CHAPTER 60

TSUNAMI INUNDATION PREDICTION

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ABSTRACT

This paper concerns the run-up and inundation characteristics of tsunami surges. The forces and moments produced by the waves are not discussed, however, the proposed technique does provide the necessary information for their determination. The method relies on the knowledge of the wave elevation at the coast (as determined from historical data or other means) and an estimation of the bed roughness. The considerations and calculations involved in determining these parameters are discussed in detail. Twenty-four observed run-ups on the island of Hawaii in the case of the 1946 Aleutian tsunami, and 18 run-ups on the island of Maui for the Chilean 1960 tsunami are used to illustrate the technique. Methods are also presented to predict the shoreline heights and extent of inundation of tsunami surges where historical data is not available.

INTRODUCTION

This paper presents the theory and technique for determining the extent and profile of flooding due to tsunamis as affecting the coastal zone, especially the State of Hawaii. An earlier paper by Bretschneider and Wybro (1973) gave results for tsunami flooding over a flat, dry bed as in Hilo, Hawaii. These results are extended for the case of sloping land profiles and also for composite slopes consisting of a flat foreshore and sloping backshore.

To determine the wave height at the coastline, it is necessary to select an appropriate value of a roughness parameter, namely Manning's n, to account for the frictional effect of the bed on the inland advance of the surge. From this, then, flooding under existing or improved conditions can be determined. For improved conditions, it should be expected that the extent of flooding will be greater than that which occurred during past conditions.

The distribution of coastline elevations obtained from historical data provide an extrapolation base for which coastline elevation at intermediate locations can be determined.

BASIC EQUATIONS

The frictional shear stress opposing the flow of water is given by the Darcy-Weisbach (1858) relationship

$$\tau = \frac{f\rho u^2}{2} \tag{1}$$

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where τ is the bottom shear stress, ρ is the fluid density, u is the mean velocity, and f is a non-dimensional friction coefficient. Because the frictional coefficient is dependent not only on the bed roughness, but also on the depth of fluid, various laws have been formulated to attempt to separate these effects.

The most common formulation that attempts to correct this defect is Manning's formula

$$u = \frac{1.486}{n} R^{2/3} S^{1/2} \tag{2}$$

where R is the hydraulic radius (feet), S is the slope of the water surface, and n is Manning's n which is a measure of the roughness of the boundary and has dimensions ($ft^{-1/3}$ sec). The hydraulic radius is defined by

$$R = A/P \tag{3}$$

where A is the cross-sectional area of flow and P is the wetted perimeter. R is equal to the depth of water, h, in the case of very wide channels.

The Chezy-Kutter formula is given by

$$u = C_h\sqrt{RS} \tag{4}$$

where C_h is the Chezy coefficient ($ft^{1/2}$ sec) and is related to Manning's n by

$$C_h = \frac{1.486}{n} R^{1/6} \tag{5}$$

Manning's n is related to the Darcy-Weisbach f as follows:

$$f = 0.9\ g\ n^2\ h^{-1/3} \tag{6}$$

Hence, if Manning's n is only a function of bed roughness, then the friction factor f varies inversely to the 1/3 power of the water height. Bretschneider and Wybro (1973) showed that Manning's n for any particular roughness remains essentially constant with water depth. This is particularly so for values of n less than 0.04 and water heights between 5 and 50 feet.

Values of Manning's n can be found in numerous references such as Creager and Justin (1950) or Parsons (1965) in the case of effects of vegetation.

The classical long wave equations consist of the momentum and continuity equations which are, respectively:

$$\frac{\partial u}{\partial t} + u\frac{\partial u}{\partial x} = -g\frac{\partial h}{\partial x} - gm \quad \frac{-g\ u|u|}{C_h^2(d_o+h)} \tag{7}$$

$$B\frac{\partial h}{\partial t} + \frac{\partial}{\partial x}(Au) = 0 \tag{8}$$

where: A is the cross sectional area of flow
B is the surface width of flow
d_o is the natural water depth
h is the depth of water above d_o
m is the bottom slope

and differentiation with respect to x and t denote flow direction and time, respectively.

The last term in equation (7) accounts for frictional dissipation and can be rewritten in terms of Manning's n as

$$\frac{g\,u|u|}{C_h^2(d_o+h)} = \frac{n^2 g|u|u(d_o+h)^{-4/3}}{(1.486)^2} \tag{9}$$

SOLUTIONS TO EQUATIONS

We will first consider the special case of a tsunami surge propagating inland over an open coast and a flat dry bed. This case was examined earlier by Bretschneider and Wybro (1973).

The momentum equation becomes (for $m = 0$, and $u = F\sqrt{gh}$ where F = constant and assuming negligible local accelerations in comparison to the convective terms)

$$\frac{\partial h}{\partial x} = -\frac{n^2\,u^2\,h^{-4/3}}{(F^2/2+1)(1.486)^2} \tag{10}$$

for which it is found:

$$\frac{h}{h_o} = \left[1 - \left(\frac{x}{x_R}\right)\right]^{3/4} \tag{11}$$

and

$$x_R = \frac{3}{4}\,\frac{h_o^{4/3}}{F^2\,n^2}\cdot\frac{(1.486)^2}{g}\left(\frac{F^2}{2}+1\right) \tag{12}$$

where h_o is the initial surge depth, h is the surge depth after travelling x feet inland, and x_R is the distance to complete dissipation (see Fig. 1). Note that equation (12) differs from that of Bretschneider and Wybro (1973) in the constant term 6.5. However, the earlier report fitted inundation profiles to the observed data, whence Manning's n was determined. To convert the Manning's n so determined to the correct value in equation (12) involves multiplying by the factor $\sqrt{3} = 1.73$. The resultant inundation curves are hence identical, except the Manning's n obtained in the earlier work are to be corrected.

