	8.518
.1220	.1598	1.004	.7634	1.182	1.5479	.6460	2.008	3.659	3.793	.7745	.5913	.9196	8.468
.1230	.1607	1.010	.7656	1.190	1.5546	.6433	2.019	3.699	3.832	.7729	.5918	.9192	8.419
.1240	.1615	1.015	.7678	1.198	1.5605	.6407	2.030	3.740	3.871	.7713	.5922	.9189	8.371
.1250	.1624	1.020	.7700	1.207	1.5674	.6381	2.041	3.782	3.912	.7698	.5926	.9186	8.324
.1260	.1632	1.025	.7721	1.215	1.5734	.6356	2.051	3.824	3.952	.7682	.5931	.9182	8.278
.1270	.1640	1.030	.7742	1.223	1.5795	.6331	2.061	3.865	3.992	.7667	.5936	.9178	8.233
.1280	.1649	1.036	.7763	1.231	1.5862	.6305	2.072	3.907	4.033	.7652	.5940	.9175	8.189
.1290	.1657	1.041	.7783	1.240	1.5927	.6279	2.082	3.950	4.074	.7637	.5944	.9172	8.146
.1300	.1665	1.046	.7804	1.248	1.5990	.6254	2.093	3.992	4.115	.7621	.5948	.9169	8.103
.1310	.1674	1.052	.7824	1.257	1.6060	.6228	2.104	4.036	4.158	.7606	.5951	.9166	8.061
.1320	.1682	1.057	.7844	1.265	1.6124	.6202	2.114	4.080	4.201	.7591	.5954	.9164	8.020
.1330	.1691	1.062	.7865	1.273	1.6191	.6176	2.125	4.125	4.245	.7575	.5958	.9161	7.978
.1340	.1699	1.068	.7885	1.282	1.6260	.6150	2.135	4.169	4.288	.7560	.5961	.9158	7.937
.1350	.1708	1.073	.7905	1.291	1.633	.6123	2.146	4.217	4.334	.7545	.5964	.9156	7.897
.1360	.1716	1.078	.7925	1.300	1.640	.6098	2.156	4.262	4.378	.7530	.5967	.9154	7.857
.1370	.1724	1.084	.7945	1.308	1.647	.6073	2.167	4.309	4.423	.7515	.5969	.9152	7.819
.1380	.1733	1.089	.7964	1.317	1.654	.6047	2.177	4.355	4.468	.7500	.5972	.9150	7.781
.1390	.1741	1.094	.7983	1.326	1.660	.6022	2.188	4.402	4.514	.7485	.5975	.9148	7.744
.1400	.1749	1.099	.8002	1.334	1.667	.5998	2.198	4.450	4.561	.7471	.5978	.9146	7.707
.1410	.1758	1.105	.8021	1.343	1.675	.5972	2.209	4.498	4.607	.7456	.5980	.9144	7.671
.1420	.1766	1.110	.8039	1.352	1.681	.5947	2.219	4.546	4.654	.7441	.5982	.9142	7.636
.1430	.1774	1.115	.8057	1.360	1.688	.5923	2.230	4.595	4.663	.7426	.5984	.9141	7.602
.1440	.1783	1.120	.8076	1.369	1.696	.5898	2.240	4.644	4.751	.7412	.5986	.9140	7.567
.1450	.1791	1.125	.8094	1.378	1.703	.5873	2.251	4.695	4.800	.7397	.5987	.9139	7.533
.1460	.1800	1.131	.8112	1.388	1.710	.5847	2.261	4.746	4.850	.7382	.5989	.9137	7.499
.1470	.1808	1.136	.8131	1.397	1.718	.5822	2.272	4.798	4.901	.7368	.5990	.9136	7.465
.1480	.1816	1.141	.8149	1.405	1.725	.5798	2.282	4.847	4.951	.7354	.5992	.9135	7.432
.1490	.1825	1.146	.8166	1.415	1.732	.5773	2.293	4.901	5.001	.7339	.5993	.9134	7.400
.1500	.1833	1.152	.8183	1.424	1.740	.5748	2.303	4.954	5.054	.7325	.5994	.9133	7.369
.1510	.1841	1.157	.8200	1.433	1.747	.5723	2.314	5.007	5.106	.7311	.5994	.9133	7.339
.1520	.1850	1.162	.8217	1.442	1.755	.5699	2.324	5.061	5.159	.7296	.5995	.9132	7.309
.1530	.1858	1.167	.8234	1.451	1.762	.5675	2.335	5.115	5.212	.7282	.5996	.9132	7.279
	.1866	1.173	.8250	1.460	1.770	.5651	2.345	5.169	5.265	.7268	.5996	.9132	7.250
550	.1875	1.178	.8267	1.469	1.777	.5627	2.356	5.225	5.320	.7254	.5997	.9131	7.221
.1560	.1883	1.183	.8284	1.479	1.785	.5602	2.366	5.283	5.376	.7240	.5998	.9130	7.191
.1570	.1891	1.188	.8301	1.488	1.793	.5577	2.377	5.339	5.432	.7226	.5999	.9129	7.162
.1580	.1900	1.194	.8317	1.498	1.801	.5552	2.387	5.398	5.490	.7212	.5998	.9130	7.134
.1590	.1908	1.199	.8333	1.507	1.809	.5528	2.398	5.454	5.544	.7198	.5998	.9130	7.107
.1600	.1917	1.204	.8349	1.517	1.817	.5504	2.408	5.513	5.603	.7184	.5998	.9130	7.079
.1610	.1925	1.209	.8365	1.527	1.825	.5480	2.419	5.571	5.660	.7171	.5998	.9130	7.052
.1620	.1933	1.215	.8381	1.536	1.833	.5456	2.429	5.630	5.718	.7157	.5998	.9130	7.026
.1630	.1941	1.220	.8396	1.546	1.841	.5432	2.440	5.690	5.777	.7144	.5998	.9130	7.000
.1640	.1950	1.225	.8411	1.555	1.849	.5409	2.450	5.751	5.837	.7130	.5998	.9130	6.975
.1650	.1958	1.230	.8427	1.565	1.857	.5385	2.461	5.813	5.898	.7117	.5997	.9131	6.949
.1660	.1966	1.235	.8442	1.574	1.865	.5362	2.471	5.874	5.959	.7103	.5996	.9132	6.924
.1670	.1975	1.240	.8457	1.584	1.873	.5339	2.482	5.938	6.021	.7090	.5996	.9132	6.900
.1680	.1983	1.246	.8472	1.594	1.882	.5315	2.492	6.003	6.085	.7076	.5995	.9133	6.876
.1690	.1992	1.251	.8486	1.604	1.890	.5291	2.503	6.066	6.148	.7063	.5994	.9133	6.853
.1700	.2000	1.257	.8501	1.614	1.899	.5267	2.513	6.130	6.212	.7050	.5993	.9134	6.830
.1710	.2008	1.262	.8515	1.624	1.907	.5243	2.523	6.197	6.275	.7036	.5992	.9135	6.807
.1720	.2017	1.267	.8529	1.634	1.915	.5220	2.534	6.262	6.342	.7023	.5991	.9136	6.784
.1730	.2025	1.272	.8544	1.644	1.924	.5197	2.544	6.329	6.407	.7010	.5989	.9137	6.761
.1740	.2033	1.277	.8558	1.654	1.933	.5174	2.555	6.395	6.473	.6997	.5988	.9138	6.738
.1750	.2042	1.282	.8572	1.664	1.941	.5151	2.565	6.465	6.541	.6984	.5987	.9139	6.716
.1760	.2050	1.288	.8586	1.675	1.951	.5127	2.576	6.534	6.610	.6971	.5985	.9140	6.694
.1770	.2058	1.293	.8600	1.685	1.959	.5104	3.586	6.603	6.679	.6958	.5984	.9141	6.672
.1780	.2066	1.298	.8614	1.695	1.968	.5081	2.597	6.672	6.747	.6946	.5982	.9142	6.651
.1790	.2075	1.304	.8627	1.706	1.977	.5058	2.607	6.744	6.818	.6933	.5980	.9144	6.631
.1800	.2083	1.309	.8640	1.716	1.986	.5036	2.618	6.818	6.891	.6920	.5979	.9145	6.611
.1810	.2092	1.314	.8653	1.727	1.995	.5013	2.629	6.890	6.963	.6907	.5977	.9146	6.591
.1820	.2100	1.320	.8666	1.737	2.004	.4990	2.639	6.963	7.035	.6895	.5975	.9148	6.571
.1830	.2108	1.325	.8680	1.748	2.013	.4967	2.650	7.038	7.109	.6882	.5974	.9149	6.550
.1840	.2117	1.330	.8693	1.758	2.022	.4945	2.660	7.113	7.183	.6870	.5972	.9150	6.530
.1850	.2125	1.335	.8706	1.769	2.032	.4922	2.671	7.191	7.260	.6857	.5969	.9152	6.511
.1860	.2134	1.341	.8718	1.780	2.041	.4899	2.681	7.267	7.336	.6845	.5967	.9154	6.492
.1	.2142	1.346	.8731	1.791	2.051	.4876	2.692	7.345	7.412	.6832	.5965	.9155	6.474
	.2150	1.351	.8743	1.801	2.060	.4854	2.702	7.421	7.488	.6820	.5963	.9157	6.456
90	.2159	1.356	.8755	1.812	2.070	.4832	2.712	7.500	7.566	.6808	.5961	.9159	6.438

APPENDIX 1. (Continued)

d/L_o	d/L	$2\pi d/L$	tanh $2\pi d/L$	sinh $2\pi d/L$	cosh $2\pi d/L$	K	$4\pi d/L$	sinh $4\pi d/L$	cosh $4\pi d/L$	n	C_G/C_o	H/H_o	M
.1900	.2167	1.362	.8767	1.823	2.079	.4809	2.723	7.581	7.647	.6796	.5958	.9161	6.421
.1910	.2176	1.367	.8779	1.834	2.089	.4787	2.734	7.663	7.728	.6784	.5955	.9163	6.403
.1920	.2184	1.372	.8791	1.845	2.099	.4765	2.744	7.746	7.810	.6772	.5952	.9165	6.385
.1930	.2192	1.377	.8803	1.856	2.108	.4743	2.755	7.827	7.891	.6760	.5950	.9167	6.368
.1940	.2201	1.383	.8815	1.867	2.118	.4721	2.765	7.911	7.974	.6748	.5948	.9169	6.351
.1950	.2209	1.388	.8827	1.879	2.128	.4699	2.776	7.996	8.059	.6736	.5946	.9170	6.334
.1960	.2218	1.393	.8839	1.890	2.138	.4677	2.787	8.083	8.145	.6724	.5944	.9172	6.317
.1970	.2226	1.399	.8850	1.901	2.148	.4655	2.797	8.167	8.228	.6712	.5941	.9174	6.300
.1980	.2234	1.404	.8862	1.913	2.158	.4633	2.808	8.256	8.316	.6700	.5938	.9176	6.284
.1990	.2243	1.409	.8873	1.924	2.169	.4611	2.819	8.346	8.406	.6689	.5935	.9179	6.268
.2000	.2251	1.414	.8884	1.935	2.178	.4590	2.829	8.436	8.495	.6677	.5932	.9181	6.253
.2010	.2260	1.420	.8895	1.947	2.189	.4569	2.840	8.524	8.583	.6666	.5929	.9183	6.237
.2020	.2268	1.425	.8906	1.959	2.199	.4547	2.850	8.616	8.674	.6654	.5926	.9186	6.222
.2030	.2277	1.430	.8917	1.970	2.210	.4526	2.861	8.708	8.766	.6642	.5923	.9188	6.206
.2040	.2285	1.436	.8928	1.982	2.220	.4504	2.872	8.803	8.860	.6631	.5920	.9190	6.191
.2050	.2293	1.441	.8939	1.994	2.231	.4483	2.882	8.897	8.953	.6620	.5917	.9193	6.176
.2060	.2302	1.446	.8950	2.006	2.242	.4462	2.893	8.994	9.050	.6608	.5914	.9195	6.161
.2070	.2310	1.451	.8960	2.017	2.252	.4441	2.903	9.090	9.144	.6597	.5911	.9197	6.147
.2080	.2319	1.457	.8971	2.030	2.263	.4419	2.914	9.187	9.240	.6586	.5908	.9200	6.133
.2090	.2328	1.462	.8981	2.042	2.274	.4398	2.925	9.288	9.342	.6574	.5905	.9202	6.119
.2100	.2336	1.468	.8991	2.055	7.285	.4377	2.936	9.389	9.442	.6563	.5901	.9205	6.105
.2110	.2344	1.473	.9001	2.066	2.295	.4357	2.946	9.490	9.542	.6552	.5898	.9207	6.091
.2120	.2353	1.479	.9011	2.079	2.307	.4336	2.957	9.590	9.642	.6541	.5894	.9210	6.077
.2130	.2361	1.484	.9021	2.091	2.318	.4315	2.967	9.693	9.744	.6531	.5891	.9213	6.064
.2140	.2370	1.489	.9031	2.103	2.329	.4294	2.978	9.796	9.847	.6520	.5888	.9215	6.051
.2150	.2378	1.494	.9041	2.115	2.340	.4274	2.989	9.902	9.952	.6509	.5884	.9218	6.037
.2160	.2387	1.500	.9051	2.128	2.351	.4253	2.999	10.01	10.06	.6498	.5881	.9221	6.024
.2170	.2395	1.506	.9061	2.142	2.364	.4232	3.010	10.12	10.17	.6488	.5878	.9223	6.011
.2180	.2404	1.511	.9070	2.154	2.375	.4211	3.021	10.23	10.28	.6477	.5874	.9226	5.999
.2190	.2412	1.516	.9079	2.166	2.386	.4191	3.031	10.34	10.38	.6467	.5871	.9228	5.987
.2200	.2421	1.521	.9088	2.178	2.397	.4171	3.042	10.45	10.50	.6456	.5868	.9231	5.975
.2210	.2429	1.526	.9097	2.192	2.409	.4151	3.052	10.56	10.61	.6446	.5864	.9234	5.963
.2220	.2438	1.532	.9107	2.204	2.421	.4131	3.063	10.68	10.72	.6436	.5861	.9236	5.951
.2230	.2446	1.537	.9116	2.218	2.433	.4111	3.074	10.79	10.84	.6425	.5857	.9239	5.939
.2240	.2455	1.542	.9125	2.230	2.444	.4091	3.085	10.91	10.95	.6414	.5854	.9242	5.927
.2250	.2463	1.548	.9134	1.244	2.457	.4071	3.095	11.02	11.07	.6404	.5850	.9245	5.915
.2260	.2472	1.553	.9143	2.257	2.469	.4051	3.106	11.15	11.19	.6394	.5846	.9248	5.903
.2270	.2481	1.559	.9152	2.271	2.481	.4031	3.117	11.27	11.31	.6383	.5842	.9251	5.891
.2280	.2489	1.564	.9161	2.284	2.493	.4011	3.128	11.39	11.44	.6373	.5838	.9254	5.880
.2290	.2498	1.569	.9170	2.297	2.506	.3991	3.138	11.51	11.56	.6363	.5834	.9258	5.869
.2300	.2506	1.575	.9178	2.311	2.518	.3971	3.149	11.64	11.68	.6353	.5830	.9261	5.858
.2310	.2515	1.580	.9186	2.325	2.531	.3952	3.160	11.77	11.81	.6343	.5826	.9264	5.848
.2320	.2523	1.585	.9194	2.338	2.543	.3932	3.171	11.90	11.93	.6333	.5823	.9267	5.838
.2330	.2532	1.591	.9203	2.352	2.556	.3912	3.182	12.03	12.07	.6323	.5819	.9270	5.827
.2340	.2540	1.596	.9211	2.366	2.569	.3893	3.192	12.15	12.19	.6313	.5815	.9273	5.816
.2350	.2549	1.602	.9219	2.380	2.581	.3874	3.203	12.29	12.33	.6304	.5811	.9276	5.806
.2360	.2558	1.607	.9227	2.393	2.594	.3855	3.214	12.43	12.47	.6294	.5807	.9279	5.796
.2370	.2566	1.612	.9235	2.408	2.607	.3836	3.225	12.55	12.59	.6284	.5804	.9282	5.786
.2380	.2575	1.618	.9243	2.422	2.620	.3816	3.236	12.69	12.73	.6275	.5800	.9285	5.776
.2390	.2584	1.623	.9251	2.436	2.634	.3797	3.247	12.83	12.87	.6265	.5796	.9288	5.766
.2400	.2592	1.629	.9259	2.450	2.647	.3779	3.257	12.97	13.01	.6256	.5792	.9291	5.756
.2410	.2601	1.634	.9267	2.464	2.660	.3760	3.268	13.11	13.15	.6246	.5788	.9294	5.746
.2420	.2610	1.640	.9275	2.480	2.674	.3741	3.279	13.26	13.30	.6237	.5784	.9298	5.736
.2430	.2618	1.645	.9282	2.494	2.687	.3722	3.290	13.40	13.44	.6228	.5780	.9301	5.727
.2440	.2627	1.650	.9289	2.508	2.700	.3704	3.301	13.55	13.59	.6218	.5776	.9304	5.718
.2450	.2635	1.656	.9296	2.523	2.714	.3685	3.312	13.70	13.73	.6209	.5772	.9307	5.710
.2460	.2644	1.661	.9304	2.538	2.728	.3666	3.323	13.85	13.88	.6200	.5768	.9310	5.701
.2470	.2653	1.667	.9311	2.553	2.742	.3648	3.334	14.00	14.04	.6191	.5764	.9314	5.692
.2480	.2661	1.672	.9318	2.568	2.755	.3629	3.344	14.15	14.19	.6182	.5760	.9317	5.684
.2490	.2670	1.678	.9325	2.583	2.770	.3610	3.355	14.31	14.35	.6173	.5756	.9320	5.675
.2500	.2679	1.683	.9332	2.599	2.784	.3592	3.367	14.47	14.51	.6164	.5752	.9323	5.667
.2510	.2687	1.689	.9339	2.614	2.798	.3574	3.377	14.62	14.66	.6155	.5748	.9327	5.658
.2520	.2696	1.694	.9346	2.629	2.813	.3556	3.388	14.79	14.82	.6146	.5744	.9330	5.650
.2530	.2705	1.700	.9353	2.645	2.828	.3537	3.399	14.95	14.99	.6137	.5740	.9333	5.641
.2540	.2714	1.705	.9360	2.660	2.842	.3519	3.410	15.12	15.15	.6128	.5736	.9336	5.633
.2550	.2722	1.711	.9367	2.676	2.856	.3501	3.421	15.29	15.32	.6120	.5732	.9340	5.624
.2560	.2731	1.716	.9374	2.691	2.871	.3483	3.432	15.45	15.49	.6111	.5728	.9343	5.616
.2570	.2740	1.722	.9381	2.707	2.886	.3465	3.443	15.63	15.66	.6102	.5724	.9346	5.608
.2580	.2749	1.727	.9388	2.723	2.901	.3447	3.454	15.80	15.83	.6093	.5720	.9349	5.600
.2590	.2757	1.732	.9394	2.739	2.916	.3430	3.465	15.97	16.00	.6085	.5716	.9353	5.592

