EVALUATION OF POSSIBLE BIAS
IN THE STORM SURGE DATA BASE DUE TO WAVE EFFECTS

by

Robert G. Dean

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DEPARTMENT OF CIVIL ENGINEERING
UNIVERSITY OF DELAWARE
NEWARK, DELAWARE
19711

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I. INTRODUCTION

Relatively few tide gage records exist along the outer shoreline for the calibration of numerical models for storm surges in the 100 year return period class. This is somewhat understandable in view of the infrequent nature of these storms, the relative narrow width of the fields of storm surge maxima and the forces which these storms can bring to bear on the stilling wells and their supporting structures (such as piers of "opportunity"). Moreover, in many cases, the primary purpose of a tide gage installation is for an improved understanding of the astronomical tidal component which requires a far shorter stilling well and less expensive system than one designed to record through and survive the elements accompanying a severe hurricane or northeaster. On the other hand, in view of the economics associated with elevated first floors of coastal structures and the associated hazard to life, it is somewhat surprising that a concerted program has not been implemented to measure storm tide elevations associated with extreme storms.

The legislation for federally subsidized insurance for coastal flooding damage requires that the lower floor be elevated above the 100-year storm surge level. Additionally, consideration of the degree to which wave effects should be included in the requirements is presently underway. Because a considerable portion of the data base for calibration of storm surge models is based on high water marks due to the relative sparsity of tide gage measurements, it is therefore relevant to examine the degree to which high water marks already include wave effects.

There are a number of known mechanisms that could account for both the incorporation of wave effects in high water marks and the complex variation of mean water level across the surf zone. A partial list of such effects includes: (1) the "pumping up" of still water levels through apertures in the structure, such as window sills and vertical gaps occurring, for example, between a door and the casing, (2) the direct effects of rainfall through structures that have lost their roofs, (3) the wave set-up (a radiation stress effect) across the surf zone, (4) the obvious direct contribution due to the wave crest if the high water mark is outside the structure, and (5) the effects of bottom shear stress and forces on vegetation or structures in reducing the wave set-up. Finally, it is noted that there is a nonlinear effect that could reduce the mean water level inside a tide gage stilling well below that outside the stilling well.

II. OBJECTIVES

The objectives of this study are to identify and describe possible effects in the data base currently available for the calibration of storm surge numerical models, to attempt to quantify the magnitudes of these effects in cases where both high water marks and tide gage records are available, and finally to develop an assessment of the effect in the current data base and to develop recommendations resulting from this assessment.

III. MECHANISMS CONTRIBUTING TO HIGH WATER MARKS AND NEARSHORE SURGE LEVEL VARIATIONS

Introduction

Due to the common meteorological cause, large waves nearly always accompany high storm tides. As these waves propagate across the shelf, energy is lost due to bottom friction and percolation and wave breaking. Inside the breaker zone, defined where the wave heights are approximately 78% of their depth, the wave energy dissipation is more rapid, resulting in a wave "set-up" across this zone. In addition, the interaction of the waves with structures can cause a bias. The following subsection identifies, describes and formulates the various possible mechanisms which could contribute to a bias in high water marks.

Formulation of Mechanisms

For purposes of later discussion, Figure 1 presents a transect across a flooded section of the shoreline and the associated still water level. Each of the mechanisms affecting the variation of this still water level inside the breaking zone and the deviation within the stilling well and inside structures is discussed below.

Variation of Still Water Level Across the Breaker Zone - Waves propagating from a generating area transport both energy and momentum. The momentum flux is a direct result of both shear stress and pressure acting on the waves within the generating area. The spatial variation of momentum flux as the waves propagate toward shore causes an associated variation in still water level which should be included explicitly in the storm surge predictions. For the simplest case of waves propagating directly toward shore, and the only energy loss due to wave breaking inside a relatively narrow surf zone, the associated effect is a set-down outside the surf zone and a set-up inside the surf zone. Denoting $\bar{\eta}$ as the deviation from a horizontal plane passing through the unaffected still water level, for this simple case, the effect (set-down) outside the surf zone is given by (Longuet-Higgins and Stewart (1964))

$$\bar{\eta} = -\frac{1}{8} H_o^2 k_o \frac{\coth^2 kh}{2kh + \sinh 2kh} \tag{1}$$

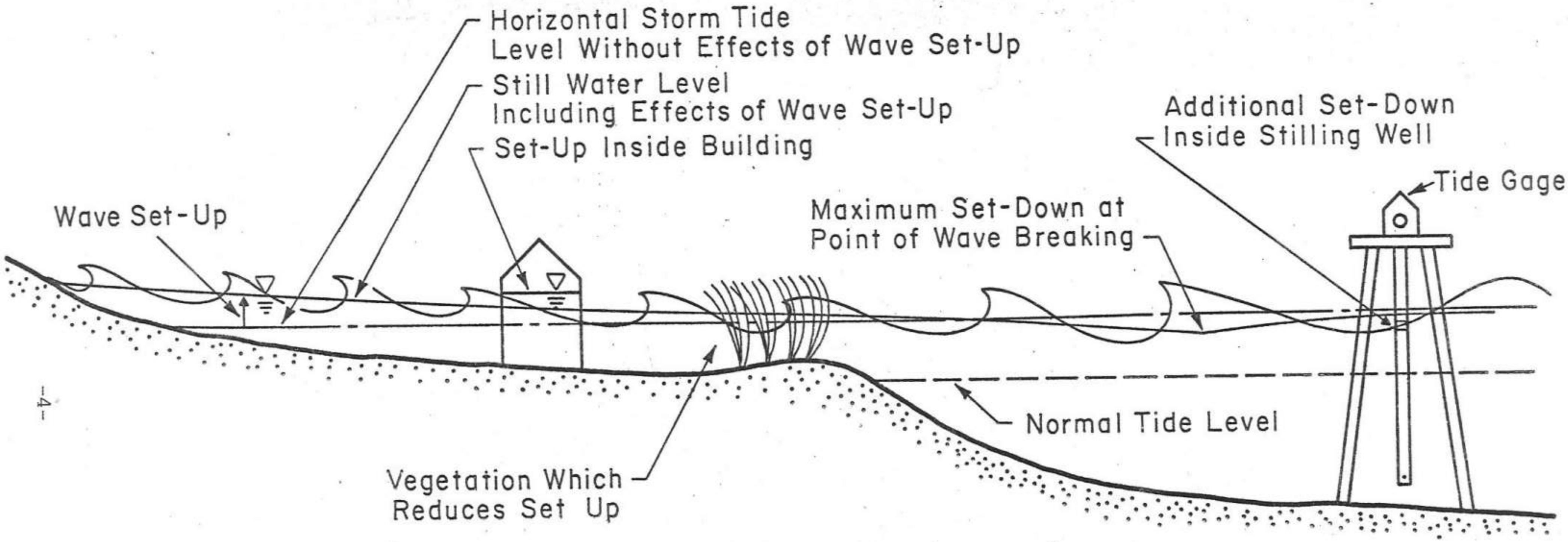


FIGURE 1. DEFINITION SKETCH ILLUSTRATING VARIOUS COMPONENTS OF WAVE SET-DOWN AND SET-UP ACROSS THE BREAKING ZONE AND INSIDE STRUCTURES.

in which H_o and k_o are the deep water wave height and wave number respectively, k is the local wave number and h is the local water depth. The radiation stress is a second-order quantity in the wave height, H. As the wave breaks, it is no longer able to transport the momentum and thus it is transferred to a mean water level gradient or set-up given approximately by

$$\bar{\eta} = \bar{\eta}_b - \frac{1}{(1 + \frac{8}{3\kappa^2})} (h - h_b) \tag{2}$$

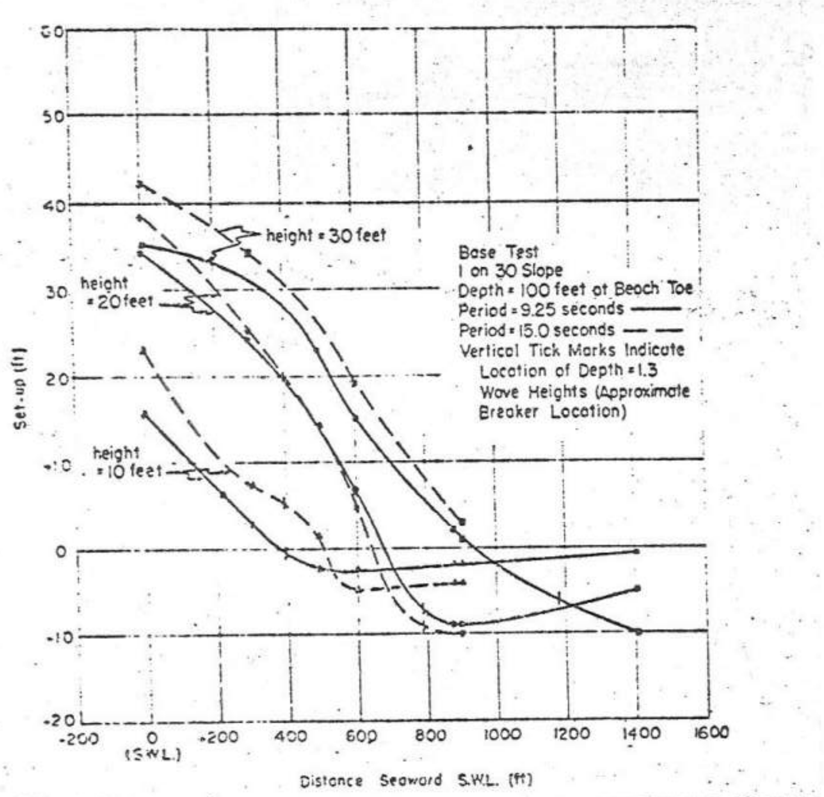
in which $\bar{\eta}_b$ is the maximum set-down at breaking as given by Eq. (1) and κ is the ratio of breaking wave height to depth ($\kappa \cong 0.78$).

Examples of the set-down and set-up in the field are provided by the measurements of Saville (1961) as presented in Figure 2a and in the laboratory measurements of Bowen, Inman and Simmons (1968) as shown in Figure 2b. Data from the field are also provided by the tide gage anomalies from the lowe east coast of Florida during the large March 1962 northeaster, see Figure 3. During this period, the waves off southern Florida were reported to be 15-20 ft. in height and the dominant winds were mild and directed offshore.

