# CHAPTER 38

## UMPQUA JETTY SURVEILLANCE PROGRAM

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## ABSTRACT

Construction methods utilized by the Portland District during its 80-year experience record of building rubblemound jetties are discussed, along with design criteria that have been developed by the Corps of Engineers for required weight of individual stones in primary cover layer of rubblemound jetties. Results of the 1963 and 1964 continuing prototype study of Umpqua River, Oregon, ocean jetty are presented. The primary purpose of the surveillance program is to improve the basic criteria for design and construction of rubblemound jetties along the Oregon coast.

## JETTY CONSTRUCTION METHODS

The Portland District, Corps of Engineers has been in the jetty construction and maintenance business since the early 1880's. Present replacement cost of the ten jetty systems at the mouths of the principal estuaries of Oregon is estimated to be over $150 million. Average annual maintenance costs are approximately $2 million. The jetties are all of the rubblemound class, some 100-percent quarry stone, some with massive concrete terminal blocks, and some with a continuous concrete cap.

Prior to 1949, stone used in construction and maintenance of the jetty systems was distributed by use of dump cars operating along a railroad trestle constructed to an elevation above the jetty crest. As the rubblemound emerged from the water, many of the stones were broken by vertical drops of up to 30 feet. Due to trestle deterioration between rehabilitation contracts, it was usually necessary to remove old and construct new trestles for each repair job. With the advent of increasingly mobile construction equipment the trend was to switch to rubber-tired truck delivery of stone. Use of the last railroad tramway in constructing or repairing a jetty on the Oregon coast was in 1951 on the Umpqua River training jetty. Direct truck delivery of stone has resulted in higher and wider jetty crests to provide suitable haul roads for a long summer working season.

Initially, the truck-haul technique consisted of stones being dumped at the advancing end of the jetty then shoved off the crest with a dozer. This produced a pellmell type of rubblemound that was an improvement over the railroad method because of the compaction effected by the haul equipment along the crest, more uniform placement along the slope, and a reduction in stone breakage. However, side-slope armor stones could not be positioned to obtain the most desirable keying effect. During this period, some resetting of armor-layer face stones was accomplished by mobile cranes.

Since 1959 the truck-haul technique has incorporated the use of crane placement of all armor stone with specifications calling for the longest axis of stone to be placed normal to the jetty surface and the stones to be placed and arranged to secure the least volume of voids in the structure.

The preceding discussion of construction methods has made liberal use of material from a paper, "'Placed-Stone Jetty," Stone-Weight Coefficients," by Kidby, Powell, and Roberts, presented at A.S.C.E. Environmental Engineering Conference in February 1963, and published by A.S.C.E. Waterways and Harbors Division as Proceedings No. 4134.

## JETTY DESIGN CRITERIA

The guide equation generally used for determination of the weight of armor stone for rubblemound jetties which best relate the principal variables (side slope, specific weight of armor stone, specific weight of fluid media, incident wave forces, shape of armor units, roughness of armor units, and degree of interlocking between armor units) effecting jetty stability is:

$$W_r = \frac{\gamma_r H^3}{K_\Delta (S_r - 1)^3 \cot\alpha}$$

where: $W_r$ = Weight of armor unit in primary cover layer, pounds,

$\gamma_r$ = Unit weight of armor unit, lbs./ft.$^3$,

$H$ = Design wave height,

$S_r$ = Specific gravity of armor unit, relative to the water in which structure is situated, $S_r = \frac{\gamma_r}{\gamma_w}$,

$\gamma_w$ = Unit weight of water; fresh water 62.4 lbs./ft.$^3$, sea water 64.0 lbs./ft.$^3$,

$\alpha$ = Angle of breakwater slope, measured in degrees from horizontal,

$K_\Delta$ = Coefficient that varies primarily with the shape of the armor units, roughness of the surface, sharpness of edges, and degree of interlocking.

The above equation was developed by Mr. R. Y. Hudson, U. S. Army Engineer Waterways Experiment Station. Details of the development can be found in Waterways Experiment Station Research Report No. 2-2 (July 1958). The development utilized a mathematical approach along with small-scale model tests to evaluate the no-damage $K_\Delta$ coefficient for various stone weights, side slopes, and design wave heights for the condition of non-breaking waves approaching the trunk at $90^{\circ}$ and no overtopping. It was found that for the conditions tested, variation of the ratio of water depth to wave length had insignificant effect upon the results. The study developed a method whereby the engineer could design a rubblemound jetty to withstand wave attack, or could make a qualitative evaluation of damage to be expected should the design wave exceed that for which a practical design could be developed.

However, with a design wave of 25 feet, which is common for Oregon jetties, side slopes limited to 1 on 2 because of available construction equipment, and with breaking wave conditions, the recommended jetty head no-damage coefficient of 2.5 for pellmell placement used in the Hudson formula yields an armor stone in the primary cover layer of 58 tons for rock weighing 170 pounds per cubic foot. Experience records show that it is economically feasible to specify a class of armor stone for the jetty head and critical points in the trunk ranging from 15 to 40 tons, with 50 percent of the stone weighing at least 22 tons. For minimum cover-layer stone weight of 22 tons (which can feasibly be accomplished) the maximum no-damage wave height would be approximately 18 feet. The 25-foot-high wave would then produce damage in the range of 10 to 20 percent, using Hudson's study as a guide.

It was felt that the Portland District's special placement method was superior to the pellmell method. Therefore, the district requested Waterways Experiment Station to determine the effect of the special placement method upon the stability coefficient, $K_\Delta$, in the Hudson formula. Results of the ensuing scale-model investigation were published in Waterways Experiment Station's Technical Report No. 2-631, July 1963, titled "Stability of South Jetty, Siuslaw River, Oregon." The published model data indicate that a no-damage, $K_\Delta$, coefficient of approximately 7 can be used for quarry-stone, special-placement, jetty head construction where the number of armor stone layers is four. With the special placement method, using $K_\Delta$ of 7, design wave height of 25 feet, and side slope of 1 on 2, the Hudson formula yields an armor unit weight in the primary cover layer of about 21 tons for 170-pounds-per-cubic-foot stone.

