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CRANE,P.A.	Pacific Gas & Electric Co.
RECIP.NAME	RECIPIENT AFFILIATION
MIRAGLIA,F.J.	Licensing Branch 3

SUBJECT: Forwards response to NRC 810309 request for info re breakwaters.Breakwater damage would not result in configuration for which intake facility safety features are not qualified.Written confirmation requested.

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PGandE

P. O. BOX 7442 • 77 BEALE STREET, 31ST FLOOR, SAN FRANCISCO, CALIFORNIA 94106
TELEPHONE (415) 781-4211 TELECOPIER (415) 543-7813

MALCOLM H. FURBUSH
VICE PRESIDENT AND GENERAL COUNSEL

ROBERT OHLBACH
ASSOCIATE GENERAL COUNSEL

CHARLES T. VAN DEUSEN
PHILIP A. CRANE, JR.
HENRY J. LaPLANTE
JOHN B. GIBSON
ARTHUR L. HILLMAN, JR.
CHARLES W. THISSELL
DANIEL E. GIBSON
ASSISTANT GENERAL COUNSEL

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March 27, 1981

RECEIVED
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U.S. NUCLEAR REGULATORY COMMISSION

Mr. Frank J. Miraglia, Jr., Chief
Licensing Branch No. 3
Division of Licensing
Office of Nuclear Reactor Regulation
U. S. Nuclear Regulatory Commission
Washington, D. C. 20555

Re: Docket No. 50-275
Docket No. 50-323
Diablo Canyon Units 1 & 2

Dear Mr. Miraglia:

The attached provides our response to Mr. R. L. Tedesco's letter dated March 9, 1981 requesting information on the breakwaters at Diablo Canyon.

Mr. Omar J. Lillevang was the consulting engineer we retained to design the breakwaters, and he is directing the ongoing studies of the recent breakwater damage. His paper entitled "A Breakwater Subject to Heavy Overtopping: Concept, Design, Construction, and Experience" is attached as Reference 1 and provides substantive material responsive to questions 1, 2, and 7.

A discussion entitled "Safety Implications of Damaged Breakwaters" is attached as Reference 2, prepared as PGandE's primary discussion on this matter, concludes that breakwater damage, whether caused by seismic or wave forces or a combination of the two, would not result in a configuration for which the safety features of the intake facility are not qualified.

The recent damage, which is bounded by previous evaluations, is therefore of no safety significance. The damaged breakwater continues to provide adequate protection to the intake facility.

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Kindly acknowledge receipt of the material listed in this letter on the enclosed copy of this letter and return it to me in the enclosed addressed envelope.

Very truly yours,

Philip A. Crane, Jr.

Attachment (40)
CC w/attachment: Service List

[illegible] [illegible]

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[illegible]

[illegible]

DIABLO CANYON UNITS 1 & 2
Dockets Nos. 50-275/323

Hydrologic Engineering Questions
Intake Cove Breakwaters

Response to the March 9, 1981 NRC Questions Regarding Breakwaters at Diablo Canyon.

Question: 1. Provide the following information relating to the hydrologic design of the breakwater system:

a. Describe the design wave field (wave height and period, direction and stillwater level) for breakwater stability. Characterize the severity of this design wave field in relation to probable maximum type events and/or statistically.

Response: The design wave field is depicted in the FSAR Appendix 2.4A, Plate IV. The figure shows the "significant" wave heights for various azimuths, wave periods and annual hours of exceedance. Significant wave height is defined in Appendix 2.4A. As discussed in Safety Implications of Damaged Breakwaters Reference 2 the highest significant wave occurs less than one-tenth of one percent of the time.

Question: 1. b. Provide detailed descriptions of the breakwater design. Describe the various layers, the materials used, the sizes, shapes and weights, and the slopes and elevations. Include drawings and cross sections.

Response: A description of the breakwater design is provided by FSAR Figures 2.4-6 through 2.4-9.

Question: 1. c. Describe the analyses and/or modeling techniques to justify the breakwater design, i.e., that the breakwater will be able to survive the design wave conditions without loss of its safety functions.

Response: Modeling techniques employed in the study of the wave action in the intake basin formed by the natural coastline and the breakwaters are described in FSAR Appendix 2.4B. A discussion of modeling employed in assessing the stability of the breakwater armor (tribars) can be found in Reference 1.

Question: 2. Provide the following information about the breakwaters as actually constructed:

a. Describe any known differences between the breakwater as designed and as constructed. Discuss the effect of any such differences on the breakwaters' ability to survive the design wave conditions.

Response: The breakwaters were constructed substantially in accordance with the plans and specifications.

The only known significant difference between the as-designed and as constructed breakwaters involves the spacing pattern employed in locating the vents through the concrete cap. Whereas the design specified vent spacing at 10 feet on centers along the cap the actual spacing pattern consisted of an equilateral triangular arrangement with each leg of the triangle approximately 10 feet in length. This difference, which was approved by the breakwater design consultant, results in increased vent areas through the cap and does not reduce, but rather may improve, the breakwaters' capability to sustain the design wave conditions. In fact, since the breakwaters were constructed, the cap vent system has been observed on several occasions satisfactorily performing its function, i.e., preventing the accumulation of energy under the cap blocks by wave-induced compression of air.

Some local sparsity of tribar coverage on the basin side of the east breakwater was noted during an inspection made in 1975. This inspection was made after minor storm wave damage was incurred at that location during the last weekend of December 1974. Remedial work at that time included rebuilding the armor to achieve not less than 85% of the theoretical pack. No subsequent damage in the rebuilt section has been noted to date.

Question: 2. b. Describe the quality assurance program used during the breakwater construction.

Response: The breakwaters were not constructed under a 10 CFR 50 Quality Assurance Program as they are not designated as safety related structures. Design was complete and construction in progress when Quality Assurance Criteria for Nuclear Power Plants were issued as Appendix B to 10 CFR 50 on June 27, 1970. Nevertheless, quality control measures were implemented during their construction. They were constructed to the plans and specifications under the supervision of a Resident Civil Engineer and one or more Field Engineers and Inspectors. Inspections were made to ensure compliance with the plans and specifications. Sufficient records were maintained to provide satisfactory evidence that the construction was consistent with specified requirements. The breakwater design consultant also made periodic inspections while construction was underway.

Quality control tests and inspections verified such items as soundness, proportions, unit weight, placement, and overall cross sections of the quarry stone elements. Armor pieces (tribars) were checked for concrete strength, form dimensions, voids, placement tolerances and each tribar was branded with a serial number and casting date. Additional quality measures involved tests on concrete materials including cement, aggregate and admixtures and controls on mix design, batching and mixing, forms, handling and placing, compacting, finishing and related items.

Question: 3. a. Provide descriptions of the actual sea and wave conditions prior to and during the time the breakwater was damaged.

Response: Visual observations of wave conditions at the breakwater location for a period of about one hour on the afternoon of January 28, 1981 yielded estimates of wave heights ranging from 4 to 18 feet and wave periods ranging from 10 to 17 seconds. The observers notes are attached as Reference 3.

Wave conditions during the morning of the 28th are recorded on photographs provided earlier. Additional displacement of some of the breakwater materials occurred subsequent to the time these photos were taken. It is believed the waves in the morning were slightly more severe than those observed in the afternoon of January 28, 1981.

Wave conditions as determined by a numerical sea state forecast for the period from 12:48 AM through 3:48 PM (local time) on January 28, 1981 are shown in the tabulation below. Time in the tabulation is based on Greenwich Mean (GMT). 0800 GMT is equivalent to 0 local time. This tabulation lists wave periods, energy, and significant wave heights in 3 hour steps for the 15 hour period involved. Note that the total energy listed with the highest significant wave for each 3 hour data set is the summation of the energy increments associated with each wave in the period bands denoted in the data set. The maximum significant wave is 19.55 feet at 9:48 AM local time.

