CHAPTER ONE HUNDRED EIGHTY SEVEN

A DETAILED MODEL STUDY OF DAMAGE TO A LARGE BREAKWATER AND MODEL VERIFICATION OF CONCEPTS FOR REPAIR AND UPGRADED STRENGTH

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ABSTRACT

Ten years after it was completed, and intact as originally built, West Breakwater at Diablo Canyon on the central coast of California was severely damaged during a wave storm in January 1981. The paper describes uncommonly detailed site investigations that followed and the development of a large three-dimensional hydraulic model for discovering the specific mechanism that precipitated the damage, and then for verification of the effectiveness of concepts for rebuilding the breakwater to resist greater storm events than had been used for the original design. Unique procedures for modelling contorted terrain, for producing reflection-corrected irregular wave systems, for eliminating abnormal waves at the start and at the end of test runs are discussed. The tested final concept for reconstruction is described and surveyed results of closely packed Tribar armoring, as reconstructed in 1983-84, are illustrated.

The authors conclude that investigations of problems involving wave attack on the termini of rubble mound breakwaters should always be undertaken with the aid of three-dimensional physical modelling unless owner and engineer are in a position knowingly to take large risks. Further, that physical modelling at suitably large scale is virtually mandated if the submerged terrain at a site is not regular.

INTRODUCTION

At dawn on January 28, 1981, the first five massive concrete blocks capping the seaward 150 feet of Diablo Canyon West Breakwater was observed to slide into the sea, in reaction to attack by a strong wave storm. Three of the other four sections, averaging 300 tons each, followed in quick succession and the fifth, now unsupported under much of its 600 square feet base at 13 feet above tidal datum, went in finally, about two months later.

The two breakwaters at Diablo Canyon Nuclear Power Plant were built in 1970-71 initially to provide sufficient wave shelter to enable

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construction of intake pumping facilities for the first two generating units at the site. Each of four sea water circulation pumps in the intake structure would deliver approximately 900 cubic feet of cold sea water per second (25.5 cubic meters per second). The rotating parts of the pumping equipment, single impeller vertical turbines, and the rotors of their electric motors would provide sufficient flywheel effect that quick suction-side variations of water level of 5 feet would be acceptable. Consequently it was determined by the project owner that the breakwaters need not provide their full sheltering effect except at such future times that construction might again be undertaken; that it would be economically appropriate to restore deteriorated breakwaters periodically rather than incur the added costs of construction to resist extreme storms of infrequent occurrence. With parts of West Breakwater extending to depths as great as 70 feet on an unprotected coast, the breakwaters that were built were massive. As shown by Figure 1, West Breakwater's trunk slopes at 2.25:1 on the seaward side and is armored by Tribars weighing 21.5 tons each that abut on concrete crest blocks 7 feet thick by 21 feet wide. Their upper surfaces are 20 feet above datum. The sheltered side of the trunk slopes at 1.5:1 and is armored by 36.8 ton Tribars. The conical terminus slopes at 3:1 and was armored by the smaller Tribars. The profile, at 20 feet above Mean Lower Low water, was not high enough to prevent heavy overtopping and the designs were developed so as to survive such overtopping during the ordinary heavy storms of each winter. The original project was described in detail by the senior author in a paper published in 1977 (4).

The design engineer, on site during the latter hours of the damaging storm, identified for the owners three fundamental questions that needed to be answered:

1. What is the extent and significance of the damage done by the storm?

2. How was the damage done; what was the mechanism of the damage?

3. What could or should be done in a reconstruction effort?

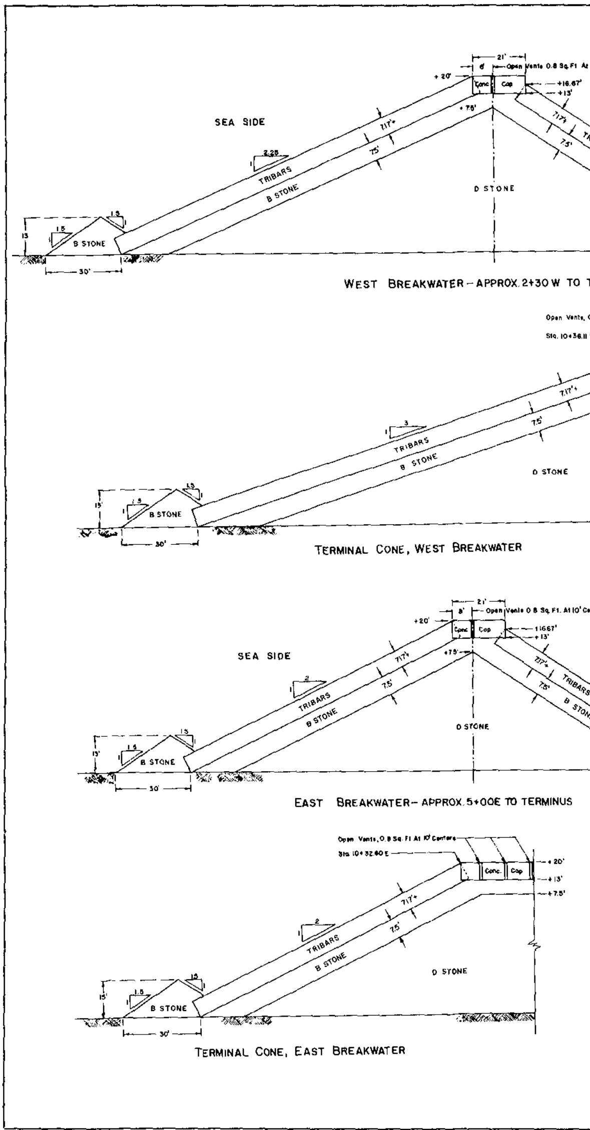
It was pointed out and accepted, that until there were solidly based answers to the first two questions, the third one at that time could only be answered by speculation. That would be unworthy, and attention therefore was initially concentrated on the prerequisites. Instructions were issued, that the answers to all three questions be pursued.

SITE AND CONDITION SURVEYS

Towill, Incorporated, with long experience at surveys in the marine environment, was engaged to provide detailed maps of the damaged areas of West Breakwater and of the location and configuration of debris shoals from the damaged areas. They also were to identify, by serial numbers that had been cast into the concrete, and to locate precisely in three dimensions all Tribars that remained intact in their original positions. Towill in turn acquired the services of engineer

Figure 1

ORIGINAL CROSS-SECTIONS, 1970-71
(Ref. 4)



DIMENSIONS

Tribars	7.17 Ft. High
B Stone Zone	7.50 Ft.
Cap Block	7 Ft. x 21 Ft.
Cap Vents	At 10 Ft. Ctrs.
Crest Elevation	+ 20 Ft.
Datum	MLLW.

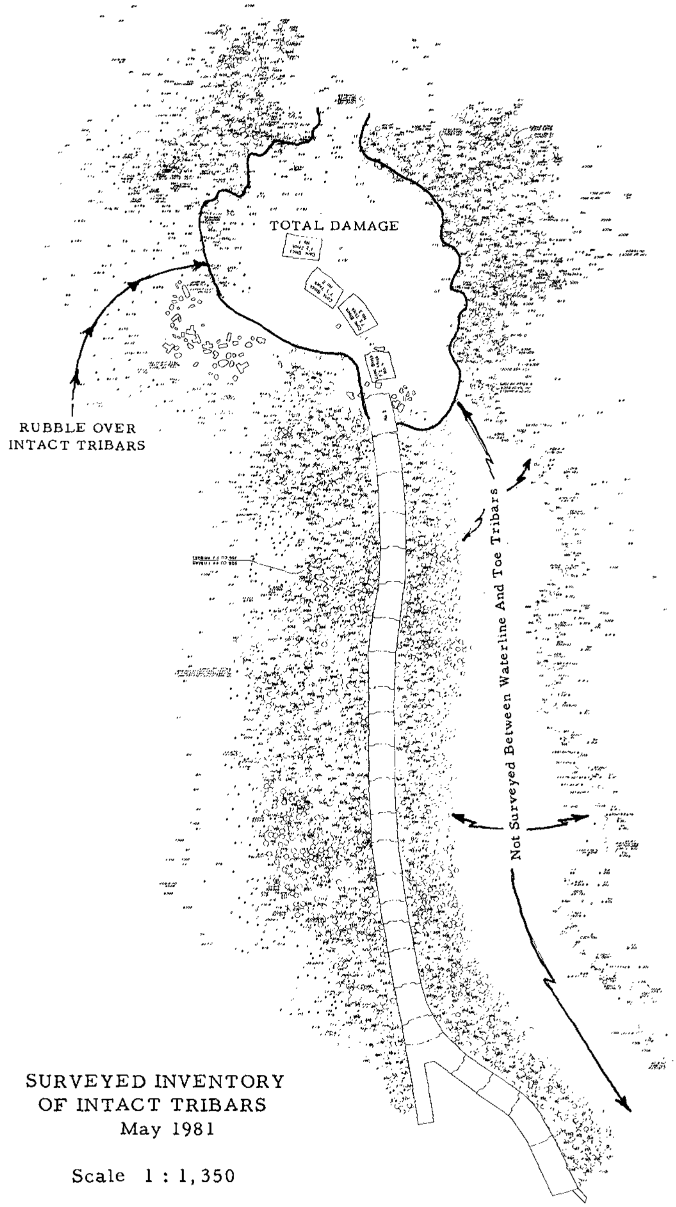
divers from Ames and Associates, Consulting Engineers, to assist in the Tribar inventory surveys. The center at the top of each leg of each Tribar was located horizontally by the intersection of lines of sight from two transits. The elevation of the top center of each leg was derived from vertical angles read from the transits, which had made their observations on a painted band on a pole held vertically by a swimmer at the water surface, while his partner, in diving gear and with cable communication with one of the transmitmen, held the pole at the required point on each leg of each Tribar. On the seaward side, landward of the damage, only the toe Tribars and the Tribars above water were surveyed. Figure 2 is reduced from a part of the resultant "map inventory" of the intact Tribars. It was of great interest, that the map showed virtually no damage had occurred below the breakwater armor zone region at 15 to 20 feet below Mean Lower Low Water. The map also showed that the debris from the damaged areas above about -15 feet had not been carried into the entrance channel between East and West Breakwaters. It had all been dropped by the waves on the channelward slope of the West Breakwater's terminal cone.