## UMPQUA PROTOTYPE STUDY

The primary purpose of the Umpqua surveillance program is to improve basic criteria for design and construction of rubblemound jetties. The location for the study at the mouth of Umpqua River, Oregon, was considered typical as to oceanographic and estuarine conditions for the coast of Oregon. Since the Umpqua south jetty was rehabilitated in 1963 to its outer end, using the special placement design and construction methods described above, it was selected for the prototype study.

The program is a continuing study to obtain a record of changes in the physical configuration of the jetty and surrounding hydrography of the beach and shore. The changes are correlated to the hydraulic forces of incident ocean waves and tidal, littoral, and river currents. Jetty configuration changes related directly to incident ocean waves of known height and duration can then be utilized to evaluate the special placement construction of the prototype armor layer, and the results compared with the previously discussed model predictions.

As-built and annual surveys were programmed to include space location of individual stone in the primary armor layer at four trunk stations and at the jetty head, average top elevations of cross sections at 500-foot intervals along the jetty, average top elevations of jetty profile, hydrography of wave approach area to the jetty, and location of mean lower low water and mean higher high water along the shoreline affected by the south jetty. Wave information was obtained from onsite wave recordings and from the Fleet Numerical Weather Facility's daily hindcast data.

## 1963-64 SURVEY COMPARISON

Hydrographic characteristics of the jetty area are shown for the 1963 and 1964 surveys on figures 1 and 2, respectively. Survey data inshore of 4 fathoms were obtained in September 1963 and May 1964 and clearly show the seasonal effect upon the bottom contours, the May configuration being the result of heavy winter storm surf and smooth September bottom resulting from the long-period low swells of the summer season. The scour hole adjacent to the north side of the jetty head has obtained depths comparable to those obtained in 1940, 2 years after the south jetty was extended in 1938. The primary cause of this scour hole appears to be ebb flow impingement upon the raised jetty. No significant bottom changes occurred seaward of 4 fathoms.

A comparison of 1963 and 1964 average top elevations of jetty cross sections and profiles showed a loss of some stone from the toe of the jetty but no discernible loss of stone above mean lower low water. There was a subsidence of the crown which indicated both jetty consolidation and loss of the road-topping material used in jetty construction.

The 1963 and 1964 coordinates and elevations for three permanently marked points on each rock in a 15- to 20-foot-wide band of the outer armor layer above mean lower low water at four jetty stations (stations 86+80, 94+00, 100+70, and 101+50) and the jetty nose centerline (table 1) were compared. A minor settlement and consolidation, with stone movement generally limited to less than 2 feet, was indicated. An exception to this occurred on the south side of the jetty at station 86+80. Several marked rocks which were originally well-keyed together had moved down the slope approximately 7 feet. Since no concentrated wave attack was evidenced from the refraction investigation of this area, and scour is shown in a hydrographic survey comparison, it is believed that the sloughing at station 86+80 resulted from toe scour. It was noted during construction of this first marked section that the contractor increased the section width which caused a slight bulge in the now-damaged area. At the time it was not considered significant enough to require rebuilding, but the misalinement is now believed to be the cause of excessive local toe scour.

