WIND SPEED = U_A = 50 MPH
FETCH = F = 500 ft.
DEPTH OF WATER = D = 5 ft

FROM SHORE PROTECTION MANUAL, PG. 3-56, FIG 3-27(a)
WAVE HEIGHT = H = 0.5 ft
PERIOD = T = 0.9 sec.

FIND BREAKING WAVE HEIGHT, H_b,
FROM FIG 7-3 PG. 7-7

$$\frac{H}{gT^2} = \frac{0.5}{32.2\,(0.9)^2} = .0192$$

THUS $\frac{H_b}{H} = 1.0$ FOR 1:10 SLOPE

$$H_b = H = 0.5 \text{ ft}$$

$$\frac{H_b}{gT^2} = \frac{H}{gT^2} = .0192$$

FROM FIG. 7-2 PG. 7-6 USING A SLOPE OF 1:10

$\alpha \approx 1.6$

$\beta = \frac{d_B}{H_B} = 1.05$

$$d_{B\,max} = \alpha H_B = 1.6\,(0.5) = 0.8$$
$$d_{B\,MIN} = \beta H_B = 1.05\,(0.5) = 0.53$$

∴ THUS WITH A SLOPE OF 1:10 BREAKING COULD OCCUR WITH A DIKE TOE DEPTH BETWEEN 0.53 - 0.8 ft.

OUR DEPTH IS 8' SO ASSUME NON-BREAKING WAVE.

NOW FIND NON-BREAKING WAVE FORCE & MOMENTS ASSUMING A VERTICAL WALL

USE METHODS DESCRIBED ON PG. 7-161
ASSUME SMOOTH WALL $\chi = 1.0$

$H_i = H = 0.5$ ft
$d = 8$ ft
$T = 0.9$ s

$$\frac{H_i}{d} = \frac{0.5}{8} = 0.0625$$

$$\frac{H_i}{gT^2} = \frac{0.5}{(32.2)(0.9)^2} = .0192$$

FROM FIG. 7-90 FOR $H_i/gT^2 = .0192$

$$\frac{h_o}{H_i} \approx 0.21$$

$$h_o = 0.21\,H_i = 0.21(0.5) = 0.105 \text{ ft}$$

FROM EQS. 7-73 & 7-74 ON PG. 7-161 AND FIG. 7-88 ON PG. 7-162

$$y_c = d + h_o + \left(\frac{1+\chi}{2}\right) H_i \qquad (7\text{-}73)$$

$$y_c = 8 + 0.105 + \left(\frac{1+1}{2}\right) 0.5$$

$$y_c = 8.6 \text{ ft}$$

$$y_t = d + h_o - \left(\frac{1+\chi}{2}\right) H_i$$

$$y_t = 8 + 0.105 - \left(\frac{1+1}{2}\right) 0.5$$

$$y_t = 7.6 \text{ ft}$$

∴ THE WALL HAS TO BE ABOUT 8.6 ft TO PREVENT OVERTOPPING (WE ARE O.K.)

FROM FIG 7-91 ON PG. 7-165, THE DIM.-LESS FORCE IS FOUND TO BE (AT WAVE CREST)

$$\frac{F_c}{w d^2} = .001$$

$$F_c = .001\, w d^2 = .001 \left(66.8 \tfrac{\text{lb}}{\text{ft}^3}\right)(8 \text{ ft})^2$$

$$F_c = 4.2 \text{ lb/ft}$$

∴ HYDRODYNAMIC FORCES ARE NEGLIGIBLE

THE AVE. STATIC PRESSURE ON THE WALL IS

$F_H = \frac{1}{2} H w = \frac{1}{2}(8 ft)(66.8 \frac{lb}{ft^3}) = 267 \ lb/ft^2$

or per linear ft

$F_H = 267 (8 ft) = 2138 \ lb/ft$

COMPARING THIS TO THE SHEARING FORCES (F_S)
CALCULATED BY ED REED & ASSOC.
(FRICTION FACTOR OF 0.4 O.K.)

