USERS MANUAL FOR WAVE HEIGHT ANALYSIS

to accompany

Methodology for Calculating Wave
Action Effects Associated With Storm Surges
(National Academy of Sciences 1977)

REVISED FEBRUARY 1981

FEDERAL EMERGENCY MANAGEMENT AGENCY
FEDERAL INSURANCE ADMINISTRATION

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(National Academy of Sciences 1977)

Revised February 1981

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FOR

FEDERAL EMERGENCY MANAGEMENT AGENCY
FEDERAL INSURANCE ADMINISTRATION

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1. INTRODUCTION

The procedures outlined in this Users Manual apply to the determination of wave heights and elevations associated with storm surge along the coasts of the Atlantic Ocean and the Gulf of Mexico. These procedures are to be used to apply the "Methodology for Calculating Wave Action Effects Associated with Storm Surges," prepared by the National Academy of Sciences (NAS)[1], in Flood Insurance Studies. The NAS report is issued separately by the Federal Insurance Administration and is to be used in conjunction with the procedures described herein. The methodology considers the effects of depth, all types of stationary obstructions, and wind fetches on wave heights and elevations. The wave crest elevations are to appear as base flood elevations on the Flood Insurance Rate Maps. The Federal Insurance Administration has a computer program available based on the NAS methodology and this manual for calculating wave heights and elevations, and flood insurance data.

2. ASSUMPTIONS OF THE METHODOLOGY

The NAS method is based on three major concepts. First, the storm surge has a breaking wave component equal to:

$$H_b = 0.78d \text{ (NAS Equation 2)} \qquad (1)$$

where H_b is the height of the breaking wave, d is the stillwater depth, and the coefficient 0.78 corresponds to the breaker height condition for a solitary wave. The elevation of the crest of a breaking wave is:

$$Z_w = S_* + 0.7H_b = S_* + 0.55d \qquad (2)$$

where Z_w is the water-surface elevation and S_* is the stillwater elevation.

The second major concept is that the breaking wave height may be diminished by dissipation of energy:

$$H_t = BH_i \text{ (NAS Equation 4)} \qquad (3)$$

where H_t is the transmitted wave height, H_i is the incident wave height, and B is an energy transmission coefficient, ranging from 0.0 to 1.0. The coefficient, B, is a function of the physical characteristics of the impediment to waves. Separate equations are used to determine B for sand dunes, dikes and seawalls, buildings, and vegetation.

The third major concept is that, in unimpeded reaches in a wind fetch zone, wave generation can result from wind energy being transferred to the water. This added energy is related to effective fetch distance and to mean depth over the fetch zone:

$$H_f = G_* d_f \qquad (4)$$

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[1] Panel on Wave Action Effects, Methodology for Calculating Wave Action Effects Associated with Storm Surges (Washington, D.C.: National Academy of Sciences, 1977).

where H_f is the regenerated wave height, G_* is an inland fetch factor which is a function of effective fetch length, and d_f is the mean depth over the fetch zone.

3. DATA PREPARATION

When an area in a community is determined to be subject to coastal wave action, a thorough reconnaissance of the area should be made. This reconnaissance should include a thorough literature search, information search, and field reconnaissance in accordance with the study procedures defined in Chapter 2, Section 2-3 of the Flood Insurance Guidelines and Specifications. Data sources collected should include the best available topographic and planimetric maps, aerial photographs, and other appropriate materials, such as beach profiles, coastal inundation maps, and historical high-water mark descriptions.

Data necessary to conduct the analysis are described below:

1. The 100-year stillwater storm tide elevations should be determined. Where appropriate, the contractor shall use available stillwater elevation information.

2. Best available topographic information should be obtained which accurately shows the following: location of the mean sea level shoreline throughout the study area; locations, elevations, leeward and seaward slopes, and lateral extents of sand dunes and other natural barriers; beach and inland ground elevations; and elevations and lateral extent of roads, bridges, seawalls, dikes, and other similar engineered features. Topographic maps with a contour interval of 5 feet or less are considered satisfactory for the wave height analysis. However, the actual crest elevations of dikes and seawalls must be determined.

3. Best available aerial photography should be obtained and/or field investigations should be conducted to determine physical parameters for vegetated zones and developed areas. Parameters include location, average effective diameter of vegetation (equivalent to a circular cylinder), average height of vegetation, width (in the direction of wave propagation), and density (average distance from center to center of trees) of the vegetated area. Location, width, and distance between buildings should also be obtained from aerial photography.

4. COMPUTATIONAL INSTRUCTIONS

a. Determination of Transect Locations

A transect is a line taken perpendicular to the average direction of the mean sea level shoreline. Wave heights and elevations are to be calculated for each obstruction, wind fetch, and area of significant change in ground elevation along the transects. Transect locations should be chosen with consideration given to the physical and cultural characteristics of the land so that they will closely represent conditions in their locality. Transects should be placed closer together in areas of complex topography, dense development, unique flooding, and where computed wave heights vary significantly between adjacent transects. Wider spacing may be appropriate in areas having more uniform characteristics. The contractor should exercise good judgment in placing the transects to avoid excessive interpolation of elevations between transects, while also avoiding unnecessary study time.

The transect line drawn on the work map should approximately represent the centerline of its reach. Along each transect, wave heights and elevations are to be computed considering the combined effects of changes in ground elevation, vegetation, and physical features. Wave heights and elevations are calculated at the end point of each fetch and obstruction along the transect in the order in which they are encountered. The end point of one fetch or obstruction becomes the beginning point of the next fetch or obstruction; the transmitted wave height from one fetch or obstruction becomes the incident wave height for the next fetch or obstruction. The stillwater elevations for the 100-year flood are to be used as the basis for these computations. Values for all parameters determined at each point of calculation should reflect the typical or average conditions throughout the width of the reach. Wave heights should be calculated to the nearest 0.1 foot, and wave elevations should be determined at whole-foot increments along the transects. The location of the 3-foot breaking wave for determining the terminus of the V Zone (areas with velocity wave action) is also to be computed at each transect.

The transect and calculations should be continued inland until (a) the wave crest elevation permanently decreases to less than 0.5 foot above the stillwater elevation, or (b) the waves meet flooding from another source (such as riverine) which determines the maximum water-surface elevation.

b. Determination of Breaking Wave Height

Equation 1 ($H_b = 0.78d$) is used to calculate the height of the breaking wave which originated over an open water body with an unlimited fetch (greater than 20 miles) and as a check for the upper limit of wave height at any point along a transect. NAS Equation 3 is used to compute the height of a wave which originated over a deep water body with a fetch length of less than 20 miles and with an incident wave height of zero.

Once an obstruction is encountered which causes energy dissipation, the equation $H_t = BH_i$ (Equation 3 above) is applied as appropriate. Beaches and sand dunes with low rising slopes will decrease the wave height but will not warrant computing a transmission coefficient, B.

c. Determination of Transmitted Wave Height at Sand Dunes and Elongated Natural Barriers

The use of NAS Equation 4 in conjunction with NAS Equations 5, 6, and 7 for energy dissipation during passage over dunes is straight forward if the dune crest, Z_b, is level. If the dune crest is uneven, the average crest elevation should be used, as indicated in the NAS report. However, if the average crest elevation itself varies along the dune length, the variation in crest elevation should be a criterion for determining the location and distance between transects.

The transmitted wave heights should be determined as follows:

1. Compute wave height (H_i)at toe of seaward slope of dune.
2. Compute transmission coefficient (B) for the dune using NAS Equations 5, 6, and 7, where d_b = depth of water at the top of the dune.
3. Compute wave height using $H_t = BH_i$ (NAS Equation 4).

The contractor should judge whether the seawardmost dunes should be considered ineffective. They should be considered ineffective if they will probably be removed by the storm before the occurrence of the peak wave height.

d. Determination of Transmitted Wave Height at Elongated Manmade Barriers

Equations 8 and 10 of the NAS report can be applied directly for computing the transmission coefficient (B) for seawalls and other similar barriers when the situation meets the described conditions. However, NAS Equation 9 as written allows for B values greater than 1.0 to be computed, thus erroneously increasing the wave height as it passes over the barrier. The Federal Insurance Administration has adopted the following equations to replace NAS Equation 9 based on rationale presented by the NAS for Equations 5-10:

$$B = \frac{(0.78d_b) + H_i}{2H_i} \quad \text{if } H_i \geq 0.78d_b \ \underline{\text{AND}} \ d_b \geq 0 \qquad (5)$$

$$B = 0.5 - \frac{(Z_b - S_b)}{H_i} \quad \text{if } S_b < Z_b < (S_b + 0.5H_i) \qquad (6)$$

Where d_b is the depth of water at the crest of the barrier, Z_b is the elevation of the crest of the barrier, and S_b is the surge elevation at the barrier.

The transmitted wave heights should be determined as follows:

1. Determine wave height (H_i) at seaward side of barrier.

2. Determine transmission coefficient (B) using NAS Equations 8 and 10, and Equations 5 and 6 above.

3. Determine transmitted wave height using $H_t = BH_i$ (NAS Equation 4).

Note: The above procedure is applicable only where the stillwater storm surge condition exists on both sides of the barrier. When the stillwater surge is contained by the barrier, any hazard that may occur from wave runup and overtopping should be computed using the procedures described in the Shore Protection Manual[2].

e. Determination of Transmitted Wave Height at Vegetated Regions:

1. Determine wave height (H_i) at seaward edge of vegetated region.

2. Determine transmission coefficient (B) using NAS Equation 11 where C_D is between 0.35 and 1.0 (depending on Reynolds number as described below). Values for the parameters D, h, b, and w of NAS Equation 11 are determined from field investigation and aerial photography.

3. Determine transmitted wave height using $H_t = BH_i$ (NAS Equation 4). When the ground elevation on the leeward, or transmitted, side of the vegetated region is greater than that on the seaward, or incident, side, NAS Equations 4 and 11 may compute a wave height greater than the maximum possible wave height for that depth. Therefore, when computing a wave height in an area with increasing ground elevations, the wave height should be checked using NAS Equation 2 ($H_b = 0.78d_t$, where d_t is the depth at the end of the vegetated region). The wave height computed using NAS Equation 2 should be used if it is less than the wave height computed using NAS Equation 4.