APPENDIX 1. (Continued)

d/L_o	d/L	$2\pi d/L$	tanh $2\pi d/L$	sinh $2\pi d/L$	cosh $2\pi d/L$	K	$4\pi d/L$	sinh $4\pi d/L$	cosh $4\pi d/L$	n	C_G/C_o	H/H_o'	M
.2600	.2766	1.738	.9400	2.755	2.931	.3412	3.476	16.15	16.18	.6076	.5712	.9356	5.585
.2610	.2775	1.744	.9406	2.772	2.946	.3394	3.487	16.33	16.36	.6068	.5707	.9360	5.578
.2620	.2784	1.749	.9412	2.788	2.962	.3376	3.498	16.51	16.54	.6060	.5703	.9363	5.571
.2630	.2792	1.755	.9418	2.804	2.977	.3359	3.509	16.69	16.73	.6052	.5699	.9367	5.563
.2640	.2801	1.760	.9425	2.820	2.992	.3342	3.520	16.88	16.91	.6043	.5695	.9370	5.556
.2650	.2810	1.766	.9431	2.837	3.008	.3325	3.531	17.07	17.10	.6035	.5691	.9373	5.548
.2660	.2819	1.771	.9437	2.853	3.023	.3308	3.542	17.26	17.28	.6027	.5687	.9377	5.541
.2670	.2827	1.776	.9443	2.870	3.039	.3291	3.553	17.45	17.45	.6018	.5683	.9380	5.534
.2680	.2836	1.782	.9449	2.886	3.055	.3274	3.564	17.64	17.67	.6010	.5679	.9383	5.527
.2690	.2845	1.788	.9455	2.904	3.071	.3256	3.575	17.84	17.87	.6002	.5675	.9386	5.520
.2700	.2854	1.793	.9461	2.921	3.088	.3239	3.587	18.04	18.07	.5994	.5671	.9390	5.513
.2710	.2863	1.799	.9467	2.938	3.104	.3222	3.598	18.24	18.27	.5986	.5667	.9393	5.506
.2720	.2872	1.804	.9473	2.956	3.120	.3205	3.610	18.46	18.49	.5978	.5663	.9396	5.499
.2730	.2880	1.810	.9478	2.973	3.136	.3189	3.620	18.65	18.67	.5971	.5659	.9400	5.493
.2740	.2889	1.815	.9484	2.990	3.153	.3172	3.631	18.86	18.89	.5963	.5655	.9403	5.486
.2750	.2898	1.821	.9490	3.008	3.170	.3155	3.642	19.07	19.10	.5955	.5651	.9406	5.480
.2760	.2907	1.826	.9495	3.025	3.186	.3139	3.653	19.28	19.30	.5947	.5647	.9410	5.474
.2770	.2916	1.832	.9500	3.043	3.203	.3122	3.664	19.49	19.51	.5940	.5643	.9413	5.468
.2780	.2924	1.837	.9505	3.061	3.220	.3106	3.675	19.71	19.74	.5932	.5639	.9416	5.462
.2790	.2933	1.843	.9511	3.079	3.237	.3089	3.686	19.93	19.96	.5925	.5635	.9420	5.456
.2800	.2942	1.849	.9516	3.097	3.254	.3073	3.697	20.16	20.18	.5917	.5631	.9423	5.450
.2810	.2951	1.854	.9521	3.115	3.272	.3057	3.709	20.39	20.41	.5910	.5627	.9426	5.444
.2820	.2960	1.860	.9526	3.133	3.289	.3040	3.720	20.62	20.64	.5902	.5623	.9430	5.438
.2830	.2969	1.866	.9532	3.152	3.307	.3024	3.731	20.85	20.87	.5895	.5619	.9433	5.432
.2840	.2978	1.871	.9537	3.171	3.325	.3008	3.742	21.09	21.11	.5887	.5615	.9436	5.426
.2850	.2987	1.877	.9542	3.190	3.343	.2992	3.754	21.33	21.35	.5880	.5611	.9440	5.420
.2860	.2996	1.882	.9547	3.209	3.361	.2976	3.765	21.57	21.59	.5873	.5607	.9443	5.414
.2870	.3005	1.888	.9552	3.228	3.379	.2959	3.776	21.82	21.84	.5866	.5603	.9446	5.409
.2880	.3014	1.893	.9557	3.246	3.396	.2944	3.787	22.05	22.07	.5859	.5600	.9449	5.403
.2890	.3022	1.899	.9562	3.264	3.414	.2929	3.798	22.30	22.32	.5852	.5596	.9452	5.397
.2900	.3031	1.905	.9567	3.284	3.433	.2913	3.809	22.54	22.57	.5845	.5592	.9456	5.392
.2910	.3040	1.910	.9572	3.303	3.451	.2898	3.821	22.81	22.83	.5838	.5588	.9459	5.386
.2920	.3049	1.916	.9577	3.323	3.471	.2882	3.832	23.07	23.09	.5831	.5584	.9463	5.380
.2930	.3058	1.922	.9581	3.343	3.490	.2866	3.843	23.33	23.35	.5824	.5580	.9466	5.375
.2940	.3067	1.927	.9585	3.362	3.508	.2851	3.855	23.60	23.62	.5817	.5576	.9469	5.371
.2950	.3076	1.933	.9590	3.382	3.527	.2835	3.866	23.86	23.88	.5810	.5572	.9473	5.366
.2960	.3085	1.938	.9594	3.402	3.546	.2820	3.877	24.12	24.15	.5804	.5568	.9476	5.361
.2970	.3094	1.944	.9599	3.422	3.565	.2805	3.888	24.40	24.42	.5797	.5564	.9480	5.356
.2980	.3103	1.950	.9603	3.442	3.585	.2790	3.900	24.68	24.70	.5790	.5560	.9483	5.351
.2990	.3112	1.955	.9607	3.462	3.604	.2275	3.911	24.96	24.98	.5784	.5556	.9486	5.547
.3000	.3121	1.961	.9611	3.483	3.624	.2760	3.922	25.24	25.26	.5777	.5552	.9490	5.342
.3010	.3130	1.967	.9616	3.503	3.643	.2745	3.933	25.53	25.55	.5771	.5549	.9493	5.337
.3020	.3139	1.972	.9620	3.524	3.663	.2730	3.945	25.82	25.83	.5764	.5545	.9496	5.332
.3030	.3148	1.978	.9624	3.545	3.683	.2715	3.956	26.12	26.14	.5758	.5541	.9499	5.328
.3040	.3157	1.984	.9629	3.566	3.703	.2700	3.968	26.42	26.44	.5751	.5538	.9502	5.323
.3050	.3166	1.989	.9633	3.587	3.724	.2685	3.979	26.72	26.74	.5745	.5534	.9505	5.318
.3060	.3175	1.995	.9637	3.609	3.745	.2670	3.990	27.02	27.04	.5739	.5530	.9509	5.314
.3070	.3184	2.001	.9641	3.630	3.765	.2656	4.002	27.33	27.35	.5732	.5527	.9512	5.309
.3080	.3193	2.007	.9645	3.651	3.786	.2641	4.013	27.65	27.66	.5726	.5523	.9515	5.305
.3090	.3202	2.012	.9649	3.673	3.806	.2627	4.024	27.96	27.98	.5720	.5519	.9518	5.300
.3100	.3211	2.018	.9653	3.694	3.827	.2613	4.036	28.28	28.30	.5714	.5515	.9522	5.296
.3110	.3220	2.023	.9656	3.716	3.848	.2599	4.047	28.60	28.62	.5708	.5511	.9525	5.292
.3120	.3230	2.029	.9660	3.738	3.870	.2584	4.058	28.93	28.95	.5701	.5507	.9528	5.288
.3130	.3239	2.035	.9664	3.760	3.891	.2570	4.070	29.27	29.28	.5695	.5504	.9531	5.284
.3140	.3248	2.041	.9668	3.782	3.912	.2556	4.081	29.60	29.62	.5689	.5500	.9535	5.280
.3150	.3257	2.046	.9672	3.805	3.934	.2542	4.093	29.94	29.96	.5683	.5497	.9538	5.276
.3160	.3266	2.052	.9676	3.828	3.956	.2528	4.104	30.29	30.31	.5678	.5494	.9541	5.272
.3170	.3275	2.058	.9679	3.851	3.978	.2514	4.116	30.64	30.65	.5672	.5490	.9544	5.268
.3180	.3284	2.063	.9682	3.873	4.000	.2500	4.127	30.99	31.00	.5666	.5486	.9547	5.264
.3190	.3294	2.069	.9686	3.896	4.022	.2486	4.139	31.35	31.37	.5660	.5483	.9550	5.260
.3200	.3302	2.075	.9690	3.919	4.045	.2472	4.150	31.71	31.72	.5655	.5479	.9553	5.256
.3210	.3311	2.081	.9693	3.943	4.068	.2459	4.161	32.07	32.08	.5649	.5476	.9556	5.252
.3220	.3321	2.086	.9696	3.966	4.090	.2445	4.173	32.44	32.46	.5643	.5472	.9559	5.249
.3230	.3330	2.092	.9700	3.990	4.114	.2431	4.185	32.83	32.84	.5637	.5468	.9562	5.245
.3240	.3339	2.098	.9703	4.014	4.136	.2418	4.196	33.20	33.22	.5632	.5465	.9565	5.241
50	.3349	2.104	.9707	4.038	4.160	.2404	4.208	33.60	33.61	.5627	.5462	.9568	5.237
.3260	.3357	2.110	.9710	4.061	4.183	.2391	4.219	33.97	33.99	.5621	.5458	.9571	5.234
.3270	.3367	2.115	.9713	4.085	4.206	.2378	4.231	34.37	34.38	.5616	.5455	.9574	5.231
.3280	.3376	2.121	.9717	4.110	4.230	.2364	4.242	34.77	34.79	.5610	.5451	.9577	5.227
.3290	.3385	2.127	.9720	4.135	4.254	.2351	4.254	35.18	35.19	.5605	.5448	.9580	5.223

Lecture 11
Coastal and Ocean Structures
Part III

Pilings and Piers

The forces on a pile depend on the size of the pile (diameter) relative to the wave length. If the pipe is very large diffraction may govern the force. For this course we will restrict our attention to small vertical piles (Diameter < 0.05 L). The definition diagram for a pile is shown in Figure 11.1 (Fig. 7-67 SPM). Additional definitions that are often used with piles are given in the APPENDIX at the end of this lecture.

For non-breaking waves the force on a small (slender) vertical pile is often estimated by the Morrison Equation:

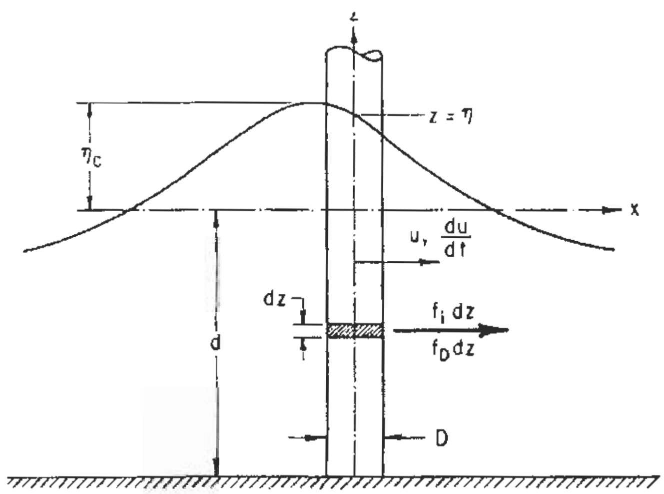


Figure 7-67. Definition sketch of wave forces on a vertical cylinder.

Slenderness Criterion $\frac{D}{L_A} < 0.05$

Figure 11.1 Slender Pile (D = pile diameter; L_A = linear wave length)

The Morrison equation has the form:

$$f = f_i + f_D = \frac{1}{4} C_M \rho \pi D^2 \frac{du}{dt} + \frac{1}{2} C_D \rho D u|u| \qquad 11.1$$

where f is the force per unit length of the pile; f_i = force due to inertia; f_D=force due to drag; u = horizontal orbital velocity; C_M = inertial or virtual mass coefficient; C_D = drag coefficient.

The Airy or linear wave length LA is given by:

$$L_A = L_o \tanh(\frac{2\pi d}{L_A}) \qquad 11.2$$

The non-linear wave length can be obtained using Dean's Stream function, Stokes or Cnoidal wave theory; Figure 11.2 (SPM) can be used to obtain an approximation of the non-linear L.

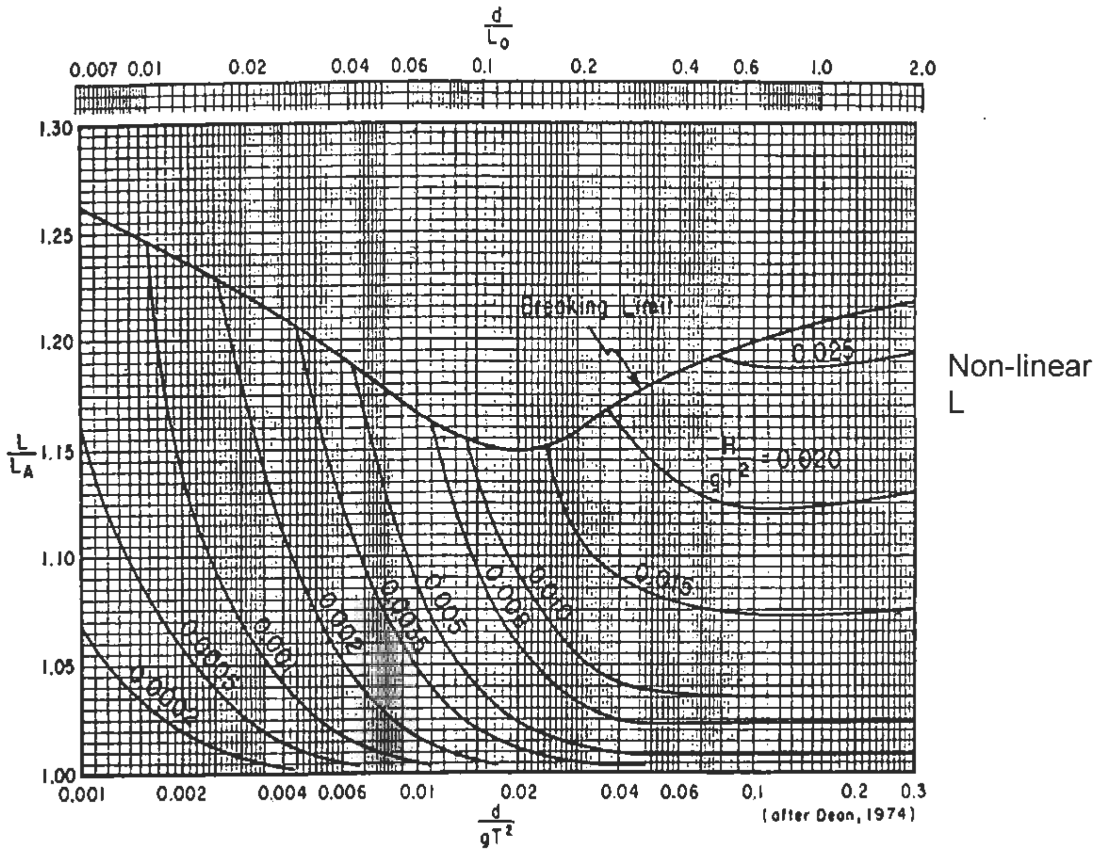


Figure 7-70. Wavelength correction factor for finite amplitude effects.

Figure 11.2 Non-linear Wave Length (SPM)

There are three types of forces that dominate on slender piles: inertia, drag and lift. Airy theory gives the following formulae for the inertia and drag forces per unit length:

Eqs. 11.3-11.6→

$$u = \frac{H}{2}\frac{gT}{L}\frac{\cosh[2\pi(z+d)/L]}{\cosh[2\pi d/L]}\cos\left(\frac{2\pi t}{T}\right) \quad (7\text{-}23)$$

$$\frac{du}{dt} \approx \frac{\partial u}{\partial t} = \frac{g\pi H}{L}\frac{\cosh[2\pi(z+d)/L]}{\cosh[2\pi d/L]}\sin\left(-\frac{2\pi t}{T}\right) \quad (7\text{-}24)$$

Introducing these expressions into equation (7-20) gives

$$f_i = C_M \rho g \frac{\pi D^2}{4} H \left\{\frac{\pi}{L}\frac{\cosh[2\pi(z+d)/L]}{\cosh[2\pi d/L]}\right\}\sin\left(-\frac{2\pi t}{T}\right) \quad (7\text{-}25)$$

$$f_D = C_D \frac{1}{2}\rho g D H^2 \left\{\frac{gT^2}{4L^2}\left\{\frac{\cosh[2\pi(z+d)/L]}{\cosh[2\pi d/L]}\right\}^2\right\}\cos\left(\frac{2\pi t}{T}\right)\left|\cos\left(\frac{2\pi t}{T}\right)\right| \quad (7\text{-}26)$$

These equations can be integrated from the seabed (-d) to the wave surface (η) as shown below:

Eq. 11.7-11.8→

$$F = \int_{-d}^{\eta} f_i dz + \int_{-d}^{\eta} f_D dz = F_i + F_D$$

$$M = \int_{-d}^{\eta}(z+d) f_i dz + \int_{-d}^{\eta}(z+d) f_D dz = M_i + M_D$$

where F, F_i and F_D are respectively the total wave force, the total inertia force and the total drag force on the pile. Similarly , M, M_i and M_D are respectively the total wave moment, the total inertia moment and the total drag moment about the base of the pile.

Figure 11.3a (Figure 3.1 in Sarpkaya and Isaacson) shows how C_D varies with Re ~ U D/(ν). Figure 11.3b shows SPM design values for C_D varies on relatively smooth circular piles. The added mass coefficient C_M is shown in Table 11.1 (Table 2.3 in Sarpkaya and Isaacson). The variation of C_M with the Keulegan-Carpenter Number (N_{kc}) is illustrated in Figure 11.4.

It is necessary to integrate Eqs. 11.3-11.6 over the length of the pile that is affected by the wave as shown by Eqs. 11.7-11.8. Based on Dean's stream function theory, the SPM gives the general integrated equations as shown Eqs. 11.9-11.12. The following are the maximum inertia and drag forces and moments with the respective integration coefficients:

Eq. 11-9-11.12→

$$F_{im} = \frac{1}{4} C_M \gamma \pi D^2 H K_{im}$$

$$F_{Dm} = \frac{1}{2} C_D \gamma D H^2 K_{Dm}$$

$$M_{im} = F_{im} d S_i$$

$$M_{Dm} = F_{Dm} d S_D$$

where K_{im}, K_{Dm}, S_i and S_D are the integration coefficients shown in Figures 11.3 to 11.6 respectively.

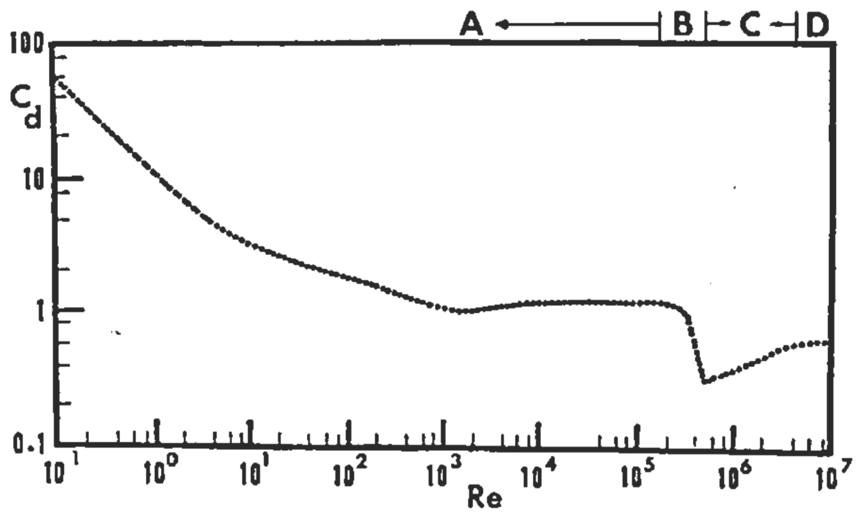


Fig. 3.1b. Drag coefficient for circular cylinders as a function of Reynolds number (Schlichting 1968).

Figure 11.3a Drag Coefficient (Sarpkaya and Isaacson)

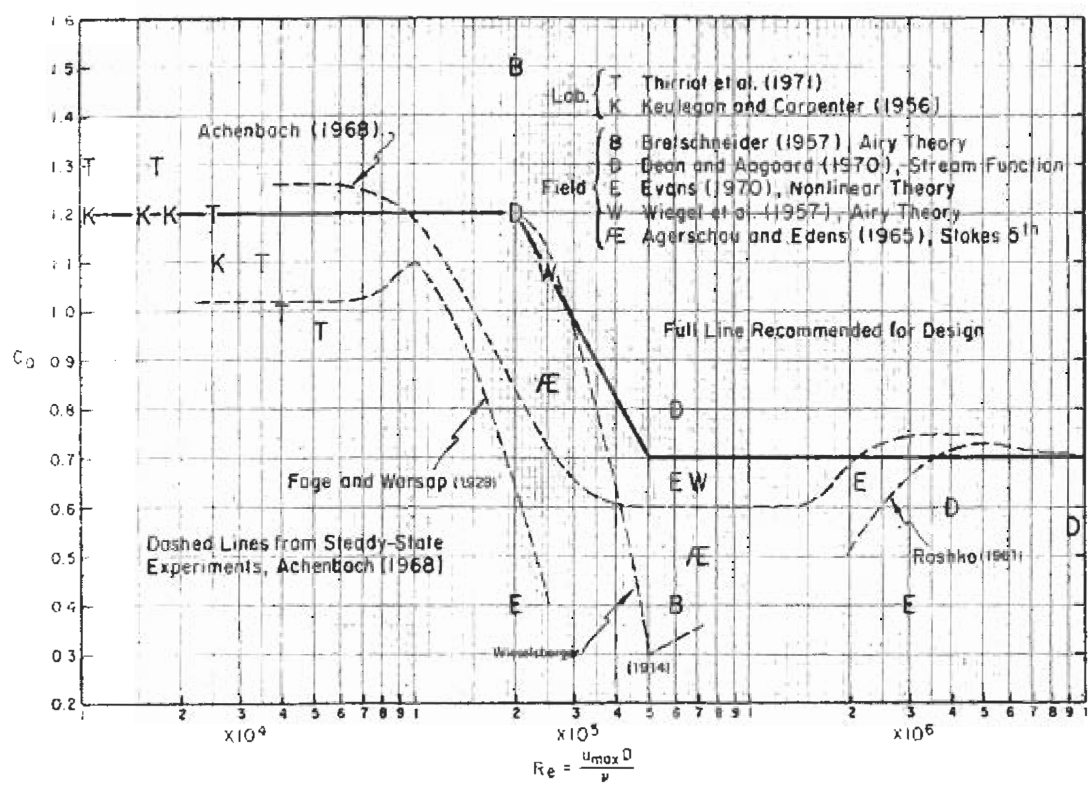
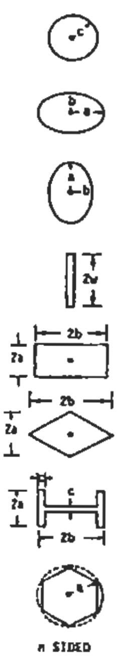


Figure 7-85. Variation of drag coefficient C_D with Reynolds number R_e.