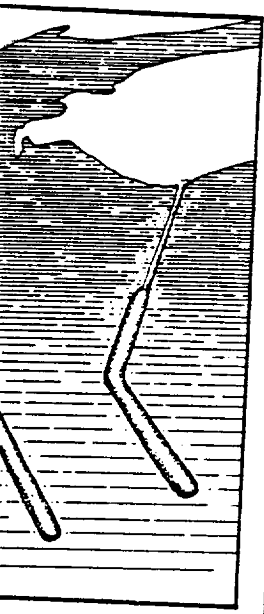
$F_S = 12{,}055$

SAFETY FACTOR $= \frac{F_S}{F_H} = \frac{12{,}055}{2138} = 5.64$

MY CALCULATIONS CONCUR.

SHORE PROTECTION MANUAL

VOLUME I



Coastal Engineering Research Center

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US ARMY ENGINEER WATERWAYS EXPERIMENT STATION

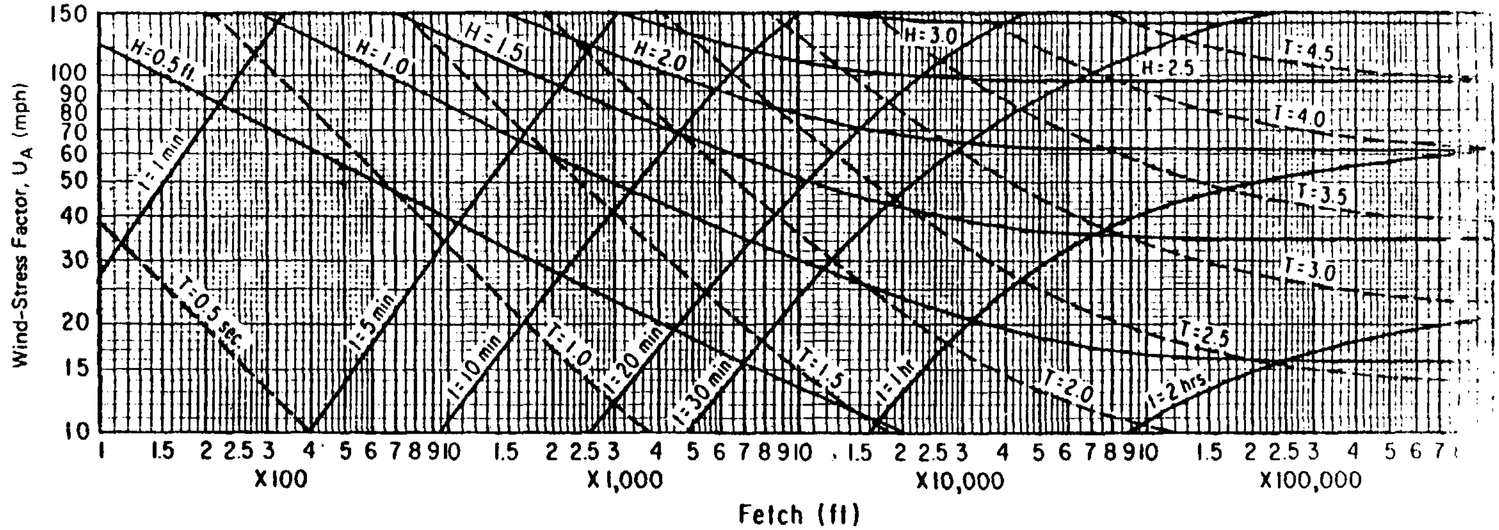
1984

Approved For Public Release; Distribution Unlimited

Prepared for

DEPARTMENT OF T[illegible] ARMY
US Army Corps [illegible]eers
Washington, D[illegible]

Note: Waves in a water depth of 5 feet with wave periods less than 1.4 secs. are considered to be deepwater waves, i.e. $d/T^2 > 2.56$.



Note: Waves in a water depth of 1.5 meters with wave periods less than 1.4 seconds are considered to be deepwater waves, i.e., $d/T^2 > 0.78$.

