4-1988

Comprehensive Condition Survey

United States Army Corps of Engineers

US Army Corps
of Engineers
Los Angeles District

Main Report and
Appendices A and B

COMPREHENSIVE CONDITION SURVEY

Inner Breakwater and the Sand Barrier
Crescent City Harbor
Del Norte County, California

DRAFT

April 1988

COMPREHENSIVE CONDITION SURVEY OF CRESCENT CITY HARBOR INNER BRAKWATER AND SAND BARRIER

Prepared by

U.S. Army Corps of Engineers

Los Angeles District

For

U.S. Army Corps of Engineers

San Francisco District

April 1988

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EXECUTIVE SUMMARY

The major storms of the winter of 1983, caused significant damage to coastal structures along the entire California coast. Prior to making repairs, a comprehensive condition survey of each structure was ordered. These surveys were comprehensive evaluations of the structures, designed to lead to (1) effective short-term repair and maintenance efforts and (2) long-term programs for inspection and monitoring of structures.

This condition survey of Crescent City Harbor Inner Breakwater and Sand Barrier was made during 1986 and the spring of 1987. It contains recommendations for repairs to some segments of the sand barrier and a recommendation for long-term monitoring.

There were very few problems identified at Crescent City Harbor Inner Breakwater caused by the 1983 winter storms. The breakwater and sand barrier were found to be generally sound. The evaluation of overall breakwater stability determined that the structures face no major long-term stability problems.

COMPREHENSIVE CONDITION SURVEY
CRESCENT CITY HARBOR INNER BREAKWATER
AND SAND BARRIER

1. INTRODUCTION

1.1 BACKGROUND TO SURVEY

In 1983, major storms caused damages to many coastal structures including the inner breakwater and the sand barrier in the Crescent City Harbor.

This condition survey has two objectives. First, the present condition and make up of the breakwater and the sand barrier has been established. Second, a long-term program for monitoring and inspection of the two structures has been developed. To meet these goals, many factors were evaluated, including wave forces, erosive currents, foundation condition and the structures themselves. A long-term benefit from this study and related study of the condition of other coastal structures, would be a better understanding of how to design, build, and maintain such structures in areas subject to very high wave forces.

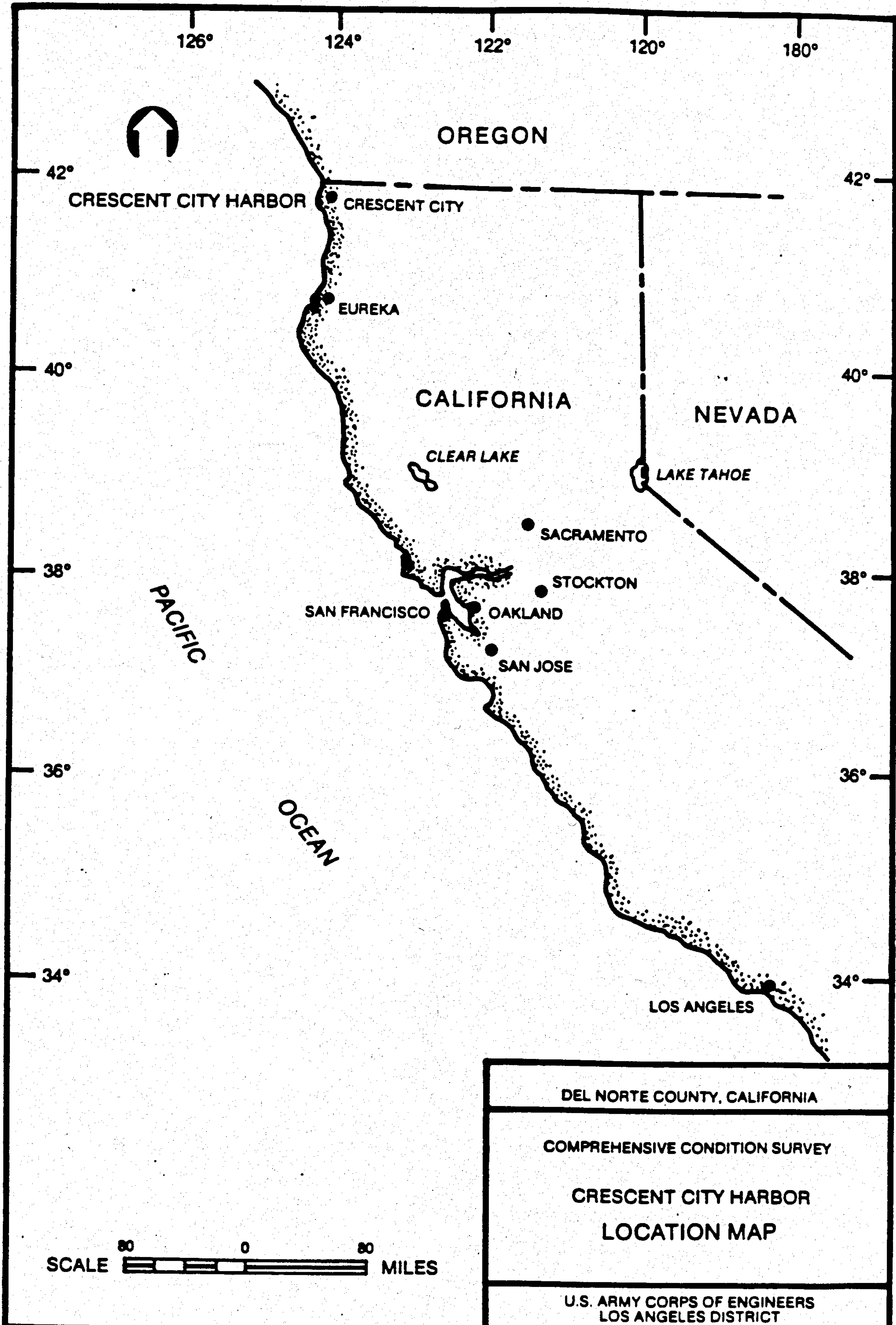
1.2 AUTHORIZATION

The report has been prepared under authority of the Public Law 98-8 (H.R. 1718); 98th Congress, First Session, March 24, 1983.

1.3 SCOPE OF STUDY

The first goal of the condition survey was to determine the present condition and make-up of the breakwater and the sand barrier. To accomplish this:

(1) An intensive review of all previous reports was made. Data on design, construction, damage history, and repairs were collected and organized. Data about materials and their sources were cataloged. Comparing historical data to field data collected during this study made it possible to determine, for example, whether stone was deteriorating (under stress). In addition, this data made it possible to track stone movement due to storms. From this review of historical data, it was possible to identify forces which had the potential to damage the breakwaters, and thus to evaluate the stability of the breakwaters.



DEL NORTE COUNTY, CALIFORNIA

COMPREHENSIVE CONDITION SURVEY

CRESCENT CITY HARBOR

LOCATION MAP

U.S. ARMY CORPS OF ENGINEERS
LOS ANGELES DISTRICT

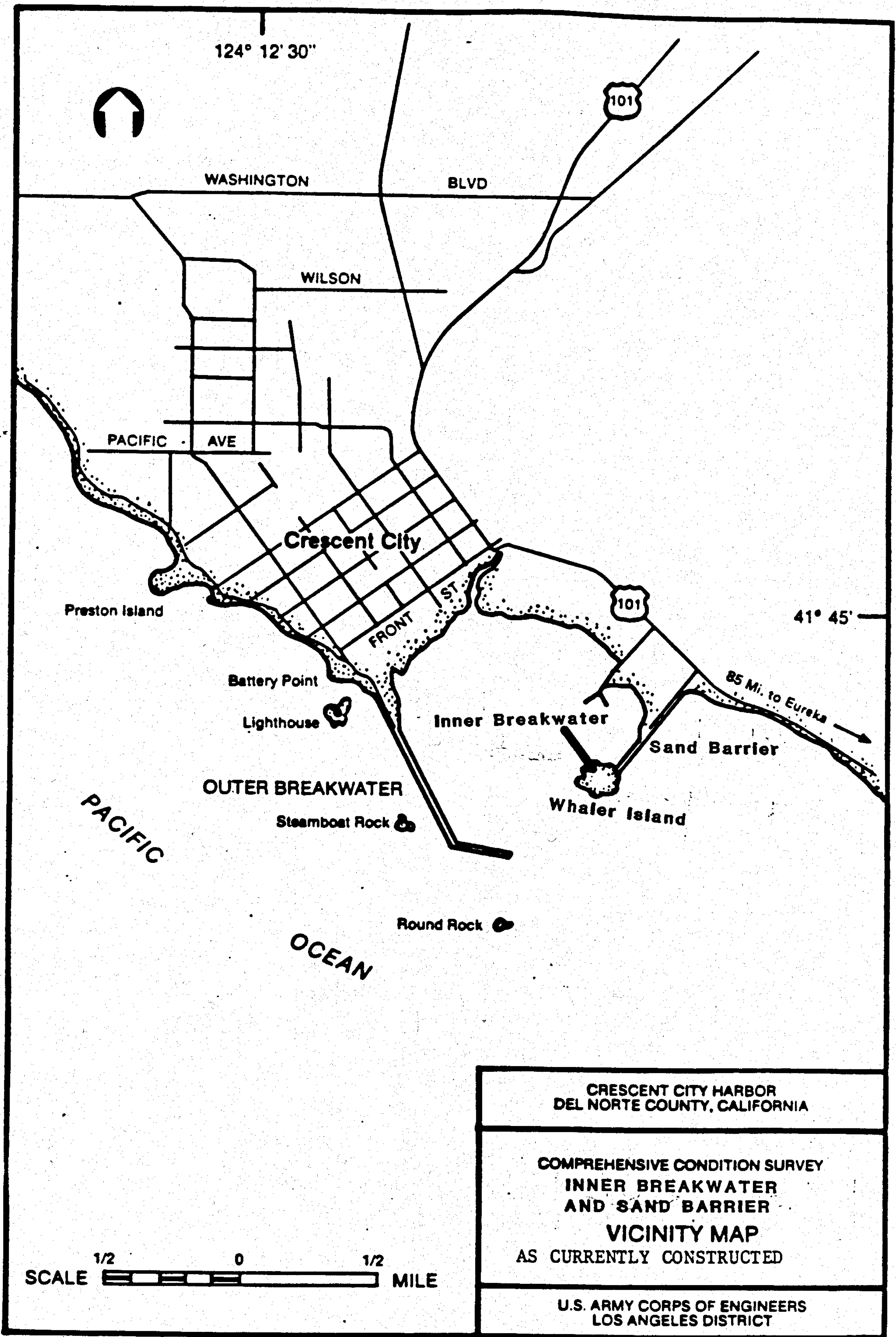

124° 12' 30"
101
WASHINGTON
BLVD
WILSON
PACIFIC
AVE
Crescent City
FRONT
ST
101
41° 45'
Preston Island
Battery Point
Lighthouse
Inner Breakwater
85 Mi. to Eureka
Sand Barrier
OUTER BREAKWATER
Whaler Island
Steamboat Rock
PACIFIC
Round Rock
OCEAN
CRESCENT CITY HARBOR
DEL NORTE COUNTY, CALIFORNIA
COMPREHENSIVE CONDITION SURVEY
INNER BREAKWATER
AND SAND BARRIER
VICINITY MAP
AS CURRENTLY CONSTRUCTED
1/2
0
1/2
SCALE
MILE
U.S. ARMY CORPS OF ENGINEERS
LOS ANGELES DISTRICT

Figure 2

(2) Extensive field studies have been undertaken, including side scan sonar and inspection dives. These studies were intended to:

(a) Determine bathymetry around the structures.

(b) Locate holes, slumps, or other major irrigularities in underwater armor position and/or slope of the breakwaters.

(c) Locate armor-loss voids.

(d) Locate blowholes and core erosion.

(e) Determine the interior composition of the structures.

(f) Identify any separations of cap from the core.

(g) Determine if materials were weathering significantly.

(3) The data has been analyzed to determine the extent and seriousness of damages and the ability of the breakwaters to withstand wave and other forces acting on it.

The Shore Protection Manual (1984), has been the basis for efforts to meet the development and rehabilitation plan for the breakwaters. In addition, many of the experts who participated in the survey were consulted and made repair recommendations.

The long-term inspection and monitoring program was then developed, taking into account the damage history of the breakwater and the sand barrier and the lessons learned during this condition survey. Routine inspection procedures were supplemented by procedures for inspection following major storms, tsunami, or seismic events. A manual is available under separate cover.

1.4 PARTICIPANTS IN THE CONDITION SURVEY AND ANALYSIS.

This has been a multi-disciplinary study with many participants, from both public and private sectors. The historical review and final analysis has been carried out by Los Angeles District staff. Field studies were conducted by:

Aerial Photography
Aero-Cartographic
Santa Rosa, California
Survey conducted in November 1983

Mapping
Aero-Cartographic; Walter Associates
Santa Rosa, California
Work completed in November 1983

Photography and Compilations
Barton Walters and Associates, Inc.
Canoga Park, California
Work done in November 1983 and March 1984

Sonar
U.S. Army Engineers Waterways Experiment Station
Coastal Engineering Research Center (CERC)
Vicksburg, Mississippi
Survey done in July 1984

Wave Analysis

U.S. Army Engineer Waterways Experiment Station

Coastal Engineering Research Center (CERC)

Vicksburg, Mississippi

Work completed in July 1984

Coastal Processes Study

Noble Consultants, Inc.

Mill Valley, California

Work completed in March 1988

2. STUDY AREA DESCRIPTION

2.1 GENERAL GEOGRAPHY AND GEOLOGY

Crescent City Harbor is located 17 miles south of the Oregon Border (fig. 1) in the lee of Battery Point. The harbor is bounded by the main or outer breakwater extending from Battery Point on the west to southside, the sand barrier on the southeast from shore out to Whaler Island and the inner breakwater extending west from Whaler Island (see fig. 2). The sand barrier was constructed in 1939 and has a length of about 2,500 feet, a crest elevation of +13 MLLW and a design crest width of 9 feet. The inner breakwater was constructed in 1946 to a length of 1100 feet and a 400-foot dog-leg extension was later constructed in 1972. The breakwater has a crest elevation of +18 feet MLLW and a crest width of 12 feet (15 feet on the extension). In this report the construction and design features of each segment of the inner breakwater is discussed separately. However, they are treated as a total unit in assessing their present condition. An assessment of the main (outer) breakwater was completed in 1984.

The entire north coast is classified as moderately seismically active, but it is not as active as areas bordering the San Andreas Fault, which ends about 100 miles to the south. There are local faults within the Crescent City areas, the largest known fault being the South Fork Mountain Fault which is the boundary between the Klamath Mountains and the Coast Ranges. No faults are known to underlie the breakwater or occur within 6 miles of it. The earthquakes predicted for this area will not have a magnitude to cause severe damage to the breakwater.

The bedrock in the bay is a heterogenous mixture of altered sedimentary rocks and intruded volcanic rocks, mostly graywackes and volcanic basalts. There is a thin layer of sediments overlying this bedrock. Sand is trapped in the harbor by the breakwater. The fact that the inner breakwater and the sand barrier lie on bedrock precludes foundation erosion and slumping.

2.2 WEATHER

The entire north coast is exposed to major Pacific Ocean storms which produce high winds and tides. Swells reaching the outer breakwater will be as high as 41 feet (7.0-foot still water level, MLLW). Air and water temperatures are relatively stable, but generally with water temperature ranging from 50°F in January to 59°F in August. Mean air temperatures range from 47°F in January to 62°F in September. Temperatures in the coastal zone reach a maximum of about 85°F and a minimum of 21°F. Prevailing winds are from the southeast during November-February, and from the north or northwest during the rest of the year. Maximum sustained wind speed is expected to be 49 knots, occurring during the winter.

2.3 BATHYMETRY

Before construction of the outer breakwater, the floor of Crescent City Harbor was bedrock with shallow patches of sand overlying the bedrock. Since construction, the bay has experienced some shoaling, although no sand buildup has been reported along the oceanside of the breakwater.

2.4 WAVE CONDITIONS

High locally-generated storm waves and high swell both reach the coast at Crescent City. The inner breakwater was designed for breaking waves of 16 feet with an occurrence interval of 100-years. Since construction there have been numerous storms with waves of 18-20 feet in height at the outer breakwater and the inner breakwater has experienced its design wave. Maximum daily wave heights of 10-15 feet have frequently been measured by a gauge located in 15 feet of water (MLLW) during the period of September 1980 to January 1983. Waves approach from the south (180°) to the northwest (315°), with waves from 190° to 270° subject to refraction by shoals, which increases wave heights. The largest waves expected to approach Crescent City are from 200° to 220°, which is coincidentally the angle most affected by shoaling, with waves amplified by as much as 31 percent.

3. CURRENT STATUS AND MAKE-UP OF CRESCENT CITY INNER BREAKWATER AND SAND BARRIER

The inner breakwater is a rubble mound structure 1100 feet long with a 400-foot dogleg extension built in 1972. The breakwater has a crest elevation of +18 feet MLLW and a crest width of 12 feet. The dogleg extension has a crest width of 15 feet. The sand barrier is also rubble mound structure was constructed in 1939 and has a length of about 2,500 feet, a crest elevation of +13 MLLW and a design crest width of 9 feet.

Both structures have been damaged several times since they were constructed. A full description of the damages and repair history for the sand barrier and the inner breakwater is found in Geotechnical Appendix, Appendix B.

3.1 PERTINENT DATA

Sand Barrier	
Length	2,500 feet
Crest Elevation	+13 feet
Slope	1 on 1.5
Inner Breakwater	
Length	2,100 feet
Crest Elevation	+18
Slope	1 on 1.5

MHHW	7.0 feet
MHW	6.3 feet
Mean tide level	3.8 feet
MLLW	0.0 feet
Extreme high water	9.1 feet
Extreme low water	-2.5 feet
Mean diurnal tidal range	7.0 feet
Extreme tidal range	12.3 feet
Design storm conditions (wave height)	
Sand barrier	16 feet
Inner breakwater	16 feet

3.2 FOUNDATION

3.2.1 Sand Barrier

The sand barrier foundation consists of sand varying in thickness from 25 feet at the shore end to 1.5 feet near the mid-point of the structure. Underlying the sand is bedrock consisting of weathered sandstone and black shale, alternately bedded.

3.2.2 Inner Breakwater

The inner breakwater is also constructed on bedrock consisting of weathered sandstone and black shale, alternately bedded.

4. PROBLEM ANALYSIS

4.1 SCOPE OF ANALYSIS

4.1.1 Two Phases of Analysis

The first phase of problem analysis involved identification and evaluation of individual damage areas. Each damaged area was explored and an appropriate repair strategy was developed. Repair and maintenance recommendations for the breakwater were based on this problem-by-problem analysis. The problems identified were predominately:

1. Missing armor units
2. Voids (both large and small)

Each of these problems is described in detail in the Geotechnical Appendix (Appendix B).

The second phase of problems analysis involved an evaluation of the breakwater's overall condition breakwater stability given the forces expected from major storms. The causes of individual problems were explored and the general condition of stone was evaluated. There were two results of this evaluation: (1) the overall present condition and stability of the breakwater was established as a baseline for future study, and (2) a detailed monitoring and inspection program was developed, one which focuses inspector's attention on the most important type of problems likely to occur at the breakwater. This monitoring program is summarized in this report. A manual for inspection and monitoring is available under separate cover.

4.1.2 Field Investigations

The field evaluations made in this condition survey included side scan sonar, bathymetry, sub-bottom profiling, above-water inspection, and diving.

4.1.3 Evaluation of Breakwater Stability

There are a number of factors to consider in evaluating the long-term stability of a rubble-mound structure, including (1) shape, weight, and condition of armor units; (2) degree of interlocking and nesting, (3) slope of the structure; (4) core condition; (5) foundation stability; (6) size and orientation of the structure to wave attack; and (7) wave dynamics.

Present analysis techniques do not provide a method for determining the forces required to displace individual units from the cover layer. Empirical methods have been developed (such as Hudson's equation) that, if used with care, will provide satisfactory estimate of the stability of the overall structure when under attack by storm waves. These methods were developed for design of new structures, not for evaluation of existing ones. Stability model testing is another technique, but it is difficult to construct a model which replicates the stability of an existing structure which has been subjected to wave attack and has fully settled.

Currently, a stability determination for an existing structure is made mainly on the basis of qualitative evaluations based on data from many sources: visual inspection, hydrographic surveys, storm damage and repair history, side-scan sonar, diver observations, aerial photography and surveys, core borings, and knowledge of wave climate. These factors are weighed by the experienced evaluator rather than being factors in a precise mathematical expression.

To evaluate the stability of the Inner Breakwater, the breakwater was divided into 5 segments. The segments were determined to have similar shapes, type and size of armor units, and repair histories.

For each segment, typical cross sections were developed from survey data, and the characteristics of core, armor stone were described. Hudson's equation was used to "back-calculate" the maximum wave the structure would likely withstand given current armor weight, condition, slope of the breakwater, and other factors.

A full description of the evaluation procedure is contained in the Coastal Processes Appendix and Geotechnical Appendix (Appendices A and B, respectively).

4.2 PROBLEMS IDENTIFIED

4.2.1 Sand Barrier

Evaluation of available data and observations of the sand barrier's present condition indicate the structure is in satisfactory condition, although it contains several deficiencies. The greatest deficiency is the reach between stations 21+95 and 24+35 where the armor was scalped in 1984 to construct a small groin nearby. Besides the scalped zones, several individual armor stones are missing on the ocean slope between stations 3 and 8. These are of limited importance at this time and do not materially affect the integrity of the structure.

4.2.2 Inner Breakwater

The results of this assessment indicate the inner breakwater is in excellent condition, capable of performing satisfactory for at least another 50 years. The one deficiency is minor erosion along the diaphragm wall from station 13+85 on the ocean side and 14+15 on the harbor side to station 15+00 near its end. Also, several dislodged stones were observed on the ocean floor near the sections repaired in 1984 and a larger zone where the armor was removed in 1972 to add the 400-foot extension.

4.3 ARMOR STONE STATIC SLOPE STABILITY

This assessment checks the stability of the slope under static conditions. The stability of the submerged portion of the armor stone slopes was analyzed in April 1986. The survey included 2-foot interval contours along the exposed sand barrier ocean side slope and along the inner breakwater harbor and ocean side slopes. In general, the breakwater and sand barrier slopes are stable.

5. RECOMMENDED REHABILITATION AND MONITORING

5.1 REHABILITATION

The recommended short-term rehabilitation program for Crescent City Harbor Inner Breakwater and Sand Barrier consists of replacement of missing armor stones for the reach between stations 21+95 and 24+35. For the inner breakwater the program is to repair the eroded section against the diaphragm using concrete.

5.2 MONITORING PROGRAM SUMMARY

The full inspection and monitoring program is described in detail in the manual for the outer breakwater and can be used for the inner breakwater and the sand barrier as well. The recommended plan is summarized here for convenience. The summary provides a general overview of the monitoring and inspection which will be required to ensure that the breakwater condition is known in time to prevent structural failures.

The recommended 10-year monitoring program will provide essential data about the breakwater and the sand barrier. The relative stability of the breakwater during the major 1983 storms will be improved upon with the knowledge gained from long-term monitoring and inspection. The proposed program is outlined in table 1.

As a part of the monitoring and inspection program, an interim report of results will be prepared at approximately 5 years into the program. A final report will summarize the findings of the 10-year program, and will make recommendations for repair, reconstruction, or improvement of the breakwaters.

5.2.1 Hydrosurvey

Because the Crescent City Inner Breakwater is built mostly on bedrock and scour is not a problem, periodic surveys are not essential and may be conducted as needed, primarily after major ocean storms.

A recommended hydrosurvey program includes survey at 200-foot intervals on both the ocean side and the harbor side of the inner breakwater and the sand barrier.

Table 1. Recommended Monitoring Program.

Task	Schedule
Prepare below-water profile of all ranges included in this report.	Every 2nd year, start 1988.
Visual inspection program	Twice a year (March & September).
Aerial Photography	After major storms or when movement is suspected.
Side Scan Sonar Survey	After major storms or when movement is suspected.
Diving Surveys	After major storms or when movement is suspected.
Map entire structure	After major storms or when movement is suspected.
Measure settlement of structure	After major storms or when movement is suspected.

5.2.2 Slope Surveys

Steep slopes are a potential problem at Crescent City Harbor, and slope surveys to identify slumping should be made every second year beginning in 1988. In addition, the slope should be checked following major storms suspected of causing damages.

5.2.3 Visual Inspection

Visual inspection by a qualified coastal engineer and geologist should be made twice a year, immediately before and immediately after the storm season (August-March, respectively). Breakwater inspection should also follow any major storm which is suspected of causing damages.

5.2.4 Aerial Photographic Program

Aerial photogrammetric techniques can be used to obtain an accurate permanent record of all visible armor units. This record can be analyzed or "surveyed" with stereoscopic photogrammetric compilation instruments to reveal the movement of individual armor units. Important items to consider are the precision of the equipment and instruments used, the skill of the photogrammetrist and pilot, ground control surveys, tidal level (should be flowen at low tide), and accuracy of the stereo photogrammetric compilation. Aerial photographs should be compiled at a scale of 1:1200. This will provide horizontal or vertical movement of armor to within ±0.3 feet and 2 foot control intervals. The aerial surveys should be conducted following major storms when damage or movement has occurred or is suspected.

5.2.5 Side Scan Sonar

This program to check the toe and side slopes should be conducted following every second major storm. Results should be checked by diving. The side scan should be done in two stages. The first stage should consist of two runs: one of the toe and another of the upper slope. These runs should cover total length of revetments. After examination of preliminary results, additional investigations will be needed for all critical locations where changes occurred.

5.2.6 Diving Program

Diving, difficult in the turbid water of the breakwater, should be used to verify possible failure means (Geotechnical Appendix B).

5.2.7 Geotechnical Monitoring

A thorough geotechnical monitoring effort is essential to identifying problem areas. Such a program should involve:

1. Measuring settlements.

2. Installation and use of a tilt monitoring system.

APPENDIX A
Coastal Processes

APPENDIX A

CONDITION SURVEY
INNER BREAKWATER AND SAND BARRIER
CRESCENT CITY HARBOR
CRESCENT CITY, CALIFORNIA

April 5, 1988

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FINAL REPORT

CONDITION SURVEY
INNER BREAKWATER AND SAND BARRIER
CRESCENT CITY HARBOR
CRESCENT CITY, CALIFORNIA

1.0 INTRODUCTION

1.1 Purpose and Scope

The purpose of this appendix is to examine the wave climate at the inner breakwater of Crescent City Harbor and to evaluate the stability of this structure. The scope of work includes the following: evaluate the deep and shallow water wave climate, including a frequency analysis, based upon available information; determine wave transformation within the harbor area as required; determine the stability of the structure, including the primary cover layers, underlayers and the toe of the structure at stations for which cross-sections are available; analyze areas of instability, if any; assess the condition of the structure; and, present recommendations for repairs and monitoring.

1.2 Location and Description

Crescent City Harbor is located within the City of Crescent City in Del Norte County, California. The harbor has three protective structures, a main or outer breakwater, an inner breakwater and the sand barrier, as shown in Figure 1. This study evaluates the inner breakwater and the revetment fronting the sand barrier. The inner breakwater is about 1500 ft in length, the main section is about 1100 ft and the dogleg measures approximately 400 ft. The main section has a crest elevation of +18 ft MLLW and a width of 15 ft, while the dogleg portion of the inner breakwater has an elevation of +15 MLLW and a crest width of 15 ft. The sand barrier is about 2500 ft in length with a revetment crest elevation of +13 ft MLLW and a crest width of 9 ft.

2.0 WAVES

2.1 Offshore Wave Data

The offshore wave data used in establishing the deepwater wave climate was obtained from Wave Information Study (WIS) Report 14 (CERC, March 1986). The data was derived from numerical hindcasting on historical wind and surface pressure records of the North Pacific between 1956 and 1975. WIS Station 6, located approximately 45 miles southwest of Crescent City at 41.08 N and 127.3 West, was used as the source of offshore wave data. The data for Station 6, which was used for this study, is included as Attachment 1. Figure 2 is a plot showing the relationship between the number of wave cases (3 hourly hindcasts over the twenty year record) and direction of approach at Station 6. As shown in Figure 2, the direction of approach for the offshore waves at Station 6 is concentrated between 180 and 360 degrees. The exposure window from which waves can approach the site is limited by the presence of Point St. George to the north, which eliminates waves approaching from northward of 315 degrees. Although waves with a direction of approach between 180 and 225 degrees have a low number of cases (low frequency of occurrence), these waves have a significant impact upon the harbor (Hales, 1985) and cannot be neglected.

The distribution of mean and largest significant wave heights by direction is shown in Figure 3. The higher mean significant wave heights and the higher of the highest significant wave heights have a direction of approach centered around the southern boundary of the exposure window, with decreasing amplitudes as the direction moves to the north. In terms of the offshore significant wave height, the direction band covering from 180 to 225 degrees is the critical direction since this segment has the highest mean and largest significant wave heights. The frequency of occurrence distribution shown in Figure 2 shows the predominant direction of approach as being from west-northwest, supporting the intuitive line of thinking, but the wave height distribution of Figure 3 seems indicative of something else. The low frequency of occurrence of waves from the south-southwest indicates that events which produced the high waves while infrequent are quite severe. Typically storms in the North Pacific have a counter-clockwise rotation. The storm generates a rotating wind field with varying directions and wind velocities. As the storm rotates it will generate waves with a direction of approach of 180 to 225 degrees. The occurrence of these events, as previously stated, is low. As shown in the plots of Figures 2 and 3, the predominate directions of approach (270-292.5), while not having the highest wave heights, still have largest significant wave heights higher than 32 ft with mean significant wave heights higher than 10 ft. Since the frequency of waves originating out of this sector is higher than from the south, this directional band must also be considered.

2.2.1 Wave Transformation

Wave transformation from deepwater up to the outer breakwater at Crescent City was performed by Hales (1985) using the numerical model RCPWAVE (Regional Coastal Processes Wave Transformation Model) developed by Ebersole, Cialone and Pratner (1985). The model predicts the transformation of monochromatic waves over a region of complex bathymetry, using a finite difference solution. The RCPWAVE model computes coefficients over the entire grid area, and not just points along the waveray. The model includes the effects of refraction, diffraction and shoaling as it computes the wave height and angle of approach throughout the wave field. The bathymetry grid used by Hales extended 93 cells (46,500 ft) offshore and 90 cells (95,000 ft) alongshore, with each cell measuring 500 ft x 500 ft. A grid with these dimensions was considered to be sufficient in providing the resolution to accurately define the bathymetry of the region.

The study done by Hales examined the wave conditions at the dogleg of the outer breakwater (see Figure 1). The RCPWAVE model was run for 224 different conditions, composed of 14 different offshore wave directions, 8 different periods, and two stillwater levels. The offshore directions for which the model was run were: 180, 200, 205, 210, 215, 220, 230, 240, 250, 260, 270, 280, 290 and 315 degrees. The periods were: 5.2, 7.1, 8.8, 10.1, 11.2 12.6, 14.4 and 16.8 seconds. The concentration of runs between 180-220 degrees resulted from an earlier analysis, which indicated an amplification of wave heights for certain periods of waves originating from this window. The periods used correspond to the mid band of the WIS offshore period groupings. the two different stillwater elevations were +10 ft and -1 ft MLLW, which represent a maximum storm surge level at high tide and a low tide for Crescent City Harbor.

The bathymetry grid was established with the offshore axis perpendicular to the outer breakwater as shown in Figure 4. To determine the wave climate inside the harbor, wave conditions must first be established outside the breakwater and then diffracted around the outer breakwater. Initially, to define the wave climate outside of the breakwater, an average of the wave height coefficients from cells I=1,2,3, and J=42,43 were computed for the cases run. The coefficients are presented in Table 1. However, due to insufficient data from these cells another grid area was chosen for determination of wave conditions outside the breakwater. The wave climate outside of the breakwater was obtained by averaging the coefficients in cells I=4, and J=39,40,41 (see Figure 4). The cells were chosen due to their location relative to the dogleg section of the outer breakwater.

Tables 2 and 3 present the wave height coefficients from Hales (1985) for stillwater levels of +10 ft MLLW and -1 ft MLLW, respectively. The coefficients in Tables 2 and 3 from 220 degrees northward are generally higher than those listed in Table 1 at an area closer to the harbor mouth. In the direction band from 180-220 degrees, the coefficients are approximately equal with some coefficients at the harbor entrance being higher and some coefficients at the dogleg being higher. Overall, the wave height coefficients at the dogleg of the outer breakwater are similar to those at the harbor entrance, therefore the dogleg coefficients can be used to describe the wave conditions in the vicinity of the harbor entrance.

In most cases there is a reduction in wave height as the wave moves from deep to shallow water. For periods higher than 10 sec and directions southward of 220 degrees there may be an amplification of the wave height, as shown in Tables 2 and 3. The amplification of these waves is caused by an offshore shoal to the southwest of Crescent City. Severe wave conditions can occur at Crescent City Harbor when this amplification occurs in conjunction with the large waves at WIS Station 6 for approach directions within this band.

Table 1

Waves by Period and Direction
Crescent City, California

Average of Coefficients of Cells at J=42,43, at I=1,2,3

Period Band (Seconds):	4.4-6.0	6.1-8.0	8.1-9.5	9.6-10.5	10.6-11.7	11.8-13.3	13.4-15.3	15.4-18.1
Mid-band Period (Seconds):	5.2	7.1	8.8	10.1	11.2	12.6	14.4	16.8
Azimuth (Degrees)								
315								
290			.65				.60	
280			.70				.67	
270			.75				.72	
260			.79				.78	
250			.80				.78	
240			.78				.72	
230			.89				.70	
220			.92				1.09	
215				.88	.93	1.08	1.24	1.19
210	.95	.90	.86	.82	.81	.88	1.10	1.31
205						.77	.88	1.18
200			.89				.76	
180							.69	

Table 2

Waves by Period and Direction
Elevation +10 mean lower low water (mllw)
Crescent City, California

Average of Coefficients of Cells at J=39,40,41, at I=4

Period Band (Seconds):	4.4-6.0	6.1-8.0	8.1-9.5	9.6-10.5	10.6-11.7	11.8-13.3	13.4-15.3	15.4-18.1
Mid-band Period (Seconds):	5.2	7.1	8.8	10.1	11.2	12.6	14.4	16.8
Azimuth (Degrees)								
315	.57	.56	.55	.54	.53	.53	.52	.52
290	.75	.66	.64	.64	.65	.65	.64	.66
280	.84	.73	.70	.70	.69	.69	.71	.74
270	.89	.78	.76	.75	.75	.75	.77	.79
260	.91	.82	.80	.79	.79	.79	.81	.85
250	.92	.84	.81	.80	.80	.81	.83	.86
240	.93	.85	.79	.76	.75	.75	.77	.80
230	.92	.86	.83	.79	.75	.71	.70	.73
220	.93	.88	.92	1.01	1.06	1.03	.89	.77
215	.93	.87	.90	.91	1.00	1.13	1.13	.96
210	.93	.85	.80	.79	.82	.98	1.18	1.18
205	.93	.85	.80	.77	.79	.79	1.00	1.24
200	.93	.85	.79	.75	.70	.69	.81	1.10
[illegible]	.92	.86	.80	.7[illegible]	.71	.64	.58	.5[illegible]

Table 3

Waves by Period and Direction
Elevation -1 mean lower low water (mllw)
Crescent City, California

Average of Coefficients of Cells at J=39,40,41, at I=4

Period Band (Seconds):	4.4-6.0	6.1-8.0	8.1-9.5	9.6-10.5	10.6-11.7	11.8-13.3	13.4-15.3	15.4-18.1
Mid-band Period (Seconds):	5.2	7.1	8.8	10.1	11.2	12.6	14.4	16.8
Azimuth (Degrees)								
315	.31	.37	.40	.42	.44	.47	.51	.53
290	.66	.62	.61	.64	.64	.64	.64	.66
280	.76	.68	.69	.69	.69	.69	.72	.75
270	.81	.74	.75	.75	.75	.76	.78	.80
260	.84	.79	.79	.78	.79	.80	.82	.87
250	.87	.82	.80	.80	.80	.81	.84	.88
240	.87	.82	.78	.76	.76	.76	.79	.82
230	.89	.84	.82	.78	.73	.70	.71	.75
220	.89	.87	.96	1.07	1.10	1.02	.86	.75
215	.89	.85	.89	.96	1.08	1.20	1.12	.91
210	.90	.83	.80	.80	.88	1.08	1.26	1.16
205	.89	.82	.79	.77	.81	.85	1.13	1.31
200	.89	.83	.78	.74	.69	.71	.90	1.24
180	.88	.81	.76	.70	.66	.61	.55	.50

2.2.2 Wave Transformation to Inner Breakwater

A wave diffraction analysis was performed to transform the waves around the outer breakwater and across the entrance channel to the inner breakwater. The diffraction analysis was based on the diffraction diagrams presented in the Shore Protection Manual (1984). Four points were picked along the length of the inner breakwater (as shown in Figure 1), for which diffraction coefficients with wave periods of 5, 10, 15 and 20 seconds and stillwater elevations of +10 ft and -1 ft MLLW were computed. At Points 1 and 2, for wave directions between 180 and 220 degrees, diffraction was first considered around the outer breakwater and then around Whaler Island; while from 230 degrees northward, diffraction was taken only around the outer breakwater. Points 3 and 4 are sufficient distance from Whaler Island such that the effect of the island on waves is minimal. Therefore, at Points 3 and 4 diffraction for all wave directions was taken only around the outer breakwater.

In transforming the waves from the outer breakwater to the inner breakwater, shoaling as well as diffraction must be considered. The relative shoaling coefficients from the outer breakwater to the inner breakwater were computed for the four different points and the two water levels. Refraction inside the entrance channel will also affect the waves as they approach the inner breakwater. The bathymetry in the entrance channel, while not varying much in depth, (from 17 to 21 ft) is contoured such that bending of the diffracted waves will occur. Qualitatively, the contours will cause the wave rays to spread out, thereby reducing the wave energy and wave height. Wave height reduction due to refraction in the entrance channel will be most evident at Points 3 and 4.

To obtain the wave height coefficients at the inner breakwater, the RCPWAVE coefficient (K_{rcp}) was multiplied by the diffraction coefficient (K_d) and the relative shoaling coefficient (K_{sr}).

$$K_{ib} = K_{rcp} * K_d * K_{sr}$$

This result was then multiplied by the offshore wave height from WIS Station 6 to obtain a resultant wave height at the structure (H_{ib}):

$$H_{ib} = H_o * K_{ib}$$

The process outlined above was used to perform the frequency analysis done for each of the points on the inner breakwater. The frequency tables shown in Tables 4-7 list the resultant wave height frequency distribution at four points along the inner breakwater using offshore wave conditions from WIS Station 6 for a stillwater level of -1 ft MLLW. Tables 8-11 list the distribution for a stillwater level of +10 ft MLLW. The wave

Table 4.

Crescent City Condition Survey

Stillwater Level = -1 ft MLLW

Point 1

Wave Height Frequencies
Percent Occurrence (X1000)

Ht, ft	Per, sec 5	10	15	20	TOTAL
.5	7481	8807	35	0	16323
1.0	1180	15612	3263	0	20055
1.5	371	7188	5927	0	13486
2.0	283	3174	6795	0	10252
2.5	97	1477	3303	3	4880
3.0	1	592	4750	5	5348
4.0	99	1263	5867	10	7239
5.0	193	717	2726	0	3636
6.0	42	382	996	0	1420
7.0	94	342	489	0	925
8.0	77	573	331	0	981
9.0	0	703	115	0	818
10.0	40	462	172	0	674
12.0	0	557	210	0	767
14.0	0	290	22	0	312
+16.0	0	107	79	0	186
+18.0	0	19	108	0	127
+20.0	0	5	30	0	35
TOTAL	9958	42270	35218	18	87464

+ Depth Limited Wave Height = 15 ft @ water depth of 18 ft
* Height is upper limit, ie .5 = 0 to .5 ft.
* Period is mean of group, ie 10 is for 7.6 - 12.5 sec.

Table 5.

Crescent City Condition Survey

Stillwater Level = -1 ft MLLW

Point 2
Wave Height Frequencies
Percent Occurrence (X1000)

Ht, ft	Per, sec 5	10	15	20	TOTAL
.5	8152	13710	272	0	22134
1.0	826	17018	8945	0	26789
1.5	339	5394	9322	3	15058
2.0	194	1220	5898	5	7317
2.5	194	771	5943	10	6918
3.0	0	689	1967	0	2656
4.0	97	929	1508	0	2534
5.0	42	825	721	0	1588
6.0	63	374	256	0	693
7.0	11	295	129	0	435
8.0	30	218	17	0	265
9.0	10	310	20	0	340
10.0	0	307	71	0	378
12.0	0	137	107	0	244
14.0	0	68	42	0	110
+16.0	0	5	0	0	5
+18.0	0	0	0	0	0
+20.0	0	0	0	0	0
TOTAL	9958	42270	35218	18	87464

\+ Depth Limited Wave Height = 15 ft @ water depth of 18 ft
* Height is upper limit, ie .5 = 0 to .5 ft.
* Period is mean of group, ie 10 is for 7.6 - 12.5 sec.

Table 6.

Crescent City Condition Survey

Stillwater Level = -1 ft MLLW

Point 3

Wave Height Frequencies
Percent Occurrence (X1000)

Ht, ft	Per, sec 5	10	15	20	TOTAL
.5	8448	22660	311	0	31419
1.0	867	11366	10722	0	22955
1.5	97	3197	13106	8	16408
2.0	100	1078	4621	10	5809
2.5	193	623	3873	0	4689
3.0	42	68	979	0	1089
4.0	171	1100	758	0	2029
5.0	40	744	376	0	1160
6.0	0	736	207	0	943
7.0	0	467	48	0	515
8.0	0	199	79	0	278
9.0	0	27	108	0	135
10.0	0	5	0	0	5
12.0	0	0	30	0	30
14.0	0	0	0	0	0
16.0	0	0	0	0	0
18.0	0	0	0	0	0
20.0	0	0	0	0	0
TOTAL	9958	42270	35218	18	87464

* Height is upper limit, ie .5 = 0 to .5 ft.
* Period is mean of group, ie 10 is for 7.6 - 12.5 sec.

Table 7.