For vegetated regions, NAS Equation 11 can be used rather directly, once the necessary parameters are determined. Note that if the vegetation is tall trees projecting above the surface of the water, h is equal to d, and the equation may be simplified. For a single row of trees parallel to the shore and perpendicular to the flow (thus w (width) equals D (diameter)), NAS Equation 11 predicts almost no energy dissipation. For example:

C_D	w=D	d=h	b	H_i	B	H_t
0.35	2	10	10	7.80	0.9988	7.79
1.0	2	10	10	4.68	0.9980	4.67
1.0	2	6	10	4.68	0.9967	4.66
1.0	5	10	10	7.80	0.9361	7.30

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[2] U.S. Army Corps of Engineers, Coastal Engineering Research Center, Shore Protection Manual (Washington, D.C., 1977).

C_D is the drag coefficient for the trees. For a circular cylinder, C_D is about 0.35 for turbulent flow and on the order of 1.0 for transitional flow.[3] For narrow stands of trees, velocity will be greater than for extended reaches of forest because there is less opportunity for energy dissipation to slow the movement of the water. Thus, a value of 1.0 for C_D may be applied if the stands of vegetation are wide, but a value of 0.35 for C_D may be used for very narrow stands.

Extended reaches of uniform forest often occur on sloping terrain, so d and h vary as a function of w. That variation should be estimated as a linear function based on topographic contours. Calculation of several points along the transect in a uniform forested reach will allow interpolation of elevations. For example, if a forested reach extends 500 feet inland from the shore, $S_* = d = 10$ feet at the shoreward edge and contours show that the 5-foot elevation is 1200 feet inland, calculations would be as follows:

$$C_D = 1.0,\ D = 2,\ h_{avg} = d_{avg} = 10\ (1 - w/4800),$$

$$d_t = 10\ (1 - w/2400),\ b = 10,\ H_i = 7.8, \text{ and}$$

w	d_{start}	H_i	d_{ave}	B	H_t	$0.78d_t$	$(0.7)\ H_t$	Z_w
0	10.00	7.8	10.00	1.00	7.80	7.80	5.46	15.5
50	10.00	7.8	9.90	0.92	7.18	7.64	5.03	15.0
200	10.00	7.8	9.58	0.74	5.77	7.15	4.04	14.0
350	10.00	7.8	9.27	0.62	4.84	6.66	3.39	13.4
500	10.00	7.8	8.96	0.52	4.06	6.18	2.84	12.8

B is non-linear. Therefore, the need for computing as a single reach or by subreaches should be based on engineering judgment. Using the above calculations, the 14.5-foot and 13.5-foot wave crest locations may be interpolated as

$$Z_w = 14.5 \text{ at } w = 50 + \frac{5}{10} \times 150 = 125 \text{ feet}$$

$$Z_w = 13.5 \text{ at } w = 200 + \frac{5}{6} \times 150 = 325 \text{ feet}$$

Note: The above procedures are applicable only to woody vegetation, which is relatively rigid. The contractor should not attempt to estimate energy loss by grasses or other herbaceous vegetation because this effect is insignificant except in shallow depths, where most of the wave energy has already been dissipated.

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[3] The change to turbulent flow occurs for a Reynolds number ($R = VD/\nu$) greater than 3×10^5 to 5×10^5. A C_D of 1.0 should normally be used unless the orbital velocity is clearly sufficient for turbulent flow.

f. **Determination of Transmitted Wave Height at Buildings:**

1. Determine wave height (H_i) at seaward side of building.

2. Determine transmission coefficient using NAS Equation 12, $B = r^{n/2}$.

   r = the average ratio of open to total space throughout a representative reach in each row of buildings.
   n = total number of rows of buildings seaward of the site. A value of 1 should be used for n if spacing between adjacent rows is greater than about 0.1 mile. In this case, dissipation of the wave should be computed at each row of houses, with wave regeneration computed between rows.

3. Determine transmitted wave height using $H_t = BH_i$ (NAS Equation 4). Where the ground elevation on the leeward side of the developed area is greater than the elevation on the incident side, the computed wave height should be checked using NAS Equation 2 ($H_b = 0.78d_t$). The contractor should judge whether buildings located below the incident wave elevation should be considered ineffective if they will probably be destroyed before the peak wave height of the storm occurs. This is particularly true for buildings located at the inland edge of the beach or on seawardmost dunes.

g. **Determination of Wave Regeneration Over Wind Fetches:**

The methodology provides for inland wave buildup from wind action over unimpeded fetch zones. The method recommended by the NAS for accomplishing this is to combine the wind effect with the existing wave height as follows:

$$H_f = \left[(Gd_f)^2 + H_i^2\right]^{\frac{1}{2}} \qquad (7)$$

where H_i is the initial wave height entering the fetch zone, d_f is the mean depth over the fetch zone, G is the fetch factor, and H_f is the final wave height at the end of the fetch zone. Figure 1, taken from the NAS report, shows the inland fetch factor, G, as a function of fetch length, L. Equation 7 (which is Equation 13 of the NAS report) is based on the concept of adding energies of two components (wind and wave) to estimate the final wave height.

Equation 7 is based on the addition of energies, but it is a linear approximation to the non-linear process. The two components are coupled. That is, the wind component depends on the initial wave height.

The problem of inland wave propagation can better be handled by introducing the concept of an effective fetch length, L_{eff}, which corresponds to the transmitted wave height, H_t, at each

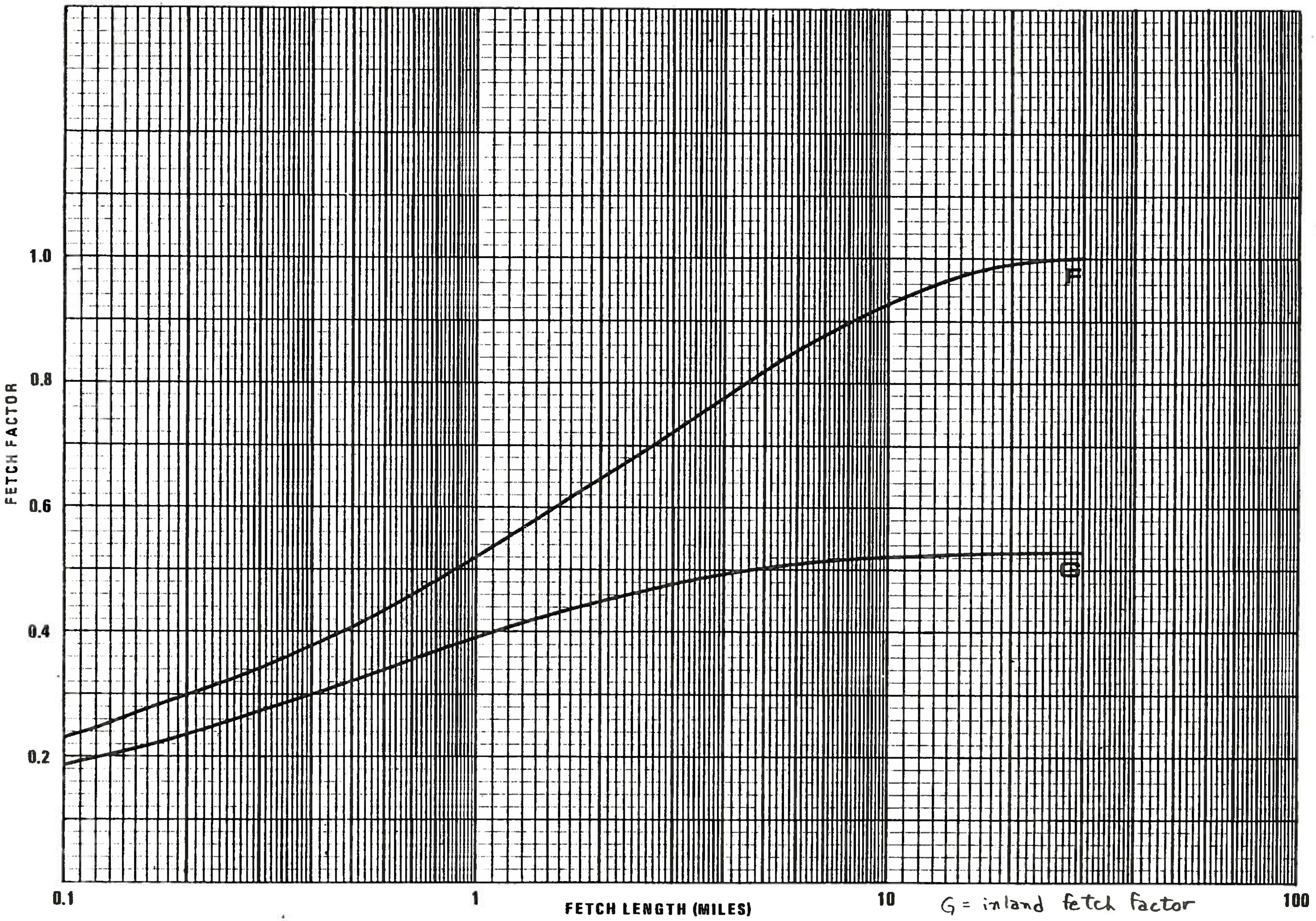


FIGURE 1

G = inland fetch factor
F pertains to deep water

point of energy dissipation. That is, L_{eff} is the fetch length which would yield a wave of height H_t. Thus, if one assumes that wave energy of a given amount is similar in spectrum, then H_t has a corresponding effective fetch length of

$$L_{eff} = G' \; (H_t/d_f) \qquad (8a)$$

where G' is the inverse function of Figure 1. The final wave height is determined by

$$H_f = G_* d_f \text{ (Equation 4 above)} \qquad (8b)$$

$$\text{where } G_* = G \text{ corresponding to } L_* \qquad (8c)$$

$$\text{and } L_* = L_f + L_{eff} \; . \qquad (8d)$$

and where d_f is the average depth throughout the fetch, L_f is the length of the open fetch following the obstruction, and L_* is the length to be used to obtain G_* from Figure 1 to compute wave height at the end of the fetch.

To compare the two methods, assume a 4-mile-long fetch, with no incident wave, with a depth of 10 feet and rows of houses at miles 1 and 2, with $B = (0.81)^{½} = 0.9$ for each row of houses. The results, using the two equations, are then:

From Equation 7					From Equations 8a-8d					
MILE	L	G	B	H_f	MILE	L_*	G_*	B	H_f	L_{eff}
1	1	0.39		3.90	1	1.00	0.39		3.90	
			0.9	3.51				0.9	3.51	0.68
2	2	0.39		5.25	2	1.68	0.435		4.35	
			0.9	4.72				0.9	3.92	1.05
4	4	0.45		6.52	4	3.05	0.47		4.70	

For a similar but unimpeded reach, G would be 0.49 and H_f would be 4.90 feet at the end of the reach.