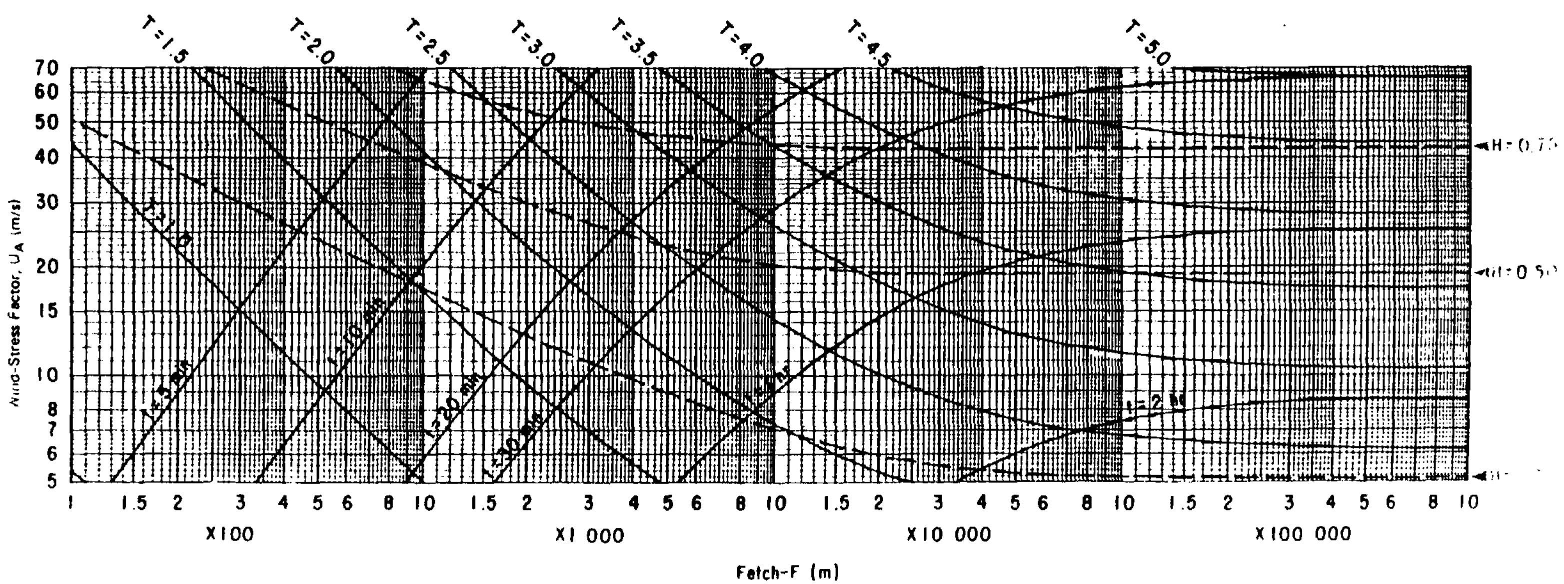


Figure 3-27. Forecasting curves for shallow-water waves; constant depths = 5 feet (upper graph) and 1.5 meters (lower graph).

