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BUILDING RESEARCH ADVISORY BOARD

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METHODOLOGY
FOR
CALCULATING WAVE ACTION EFFECTS
ASSOCIATED WITH STORM SURGES

Prepared by the
Panel on Wave Action Effects
Associated with Storm Surges
of the
Science and Engineering Program on the Prevention
and Mitigation of Flood Losses
Building Research Advisory Board
Commission on Sociotechnical Systems
National Research Council

NATIONAL ACADEMY OF SCIENCES
Washington, D.C.
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NOTICE: The project that is the subject of this report was approved by the Governing Board of the National Research Council, whose members are drawn from the Councils of the National Academy of Sciences, the National Academy of Engineering, and the Institute of Medicine. The members of the Committee responsible for the report were chosen for their special competencies and with regard for appropriate balance.

This report has been reviewed by a group other than the authors according to procedures approved by a Report Review Committee consisting of members of the National Academy of Sciences, the National Academy of Engineering, and the Institute of Medicine.

This is a report of work under Contract Number H-3568 between the Department of Housing and Urban Development and the National Academy of Sciences.

Inquiries concerning this publication should be addressed to: The Executive Director, Building Research Advisory Board, National Research Council, 2101 Constitution Avenue, N.W., Washington, D.C. 20418.

PANEL ON
WAVE ACTION EFFECTS ASSOCIATED WITH STORM SURGES
OF THE
SCIENCE AND ENGINEERING PROGRAM
ON THE
PREVENTION AND MITIGATION OF FLOOD LOSSES

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FOREWORD

The Federal Insurance Administration (FIA), U.S. Department of Housing and Urban Development (HUD), is charged with promoting the public welfare by providing insurance protection against the risks of flood and mudslide losses and with stimulating the development of sound flood plain management practices. In its effort to formulate and implement the most effective programs possible for reducing the significant annual property losses resulting from floods and mudslides, HUD entered into a contract with the National Academy of Sciences (NAS) for advice and assistance. This advice and assistance is provided through the Academy's National Research Council (NRC), specifically through the NRC Science and Engineering Program on the Prevention and Mitigation of Flood Losses administered by the NRC Building Research Advisory Board (BRAB). To date advice and assistance has been provided to the FIA on a wide variety of topics associated with FIA technical planning, programs, and practices.

This report, the seventh in the series, has been prepared by the Panel on Wave Action Effects Associated with Storm Surges in response to one specific problem posed by the FIA--how best to estimate wave action effects (limiting wave height and runup) associated with storm surges. The Board gratefully acknowledges the work of the Panel and the contribution of its members.

J. NEILS THOMPSON, Chairman
Building Research Advisory Board

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I
INTRODUCTION

A. BACKGROUND

Established by the National Flood Insurance Act of 1968 (as amended), the Federal Insurance Administration (FIA), U.S. Department of Housing and Urban Development (HUD), is responsible for promoting the public welfare by ensuring the availability of insurance protection against the risks of flood and mudslide losses and by encouraging sound flood plain management by local communities as a condition for the insurance protection. In the context of these responsibilities, the FIA has considerable opportunity to formulate programs that will reduce the annual property losses resulting from floods and mudslides.

To aid it in making the maximum feasible technical and scientific contribution to disaster mitigation, the FIA requested that the National Academy of Sciences-National Academy of Engineering-National Research Council (NAS-NAE-NRC) provide it with continuous, objective review of and advice on its current technical planning, programs, and practices. In response to this request, the NAS entered into a contract with HUD and charged its NRC Building Research Advisory Board (BRAB) with administration of a Science and Engineering Program on the Prevention and Mitigation of Flood Losses.

B. PURPOSE AND SCOPE OF REPORT

This report responds to the FIA's request (Task 7, Contract No. H-3568) for immediate assistance in ascertaining whether and, if so, how calculations of wave height and runup should be incorporated in Flood Insurance Studies (FIS) of coastal communities subject to storm-induced flooding to provide an estimate of the areal extent and height (flood elevations) of overland flows having specified recurrence intervals (i.e., the probabilities of annual occurrence stipulated in the legislation and regulations pertaining to the National Flood

Insurance Program). Specifically, the report presents a method to be used in the immediate future for estimating the wave crest elevation (n-year flood elevation) associated with the n-year storm surge crossing the open coast on the shores of bays and estuaries on the Atlantic and Gulf coasts.[1]

This report does not address the problem of whether or how estimates of the extent of runup or amount of overtopping should be incorporated in a FIS since the time alloted by the FIA for the study did not permit these matters to be considered fully.[2] The report also does not address the problems of the effect of storm wave action on buildings and structures or on land features, which are outside the scope of the FIA's request. Both problems--and their implications for the National Flood Insurance Program--merit careful consideration by the FIA in the near future.

CONDUCT OF STUDY

This report is based primarily on the deliberations of the Panel on Wave Action Effects Associated with Storm Surges at a two-day meeting in Washington, D.C., on September 9 and 10, 1976. The point of departure for the deliberations was a number of reports and papers, made available to the Panel immediately prior to the meeting, that set forth (1) three techniques for identifying coastal high-hazard zones suggested to the FIA by the U.S. Army Corps of Engineers, Galveston District, in June 1975 (referred to hereafter as the CHHZ method); and (2) modifications to those techniques suggested to the FIA

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[1]The presented method also could be used for estimating the wave crest elevation associated with the storm surge crossing the open coast on the shores of bays and estuaries on the Great Lakes coast if the fetch factors given herein (see Table 1) were revised to reflect the 100-year still-water surge height and wind speed applicable to the Great Lakes region.

[2]A rather well defined technique for determining the extent of runup for design purposes does exist; it is described in U.S. Army Coastal Engineering Research Center, Shore Protection Manual, Vol. II (Washington: U.S. Government Printing Office, 1973), pp. 15-37. While this technique is considered too elaborate for the purposes of a FIS, it might serve as the point of departure in developing a technique for the FIA's purposes.

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by Tetra Tech, Inc., in August 1976.[3] The deliberations benefited from, and the Panel greatly appreciates, the presence of the following representatives of the FIA and Tetra Tech, Inc., who enlarged upon the background of the Panel's assignment and the reports and papers made available to the Panel:[4]

Robert D. Cassell, Flood Insurance Specialist, FIA, Atlanta, Georgia

F. Melvin Crompton, Director, Engineering and Hydrology Division, FIA, Washington, D.C.

Charles A. Lindsey, Assistant Director of Technical and Review Branch, FIA, Washington, D.C.