Figure 11.3b Drag Coefficient (SPM)

Table 2.3 Added Masses of Various Bodies



SHAPE	ADDED MASS PER UNIT LENGTH ← MOTION				
CIRCLE	$\rho\pi c^2$				
ELLIPSE	$\rho\pi b^2$				
ELLIPSE	$\rho\pi a^2$				
PLATE	$\rho\pi a^2$				
RECTANGLE	a/b		a/b		
	∞	1.00 $\rho\pi a^2$	1	1.51 $\rho\pi a^2$	
	10	1.14 "	0.5	1.70 "	
	5	1.21 "	0.2	1.98 "	
	2	1.36 "	0.1	2.23 "	(Wendel 1950)
DIAMOND	2	0.85 "			
	1	0.76 "			
	0.5	0.67 "			
	0.2	0.61 "	(Wendel 1950)		
I-BEAM a/c = 2.6 b/c = 3.6		2.11 $\rho\pi a^2$	(Patton 1965)		
REGULAR POLYGON OR ↕	n = 3	0.654 $\rho\pi a^2$			
	4	0.787 "			
	5	0.823 "			
	6	0.867 "			
	∞	1.000 "	(Wendel 1950)		

C_M =Added Mass/(Displaced Mass)

Table 11.1 Inertia Coefficient (from Sarpkaya and Isaacson)

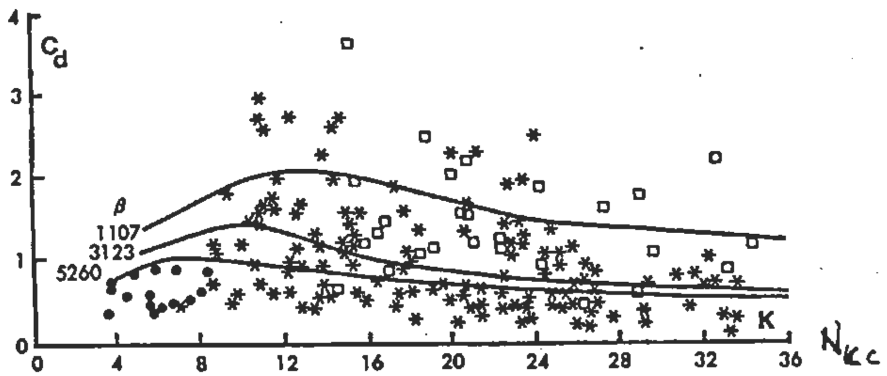


Fig. 5.2. Drag coefficients from field data, as compiled by Wiegel et al. (1957). Solid lines from Sarpkaya (1976a).

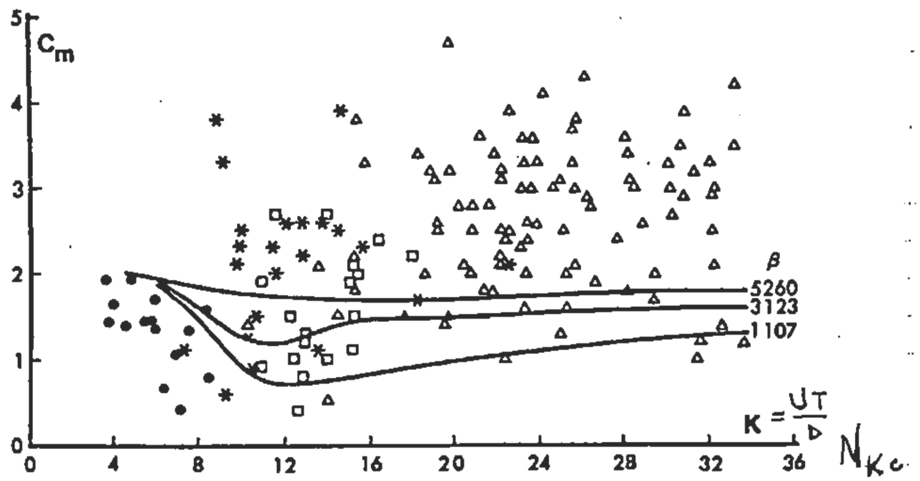


Fig. 5.3. Inertia coefficients from field data, as compiled by Wiegel et al. (1957). Solid lines from Sarpkaya (1976a).

Figure 11.4 Inertia Coefficient (from Sarpkaya and Isaacson). N_{kc} is the Keulegan-Carpenter Number defined in the Appendix to this lecture.

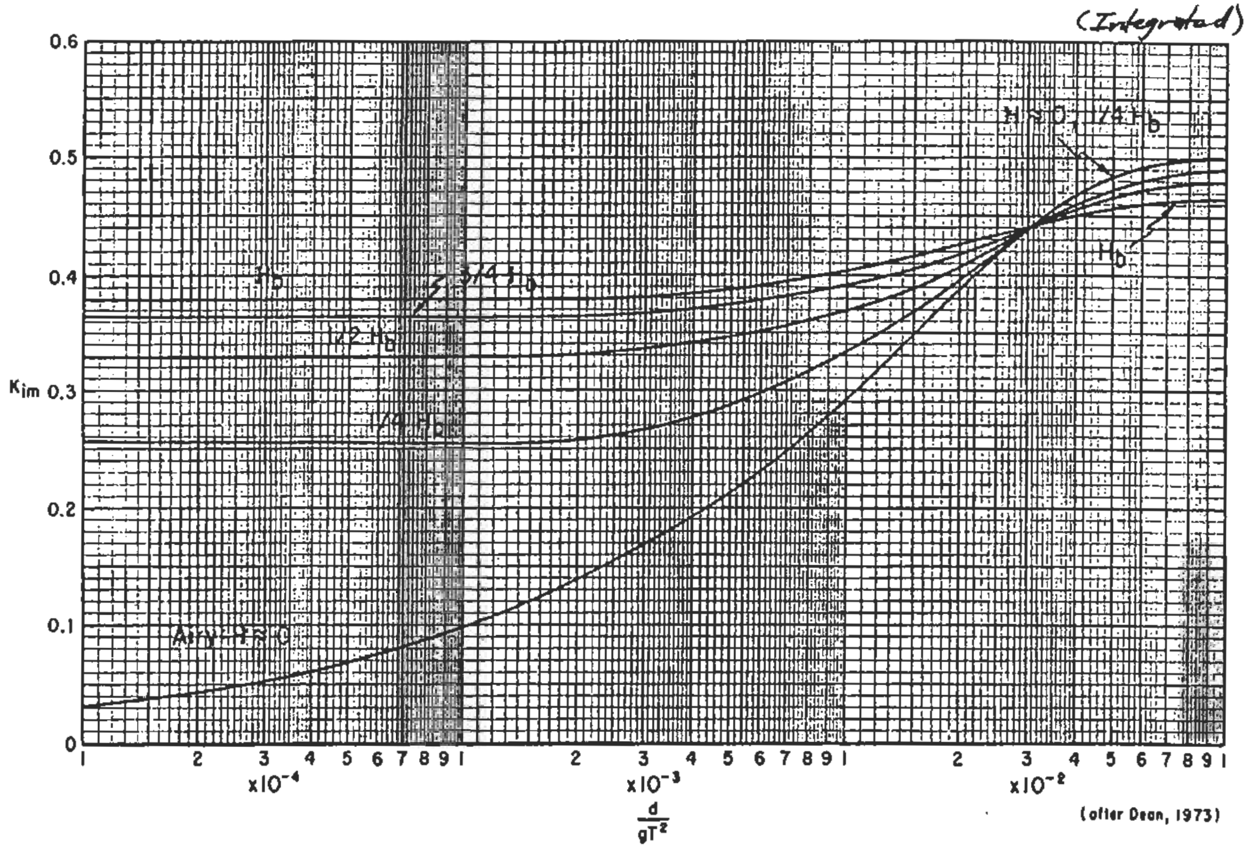


Figure 7-71. K_{im} versus relative depth, d/gT^2 .

Figure 11.5 Inertia Force Integration Coefficient

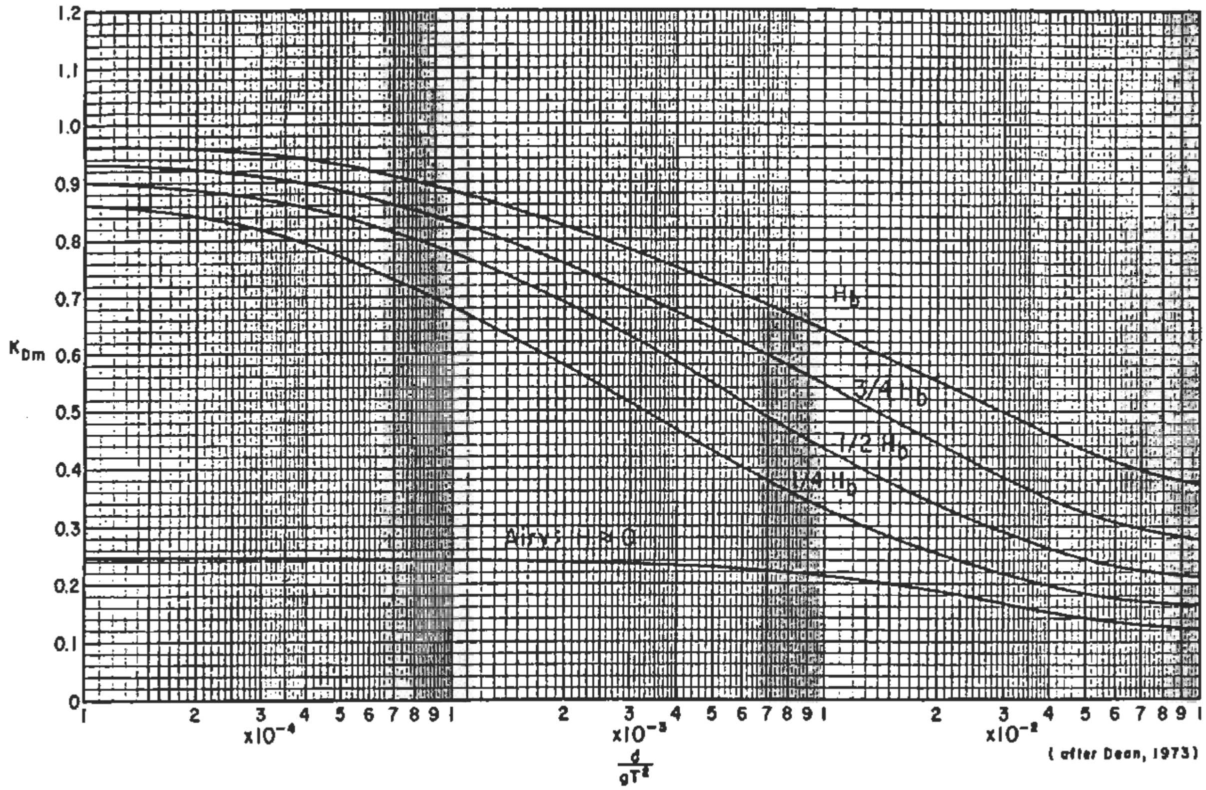


Figure 7-72. K_{Dm} versus relative depth, d/gT^2.

Figure 11.6 Drag Force Integration Coefficient

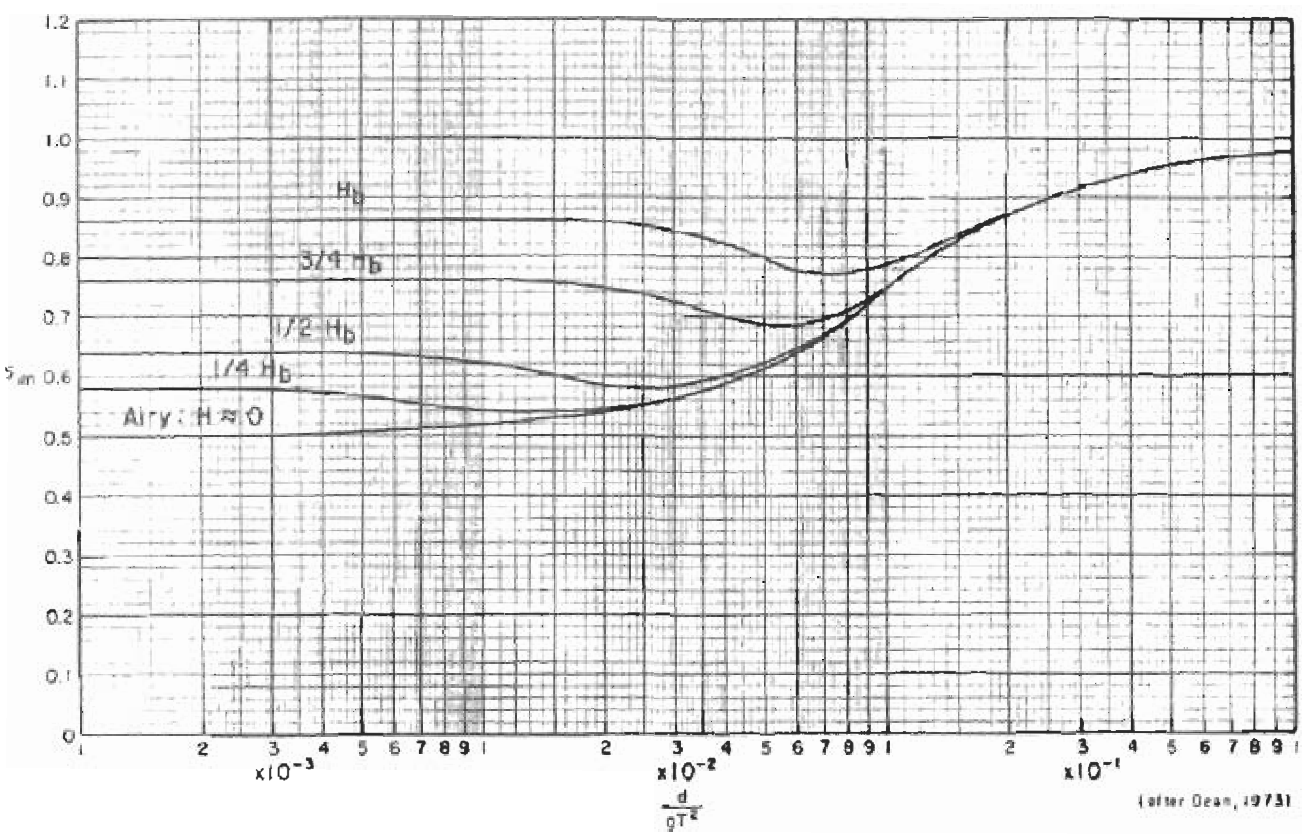


Figure 7-73. Inertia force moment arm, S_{im}, versus relative depth, d/gT^2.

Figure 11.7 Inertia Moment Integration Coefficient

The maximum inertia and drag do not occur simultaneously; therefore, the resultant is less than the sum of the two maxima. The resultant force is the computed from

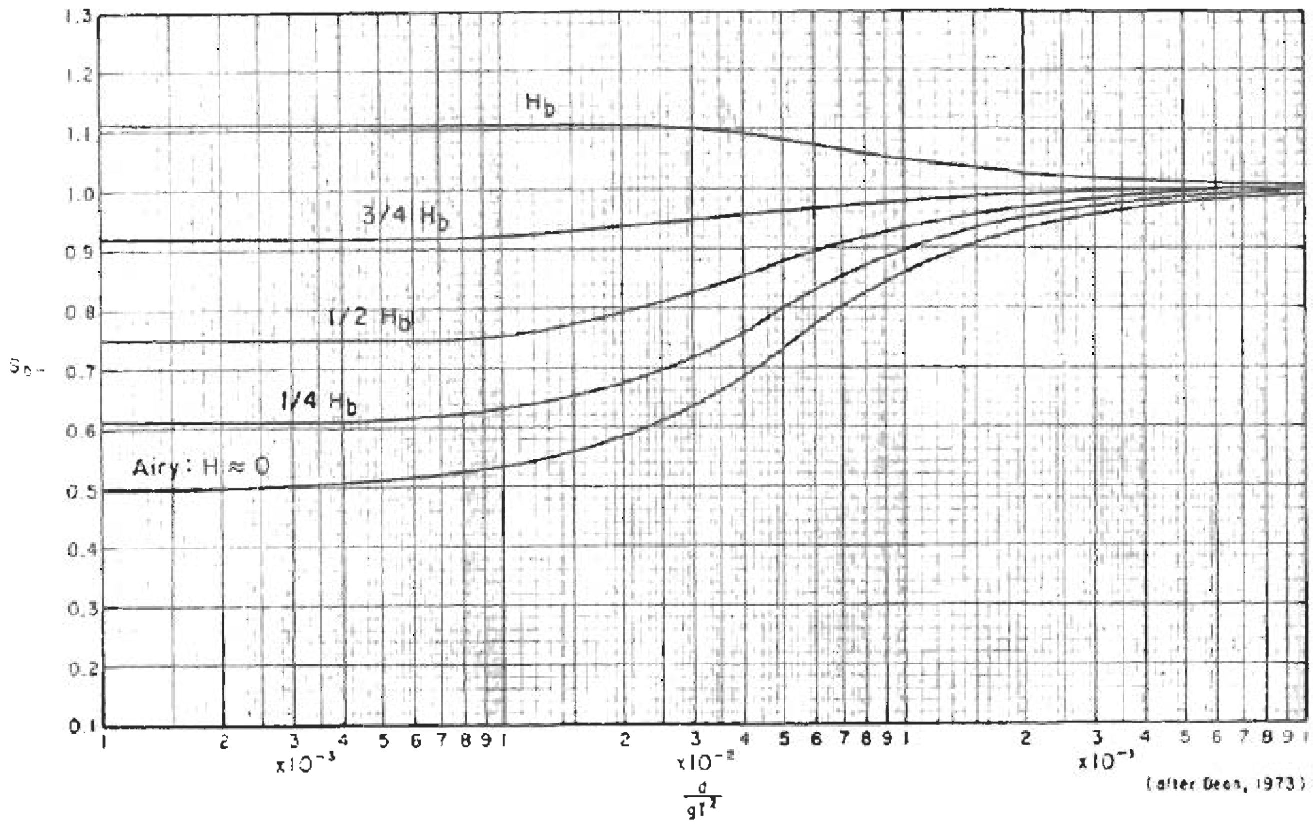


Figure 7-74. Drag force moment arm, S_{Dm}, versus relative depth, d/gT^2.

Figure 11.8 Drag Moment Integration Coefficient

The maximum inertia and drag do not occur simultaneously; therefore, the resultant is less than the sum of the two maxima. The resultant force is the computed from

$$F_R = \phi_m \gamma C_D H^2 D \qquad 11.13$$

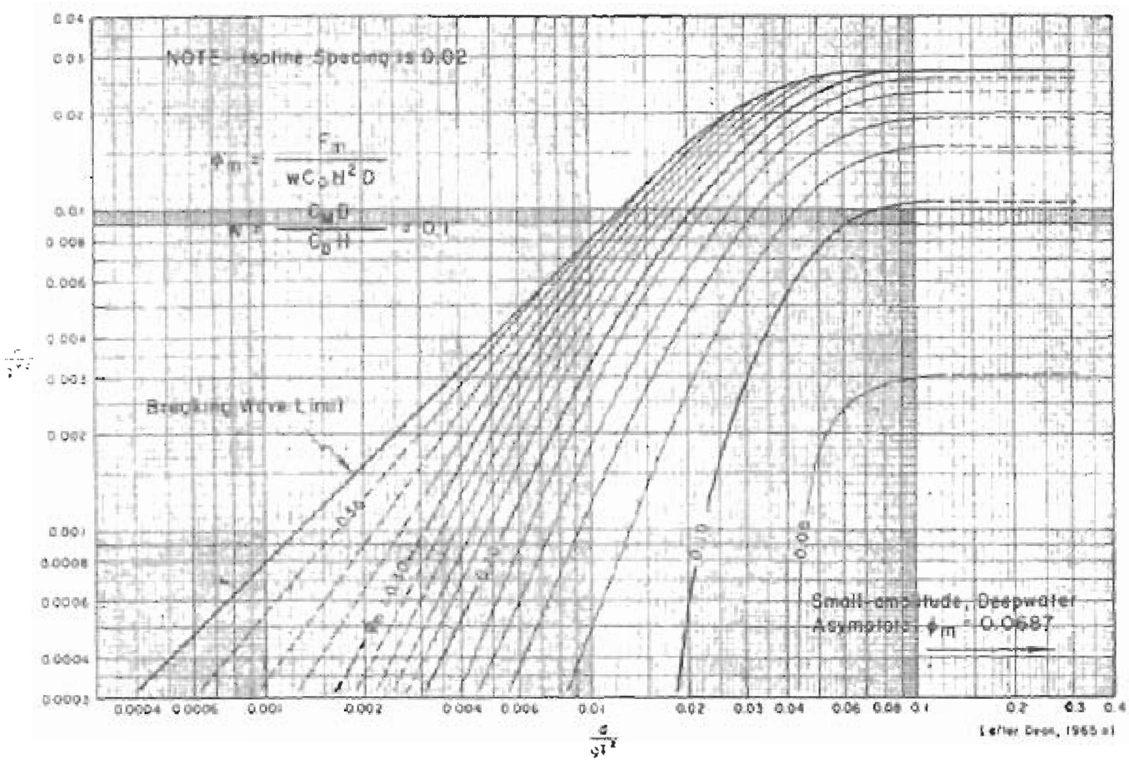
Figures 11.9a, 11.9b and 11.9c give the ϕ_m factors in terms of $d/(gT^2)$, H/gT^2 and $W=C_MD/(C_DH)$ for W=0.1, 0.5 and 1.0

The resultant moment is computed from

$$M_R = \alpha_m \gamma C_D H^2 Dd \qquad 11.14$$

Figures 11.10a, 11.10b and 11.10c give the α_m factors in terms of $d/(gT^2)$, H/gT^2 and $W=C_MD/(C_DH)$ for W=0.1, 0.5 and 1.0

$$W = \frac{C_M D}{C_D H} \qquad F_m = \phi_m w C_D H^2 D$$



W~0

Figure 7-77. Isolines of ϕ_m versus H/gT^2 and d/gT^2 ... (W = 0.1).