NUMERICAL SEA STATE FORECAST
DIABLO CANYON January 28, 1981

DATE/Time	Travel Time	PERIOD	ENERGY	TOTAL ENERGY	HEIGHT 1/3
28/8.8Z	8.8	21.76 - 21.76	74.95 - 74.95	0	0
		21.76 - 21-76	74.95 - 74.95	0	0
28/11.8Z	11.8	21.76 - 16.22	74.95 - 38.11	27.63	14.88
		21.76 - 20	74.95 - 74.11	9.63	2.25
		20 - 18	74.11 - 59.91	10.65	9.23
		18 16-22	59.9 - 38.11	16.34	11.44
28/14.8Z	14.8	21.76 - 12.93	74.95 - 16.88	43.55	18.68
		21.76 - 20	74.95 - 74.11	9.63	2.25
		20 - 18	74.11 - 59.91	10.65	9.23
		18 - 16	59.9 - 35.75	18.11	12.04
		16 - 14	35.74 - 21.18	10.92	9.35
		14 - 12.93	21.18 - 16.88	3.22	5.08
27/17.8Z	17.8	21.76 - 10.75	74.95 - 11.34	47.71	19.55
		21.76 - 20	74.95 - 74.11	.63	2.25
		20 - 18	74.11 - 59.91	10.65	9.23
		18 - 16	59.9 - 35.75	18.11	12.04
		16 - 14	35.74 - 21.18	10.92	9.35
		14 - 12	21.18 - 14.16	5.26	6.49
		12 - 10.75	14.16 - 11.34	2.11	4.11
28/20.8Z	20.8	18.72 - 9.2	67.51 - 8.4	44.33	18.84
		18.72 - 18	67.51 - 59.91	5.7	6.76
		18 - 16	59.9 - 35.75	18.11	12.04
		16 - 14	35.74 - 21.18	10.92	9.35
		14 - 12	21.18 - 14.16	5.26	6.49
		12 - 10	14.16 - 9.87	3.22	5.08
		10 - 9.2	9.86 - 8.4	1.1	2.97
28/23.8Z	23.8	16.36 - 8.04	39.64 - 6.36	25.96	14.14
		16.36 - 16	39.64 - 35.75	2.91	4.83
		16 - 14	35.74 - 21.18	10.92	9.35
		14 - 12	21.18 - 14.16	5.26	6.49
		12 - 10	14.16 - 9.87	3.22	5.08
		10 - 8.04	9.86 - 6.36	2.63	4.59

<u>Question</u>: 3. b. Provide sources of your information including instrument types, locations and accuracy. If some of the descriptions are from interpolations, estimations, and/or hindcasting justify the methods used.

<u>Response</u>: As discussed in the response to Question 3a visual observations of wave conditions during the afternoon of January 28, 1981 (approximately 7 to 10 hours after the damage is believed to have occurred) represents one source of information.

The photographic and videotape records of sea conditions represent a second source of information. Some additional displacement of the damaged section of breakwater is known to have occurred after the photos on the morning of January 28 were taken.

The third source of data on wave conditions is the numerical sea state forecast also discussed in the response to Question 3a. The method employed in this forecasting technique utilizes a fetch limited spectrum of the Pierson-Moskowitz form. It was developed through analysis of wave data collected at many California locations since the mid-1960's.

The spectra are quite wide at early stages of wave development, while they become narrower and the front steeper as the wind speeds or fetches increase. The heights and periods obtained by integrating this spectrum agree almost exactly with the sea state values predicted by the S-M-B method.

Question: 3. c. Discuss and compare the actual sea wave conditions with the design conditions.

Response: In general the wave occurring on January 28, 1981 were similar to those observed on several occasions since the breakwaters were constructed. Other than the minor damage in December 1974 summarized in the response to Question 2a and discussed in greater detail in Reference 1 no damage as a result of these earlier episodes has been observed.

The significant wave height established during the breakwater design process was 18 feet with a period of 14 seconds. The significant wave height of 19.55 feet hindcast for the January 28 storm exceeds the design significant wave by less than 9 percent.

If the wave data set containing the 19.55 feet maximum significant wave is plotted as significant wave height versus period, the resulting curve peaks at a period of 16.65 seconds within the bandwidth containing the greatest energy. The work per wave is a function of the square of the product of height and period. Thus the ratio of the square of these products for the design and recent storm waves provides an index of the relative severity of the two conditions. Calculating this ratio results in a value of 1.67. On this basis, the January 28 storm waves were significantly more severe than the design conditions. As noted earlier, waves of similar severity have occurred in the past with no resulting damage.

We have concluded that the breakwaters are capable of tolerating wave action well in excess of the design conditions without suffering damage more severe than that assumed in the safety evaluation of the Design Class I features of the intake structure.

Question: 4. Provide detailed descriptions of the damage to the breakwaters. Include drawings and photographs.

Response: The recent storm wave damage to the west breakwater is believed to have occurred during the late night and/or early morning hours of January 27 - 28, 1981. The damage extends for approximately 150 feet northerly from the center of the terminal cone and has resulted in lateral and vertical displacement of armor pieces (tribars), six concrete cap blocks, and the underlying quarry stone sections. Some of the displaced stone and armor pieces now lie within the entrance to the intake basin producing a shoaling effect while at the same time providing some degree of buttressing support for the undamaged section of the west breakwater and maintaining protection for the east breakwater.

Drawings depicting the breakwater damage are not available. Photographs and narrated videotape coverage of the scene obtained from both land based and aerial cameras have been provided earlier.

Question: 5. Provide an evaluation of the causes of breakwater damage.

Response: An evaluation of the causes of breakwater damage is not available at this time. Such an evaluation must await the completion of an extensive reconnaissance and surveying program. This program will include underwater mapping utilizing divers, side scan sonar and possibly underwater video as well as aerial photography and conventional surveying techniques. The program will require the mobilization of land, air, and water based equipment. The objective of the program is to accurately define the extend of damage and supplement the available bathymetric data base for use in model studies. This information will be used to establish or infer the probable failure mechanism. In addition, a confirmatory inspection of the apparently undamaged sections of the breakwaters will be undertaken.

The program outline is expected to require several months to complete. The schedule will be constrained by weather, sea conditions and the attendant concerns with personnel safety during survey operations.

Overall technical direction for the program will be provided by Omar J. Lillevang. Surveying and diving support personnel have been retained and initial scope of work and planning sessions have been held. The first phase of work is underway.

Question: 6. Discuss your proposed repairs to the breakwater including schedules, procedures and your quality assurance program.

Response: A determination of the most appropriate remedial work must necessarily await completion, or at least substantial completion, of the program outlined in the response to question 5. Meanwhile, construction feasibility studies and material availability determinations are progressing. Some spare tribars are on site and forms for casting new tribars are available. Our schedule objective is to commence reconstruction work in early summer 1981. A quality control program similar to that employed in the original breakwater construction will be implemented during the repair effort.

Question: 7. Discuss any previous instances or damage and/or repairs to the breakwaters including dates, descriptions of damages and causes, if known.

Response: A discussion of previous damage and repairs to the breakwaters is provided in Reference 1.

Question: 8. Provide descriptions of breakwater surveillance programs and repair procedures during plant operation.

Response: The breakwater surveillance program to be used during plant operation will be commensurate with the safety and/or operational significance of potential future breakwater damage. As discussed in Safety Implications of Damaged Breakwaters (Reference 2) the recent storm damage would not have affected plant safety or operability. This damage was identified visually. A visual surveillance program is adequate to detect significant damage which might result in a breakwater configuration for which the safety features of the intake facility are not qualified. Such a program will be implemented during plant operation.

The program will include, as a minimum, bi-monthly visual inspections from the breakwater crests with the results and observations recorded in an inspection log. In addition an inspection will be performed and documented after each heavy storm wave episode which results in significant wave overtopping of the breakwaters.

Repair procedures during plant operation will include maintaining an inventory of spare armor pieces (tribar) and a few tribar forms at the site. Additional tribar forms are available from the original breakwater contractor. Quarry stone and concrete materials are available within practical haul distances to the site.

Any design required to implement breakwater repairs will be prepared under the direction of the Engineering Department.

Contract administration and inspection for any repair activities will be the responsibility of the PGandE General Construction Department.

A quality control program similar to that employed during the original breakwater construction will be utilized in any future repair program.

A summary of the results of all inspections and repairs, if any, will be provided in the plant annual report. Should any major damage occur, the NRC Region V Division of Inspection and Enforcement, the NRC Project Manager and/or the Chief of the Hydrologic and Geotechnical Engineering Branch will be notified within 3 days of the occurrence.