When original construction of the breakwaters was under way in 1970 and 1971, and during the nearly ten years that passed before West Breakwater was damaged in January of 1981, there had been wave behavior that suggested the presence of bottom terrain features that the available hydrographic maps did now show or, if suggested by the maps, were not delineated with enough detail for adequately explaining how they affected the waves. Towill, Inc. was therefore also asked to map the ocean floor out to the 110 feet depth contour, designing their survey coverage appropriately to define contours at two-feet depth intervals, at a horizontal scale of 1"=20' (1:240) within 1,000 feet West of West Breakwater and similar radial distances from its terminus, and at 1"=100' (1:1200) the rest of the way to the limiting 110 feet depth contour. Obviously they could not resurvey the ocean floor where the breakwaters had already been built, and it was not deemed necessary to do new mapping of the intake basin, so the maps that finally were compiled were composites made up from the new and densely covered areas West and South of West Breakwater, and the earlier surveys which had been done in 1967 by the same firm. Figure 3 is a reduction of the composite map. It represents sea floor terrain over a width from East to West of one mile, or 1600 meters, and a width at its greatest of 0.7 miles, or 1100 meters. The sea floor is essentially free of sediments above elevation -70 feet and the details of its features are extremely contorted and abrupt in relief. A massive rock mound referred to as "The Wash Rock", just 350 feet West of West Breakwater, proved to be so steep that two-feet contours could not be drawn at its flanks as separately distinguishable lines, so there the contours are drawn only for each 10 feet.

THE STORM, IN DEEP WATERS

With the peak of the storm of January 28, 1981 occurring in the early morning hours, there were no qualified engineers on site at that time to note technical details of the wave attacks. Hindcasts from meteorological data were acquired therefore from marine meteorologist R. Rea Strange III. He described the deep water wave system offshore

Figure 2


TOTAL DAMAGE
RUBBLE OVER
INTACT TRIBARS
Not Surveyed Between Waterline And Toe Tribars
SURVEYED INVENTORY
OF INTACT TRIBARS
May 1981
Scale 1 : 1,350

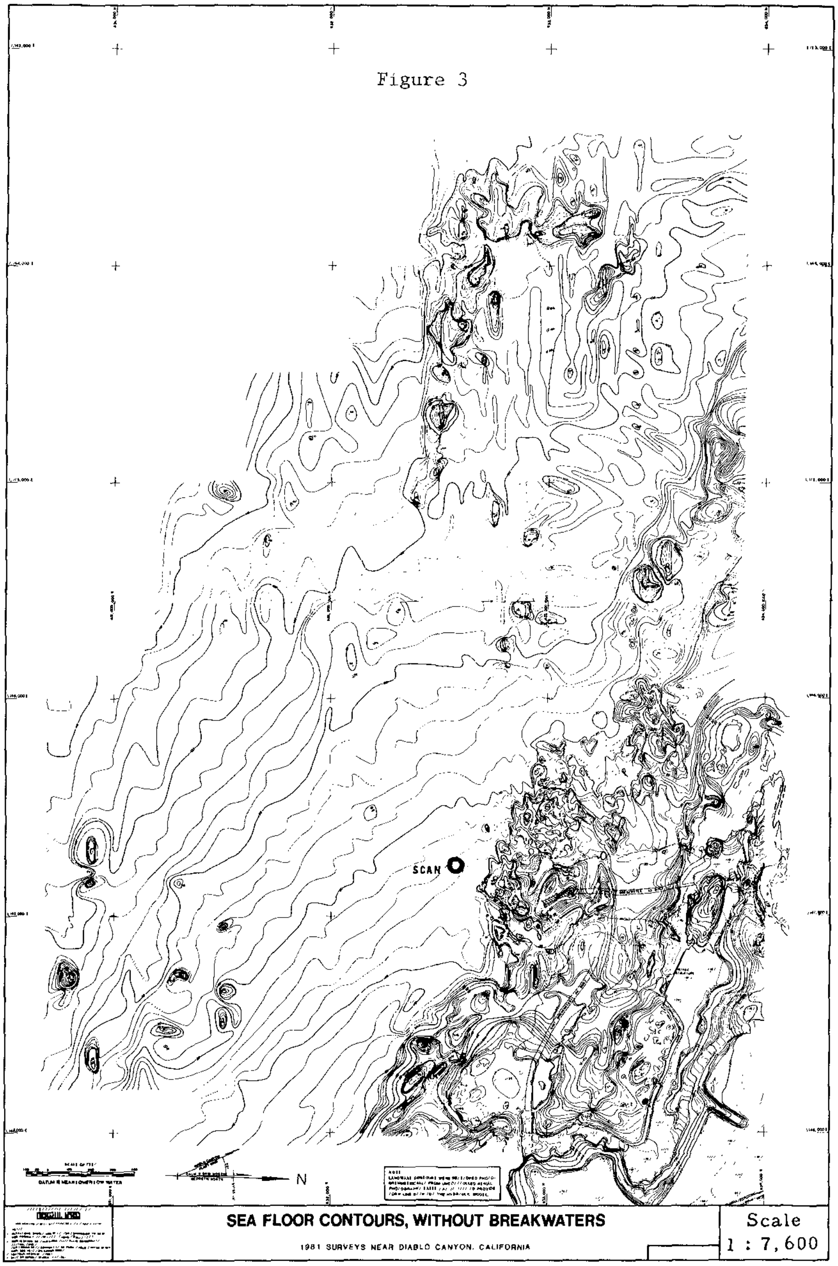

SCAN
N
SEA FLOOR CONTOURS, WITHOUT BREAKWATERS
1981 SURVEYS NEAR DIABLO CANYON, CALIFORNIA
Scale
1 : 7,600

Figure 3

as being composed of seas that were being generated by a local storm, with winds from azimuth 225°, which for 12 hours had "significant" heights of 13.5 feet, and of swells from a storm 900 miles distant at azimuth 260°. The swells rose very abruptly after the seas had begun to drop. The maximum significant height of the combined swells and seas was 20.8 feet; the peak of the combined spectra was 17.5 seconds. The significant height of the swell component was 19.9 feet, of the seas, 6.1. The periods, respectively, were 17.5 and 8.5 seconds. About two weeks after submitting his hindcasts, Mr. Strange discovered the existence of a wave data buoy, anchored in deep water by the National Oceanic and Atmospheric Administration at a location only 18.6 nautical miles from Diablo Canyon Site. Apparently the buoy had recorded the waves on January 28, 1981, without any lapses. The investigators acquired duplicate magnetic tapes of the data transmitted by the NOAA buoy, number 46011, that had been recorded at its shore station in Mississippi. They were not raw data, so a factual water surface time profile could not be taken from the tape. Instead, the tape had spectral data which had been refined at the shore station from partially processed accelerometer records compiled by a mircoprocessor aboard the buoy. The raw data were processed aboard the buoy for each 20 minutes of record. They were sampled each two-thirds of a second. When the twenty minutes of partially processed data were transmitted to shore by radio satellite, records for the next 20 minutes were processed on the buoy, eradicating what had been there before.

Figure 4 shows the hourly variations of the NOAA buoy's spectral energy data, by separate frequency bands, through the two days of the storm. The presence and persistence of energy in the lower frequencies is worthy of note. Figure 5 breaks the energy data down to show the variations during the storm's strongest hours of the significant wave height (H_{33}) as calculated from total energy, of the period at the peak of the spectral energy, of the deep water wave length for the peak energy component, and of the variation of H_{33}^2L, a characterization of the energy per wave in the deep water condition.

The investigators had no basis for confirming in detail how well the accelerations of the NOAA buoy were interpreted to yield the energy data it preserved, so no conclusions could be reached as to the comparative qualities of the buoy's data and the hindcast data. If all else should be equal, there were advantages in the buoy recordings in their being available at 20-minute intervals instead of the 3 hours for hindcasts, and they did of course give clear indications of the time of rise and fall of the storm. They could not, however, provide any information as to the directions of the waves. The hindcasts did provide that information.

THE STORM, IN SHALLOW WATERS

The NOAA buoy was located in 133 fathoms depth. Thus, with normal criteria considered, its recordings were deep water data for all wave frequencies above 0.055 Hertz (T=18 seconds). Hindcasts also yield deep water data, so it was necessary to determine the adjustments that should be made to the deep water storm descriptions in order to describe the waves at Diablo Canyon site that had damaged West

Figure 4

GROWTH AND DECLINE OF ENERGY
WITHIN SPECIFIC FREQUENCY BANDS
AT BUOY 46011
JANUARY 28-29 1981

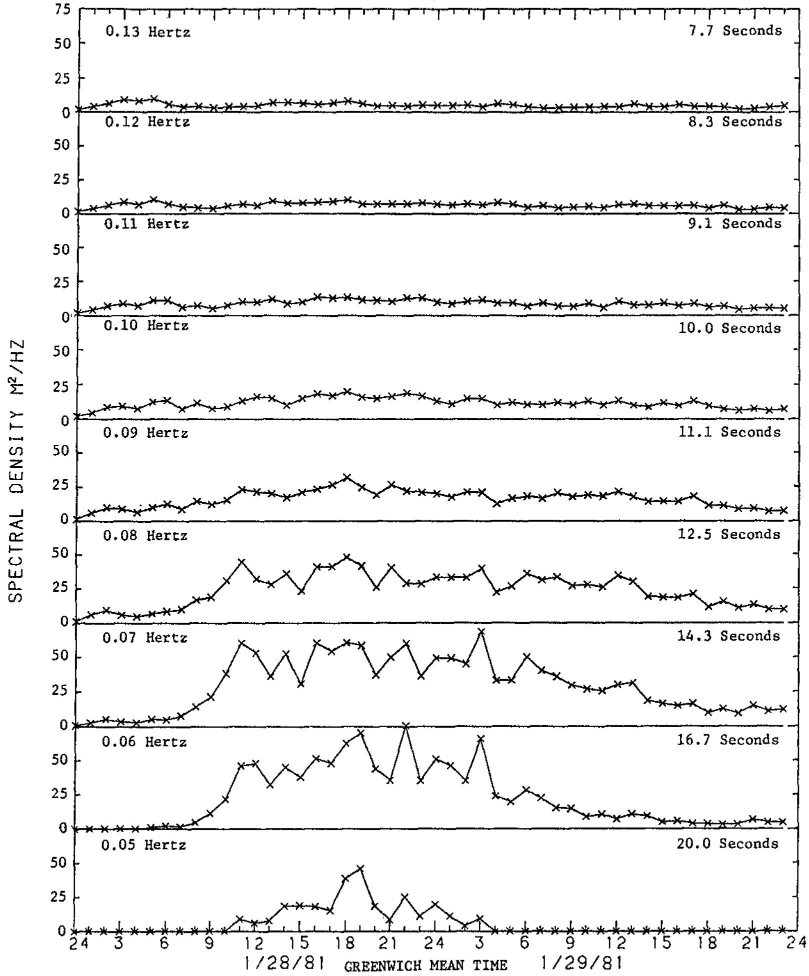
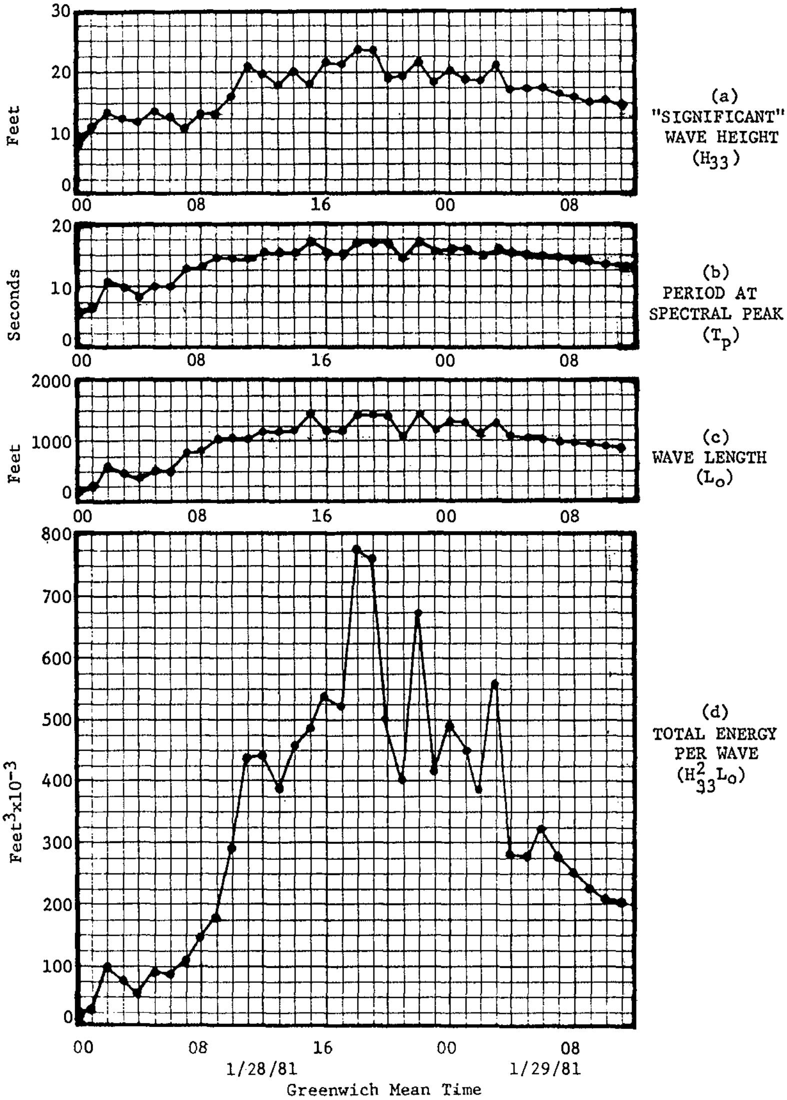