Earl Moss, Deputy Director, Engineering and Hydrology Division, FIA, Washington, D.C.

Frank Tsai, Hydraulic Engineer, FIA, Washington, D.C.

David Divoky, Associate Director, Engineering Division, Tetra Tech, Inc., Pasadena, California

Li-San Hwang, Vice President, Tetra Tech, Inc., Pasadena, California

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[3]The three techniques suggested by the Corps are described in a report entitled Guidelines for Identifying Coastal High Hazard Zones submitted by the Corps to the FIA in June 1975. The techniques are: (a) an analytical approach for identifying the coastal high-hazard zone (CHHZ) in sparsely developed coastal areas along the Atlantic and Gulf coasts that are subject to inundation by hurricane surge, (b) an abbreviated form of the analytical approach for identifying the CHHZ in the same locations for which the analytical approach is applicable, and (c) an empirical method for identifying the CHHZ in highly developed areas along the Atlantic and Gulf coasts that are subject to inundation by a hurricane surge.

The modifications suggested by Tetra Tech, Inc., are described in a technical note entitled Treatment of Wind Waves in Coastal Flood Insurance Studies submitted by the firm to the FIA in August 1976. The modifications to the analytical approaches (abbreviated and unabbreviated) essentially involve differences in: (a) selecting the wind field associated with height of storm waters (the surge caused by a hurricane plus height of astronomical tide) having a given probability of occurrence, (b) selecting the fetches to be studied, (c) accounting for variations in water depths along the fetches, (d) accounting for wave energy damping, and (e) selecting the shape of the wind wave to be used to determine maximum wave height. Tetra Tech, Inc., also recommends that one of the analytical approaches be used in highly developed areas instead of the empirical approach.

[4]Also attending the first day of the meeting as an observer was Robert M. Sorenson, U.S. Army Coastal Engineering Research Center, Ft. Belvoir, Virginia.

D. ORGANIZATION OF THE REPORT

The essence of the Panel's judgments concerning whether and how calculations of wave height and runup should be incorporated into FIS of coastal communities subject to storm-induced flooding is presented in the following section of this report together with a brief explanation of the Panel's thinking in arriving at these decisions. An appendix presents a method for assessing wave energy losses due to propagation through or over vegetation and a glossary of terms is included.

II
EXECUTIVE SUMMARY

Based on its deliberations, the Panel on Wave Action Effects Associated with Storm Surges has concluded that the FIA should include prediction of wave height in FIS of coastal communities subject to storm-induced flooding and should report the estimated wave crest elevation as the flood elevations of overland flows at recurrence intervals stipulated in the National Flood Insurance Program. The Panel also has concluded that the state of the art does not now permit wave heights associated with storm-induced overland flows to be predicted probabilistically and that even rigorous application of existing methods for deterministically predicting wave heights in transitional- and shallow-water areas is not appropriate in the conduct of FIS of coastal communities.

The Panel has recommended a method for use by the FIA in the immediate future for estimating the wave crest elevation (_n_-year elevation[1]) associated with the _n_-year storm surge crossing the open coast on the shores of bays and estuaries on the Atlantic and Gulf coasts. The proposed method includes means for taking account of varying fetch lengths, barriers to wave transmission, and the regeneration of waves likely to occur over flooded land areas. The method assumes a high correlation between _n_-year wave heights and _n_-year still-water level and that the estimate of the _n_-year still-water elevation (astronomical tide, surge, and setup) in a FIS: (1) is calculated independently in a rational, defensible manner and (2) does not already include contributions due to wave runup either as a result of the mathematics of the predictive model used or as a result of the data used to calibrate the predictive model for use in the particular location. The method also could be used on Great Lakes coasts

[1]As part of a FIS, it is necessary to estimate flood elevations having different probabilities of occurrence, i.e., 10-, 50-, 100-, and 500-year. The method proposed is applicable for any _n_-year probability.

if the fetch factors presented in Table 1 were revised to reflect the 100-year still-water surge height and wind speed applicable to the Great Lakes region. The method is not suitable for use on Pacific Ocean coasts because the n-year still-water level on these coasts is primarily a function of astronomical tide and tsunamis rather than storm occurrence and, thus, the n-year wave heights are only weakly correlated, if at all, with the n-year still-water level.[2]

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[2]Logic suggests that a suitable method for estimating n-year wave heights on the Pacific Coast (including Alaska and Hawaii) could be developed by an appropriate application of the joint probability method, but time did not permit the investigations of such a method as part of this study.

III

CONCLUSIONS, RECOMMENDATIONS, AND RATIONALE

A. PRINCIPLE

The Panel has concluded that the FIA should include prediction of wave height in FIS of coastal communities subject to storm-induced flooding and should report the estimated wave crest elevation as the flood elevations of overland flows at recurrence intervals stipulated in the National Flood Insurance Program.

At the present time, the FIA explicitly recognizes that wave action can occur in certain portions of a coastal community subject to 100-year storm-induced flooding, and it identifies these areas on the Flood Insurance Map of the community as Zone V, an area of special flood hazard due to the potential for inundation by tidal floods with velocity. This designation generally is applied to those areas where the still storm-water height (height of astronomical tide plus surge) is sufficient to support at least a 3-foot wave, assuming, of course, that there is sufficient fetch to generate such waves.[1] In these areas, the FIA establishes flood insurance premium rates that are 50 percent higher than those in Zone A, an area of special flood hazard due to the potential inundation by tidal floods without velocity. However, the FIA does not report the height of waves for Zone V but rather the still storm-water elevation just as it does for Zone A, and this reported elevation frequently becomes the elevation subsequently stipulated in community building and land-use regulations as the minimum elevation of the first habitable floor of new construction. Since there is a pronounced tendency for buildings and structures to be constructed to meet the minimum requirements of building and land-use

[1]One rationale for the choice of the 3-foot wave is set forth in Corps of Engineers (Galveston District), Guidelines for Identifying Coastal High Hazard Zones, "Appendix B: Criteria Relating to the Adoption of the 3-Foot Breaking Wave" (Galveston: Corps of Engineers, June 1975).

regulations, a significant number of people owning or occupying such buildings and structures unknowingly could be accepting a high degree of flood-related structure and personal hazard.