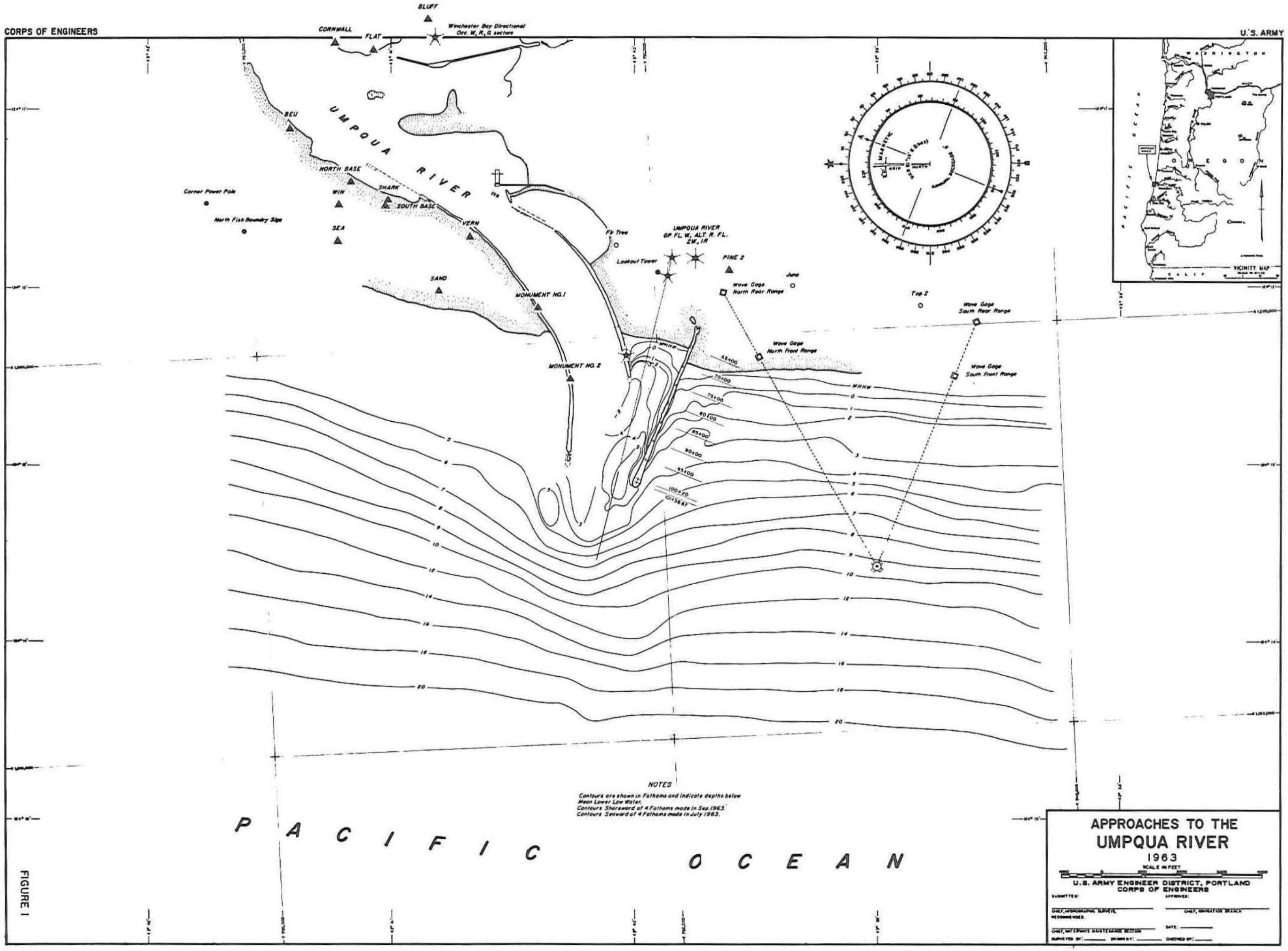



Figure 1. Area Plan with 1963 Hydrographic Features

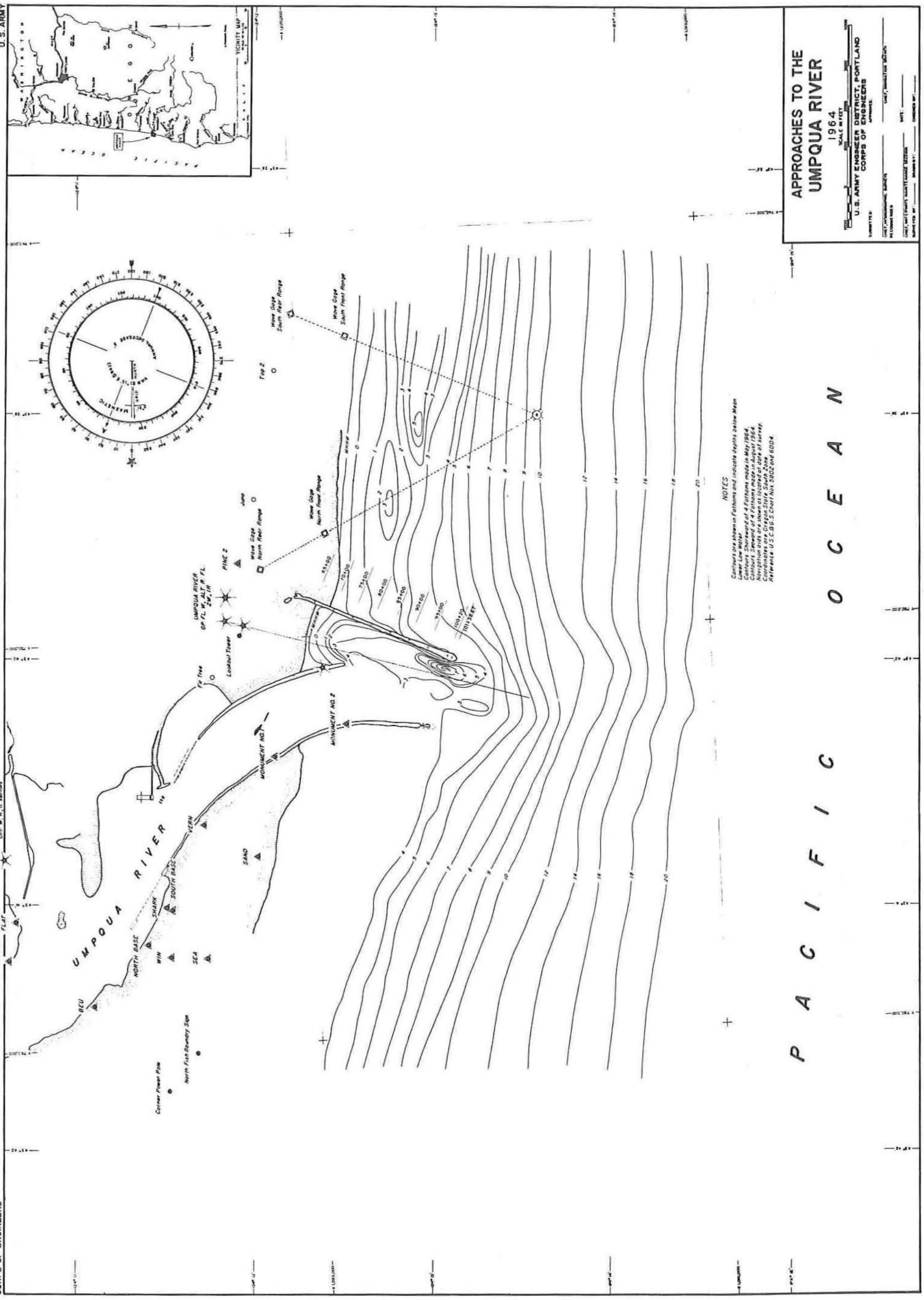



Figure 2. Area Plan with 1964 Hydrographic Features

TABLE 1

1963-64 Rock Movement of Outer Armor Layer of Jetty Head

UMPQUA RIVER, OREGON
SOUTH JETTY SURVEILLANCE PROGRAM

JETTY STATION ________
SECTION Y-1
LOCATION NOSE CENTER LINE

| ROCK NO. | WT. IN TONS | DENSITY | 1963 BASE COORDINATES | | 1963 BASE | 1964 CHANGE FROM BASE | | | 1965 CHANGE FROM BASE | | | 1965 ANNUAL CHANGE | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | N | E | EL. | N | E | EL. | N | E | EL. | N | E | EL. |
| 1A | 41 | 170 | 750674.27 | 1016437.87 | 26.56 | -0.3 | 0.6 | -1.5 | | | | | | |
| B | | | 68.02 | 31.57 | 25.99 | 0.6 | 0.0 | -1.0 | | | | | | |
| C | | | 74.14 | 31.95 | 26.45 | -0.2 | 0.5 | -1.0 | | | | | | |
| 2A | 20 | 170 | 750666.64 | 1016429.39 | 21.56 | 0.0 | -0.3 | -0.9 | | | | | | |
| B | | | 67.15 | 27.09 | 22.41 | 0.2 | -0.4 | -1.7 | | | | | | |
| C | | | 69.43 | 26.29 | 21.15 | 0.1 | 0.0 | -1.8 | | | | | | |
| 3A | 29 | 170 | 750675.37 | 1016427.89 | 22.79 | -0.1 | 1.0 | -1.0 | | | | | | |
| B | | | 71.98 | 25.40 | 19.38 | 0.2 | 0.5 | -0.6 | | | | | | |
| C | | | 74.19 | 22.50 | 19.38 | -0.5 | 1.4 | -0.3 | | | | | | |
| 4A | 39 | 170 | 750669.81 | 1016422.55 | 17.85 | -0.6 | 0.1 | -0.6 | | | | | | |
| B | | | 68.56 | 19.61 | 20.27 | 0.0 | -0.6 | -0.9 | | | | | | |
| C | | | 71.62 | 19.60 | 18.13 | 0.0 | -0.5 | -0.9 | | | | | | |
| 5A | 34 | 170 | 750677.75 | 1016418.20 | 17.97 | - | - | - | | | | | | |
| B | | | 76.27 | 16.14 | 16.39 | 0.6 | 0.0 | -1.2 | | | | | | |
| C | | | 79.94 | 15.53 | 16.15 | 0.6 | 0.3 | -1.5 | | | | | | |
| 6A | 33 | 170 | 750674.52 | 1016411.93 | 14.77 | 0.5 | 0.2 | -1.1 | | | | | | |
| B | | | 74.72 | 08.80 | 14.65 | 0.9 | 0.2 | -1.2 | | | | | | |
| C | | | 78.67 | 09.59 | 13.61 | -0.1 | 1.6 | -1.2 | | | | | | |
