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Determination of Mooring Load and Transmitted Wave Height for a Floating Tire Breakwater

by

Michael L. Giles and James W. Eckert

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Floating tire breakwaters (FTB) are being used to protect and improve small-craft harbors, and as the need for additional mooring space increases, FTB's are often being placed in locations exposed to larger waves. Other uses for FTB's include protection of construction operations, protection of dredges, and beach stabilization. Methods for predicting the transmitted wave height, as well as for determining the anchor loading for the Goodyear module FTB, are presented.

(continued)

These methods are based on laboratory tests that used full-scale monochromatic wave conditions typical of partially sheltered bodies of water.

Wave transmission is given as a function of the ratio of the breakwater width to incident wavelength. The mooring load is also given as a function of incident wave height. Design curves and procedures are presented for determining the breakwater width required to obtain a desired degree of wave attenuation, and for determining the mooring loads for each anchor line. Various anchor types are discussed to aid in the design of an anchor system.

PREFACE

This report presents techniques for estimating wave transmission and anchor loading for the Goodyear Module Floating Tire Breakwater when subject to given incident wave conditions. A discussion of anchor selection is also presented.

The report was prepared by Michael L. Giles, while a member of the Coastal Structures Branch, and James W. Eckert, Coastal Design Criteria Branch, under the general supervision of Dr. R.M. Sorensen, Chief, Coastal Structures Branch.

The authors acknowledge the useful suggestions provided by R.A. Jachowski and Dr. F.E. Camfield of the Coastal Design Criteria Branch.

Comments on this publication are invited.

Approved for publication in accordance with Public Law 166, 79th Congress, approved 31 July 1945, as supplemented by Public Law 172, 88th Congress, approved 7 November 1963.

Ted E. Bishop

TED E. BISHOP
Colonel, Corps of Engineers
Commander and Director

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CONVERSION FACTORS, U.S. CUSTOMARY TO METRIC (SI) UNITS OF MEASUREMENT

U.S. customary units of measurement used in this report can be converted to metric (SI) units as follows:

Multiply	by	To obtain
inches	25.4	millimeters
	2.54	centimeters
square inches	6.452	square centimeters
cubic inches	16.39	cubic centimeters
feet	30.48	centimeters
	0.3048	meters
square feet	0.0929	square meters
cubic feet	0.0283	cubic meters
yards	0.9144	meters
square yards	0.836	square meters
cubic yards	0.7646	cubic meters
miles	1.6093	kilometers
square miles	259.0	hectares
knots	1.852	kilometers per hour
acres	0.4047	hectares
foot-pounds	1.3558	newton meters
millibars	1.0197×10^{-3}	kilograms per square centimeter
ounces	28.35	grams
pounds	453.6	grams
	0.4536	kilograms
ton, long	1.0160	metric tons
ton, short	0.9072	metric tons
degrees (angle)	0.01745	radians
Fahrenheit degrees	5/9	Celsius degrees or Kelvins[1]

[1]To obtain Celsius (C) temperature readings from Fahrenheit (F) readings, use formula: $C = (5/9)\ (F - 32)$.

To obtain Kelvin (K) readings, use formula: $K = (5/9)\ (F - 32) + 273.15$.

SYMBOLS AND DEFINITIONS

c_u	undrained shear strength
D	pile diameter
d	water depth
E	modulus of elasticity
e	lever arm of lateral load above firm soil
F_s	factor of safety
F_t	lateral mooring-line load (tension)
f	depth of maximum bending
g	acceleration of gravity
H_i	incident wave height
H_t	transmitted wave height
I	moment of inertia of pile cross section
K_p	coefficient of passive Earth pressure
K_t	transmission coefficient; H_t/H_i
L	incident wavelength
ℓ	embedment length of short-pile anchor
ℓ_t	total required length of pile anchor
M_{yield}	yielding moment of pile section
n_h	constant of horizontal subgrade reaction
P	design lateral load on pile
q	length of pile below maximum moment point
T	wave period
W	breakwater width measured in the direction of wave travel
W_t	weight of concrete anchor
w_c	unit weight of concrete
w_s	unit weight of soil
w_w	unit weight of water
η	short-pile coefficient
μ	coefficient of static friction
ϕ	internal friction angle

DETERMINATION OF MOORING LOAD AND
TRANSMITTED WAVE HEIGHT FOR A FLOATING TIRE BREAKWATER

by
Michael L. Giles and James W. Eckert

I. INTRODUCTION

This report presents methods for predicting the transmitted wave height and required anchor capacity for a floating tire breakwater (FTB) using the FTB module concept proposed by the Goodyear Tire and Rubber Co. (Candle and Fischer, 1977). The methods are based on prototype-scale wave tank tests of the Goodyear module FTB (Giles and Sorensen, 1978).