B. STATE OF THE ART

The Panel also has concluded that it is not now feasible to predict probabilistically wave heights associated with storm-induced overland flows. Additionally, the Panel has concluded that the rigorous application of existing methods for predicting deterministically wave heights in transitional- and shallow-water areas is not appropriate in the conduct of FIS of coastal communities subject to storm-induced flooding.

In having a FIS conducted, the FIA presently seeks to have the areal extent and height of inland flooding having a given probability of annual occurrence established on the basis of flooding that would be caused by individual hurricane-induced surges (with astronomical tide superimposed thereon) whose temporal and spatial (height and alongshore spread) characteristics and attendant wind fields are defined probabilistically.[2] The FIA achieves this by requiring that SPLASH[3] or comparable models and the method of joint probabilities be used in the conduct of a FIS to assign a probability of occurrence to the height of a surge produced by a hurricane and the total height of the resulting storm waters (i.e., surge plus astronomical tide).

However, because the models being used to not take into account the short-term water surface oscillations (3- to 20-second period waves) caused by the wind

[2]The rationale for this approach is discussed in Panel on Coastal Surges from Hurricanes, Methodology for Estimating the Characteristics of Coastal Surges from Hurricanes (Washington, D.C.: National Academy of Sciences, 1975), pp. 16-19.

[3]Chester P. Jelesnianski, "SPLASH (Special Program to List Amplitudes of Surges from Hurricanes), Part I--Landfall Storms," NOAA Technical Memorandum NWS TDL-46, 1972 and "SPLASH (Special Program to List Amplitudes of Surges from Hurricanes), Part II--General Track and Varient Storm Conditions," NOAA Memorandum NWS TDL-52, Mar. 1974. These works are a refinement of the following two publications: C.P. Jelesnianski, "Numerical Computations of Storm Surges Without Bottom Stress," Monthly Weather Review, XCIV (June 1966): 379-94, and "Numerical Computations of Storm Surges with Bottom Stress," Monthly Weather Review, XCIV, (Nov. 1967): 740-56.

acting directly on the water surface in transitional- or shallow-water areas, the surge and resulting storm water heights determined are essentially still-water elevations (i.e., tide heights above local sea level datum).[4] Presumably, an appropriate state-of-the-art spectral wave generation model (Tetra Tech, Inc., suggests one based on the work of Collins and Weir[5] but the work of others such as that of Resio and Vincent[6] might be equally valid) could be combined with the storm surge model so that wave heights as well as still-water heights could be computed for each storm modeled and then summed to obtain the needed frequency distribution. Nevertheless, the adequacy of such combinations of surge and wave generation models has yet to be demonstrated; indeed, it is not clear at this time which wave generation model concept, if any, should be developed and combined with surge models to yield reliable forecasts of wave heights associated with storm-induced overland flows. In the interest of fulfilling its long-term responsibilities, the FIA should evaluate and, if possible, sponsor research and development activities exploring these concepts.

Seemingly, an immediate solution would be to use the still-water heights determined using surge models and the joint probability approach in conjunction with the current technique for forecasting waves deterministically in

[4]There is some question, however, about the extent to which the forecasted still-water heights actually inadvertently include wave heights as a result of the data used to calibrate the models. It seems, for example, that the tide frequency curve for Cedar Key, Florida, produced by use of the SPLASH model (Francis P. Ho and Robert J. Tracey, Storm Tide Frequency Analysis for the Gulf Coast of Florida from Cape San Blas to St. Petersburg Beach, NOAA Technical Memorandum NWS HYDRO-20, April 1975, p. 34) overstates by 4 to 5 feet the tide gauge readings for Hurricanes Alma (1966) and Agnes (1972) while matching very closely observed high-water marks that could have been made by propagating waves.

[5]J. I. Collins and W. Weir, Prediction of Shallow-Water Spectra, Tetra Tech, Inc., Report No. TC-164 for Naval Ship Research and Development Laboratory, Contract No. N61339-69-C-0237 (Pasadena, Calif.: Tetra Tech, Inc., 1971). This material is condensed in J. Ian Collins, "Prediction of Shallow-Water Spectra," Journal of Geophysical Research, 77, (May 20, 1972): 2694-2706.

[6]See Resio and Vincent, Waterways Experiment Station Technical Report H-76-1: Design Wave Information for the Great Lakes, Report 1: Lake Erie (January 1976) and Report 2: Lake Ontario (March 1976); also Resio and Vincent, Waterways Experiment Station Miscellaneous Paper H-76-12: Estimation of Winds Over the Great Lakes (June 1976).

transitional- and shallow-water areas (i.e., the charts and graphs contained in the Shore Protection Manual[7]), and this is the thrust of the analytical approaches suggested by the Corps of Engineers and Tetra Tech, Inc.[8] These approaches, however, are beset with two inherent problems that belie the results of sophisticated calculations obtained by the rigorous application of the charts and graphs in the Shore Protection Manual in conjunction with the still-water heights obtained from surge models and the joint probability approach.

First, the height of waves that theoretically can be produced in transitional or shallow water of a given depth depends significantly on assumptions made about the wind field operating and the fetch available. The CHHZ method proposes that a unique landfalling hurricane bearing no particular relationship to the cause of the storm water height be chosen in a standardized way. The Tetra Tech method proposes that: (1) a unique relationship between peak surge and maximum onshore wind speed be assumed, (2) surge models be used to derive frequency distribution curves for peak surge levels versus peak wind speed while the other storm characteristics (central pressure depression, radius to maximum wind, forward speed, and path) are held constant, and (3) the wind speed yielding the given surge height be selected for use in forecasting the waves. Neither approach is particularly defensible because the depth of storm waters (surge plus astronomical tide) having a given probability of annual occurrence is not relatable to a unique wind field or fetch--i.e., the depth of storm waters having a given probability of annual occurrence is not attributable to a particular storm but rather is the depth whose probability of occurrence reflects the outcome of the possibilities of strong and weak, nearby and distant, alongshore and landfalling storms in the vicinity of the community.

Second, the height of waves that theoretically can be produced in transitional or shallow waters for a given wind field and fetch is significantly dependent

[7]U.S. Army Coastal Engineering Research Center, Shore Protection Manual Vol. I, (Washington: U.S. Government Printing Office, 1973), pp. 33-69.