| 7A | 32 | 170 | 750686.64 | 1016414.81 | 13.25 | 0.5 | 0.6 | -1.2 | | | | | | |
| B | | | 81.72 | 13.04 | 14.03 | 0.6 | 0.5 | -1.6 | | | | | | |
| C | | | 85.19 | 11.05 | 15.50 | - | - | - | | | | | | |
| 8A | 29 | 170 | 750691.34 | 1016411.26 | 12.15 | 1.6 | -0.6 | -2.0 | | | | | | |
| B | | | 86.73 | 07.71 | 12.40 | 2.1 | -1.2 | -2.5 | | | | | | |
| C | | | 91.19 | 06.60 | 10.58 | - | - | - | | | | | | |
| 9A | 36 | 162 | 750683.93 | 1016406.90 | 10.83 | 0.2 | 0.6 | -0.8 | | | | | | |
| B | | | 82.74 | 03.58 | 11.92 | 0.3 | 0.6 | -0.8 | | | | | | |
| C | | | 86.90 | 04.25 | 11.48 | 0.4 | 0.7 | -0.9 | | | | | | |

Note: Density in lbs. per cu. ft. Elevations in feet MLLW. Changes in feet.

WAVE GAGE-WAVE DATA

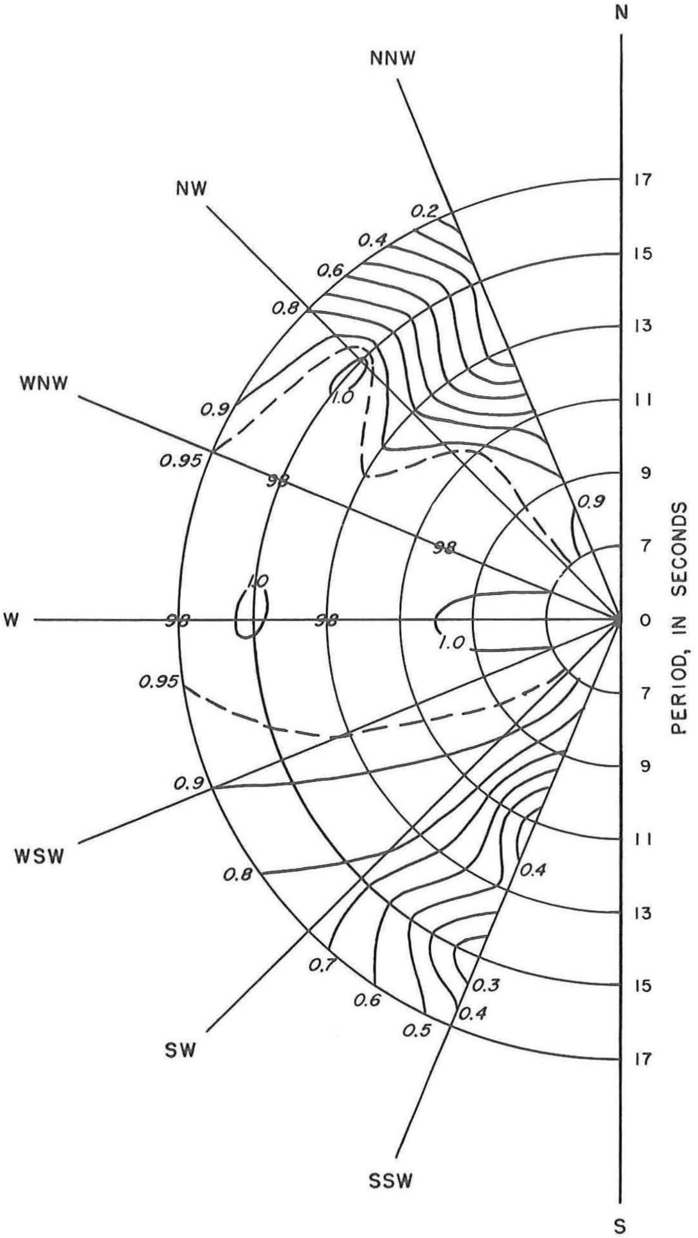
The wave-gage pressure cell is located one mile south of the south jetty where the water depth is 60 feet. The cell is 40 feet off bottom at the top of a steel support pile. A four-conductor marine cable connects the cell with a pen recorder and a magnetic tape recorder which are located ashore. The magnetic tape recorder is programmed to run continuously, while the pen recorder operates 20 minutes every 4-hour period. The records are put through the Coastal Engineering Research Center's wave analyzer from which significant wave heights, periods, and durations are obtained for the wave-gage location. The deepwater wave direction for any particular set of storm waves is interpolated from Fleet Numerical Weather Facility hindcast data. Refraction coefficients (figures 3 to 6) at the wave-gage location and jetty stations are applied to the significant wave heights at the wave-gage location along with proper shoaling coefficients to obtain the corresponding significant wave heights at the jetty stations under consideration. Should direction of wave attack at the jetty be required, figures 7 to 9 are consulted.

