Miami Beach

Part 4

COASTAL STRUCTURES AND RELATED PROBLEMS

Miami Beach

CHAPTER 41
WAVE RUN-UP ON COMPOSITE SLOPES

Thorndike Saville, Jr.
U. S. Beach Erosion Board, Corps of Engineers,
Department of the Army

ABSTRACT

A method is presented for determining wave run-up on composite slopes from laboratory-derived curves for single slopes. The method is one of successive approximations and involves replacement of the actual composite slope with a hypothetical single slope obtained from the breaking depth and an estimated run-up value. Comparison of predicted values is made with actual laboratory data.

Accurate design data on the height of wave run-up is needed to determine design crest elevations of protective structures subject to wave action such as seawalls, beach fills, and dams. Such structures are normally designed to prevent wave overtopping with consequent flooding on the landward side and, if of an earth type, possible failure by rear face erosion. Wave run-up (the vertical height to which water from a breaking wave will rise on the structure face) therefore, has an important bearing on the final determination of crest elevation or freeboard.

Apart from the safety factor, decisions as to the necessary crest elevation frequently have considerable economic implication also, as for example in the levees presently being designed for protection with the planned raised water levels in Lake Okeechobee (Florida) where it has been estimated that each additional foot of levee elevation required will cost several million dollars.

Much study by models has recently been devoted to the problem of run-up on structures, both in this country and abroad. The problem for smooth impermeable structures of constant slope has been discussed previously (Saville, 1956). Savage (1957), more recently, has given data on run-up on roughened and permeable structures, but still of a constant slope. Some information has also been given (Saville, 1956) for composite slopes made up of a smooth impermeable structure slope rising from a smooth 1 on 10 (beach) slope which is at or below the still water level. Few structures, however, fit exactly the cases reported, and interpolation or extrapolation of the curves is relatively difficult. Consequently resort frequently is still made to an exact model study of a planned structure to obtain the design values of wave run-up. This is particularly the case where more complex composite slope structures, such as those with berms, are being considered.

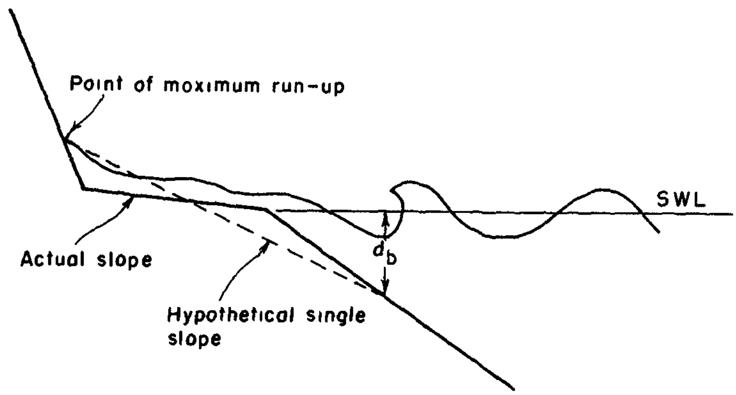

Fig. 1. Schematic of hypothetical single slope for use in determining run-up for composite slopes.