A BREAKWATER SUBJECT TO HEAVY OVERTOPPING:

CONCEPT, DESIGN, CONSTRUCTION AND EXPERIENCE

by

Omar J. Lillevang, F. ASCE
Consulting Engineer
Los Angeles, California

Prepared for "Ports 77", ASCE Specialty Conference of the
Waterway, Port, Coastal and Ocean Division
Long Beach, California

March 10, 1977

REFERENCE 1

The writer acknowledges the pleasure derived from his engagement by Pacific Gas and Electric Company on its Diablo Canyon Project Intake Facilities, and the Company's cooperation in releasing for publication the subject matter of this paper.

APPENDIX 1.— REFERENCES

1. Gaillard, D. D., *"Wave Action in Relation to Engineering Structures"*, Professional Papers, No. 31, Corps of Engineers, U.S. Army, 1903, Chapter VIII.

2. National Marine Consultants, Inc., *"Wave Statistics for Seven Deep Water Stations Along the California Coast,"* U.S. Army Corps of Engineers, Los Angeles, December 1960.

3. Raichlen, F., *"A Model Study of the Breakwater for the Diablo Canyon Site of the Pacific Gas and Electric Company Nuclear Power Plant,"* Omar J. Lillevang, Consulting Engineer, Los Angeles, California, August 18, 1969.

4. Raichlen, F., discussion of "Armor Stability of Overtopped Breakwater", by Patrick R. Lording and John R. Scott, *Journal of the Waterways, Harbors and Coastal Engineering Division*, ASCE, Vol. 98, No. WW2, Proc. Paper 8138, May 1972, pp. 273-279.

5. Vanoni, V. A. and Raichlen, F., *"Laboratory Design Studies of the Effect of Waves on a Proposed Island Site for a Combined Nuclear Power and Desalting Plant,"* KH-R-14, California Institute of Technology, Pasadena, California, July 1966.

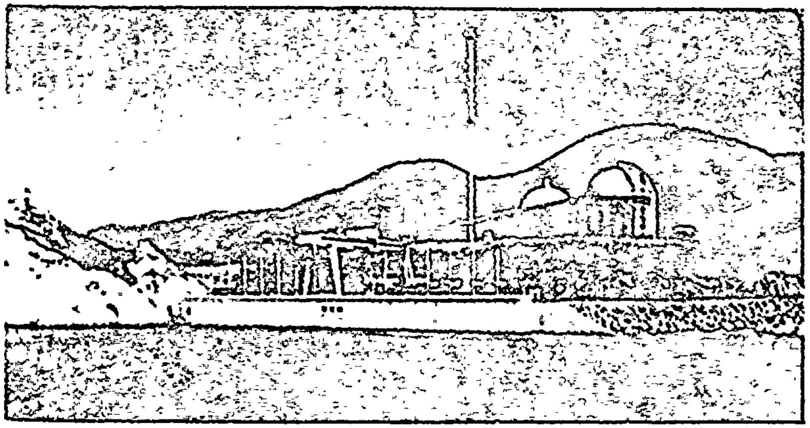


Fig. 21 Combined Pump Stations 1 and 2, Completed

1. Designing armor analytically for back slopes of breakwaters that will be heavily overtopped is not presently feasible. Although progress by investigators in this realm of civil engineering would be a valuable addition to the body of knowledge, the writer believes it should still be backed by practical and thoughtful model verification. Meanwhile, the knowledgeable use of scale modelling is the best available device, and should by all means by employed.

2. Detailed surveys of site bathymetry, suited in detail to the nature of the site, should always be acquired. Time and money spent in acquiring them should be viewed as prudent investment. They are a major factor for minimizing costs of construction and for avoidance of future problems.

3. Where monoliths are appropriate on a breakwater crest, means, and meticulous care in achieving them, to prevent accumulation of energy by compression of air under them need to be taken.

4. Tribars in single layer geometric patterns entail a smaller amount of concrete and a lesser number of units to place than any other precast element commonly used for breakwaters. They can be used to create an excellent armor, provided rigorously strict and alert control of quality in construction is exercised, as it was at Diablo Canyon, by willing contractors and by fair and attentive discipline exercised by the owner's construction supervisory forces. The cost of the latter is one the owner must unstintingly commit itself to bear, or the writer would counsel other armoring techniques.

A BREAKWATER SUBJECT TO HEAVY OVERTOPPING; CONCEPT, DESIGN, CONSTRUCTION AND EXPERIENCE[a]

By Omar J. Lillevang, F. ASCE[1]

INTRODUCTION

Breakwaters on the central coast of California were built in 1971-72, to create a suppressed wave environment at the intakes for water pumped from the sea to condense steam in a very large thermal electric power station. They were conceived with the expectation they would be overtopped several times each typical year, and very heavily during less frequent but expected greater wave storms. Their design, construction, service record and an overhaul episode are described, including discussion of the peculiarities of achieving stable armor on the basinward side of heavily overtopped breakwaters, where massive volumes of water plunge and attack the armoring at depth. Also discussed and illustrated is the phenomenon of compression by waves of air under massive monolithic caps on breakwaters, with description of means employed in the instant case to frustrate this probable major contributor to instability of seemingly immovable masses.

Although the following paragraphs relate to breakwaters erected to create a relatively quiet forebay for pumps, they are equally applicable to breakwaters built for any purpose. Little can be found in existing technical literature on the back slope stability question and the model tests described briefly herein, and the story of repairs found necessary where field conditions were different from what was modelled for design, hopefully will be useful additions. If the description of measures taken, and performance observed, to keep 200-ton monoliths from acting like "Hovercraft", motivates others to share comparable theories and observations, on that score alone this description will have been justified.

---

[a]Prepared for "Ports 77", ASCE Specialty Conference of the Waterway, Port, Coastal and Ocean Division, Long Beach, Calif., Mar. 9-11,1977.

[1]Consulting Engineer, Los Angeles, California

THE SITE

In 1965 Pacific Gas and Electric Company, headquartered in San Francisco, California, began studies and negotiations and eventually acquired a site for a new nuclear generating plant at the mouth of Diablo Canyon, in San Luis Obispo County, California. Almost exactly halfway between Los Angeles and San Francisco, the site is on the open coast of the Pacific at Lat. 35°12.5'N, Long. 121°51.5'W, and lies midway in a bluff-fringed reach of coast 11 miles long where trespassing prohibitions had been strictly enforced by the landowners and the land was used for ranging cattle. Except for deep sea fishermen, and occupants of an occasional low-flying aircraft, few people other than the owners had seen the site. Fig. 1 shows an air view looking ESE toward Point San Luis. It was planned to build pump stations in the smaller cove, just beyond the larger one in the foreground, for taking cold sea water to the steam condensers of the plants. It was also planned to return spent water to the sea at the head of the larger cove. There were no beaches below the sea cliffs. Occasional talus piles temporarily lay between the cliffs and the water, that storm waves soon removed to the abruptly deep bottom. The 60-feet depth contour in the sea lies only 600 feet offshore and the 100-fathom contour is less than 5 miles from the general coastline. It is apparent on Fig. 2, from the coastal locality map, that no geographic features shelter the site from Pacific storms and swells between SSW and WNW, the sector from which waves come sweeping across the Pacific toward Diablo Canyon from storms ranging all the way from south of Chile to the north of the Japanese archipelago.

Fig. 1 Diablo Canyon Site

less than 85 per cent of the theoretical pack. Concrete was pumped under the bottom two ranks of Tribars, a very stiff plastic mix, filling in around all their legs and up to center height of each armor piece in those two ranks. No single tribar then had to resist overturning alone; each was given interlock assistance from its two lateral neighbors, and its feet were embedded in a continuous mat so it could not rise and roll away from the rest. Excellent stability of this measure had been demonstrated in the model tank and it provided a simple practical measure for carrying out.

A different contractor did the repair work, showing great interest and carefully executing the work. Use of pumpcrete was insisted upon, with a diver controlling the discharge hose so its delivery was always submerged in the mass. When "fat" plastic mixes are used and there is only slight water motion, a diver can virtually build "stalagmites" of fresh concrete by this technique that do not slump before hardening begins. This part of the work was intensively monitored by marine biologist divers, and loss of cement to the surroundings was so slight as to raise no objections.