The surveillance program is dependent upon reliable deepwater wave heights, periods, directions, and durations offshore from the jetty. Until the summer of 1965, very little data of these types have been measured. The most favorable of sea conditions are required for wave-gage installation and repair work. Since the program was initiated, the gage has been operable only during the month of August 1964. Significant wave heights, periods, directions, and durations during the fall of 1963 and winter of 1963-64 were interpolated from Fleet Numerical Weather Facility data. Seas during October 1963 exceeded 14 feet from southwest, in November from south-southwest, in January 1964 from west, in March from west and west-southwest, and in April from north-northwest. Swells exceeded 14 feet in both October and November from northwest through west, and in April from west. No correlation between wave heights and jetty damage was made as it appeared that the damage that did occur was from scour rather than from direct wave attack.

It is believed that, after several more years of surveillance, analysis of program data will produce knowledge of the exact factors which caused the major deterioration of each jetty area or zone. With this knowledge, evaluations can be made of rubblemound jetty design and construction methods such as layout with respect to beachline; layout with respect to estuarine flow; need for filter blanket as related to toe scour; effects of irregular alinement of jetty toe; and, of course, the various variables of the Hudson design formula with emphasis on evaluation of the special placement of armor layer stone on the stability coefficient, $K_\Delta$.

REFERENCES

Kidby, H. A.; Powell, S. B.; and Roberts, A. L. (1963). "Placed-stone" jetty stone-weight coefficients, Proceedings No. 4134, A.S.C.E. Waterways and Harbors Division.