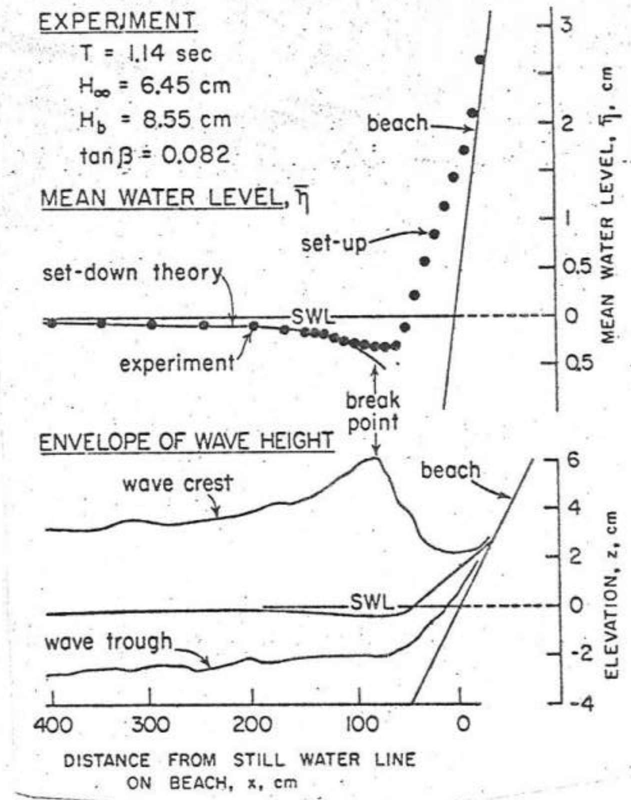
In a more general and realistic case, wave set-up can occur due to a number of causes other than wave breaking. In particular, net bottom shear stresses and or net forces on vegetative elements can contribute to the wave set-up. Denoting F as the average force on the water column per unit plan area, the relationship is

$$\frac{\partial \bar{\eta}}{\partial x} = - \frac{1}{\rho g (h + \bar{\eta})} [\frac{\partial S_{xx}}{\partial x} - F(x)] \tag{3}$$

which, in general, must be solved numerically. In Eq. (3), S_{xx} is the momentum



2-a) Wave Set-Down and Set-Up in the Field as Reported by Saville (1961).



2-b) Wave Set-Down and Set-Up in the Laboratory as Reported by Bowen, Inman and Simmons (1965).

FIGURE 2. EXAMPLES OF WAVE SET-DOWN AND SET-UP IN THE FIELD AND LABORATORY.

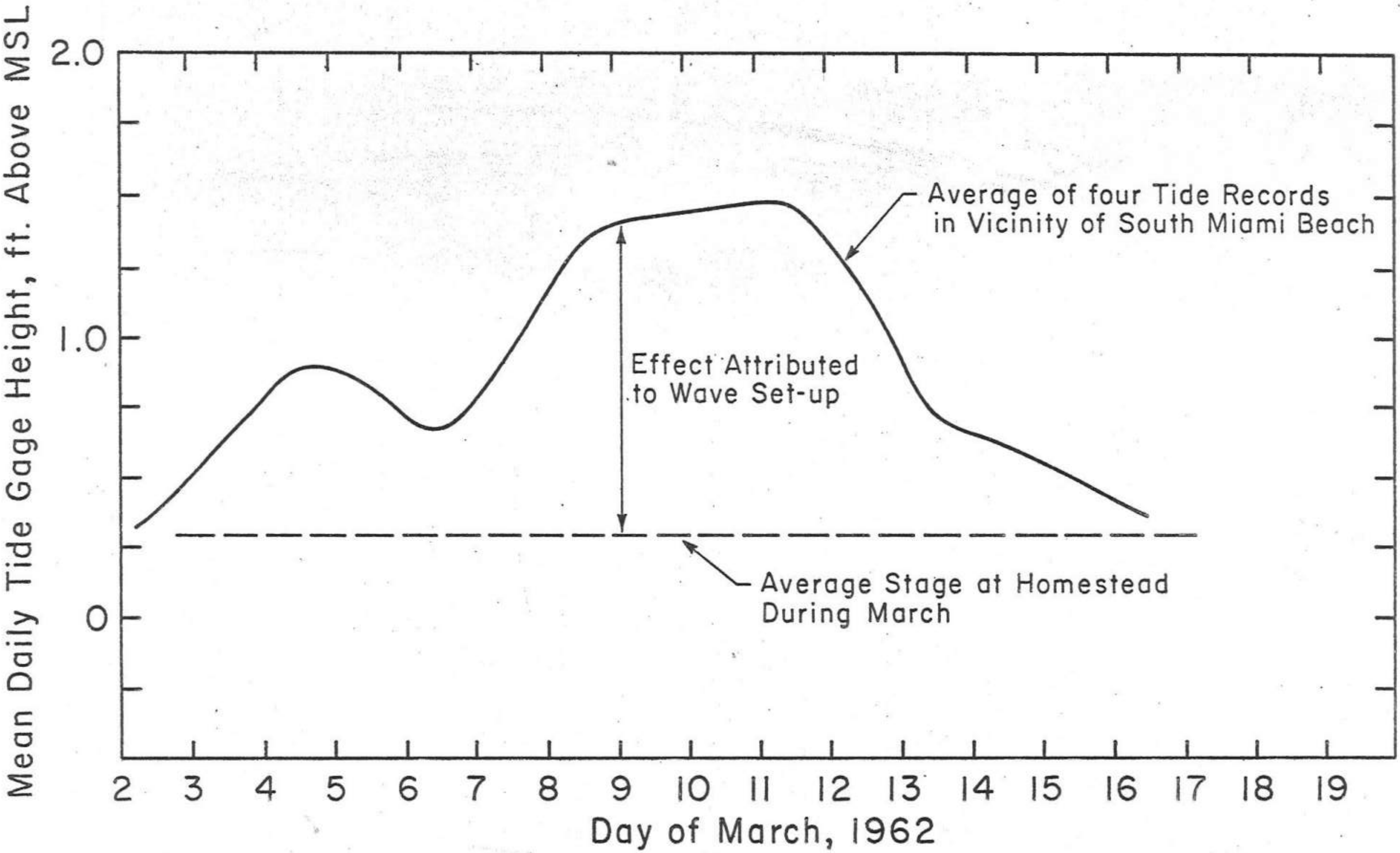


FIGURE 3. FIELD DATA ILLUSTRATING WAVE SET-UP ALONG LOWER EAST COAST OF FLORIDA DUE TO MARCH, 1962 STORM, (Adapted From Lazarus and Nowlin, 1966).

flux and for waves propagating directly toward shore, is given by

$$S_{xx} = E[\frac{1}{2} + \frac{2kh}{\sinh 2kh}] \tag{4}$$

where E represents the average wave energy per unit surface area, i.e.

$$E = \rho g \frac{H^2}{8} \tag{5}$$

As noted previously, the anomalies noted here as $\bar{\eta}$ are real effects on the still water level and should not be considered as extraneous to the storm tide problem. Examples will be presented later demonstrating the magnitudes of these effects.

Anomalous Effects on High Water Marks Inside a Structure

There are at least four possible causes of anomalously high water marks inside a building. These include: (1) set-up due to flows through vertical slits, such as between doors, (2) set-up due to wave reflection, (3) flows over such features as window sills, and (4) direct rainfall. Each of these effects is discussed and formulated in the following paragraphs.

(1) Set-up Due to Vertical Slits - If a vertical slit is present, then flow into the structure will occur during the presence of the wave crest and outflow will occur during the presence of the trough. It is clear on intuitive grounds that a net set-up of water inside the building will occur as the inflow area to first order is: depth plus wave amplitude and the outflow area is: depth minus wave amplitude. This phenomenon is formulated in Appendix I and it is found that the nonlinear form is not readily solvable. Instead a linear version is developed which yields approximately

$$\bar{z}_1 = \frac{H^2}{16h} \tag{6}$$

in which $\overline{z}_1$ represents the set-up. Thus, it appears that this effect would not be very important as a wave height of 80% of the depth would only cause a set-up of 5% of the wave height. As an example, a wave of 8 ft. height in 10 ft. of water would only yield a 0.4 ft. set-up. It is noted that wave reflection from the building could yield effects not accounted for here which are believed to be sufficiently large to increase the above estimate of set-up to on the order of one foot. If a standing wave height equal to twice the incident height is considered, the set-up determined from Eq. (6) would be 1.6 ft.

(2) <u>Set-up Due to Wave Reflection</u> - Waves reflecting from a vertical wall cause a second-order set-up at the wall, $\overline{z}_2$, given, for shallow water conditions, by

$$\overline{z}_2 = \frac{H^2}{8h} \qquad (7)$$

which is just twice the value given by Eq. (6) and thus could readily amount to approximately one foot for the previous example.

(3) <u>Flows Over Horizontal Sills</u> - Consider the case of a window sill at an elevation below the wave crest level. It is clear that there would be more inflow area than outflow area and thus in order to achieve a steady state, there must be a set-up inside the building. In fact, in the absence of other outflow areas, the mean set-up must always be above the elevation of the sill. Appendix II presents an approximate treatment for the case in which the sill is above the wave trough level and the inflow velocities are approximated by the shallow water wave equations and the outflow velocities by the sharp-crested weir equation with critical depth occurring over the sill. Figure 4 presents the results in dimensionless form.

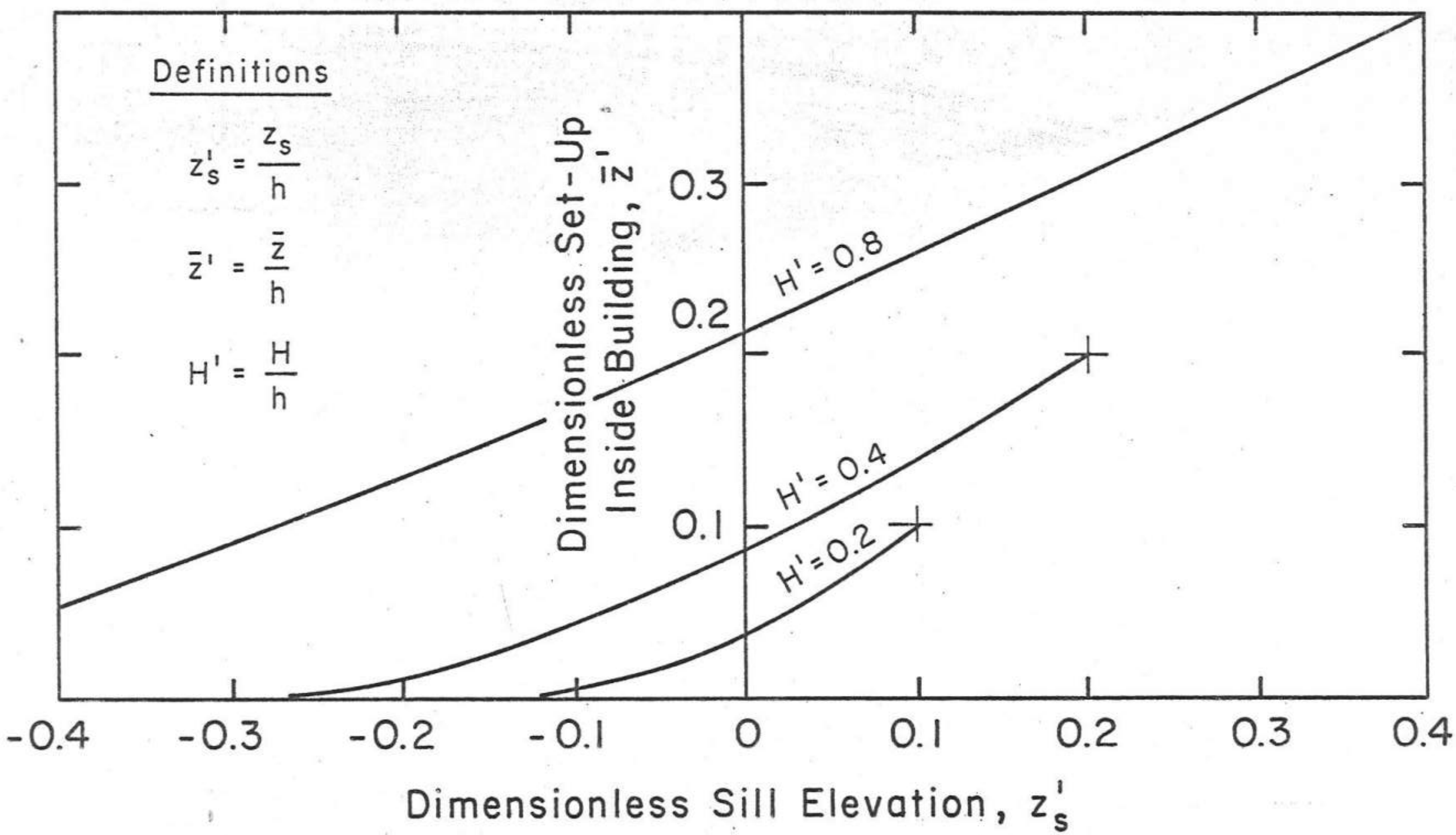


FIGURE 4. DIMENSIONLESS SET-UP, $\bar{z}'$, INSIDE BUILDING AS AFFECTED BY DIMENSIONLESS SILL ELEVATION, z'_s, AND DIMENSIONLESS WAVE HEIGHT, H' (See Figure II-1 for Definitions of Terms).