Figure 5

WAVE TRAIN CHARACTERISTICS AT NOAA BUOY 46011
DURING PEAK OF STORM ON JANUARY 28-29, 1981



Breakwater. To enable making the appropriate adjustments, to deep water waves of any period from any azimuthal direction, a numerical model of the ocean floor was created and used with a refraction program elaborated by Dr. R.C.Y. Koh at California Institute of Technology from concepts developed by Professor Fredric Raichlen and published in 1970 (1). The model extends 15 miles from the coast out to the 206 fathoms depth contour and reaches from 35 miles downcoast from Diablo Canyon to 6 miles upcoast. Elevation data were entered at grid intervals of 250 feet between the coast and about the 40 fathoms curve. Between the 40 fathoms contour and 75 fathoms, data were entered for each 500 feet, and seaward from the 75 fathoms depth the elevations were stored for each 1,000 feet. Twenty thousand such points were input from manual takeoffs, using the new maps by Towill Inc. close in and several of the "smooth sheets" from the mid-1930s archives of the United States Coast and Geodetic Survey for the rest of the modelled area that would be used for wave refraction calculations. Seventeen thousand more elevations were computer interpolated between the input points and stored to define a "fine grid" of 250 feet mesh out to the 40 fathoms area and to define a coarse grid of 500 feet spacing for the rest of the model. Unused areas of a rectangular outline enclosing the above also had to be stored in the model, so pseudo elevations were used for such points. Thus the model consequently contained 109,000 data points. Figure 6 provides a locality map showing the Diablo Canyon Site between Point Buchon at the North and Point Arguello at the South, the NOAA buoy and the physical extent of the numerical model.

Fan diagrams were used, with a ray emanating from each 2.5 degrees of azimuth at an arbitrary point named SCAN at 75 feet depth, close to West Breakwater, to discover the deep water locations from which to propagate waves of selected deep water directions at various periods or frequencies that would bracket the coastal site. The location of SCAN is shown on Figure 3. With tabular as well as graphic output from the program to work from, representing calculations for over 500 wave refraction rays, a matrix was compiled that can be interpolated to give the refracted energy or height and the refracted azimuth at SCAN of waves of any deep water period up to 22 seconds with any deep water azimuth between 180 and 300 degrees.

With use of the numerical refraction model and accepting the hindcast estimates of direction in deep water of the storm waves, the January 28, 1981 recordings by the NOAA buoy were transformed to represent shallow water waves at Diablo Canyon Site. Figure 7 shows both the deep water spectra and the spectra of the refracted waves at the 100 feet depth locality. The data for the 100 feet depth condition were used later for programming random wave generation in the physical hydraulic scale model that is described following.

THE HYDRAULIC MODEL

The extremely contorted submarine terrain that has previously been shown on Figure 3 was a major contributor to the assumptions that complex three-dimensional wave phenomena are present at Diablo Canyon Site that would be best reproduced by competent three-dimensional physical modelling. A new basin to hold such a model was built in Escondido,

Figure 6

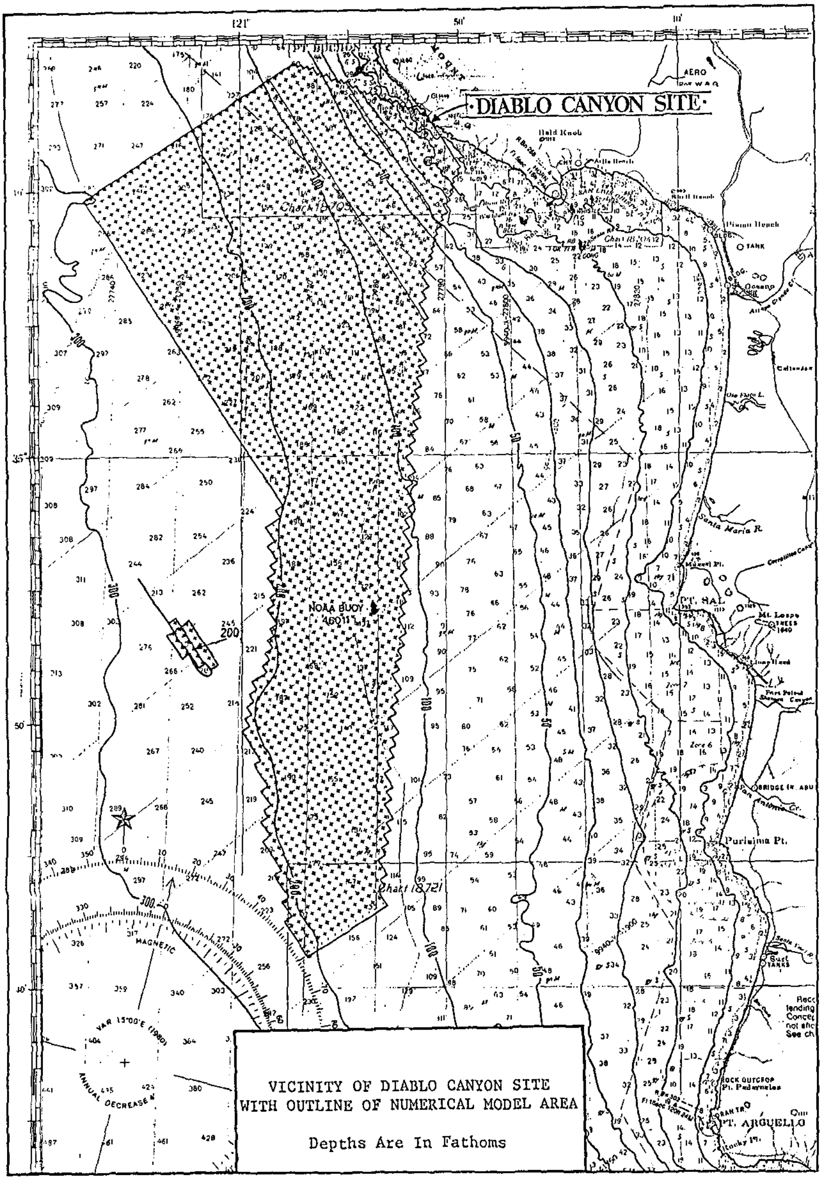
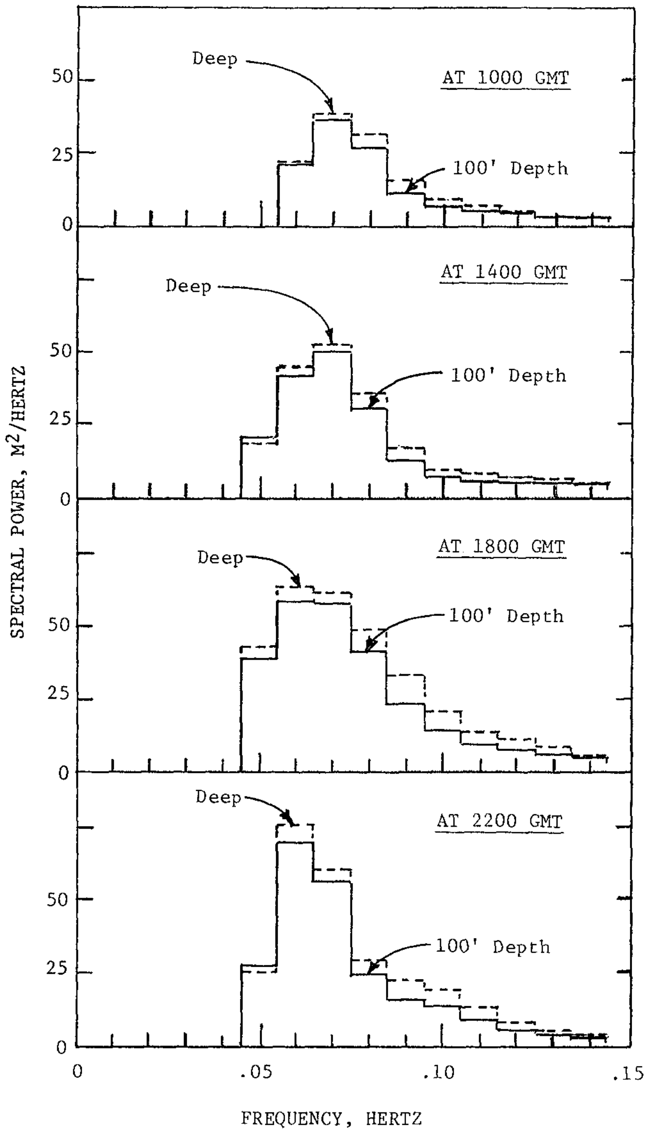

DIABLO CANYON SITE
NOAA BUOY 46011
PT. SAL
Purisima Pt.
PT. ARGUELLO
Santa Maria R.
VICINITY OF DIABLO CANYON SITE
WITH OUTLINE OF NUMERICAL MODEL AREA
Depths Are In Fathoms

Figure 7

WAVE SPECTRA IN DEEP AND IN SHALLOW WATER
FROM NOAA BUOY RECORDS ON JANUARY 28, 1981



California, alongside Offshore Technology Corporation's existing facilities. With a model scale of 1 to 45, the terrain above the 100 feet contour that is mapped on Figure 3 was fitted into the 80 feet by 120 feet basin. The basin's walls were at the edge of the Figure 3 map.