CONCLUSION

Figure 20 shows the completed breakwaters protecting the cylindrical cell gravity cofferdams behind which the first two pump stations were built in the dry. Figure 21 is a photograph of the completed structure for those two plants. The photographs are a conclusion, of sorts, to this presentation. However, a few other points can be made from the experience:

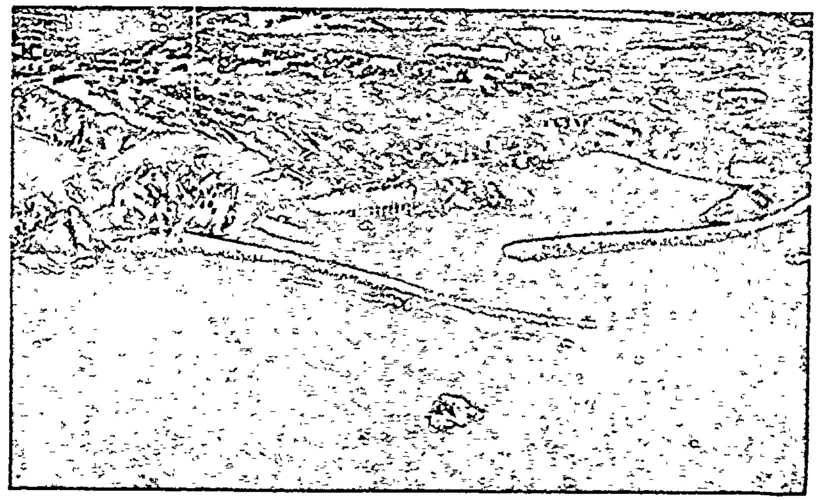


Fig. 20 Completed Basin, Pump Station Cofferdam Unwatered

the unprotected mound at that time by waves, before "B" stone could be laid over the core to hold it. It was suspected the nine "B" stones that were mapped on the basin floor were also of that earlier origin, not the product of the recent episode.

Diagnosis.—The abruptness of drop of the sea floor just basinward of the armor's toe of slope had not been delineated by the surveys shown on Figs. 3 and 5. At that time it was hazardous, virtually all of the time, to operate a sounding boat at that location, and interest was in fact concentrated at the niche sites that were in mind for the pump stations. Cross-sections taken during construction did discover the abruptness of the escarpments, but they were not compiled into a contour map. They had shown, however, that there was not enough ledge left to keep a dike of "B" stone over the basinward toe of the Tribar armor, and none was placed. The terrain features along the damaged area were probably critically involved in the situation.

By counting the number of Tribars visible in a 400-foot length of the seaward side of East Breakwater on the aerial photograph, Fig. 18, it was found the packing on that seaward side was better than 90% of the theoretically perfect geometric pattern. The detailed surveyor's map of Tribars on the damaged side implied the packing had been less than 60 percent in the 80 feet of length that had been damaged on the basin side. Although no Tribars had yet been tilted either side of the 80-foot damaged reach, the packing was shown by the map to be a relatively sparse 79 per cent for another 50 feet to the West and 75 per cent on 35 feet of armor East of the damaged area. Although parts of the "B" stone in the whole 165-foot reach seemed to have been placed too steeply, it was concluded the main cause of the damage was the shallow elevation of the rocky ridge on which the starter row of Tribars had been laid. With the two down-slope legs of the Tribars in the lowest rank resting at -10 to -15 feet, their upslope legs must have extended up to between -1 and -6 feet elevation. This is the region of greatest vulnerability to the plunging jet of overtopping waves, and there was no support from down-slope to keep the lowest Tribars from tilting out of their placed position. When they did tilt, the pieces above began following down the slope, most of them not tipping because they found support before the storm could persist long enough for the damage to progress further.

Prescription.—Three concepts for repair were submitted to model tests, again made by Dr. Raichlen. Before they were tested he constructed the new model with the same placement densities as the surveys showed had been achieved in the original construction. The terrain features on the basin side were incorporated in the model and the hindcast wave characteristics of the storm which had revealed the damage were reproduced for the tests. Damage developed that was very comparable with what had occurred in nature, verifying the model's reliability. The three concepts for repair were then committed to testing. One performed very well, and was recommended and adopted.

All Tribars were removed from the damaged area, a few feet of breakwater length at a time. The "B" stone slope was improved, to assure that at no place would it slope steeper than 1.5:1. Additional Tribars were brought from left-over storage and the armor was re-built to achieve not

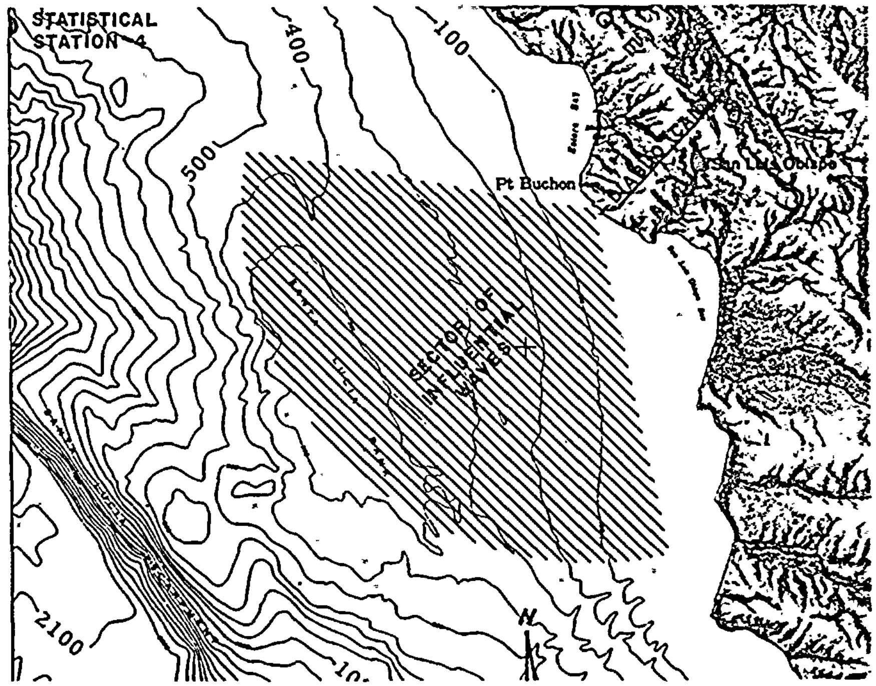


Fig. 2 Coastal Locale of Diablo Canyon Site
Scale 1"=21 Naut.Mi. Depths in Fathoms

SEA WATER·INTAKES

Pump Stations.— Long-range plans for the site when it was acquired were for as many as six generating units eventually to be built. Preliminary planning for the first two units had established a design flow of 808,000 gallons per minute of cooling water delivery to each unit's condensers (183,500 m^3/hr/unit). Preliminary pump station designs showed the intake portals needed to extend to roughly 25 feet below low water and to be 100 feet long, more or less. There were six niches in the cliffs bordering the intake cove, where the eventually needed pumping structures could be "tucked in".

Bottom Surveys.— When the cove was being sounded it proved unsafe to run the survey boats close in, so detailed soundings in front of the intended locations for the first stations were made by hovering helicopters from which weighted marked cables were lowered to the cove's floor and surveyed. The bathymetry on Fig. 3 is compiled from the bathymetric surveys. The six proposed pump structure sites in the niches are shown and the lightly stippled areas in front of each show where the surveys found it would be necessary to excavate ramped sub-