Because of the ease of module construction and availability of used tires, this type of FTB provides an alternative means for sheltering shorelines, docks, and boats from both storm and normal wave conditions. In comparison to other types of floating or fixed breakwater structures, the proposed module design has a relatively low cost. Because floating breakwaters are most effective for short-period waves, this type of FTB may best be used as protection for harbors of refuge and for shorelines in which the waves are limited by fetch or water depths such as in large coves, estuaries, and reservoirs.

The FTB is assembled using individual 18-tire modules (Fig. 1) measuring approximately 6.5 by 7.0 by 2.5 feet (2.0 by 2.2 by 0.8 meters). The modules are constructed by stacking the tires in a 3-2-3-2-3-2-3 combination and threading tying lines through the tires as they are stacked. An evaluation of various types of tying materials for both freshwater and saltwater environments (Davis, 1977) has found that conveyor belting and unwelded open-link chain were the optimum choices for corrosional resistance. Typically, the FTB has flotation material added to the crown of each tire and two 2-inch-diameter (5 centimeters) holes are punched in the bottom of each tire. The use of flotation material, such as rigid urethane or polystyrene, will maintain uniform flotation of the breakwater and will permit the use of severely damaged tires which otherwise could not be used. The holes are to reduce the amount of sand and debris which may accumulate in the tires. Additional details on the construction of the individual modules and assembly of the modules to form a complete breakwater are presented by Candle and Fischer (1977) and by Kowalski and Ross (1975).

The data and design curves presented are applicable for wave heights up to about 4.5 feet (1.4 meters), wavelengths between 30 and 165 feet (9.5 and 50 meters), and water depths between 6.5 and 13 feet (2 and 4 meters). If design conditions are significantly different, then care and engineering judgment should be used in applying these design procedures.

II. DETERMINATION OF BREAKWATER WIDTH

For specific site conditions, given the design incident significant wave height, H_i, wave period, T, water depth, d, and transmitted

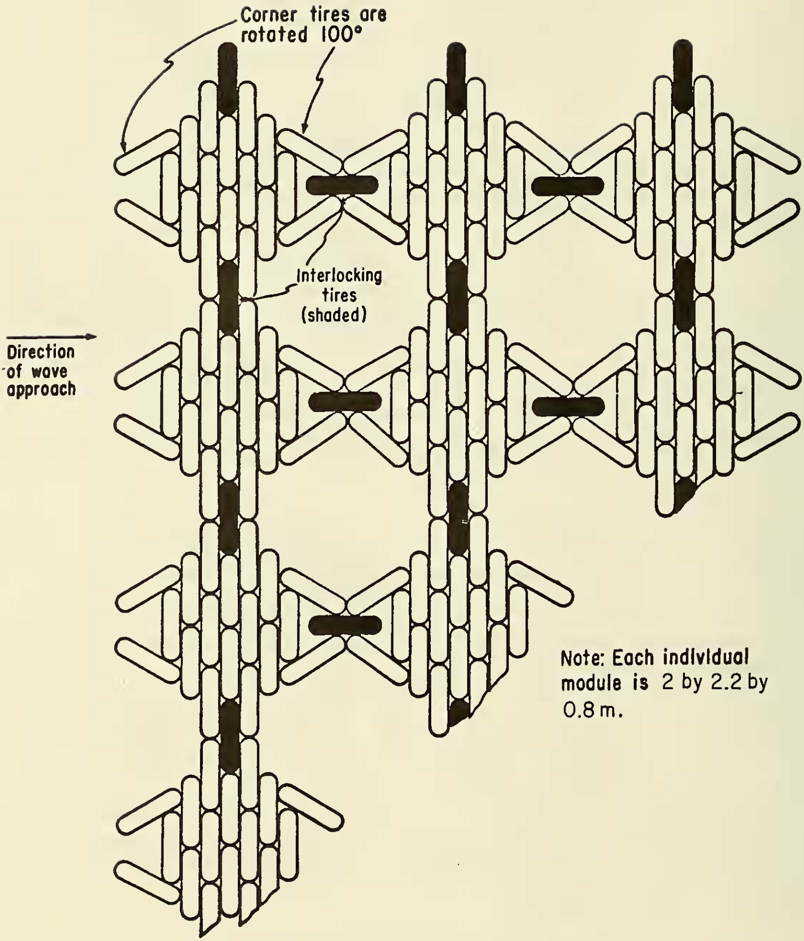

Figure 1. Section of assembled breakwater composed of individual modules.