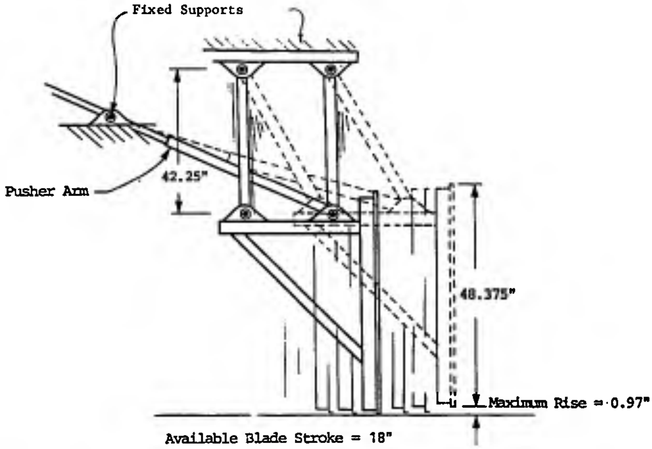
Several known techniques for molding the terrain accurately in the model were considered, but the great irregularity of the contours and the firm commitment to make the model as accurate a representation of the mapped features as possible, ruled those traditional methods out and a unique approach was developed. First, photographic enlargements of each 150' x 200' rectangular extent of the maps on sheets 40 by 54 inches in size were printed at 1:45 scale. Those sheets then were glued as a controlled mosaic on the floor of the model basin. Along every contour, typically at about eight-inch intervals, angle-iron clips were fastened to the floor with power-actuated stud drivers, the clips being so placed that their standing legs were tangent to the contour line. Galvanized steel strips, or "ribbons", were then bowed to follow each contour on the map and were spot welded to the angles after a surveyor had verified, by differential levelling, that the upper edges of each of the ribbons were positioned accurately to scale at the elevation defined by the contour. At single points representing peaks or hollows in the terrain, or at features where contours were too closely spaced or too tightly curved to leave room for the angles and ribbons, 10-inch galvanized spikes that are sold for eaves gutter fastening were driven to prescribed model elevations so they could be used like grade hubs. With the ribbons and spikes properly in place, sand was laid on the basin floor up to within 2 or 3 inches of the surfaces represented by the upper edges of the ribbons and the spikes, and then the remaining 2 or 3 inches were filled with sand-cement concrete and shaped to the contour controls by trowelling. Figure 8 shows the angles fastened along contours of the mosaic, steel ribbons temporarily

clamped to the angles, surveyors confirming the accuracy of the clamping and technicians with a portable spot welder fastening the ribbons permanently in place.

Four existing wave makers, each 11 feet long, were re-built to a modified configuration and a fifth was built to the new design, in which the moving blade's suspension was an articulated parallelogram support that caused the blade to remain aligned vertically throughout its stroke rather than swinging through a long radius arc from a single axis, like a door on edge. The re-built linkage is shown schematically by Figure 9. Screw jack castered wheels were included that could be

Figure 9

ARTICULATION OF WAVE BLADES



cranked down to make moving the modules relatively easy. Thus various alignments of the wave machinès could be achieved for propagating waves across the model terrain toward the breakwaters. The five modules were aligned with a straight front and their blades were moved identically and simultaneously to produce continuous wave crests 55 feet long. Perpendicular to the 55 feet long blade at each end a wave guide fence that self-adjusted to profile features was extended toward the area under test, to control losses of wave energy laterally from the waves. Wherever reflections were seen that were caused by model boundaries but would not occur at the coastal site, open barred cages filled with lathe shavings of stainless steel were deployed to absorb the energy that otherwise would be reflected.

The wave blades were moved through the water by two-way hydraulic cylinders that tapped an open loop of a circulating oil supply at near constant pressure. Servo valves controlled the displacement time-history of the blade and these valves functioned in response to a voltage time-history that was constructed from spectra, either hindcast or measured, using the method proposed in 1970 by Goda (2). The variable voltage signals were recorded and played back during wave generation on conventional C-60 audio tape cassettes.

The testing program was devised to determine the effects on the model breakwaters of either uniform waves or of trains of random waves of equivalent energy. Except for the largest wave periods, where six or seven waves was the limit, the uniform waves were sent against the model breakwaters in bursts of nine and the machines then were stopped until the basin was suitably calm again. Then another burst of nine would continue the test. Two interesting techniques that had evolved in previous studies at California Institute of Technology were used with these uniform waves that were important improvements over traditional ones. First, instead of sinusoidal waves, which would not be a stable profile for gravity waves in depths as shallow as 100 feet, the variable voltage signals to the wave makers were programmed so that cnoidal wave profiles were produced within very short intervals of travel away from the blades of the wave machines, usually before those waves began to cross the modelled sea floor terrain. The other innovation eliminated a nuisance that historically has confused interpretations of model experiments. The first wave or two and the last one in a test run are almost always abnormal. Often the last wave has been compounded by reflections off of the blade and can be a high energy "rogue" that can do damage that the preceding train's series of prescribed waves had not been capable of doing. A simple concept was employed that essentially eliminated these first and last wave anomalies. No matter what phase of generating motion the blade might be in when a test run was completed, and a halt command was sent, sensing circuitry automatically delayed the effect of the halt instruction until the blade had completed its programmed traversal to its fully retracted position, and only then did it stop moving. At that phase of wave generation the blade's translatory velocity is zero; extraordinary waves do not result and start-up for following sets of waves is smooth and free of the lurching that is frequent when reactions to instantaneous demands of the system occur for motion.

Capacitance type probes were utilized to sense wave heights at various separate locations within the basin. The capacitance probes were essentially copper rods which had been insulated by continuous heat shrink tubing. As a probe becomes immersed in water, a change in capacitance is sensed between the copper rod core and the surrounding body of water. This change in capacitance is sensed by an electronic circuit which transforms it to a change in voltage which then is correlated with different degrees of immersion so that the change in water level due to the passing of the wave is sensed and related to wave height. As many as sixteen wave probes were deployed in the model basin at selected locations in different runs. They were mounted on tripod stands with adjustable legs that facilitated placement in various water depths and over the complex bottom topography. To relate the signal from a wave probe to the change in water level each probe was frequently calibrated by use of centrally controlled motors that drove them up and down in the water for known amounts of displacement. Information on the voltage reading of the probe and the displacement was supplied to a Tektronix 4052 computer which calculated the calibration value for each wave probe as a function of voltage and displacement. The results were highly linear and stable with time.

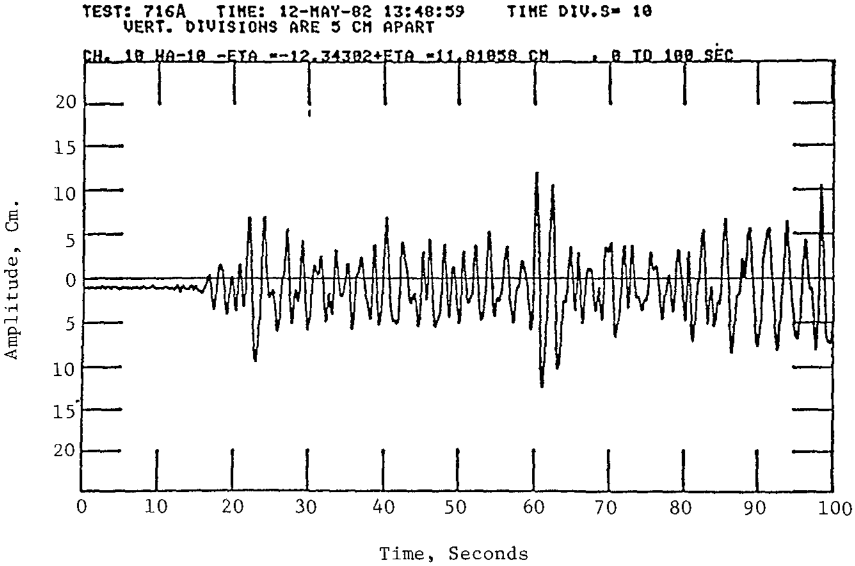
Measured wave heights were compiled by digital sampling, utilizing the Tektronix 4052 computer. Up to 16 channels of analog wave information were converted to digital form and were input to the computer. Once detected and processed by computer software, the information could then be permanently stored on floppy disk and also be presented in a hardcopy form for evaluation, through a graphics printer. The data were taken in unfiltered form and sampled at a rate of 10 Hertz. Lengths of test records were limited to the maximum number of samples that could be stored in the memory of the computer. A total of 128,000 samples could normally be stored using 12-bit format, limiting the maximum duration for compiling a record of a wave train to approximately 13 minutes of a test. With a slightly less accurate 8-bit format 256,000 samples could be stored, which made it possible to record as much as 26 minutes of testing.

Standard data that was output for both tests with regular waves and tests using irregular waves included wave time histories at each of the wave probes and basic statistical information on the waves that were measured including significant wave height and peak wave height. An example of a series of waves recorded during a test is shown by Figure 10, along with statistical information derived by the computer during the test. More sophisticated analyses that were also made from the probe records included presentation of actually measured wave spectra and the decomposition of the wave signals from two or more of the probes into incident and reflected wave components.

All relevant wave data were stored in computer compatible form for easy retrieval and analysis. Data were read to the Tektronix 4907 floppy disk memory unit, which transcribed the information onto 8 inch soft disks. The data were stored in an uncalibrated form with calibration factors stored in a separate header. The calibration factors were stored with each test run, rather than in one file only, to minimize or avoid any wrong calibration factors inadvertently being applied to

Figure 10

SPECIMEN RECORDING OF MODEL WAVES



the raw data.

When test runs were made that used random waves the succession of wave bursts and intervening rests that were used for the uniform wave tests was not acceptable. The random wave sequences ran for tens of model minutes. With periods at peak energy of the random waves being 12 to 14 prototype seconds, which would be 1.8 to 2.1 seconds in the model, roughly 300 waves would be generated each ten minutes in the model. Such prolonged episodes provided considerable opportunity for wave reflections to build up between the model breakwaters and the wave machines. Routines were devised to adapt concepts to a three-dimensional wave basin that were described by Goda (3) for two-dimensional wave tests in flumes, for separating reflected wave components from the incident. The procedures involved comparing the histories of the water surfaces at two probes closely located one to the other along the direction of wave motion and near the wave machine, determining the separate incident and reflected spectra by harmonic analysis and computing adjusted signals to send to the servomechanisms to produce the effect of the desired net incident wave spectrum.