The final height for our example problem for inland propagation across both obstacles and open spaces is 4.70 feet using Equations 8a-8d, slightly less than the value of 4.90 feet which would be obtained for a 4-mile reach without obstacles. However, if the original NAS Equation 13 was applied (Equation 7 above), an inconsistent value of 6.52 feet would be obtained.

The effective length approach suggested here yields an answer which asymptotically approaches the maximum possible height. This method is suggested for computing inland wave buildup because it is consistent with, and can be incorporated into, the overall framework of the NAS method. Assumptions adopted by the NAS, including a reduced wind velocity inland, still stand.

The method, with revised equations, should be applied as follows for inland (overland) fetches:

1. Determine length of open fetch (L_f). Do not determine regeneration for fetches of less than 0.1 mile with G_{eff} greater than 0.4 because, under those conditions, an insignificant amount of energy is added.

2. Determine wave height at the start of fetch area, H_i. If fetch begins immediately after an obstruction, H_i of the fetch calculation equals H_t of the obstruction calculation.

3. Determine G_{eff} using (H_i/d_f) where d_f is the mean depth over the fetch zone. Regeneration should not be considered where G_{eff} equals or exceeds 0.53 for inland fetches. Where d_f varies significantly, the fetch should be subdivided.

4. Apply G_{eff} to Figure 1 to find L_{eff}.

5. Sum $L_f + L_{eff}$ to obtain L_*.

6. Apply L_* to Figure 1 to obtain G_*.

7. Determine transmitted wave height using Equation 4, $(G_* \times d_f) = H_f$.

When considering shallow inland lakes, streams, and lagoons, assume the normal water-surface elevation as the ground elevation and compute wave height using the inland fetch equation as described above.

The effective length approach can also be used to compute wave buildup in deep water which may be found inland of obstacles such as barrier islands and coral reefs. In this case, the effective length is given by

$$L_{eff} = F' \; (H_t/0.78S_1) \qquad (9)$$

where F' is the inverse of Figure 1 and S_1 is the stillwater storm tide depth at the normal mean sea level shoreline.

The difference between the F and G curves in Figure 1 results from differing assumptions concerning wind velocity and depth of fetch zone. In the transition region from open water to over land, neither the F nor G curve, nor the corresponding equations, apply exactly.

To determine wave regeneration over overwater fetches, follow a similar procedure as above except compute F_{eff} using

$$F_{eff} = H_i/(0.78 \times S_1) \qquad (10)$$

and calculate the transmitted wave height using

$$H_t = F_* \times 0.78 \times S_1 \qquad (11)$$

WAVE HEIGHT ANALYSIS COMPUTATION SHEET

COMMUNITY NAME ALABAMA COAST
TRANSECT NO. A7
STILLWATER ELEVATION (S_*) 11.3'
ZONE V/ZONE A BOUNDARY 1140' inland

REFERENCE SOURCES:
MAPS: RS&H 15,29
PHOTOS: USACE 53-29,30
OTHER: USACE Beach Profile 2B-A0+00

COMPUTED BY: MB
DATE: 1/7/80
CHECKED BY: JB
DATE: 1/7/80

STATION (FEET)	DESCRIPTION	L_f	L_{eff}	L_*	G_{eff} F_{eff}	G_* F_*	Z_{g*},Z_b	d,d_f,d_b	H_i	C_D	h	D	w	b	r	n	B	H_b,H_t,H_f	Z_w
0	Shoreline						0	11.3	—									8.81	17.5
150	Dune						4.4	6.9	8.81								.61	5.37	15.1
350	Fetch	.04					4.9	6.7	5.37									5.23	15.0
400	Building						4.9	6.4	5.23						.61	1	.78	4.08	14.2
500	Fetch	.02					6.0	5.9	4.08									4.08	14.2
570	Fetch	.01					5.0	5.8	4.08									4.08	14.2
600	Building						5.0	6.3	4.08						.55	1	.74	3.02	13.4
1070	Fetch	.09	1.9	1.99	.444	.45	4.0	6.8	3.02									3.06	13.4
1140	Building						4.0	7.3	3.06						.43	1	.66	2.02	12.7
1420	Fetch	.05					3.0	7.8	2.02									2.02	12.7
1490	Vegetation						2.2	8.7	2.02	.35	8.7	1	70	5			.98	1.98	12.7
1810	Fetch	.06					3.6	8.4	1.98									1.98	12.7
1860	Vegetation						3.6	7.7	1.98	.35	7.7	1	50	5			.98	1.94	12.7
2280	Fetch	.08					1.2	8.9	1.94									1.94	12.7
2370	Vegetation						1.4	10.0	1.94	.35	10.0	1	90	5			.97	1.88	12.6
2430	Fetch	.01					1.7	9.8	1.88									1.88	12.6
2530	Vegetation						2.0	9.4	1.88	.35	9.4	1	100	5			.97	1.82	12.6
3200	Fetch	.13	.115	.245	.196	.255	2.0	9.3	1.82									2.37	13.0
3260	Vegetation						2.0	9.3	2.37	.35	9.3	1	60	5			.98	2.32	12.9
3710	Fetch	.09	205	.295	.239	.27	1.2	9.7	2.32									2.62	13.1

WAVE HEIGHT ANALYSIS COMPUTATION SHEET

COMMUNITY NAME ALABAMA COAST

TRANSECT NO. A 1 (continued)

STILLWATER ELEVATION (S_*) 11.3'

ZONE V/ZONE A BOUNDARY

REFERENCE SOURCES:

MAPS:

PHOTOS:

OTHER:

COMPUTED BY:

DATE:

CHECKED BY:

DATE:

STATION (FEET)	DESCRIPTION	L_f	L_{eff}	L_*	G_{eff} F_{eff}	G_* F_*	Z_g, Z_b	d, d_f, d_b	H_i	C_D	h	D	w	b	r	n	B	H_b, H_t, H_f	Z_w
3810	Vegetation						1.2	10.1	2.62	.35	10.1	1	100	5			.96	2.52	13.1
3960	Fetch	.03					3.9	8.8	2.52									2.52	13.1
4080	Vegetation						6.0	6.4	2.52	35	6.4	1	120	5			.93	2.35	13.0
5080	Vegetation						4.0	6.3	2.35	1.0	6.3	1	980	5			.39	.92	11.9
6180	Vegetation						10.0	4.3	.92	1.0	4.3	1	1120	5			.50	.46	11.6
6200	Ground Elevation = Surge Elevation						11.3	0	.46									0	11.3

5. SPECIAL SITUATIONS

a. Changing Surge Elevations

Surge elevations may vary moving inland as well as alongshore. Thus, a single transect may encounter more than one set of surge elevations. Surge elevations may change over land, over water, at any type of fetch or obstruction. A possible approach would be to treat the situation as a linear transition over some logical transition zone by locating two points along the transect between which the surge elevation will change. These two points would identify the transition area. The beginning point of the transition area is the end point of the previous fetch or obstruction in which the old surge elevation was fully effective. The end point of the transition area is the point at which the new surge elevation becomes fully effective. Increasing the length of the transition area causes a more gradual transition between the old and new surge elevations. If the transition area includes more than one fetch or obstruction, that portion of the surge change which occurs in each fetch or obstruction must be incorporated into the calculations for that fetch or obstruction.

b. Multiple Flooding Sources

Spits, barrier islands, peninsulas, and other landforms with large water bodies on more than one side may be subject to storm surges and waves approaching from more than one direction. Wave heights and elevations should be calculated along transects taken perpendicular to shorelines of all coastal flooding sources. Wave elevations on the landmass should represent worst-case situations with maximum possible elevations shown in each area. Good judgment should be used when delineating elevations expected to exist at the end of peninsulas and barrier islands where alongshore variations in surge elevations may occur. Delineations should decrease by whole-foot increments around the end of the landmass when maximum wave crest elevations of the two flooding sources are unequal.

C. Restricted Inlets

Narrow, restricted inlets such as stream channels oriented perpendicular to the shoreline create a narrow wind fetch area over the stream which may allow waves to continue inland until the channel changes direction and is no longer perpendicular to the shoreline. At this point, the waves would dissipate against the rising ground elevations at the convex channel bank. However, some energy dissipation will occur due to roughness along the channel banks where the channel is perpendicular to the shoreline. How extensively the channel bank dissipation affects the entire wave-front elevation depends in part on the ratio of width of stream to length of overstream fetch. Wave elevations delineations should reflect this expected behavior and should tie in logically with delineations on either side of the stream channel.

Concave?
where waves
jump bank?

In narrow bays and estuaries not oriented perpendicular to an unlimited fetch zone and where fetch lengths exceed fetch widths, the fetch factor curves provided by the NAS should be adjusted in the process of determining wave generation. The user is referred to the effective fetch length computation procedures provided in Chapter 3, Volume II, of the Shore Protection Manual.

Wave refraction and reflection may be disregarded in considering wave travel upstream along a channel, although the qualitative consideration of these effects will aid in assessing the wave dissipation as mentioned earlier. The complex spectrum of wave energy transmitted precludes a simple assessment of refraction and reflection effects.