Solutions to equations (11) and (12) are depicted graphically in Figures 2 through 5. Each figure, for a particular roughness value, describes the envelope curve of the surge height as it propagates inland for various initial heights and for Froude numbers F = 1 and F = 2.

In the theoretical dam break problem, considering frictionless flow, Keulegan (1949) arrives at F = 2. Experiments by Fukui, et al., (1963) found that F = 1.73 for a bore propagating over a dry bed with n = 0.013. More recent experiments by Cross (1966) for tsunami surges finds F = 1.41 for a bed roughness of n = 0.02.

Miller's (1968) experiments indicated that the characteristic Froude numbers are functions not only of the roughness, n, but also of the ratio of surge depth to depth of water prior to sloping bottom. The relationships F = f(n, h/d) was not determined, however. It appears from Figures 2 through 5 that F = 2 causes greater frictional dissipation than F = 1 because of higher fluid velocities. It is, hence, conservative to use F = 2 to predict shoreline elevations from run-up data and for conservative purposes, until more exact information is known, it appears desirable to use F = 2 (as, for instance, if forces are calculated).

In the case of surges advancing over a sloping bottom, equation (7) becomes (neglecting local acceleration and friction)

$$u \frac{\partial n}{\partial x} = -g \tan \alpha - g \frac{\partial h}{\partial x} \tag{13}$$

where α is the bottom slope. For the case of $u = F\sqrt{gh}$ then

$$\frac{\partial h}{\partial x'} = - \frac{\sin \alpha}{(F^2/2+1)} \tag{14}$$

where x' is the coordinate along the flow length. Upon integrating, the expression for the vertical runup, R", becomes

$$R'' = h_c + h_t + \frac{u_t^2}{2g} \tag{15}$$

where h_c is the vertical rise of the berm or beach (Fig. 1) and h_t and u_t are the surge height and surge velocity, respectively, at the toe of the slope.

Equation (15) can be compared to the value $u_t^2/2g$ arrived at by Freeman and LeMehaute (1964) where $h_c = 0$ and also $h_t = 0$ as a result of the first-order, non-linear long wave analysis. In this analysis, the surge collapses at the shoreline to a thin sheet of water with a velocity u_t.

Note that for $h_c = 0$ and F = 2, R" = 3 or the vertical rise in water will triple.

It appears that for tsunami surges, the inclusion of h_t in equation (12) is justified as it is well known that the surge height does not collapse at the shoreline. This was experimentally justified by Miller (1968).

Cross (1966) found experimentally that the rise of water on a vertical wall is approximately 1.33 $u_t^2/2g$ for a dry bed. In equation (15) with F = 1.41 and h_c = 0 yields $R'' = 1.9\, u_t^2/2g$ which is conservative.

In the case of a roughened slope, equation (7) becomes the governing equation, and equation (14) becomes (neglecting the local acceleration)

$$\frac{\partial h}{\partial x'} = -\{\sin\alpha + \frac{n^2 g F^2 h^{-1/3}}{(1.486)^2} \cos\alpha\} [F^2/2 + 1]^{-1} \qquad (16)$$

and the runup, R, is found by numerical integration. This has been done for various slopes (2.5° and 5°), various roughness (n = 0.025, 0.035, 0.045, and 0.055) and a range of surge depths, h_t. The results are depicted in Figures 6 and 7, wherein R is plotted versus h_t for various Manning's n values.

For composite profiles (horizontal foreshore and sloping backshore), Figures 2 through 5 can be used to determine h_t, and this value is entered in Figures 6 or 7 with the corresponding Manning's n value to determine the run-up, R. Note that for the composite slopes, the roughness of the sloping portion need not be the same as the flat portion.

APPLICATION TO HISTORICAL DATA

Bretschneider and Wybro (1973) investigated the results of the tsunamis of 1946 and 1960 for the case of Hilo, Hawaii using the flat bottom equations. It was found that the land profiles were ideally suited due to the very gentle grade and an excellent comparison was obtained between the computed and observed inundation profiles. In this section, the analysis is extended to include composite land profiles whence equations (10) and (11) are used along with Figures 6 and 7. The areas of interest are the north coast of the island of Hawaii as experienced by the 1946 Aleutian tsunami and the southwest coast of the island of Maui for the case of the 1960 Chilean tsunami.

Island of Hawaii

Figure 8 shows the wave advancement of the 1946 tsunami as affecting the north coast of the island of Hawaii. The solid lines represent the wave fronts, the light dashed lines represent orthogonals and the heavy dashed lines represent orthogonal intersections, as was computed by Shepard, et al. (1950). The region of interest extends from Upolu Point in the west to Kumukahi Point in the east, a coastal distance of roughly 100 miles. The reported run-ups, also shown, were obtained from the Tsunami Research Center, University of Hawaii as documented on 1:24,000 scale USCGS quadrangle charts. The land profiles were obtained from these charts and the coastal terrain and roughness conditions were estimated from a field survey coupled with infrared aerial photographs taken from U-2 flights in 1974 and 1975.