[8]See Section I, footnote 4.

on assumptions made about the depth of water available and the degree of dampening caused by the roughness of the bottom and the presence of grass, trees, and other impediments to flow in the water. Both the CHHZ and the Tetra Tech methods propose that, in consonance with general FIA guidelines for the conduct of a FIS, needed topographic and bathymetric data be derived largely from existing map sources (e.g., U.S. Geological Survey quadrangle maps at a scale of 1:24000) and that major field surveys not be conducted. The two methods differ in their treatment of dampening: The CHHZ method adopts the Shore Protection Manual charts and graphs that are based on a constant bottom friction factor and proposes to account for the effects of marsh grasses and other ground cover by reducing the depth of water available by the average height of the ground cover. The Tetra Tech method proposes using the basic wave forecasting equations on which the Shore Protection Manual charts and graphs are based and variable friction factors and dampening coefficients to suit the local situation. Both approaches seemingly overlook the effect of the considerable uncertainty involved in the basic data being used (i.e., topographic, bathymetric, and still-water height of surge and storm waters) on the resulting estimate of wave height, no matter how rigorously computed.

C. ESTIMATING WAVE CREST ELEVATIONS

To determine and report the _n_-year flood elevation in a community on the coasts of the Gulf of Mexico and Atlantic Ocean, the Panel recommends that the FIA define the _n_-year flood elevation at a site as the elevation at the crest of waves that can exist superimposed on the _n_-year still-water storm tide level at the site and compute the _n_-year flood elevation at the site, Z_w, according to the equation:

$$Z_w = S_* + 0.7 H_*, \tag{1}$$

where S_* is the still-water storm tide elevation at the site above the local sea level datum for the _n_-year flood conditions (as determined by the use of SPLASH or comparable models and the method of joint probabilities) and H_* is

the wave height at the site.[9] (See the Glossary for a definition of all terms used.)

The evaluation of H_* should be carried out by a succession of steps starting with the calculation of the height of the initial wave height, H_1, as it crosses the position of the normal mean sea level shore line (Eq. 3), followed by the calculation of the wave height transmitted past each type of obstruction (Eqs. 5 through 12) including any augmentation of wave energy due to winds acting on significant reaches of flooded land that lie seaward of the site in question (Eq. 13).[10] The upper limit for H_* is the breaker height:

$$H_{*b} = 0.78\ d_*, \qquad (2)$$

where d_* is the still-water depth at the site or $S_* - Z_{g*}$ with Z_{g*} being the ground elevation at the site. The waves transmitted to the site generally may be lower than this limiting value, particularly if the site is partially protected from the open sea by either natural or man-made obstructions. Three types of obstruction (these together with reach of flooded area for which wave generation may be significant are depicted on a hypothetical profile normal to a shoreline in Figure 1) should be considered:

1. Elongated natural or man-made barriers such as dunes, bars, and breakwaters that occur seaward or bayward of the site in question;
2. Vegetated regions such as dense mangrove marsh or dense wooded areas that lie seaward or bayward of the site in question; and
3. Buildings that extend to ground level (excluding those on pilings for which the lower floor level is above the potential wave crest elevation) and could obstruct the transmission of wave energy to the site in question.

In cases where the seaward fetch is essentially unlimited, the wave height, H_1, at the normal mean sea level shore line position accompanying the _n_-year storm tide elevation should be taken as the breaking wave height, $0.78\ S_1$,

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[9]The rationale for Eq. 1 through 8 begins on page 14 of this report.

[10]These obstructions might occur in any combination or order. If the H_* determined by these series of calculations is greater than H_{*b} (Eq. 2), then H_{*b} should be taken as the _n_-year flood elevation at the site (i.e., H_* in Eq. 1 should be taken as H_{*b}).

at that position. In cases where the fetch is limited (e.q., for bays or estuaries), the height should be taken as:

$$H_1 = 0.78\ F\ S_1, \tag{3}$$

where F is a fetch factor given in Table 1 and S_1 is the still-water storm tide elevation at the normal mean sea level shore line as shown on Figure 1.

TABLE 1 Fetch Factor F as a Function of Fetch[a]

Fetch (Statute Miles)	F (Fetch Factor)
1/8	0.25
1/4	0.32
1/2	0.41
1	0.52
2	0.65
4	0.78
10	0.93
>20	1.00

[a]For convenience, a plot of F versus fetch is given in Figure 2. F for 1- and 2-mile fetches in Table 1 and Figure 2 are smoothed values derived from data presented in Table 3.

For transmission past a given obstruction, the transmitted wave height, H_t, should be taken as:

$$H_t = B\ H_i \tag{4}$$

where H_i is the incident wave height and B is the transmission coefficient evaluated as described below.

For elongated natural barriers such as dunes:

$$B = 1 \text{ if } H_i < 0.78\ d_b, \tag{5}$$

$$B = \frac{0.78\ d_b}{H_i} \text{ if } H_i > 0.78\ d_b, \text{ or} \tag{6}$$

$$B = 0 \text{ if } Z_b > S_b, \tag{7}$$

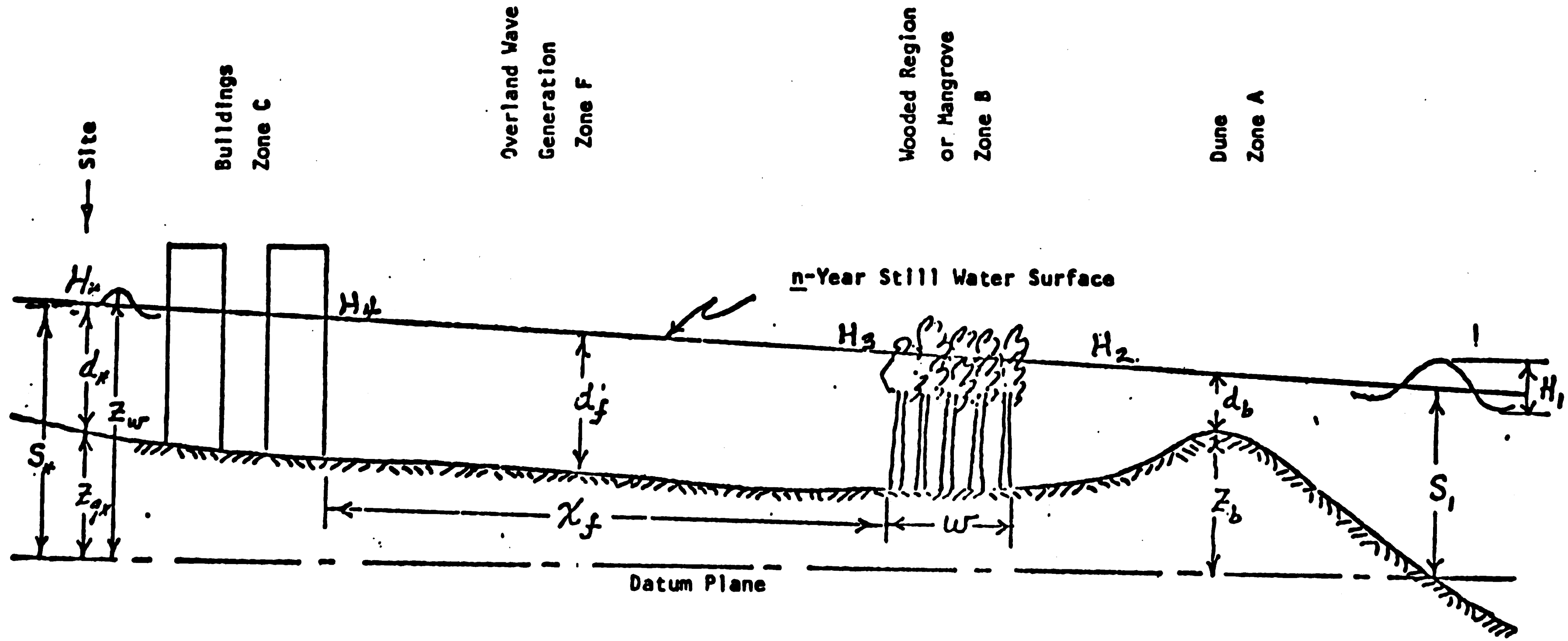

FIGURE 1 Hypothetical Profile Normal to a Shoreline

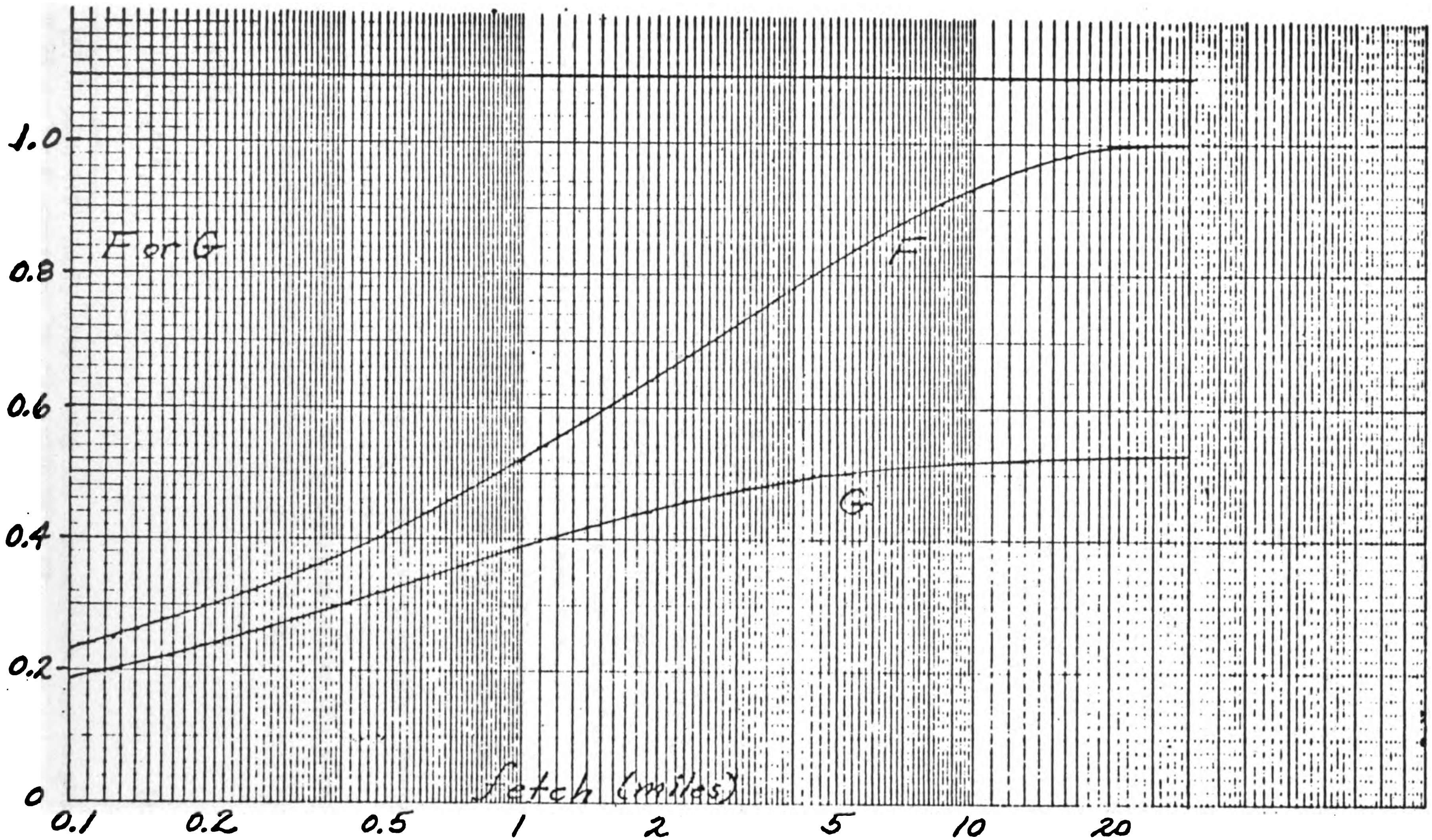

FIGURE 2 Plot of Fetch Factor (F) and Inland Fetch Factor (G) Versus Fetch

where Z_b is the alongshore average elevation of the barrier, S_b is the value of S at the barrier, and d_b is the average still-water depth, $(S_b - Z_b)$.[11]

For elongated man-made barriers such as dikes and seawalls:

$$B = 1 \text{ if } H_i < 0.78\, d_b, \tag{8}$$

$$B = \frac{1}{H_i}\,(0.78\, d_b + 0.5\, H_i) \text{ if } H_i > 0.78\, d_b, \text{ or} \tag{9}$$

$$B = 0 \text{ if } Z_b > S_b + 0.5\, H_i, \tag{10}$$

where Z_b, S_b, and d_b are as above.