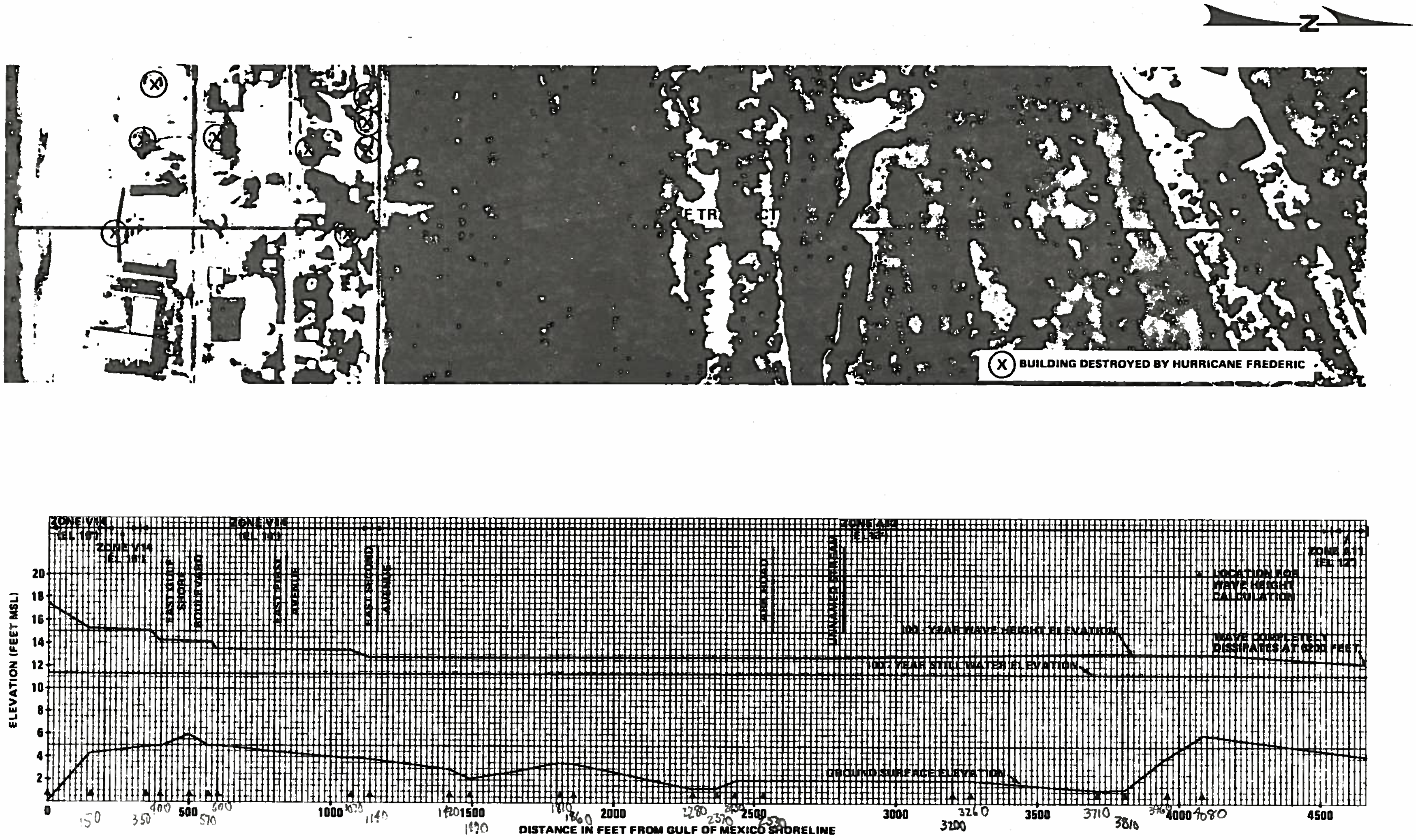
6. SAMPLE CALCULATIONS

A set of sample calculations is included to illustrate the application of the equations at computation points along a sample transect (Table 1). The calculations are recorded on a computation sheet. A similar computation sheet should be used to document all calculations of wave height analyses. A blank computation sheet is included as Attachment A.

A wave elevation profile is also included (Figure 2). This shows the effects of obstructions, wind fetches, and changing ground elevation on wave elevations along the sample transect.

The sample transect was taken on the Alabama coast, shortly after the area was struck by Hurricane Frederic. The aerial photograph in Figure 2 indicates which structures were removed by Hurricane Frederic and, therefore, considered ineffective in the analysis.

SAMPLE TRANSECT
FIGURE 2



ee preceding computation sheets

WAVE HEIGHT ANALYSIS COMPUTATION SHEET

COMMUNITY NAME ______________________

TRANSECT NO. ______________________

STILLWATER ELEVATION (S_*) ______________

ZONE V/ZONE A BOUNDARY ______________

REFERENCE SOURCES:

MAPS: ______________________

PHOTOS: ______________________

OTHER: ______________________

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STATION (FEET)	DESCRIPTION	L_f	L_{eff}	L_*	G_{eff} F_{eff}	G_* F_*	Z_{g*}, Z_b	d, d_f, d_b	H_i	C_D	h	D	w	b	r	n	B	H_b, H_t, H_f	Z_w

ATTACHMENT A

SECTION 3

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
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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

CONTENTS

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~~ OR 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.

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

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[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. ? [80, 60 mph ?]

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

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[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

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[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

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[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

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[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:

$$Z_w = S_* + 0.7\,H_*, \qquad (1)$$

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:

$$H_{*b} = 0.78\ d_*, \tag{2}$$

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 <u>n</u>-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 1~~4~~7 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, 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:

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

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

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

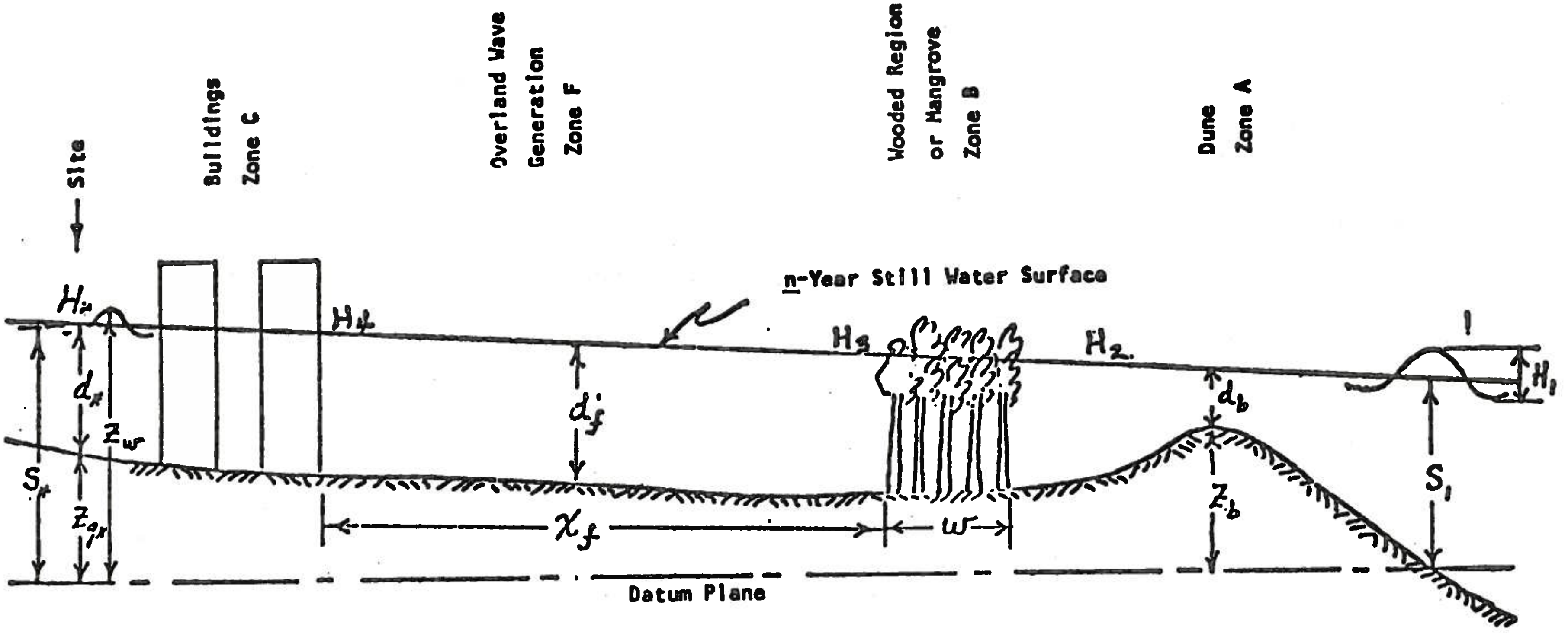

FIGURE 1 Hypothetical Profile Normal to a Shoreline

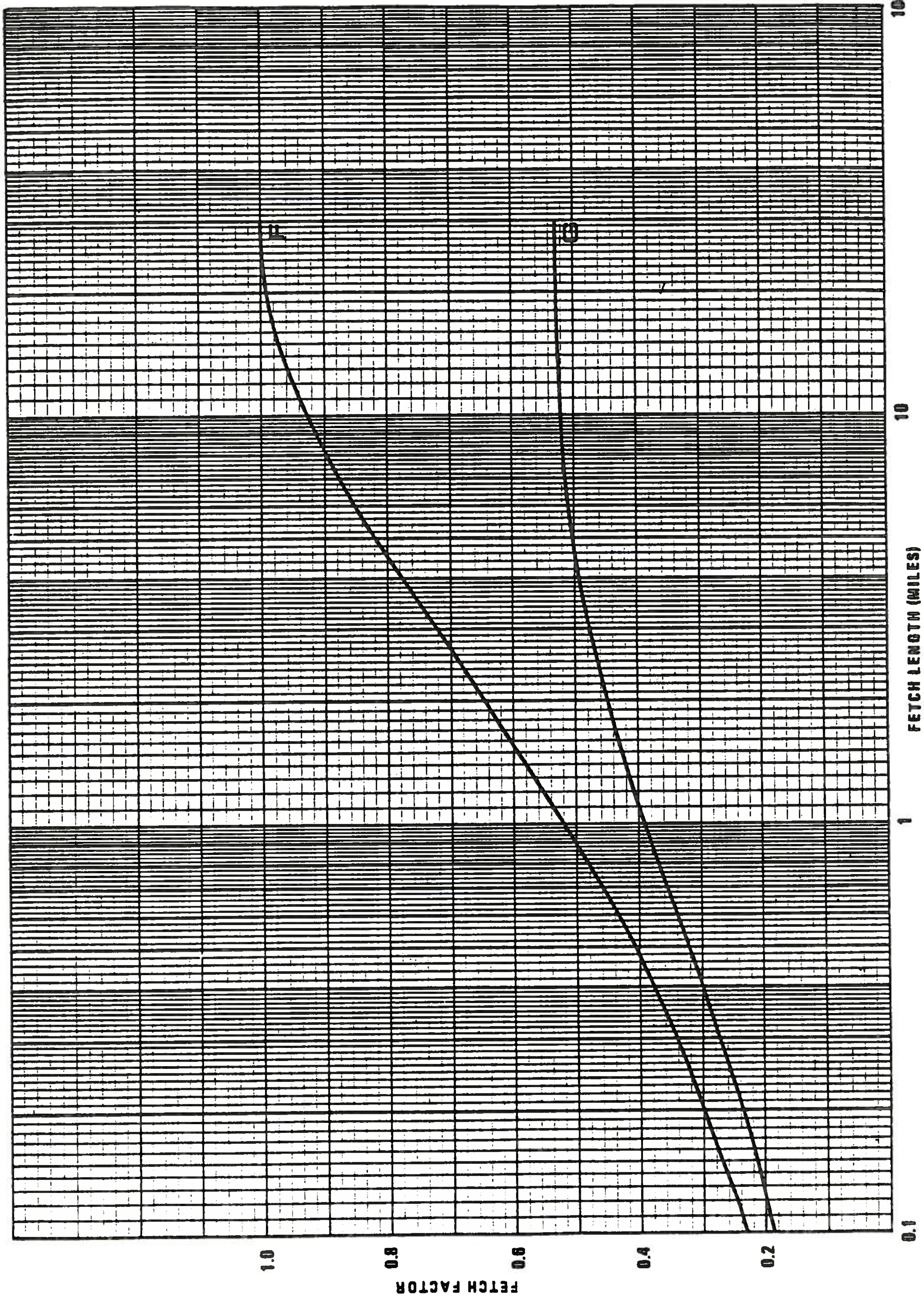

FIGURE 2

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:

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

See pp 7, 9 in
Dames & Moore, 198

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. Rationale

Assuming that the n-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 $\overset{.5,\ .68}{\text{short}}$ and $\overset{.5,\ 1.0}{\text{long}}$ 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).