(4) Direct Effect of Rainfall Through Damaged Roofs - There is the possibility that the direct rainfall accompanying hurricanes could cause an increased water level inside structures if the roof was open to the precipitation. A review of the extremes of rainfall occurring during a 24 hour period (Jennings, 1952) indicate that for the states affected by hurricanes, the maximum of 21.4 inches per 24 hour period occurred in Louisiana in the month of June. Even if this rate were doubled to approximately 2 inches per hour, it would appear to be relatively unimportant compared to other mechanisms identified.

(5) Reduction of Water Level Inside a Tide Gage Stilling Well - Progressive water waves cause a reduction in the mean pressure at a given depth. This effect would cause the still water level inside a stilling well to be below that outside by, $\overline{z}$, expressed as

$$\overline{z} = - \frac{\overline{w^2}}{g} \tag{8}$$

in which g is the gravitational constant, $\overline{w^2}$ is the mean square of the vertical velocity component of the waves. According to small amplitude wave theory,

$$\overline{w^2} = \frac{H^2}{8} \sigma^2 \left(\frac{S}{h}\right)^2$$

where σ is the wave angular frequency (= 2π/wave period), and S is the distance above the sea bottom of the orifice through which water flows into and out of the stilling well.

IV. SAMPLE COMPUTATIONS OF MECHANISMS WHICH COULD BIAS HIGH WATER MARKS

Introduction

In the example computations, the following wave characteristics will be considered

	Cases (a), (b), (c)	Case (d)
Breaking Height:	H_b = 8 ft.	H_b = 16 ft.
Breaking Depth:	h_b = 10 ft.	h_b = 20 ft.
Wave Period:	T = 10 sec.	T = 10 sec.

along with the profile in Figure 5. Equations presented in the preceeding section are used for the example computations.

Variation of Still Water Level Across the Surf Zone

Several examples will be computed:

Case (a) - Wave set-down and set-up are computed for $H = 0.78\ h$ across the breaking zone and no vegetative or bottom shear stress forces are considered. The initial set-down is $\overline{\eta} = -0.4$ ft. and the maximum set-up is approximately 2.0 ft. The results are plotted as Curve a) in Figure 5.

Case (b) - Conditions are the same as for Case (a) except the effect of a net bottom shear stress was included. Based on Case 2-D of the Stream Function Tables (Dean, 1974), the average bottom shear stress which is directed in an offshore direction on the water column, is

$$\tau = \frac{-\rho f}{2}(10.18)\left(\frac{H}{T}\right)^2 \qquad (10)$$

Using a friction coefficient value of 0.02, the set-up was computed by the following finite difference equation

$$\overline{\eta}^{j+1} = \overline{\eta}^{j} + \Delta x[1.86 \times 10^{-3} - 1.564 \times 10^{-5}(h^{j} + \overline{\eta}^{j})] \qquad (11)$$

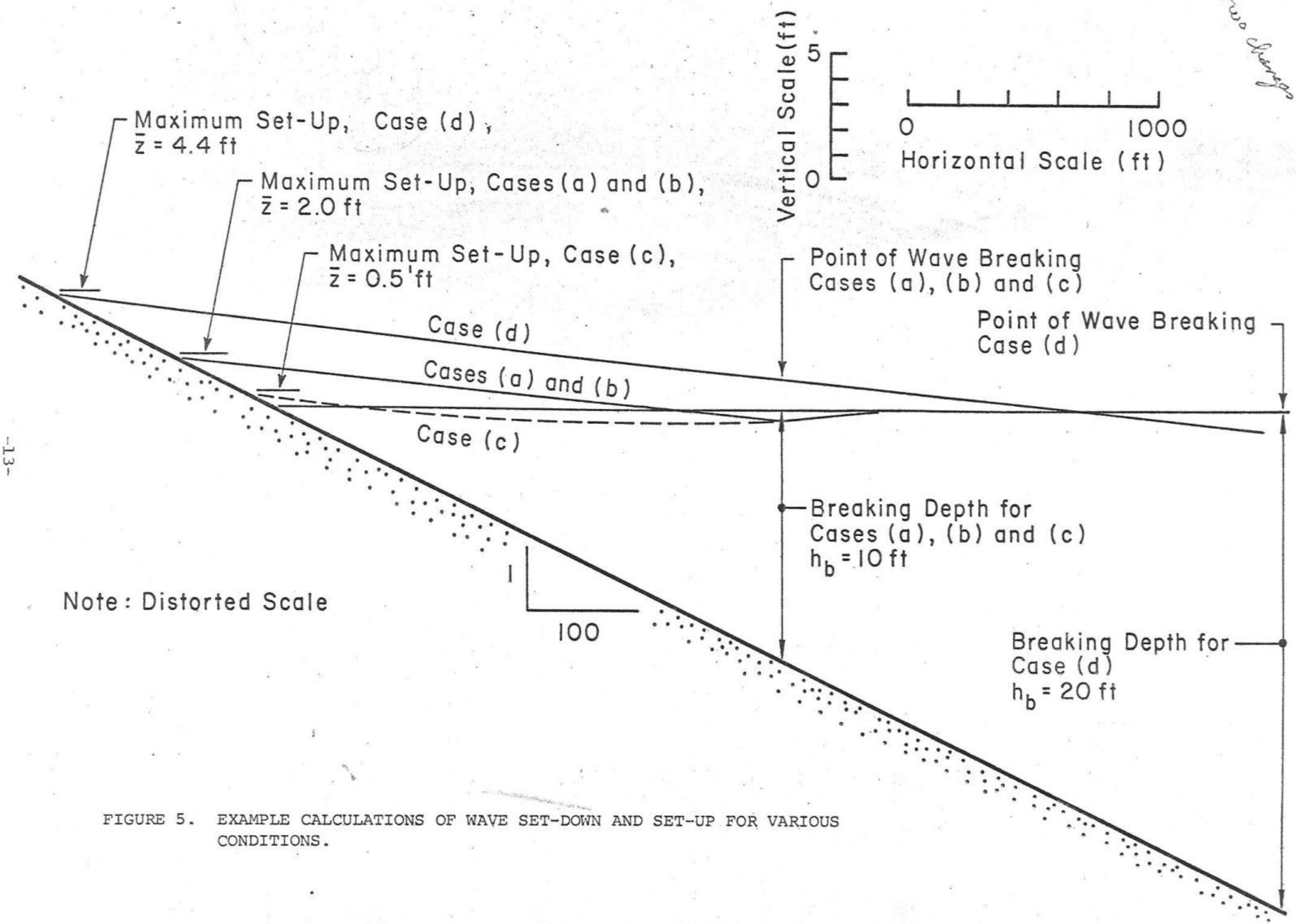


FIGURE 5. EXAMPLE CALCULATIONS OF WAVE SET-DOWN AND SET-UP FOR VARIOUS CONDITIONS.

in which the j superscript denotes the j^{th} grid and $\Delta x = x^{j+1} - x^j$ was taken as 50 ft. It was found that the resulting set-up (Curve b) agreed within 0.1 ft. of that for Case a).

Case (c) - The wave conditions were the same as for Cases (a) and (b). Landward of the breaker line, vegetation, consisting of a two inch diameter tree trunk is considered to be present on a spacing of 4 ft. center-to-center. Based on Case 2-D of the Stream Function tables and retaining all nonlinear effects, this led to an average drag force per unit plan area, $\overline{F}$, of

$$\overline{F} = -0.38 \frac{H^3}{T^2} \tag{12}$$

and the finite difference equation is given by

$$\overline{\eta}^{j+1} = \overline{\eta}^j + \Delta x[1.86 \times 10^{-3} - 2.83 \times 10^{-5}(h^j + \overline{\eta}^j)^2] \tag{13}$$

In this case, the maximum set-down, $\overline{\eta}$, is -0.56 ft. and the maximum set-up is 0.5 ft., a considerable reduction from the value of 2.0 ft.

Case (d) - The final case considered was for a wave of twice the breaking height and depth as for Cases (a), (b) and (c) and for no vegetative effects. The maximum set-down is 0.8 ft. and the maximum set-up is 4.4 ft.

Set-Up Inside a Building

Set-Up Due to Flow Through a Vertical Slit - Consider a wave height and water depth of 8 and 10 ft. respectively. The set-up, $\overline{z}$, inside a building is expressed by Eq. (6)

$$\overline{z} = \frac{H^2}{16h} = \frac{(8)^2}{(16)(10)} = 0.4 \text{ ft.}$$

<u>Set-Up Due to Wave Reflection</u> - For the same conditions as in the preceding example, the set-up as given by Eq. (7) is

$$\bar{z} = \frac{H^2}{8h} = \frac{(8)^2}{(8)(10)} = 0.8 \text{ ft.}$$

<u>Set-Up Due to Flow Over a Sill</u> - Consider the same wave conditions as in the preceding examples and a window sill at an elevation of 13 ft. above ground level. The set-up as determined from Figure 4 is

$$\bar{z} = 3.5 \text{ ft.}$$

<u>Set-Up Due to Rainfall</u> - As noted previously, the maximum rainfall rate is on the order of two inches per hour and thus the set-up should be reasonably small compared to other effects discussed.

<u>Set-Down Inside a Stilling Well</u> - Suppose that the stilling well orifice through which flow occurs is at an elevation $S = 6$ ft. above the bottom in a total water depth $h = 20$ ft. and that the wave height, $H = 16$ ft., and the wave period $T = 10$ sec., then according to Eqs. (8) and (9)

$$\bar{z} = -\frac{H^2}{8g}\left(\frac{2\pi}{T}\right)^2\left(\frac{S}{h}\right)^2 = -\frac{(16)^2}{8(32.2)}\left(\frac{2\pi}{10}\right)^2\left(\frac{6}{20}\right)^2 = -0.035 \text{ ft.}$$

which is negligibly small in comparison to other effects.