wave height, H_t; the wavelength, L, and transmission coefficient, K_t, can be determined. The wavelength, L, can be determined for a given wave period, T, and water depth, d, by use of Figure 2 or the equation.

$$L = \frac{gT^2}{2\pi} \tanh\left(\frac{2\pi d}{L}\right). \tag{1}$$

The transmission coefficient, K_t, can be determined using the following relationship:

$$K_t = \frac{H_t}{H_i}. \tag{2}$$

Knowing the wavelength, L, and the transmission coefficient, K_t, the required breakwater width can be determined from Figure 3. Since the overall efficiency of the breakwater decreases as the breakwater width increases, the width determined from Figure 3 may be too small if the indicated breakwater width is much greater than the maximum tested length of 42 feet (12.8 meters).

III. DETERMINATION OF MOORING LOADS

Giles and Sorensen (1978) found that the mooring-line load for the FTB system is essentially a function of the incident wave height. Figure 4 provides a procedure for determining the mooring-line load for a given incident wave height.

Since the design curve is based on limited data, care and engineering judgment should be used in extrapolating the curve for wave heights greater than 4.5 feet. Also, if the breakwater width to wavelength ratio is greater than 1.4 the actual load on the anchor may be slightly higher due to additional modules being added.

Figure 4 is applicable to mooring lines placed on a slope of 1 on 7. If steeper mooring-line slopes are used, then the loads would be slightly higher and proportional to the change in the tangent of the slope.

The rear mooring system should be designed for the largest force determined by either the force of the largest wave coming from the shoreward direction or 20 percent of the seaward force, whichever is greater.

IV. SELECTION OF A MOORING SYSTEM

1. Selection of the Mooring Line and Hardware.

To minimize the vertical load on the anchor, the mooring line should have a minimum length of approximately eight times the maximum expected water depth and the anchor should be positioned seven times the maximum water depth from the breakwater. The anchor holding capacity must exceed