UMPQUA RIVER, OREGON
SOUTH JETTY

REFRACTION COEFFICIENTS
AT WAVE GAGE

FALL 1964

Figure 3. Refraction Coefficients at Wave Gage

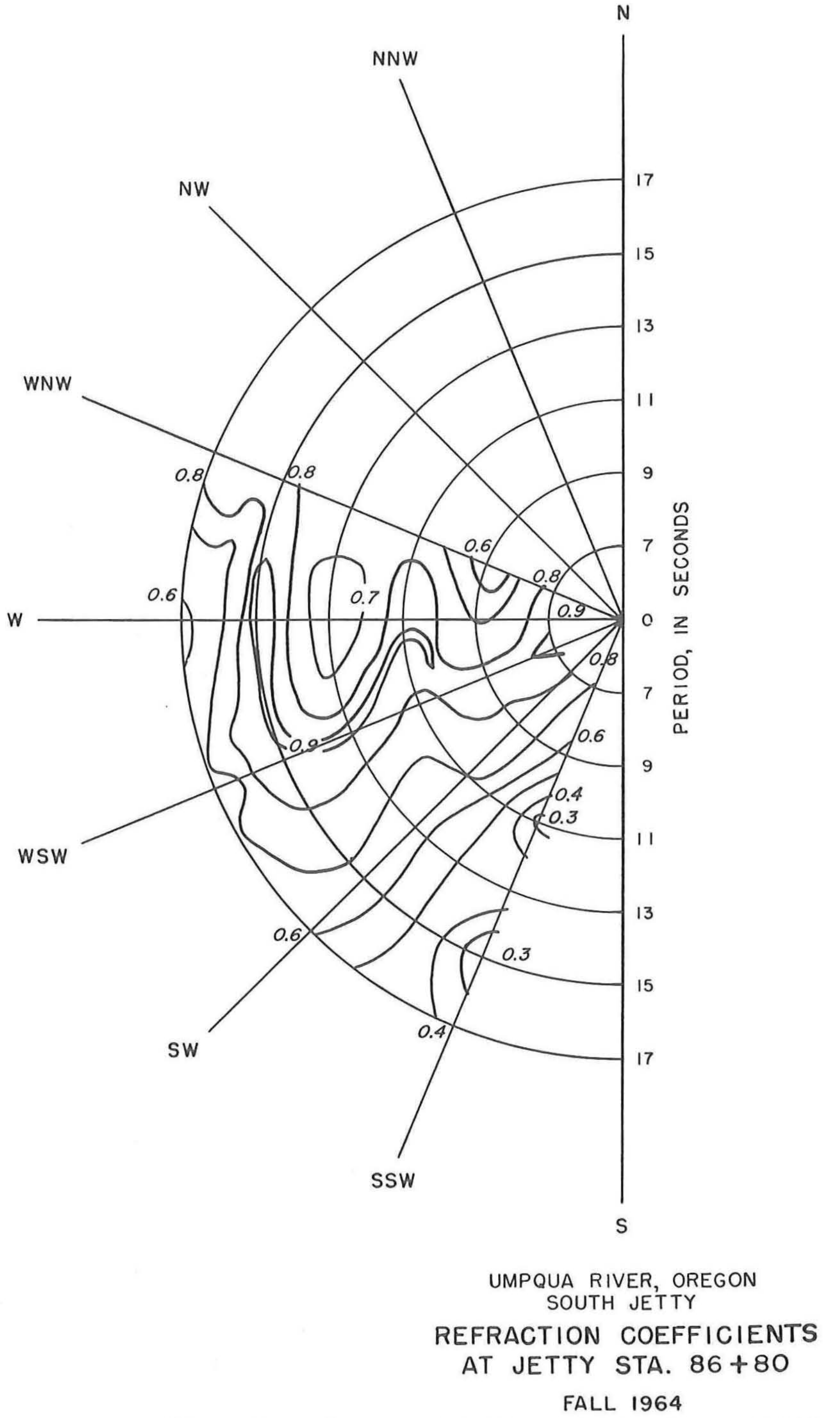
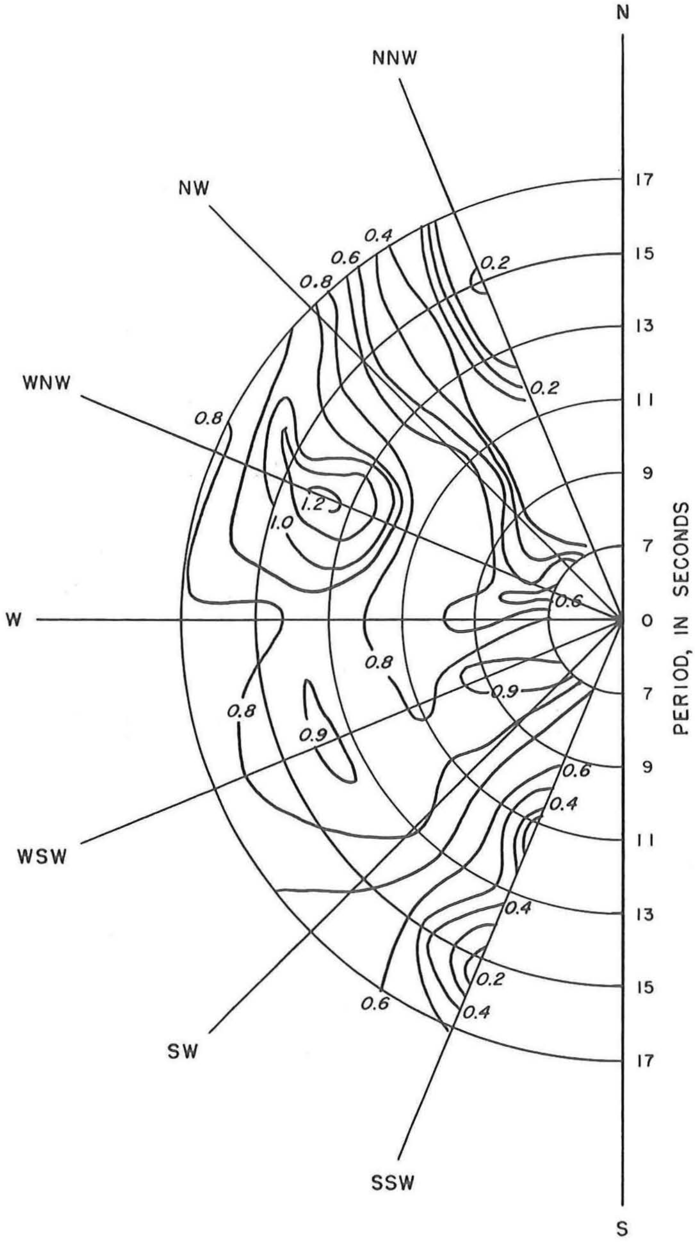
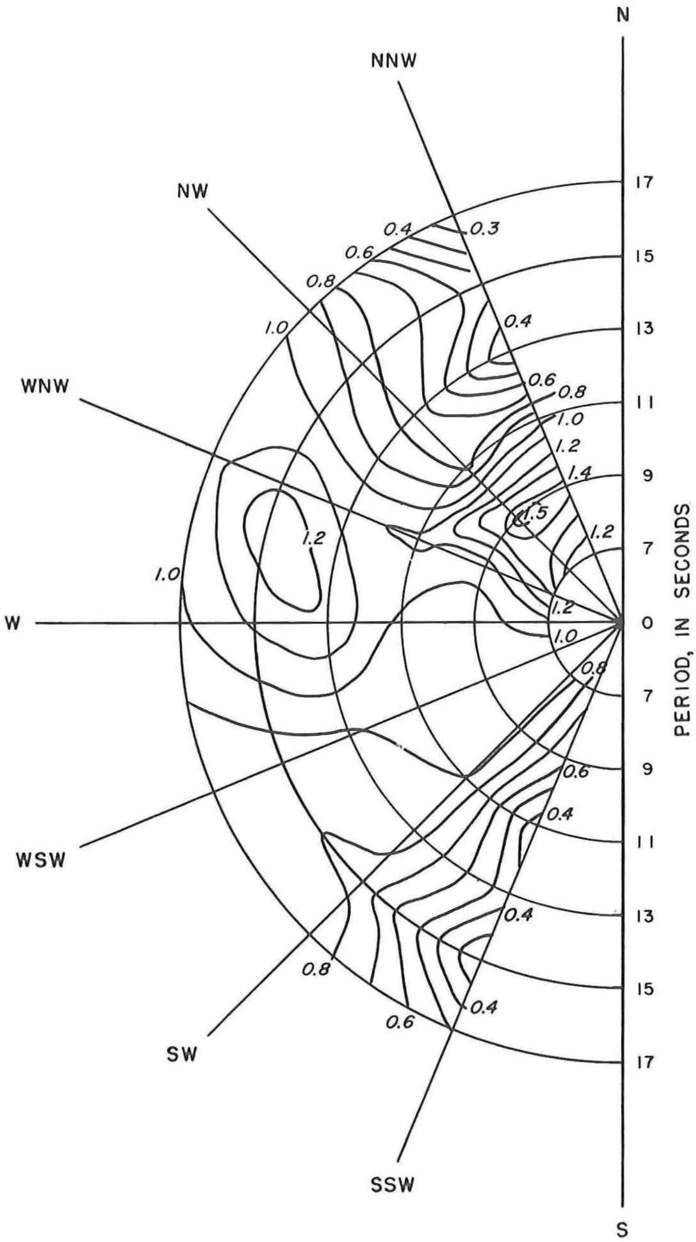