For vegetated regions:

$$B = \left[1 + \frac{1}{3\pi}\, C_D\, H_i\, h\, D\, w/(b^2 d^2)\right]^{-1}, \tag{11}$$

where C_D is the drag coefficient for the obstructing elements (of order unity),
- d is the mean depth of water for the vegetated region,
- h is the mean wetted height of obstructing elements (thus, actual mean height if fully submerged or d if not submerged),
- D is the mean effective diameter of obstructing elements (diameter of an equivalent circular cylinder having the same projected area in the direction of wave propagation),
- b is the mean horizontal spacing of obstructing elements measured between centers, and
- w is the width of the vegetated zone, measured along the direction of wave propagation (normal to shore).

For buildings:

$$B = r^{n/2}, \tag{12}$$

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[11] Eqs. (5) through (10) imply simply that the transmitted wave height is either H_i, $0.78\, d_b$, or 0 according to the value of d_b/H_i.

where r is the average ratio of open distance between buildings to total distance measured parallel to shore and n is the number of rows of buildings seaward of the site.

To account for inland wave generation that might take place in the wind fetch zone, f, depicted in Figure 1, it is recommended that the augmentation of wave height be computed by a procedure in which the depth of flooding and fetch length govern the added wave generation (the depth of flooding being correlated to wind conditions). For this case, the wave height at the end of the inland fetch, H_f, should be computed by:

$$H_f = \left[(G\ d_f)^2 + H_i^2 \right]^{1/2}, \tag{13}$$

where H_i is the initial wave height entering the fetch zone, d_f is the mean depth over the fetch zone, and G is a function of fetch distance x_f given in Table 2.

TABLE 2 Fetch Factor G as a Function of Fetch x_f[a]

x_f (Statute)	G
1/8	0.20
1/4	0.26
1/2	0.32
1	0.39
2	0.45
4	0.49
10	0.52
>20	0.53

[a]For convenience, a plot of G versus fetch is given in Figure 2.

1. <u>Rationale</u>

Assuming that the <u>n</u>-year still-water storm tide elevation has been determined by some appropriate means and that the height and areal extent of resulting overland flooding has been postulated, the objective of the method recommended by the Panel is to determine the reasonable added height of water that may occur in and around structures due to waves generated by the action of the wind on the

surface of the flood waters. The waves being studied fall basically into two classes: (a) those generated over the open waters of the Atlantic Ocean or the Gulf of Mexico and reaching the shore coincident with the storm surge, and (b) those generated over the interior coastal waters by the storm winds accompanying the storm surge. While the mechanics of generation of the two classes of wave are the same, class a waves will almost always be much higher waves and of longer duration than class b waves because of the greater fetches and depths available in Atlantic and Gulf areas.

The basic premise of the recommended procedure is that both the _n_-year still-water tide elevation and the waves are primarily _related to a common origin_--i.e., storm wind conditions. Moreover, it is desirable to derive wave heights that reflect the same _n_-year recurrence as the storm tide, and a simple way of doing this is to relate the wave conditions primarily to the _n_-year storm tide elevation rather than to any one particular storm tide elevation. This premise is considered valid only for the Atlantic Coast, the Gulf Coast, and the Great Lakes; it is _not_ recommended for the West Coast of the United States or for the coasts of Hawaii or Alaska where flood levels and waves are not necessarily directly related.

It must be emphasized that the recommended procedure, properly reflecting the physical principles involved, is highly simplified. In addition to assumptions presented below it should be noted that the effect of shoaling and refraction on wave height has been ignored in the recommended interim procedure. It is felt that these refinements are not warranted within the context of a FIS.

a. _Eq. (1)_.

The portion of the wave height above still-water level for a wave of period T, height H, in a depth d depends in general upon d/T^2 and H/T^2 and, possibly, the slope of the sea bed. For very small bottom slope, the dependence of the relative crest elevation η_c/H on d/T^2 and H/T^2, where η_c is the crest elevation above still-water level, is given in Figure 7-41 of the _Shore Protection Manual_. For short period waves (d/T^2 >3 ft/sec^2), η_c/H varies from 0.5 to 0.68, the upper limit being for breaking waves. For very long period waves, η_c/H varies

between 0.5 and 1.0, the upper limit being for extremely long period breaking waves. Actual wind-induced waves represent a composite spectrum of waves of different periods and associated amplitudes. In the interim procedure, the wave period is not considered explicitly. As a compromise value for η_c/H, considering that many periods and relative wave heights are represented, the average η_c/H for the four extremes for short and long period waves discussed above is taken; this yields 0.67, which is rounded to 0.7 and somewhat favors the higher waves. This is the basis for the term 0.7 H_* in Eq. (1).

b. Eq. (2)

As an individual wave in a wave train moves ashore (i.e., into progressively shallower water), it finally reaches a depth that is too shallow to maintain it and the wave breaks, thereby dissipating most of its energy and losing most of its height. This height, the breaking height of waves, is the maximum height of wave (from crest to trough) that can exist in water of a particular still-water depth. The value of H/d for breaking generally depends upon the relative depth, d/T^2, as well as the bottom slope as given, for example, in Figure 2-66 of the Shore Protection Manual. For the purpose of the interim procedure, the chosen d/H for breaking is 1.28, which corresponds to H/d for breaking of 0.78. This value is adopted in Eq. (2) and elsewhere for the breaking condition. It happens to correspond to the breaker height condition for a solitary wave.

c. Eq. (3) and Table 1

Although Eq. (2) holds for unobstructed open coast regions (i.e., those exposed to essentially unlimited fetches over great depths of water), some modifications are necessary where available fetches and depths of water would generate waves lower than the breaking height. This modification can be achieved by introducing a fetch coefficient. The coefficients presented in Table 1 are based upon use of Figures 3-21 to 3-30 of the Shore Protection Manual in an evaluation of the ratio of the maximum wave height to the breaking wave height assuming: (1) a still-water storm tide height of 14 feet (assumed to be a typical 100-year still-water storm tide), (2) a typical mean no-storm depth of bay of 12 feet, and (3) a wind speed of 80 mph. In this evaluation, it is also necessary to decide which wave in

the estimated wave train is to be used in setting the flood level increment above the still-water surge level.[12] For the purposes at hand, it is considered that a wave approaching--but lower than--the average height of the 1 percent highest waves is a proper "controlling wave." Thus, the controlling wave height, H_c, is assumed as:

$$H_c = 1.6\ H_s, \tag{14}$$

where H_s is the significant wave height.