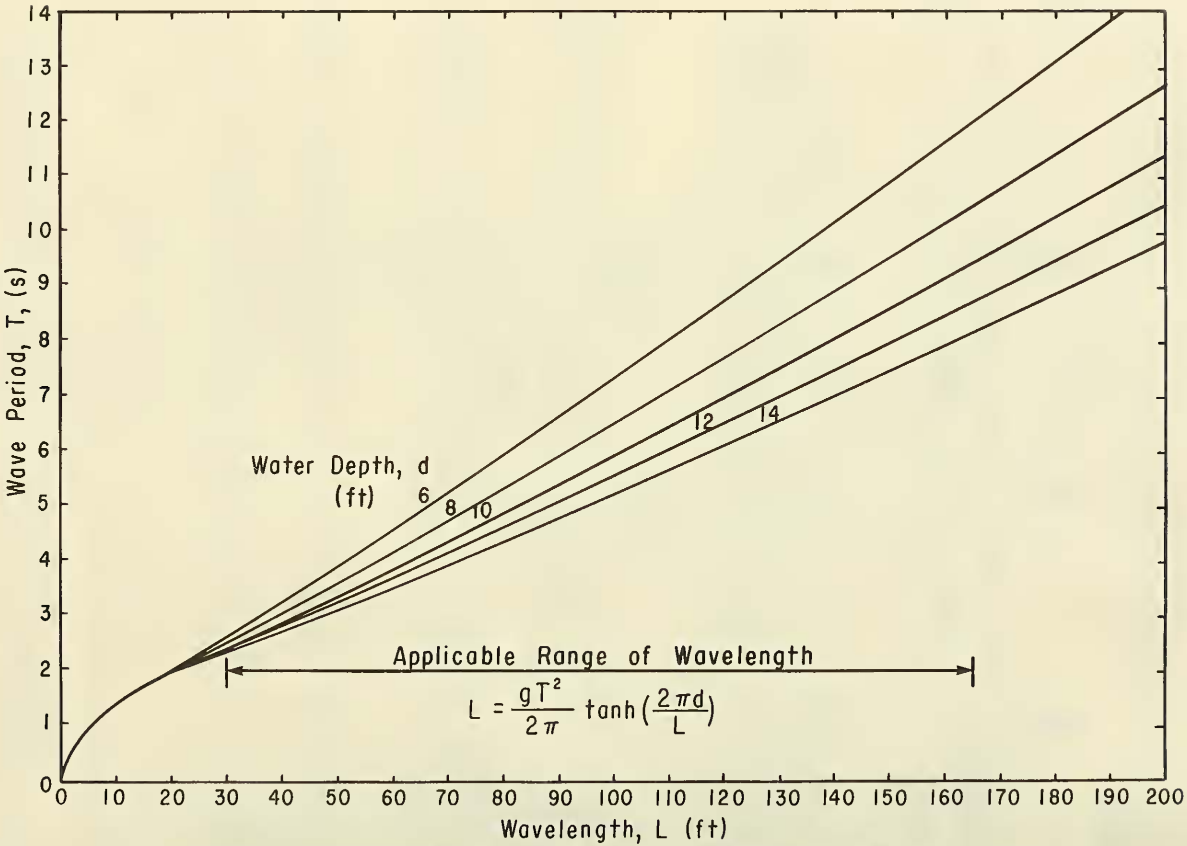

Figure 2. Relationship between wave period, wavelength, and water depth.

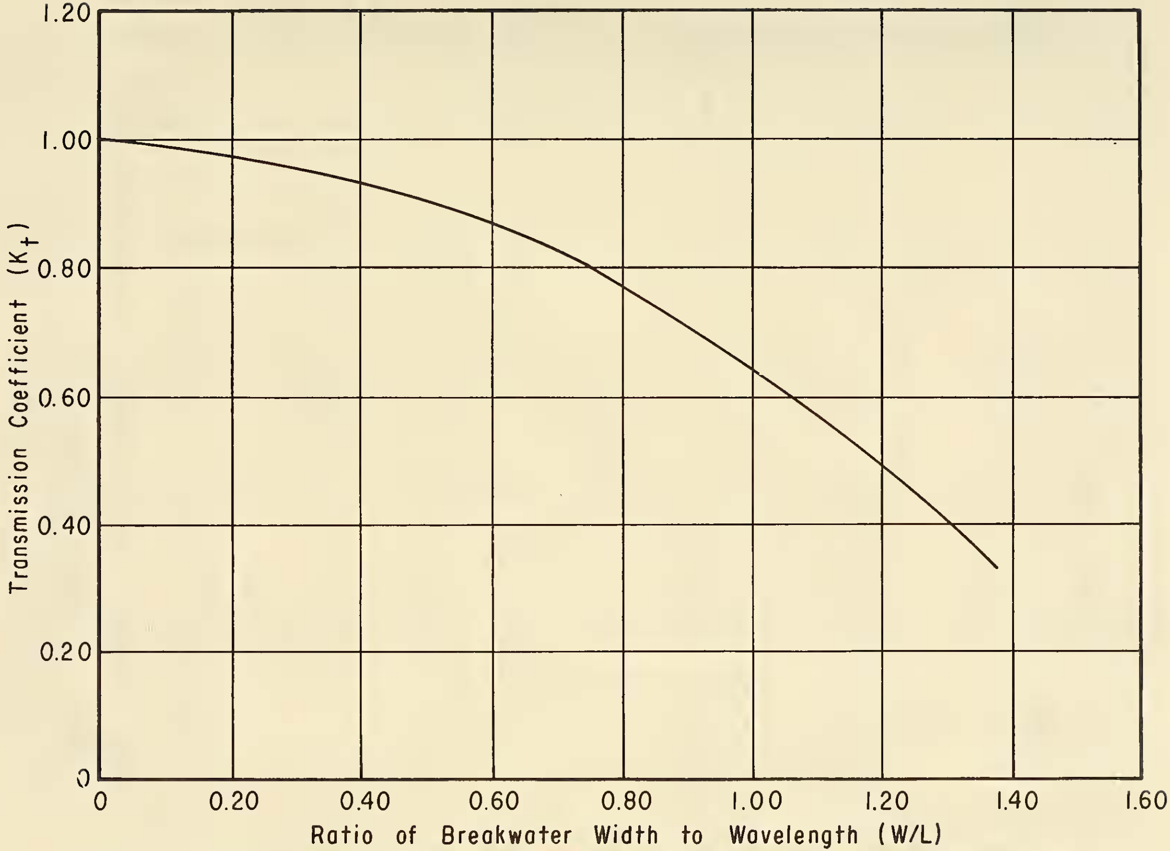

Figure 3. Design curve for predicting the transmission coefficient knowing the W/L ratio.