Crescent City Condition Survey

Stillwater Level = -1 ft MLLW

Point 4

Wave Height Frequencies
Percent Occurrence (X1000)

Ht, ft	Per, sec 5	10	15	20	TOTAL
.5	8636	23498	2546	0	34680
1.0	779	11990	12970	3	25742
1.5	290	2710	12277	15	15292
2.0	139	1146	4830	0	6115
2.5	63	818	1247	0	2128
3.0	11	938	729	0	1678
4.0	40	695	391	0	1126
5.0	0	401	141	0	542
6.0	0	51	87	0	138
7.0	0	23	0	0	23
8.0	0	0	0	0	0
9.0	0	0	0	0	0
10.0	0	0	0	0	0
12.0	0	0	0	0	0
14.0	0	0	0	0	0
16.0	0	0	0	0	0
18.0	0	0	0	0	0
20.0	0	0	0	0	0
TOTAL	9958	42270	35218	18	87464

* Height is upper limit, ie .5 = 0 to .5 ft.
* Period is mean of group, ie 10 is for 7.6 - 12.5 sec.

Table 8.

Crescent City Condition Survey

Stillwater Level = +10 ft MLLW

Point 1

Wave Height Frequencies
Percent Occurrence (X1000)

Ht, ft	Per, sec 5	10	15	20	TOTAL
.5	6451	5601	35	0	12087
1.0	2185	18703	3264	0	24152
1.5	396	7303	5918	0	13617
2.0	283	3058	6803	0	10147
2.5	97	1593	6421	3	8116
3.0	1	592	4787	5	5380
4.0	99	1270	3058	0	4437
5.0	193	710	2596	10	3499
6.0	42	690	817	0	1549
7.0	94	555	674	0	1323
8.0	77	63	225	0	365
9.0	0	740	171	0	911
10.0	40	569	141	0	750
12.0	0	574	94	0	668
14.0	0	217	86	0	303
+16.0	0	27	98	0	125
+18.0	0	5	30	0	35
+20.0	0	0	0	0	0
TOTAL	9958	42270	35218	18	87464

+ Depth Limited Wave Height = 15 ft @ water depth of 18 ft
* Height is upper limit, ie .5 = 0 to .5 ft.
* Period is mean of group, ie 10 is for 7.6 - 12.5 sec.

Table 9.

Crescent City Condition Survey

Stillwater Level = +10 ft MLLW

Point 2

Wave Height Frequencies
Percent Occurrence (X1000)

Ht, ft	Per, sec 5	10	15	20	TOTAL
.5	7185	11138	69	0	18392
1.0	1793	19462	9148	0	30403
1.5	339	4946	8874	3	14162
2.0	194	1796	6335	5	8330
2.5	1	985	5954	10	6950
3.0	193	476	2131	0	2800
4.0	97	1131	1518	0	2746
5.0	42	622	548	0	1212
6.0	63	379	277	0	719
7.0	11	385	124	0	520
8.0	30	277	14	0	321
9.0	10	294	77	0	381
10.0	0	166	62	0	228
12.0	0	140	45	0	185
14.0	0	50	42	0	92
+16.0	0	23	0	0	23
+18.0	0	0	0	0	0
+20.0	0	0	0	0	0
TOTAL	9958	42270	35218	18	87464

+ Depth Limited Wave Height = 15 ft @ water depth of 18 ft
* Height is upper limit, ie .5 = 0 to .5 ft.
* Period is mean of group, ie 10 is for 7.6 - 12.5 sec.

Table 10.

Crescent City Condition Survey

Stillwater Level = +10 ft MLLW

Point 3

Wave Height Frequencies
Percent Occurrence (X1000)

Ht, ft	Per, sec 5	10	15	20	TOTAL
.5	8394	18185	311	0	26890
1.0	638	15842	10711	0	27191
1.5	380	2563	11923	8	14874
2.0	100	1491	5607	10	7208
2.5	193	748	4081	0	5022
3.0	42	241	978	0	1261
4.0	160	1005	787	0	1952
5.0	41	820	371	0	1232
6.0	10	730	141	0	881
7.0	0	396	89	0	485
8.0	0	175	91	0	266
9.0	0	51	98	0	149
10.0	0	18	0	0	18
12.0	0	5	30	0	35
14.0	0	0	0	0	0
16.0	0	0	0	0	0
18.0	0	0	0	0	0
20.0	0	0	0	0	0
TOTAL	9958	42270	35218	18	87464

* Height is upper limit, ie .5 = 0 to .5 ft.
* Period is mean of group, ie 10 is for 7.6 - 12.5 sec.

Table 11.

Crescent City Condition Survey

Stillwater Level = +10 ft MLLW

Point 4

Wave Height Frequencies
Percent Occurrence (X1000)

Ht, ft	Per, sec 5	10	15	20	TOTAL
.5	8133	21516	378	0	30027
1.0	1282	13704	15130	0	30116
1.5	289	2787	11421	8	14505
2.0	113	1326	4687	10	6136
2.5	27	791	1932	0	2750
3.0	74	747	878	0	1699
4.0	40	882	555	0	1477
5.0	0	364	150	0	514
6.0	0	130	45	0	175
7.0	0	18	42	0	60
8.0	0	5	0	0	5
9.0	0	0	0	0	0
10.0	0	0	0	0	0
12.0	0	0	0	0	0
14.0	0	0	0	0	0
16.0	0	0	0	0	0
18.0	0	0	0	0	0
20.0	0	0	0	0	0
TOTAL	9958	42270	35218	18	87464

* Height is upper limit, ie .5 = 0 to .5 ft.
* Period is mean of group, ie 10 is for 7.6 - 12.5 sec.

height distribution for a stillwater of -1 ft MLLW at Point 1 on the inner breakwater shows wave heights in the 20 foot class, which represents all wave heights higher than 18 feet. Wave heights of this magnitude should not occur at Point 1 due to depth limiting conditions (see Section 2.3). In performing the frequency analysis, wave refraction in the entrance channel was not considered which should reduce the wave heights.

At Point 1 the wave height distribution shows a shift to higher wave heights at the -1 ft MLLW stillwater elevation when compared to the distribution of the +10 ft MLLW. At a stillwater elevation of -1 ft MLLW, 1.43% of the waves are greater than 10 ft; while at the +10 ft MLLW level, 1.13% of the waves are greater than 10 ft. The difference in the percent occurrence of wave heights greater than 10 ft between the two water elevations is insignificant and the frequency of occurrence of these can be equated for the two water elevations. The principle difference between the two distributions is that the -1 ft MLLW has wave heights in the +20 ft wave class and the +10 ft MLLW does not. The frequency of occurrence of the +20 ft events is 0.04%.

The distribution of wave heights at Point 2 for both stillwater elevations are basically equal. The maximum wave height is in the 16 ft class for both water elevations, and the percent occurrence of events with wave heights greater than 10 ft is 0.36% and 0.30% for the -1 ft MLLW and the +10 ft MLLW stillwater elevations, respectively. The distribution at Point 3 shows the same characteristics as the distribution at Point 2. The maximum wave height for both elevations is in the 12 ft group and the percent occurrence of events with wave heights greater than 10 ft is 0.03% and 0.04% at stillwater elevations of -1 ft MLLW and +10 ft MLLW, respectively. At point 4 the maximum wave height lies in the 7.0 ft group and has a frequency of occurrence of 0.02% at a stillwater elevation of -1 ft MLLW. At a stillwater elevation of +10 ft MLLW, the maximum wave height is in the 8 ft class and has a frequency of occurrence less than 0.01%.

Essentially, the stillwater elevation has no effect on significant wave height distribution seen at the inner breakwater. The stillwater elevation will become important in determining the depth limited waves at the structure.

2.2.3 Wave Transformation to the Sand Barrier

The exposure window of the sand barrier is significantly less than that for the harbor entrance. The northward limit of the window is limited by the alignment of the sand barrier (approximately 215.5 deg) and the presence of the outer breakwater (see Figure 1). The wave window for the sand barrier is from 180 to 215 degrees. The combined shoaling/refraction coefficients for the sand barrier were obtained using Figure 5

(Shore Protection Manual, 1984). A shallow water wave angle was obtained from the RCPWAVE output and then a deepwater angle was back-calculated using the curves on the graph. Using the deepwater wave angle, the wave period, and a water depth of 15 ft, the combined refraction/shoaling coefficient (K_rKs) was then determined. The RCPWAVE shallow water wave angles were obtained for the same conditions as listed in Table 1. Angles for other conditions were linearly interpolated. Table 12 lists the estimated wave height coefficients at the mid-point of the sand barrier for wave directions of 180 and 215 degrees.

Table 12.

Wave Height Coefficients at Sand Barrier

Direction (deg)	Period (sec)	K_rK_s, @ d=10' MLLW		K_rK_s, @ d=5' MLLW	
		SWL=+10'	SWL=-1'	SWL=+10	SWL=-1'
180	5	1.25	1.60	1.40	1.90
	10	1.1	1.30	1.13	1.55
	15	1.3	1.50	1.40	1.80
	20	1.45	1.75	1.54	2.30
215	5	0.92	1.60	0.92	1.90
	10	1.10	1.30	1.13	1.55
	15	1.30	1.50	1.40	1.80
	20	1.45	1.75	1.53	2.30

The wave height coefficients obtained for the sand barrier (Table 12) are higher than those at the harbor entrance (Tables 2 & 3) for the same directions. Similar to the analysis for the inner breakwater, Whaler Island will impact the wave conditions along the sand barrier. Refraction and diffraction near the island will decrease heights compared to that indicated by the coefficients in Table 12.

2.3 Depth Limited Waves

In view of the large wave height coefficients calculated for both the inner breakwater and the sand barrier, consideration was given to the maximum wave height that could be supported by the local water depth.

As waves move shoreward their maximum height is limited prior to breaking and is a function of wave period, water depth and sea floor slope. The breaking wave height is described by the equation from the Shore Protection Manual (1984).

$$H_b = \frac{d_b}{1 + \frac{da}{gt^2}}$$

$$a = 43.75(1 - e^{-19m})$$

$$b = 1.56/(1 - e^{-19.5m})$$

A breaking wave height estimate for the inner breakwater and sand barrier was prepared by CERC at the request of the Los Angeles District in January of 1987. The CERC estimate, included as Attachment 2, predicts a breaking height of 11 ft in 13 ft of water at the sand barrier. At a +10 ft MLLW stillwater level, a water depth of 13 ft is about 80 ft from the sand barrier, at its closest. This results in a wave height estimate that is overly conservative since the wave breaks at a distance seaward far from the structure. At the same stillwater elevation the 11 ft contour is about 40 ft from the breakwater, which is still a fair distance from the structure. Therefore the breaking wave height calculated based on this depth will be conservative.

The sand barrier is provided with two sources of natural protection against wave attack. The trunk of the structure is protected by the accumulation of material on the seaward side. The depths in this area will prevent large waves from reaching the structure. The protection provided by the shoal extends from the shore out to about Station 17+00. From Station 17+00 to Whaler Island, the sand barrier is sheltered by the finger of Whaler Island that extends westward (see Figure 6). The bathymetry between Station 17+00 and Whaler Island will support a breaking wave height of 9.6 ft. It is unlikely, however that this wave would occur due to the natural protection provided by the projection of Whaler Island.

At the inner breakwater a depth of 18 ft was chosen as the controlling depth for the depth limited wave height. The 18 ft depth contour at a +10 ft MLLW tide runs along the breakwater toe for the entire length of structure (see Figure 6). A breaking wave at this contour would directly impact upon the structure, resulting in the most severe condition to be experienced by the inner breakwater. The analysis shows that the maximum breaking wave height is approximately 15 ft and therefore should be considered to be the controlling wave height.

The wave height estimate performed by CERC for the Los Angeles District estimates a non-breaking wave of 16 ft for the inner breakwater. It was assumed that diffraction around the breakwater would reduce the wave amplitude by about 50%. The wave height was based on WIS Station 6 data and the RCPWAVE wave height coefficients for directions south of 220 degrees, which resulted in a wave height of 32 ft at the harbor entrance.

3.0 STABILITY AND CONDITION OF STRUCTURE

3.1 Stability

The stability of a structure, while dependant on many factors, can be grouped into three main areas. The first is the breakwater itself. The breakwater should be examined in terms of its armor stone size and shape, the armor layer thickness, placement of the armor units, degree of interlocking among the armor units, slope of the structure, overall dimensions of the structure (height, width), crown type, condition of the core and the quality of the construction. The second area is the local sea floor. The local bathymetry can have the effect of concentrating or dispersing the wave energy approaching the site. In addition, the sea floor material must be able to carry the load placed upon it by the breakwater without suffering excessive settlement or risk of a slip failure. The third area to consider is the wave climate in the vicinity of the structure. The breakwater must ultimately be designed to withstand the waves that will impact upon the structure whether these are breaking or non-breaking waves.

It is not possible to assign each factor a given weight in the design or analysis phase to predict the stability. In view of this, empirical methods of determining the required armor stone weight to provide a satisfactory degree of stability have been developed (Shore Protection Manual, 1984). The empirical formula is based on extensive small scale model testing and a small amount of large scale testing performed by the U.S. Army Corps of Engineers, Waterways Experiment Station (WES). The armor stone weight obtained through this empirical formula is a function of the wave height, the unit weight of the stone, the slope of the structure and a stability coefficient obtained from model testing (shown in Table 13). The formula presented by WES has the following form:

$$W = \frac{W_r H^3}{K_d \; (S_r - 1)^3 \; \mathrm{COT} \; 0}$$

Where W = weight of individual armor stone (pounds)
w_r = unit weight of rock (pounds per cubic foot)
H = wave height (ft)
S_r = specific gravity of rock (w_r/w_w)
0 = slope of structure measured from horizontal
K_d = stability coefficient from model testing

Stability of the armor units was considered at the four stations selected along the inner breakwater for the wave height analysis (see Figure 1). Points 1, 2, and 3 are all located near where repair work has been performed in the past, and Point 4 was

Table 13

Armor Stone Stability Criteria

Suggested K_D Values for use in determining armor unit weight[1].

No-Damage Criteria and Minor Overtopping							
Armor Units	n[3]	Placement	Structure Trunk K_D[2] Breaking Wave	Structure Trunk K_D[2] Nonbreaking Wave	Structure Head K_D Breaking Wave	Structure Head K_D Nonbreaking Wave	Structure Head Slope Cot θ
Quarrystone							
Smooth rounded	2	Random	1.2	2.4	1.1	1.9	1.5 to 3.0
Smooth rounded	>3	Random	1.6	3.2	1.4	2.3	5
Rough angular	1	Random[4]	[4]	2.9	[4]	2.3	5
Rough angular	2	Random	2.0	4.0	1.9	3.2	1.5
					1.6	2.8	2.0
					1.3	2.3	3.0
Rough angular	>3	Random	2.2	4.5	2.1	4.2	5
Rough angular	2	Special[6]	5.8	7.0	5.3	6.4	5
Parallelepiped[7]	2	Special[1]	7.0 – 20.0	8.5 – 24.0	—	—	
Tetrapod and Quadripod	2	Random	7.0	8.0	5.0	6.0	1.5
					4.5	5.5	2.0
					3.5	4.0	3.0
Tribar	2	Random	9.0	10.0	8.3	9.0	1.5
					7.8	8.5	2.0
					6.0	6.5	3.0
Dolos	2	Random	15.8[8]	31.8[8]	8.0	16.0	2.0[9]
					7.0	14.0	3.0
Modified cube	2	Random	6.5	7.5	—	5.0	5
Hexapod	2	Random	8.0	9.5	5.0	7.0	5
Toskane	2	Random	11.0	22.0	—	—	5
Tribar	1	Uniform	12.0	15.0	7.5	9.5	5
Quarrystone (K_{RR}) Graded angular	–	Random	2.2	2.5	—	—	

1 CAUTION: Those K_D values shown in *italics* are unsupported by test results and are only provided for preliminary design purposes.

2 Applicable to slopes ranging from 1 on 1.5 to 1 on 5.

3 n is the number of units comprising the thickness of the armor layer.

4 The use of single layer of quarrystone armor units is not recommended for structures subject to breaking waves, and only under special conditions for structures subject to nonbreaking waves. When it is used, the stone should be carefully placed.

5 Until more information is available on the variation of K_D value with slope, the use of K_D should be limited to slopes ranging from 1 on 1.5 to 1 on 3. Some armor units tested on a structure head indicate a K_D-slope dependence.

6 Special placement with long axis of stone placed perpendicular to structure face.

7 Parallelepiped-shaped stone: long slab-like stone with the long dimension about 3 times the shortest dimension (Markle and Davidson, 1979).

8 Refers to no-damage criteria (<5 percent displacement, rocking, etc.); if no rocking (<2 percent) is desired, reduce K_D 50 percent (Zwamborn and Van Niekerk, 1982).

9 Stability of dolosse on slopes steeper than 1 on 2 should be substantiated by site-specific model tests.

Reference: Shore Protection Manual, 1984 selected because typically the head section of breakwaters are critical sections. Table 14 shows the wave height, stability coefficient, slope of the structure and the specific gravity of the armor stone used in computing the armor stone size using the empirical formula. The wave height used in computing the armor stone size at each point was the maximum wave height at that point based on the frequency analysis, except when the wave height from the frequency analysis was larger than the depth limited wave, in which case the depth limited wave height was used. At Stations 1 and 2 the frequency analysis showed wave heights in the +20 ft height range, however the depth limited wave is 15 ft, therefore the depth limited wave height was used in determining the required stone size. The required armor stone size for these two stations is about 20 tons, which is larger than the existing stone size as stated in the Geotechnical Appendix. As discussed in the section pertaining to wave transformation to the inner breakwater, Whaler Island has a significant impact on reducing wave heights at Points 1 and 2. The existing armor stone in this area has an average stone size of 6 tons and a maximum stone size of 17 tons. A detailed history of construction and maintenance given in the Geotechnical Appendix reveals that no repair work was performed on the inner breakwater between its construction, in 1949, and 1983 when it suffered damage. During the interval between 1949 and 1983, severe waves from the critical directions and periods did occur and no damage was sustained. The most frequent waves at the project site are from the west-northwest and are significantly reduced by the time they reach the inner breakwater.

At Points 3 and 4 the wave heights are not as high as they are at Points 1 and 2 due to the effects of diffraction and shoaling. The waves reaching these points are also probably reduced by refraction in the entrance channel. The required armor stone weights calculated for these points was computed using the highest wave heights obtained for the wave height distributions at each point (see Tables 4-11). The required stone sizes fall within the design range and the field measured range. The original design called for armor stone with a minimum size of 5.8 tons and an average of 8.3 tons. Subsequent repairs called for a minimum size of 9 tons and an average of 12.8 tons. The breakwater extension was constructed using a minimum armor stone size of 9 tons and an average size of 11 tons.

The performance history of the inner breakwater seems to indicate the structure has a high degree of stability. The infrequent high waves from the south to southwest can cause damage to the structure, however the frequency of their occurrence is low. The 15 ft breaking wave is the maximum depth supported wave height at these stations and is therefore conservative.

Table 14.

Armor Stone Weights On Inner Breakwater

Point	Slope (1V on _ H)	K_d	S_r	wr (pcf)	H (ft)	W (tons)
1	1.5	2.0	2.7	178.8	15*	20
2	1.5	2.0	2.7	178.8	15*	20
3	2.0	2.0	2.7	178.8	12+	7.6
4	2.5	1.6	2.7	178.8	8+	2.2

*Depth limited wave @ d=18'
+Maximum wave from frequency analysis

The sand barrier was constructed using an armor stone size of 4 to 6 tons. Subsequent repair work used 3-7 ton stones. The required stone size for the maximum breaking wave height of 9.6 ft is about 6.2 tons. This value should be conservative since this stone size was calculated using the maximum depth supported wave height in 11 ft of water. The frequency with which a wave of this height occurs is very low.

3.2 Condition

Information on the condition of the inner breakwater and sand barrier were obtained from the Geotechnical Appendix.

3.2.1 Inner Breakwater

The inner breakwater is in excellent condition above water with only very minor deficiencies. Since the original construction in 1946, and the 400 ft extension in 1972, the structure has changed little. The original section (1120 feet long) is constructed of Whaler Island greenstone, and is in excellent condition. The edges of the armor rock at the water line have rounded somewhat. A bedrock knoll, which is an extension of Whaler Island, is built into the ocean side of the structure between Stations 1+00 and 2+50. It rises to near the crest elevation at Station 2+80 and extends laterally to 30 ft from the centerline. The only repairs that have been made to the structure occurred in 1984 at the three locations shown in Table 15; the present stationing is offset 20 feet from the original.

Table 15.

1984 Inner Breakwater Repairs

Present Stationing	Original Stationing	Area Repaired
3+30 to 3+65	3+50 to 3+85	across crest
3+95 to 4+55	4+15 to 4+75	across crest
10+70 to 11+40	10+90 to 11+60	ocean slope

In addition to the above repairs, extra stone has been placed on the sea side corner between the knoll and the foot of the breakwater.

Throughout the breakwater, the above-water slopes are regular and contain no pockets, holes or missing armor. At about Station 6+25, the ocean slope has a slight jog, and steepens slightly up-station. A concrete diaphragm wall, approximately 2 feet wide, extends from Station 10+95 to 15+15 along the centerline. The wall is in excellent condition and contains only one crack. The crest stone adjacent to the concrete wall tends to be smaller than the design in other areas because of the wall, thus numerous cobble sizes occur in this area. This small stone has washed out along the wall from Station 13+85 to 15+00 on the ocean side, and at Station 14+15 on the harbor side, leaving up to 3 ft of the wall exposed. This condition is not considered serious and in need of immediate repair.

Left over road base from the 1984 repair, caps the crest for most of its length, except between Stations 6+25 to 9+75, and beyond Station 11+10 where no road was constructed. Numerous small scraps of weathered and unusual rock types exist in the reach between Stations 6+25 and 9+75, inferring road base once existed there. This has been mostly washed off.

A large part of the harbor slope above approximately Station 12+00 contains small angular rock from the crest to the water line where a small bench has formed. The bench is especially distinct around Station 14+00. The stone that forms the bench is not core material, but is small crest material that was placed adjacent to the concrete diaphragm that has been washed out.

A side scan survey was performed on July 25, 1986 (see Geotechnical Appendix). The results of the survey indicate that no deficiencies exist in the below-water slopes. A small detached bedrock pinnacle about 40 feet in diameter occurs around Station 4+75 roughly 20 feet from the breakwater toe on the seaward side. The toe and slope, for the most part, are slightly undulating with patches of detached stones along the toe around Stations 3+50 and 10+50. The former location is the site of one of the 1984 repairs. The latter patch is likely from the 1972 construction when the armor was removed from the original head at Station 11+20 to join the extension to the trunk. Around the present head and along the harbor side, no unusual features were noted. Kelp on the harbor side masks the slope of the breakwater between Station 14+00 and 15+00.

The ocean floor is mostly exposed bedrock from Station 4+00 to about Station 7+00. Beyond Station 7+00, the floor becomes more sandy with some silt. Almost all stone on the ocean floor was armor size except for a patch of one-foot diameter stone in a swale 4 feet across at about Station 11+50. This stone had been there for sometime as it was covered with marine growth and kelp. A patch of armor stone at Station 10+50 extended out some 50 feet from the toe of the breakwater, was lying on the sand and as with the other stone, indicated little change has occurred in the floor on the ocean side. In the slope itself, no holes, slumps or other irregularities were noted. Around the head into the harbor, shoaling has raised the floor from elevations of -20 and -30 ft to about -6 ft MLLW. No diving was performed between the end of the breakwater and Station 14+00 because of the heavy kelp, shallow floor and very bad visibility. It was noted, however, that armor on the harbor side protruded through the sand and that the sand contained more silt than on the ocean side.

3.2.2 Sand Barrier

Since constructed in 1939, harbor improvements and shoaling have covered much of the sand barrier. The only portion visible is the crest and the ocean slope above elevation -1 ft MLLW. For the most part, the sand barrier is in good condition and has incurred little deterioration since last repaired in 1965. The barrier was designed to be 2,640 feet long, however, its present length is approximately 2,500 feet. The shore end just before Station 0+10 appears to be the original end, but the Whaler Island end is obscured. Quarrying and subsequent harbor construction have modified that end and, above Station 24+35, the armor is smaller and blends with scattered piles of quarry stone, road base and rock on the adjoining beach. Between Stations 21+95 and 24+35, the armor has been scalped and used to construct a short groin from Whaler Island about 200 ft to the south. The scalped slope is about 1V:3H, covered mostly with core size material from gravel to 6 inches. An estimated one-third of the slope contains scattered remnants of small armor, generally about 2 cubic feet (350 lbs) with the largest size being 10 cubic feet (1700 lbs). Apparently the scalping, which occurred in 1984, was allowed because of the area's protected location by the island and the groin. This end has sustained no storm damage in the past 2 years.

The crest of the revetment along the sand barrier averages about 10 ft in width from the edge of the adjoining road fill. The crest is full width at about Station 3+00, narrowing to a one-stone width of 3-4 ft at the shoreward end of the barrier. The sand barrier has been repaired twice, both times to the crest. The first repair, done in 1949, extended from about Station 11+00 to 15+00 (1949 stationing of 9+75 to 13+75). The second repair, done in 1952, covered from Station 15+00 to 24+00. As mapped in 1986, the actual 1949 repair began at Station 9+75 instead of 11+00. The present up-station end abuts the 1952 repair at Station 14+50. The actual 1952 repair, in present stationing, extends from Station 14+50 to beyond 21+95 and may have included 50 ft of the 1949 repair. Another patch of the 1952 repair extends from Station 9+25 to 9+75 (present stationing). During the 1952 repair work, the crest in the repair sections was raised to a full +13-foot elevation. It is not known when the remainder of the crest was raised to a full +13 feet MLLW, although the 1949 repair also appears to have raised that reach of the crest. From Station 17+65 down to about 11+00, pieces of greenstone from Whaler Island to 6-cubic foot size are scattered along the crest near the road. The stone may be related to the road construction rather than barrier repairs.

Throughout the ocean slope of the sand barrier, the armor stone is in very good condition, especially below the mean high tide line where the surface deterioration is light; although the edges may be slightly rounded, pieces remain angular. The armor is significantly rounded in a small zone around Station 21+00 where cobbles on the adjoining beach have worn them. Overall, little armor is missing on the barrier and is limited to between Stations 3+00 and 8+00, except for the area scalped for armor beyond Station 21+95. Single stones are missing at Stations 3+30, 6+00, 6+45, 7+35 and 7+85, and two stones are missing at Station 4+60 where some smaller stone is exposed. The small stone has been exposed for some time, as they are rounded, but does not indicate a weak area in the structure. Below water investigations were not conducted for the sand barrier due to shoaling that has occurred on the seaward slope, and the harbor slope has been backfilled.

4.0 CONCLUSIONS

4.1 Inner Breakwater

In general the structure is in satisfactory condition. The only repairs that are recommended at this time would be to replace material that has been washed out adjacent to the concrete diaphragm wall between stations 13+85 and 15+00 on the seaward side and between stations 14+15 and 15+00 on the harbor side. A monitoring program for the inner breakwater could easily be tied into the existing monitoring program for the outer breakwater. It would be cost effective for surveys of both structures to be done at the same time, which would include aerial photographs and ground survey. The frequency with which the inner breakwater should be surveyed should be less than that of the outer breakwater, perhaps every 3-5 years. A visual inspection of the breakwater should be made at least yearly, and after severe storm events. This inspection can be made by the local authorities who can then report their findings to the responsible district personnel.

4.2 Sand Barrier

The sand barrier is in satisfactory condition. Shoaling of material on the seaward side of the barrier serves to protect the structure against direct impact of waves. The protection provided by the sediment accumulation is not permanent and can vary seasonally. It is therefore recommended that the barrier be repaired along its entire length, where deficiencies exist. Armor stone should be replaced at stations 3+30, 4+60, 6+00, 6+45, 7+35 and at 7+85. The reach between stations 21+85 and 24+35 on the seaward side should be evaluated in terms of its erosion potential and possible effects on the stability of the structure.

The monitoring program for the sand barrier should coincide with the program for the inner breakwater. Surveys and evaluations should be conducted at the same time.

ATTACHMENT 1

STATION 6 41.08N 127.34W AZIMUTH(DEGREES) = 0.
PERCENT OCCURRENCE(X1000) OF HEIGHT AND PERIOD BY DIRECTION

HEIGHT(METRES) — PEAK PERIOD(SECONDS)

HEIGHT(METRES)	4.4-6.0	6.1-8.0	8.1-9.5	9.6-10.5	10.6-11.7	11.8-13.3	13.4-15.3	15.4-18.1	18.2-22.2	22.3-LONGER	TOTAL
0.-0.9	5	.	.	.	.	.	.	.	.	.	5
1.0-1.9	34	49	10	.	.	.	.	.	.	.	93
2.0-2.9	66	1322	5	.	1	1	.	.	.	.	1395
3.0-3.9	.	1177	154	.	5	.	8	.	.	.	1344
4.0-4.9	.	23	395	3	.	.	.	1	.	.	422
5.0-5.9	.	.	112	18	.	.	.	.	.	.	130
6.0-6.9	.	.	1	30	3	.	.	.	.	.	34
7.0-7.9	.	.	.	.	.	.	.	.	.	.	0
8.0-8.9	.	.	.	.	.	.	.	.	.	.	0
9.0-9.9	.	.	.	.	.	.	.	.	.	.	0
10.0+	.	.	.	.	.	.	.	.	.	.	0
TOTAL	105	2571	677	51	9	1	8	1	0	0	

MEAN HS(M) = 3.2 LARGEST HS(M)= 6.8 MEAN TP(SEC)= 7.3 NO. OF CASES= 2007.

STATION 6 41.08N 127.34W AZIMUTH(DEGREES) = 22.5
PERCENT OCCURRENCE(X1000) OF HEIGHT AND PERIOD BY DIRECTION

HEIGHT(METRES) — PEAK PERIOD(SECONDS)

HEIGHT(METRES)	4.4-6.0	6.1-8.0	8.1-9.5	9.6-10.5	10.6-11.7	11.8-13.3	13.4-15.3	15.4-18.1	18.2-22.2	22.3-LONGER	TOTAL
0.-0.9	.	.	.	.	.	.	.	.	.	.	0
1.0-1.9	.	6	1	.	.	.	.	.	.	.	7
2.0-2.9	.	71	3	.	.	.	.	.	.	.	74
3.0-3.9	.	90	10	.	.	.	.	.	.	.	100
4.0-4.9	.	5	37	.	.	.	.	.	.	.	42
5.0-5.9	.	.	10	.	.	.	.	.	.	.	10
6.0-6.9	.	.	.	.	.	.	.	.	.	.	0
7.0-7.9	.	.	.	.	.	.	.	.	.	.	0
8.0-8.9	.	.	.	.	.	.	.	.	.	.	0
9.0-9.9	.	.	.	.	.	.	.	.	.	.	0
10.0+	.	.	.	.	.	.	.	.	.	.	0
TOTAL	0	172	61	0	0	0	0	0	0	0	

MEAN HS(M) = 3.3 LARGEST HS(M)= 5.4 MEAN TP(SEC)= 7.5 NO. OF CASES= 139.

STATION 6 41.08N 127.34W AZIMUTH(DEGREES) = 45.0
PERCENT OCCURRENCE(X1000) OF HEIGHT AND PERIOD BY DIRECTION

HEIGHT(METRES) — PEAK PERIOD(SECONDS)

HEIGHT(METRES)	4.4-6.0	6.1-8.0	8.1-9.5	9.6-10.5	10.6-11.7	11.8-13.3	13.4-15.3	15.4-18.1	18.2-22.2	22.3-LONGER	TOTAL
0.-0.9	.	.	.	.	.	.	.	.	.	.	0
1.0-1.9	.	.	1	.	.	.	.	.	.	.	1
2.0-2.9	.	5	6	.	.	.	.	.	.	.	11
3.0-3.9	.	.	6	.	.	.	.	.	.	.	6
4.0-4.9	.	.	6	.	.	.	.	.	.	.	6
5.0-5.9	.	.	.	.	.	.	.	.	.	.	0
6.0-6.9	.	.	.	.	.	.	.	.	.	.	0
7.0-7.9	.	.	.	.	.	.	.	.	.	.	0
8.0-8.9	.	.	.	.	.	.	.	.	.	.	0
9.0-9.9	.	.	.	.	.	.	.	.	.	.	0
10.0+	.	.	.	.	.	.	.	.	.	.	0
TOTAL	0	5	19	0	0	0	0	0	0	0	

MEAN HS(M) = 3.2 LARGEST HS(M)= 4.6 MEAN TP(SEC)= 8.2 NO. OF CASES= 16.

STATION 6 41.08N 127.34W AZIMUTH(DEGREES) = 67.5
PERCENT OCCURRENCE(X1000) OF HEIGHT AND PERIOD BY DIRECTION

HEIGHT(METRES) — PEAK PERIOD(SECONDS)

HEIGHT(METRES)	4.4-6.0	6.1-8.0	8.1-9.5	9.6-10.5	10.6-11.7	11.8-13.3	13.4-15.3	15.4-18.1	18.2-22.2	22.3-LONGER	TOTAL
0.-0.9	.	.	.	.	.	.	.	.	.	.	0
1.0-1.9	.	.	.	.	.	.	.	.	.	.	0
2.0-2.9	.	.	.	.	.	.	.	.	.	.	0
3.0-3.9	.	3	.	.	.	.	.	.	.	.	3
4.0-4.9	.	1	.	.	.	.	.	.	.	.	1
5.0-5.9	.	.	.	.	.	.	.	.	.	.	0
6.0-6.9	.	.	.	.	.	.	.	.	.	.	0
7.0-7.9	.	.	.	.	.	.	.	.	.	.	0
8.0-8.9	.	.	.	.	.	.	.	.	.	.	0
9.0-9.9	.	.	.	.	.	.	.	.	.	.	0
10.0+	.	.	.	.	.	.	.	.	.	.	0
TOTAL	0	4	0	0	0	0	0	0	0	0	

MEAN HS(M) = 3.4 LARGEST HS(M)= 4.1 MEAN TP(SEC)= 7.6 NO. OF CASES= 3.

STATION 6 41.08N 127.34W AZIMUTH(DEGREES) = 90.0
PERCENT OCCURRENCE(X1000) OF HEIGHT AND PERIOD BY DIRECTION

HEIGHT(METRES)	PEAK PERIOD(SECONDS) 4.4-6.0	6.1-8.0	8.1-9.5	9.6-10.5	10.6-11.7	11.8-13.3	13.4-15.3	15.4-18.1	18.2-22.2	22.3-LONGER	TOTAL
0.-0.9	.	.	.	.	.	.	.	.	.	.	0
1.0-1.9	.	.	.	.	.	.	.	.	.	.	0
2.0-2.9	.	.	.	.	.	.	.	.	.	.	0
3.0-3.9	.	6	.	.	.	.	.	.	.	.	6
4.0-4.9	.	.	.	.	.	.	.	.	.	.	0
5.0-5.9	.	.	.	.	.	.	.	.	.	.	0
6.0-6.9	.	.	.	.	.	.	.	.	.	.	0
7.0-7.9	.	.	.	.	.	.	.	.	.	.	0
8.0-8.9	.	.	.	.	.	.	.	.	.	.	0
9.0-9.9	.	.	.	.	.	.	.	.	.	.	0
10.0+	.	.	.	.	.	.	.	.	.	.	0
TOTAL	0	6	0	0	0	0	0	0	0	0	

MEAN HS(M) = 3.4 LARGEST HS(M)= 3.6 MEAN TP(SEC)= 7.6 NO. OF CASES= 4.

STATION 6 41.08N 127.34W AZIMUTH(DEGREES) =112.5
PERCENT OCCURRENCE(X1000) OF HEIGHT AND PERIOD BY DIRECTION

HEIGHT(METRES)	PEAK PERIOD(SECONDS) 4.4-6.0	6.1-8.0	8.1-9.5	9.6-10.5	10.6-11.7	11.8-13.3	13.4-15.3	15.4-18.1	18.2-22.2	22.3-LONGER	TOTAL
0.-0.9	.	.	.	.	.	.	.	.	.	.	0
1.0-1.9	.	.	.	.	.	.	.	.	.	.	0
2.0-2.9	.	.	.	.	.	.	.	.	.	.	0
3.0-3.9	.	1	.	.	.	.	.	.	.	.	1
4.0-4.9	.	.	1	.	.	.	.	.	.	.	1
5.0-5.9	.	.	.	.	.	.	.	.	.	.	0
6.0-6.9	.	.	.	.	.	.	.	.	.	.	0
7.0-7.9	.	.	.	.	.	.	.	.	.	.	0
8.0-8.9	.	.	.	.	.	.	.	.	.	.	0
9.0-9.9	.	.	.	.	.	.	.	.	.	.	0
10.0+	.	.	.	.	.	.	.	.	.	.	0
TOTAL	0	1	1	0	0	0	0	0	0	0	

MEAN HS(M) = 4.1 LARGEST HS(M)= 4.3 MEAN TP(SEC)= 7.9 NO. OF CASES= 2.

STATION 6 41.08N 127.34W AZIMUTH(DEGREES) =135.0
PERCENT OCCURRENCE(X1000) OF HEIGHT AND PERIOD BY DIRECTION

HEIGHT(METRES)	PEAK PERIOD(SECONDS) 4.4-6.0	6.1-8.0	8.1-9.5	9.6-10.5	10.6-11.7	11.8-13.3	13.4-15.3	15.4-18.1	18.2-22.2	22.3-LONGER	TOTAL
0.-0.9	.	.	.	.	.	.	.	.	.	.	0
1.0-1.9	.	.	.	.	.	.	.	.	.	.	0
2.0-2.9	.	.	.	.	.	.	.	.	.	.	0
3.0-3.9	.	.	.	.	.	.	.	.	.	.	0
4.0-4.9	.	.	1	.	.	.	.	.	.	.	1
5.0-5.9	.	.	.	.	.	.	.	.	.	.	0
6.0-6.9	.	.	.	.	.	.	.	.	.	.	0
7.0-7.9	.	.	.	.	.	.	.	.	.	.	0
8.0-8.9	.	.	.	.	.	.	.	.	.	.	0
9.0-9.9	.	.	.	.	.	.	.	.	.	.	0
10.0+	.	.	.	.	.	.	.	.	.	.	0
TOTAL	0	0	1	0	0	0	0	0	0	0	

MEAN HS(M) = 4.5 LARGEST HS(M)= 4.5 MEAN TP(SEC)= 8.2 NO. OF CASES= 1.

STATION 6 41.08N 127.34W AZIMUTH(DEGREES) =157.5
PERCENT OCCURRENCE(X1000) OF HEIGHT AND PERIOD BY DIRECTION

HEIGHT(METRES)	PEAK PERIOD(SECONDS) 4.4-6.0	6.1-8.0	8.1-9.5	9.6-10.5	10.6-11.7	11.8-13.3	13.4-15.3	15.4-18.1	18.2-22.2	22.3-LONGER	TOTAL
0.-0.9	.	.	.	.	.	.	.	.	.	.	0
1.0-1.9	.	.	.	.	.	.	.	.	.	.	0
2.0-2.9	.	6	.	.	.	.	.	.	.	.	6
3.0-3.9	.	1	.	.	.	.	.	.	.	.	1
4.0-4.9	.	.	10	.	.	.	.	.	.	.	10
5.0-5.9	.	.	3	.	.	.	.	.	.	.	3
6.0-6.9	.	.	1	6	3	.	.	.	.	.	10
7.0-7.9	.	.	.	1	.	.	.	.	.	.	1
8.0-8.9	.	.	.	.	1	.	.	.	.	.	1
9.0-9.9	.	.	.	.	.	3	.	.	.	.	3
10.0+	.	.	.	.	.	3	.	.	.	.	3
TOTAL	0	7	14	7	4	6	0	0	0	0	

MEAN HS(M) = 5.8 LARGEST HS(M)= 10.3 MEAN TP(SEC)= 9.3 NO. OF CASES= 26.

STATION 6 41.08N 127.34W AZIMUTH(DEGREES) =180.0
PERCENT OCCURRENCE(X1000) OF HEIGHT AND PERIOD BY DIRECTION

HEIGHT(METRES)	4.4-6.0	6.1-8.0	8.1-9.5	9.6-10.5	10.6-11.7	11.8-13.3	13.4-15.3	15.4-18.1	18.2-22.2	22.3-LONGER	TOTAL
0. -0.9	.	.	.	.	.	.	.	.	.	.	0
1.0-1.9	.	.	.	.	.	.	.	.	.	.	0
2.0-2.9	1	17	5	.	.	.	.	.	.	.	23
3.0-3.9	.	11	17	1	3	.	.	.	.	.	32
4.0-4.9	.	10	30	6	25	.	3	.	.	.	74
5.0-5.9	.	.	59	39	3	.	5	.	.	.	105
6.0-6.9	.	.	5	56	18	1	.	.	.	.	80
7.0-7.9	.	.	.	8	42	6	.	.	.	.	55
8.0-8.9	.	.	.	.	16	15	.	.	.	.	33
9.0-9.9	.	.	.	.	5	17	.	.	.	.	22
10.0+	.	.	.	.	.	25	5	.	.	.	30
TOTAL	1	38	116	110	114	64	13	0	0	0	

MEAN HS(M) = 6.2 LARGEST HS(M)= 11.2 MEAN TP(SEC)= 10.1 NO. OF CASES= 275.

STATION 6 41.08N 127.34W AZIMUTH(DEGREES) =202.5
PERCENT OCCURRENCE(X1000) OF HEIGHT AND PERIOD BY DIRECTION

HEIGHT(METRES)	4.4-6.0	6.1-8.0	8.1-9.5	9.6-10.5	10.6-11.7	11.8-13.3	13.4-15.3	15.4-18.1	18.2-22.2	22.3-LONGER	TOTAL
0. -0.9	.	.	.	.	.	.	.	.	.	.	0
1.0-1.9	.	.	.	.	.	.	.	.	.	.	0
2.0-2.9	6	18	23	3	1	.	.	.	.	.	51
3.0-3.9	.	63	30	18	3	.	.	.	.	.	114
4.0-4.9	.	30	152	11	35	18	1	.	.	.	247
5.0-5.9	.	.	154	90	42	22	8	.	.	.	316
6.0-6.9	.	.	29	126	92	17	3	.	.	.	267
7.0-7.9	.	.	1	39	138	11	10	.	.	.	199
8.0-8.9	.	.	.	.	56	61	.	.	.	.	117
9.0-9.9	.	.	.	.	1	41	.	.	.	.	42
10.0+	.	.	.	.	.	15	30	.	.	.	45
TOTAL	6	111	389	287	368	185	52	0	0	0	

MEAN HS(M) = 6.0 LARGEST HS(M)= 13.6 MEAN TP(SEC)= 10.1 NO. OF CASES= 829.