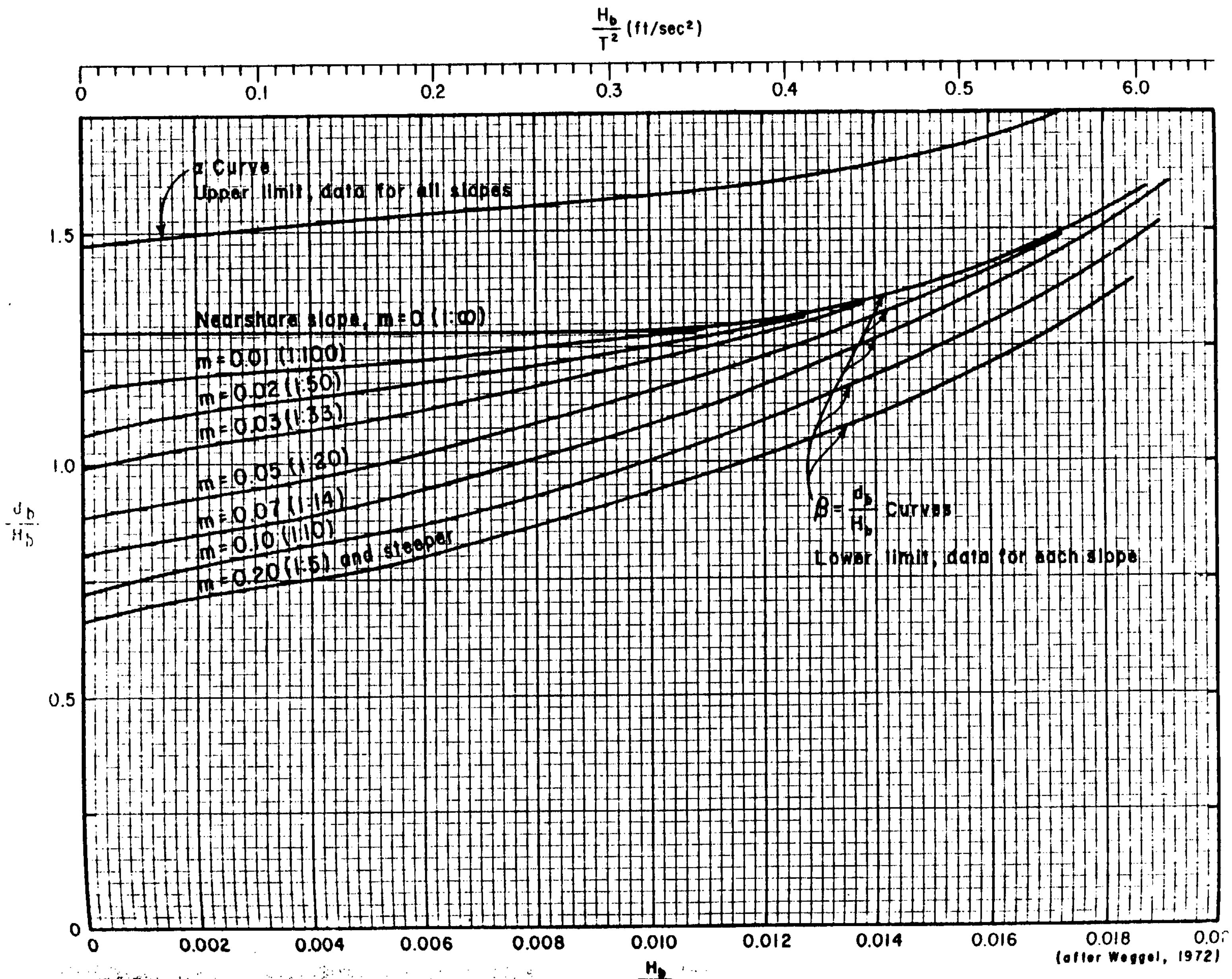


Figure 7-[illegible] α and β versus H_b/gT^2

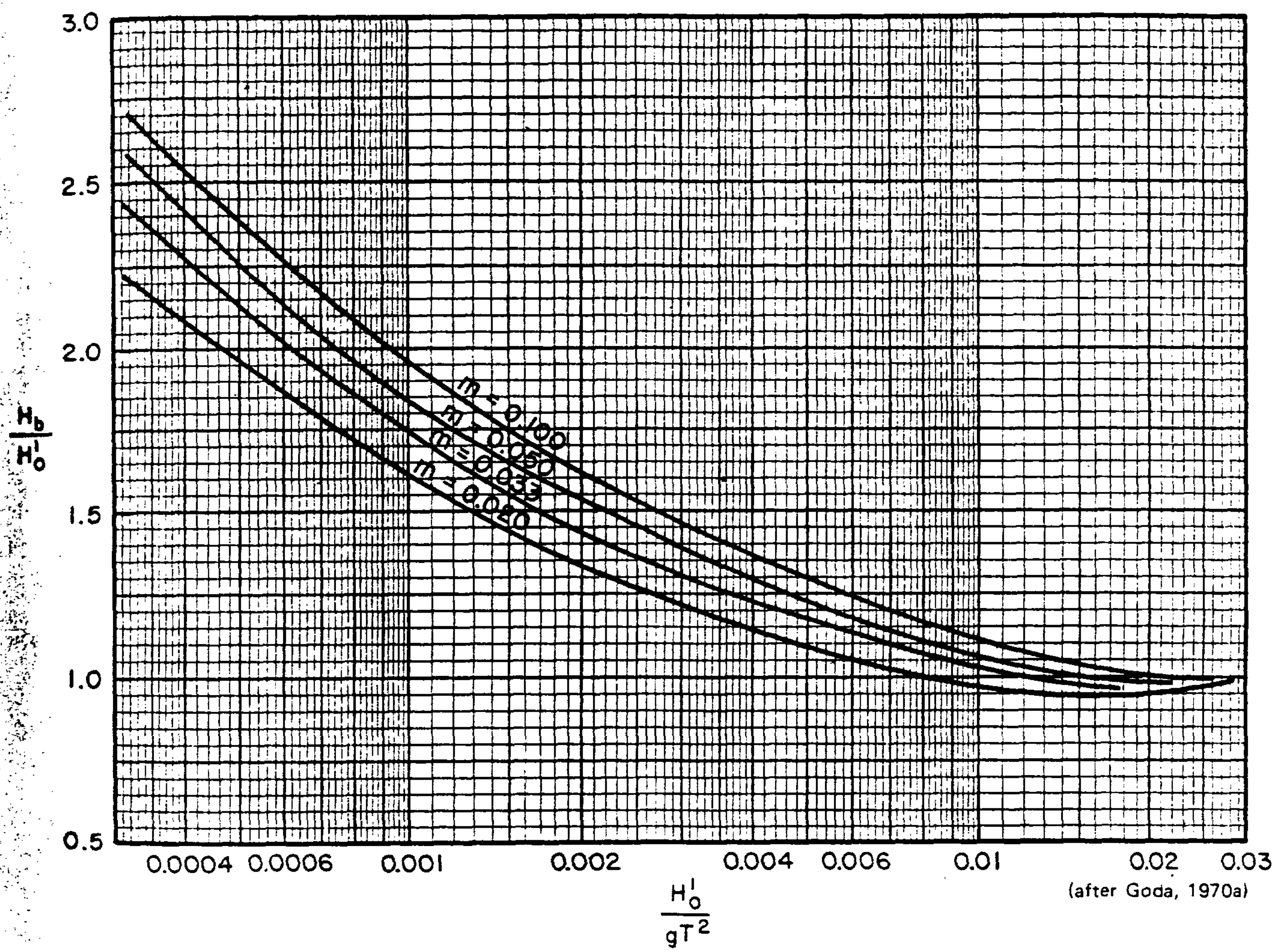


Figure 7-3. Breaker height index H_b/H_o' versus deepwater wave steepness H_o'/gT^2 .

2. Nonbreaking Wave Forces on Walls.

[illegible] an analysis of wave forces on structures, a distinction [illegible] between the action of nonbreaking, breaking, and broken waves (see [illegible] Selection of Design Waves). Forces due to nonbreaking waves are primarily hydrostatic. Broken and breaking waves exert an additional force due to the dynamic effects of turbulent water and the compression of entrapped air pockets. Dynamic forces may be much greater than hydrostatic forces; therefore, structures located where waves break are designed for greater forces than those exposed only to nonbreaking waves.

b. Nonbreaking Waves. Typically, shore structures are located in depths where waves will break against them. However, in protected regions, or where the fetch is limited, and when depth at the structure is greater than about 1.5 times the maximum expected wave height, nonbreaking waves may occur.

Sainflou (1928) proposed a method for determining the pressure due to nonbreaking waves. The advantage of his method has been ease of application, since the resulting pressure distribution may be reasonably approximated by a straight line. Experimental observations by Rundgren (1958) have indicated Saniflou's method overestimates the nonbreaking wave force for steep waves. The higher order theory by Miche (1944), as modified by Rundgren (1958), to consider the wave reflection coefficient of the structure, appears to best fit experimentally measured forces on vertical walls for steep waves, while Sainflou's theory gives better results for long waves of low steepness. Design curves presented here have been developed from the Miche-Rundgren equations and the Sainflou equations.

c. Miche-Rundgren: Nonbreaking Wave Forces. Wave conditions at a structure and seaward of a structure (when no reflected waves are shown) are depicted in Figure 7-88. The wave height that would exist at the structure if the structure were not present is the incident wave height H_i. The wave height that actually exists at the structure is the sum of H_i and the height of the wave reflected by the structure H_r. The wave reflection coefficient χ equals H_r/H_i. Wave height at the wall H_w is given as