Figure 4. Refraction Coefficients at Jetty Station 86+80




UMPQUA RIVER, OREGON
SOUTH JETTY
REFRACTION COEFFICIENTS
AT JETTY STA. 94+00
FALL 1964

Figure 5. Refraction Coefficients at Jetty Station 94+00




UMPQUA RIVER, OREGON
SOUTH JETTY

REFRACTION COEFFICIENTS
AT JETTY STA. 101+50

FALL 1964

Figure 6. Refraction Coefficients at Jetty Head (Station 101+50)

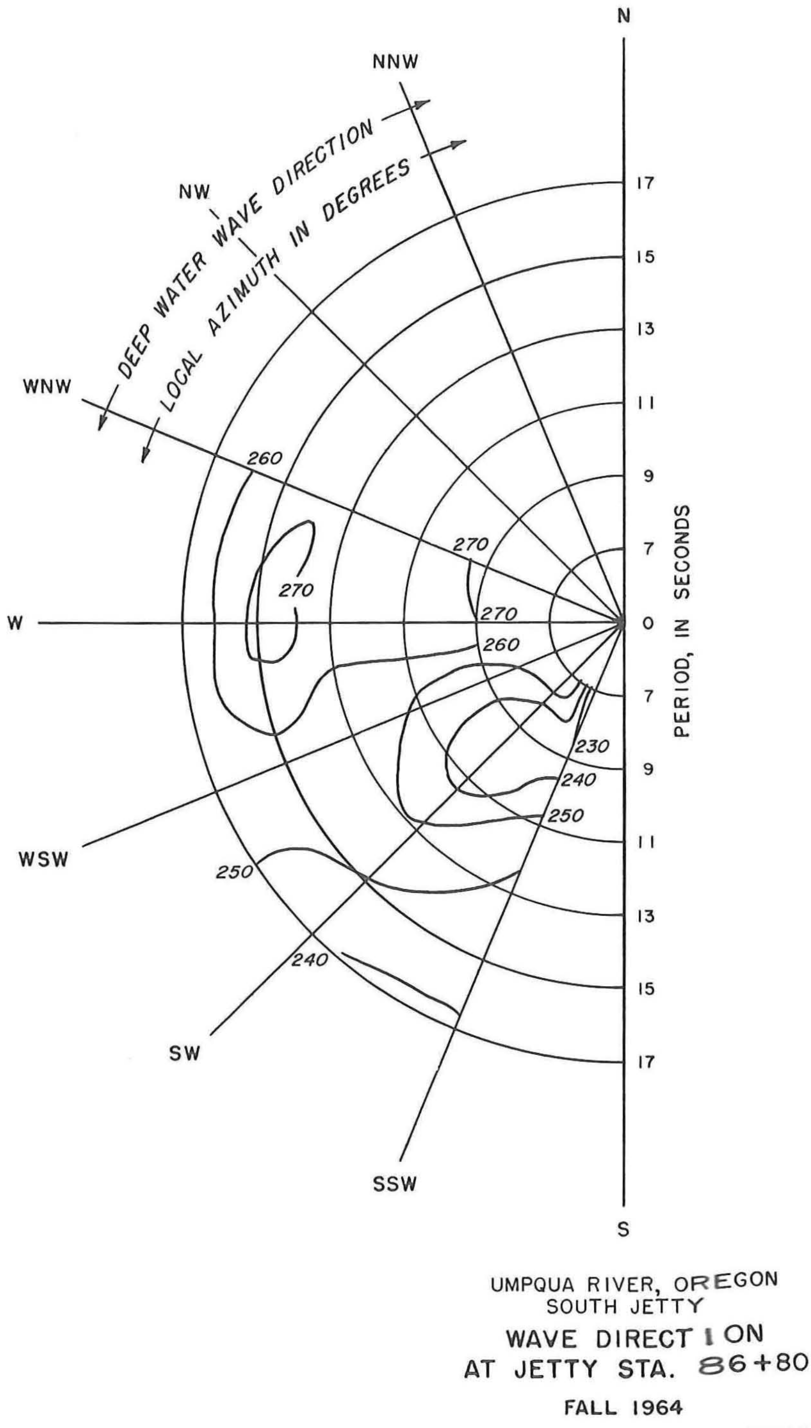



Figure 7. Wave Direction at Jetty Station 86+80

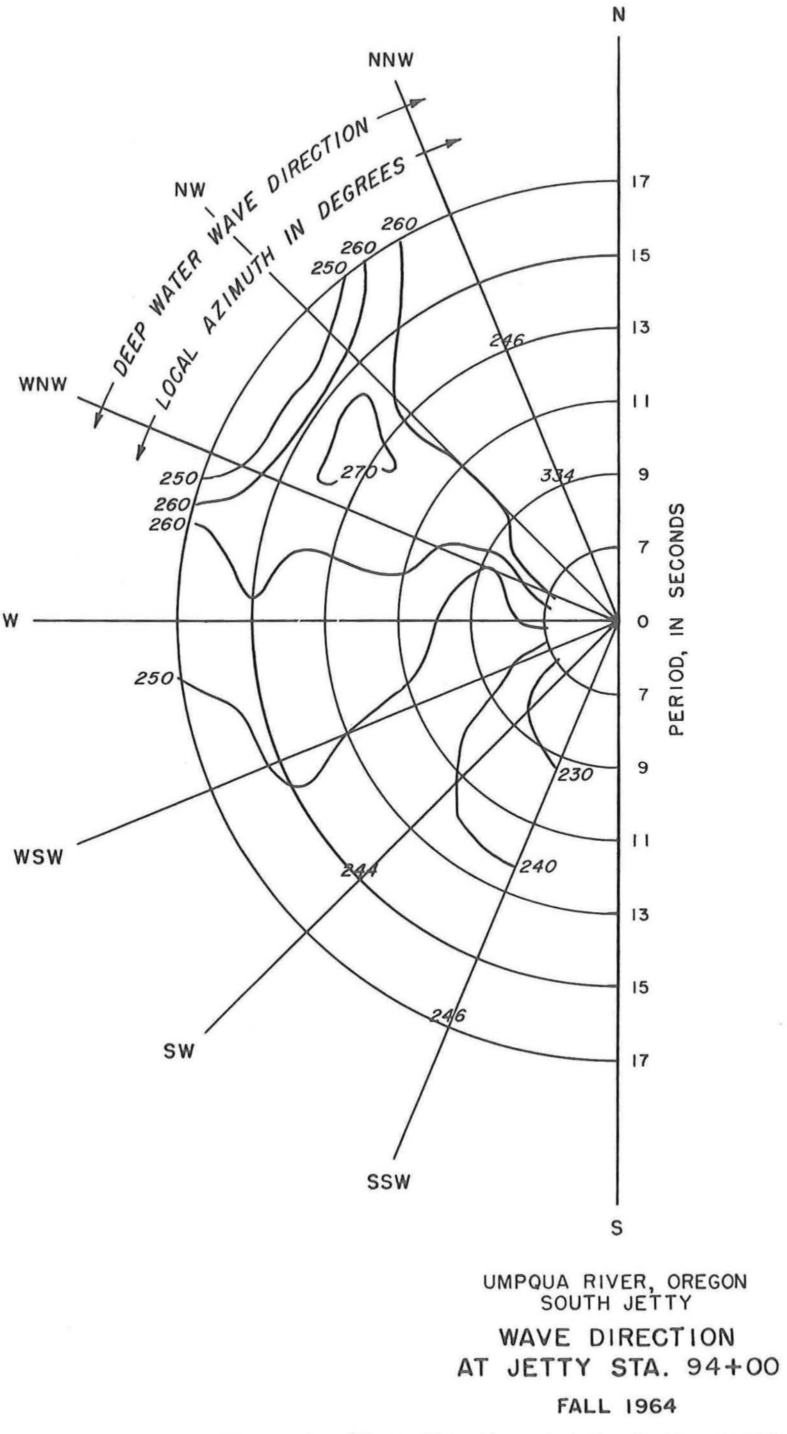