V. EVALUATION OF WAVE EFFECTS IN EXISTING STORM SURGE DATA BASE

Introduction

The preceding sections have demonstrated that there are several mechanisms that could result in the contribution of wave effects to high water marks. Some of these effects, such as wave set-up, within the breaking zone, are valid contributors to the storm tide values, provided they are applied at the location of their occurrences. Others are anomalous such as demonstrated for the set-up inside a building due to wave-induced flow over a sill.

In this section some aspects of the available data base will be examined in an attempt to establish whether or not wave effects are already incorporated to some degree. Where possible, the focus will be on a comparison between high water marks and tide gages in close proximity to the locations where the high water marks were determined. By far the greatest source of storm surge data is in the form of high water marks, not storm surge data. In 1972, C. P. Jelesniaski assembled data on "observed peak surges" to compare with the predictions of his SPLASH numerical model. In describing these peak surge data, Jelesniaski noted (p. 27):

> "The data in Table 3 leave much to be desired. Although some of the peak surge values came from tide gage records, most were from post-storm surveys of high water marks. In general, it was not possible to take the stage of the astronomical tide into consideration in evaluating the effects of the storm. It is unlikely that the actual peak surge would coincide with one of the small number of high water marks measured. . ."

This comment by Jelesniaski in 1972 presents a fairly accurate portrayal of the storm surge data base today (1979).

Individual Storms

An attempt was made to locate high water marks and tide gage data associated with particular storms. In this section, comparisons are presented from five individual storms. It is found that the quality of high water mark information depends strongly on those responsible for its collection, assessment and presentation. In some cases, the high water mark data were accepted with hardly any apparent comparison/adjustment with what appear to be quality tide gage data. In other cases, the close correspondence between the high water mark data and the available tide gage data demonstrates that the two sources were somehow merged to develop the final high water mark results.

Hurricane Alma, June 1966, Cedar Key, Florida - Hurricane Alma passed approximately 50 n. mi. to the west of Cedar Key, Florida, making landfall near St. Marks, Florida. The two data sources are the tide gage record and a high water mark reported in Ho and Tracey (1975). The tide gage at Cedar Key is located on the end of a pier and although there are several islands offshore, it is not believed that the tide gage would be influenced significantly by their presence. More relevant, of course, the high water mark was undoubtedly obtained landward from the tide gage and thus also would presumably include any sheltering, and wave set-up effects. Figure 6 presents the tide record as provided by the National Ocean Survey and modified to Mean Sea Level datum. It is seen that the maximum recorded tide, including the astronomical component is approximately 6.2 ft. whereas the high water mark reported by Ho and Tracey is approximately 10 ft. The location of the high water mark was not determined in the present study and it is possible that both the high water mark and the tide gage data are correct but were just measured at different positions along the profile. However, the recommended tidal stage vs. return period developed by Ho and Tracey relied on the high water mark data as shown in Figure 7. This figure is based on the

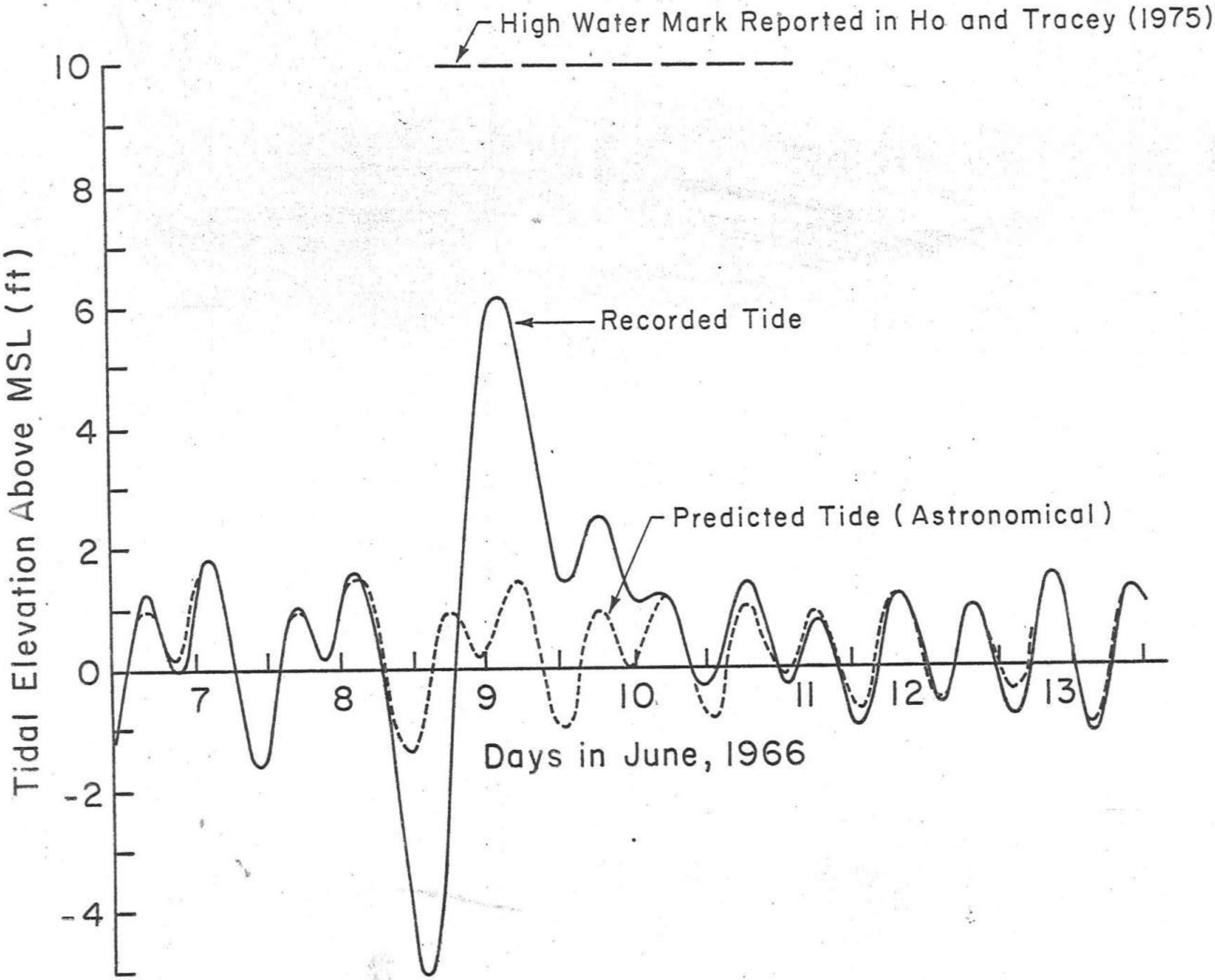


FIGURE 6. COMPARISON OF RECORDED AND PREDICTED TIDES AND REPORTED HIGH WATER MARK. CEDAR KEY, FLORIDA, HURRICANE ALMA, JUNE, 1966.

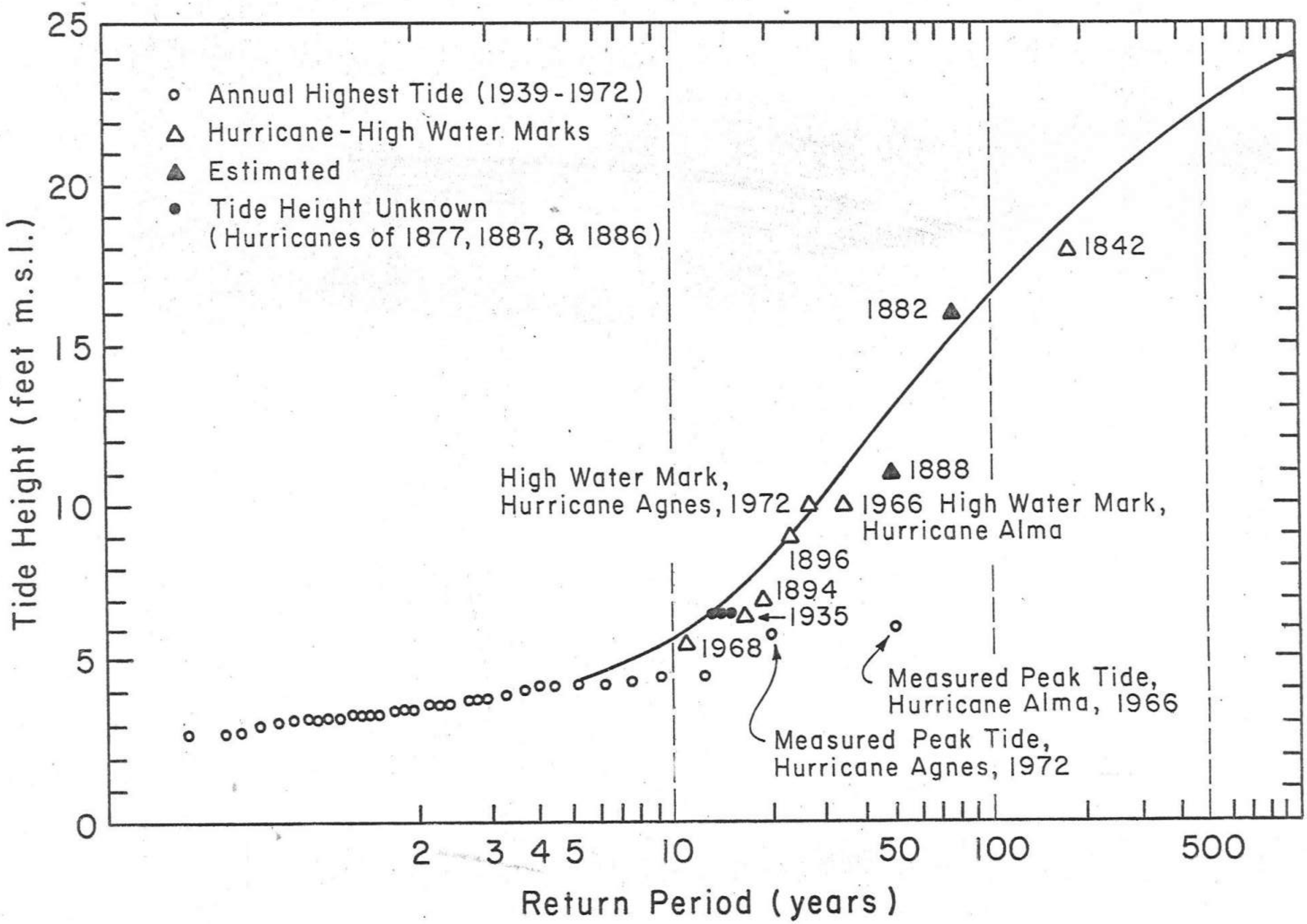


FIGURE 7. TIDE HEIGHT VS. RETURN PERIOD RELATIONSHIP AS RECOMMENDED BY HO AND TRACEY (1975). (Note Conformance of Recommended Curve to High Water Marks and Differences Between High Water Marks and Measured Peak Tides for Hurricanes Alma (1966) and Agnes (1972).

results of combining the storm surge nomographs developed by Jelesniaski with the historical statistics of the hurricane parameters. If it is assumed that the hurricane statistics are correct, it follows that the SPLASH model on which the storm surge nomographs are based appears to include a bias due to wave effects in the storm surge data base.

It is clear that wave effects have already been incorporated to an unknown degree in the recommended storm surges presented in Figure 7.