STATION 6 41.08N 127.34W AZIMUTH(DEGREES) =225.0
PERCENT OCCURRENCE(X1000) OF HEIGHT AND PERIOD BY DIRECTION

HEIGHT(METRES)	4.4-6.0	6.1-8.0	8.1-9.5	9.6-10.5	10.6-11.7	11.8-13.3	13.4-15.3	15.4-18.1	18.2-22.2	22.3-LONGER	TOTAL
0. -0.9	.	.	.	.	.	.	.	.	.	.	0
1.0-1.9	1	1	.	.	.	.	.	.	.	.	2
2.0-2.9	23	75	42	6	13	8	.	.	.	.	167
3.0-3.9	.	193	107	107	54	107	5	.	.	.	573
4.0-4.9	.	94	287	78	142	121	30	.	.	.	752
5.0-5.9	.	3	237	203	80	128	37	.	.	.	688
6.0-6.9	.	.	54	242	219	94	66	1	.	.	676
7.0-7.9	.	.	1	65	229	106	53	5	.	.	459
8.0-8.9	.	.	.	5	78	85	30	5	.	.	203
9.0-9.9	.	.	.	.	5	46	10	.	.	.	61
10.0+	.	.	.	.	.	22	10	.	.	.	32
TOTAL	24	366	728	706	820	717	241	11	0	0	

MEAN HS(M) = 5.5 LARGEST HS(M)= 13.1 MEAN TP(SEC)= 10.4 NO. OF CASES= 2125.

STATION 6 41.08N 127.34W AZIMUTH(DEGREES) =247.5
PERCENT OCCURRENCE(X1000) OF HEIGHT AND PERIOD BY DIRECTION

HEIGHT(METRES)	4.4-6.0	6.1-8.0	8.1-9.5	9.6-10.5	10.6-11.7	11.8-13.3	13.4-15.3	15.4-18.1	18.2-22.2	22.3-LONGER	TOTAL
0. -0.9	.	.	.	.	.	.	.	.	.	.	0
1.0-1.9	1	34	85	29	8	10	.	.	.	.	167
2.0-2.9	30	116	304	217	237	44	.	5	.	.	953
3.0-3.9	1	282	116	220	545	562	77	3	.	.	1806
4.0-4.9	.	95	268	83	431	756	263	3	.	.	[illegible]
5.0-5.9	.	1	205	188	191	641	441	15	.	.	[illegible]
6.0-6.9	.	1	65	220	169	152	278	17	.	.	902
7.0-7.9	.	.	3	23	118	66	177	41	.	.	428
8.0-8.9	.	.	.	.	39	27	46	13	.	.	125
9.0-9.9	.	.	.	.	1	15	5	5	.	.	26
10.0+	.	.	.	.	.	.	10	6	.	.	16
TOTAL	32	529	1046	980	1739	2273	1297	108	0	0	

MEAN HS(M) = 4.6 LARGEST HS(M)= 12.0 MEAN TP(SEC)= 11.3 NO. OF CASES= 4694.

STATION 6 41.08N 127.34W AZIMUTH(DEGREES) =270.0
PERCENT OCCURRENCE(X1000) OF HEIGHT AND PERIOD BY DIRECTION

HEIGHT(METRES)	4.4-6.0	6.1-8.0	8.1-9.5	9.6-10.5	10.6-11.7	11.8-13.3	13.4-15.3	15.4-18.1	18.2-22.2	22.3-LONGER	TOTAL
0.-0.9	.	3	5	.	.	.	.	.	.	.	8
1.0-1.9	71	207	622	296	59	34	.	.	.	.	1289
2.0-2.9	23	165	838	1533	1413	617	106	6	.	.	4701
3.0-3.9	.	171	169	497	1858	2464	634	51	.	.	5844
4.0-4.9	.	54	265	104	576	3105	1635	70	.	.	5809
5.0-5.9	.	.	85	71	183	850	1471	164	.	.	2824
6.0-6.9	.	.	11	58	82	133	595	246	.	.	1125
7.0-7.9	.	.	.	8	32	17	123	118	.	.	298
8.0-8.9	.	.	.	.	6	18	47	65	.	.	136
9.0-9.9	.	.	.	.	1	3	.	.	.	.	4
10.0+	.	.	.	.	.	.	1	1	.	.	2
TOTAL	94	600	1995	2567	4210	7241	4612	721	0	0	

MEAN HS(M) = 3.9 LARGEST HS(M)= 10.8 MEAN TP(SEC)= 11.8 NO. OF CASES= 12896.

STATION 6 41.08N 127.34W AZIMUTH(DEGREES) =292.5
PERCENT OCCURRENCE(X1000) OF HEIGHT AND PERIOD BY DIRECTION

HEIGHT(METRES)	4.4-6.0	6.1-8.0	8.1-9.5	9.6-10.5	10.6-11.7	11.8-13.3	13.4-15.3	15.4-18.1	18.2-22.2	22.3-LONGER	TOTAL
0.-0.9	10	15	68	.	.	.	.	.	.	.	93
1.0-1.9	237	691	2464	970	434	34	17	1	.	.	4848
2.0-2.9	116	617	2150	3158	4467	2168	246	27	.	.	12949
3.0-3.9	.	261	260	554	2313	4671	1473	99	.	.	9631
4.0-4.9	.	25	224	128	533	2005	2414	220	10	.	5559
5.0-5.9	.	.	66	107	150	448	1059	299	.	.	2129
6.0-6.9	.	.	1	34	46	135	306	227	.	.	749
7.0-7.9	.	.	.	.	8	11	42	63	.	.	124
8.0-8.9	.	.	.	.	.	.	13	1	.	.	14
9.0-9.9	.	.	.	.	.	.	15	.	.	.	15
10.0+	.	.	.	.	.	.	.	.	.	.	0
TOTAL	363	1609	5233	4951	7951	9472	5585	937	10	0	

MEAN HS(M) = 3.2 LARGEST HS(M)= 9.9 MEAN TP(SEC)= 11.3 NO. OF CASES= 21118.

STATION 6 41.08N 127.34W AZIMUTH(DEGREES) =315.0
PERCENT OCCURRENCE(X1000) OF HEIGHT AND PERIOD BY DIRECTION

HEIGHT(METRES)	4.4-6.0	6.1-8.0	8.1-9.5	9.6-10.5	10.6-11.7	11.8-13.3	13.4-15.3	15.4-18.1	18.2-22.2	22.3-LONGER	TOTAL
0.-0.9	39	13	10	.	.	.	.	.	.	.	62
1.0-1.9	826	959	1216	299	135	35	.	5	.	.	3475
2.0-2.9	1043	2337	1353	1853	1413	186	56	.	.	.	8241
3.0-3.9	1	913	215	207	942	763	200	3	3	.	3247
4.0-4.9	.	54	172	30	87	174	155	8	5	.	685
5.0-5.9	.	.	22	6	.	8	70	8	.	.	114
6.0-6.9	.	.	.	.	.	.	15	3	.	.	18
7.0-7.9	.	.	.	.	.	.	.	.	.	.	0
8.0-8.9	.	.	.	.	.	.	.	.	.	.	0
9.0-9.9	.	.	.	.	.	.	.	.	.	.	0
10.0+	.	.	.	.	.	.	.	.	.	.	0
TOTAL	1909	4276	2988	2395	2577	1166	496	27	8	0	

MEAN HS(M) = 2.5 LARGEST HS(M)= 6.4 MEAN TP(SEC)= 8.8 NO. OF CASES= 9270.

STATION 6 41.08N 127.34W AZIMUTH(DEGREES) =337.5
PERCENT OCCURRENCE(X1000) OF HEIGHT AND PERIOD BY DIRECTION

HEIGHT(METRES)	4.4-6.0	6.1-8.0	8.1-9.5	9.6-10.5	10.6-11.7	11.8-13.3	13.4-15.3	15.4-18.1	18.2-22.2	22.3-LONGER	TOTAL
0.-0.9	8	.	.	.	.	.	.	.	.	.	8
1.0-1.9	472	210	53	1	.	.	.	.	.	.	736
2.0-2.9	713	3995	152	71	66	8	5	.	.	.	5010
3.0-3.9	.	1925	112	27	63	66	15	.	.	.	2208
4.0-4.9	.	159	369	.	5	27	13	3	1	.	577
5.0-5.9	.	1	37	3	.	10	1	.	.	.	52
6.0-6.9	.	.	3	6	.	.	.	.	.	.	9
7.0-7.9	.	.	.	.	.	.	.	.	.	.	0
8.0-8.9	.	.	.	.	.	.	.	.	.	.	0
9.0-9.9	.	.	.	.	.	.	.	.	.	.	0
10.0+	.	.	.	.	.	.	.	.	.	.	0
TOTAL	1193	6290	726	108	134	111	34	3	1	0	

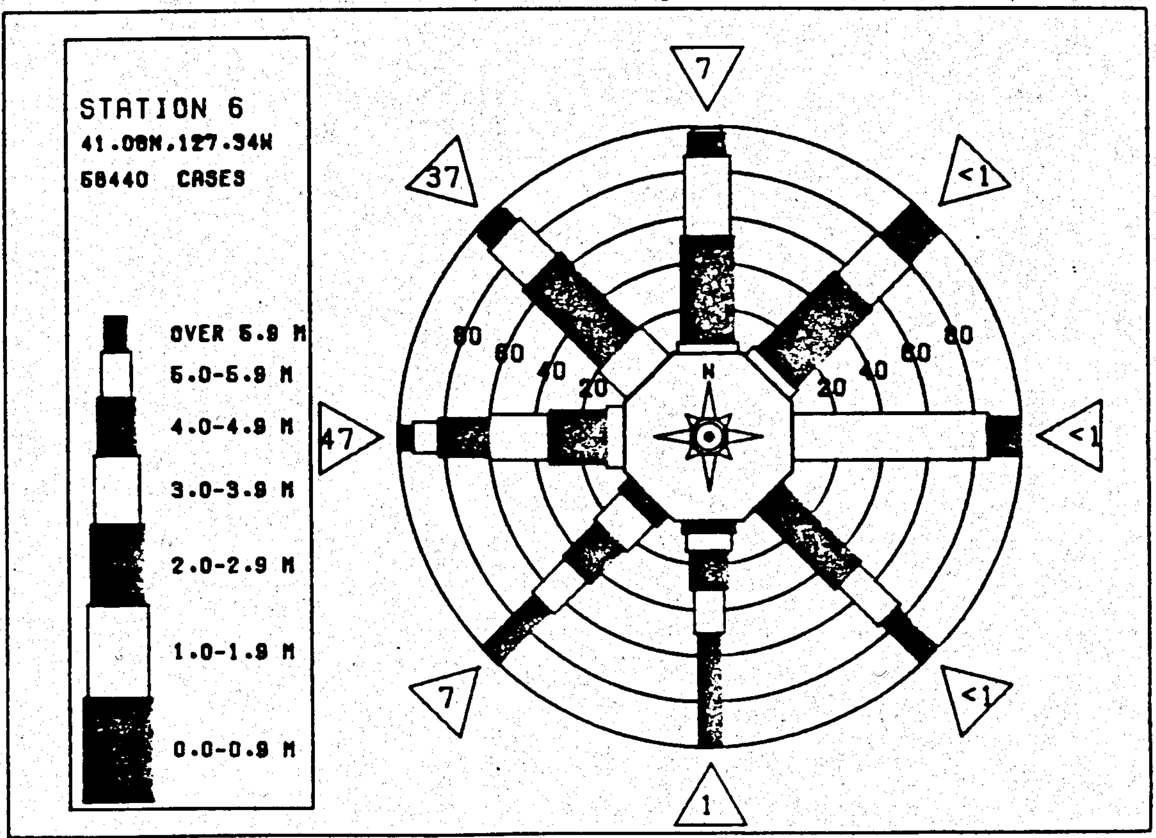
MEAN HS(M) = 2.8 LARGEST HS(M)= 6.6 MEAN TP(SEC)= 7.0 NO. OF CASES= 5035.

STATION 6 41.08N 127.34W FOR ALL DIRECTIONS

PERCENT OCCURRENCE(X100) OF HEIGHT AND FERICD FOR ALL DIRECTIONS

HEIGHT(METRES)	PEAK PERIOD(SECONDS)										TOTAL
	4.4-6.0	6.1-8.0	8.1-9.5	9.6-10.5	10.6-11.7	11.8-13.3	13.4-15.3	15.4-18.1	18.2-22.2	22.3-LONGER	
0.-0.9	6	3	8	.	.	.	.	.	.	.	17
1.0-1.9	164	216	445	159	63	11	1	.	.	.	1059
2.0-2.9	202	875	488	664	761	303	41	3	.	.	3357
3.0-3.9	.	510	120	163	579	863	241	15	.	.	2491
4.0-4.9	.	55	222	44	183	620	451	30	1	.	1606
5.0-5.9	.	.	99	73	65	210	309	48	.	.	804
6.0-6.9	.	.	17	78	63	53	126	49	.	.	385
7.0-7.9	.	.	.	14	56	22	40	22	.	.	154
8.0-8.9	.	.	.	.	20	20	13	8	.	.	61
9.0-9.9	.	.	.	.	1	12	3	.	.	.	16
10.0+	.	.	.	.	.	6	5	.	.	.	11
TOTAL	372	1659	1399	1215	1791	2120	1230	175	1	0	

MEAN HS(M)= 3.5 LARGEST HS(M)= 13.6 MEAN TP(SEC)= 10.4 TOTAL CASES= 58440.



MEAN HS(METRES) BY MONTH AND YEAR

WIS STATION 6 (41.08N 127.34W)

MONTH

YEAR	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP	OCT	NOV	DEC	MEAN
1956	5.2	4.2	4.0	3.1	2.9	2.8	2.5	2.3	2.1	2.8	3.1	3.8	3.2
1957	3.6	4.0	4.0	3.3	2.9	2.2	2.3	2.0	2.6	3.2	3.9	5.2	3.3
1958	6.2	5.5	4.2	3.9	2.5	2.2	2.6	2.3	2.4	3.0	4.1	4.5	3.6
1959	4.7	4.8	3.6	3.6	2.6	2.2	2.7	2.3	2.8	3.3	3.8	4.5	3.4
1960	4.9	5.2	3.2	3.3	2.7	2.6	2.5	2.4	2.2	3.1	4.9	4.6	3.5
1961	5.4	4.9	4.3	3.4	2.8	2.7	2.2	1.8	2.5	3.2	3.9	4.1	3.4
1962	3.6	4.2	3.7	2.9	2.2	2.2	2.5	2.1	2.4	3.7	4.8	5.0	3.3
1963	3.6	5.1	4.0	3.0	2.5	2.7	2.2	1.7	2.4	4.3	4.3	4.8	3.4
1964	5.8	3.7	4.0	3.6	2.6	2.4	2.7	2.3	2.4	2.8	4.3	5.5	3.5
1965	4.3	3.3	3.2	3.3	2.9	3.0	2.4	1.9	2.5	3.3	4.7	5.2	3.3
1966	5.6	4.3	4.4	2.9	3.0	2.2	2.4	2.5	2.3	2.6	3.9	5.2	3.4
1967	4.6	3.8	3.7	3.2	2.2	2.2	2.1	1.9	2.7	3.6	4.2	5.0	3.3
1968	4.8	4.7	4.2	2.9	2.1	2.2	2.3	2.1	2.1	3.3	4.3	4.8	3.3
1969	3.7	5.5	4.1	4.0	2.9	2.4	2.5	2.2	2.5	3.4	4.1	6.5	3.7
1970	5.4	5.1	4.4	3.7	2.6	2.4	2.5	2.2	2.4	3.4	4.3	5.5	3.7
1971	4.3	3.9	4.4	3.8	2.8	2.5	2.1	2.3	2.7	2.9	4.4	4.5	3.4
1972	5.2	4.7	4.2	3.9	3.0	2.9	2.8	2.2	2.4	3.0	5.2	5.1	3.7
1973	5.6	4.8	4.8	3.8	3.2	3.1	2.9	2.7	2.9	3.0	4.7	5.9	4.0
1974	5.3	4.6	4.9	3.9	3.3	3.0	2.4	2.3	2.3	3.1	4.4	5.2	3.7
1975	4.0	4.1	4.1	2.7	2.7	2.3	2.0	2.0	1.8	3.3	4.0	4.0	3.1
MEAN	4.8	4.5	4.1	3.4	2.7	2.5	2.4	2.2	2.4	3.2	4.3	5.0	

LARGEST HS(METRES) BY MONTH AND YEAR

WIS STATION 6 (41.08N 127.34W)

MONTH

YEAR	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP	OCT	NOV	DEC
1956	9.9	8.0	6.4	4.8	4.5	5.1	4.1	4.4	3.5	4.6	4.4	4.9
1957	6.5	9.4	5.8	4.6	4.1	3.5	5.4	3.7	5.3	5.4	6.1	8.2
1958	8.9	8.2	7.7	7.7	3.7	3.5	3.8	3.3	3.7	5.6	6.5	6.9
1959	8.9	9.8	5.6	6.0	4.6	3.2	4.4	3.4	4.2	5.4	6.5	7.4
1960	9.9	10.3	6.4	4.5	3.6	4.4	4.1	3.5	4.5	6.0	9.8	7.5
1961	8.4	7.0	6.8	6.8	3.5	3.8	3.8	3.3	4.9	6.1	6.8	7.4
1962	5.7	6.9	5.8	4.2	3.4	3.6	3.9	3.1	4.8	6.8	7.8	11.9
1963	8.5	8.7	7.4	4.5	5.0	4.5	3.8	3.2	3.2	8.6	6.1	9.7
1964	9.9	5.0	6.1	5.5	5.3	3.6	4.2	4.0	4.1	4.0	6.5	10.7
1965	6.6	5.5	5.1	4.9	4.2	4.4	3.5	3.6	4.8	5.6	10.8	9.6
1966	9.0	7.0	6.6	4.6	5.0	3.1	3.5	3.5	3.8	4.5	6.6	8.4
1967	7.4	5.3	7.1	5.0	3.3	3.6	3.3	2.9	4.5	6.7	6.2	8.6
1968	11.2	8.2	5.6	4.8	3.0	3.4	3.3	3.8	3.9	5.3	6.0	9.7
1969	6.0	8.1	6.5	6.3	4.9	3.7	4.4	3.5	3.5	5.1	9.0	13.1
1970	9.0	7.7	6.6	5.4	4.2	4.7	3.4	3.3	4.6	6.2	6.4	8.7
1971	10.7	6.1	8.3	5.3	4.4	3.3	3.5	3.9	5.9	4.9	7.8	7.4
1972	9.9	7.1	6.3	7.7	4.7	5.7	6.0	4.0	3.5	6.0	6.8	11.0
1973	9.0	8.1	6.7	5.3	6.4	5.4	5.6	4.9	4.4	4.2	7.5	10.9
1974	13.6	7.5	9.3	7.0	5.4	5.0	5.0	4.4	4.8	6.3	6.7	7.3
1975	6.5	7.7	8.1	3.6	4.8	5.3	2.8	3.5	3.8	6.7	7.0	8.5

20 YR. STATISTICS FOR PACIFIC STATION 6 (41.08N 127.34W)

MEAN SIGNIFICANT WAVE HEIGHT(METRES)=	3.5
MEAN PEAK WAVE PERIOD (SECONDS)=	10.5
MOST FREQUENT 22.5(CENTER) DIRECTION BAND (DEGREES)=	292.5
STANDARD DEVIATION OF HS(METRES)=	1.4
STANDARD DEVIATION OF TP(SECONDS)=	2.6
LARGEST HS(METRES)=	13.6
TP (SECONDS)ASSOC. WITH THE LARGEST HS=	14.3
AVE. DIRECTION (DEGREES) ASSOC. WITH THE LARGEST HS=	204.0
DATE OF LARGEST HS OCCURRENCE IS(YR,MO,DA,HR)	74011512

ATTACHMENT 2

DRAFT

WAVE HEIGHT ESTIMATE

SAND BARRIER AND INNER BREAKWATER

CRESCENT CITY, CALIFORNIA

The coastal engineering works of improvement at Crescent City, California, have experienced periodic and recurring structural damages since their initial construction. The outer breakwater, which receives the overwhelming majority of wave energy arriving from the North Pacific, has suffered the greater damage. The inner breakwater, which is significantly shielded by the outer breakwater, has experienced considerably less damage, although occasional large damaging waves are able to enter the harbor complex and cause accrued stone displacement to this structure also. The sand barrier is subjected to waves arriving from a southerly direction which strike the armor stone at an angle and create an unraveling effect, with resulting stone displacement. Because this is a shallow water surf zone location, the waves which strike the sand barrier often have breaking-wave characteristics, requiring larger stone for stability than non-breaking waves.

Sand Barrier

Deepwater wave hindcast statistics for the 20-year period 1956-1975 by US Army Engineer Waterways Experiment Station (WES) Wave Information Study (WIS) Sea-State Engineering Analysis System (SEAS), Station P 01-06, indicate that waves arrive from the open ocean direction of 180 deg azimuth with heights up to 30 ft and periods up to 13 sec. As these waves approach the structure and shoreline, changes occur due to refraction and shoaling which significantly alter these deepwater characteristics. The waves which ultimately strike the sand barrier are depth-limited waves whose height depends entirely on the water depth at the structure, d, the slope of the beach in front of the structure, m, and the period of the approaching wave train. The expression relating these independent variables is given by <u>Shore Protection Manual</u> (CERC 1984) as:

$$H = \frac{d\,b}{1 + \frac{d\,a}{g\,T^2}}$$

where

$$a = 43.75\,(1 - e^{-19m})$$

and

$$b = \frac{1.56}{(1 + e^{-19.5m})}$$

FROM (Name)	OFFICE SYMBOL	TELEPHONE NO.	RELEASER'S SIGNATURE		
Lynn Hales	WESCR-P	FTS 542-3207	Lyndell Z. Hales		
TO (Name)	OFFICE SYMBOL	TELEPHONE NO.	# PAGES	PRECEDENCE	DTG
G. Mikhchi	SPL PD-C	FTS 798-5403	5	1	15 Jan 87

DRAFT

The depth of water at the toe of the structure varies along the structure spatially, and also varies with time at specific locations along the structure, as differing wave climates induce differing current effects which tend to alternately scour and fill depressions. These locally varying depths do not in themselves cause instability to the structure, but do affect the height of the wave at that particular location. From the most recent nautical charts of the region, it appears the greatest water depth along the sand barrier is approximately 3 ft mean lower low water (mllw). The most damaging condition will occur at the greatest storm tide elevation; therefore, by utilizing a storm tide of +10 ft mllw, the depth of water for estimating breaking wave conditions at the structure is 13 ft. The bottom slope in this vicinity can be approximated as $m = 0.0107$. For the wave period band centered about 12.5 sec (11.8-13.3 sec) (WES Miscellaneous Paper CERC-85-3, "Water Wave Refraction/ Diffraction/Shoaling Investigation, Crescent City, California,") (CERC 1985), the following parameters are obtained,

$a = 8.05$
$b = 0.86$
$d = 13$ ft
$m = 0.0107$
$T = 12.2$ sec

resulting in the breaking wave height at the structure,

$H = 11.0$ ft, (breaking wave)

for the greatest water depth along the sand barrier at extreme high storm tide of +10 ft mllw.

The stone size for stability, W, required to withstand a breaking wave height of 11.0 ft can be estimated from CERC (1984)

$$W = \frac{w H^3}{K (S - 1)^3 \cot}$$

where

H = wave height = 11.0 ft
w = unit weight of rock = 165 lb/cu ft
K = Stability Coefficient = 2.0 for breaking wave on a structure trunk
S = specific gravity of armor unit = 1.65
$\cot$ = angle of seaside slope of structure = 1.5

~~There~~ This results in an average stone size of 8.1 ton, say

$W = 8$ tons

When the cover layer is two quarrystones in thickness, the stones comprising the primary cover layer can range from about $0.75\ W$ to $1.25\ W$, with about 50 percent of the individual stones weighing more than W. This indicates the minimum size stone should be about 6 tons.

DRAFT

Inner Breakwater

Waves which strike the inner breakwater arrive from south of about 220 deg azimuth, diffract around the outer structure, and propagate through the entrance channel to the harbor complex toward the inner breakwater. SEAS statistics and CERC (1985) indicate that waves can approach the entrance to the harbor complex with significant heights up to 32 ft. Diffraction through the entrance channel causes a reduction in wave height of approximately 50 percent along the inner breakwater structure. There results a non-breaking wave condition where the significant wave height for the maximum storm tide of +10 ft mllw is

H = 16.0 ft

Previous physical model tests utilizing earlier wave hindcast data from a much more limited time period had indicated the maximum wave height at the inner breakwater would be around 15 ft. It is believed the SEAS statistics are far more comprehensive than the previous hindcast, and the results of CERC (1985) indicate these deepwater waves arrive at the entrance channel essentially unattenuated in height, although the frequency of their occurrence is low. Refraction alters the direction of approach slightly, so that the waves pass into the harbor complex from a direction essentially perpendicular to the outer breakwater extension. For a non-breaking wave height of 16 ft, and cot = 1.75 , the stone size for stability is W = 10.7 ton, say

W = 11 tons

There exist other waves in the spectrum which arrive with heights in excess of these significant heights, although their frequency of occurrence is exceedingly small. Their actual heights, however, can be up to twice the significant height. Even though they occur very rarely with short durations, their presence is occasionally sufficient to dislodge individual stones of the structure which leads to later dislocation of additional stones. For this reason it would not be unreasonable to have a range of stone sizes which vary from 0.75 W to 1.25 W .

Summary and Conclusions

Based on this preliminary analysis of existing data, the following conclusions have been reached. It is emphasized that a far more extensive investigation should be conduction prior to any rehabilitation structural works at either the sand barrier or the inner breakwater. WES can perform a more comprehensive in-depth analysis in a relatively short time frame which will provide more definitive results, if requested. At present, the preliminary conclusions are:

DRAFT

Sand Barrier

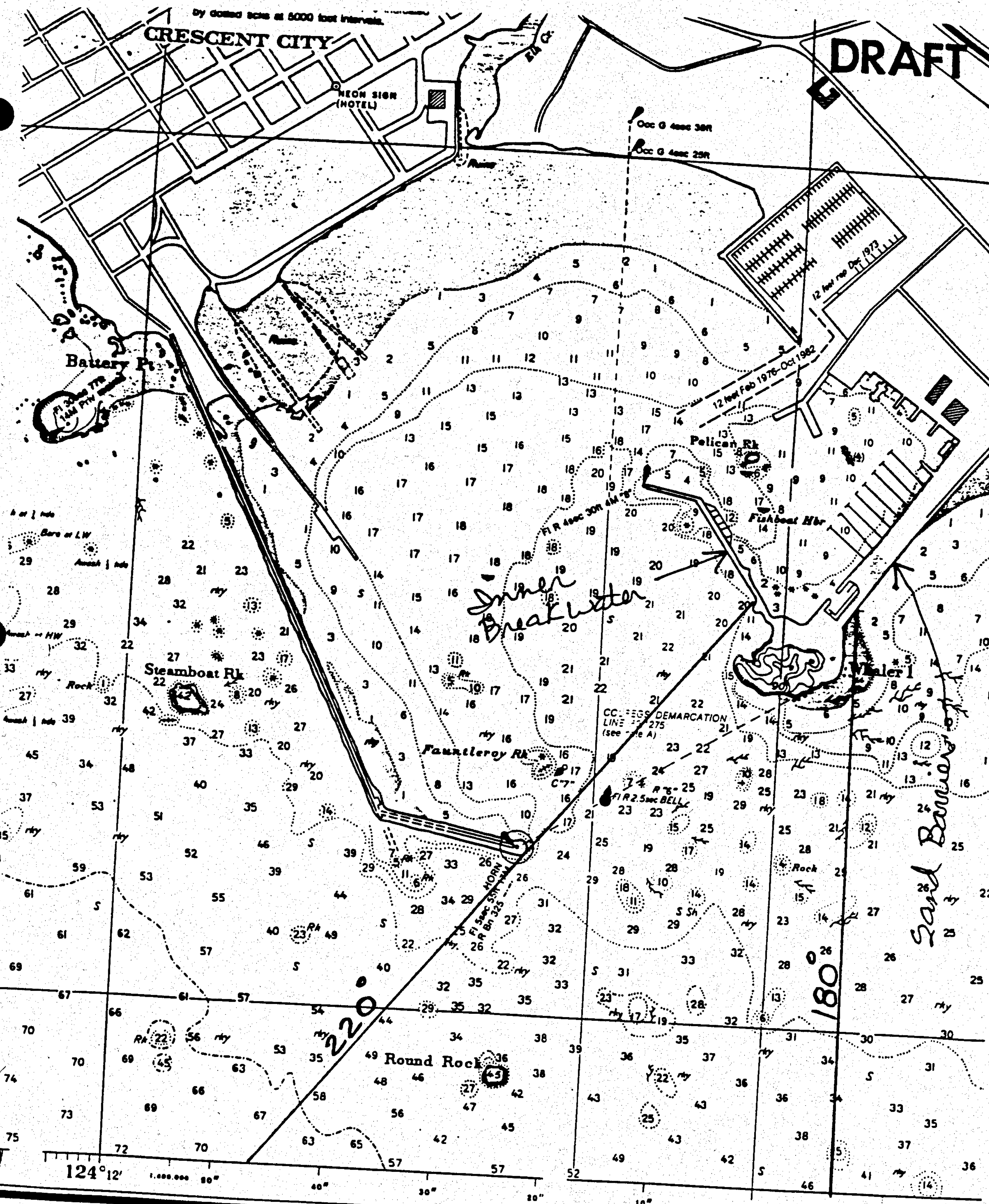
H = 11.0 ft (breaking wave height, K = 4)
W = 8 tons (average stone size)
d = 13 ft (storm tide elevation = +10 ft mllw)

Inner Breakwater

H = 16.0 ft (non-breaking wave height, K = 2)
W = 11 tons (average stone size)
d = 26 ft (storm tide elevation = +10 ft mllw)

Lynn Hales

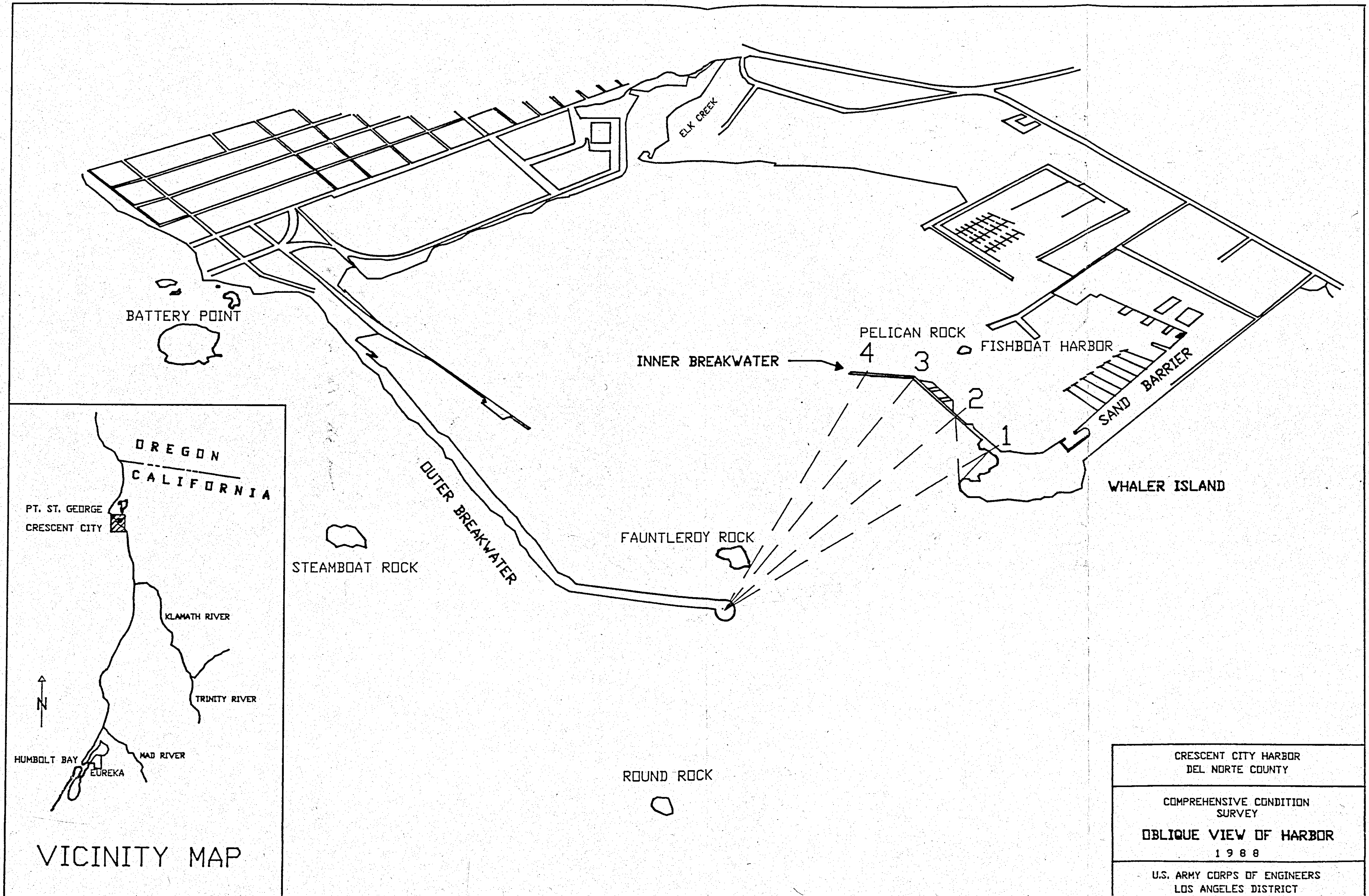
Lyndell Z. Hales, PhD, P.E.
Research Hydraulic Engineer
Coastal Engineering Research Center
US Army Engineer Waterways Experiment Station



CAUTION

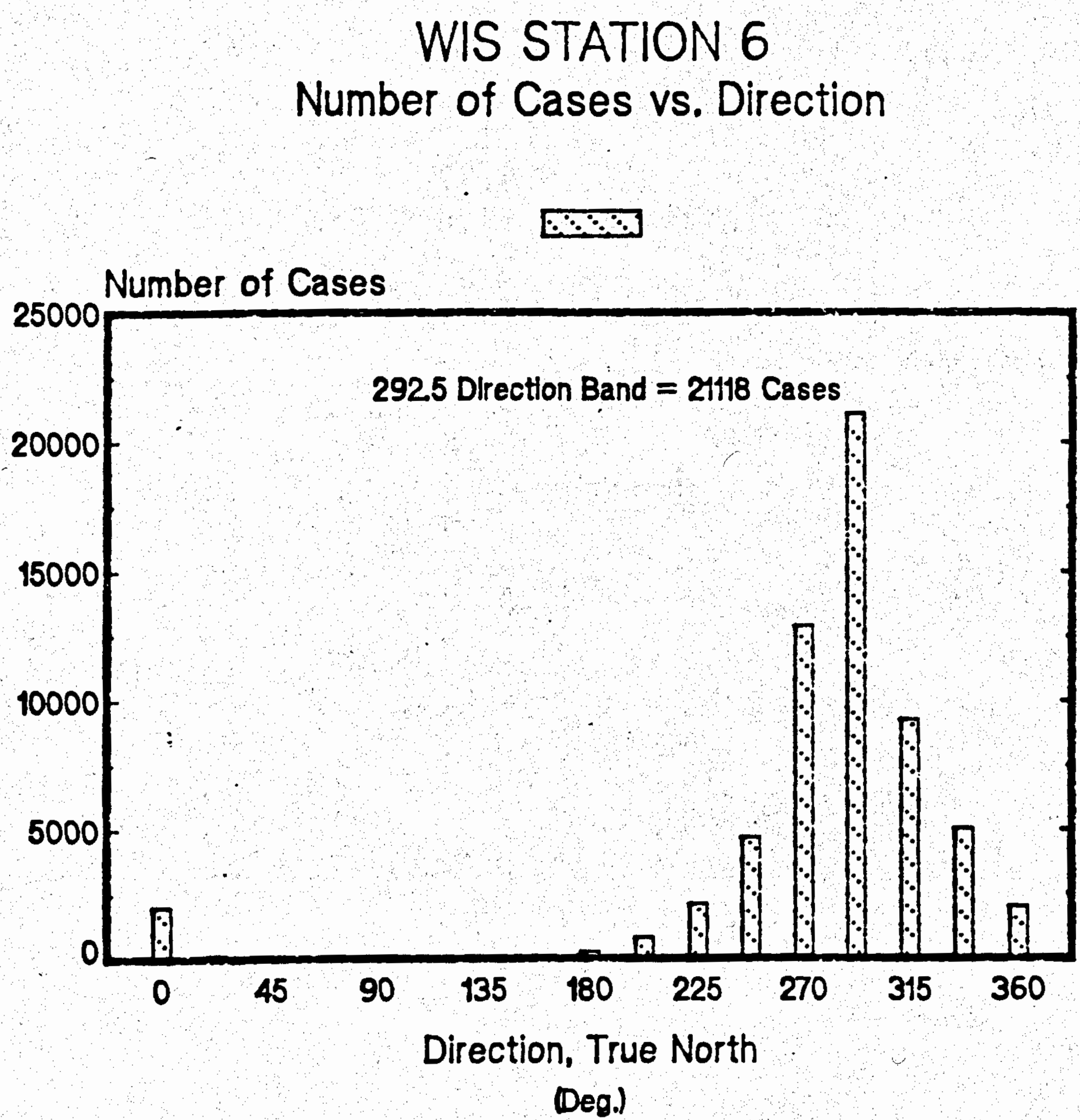
This chart has been corrected from the Notice to Mariners published weekly by the Defense Mapping Agency Hydrographic/Topographic Center and the Local Notice to Mariners issued periodically by each U.S. Coast G...