With this relationship selected, the first three columns of Table 3 were constructed using the shallow-water wave generation curve from the Shore Protection Manual. Since the controlling wave will break when it reaches a height equal to about 0.8 of the depth of water, the fourth column in the table was prepared based on the relation:

$$\text{Breaking depth} = d_c = H_c/0.8. \tag{15}$$

Assuming that the most severe conditions of generation in interior waters are a 26-foot depth of water (12-foot chart depth plus 14-foot still-water surge height) and an 80 mph wind, the maximum controlling wave height is considered to be 11.7 feet for fetches of 20 miles or more. For shorter fetches, the controlling wave heights would be limited by the fetch to the heights shown in the third column of Table 3. Thus, the shorter fetches in the first column would reduce the heights of the maximum controlling wave (11.7 feet) by the factor

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[12]The spectrum of waves in a wave train represent a wide range of wave heights. Studies have identified certain interrelationships of the waves in a wave train. Most of the wave generation theory is based on estimating a wave known as the "significant wave," which is a wave whose height is equal to the average height of the one-third highest waves in the spectrum. The relation of the height of other waves in the spectrum to the height of this significant wave has been found to be as follows:

Mean wave height = H_{50} = 0.62 H_s,

Significant wave height = $H_{1/3}$ = H_{33} = H_s,

Average height of 10 percent highest waves = H_{10} = 1.27 H_s,

Average height of 1 percent highest waves = H_1 = 1.67 H_s.

shown in the last column of Table 3. This last column, presents the fetch coefficients that were plotted in Figure 2 and a smooth curve drawn; the values appearing in Table 1 were then taken from Figure 2.

TABLE 3 Derivation of Numerical Value of Fetch Coefficient

Fetch (Statute) Miles	Significant Wave Height, H_s (ft)	Controlling Wave Height, $H_c = 1.6\ H_s$ (ft)	Breaking Depth of Controlling Wave, $d_c = H_c/0.8$ (ft)	Fetch Coefficient $F = d_c/14.6$[a]
1/8	1.8	2.9	3.6	0.25
1/4	2.3	3.7	4.6	0.32
1/2	3.0	4.8	6.0	0.41
1	3.7	5.9	7.4	0.51
2	4.9	7.8	9.8	0.67
4	5.7	9.1	11.4	0.78
10	6.8	10.9	13.6	0.93
>20	7.3	11.7	14.6	1.00

[a] The fetch coefficient serves to reduce the maximum breaking depth of controlling wave, d_c, of 14.6 feet for fetches of 20 miles or more to a proper value for fetches of less than 20 miles.

d. Eqs. (5) Through (10)

Elongated natural barriers cause significant energy dissipation by triggering the breaking of waves whose heights exceed 78 percent of the depth of water over the top of such barriers, assuming that the storm tide elevation does exceed the barrier elevation. If the barrier elevation exceeds the storm mean water level, S, then it is assumed that essentially no wave energy is transmitted shoreward of the barrier. While wave overtopping can exist, this contributes water shoreward of the barrier but little wave energy. It is assumed that S is the same on either side of the barrier, provided the barrier is not a dike enclosing the site in question. However, the effect on incident waves of elongated man-made barriers is not as great as is the effect of elongated natural barriers. Laboratory wave tests have shown that for thin barriers, such as seawalls and dikes, the transmitted wave height can be on the order of 60 percent of the incident wave height even with barriers extending almost to the water surface. Therefore, for man-made barriers, it was decided to recognize that the transmitted wave height could easily be 50 percent of the incident wave height and Eqs. (6) and (7) were thus adjusted.

e. Eq. (11)

The basis for Eq. (11) is that: (1) the vegetation present will not be changed prior to the storm and the essential hydrodynamic drag characteristics will remain constant during the storm, (2) the vegetation matrix can be represented by an equivalent "stand" of equally spaced circular cylinders, (3) the cylinders are not so dense that they interact, (4) the application of shallow water wave theory is justified to approximate the horizontal water particle velocity as simple harmonic, and (5) the energy loss due to a single circular cylinder acted upon by an oscillatory flow field is due to drag forces only and equivalent to the case of a cylinder oscillating in otherwise still water.[13] Considerable care needs to be given to selecting the vegetation characteristics and to ensuring that the probability is minimal that the vegetation will be intentionally removed (or the damping effect reduced) in the course of time or that the vegetation effects would be markedly reduced during a storm through erosion, uprooting, or breakage.

f. Eq. (12)

Eq. (12) is based on simplifying assumptions: (1) that the fraction of the total wave energy transmitted inland past a given row of buildings is r times the incident energy; (2) that the transmitted energy is immediately redistributed laterally upon passing each row of buildings, and (3) that the wave height is directly proportional to the square root of the wave energy. Secondary forward scattering of energy due to re-reflection from the back sides of buildings, which would tend to increase the net transmission, is ignored in this simplified approach; however, energy dissipation also is ignored and this would tend to offset the effect of secondary forward scattering.

g. Eq. (13)

The quantity ($G\ d_f$) in Eq. (13) is the wave height that would exist at the end of the inland fetch in the absence of any initial wave height. Since the waves generated in the new fetch generally will have a different spectrum and mean period from that of the incident waves, the most rational way of combining these is on the basis of the sum of the energy of each, the energy being taken proportional to the square of the wave height.

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[13]The details of the derivation of Eq. (11) are contained in the Appendix to this report.