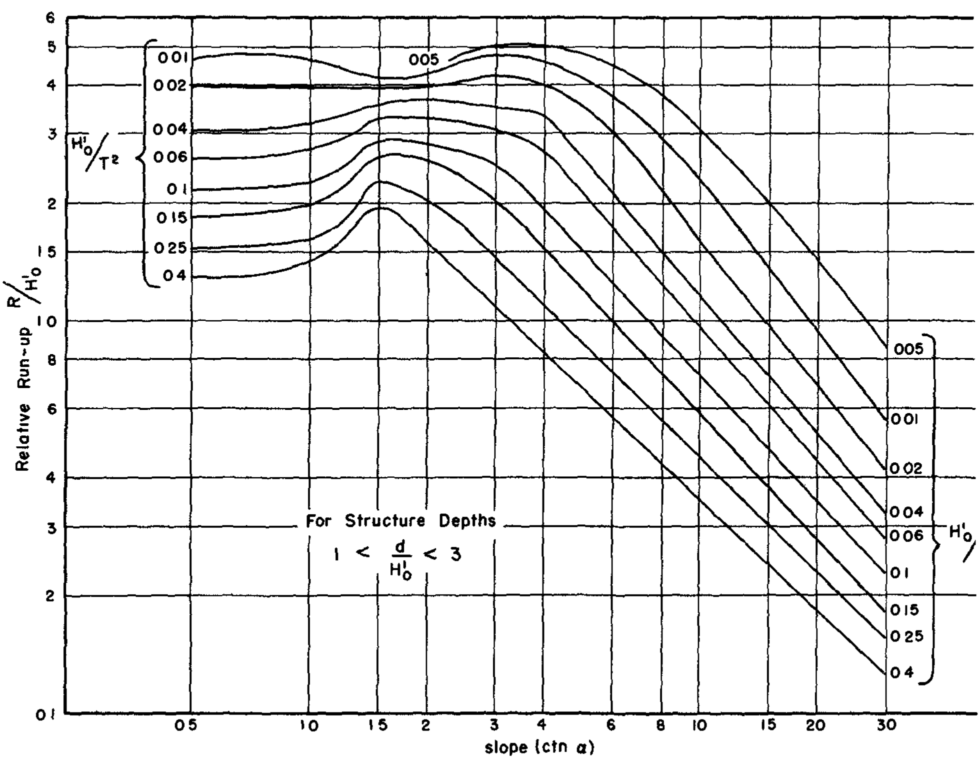

Fig. 2. Run-up on sloped structures.

However, an analysis of existing data shows that this data may be used to predict relatively accurate values of wave run-up for any slope if the actual composite slope is replaced by a hypothetical single constant slope; this hypothetical slope is obtained from the breaking depth and an estimated value of wave run-up. Such a case is shown in Figure 1 where a composite slope consisting of a beach slope, a very gently sloping berm, and a steep structure slope is replaced (dashed line) by a single hypothetical slope extending from the breaking point to an arbitrarily estimated point of maximum run-up. Using this hypothetical slope, a value of run-up may be determined by interpolation from the earlier data. In the general case, the value of run-up determined will be somewhat different from that initially chosen to obtain the hypothetical single slope; the process is then repeated using the new value of run-up to obtain a new single slope value, which in turn determines a new value of run-up. The process is repeated until identical values are obtained for two successive trials.

In order to make the interpolation between the earlier curves somewhat simpler, these curves have been replotted as shown in Figure 2. The case of structure depth, that is depth of water at the toe of the structure, between one and three wave heights is the only one presented, as this range contains the breaking point which would be at the toe of the hypothetical single slope structure used here. The figure shows relative run-up (R/H_o') as a function of structure slope for various values of wave steepness (H_o'/T^2), where R is the wave run-up (the vertical height above still water level to which water will rise on the face of the structure), H_o' is the equivalent deep water wave height, and T is the wave period. It should be noted that the values are given in terms of the deep water wave height (corrected for refraction), H_o'. This value, if not known initially, may be obtained from the non-breaking wave height in any depth of water by using tables of functions of d/L_o (relative depth) as, for example, given by Wiegel (1948) and later reproduced by the Council on Wave Research (1954) and the Beach Erosion Board (1954); or from the breaking depth or breaking height as given by the solitary wave equations (Munk, 1949). These, as rearranged to utilize the generally more available value of wave period, T, rather than the deep water wave length, L_o, are:

$$d_b = 1.28H_b \quad \text{and}$$

$$H_o' = 1.5d_b \, (H_o'/T^2)^{1/3}$$

where d_b and H_b are respectively the breaking depth and height. These same solitary wave equations may also be used to obtain the breaking depth for use in determining the hypothetical single slope used to replace the actual composite slope.