It is found that the land profiles for a majority of the locations can be very adequately described by the composite slope method, using backshore slopes of 0.5° up to 10° with an average of about 2° to 3°. The steep, nearly vertical cliffs that are common to this coastline were not used as these regions are inaccessible, hence, no run-ups were reported.

The reported run-ups are plotted in Figure 9 (solid circles) versus coastal distance measured west to east. These run-ups are projected back to the coast by means of Figures 2 through 7 using the iteration method described in the previous section and where the values of Manning's n are determined from Table I. The coastal elevations are indicated by open circles in Figure 9. Table II lists the run-ups, inundations, coastal elevation and Manning's n determined at each location.

A somewhat smooth curve can be passed through the coastline distribution, and on the basis of this curve, an equivalent, uniform wave of height z_r in open ocean is

$$z_r = \sqrt{\frac{1}{\ell} \int_0^L z_o^2 \, ds} \tag{17}$$

where ℓ = projected distance of coast along the wavefront, ds is the increment of length along the shoreline, and L is the length of the coast. Note that z_r is a fictitious height in that shoaling has already been taken into account. In this case, projecting to the 6:30 a.m. wavefront (Fig. 8) gives ℓ = 80 miles, L = 100 miles, and z_r = 46 feet. On this basis the refraction coefficient is

$$K_R = \frac{z_o}{z_r} \tag{18}$$

The distribution of K_R as obtained by equation (18) is compared to the values obtained from the refraction diagram, Figure 8. The results are shown in Figure 10. The agreement is good, and both curves exhibit the same trends. The discrepancies can be attributed to the basic assumptions involved in arriving at the frictional effect (equation (16)) and the neglect of reflection and diffraction effects.

The determination of the appropriate friction factor involves some subjectivity, especially in the case where the flow length is composed of various roughnesses. The values in Table I were, however, determined not only from a literature review, but also from the fitting of envelope curves to the known inundation profiles (such as Hilo) and hence determining Manning's n. For example, Kolekole Beach Park is situated somewhat midway along the northern coast (see Fig. 11). Field observations made after the 1946 tsunami show that the wave overtopped and destroyed the existing railroad bridge and overtopped the Nanahoa highway bridge by three feet. A fitting of the inundation profile using Figures 2 through 7 determined Manning's n to be 0.04 which is as expected from Table I.

Suppose that the terrain was cleared to an open area with finely cut grass, etc., whence n = 0.025. If the same tsunami surge advanced inland under the new conditions, then it would have overtopped the highway bridge by 11 feet and the vertical run-up would be 30 feet instead of 17 feet. This exemplifies the changes in inundation characteristics corresponding to terrain changes as, for instance, when development occurs.

TABLE I

SUGGESTEO VALUES OF MANNING'S n FOR VARIOUS COASTAL TERRAIN CONOITIONS

n	Conditions
0.015 - 0.025	Very smooth (mud flats, ice, well maintained concrete paved ways, beaches of fine sand)
0.025 - 0.030	Smooth (dried earth, coarse sand beaches, badly maintained concrete paved ways, very thin lawn grass up to 1 cm high)
0.030 - 0.035	Average for developed areas (lawn grass up to 5 cm high, gravel, presence of some buildings, houses, and other obstructions)
0.035 - 0.045	Open coast, relatively smooth and open area (grass up to 10 cm, sparse population of trees*, sparse bush, even bottom)
0.045 - 0.055	Moderately rough open coastal areas (thick grass, uneven bottom consisting of large rocks, coral, etc., presence of trees with low foliage, brush, lava rock, etc.)
0.055 - 0.070	Unusually rough coastal areas (dense brush, dense tree population with exposed roots, coarse lava rock formations)

*Trees with high foliage such that only trunks are exposed to flow

TABLE II

HEIGHT, RUN-UP, AND INUNDATION OF 1946 TSUNAMI
ALONG NORTH COAST OF ISLAND OF HAWAII

Location No. (west to east)	Run-up, R (ft)	Inundation, X_L (ft)	Manning's n ($ft^{1/6}$)	Elevation at Coast, Z_o (ft)
1	18	100.	.052	20.
2	40	80.	.035	42.
3	(33)	80.	.045	(34.)
4	50	100.	.045	52.
5	26	4600.	.045	60.
6	40	500.	.045	46.
7	23	5200.	.026	42.
8	40	2300.	.024	50.
9	36	600.	.055	53.
10	28	100.	.035	31.
11	30	200.	.043	33.
12	38	150.	.035	39.
13	(23)	75.	.045	(25.)
14	38	200.	.045	40.
15	37	150.	.050	40.
16	37	50.	.045	39.
17	27	230.	.050	33.
18	34	70.	.043	36.
19	38	100.	.035	39.
20	35	100.	.045	38.
21	20	1800.	.031	30.
22	23	1500.	.029	32.
23	18	360.	.065	32.
24	19	250.	.050	28.