Figure 8. Wave Direction at Jetty Station 94+00

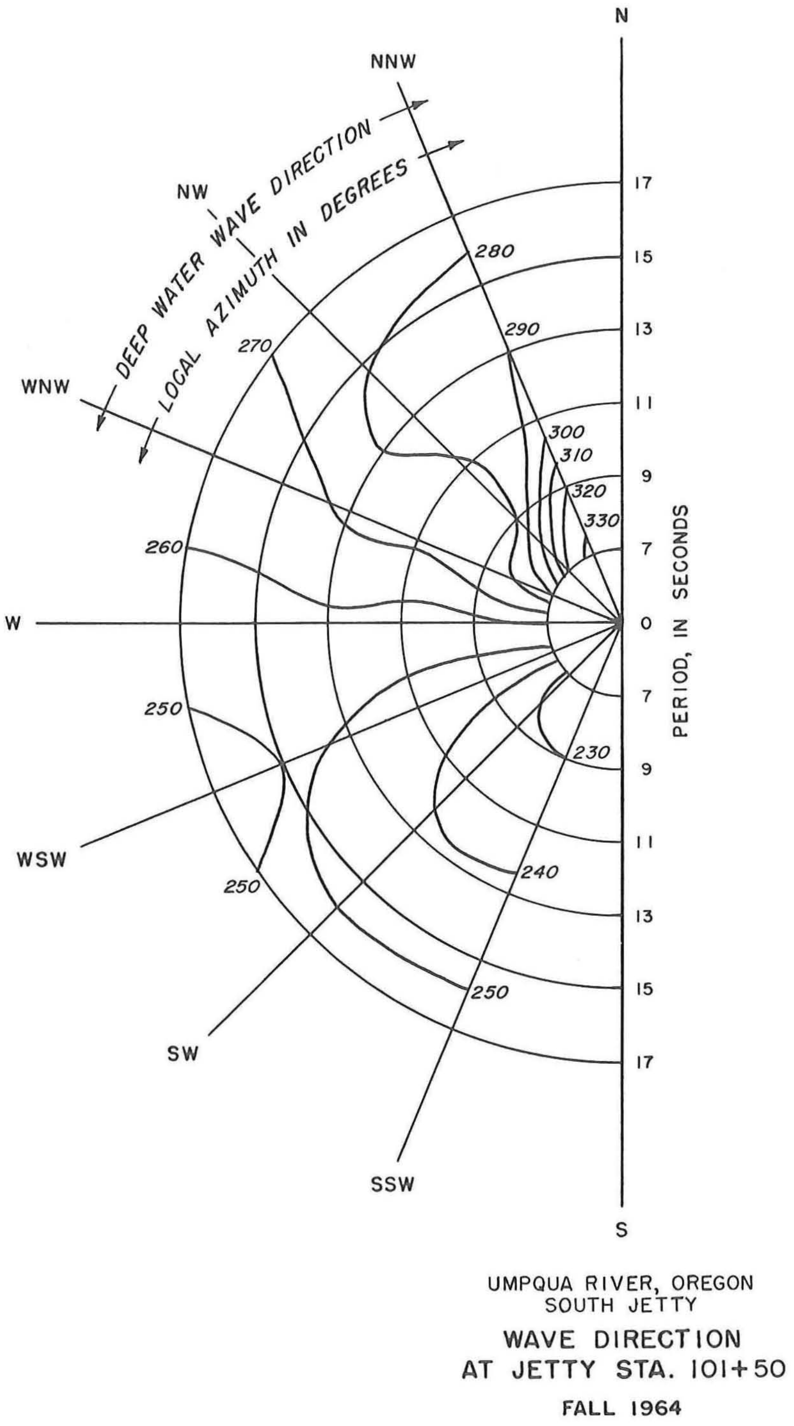



Figure 9. Wave Direction at Jetty Head (Station 101+50)

Waterways Experiment Station (1958). Design of quarry-stone cover layers for rubblemound breakwaters, Research Report No. 2-2, Corps of Engineers, Vicksburg, Mississippi.

Waterways Experiment Station (1963). Stability of south jetty, Siuslaw River, Oregon, Technical Report No. 2-631, Corps of Engineers, Vicksburg, Mississippi.

ACKNOWLEDGMENT

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