Hurricane Agnes, June 1972, Cedar Key, Florida - Hurricane Agnes passed approximately 130 n. mi. to the west of Cedar Key with landfall between Panama City and Port St. Joe, Florida. The types of data available are the same as in the previous example: a tide gage record and a single high water mark. The tide gage recording as obtained from the National Ocean Survey and adjusted to MSL datum is presented in Figure 8 which also shows the high water mark of approximately 10 ft. as reported by Ho and Tracey. The comments for these data are much the same as presented for Hurricane Alma. Both the high water mark and the tide gage recording could be correct if, for example, the high water mark was measured at a location where the waves had caused substantial set-up. On the other hand, it is also reasonably likely that the high water mark is contaminated by wave effects due to interaction with the structure. If the high water mark was obtained from near the limit of the inundation and where considerable wave set-up had occurred, it would be inappropriate to apply these results to a region near the normal shoreline where in a severe storm, set-up would not be nearly as substantial.

Hurricanes Alma and Agnes, Apalachicola Bay, Florida - It appears that high water marks have been weighted heavily in Ho and Myers' development of the recommended tide height vs. return period relationship as presented in Figure 9. The two highest tide gage levels are from a 1950 hurricane and Hurricane Agnes

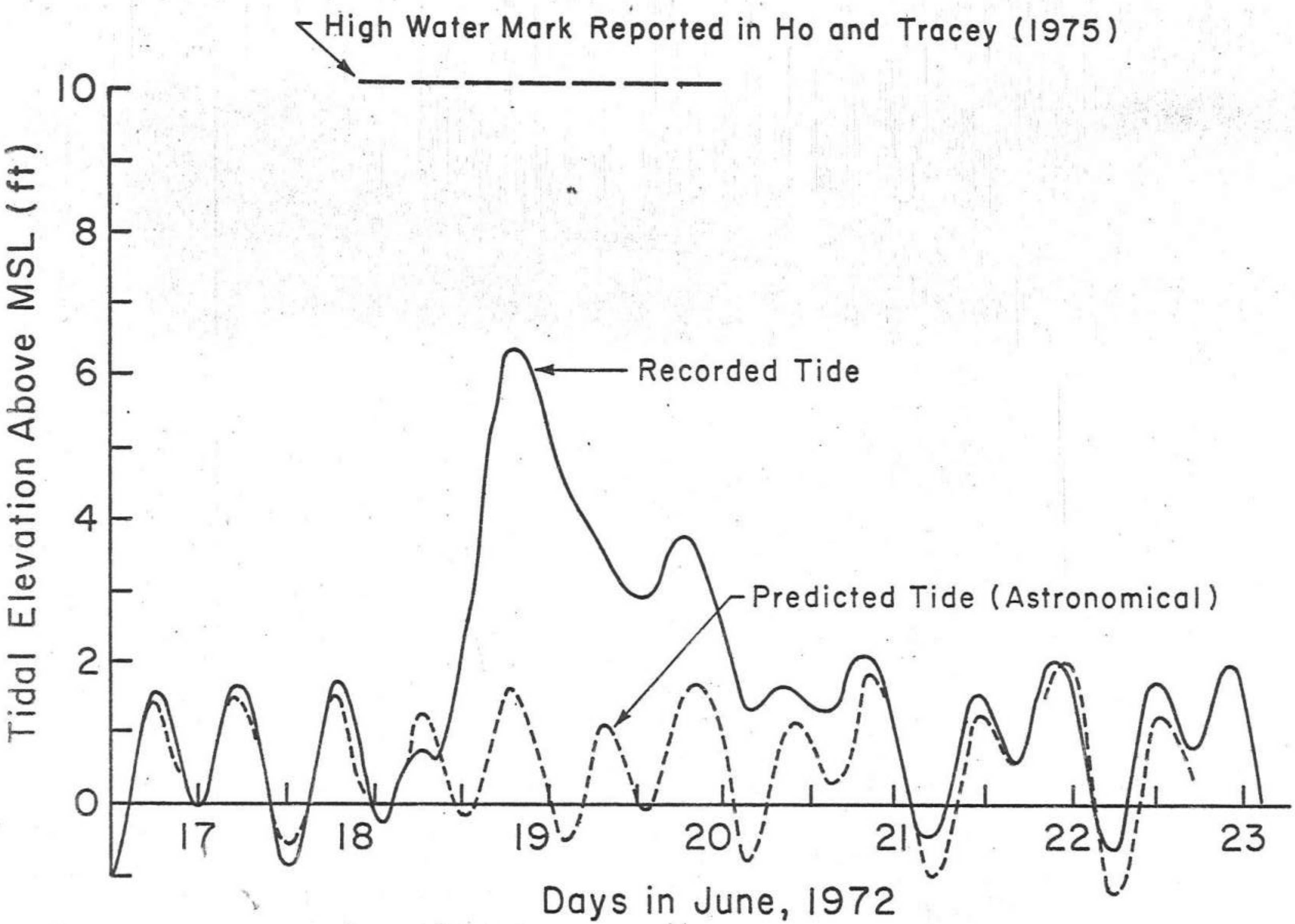


FIGURE 8. COMPARISON OF RECORDED AND PREDICTED TIDES AND REPORTED HIGH WATER MARK. CEDAR KEY, FLORIDA, HURRICANE AGNES, 1972.

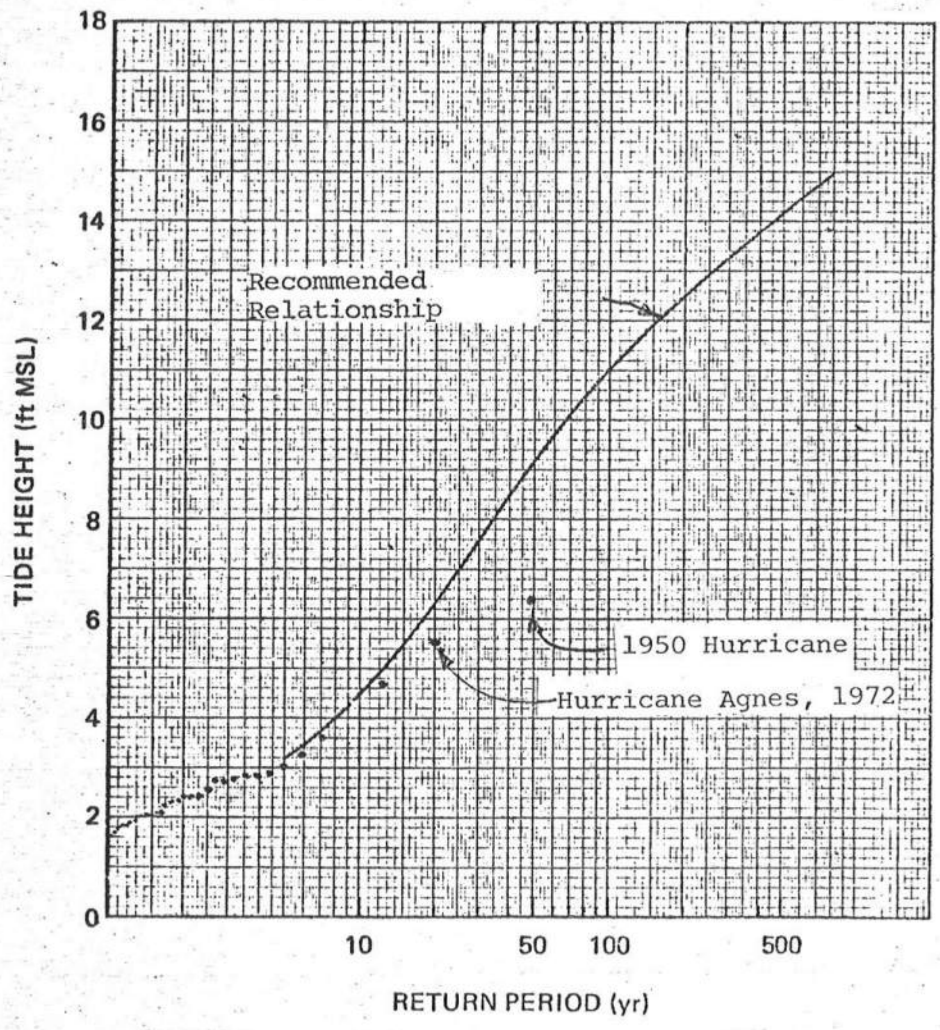


FIGURE 9. COMPARISON OF TIDE HEIGHT VS. RETURN PERIOD RELATIONSHIP AS RECOMMENDED BY HO AND MYERS (1975) WITH MEASURED PEAK TIDES AT APALACHICOLA, FLORIDA.

in 1972. At the 50 year return period, the recommended tide height is approximately 2.8 ft. higher than the data.

Hurricane Hazel, October 1954, North Carolina and South Carolina - Hurricane Hazel made landfall near the North Carolina-South Carolina border and proceeded inland passing to the west of Washington, D. C. and Baltimore. The data for this storm consist of high water marks as published by Myers (1975) and peak storm surges determined from tide gage recordings with the astronomical tidal component subtracted out as presented by Harris (1963). The high water marks are presented in Figure 10, in which the abscissa represents the distance in nautical miles from Myrtle Beach, S. C. The symbols distinguish whether the high water marks were determined along the coast or in bays and rivers. It is of interest that for this storm there seems to be little average difference for the data from the coast and inland locations. Also presented in Figure 8 are the peak storm surges as determined by Harris (1963) without the contributing effect of astronomical tides. In addition, an attempt has been made to establish a reasonable upper limit of the tide gage value including the effect of the astronomical tide. In particular, the symbol x represents the sum of the peak surge value (as determined by Harris) and the difference between one-half the spring tidal range and the mean tide level as determined from the tide tables.

Two comments are in order from Figure 10: (1) The agreement between high water marks along the coast and inland is surprisingly good, and (2) There appears to be a substantial contribution in the high water marks due to wave effects.