Figure 11.9a ϕ coefficient for resultant inertia and drag forces. W=0.1

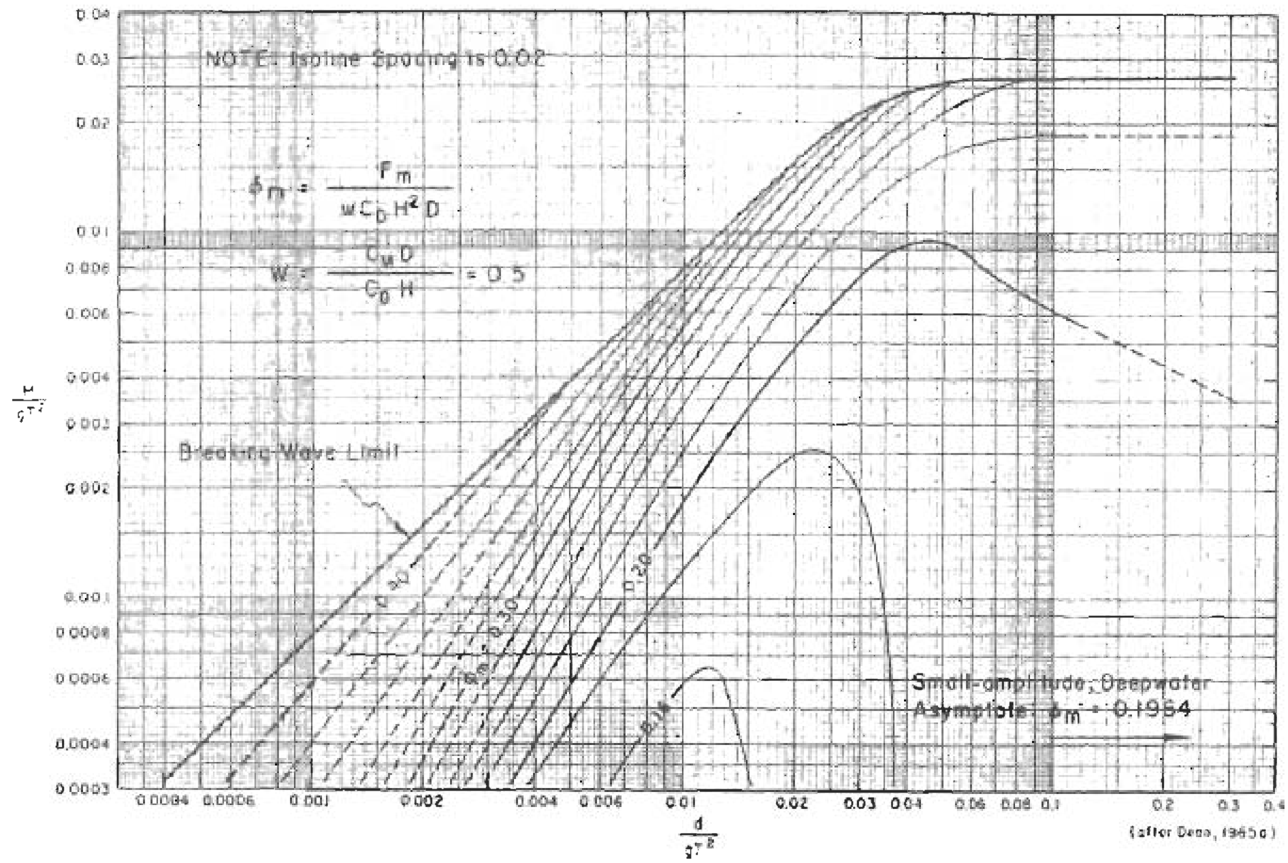


Figure 7-78. Isolines of ϕ_m versus H/gT^2 and d/gT^2 ... (W = 0.5) .

Figure 11.9b ϕ coefficient for resultant inertia and drag forces. W=0.5

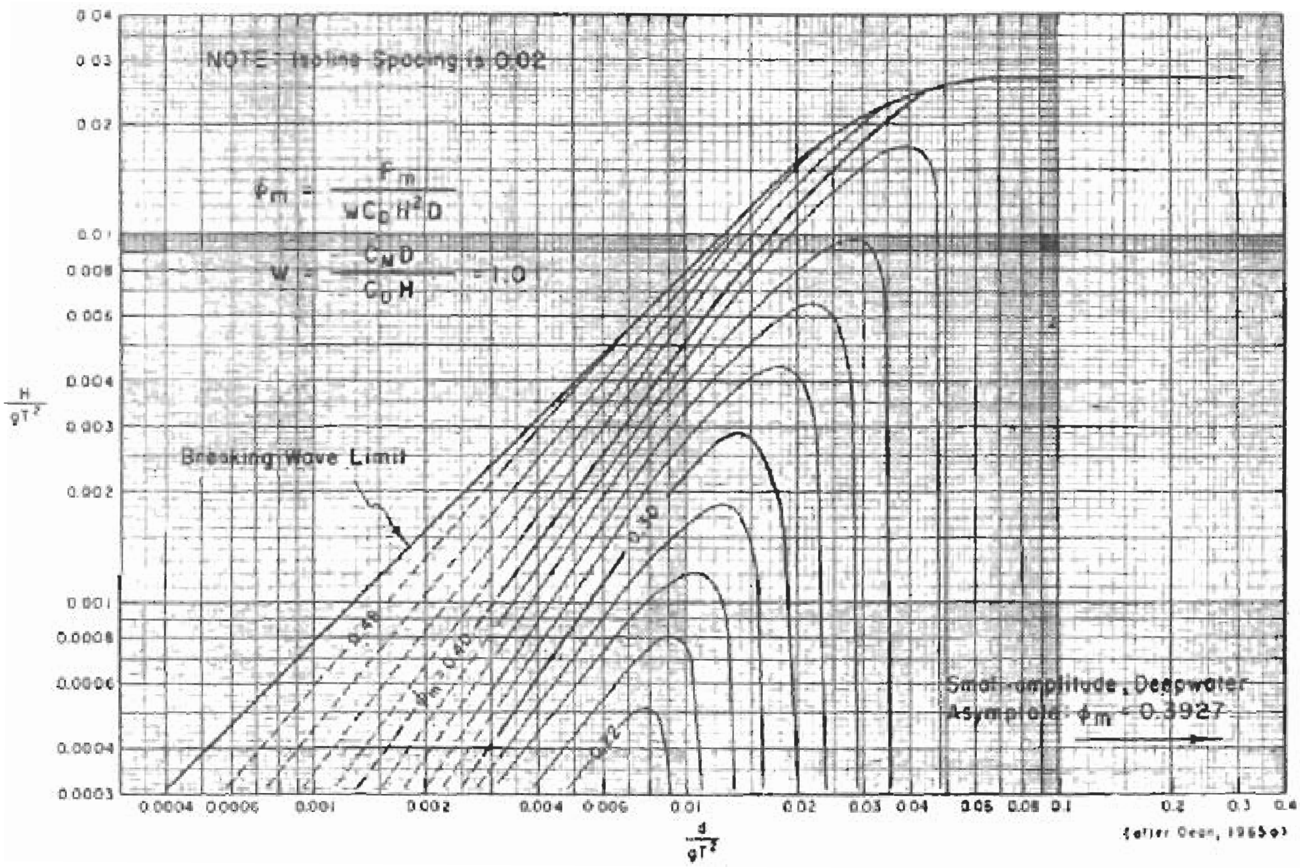
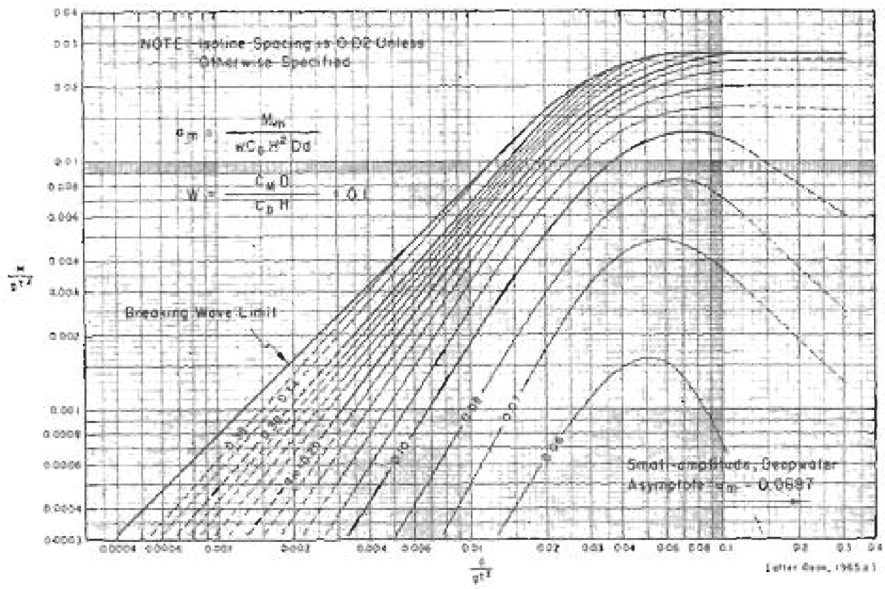


Figure 7-79. Isolines of ϕ_m versus H/gT^2 and d/gT^2 ... (W = 1.0) .

Figure 11.9c ϕ coefficient for resultant inertia and drag forces. W=1

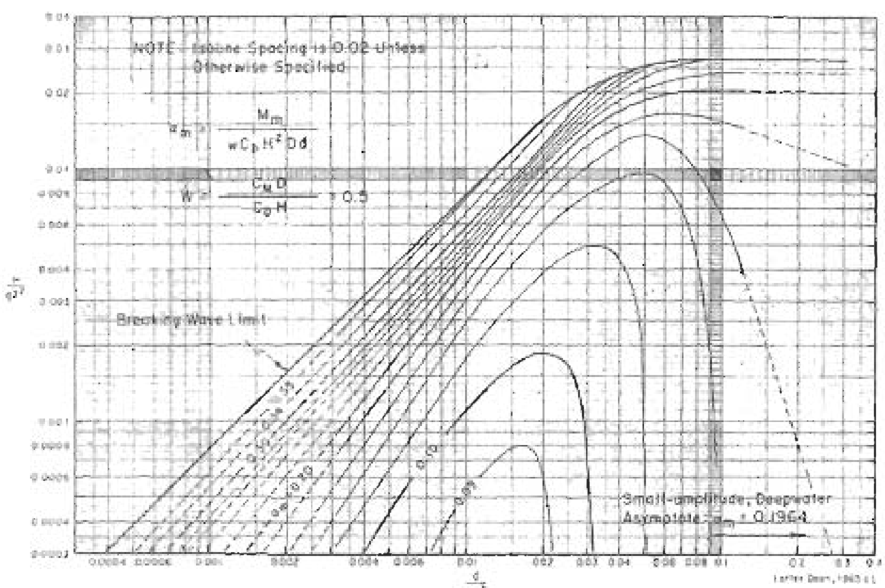
$$W = \frac{C_M \ D}{C_D \ H}$$



W~0.1

Figure 7-81. Isolines of α_m versus H/gT^2 and d/gT^2 ... (W = 0.1).

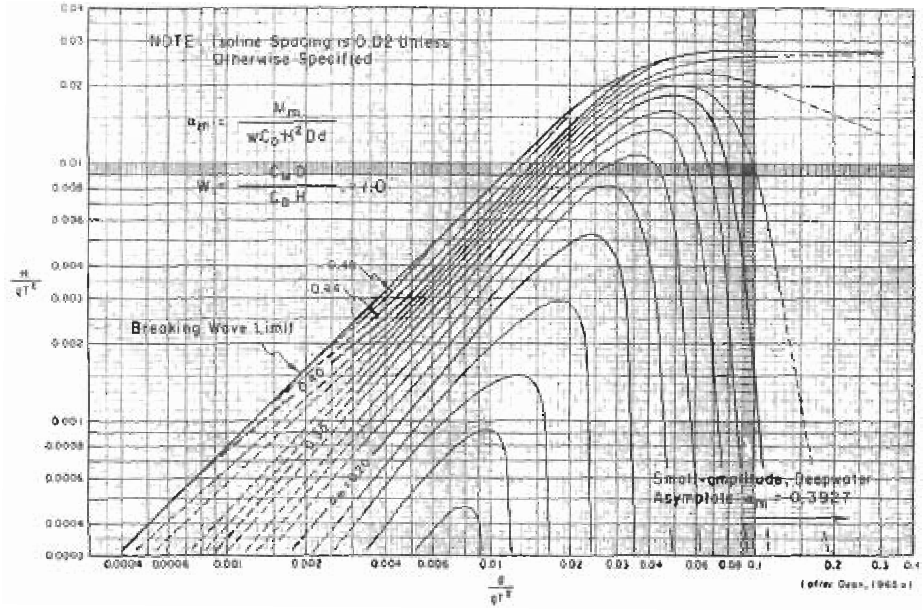
Figure 11.10a α coefficient for resultant inertia and drag moments. W=0.1



W~0.5

Figure 7-82. Isolines of α_m versus H/gT^2 and d/gT^2 ... (W = 0.5).

Figure 11.10b α coefficient for resultant inertia and drag moments. W=0.5



W~1

Figure 7-83. Isolines of α_m versus H/gT^2 and d/gT^2 ... (W = 1.0) .

Figure 11.10c α coefficient for resultant inertia and drag moments. W=1

Lift

Flow past a pile can also induce a lateral force (lift) that is roughly related to the Keulegan-Carpenter Number = Umax T/D as shown in Figure 11.11 (Figure 7-83 SPM). The maximum lift force can be represented by

$$F_L = \frac{1}{2}\gamma C_L H^2 D K_{Dm} \qquad 11.15$$

where C_L is the lift coefficient. For large N_{kc} the C_L/C_D ~0.7 to 1

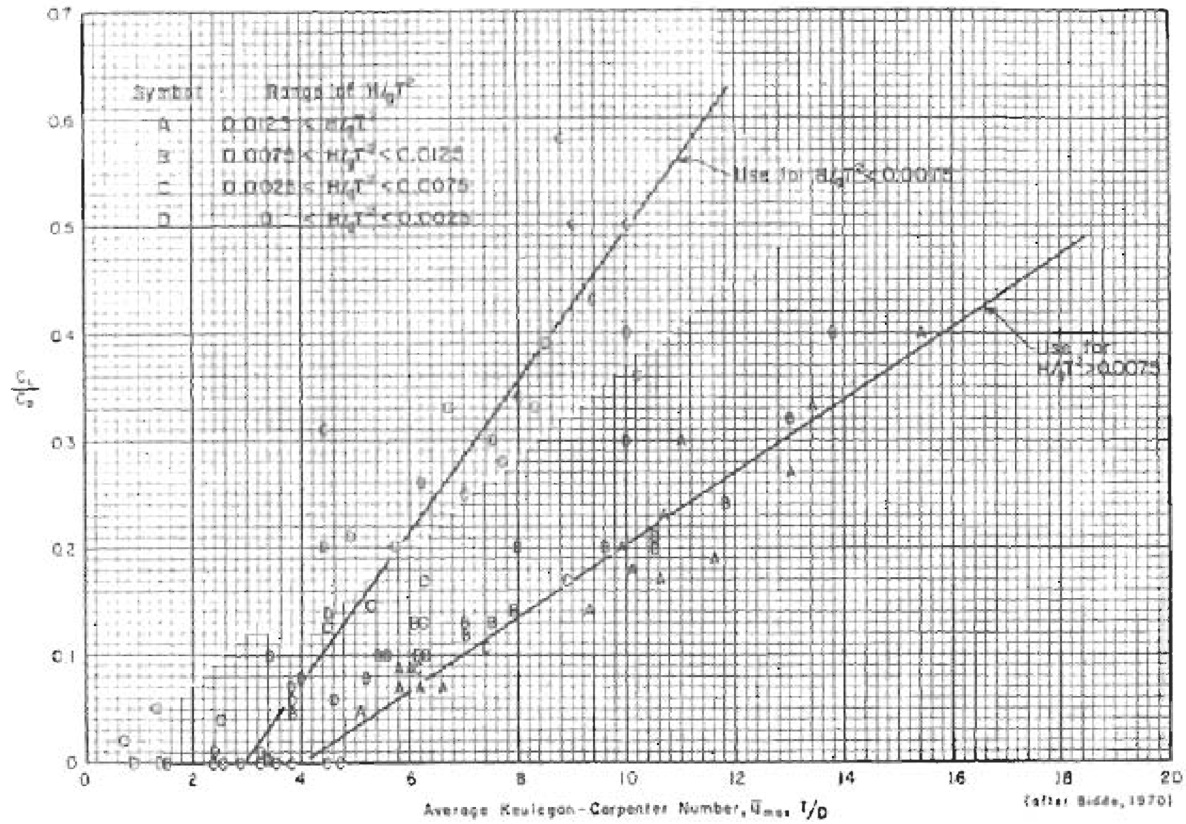


Figure 7-84. Variation of C_L/C_D with Keulegan-Carpenter number and H/gT^2.

Figure 11.11 Lift Coefficient as a Function of Keulegan-Carpenter Number.

The lift force is associated with alternating shedding vortices which have a characteristic frequency f_o that can be estimated by the Strouhal Number = f_o D/U as shown in Figure 11.12, 11.13 and Table 11.2 (Fig 3.1a, 3.2 and 3.9 Sarpkaya and Isaacson).

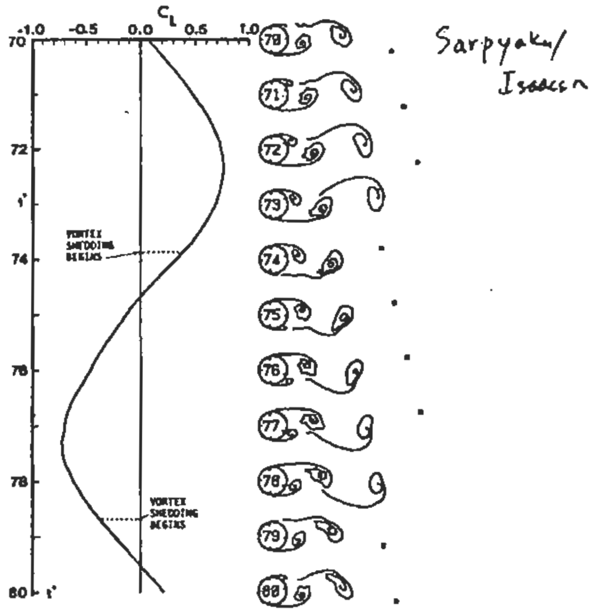


Fig. 3.9. Evolution of lift and wake in the later stages of impulsively-started flow (Sarpkaya and Shoaff 1979a).

Figure 11.12 Lift Coefficient and Shedding Vortices

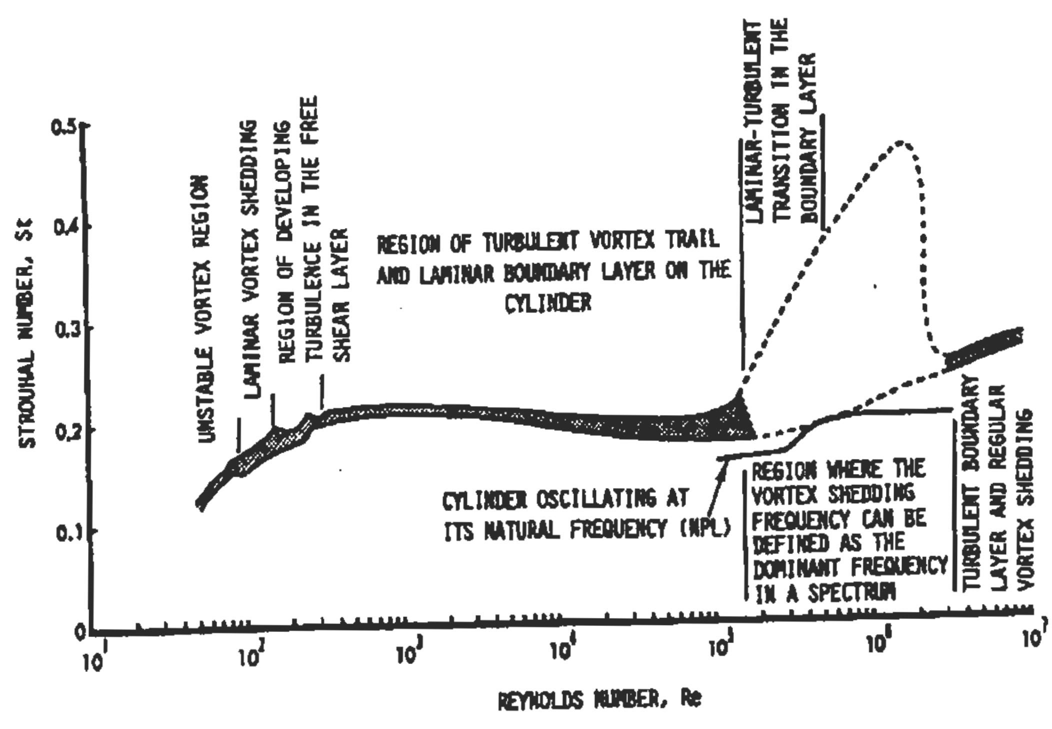


Fig. 3.2. The Strouhal-Reynolds number relationship for circular cylinders (Lienhard 1966).