Although actual verification of this method is not shown until later in the paper, to illustrate the method, an actual design example is worked out below for the Jefferson Parish levee on Lake Ponchartrain outside New Orleans, Louisiana. A schematic diagram of the existing levee is shown in Figure 3. The problem was whether this levee would be overtopped by hurricane waves on Lake Ponchartrain, and, if so, how high the existing levee would have to be raised to prevent overtopping. The wind tide level under hurricane winds for a particular choice of design storm was determined as +8 feet mean sea level, as indicated on the figure. It was estimated that waves 7 feet high and of 6.7-second period would be observed one mile offshore where the water depth is 19 feet. The equivalent deep water wave height H_o' may then be found as 7.38 feet by obtaining a value of H/H_o' from tables of functions of d/L_o. For this particular case there are three values of wave run-up that need to be computed. These are (1) that resulting from the 7-foot incident wave breaking on the 1 on 130 slope, (2) that from the smaller wave propagated over the 20-foot berm in 6 feet of water and breaking on the 1 on 4 slope, and (3) that from the still smaller wave propagate across the 40-foot berm in 3 feet of water and breaking directly on the 1 on 8 levee slope. They will be computed below in that order.

(1) Run-up on 1:8 slope for wave breaking on 1:130 slope:
Compute $H_o'/T^2 = 0.164$
From the solitary wave equation (above) compute the depth of breaking, $d_b = 8.99'$
Assume run-up on the 1:8 slope as any value, say 2'
Compute a hypothetical single slope as a vertical rise from -0.99' msl (breaking depth) to +10' (crest of run-up) in a horizontal distance of 40' (1:8 slope = 8 x 5) plus 40' (berm at 3' depth) plus 12' (1:4 slope) plus 20' (berm at 6' depth) plus 389' (1:130 slope = 130 x 2.99) or slope = 10.99/501 = 1:45.6
From Figure 2 (extrapolated) determine $R/H_o' = 0.115$ and compute $R = 0.85'$
Repeat the above computations assuming R = 0.8'; then slope = 9.79/491.4 = 1:50.2
From Figure 2 (extrapolated) determine $R/H_o' = 0.11$; then R = 0.8 approximately. As the computed value of 0.8' agrees with the assumed, then this value is the final computed run-up for thes particular assumptions as to breaking condition.

(2) Run-up on 1:8 slope from stable wave on 20-foot berm where d = 6':
From $d_b = 6'$ using the solitary wave equation, and the same wave period, T = 6.7 seconds, compute $H_o' = 4.02'$ and $H_o'/T^2 = 0.0895$
Assume the wave breaks just at the toe of 1:4 slope
Assume run-up as any value, say, 4 feet
Compute a hypothetical single slope as a vertical rise from +2' msl (depth of breaking) to +12' (crest of run-up) in a horizontal distance of 12' (1:4 slope) plus 40' (berm) plus 56' (1:8 slope = 8 x 7) or slope = 10/108 = 1:10.8
From Figure 2, $R/H_o' = 0.71$ and R = 2.85'
Assume R = 2.85 and repeat, computing slope = 8.85/98.8 = 1:11.2 and $R/H_o' = 0.68$ from which R = 2.74'
Assume R = 2.74' and repeat, computing R again = 2.74', which becomes the final run-up value.