SOUNDINGS IN FEE



ELK CREEK
BATTERY POINT
PELICAN ROCK
FISHBOAT HARBOR
INNER BREAKWATER
4
3
2
1
SAND BARRIER
WHALER ISLAND
OUTER BREAKWATER
FAUNTLEROY ROCK
STEAMBOAT ROCK
ROUND ROCK
OREGON
CALIFORNIA
PT. ST. GEORGE
CRESCENT CITY
KLAMATH RIVER
TRINITY RIVER
N
HUMBOLT BAY
MAD RIVER
EUREKA
VICINITY MAP
CRESCENT CITY HARBOR
DEL NORTE COUNTY
COMPREHENSIVE CONDITION SURVEY
OBLIQUE VIEW OF HARBOR
1988
U.S. ARMY CORPS OF ENGINEERS
LOS ANGELES DISTRICT

FIGURE 1



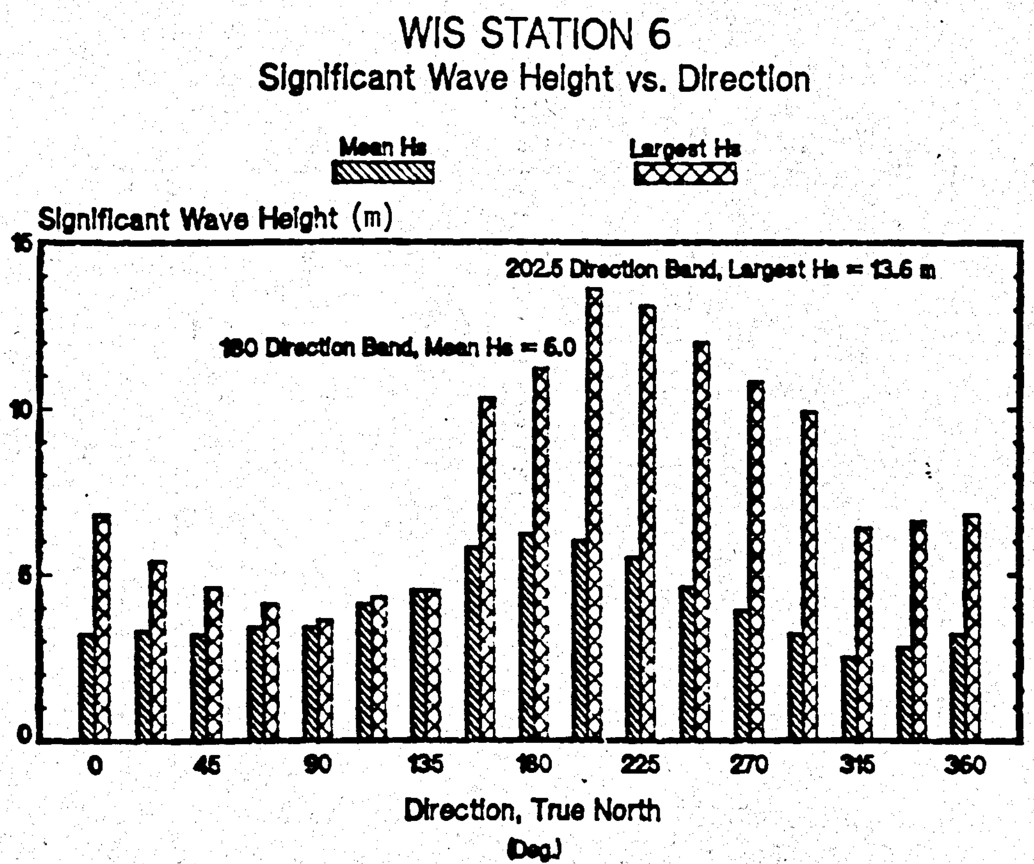
CRESCENT CITY HARBOR
DEL NORTE COUNTY

COMPREHENSIVE CONDITION SURVEY

WIS STATION 6
NUMBER OF CASES vs. DIRECTION
1 9 8 8

U.S. ARMY CORPS OF ENGINEERS
LOS ANGELES DISTRICT

FIGURE 2



CRESCENT CITY HARBOR
DEL NORTE COUNTY

COMPREHENSIVE CONDITION SURVEY

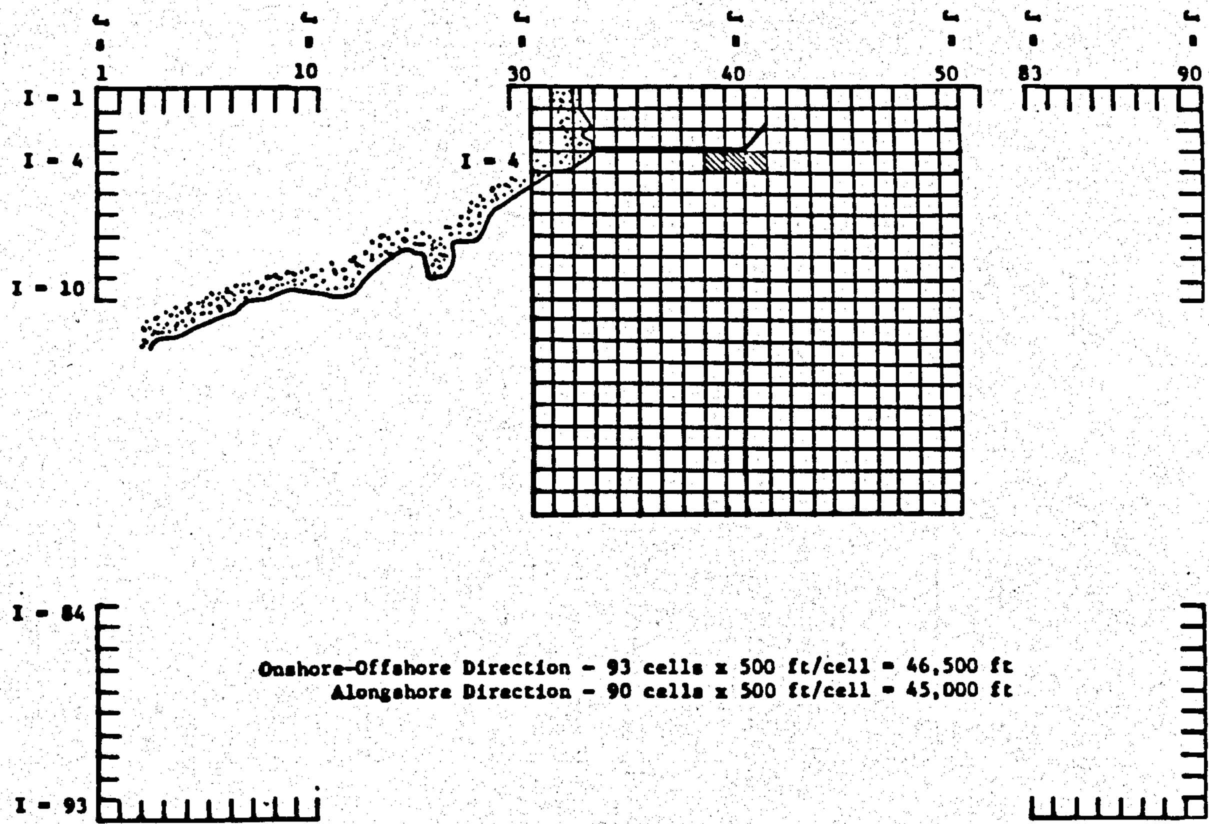
WIS STATION 6

SIGNIFICANT WAVE HEIGHT vs. DIRECTION

1988

U.S. ARMY CORPS OF ENGINEERS
LOS ANGELES DISTRICT

FIGURE 3



LEGEND

Grid cells used in analysis

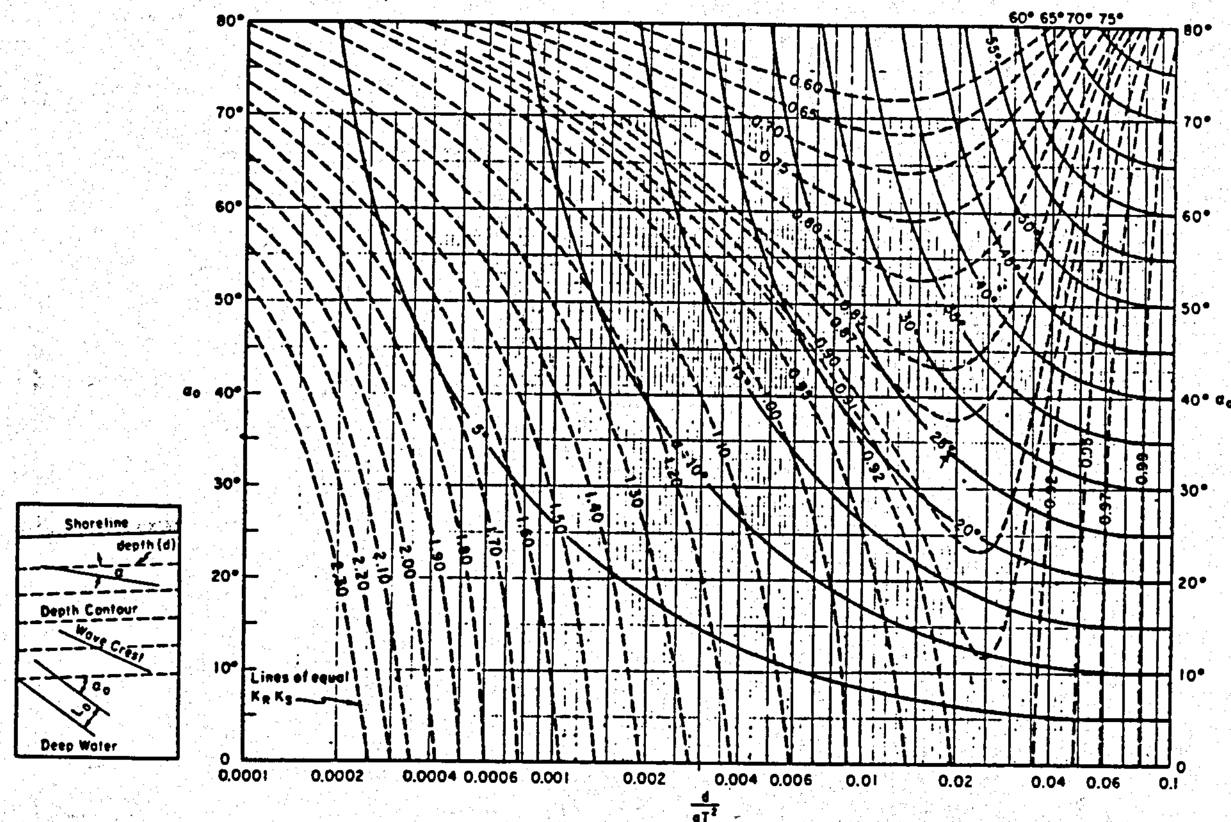
Reference: Hales, 1985

CRESCENT CITY HARBOR
DEL NORTE COUNTY

COMPREHENSIVE CONDITION SURVEY
RCPWAVE BATHYMETRIC GRID
1 9 8 8

U.S. ARMY CORPS OF ENGINEERS
LOS ANGELES DISTRICT

FIGURE 4



Change in wave direction and height due to refraction on slopes with straight, parallel depth contours including shoaling.

Reference: Shore Protection Manual, 1984

CRESCENT CITY HARBOR
DEL NORTE COUNTY

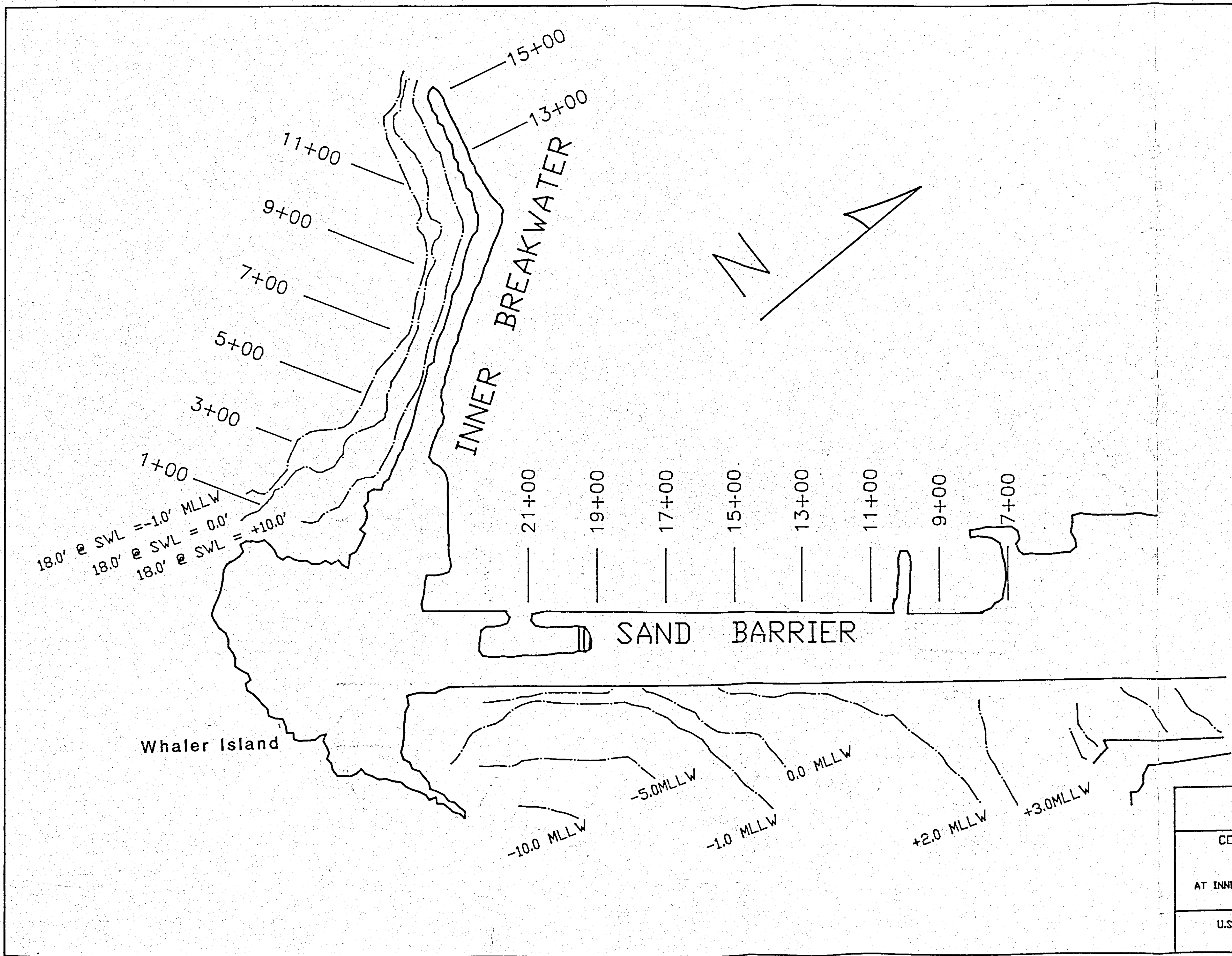
COMPREHENSIVE CONDITION SURVEY

REFRACTION AND SHOALING COEFFICIENTS

1 9 8 8

U.S. ARMY CORPS OF ENGINEERS
LOS ANGELES DISTRICT

FIGURE 5



CRESCENT CITY HARBOR
DEL NORTE COUNTY

COMPREHENSIVE CONDITION SURVEY

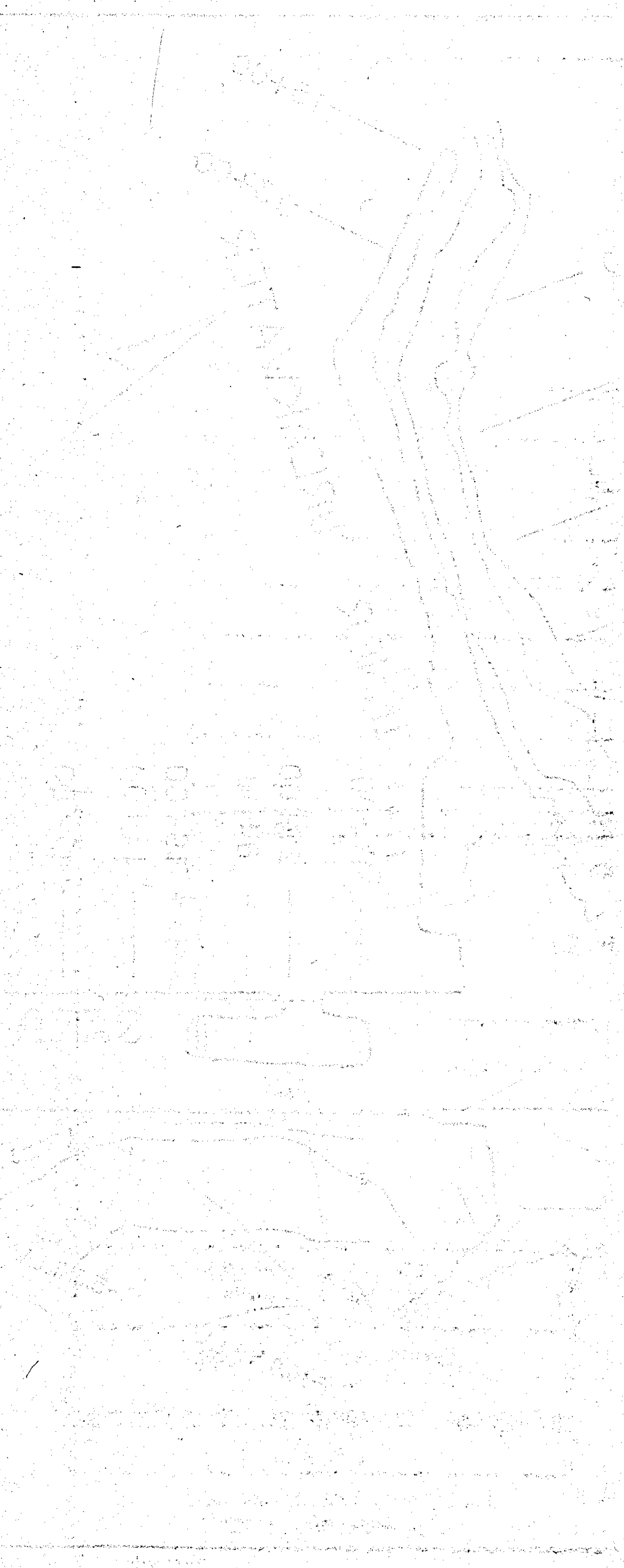
DEPTH CONTOURS

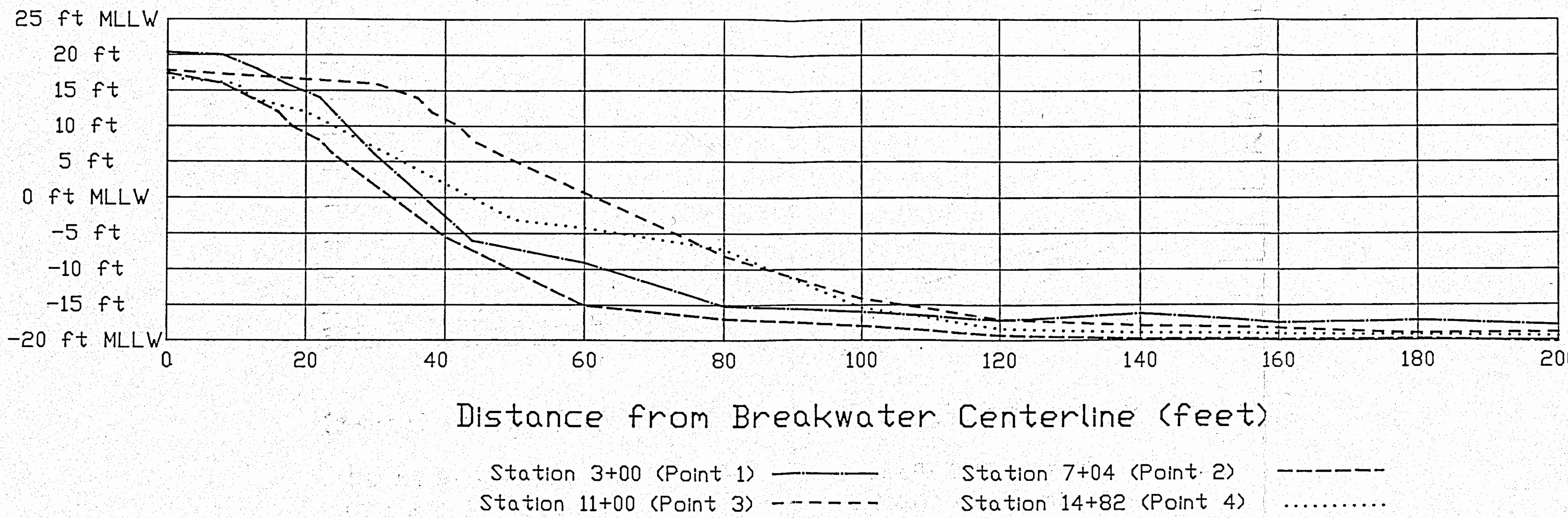
AT INNER BREAKWATER AND SAND BARRIER

1 9 8 8

U.S. ARMY CORPS OF ENGINEERS
LOS ANGELES DISTRICT

FIGURE 6





CRESCENT CITY HARBOR
DEL NORTE COUNTY

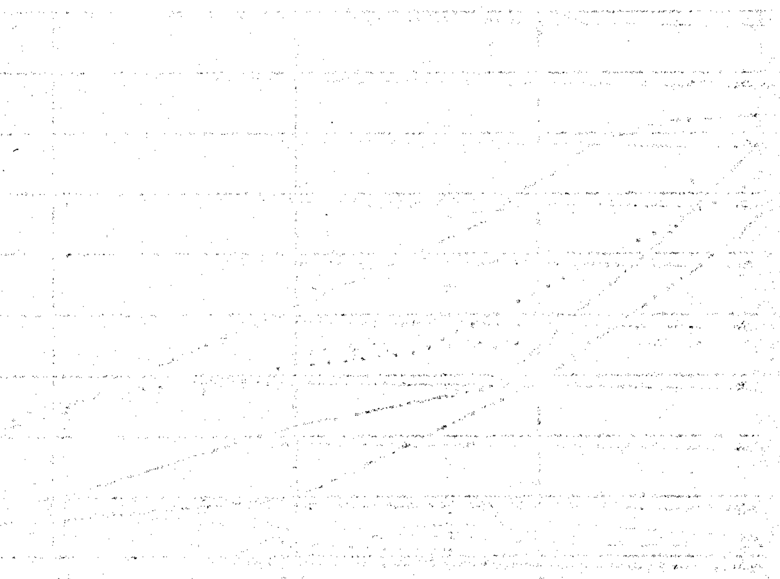
COMPREHENSIVE CONDITION SURVEY

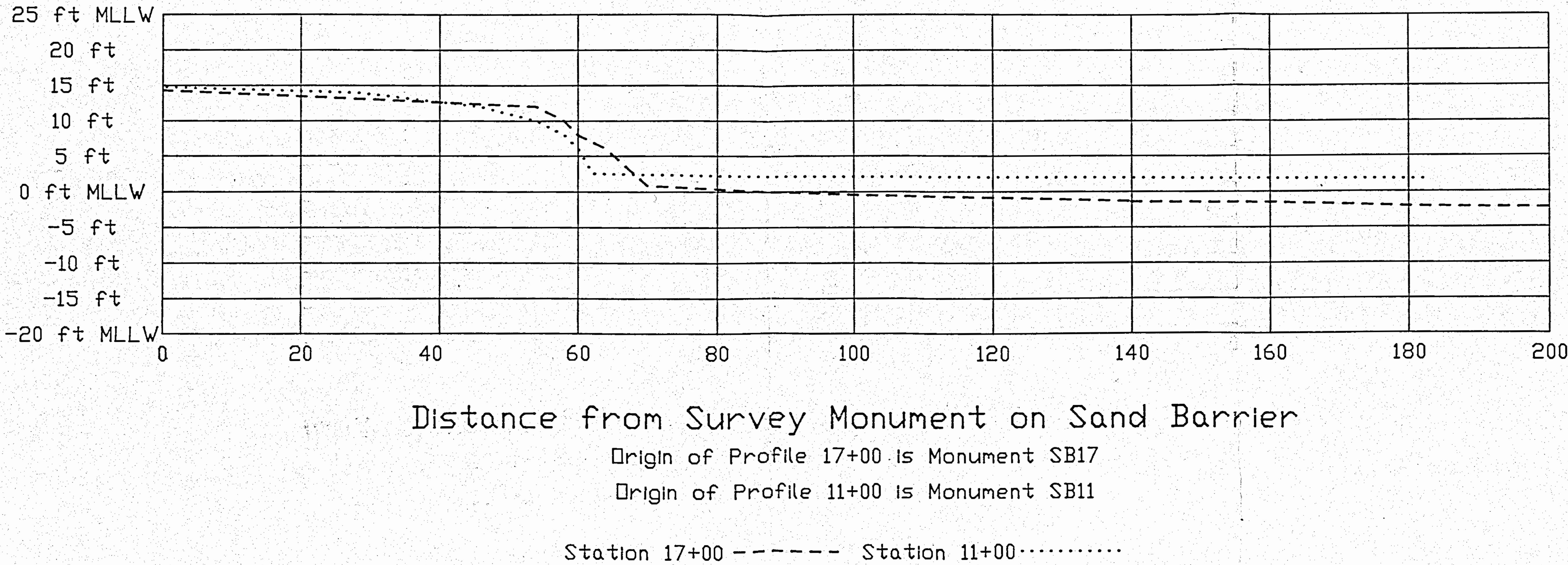
INNER BREAKWATER CROSS SECTIONS

1 9 8 8

U.S. ARMY CORPS OF ENGINEERS
LOS ANGELES DISTRICT

FIGURE 7





CRESCENT CITY HARBOR
DEL NORTE COUNTY

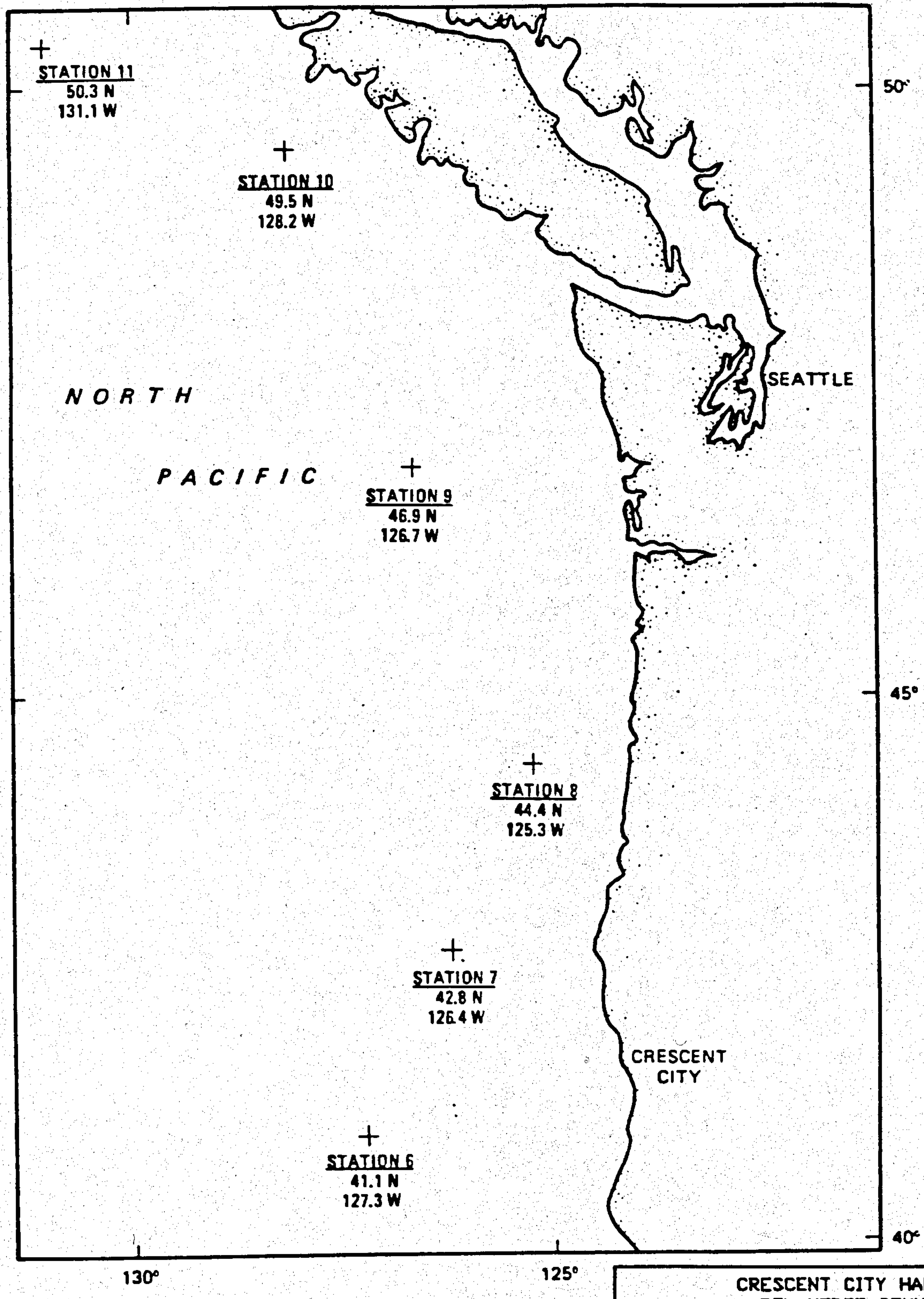
COMPREHENSIVE CONDITION SURVEY

SAND BARRIER CROSS SECTIONS

1988

U.S. ARMY CORPS OF ENGINEERS
LOS ANGELES DISTRICT

FIGURE 8



CRESCENT CITY HARBOR
DEL NORTE COUNTY

COMPREHENSIVE CONDITION
SURVEY

LOCATION OF WIS STATION 6

1 9 8 8

U.S. ARMY CORPS OF ENGINEERS
LOS ANGELES DISTRICT

FIGURE 9

APPENDIX B

DRAFT Geotechnical Report

COMPREHENSIVE CONDITION SURVEY

GEOTECHNICAL APPENDIX

Crescent City Harbor
Sand Barrier
Inner Breakwater
Del Norte County, California

U.S. ARMY CORPS OF ENGINEERS
SAN FRANCISCO DISTRICT

prepared by

U.S. ARMY CORPS OF ENGINEERS
LOS ANGELES DISTRICT

March 1987

COMPREHENSIVE CONDITION SURVEY
Crescent City Harbor
Sand Barrier and Inner Breakwater
GEOTECHNICAL APPENDIX

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PLATES

ATTACHMENTS

PHOTOGRAPHS

Comprehensive Condition Survey

Geotechnical Appendix

Crescent City Sand Barrier and Inner Breakwater

1. INTRODUCTION

1.1 Purpose and Scope.

The purpose of this appendix is to present an assessment of the geotechnical condition of the Crescent City Harbor sand barrier and the inner breakwater. The assessment was made in 1986 by the Los Angeles District at the request of the San Francisco District. The scope of this appendix is threefold: First, to determine the present condition and make-up of the two structures, secondly, to develop a rehabilitation program to address necessary repairs and thirdly, develop a monitoring and maintenance program to outline expected repairs and preventative maintenance measures for an additional 50-year period of service.

1.2 Location and Description.

Crescent City Harbor encloses a portion of the coastline on the southern edge of Crescent City, see plate 1. Other than the shore, the harbor is bounded by the main, or outer breakwater, extending from Battery Point on the west to south side, the sand barrier on the southeast from shore out to Whaler Island and the inner breakwater extending west from Whaler Island. The sand barrier was constructed in 1939 and has a length of about 2,500 feet, a crest elevation of +13 MLLW and a design crest width of 9 feet. The inner breakwater was constructed in 1946 to a length of 1100 feet and a 400-foot dog-leg extension was later constructed in 1972. The breakwater has a crest elevation of +18 feet MLLW and a crest width of 12 feet (15 feet on the extension). In this report the construction and design features of each inner breakwater segment are discussed separately. However, they are treated as a single unit in assessing their present condition. An assessment of the main (outer) breakwater was completed in 1984.

2. GEOTECHNICAL SURVEY PROGRAM

2.1 General.

The geotechnical investigations employed for the determination of the present condition and make-up, include an assessment of the interior and exterior of the two structures, the foundation conditions, design and the rock performance (both armor and core). The investigations included both field and office studies. Related work, but not necessarily geotechnical, includes evaluation of recent surveys (both above and below water), bathymetry and photography (both aerial and ground).

2.2 Office Studies and Literature Search.

In order to fully assess the existing condition of the sand barrier and the inner breakwater, first, a review of available literature was conducted to determine the sources of the stone used, design criteria, the methods of construction and the locations of subsequent repairs and any other pertinent information relevant to their construction and maintenance. The Annual Reports of the Chief of Engineers, the San Francisco District Chronicle "Engineers at the Golden Gate" (Hagwood, 1981) and district files and drawings provided most of the information. Additional information was obtained from library records of Crescent City's newspapers, the Triplicate and the Crescent American, personal communications with involved personnel and the Federal Archives. The accumulated data, combined with field investigation results, were assessed to determine the present condition of the structures. It was recognized early in the task that some data conflicted and that other were irretrievably lost such as the actual dates and locations for the sand barrier repairs and the stone sources used for those repairs.

2.3 Field Investigations.

2.3.1 Condition Mapping. For this study, San Francisco District re-surveyed the sand barrier and inner breakwater, setting monuments on 200-foot centers the full length of both structures. New stationing was also established. For the sand barrier, the new stationing is within a couple of feet of the original. However, on the inner breakwater, the original stations 0+00 to 11+20 on the trunk and 0+00 to 4+00 on the extension were combined into one reach from station 0+00 to 15+20. This new stationing is also 20 feet off from the original so that original station 11+20 is now 11+00. Following completion of the new survey, the above-water features of both structures were assessed using the monuments for control. On 30 April and 25 July 1986, the structures were fully inspected from station 0+00 at the foot of the inner breakwater to 15+20(+) at the head and below station 0+00 at the foot of the sand barrier to beyond 24+00 (the last monument) at Whaler Island. The mapping consisted of noting rock types and their condition, areas and condition of past repairs where identifiable, general areas presently missing armor, possible deviations from the design, areas of transmissibility through the structures and abnormal spacing or gaps between armor stones. The results are plotted on the new survey sheets at a reduced scale of 1" = 80' with an accuracy of about 5 feet, see plates 2 and 6.

2.3.2 <u>Side Scan Sonar</u>. This investigation was employed to survey the toe of the inner breakwater, its underwater slope and the adjoining ocean floor. The purpose was to locate irregularities in the structure such as slumps, dislodged armor, pockets of small rock and other such features. Originally, side scan was not considered as a formal investigation for this assessment because of the limited area for which it could be used. However, the Waterways Experiment Station was in the harbor demonstrating side scan on the outer breakwater to SPN and the inner breakwater was included. Because no electronic positioning was used, the position of the structure's toe could not be plotted with reasonable accuracy.

Side scan sonar is a marine geophysical system which graphically potrays ocean bottom features similar to an oblique aerial photograph. The system transmits simultaneously, from transducer elements in a towfish, two 100-kHz bursts of sound in fan-shaped beams. Each beam is oriented at right angles to the survey vessel trackline. Reflected signals from the ocean floor are detected by the towfish and electronically processed to produce sonographs. The side scan system used in this survey was an EG&G with a Model 260 plotter. Six passes were made around the inner breakwater from about station 2+00 on the ocean side to about 3+50 on the harbor side. All passes were made with the towfish about mid-depth and the data plotted on a 50-meter scale. No side scan sonar was conducted on the sand barrier because the ocean side toe is above water at a -1 foot MLLW tide.

2.3.3 Diving. Diving was conducted only on the ocean side of the inner breakwater. No diving was conducted on the sand barrier since the complete harbor side is inaccessible and the exposed ocean side is visible above water at low tide. The purpose of the diving was to inspect the underwater slopes, supplementing the side scan sonar results, and note any features which would help determine the structure's physical condition. The existence of such features as slumps, displaced armor, exposed core material, extracted core on the slope or ocean floor and any other deficiencies were also ascertained.

Diving was accomplished over roughly the outer half of the inner breakwater, i.e., from station 4+00 on the ocean side to 14+00 on the harbor side. The oil dock and Coast Guard facilities as well as the kelp and shallow water prevented further inspection of the harbor side. From the head to the Coast Guard dock which is not a Corps of Engineers constructed facility, the harbor floor elevation is less than -6 MLLW and faintly visible from above water at low tide.

2.3.4 Armor Study. The armor study involved several tasks including the quality and size of the armor and location and history of various rock types used. No physical studies were made of the core materials, but rather, they were evaluated based on design and construction data and the present condition of the structures, namely, lack of distress in the core zone as visible on the exterior.

2.3.4.1 Rock Quality. Assessment of the quality of the rock forming the inner breakwater and the sand barrier was made by (1) determining the stages of construction and repairs and locations of the rock sources (quarries) for them (2) determining the present condition of the respective materials at their source and (3) identifying the respective rock types in the structures and evaluating their condition. To perform this task, the histories of the sand barrier and the inner breakwater were re-constructed and the stone sources identified. This was followed by inspections of both the quarries and the structures. The latter's side slopes and crests were photographed at scattered locations to record the condition of the various materials for future evaluation.

2.3.4.2 Armor Stone Gradation. A study to establish the gradation of the armor stone was conducted in order to evaluate the ability of the structures to perform in accordance with design criteria. The analysis covered the area of each structure above the prevailing water surface from the head to the point of contact with the shore. Usually to effectively estimate gradations of large numbers of stone over long reaches, aerial photographs are taken of the armor slopes and stone dimensions are measured from the photographs. During August 1986, when the field investigations were conducted, the weather conditions did not allow for aerial photography and therefore photographs were taken from a boat. As with aerial photographs, dimensions were taken and used to compute the stone gradations using a volume-dimension relationship previously established with field data. A statistical procedure was used, which correlated stone dimensions in the field with data obtained from the photographs. Details of this methodology are discussed in Attachment 1. The methodology has been used for previous condition surveys and gives results which nearly reflect the actual stone weights. The results have proved to be consistent and accurately reflect the relative distribution of stone weights. Armor sizes could not be determined underwater, except by divers along the inner breakwater toe.

3. GEOTECHNICAL SURVEY RESULTS

3.1 Background Data.

3.1.1 Physiography. California is divided into 11 distinct geomorphic provinces. Crescent City is located in the Coast Ranges province at its northern boundary with the Klamath Mountains province. Although both provinces are characterized by rugged topography and bold steep cliffs along the shore, the Klamaths contain flatter upper slopes and crests, approximating a general but well dissected plain. The city occupies a wide terrace some 50 feet above the ocean. The terrace projects a maximum 6 miles from the general mountain front; an unusual width for the province whose terraces are narrow and short. The shoreline is typical of the northern coast in being rugged, very steep and peppered with sea stacks, "rocks" and islands. The south side of the Crescent City terrace dips gently to the shoreline which has a small bay and a sandy beach. The harbor occupies the north and west side of the bay against a rock headland known as Battery Point. The harbor is then contained by three protective dikes known as the outer breakwater, inner breakwater and the sand barrier. The most important structure is the outer breakwater which extends southeasterly from Battery Point to water depths of -30 to -40 feet MLLW. The other two structures are connected to Whaler Island, a "rock" some 100 feet high, 1/2 mile off-shore and east of Battery Point. The sand barrier extends from shore to the island on a submerged tombolo or ridge. Presently, construction on the harbor side has covered that side of the barrier to the crest elevation at +13 feet MLLW. Shoaling on the ocean side has raised the toe from an original elevation of about -7 feet to a low point of about -1 foot MLLW located towards the island. The inner breakwater extends northwesterly from the island perpendicular to the sand barrier across water depths of about -20 feet MLLW. The harbor floor including that under the breakwater is very irregular and composed mostly of bedrock; patches of sand on the bedrock are variably thick, especially against the breakwater, and occasionally need dredging. Under the sand barrier, the ridge, composed of sand, varies in thickness from 1.5 to 10 feet over the bedrock.

3.1.2 Geology. The Klamath Mountains are composed of mostly pre-Paleozoic and Paleozoic age rocks while the Coast Ranges contain younger Jurassic to Cretaceous age rocks known as the Franciscan formation. At the harbor, the terrace and adjacent shore are underlain by Franciscan rocks which are exposed in the cliffs and on the ocean floor. The formation is a heterogeneous mixture of altered sedimentary rocks and intruded volcanics. The sedimentary rocks are graywackes, a type of sandstone, and a few zones of altered shale. These are mixed with bodies of volcanics which were basalts and are now altered to greenstones. Some of the graywackes resemble greenstones and are difficult to be distinguished from them. Most of the bedrock is hard, resistant to erosion and sufficiently durable to be used in construction of coastal structures. Whaler Island is composed mostly of greenstone which was used to construct the initial leg of the inner breakwater. Other hills along the shore locally supplied similar material for other structures in the harbor.

The bedrock is overlain by sediments of late Tertiary age on the terraces; the oldest being the St. George formation of shallow marine origin. Lying unconformably on it is the Battery formation which caps the terrace locally.

This formation is composed of non-marine and weakly indurated conglomerates, sandstones and siltstones. Along the beaches and near shore, recently deposited sand covers the bedrock in pockets of variable thicknesses and in the tombolo under the sand barrier. The general littoral drift is from the north, although construction of the outer breakwater locally altered the pattern. This alteration causes shoaling in the harbor and also against the outside of the sand barrier.

3.1.3 Faults and Seismicity. Regionally, the geologic structure and seismicity is governed by plate tectonics as is the rest of the west coast. The junction between three of the plates forming the earth's crust forms an escarpment extending west off Cape Mendocino less than 100 miles south of Crescent City. The Pacific Plate to the south is moving laterally against the Gorda Plate to the north. The Gorda Plate, in turn, is being subducted or thrust under the North American Plate which forms most of the North American continent. The junction between the latter two plates extends roughly northwest 50 miles offshore from Crescent City along the Continental Margin. The shelf between the margin and shore off northern California is a region of structural transition containing northwest trending faults. These latter faults are caused by lateral movement along the San Andreas fault south of Cape Mendocino into the shelf to the north. Most seismic activity is generated along the Continental Margin and the San Andreas to the south. The San Andreas fault, however, is considerably more active than the continental margin. Around Crescent City, faults occur, but none are known to underlie the harbor or occur within 6 miles of it. The largest fault is the South Fork Mountain fault which forms the boundary between the Klamath Mountains and the Coast Ranges provinces. However, activity on it is significantly less than on the San Andreas and other zones to the south. Since the degree of regional seismicity is considered moderate, Crescent City is on the borderline between seismic zones 2 and 3 on Algermissen's seismic risk map. Earthquakes will occur within the next 50 years, but realistically, not greater than in the Richter magnitude 5 range. This would not cause damage to either the sand barrier or the inner breakwater other than dislodgement of occasional stones. The most likely seismic impact to the structures would be from activity around Cape Mendocino. However, even though the event could be greater than a magnitude 5, attenuation to Crescent City would subdue the energy to non-destructive levels; although minor damage, as mentioned above, is possible.

3.1.4 Design Armor Stone Gradations. The design stone gradations for the structures are based on data obtained from drawings of previous construction and repairs, see plates 4, 7, 9, and 10. The outer zone of the sand barrier consists of "A" rock ranging in size from 4 to 6 tons. Subsequent repairs consisted of stone from 3 to 7 tons in size. The original inner breakwater armor consisted of the following: ocean side, Class "A" stone of 70 cubic feet (5.8 tons) or larger with an average of 100 cubic feet (8.3 tons); and harbor side, Class "B" stone with a range from 25 to 70 cubic feet (2 to 5.8 tons) with 50 percent of the stone larger than 50 cubic feet (4 tons). Subsequent inner breakwater repair consisted of the following: ocean side, Class "A" stone of 110 cubic feet (9 tons) or larger with an average of 155 cubic feet (12.8 tons); and harbor side, Class "B" stone with a range from 25 to 70 cubic feet (2 to 5.8 tons) of 50 percent of the stone larger than 50 cubic feet (4 tons). Armor stone for the inner breakwater extension consisted of a minimum size of 9 tons, an average size of 11 tons and a maximum size of 13 tons.

3.1.5 Construction and Maintenance History.

3.1.5.1 Sand Barrier.

1935- Construction of the sand barrier was authorized in House Document No. 40. After the main breakwater was completed in 1930, the harbor began shoaling with about 180,000 cubic yards of sand a year. The predominent sand movement is north from the adjacent long beach down coast. Initial shoaling steepened the beaches in the harbor out to about -12 feet MLLW then began spreading over the entire harbor hampering ship movement. It was expected that a sand barrier would be effective for only about 11 years.

1938- In April, Congress appropriated $135,000 to construct the barrier. In May, the Corps of Engineers mapped the project area and conducted probings on the submerged tombolo along the barrier alinement between Whaler Island and shore. The exploration indicated the barrier would be underlain by from 1.5 to 25.0 feet of sand over bedrock, see plate 3. Also, at this time, the Harbor District negotiated with Eric Lynders to purchase Whaler Island and made a contract with Hobbs-Wall and Company to furnish stone (royalty-free for the U.S.) for $0.10 a ton from Preston Island quarry on the coast 2 miles north. A road was graded into the quarry to replace the train trestle previously used. In December, the contractor, Hanrahan and Connelly, started construction of the sand barrier which was expected to take 6 to 8 months to complete. The Contractor's bid for the stone was $1.78 a ton placed.

1939- The barrier was completed by mid-year with a total of 67,458 tons of stone at a contract cost of $106,516.75. The final section constructed was a modification of the design section and is as shown on plate 3.

1949- The OCE annual report for FY-1949 and drawing no. 8-9-27, see plate 4, indicate repairs were made to the sand barrier crest. Roughly between stations 11+00 and 15+00, 2,163 tons of stone were placed at a cost of $10,721.15. No details were found on the contractor or the quarry from which the material was obtained. No mention of the repairs was made in either Crescent City newspaper other than on February 8, 1952, the Triplicate notes that the sand barrier again needed repairs, and had been repaired in 1946 (sic) about the time the inner breakwater was constructed. The actual repair date is vague, but most likely was late spring of 1949, at the start of construction of the outer breakwater extension by Macco and Morrison-Knudsen.

1952- Funds in the amount of $35,000 were appropriated for repairs to the sand barrier. Again, no details of the repair are available other than the Triplicate noted on March 28 that the contractor was waiting for the weather to improve in order to start work. It is noted in an SPN office memo dated 14 April 1952, that the proposed advertising date for the repairs was 1 May 1952. Ultimately, 2,382 cubic yards of stone at a contract cost of $50,622.62 were placed in a 900-foot long reach extending shoreward from Whaler Island, see plate 5; these repairs modified the crest by raising it to the top of the rock parapet at elevation +13.

1961- A DF dated 17 August 1961 presents stone requirements for maintenance to the sand barrier as 51 tons (30 cubic yards) for core material and 81 tons (48 cubic yards) for armor. No locations or other documentation is given for the need of these repairs.

1965- A 1962 inspection report states that the barrier had sustained damage since last repaired in 1952. Also, the crest elevation in several locations, had lowered to between +6 and +8 feet MLLW (from a modified elevation of +13 feet MLLW). An interim report for navigation published in 1965 states that the sand barrier has not measurably reduced shoaling in the harbor, although it has offered protection from high waves. No mention is made of damage from the 1964 tsunami.