Figure 11.13 Dimensionless Frequency for Flow Around Circular Piles (Sarpkaya & Isaacson)

	A Subcritical	B Critical	C Supercritical	D Post-supercritical
Boundary layer	laminar	transition	turbulent	turbulent
Separation	about 82 deg.	transition	120 - 130 deg.	about 120 deg.
Shear layer near separation	laminar		laminar separation, bubble turbulent reattachment	turbulent
Strouhal number	$S = 0.212 - \frac{2.7}{Re}$	transition	0.35 - 0.45	about 0.29
Wake	$Re < 60$ laminar; $60 < Re < 5000$ vortex street $Re > 5000$ turbulent	not periodic		
Approximate Re range	$< 2 \times 10^5$	2×10^5 to 5×10^5	$5 \times 10^5 - 3 \times 10^6$	$> 3 \times 10^6$

Fig. 3.1a. Incompressible flow regimes and their consequences.

Table 11.2 Regimes of Flow Around Circular Piles (Sarpkaya and Isaacson)

Effect of Roughness

If the pile is rough e.g. coated with barnacles, the C_D increases ~ 1.3-1.5
The C_M ~ 1.2-1.6 for circular piles with N_{kc} up to 100

Large Diameter Piles

Figure 11.14 shows the dominant processes that affect the flow around piles. For small D/L we have inertia and drag as dominant forces; however, as D/L increases diffraction become more important.

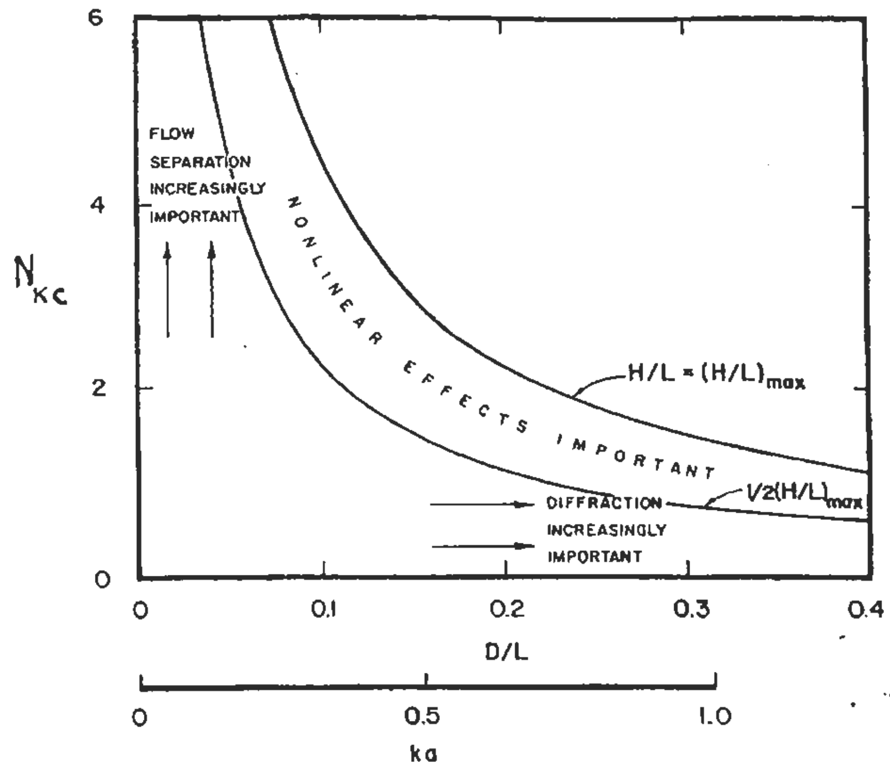


Fig. 6.1. Wave force regimes (Isaacson 1979a).

Figure 11.14 Flow Regimes as a Function of D/L (Isaacson)

Eq. 11.17→

$$F_{max} = K_{dF}\gamma Had\frac{\tanh(kd)}{kd} = \frac{1}{2}K_{dF}\gamma HDL\frac{\tanh(kd)}{2\pi}$$

where

$k = wave.number = 2\pi / L$

$a = D/2$

K_{dF} is can be obtained from Figure 11.15

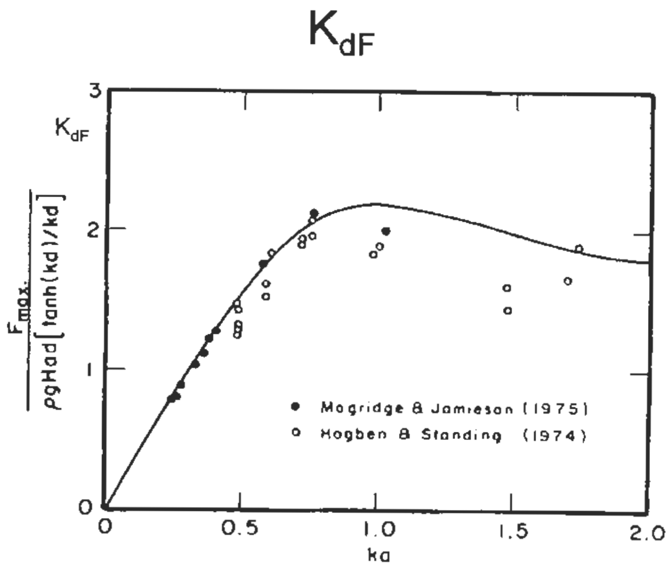


Fig. 6.6. Comparison of the closed-form solution for the wave force on a vertical circular cylinder with the experimental results of Hogben and Standing (1974) and Mogridge and Jamieson (1975).

Figure 11.15 Large Diameter Isolated Circular Piles or Caissons

Inclined Piles

WAVE FORCES ON SMALL BODIES 329

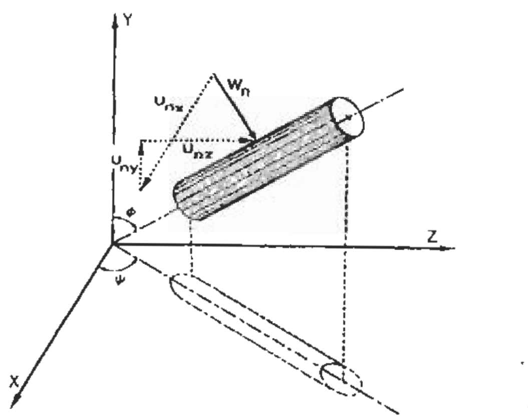


Fig. 5.9. Definition sketch for an inclined cylinder.

Figure 11.16 Inclined Member

The force on an inclined member can be found from the vector equation:

$$\underline{f} = \frac{1}{4} C_M \rho \pi D^2 \frac{d\underline{U}_n}{dt} + \frac{1}{2} C_D \rho D |\underline{U}_n| \underline{U}_n$$

$$\underline{U}_n = \begin{bmatrix} u_{nx} \\ u_{ny} \\ u_{nz} \end{bmatrix} \qquad 11.18$$

$$\underline{f} = \begin{bmatrix} f_x \\ f_y \\ f_z \end{bmatrix}$$

APPENDIX

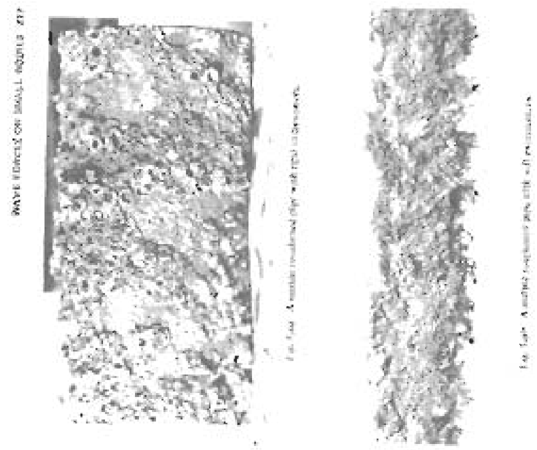
Definitions

- Reynolds Number=
  $Re = UD/\nu$
- Keulegan-Carpenter Number N_{KC}=UT/D
- Strouhal Number
  $S_t = f_o D/U$
- U = maximum horizontal orbital velocity =umax
- f_o = shedding frequency= $S_t U/D$

$$u_{\max} \cong \frac{\pi H L_o}{T L_A}$$

Roughness

HARD



SOFT

SPM

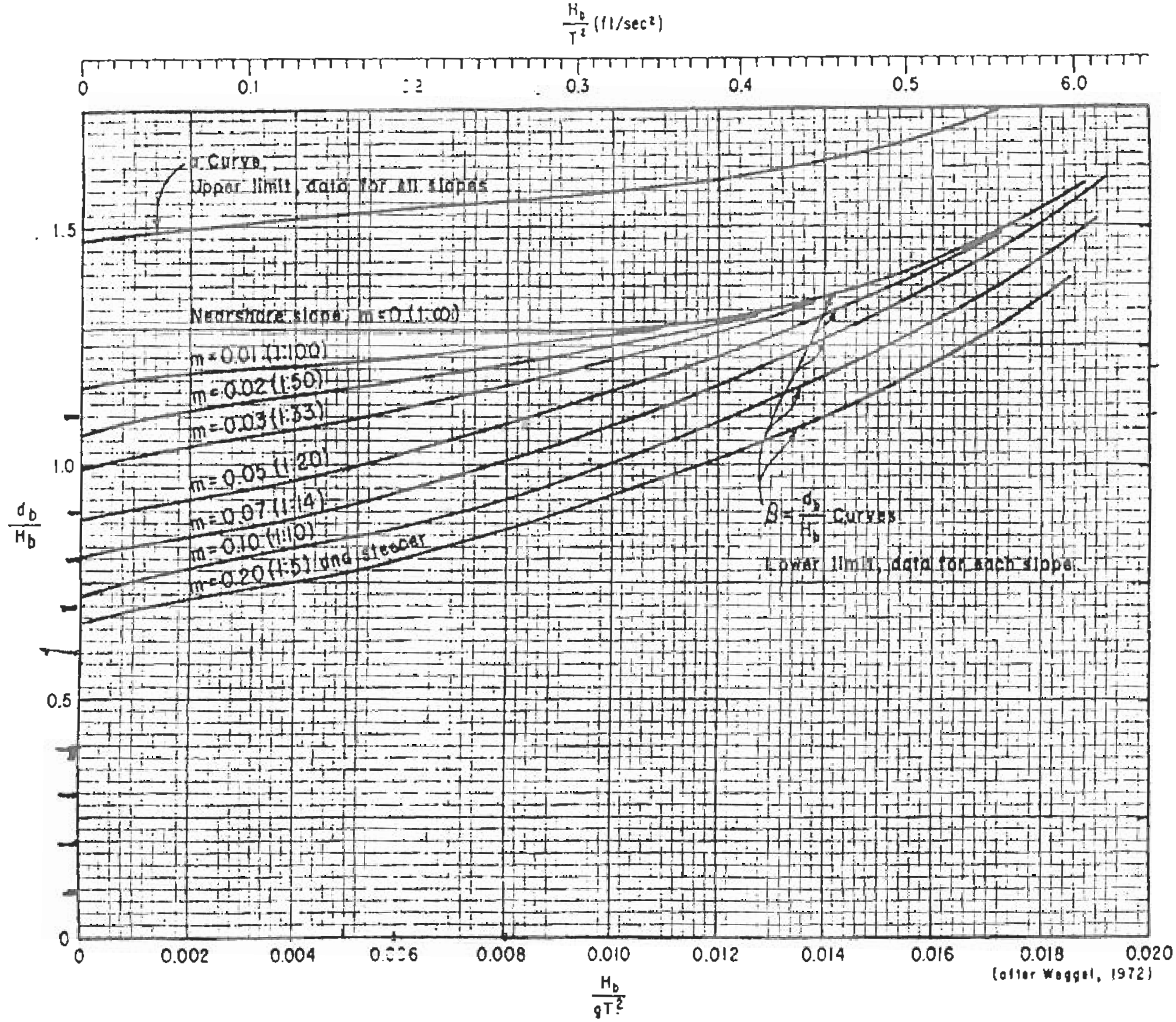
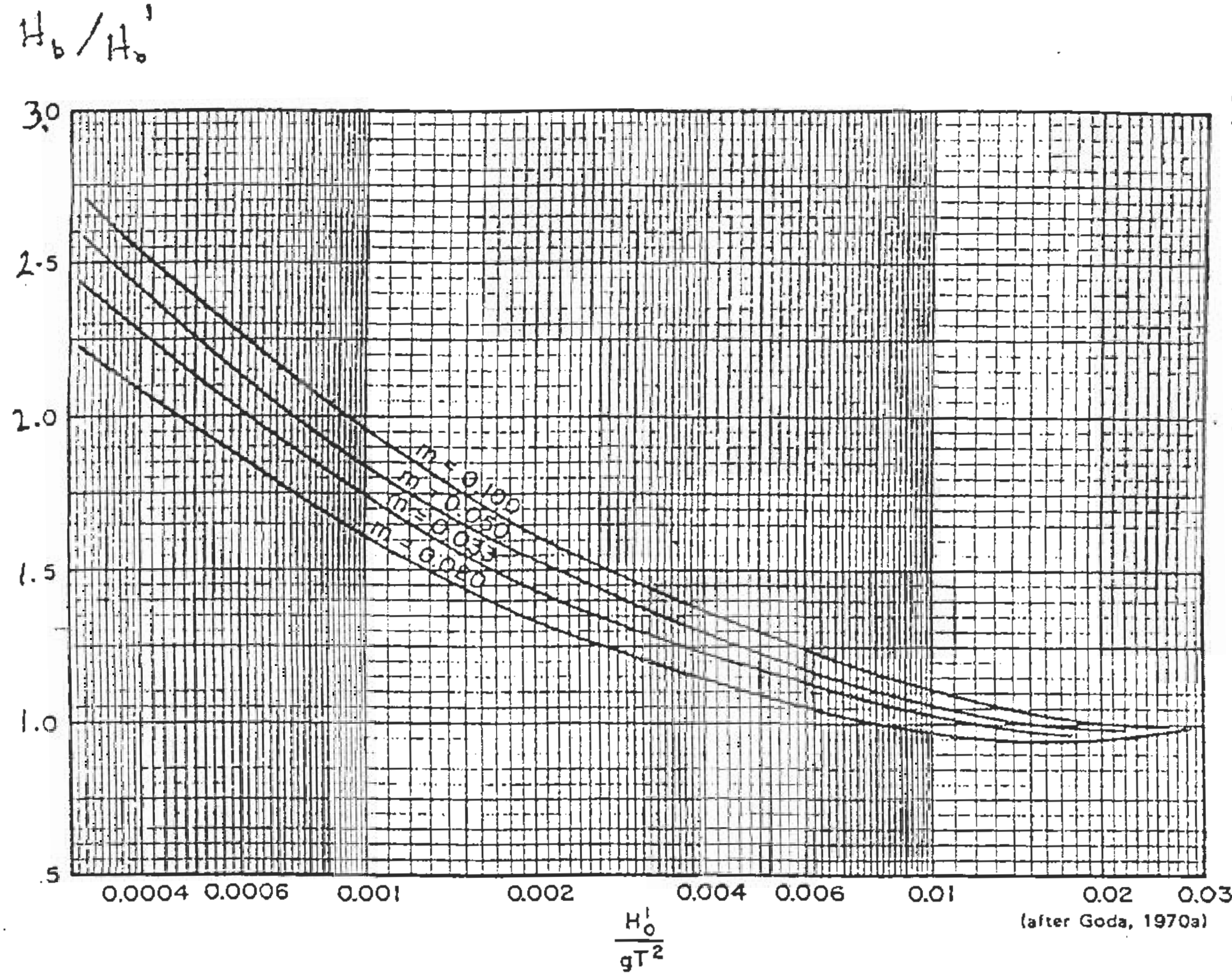


Figure 7-2. α and β versus H/gT^2 .

>pM



ure 7-3. Breaker height index H_b/H_o' versus deepwater wave steepness H_o'/gT^2 .

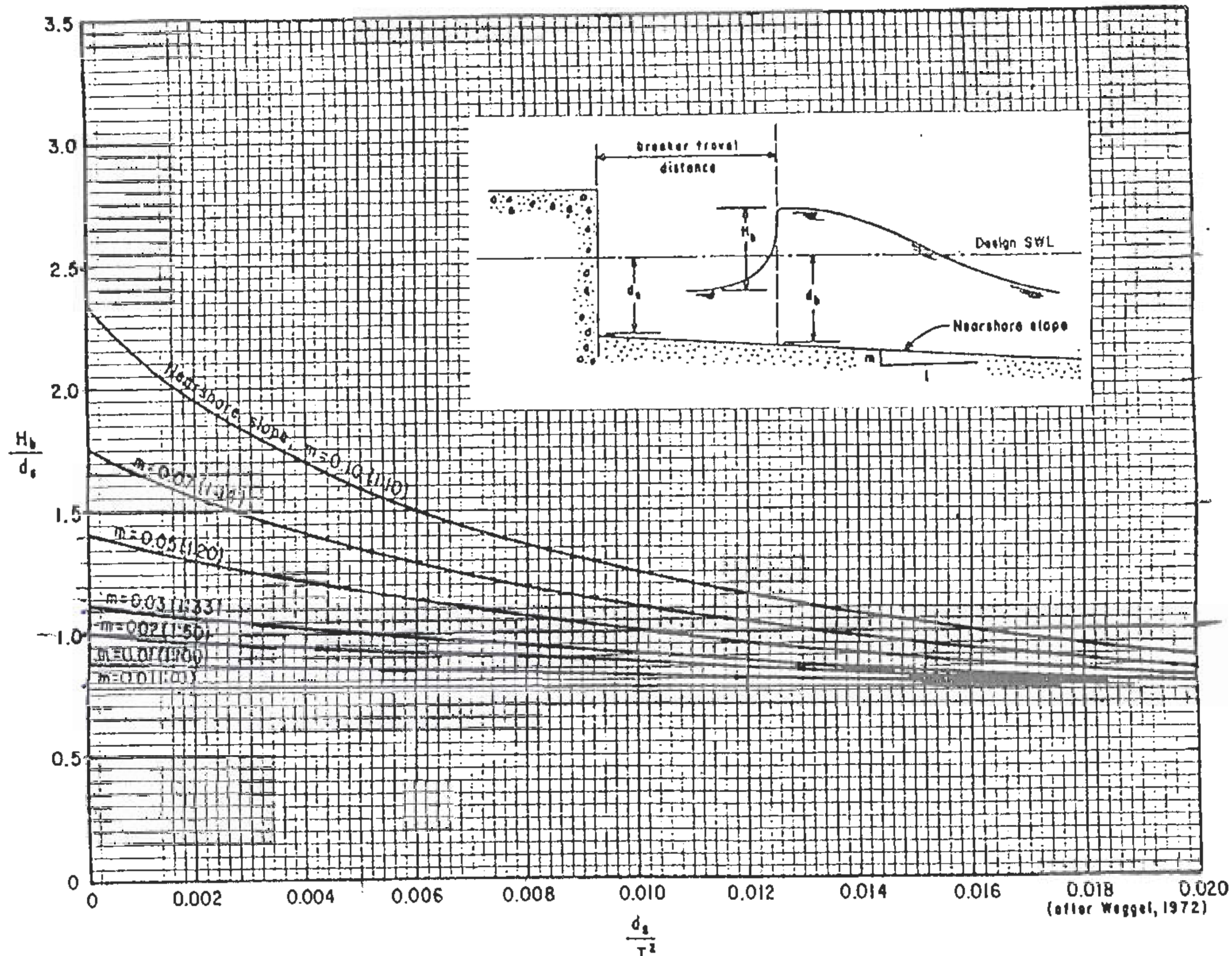


Figure 7-4. Dimensionless design breaker height versus relative depth at structure.