An office trailer placed alongside the test basin housed all central electronic gear and control stations, as well as computers, plotters, printers and physical storage of records. Video recording terminals and monitors, with titling equipment and cameras, both video and film, standby equipment, etc., were housed in a smaller portable building alongside. Two-way speaker-microphone stations were installed to cover the whole model basin and in both portable buildings, so that all personnel could hear or contribute to voice communication.

Catwalks and booms, adjustable for height and rotatable from a mainmast to any location were cantilevered from the mast at the basin's edge. Fixed video and film cameras were mounted which could be activated remotely. All test runs included a recorded running commentary by the test engineers that later was transcribed. Parts of the commentary also were on the video cassettes that were recorded continuously during every test run. Parts of selected tests also were recorded on moving picture film exposed at 128 frames per second. When exposed at normal projection speeds, the moving images slowed to approach full-scale time of wave motion. That made more acute observation possible of fast acting events that had occurred in the model.

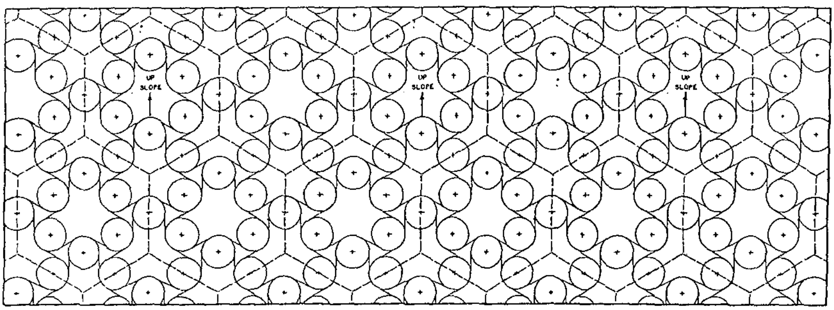
The model breakwaters were built over the molded sea floor terrain in close compliance with the original design. The core in the model breakwaters was made of coarse sand that was generally larger than the 1:45 linear model scaling of the prototype material would call for. That was deliberate, so viscous effects with smaller grain sizes would not distort the experimental results. The intermediate zone between the core and the Tribar armor layer, either B Stone or E Stone, was modelled to scale of volumes, i.e. $1/45^3$. The gradation called for the median stone's volume to be twice the volume of the minimum piece and the maximum stone to be twice the volume of the median. In either B or E Stone, the median size was 10 per cent of the volume of the Tribar armor piece. That gradation produces a voids volume approximating 45 per cent of gross volume occupied by the mixture. With nominal diameters of the scaled-down stones ranging between 2.1 and 3.3 centimeters for the smaller B Stone and between 2.5 and 3.9 centimeters for the E Stone it was determined that viscosity effects would be unimportant so it was scaled linearly. The model B and E Stone mixtures were made from crushed rock of appropriate specific weight. Proper shape and size distributions were obtained by hand selection and weighing each individual stone and classifying into two bins, using median size as the separation. Then equal weights of the two classifications were combined and mixed to produce a practical model gradation of the specified stone.

The model Tribars were cast under tightly controlled procedures in RTV molds, each of which was discarded after about 80 castings had been made from it. Thermal-setting resins were used to which industrial grade barium sulfate (barite) was added to obtain the correct specific gravity. Liquid catalyst was added to harden the pour before the finely divided weighting mineral could settle out of the honey consistency resin. Air that was entrained during the mixing of the parts was extracted from the catalyzed mixture under a vacuum bell jar before the mixture was poured into the molds. The heat of the catalyst is ineffectual, however, where the mold and the resin interface, so external heat is necessary to harden the resin there. To provide the surface heat the molds were first placed in controlled temperature ovens and then were removed just as the resin-barite-catalyst mixture was ready for pouring. Minimal flashing that resulted was removed from the castings after they were taken from the mold. After the hardened pouring chimney material had also been ground away each of the Tribars was weighed and one in ten was tested for specific gravity. No pieces were overweight and the specific gravity was almost without exception on target. None of the Tribars of 296 cubic feet size was as much as 3 per cent off on weight and the pieces of most accurate weight, ranging between 0 and 0.14% underweight, were code painted blue and were used in the model where surveys had shown removals by the January 28, 1981 wave storm. Those that were underweight by between 0.14 and 1.00% were painted green and were placed adjoining the historically damaged zones. The rest, sometimes painted black but mostly left in their unpainted milk chocolate brown color, were used elsewhere on West Breakwater beyond expected aggravated damage and on East Breakwater.

Figure 11 illustrates one of two possible geometric patterns for placing Tribars that represents the so-called 100 per cent pack. All pieces are in juxtaposition. In that configuration, one Tribar occupies $5.008d^2$ units of area, d being the common diameter of each cylindrical leg or spoke in the Tribar. The theoretical perfect pack, at

Figure 11

GEOMETRICAL ARRANGEMENT OF TRIBARS IN 100% PACK



100 per cent, cannot be achieved in actual construction but tight packing is an important objective for a Tribar armor zone. With close fitting, each Tribar is restrained by its abutting neighbors from tilting out of the "fabric" if it becomes unstable due to uneven support by the stones underneath and suffers displacing forces from the moving water. With the possibility under consideration that reconstruction specifications might require the builder to document achievement of a 90 per cent pack, the model breakwater armoring was placed to approximate 85 per cent, a consciously conservative modelling provision. Limits were also imposed on how much the legs of one Tribar could extend above or below those of an adjoining one, and as to how far the plane defined by the

ends of the three legs of any Tribar could tilt from the surface slopes defined by the design cross-sections.

A fast hardening and water resistant material named "Duracal" by one of several firms marketing it, and normally used for thin patches on deteriorated surfaces of concrete slabs, curbs or steps, was used for casting the model crest blocks. Barite was blended in to bring the material to appropriate density. The cap blocks were cast in place on the model breakwater, with a double layer of woven plastic screen cloth overlaid with a paper towel sheet being used to keep the simulated concrete from intruding the voids in the quarrystones that support the crest block. In the prototype, chain link fencing mesh had been used, with asphaltic felt paper as an overlay, for this same purpose. After the blocks had hardened and their side forms had been taken away, each block was carefully lifted out, the mesh and paper towel membrane were removed, the tops were sanded smooth and the vent holes specified in the design were drilled. Each block was then put very carefully back in its original place. Each time that the breakwater was restored, in preparation for another test, new cap blocks were poured in place to replace the ones that the waves had removed during the preceding test. The displaced ones were not used again.

Figure 12 shows several of the features that have been discussed. The model's West Breakwater can be seen, ready for the first tests. The extreme relief characteristics of the terrain features are evident. Five of the wave recording probes on their tripods, with full-thread rods for adjustable legs, are aligned just West of West Breakwater, one of the profile-accommodating wave guide fences for containing the ends of waves is seen in the background, as is one of the crates containing stainless steel shavings for absorbing unwanted reflections at borders of the model. Figure 13 is a view from East of the Intake Structure of a test in progress. Water in the model basin is at extreme high tide stage +7.5 feet, while uniform waves 19 feet high and with 16 seconds period are attacking from a shallow water azimuth of 249 degrees. The catwalk for observations and camera station is also to be seen. Due to either a kind providence or Southern California weather this outdoor operation was at no time shut down because of wind conditions compromising testing conditions. On only one occasion was it necessary briefly to suspend progress due to rain. Much of the restructuring of the model breakwater extended into night shifts and on occasion test runs were made on the night shift.

THE MODEL CONFIRMED - THE DAMAGE MECHANISM IDENTIFIED

Five separate test runs were made that each confirmed the model's ability to reproduce the damage that was done to West Breakwater on January 28, 1981. The model was also consistent in that each of the tests produced the same damage. It was of considerable interest that essentially identical results were displayed by the three tests that were made with successive bursts of nine uniform height waves and that were made by the two continuous tests where the breakwaters were subjected to attack by random height waves. The wave series in the random heights tests carefully reproduced the energy characteristics at the 100 feet depth region of the model that had been derived from the

Figure 12

DETAIL OF THE MODEL

Figure 13

TESTING UNDER WAY

records made during the January 28, 1981 storm by the NOAA wave buoy. In all five of the test runs the wave machines were oriented to agree with azimuths that were derived by use of the numerical model for refraction, with input of Strange's hindcasts of deep water azimuths for that storm. Tide variations for January 28, 1981 at the site were not great through the hours of the storm's stronger phases, so no variation of water level was used during these five test runs, all being made with a tide stage in the model basin representing elevation +3.7 feet.

The damage sequence began with displacement by the waves of a Tribar on the edge of the armor pattern, at a specific point at the toe of the slope of the terminal cone. That specific location is where a blunt rocky ridge with its crest at elevation -15 feet was partly covered by the breakwater. Water rushing back down the conical surface of the breakwater's terminal cone, after passage of the preceding wave crest, proved to be capable of shifting the bottom row Tribar away from those upslope Tribars against which it had been tightly fitted during construction. After being moved downslope, away from the abutting support of the upslope Tribars the force of a following wave's uprush was sufficient to lift the shifted Tribar off of its base and then to tumble it across the breakwater's conical terminal surface toward deeper water on the sheltered quarter of the cone. Tribars that had been installed at the same -15 feet elevation either side of this localized high toe were not moved because they were in the middle of a pattern of Tribars and packed closely, so that freedom to move sufficiently to be tipped out was not available. The Tribars that shifted and then came out did so because of what was clearly an edge phenomenon. After the first Tribar was carried away its former immediate upslope neighbors became edge pieces and in succession they were loosened downslope by downrushing phases of the waves and then, newly without buttressing restraint from their former upslope neighbors, they were tilted out and carried over to a developing shoal of Tribar debris on the back slope. Every removal of a Tribar expanded the "unhemmed" edge of the pattern of remaining intact elements and the progress of damage increased rapidly as the perimeter of unsupported Tribars grew longer.

For some time the stripped off area's "downstream" boundary did not grow beyond the conical element of the terminus that paralleled the crests of the waves. That was the high point of the armoring crossed by the transiting waves. Beyond that "ridge line" there was no backrush of water after a wave's passage. The only forces by moving water that was felt by those Tribars tended to drive the edge pieces more firmly against their buttressing neighbors. In later tests that were carried on with long durations those edge Tribars did finally come loose, but due to a different cause. They tipped backward into the stripped off area after persisting high velocity forward wash of the waves eroded the stripped off area to a lowered profile. That eventually under-cut the quarrystone that had been supporting those Tribars, and they fell back and then were carried away forward to the debris shoal.