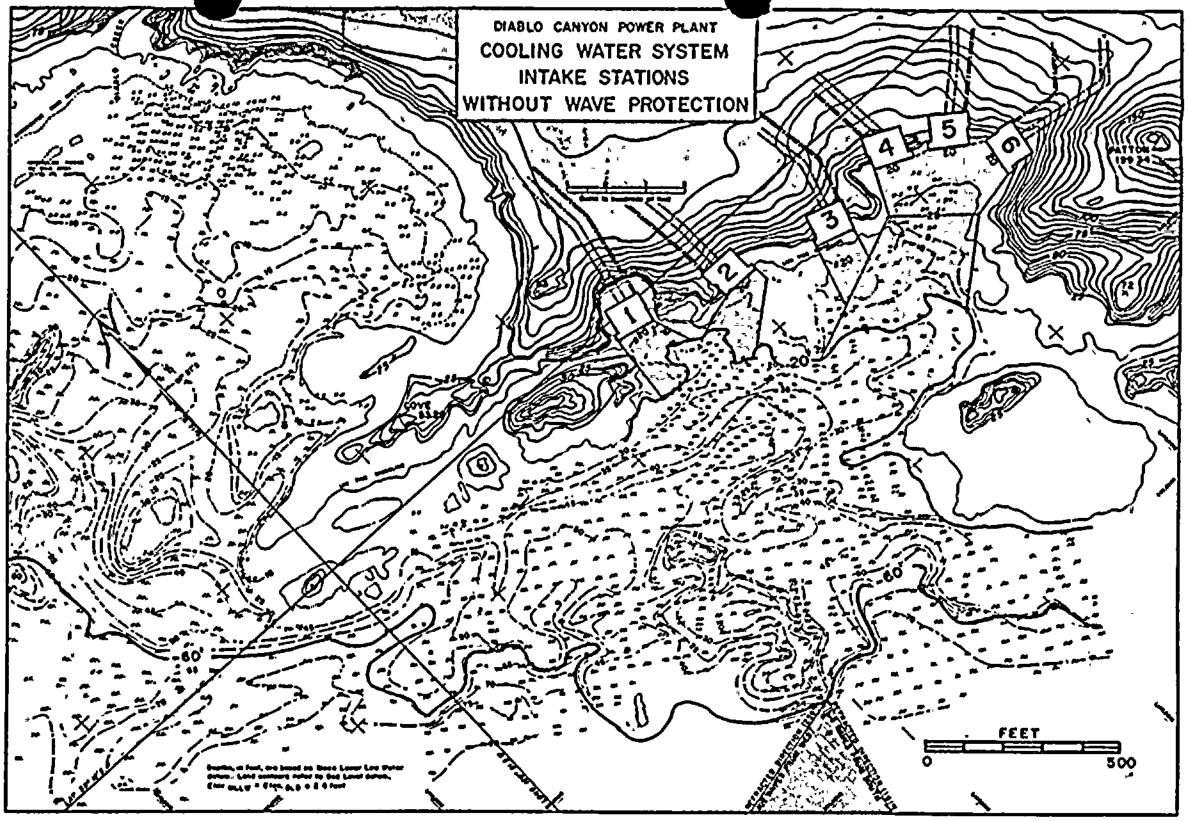


Fig. 3 Initial Concept of Unsheltered Intake Structures

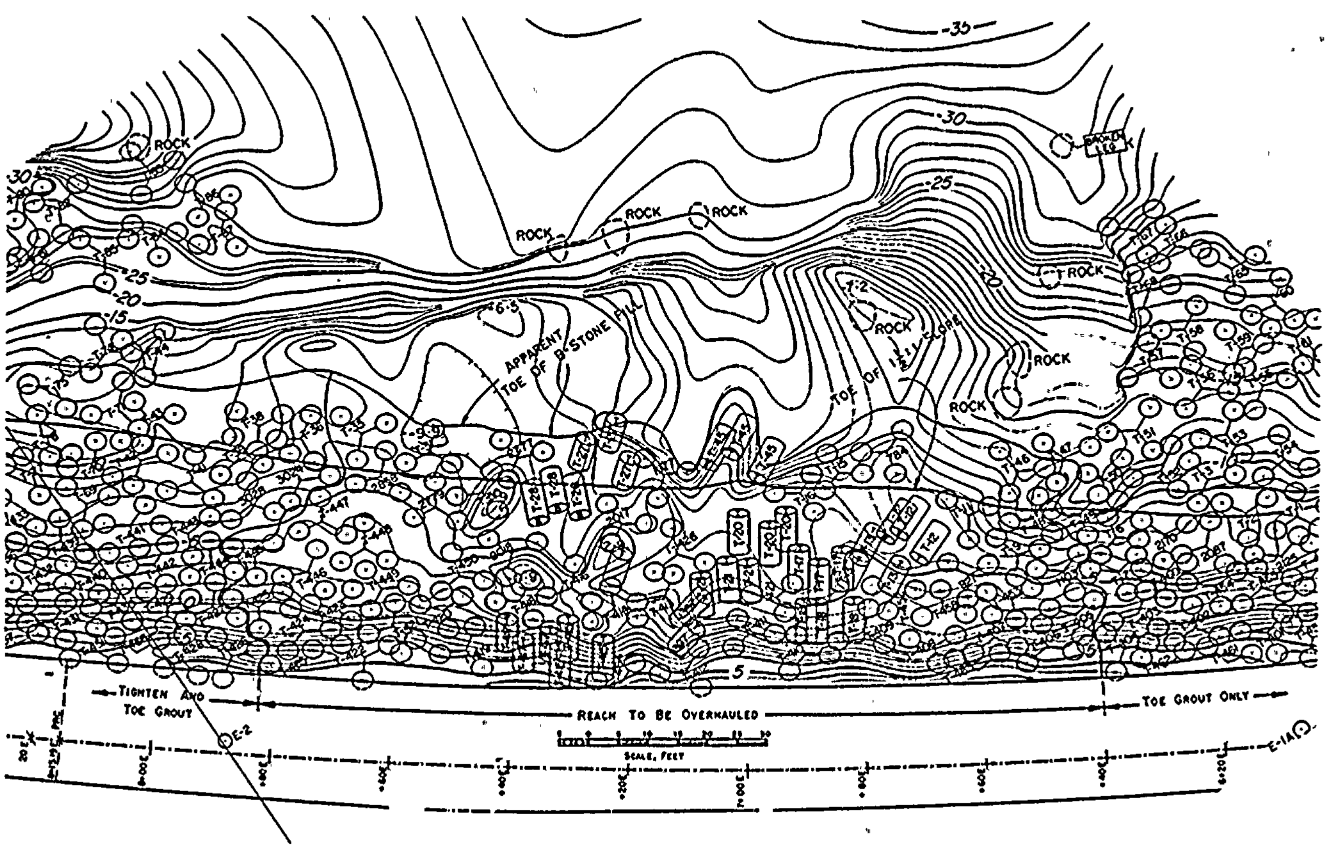


Fig. 19 Surveyed Positions of Individual Tribars, January 1975

Table 1

DURATIONS OF WAVES FROM WNW AT OFFSHORE STATION 4
DURING A TYPICAL SEPTEMBER

Durations Tabulated As Percentages of 720 Hours

Swells and Seas Combined									
Period, Sec. / Feet, Sig. Ht.	4-6	6-8	8-10	10-12	12-14	14-16	16-18	18+	Σ
1-2.9		2.7	8.6	5.1	.8	.8	.5	.3	18.8
3-4.9	.5	.8	4.6	2.9	1.6	.8	.5	.3	12.0
5-6.9			1.1	1.3	.3		.3		3.0
7-8.9				1.6	.8	.3	.3		3.0
Σ	.5	3.5	14.3	10.9	3.5	1.9	1.6	.6	36.8

of that occurrence is shown as a percentage of the number of hours in the month. Swells originating from different storms in separate parts of the ocean can and do arrive simultaneously at the reference point. Thus the occurrences for swells, tabulated as percentages, often add to more than 100. When, as was done in the Diablo studies, the swells and seas percentages are combined, the totals therefore exceed 100 by even greater amounts. Thus, what was used from the statistical reports tabulated for Station 4 actually could be called wave hours, not time. The total for all directions that were reported for September, for example, was 196.1%. That is to say there were 1412 wave hours tabulated for that month.

"Constructed" On-Site Statistics.—Refraction calculations were made to determine the amounts of changes in height and direction of the Station 4 waves that take place as the waves advance over the rising submarine terrain features toward the coast. Using resultant curves of Direction at Station 4 vs. Direction at Diablo Canyon Site, and of Height in "Deep water" vs. Height at various inshore locations at Diablo Canyon Site, it was found that for over 1700 hours in the typical year the "Significant" height of the wave trains would exceed 3 feet, and that "Significant" heights of 18 feet at the headlands of the Intake Cove had to be expected in such a year. The lesser depths within the cove and directly in front of the niches in the bluff reduced the waves that passed the headlands only to the extent of breaking the ones that were more than 15 feet high.

The "Significant" height of any wave train, be it one of large storm waves or a train of low height waves during fair weather, is

available on horizontal movement, because precise control triangulation procedures have not been followed.