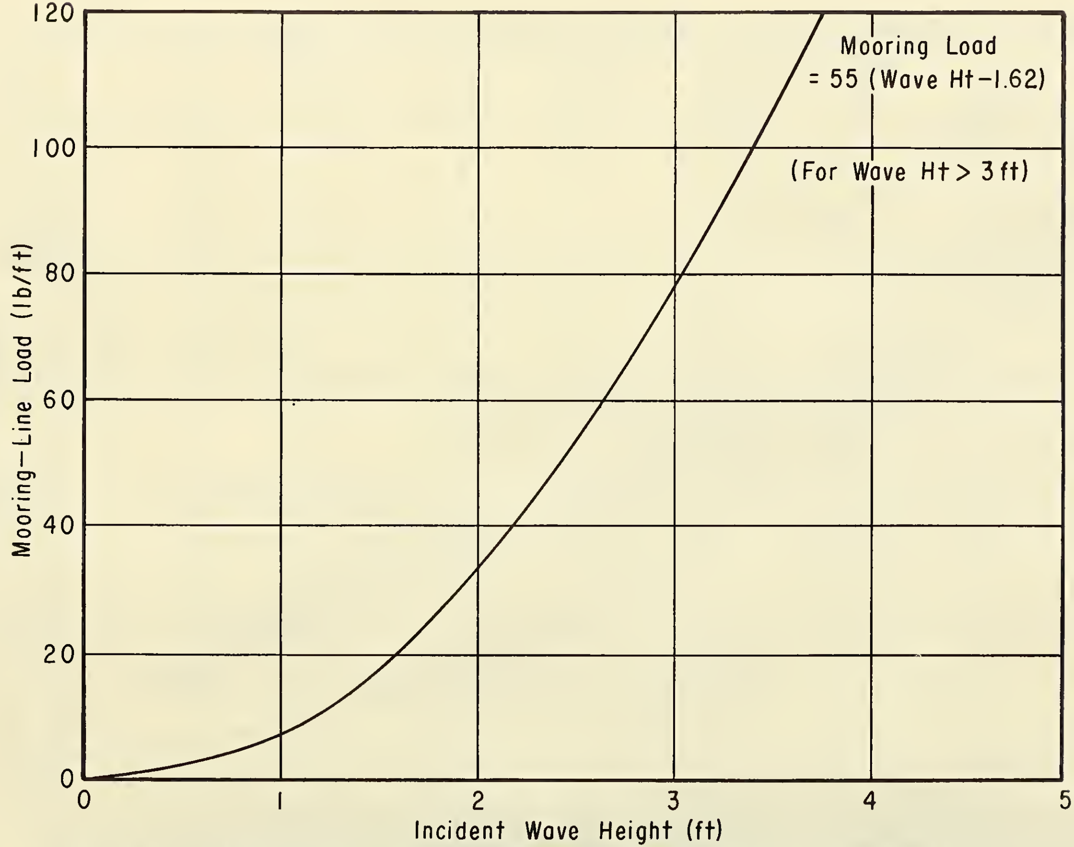

Figure 4. Design curve for predicting mooring loads per foot of breakwater length for a given incident wave height.

the maximum force expected to occur. Since the forces are cyclic, all the connections between the breakwater and anchor should be as flexible and free-moving as possible. Therefore, it is suggested that either galvanized steel or wrought-iron chain be used. Wire cable has been used, but the cable is subject to both axial fatigue and corrosion weakening. All connections should also be oversized to allow for corrosion and wear. Secondary methods of connection, such as cotter pins and extra nuts, should be used to prevent disconnection.

2. Selection of the Anchor.

The selection of the type of anchor depends on the maximum mooring force, bottom conditions (i.e., mud, sand, or rock bottom), and the various available methods for placing the anchor. The four basic anchor types normally used with floating breakwaters are deadweight anchors, embedment anchors, screw anchors, and pile anchors.