$\overline{n}$ = 0.04236, mean of n

σ_n = 0.01006, standard deviation of n

$\overline{n}_r$ = 0.04354, root mean square of n

Island of Maui

A second coastline is examined in detail, that being the southwest coast of the island of Maui, a coastal distance of roughly 35 miles (see Fig. 12). In this case the 1960 tsunami, which originated from Chile, is examined in much the same manner as the previous case. The reported run-ups were substantially lower due to the sheltering effect of the island of Kahoolawe, however, there is a preponderance of data. This coastline is also ideally suited for the composite slope model.

The results of this analysis are depicted in Figures 13 and 14 and Table III where generally good agreement is obtained between the expected and calculated refraction effects. The discrepancy in the southeast region (right hand side, Fig. 14) can be attributed to diffraction effects due to Kahoolawe.

The method of fitting curves to the distribution of coastline elevations allows for the determination of heights at intermediate locations having no historical data. This is a common occurrence, especially for newly developed areas.

SUMMARY AND CONCLUSIONS

The prediction of the height and extent of flooding due to tsunamis should take into account the initial height, Green's law, convergences or divergences, and friction effects. The equations developed in this report take all of the above into account except convergences and divergences, i.e. they are valid only for the open coastline.

It appears from an analysis of historical run-up data that frictional effects can account for the generally wide data scatter, and on this basis coastline elevations can be determined at locations other than where data is present. When these values have been determined for existing conditions, it is then possible to predict flooding contours under existing conditions and also for extreme conditions. If the existing conditions are changed because of development, then it will be important to select the appropriate value of Manning's n in order to predict the extent of normal and extreme flooding under the improved conditions.

Although the technique has been tested and verified by field conditions, it appears that experimental justification is necessary, especially in regards to the following:

1. The Froude number relationship in regards to bore height and roughnesses.
2. The constancy of the Froude number over the flow length, especially in the case of composite slope.
3. The quantitative criteria for bore formation.

ACKNOWLEDGEMENTS

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TABLE III

HEIGHT, RUN-UP, ANO INUNDATION OF 1960 TSUNAMI

ALONG SOUTHWEST COAST OF MAUI

Location No. (west to east)	Run-up, R (ft)	Inundation, X_L (ft)	Manning's n ($ft^{1/6}$)	Elevation at Coast, Z_o (ft)
1	7	1600.	.038	11.5
2	9	2200.	.026	11.8
3	9	250.	.052	10.8
4	9	220.	.043	9.8
5	8	1000.	.026	9.1
6	9	300.	.035	9.3
7	8	200.	.043	9.0
8	7	100.	.060	10.5
9	11	150.	.026	11.5
10	7	400.	.035	10.5
11	8	1500.	.021	9.1
12	8	500.	.024	9.0
13	5	650.	.035	8.8
14	4	1000.	.026	9.0
15	5	400.	.047	9.1
16	7	200.	.052	9.2
17	9	100.	.035	10.0
18	9	500.	.031	11.0

$\bar{n}$ = 0.03686, mean of n

σ_n = 0.01101, standard deviation of n

$\bar{n}_r$ = 0.03847, root mean square of n

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LIST OF SYMBOLS

A	cross sectional area of flow
B	surface width of flow
C_h	Chezy coefficient
F	Froude number
K_R	refraction coefficient
L	length of coastline
P	wetted perimeter
R	hydraulic radius, vertical run-up
R'	vertical run-up not including slope effect
R''	vertical run-up not including frictional effect
S	slope of water surface
d_o	natural water depth
f	Darcy-Weisbach friction factor
h	surge height
h_c	height of flatland above MSL
h_o	depth of surge at coastline
h_t	depth of surge at toe of slope
ℓ	projected distance of coast along wavefront
m	slope of backshore
n	Manning's n
t	time
u	surge velocity
u_t	surge velocity at toe of slope
x	coordinate measured inland from coast
x_R	extent of inundation
z_o	total depth of surge above MSL at coast
z_r	equivalent unrefracted wave height
α	angle of backshore
ρ	density of fluid
τ	bottom shear stress

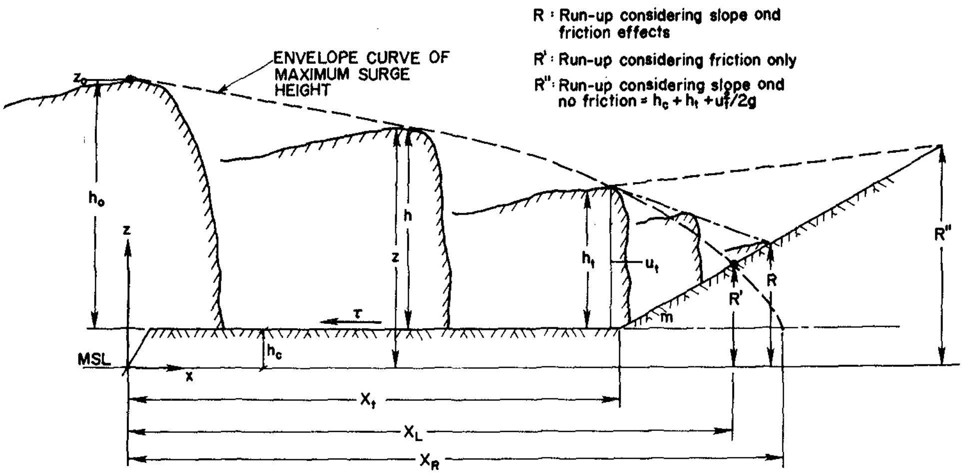


FIGURE 1 SCHEMATIC DIAGRAM OF BORE ADVANCEMENT AND DEFINITION OF SYMBOLS

FIGURE 2 ENVELOPE CURVES OF WATER SURFACE ELEVATION, h, VERSUS INLAND TRAVEL DISTANCE, x, FOR SEVERAL COASTAL ELEVATIONS. MANNING'S n = 0.025 FOR F = 2 AND MANNING'S n = 0.017 FOR F = 1.