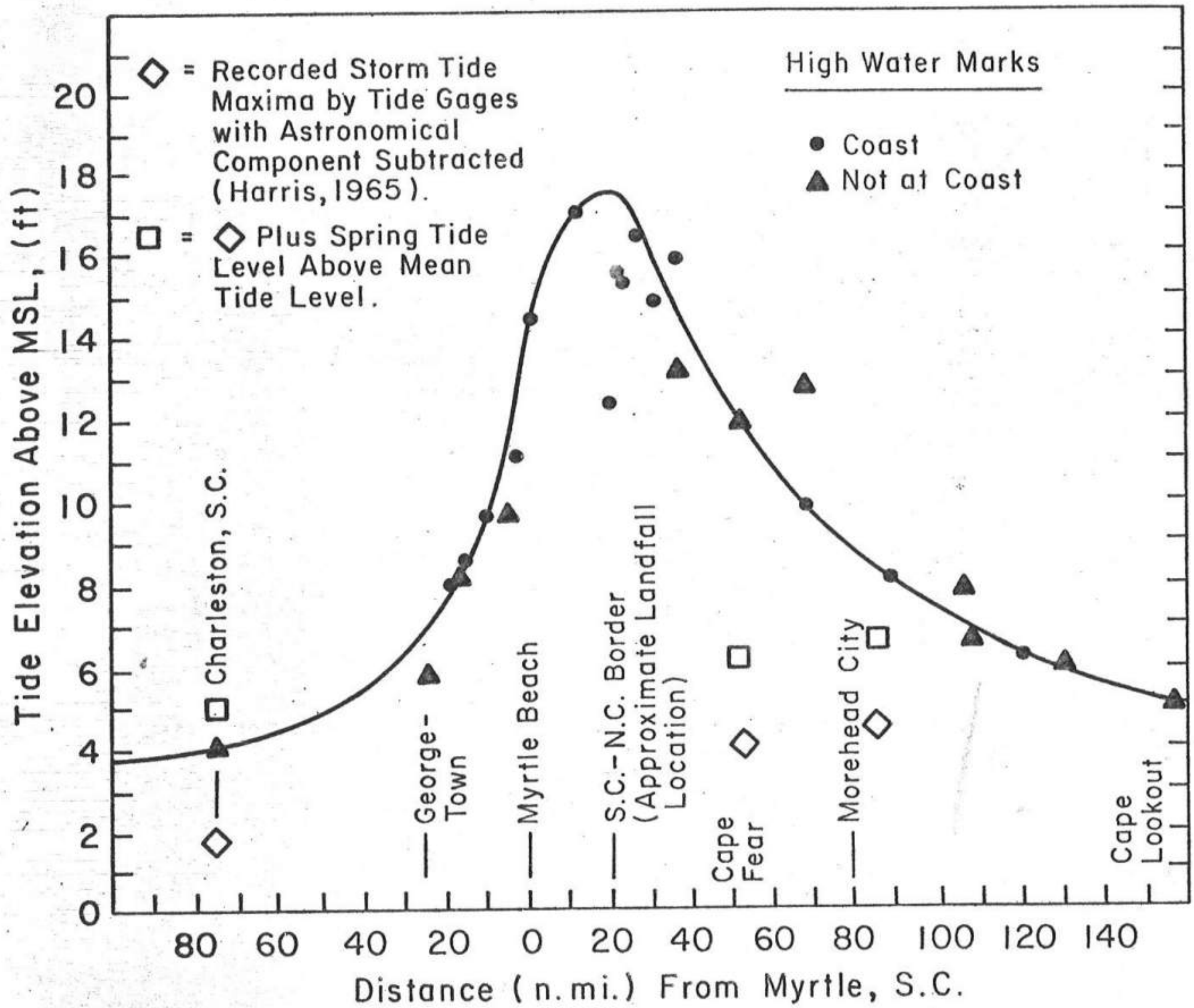


FIGURE 10. COMPARISON OF HIGH WATER MARKS WITH TIDE GAGE DATA. HURRICANE HAZEL, OCTOBER 14-15, 1954. NORTH CAROLINA AND SOUTH CAROLINA. (Adapted From Myers, 1975).

Hurricane Donna, September 1960, North Carolina - The track of Hurricane Donna in the vicinity of North Carolina was slightly East of North. The hurricane made landfall near Holden Beach, North Carolina, traversed across Pamlico Sound and Morehead City, then exited near the North Carolina-Virginia Border. The high water mark data are those published by Harris (1963) and were measured by the Norfolk and Wilmington Districts of the U. S. Army Corps of Engineers. Harris also presents the Morehead City tide gage record from which the astronomical tidal component has been subtracted. The peak of this "storm surge" component is 4.1 ft. Figure 11 presents the high water marks as a function of distance north along the coastline from the North Carolina-South Carolina border. The symbols associated with the high water marks designate whether they were measured at the coast or "not at the coast", i.e. in bays and rivers. In this case, the values determined along the coast appear to be somewhat higher than the others, although there is considerable scatter in the reported results. Also shown in Figure 11 is the maximum storm surge component reported by Harris (1963) and a value which represents an attempt to establish a reasonable upper limit of the tide gage including the contribution of the astronomical tide. In particular, the symbol x represents the sum of the peak storm surge value (determined by Harris) and the difference between half the spring tidal range and the mean tide level as determined from Tide Tables. The most logical conclusion to be drawn from Figure 9 is that there is a high probability of significant wave bias in the high water marks.

Figure 12 presents the 100 year return period tide height as recommended by Ho and Tracey. This is based on the historical hurricane statistics and the nomographs by Jelesniaski using the numerical model, SPLASH. The high water marks of Hurricane Hazel (1954) appear to have been instrumental in developing this curve. The tide data from Hurricanes Hazel and Donna have been added to this curve.

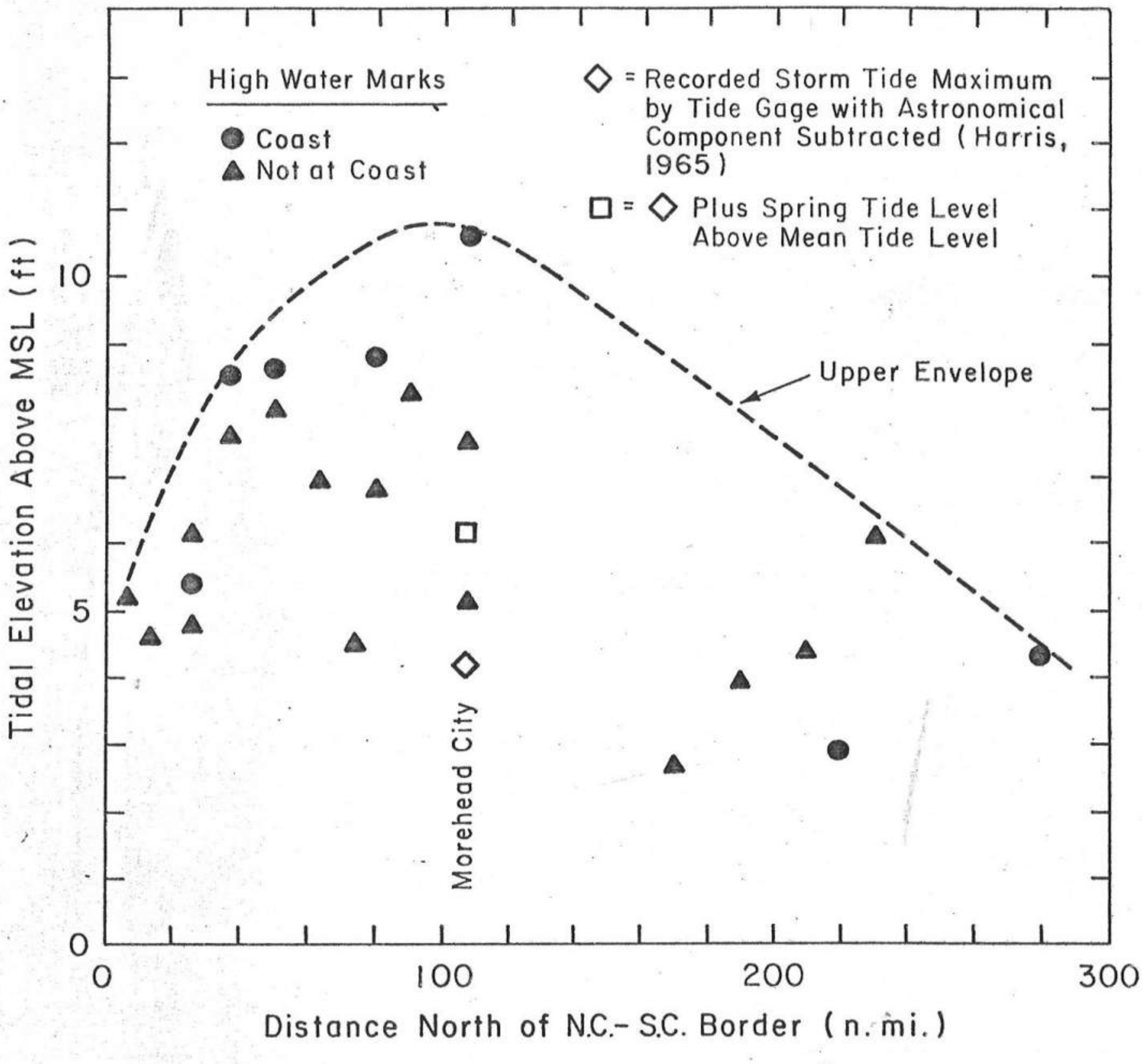


FIGURE 11. COMPARISON OF HIGH WATER MARKS WITH TIDE DATA. HURRICANE DONNA, SEPTEMBER 9-13, 1960, NORTH CAROLINA.

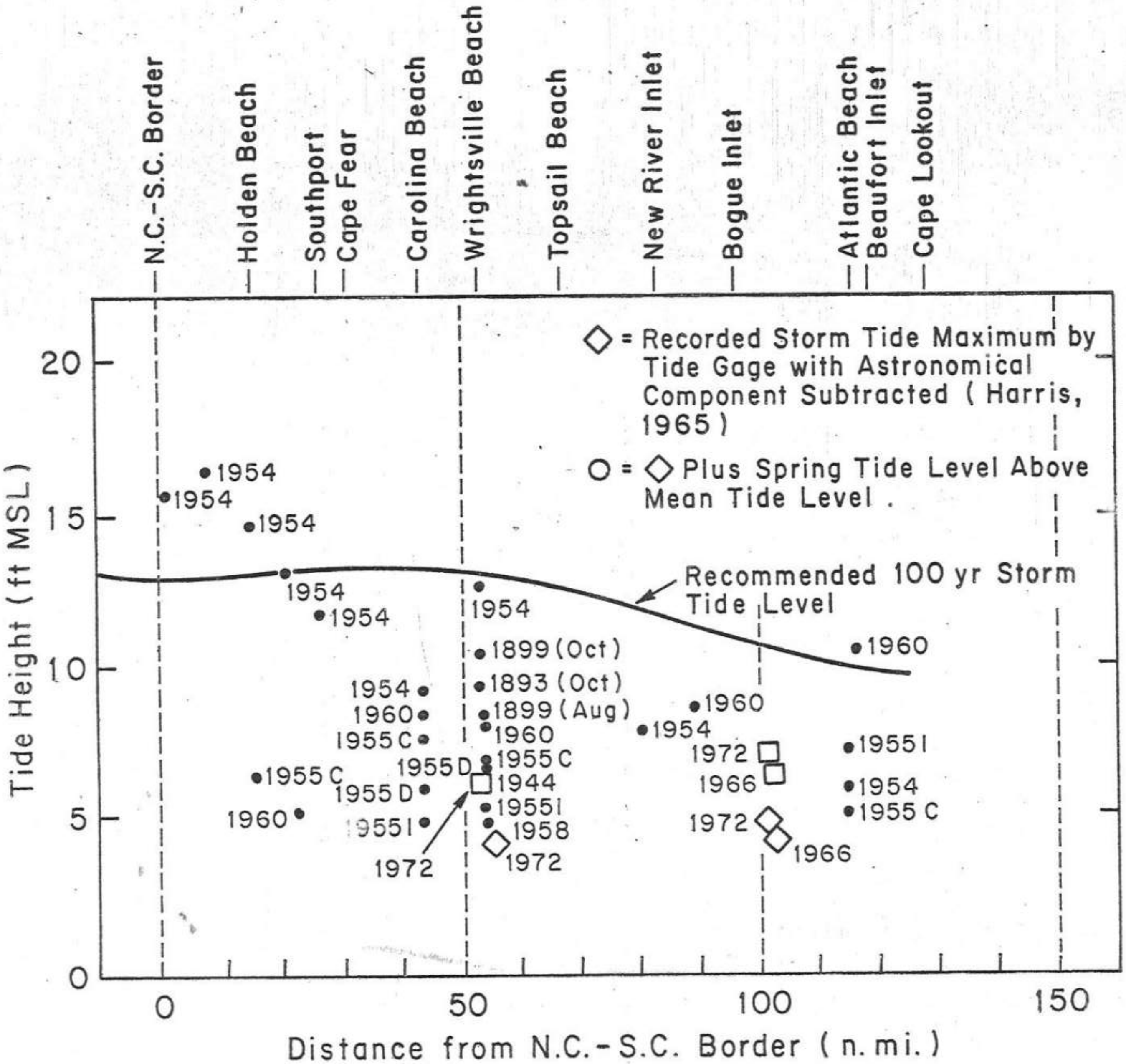


FIGURE 12. RECOMMENDED TIDE HEIGHTS ASSOCIATED WITH A 100 YEAR RETURN PERIOD. BASED ON HIGH WATER MARKS. (From Ho and Tracey, 1975).