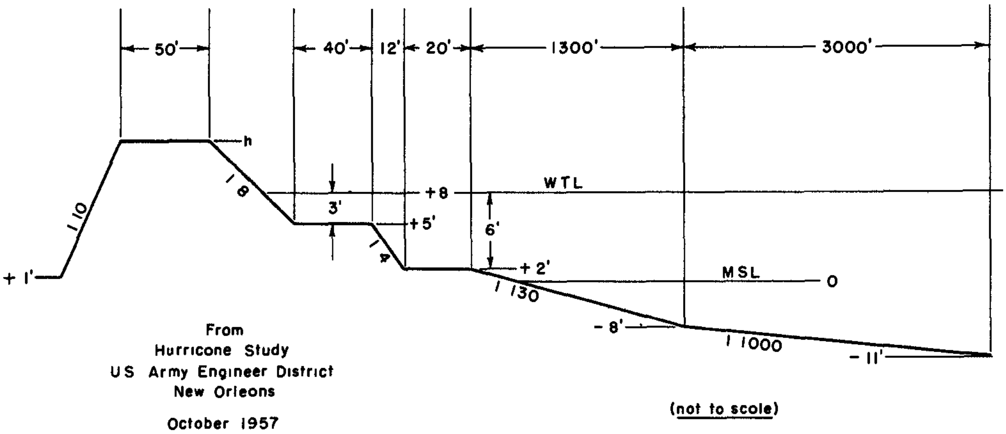

Fig. 3. Schematic of Jefferson Parish levee.

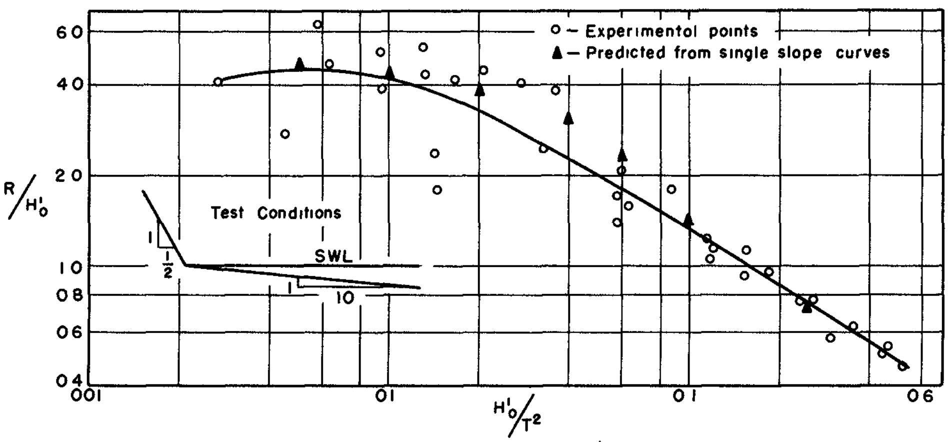

Fig. 4. Wave run-up, composite slope (1 : 1/2 above SWL, 1:10 below SWL).

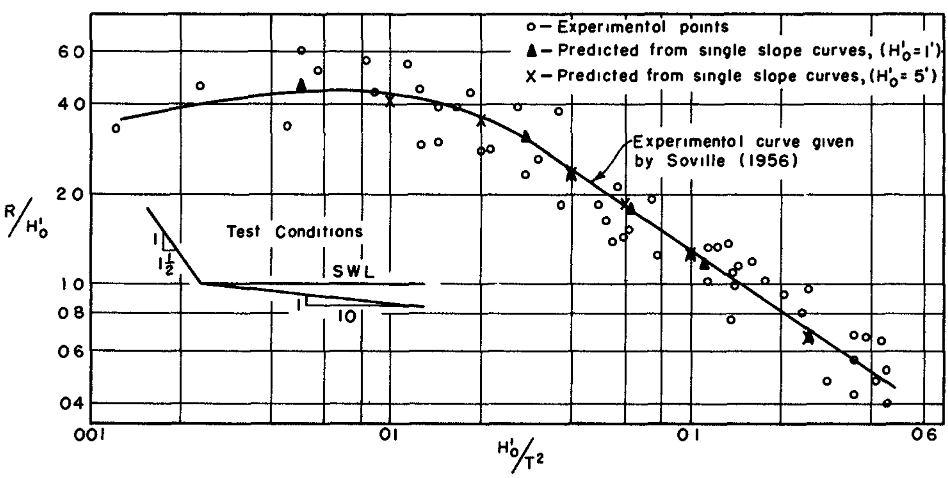

Fig. 5. Wave run-up, composite slope (1 : 1 1/2 above SWL, 1:10 below SWL).