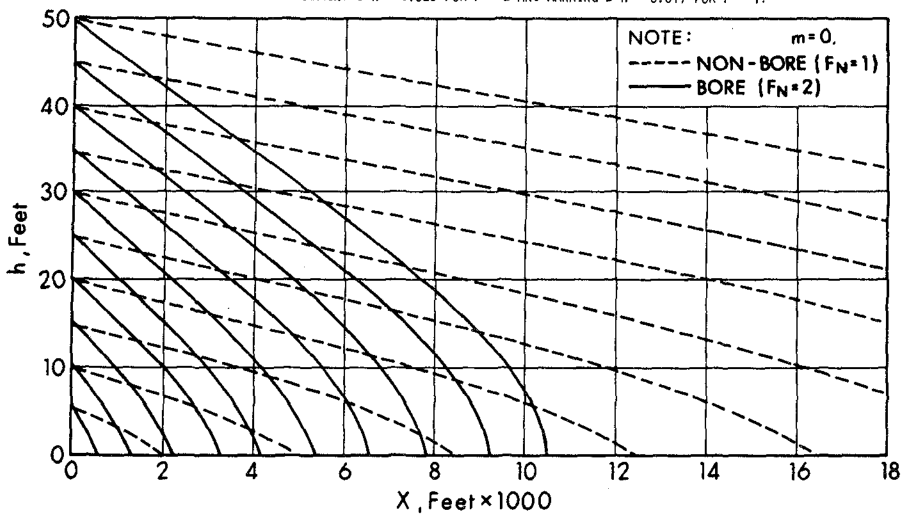


FIGURE 3 ENVELOPE CURVES OF WATER SURFACE ELEVATION, h, VERSUS INLAND TRAVEL DISTANCE, x FOR SEVERAL COASTAL ELEVATIONS.
MANNING'S n = 0.035 FOR F = 2 AND MANNING'S n = 0.025 FOR F = 1.

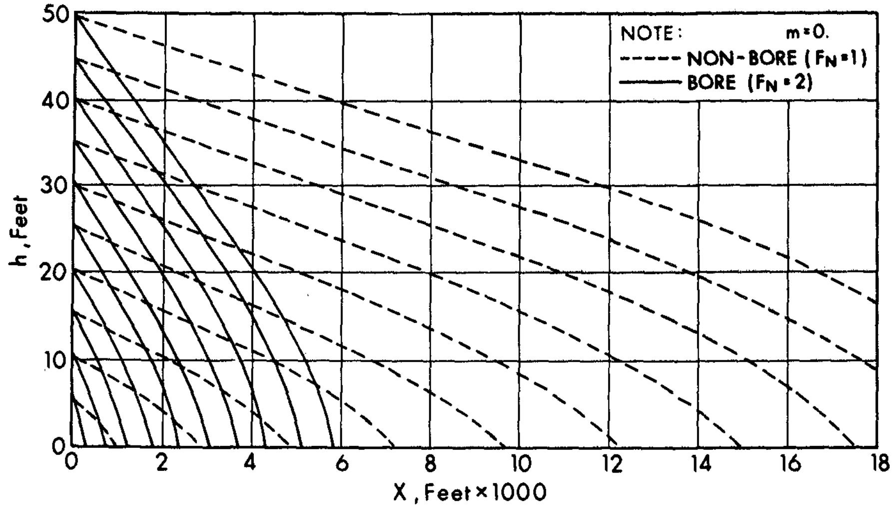


FIGURE 4 ENVELOPE CURVES OF WATER SURFACE ELEVATION, h, VERSUS INLAND TRAVEL DISTANCE, x, FOR SEVERAL COASTAL ELEVATIONS.
MANNING'S n = 0.045 FOR F = 2 AND MANNING'S n = 0.032 FOR F = 1.