$$H_w = H_i + H_r = (1 + \chi)\, H_i \tag{7-72}$$

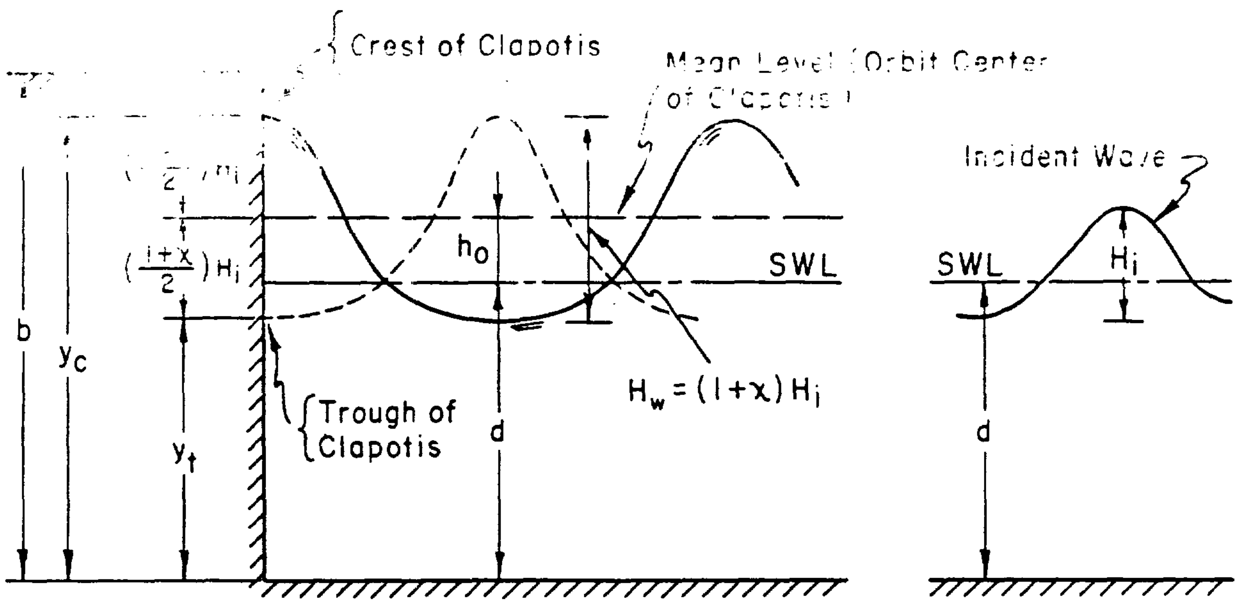
If reflection is complete and the reflected wave has the same amplitude as the incident wave, then $\chi = 1$ and the height of the *clapotis* or *standing wave* at the structure will be $2H_i$. (See Figure 7-88 for definition of terms associated with a clapotis at a vertical wall.) The height of the clapotis crest above the bottom is given by

$$y_c = d + h_o + \frac{1 + \chi}{2} H_i \tag{7-73}$$

where h_o is the height of the clapotis orbit center above SWL.

The [illegible] the clapotis trough above the bottom is given by

$$y_t = d + h_o - \frac{1 - \chi}{2} H_i \tag{7-74}$$



d = Depth from Stillwater Level

H_i = Height of Original Free Wave (In Water of Depth, d)

χ = Wave Reflection Coefficient

h_o = Height of Clapotis Orbit Center (Mean Water Level at Wall) Above the Stillwater Level (See Figures 7-90 and 7-93)

y_c = Depth from Clapotis Crest = $d + h_o + \left(\frac{1+\chi}{2}\right) H_i$

y_t = Depth from Clapotis Trough = $d + h_o - \left(\frac{1+\chi}{2}\right) H_i$

b = Height of Wall

Figure 7-88. Definition of Terms: nonbreaking wave forces.

The reflection coefficient, and consequently clapotis height and wave force, depends on the geometry and roughness of the reflecting wall and possibly on wave steepness and the "wave height-to-water depth" ratio. Domzig (1955) and Greslou and Mahe (1954) have shown that the reflection coefficient decreases with both increasing wave steepness and "wave height-to-water depth" ratio. Goda and Abe (1968) indicate that for reflection from smooth vertical walls this effect may be due to measurement techniques and could be only an apparent effect. Until additional research is available, it should be assumed that smooth vertical walls completely reflect incident waves and $\chi = 1$. Where wales, tiebacks, or other structural elements increase the surface roughness of the wall by retarding vertical motion of the water, a lower value of χ may be used. A lower value of χ also may be assumed when the wall is built on a rubble base or when rubble has been placed seaward of the structure [illegible]