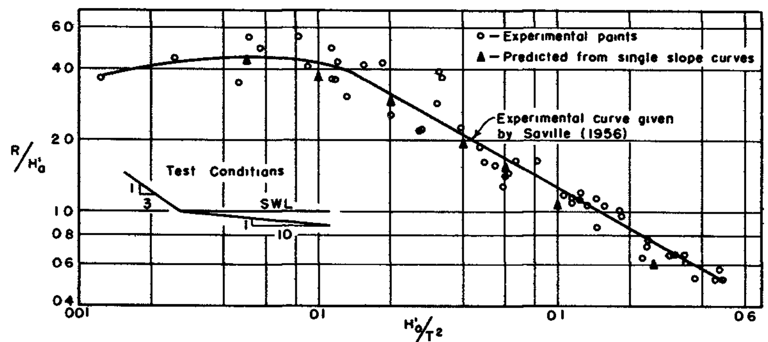

Fig. 6. Wave run-up, composite slope (1:3 above SWL, 1:10 below SWL).

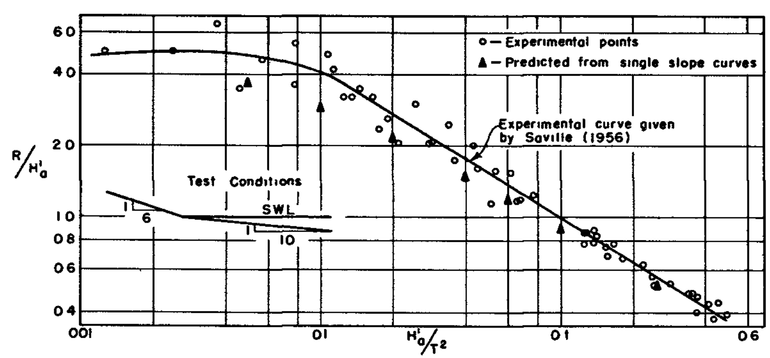

Fig. 7. Wave run-up, composite slope (1:6 above SWL, 1:10 below SWL).

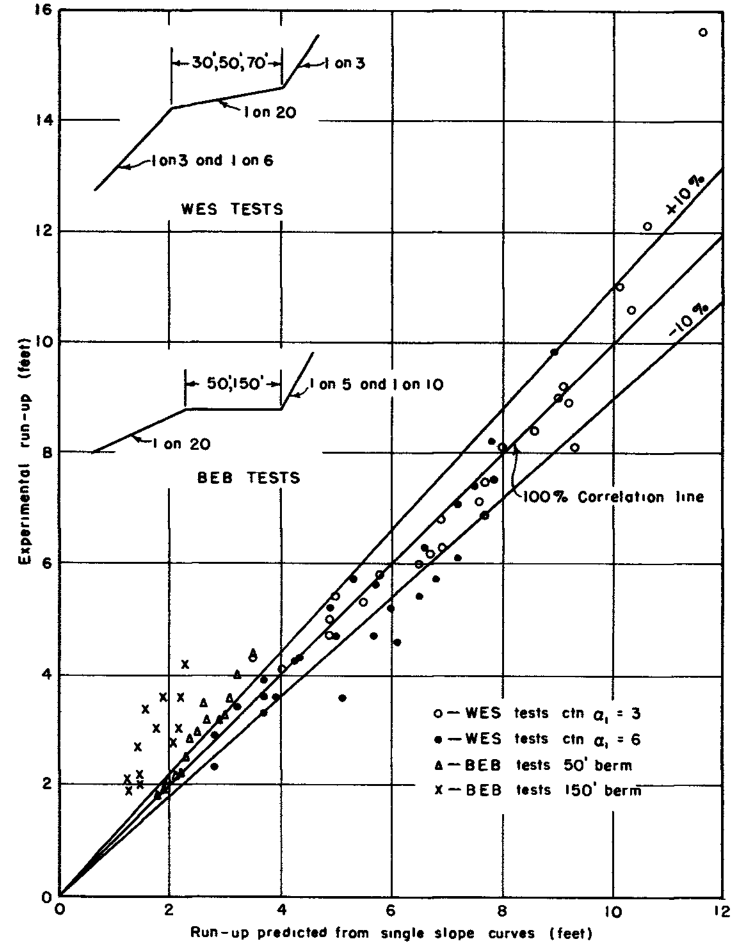

Fig. 8. Comparison with experimental values of run-up for slopes with berms obtained by