The factor G was determined in a manner somewhat similar to that by which factor F (Eq. (3), Table 1, and Figure 2) was determined. A wind speed of 60 mph, which is 75 percent of that used in deriving F, is employed over the inland fetch assuming a flood depth of 10 feet. The values of significant wave height H_s were determined from the 1975 corrected version of Figure 3-22 of the Shore Protection Manual for each fetch. The values of H_s were multiplied by 1.6 to obtain the controlling wave height and G was determined by dividing the foregoing product by the 10-foot depth. The resulting G values are given in Table 2. While these have been determined for a specific depth and wind speed, it is recommended that these be used for general flood depths. For example, if one applies Table 2 to a situation in which d = 5 feet and $x_f \geq 20$ miles, the resulting H_f is 2.6 feet if $H_i = 0$. This corresponds to a control wave height obtained for a wind speed of 42 mph for d = 5 feet (Figure 3-2 of the Shore Protection Manual, 1975 revision). On the other hand, with the same fetch and d = 20 feet, H_f = 10.6 feet, which corresponds to a wind speed of 85 mph and a depth of 20 feet. Thus the recommended procedure, which relates H_f to the depth for a given fetch, implies a direct correlation between wind and flood depth, as indeed should be the case.

2. Example Calculations

a. Vegetated Regions

Two example calculations using Eqs. (4) and (11) for mangrove and one example for pine forest are given in Table 4. In each of these examples the drag coefficient is taken as unity; this is recommended in actual application.

b. Complete Example

As an example for evaluation of H_*, assume the situation depicted in Figure 1 where:

Seaward fetch = 2 miles

S_1 = 16 feet

d_b = 12 feet

Zone b:

w = 500 feet

D = 0.2 feet

b = 1.0 feet

h = 12 feet

d = 12 feet

Zone f:

d_f = 10 feet

x_f = 4 miles

Zone c:

r = 0.5

n = 3

Site:

d_* = 7 feet

S_* = 17 feet

TABLE 4 Examples of Wave Height Reduction Due to Vegetation

		Vegetation Characteristics (ft)				Wave Characteristics (ft)		
Case	Vegetation Type	D	b	h	w	d	H_i	H_t
1.	Mangrove[a] (over full depth)	0.2	1	10	100	10	7	2.82
2.	Mangrove[a] (over partial depth)	0.2	1	6	100	12	7	4.32
3.	Pine forest	1	10	12	1000	12	9	5.01

NOTE: C_D = 1 in all examples.

[a]Characteristics of mangrove selected attempt to include effects of branches.

Solution:

From Table 1 or Figure 2 and Eq. (3):

$H_1 = 0.78 \times 0.65 \times 16 = 8.1$ feet.

From Eqs. (4) and (5), $0.78\ d_b = 9.4$ feet; therefore,

$H_2 = 8.1$ feet.

From Eqs. (4) and (11) using $C_D = 1$:

$$H_3 = \left[\frac{8.1}{1 + \frac{1}{3\pi}\ 8.1 \times 12 \times 0.2 \times 500/(1 \times 12)^2}\right] = 1.0 \text{ feet.}$$

From Eq. (8) and Table 11 or Figure 2:

$$H_4 = H_f = \left[(0.49 \times 10)^2 + (1)^2 \right]^{1/2} = 5.0 \text{ feet,}$$

which is less than the breaking height $0.78\, d_f = 7.8$ feet and therefore allowable. Finally, from Eqs. (4) and (13):

$$H_* = (0.5)^{3/2} \times 5.0 = 1.8 \text{ feet,}$$

which is less than $H_{b*} = 5.5$ feet.

Therefore, by Eq. (1):

$$Z_w = 17.0 + 0.7 \times 1.8 = 18.3 \text{ feet.}$$

GLOSSARY

b = mean horizontal spacing of obstructing elements measured between centers

B = transmission coefficient

C_D = drag coefficient for the obstructing elements (of order unity)

d = mean depth of water for the vegetated region

d_* = still-water depth at site

d_b = still-water depth over elongated barrier

d_f = still-water depth over inland fetch area

D = mean effective diameter of obstructing elements (diameter of an equivalent circular cylinder having the same projected area in the direction of wave propagation)

F = fetch factor

G = inland fetch factor

h = mean wetted height of obstructing elements (thus, actual mean height if fully submerged or d if not submerged)

H_f = wave height at end of inland fetch

H_i = wave height in front of elongated barrier, vegetated area, buildings, or inland fetch area

H_t = wave height behind elongated barrier

H_1 = wave height at the normal mean sea level shore line

H_* = wave height at site

H_{*b} = breaker wave height at site

n = number of rows of buildings seaward of site

r = average ratio of open distance between buildings to total distance parallel to shore

S_b = still-water storm tide elevation at elongated barrier

S_1 = still-water storm tide elevation at the normal mean sea level shore line

S_* = n-year still-water storm tide elevation at site

w = width of the vegetated zone, measured along the direction of wave propagation (normal to shore)

x_f = length of inland fetch
Z_b = average elevation of elongated barrier
Z_{g*} = ground elevation at site
Z_w = n-year flood elevation at site

APPENDIX
WAVE ENERGY LOSSES DUE TO PROPAGATION THROUGH OR OVER VEGETATION

A. INTRODUCTION

To investigate the energy losses resulting as a wave propagates through or over vegetation, the equivalent problem of energy losses due to drag forces on an element oscillating in still water are derived. Considering shallow water waves, these results are applied to the case of a vegetative stand that is approximated by a series of equally spaced vertical circular cylinders.

B. METHODOLOGY

1. Energy Losses Due to Drag Forces on an Element Oscillating in Still Water

Consider a vertical circular cylinder of diameter D and height h oscillating horizontally in still water. The instantaneous rate of energy loss $\dot{e}$ is:

$$\dot{e} = F_D(t)\ U(t), \tag{1}$$

in which F_D is the drag force and U is the speed of the cylinder. The drag force is given by:

$$F_D = \frac{C_D \rho D}{2} U(t)|U(t)|h, \tag{2}$$

where ρ is the mass density of water, C_D is the drag coefficient, and the velocity is presumed to be simple harmonic with amplitude U_m, $U = U_m \cos \sigma t$, and $\sigma (= 2\pi/T)$ is the angular frequency and T the period of oscillation. It is noted that the cylinder would also experience an instantaneous inertia force component; however, this would be out of phase with the velocity, U, and therefore would not contribute to the net energy loss. The time-averaged energy loss $\bar{\dot{e}}$ is:

$$\bar{\dot{e}} = C_D \frac{\rho D}{2} h\ \overline{U^2(t)|U(t)|}, \tag{3}$$