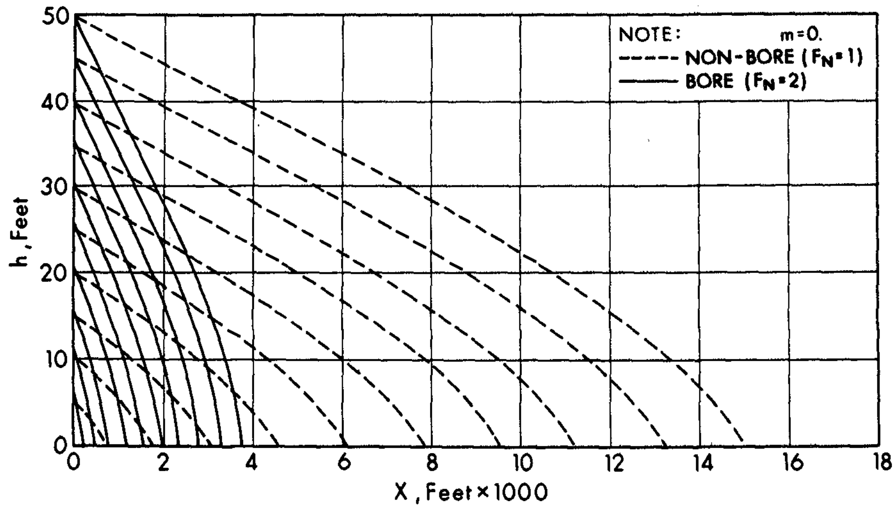


FIGURE 5 ENVELOPE CURVES OF WATER SURFACE ELEVATION, h, VERSUS INLAND TRAVEL DISTANCE, x, FOR SEVERAL COASTAL ELEVATIONS. MANNING'S n = 0.055 FOR F = 2 AND MANNING'S n = 0.039 FOR F = 1.

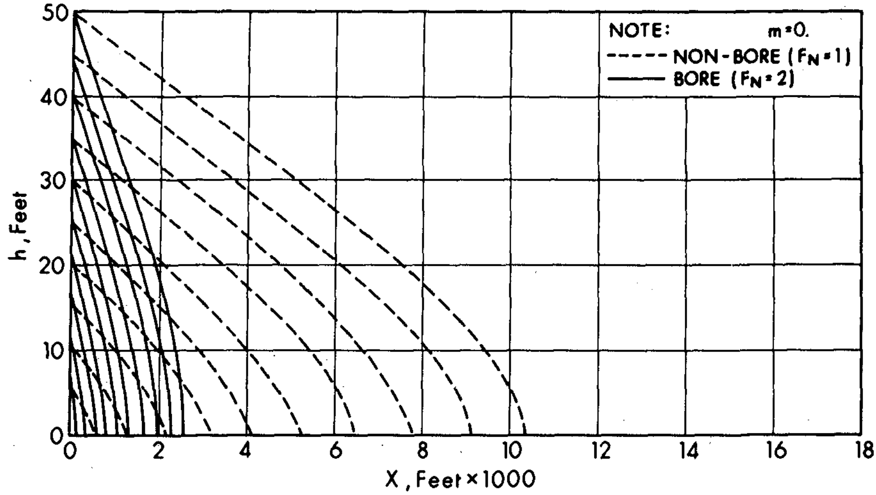


FIGURE 6 RUN-UP, R, VERSUS DEPTH OF SURGE AT TOE, h_t, FOR VARIOUS MANNING'S n. BED SLOPE = 2.5°.

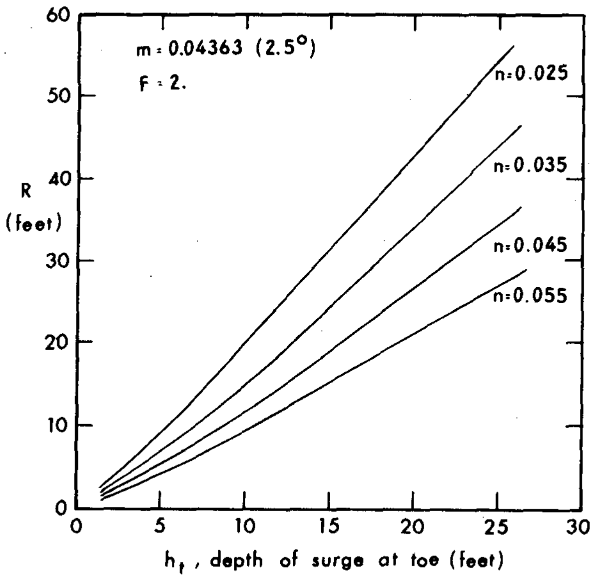


FIGURE 7 RUN-UP, R, VERSUS DEPTH OF SURGE AT TOE, h_t, FOR VARIOUS MANNING'S n. BED SLOPE = 5°.