The most commonly used anchor is the concrete block deadweight anchor which is usually cast at the site. The design anchor weight (W_t) of these anchors can be determined by the following relationship based on a static analysis:

$$W_t = \frac{F_t F_s}{\mu\left(1 - \frac{w_w}{w_c}\right)} \tag{3}$$

where

μ = the coefficient of static friction

W_t = the total weight of concrete anchor in air

w_w = the unit of weight in water

w_c = the unit weight of concrete

F_t = the lateral mooring-line load

F_s = the factor of safety

The embedment anchor is often used by small-boat operators. The holding capacity of embedment anchors vary with the type or marine soil and embedment anchor design. This is discussed by Taylor and Lee (1972) and Berteaux (1976).

Vesic (1971) and Jenkins (1976) describe methods for determining the holding capacity of the screw anchor in various soil types. They indicate that the maximum holding capacity is equivalent to the capacity developed by a short pile of equal length. Disadvantages of the screw anchor are that they usually have a short length and are difficult to install in firm marine soils.

When the available equipment and materials suggest a pile anchor system be used to hold the floating breakwater, the required design may be accomplished by one of the methods described below. Anchor piles are designed by finding the ultimate lateral resistance of the pile-soil system and increasing the lateral mooring load, F_t, by a safety factor, F_s, to find the design lateral load on the pile, P; i.e.,

$$P = F_t F_s \ . \tag{4}$$

The ultimate lateral resistance of the anchor pile is reached when either the passive strength of the surrounding soil is exceeded or when the yielding moment of the pile section is reached.

Simple design methods, as described in Broms (1964a, 1964b), are divided according to the soil characteristics (cohesionless or cohesive) and by the pile characteristics (short-rigid piles or long-flexible piles). Only the cohesionless and cohesive short-rigid pile cases are included here because these will normally suffice for anchor piles for floating breakwaters.

In considering cohesionless soils (i.e., sands), the definition of long versus short piles depends on the calculation of the dimensionless term, $\eta \ell$, where:

$$\eta = \left[\frac{n_h}{EI}\right]^{1/5} \tag{5}$$

and ℓ is the pile length. This term includes the pile section stiffness (EI) and the constant of horizontal subgrade reaction, n_h, which is a function of the soil only. Values of n_h are given in the Table; when $\eta \ell$ is less than 2.0 the pile is considered short and rigid and when $\eta \ell$ is greater than 4.0 the pile is long and flexible.

Table. Values of n_h (from Terzaghi, 1955).

Embedded soil condition	Relative density (tons/ft^3)		
	Loose	Medium	Dense
Above water table	7	21	56
Below water table	4	14	34

The short-rigid pile is assumed not to bend when laterally loaded but will rotate about a point approximately 1/3 to 1/4 its length above the pile tip. The soil reaction increases with depth below the firm bottom as shown in Figure 5.

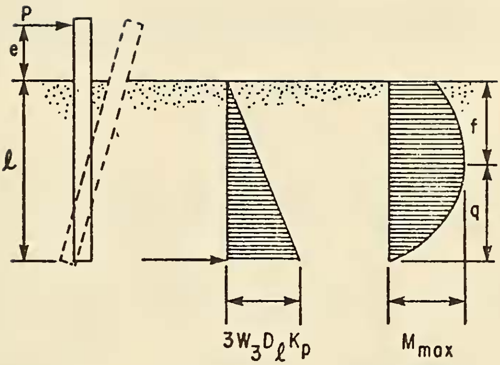

Figure 5. Failure modes for a short-rigid pile in cohesionless soil (after Broms, 1964b).

Because anchor piles are designed for the soil's ultimate lateral resistance rather than deflection of the pile head as in structural piles, the design is predicated on sufficiently large deflection to develop the full passive resistance. This is defined, based on comparison with test data, as three times the Rankine passive Earth pressure from the ground surface to the center of rotation (Broms, 1964b). The resulting equation for ultimate lateral resistance is:

$$P = \frac{w_s \; D \; \ell^3 K_p}{2(e + \ell)} \tag{6}$$

where

P = design lateral load ($P = F_t F_S$, eq. 4)

D = characteristic width of pile (the diameter for pipe pile)

w_s = unit weight of soil

e and ℓ = defined in Figure 5

K_p = Rankine's coefficient of passive Earth pressure;

$$K_p = \frac{1 + \sin \phi}{1 - \sin \phi} \tag{7}$$

ϕ = angle of internal friction of sand

Equation (6) may be solved by iteration as shown in the example.