1970- A letter from the Crescent City Harbor Commission to the San Francisco District states that the easterly section of the barrier at Whaler Island is in need of repair and that sections are 4 feet below grade. Photos taken from Whaler Island show dredge fill along the harbor side from shore out about 500 feet, but no indication of damage. A DF of a Corps of Engineers inspection of 17 June 1970 did not mention any deficiencies in the sand barrier other than stones on the crest near shore were shattered possibly due to heat cracking them when driftwood was burned after the 1964 tsunami.

1972- The Harbor Commission applied for construction of a seawall in the harbor 200 feet from the barrier and parallel with it. The wall is to extend to Whaler Island and the space between filled with 76,000 cubic yards of dredged sand.

1981- Storms caused damage to the access road and parking area recently constructed adjacent to the sand barrier crest, the barrier itself was not damaged. See figure 1 for a typical cross-section of the sand barrier at the present time.

1983- A shoaling study conducted by the San Francisco District reports that average shoaling was approximately 180,000 cy annually prior to the construction of the sand barrier and has averaged between 80,000 and 100,00 cy since that time.

1984- Armor was scalped off the westerly 250 feet or so of the barrier and used to start a groin from Whaler Island a couple hundred feet south of the barrier.

3.1.5.2 Inner Breakwater.

1939- A proposal was made for harbor improvements to include another breakwater extending northwesterly from Whaler Island towards Flat Rock for a distance of about 1100 feet.

1945- Construction of the inner breakwater was authorized on March 2, 1945 in a report on file in the Office, Chief of Engineers, the same date as House Document No. 688, 76th Congress, 3rd session authorized extending the main breakwater to Round Rock. The original design called for a structure 1100 feet long with a crest width of 4 feet at a crest elevation of +20 feet MLLW.

1946- Contract was awarded to Basalt Rock Company of Napa, California, who obtained the stone for the structure from Whaler Island. The first quarry blast was on May 17 for which the city had a big celebration. The breakwater was constructed with 86,280 cubic yards of stone for a length of 1120 feet, a crest width of 15 feet, and in water depths to a maximum -20 feet MLLW, see plate 7. The final contract cost was $252,186.

1965- An SPN interior report states that the inner breakwater has had no maintenance since constructed. This is because the structure is protected by the outer breakwater, however, some surging between the two structures was causing problems to facilities and boats in the harbor. To rectify this, a 300-foot long extension was proposed. The new section was to be semi-pervious to most wave period ranges and essentially the same design as the existing breakwater since it has been trouble free.

1970- An inspection of the inner breakwater indicated it was in good condition without any need for repair.

1972- Contract No. DACW07-72-C-0026 was awarded to the Silverberger Corp. Inc., to build a 400-foot long extension at a bid price of $966,740.00. The specifications called for placing 102,000 tons of stone and removing and replacing 2,900 tons of armor to tie the extension to the existing breakwater, see plates 8 and 9. Specified stone sizes are shown on these plates.

Stone for the first half of the extension was greenstone from the McVay quarry, depleting it. The outer half of the extension is composed of gabbro from the Gardner Ridge (Bankus) quarry also near Brookings, Oregon. In an inspection on 6 April 1973, the extension was complete except for cleanup. It was noted that the rock sizes and slopes conformed to the plans and specifications. The actual contract cost and final quantity of stone for the extension was not given in the 1973 Annual Report of the Chief of Engineers.

1977- An inspection made of the inner breakwater found it was in good condition. On the extension, some of the fill material bordering the concrete diaphragm had eroded leaving the diaphragm some 2 inches higher than the crest.

1980- A post-earthquake inspection of the inner breakwater on 2 December 1980 indicated the strong shock that affected northwest California dislodged 3 stones from the structure's crest.

1983- Storms in early 1983 caused slight damage to the inner breakwater which was repaired by Contractor A. K. Tonkin under contract no. DACW07-83-C-0025. Repairs were made at three locations comprising a final quantity of 1,441 tons of "A" stone and 399 tons of "C" stone; another 640 tons of existing "A" stone were reset, see plate 10. The new material was greenstone from Tonkin's Liscom Hill quarry near Blue Lake. Total contract cost was $75,458.13.

3.1.6 Quarries.

To better evaluate the performance of the stone materials comprising the sand barrier and the inner breakwater, the quarries supplying the stone were also inspected. The intent was to compare the performance of the materials in the quarries with their performance on the structures and in the outer breakwater, if they had been used there also. For the sand barrier, one quarry, Preston Island, was used in the original construction in 1939 and possibly for the 1949 repairs. Presumeably Sugar Loaf quarry was used for repairs in 1952, although no reference to the source has been found. Greenstone from Whaler Island was used to construct the original section of the inner breakwater in 1946. When it was extended in 1972-1973, greenstone was obtained from McVay quarry for the first half and gabbro from Gardner Ridge in Oregon for the last half. Armor for the 1983 inner breakwater repairs is greenstone from Liscom Hill quarry. The following paragraphs describe the observed conditions in each quarry and laboratory data is given, when available. Materials from Preston, Sugar Loaf and Liscom Hill quarries also were used in the outer breakwater. The 1984 evaluation presented in that Condition Survey report are restated in this appendix with updated information where available. Quarry locations, except for Liscom Hill, are presented on plate 1.

PRESTON ISLAND QUARRY. Material from this source was used to construct the sand barrier in 1938-1939 and for construction and maintenance of the outer breakwater between 1923 and 1930 and again in 1947. The source was apparently depleted in 1949 with the repair of the sand barrier. The quarry site is accessible by a paved road off Pebble Beach Drive about 2 miles north of the sand barrier. The quarry was a sea stack some 200 yards across and 100 feet high projecting from the shore. Presently, all that remains is a floor at about elevation +10 MSL.

The rock is greenstone (metabasalt), light green in color with random white quartz seams up to 2 inches wide. In the outer breakwater armor, in lesser amounts, is dark gray medium grained graywacke and a few pieces of a dark maroon-colored shale with random white quartz seams, but only the greenstone was observed in the sand barrier. In the quarry, the shale occurs in irregular bands that are totally sheared, comprising over 1/3 of the floor. Interestingly, there is no waste at the site. It is speculated that the waste, including the sheared shale, was placed in the sand barrier core zone. The exposed greenstone has deteriorated somewhat by dissolution or mechanical erosion, giving the rock a "fuzzy" appearance. Although pieces remain angular, no sharp edges exist and surfaces show considerable etching. Laboratory data indicate that the greenstone is not similar to that in the

Trinidad and Liscom Hill quarries. Of the Preston Island greenstone, two samples tested have a bulk SSD specific gravity of 2.63 and 2.69 with a relatively high absorption of 3.0 and 2.8 percent, respectively. The greenstones from Trinidad and Liscom Hill have a considerably higher specific gravity of 2.85 and 3.15, respectively, with absorption about 0.1 percent. The Preston Island greenstone was less dense and more "granular" which, with a slight difference in the chemical composition, accounts for its deterioration on exposed surfaces.

SUGAR LOAF QUARRY. The quarry site is located on Point St. George about 4 miles north of the sand barrier. The source was yet another shoreline "rock" that has been leveled to about +10 feet elevation. Material was apparently used for sand barrier repairs in 1952. The rock in the old quarry floor is nearly massive graywacke with several conglomerate zones and shale beds. The exposed rock is light gray green to brown, massive, hard and moderately dense and in excellent condition. Surfaces exhibit no deterioration, edges are sharp and no etching was observed. Along the water line, only slight rounding of the edges has developed. In zones with shale clasts, the softer shale has eroded out giving the graywacke a vesicular appearance. Some of the rock exhibits open seams which result from eroding of dissolvable material in the strong planar joints. Considerable splitting of the rock occurs within the quarry floor much of which is a result of the blasting, but these cannot be distinguished from those split by weathering. Laboratory testing results indicate the bulk SSD specific gravity is about 2.72 with a 0.1 percent absorption.

About half a mile south of Sugar Loaf, there is an unidentifiable quarry for which no data or description exists in the San Francisco District quarry files. This may be the McNamara quarry, and is not a "rock" as the other quarries were. The material is again graywacke, similar to the Sugar Loaf material, but is distinctly bedded, the thickest stratum being about three feet. However, because the bedding limited the sizes available for armor, it is doubtful whether any was used for crest armor repair in the sand barrier.

WHALER ISLAND QUARRY. Material from the island was used to construct the initial 1120-foot long segment of the inner breakwater. The breakwater is the only known project using stone from this source. Although the Corps of Engineers has a perpetual easement from the city for material, encroachment of harbor facilities and environmental considerations likely pre-empt further exploitation of the source. Some 86,280 cubic yards of stone were placed in the breakwater. The rock is greenstone, a little more dense than that from Preston Island. It is reported that graywacke also occurs on the island, but none occurs in the quarry except for a few pieces from another source dumped along the toe. The rock is a mottled gray green with color variations to bluish green and almost a tan. Joint and fracture surfaces are strongly coated with rust and oxidation which tends to hide the natural rock color. Surface coloring is what distinguishes the appearance of the Whaler Island greenstones, from other greenstones, although the Liscom Hill stone is somewhat similar. Of the three faces in the quarry, the most material was taken from the center. When inspected in April 1986, none of the rock has deteriorated since exposed 40 years ago, although the surfaces have slightly discolored. No original test data or quarry evaluations could be found. A sample of the material was tested at SPD laboratory and found to have a bulk specific gravity of 2.67 and an absorption of 0.4 percent; no abrasion test was made. The principal constituents are chlorite, quartz and plagioclose feldspar, see Attachment No. 2. Numerous fine sub-parallel veins, containing mostly iron oxide, dissect the rock.

McVAY QUARRY. Construction of the inner breakwater extension in 1972 was begun with stone from McVay quarry, depleting it. The site, another sea stack located on the shore 2.5 miles south of Brookings, is now leveled and the only remnant is a small ridge of waste. Houses now dot the surrounding area, pre-empting further excavation below the ground surface. The floor and waste pile are mostly grown over, but the rock visible is a hard and dense undeteriorated greenstone. Although no specific laboratory data was found, in the 1972 Basis for Design it was reported that the specific gravity (Bulk SSD) was 2.67 to 2.79, absorption was 0.5 to 1.3 percent and the abrasion loss (L.A. Rattler) was 7.9 to 13.1 percent. These data were furnished by the North Pacific Division as the Portland District used McVay stone prior to 1965.

GARDNER RIDGE (BANKUS) QUARRY. The outer half of the inner breakwater extension was completed with material from this quarry which is located a few miles northeast of Brookings along the Chetco River. The quarry still contains material, but appears to not have been in operation for a few years, possibly since 1973. According to SPD laboratory testing, the rock is a mottled dark green meta-hornblende gabbro with a specific gravity (Bulk SSD) of 2.82, absorption of 0.1 percent and a magnesium sulfate loss of 0.9 percent. It is hard, dense and dissected with fractures which have been sealed with calcite and chlorite. The 1972 Basis for Design also states that the abrasion loss (L.A. Rattler) is 9.1 to 10.8 percent, as conducted by North

Pacific Division laboratory. An inspection of the quarry faces and several stock piles in April 1986 indicated that the rock has not deteriorated after 14 years exposure; it is fresh, durable and the edges remain sharp.

LISCOM HILL QUARRY. For the 1983 repairs, stone was placed from Liscom Hill quarry, located near Blue Lake. The rock is greenstone used frequently for repairs on the Humboldt Bay jetties since the 1960's and on the Crescent City outer breakwater in 1979 and 1985-86. The rock is a dark gray-green color with scattered white quartz streaks, hard, somewhat brittle and very dense. It has a specific gravity (bulk SSD) of 2.88 to 3.15, absorption of 0.2 percent, abrasion loss (L.A. Rattler) of 14.9 percent, magnesium sulfate loss of 0.9 percent and no deterioration by the wetting and drying test with either fresh or salt water. Material on the jetties and outer breakwater inspected in 1984 showed no evidence of deterioration. When the quarry was inspected in 1985, it was nearly depleted and material was being extracted for the outer breakwater repair.

3.1.7 Foundation Conditions.

3.1.7.1 Sand Barrier. The sand barrier foundation consists of sand varying in thickness from 25 feet at the shore end to 1.5 feet near the midpoint of the structure. Underlying the sand is bedrock consisting of weathered sandstone and black shale, alternately bedded; plate 3 shows a profile of the foundation.

3.1.7.2 Inner Breakwater. The inner breakwater is also constructed on bedrock consisting of weathered sandstone and black shale, alternately bedded. In part, this rock is covered by thin deposits of sand up to 10 feet thick. Diving investigations, which will be discussed later, give indication that the foundation is exposed bedrock from Whaler Island to about station 7+00. Recently drilled borings (1979) in the inner harbor adjacent to the inner breakwater further reveal the sand deposit is loose and also consists of gravels and shell fragments, varying amounts of organic material and occasional cobbles and boulders. It also reveals the bedrock underlying the sand deposit is a soft bedrock, composed mainly of shale which is often sheared to claylike consistency, containing frequent slickensides, and closely spaced fractures and joints.

3.2 Field Investigation Results.

3.2.1 Above-Water Conditions.

3.2.1.1 Sand Barrier. Since constructed in 1939, harbor improvements and shoaling have covered much of the barrier. The only portion visible is the crest and the ocean slope above elevation -1 MLLW, see figure 1. The toe was fully exposed on both mapping days 30 April and 25 July 1986, when the low tide was -0.5 feet MLLW.

For the most part, the sand barrier is in good condition and has incurred little deterioration since last repaired in 1952. The barrier was designed to be 2,640 feet long, however, its present length is approximately 2,500 feet. The shore end just before station 0+10 appears to be the original end, but the Whaler Island end is obscured. Quarrying and subsequent harbor construction have modified that end and, above station 24+35, the armor is smaller and blends with scattered piles of quarry stone, road base and rock on the adjoining beach, see photo 1. Between stations 21+95 and 24+35, the armor has been scalped and used to construct a short groin from the island about 200 feet south, see photos 1 thru 4. The scalped slope is about 1V:3H covered mostly with core size material from gravel to 6 inches. An estimated one-third of the slope contains scattered remnants of small armor, generally about 2 cubic feet with the largest size being 10 cubic feet. The few larger pieces are the original Preston Island greenstone, but the 2-cubic foot stone is darker and more dense, similar to the Whaler Island stone. Much of the exposed small stone is also darker greenstone and, therefore, likely is not core material, but was added after the scalping. Approximatley half of it is weathered brown and slightly rounded as if it was re-cycled quarry waste. The material does not appear to extend under the in-place armor down station of 21+95. Apparently the scalping occurred in 1984 and because of the area's protected location by the island and the groin, this end has sustained no storm damage in the past 2 years.

The crest averages about 10 feet in width, being slightly wider or narrower from the edge of the adjoining road fill. The crest is full width above station 3+00, see photo 5, narrowing down station to a one-stone width at the shoreward end. The sand barrier has been repaired twice, both times to the crest. According to the contract plans, see plate Nos. 4 and 5, the 1949 repair, although vague, extended from about station 11+00 to 15+00 and the 1952 repair from station 15+00 to 24+00. As mapped in 1986, see plate No. 2, the actual 1949 repair bagan at station 9+75 instead of 11+00. The present up station end abuts the 1952 repair at station 14+50. The actual 1952 repair, then, extends from station 14+50 to beyond 21+95 and may have included 50 feet of the 1949 repair. Another patch of the 1952 repair extends from station 9+25 to 9+75. Included with the 1952 repair, that portion of the crest was raised to a full +13-foot elevation behind the parapet, see plate No. 5. It is not known when the remainder of the crest was raised to a full +13 feet MLLW, although the 1949 repair also appears to have raised that reach of the crest. The crest contains Whaler Island stone down station of 9+25 to shore, which must have been placed some time after 1972 when the island was first quarried. The 1949 repair was made with Preston Island greenstone similar to the stone comprising the original structure. The 1952 repair was made with graywacke which is easily distinguishable from the greenstone, see photos 6 and 7. The graywacke is a gray brown with occasional white seams and strong very planar joints which form flat faces. The greenstone is more of a gray green, but frequently the surfaces are oxidized a darker color above the surf zone, resembling the graywacke. The surfaces, however, are not usually planar and are rough or etched from surface disintegration. Most of the graywacke has not deteriorated, but an estimated 10 percent have split, see photo 8. It is also estimated about half of the graywacke has through-going seams with a potential for splitting. Pieces range in size from 75 to 100 cubic feet with a few pieces as small as 10 and as large as 150 cubic feet. From station 17+65 down to about 11+00, pieces of greenstone from Whaler Island to 6-cubic foot size are scattered along the edge of the road. This material is in excellent condition and shows no signs of deterioration. The stone is likely related to the road construction rather than barrier repairs, since none is found on the barrier itself.

Throughout the ocean slope, the Preston Island greenstone is in very good condition, especially below the mean high tide line where the surface deterioration is light; although the edges may be slightly rounded, pieces remain angular. The armor is significantly rounded in a small zone around station 21+00 where cobbles on the adjoining beach have worn them. Overall, little armor is missing on the barrier and is limited to between stations 3+00 and 8+00, outside of the area scalped of armor above station 21+95. Single stones are missing at stations 3+30, 6+00, 6+45, 7+35 and 7+85, see photo 9, and two stones are missing at station 4+60 where some smaller stone (core?) is exposed, see photo 10. The small stones had been exposed for some time, as they were rounded, and do not indicate a weak area in the structure.

3.2.1.2 Inner Breakwater. The breakwater is in excellent condition above water with only very minor deficiencies. Since constructed in 1946 and extended in 1972, the structure has changed little, see photos 11 thru 14. Two modifications are the Coast Guard facility which covers the lower two-thirds of the harbor side slope for about the first 600 feet and fuel dock facilities attached to the harbor side between stations 7+50 and 10+00; the latter has little altered the breakwater, see photo 12. The trunk (1120 feet long) is constructed of Whaler Island greenstone, see photo 15. The stone is in excellent condition and contains no deterioration or wearing; edges have rounded little along the water line. Between stations 1+00 and 2+50 on the ocean side, a bedrock knoll outcrops on the ocean side; an extension of Whaler Island. It rises to near crest elevation at station 2+80 and extends laterally to 30 feet from the centerline. The only repairs made to the structure were in 1984 at the following three locations:

Present Stationing	Original Stationing
3+30 to 3+65	3+50 to 3+85 = across crest
3+95 to 4+55	4+15 to 4+75 = across crest
10+70 to 11+40	10+90 to 11+60 = ocean slope

The repairs are not readily distinguishable as the stone used from Liscom Hill is similar appearing to the Whaler Island stone. In addition to the above repairs, extra stone has been placed in the sea side corner between the knoll and the foot of the breakwater. In the repair around station 11+00, the Liscom Hill stone has been somewhat mixed with the McVay stone on the extension and the Whaler Island stone further complicating distinguishing between the three greenstones. From stations 11+00 to 13+00, the breakwater is composed of the McVay greenstone which shows no signs of deterioration other than the usual very minor rounding at the water line, see photo 17. Beyond station 13+00 to the head of the breakwater, the structure is composed of gabbro from the Gardner Ridge (Bankus) quarry. This material also is in excellent condition exhibiting no deterioration and remaining hard, in fact and durable, see photo 16.

Throughout the breakwater, the above-water slopes are regular and contain no pockets, holes or missing armor. At about station 6+25, the ocean slope takes a slight jog steepening slightly up station. A concrete diaphragm wall approximately 2 feet wide extends from station 10+95 to 15+15 along the centerline, see photo 17. The wall is in excellent condition and contains only one crack which is thin. The crest stone tends to be smaller because of the wall and numerous cobble sizes occur along it. Stone has washed out along the wall from station 13+85 to 15+00 on the ocean side and 14+15 on the harbor side leaving the wall exposed up to 3 feet high, see photo 18. Road base from the 1984 repairs caps the crest except from station 6+25 to 9+75; above station 11+10 no road was constructed. Numerous small scraps of weathered and unusual rock types exist in the reach between stations 6+25 and 9+75 inferring road base once existed there also, but has been partially washed off. A large part of the harbor slope above approximate station 12+00 contains small angular rock from the crest to the water line where a small bench has formed, see photo 14. The bench is especially distinct around station 14+00; the stone is not core material, but small crest material from adjacent to the diaphragm and old road base.

3.2.2 Below-Water Conditions.

3.2.2.1 Sand Barrier. Because the harbor slope is covered with fill for the harbor facilities and the ocean slope has shoaled to elevation -1 foot MLLW, no underwater investigations were conducted.

3.2.2.2 Inner Breakwater. The side scan survey conducted on 25 July 1986 indicated no deficiencies exist in the below-water slopes. On the ocean side, the rock knoll is easily distinguishable from the "nubby" signature of the breakwater armor on the records by its more smooth and massive face. A small detached bedrock pinnacle about 40 feet in diameter occurs around station 4+75 and roughly 20 feet from the breakwater toe. The toe and slope for the most part, are slightly undulating with patches of detached stones along the toe around stations 3+50 and 10+50. The former location is the site of one of the 1984 repairs and is probably from storm damage. The latter patch is likely from construction in 1972 when the armor was removed from the original head at station 11+20 to join the extension to the trunk. Around the present head and along the harbor side, no unusual features were noted, although kelp on the harbor side down station from 15+00 masks the slope.

The divers confirmed the data obtained by the side scan sonar. They found the ocean floor was mostly exposed bedrock from the beginning of the dive at station 4+00 to about station 7+00. Up station from there, the floor becomes more sandy with some silt. Almost all stone on the ocean floor was armor size except for a patch of one-foot diameter stone in a swale 4 feet across at about station 11+50. This stone had been there for sometime as it was covered with marine growth and kelp. A patch of armor stone at station 10+50 extended out some 50 feet was lying on the sand and as was the other stone, indicating little change has occurred in the floor on the ocean side. In the slope itself, no holes slumps or other irregularities were noted. Around the head into the harbor, shoaling has raised the floor from elevations of -20 to -30 feet to about -6 feet MLLW. Diving was discontinued down station of 14+00 because of the heavy kelp, shallow floor and very bad visibility. It was noted, however, that armor on the harbor side protuded through the sand and that the sand contained more silt than that on the ocean side.

3.2.3 Armor Stone Gradation.

3.2.3.1 Sand Barrier. Five stations were surveyed for stone size determination on the ocean side of the structure. Because of inaccessibility to take photographs of the middle reaches of the structure, visual observations were used to compare the relative sizes of stone with adjacent reaches. It was determined that the stone gradations are representative of the average stone sizes of stations 9+00 and 20+00. Table 1 lists the stone gradations and the average was computed by using a weighted average of results obtained from evaluation of the entire structure. With the exception of station 24+00, ten percent of the stone weighs less than 0.5 tons, the average stone weight is about 1.6 tons, and the maximum stone weight is about 4 tons.

3.2.3.2 Inner Breakwater. Eight stations were surveyed for stone size determination for the inner breakwater. Gradation results for the survey are included on Table 1. For the harbor side of the structure, between stations 0+00 and 10+00, the gradations are only representative of the stone believed to be part of the original construction, in that the majority of this stone has been either removed or subsequently covered with smaller sized rubble and material representing that of typical quarry waste. The gradation survey for the harbor side of the structure, at stations progressing beyond the extension (slopes meeting the water), revealed 10 percent of the stone to weigh less than 1 ton, the average stone weight to be about 3.2 tons, and the maximum stone size to be about 7.5 tons. The ocean side of the structure survey had 10 percent of the stone weigh less than 1.5 tons, the average stone size to be about 5 tons, and the maxmum stone size to be about 17 tons. Station 11+00 of the ocean side revealed the finest material. Without this station included in the average, 10 percent of the stone weighs less than 2 tons and the average stone weight is about 6 tons.

4. DESIGN ASSESSMENT.

4.1 Settlement.

Settlement of the structures can take place due to subsidence, consolidation or migration of the foundation soils, or consolidation of the rubblestone within the structure itself. Limited survey data is available for the inner breakwater and only current survey data is available for the sand barrier. This data along with the information known about the foundation conditions are analyzed in order to assess settlement of the structures.

4.1.1 Subsidence. Subsidence is a form of regional settlement in which areas considerably larger than that of these structures lower in elevation. The most common causes are from fluid extraction (oil or water) which consolidates the underlying strata, or tectonic, resulting from differential movement of the earth's crust. Although there is likely tectonic movement occurring, it's rate is slow enough such that it would not effect the structures during their life span. Fluid extraction is also not a factor, since the bedrock consists of well consolidated materials and no extraction from the bedrock occurs locally. No other subsidence-causing mechanisms are known to be within close proximity to the harbor, indicating subsidence does not influence the structures.

4.1.2 Settlement During Construction. As previously mentioned, the inner breakwater was founded on a sand layer, (0 to 10 feet thick) underlain by bedrock. During construction, the stone being placed likely induced an initial settlement in the unconsolidated sands. This settlement took place entirely during construction as this immediate settlement is characteristic of coarse-grained foundation soils. The diving investigation revealed that from station 4+00 of the inner breakwater to about station 7+00 the ocean floor is mostly exposed bedrock which indicates the foundation in this area is also likely to be bedrock. There would be no foundation settlement where the structure is constructed directly on bedrock.

The sand barrier is constructed on a sand layer (0 to 25 feet thick) underlain by weathered sandstone and shale. During construction, the stone being placed also induced a settlement in this unconsolidated sand layer.

4.1.3 Post-Construction Settlement. Very limited survey data is available on the structures. Prior to the April 1986 survey, the only survey data available pertains to crest elevation at two locations of the inner breakwater. The available survey data for the inner breakwater and sand barrier are summarized in tables 2 and 3. None of the benchmarks have been surveyed more than once so the only settlement analyses which can be made are a comparison between the design elevations and the present survey data.

4.1.3.1 Sand Barrier. The design crest elevation of the sand barrier is +13 feet MLLW. Some of the survey data of April 1986 (monuments SB-1 through SB-24) were taken along the road approximately 15 to 30 feet north of the sand barrier crest centerline. These data showed elevations ranging from +13.5 to +14.9 feet MLLW. Additional survey measurements were made along the sand barrier crest centerline but are not identified with monuments. These data are presented in table 3 and show a range of elevations along the crest from +12.5 feet at station 18+85 to +14.9 feet MLLW at station 1+00. Since the foundation conditions are similar to that of the inner breakwater and this structure does not impose as large a load on the foundation as the inner breakwater, the sand barrier probably has not settled. It is likely the variations (-0.5 to +1.9 feet) in crest elevation are due to construction and repair.

4.1.3.2 Inner Breakwater. The design crest elevation of the inner breakwater is +18 feet MLLW. The recent survey data indicate the crest ranges in elevation from 16.8 feet at the head to 20.4 feet MLLW at station 3+00. The diaphragm wall, constructed through the armor to the top of the inner breakwater extension's core zone, has only one crack. Had there been differential settlement in the foundation or, consolidation of the rubblestone within the structure, or migration of foundation materials, more severe cracking would have been evident. It is probable that the structure was not constructed to an exact design crest elevation of +18 feet MLLW and there has been no settlement.

4.2 Stability.

The stability of the structures against wave velocities and pressures is a concern of both the coastal engineer and the geotechnical engineer. This geotechnical appendix presents a qualitative assessment of stability based on a nonrigorous method to illustrate the relative destabilizing effects of wave velocity and pressure. The wave velocities and pressures, as determined from the limited wave data made available, are utilized in this appendix as only one factor in assessing the modes of failure and performance of the structures. Quantitative conclusions are not to be drawn from this cursory analysis, rather the intent of this stability assessment is to identify possible modes of wave attack and their effects on the condition of the structure, and to provide recommendations for addressing these conditions in future repairs or designs.

4.2.1 Wave Characteristics. This stability assessment is based upon wave data by WES (Wave Height Estimate, Sand Barrier and Inner Breakwater, Crescent City, California, 1987). Waves arrive from the south at about 220 degrees azimuth, diffract around the outer breakwater and propagate through the entrance channel, striking the inner breakwater. Non-breaking wave conditions prevail where the significant wave height (at a maximum storm tide of +10 feet MLLW) is 16 feet, with a period of 12.2 seconds. Waves striking the sand barrier arrive from the open ocean at an azimuth of 180 degrees. Shoaling significantly alters these waves. Due to the shallow water surf characteristics at this location, waves which strike the sand barrier often have breaking-wave characteristics. The most damaging condition will occur at a maximum storm tide of +10 feet MLLW and a period of 12.2 seconds. This will result in a breaking wave height of 11 feet at the sand barrier.

4.2.2 <u>Stability Against Sliding From Wave Impact</u>. Based on wave pressure methodology developed by Gaillard and Molitor (1935), the inner breakwater and extension were assessed for lateral stability against wave pressures. This method computes the total (static and dynamic) pressure imposed by an unobstructed wave striking a vertical wall. Since the breakwater and sand barrier have inclined side slopes rather than vertical walls, the pressure would be decreased for the case of the inclined side slopes. Additionally, the wave pressures are calculated assuming the waves strike normal to the structures. Any deviation from the normal angle of wave attack would decrease the wave pressures. Therefore, the pressures obtained in this method are conservative and can be safely used. For the structures to be considered laterally stable, the frictional forces which develop between the structure and foundation should exceed the wave forces by a substantial amount. This assessment is not made for the sand barrier because its harbor side has been backfilled and would not be susceptible to lateral movement by the comparatively small wave pressures acting against it. To determine the factor of safety against sliding for the inner breakwater, the following assumptions were made:

(1) The height of the structures are equal to the differences in elevation between the foundation and the crest of the structures as determined from typical cross-sections.

(2) The specific gravity of the rock is 2.67 for both the original inner breakwater (constructed from Whaler Island quarry greenstone) and inner breakwater extension (constructed from McVay quarry greenstone and Gardner Ridge quarry gabbro).

(3) The weight of the rock below the water is decreased by 64 pcf due to its submersion.

(4) The significant wave height is that characterized in paragraph 4.2.1.

(5) The coefficient of the friction between the stone and foundation is 0.4.

Using the above assumptions, the factors of safety against sliding are 2.3 for the inner breakwater trunk and 3.1 for the extension. Figure 2 graphically depicts the wave pressure distribution at the structure. Given the conservative assumptions of this assessment and that the factors of safety are greater than 1.0, sliding is not a stability concern.

4.2.3 Stability of Armor Stone Against Wave-Induced Shear Forces.

4.2.3.1 Overtopping. When a wave overtops a rubblemound structure, the impact point on the back slope of the structure is critical as the remaining energy in the water will be dissipated at that point, possibly displacing armor or capstone from the harbor side face. The forces acting at this impact point have contributions from horizontal and vertical components for velocity. The horizontal component will not exceed the wave propagation velocity while the vertical velocity is due to gravity. The impact forces from these overtopping waves are likely to be large enough to displace the existing armor stones from the harbor side slopes at the inner breakwater. Thus far, this structure has not been damaged from overtopping waves.

At the sand barrier, overtopping waves have caused damage prior to 1949 and in 1952, see plates 4 and 5 for typical cross-sections of necessary repairs. Orientation of the sand barrier is such that wave attack normal to it is impossible. However, southerly waves build up as they travel toward shore along the seaward face of the structure causing significant overtopping and have previously resulted in displacement of the armor stone. The harbor side slope is no longer subject to the overtopping waves because of the recently-constructed adjacent road and parking area.

4.2.3.2 Drawdown. The porosity of a rubblemound structure generally is a factor in the stability of the ocean side slope during drawdown because the seepage forces into the slope tend to increase the apparent weight of the armor stone. For small waves with low propagation velocities, the small amount of water remaining on the slope face after the passage of the wave will quickly disappear through the voids. However, large waves with high propagation velocities fill the voids and cover the slope face and crest with water. In addition, airborne water falls back on the structure. Because of the high propagation velocity, the wave trough has already arrived at the structure, allowing the water on the slope and the returning airborne water to run down the slope. The velocity of the water running down the slope is a function of the static head of water remaining on the slope after the passage of the wave. The velocity as determined by the static head is the same as the vertical component determined for overtopping. Therefore, drawdown velocities will be less than the resultant velocities occuring for the overtopping case at the inner breakwater and inner breakwater extension. The effect of drawdown velocities should not effect the stability of the inner breakwater.

At the sand barrier, drawdown velocities are not likely to create forces large enough to displace the existing armor stones and should not be a threat to its stability.

4.2.3.3 Runup Velocities. As a wave makes contact with the structure, water runs up the ocean side slope trying to loosen and displace the armor stone. The runup velocity is a component of the wave's horizontal propagation velocity as determined in the assessment for stability against sliding. The stones near the top of the slope would experience the maximum runup velocity equal to the wave's horizontal propagation velocity. These are the probable wave forces which displaced armor stone near the crest of the inner breakwater when it was damaged in 1983. Runup velocities are likely to cause the largest wave forces acting on the ocean side slope for the inner breakwater. These forces would likely be large enough to displace the smaller-sized existing armor stone used in constructing the inner breakwater and a more significant amount of existing armor stone at the inner breakwater extension.

At the sand barrier, runup velocities from the breaking waves will be less than the velocity of wave as it initially impacts the structure. Although less critical than the forces induced by the breaking wave at impact, these forces are likely to be large enough to displace the existing armor stone.

4.2.3.4 Impact of Breaking Waves. This particular assessment pertains to the breaking wave conditions present at the sand barrier. As a breaking wave strikes a rubblemound structure, the energy in the wave is dissipated and has an unraveling effect on the armor stone, tending to displace it. This mode of failure was likely exhibited when the sand barrier was damaged in the late 1940's (see plate 4) and again prior to the 1952 repairs. Displacement of existing armor stone due to the impact of an 11-foot breaking wave can be expected and represents the most critical mode of failure at the sand barrier.

4.2.3.5 Scour. The side scan sonar survey indicated that there was no evidence of scour at the inner breakwater and that there is shoaling around the head into the harbor. This was confirmed by the diving investigations. The sand barrier has been effected by shoaling, which protects its toe from potential scour.

4.2.4 Armor Stone Static Slope Stability. This assessment checks the stability of the slope under static conditions. The stability of the submerged portion of the armor stone slopes was analyzed using the infinite slope method assuming that the internal angle of friction ϕ (phi) for the stone is 45 degrees. The survey conducted in April 1986 included 2-foot interval contours along the exposed sand barrier ocean side slope and along the inner breakwater harbor and ocean side slopes. Evaluation of these data indicate the side slopes are generally flatter than specified in the design. The design slopes of these structures were used in this analysis and are shown on plates 5, 7, and 9. The factors of safety are 1.3 to 2.0 at the inner breakwater and 1.3 at the sand barrier. In general, the breakwater and sand barrier slopes are stable.

4.2.5 Bearing Capacity. In a letter dated 1950, it was reported by SPD that the bearing capacities of the foundation materials at both structures are not known. The inner breakwater is built on a variable layer of sand (0 to 10 feet thick) over rock, while the sand barrier is also built on a variable layer of sand (1.5 to 25 feet thick) over rock. The soil pressures applied by the structures to the foundation are calculated by assuming the water surface is at +0 feet MLLW, the specific gravity of the structures are 2.67, and the void ratio of the structures are 0.4. Conservatively estimated soil pressures (3.1 and 1.6 ksf for the inner breakwater and sand barrier, respectively) are not likely to exceed the bearing capacity of sand and definitely will not exceed the bearing capacity of the bedrock.

4.2.6 Migration. When a layer of coarse material (soil or rock) overlays a layer of finer material, it is possible the fine material may migrate through the voids of the coarse material. Compatibility between these materials such that the finer material does not migrate through the voids of the coarse material can be checked by criteria presented in the Shore Protection Manual. At the inner breakwater, foundation soils can migrate through the corestone due to the absence of a filter layer. Migration could have occurred during construction until the stone was embedded in the sand. However, it has not likely happened since because the tidal and wave action hasn't appeared to have disturbed the foundation soils near the toe. The inner breakwater extension was constructed on a filter layer which prevents the migration of foundation soils through the structure. The various layers of rubblestone used in the inner breakwater satisfy the criteria such that migration of rubblestone within the structure won't occur. Migration of sands, transmitted through the structure via wave action, is not prevented.

Foundation soils can potentially migrate through the sand barrier due to the absence of a filter layer, however, shoaling at the toe of the structure protects the foundation materials from wave action and migration is not likely to occur. Migration could have occurred during construction until the stone was embedded in the sand and after construction, until shoaling had built up to protect the foundation materials. Currently there is approximately 4 to 8 feet of shoaling built up at the toe of the structure between stations 8+00 and 24+00. Rubblestone within the structure satisfy the criteria such that migration of rubblestone within the structure won't occur. The main purpose of the sand barrier was to reduce shoaling in the inner harbor area which was occurring after construction of the outer breakwater. Sand dredged from the inner harbor area has been deposited as fill adjacent to the sand barrier crest in order to construct the parking lot. This fill should substantially reduce the shoaling in the inner harbor due to transmission through the sand barrier.

5. SUMMARY AND CONCLUSIONS

5.1. Sand Barrier.

Evaluation of available data and observations of the sand barrier's present condition indicate the structure is in satisfactory condition, although it contains several deficiencies, most of which are minor. The greatest deficiency is the reach between stations 21+95 and 24+35 where the armor was scalped in 1984 to construct a small groin nearby. The exposed slope is composed of mostly sizes less than 6-inch diameter with a few scattered remnants of small armor. Since this reach abuts Whaler Island, it is somewhat protected by the island and also by the new groin, however, a coastal engineering analysis needs to be made of the vulnerability of this reach. Although the barrier was designed to be 2,640 feet long, subsequent quarrying of Whaler Island and construction of harbor facilities have obscured the seaward end. The structure now appears to be not over 2,500 feet long; current stationing is within a foot or two of the original. Since the barrier's construction in 1939, much of the structure has been buried with harbor facilities and shoaling; only the ocean slope above elevation -1 MLLW and the crest are exposed. Presumably during the repairs in 1949 and 1952, the crest has been modified by filling in behind the original rock parapet raising the crest from an elevation of +10 feet to a full +13 feet MLLW.

The stone comprising the armor is greenstone from Preston Island with a veneer of graywacke from Sugar Loaf quarry on the crest, mostly between stations 14+50 and 21+95. Both materials are serving well and can be expected to remain in satisfactory condition for another 50 years. The greenstone has slightly deteriorated on the surface giving it an "etched" appearance and the edges have rounded slightly, however, it basically remains durable, angular and of adequate size. The graywacke comprises a very minor percentage of the structure. It also remains durable although an estimated 10 percent has split along strong seams. It has also been estimated that roughly half of the graywacke contains these strong seams, but it is deduced that if they have not split after being in place 34 years, they likely will not. The splitting is attributed to weak material in the seams, as splitting was also observed in the quarry, and not to burning of driftwood as was once surmised. It is assumed that the core material is the same Preston Island greenstone as the armor. None has been observed although the small material in the scalped area may be core material. The quarry floor contains numerous zones of a weak altered shale and there is no waste at the quarry. The possibility exists that this material may be in the core. Although undesirable, if it does exist, it is not affecting the performance of the core and, therefore, is not a concern. Besides the scalped zone, several individual armor stones are missing on the ocean slope between stations 3+00 and 8+00. These are of no importance and not materially affecting the integrity of the structure.

Since the sand barrier was last repaired in 1952, subsequent inspections eluded to settlement of as much as 6 feet. Evaluation of the topography developed for this survey indicates no such condition exists and the barrier crest is fully above elevation +12 feet and mostly about +13 feet MLLW. The lowest crest elevation surveyed is +12.5 feet MLLW, 0.5 feet lower than the modified crest height. This settlement is most likely due to a deviation from design height during construction and repairs. Future settlement is not expected.

The waves approaching the sand barrier are influenced by a shallow water surf zone and consequently they are often breaking waves. Previous damage was incurred to the sand barrier (late 1940's and 1952) from waves overtopping the structure and the breaking waves unraveling the ocean side armor stone. Overtopping waves are no longer a threat to the harbor side slope since it was backfilled and constructed upon. However, the sand barrier is still subjected to the unraveling effects of the breaking waves. The significant maximum breaking wave expected at the sand barrier represents the condition which is most threatening to the structure and it appears the existing armor stone is not large enough to resist displacement. This condition doesn't warrant design changes to the existing structure due to its satisfactory performance.

The foundation consists of alternately bedded weathered sandstone and black shale, overlain by a sand layer varying in thickness from 1.5 to 25 feet. These materials are satisfactory and should not consolidate or exhibit bearing failure. Migration of sandy foundation material is possible at the structure due to the absence of a filter layer. However, shoaling at the toe of the structure protects the foundation from wave action and this is not likely to occur.

5.2. Inner Breakwater.

The results of this assessment indicate the inner breakwater is in excellent condition, capable of performing satisfactorily for at least another 50 years. The one deficiency is minor erosion along the diaphragm wall from station 13+85 on the ocean side and 14+15 on the harbor side to station 15+00 near its end. The wall is exposed up to 3 feet high where the adjacent small rock used to form the wall has washed out. The eroded material, together with old road material on the crest, have collected on the harbor slope and formed a slight bench around the water line, particularly up station of about 11+00. Down station of 11+00, the fuel dock and especially the U.S. Coast Guard facilities have altered a good portion of the harbor slope. Out to the dog-leg at station 11+00, the breakwater is composed of Whaler Island greenstone, a very dense and durable material that exhibits no deterioration other than very minor rounding of edges in the surf zone. The last 400 feet of the breakwater is composed first of 200 feet of McVay quarry greenstone and then of Gardner Ridge gabbro. Both materials are also in excellent condition with no evidence of deterioration after being in place since 1972. The Liscom Hill greenstone used for repairs in 1984 has an excellent service record on other coastal projects and is expected to continue satisfactory service for at least another 50 years. The structure itself contains no irregularities other than the minor erosion along the diaphragm. The slopes and crest contain no missing armor, slumps, dislodged armor or extracted core material. On the ocean side, the slope steepens slightly at station 6+25 above water, but this cannot be detected below water. Several dislodged stones were observed on the ocean floor near the sections repaired in 1984 and a larger zone where the armor was removed in 1972 to add the 400-foot extension. The floor itself is exposed bedrock similar to the knob that forms part of the breakwater around station 2+00. Up station from about station 7+00 on the ocean side, the floor becomes covered with silty sand. Around the head into the harbor, the sand thickness increases markedly, raising the floor elevation from -20 feet and -30 to -6 feet MLLW with the sand becoming finer grained and more silty; kelp also becomes prevalent and obscures the breakwater slope. It was observed that stones on the ocean side were resting on the sand while those along the harbor toe were protruding through it.

Because of the bedrock foundation, the inner breakwater has not experienced settlement in the foundation. The structure appears stable as the concrete diaphragm had only one crack. The design crest elevation for this structure is +18 feet MLLW and recent survey data (April 1986) revealed the lowest crest elevation is +16.8 feet MLLW. The apparent settlement of up to 1.17 feet is likely caused by a deviation from design height during construction. Future settlement of the structure is not expected.