Hurricane Audrey, June 1957, Texas and Louisiana - Hurricane Audrey made landfall near the outlet of Sabine Lake which forms the boundary between Texas and Louisiana. The available data consist of high water marks and tide gage records presented by Harris. These data appear to have been "merged" in the sense that 30 or so of the peak tides obtained from tide gages also appear as high water marks. The highest reported peak surge from a tide gage station is 12.1 ft. and is from a gage at the mouth of the outlet of Calcasieu Lake where it is reported that the tide gage was destroyed by the storm; however, "a portion of it was reconstructed from the log of the nearby Coast Guard Station on Monkey Island . . ." (Harris, 1963, p. 107). The details of this "reconstruction" are not stated. It is of interest that approximately 17 n. mi. inland the peak recorded tide measured by a gage that survived was 6.7 ft. and still further inland the peak tide increased to 7.7 ft. at 33 n. mi. inland and was 7.0 ft. at 42 n. mi. island. As a summary statement to the review of the high water marks developed from this storm it is impossible to determine whether the maximum reported tide of 12.1 ft. contained anomalous wave effects; however, the basis for this level does suggest that it could contain considerable subjectivity. Where tide records existed, there appears to have been care taken in using these to "screen" the high water marks prior to their reporting.

General Comments Concerning the Data Base Assembled by Jelesniaski in 1972

In 1972, C. P. Jelesniaski assembled the peak storm tides from 43 hurricanes occurring between 1893 and 1957. Jelesniaski's comments concerning the dominant source of these data being from high water marks were quoted previously in this report (p. 16).

In an attempt to assess the significance of the high water marks on the higher peak tides reported, the first, second and third highest storm tides from each decade were examined to determine their source where possible. The results of this examination are presented in Table I. The basis for deducing

TABLE I

EVALUATION OF SOURCES OF PEAK STORM TIDE DATA
INCLUDED IN JELESNIANSKI'S SUMMARY OF 1972

Decade	No. of Storm Tides in Summary	Highest Value in Decade		Second Highest Value in Decade		Third Highest Value in Decade	
		Storm Tide (ft.)	Source	Storm Tide (ft.)	Source	Storm Tide (ft.)	Source
1891-1900	3	14.6	HWM	9.3	HWM	5.3	HWM
1901-1910	2	10.0	HWM	7.4	HWM	-	-
1911-1920	6	13.9	HWM	9.0	HWM	7.1	HWM
1921-1930	6	10.9	HWM	10.4	HWM	9.8	HWM
1931-1940	6	13.0	HWM	9.0	HWM	8.0	HWM
1941-1950	16	14.0	HWM	10.9	HWM	10.6	HWM
1951-1960	4	12.8	HWM	12.5	TGR*	7.1	HWM

*TGR - Tide Gage Installation Destroyed. Record "Reconstructed" From Log at Nearby Coast Guard Station.

that the data were determined from high water marks rather than tide records was the examination of the rather thorough presentation of storm surge records by Harris (1963) and by the assumption that prior to 1926 (the first storm included by Harris) there are no tide gage records pertaining to hurricanes. It is surprising that of 20 peak surges, only one is based on a tide recording and that is the aforementioned "reconstructed" record from nearby observations that occurred in Hurricane Audrey.

The degree to which the data summarized by Jelesniaski have been used in calibrating SPLASH is not known. Jelesniaski has demonstrated that nomographs developed through the use of SPLASH do agree on the average fairly well with these peak surges. Nor is the degree known to which other numerical models in use are affected by the wave-related contamination that appears to be present in the high water mark data. The draft reports describing the Tetra Tech model ("Coastal Flooding Handbook", Volumes I and II, May 1977) do not mention calibration or verification.

VI. SUMMARY, CONCLUSIONS AND RECOMMENDATIONS

Summary

This study has been carried out to assess the validity of high water marks as representative of the maximum local still wave levels associated with a storm tide. A number of possible mechanisms have been identified, formulated and quantified through example calculations. In addition the correspondence between high water marks and tide gage recordings has been compared for five hurricanes. Finally, the largest of the 43 peak storm tides published by Jelesniaski are examined to determine the prevalence of high water mark versus tide gage data.

Of the possible sources of bias in high water mark data, it appears that the two most significant are set-up inside buildings due to flow over sills, and wave set-up inside the surf zone. The latter effect is a valid contribution if the results are applied at the location of their measurement. However, it is more likely that the high water marks would be developed from the higher portion of the profile where buildings were left intact and where wave set-up is substantial. In four of the five storms examined it appears that the high water marks were contaminated substantially by wave effects and in some cases, it is evident that this bias was extended to the recommended level of the 100 year storm tide. In some storms, high water mark data appear to be "merged" with the available tide gage records. For other storms, the data appear to have been regarded as unrelated.

There does not appear to be any published verification of the Tetra Tech storm surge model. The degree to which the bias in the high water marks is present in this storm surge model is not known.

Conclusions and Recommendations

It is concluded that there is significant but not consistent bias in the available reported high water marks due to contamination by wave effects.

It is recommended that a concerted program be initiated this year to:

a) Establish a field program to collect high quality storm surge data from tide gages. This should include both fixed and portable installations.

b) Establish a uniform procedure for reporting high water data. This should include a profile through the location of the high water mark and topography from the shoreline and any relevant conditions of the buildings, e.g. sills and vegetation across the profile.

c) Develop a program in which the details of the historical tide gage data base would be examined thoroughly for the purpose of serving as a data base for calibrating and verifying storm surge models.

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APPENDIX I - WATER LEVEL SET-UP INSIDE BUILDINGS DUE TO FLOW THROUGH VERTICAL SLITS

Introduction

One possible contribution to the anomalous set-up inside buildings is that due to the wave-induced flow of water through vertical slits. The total inflow depth (depth + wave amplitude) is greater than the total outflow depth (depth - wave amplitude). Thus for steady state conditions, there is a set-up of water inside the building and an outflow duration which exceeds the inflow duration. In this appendix, this mechanism is formulated, linearized and a simple solution obtained.

Formulation and Solution

Consider a vertical slit of width, W, a water depth, h, a wave height, H, and a set-up inside the building of $\bar{z}$, see Figure I-1. A reasonable approximation for the velocity, u, through the slit can be obtained from the Bernoulli equation as

$$\left.\begin{aligned} u &= \sqrt{2g|\eta - \bar{z}|} \cdot \operatorname{sign}(\eta - \bar{z}), \quad z < \bar{z} \text{ or } z < \eta \\ u &= \sqrt{2g|\eta - z|} \cdot \operatorname{sign}(\eta - \bar{z}), \quad \eta < z < \bar{z} \text{ or } \bar{z} < z < \eta \end{aligned}\right\} \tag{I-1}$$

For steady state conditions, the equation for zero net flow over one wave period, T, is given by

$$0 = \int_0^T \left[\int_{-h}^{z_A} W\sqrt{2g|\eta - \bar{z}|}\, dz + \int_{z_A}^{z_B} W\sqrt{2g|\eta - z|}\, dz \right] \cdot \operatorname{sign}(\eta - \bar{z})\, dt \tag{I-2}$$

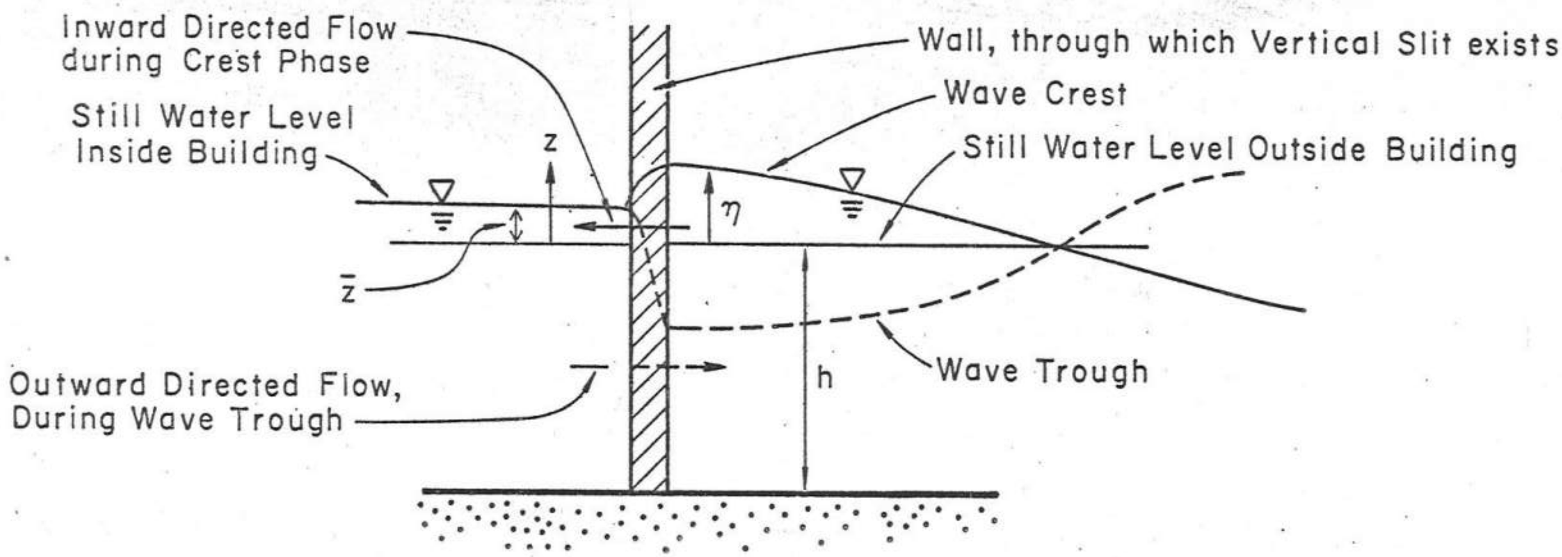

FIGURE I-1. DEFINITION SKETCH FOR MEAN WATER LEVEL SET-UP DUE TO FLOW THROUGH A VERTICAL SLIT.

in which

$$\left.\begin{aligned} z_A &= \begin{cases} \bar{z} \text{ if } \eta > \bar{z} \\ \eta \text{ if } \eta < \bar{z} \end{cases} \\ z_B &= \begin{cases} \eta \text{ if } \eta > \bar{z} \\ \bar{z} \text{ if } \eta < \bar{z} \end{cases} \end{aligned}\right\} \tag{I-3}$$

In Eq. (I-2), it is assumed that the plan area inside the building is so large that the set-up level, $\bar{z}$, inside the building is essentially constant over one wave period.