When e, the lever arm of the load applied above the firm bottom, is zero the equation may be solved directly for required pile length as:

$$\ell = \left[\frac{2P}{w_s \ DK_p}\right]^{1/2} . \tag{8}$$

This method of analysis has been predicated on the maximum bending moment in the loaded pile not exceeding the piling sections maximum yield moment. This is considered a safe assumption for the typical short anchor pile problem.

When the foundation soils at the breakwater site are clay, the method in Broms (1964a) is used for determining the ultimate lateral resistance of a rigid-pile anchor under lateral load. As before, the pile is assumed to rotate without bending around a point in the lower half of the pile (see Fig. 6). The length of pile required is

$$\ell_t = e + 1.5D + f + q . \tag{9}$$

(See Fig. 6 for definition of terms.) The maximum moment occurs at (f + 1.5D) below the firm soil level where

$$f = \frac{P}{9 \ c_u D} . \tag{10}$$

The term c_u is the undrained cohesive strength of the clay. Care should be taken in the determination of the value of c_u to reflect the dynamic nature of the lateral load. The length, q, is the length of

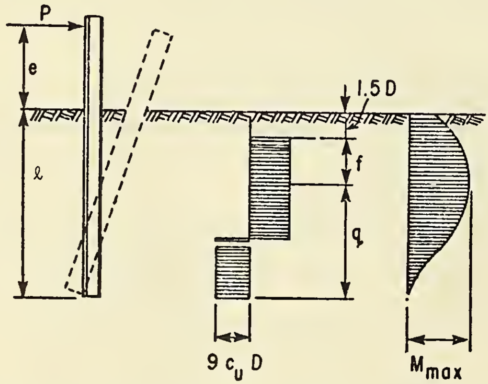

Figure 6. Failure mode for a short-rigid pile in cohesive soil (after Broms 1964a).

pile needed below the point of maximum moment to achieve the ultimate lateral resistance, and may be calculated as

$$q = \frac{P(e + 1.5D + 0.5f)^{1/2}}{2.25D} \qquad (11)$$

and the required pile length ℓ_t may be calculated directly from equation (9).

The choice of anchor type should be based on the design loading, the soil conditions, and the available method of placement. No one anchor type is universally suited for all conditions, and each type (or other types not mentioned) should be considered for a particular application and location.

* * * * * * * * * * * * * * V. EXAMPLE PROBLEM * * * * * * * * * * * * * *

GIVEN: The typical significant wave height and wave period observed at a site during summer storm conditions are 3.0 feet and 3.0 seconds, respectively, in 6.5 feet of freshwater.

FIND:

(1) The width of the Goodyear module FTB that reduces the 3.0-foot incident wave height to a 1.0-foot transmitted wave height;

(2) the expected mooring load (mooring lines are placed on a 1 on 7 slope);

(3) the required weight and volume of a mass concrete anchor; and

(4) the required length of an anchor pile.

SOLUTION:

(1) Find the width of a breakwater that reduces the incident wave height to a 1.0-foot transmitted height.

(a) Compute the allowable transmission coefficient, K_t, using equation (2):

$$K_t = \frac{H_t}{H_i} = \frac{1.0 \text{ ft}}{3.0 \text{ ft}} = 0.33 .$$

(b) Determine the incident wavelength, L, using equation (1) or Figure 2. From Figure 2 for T = 3.0 seconds and d = 6.5 feet,

$$L = 37 \text{ ft} .$$

(c) Determine the W/L ratio for the required K_t and compute the breakwater width. From Figure 3, where $K_t = 0.33$, $W/L = 1.38$. Thus, the required breakwater width, $W = (W/L)(L) = (1.38)(37 \text{ feet}) = 51$ feet.

(d) Determine the number of modules required which is equal to W (module width) or 51 feet per 7.0 feet per module = 7.3 modules required. Thus, the breakwater would have to be at least eight modules wide to obtain the desired wave height reduction.

(2) Determine the mooring load. Using Figure 4 and an incident wave height equal to 3.0 feet, the design load is 77 pounds per lineal foot of breakwater parallel to the wave crest between the anchor lines.