The foundation consists of alternately bedded weathered sandstone and black shale, overlain by a sand layer up to 10 feet thick. The foundation from Whaler Island to about station 7+00 is most likely exposed bedrock. The foundation materials are satisfactory and should not consolidate or exhibit bearing failure. Migration of sandy foundation material is possible at the original inner breakwater due to the absence of a filter layer. However, diving investigations revealed no evidence of scour or disruption of ocean floor materials around the structure so migration of these soils is unlikely.

Wave characteristics are such that the significant maximum wave at the inner breakwater would be a non-breaking wave. The most threatening failure mode to the harbor side slope is waves overtopping the structure. These wave forces are likely to be large enough to displace the existing armor stones. Forces due to wave runup are the most threatening forces to the ocean side slope. These would probably be large enough to displace a significant amount of the existing armor stone. Runup wave forces are the likely forces which damaged the structure in 1983. However, it is unlikely the structure has been subjected to the full magnitude of forces which the maximum significant wave would impose. Past performance of the structure indicates the overall stability of the structure is adequate under the wave conditions experienced to date. Even though these forces could displace armor stone, the structure has performed adequately and does not warrant changes to its existing design.

5.3 Armor Gradation Assessments.

The following table summarizes the design gradations and results of the survey gradations for armor stone on the surfaces of the structures. Surveyed results represent the overall gradations of the structures and do not take into account the different gradations used for repairs or those used above and below elevation 0.0 feet MLLW.

Structure and Location	Design Stone Sizes (tons) Minimum	Range	Average	Survey Stone Sizes (tons) Minimum	Range	Average
Inner Breakwater						
Original harbor side		2-5.8	4.0*		0.5- 6	1.5
Original ocean side	5.8		8.2	2.5		6.0
Repair harbor side		2-5.8	4.0*			
Repair Ocean side	9.0		12.8			
Extension, above E1.0'		9-13	11.0			
Extension, below E1.0'		2- 9	4.5			
Extension, harbor side					1.2- 7	3.2
Extension, ocean side					1.5-14	4.5
Sand Barrier						
Original constr.		4- 6				
Repair constr.		3- 7			0.5- 4	1.6

* median stone weight

In general, the results indicate the stone for the structures to be lighter than the various gradations required in the design plans. The sand barrier stone was found to be significantly lighter in overall gradation. For the breakwater, the absence of large stone used for repairs is apparent and the overall gradations appear slightly less in weight than the minimum gradations required for the original construction.

6. RECOMMENDATIONS

6.1. General.

As a result of this condition survey, the following recommendations are provided for rehabilitation and future monitoring and maintenance of the Crescent City Harbor sand barrier and inner breakwater. These recommendations include discussions of those repairs which should be made or can be deferred, what monitoring should be established to fully define the condition of the structures, and what type of maintenance routine should be established.

6.2. Rehabilitation.

The following two paragraphs list seven areas of the sand barrier and two areas of the inner breakwater requiring repairs using appropriate armor stone. Also listed for the sand barrier is a reach where a field evaluation is required.

Although, as discussed in the report, some local stone displacement has occurred, armor stone still remains in sufficient quantity to retain interior stone. Therefore immediate repairs to the sand barrier and inner breakwater are not required and can be deferred until future damages or analyses finally warrant them. It is also recommended that replacement stone for the sand barrier be larger than the armor stone specified in the original design. A minimum average stone size of 8 tons, having a range of 6 to 10 tons, should be specified for the armor stone repair. Replacement stone for the inner breakwater should meet the gradation and quality requirements of the stone specified for the inner breakwater extension. Assessing the scalped area on the sand barrier is beyond the scope of the geotechnical appendix.

6.2.1 Sand Barrier.

1. Station 3+30
   - Replace one missing armor stone
2. Station 4+60
   - Replace two missing armor stones
3. Station 6+00
   - Replace one missing armor stone

4. Station 6+00

   - Replace one missing armor stone

5. Station 6+45

   - Replace one missing armor stone

6. Station 7+35

   - Replace one missing armor stone

7. Station 7+85

   - Replace one missing armor stone

8. Station 21+85 to 24+35

   - Evaluate section scalped of armor

6.2.2 Inner Breakwater.

1. Station 13+85 to 15+00/ocean side

   - Repair the eroded section against diaphragm with concrete

2. Station 14+15 to 15+00/harbor side

   - Repair the eroded section against diaphragm with concrete

6.3 Monitoring.

A monitoring program should be established for the Crescent City Harbor sand barrier and inner breakwater. On some regular basis, recorded site visits should be conducted (suggest: 5-year intervals and following damaging storms) which would be supplemented with aerial photography and ground survey (suggest: 10-year intervals). Any deficiencies noted in the regularly scheduled monitoring should be investigated, as necessary, with supplemental diving or bathymetric surveys. The use of side scan sonar to observe conditions below water is limited to a small portion of the inner breakwater and would not be practical, unless it is done in conjuction with similar surveys on the outer breakwater. It is also recommended that the amount of split graywacke on the sand barrier crest be re-estimated for growth every 10 years. No instrumentation has been found to be necessary. It is expected that as a result of monitoring, additional recommendations may be necessary in regard to maintenance or latent deficiencies.

6.4. Long-Term Maintenance.

No scheduled geotechnical long-term maintenance is recommended for either structure. Repairs should continue to be made on an "as needed" basis.

The only maintenance anticipated for the sand barrier and inner breakwater will be the occasional replacement of armor stone. Replacement criteria should be established that require the immediate addition of armor stone whenever either structure has exposed interior stone. No design changes are necessary and replacement armor stone should meet the gradations recommended in section 6.2.

7. SELECTED BIBLIOGRAPHY

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8. U.S. Army Corps of Engineers, San Francisco District, May 1946, Definite Project Report--Crescent City Harbor, California.

9. U.S. Army Corps of Engineers, San Francisco District, February 1965 Interim Report-- Crescent City Harbor, California, for Navigation.

10. U.S. Army Corps of Engineers, San Francisco District, 6 April 1973, DF--Inner Breakwater Inspection.

11. U.S. Army Corps of Engineers, San Francisco District, 31 May 1977, DF--Inner Breakwater Inspection.

12. U.S. Army Corps of Engineers, San Francisco District, 6 Feb. 1981, DF--Condition of Sand Barrier.

TABLES

TABLE 1
CRESCENT CITY SAND BARRIER AND INNER BREAKWATER
MEASURED ARMOR STONE GRADATIONS, AUGUST 1986

SAND BARRIER

Weight (Tons)	Percent Smaller by Weight at Each Station						Range		Average
	1+00	4+99	9+00	20+00	24+00		Min	Max	
5.0	100	100	100	100	100	Note: Station	100	100	100
2.5	78	44	91	72	100	24+00 is not	44	91	71
1.2	67	10	24	50	100	included in the	10	67	38
0.8	41	0	15	15	100	range and	0	41	18
0.4	22	0	1	3	72	average. The	0	22	7
0.2	1	0	0	0	33	station is not	0	1	0
0.1	0	0	0	0	13	representative.	0	0	0

INNER BREAKWATER - OCEAN SIDE

Weight (Tons)	Percent Smaller by Weight at each Station								Range		Average
	1+00	3+00	5+00	7+00	9+00	11+00	13+00	15+00	Min	Max	
20.0	100	100	100	100	100	100	100	100	100	100	100
15.0	86	100	100	88	87	100	100	100	86	100	95
10.0	63	100	100	80	71	100	88	100	63	100	88
7.5	63	62	90	64	59	100	61	87	59	100	73
5.0	41	49	73	23	33	100	61	57	23	100	55
2.5	6	19	19	5	8	74	20	24	5	74	22
1.2	1	5	5	2	0	17	2	6	1	17	5
0.8	0	1	2	1	0	9	1	1	0	9	2
0.4	0	0	0	0	0	1	0	0	0	1	0
0.2	0	0	0	0	0	0	0	0	0	0	0
0.1	0	0	0	0	0	0	0	0	0	0	0

INNER BREAKWATER - HARBOR SIDE

Weight (Tons)	Percent Smaller by Weight at each Station								Range		Average
	1+00	3+00	5+00	7+00	9+00	11+00	13+00	15+00	Min	Max	
7.5	100	100	100	100	100	100	100	100	100	100	100
5.0	100	100	100	100	89	100	62	100	62	100	94
2.5	100	100	90	59	37	70	23	47	23	100	66
1.2	69	20	43	16	11	26	6	7	6	69	25
0.8	69	17	22	6	7	13	1	3	1	69	17
0.4	27	5	3	2	1	1	0	1	0	27	5

TABLE 2

INNER BREAKWATER SURVEY ELEVATIONS (feet, MLLW)

DATE OF SURVEY	MONUMENT and STATION										
	IB-1 1+00.00	ROK[1] 2+38.13	IB-3 3+00.00	4+99	IB-5 4+99.83	IB-7 7+04.45	IB-9 9+05.59	10+06	IB-11 11+00.00	IB-13 12+84.35	IB-15 14+82.42
April 1952		20.94									
May 1964		20.94									
August 1965								16.97			
Not Given[2] (Repair 1983)		20.94		17.56				16.97	(R)[3]		
April 1986	20.19	20.94	20.35		19.77	17.43	17.46		17.88	17.93	16.83

1 - 'ROK' is project benchmark.

2 - This data is from April 1968 Drawing; Horizontal and Vertical Control.

3 - (R): This area was repaired on date indicated.

Note: April 1986 Survey is approximately 20' offset from original. For example, the present station 11+00 was originally 11+20.

SAND BARRIER SURVEY ELEVATIONS (feet MLLW)

(These monuments are located 15-30 feet north of the crest centerline)

DATE OF SURVEY	MONUMENT and STATION												
	SB-1 1+00.00	SB-3 3+00.00	SB-5 4+99.99	SB-7 7+00.01	SB-9 9+00.01	SB-11 11+00.00	SB-13 12+99.96	SB-15 14+99.96	SB-17 16+99.96	SB-18 18+07.90	SB-20 19+99.87	SB-22 21+99.84	SB-24 23+99.80
April 1986	14.91	14.77	14.45	14.20	14.18	14.78	14.85	14.66	14.28	13.94	13.83	13.52	14.26

TABLE 3

SAND BARRIER SURVEY ELEVATIONS, APRIL 1986 (feet, MLLW)

(Located approximately along centerline of sand barrier)

Approximate Station	Elevation (feet, MLLW)
1+00	14.91
1+70	14.4
2+25	14.1
2+75	13.8
3+60	14.2
4+55	14.1
5+45	14.1
6+70	13.8
7+60	13.6
16+50	12.6
18+85	12.5
19+80	12.7
20+55	12.9
21+15	12.8
22+60	13.2

FIGURES

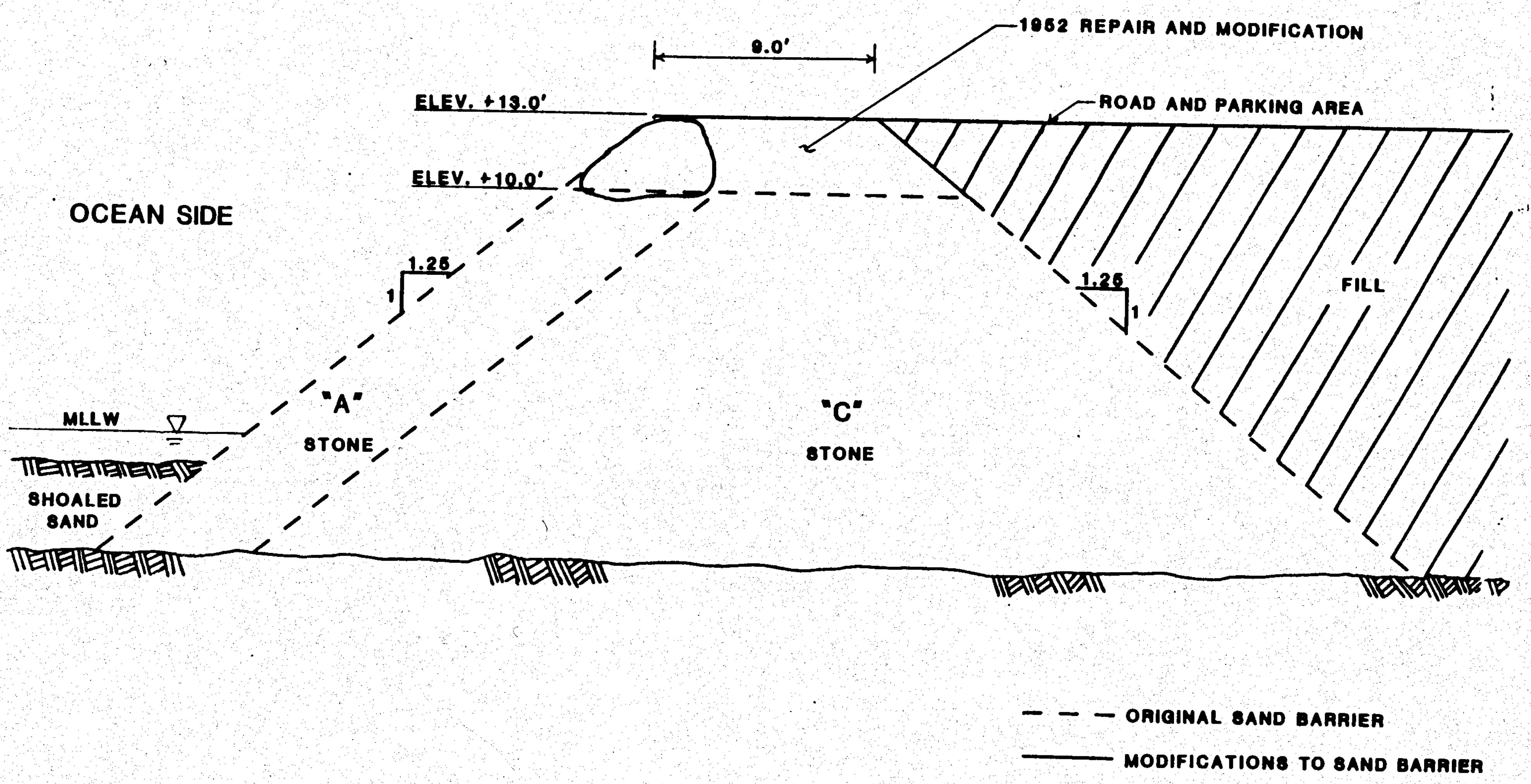
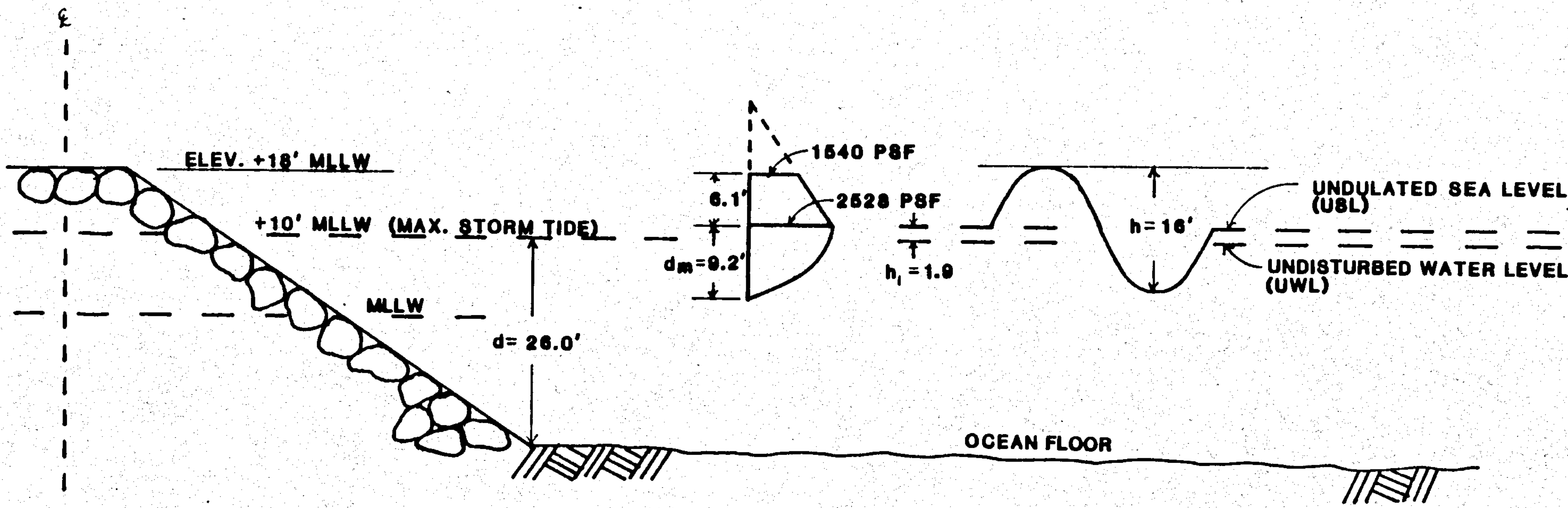


Figure 1. Typical Cross Section of Sand Barrier.
(At Present)



h = wave height

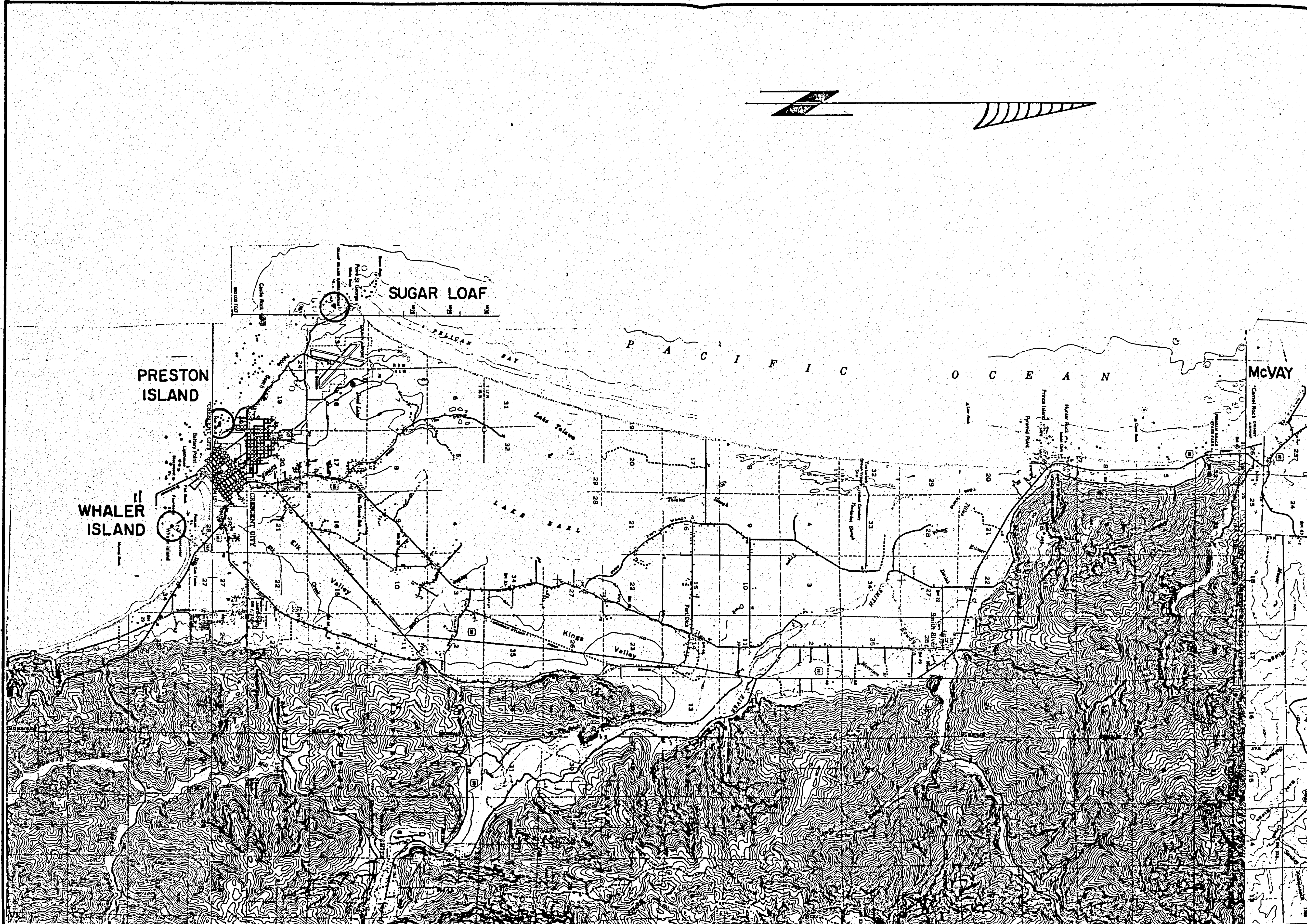
h_1 = height of USL-UWL

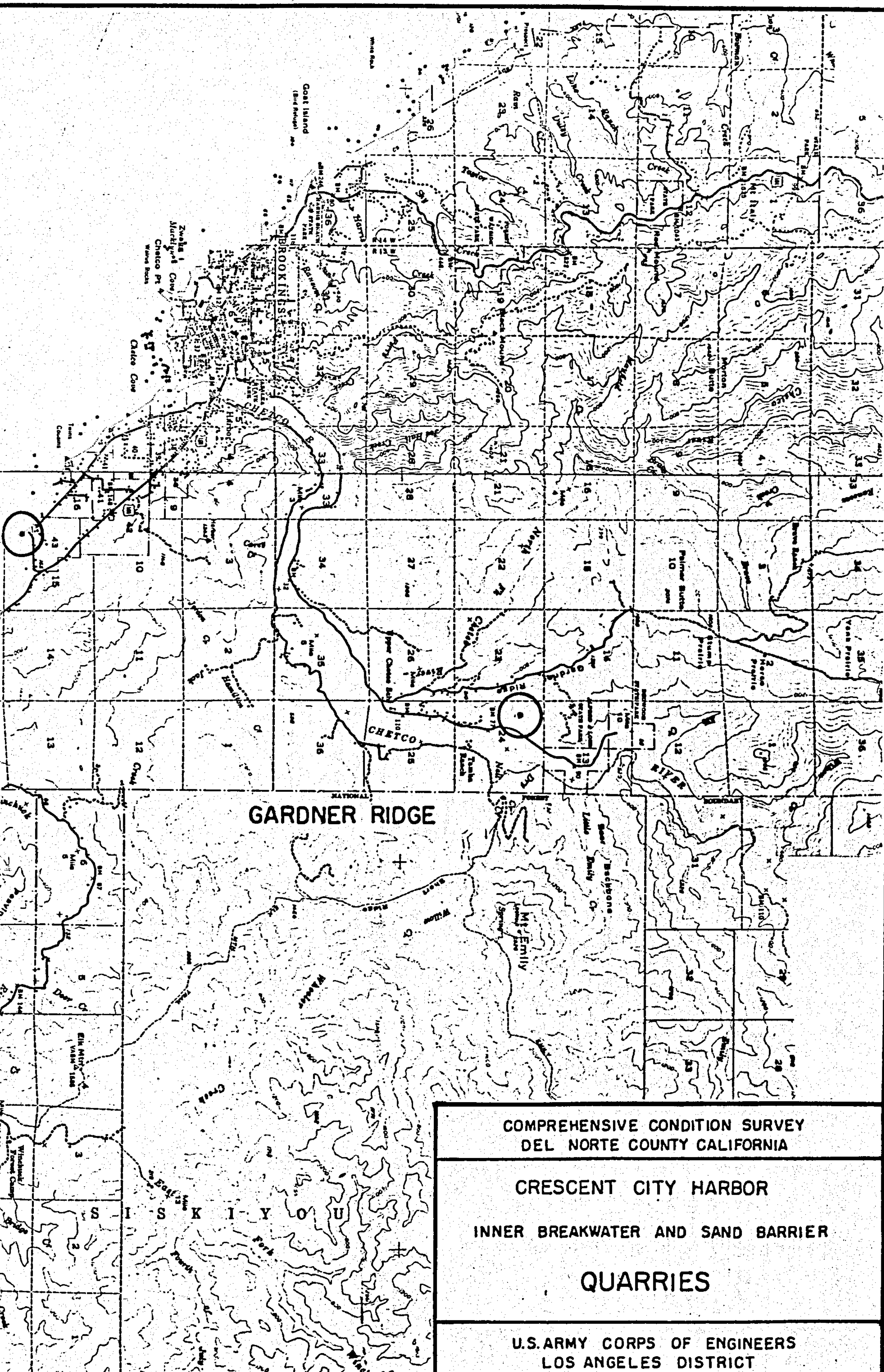
d = depth of water at toe

d_m = maximum depth of pressure

Figure 2. Wave Pressure Distribution at Inner Breakwater.

PLATES


SUGAR LOAF
PRESTON ISLAND
WHALER ISLAND
McVAY
PACIFIC OCEAN
PELICAN BAY
LAKE EARL
Lake Talawa
CRESCENT CITY
Battery Point
Elk Valley
Kings Valley
Smith River



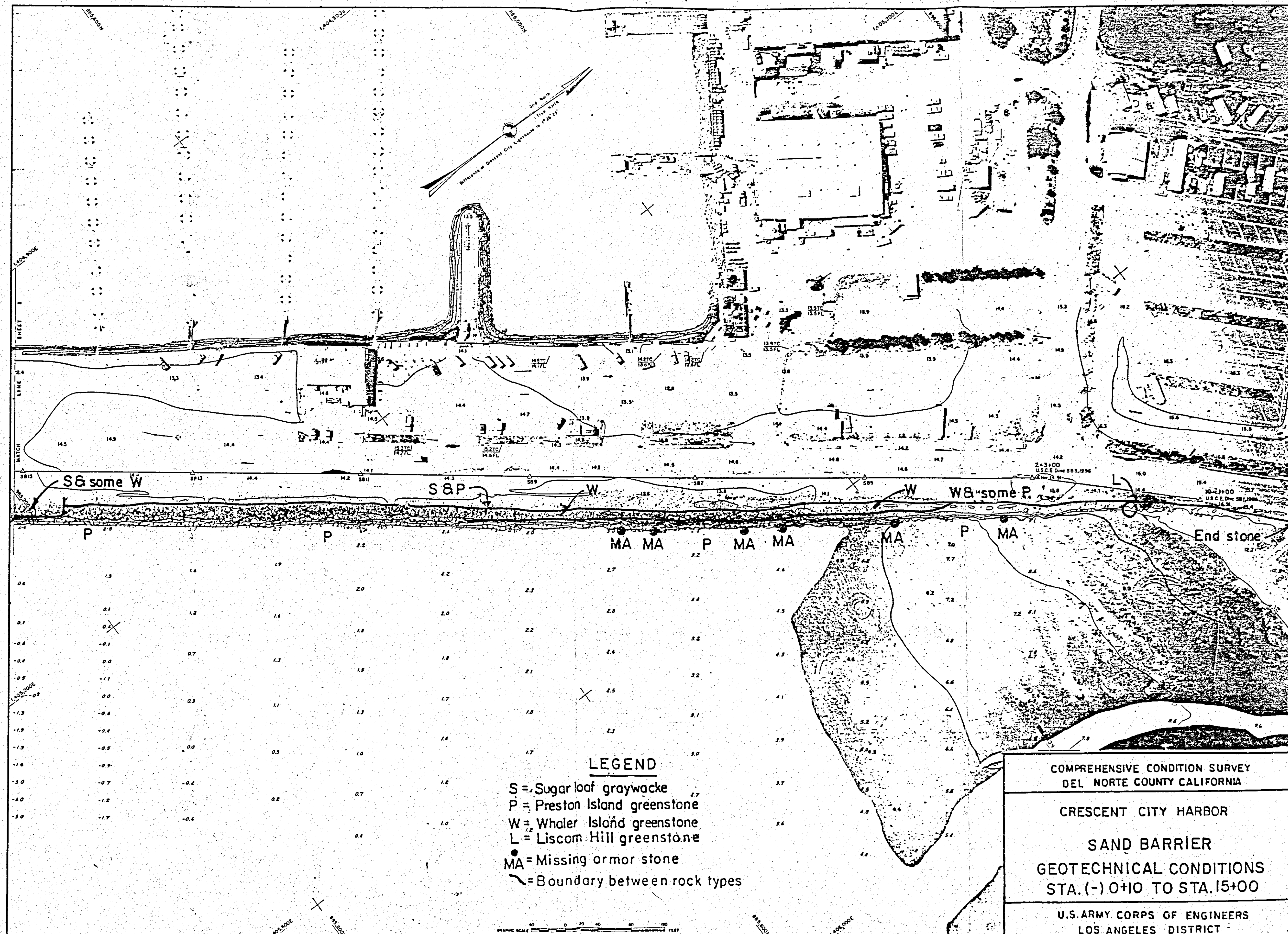
COMPREHENSIVE CONDITION SURVEY
DEL NORTE COUNTY CALIFORNIA

CRESCENT CITY HARBOR

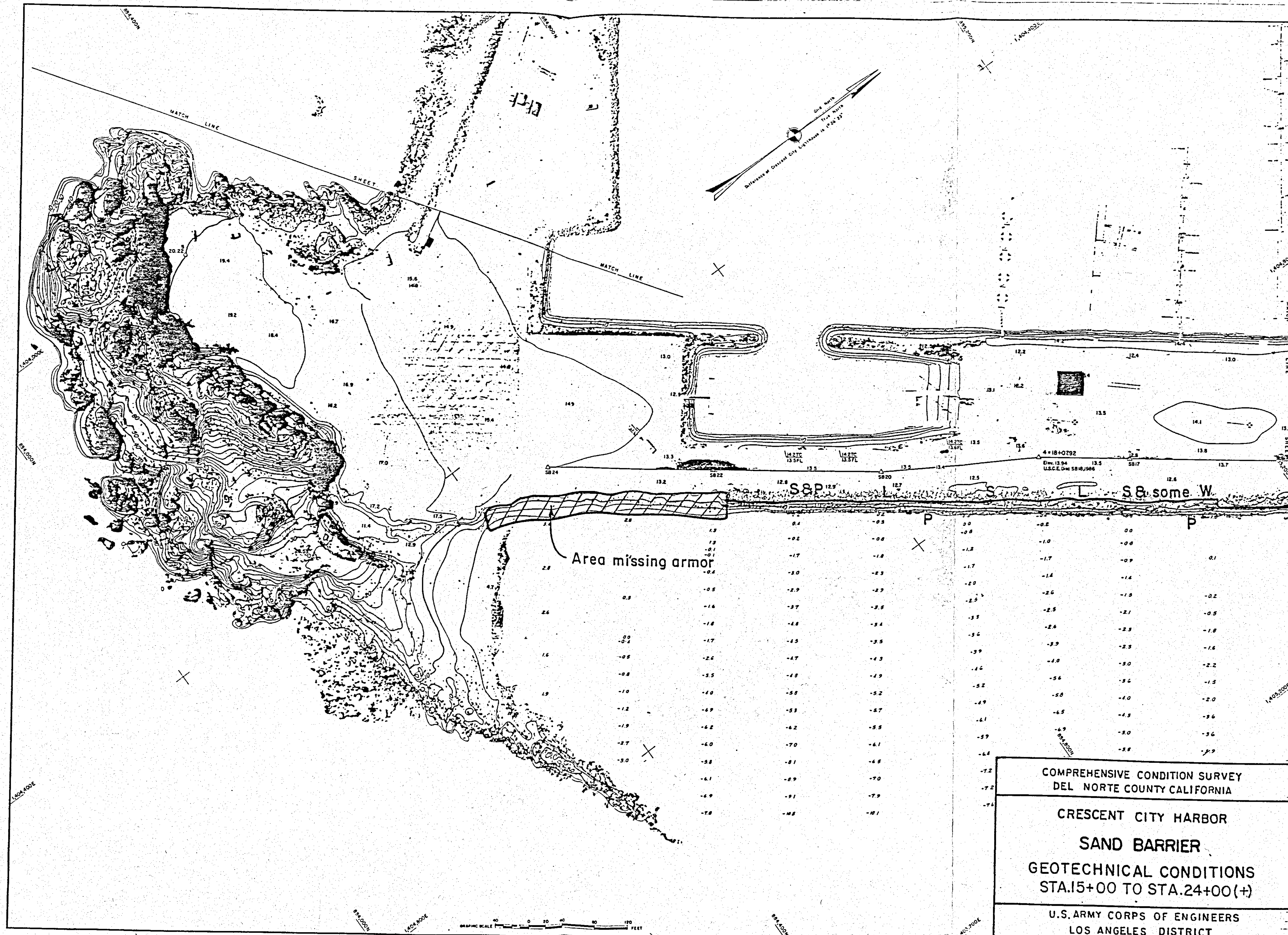
INNER BREAKWATER AND SAND BARRIER

QUARRIES

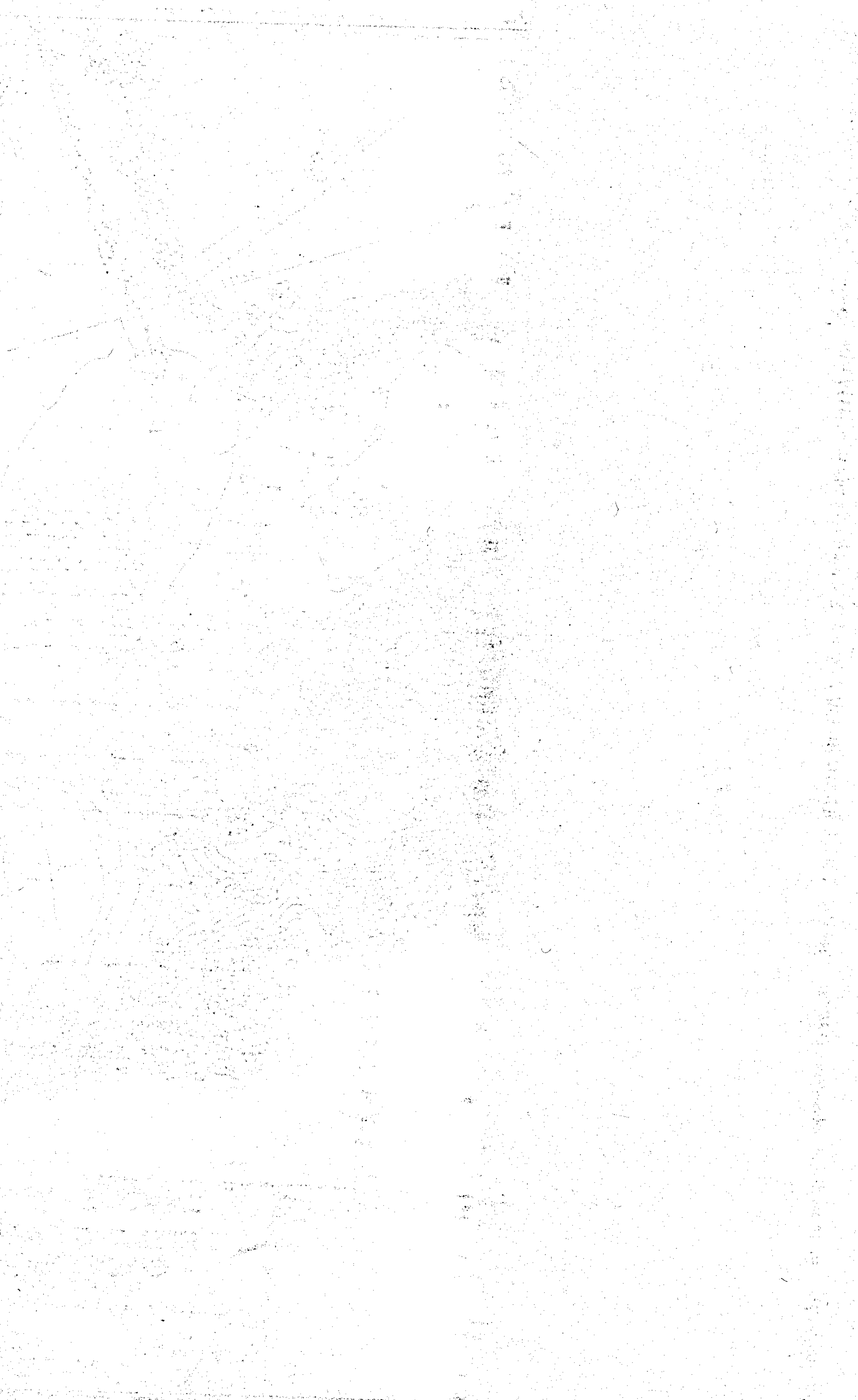
U.S. ARMY CORPS OF ENGINEERS
LOS ANGELES DISTRICT

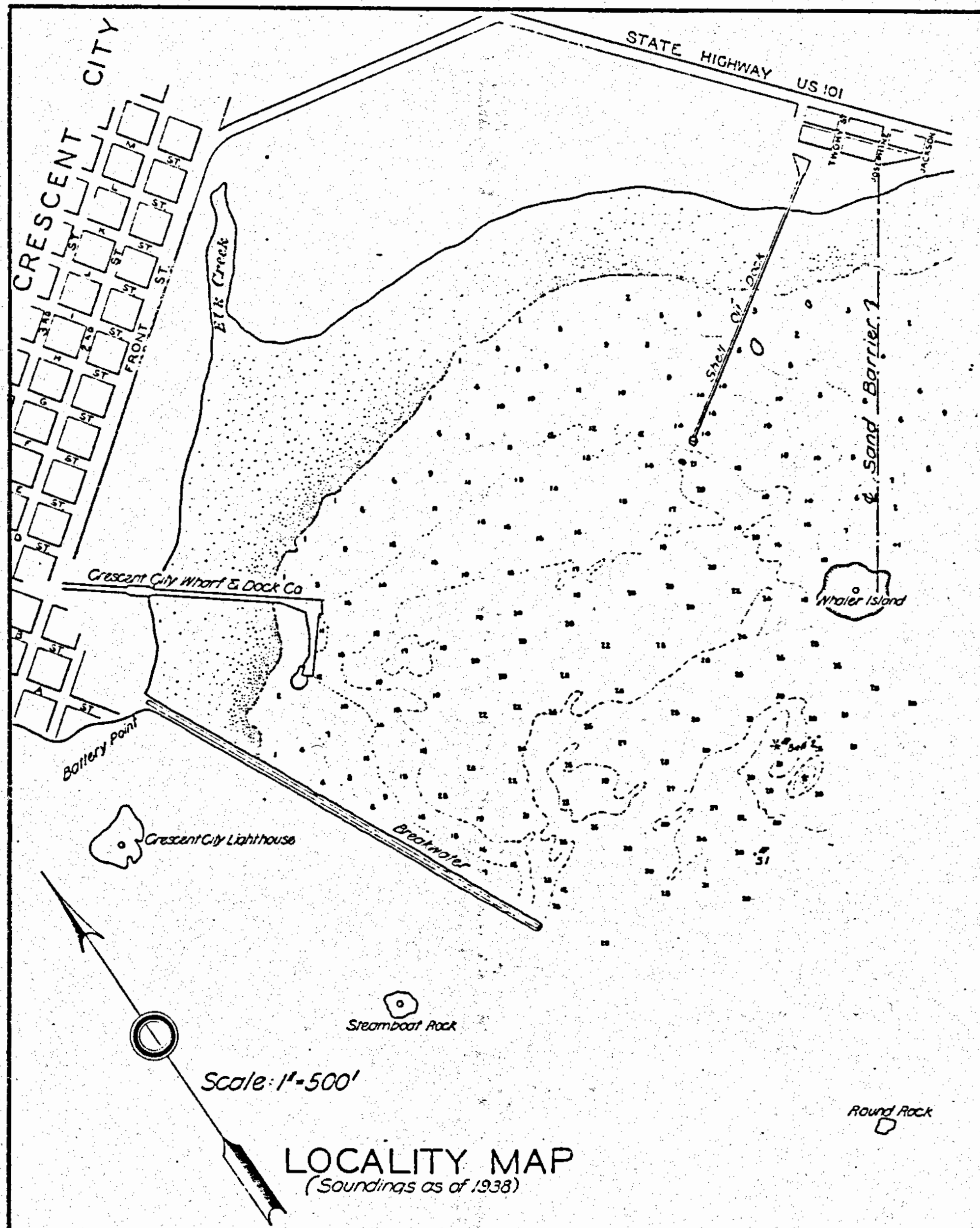


S& some W
S&P
W
W& some P
L
P
MA
End stone
LEGEND
S = Sugar loaf graywacke
P = Preston Island greenstone
W = Whaler Island greenstone
L = Liscom Hill greenstone
MA = Missing armor stone
= Boundary between rock types
COMPREHENSIVE CONDITION SURVEY
DEL NORTE COUNTY CALIFORNIA
CRESCENT CITY HARBOR
SAND BARRIER
GEOTECHNICAL CONDITIONS
STA. (-) 0+10 TO STA. 15+00
U.S. ARMY CORPS OF ENGINEERS
LOS ANGELES DISTRICT



APPENDIX B PLATE 2 SHEET 2 OF 2



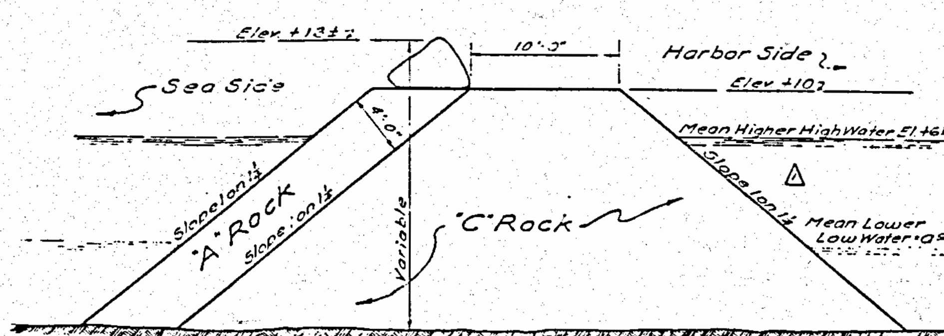


LOCALITY MAP
(Soundings as of 1938)

LOG OF JET PROBINGS

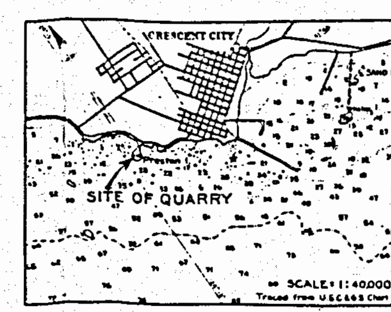
No samples were taken.