POST-CONSTRUCTION OVERHAUL

Discovery of Displaced Tribars.—A sequence of lesser storm waves occurred at the site during the last week-end of December, 1974. On Monday morning after, engineers for the owner noted several Tribars rolled on their sides, just at and immediately below the water surface, midway out the length of East Breakwater. Dr. Raichlen was able to accompany the writer on a quick inspection trip to the site and SCUBA divers were there with a diver's television camera and monitor to assist. In an 80-foot reach of the breakwater, ten tipped-on-edge Tribars could be seen and, generally, the other Tribars in the disturbed area appeared to have moved downslope, away from the cap, distances of a foot or more. None seemed to be broken, and the divers were able to determine none had rolled out of the breakwater to the basin floor.

Survey and Inspection of Damage.—Figure 18 is part of an aerial photograph by Pafford and Associates, Surveyors, taken a few days later. That firm was engaged to make detailed surveys of the damaged section, so informed diagnoses could be attempted and prescriptions could be made. The surveyors used two wet-suit divers, one equipped for phone communication, to assist in survey positioning by instrument of every Tribar on the basin side in a 240-foot reach of East Breakwater, and to examine the basin floor for rock debris, if any, and to report in detail any other evidence of interest that might be discovered. The X, Y and Z coordinates of the center of the end of each leg of each Tribar in the survey area was determined from slope angles and horizontal angles measured by each of two theodolites, set up over monuments on the cap. To center the rod that was observed from the two instruments, the phone-equipped diver found each leg and slipped a light 3-spoked centering frame on it. At the frame's hub a universal joint from a mechanic's socket set had been welded, and the rod was set in that joint. When the frame was in place, the diver phoned that situation to one of the instrument men, who in turn told the other man in the water. The latter then swam the rod to plumb, guided by a bull's-eye level attached to the rod. Both instruments were immediately brought to bear on a band attached to the rod at a fixed height above the Tribar leg's center. Figure 19 is the map derived from the survey. Except for the tilted Tribars, only the tops are delineated. Knowing from the survey data what X, Y and Z were for the top of each leg of a Tribar permitted calculating its attitude angle and, therefrom, the elevation and position of the lower end of each leg. From this computational procedure, elevations for plotting the contours on which the Tribars rested were derived.

Nine "B" stones were found on the basin floor but no Tribars had rolled there, in spite of the fact 10 to 15 foot high escarpments dropped off into deeper water just past the toe of the breakwater. There was apparent core material, pieces of rock 4 to 6 inches nominal diameter, on the basin floor, but marine life on that material suggested it had been there since construction was under way, presumably washed off

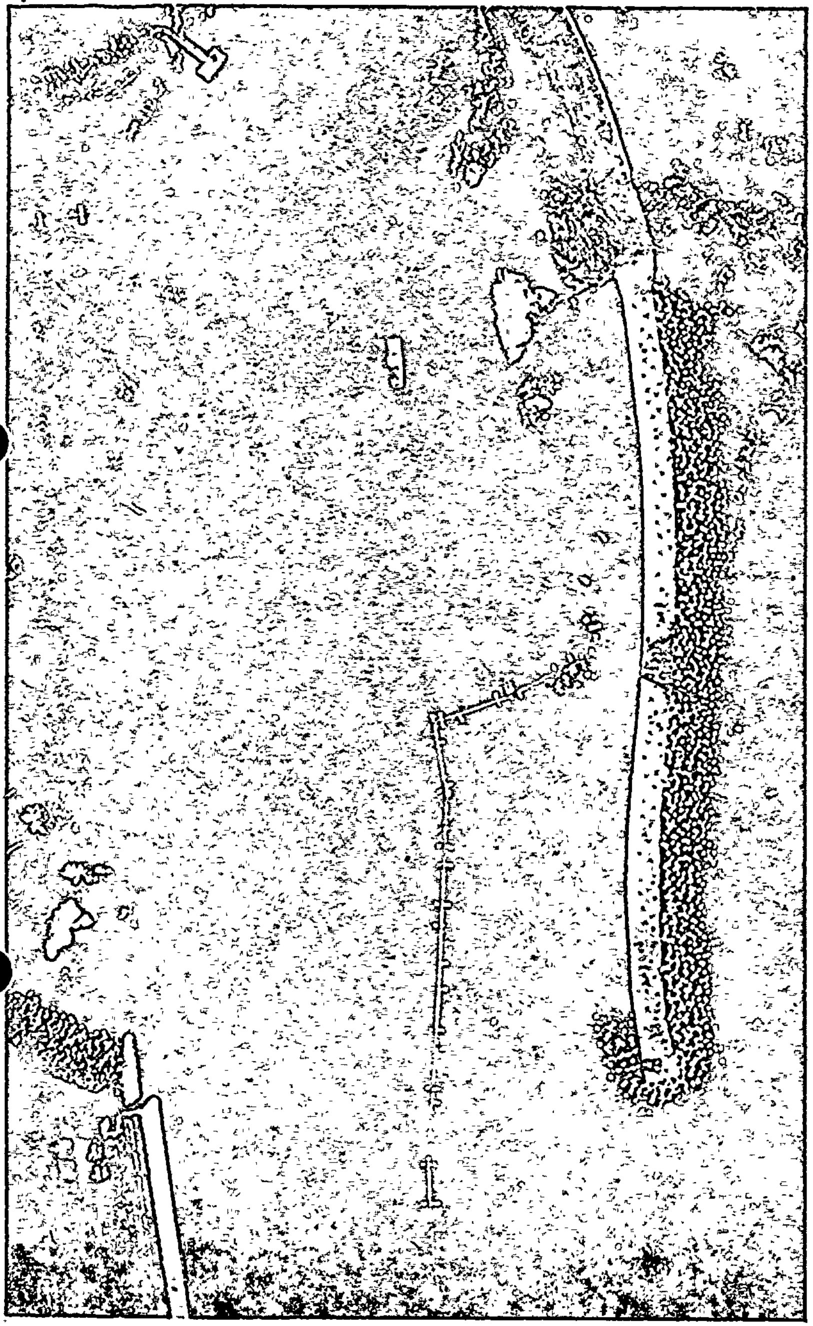


Fig. 18 East Breakwater, 24 January 1975

marine channels from the 20 feet depth contour to the portals, to enable flow of water from the cove toward the pump stations. The bottom proved to be rock and its features to be so rough and intermittent that definitive contours could not be interpolated from the soundings, even though the soundings were densely spaced. Reconnaissance dives found 15 to 20 feet vertical or overhanging escarpments in the submarine terrain, and many pinnacles and ridges and mounds. Deep crevices held sparse accumulations of aggregate-size gravels, but the divers found no sediments finer than pea gravel, and hardly any that fine. Contours are shown on Fig. 3, but they do not show the terrain in detail. They indicate gross forms only, to help in visualizing the area.

Tolerable Wave Motion.— The preliminary plans with the tucked-in pumping stations assumed waves at the intake portals would have been reduced from their heights in the open sea to 5 feet or less at the niches. The circulating water system designers considered such wave amplitudes acceptable. Nothing in a long series of former assignments to devise cooling water source systems had prepared the writer for anything except pumping system designers who insisted wave motion at the intake portals should be well under a foot. With quick changes in suction level, the traditional battery of pumps might otherwise alternately race and labor as their suction lift varied up and down, with each wave. Small changes in that level keep speed variations minimal for conventional pump installations, and transient surge pressures in the system then remain tolerably low. The new Diablo scheme departed from the traditional concept of a battery of smaller pumps. Instead, only two pumps per generator would be used. They had to be big to deliver over 400,000 gpm each at a total discharge heads of nearly 100 feet. Consequently their rotating parts, the pump impeller and the motor's rotor, would store large resources of momentum when pumping. Then when the suction level varied quickly there could be negligible slowing or speedup of pump rotation, the flywheel effect could keep surge pressures insignificant.

WAVE STUDIES

Published Deep Ocean Statistics.—With the preliminary plans established, the power company retained special services to derive wave loads for the pump structures and to provide advice concerning concepts and construction methods. The first steps were directed toward compiling a wave climate description for the site. Hindcast wave statistics at seven arbitrarily spaced positions, in very deep water several miles off the California coast, were prepared by contract a number of years ago for the Army Corps of Engineers by National Marine Consultants (2). The nearest of the 7 arbitrarily located statistical stations to the Diablo Canyon site is Station 4, shown at upper left on Fig. 2, at Latitude 35.5°N, Longitude 122°W. Depth at Station 4 is approximately 800 fathoms. At that depth neither wave direction nor height is affected by the sea floor's topography; all waves except tsunamis are "deep water" waves.