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b. <u>Eq. (2)</u>

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 <u>Shore Protection Manual</u>. 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. <u>Eq. (3) and Table 1</u> – lesser breaking wave heights with limited wave generation.

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 <u>Shore Protection Manual</u> 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:

$$H_c = 1.6\ H_s, \qquad \approx H_{1.4} \tag{14}$$

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:

INCONSISTENT NOTATION: SHOULD BE DENOTED BY H_{100}

Mean wave height = $\boxed{H_{50}} = 0.625\ H_s$,

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 = $H_1 = 1.67\ H_s$.

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TABLE 3 SIMPLIFIED

Table 3 in the 1977 Methodology has 5 columns of numbers (see copy attached). The footnote says that the Fetch coefficient F is the ratio of depths, but the rationale for this is difficult to grasp. Actually, the F is the ratio of significant heights, according to the assumptions used.

By examining Column 2 in Table 3, it is evident that the fetch coefficient, F, is the value of H_s divided by H_s for fetches greater than 20 miles, i.e., the H_s divided by 7.3 feet. The fetch factor, F, is thus understood to be a ratio of the wave height at a short fetch to the wave height at an infinitely long fetch (this sentence could replace the footnote now with Table 3).

Since the depth depends on the height, the controlling depth also changes in proportion to fetch factor. (See Table 3 to verify that the controlling depth (d_c) is merely twice the significant height (H_s).) This makes more physical sense to me, but it may not matter to someone who is already used to the methodology.

Attachment:
  Table 3, p 21, Methodology

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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

see ①②③ p 19

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):

$$H_1 = 0.78 \times 0.65 \times 16 = 8.1 \text{ feet.}$$

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

$$H_2 = 8.1 \text{ feet.}$$

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_* = $\underline{n}$-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 $\mathring{e}$ is:

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

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 $\bar{\mathring{e}}$ is:

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

where the overbar denotes time averaging. The result is:

$$\bar{\dot{e}} = \frac{4}{3\pi} U_m^3 \frac{C_D \rho D}{2} h, \qquad (4)$$

which is the average energy loss associated with the oscillation of a single cylinder. In the next section these results will be combined with the energy flux relationships to result in a wave height attenuation relationship.

2. Wave Energy Flux Relationship

If $-\bar{\dot{E}}$ represents the average wave energy dissipation rate per unit surface area, the equation governing wave energy is:

$$\frac{\partial (E\, C_G)}{\partial x} = -\bar{\dot{E}}, \qquad (5)$$

where $E = \rho \frac{gH^2}{8}$, g is the gravitational constant, and C_G is the group velocity (C_G for shallow water waves is $\sqrt{gd}$). If the average horizontal spacing of the obstructing element is b, the number per unit area is $1/b^2$ and the average energy loss per unit area is:

$$\bar{\dot{E}} = \frac{1}{b^2}\, \bar{\dot{e}} = \frac{4}{3\pi} \frac{U_m^3}{b^2} \frac{C_D \rho D}{2} h. \qquad (6)$$

Combining Eqs. (5) and (6), assuming uniform depth over the length, w, of the obstructing region and introducing the shallow water approximation for maximum water particle velocity, U_m, where $U_m = \frac{H}{2}\sqrt{\frac{g}{d}}$, wave height H_2 just landward of the obstructing region is expressed in terms of the wave height H_1 just seaward of the obstructing region as:

$$H_2 = \frac{H_1}{1 + \frac{1}{3\pi} H_1 \frac{C_D\, h\, Dw}{b^2 d^2}} . \qquad (7)$$

*U.S. GOVERNMENT PRINTING OFFICE: 198[illegible]-0-523-860/232

TABLE OF CONTENTS

INTRODUCTION

This publication contains three separate documents concerning the calculation of wave heights in coastal high hazard areas.

The first document, "Field Manual for Estimating Wave Heights in Coastal High Hazard Areas in Atlantic and Gulf Coast Regions" was originally prepared by the Federal Insurance Administration (FIA) for use with the FIA individual insurance rating system in FIA-designated V zones. It is a simplified and approximate version of the method FIA uses in flood insurance studies. The detailed methodology is provided by the second and third documents.

The "Users Manual for Wave Height Analysis" is a practical guide to the application of the methodology.

The "Methodology for Calculating Wave Action Effects Associated with Storm Surges" serves as the technical background and support for the approach.

All three documents should be useful to those who must determine the extent and elevations of coastal high hazard areas for purposes of implementing FEMA's regulations at 44 CFR Part 9 and 10, with respect to Executive Orders 11988 and 11990.

SECTION 1

FIELD MANUAL FOR ESTIMATING WAVE HEIGHTS

IN

COASTAL HIGH HAZARD AREAS

IN

ATLANTIC AND GULF COAST REGIONS

March 1981

FEDERAL EMERGENCY MANAGEMENT AGENCY

FEDERAL INSURANCE ADMINISTRATION

CONTENTS

ESTIMATING WAVE HEIGHTS IN COASTAL HIGH HAZARD ZONES

INTRODUCTION

The elevation of coastal flood waters along the Gulf of Mexico and the Atlantic Ocean consists of the combined height of three factors: (1) astronomical tides; (2) storm surges; and (3) wind generated waves.

To implement the National Flood Insurance Program, the Federal Insurance Administration has published Flood Insurance Rate Maps (FIRM) showing Base Flood Elevations that have been adopted as minimum criteria for various requirements of the program. For many coastal communities, the Base Flood Elevations shown on the published FIRMs are stillwater elevations which include only the effects of the tide and surge, and not the height of wind generated waves.*

This Manual contains approximate procedures for rapidly estimating wave heights associated with the Base Flood in communities for which the published FIRMs lack wave height determinations. More detailed procedures may be found in the "Users Manual for Wave Height Analysis," Federal Insurance Administration, February 1981, and in the "Methodology for Calculating Wave Action Effects Associated with Storm Surges," National Academy of Sciences, 1977, upon which this general approach is based.

GENERAL APPROACH

The height of waves at an inland site is determined mainly by the Base Flood Elevation and terrain conditions between the shoreline and the site. This Manual outlines a set of sequential calculations that permit an estimate of the cumulative effects of the principal obstructions to wave transmission.

In performing the sequential calculations, a beginning wave height is calculated at the shoreline; this wave height is continued inland and becomes the <u>arriving</u> wave height at the first obstruction encountered. The <u>leaving</u> wave height is calculated for the first obstruction, and this becomes the <u>arriving</u> wave height for the next obstruction encountered. The procedure is repeated for each succeeding obstruction until the building site is reached.

PROCEDURES FOR ESTIMATING WAVE HEIGHT

A. Define Flood Reach

On a suitable map, draw a line through the site parallel to the general shoreline trend and extending 250 feet to each side of the

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*Where wave heights are included in the Base Flood Elevation, this is noted in the legend box of the Flood Insurance Rate Map, and use of this Manual is not required.

site. Extend the end points directly to the shoreline. This defines a rectangular area 500 feet wide in which obstructions are to be considered. For shorelines less than 5 miles behind a barrier island, the flood reach should be extended across the island to the open coast.

B. Determine Beginning Wave Height

1. Open Coast

For shorelines on the open coast, such as barrier islands and mainland areas fronting directly on the Ocean or Gulf, multiply the Base Flood Elevation by .78. This is the beginning wave height on an open coast.

2. Non-Open Coasts

(a) Bays, Lagoons, Estuaries

For shorelines not on the open coast or a protected coast, estimate the width of the bay, lagoon, or estuary, in miles, in a direction perpendicular to the shoreline. (If this distance is more than 10 miles, treat it as open coast.)

Using this width, determine the fetch factor from Table 1. Multiply the Base Flood Elevation by this factor. This is the beginning wave height on bays, lagoons, and estuaries and other indentations of the coast.

(b) Protected Coasts

Protected coasts are mainland shores that are 5 miles or less behind a barrier island. In such cases, the beginning wave height is calculated at the open coast of the barrier island.

C. Determine Effects of Obstructions and Fetches

Three types of obstructions and one type of fetch are considered. The type that best fits should be chosen for each calculation, and only those obstructions judged capable of withstanding the Base Flood should be considered.

The obstructions and fetch, if any, should be located and labelled sequentially on the map. The information used in the calculations should be entered on the forms provided at the back of the Manual.

1. Continuous Linear Obstructions

(a) Description

Continuous linear obstructions are man-made or natural features trending at least 500 feet parallel or subparallel to the shoreline trend. Two types of continuous linear obstructions are considered: natural features (such as sand dunes) and man-made features (such as seawalls, dikes, road embankments, levees, and attached buildings).

(b) Information Needed

The following information is needed to determine the height of the waves leaving continuous linear obstructions:

(1) Arriving wave height in feet
(2) Base Flood Elevation in feet above sea level
(3) Average elevation of top of obstruction in feet above sea level
(4) Elevation of ground surface at leeward edge of man-made features in feet above sea level

(c) Procedures for Natural Features

(1) If the top of the obstruction is higher than the Base Flood Elevation, then the leaving wave height is 0.