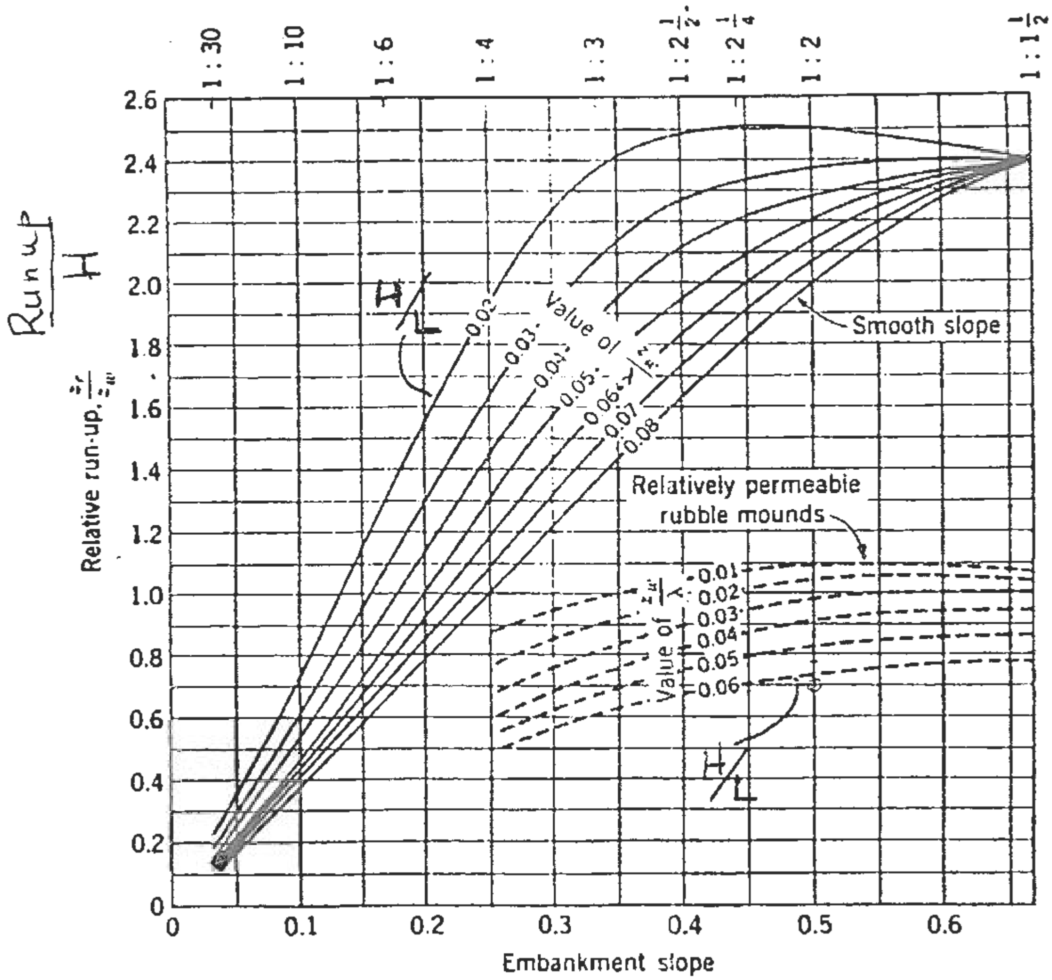


FIG. 7-13 Wave run-up ratios vs. wave steepness and embankment slopes. (From Saville, McClendon, and Cochran)

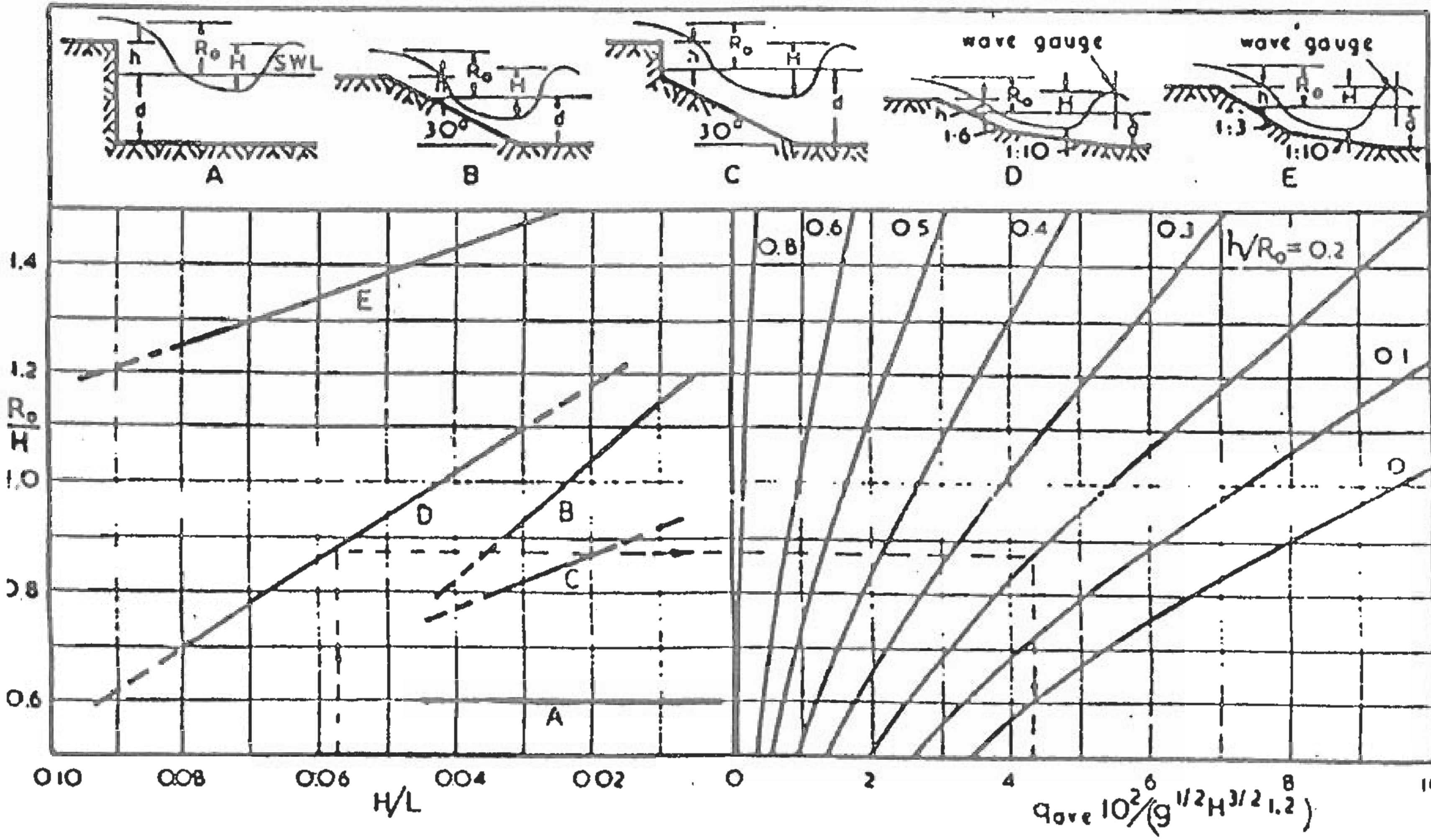


Fig. 7-24. Average overtopping discharge q_{ave} per unit length of walls illustrated.

h/R_o = (CREST EL − SWL)/ RUNUP for overtopping

After Silvester

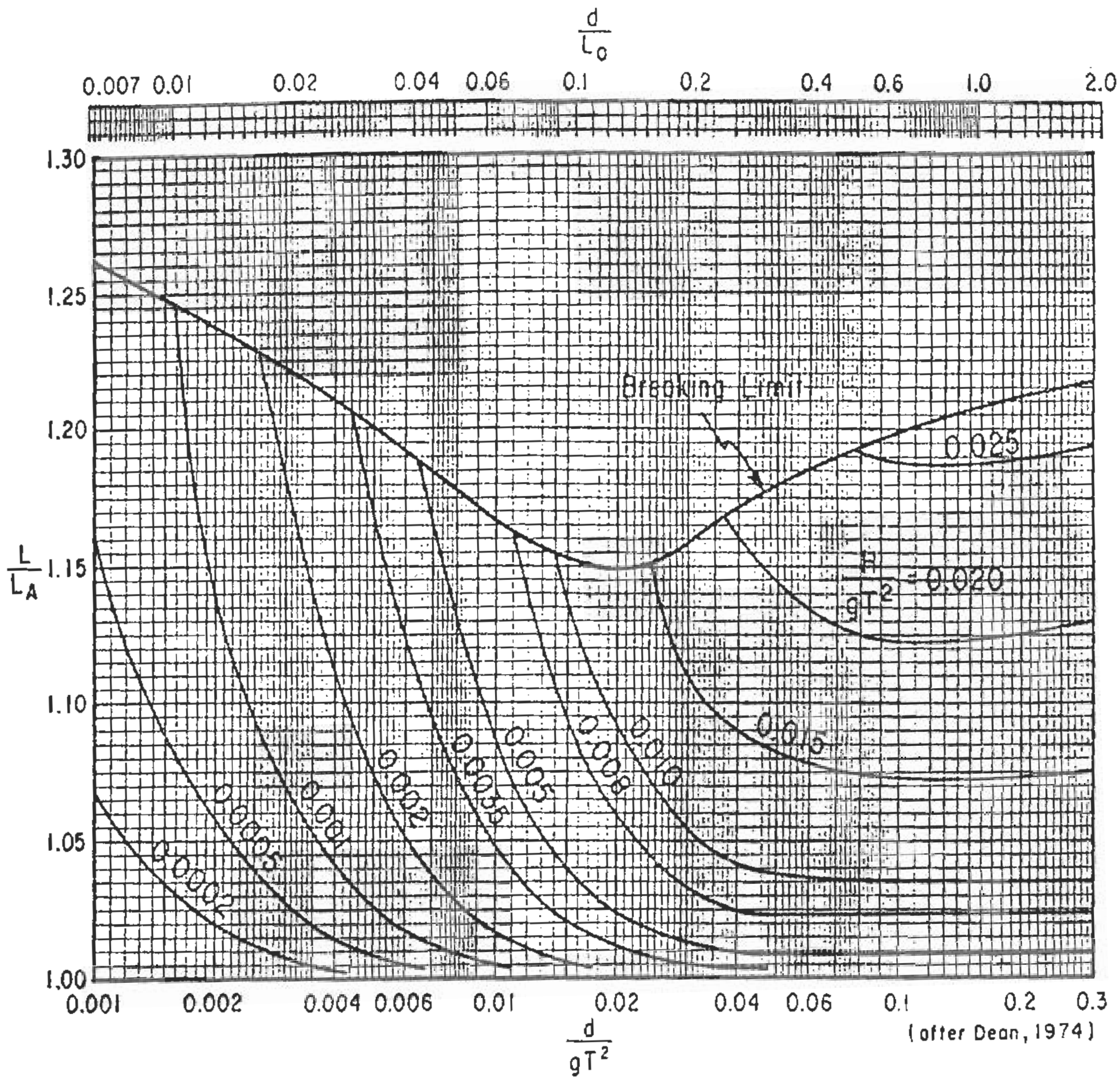


Figure 7-70. Wavelength correction factor for finite amplitude effects.

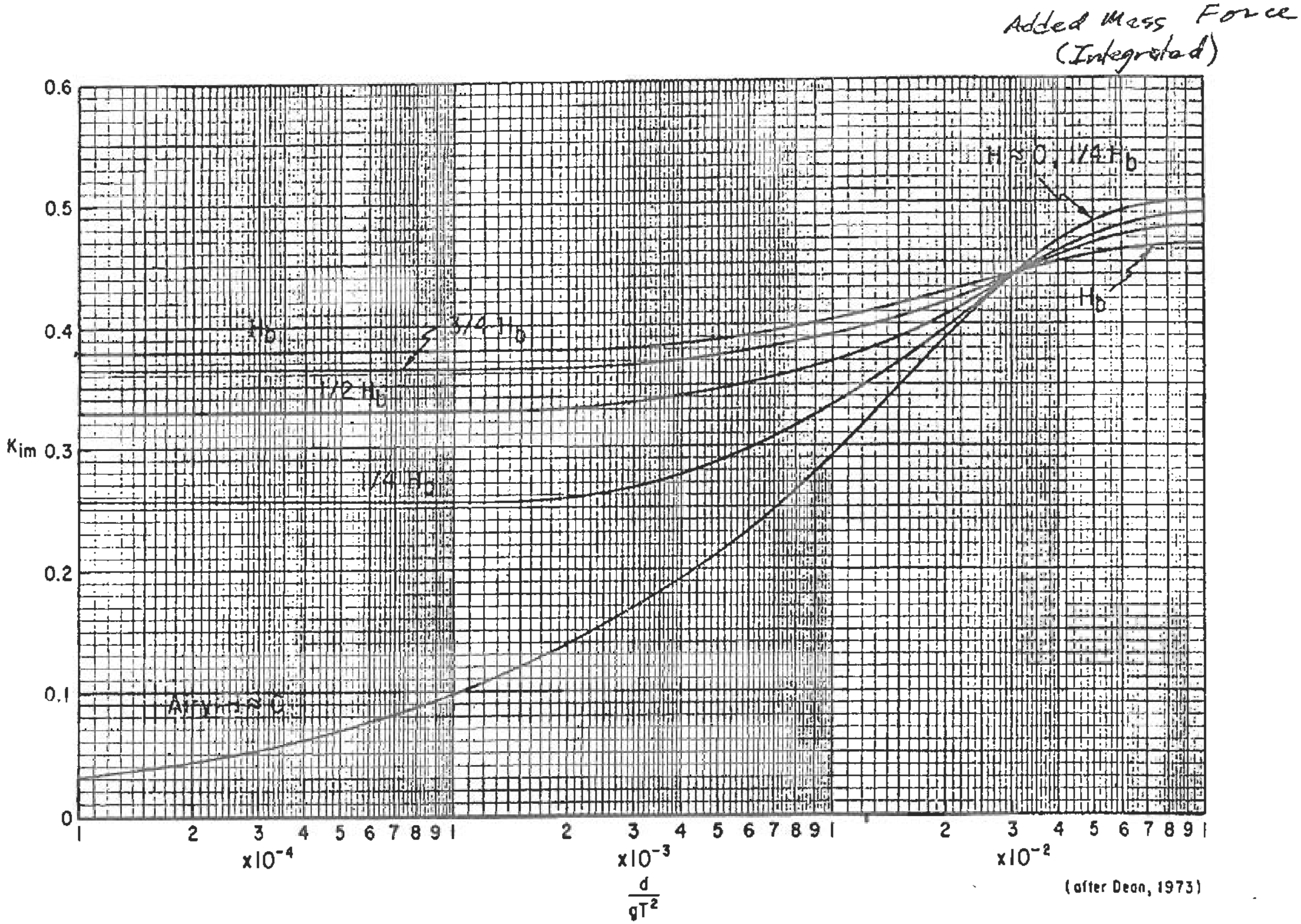


Figure 7-71. K_{im} versus relative depth, d/gT^2 .

Drag Force
(Integrated)

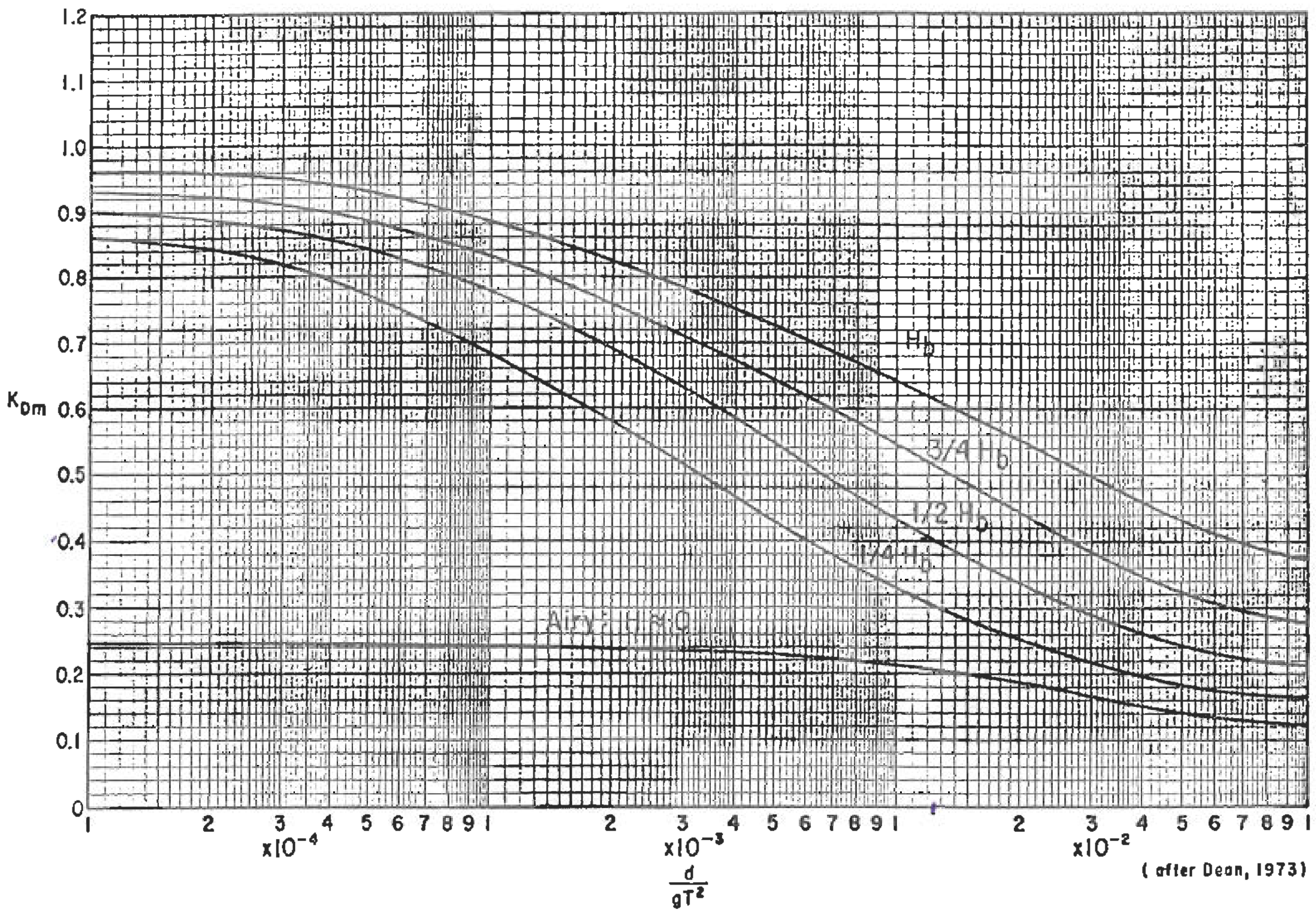


Figure 7-72. K_{Dm} versus relative depth, d/gT^2.

Integrated Moment due to Added Mass

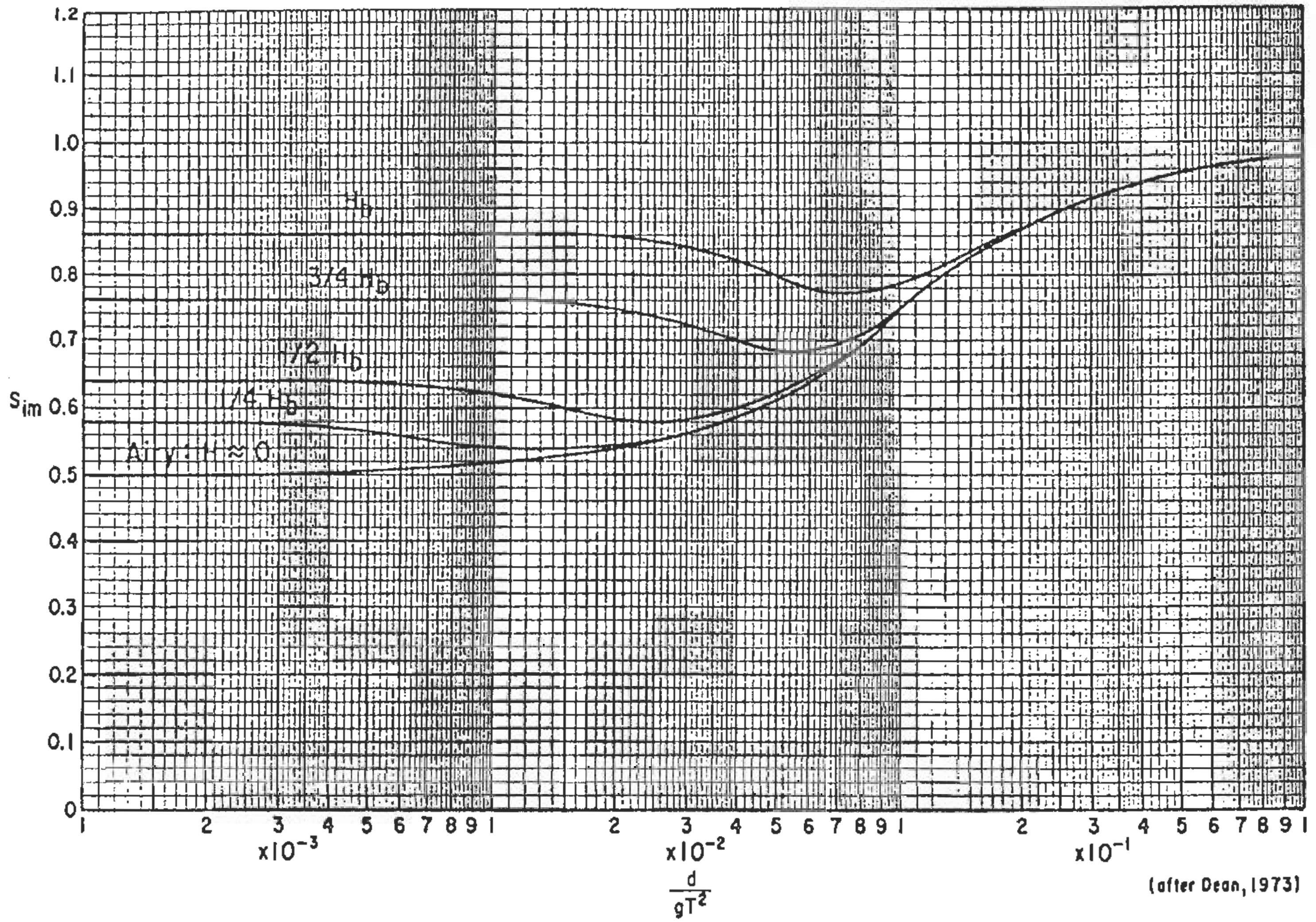


Figure 7-73. Inertia force moment arm, S_{im} , versus relative depth, d/gT^2 .