Once the rapid removal of the Tribar armor had grown to an area roughly elliptical in outline, with the major axis of the ellipse extending from the -15 feet initial point up to the near corner of the end cap block (Cap Block No. 1), the rapid growth continued at its edge toward the breakwater's root. The other edge, along the "ridge line" of the cone as described above, did not grow very much at that phase. At the lower parts of the growing edge of the ellipse uprushing waves passed over Tribars whose upslope neighbors had already been carried away. Those lower Tribars were dislodged by the uprushing waves, transported upslope to come to rest momentarily against the crest block for a wave or two, rolling short distances up and down slope. They were then carried parallel with the breakwater's axis toward the breakwater's end and thence across the terminal cone for eventual deposition in the debris shoal. As the Tribars at the bottom parts of the ellipse came away, Tribars on the upper reaches of that edge lost support from below and were dislodged, and the cycle continued. The edge toward the breakwater's root of the stripped off area maintained its curved configuration as the damaged area grew.

Generally, there was no detectable rocking or movement of the crest block above the area currently being stripped of Tribars until right after all of the Tribars immediately below that block had been carried away. At about that time the full undiffused force of the uprushing waves was impacting against the 7 feet vertical extent of the block. The block could be seen to rock upward very slightly as each wave struck and was pivoted more or less on the opposite edge of its base. Simultaneously there was a violent diversion of energy downward and upward at the block face. The upward diversion was visibly evident by the splash from the wave. The downward diversion caused vertical downward jet just as the confining weight of the crest block on its foundation quarrystones was momentarily relaxed through the upward rocking of the block. The large quarrystones on which the cap normally was supported were quickly picked away in the split second that the block tended to rock upward. The progressive undercutting soon removed a third or more of the support and then the block slid toward the sea, badly tilted and no longer an effective element of the breakwater.

Figure 14, in three parts, shows in (a) and (c) the damage in the model after eight crest blocks had been dislodged after an extended run during the first model confirmation test. Part (b) is a reduced size excerpt from the map of the surveys of condition that were made at Diablo Canyon Site just after the storm of January 28, 1981. The curved and rising edge of the stripped away areas are seen in all three illustrations. Figure 15 is reproduced from the test records and shows outlines of the growing area of Tribar removals in the model that were sketched from vertical photographs at nine stages of the testing. The position of the dislodged crest blocks is not sketched, but as noted in Figure 15 Blocks 1 through 12 were dislodged in this test. It was continued well beyond the conditions that reproduced the historical damage, primarily to see if the storm in nature might have destroyed West Breakwater completely, had it persisted long enough.

Test 1, which Figure 15 summarizes, was made with regular waves that were delivered in bursts of nine. As shown by the Figure the test began with 54 waves that increased from 13 to 17 feet in prototype height and with a period in prototype time of 16 seconds. No damage resulted. Then, without changing the period, the wave height was

Figure 14

DAMAGE IN NATURE AND IN THE MODEL

Model, Test 1

Model, Test 1

Survey At Site
1981

Figure 15

SUMMARY, TEST 1

OATE: 1/11/82 - 1/18/82

RUNS: 526-533ZZZ

AZIMUTH: 249°

TIDE LEVEL: + 3.7' MLLW

TEST WAVE: Regular Waves; Wave Height = 13 to 20'; Period = 16 sec; 9 Waves Per Burst

BREAKWATER CONFIGURATION #1: Breakwater built as originally constructed; no modifications.

SUMMARY:

Wave Height(ft.)	No. of Waves	Damage
13-17	54	None
20	180	Damage started on ends of cone between waves 63 & 72 (1) and continued with each burst: Area 2 - between waves 99 and 108 Area 3 - between waves 135 & 144 Area 4 - between waves 171 & 180 Capblock 1 moved between waves 135 & 144 Capblock 2 moved between waves 162 & 171 Capblock 3 moved between waves 171 & 180
19	765	Oamage area 5 occurred between waves 45 & 54 Area 6 - between waves 162 & 171 Area 7 - between waves 243 & 252 Area 8 - between waves 423 & 432 Area 9 - between waves 630 & 639 Capblock 4 moved between waves 27 & 36 Capblock 5 moved between waves 81 & 90 Capblock 6 moved between waves 90 & 99 Capblock 7 moved between waves 162 & 171 Capblock 8 moved between waves 207 & 216 Capblock 9 moved between waves 216 & 225 Capblock 10 moved between waves 225 & 234 Capblock 11 moved between waves 423 & 432 Capblock 12 moved between waves 513 & 522

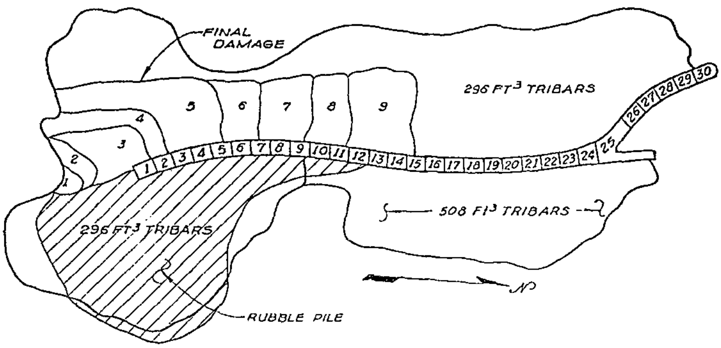


Table A

RATES AND EXTENT OF DAMAGE DURING TEST NO. 1
RELATIVE TO NUMBERS OF UNIFORM WAVES

Areas Stripped of Tribars		Cumulative Number of Waves	Crest Blocks Displaced	
Area No. (Fig. 15)	No. of Waves Per Area		No. of Waves Per Block	Block Number
1	72	72		
2	36	108		
3	36	144	144	1
		171	27	2
4	36	180	9	3
		216	36	4
5	54	234		
		270	54	5
		279	9	6
6	117	351	72	7
		396	45	8
		405	9	9
		414	9	10
7	261	432		
8	180	612	198	11
		702	90	12
9	207	819		

increased to 30 feet and shortly damage began to develop. Table A is a timing breakdown of the data in Figure 15. The center column shows the total number of large waves (greater than 17 feet) that had attacked the breakwater during Test 1 when each numbered area on Figure 15 was observed to have developed, and when each crest block was displaced.

The damage to the Tribar armor grew more and more slowly as the Tribar removals passed the general locality of Crest Block Number 5. By the time the seaward face of the breakwater was denuded as far back as the limit of Area 9, at Block Number 15, the rate of growth of the damage was hardly perceptible. Reasons for this were not rigorously investigated but are believed to be uniquely related to the site, with particular relationship to the path the waves moved over submerged terrain features toward the landward part of West Breakwater's alignment.

Test runs 2 and 3 duplicated Run Number 1. Runs 4 and 5 also tested the original configuration of the breakwaters but random waves were generated instead, in which the spectral energy equalled the energy at SCAN that was derived from the records made by the NOAA buoy during the actual storm of January 28, 1981. The synthesized trains of random height waves that were generated in the model differed however between Run Number 4 and Run Number 5 in one respect. The wave train of Test Number 5 was one of "high groupiness" and the one generated for Test Number 5 was of "low groupiness". The groupiness factor relates to the concept that the higher waves in a random set often are found together in the train. If the large amplitude waves occur together, that can be referred to as high groupiness, but if the high waves are dispersed more evenly it can be referred to as low groupiness.

There was no difference to be seen in any of the five tests as to how and exactly where the Tribar removals began, or in how the stripping away of Tribars progressed. Neither were any differences apparent in the tests as to the sequence of events that led to displacement of each crest block. There were noticeable differences shown in the resistance to displacement by individual crest blocks and in exactly where they had come to rest when a test was completed. Those differences were known to relate to the impossibility of duplicating exactly the quarrystone beddings on which the blocks were poured.

Those first five tests, with the breakwaters modelled as the prototype originally had been designed, and attacked in the model by waves that are believed to be closely comparable in energy with the historical wave event of January 27, 1981, produced essentially identical results in steps that were closely alike. The results they produced closely matched the conditions that site surveys showed immediately after the 1981 storm and in each test the damage began at exactly the same point.

HEMMING THE EDGE

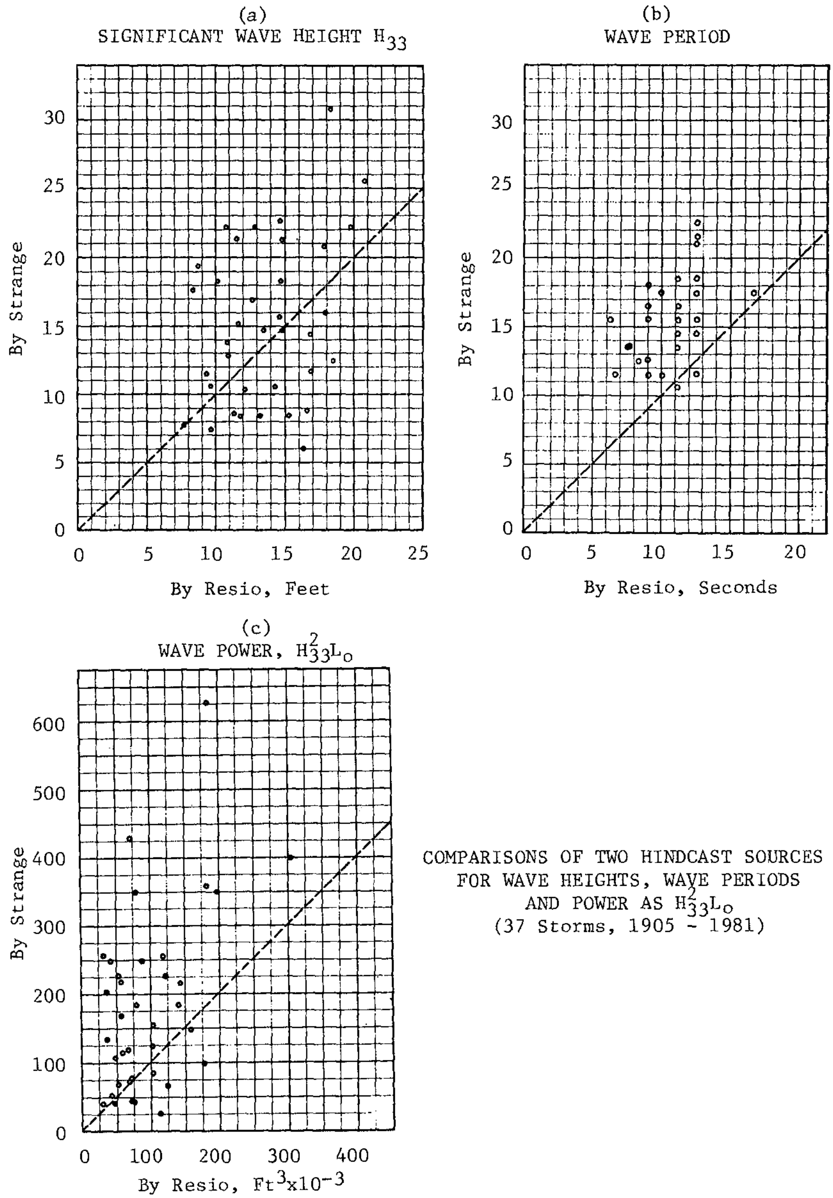
The edge Tribars at the -15 feet locality of West Breakwater's terminal cone were given restraint against downslope displacement by the downrushing of waves by embedding their legs in simulated pumped-in-place concrete. Two rows of Tribars at that locality of the toe of the slope were embedded to a thickness equal to one-half the Tribars' height. This procedure was first used in 1975 to solve an edge restraint problem on the toe of East Breakwater's sheltered side and proved to be effective and was found in 1981 to be entirely intact (4, pgs 87-91). To be conservative, the embedment was continued along the toe of slope until it reached to the -30 feet contour on both sides of the -15 feet rock. The whole breakwater was again rebuilt, otherwise exactly according to the original design, and again tested with both uniform wave trains of up to 21 feet heights and random wave trains like those of January 28, 1981, in which the significant height of the wave train was 19 feet. The results at the embedded toe were good, the edge Tribars were successfully restrained from shifting.