(2) If the obstruction is lower than the Base Flood Elevation, multiply the difference between the Base Flood Elevation and the top of the barrier by .78. Compare this to the arriving wave height. The smaller of the two values is the leaving wave height.

(d) Procedures for Man-Made Features

(1) When the obstruction is lower than the Base Flood Elevation:

(a) Divide the difference between the Base Flood Elevation and the elevation of the obstruction by the arriving wave height to obtain a ratio. Round this to the nearest hundredth and use Table 2 to find a transmission coefficient. Multiply the arriving wave height by this coefficient.

(b) Multiply the difference between the Base Flood Elevation and the average ground elevation at the leeward edge of the obstruction by .78.

(c) The leaving wave height is the smaller of these two calculations.

(2) When the obstruction is <u>higher</u> than the Base Flood Elevation:

   (a) Divide the difference between the elevation of the obstruction and the Base Flood Elevation by the arriving wave height. Subtract this from 0.5 to obtain a transmission coefficient, and multiply the arriving wave height by this coefficient.

   (b) Multiply the difference between the Base Flood Elevation and the ground elevation at the leeward edge of the obstruction by .78.

   (c) The leaving wave height is the smaller of these two calculations.

2. Discontinuous Linear Obstructions

(a) Description

Discontinuous linear obstructions are mainly rows of detached buildings trending at least 500 feet parallel or subparallel to the shoreline trend. Where rows are more than 500 feet apart, they should be treated as separate obstructions.

(b) Information Needed

The following information is needed to determine the height of waves leaving discontinuous linear obstructions:

(1) Arriving wave height in feet
(2) Base Flood Elevation in feet above sea level
(3) Average <u>total</u> open space between buildings within rows in feet
(4) Number of rows of buildings
(5) Average elevation of ground surface at leeward edge of obstruction in feet above sea level

(c) Procedures

(1) Divide the average open space within the rows by 500 to obtain a ratio. Round to the nearest tenth and, using the number of rows, find a transmission coefficient from Table 3. Multiply the arriving wave height by this coefficient.

(2) Multiply the difference between the Base Flood Elevation and the average ground elevation at the leeward edge of the obstruction by .78.

(3) The leaving wave height is the smaller of these two calculations. If the ground elevation at the leeward edge is greater than the Base Flood Elevation, the leaving wave height is 0.

3. Vegetation Obstructions

(a) Description

Vegetation obstructions are classified as dense brush or mangrove; forest; and scattered trees and brush. To be considered, they must extend across the entire flood reach. Non-rigid herbaceous vegetation, such as grasses, should not be considered.

(b) Information Needed

The following information is needed to determine the height of the waves leaving a vegetation obstruction:

(1) Arriving wave height in feet
(2) Base Flood Elevation in feet above sea level
(3) Average ground surface elevation in vegetated area in feet above sea level
(4) Average height of vegetation in feet
(5) Width of vegetated area in feet perpendicular to shoreline
(6) Elevation of ground surface at leeward edge of vegetated area in feet above sea level

(c) Procedures - Vegetation higher than Base Flood Elevation

(1) Divide the arriving wave height by the difference between the Base Flood Elevation and the ground elevation in the vegetated area to obtain a ratio. Use this ratio and the width of the vegetated area to find the transmission coefficient in Table 4. Multiply the arriving wave height by this coefficient.

(2) Multiply the difference between the Base Flood Elevation and the elevation at the leeward edge of the vegetated area by .78.

(3) The leaving wave height is the smaller of the two calculations. If the ground elevation at the leeward edge is greater than the Base Flood Elevation, the leaving wave height is 0.

(d) Procedures - Vegetation lower than Base Flood Elevation

(1) Multiply the arriving wave height by vegetation height and divide by the square of the difference between the Base Flood Elevation and the average ground elevation

within the vegetated area. Use this ratio and the width of the vegetated area to find the transmission coefficient in Table 4. Multiply the arriving wave height by this coefficient.

(2) Multiply the difference between the Base Flood Elevation and the ground elevation at the leeward edge of the vegetated area by .78.

(3) The leaving wave height is the smaller of the two calculations. If the ground elevation at the leeward edge is greater than the Base Flood Elevation, the leaving wave height is 0.

4. Fetches

(a) Description

A fetch is an area in which there are no obstructions to waves. Only one type of fetch is considered: bodies of water less than 5 miles wide in the flood reach behind a barrier island.

(b) Information Needed

The following information is needed to determine the height of the wave leaving the water body:

(1) Arriving wave height in feet
(2) Base Flood Elevation in feet above sea level
(3) Width of water body in miles

(c) Procedures

Divide the arriving wave height by the Base Flood Elevation to obtain a ratio. Use this ratio, and the width of the water body, to find the fetch factor in Table 5. Multiply the Base Flood Elevation by this fetch factor to obtain the height of the waves leaving the water body.

D. Determine Wave Height at Site

The wave height at the site is estimated for the lowest point at the site.

1. Information Needed

The following information is needed to make the final estimate of wave height at the site:

(a) Arriving Wave Height
(b) Base Flood Elevation in feet above sea level
(c) Site elevation in feet above sea level

2. Procedure

Multiply the difference between the Base Flood Elevation and the site elevation by .78. Compare this to the arriving wave height. The smaller of the two values is the estimated wave height at the site.

E. Determine Wave Crest Elevation at Site

Multiply the wave height at the site by 0.7 and add this to the stillwater Base Flood Elevation. This is the wave crest elevation.

Table 1

Fetch Factor for Beginning Wave Height

Width of Water Body*	Fetch Factor
0.5 - 1.0	.35
1.0 - 1.5	.43
1.5 - 2.0	.48
2.0 - 2.5	.52
2.5 - 3.0	.55
3.0 - 3.5	.58
3.5 - 4.0	.59
4.0 - 4.5	.61
4.5 - 5.0	.62
5.0 - 6.0	.65
6.0 - 7.0	.67
7.0 - 8.0	.69
8.0 - 9.0	.70
9.0 - 10.0	.72

*In Miles

Table 2

Transmission Coefficient for Man-Made Linear Obstructions

Ratio	Transmission Coefficient
.10	.54
.15	.56
.20	.58
.25	.60
.30	.62
.35	.64
.40	.66
.45	.68
.50	.69
.55	.71
.60	.73
.65	.75
.70	.77
.75	.79
.80	.81
.85	.83
.90	.85
.95	.87
1.00	.89
1.05	.91
1.10	.93
1.15	.95
1.20	.97
1.25	.99
1.28	1.00

Table 3

Transmission Coefficient for Detached Buildings

Number of Rows

Ratio	1	2	3	4	5
.1	.32	.10	0	0	0
.2	.45	.20	.10	0	0
.3	.55	.30	.16	.10	0
.4	.63	.40	.25	.16	.10
.5	.71	.50	.35	.25	.18
.6	.77	.60	.46	.36	.28
.7	.84	.70	.59	.49	.41
.8	.89	.80	.72	.64	.57
.9	.95	.90	.85	.81	.77

Table 4

Transmission Coefficients for Vegetation

Dense Brush Including Mangrove

Ratio*	Width (Feet)							
	25	50	100	200	400	600	800	1000
.1	.95	.90	.82	.70	.54	.44	.37	.32
.2	.90	.82	.70	.54	.37	.28	.23	.19
.3	.86	.76	.61	.44	.28	.21	.16	.14
.4	.82	.70	.54	.37	.23	.16	.13	.11
.5	.79	.65	.49	.32	.19	.14	.11	.09
.6	.76	.61	.44	.28	.16	.12	.09	.07
.7	.73	.57	.40	.25	.14	.10	.08	.06
.78	.71	.55	.38	.23	.13	.09	.07	.06

Forest

Ratio*	Width (Feet)							
	25	50	100	200	400	600	800	1000
.1	.99	.99	.99	.98	.96	.94	.92	.90
.2	.99	.99	.98	.96	.92	.89	.85	.82
.3	.99	.98	.97	.94	.89	.84	.80	.76
.4	.99	.98	.96	.92	.85	.80	.75	.70
.5	.99	.97	.95	.90	.82	.76	.70	.65
.6	.98	.97	.94	.89	.80	.72	.66	.61
.7	.98	.96	.93	.87	.77	.69	.63	.57
.78	.98	.96	.92	.86	.75	.67	.60	.55

Scattered Trees or Shrubs

Ratio*	Width (Feet)							
	25	50	100	200	400	600	800	1000
.1	.99	.99	.99	.99	.98	.97	.96	.95
.2	.99	.99	.99	.98	.96	.94	.92	.90
.3	.99	.99	.98	.97	.94	.91	.89	.86
.4	.99	.99	.98	.96	.92	.89	.85	.82
.5	.99	.99	.97	.95	.90	.86	.82	.79
.6	.99	.98	.97	.94	.89	.84	.80	.76
.7	.99	.98	.96	.93	.87	.82	.77	.73
.78	.99	.98	.96	.92	.86	.80	.75	.71

*If ratio is greater than .78, use coefficients for .78

Table 5

Fetch Factors for Protected Coasts

Ratio	Width (Miles)										
	<.5	.5	1.0	1.5	2.0	2.5	3.0	3.5	4.0	4.5	5.0
<.2	*	.32	.41	.46	.50	.54	.56	.59	.60	.62	.64
.2	.20	.35	.42	.47	.51	.54	.57	.59	.61	.63	.64
.3	.30	.39	.45	.50	.53	.56	.58	.60	.62	.63	.65
.4	.40	.46	.50	.53	.56	.58	.60	.62	.64	.65	.66
.5	.50	.53	.56	.58	.60	.62	.64	.65	.66	.67	.68
.6	.60	.62	.63	.65	.66	.67	.68	.69	.70	.70	.71
.7	.70	.71	.71	.72	.72	.73	.73	.74	.74	.74	.75
.78	.78	.78	.78	.78	.78	.78	.78	.78	.78	.78	.78

*If ratio is less than .20, and width is less than .5 mile, leaving wave height should be made equal to the arriving wave height.

NAME______________________________

DATA FORM FOR BEGINNING WAVE HEIGHT

Open Coast

1. Base Flood Elevation.........______
2. Beginning Wave Height:
   (Line 1 x .78)..............______

Bays, Estuaries, etc.