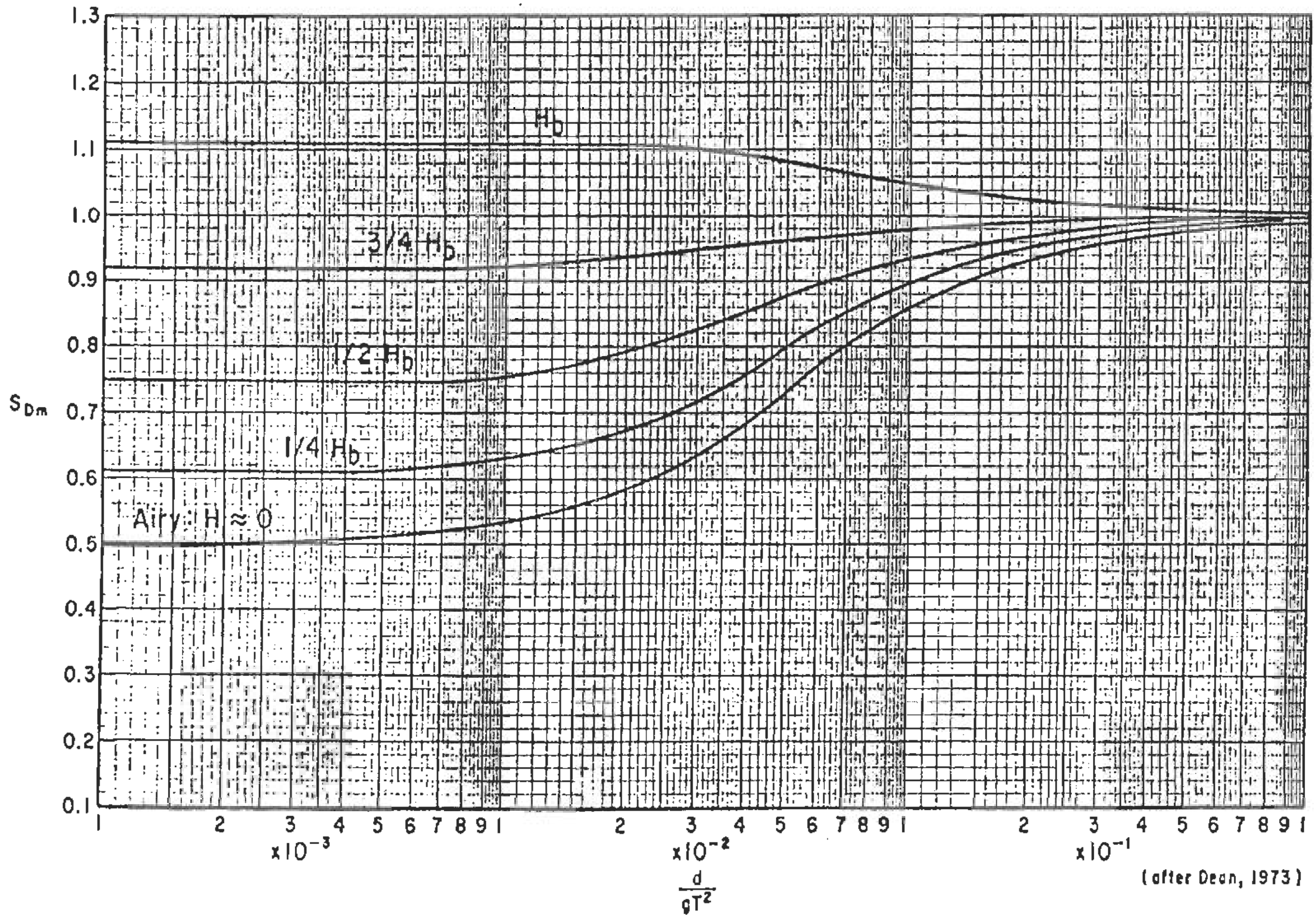


Figure 7-74. Drag force moment arm, S_{Dm}, versus relative depth, d/gT^2.

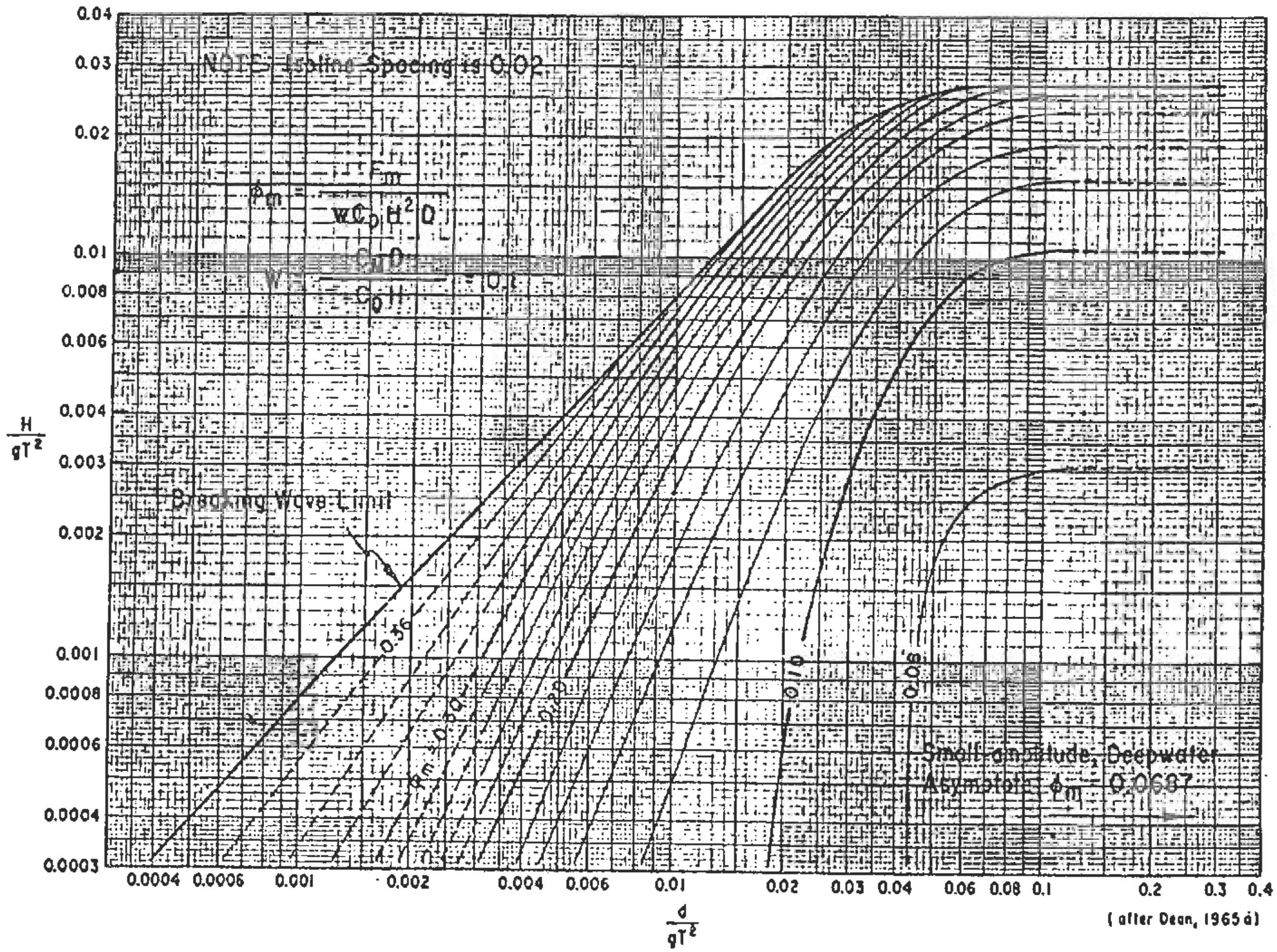


Figure 7-77. Isolines of ϕ_m versus H/gT^2 and d/gT^2 ... ($W = 0.1$) .

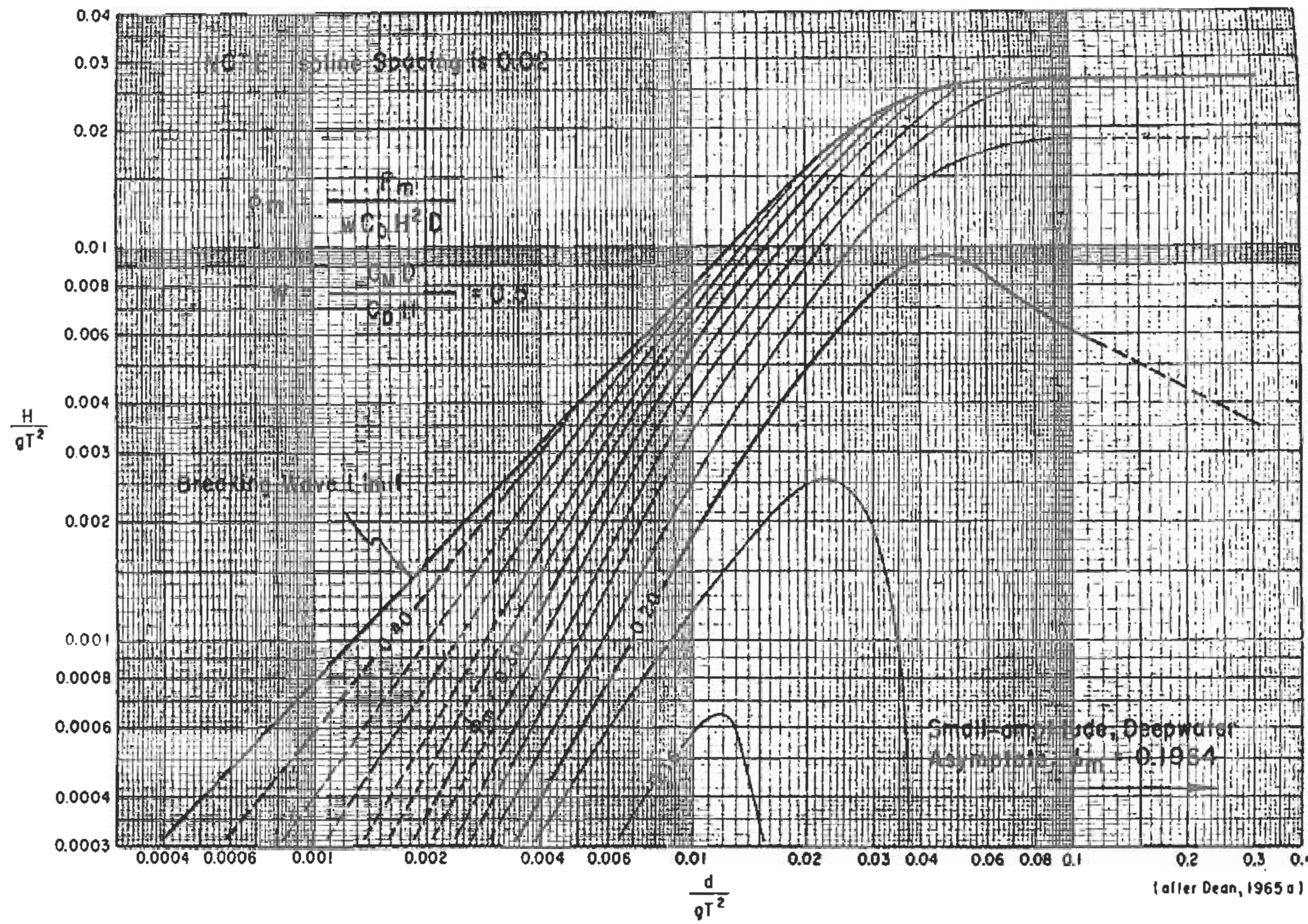


Figure 7-78. Isolines of ϕ_m versus H/gT^2 and d/gT^2 ... (W = 0.5) .

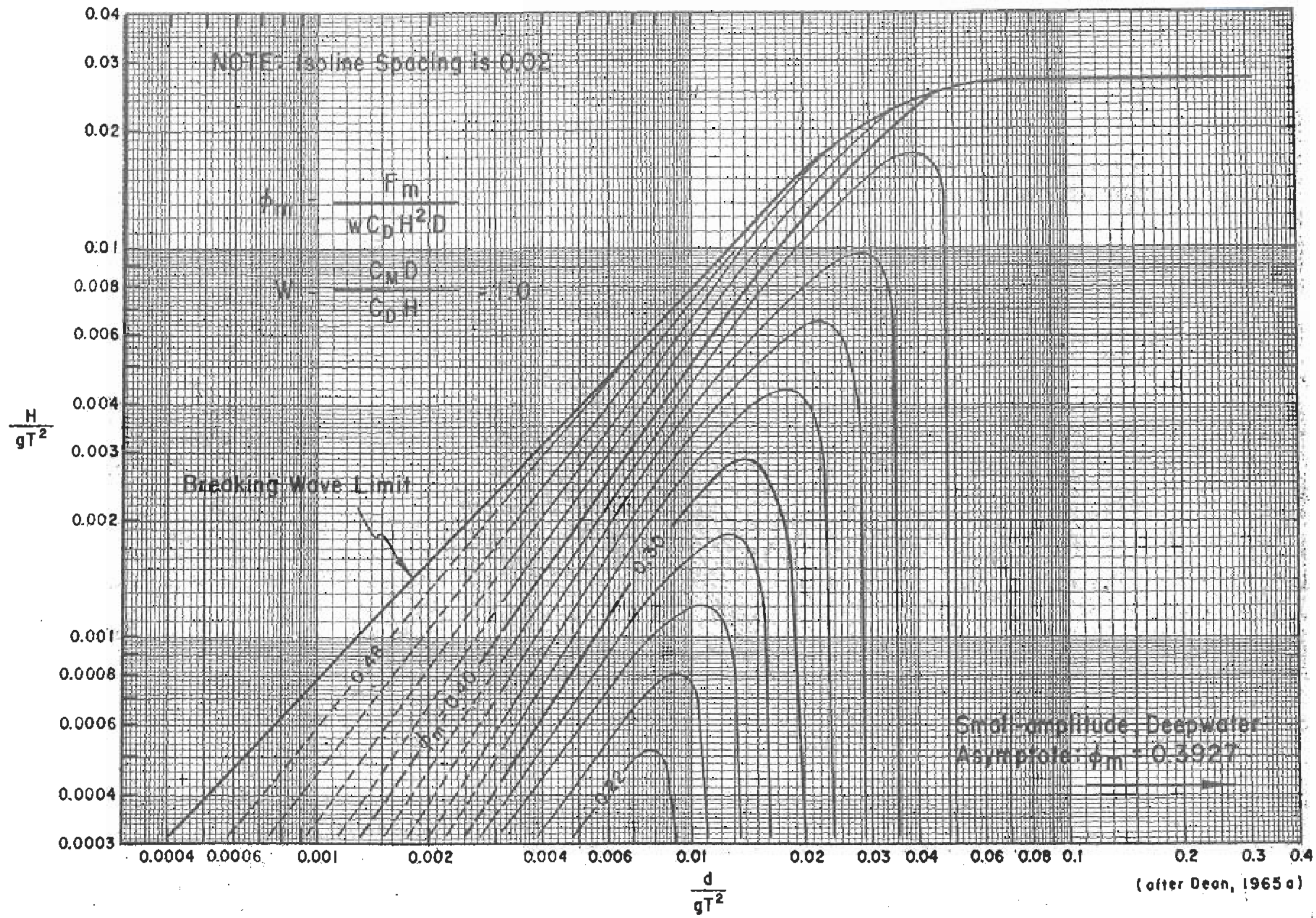


Figure 7-51. Isolines of ϕ_m Versus H/gT^2 and d/gT^2 ..... (W=1.0)

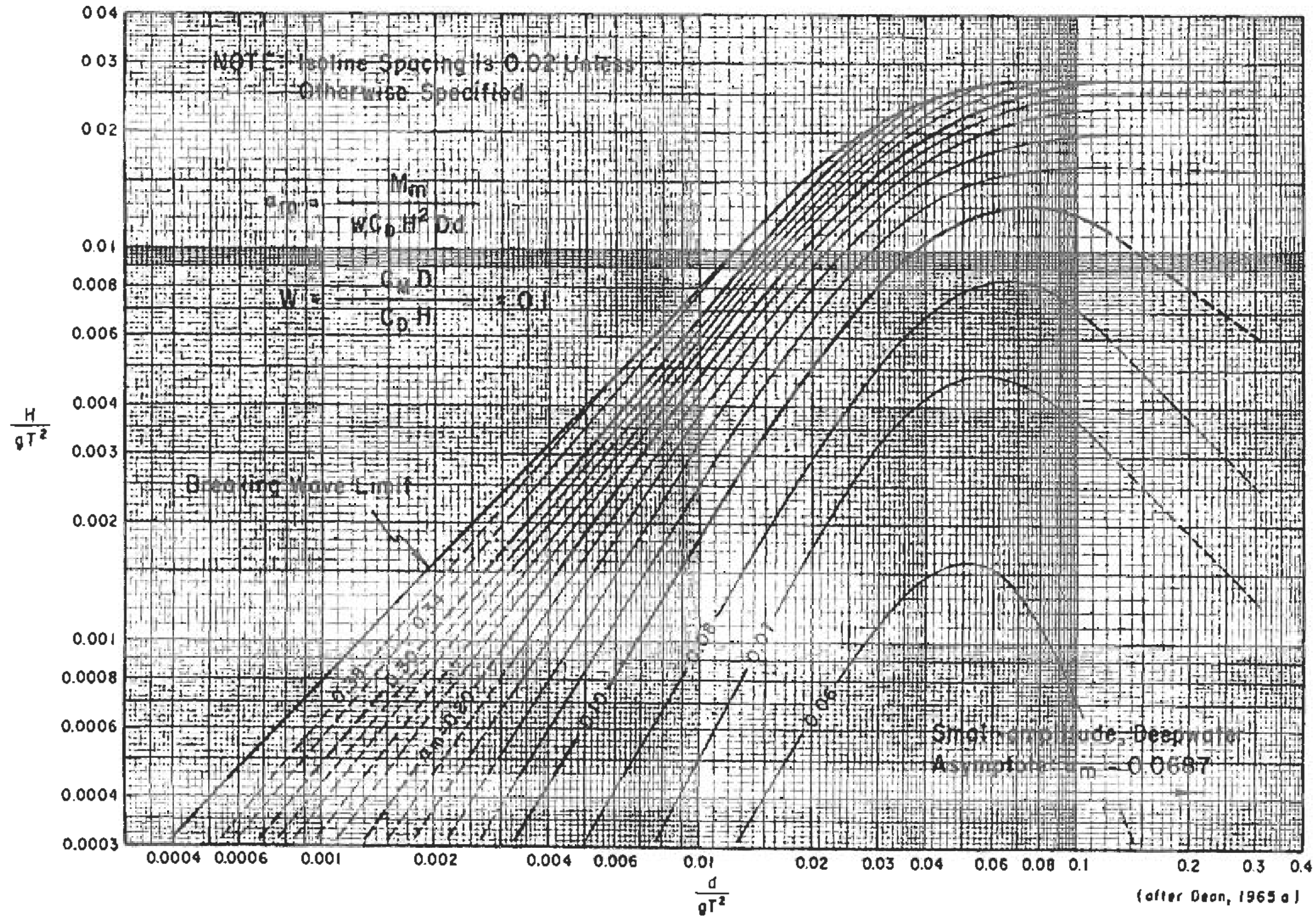


Figure 7-81. Isolines of α_m versus H/gT^2 and d/gT^2 ... (W = 0.1).