Approximating the water surface fluctuations η as

$$\eta = \frac{H}{2} \sin \sigma t \tag{I-4}$$

and casting all lengths and times in Eq. (I-2) in non-dimensional form by normalizing by the depth, h, and wave period, T, respectively and denoting dimensionless quantities by primes,

$$0 = \int_0^1 \left[\int_{-1}^{z'_A} \sqrt{(\eta' - \bar{z}')}\, dz' + \int_{z'_A}^{z'_B} \sqrt{|\eta' - z'|}\, dz' \right] \cdot \text{sign}(\eta' - \bar{z}')\, dt' \tag{I-5}$$

Eq. (I-5) can be expressed more explicitly by separating the integrals over the times of inflow and outflow

$$\left.\begin{aligned} 0 = &\int_{t'_1}^{t'_2} \left[\int_{-1}^{\bar{z}'} \sqrt{\eta' - \bar{z}'}\, dz' + \int_{\bar{z}'}^{\eta'} \sqrt{\eta' - z'}\, dz' \right] dt' \\ &- \int_{t'_2}^{1+t'_1} \left[\int_{-1}^{\eta'} \sqrt{\bar{z}' - \eta'}\, dz' + \int_{\eta'}^{\bar{z}'} \sqrt{z' - \eta'}\, dz' \right] dt' \end{aligned}\right\} \tag{I-6}$$

in which the first integral over time represents the inflow and the second the compensating outflow, and

$$
\left.\begin{aligned}
t'_1 &= \frac{1}{2\pi}\sin^{-1}\left(\frac{2\overline{z}'}{H'}\right) \\
t'_2 &= \frac{1}{2} - \frac{1}{2\pi}\sin^{-1}\left(\frac{2\overline{z}'}{H'}\right)
\end{aligned}\right\} \qquad \text{(I-7)}
$$

Unfortunately, Eq. (I-6) is not amenable to analytical solution; however, it is clear from this equation that

$$
\left.\begin{aligned}
\overline{z}' &= f(H') \\
\text{or } \overline{z}' &= hf(H/h)
\end{aligned}\right\} \qquad \text{(I-8)}
$$

An approximate but more direct solution can be determined by linearizing the flow velocities expressed as Eq. (I-1),

$$
\left.\begin{aligned}
u &= A(\eta - \overline{z}), \ z < \overline{z} \text{ or } z < \eta \\
u &= A(\eta - z), \ \eta < z < \overline{z}, \text{ or } \overline{z} < z < \eta
\end{aligned}\right\} \qquad \text{(I-9)}
$$

Utilizing Eq. (I-9), the approximate solution is

$$
\overline{z}' = \frac{H'^2}{16} \qquad \text{(I-10)}
$$

or in dimensional form

$$
\boxed{\overline{z} = \frac{H^2}{16h}} \qquad \text{(I-11)}
$$

APPENDIX II - WATER LEVEL SET-UP INSIDE BUILDINGS DUE TO FLOW OVER SILLS

Introduction

During hurricanes, buildings may suffer partial damage such as window breakage, etc., but may still retain their overall integrity such that recovery of high water marks following storms is possible. If a window is facing the incoming waves and is broken such that its sill lies below the wave crest level, and if this is the only flow communication with the outside, it is clear that there will be a wave-induced mean water level set-up inside the building due to the wave crest flowing over the sill and the lesser depth of return flow during the time of wave trough. It follows that the level of the mean water set-up inside the building will always be greater than the sill elevation and that for the limiting case of the sill elevation at the crest elevation, the set-up elevation would be equal to that of the wave crest.

Formulation and Solution

Consider a window sill of elevation, z_s, above the wave trough level, a set-up inside the building of, $\bar{z}$, and a wave height of height, H, propagating in a water depth, h. In the following development, it is assumed that small amplitude wave theory will suffice for our purposes and, as before, that the plan area inside the building is so large that the set-up level, $\bar{z}$, inside the building is essentially constant over one wave period, see Figure II-1 for a definition sketch.

It will be assumed that the inflow velocities can be described by the small amplitude shallow water relationship

$$u = \eta \sqrt{\frac{g}{h}} \qquad \text{(II-1)}$$

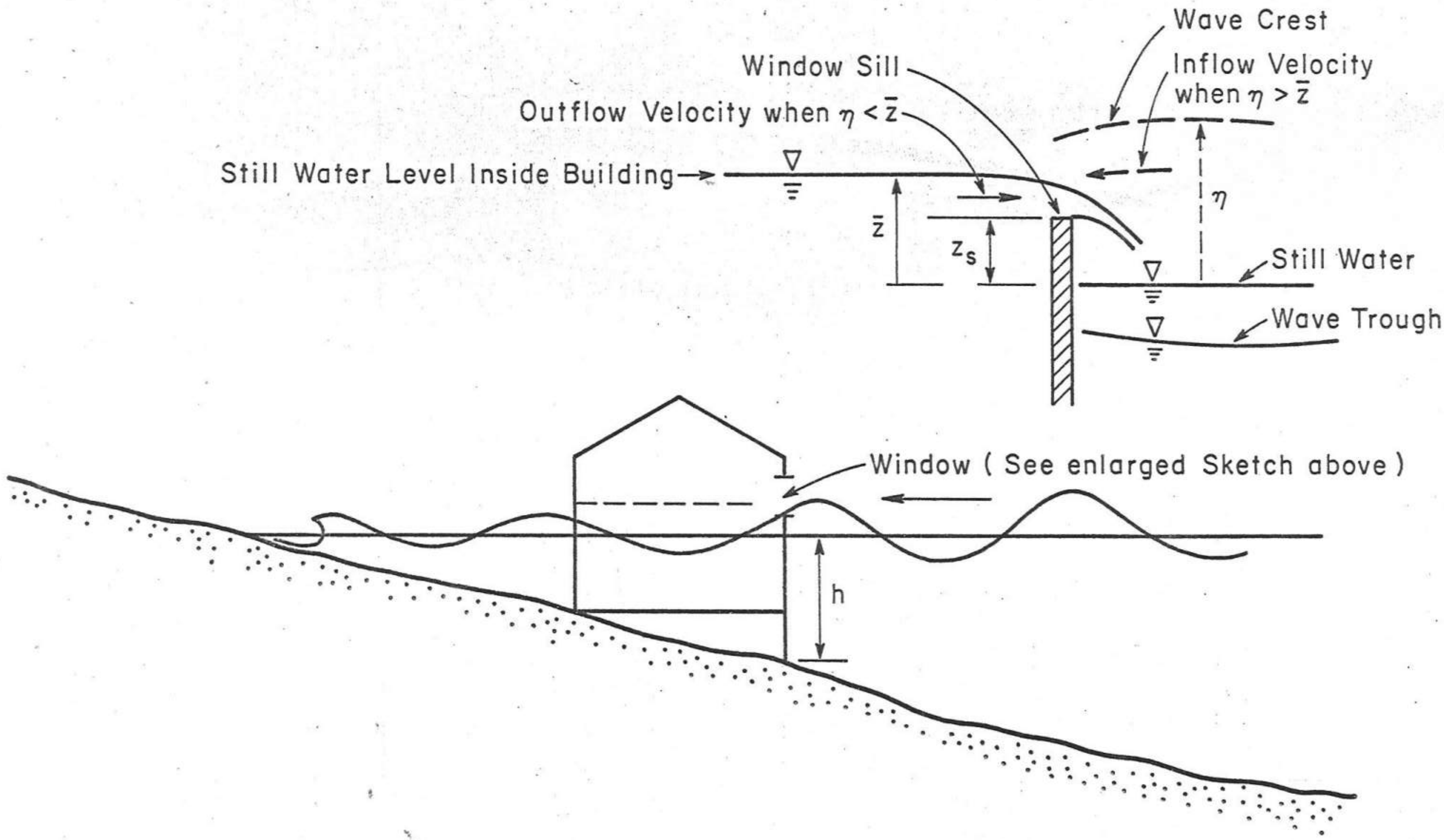

FIGURE II-1. DEFINITION SKETCH FOR WAVE INDUCED SET-UP INSIDE BUILDINGS DUE TO FLOW OVER A SILL.

in which η is the wave displacement outside the building and given by

$$\eta = \frac{H}{2} \sin \sigma t \qquad \text{(II-2)}$$

and that inflow occurs during the time interval when $\eta > \overline{z}$. The outflow is treated as that over a sharp crested weir in which case critical flow conditions would exist over the weir,

$$u = \left(\frac{2}{3}\right)^{\frac{1}{2}} \sqrt{g(\overline{z} - z_s)} \qquad \text{(II-3)}$$

and this flow occurs when $\eta < \overline{z}$. The steady state requirement of no net inflow, Q, can be expressed as

$$Q = \int_{t_1}^{t_2} \eta \sqrt{\frac{g}{h}}(\eta - z_s)dt - \left(\frac{2}{3}\right)^{3/2} \sqrt{g}(\overline{z} - z_s)^{3/2} [T - (t_2 - t_1)] = 0 \qquad \text{(II-4)}$$

in which

$$t_1 = \frac{T}{2\pi} \sin^{-1}\left(\frac{2\overline{z}}{H}\right), \; t_2 = \frac{T}{2} - t_1$$

It is helpful in the interpretation of Eq. (II-4) to cast it in dimensionless form using the water depth, h, as the length scale and the wave period T as the time scale. The resulting equation is

$$\int_{t_1'}^{t_2'} [\eta'^2 - \eta' z_s']dt' - \left(\frac{2}{3}\right)^{3/2} (\overline{z}' - z_s')^{3/2} [1 - (t_2' - t_1') = 0 \qquad \text{(II-5)}$$

It is evident from the form of Eq. (II-5) that, $\overline{z}'$, the dimensionless set-up depends only on two dimensionless variables, H' and z_s', i.e.

$$\overline{z}' = f(H', z_s')$$ (II-6)

or in dimensional form

$$\frac{\overline{z}}{h} = f\left(\frac{H}{h}, \frac{z_s}{h}\right)$$ (II-7)

The objective of the remainder of this appendix is to quantify this relationship, at least in graphical form.

The integration in Eq. (II-5) can be carried out yielding

$$\left(\overline{z}' - z_s'\right)^{3/2} = \left(\frac{3}{2}\right)^{3/2} \frac{(A + B)}{[1 - (t_2' - t_1')]}$$ (II-8)

in which

$$\left.\begin{aligned} A &\equiv \frac{H'^2}{8}\left[(t_2' - t_1') - \frac{(\sin 4\pi t_2' - \sin 4\pi t_1')}{4\pi}\right] \\ B &\equiv \frac{z_s' H'}{4\pi}[\cos 2\pi t_2' - \cos 2\pi t_1'] \end{aligned}\right\}$$ (II-9)

It was not possible to solve Eq. (II-8) analytically; therefore, it was solved by an iterative procedure. The results have been presented as Figure 4 with the dimensionless still elevation, z_s', as the abscissa, the dimensionless set-up, $\overline{z}'$, as the ordinate and for three values of dimensionless wave height.