UPGRADED RECONSTRUCTION GOALS

The weak link in West Breakwater's resistance to the original design goals had been found and was remedied by the pumped concrete "hem" at the high level toe of the terminal slope. However, with prolonged attack in the model by the January 28, 1981 peak wave train it was eventually able to remove a cluster of three or four Tribars on the far side of the terminus. Once such a cluster of Tribars had tediously been removed by prolonged attack, another analogy to the behavior of the ravellable fabrics was evident. Like a hole in the middle of some fabric meshes, the cluster of stripped away Tribars grew rapidly. It was obserably the edge effect again. With clusters of only 2 or possibly 3 Tribars removed by waves the perimeter Tribars around the cluster area appeared to be giving mutual restraint, each to the adjoining two, against being shifted by moving water. It was much as arch stones carry higher structural loads laterally and then down to a masonry building's foundation. But when the three or four Tribars were gone such arching action between peripheral remaining pieces diminished and the wave overwash shifted, then tipped, then rolled out and carried away the edge pieces in rapid succession. It was evident that the January 28, 1981 storm, which had at its peak an $H^2_{33}L$ value per wave of 350,000 at SCAN as contrasted with the original design's value of 205,400, would eventually have damaged West Breakwater if the "weak link" at the high edge of the terminus' toe had not existed at all.

The project owners needed to know the probable recurrence interval for the 1981 storm in order to have a decision basis for questions of upgrading vs. reproducing the original structure. Identical wave hindcast assignments were separately given to two qualified marine meteorologists, to identify all storms in the weather maps that had been compiled since the earliest days that had inferrable potential for severe attack in the inshore waters at Diablo Canyon Site. They collaborated only in searches of the records, beginning with those of 1899 and continuing through January of 1981, and reached a consensus on forty events. It was understood that either hindcaster was free to make further additions or deletions as he got deeper into his assignment. R. Rea Strange III compiled 47 hindcasts of his expanded list of storms and D. T. Resio submitted 41. The results from the two efforts were carefully compared and, as shown by Figure 16(a), their wave height hindcast data were fairly well centered on a 1:1 comparison line, though quite scattered. Figure 16(b) however shows consistent

Figure 16



COMPARISONS OF TWO HINDCAST SOURCES FOR WAVE HEIGHTS, WAVE PERIODS AND POWER AS $H_{33}^2L_o$ (37 Storms, 1905 - 1981) disagreement between the peak frequencies of the spectra from the two hindcasters. In most storms the frequency according to Strange was lower than the values from Resio. Each hindcaster's data are also compared by using the wave periods to calculate wave lengths and thus to derive the H_{33}^2L characteristics at the peak of each storm. That comparison appears in Figure 16(c). With the typically larger wave periods thus affecting the comparison it was clear that use of Strange's hindcasts would be the more conservative choice. That the periods hindcast by Strange also compared satisfactorily with the periods found in the few buoy records of storms that had also been independently hindcast, confirmed the conclusion that Strange's results should be used in the present studies when wave storms that had not been recorded would be generated in the model basin.

The most severe storm in the hindcast report, between 1899 and 1981, occurred on March 13, 1905. According to Professor Leon E. Borgman, who analyzed the storms statistically for the principal investigator, that storm has a probable return frequency of 100 years at SCAN if the wave height is used for ranking, and of 80 years when the H_{33}^2L characteristic is used to rank the storms' strength. The respective values for the January 28, 1981 storm he found to be 13 and 18 years. The original design criteria would fall on Borgman's return frequency curves at 8 years for wave height and 8 years also for H_{33}^2L.

The project owner instructed that an upgraded design be developed for West Breakwater, to provide a level of resistance to storm attack that would provide undiminished shelter for the intake basin on a continuous basis. An important constraint on an upgraded West Breakwater design was however defined by the owner, namely that for environmental concerns a rebuilt breakwater had to fit within the cross-section, profile and planform limits of the original structure. With that limitation sharply in focus, the investigators tested to see how strong a storm an upgrade concept might be able to resist. After proving the effectiveness of embedding the toe Tribars at the high parts of the terminal cone, the West Breakwater model was rebuilt with 36.8 tons Tribars in the damaged area instead of the 21.5 tons pieces of the original design and of all of the preceding tests. The model was then subjected to several tests with the January 28, 1981 spectrum of irregular waves, and with regular waves whose height was built up from burst to burst until they were on the order of the highest 1 per cent of the waves in the corresponding irregular wave train, or even higher. Eventually in each test several Tribars did come out of the pattern on the back side of the terminal cone and then, as described before, the damage spread quickly. The embedment concept was then adapted to this type of damage evolution, by creating parallel "ribs" of embedded Tribars across the terminal cone. Gaps were left between parallel adjoining ribs so that air that might otherwise be trapped by uprushing waves in the voids of the quarrystone supporting the Tribars could vent harmlessly away. Like rows of stitching across a banner to limit tearing or unravelling of the fabric, the ribs minimized or eliminated the development of clustered Tribar removals. Consequently edge removals of intact Tribars did not grow after single Tribars might be carried away from the gaps between ribs.

Compressive strength of concrete in the prototype specifications was set at 4,500 pounds per square inch at 28 days. Scale strength for the 1:45 model would then be 100 pounds per square inch. A mixture was prepared for the model concrete that was composed of Plaster of Paris, sand, barite and water. It could easily be made to the correct density, had good pouring qualities and the correct scaled compressive strength. However, it was found after several test runs that extended immersion caused the material to become so weak that it could easily be crumbled by pinching it between finger and thumb. Its use was discontinued. Interestingly, however, the embedment functioned effectively in its saturated weak condition, restraining the model Tribars from movement by big waves. Although the embedment concrete undoubtedly should be strong, to achieve longevity of the embedment, the compressive loading on the mass of embedment concrete by Tribars that are being attacked by the waves is low. The function of the embedment is simply to provide a passive restraint against initial movement.

Instead of the Plaster of Paris mixture another material named "Modcrete" was used for both the toe embedment and the rib embedment of Tribars in all the remaining tests. It was developed for the work by personnel of ARCTEC, Incorporated, by adapting and altering that firm's model scaled ice compound, changing its strength and elastic properties by adjustment of proportions of its ingredients and weighting it with barite to achieve the correct specific weight.

A pattern of ribs evolved as the tests proceeded with the 1981 storm conditions. Those ribs stabilized the breakwater for that storm condition, but survival of higher attack levels was desired.

Table B shows the characteristics at the 100 feet depth locality of ten greatest storms at Diablo Canyon Site among the 47 that were selected and hindcast by Strange.

Table B

CHARACTERISTICS AT 100 FEET LOCALITY OF TEN GREATEST STORMS

Date	H_{33} Ft.	T_p Sec.	Azimuth	$H^2_{33}L$
3/13/05	30.8	14.5	233°	626,500
12/06/69	20.0	22.5	249°	428,900
4/05/58	22.1	17.5	260°	398,900
2/03/15	25.4	12.5	247°	357,300
1/28/81	20.7	17.5	249°	350,000
2/17/80	22.1	15.5	242°	348,800
12/28/31	17.7	17.5	247°	255,900
2/09/60	22.6	11.5	264°	255,100
2/10/63	19.4	14.5	246°	248,500
1/06/39	16.9	18.5	260°	248,000

The hindcasts were reviewed to identify the historical storms that had been most severe from each of four subdivisions of the whole sector from which any damaging waves had reached the site. Three of the four sub-sectors had storms among the ten most severe in the 81 years covered by the hindcasts. In the remaining sub-sector, the most southerly one, the worst storm was that of January 25, 1914, It was ranked as the 17th strongest of the 47 in the 80 years of hindcasts. The events in March 1905, January 1914, February 1960 and January 1981 were selected for the four events that should be reproduced in continuing test series to discover whether or not additional measures for upgrading West Breakwater should be taken and, if so, to test such measures for effectiveness.

Table C presents the maximum wave conditions from each of the four sub-sectors of approach to which the model breakwaters were exposed. The sets of tests were not done in the order they appear in the table. Each set began with runs using small regular waves and then they were built up toward the maxima until substantial damage developed. At that point the run was halted. The damage was mapped by a surveyor, photos were taken and modifications of the rib patterns were decided upon. The breakwater was then rebuilt accordingly. The maximum heights of regular waves that were reached generally approximated or exceeded the 1 per cent wave height for the hindcast waves of the maximum historical storm for that sector of approach.

After the attacks by the maximum hindcast storm spectra from the four sub-sectors of approach had been made, and in each test the embedment ribs had been improved so that West Breakwater survived without progressive damage, the 1905 storm spectrum was generated from each of the four sectors at several tide stages for each direction. As indicated by the H^2L factors shown in the last column at the right of Table C, those tests using the 1905 spectrum substantially exceeded the attack levels of the historical storms in each of the other three sectors. Furthermore, the durations of the test runs from the 1905 storm in its own sector, 233°, at fixed tide levels rather than naturally transient levels, constituted attacks that were more drastic than the historical. Single Tribars at randomly scattered locations were removed by waves during some of the 1905 storm tests, two adjoining Tribars in one case, but none of those areas then grew larger during persistent continuing attack by these final extreme condition tests. In fact, these small opened areas shrank slightly, enough that a Tribar could not be reinserted in the remaining space.