1. Base Flood Elevation..............______
2. Width of Water Body...............______
3. Fetch Factor (From Table 1).......______
4. Beginning Wave Height
   (Line 1 x Line 3)................______

DATA FORM FOR FETCH ON PROTECTED COAST

1. Arriving Wave Height.........______
2. Base Flood Elevation.........______
3. Width of Water Body..........______
4. Line 1 ÷ Line 2 =.................______
5. Fetch Factor (From Table 5).......______
6. Leaving Wave Height
   (Line 2 x Line 5) = ...______

DATA FORM FOR WAVE HEIGHT AT SITE

1. Arriving Wave Height.........______
2. Base Flood Elevation.........______
3. Site Elevation...............______
4. Line 2 - Line 3 = ...______
5. Line 4 x .78 = ...______
6. Wave Height at Site
   (Smaller of Lines 1 and 5).......______

DATA FORM FOR WAVE CREST ELEVATION AT SITE

1. Wave Height at Site..........______
2. Base Flood Elevation.........______
3. Line 1 x 0.7 = ...______
4. Wave Crest Elevation
   (Line 2 + Line 3) = ...______

NAME ______________________________

DATA FORMS FOR NATURAL LINEAR OBSTRUCTIONS

OBSTRUCTION NO.________ DESCRIPTION ______________________________

1. Arriving Wave Height..........._____
2. Base Flood Elevation..........._____
3. Elevation of Obstruction........_____
4. Line 2 - Line 3 = ..._____
5. Line 4 x .78 = ..._____
6. Leaving Wave Height (Smaller of Lines 1 and 5)......._____

OBSTRUCTION NO.________ DESCRIPTION ______________________________

1. Arriving Wave Height..........._____
2. Base Flood Elevation..........._____
3. Elevation of Obstruction........_____
4. Line 2 - Line 3 = ..._____
5. Line 4 x .78 = ..._____
6. Leaving Wave Height (Smaller of Lines 1 and 5)......._____

OBSTRUCTION NO.________ DESCRIPTION ______________________________

1. Arriving Wave Height..........._____
2. Base Flood Elevation..........._____
3. Elevation of Obstruction........_____
4. Line 2 - Line 3 = ..._____
5. Line 4 x .78 = ..._____
6. Leaving Wave Height (Smaller of Lines 1 and 5)......._____

OBSTRUCTION NO.________ DESCRIPTION ______________________________

1. Arriving Wave Height..........._____
2. Base Flood Elevation..........._____
3. Elevation of Obstruction........_____
4. Line 2 - Line 3 = ..._____
5. Line 4 x .78 = ..._____
6. Leaving Wave Height (Smaller of Lines 1 and 5)......._____

OBSTRUCTION NO.________ DESCRIPTION ______________________________

1. Arriving Wave Height..........._____
2. Base Flood Elevation..........._____
3. Elevation of Obstruction........_____
4. Line 2 - Line 3 = ..._____
5. Line 4 x .78 = ..._____
6. Leaving Wave Height (Smaller of Lines 1 and 5)......._____

NAME______________________________

DATA FORMS FOR MAN-MADE LINEAR OBSTRUCTIONS

OBSTRUCTION NO.__________ DESCRIPTION______________________________

1. Arriving Wave Height............______
2. Base Flood Elevation............______
3. Elevation of Obstruction...........______
4. Elevation, Leeward Edge............______

(a) If Line 3 is <u>Less</u> than Line 2:

5a. Line 2 - Line 3 = ...______
6a. Line 5a ÷ Line 1 = ...______
7a. Transmission Coefficient..______
8a. Line 1 x Line 7a = ...______
9a. Line 2 - Line 4 = ...______
10a. Line 9a x .78 = ...______
11a. Leaving Wave Height (Smaller of Lines 8a and 10a)................______

(b) If Line 3 is <u>More</u> than Line 2:

5b. Line 3 - Line 2 = ...______
6b. Line 5b ÷ Line 1 = ...______
7b. 0.5 - Line 6B = ...______
8b. Line 1 x Line 7b = ...______
9b. Line 2 - Line 4 = ...______
10b. Line 9b x .78 = ...______
11b. Leaving Wave Height (Smaller of Lines 8b and 10b)..................______

OBSTRUCTION NO.__________ DESCRIPTION______________________________

1. Arriving Wave Height............______
2. Base Flood Elevation............______
3. Elevation of Obstruction...........______
4. Elevation, Leeward Edge............______

(a) If Line 3 is <u>Less</u> than Line 2:

5a. Line 2 - Line 3 = ...______
6a. Line 5a ÷ Line 1 = ...______
7a. Transmission Coefficient..______
8a. Line 1 x Line 7a = ...______
9a. Line 2 - Line 4 = ...______
10a. Line 9a x .78 = ...______
11a. Leaving Wave Height (Smaller of Lines 8a and 10a)................______

(b) If Line 3 is <u>More</u> than Line 2:

5b. Line 3 - Line 2 = ...______
6b. Line 5b ÷ Line 1 = ...______
7b. 0.5 - Line 6B = ...______
8b. Line 1 x Line 7b = ...______
9b. Line 2 - Line 4 = ...______
10b. Line 9b x .78 = ...______
11b. Leaving Wave Height (Smaller of Lines 8b and 10b)..................______

NAME________________________

DATA FORMS FOR DETACHED BUILDINGS

OBSTRUCTION NO.__________ DESCRIPTION______________________________

1. Arriving Wave Height............______
2. Base Flood Elevation............______
3. Open Space Per Row..............______
4. Number of Rows..................______
5. Elevation, Leeward Edge.........______
6. Line 3 ÷ 500....................______
7. Transmission Coefficient (From Table 3)..................______
8. Line 1 x Line 7 = ...______
9. Line 2 - Line 5 = ...______
10. Line 9 x .78 = ...______
11. Leaving Wave Height (Smaller of Lines 8 and 10)...............______

OBSTRUCTION NO.__________ DESCRIPTION______________________________

1. Arriving Wave Height............______
2. Base Flood Elevation............______
3. Open Space Per Row..............______
4. Number of Rows..................______
5. Elevation, Leeward Edge.........______
6. Line 3 ÷ 500....................______
7. Transmission Coefficient (From Table 3)..................______
8. Line 1 x Line 7 = ...______
9. Line 2 - Line 5 = ...______
10. Line 9 x .78 = ...______
11. Leaving Wave Height (Smaller of Lines 8 and 10)...............______

OBSTRUCTION NO.__________ DESCRIPTION______________________________

1. Arriving Wave Height............______
2. Base Flood Elevation............______
3. Open Space Per Row..............______
4. Number of Rows..................______
5. Elevation, Leeward Edge.........______
6. Line 3 ÷ 500....................______
7. Transmission Coefficient (From Table 3)..................______
8. Line 1 x Line 7 = ...______
9. Line 2 - Line 5 = ...______
10. Line 9 x .78 = ...______
11. Leaving Wave Height (Smaller of Lines 8 and 10)...............______

OBSTRUCTION NO.__________ DESCRIPTION______________________________

1. Arriving Wave Height............______
2. Base Flood Elevation............______
3. Open Space Per Row..............______
4. Number of Rows..................______
5. Elevation, Leeward Edge.........______
6. Line 3 ÷ 500....................______
7. Transmission Coefficient (From Table 3)..................______
8. Line 1 x Line 7 = ...______
9. Line 2 - Line 5 = ...______
10. Line 9 x .78 = ...______
11. Leaving Wave Height (Smaller of Lines 8 and 10)...............______

NAME______________________________

DATA FORMS FOR VEGETATION OBSTRUCTIONS

OBSTRUCTION NO.________ DESCRIPTION______________________________

1. Arriving Wave Height...........______
2. Base Flood Elevation...........______
3. Elevation, Vegetated Area.......______
4. Vegetation Height..............______
5. Vegetation Width..................______
6. Elevation, Leeward Edge...........______
7. Line 2 - Line 3 = ...______

(a) If Line 4 is <u>More</u> than Line 7:

8a. Line 1 ÷ Line 7 = ...______
9a. Transmission Coefficient
(Table 4)................______
10a. Line 1 x Line 9a = ...______
11a. Line 2 - Line 6 = ...______
12a. Line 11a x .78 = ...______
13a. Leaving Wave Height
(Smaller of Lines 10a
and 12a)................______

(b) If Line 4 is <u>Less</u> than Line 7:

8b. Line 1 x Line 4 = ...______
9b. Line 7 x Line 7 = ...______
10b. Line 8b ÷ Line 9b...........______
11b. Transmission Coefficient
(Table 4..................______
12b. Line 11b x Line 1 = ...______
13b. Line 2 - Line 6 = ...______
14b. Line 13b x .78 = ...______
15b. Leaving Wave Height
(Smaller of Lines 12b and
14b).....................______

OBSTRUCTION NO.________ DESCRIPTION______________________________

1. Arriving Wave Height...........______
2. Base Flood Elevation...........______
3. Elevation, Vegetated Area.......______
4. Vegetation Height..............______
5. Vegetation Width..................______
6. Elevation, Leeward Edge...........______
7. Line 2 - Line 3 = ...______

(a) If Line 4 is <u>More</u> than Line 7:

8a. Line 1 ÷ Line 7 = ...______
9a. Transmission Coefficient
(Table 4)................______
10a. Line 1 x Line 9a = ...______
11a. Line 2 - Line 6 = ...______
12a. Line 11a x .78 = ...______
13a. Leaving Wave Height
(Smaller of Lines 10a
and 12a)................______

(b) If Line 4 is <u>Less</u> than Line 7:

8b. Line 1 x Line 4 = ...______
9b. Line 7 x Line 7 = ...______
10b. Line 8b ÷ Line 9b...........______
11b. Transmission Coefficient
(Table 4..................______
12b. Line 11b x Line 1 = ...______
13b. Line 2 - Line 6 = ...______
14b. Line 13b x .78 = ...______
15b. Leaving Wave Height
(Smaller of Lines 12b and
14b).....................______

SAMPLE CALCULATIONS

NAME JANE SMITH

DATA FORM FOR BEGINNING WAVE HEIGHT

Open Coast

1. Base Flood Elevation......... 12.5'
2. Beginning Wave Height:
   (Line 1 x .78).............. 9.75'

Bays, Estuaries, etc.