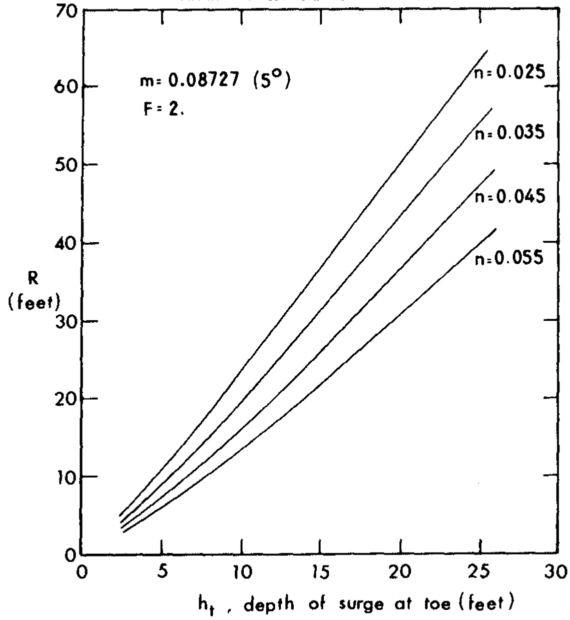


FIGURE 8 NORTH COAST, ISLAND OF HAWAII, SHOWING 1946 WAVE ADVANCE AND OBSERVED RUN-UP HEIGHTS

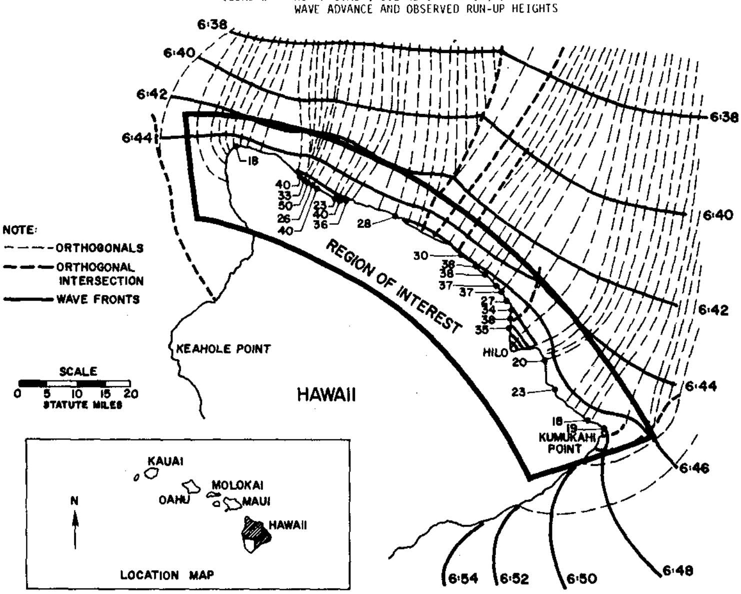


FIGURE 9 OBSERVED RUN-UPS AND PREDICTED ELEVATIONS AT COAST VERSUS COASTAL DISTANCE ALONG NORTH SHORE OF HAWAII FOR 1946 TSUNAMI

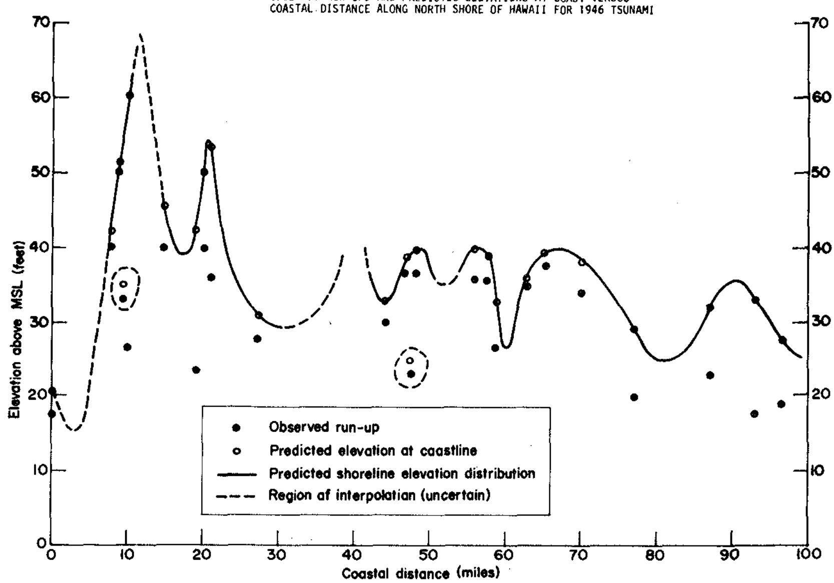


FIGURE 10 COMPARISON BETWEEN REFRACTION COEFFICIENT AS DETERMINED FROM EQUATION (19) AND REFRACTION ANALYSIS FOR THE NORTH COAST, ISLAND OF HAWAII, FOR 1946 TSUNAMI

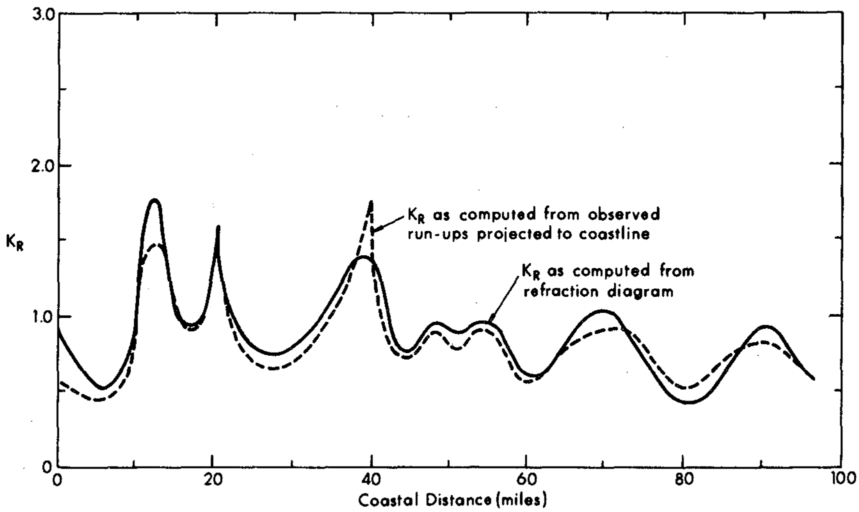
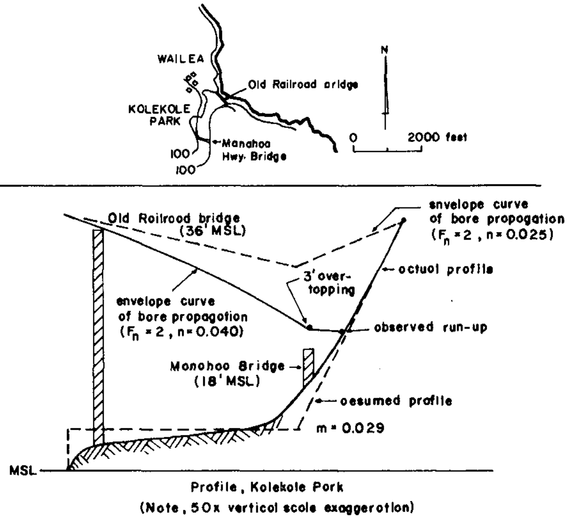