Figure 17 shows the plan that was recommended for the restoration and upgrading of West Breakwater; Figure 18 presents the related cross-sections. During the final verification testing, when extreme storm conditions were imposed, some vulnerability of the Tribars was seen at two shallow toe features and at the emergent edge of the armoring where East Breakwater springs from a reef at its root. As shown on Figures 17 and 18 embedment of the toe at those locations was included in the recommended plan. The recommendations were adopted and the breakwater was reconstructed accordingly. Packing the Tribars to 90 per cent of the theoretically perfect pack was specified. As each Tribar was set in place the three coordinate values for the center at the top of each of the 3 legs of the piece were determined by instrument surveys and field computer, to determine if an acceptable placement had been achieved before the Tribar was released from the setting crane's tackle. No new work shift was started by the contractor until it had

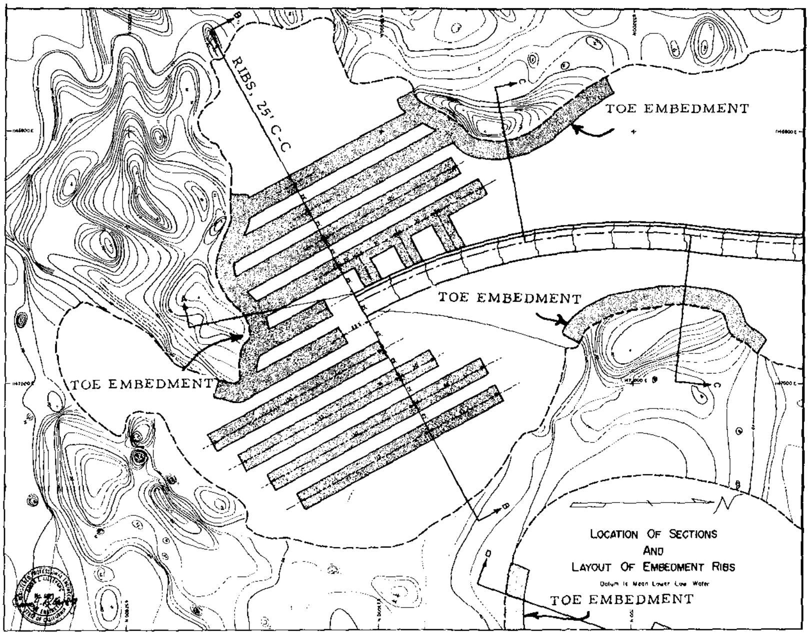
Table C

MAXIMUM CONDITIONS GENERATED AT 100 FEET DEPTH LOCALITY
DURING 47 TESTS ON DIABLO MODEL BREAKWATERS

Azimuth At 100' Depth	REGULAR WAVES				IRREGULAR WAVES				
	Tide (Ft)	H (Ft)	T (Sec)	Length (Ft)	Spectrum Of:	Tide (Ft)	H_{33} (Ft)	T_p (Sec)	$\frac{H_{33}^2 L}{1000}$
270°	+7.5	37	18	987	2/09/60	+7.5	24	18	568
	-2.0	41	18	948		-2.0	25	18	593
	+7.5	37.5	11.5	564					
	-2.0	39	11.5	548					
					3/13/05	+7.5	30	14.5	687
						+5.3	32	14.5	775
						+2.8	30	14.5	675
						0.0	31	14.5	713
						-2.0	34	14.5	851
249°	+3.7	33	16	848	1/28/81	+7.5	25.5	17.5	621
						+3.7	26	17.5	636
						-2.0	24	17.5	529
					3/13/05	+7.5	32	14.5	782
						-2.0	31	14.5	708
233°	+7.5	35	14.5	763	3/13/05	+7.5	31	14.5	733
	-2.0	45	14.5	736		-2.0	29	14.5	619
217°	+7.5	35	12.5	631	1/25/14	+3.7	19.5	12.5	237
	+3.7	35.5	12.5	624					
	-2.0	35	12.5	611					
					3/13/05	+7.5	30	14.5	681
						+5.3	27	14.5	552
						+2.8	25.5	14.5	488
						0.0	25	14.5	464
						-2.0	25	14.5	460

Figure 17

RECOMMENDED PLAN FOR RIBS AND TOE EMBEDMENT



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been demonstrated to the Resident Engineer by computer analysis that the preceding shift's placing results had met or exceeded the packing requirement that was specified. Figure 19 is printed at greatly reduced scale from the map at 1"-20' (1:240). It shows a computer plot of the surveyed Tribars as built in the prototype in 1983-84.

CONCLUSION

The solution for upgrading the breakwater's resistance to storms that has been described is by no means conventional; that is mostly due to the constraint that had been laid down, that the original alignment, profile and cross-section limits had to enclose the upgraded structure. However, the unique character of the terrain effects at the site and the conventional influences of cost for larger structures in the sea might well have made this an appropriate solution even if the environmentally related limits had not been imposed.

The authors conclude that investigations of problems involving wave attack on the termini of rubble mound breakwaters should always be undertaken with the aid of three-dimensional physical modelling unless

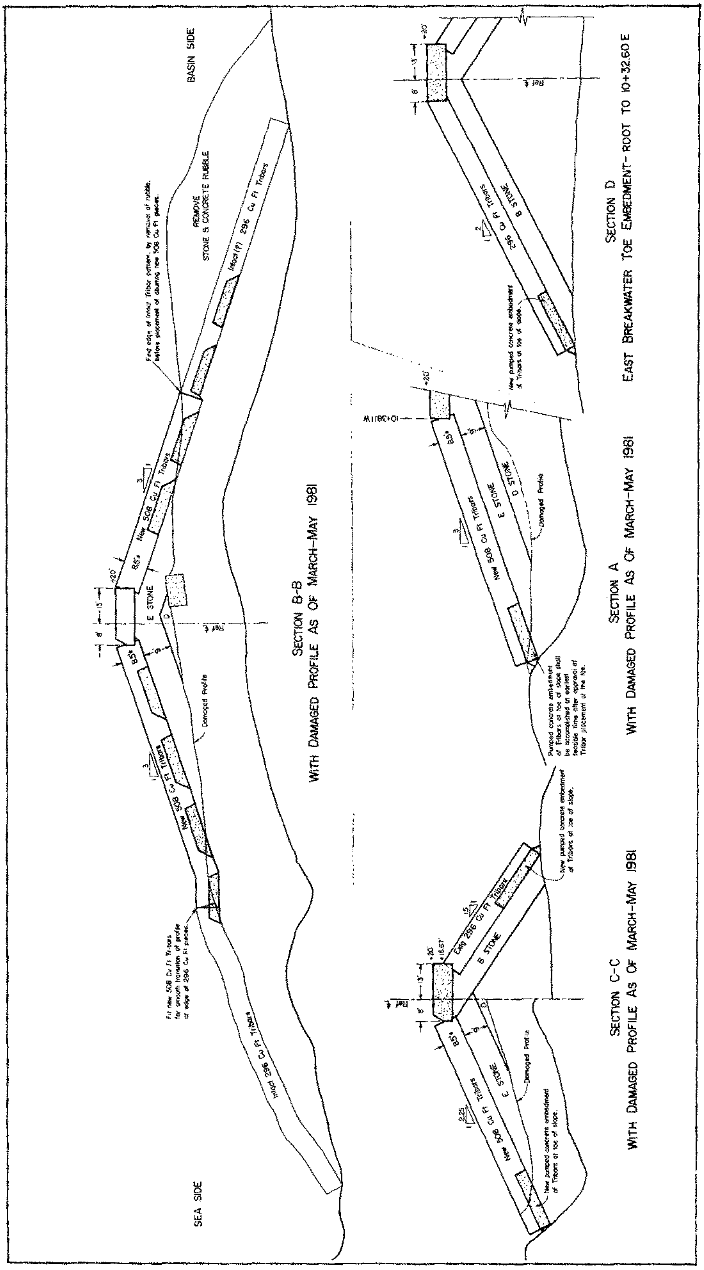
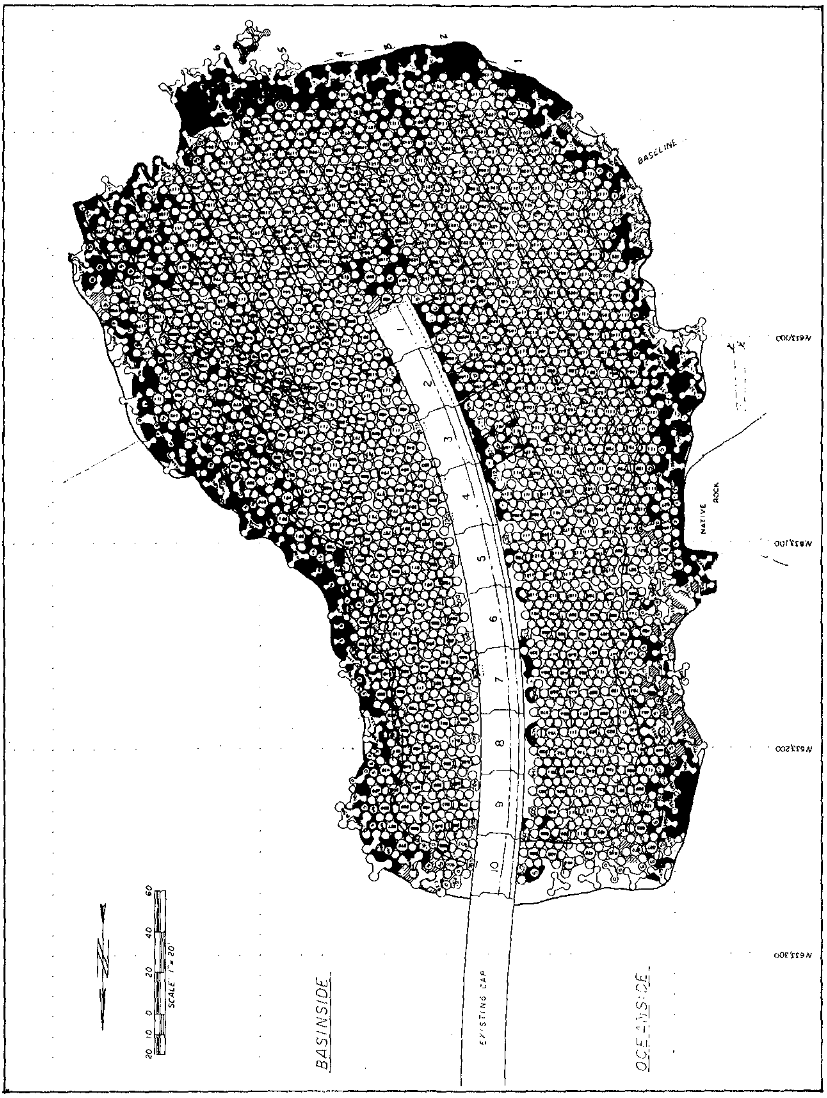


Figure 18

CROSS-SECTIONS SHOWING RECOMMENDED RIBS AND TOE EMBEDMENTS

Figure 19

SURVEYED POSITIONS OF TRIBARS
AS BUILT AT DIABLO CANYON WEST BREAKWATER
1984



owner and engineer are in a position knowingly to take large risks. Further, that physical modelling at suitably large scale is virtually mandated if the submerged terrain at a site is not regular.

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Michele M. Monde, A.M.ASCE, with the authors, completed the team of professionals that carried out the model studies to which this paper relates.

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