(3) <u>Run-up on 1:8 slope from stable wave on 40-foot berm where $d = 3'$:</u>
From $d_b = 3'$ and $T = 6.7$ seconds, compute $H_o' = 1.43'$, and
$H_o'/T^2 = 0.0318$
From Figure 2, $R/H_o' = 1.67$ and $R = 2.39'$

The design run-up for this wave condition is then the maximum of these three values or 2.74 feet. It may be noted that this value is not for the full sized hurricane wave breaking on the outer slope, but for a somewhat smaller wave in the spectrum (or a reformed wave) which can propagate as a stable wave over the deeper berm, and break on the 1:4 slope.

Comparisons have been made for values computed by this method with those determined experimentally in wave flumes for certain cases. These are composite slopes made up of smooth constant impermeable slopes of 1 on 6, 1 on 3, 1 on $1\frac{1}{2}$, and 1 on $\frac{1}{2}$ above still water level and a constant beach slope of 1 on 10 below still water level. The comparisons for these cases are shown in Figures 4 - 7. The experimental data and curves for the 1 on 6, 1 on 3, and 1 on $1\frac{1}{2}$ slopes are those previously reported (Saville, 1956) and data for the 1 on $\frac{1}{2}$ slope are additional unpublished data obtained at the Beach Erosion Board; the curves in all cases were drawn by eye through the general center of the experimental points. Actually the points determined by the hypothetical slope method may be connected to form a curve which can be compared with the experimental curve. As the plotted points represent dimensionless quantities the actual wave height used in determining the points makes no difference in the curves obtained. This may be seen in Figure 5 where values determined from waves of both 1 and 5-foot height are shown. The points or curves predicted by this method vary from somewhat above the experimental curve for the 1 on $\frac{1}{2}$ slope, to almost exactly on the curve for the 1 on $1\frac{1}{2}$ slope, to somewhat below the curve for the 1 on 3 and 1 on 6 slopes. In every case, however, the predicted points lie within the scatter pattern of the experimental points. The deviation of the predicted values from the previously drawn experimental curves is generally within 10 percent with a maximum deviation of about 25 percent.

In addition, a comparison was made with run-up data for a number of structures having berms. These comparisons are shown in Figure 8 where the actual prototype value of predicted run-up is compared with the experimentally determined values from model studies for the Lake Okeechobee levee design reported by Hudson, Jackson and Cuckler (1957) and for beach dune design reported by Savage (1957). The former were tests made at the Waterways Experiment Station of the Corps of Engineers in Vicksburg, Mississippi and involved underwater slopes (α_1) of 1 on 3 and 1 on 6, berms of 30, 50, and 70-foot width on a 1 on 20 slope with the toe at still water level, and upper structure slopes of 1 on 3 (see Figure 8). Those reported by Savage were tests carried out at the Beach Erosion Board and involved an underwater beach slope of 1 on 20, horizontal berms of 50 and 150-foot width, and dune slopes of 1 on 5 and 1 on 10 (see Figure 8). There was also an outer bar involved in these latter tests; the values used herein were restricted to the cases where the wave did not break on passing over the bar. Waves breaking on the bar could also have been used, but additional computations would have been necessary to determine whether the run-up was due to the wave breaking on the bar or to the reformed waves generated in the water area shoreward of the bar. Similarly usage was also restricted to the cases of still water depths over the berm of -2, -1, 0, and 1 feet in an attempt to ensure that the run-up was due to the wave breaking on the beach slope rather than to a reformed smaller wave propagating in the water over the berm.