Assuming an anchor spacing of 50 feet, the total mooring-line load per anchor is:

$$F_t = 77 \text{ lb/ft} \times 50 \text{ ft} = 3,850 \text{ lb} .$$

NOTE.--The mooring-line load from Figure 4 is used as the lateral mooring-line load because they are essentially equal for the 1 on 7 slope specified.

(3) Design of a mass concrete anchor. Since the bottom is assumed to be level firm sand, the coefficient of static friction, μ, is assumed to be 0.4. Also, the assumed unit weight of concrete in air, w_c, is 150 pounds per cubic foot and the unit weight of freshwater, w_w, is 62.4 pounds per cubic foot. Substituting the equation (3) and solving for the total mass weight of the anchor, W_t,

$$W_t = \frac{F_t F_S}{\mu\left(1 - \dfrac{w_w}{w_c}\right)}$$

Thus, using $F_S = 1.5$ and $F_t F_S = 3,850 \times 1.5 = 5,875$ lb (or 6,000 lb)

$$W_t = \frac{6,000 \text{ lb}}{(0.4)(1 - 0.416)}$$

$$W_t = 25,685 \text{ lb } (12.8 \text{ tons}) \text{ for each 50-foot section} .$$

The volume is

$$\frac{25,685 \text{ lb}}{150 \text{ lb/ft}^3} = 171.2 \text{ ft}^3 \text{ (approximately 5-foot 7-inch cube)} .$$

(4) Design length of a pile anchor. For bottom firm sand (medium relative density), n_h = 14 tons per cubic foot (see the Table), solve for K_p using ϕ which must be known from soil sampling or be estimated (here assume $\phi = 30°$).

$$K_p = \frac{1 + \sin \phi}{1 - \sin \phi} = \frac{1 + 0.5}{1 - 0.5} = 3.0 \ .$$

Select a 16-inch (1.33 foot) steel pipe pile with a 0.1-foot wall thickness.

$$E \text{ for steel} = 2.16 \times 10^6 \text{ tons/ft}^2$$

I of cross section = 0.094 ft (from American Institute of Steel Construction, 1973) .

Assume solid bottom is 2.0 feet below anchor line attachment point:

$$e = 2 \text{ ft} \ .$$

Solving for the characteristic length:

$$\eta = \left[\frac{n_h}{EI}\right]^{1/5} = \left(\frac{14 \text{ tons/ft}^3}{(2.16 \times 10^6 \text{ ton/ft}^2)(9.4 \times 10^{-2} \text{ ft}^4)}\right)^{1/5}$$

$$\eta = 0.147 \frac{1}{\text{ft}} \ .$$

Because by definition $\eta \ell \leq 2.0$ for the rigid-pile case, try first values of $\ell < 13.6$ feet and use the rigid-pile analysis for cohesionless material to find actual ℓ using equation (6) and rewriting as

$$\frac{2P}{w_s \, DK_p} = \frac{\ell^3}{(e + \ell)} \ .$$

Substituting the known values and assuming w_s = 60 pounds per cubic foot for submerged unit weight of sand:

$$\frac{\ell^3}{(2 + \ell)} = \frac{2 \times 6{,}000 \text{ lb}}{\left(60 \dfrac{\text{lb}}{\text{ft}^3}\right)(1.33 \text{ ft})(3)} = 50.12 \ .$$

Then, by substituting for ℓ as follows:

Assume ℓ(ft)	Calculate $\ell^3/(2 + \ell)$	Compare to required value
7	38.111	< 50.12
8	51.20	> 50.12 by small amount
9	66.27	> 50.12

Note that ℓ = 8 feet is less than 13.6 feet (upper limit for rigid-pile analysis). Therefore, use ℓ = 8 feet and add e = 2 feet to obtain the total pile length ℓ_t = 10 feet. If scour is expected, the pile should be driven to the design depth below the maximum predicted scour elevation.

* *

VI. SUMMARY

Methods for determining the transmitted wave height and required anchor holding capacity for a floating tire breakwater using the proposed Goodyear FTB module design were presented. Application of these methods is intended to give conservative results.

The discussion on various types of anchors was included to provide general guidance on anchor types as well as references for further information on anchor design.

As a practical guide for the design of an FTB mooring system, an assumed harbor of refuge breakwater is considered as a design example.

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