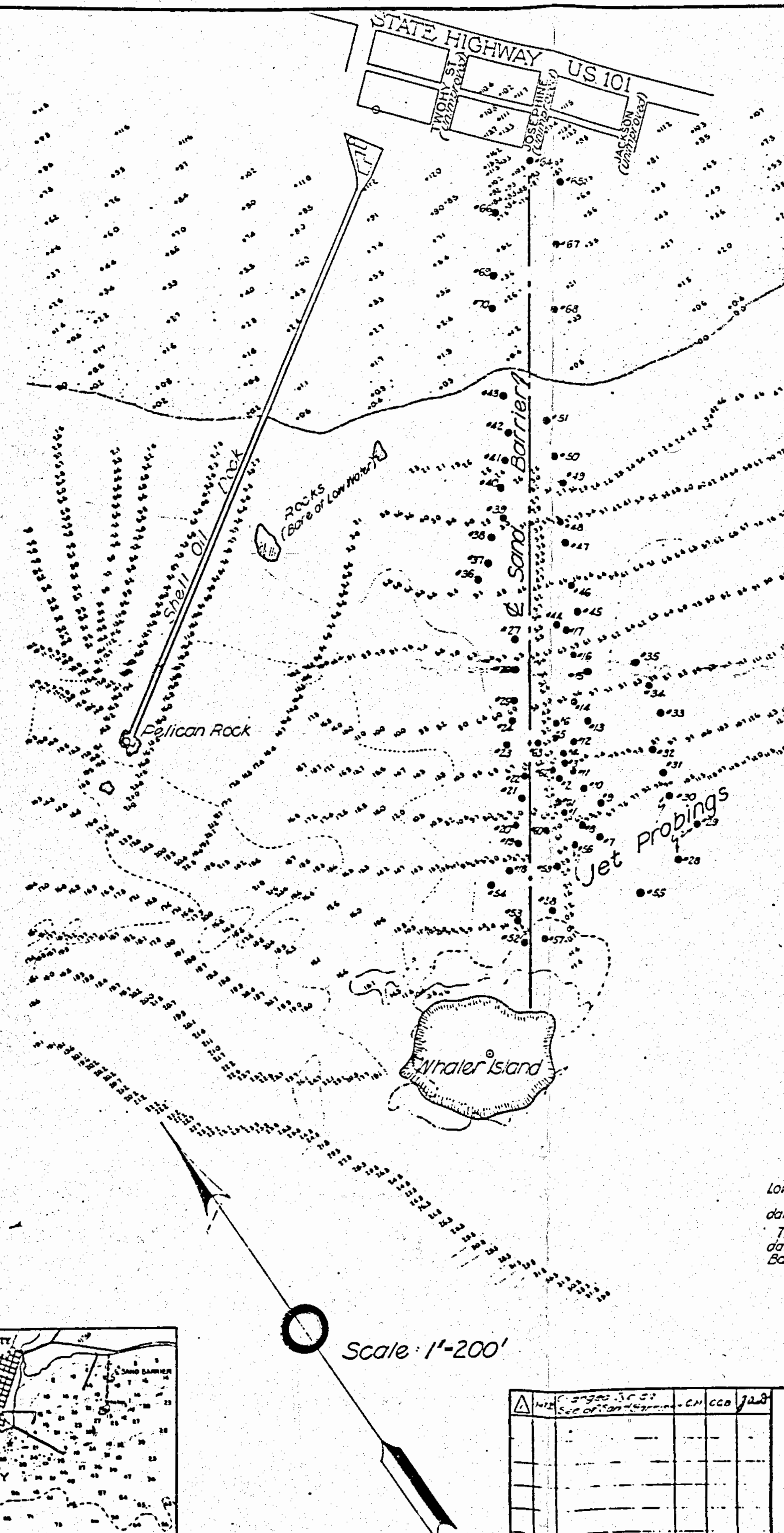
Hole	Depths (Ft) Sand	Depths (Ft) Rock	
1	7.2	8.2	All Probings thru sand to rock
2	7.2	7.5	
3	7.1	7.6	
4	6.8	11.3	
5	5.8	12.3	
6	5.4	8.3	
7	5.3	7.8	
8	6.1	7.3	
9	6.4	7.3	
10	7.3	8.0	
11	7.1	7.3	
12	6.8	7.3	
13	5.3	7.3	
14	5.4	8.4	
15	6.2	6.9	
16	5.8	8.4	
17	6.5	8.8	
18	3.6	3.6	
19	5.4	10.4	
20	6.6	9.9	
21	6.6	8.8	
22	6.4	10.4	
23	6.2	10.4	
24	5.7	10.4	
25	5.8	9.4	
26	5.4	9.4	
27	5.5	12.4	
28	9.3	9.8	
29	12.3	12.3	
30	10.3	10.3	
31	10.1	10.1	
32	8.1	9.2	
33	7.1	8.1	
34	9.1	9.6	
35	8.1	8.1	
36	4.4	9.4	
37	4.4	11.4	
38	3.8	10.8	
39	3.3	12.8	
40	2.3	13.3	
41	2.2	12.3	
42	2.2	8.3	
43	1.3	7.3	
44	4.7	4.7	
45	3.4	8.2	
46	4.7	8.2	
47	4.1	8.7	
48	3.4	13.2	
49	3.0	13.2	
50	2.4	12.7	
51	1.8	5.2	
52	2.1	5.1	
53	2.8	11.1	
54	3.1	8.1	
55	2.6	2.6	
56	3.1	4.9	
57	0.0	7.1	
58	2.1	5.6	
59	3.3	8.1	
60	6.1	7.1	
61	7.6	8.5	
62	7.1	8.1	
63	5.6	9.1	
64	+11.0	15.0	1 to 2 Ft of gravel on top of rock
65	+8.9	15.1	
66	+8.1	15.9	
67	+5.3	14.7	Sand caved in
68	+2.8	14.2	2 Ft of gravel on top of rock
69	+4.5	14.5	
70	+2.4	13.6	



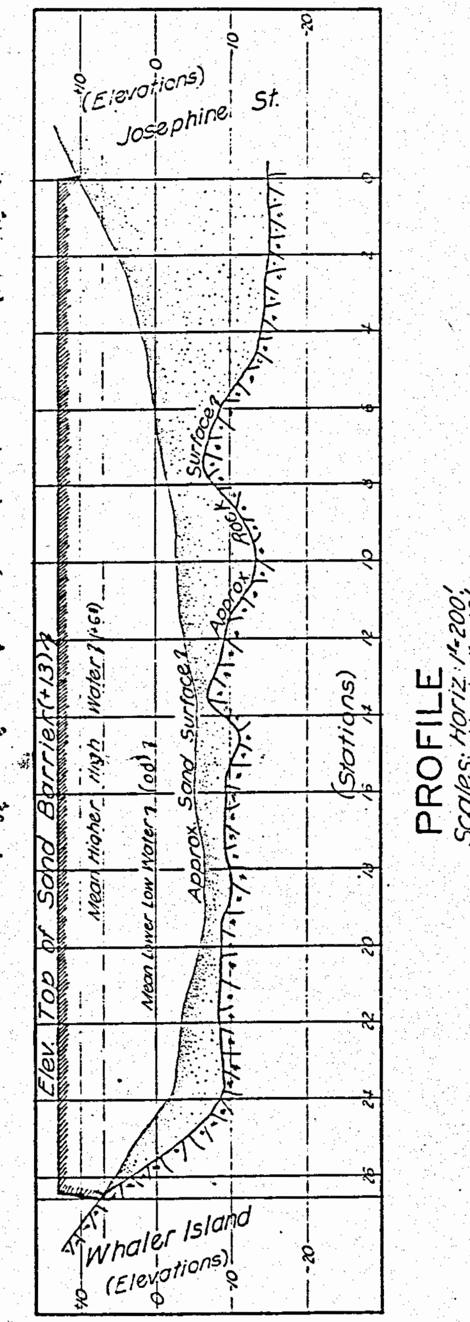
Scale 1"=5'

TYPICAL CROSS-SECTION OF SAND BARRIER





GENERAL PLAN



PROFILE
Scales: Horiz. 1"=200'
Vert. 1"=10'

NOTES

Soundings are in feet & tenths and refer to the plane of Mean Lower Low Water.

Figures preceded by a plus sign denote elevations above the datum plane.

This Map is the one referred to in paragraph 1-03 of the Specifications dated Aug. 15, 1938, for furnishing rock and constructing a Sand Barrier in Crescent City Harbor, California.

REV NO	DATE	REVISION	REV BY	CHK BY	APP BY
1		Changes Sec of Sand Barrier	CH	CCB	JAD

SAND BARRIER
CRESCENT CITY HARBOR
CALIFORNIA

SCALES AS SHOWN

U. S. ENGINEER OFFICE, SAN FRANCISCO, CALIF. AUGUST 15, 1938

SUBMITTED

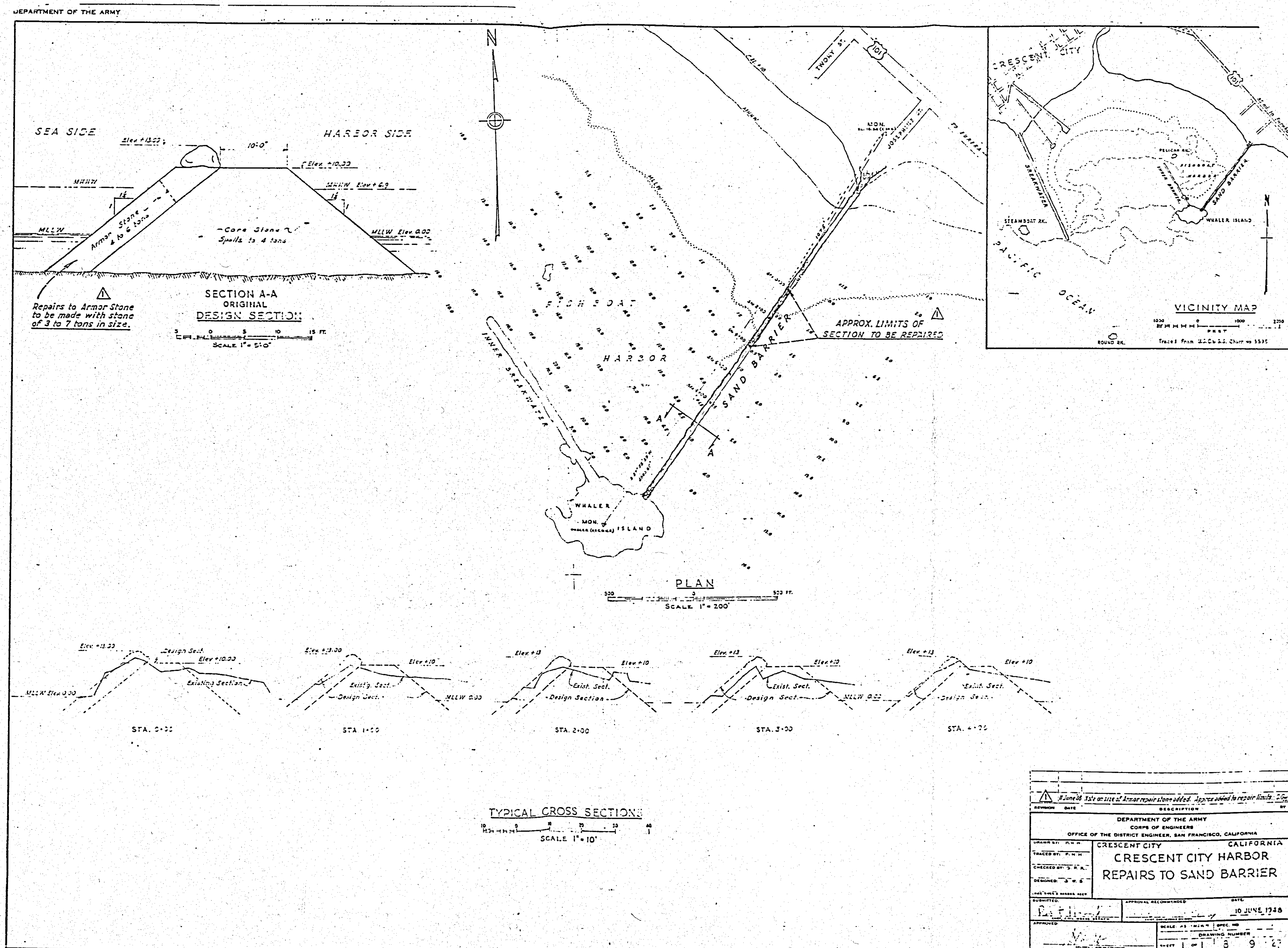
SENIOR ENGINEER

APPROVED

CAPT. CORPS OF ENGINEERS

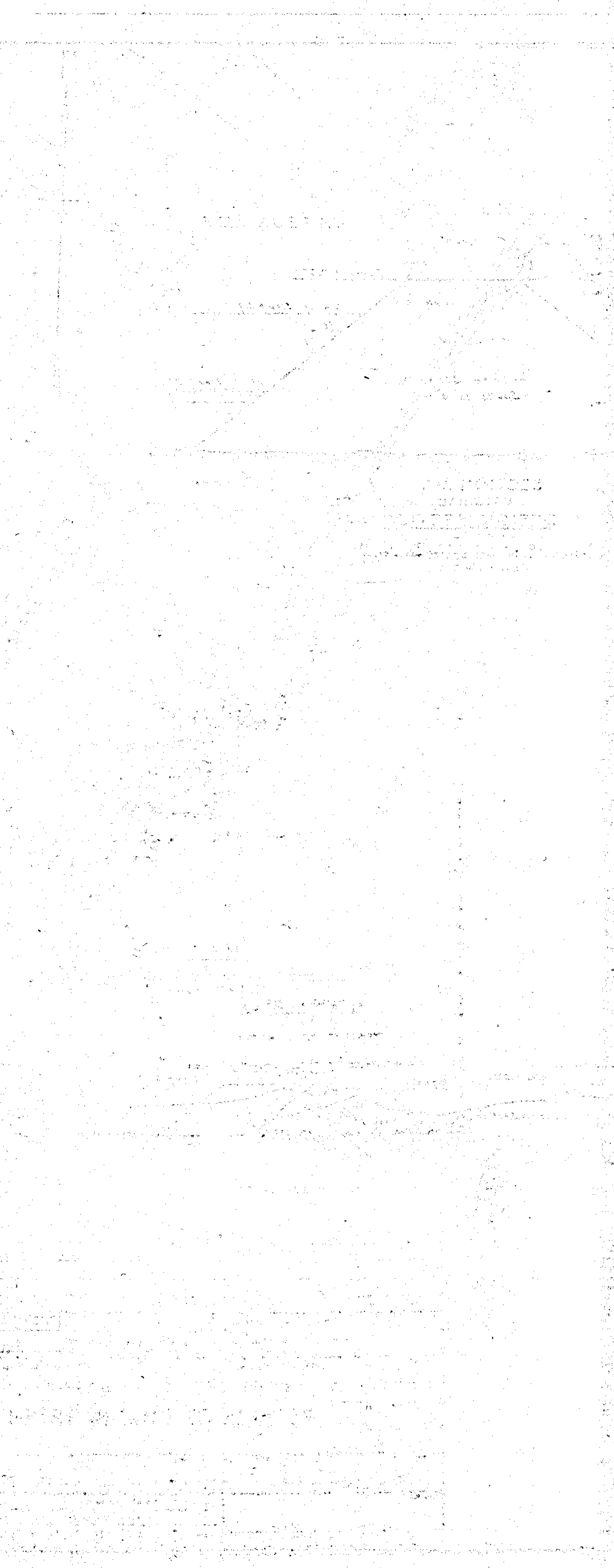
DRAWN R.L.P.
TRACED R.L.P.
CHECKED C.C.E.

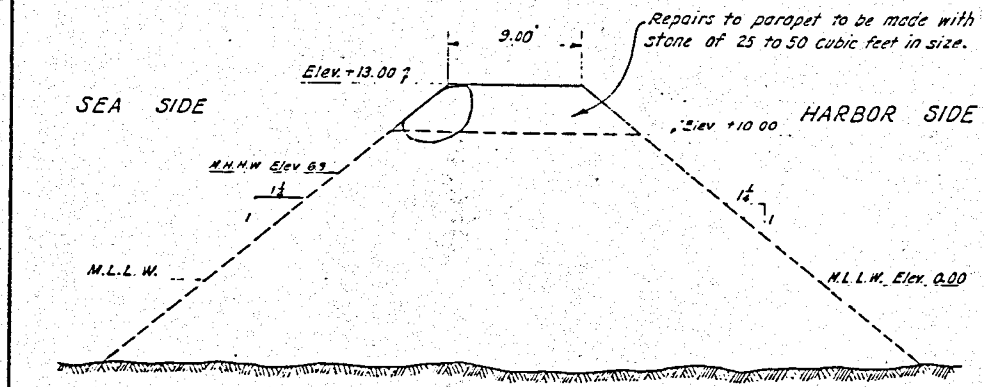
DOC FILE	FILE NO	DIV.	SHEET
	8	9	18



APPENDIX B

PLATE 1



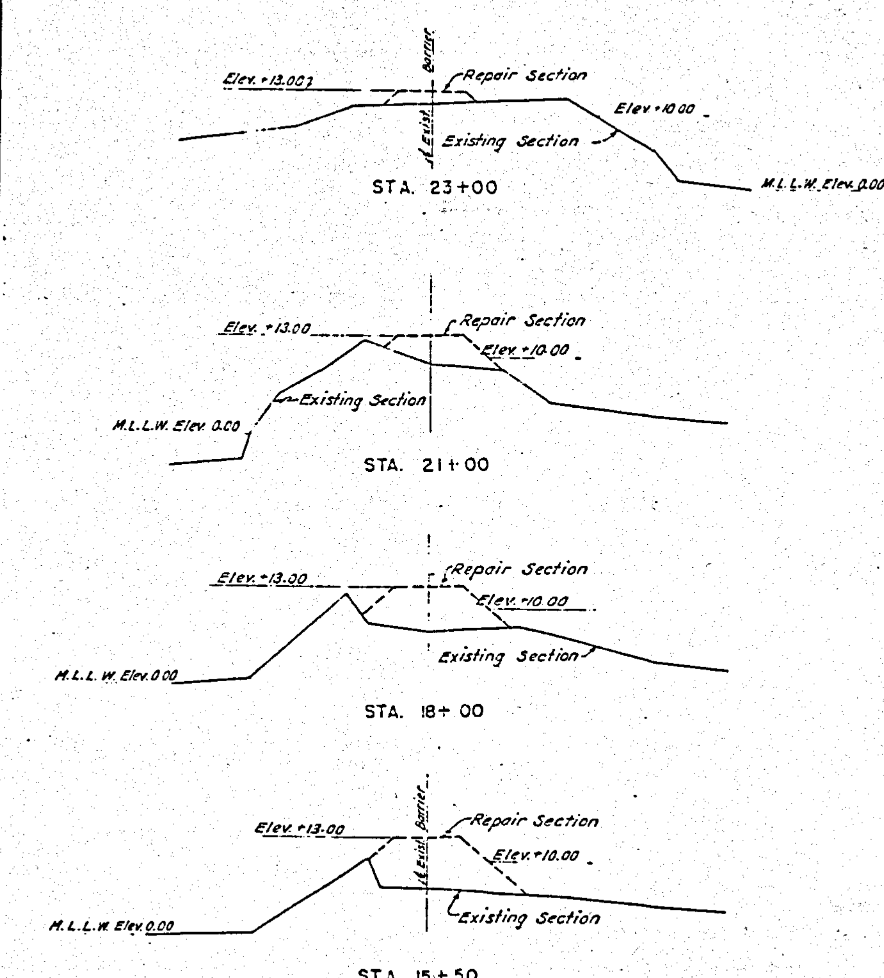


CROSS SECTION OF SAND BARRIER

LEGEND
Original design section shown thus ----
Repair work section limits shown thus ——

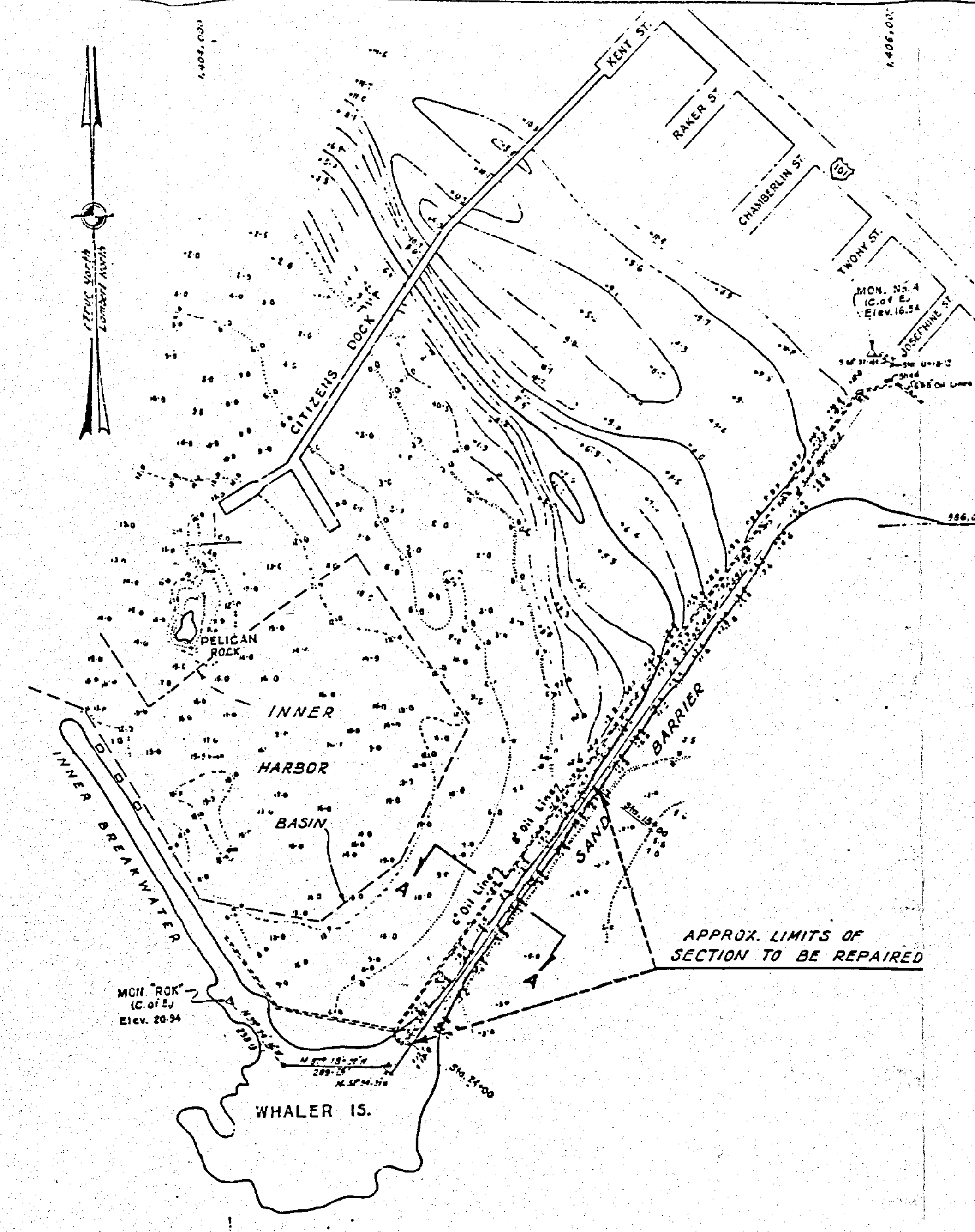
SECTION A–A

SCALE 1" = 5'



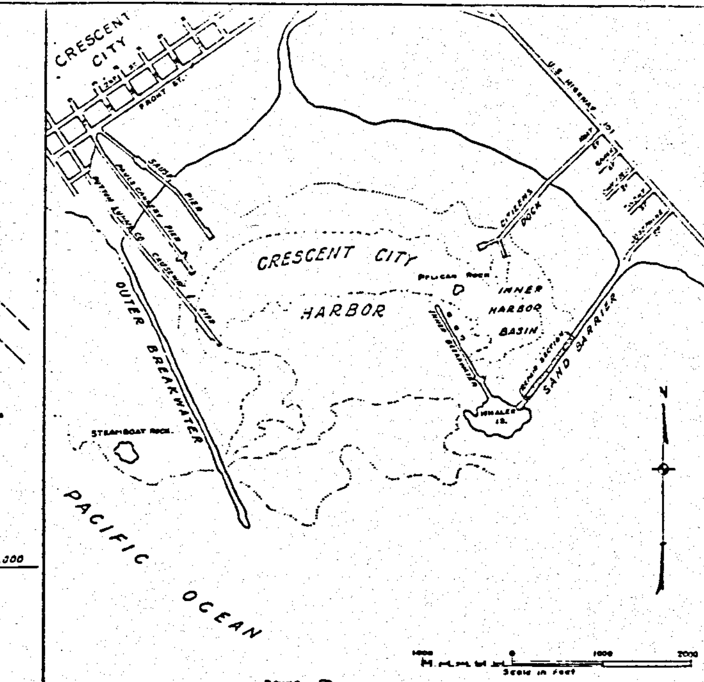
TYPICAL CROSS SECTIONS

SCALE 1" = 10'

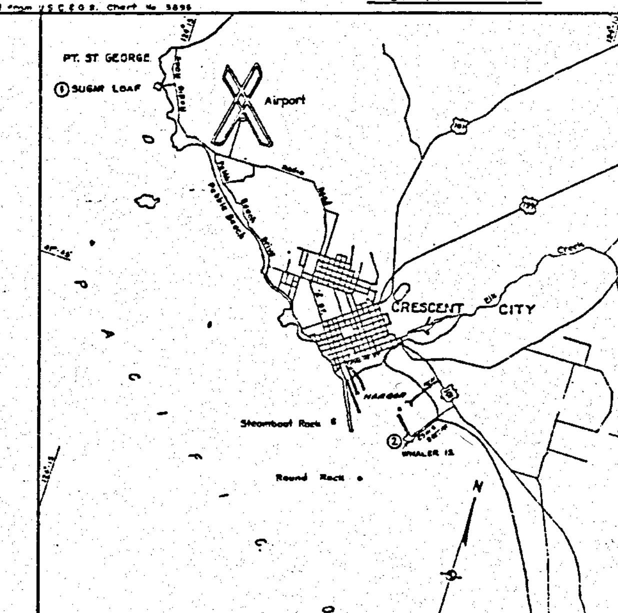


PLAN

SCALE 1" = 200'



VICINITY MAP



QUARRY SITES
VICINITY OF
CRESCENT CITY, CALIFORNIA

① Sugar Loaf
② Whaler Island

NOTES:
Surveyed by the Corps of Engineers on 1 & 2 April 1952. Soundings, Elevations and Bench Marks are referred to the Datum of M.L.L.W. of the locality. Additional Cross Sections are available for examination in San Francisco District Office.

REVISION	DATE	DESCRIPTION	BY

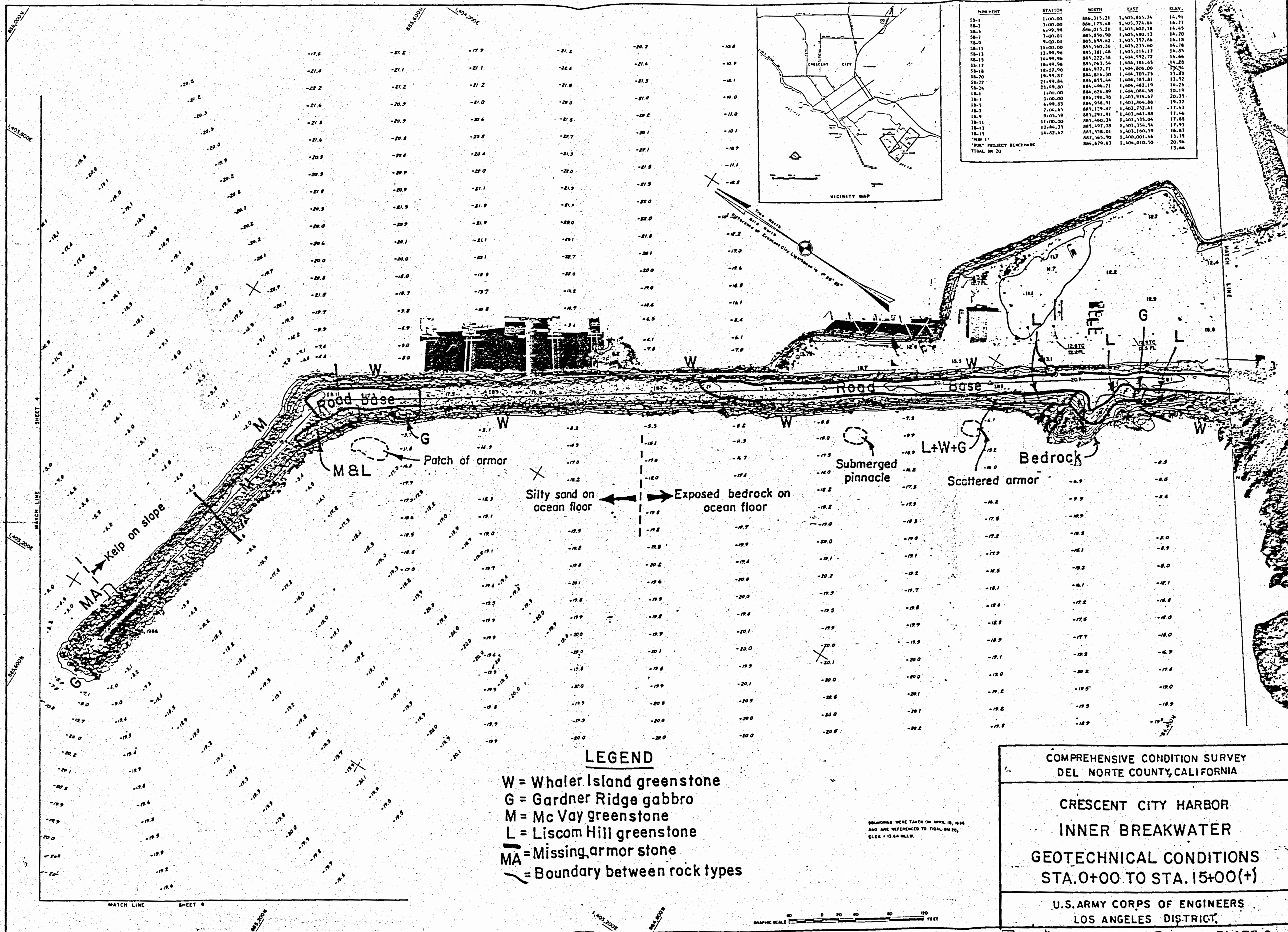
CORPS OF ENGINEERS
U. S. ARMY
OFFICE OF THE DISTRICT ENGINEER, SAN FRANCISCO DISTRICT
SAN FRANCISCO, CALIFORNIA

DRAWN BY: C.W.H
TRACED BY: C.W.H
CHECKED BY: J.A.S
DESIGNED: J.A.S.

CRESCENT CITY CALIFORNIA
CRESCENT CITY HARBOR
REPAIRS TO SAND BARRIER

SUBMITTED: [signature]
APPROVAL RECOMMENDED: [signature]
DATE: 2 MAY 1952
APPROVED: [signature]

SCALE: AS SHOWN
DRAWING NUMBER
SHEET 1 OF 1 | 8 | 9 | 36



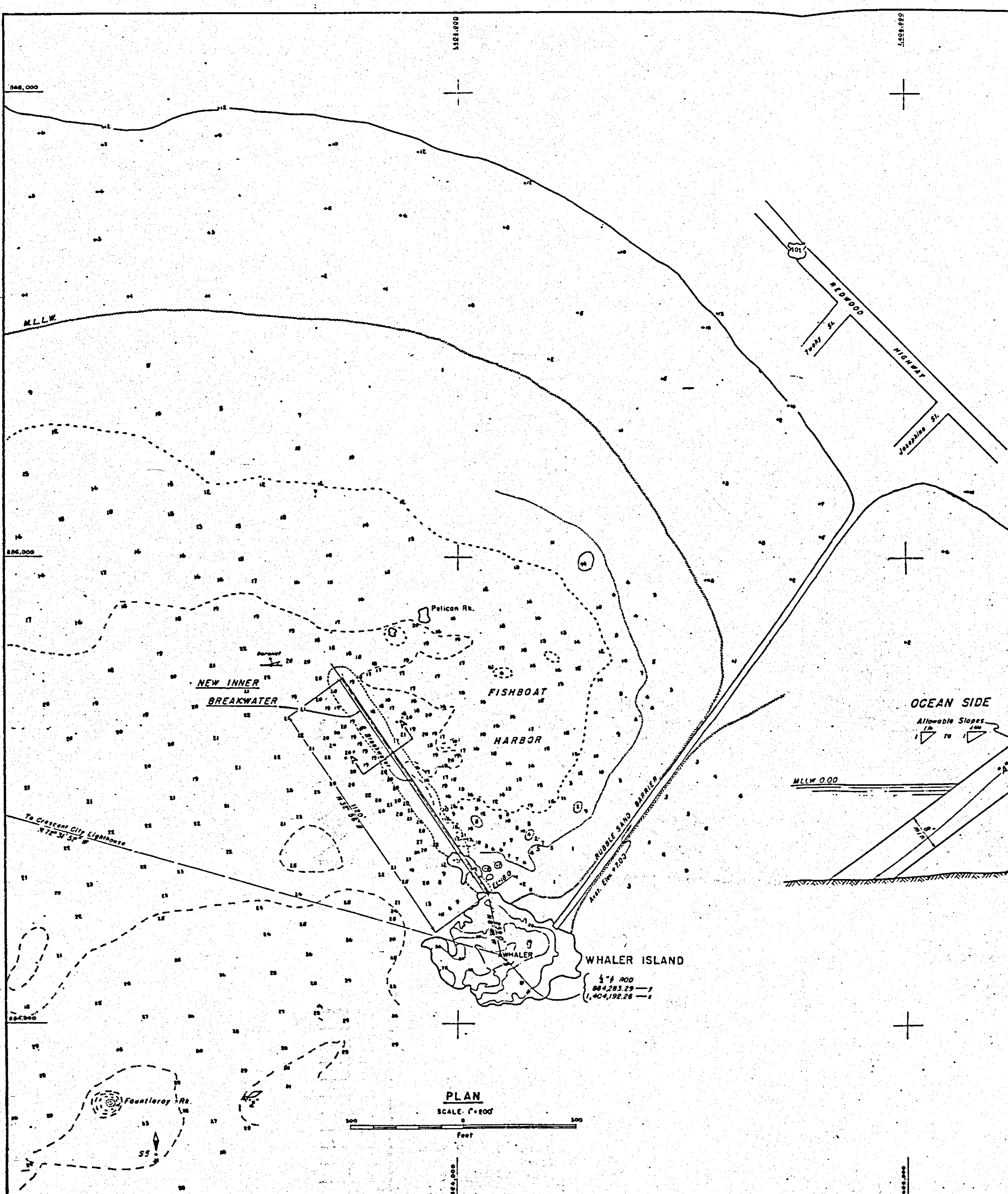
MONUMENT	STATION	NORTH	EAST	ELEV.
SB-1	1+00.00	886,315.21	1,405,865.74	14.91
SB-3	3+00.00	886,173.48	1,405,724.64	14.77
SB-5	4+99.99	886,015.21	1,405,602.38	14.45
SB-7	7+00.01	885,856.90	1,405,480.13	14.20
SB-9	9+00.01	885,698.62	1,405,357.86	14.18
SB-11	11+00.00	885,540.36	1,405,235.60	14.78
SB-13	12+99.96	885,381.48	1,405,114.17	14.85
SB-15	14+99.96	885,222.58	1,404,992.72	14.66
SB-17	16+99.96	885,063.54	1,404,781.45	14.28
SB-18	18+07.90	884,977.71	1,404,806.00	~~[illegible]96~~
SB-20	19+99.87	884,814.30	1,404,705.25	13.83
SB-22	21+99.84	884,655.44	1,404,583.81	13.52
SB-24	23+99.80	884,496.71	1,404,462.19	14.26
IB-1	1+00.00	884,624.89	1,404,084.58	20.19
IB-3	3+00.00	884,791.96	1,403,974.67	20.35
IB-5	4+99.83	884,958.91	1,403,864.86	19.77
IB-7	7+04.45	885,129.87	1,403,752.41	17.43
IB-9	9+05.59	885,297.91	1,403,641.88	17.66
IB-11	11+00.00	885,460.34	1,403,535.04	17.88
IB-13	12+84.35	885,497.78	1,403,354.54	17.93
IB-15	14+82.42	885,538.01	1,403,160.59	16.83
"MON 1"		887,565.90	1,400,001.46	13.79
"ROK" PROJECT BENCHMARK		884,679.63	1,404,010.50	20.94
TIDAL BM 20				13.64

COMPREHENSIVE CONDITION SURVEY
DEL NORTE COUNTY, CALIFORNIA

CRESCENT CITY HARBOR
INNER BREAKWATER
GEOTECHNICAL CONDITIONS
STA. 0+00 TO STA. 15+00(+)

U.S. ARMY CORPS OF ENGINEERS
LOS ANGELES DISTRICT

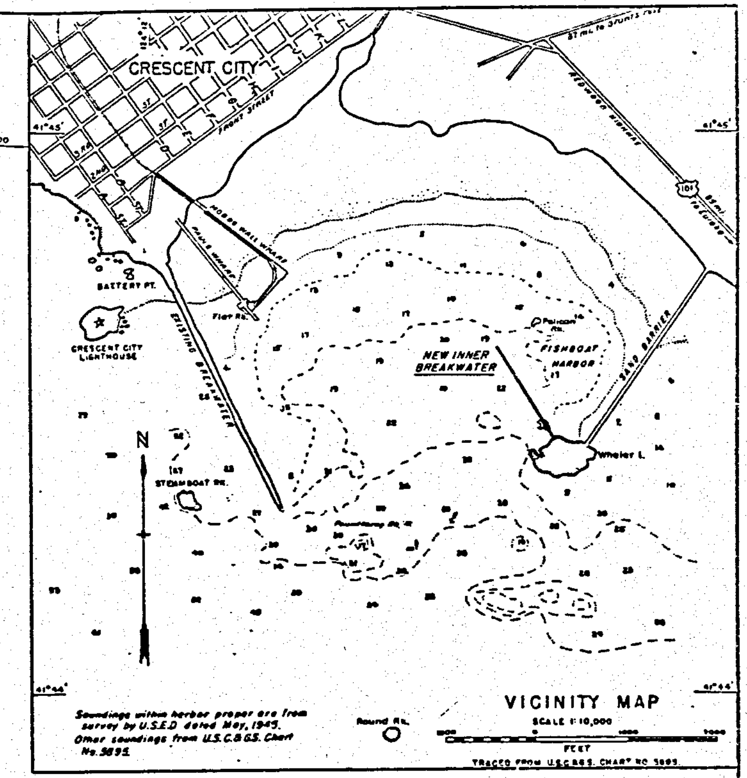
APPENDIX B PLATE 8



PLAN

SCALE: 1" = 200'

Feet

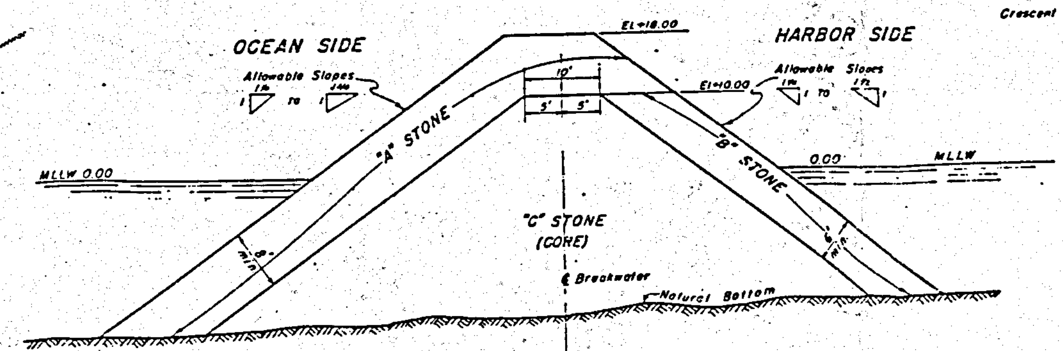


VICINITY MAP

SCALE 1:10,000

Notes:

1. Soundings are in feet and refer to the plane of MLLW.
2. Elevations above datum are prefixed with a plus(+) sign.
3. Soundings are from survey by U.S.E.D. dated May, 1945.
4. Breakwater Center Line to be staked by the United States.
5. ~~Royalty-free stone may be obtained~~ [illegible]
6. Plane coordinates are based on the system described in Special Publication No. 202 of the U.S.C.&G.S. Chart (1936) for Zone I, California.
7. This is the map referred to in paragraph (SC-4) of specifications dated 11 March 1946 for construction of Inner Breakwater, Crescent City, Calif.



SECTION A-A

(TYPICAL)

SCALE: 1" = 10'

SIZES OF STONE		
CLASS	RANGE	GRADING
A	70 cu. ft. or larger	100 cu. ft. average
B	25 to 70 cu. ft.	Over 50% – larger than 50 cu. ft.
C	Fines to 36 cu. ft.	Over 40% – 3 cu. ft. or larger Over 60% – 1 cu. ft. or larger Not over 5% – 0.03 cu. ft. or smaller

REVIEWED

DATE 3-7-46

CRESCENT CITY HARBOR, CALIF.