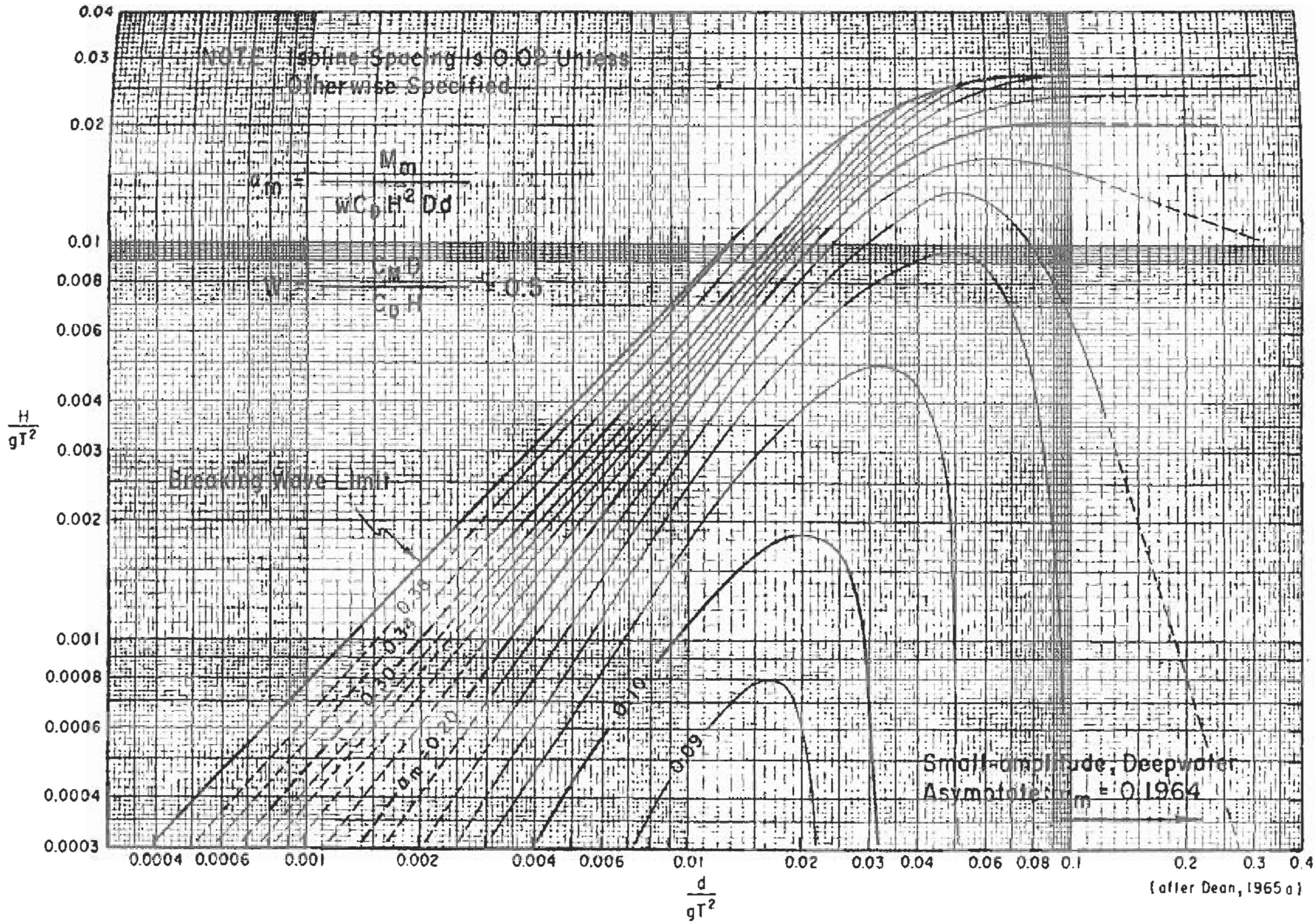


Figure 7-82. Isolines of α_m versus H/gT^2 and d/gT^2 ... (W = 0.5) .

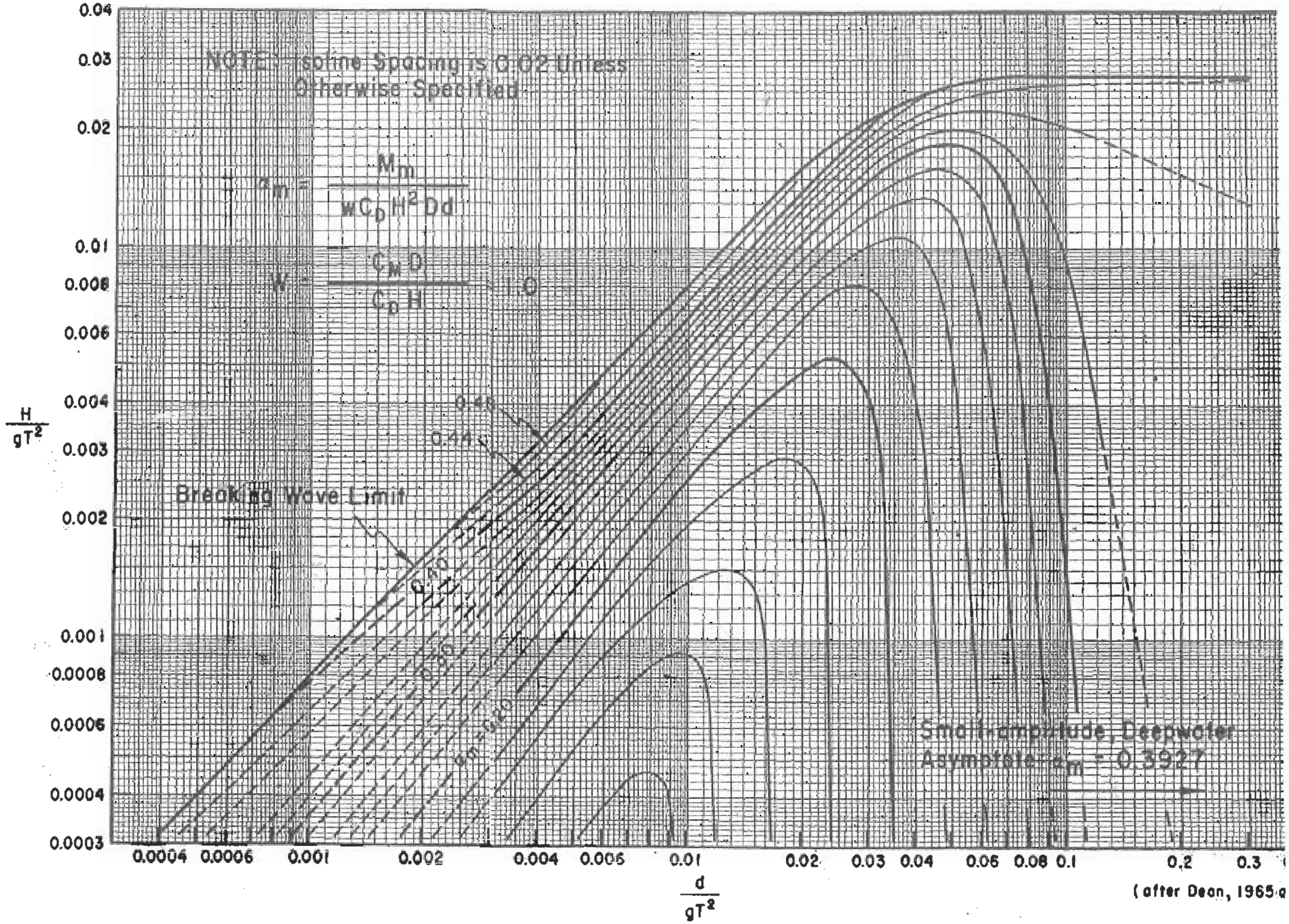


Figure 7-55. Isolines of α_m Versus H/gT^2 and d/gT^2 ..... (W=1.0)

SPM

NON-BREAKING WAVES Kf = 1.0

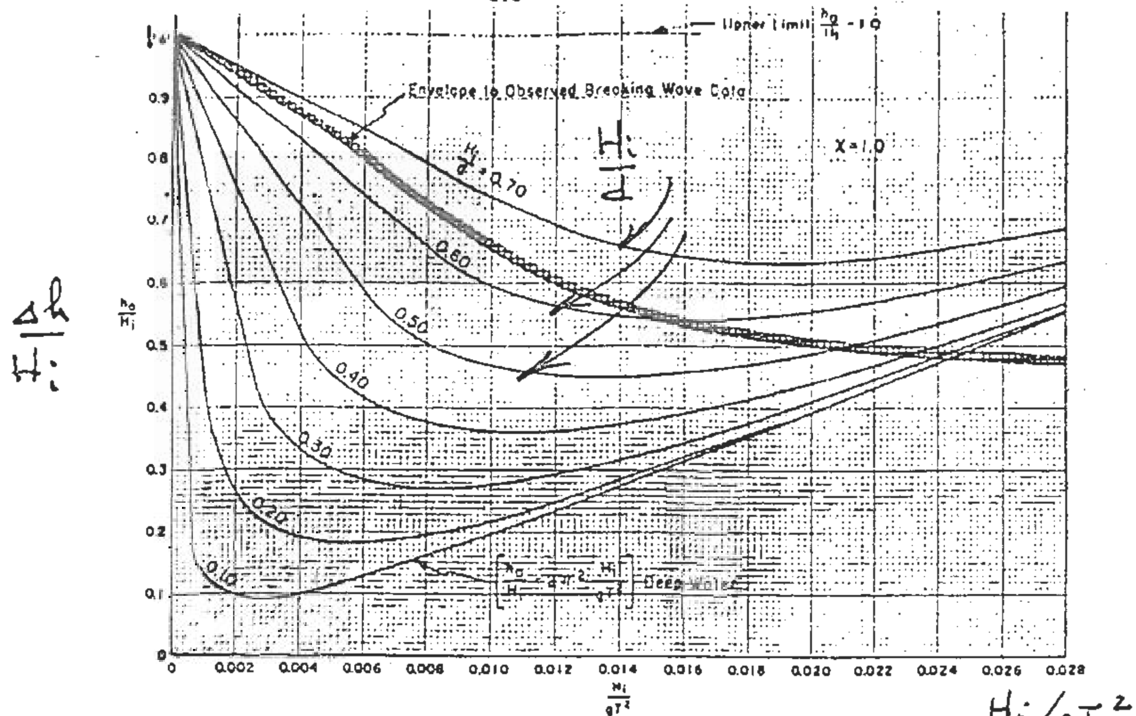
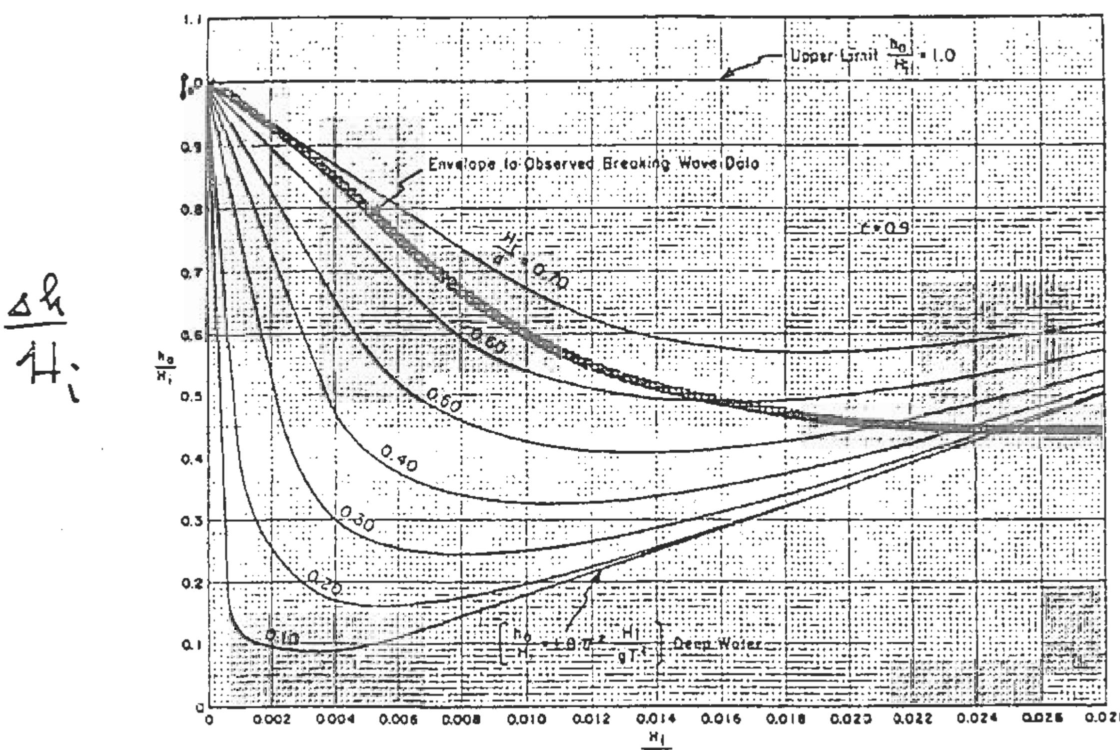


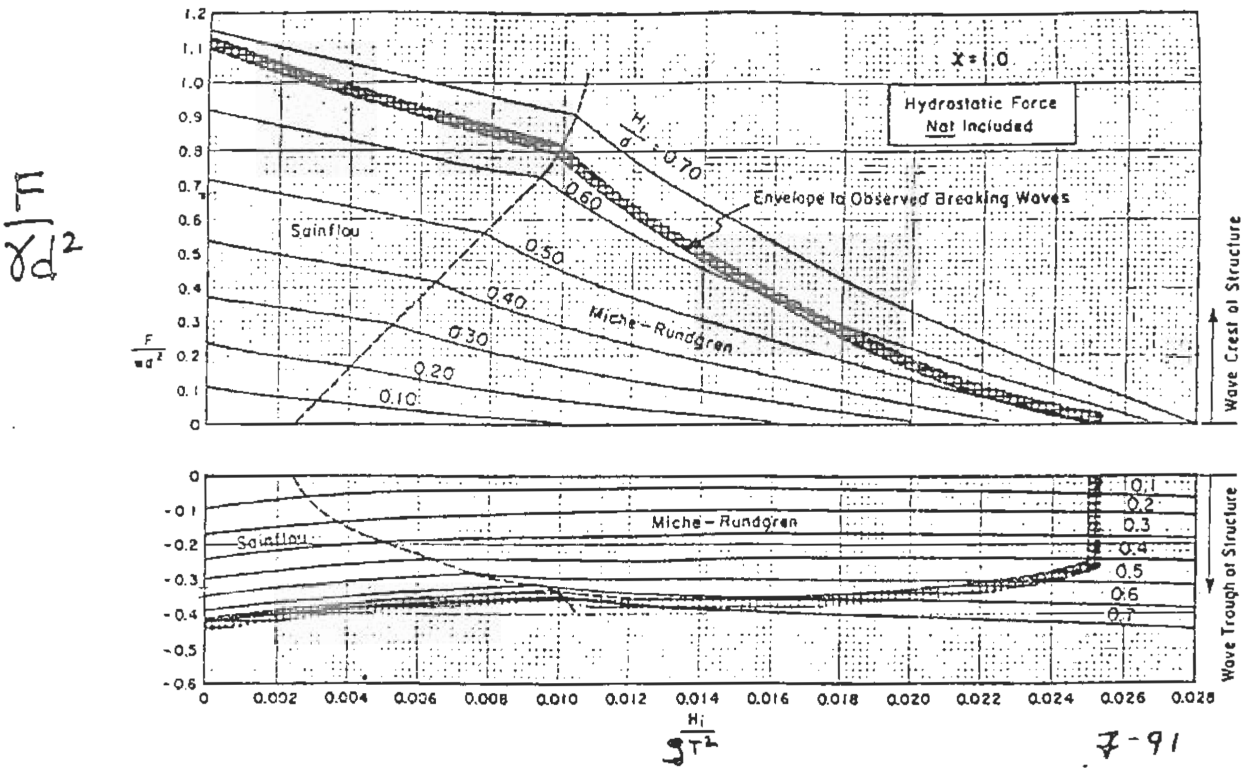
Figure 7-90. Nonbreaking waves; χ = 1.0 .



NON-BREAKING WAVES Kf = 0.90

SPM Fig 7-93

NON-BREAKING WAVES Kf = 1.0 [Net Force & Net Moment]



7-91

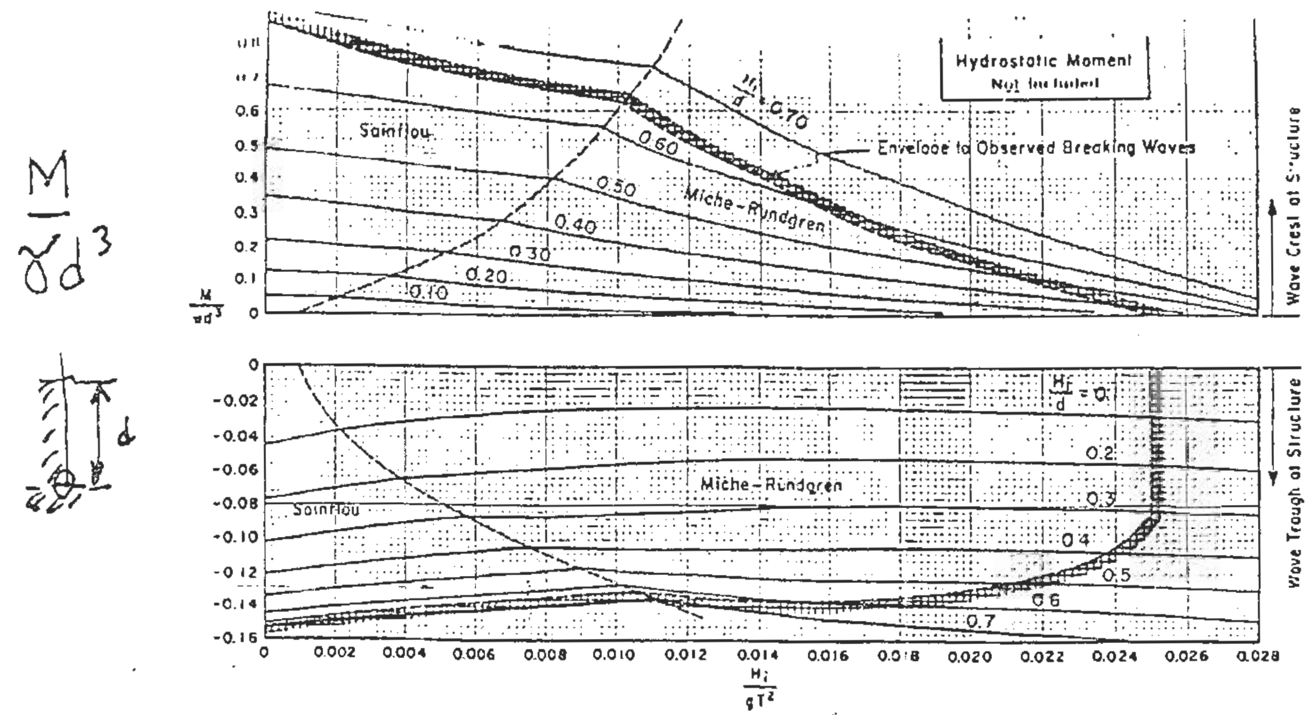


Figure 7-92. Nonbreaking wave moment; x = 1.0 .

7-92

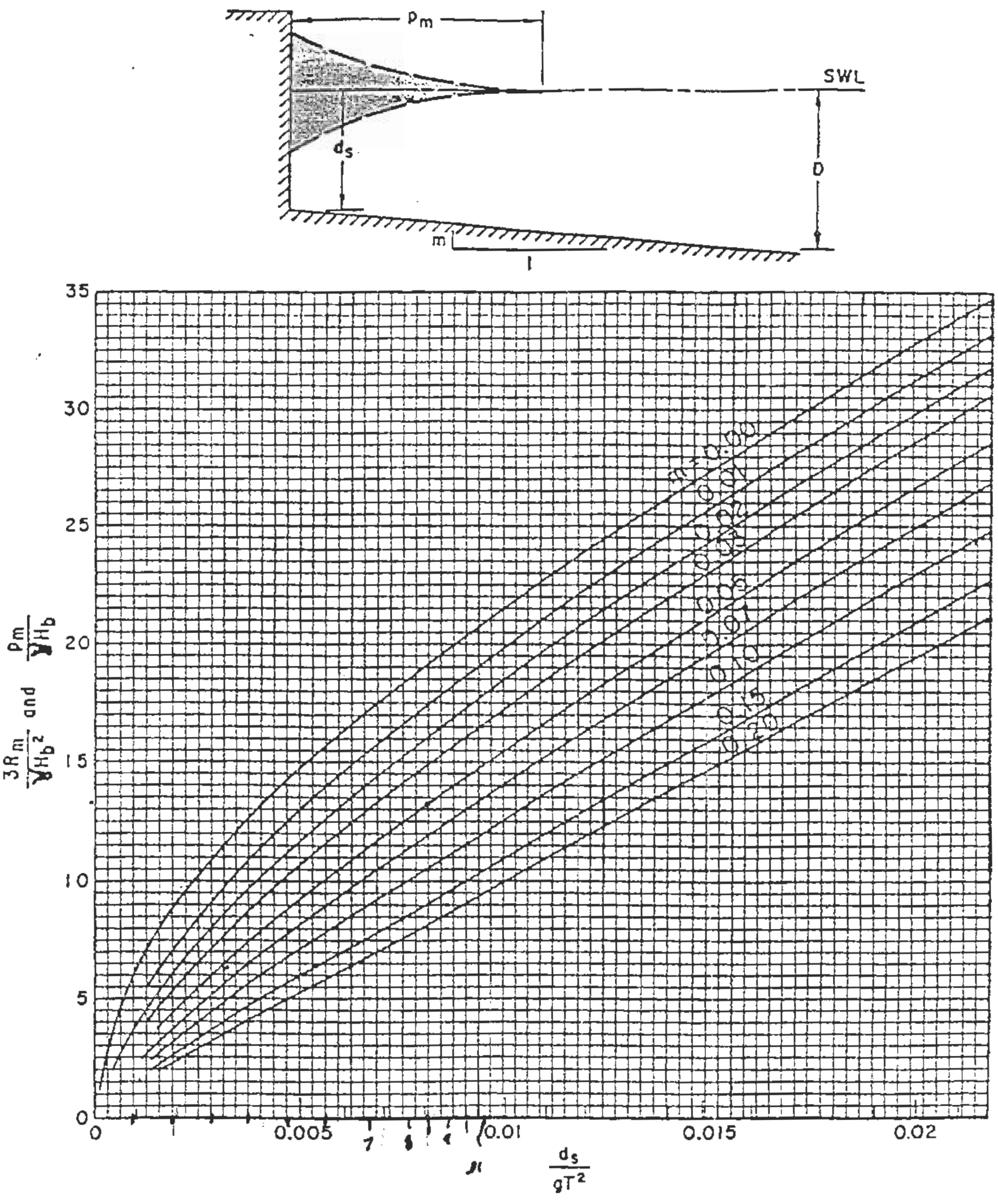


Figure 7-100. Dimensionless Minikin wave pressure and force.