Wave data are tabulated in the Army's report for each of the 12 months of a "typical" year, showing occurrence of seas and swells separately, and in 22.5° directional groups. Height vs. period subdivisions are made in the tabulations; for each subdivision the duration more costly, would have been a permanent widening of the whole breakwater, placed on the basin side of the cross-section. However, such a change would have encroached on the basin unacceptably, raising velocities through the entrance and otherwise frustrating desired functional objectives, so the alternative was not acceptable. In other circumstances, however, the economic values possible in use of larger quantities of material for special reasons should not be forgotten.

Quantities and Breakage.—The West Breakwater was armored by 2,300 Tribars of the 21.5-tons size and 300 of those weighing 37.1 tons. Thirteen hundred of the smaller Tribars went on East Breakwater. Casting of Tribars was paced at 40 per 8-hour shift. Average placing rate was 30 pieces per 10-hour shift; a maximum shift put 71 in place in 10 hours.

Sixteen Tribars were broken during construction of East Breakwater and 20 of the smaller pieces at West Breakwater, an overall average of 1 per cent. Only six of the large tribars broke, all in one event. They were on a narrowly identified area, extending from the crest to near the basin floor. Apparently a key armor piece had been supported by a sharp point of rock, which broke off after some time and caused a sharp downslope consolidation or shift of the armor above. On the order of 15 Tribars shifted and impacted one another. All armor pieces were marked when cast with identifying numbers, and each one's history had been carefully kept. A review of the records showed a report, by a night shift inspector, that several of the large Tribars had been dropped hard at the casting yard when a crane oiler, learning to operate, was loading Tribars onto the carrier trailers. The crane tilted on its tracks under the load of the large pieces, and the nervous learner had let go his hoist brake several times, in fear otherwise his machine would be turned over. All of the Tribars that broke in the subsidence event bore the numbers of six of the pieces the inspector had reported being roughly handled.

Approximately 277,000 tons of core material, "D" stone, went into the two breakwaters and 185,000 tons of "B" and "E" stone. Average placing rate for all classes of stone was 1,500 tons per 10-hour shift. The best 10 hours saw 3,500 tons placed.

Total payment to the contractor was $11,900,000. It included costs of winter shutdown and of removals of temporary work in the spring, when operations resumed.

Settlement.—The breakwaters were completed and accepted as of January 21, 1972. Brass cap monuments were set in April 1972, and periodically their elevations have been measured. By January of 1974 the lowest cap elevation was +19.36 feet. Stated as a per cent of change with regard to the vertical thickness of fill beneath the cap, on centerline, no consolidation exceeded 1.5 per cent in the two-year "seasoning" period. Visual examination of the construction joints along the concrete caps show only two or three to have opened since they were made, and in those instances the opening is no more than the thickness of a doubled file card. No differential vertical movement at joints between crest blocks is visually apparent after 5 years, even at the joints that have opened slightly. There are no definitive measurements

precisely defined as being the average height of the highest one third of any observed succession of waves. If the observed sample is 100 waves, one of them statistically can be expected to have a height at least 1.67 times as high as the "Significant" height. Thus during the 1,700 wave hours in a typical year that "Significant" heights of 3 feet or more could occur it was apparent that individual waves of 5 feet and higher would be experienced over that same duration of time.

Need for Wave Shelter.—Such wave conditions suggested frequent shut-down of construction work in the sea, and the trains of higher waves that had to be expected each year were clearly capable of wreaking heavy damage on incomplete work. It was reported, and the project owner agreed, that no scheduled construction completion dates could be relied upon and the cost of completing construction of work in such a wave environment couldn't be defined, if it were attempted without some type of wave screening in front of the work site. It was also apparent from the new wave studies that operating conditions would be unsatisfactory a significant amount in a year's time if shelter were not provided.

Figure 4 illustrates important aspects of the wave climate calculated for the site, and the mitigation available with breakwaters. The maximum "Significant" heights of refracted waves at the site were extracted from the calculations and plotted against their respective periods, separately for each 22.5° sector centered West, West Southwest, Southwest, South Southwest and South. Those plots appear as five skewed grid graphs. The upper curves show maximum "Significant" heights for each wave period at the cove's headlands. The dot-shaded upper curves are the same waves at the -20 feet contour in the cove, if no breakwaters were built. The two curves are identical, for practical purposes, except where the larger waves would break at depths of 20 feet or less.

Alongside to the right of each $H_{33_{max}}$ vs. T graph is a duration plot of "Significant" heights for all waves originating in the represented sector of deep water directions. At the bottom of Fig. 4 appears H_{33} vs. Duration, combining the wave durations from all of the 5 sectors. The line above the hatched area represents height durations if breakwaters were provided and the upper line is height durations if no breakwaters were provided.

THE REVISED CONCEPT

Incremental Construction Infeasible.—It was apparent the wave screening needed for the initial units' pump stations would be needed for all the pump stations the full development of the site would require. No practical way appeared feasible for building the first pump stations behind a partial shelter and plan on increasing the shelter each time a future unit was needed. Therefore it was decided the breakwaters should be designed and built for sheltering the whole intake cove, i.e. to be built now for future needs as well as for the immediate.

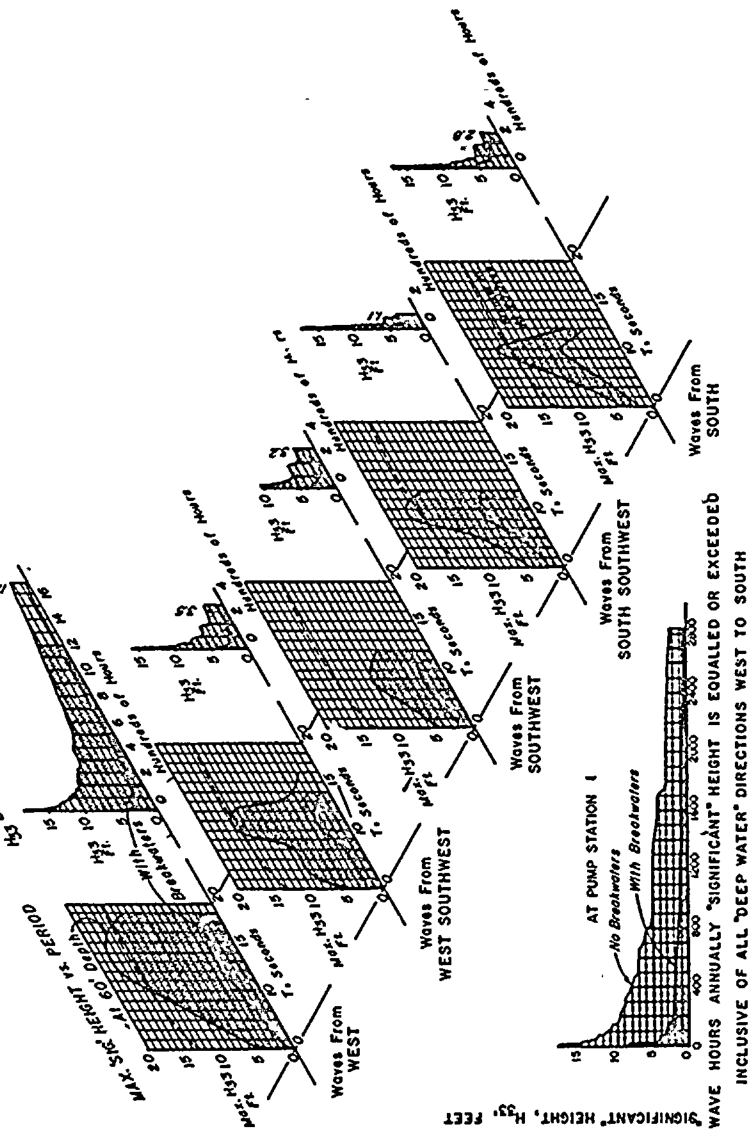


Fig. 4 Heights and Durations of Waves at Pump Station 1, With and Without Shelter