INNER BREAKWATER

GENERAL PLAN AND DETAILS

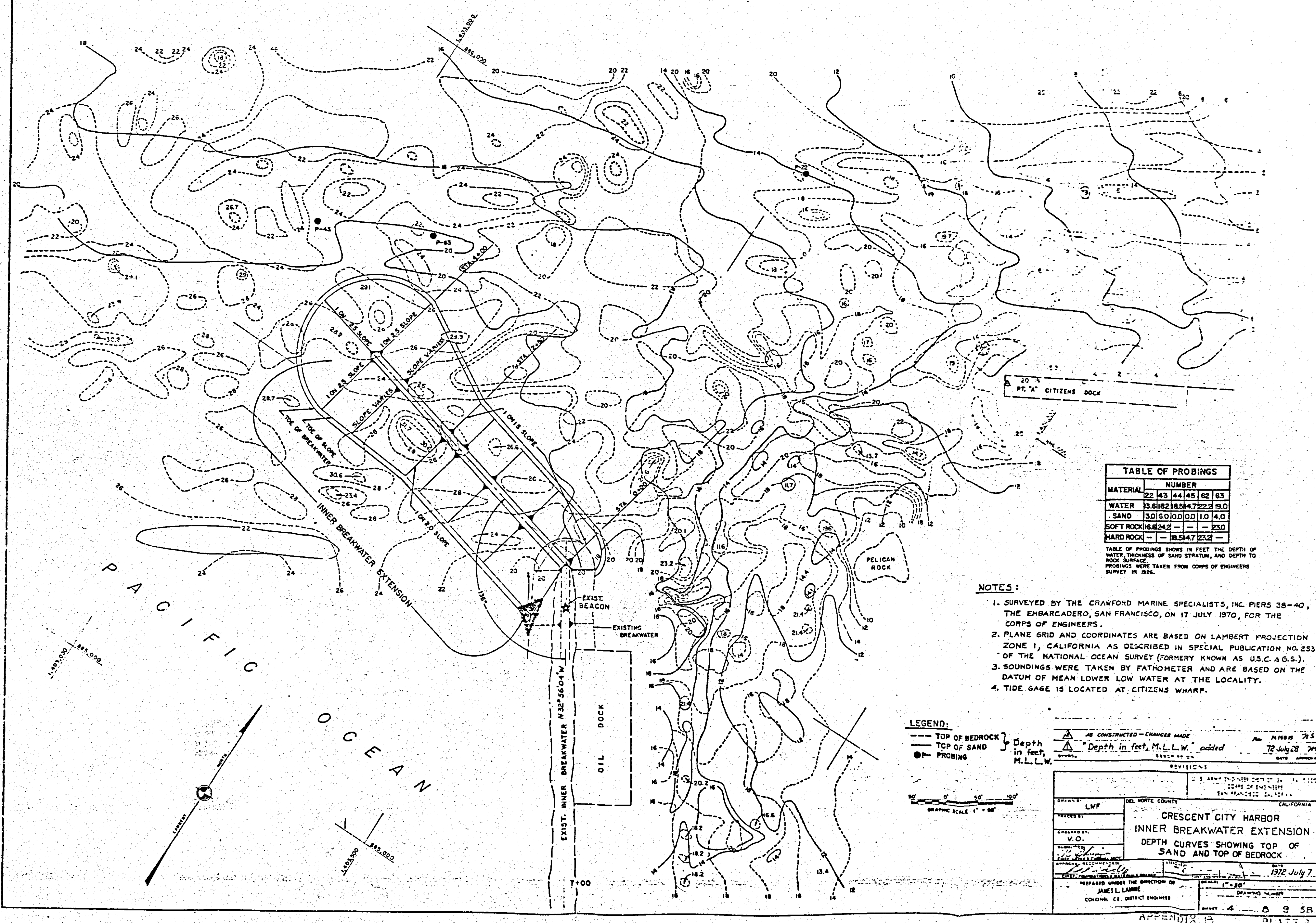
SCALE: AS SHOWN DATE: 8 MARCH 1946 SHEET NO. 1 OF 1

U. S. ENGINEER OFFICE, SAN FRANCISCO, CALIFORNIA

SUBMITTED R. Hornigoff APPROVAL RECOMMENDED APPROVED J. H. Veal

DESIGNED G.P.R. CHECKED DRAWN TRACED P.R. W.C.D.

FILE	DIV.	PROJECT
8	9	22



NOTES:

1. SURVEYED BY THE CRAWFORD MARINE SPECIALISTS, INC. PIERS 38-40, THE EMBARCADERO, SAN FRANCISCO, ON 17 JULY 1970, FOR THE CORPS OF ENGINEERS.
2. PLANE GRID AND COORDINATES ARE BASED ON LAMBERT PROJECTION ZONE I, CALIFORNIA AS DESCRIBED IN SPECIAL PUBLICATION NO. 253 OF THE NATIONAL OCEAN SURVEY (FORMERY KNOWN AS U.S.C. & G.S.).
3. SOUNDINGS WERE TAKEN BY FATHOMETER AND ARE BASED ON THE DATUM OF MEAN LOWER LOW WATER AT THE LOCALITY.
4. TIDE GAGE IS LOCATED AT CITIZENS WHARF.

TABLE OF PROBINGS

MATERIAL	NUMBER					
	22	43	44	45	62	63
WATER	13.6	18.2	18.5	14.7	22.2	19.0
SAND	3.0	6.0	0.0	0.0	1.0	4.0
SOFT ROCK	16.6	24.2	—	—	—	23.0
HARD ROCK	—	—	18.5	14.7	23.2	—

TABLE OF PROBINGS SHOWS IN FEET THE DEPTH OF WATER, THICKNESS OF SAND STRATUM, AND DEPTH TO ROCK SURFACE.
PROBINGS WERE TAKEN FROM CORPS OF ENGINEERS SURVEY IN 1926.

REVISIONS:
2 AS CONSTRUCTED — CHANGES MADE
1 "Depth in feet, M.L.L.W." added — 72 July 28

DRAWN BY: LMF
CHECKED BY: V.O.

DEL NORTE COUNTY — CALIFORNIA

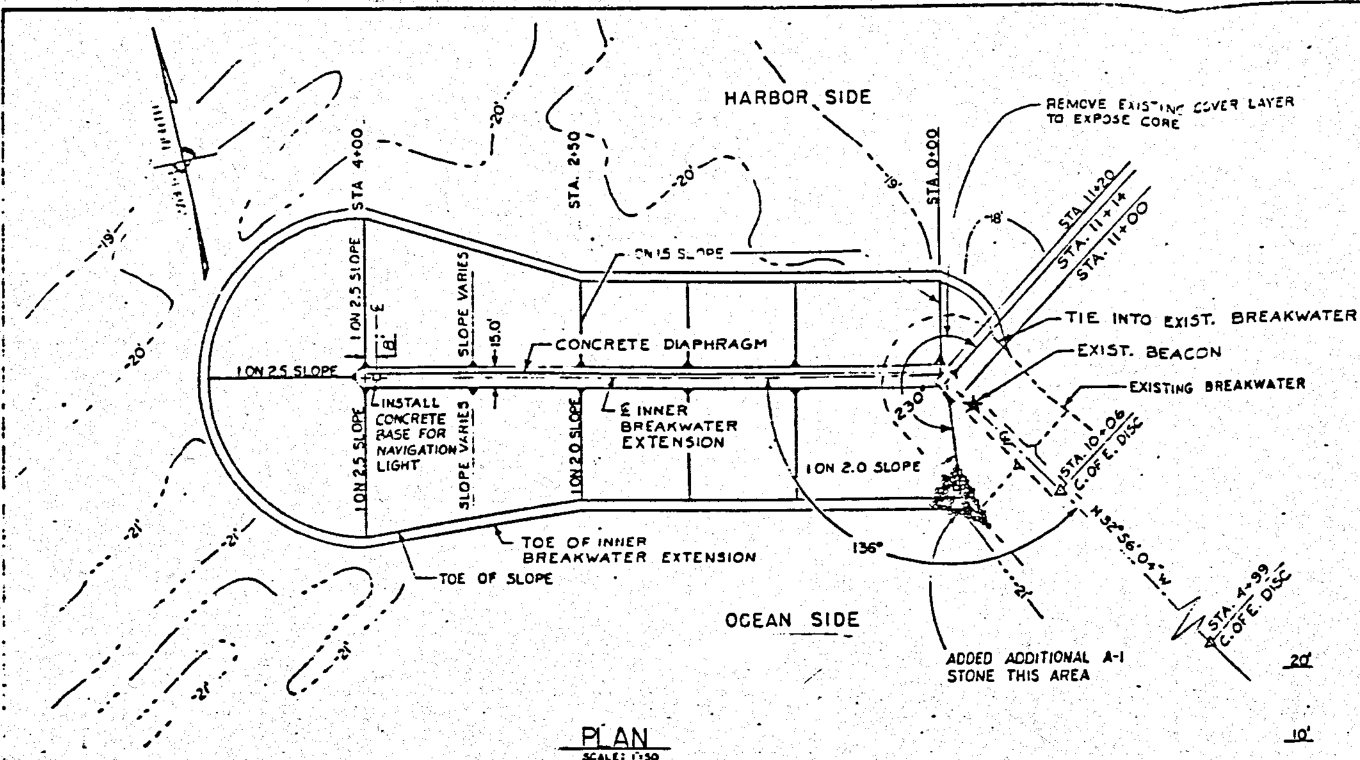
CRESCENT CITY HARBOR
INNER BREAKWATER EXTENSION
DEPTH CURVES SHOWING TOP OF SAND AND TOP OF BEDROCK

DATE: 1972 July 7

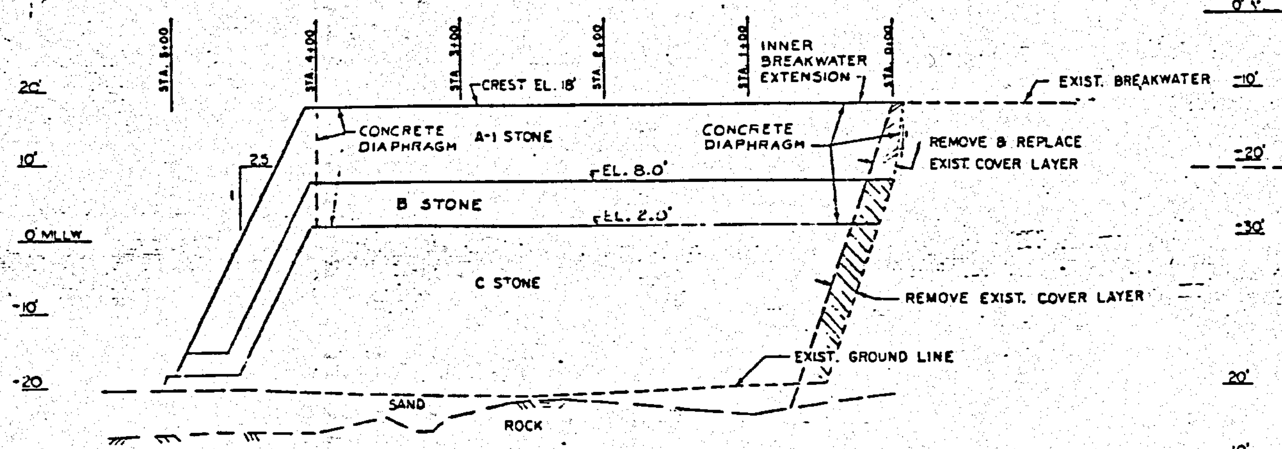
PREPARED UNDER THE DIRECTION OF JAMES L. LAMMIE, COLONEL C.E. DISTRICT ENGINEER

SCALE: 1" = 80'

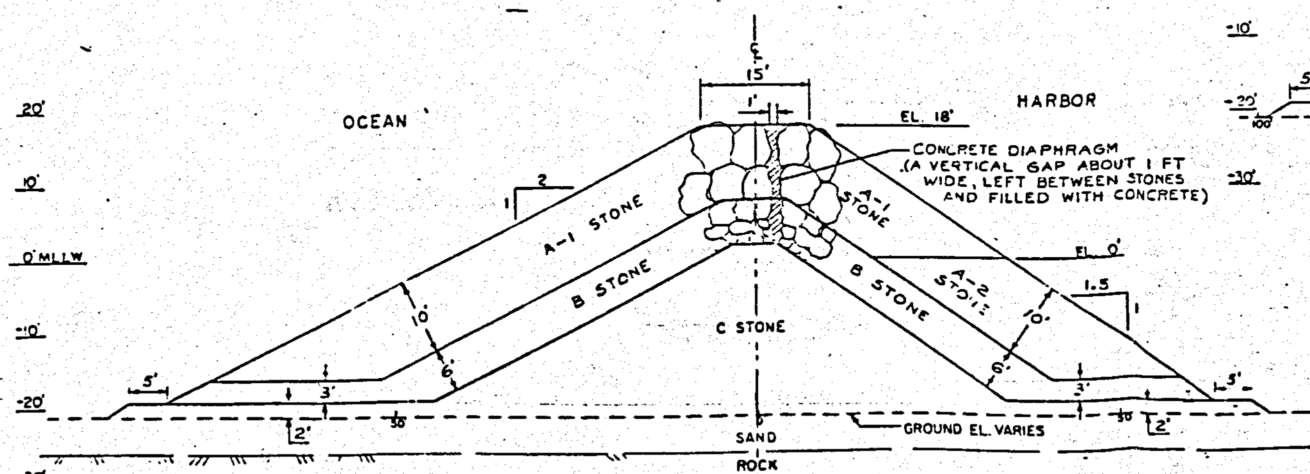
SHEET 4 — 8 9 58



PLAN
SCALE: 1"=50'



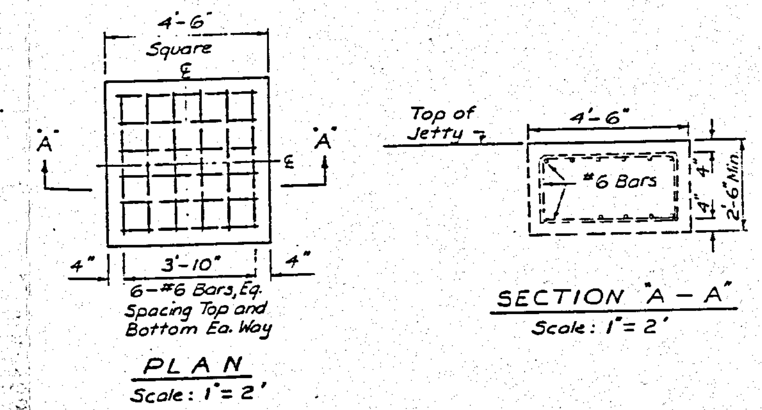
PROFILE



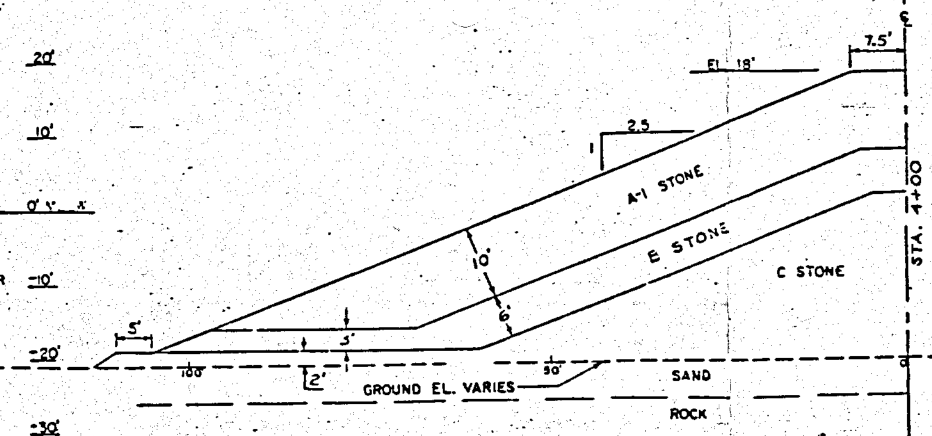
TRUNK SECTION, STA. 0+00 TO STA. 2+50
SCALE: 1"=10'

Notes:

1. Concrete base may be cast integrally with the adjacent concrete diaphragm.
2. Anchor bolt assembly and template will be furnished.
3. Anchor bolt assembly, not shown, to be embedded in concrete with the reinforcing steel.



CONCRETE NAVIGATION LIGHT BASE REINFORCEMENT DETAILS



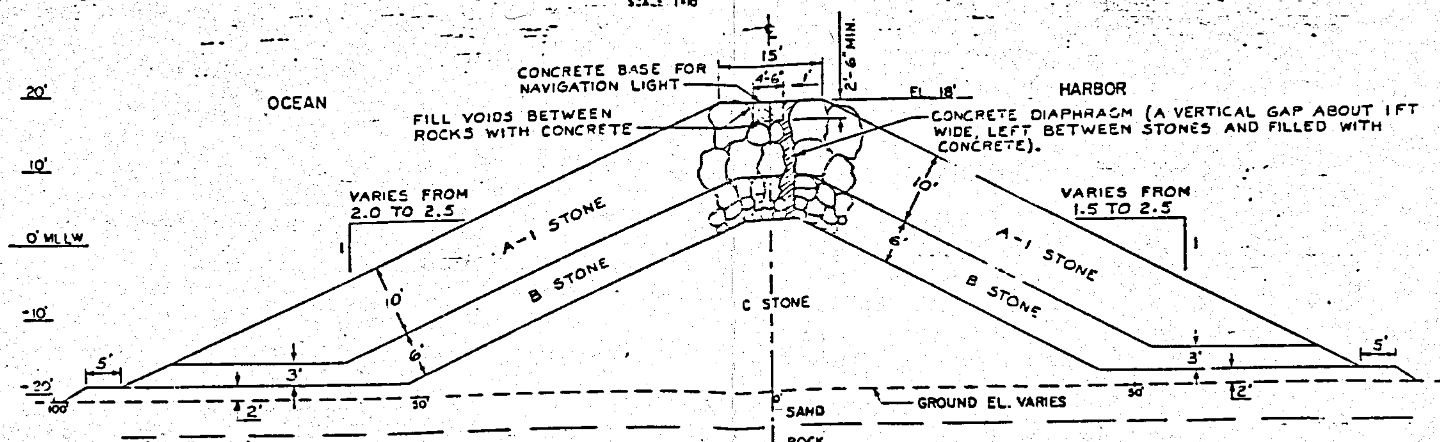
CONICAL HEAD SECTION, STA. 4+00
SCALE: 1"=10'

STONE SIZES

TYPE OF STONE	MIN. SIZE	AVE. SIZE	MAX. SIZE
A-1	9 TONS	11 TONS	13 TONS
A-2	2.0 TONS	4.5 TONS	9.0 TONS
B	100 LBS.	1.0 TON	2.5 TONS
C	FINES*	25 LBS.	200 LBS.

*NOT MORE THAN 5% PASSING THE NO. 4 SIEVE.

NOTE:
WEIGHTS OF STONE ARE BASED ON SPECIFIC GRAVITY OF 2.74, SEE SECTION 4-3.8 OF THE SPECIFICATIONS.



TRANSITION SECTION, STA. 2+50 TO STA. 4+00
SCALE: 1"=10'

GRAPHIC SCALE 1" = 10'
GRAPHIC SCALE 1" = 50'

1	AS CONSTRUCTED - CHANGES MADE	74 FEB 15

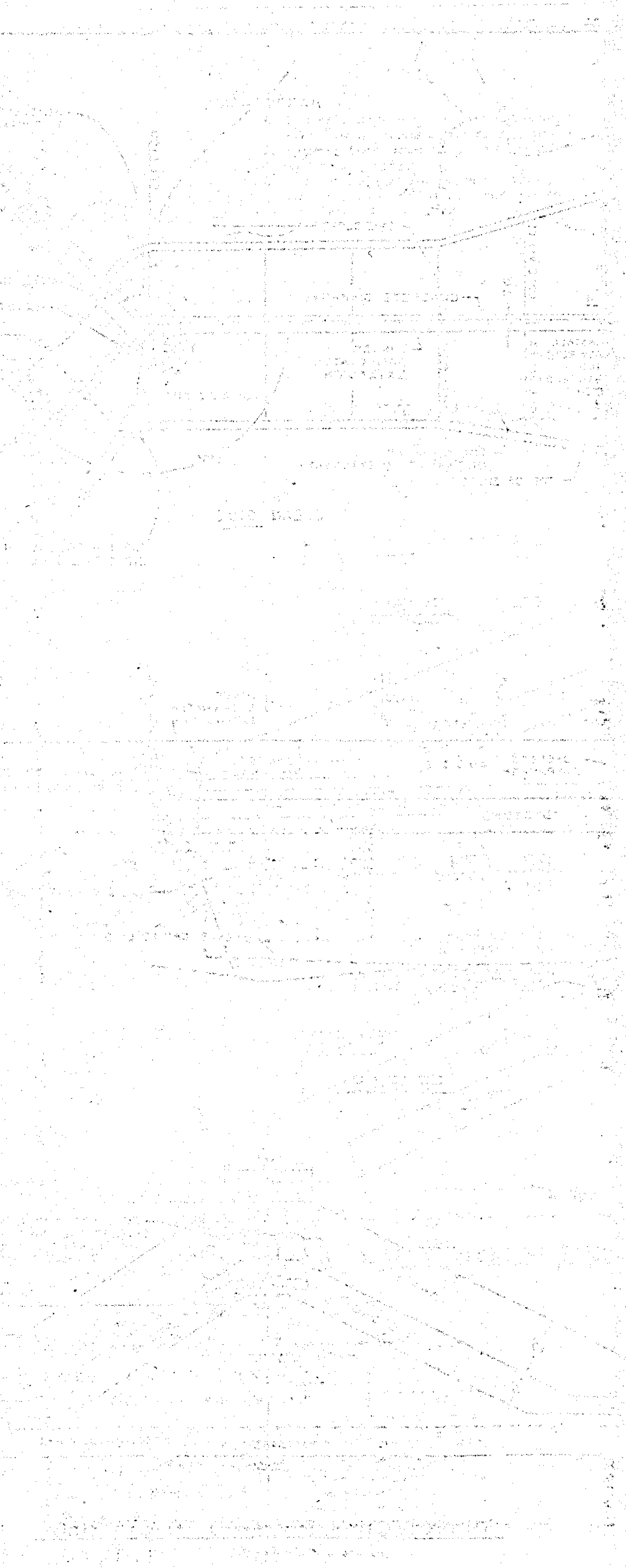
REVISIONS

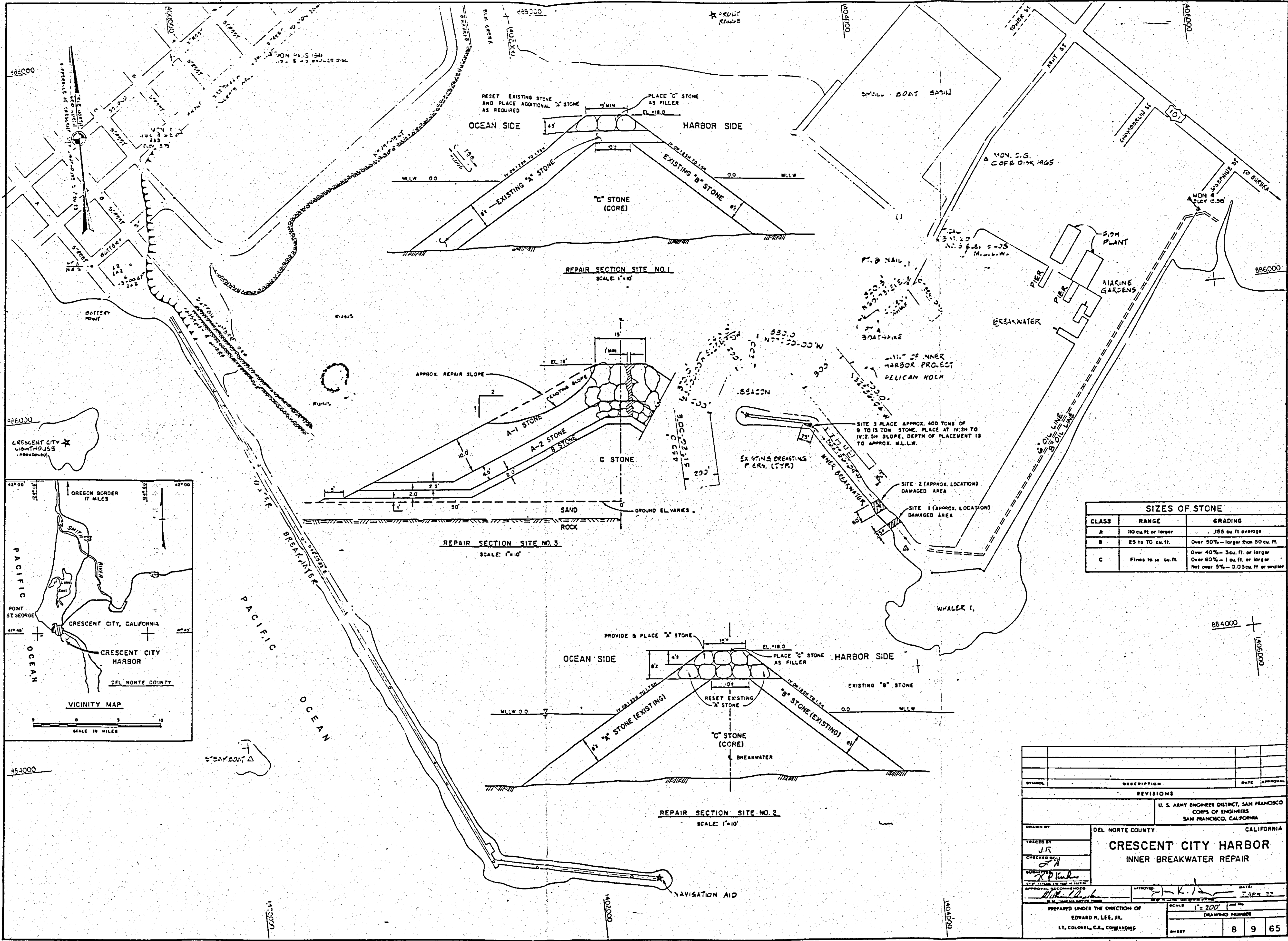
U. S. ARMY ENGINEER DISTRICT, SAN FRANCISCO
CORPS OF ENGINEERS
SAN FRANCISCO, CALIFORNIA

DEL NORTE COUNTY — CALIFORNIA

CRESCENT CITY HARBOR
INNER BREAKWATER EXTENSION
PLAN, PROFILE AND TYPICAL SECTIONS

DRAWN BY J.K.C. — CHECKED BY V.O.
DATE 1972 July 7
PREPARED UNDER THE DIRECTION OF JAMES L. LAMMIE, COLONEL, CE, DISTRICT ENGINEER
SCALE AS SHOWN
SHEET 3 — DRAWING NUMBER 8-9-58





SIZES OF STONE		
CLASS	RANGE	GRADING
A	110 cu.ft. or larger	.155 cu.ft. average
B	25 to 70 cu.ft.	Over 50% – larger than 50 cu. ft.
C	Fines to 14 cu.ft.	Over 40% – 3cu. ft. or larger Over 60% – 1 cu.ft. or larger Not over 5% – 0.03cu. ft or smaller

ATTACHMENTS

ATTACHMENT 1

COMPREHENSIVE CONDITION SURVEY
METHODOLOGY OF ARMOR STONE GRADATION

PURPOSE AND SCOPE

The purpose of this attachment is to summarize the methodology used for determining gradations of armor stone above water on coastal structures. The gradations are used to evaluate armored coastal structures and their ability to perform in accordance with current design criteria. This methodology entailed physical measurement of stone samples, photography of the complete structures, and determination of the size of stone from the photographs. It covers each structure from its head to its point of contact with the shore. Pertinent data used in the study were the size, shape, specific gravity, and weight of the armor stone. Grouted stone and stone not visible at the surface are not included in the gradation analysis.

INTRODUCTION

Armor stone surveys require methods that give consistent results of stone sizes on coastal structures. This study is part of the final phase of development for a standard method which would be employed to evaluate armor stone on all coastal structures. The first phase was carried out as part of the comprehensive condition survey for the Humboldt Bay Jetties and Crescent City Outer Breakwater. This phase, described herein, is used for the subsequent condition surveys conducted since July 1985.

The determination of gradations of large in-place armor stone can be a difficult and costly procedure. Currently, no attempts, prior to the development of the method presented herein, are known to determine gradations other than by physically removing and weighing individual stones. Removing and weighing individual stones would be prohibitively expensive and runs the risk of being unrepresentative if enough areas are not sampled. Stone gradation records of past construction and repair can be used to confirm results obtained, if they are complete and well documented. However, construction and repair records are often sketchy and vague. In light of this, the most effective basis for the evaluation method is aerial photography with ground control and sampling. Photography taken from boats has given comparable results.

On projects where stone gradation data may have been well documented during construction, the standard method would be further refined, if necessary, to obtain results comparable to known data. In every use of this method, data collection and sampling would be performed to arrive at correlations suitable for obtaining armor stone gradations at a particular project.

FIELD PROCEDURES

General. Aerial photographs of the armor stone were taken perpendicular to the plane of the structures slope. To interpret the photographs properly, ground control points were marked and premark angles for scale determination were placed prior to photographing. Also, three dimensions were measured, by hand, of selected samples of the armor stone at several stations.

Sample Measurements. As a control, three dimensions were measured on armor stones in selected sampling locations on the structures. More than ten stones were measured at each location on the structure(s). Dimensions on these stones, in mutually perpendicular planes, were measured with a tape and were noted as to which were visible when viewed from above. Large numbers were then painted on the stones for identification in the aerial photographs.

In order to obtain a shape-volume-weight relationship for stone, a sample consisting of 45 stones of known weight and specific gravity from the Los Angeles and San Gabriel Rivers was analyzed. Dimensions of these stones were also taken in three mutually perpendicular planes and the shape-weight relationship was established for the methodology.

Marker Layout. Occasional 100-foot intervals were marked with white paint along the structure. Premark rectangles were placed at representative locations on the armor stone slope to provide a reference for size comparison and scale determination in the photographs. The premark rectangles were constructed of two-inch diameter P.V.C. pipe to form a ten by twenty foot shape, and were marked with black stripes at two-foot intervals.

Photography. Photographs were taken using a 35 millimeter camera. Aerial photographs were taken perpendicular to the plane of the structure's slope to avoid excessive foreshortening effects. Overlapping, or panoramic photographs were taken at approximately 75-feet altitude resulting in an average photograph scale of about one inch to 20 feet.

DATA ANALYSIS

General. Using data of known stone dimensions, specific gravities, and weights, statistical analyses were performed to find correlations between the two dimensions that are visible from photographs and the actual weight of the stone. This required establishment of a relationship of the shape of a stone and its weight, with the dimensions of a stone measured in the photograph. Dimensions were taken from photographs of armor stone in the study area. When the established relationships were applied to these dimensions, the estimated armor stone gradation was found from photographed portions of the structure.

Theory of shape and weight relationships. The dimensions of a stone are defined as the outside dimensions in three mutually perpendicular planes, with Z being the longest, X the shortest, and Y the intermediate dimension (i.e. the dimensions of a circumscribing box).

Stone shapes may vary extremely, from cubical to spherical to flat or any combination thereof. Further, as discovered with the stone samples from the Los Angeles and San Gabriel Rivers, two stones with the same circumscribed dimensions may have significantly different weights. However, by using a sufficiently large sample of stones of known dimensions and weights, a "true" average relationship between three measured dimensions and actual weight can be determined. The steps involved were as follows:

(1) First, the actual gradation, by volume, of the stone sample was plotted using the known weights and specific gravity, where

$$V_a = W / (S.G.)(w)$$

where: V_a = volume
S.G. = specific gravity
w = weight of water = 62.4 lb/cf
W = weight of stone

(2) Then, the volume of a box of the dimensions of each stone was calculated by:

$$V_b = ZYX$$

where: Z,Y,X = the three stone dimensions

(3) Finally, a gradation was plotted using the volumes of these boxes, and a linear regression analysis was performed on a set of corresponding points from the two gradations. The linear regression analysis relates the box and actual volumes by the following:

$$V_a = 0.3V_b + 0.34 \quad \text{(V in cu. ft.)} \qquad \text{eq. 1}$$

Or by substituting, $V_b = XYZ$:

$$V_a = 0.3\,(XYZ) + 0.34 \qquad \text{eq. 2}$$

Theory of visible and actual dimension relationships. It has been observed that the "visible" dimensions measured from a photograph of a stone on a coastal structure often vary significantly from the "actual" dimensions of the stone measured in the field. This variance results from the foreshortening effect in stone faces not perpendicular to the line of sight of the photograph and from partial obscuring of one stone by another. The stone samples which were measured on the structure were identified from the photograph by a large painted number, and were used to determine a relationship between the "visible" and "actual" dimensions of a stone.

Since only two dimensions can be measured from a photograph, the intermediate, unknown, dimension is described as a function of the average of the largest and smallest dimensions, and a shape factor q and is represented by the following relationship.

$$Y = q\,(Z+X)\,/\,2$$

Substituting this relationship into equation 2 gives:

$$V_a = 0.15q\,(Z+X)\,ZX + 0.34 \qquad \text{eq. 3}$$

Using equation 2 above, the volumes were computed and a gradation plotted for the three dimensions measured in the field. Using equation 3 above, the volumes were computed and a gradation plotted for the two dimensions measured from the photographs. A linear regression analysis was then performed on a set of corresponding points from the two gradations. The linear regression results verified the following relationship which allows the use of a variable correction factor for different structures:

$$V_a = V_p\,/\,S$$

where: S = the gradation correction (shift) factor

V_p = volume obtained from the photograph measurements

Substituting this correction relationship into equation 3 yields:

$$V_a = 0.15\,(q/S)\,(Z+X)\,ZX + 0.34\,/\,S \qquad \text{eq. 4}$$

Obtaining the weight via specific gravity yields:

$$W_a = (\text{S.G.})\,\frac{62.4\ \text{lb/cf}}{2000\ \text{lb./ton}}\left[(0.15)\frac{q}{S}(Z+X)ZX + \frac{0.34}{S}\right]\ \text{cf}$$

Or:

$$Wa = (\text{S.G.})\left[0.00468\frac{q}{S}(Z+X)ZX + \frac{0.01}{S}\right] \qquad \text{eq. 5}$$

where: W_a = estimated weight of a stone, in tons, measured from a photograph.

q = $Y / (Z+X/2)$ = 'shape factor' representing the variation of actual intermediate dimension from the average of Z and X.

S = V_p / V_a = gradation correction factor representing variation of gradation obtained from photographs versus gradations obtained from the field.

Procedure. Short reaches of approximately 40 feet were selected at about 200-foot intervals along each structure. Each reach included at least 25 stones to ensure that a representative gradation could be obtained. The two visible dimensions of every stone was measured within each reach.

A table was constructed for each structure, where the stone sizes were compiled corresponding with the two visible dimensions. The weights were then computed by equation 5 above, using a visible to actual correction factor "s" and a shape factor "q" and the specific gravity typical of the particular structure. These weights were then compiled into representative gradations of the armor stone at each reach. These results, along with the average and range of values, are presented in the main body of this report.

ATTACHMENT 2

LAT: 41 45'N	INDEX NO.:	RIPRAP DATA SHEET	TESTED BY: SPDL
	LONG.: 124 10'W		DATE: August 1986

LAB SYMBOL NO.: CCBCS-WI-86 TYPE OF MATERIAL: Loose Rock Material

Project: Crescent City Harbor, Inner Breakwater Condition Survey

Authorization: DA Form No. E86-86-0113, dated 1 August 1986
Samples Received: 1 August 1986
Source: Whaler Island Quarry
Location: Crescent City, T16N R1W

PROCESSING BEFORE TESTING: None

GEOLOGICAL FORMATION AND AGE: Jurassic-Triassic Fransican Formation

TEST METHOD	RESULTS
BULK SPECIFIC GRAVITY, SSD, (CRD-C 107)	2.67
ABSORPTION, % (CRD-C 107)	0.40%
WT. AV. % LOSS, 5 CYC. $MGSO_4$ (CRD-C 137)	N/A
L.A. ABRASION LOSS, % (CRD C 145 OR RTH-115), GRADING 1.	N/A
UNIT WT., LB/CU FT (CRD-C 107)	
WETTING AND DRYING, %, 35 CYCLES	N/A
FREEZE AND THAW, % (CRD-C 144) 20 CYCLES	
EXPANSION IN ETHYLENE, GLYCOL (CRD-C 148)	

PETROGRAPHIC DATA (CRD-C 127) Meta Basalt (Greenstone)

Marcoscopic: Sample is greenish gray (5GY 6/1) on the fresh surface and mottled moderate brown (5YR 4/4) and greenish gray (5GY 6/1) on weathered surface. The texture is aphanitic with the principal constituents consisting of chlorite, quartz and plagioclase feldspar. The sample exhibits no weathering and is very hard and strong. Numerous sub-parallel veins bisect the sample with iron oxide as the principal mineral.

Microscopic: Sample consists of subparallel microlites and irregular masses of plagioclase feldspar of albite composition interlocked with irregular masses of chlorite, quartz, iron oxide, and clinozoisite. Pumpellyite occurs as needles within the quartz masses indicating metamorphism to greenschist facies. Numerous micro-fractures bisect the sample in random orientations and are filled with an aphanitic opaque material. This aphanitic material contains fragments of the surrounding rock and appears to follow the micro-fracturing pattern.

REMARKS

PHOTOGRAPHS

1. SAND BARRIER. The barrier from the Whaler Island towards shore. Note harbor facilities up to the crest and the area scalped of armor just above the car. April 1986

2. SAND BARRIER. The barrier from about the middle looking towards Whaler Island. Note smaller stone below about elevation +6 MLLW on the harbor slope and the irregularity of the slope. June 1970

3. SAND BARRIER. Groin on right constructed from armor scalped from end of barrier on the left. April 1986

4. SAND BARRIER. End of armor at station 21+95 and scalped slope on the left. April 1986

5. SAND BARRIER. Crest at station 9+00 with rock from harbor facilities on the right. Note the small depression in left center where the cres was not filled behind the original parapet wall. April 1986

6. SAND BARRIER. Crest at station 21+80. Greenstone on the right and graywacke on the left. July 1986

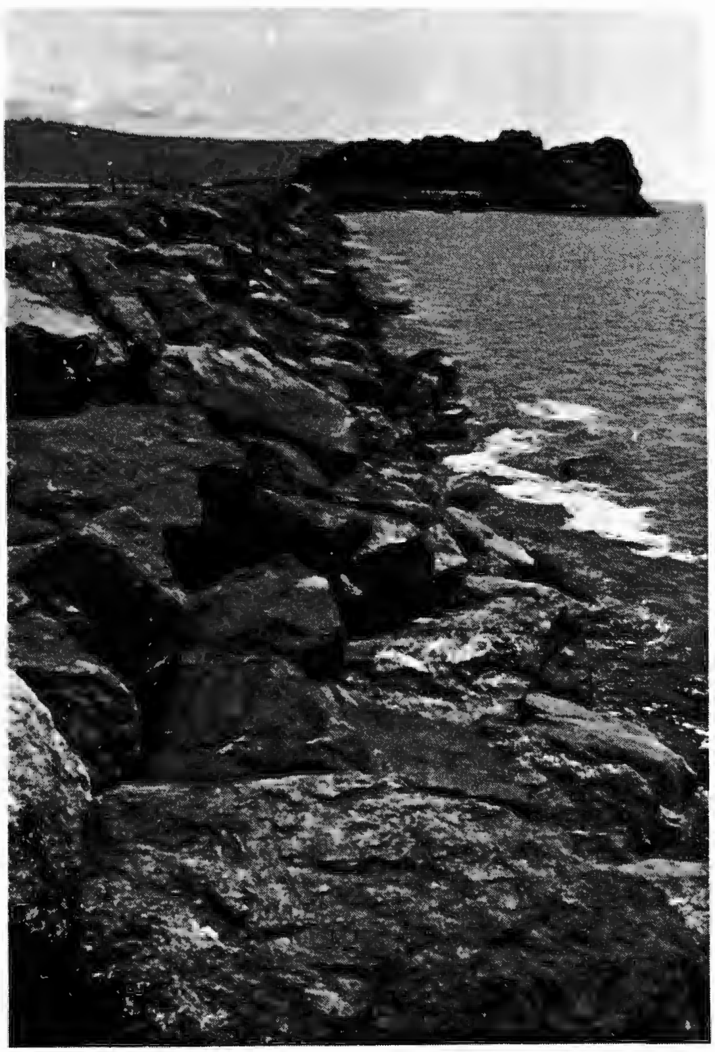
7. SAND BARRIER. Crest at station 21+50. Mottled greenstone armor surrounded by graywacke. Note the seams in the graywacke. July 198

8. SAND BARRIER. Split graywacke at station 14+95. April 1986

9. SAND BARRIER. Missing armor near station 6+00. April 1986

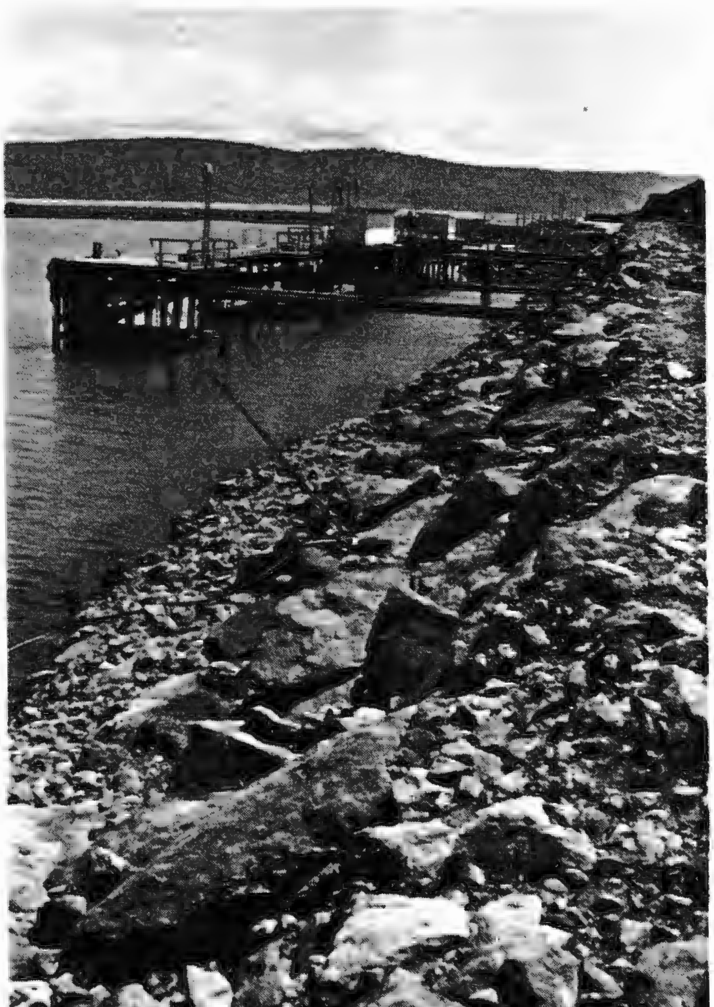
10. SAND BARRIER. Missing armor at station 7+85. Note rounded small stone in hole. April 1986

11. INNER BREAKWATER. Ocean side from Whaler Island. April 1986



12. INNER BREAKWATER. Ocean slope towards Whaler Island from station 10+00. Note smoothness of the slope. April 1986

13. INNER BREAKWATER. Ocean slope up station from station 10+00. April 1986



14. INNER BREAKWATER. Harbor slope down station from station 11+00. Note small stone "bench" near the water line. April 1986

15. INNER BREAKWATER. Ocean slope at station 3+00. Rock is Whaler Island greenstone. July 1986

16. INNER BREAKWATER. Gardner Ridge gabbro on the center line of the head. April 1986

17. INNER BREAKWATER. Ocean side of the diaphragm at station 11+25. Rock is McVay greenstone. April 1986

18. INNER BREAKWATER. Eroded crest against the ocean side of the diaphragm, from station 14+40 towards the head. April 1986