1. Base Flood Elevation.............. ______
2. Width of Water Body............... ______
3. Fetch Factor (From Table 1)....... ______
4. Beginning Wave Height
   (Line 1 x Line 3)................ ______

DATA FORM FOR FETCH ON PROTECTED COAST

1. Arriving Wave Height......... 2.24'
2. Base Flood Elevation......... 12.5'
3. Width of Water Body.......... .47
4. Line 1 ÷ Line 2 =................. .2
5. Fetch Factor (From Table 5)....... .35
6. Leaving Wave Height
   (Line 2 x Line 5) = ... 4.38'

DATA FORM FOR WAVE HEIGHT AT SITE

1. Arriving Wave Height......... .12'
2. Base Flood Elevation......... 12.5'
3. Site Elevation............... 7.0'
4. Line 2 - Line 3 = ... 5.5'
5. Line 4 x .78 = ... 4.3'
6. Wave Height at Site
   (Smaller of Lines 1 and 5)....... .12'

DATA FORM FOR WAVE CREST ELEVATION AT SITE

1. Wave Height at Site.......... .12'
2. Base Flood Elevation......... 12.5'
3. Line 1 x 0.7 = ... .08
4. Wave Crest Elevation
   (Line 2 + Line 3) = ... 12.58'

NAME JANE SMITH

DATA FORMS FOR NATURAL LINEAR OBSTRUCTIONS

OBSTRUCTION NO. 2 DESCRIPTION SAND DUNE

1. Arriving Wave Height............4.25'
2. Base Flood Elevation............12.5'
3. Elevation of Obstruction........12'
4. Line 2 - Line 3 = ...5'
5. Line 4 x .78 = ...39'
6. Leaving Wave Height
   (Smaller of Lines 1 and 5).......39'

OBSTRUCTION NO.______ DESCRIPTION______

1. Arriving Wave Height............______
2. Base Flood Elevation............______
3. Elevation of Obstruction........______
4. Line 2 - Line 3 = ...______
5. Line 4 x .78 = ...______
6. Leaving Wave Height
   (Smaller of Lines 1 and 5).......______

OBSTRUCTION NO.______ DESCRIPTION______

1. Arriving Wave Height............______
2. Base Flood Elevation............______
3. Elevation of Obstruction........______
4. Line 2 - Line 3 = ...______
5. Line 4 x .78 = ...______
6. Leaving Wave Height
   (Smaller of Lines 1 and 5).......______

OBSTRUCTION NO.______ DESCRIPTION______

1. Arriving Wave Height............______
2. Base Flood Elevation............______
3. Elevation of Obstruction........______
4. Line 2 - Line 3 = ...______
5. Line 4 x .78 = ...______
6. Leaving Wave Height
   (Smaller of Lines 1 and 5).......______

OBSTRUCTION NO.______ DESCRIPTION______

1. Arriving Wave Height............______
2. Base Flood Elevation............______
3. Elevation of Obstruction........______
4. Line 2 - Line 3 = ...______
5. Line 4 x .78 = ...______
6. Leaving Wave Height
   (Smaller of Lines 1 and 5).......______

NAME Jane Smith

DATA FORMS FOR DETACHED BUILDINGS

OBSTRUCTION NO. 3 DESCRIPTION Buildings

1. Arriving Wave Height............ .39'
2. Base Flood Elevation............ 12.5'
3. Open Space Per Row.............. 150'
4. Number of Rows.................. 2
5. Elevation, Leeward Edge......... 5.5'
6. Line 3 ÷ 500.................... .3
7. Transmission Coefficient (From Table 3).................. .30
8. Line 1 x Line 7 = ... .12'
9. Line 2 - Line 5 = ... 7.0'
10. Line 9 x .78 = ... 5.5'
11. Leaving Wave Height (Smaller of Lines 8 and 10).............. .12'

OBSTRUCTION NO.______ DESCRIPTION______

1. Arriving Wave Height............______
2. Base Flood Elevation............______
3. Open Space Per Row..............______
4. Number of Rows..................______
5. Elevation, Leeward Edge.........______
6. Line 3 ÷ 500....................______
7. Transmission Coefficient (From Table 3)..................______
8. Line 1 x Line 7 = ...______
9. Line 2 - Line 5 = ...______
10. Line 9 x .78 = ...______
11. Leaving Wave Height (Smaller of Lines 8 and 10)..............______

OBSTRUCTION NO.______ DESCRIPTION______

1. Arriving Wave Height............______
2. Base Flood Elevation............______
3. Open Space Per Row..............______
4. Number of Rows..................______
5. Elevation, Leeward Edge.........______
6. Line 3 ÷ 500....................______
7. Transmission Coefficient (From Table 3)..................______
8. Line 1 x Line 7 = ...______
9. Line 2 - Line 5 = ...______
10. Line 9 x .78 = ...______
11. Leaving Wave Height (Smaller of Lines 8 and 10)..............______

OBSTRUCTION NO.______ DESCRIPTION______

1. Arriving Wave Height............______
2. Base Flood Elevation............______
3. Open Space Per Row..............______
4. Number of Rows..................______
5. Elevation, Leeward Edge.........______
6. Line 3 ÷ 500....................______
7. Transmission Coefficient (From Table 3)..................______
8. Line 1 x Line 7 = ...______
9. Line 2 - Line 5 = ...______
10. Line 9 x .78 = ...______
11. Leaving Wave Height (Smaller of Lines 8 and 10)..............______

NAME Jane Smith

DATA FORMS FOR MAN-MADE LINEAR OBSTRUCTIONS

OBSTRUCTION NO.______ DESCRIPTION Seawall

1. Arriving Wave Height............ 9.75'
2. Base Flood Elevation............ 12.5'
3. Elevation of Obstruction........... 7.0'
4. Elevation, Leeward Edge............ 5.0'

(a) If Line 3 is <u>Less</u> than Line 2:

5a. Line 2 - Line 3 = ... 5.5'
6a. Line 5a ÷ Line 1 = ... .56
7a. Transmission Coefficient.. .71
8a. Line 1 x Line 7a = ... 6.9'
9a. Line 2 - Line 4 = ... 7.5'
10a. Line 9a x .78 = ... 5.9'
11a. Leaving Wave Height (Smaller of Lines 8a and 10a)................ 5.9'

(b) If Line 3 is <u>More</u> than Line 2:

5b. Line 3 - Line 2 = ...______
6b. Line 5b ÷ Line 1 = ...______
7b. 0.5 - Line 6B = ...______
8b. Line 1 x Line 7b = ...______
9b. Line 2 - Line 4 = ...______
10b. Line 9b x .78 = ...______
11b. Leaving Wave Height (Smaller of Lines 8b and 10b)..................______

OBSTRUCTION NO.______ DESCRIPTION Seawall

1. Arriving Wave Height............ 9.75
2. Base Flood Elevation............ 12.5
3. Elevation of Obstruction........... 14'
4. Elevation, Leeward Edge............ 5.0'

(a) If Line 3 is <u>Less</u> than Line 2:

5a. Line 2 - Line 3 = ...______
6a. Line 5a ÷ Line 1 = ...______
7a. Transmission Coefficient..______
8a. Line 1 x Line 7a = ...______
9a. Line 2 - Line 4 = ...______
10a. Line 9a x .78 = ...______
11a. Leaving Wave Height (Smaller of Lines 8a and 10a)................______

(b) If Line 3 is <u>More</u> than Line 2:

5b. Line 3 - Line 2 = ... 1.5'
6b. Line 5b ÷ Line 1 = ... .15
7b. 0.5 - Line 6B = ... .35
8b. Line 1 x Line 7b = ... 3.41'
9b. Line 2 - Line 4 = ... 7.5
10b. Line 9b x .78 = ... 5.85'
11b. Leaving Wave Height (Smaller of Lines 8b and 10b).................. 3.41'

NAME JANE SMITH

DATA FORMS FOR VEGETATION OBSTRUCTIONS

OBSTRUCTION NO. 1 DESCRIPTION VEGETATION - MANGROVE

1. Arriving Wave Height........... 9.75'
2. Base Flood Elevation........... 12.5'
3. Elevation, Vegetated Area....... 6'
4. Vegetation Height.............. 5'

(a) If Line 4 is More than Line 7:

8a. Line 1 ÷ Line 7 = ... ______
9a. Transmission Coefficient (Table 4)................ ______
10a. Line 1 x Line 9a = ... ______
11a. Line 2 - Line 6 = ... ______
12a. Line 11a x .78 = ... ______
13a. Leaving Wave Height (Smaller of Lines 10a and 12a)................ ______

5. Vegetation Width.................. 250'
6. Elevation, Leeward Edge........... 5'
7. Line 2 - Line 3 = ... 6.5'

(b) If Line 4 is Less than Line 7:

8b. Line 1 x Line 4 = ... 48.75 ft^2
9b. Line 7 x Line 7 = ... 42.25 ft^2
10b. Line 8b ÷ Line 9b........... 1.15
11b. Transmission Coefficient (Table 4.................. .23
12b. Line 11b x Line 1 = ... 2.24'
13b. Line 2 - Line 6 = ... 7.5'
14b. Line 13b x .78 = ... 5.85'
15b. Leaving Wave Height (Smaller of Lines 12b and 14b)..................... 2.24'

OBSTRUCTION NO. 4 DESCRIPTION VEGETATION - PINE FOREST

1. Arriving Wave Height........... .12'
2. Base Flood Elevation........... 12.5'
3. Elevation, Vegetated Area....... 6'
4. Vegetation Height.............. 25'

(a) If Line 4 is More than Line 7:

8a. Line 1 ÷ Line 7 = ... .02
9a. Transmission Coefficient (Table 4)................ .98
10a. Line 1 x Line 9a = ... .12'
11a. Line 2 - Line 6 = ... 6.0'
12a. Line 11a x .78 = ... 4.7'
13a. Leaving Wave Height (Smaller of Lines 10a and 12a)................ .12'

5. Vegetation Width.................. 500'
6. Elevation, Leeward Edge........... 6.5'
7. Line 2 - Line 3 = ... 6.5'

(b) If Line 4 is Less than Line 7:

8b. Line 1 x Line 4 = ... ______
9b. Line 7 x Line 7 = ... ______
10b. Line 8b ÷ Line 9b........... ______
11b. Transmission Coefficient (Table 4.................. ______
12b. Line 11b x Line 1 = ... ______
13b. Line 2 - Line 6 = ... ______
14b. Line 13b x .78 = ... ______
15b. Leaving Wave Height (Smaller of Lines 12b and 14b)..................... ______

SECTION 2