[illegible] wall are shown in Figure 7-89. When the crest is at the wall, pressure

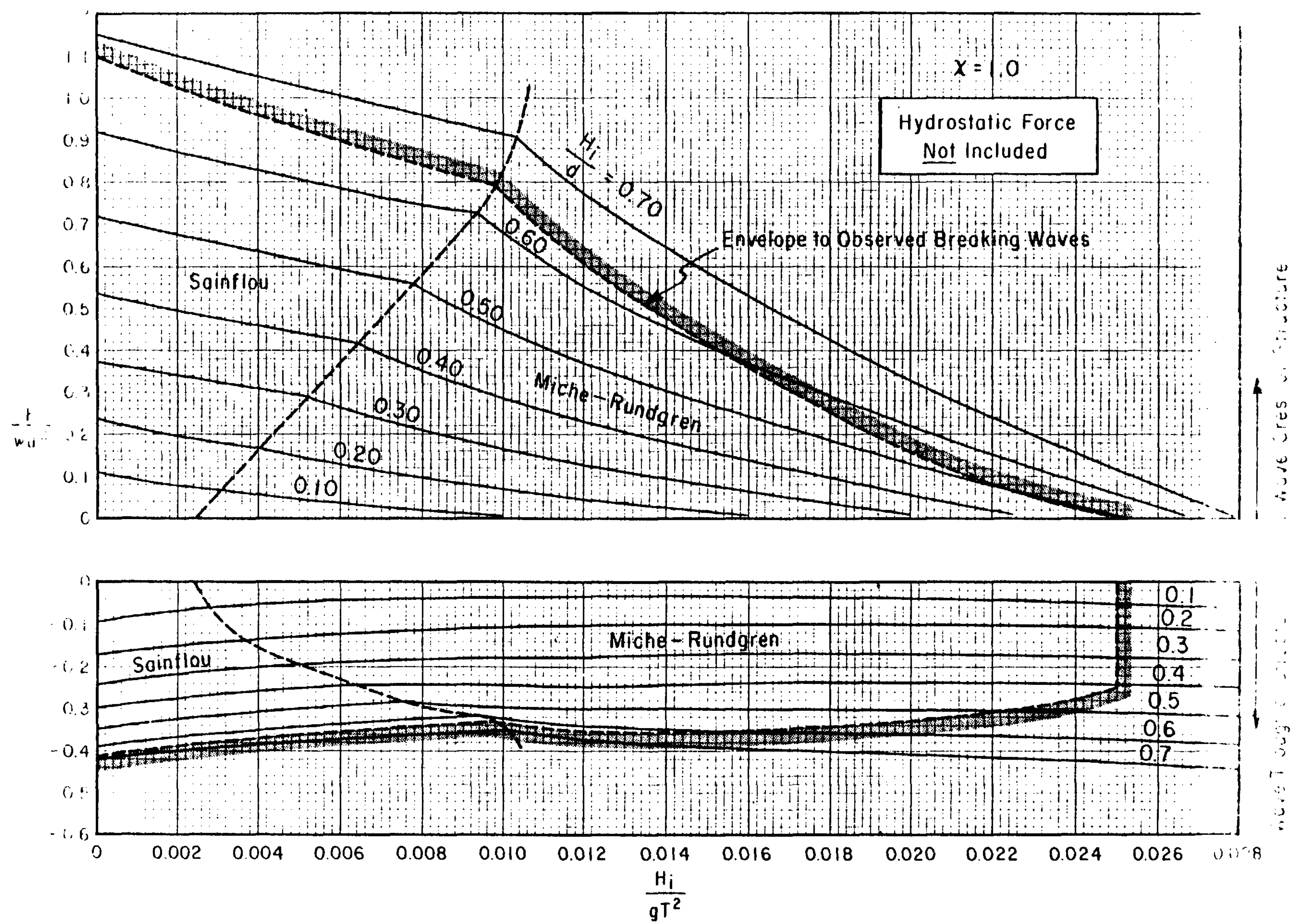


Figure 7-91. Nonbreaking wave forces; $\chi = 1.0$.