FIGURE 11 THEORETICAL INUNDATION PROFILE OF 1946 TSUNAMI (SOLID CURVE) FOR KOLEKOLE BEACH PARK, ISLAND OF HAWAII, SHOWING OBSERVED ELEVATIONS



Profile, Kolekole Park
(Note, 50x vertical scale exaggeration)

FIGURE 12 SOUTHWEST COAST, ISLAND OF MAUI, SHOWING 1960 WAVE ADVANCE AND OBSERVED RUN-UP HEIGHTS

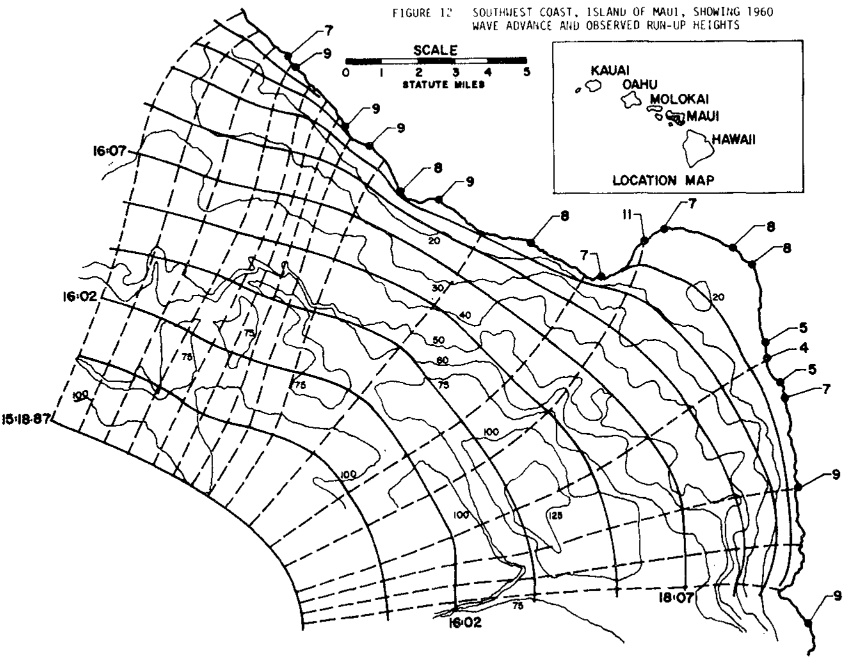


FIGURE 13 OBSERVEO RUN-UPS ANO PREOICTEO ELEVATION AT COAST VERSUS COASTAL OISTANCE ALONG SOUTHWEST COAST OF MAUI FOR 1960 TSUNAMI

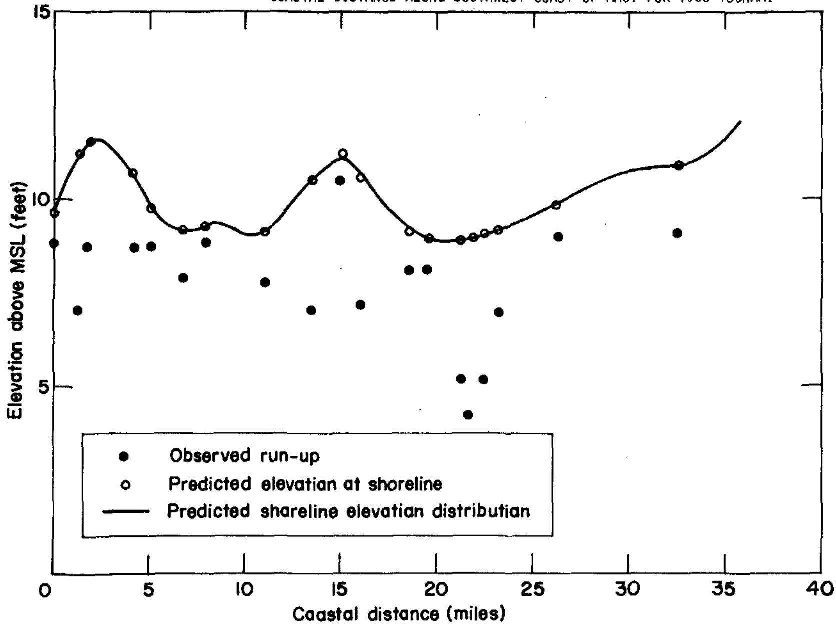


FIGURE 14 COMPARISON BETWEEN REFRACTION COEFFICIENT AS DETERMINED FROM EQUATION (19) AND REFRACTION ANALYSIS FOR SOUTHWEST COAST, ISLAND OF MAUI, FOR 1960 TSUNAMI