As may be noted from Figure 8, the agreement of the predicted with the experimental values is fairly good except for the case of the 150-foot berm, where experimental values were considerably higher than those predicted. It is interesting to note that for these cases essentially the same value of run-up was obtained for both the 50-foot and 150-foot berm width. The maximum difference between the two was 0.2 feet for run-ups ranging from 1.9 to 4.2 feet. This would seem to imply that after a berm has reached a certain width, further widening has no significant effect in reducing wave run-up - - at least for horizontal berms. This possibility has previously been indicated by researchers in The Netherlands (van Asbeck, Ferguson, and Schoemaker, 1953) in stating that berms wider than about one-fourth of the wave length, while still reducing the wave uprush, do so at a lesser rate. For the tests reported here one-fourth of the wave length is between 40 and 50 feet. This reduction in effect of berm width may be because in the laboratory tests at least, a definite "set-up" of water occurre on the berm. This "set-up" or increase in mean water level is caused by the forward transport of water by the waves and, for these tests, ranged between 0.9 and 2.4 feet with an average value of 1.7 feet and a most frequent value of 1.8 feet. This "set-up" increased the water depth over the berm appreciably, and in many cases the run-up measured may have been due more to reformed waves or surges in this increased depth than to the actual uprush of the wave. This is partially substantiated by the fact that experimental values for the higher berms (at or above still water level) are more nearly approached by th predicted values than are those for the lower berms where a greater water depth is observed. This "set-up" phenomenon appears to be much more apparent for horizontal berms than for sloping berms, where the water pushed forward by the wave may flow back much more readily. No mention of this occurrence was made in the Vicksburg tests, and the difference between predicted andobserved values for these tests did not appear to be affected by the berm width (which varied from 30 to 70 feet).

Referring again to Figure 8, some 72 percent of the experimental values lie within ± 10 percent of the predicted values if the points for the 150-foot berm are ignored; if these points are included, then 61 percent of the experimental values are within ± 10 percent of the predicted.

In conclusion, a method for predicting wave run-up on any type of composite sloped impermeable structure has been presented. The accura of the method, as judged by comparison with experimentally observed values obtained from laboratory tests, is regarded as satisfactory. It is felt that use of the method will simplify design determination of run-up for many structures. However, further tests are needed to define those cases where width of horizontal berm becomes great enough to affect the validity of the method.

REFERENCES

Beach Erosion Board (1954). Shore Protection Planning and Design: Beach Erosion Board Technical Report No. 4.

Hudson, R. Y., R. A. Jackson, and R. E. Cuckler (1957). Wave Run-up and Overtopping, Levee Sections, Lake Okeechobee, Florida: Waterways Experiment Station, Technical Report No. 2-449.

Munk, W. H. (1949). The Solitary Wave Theory and its Application to Surf Problems: Ann., N.Y. Acad. Sci., Vol. 51, Art. 3, pp. 376-424.

Savage, R. P. (1957). Model Tests of Wave Run-up for Hurricane Project: Bulletin, Beach Erosion Board, V. 11, No. 1, pp. 1-12.

Savage, R. P. (1957). Wave Run-up on Roughened and Permeable Slopes: Paper presented at October 1957 meeting Am. Soc. Civ. Engrs.

Saville, T., Jr. (1956). Wave Run-up on Shore Structures: Proc. Am. Soc. Civ. Engrs., Separate No. 925, Jour. Waterways Div., V. 82, No. WW2.

Wiegel, R. L. (1948). Oscillatory Waves: Bulletin, Beach Erosion Board, Special Issue No. 1.

Wiegel, R. L. (1954). Gravity Waves, Tables of Functions: Council on Wave Research, Engineering Foundation.

Van Asbeck, W. F., H. A. Ferguson, and H. J. Schoemaker (1953). New Design of Breakwaters and Seawalls with Special Reference to Slope Protection: XVIIIth International Navigation Congress, Rome, Section II, Question I, pp. 169-198.