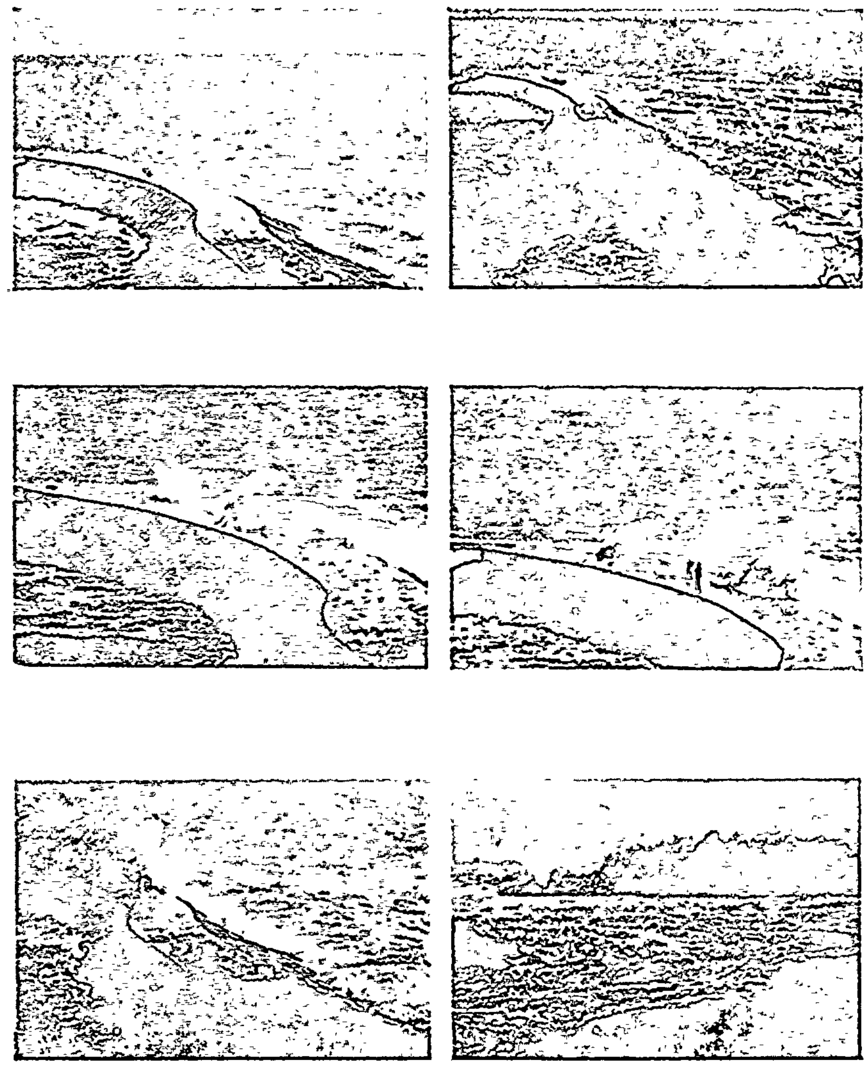


Fig. 17 Wave Storm 10 November 1971, West Breakwater
Showing Pressure Relief Through Cap Block Vents

$H_{33} \approx 10'$, $T \approx 14$ Sec. Tide $\approx +4.5' \pm$

cable clamps. Core material was not released from the cable mesh pouch until the crane operator sensed the load was against already-placed material, and properly positioned to fill-in a low area. Then two of the four suspender cables were let go, and the net was slowly pulled up at one side to release the material. The technique avoided nearly all washing of fines from the core mixture, because it nearly eliminated free fall of the mixture through the water. The "B" and "E" stone had no fines, of course. Use of the net for that material let a whole truckload be placed in one swing of the crane, so the net's use for that rock was a cost advantage for the contractor.

PERFORMANCE OF VENT STACKS

Before the contractor was quite finished building West Breakwater, (the East structure was finished) a wave storm occurred. Typical of many occurrences on the California coast when swells from distant storms reach the shore, the local weather was clear and bright, with light breeze. These swells approached at the basin from about 265°, just below west, at a tide stage of about +4.5 feet, with 14 seconds period and a "significant" height estimated by hindcast at 10 feet. The late Clyde A. Hollcraft was on site and photographed the occurrence for the writer. Noteworthy is the performance of the cap vents, which is graphically evident in the series of 6 photographs of Fig. 17. Although looking like water in most of the photographs, the emissions were mostly air, carrying moisture along as spray particles or froth. Careful examination of the original slides detects high velocity air also emitting horizontally from under the cap toward the basin, relief of pressure through the 5.5-foot "window" of porous "B" and "E" stone that had been provided above the top of the relatively low porosity core and the under surface of the cap. It was a most satisfying event. The writer was comforted by the demonstration that venting freedom had been accomplished, and glad the contractor and the inspectors were still there, to see the nagging for careful compliance had been for a real purpose.

CONSTRUCTION

Casting and Placing Tribars.— The Tribars were manufactured and stored by the contractor on 15 acres of fill in Diablo Canyon, 1.6 miles via on-site roadways from the intake basin. The fill had been constructed as the site for the generating plant's main switchyards. When needed for placement on the breakwaters, Tribars were loaded by a small crane to low multi-wheel trailers and taken to the placing crane on the breakwater. The placing crane was a very large pneumatic-tired rig, P&H 2500, using outrigger pads for overturning stability. Reaches with the 21.5-ton Tribars were as much as 170 feet to place the toe of the terminal 3:1 slope. The prescribed final width of the breakwater crest was too narrow for the outriggers. Therefore the contractor steepened the upper part of the seaward slope to create an interim widening to make a work platform. Tribars were brought up in the initial phase only to the level where the steepening began. When all Tribars were acceptably in place up to that level on the breakwater the steepening rocks were removed, beginning at the seaward end and working landward, and the remainder of the armor was placed as the crane backed away. An alternative, that was considered by the contractor to be no

Basin Layout.—The entrance to the contemplated intake basin needed to be large enough, hydraulically, to convey the ultimately needed flow of water at velocities that would minimize ingestion of flotsam, such as storm-uprooted seaweed rafts, and silts carried along the coast by ocean currents when storm discharges of the adjoining creeks were delivering sediments to the sea. Yet, the entrance also needed to be as narrow as possible, in order to minimize wave penetration. A maximum velocity of 2 feet per second at lowest predicted tide was selected as the basis in the layout studies for the entrance geometry. Figure 5 illustrates the breakwaters as they were located for making detailed design studies. Both breakwaters had been delineated with straight alignments during early comparative studies, but essentially in the same positions as the curved layouts of Figure 5. Esthetics reviews produced requests for sinuosity in the breakwater placements. Trial layouts with curved alignments showed lesser volumes of stone would be required, and the curved layouts were entirely acceptable from a functional viewpoint. A cross-hatched sector below the entrance shows the extremes in direction that waves of 10 seconds period would come to the 60 feet depth curve.

Design Waves.—The design wave for West Breakwater was identified by the data, and from Fig. 4, as having a period of 14 seconds and a "Significant" height of 18 feet, i.e. H_{01}= 30 feet. As shown on Fig. 4, its direction in deep water is 270° (West). Refraction turns it so its direction at West Breakwater is approximately from azimuth 250°, essentially perpendicular to the seaward end of the structure. East Breakwater has some protection against Westerly waves from the overlapping extent of West Breakwater, but that was a lesser reason for the overlap. The primary purpose of extending West Breakwater well seaward was to force floating storm-uprooted and wind driven seaweed rafts, and suspended sediments from watershed freshets, away from the basin entrance that otherwise could be entrained by accelerations of velocity into the basin. It is of interest to note the wind-driven streaks of sea foam on Fig. 1, in this regard. With relatively small variations in direction, such indications of drift are to be seen on photographs of these waters taken at many seasons of any year.

The diffracted height of the westerly design wave reaching the East Breakwater trunk is a very impressive wave. After diffracting behind West Breakwater, its 30-foot design wave is still 15 feet high where it is intercepted by East Breakwater, about midway between that structure's terminus and its islet root. However, as shown by Fig. 4, an even higher wave against East Breakwater had to be anticipated, coming from South, with "Significant" height at the site of 17 feet and 11 seconds period. Its 1 in 100 probable height would figure to be about 28 feet. However, East Breakwater's design wave would reach the structure at a depth where breaking begins. This condition caused its design height to be adjusted upward slightly to 28.7 feet.

Breakwater Armor.—Using the average height of the highest 1% of waves in the design wave train, i.e. H_{01}, for calculation of armor characteristics, preliminary comparisons using Hudson's Formula indicated armor sizes for each of several assumed seaward slopes and each of three armor types as shown by Table II:

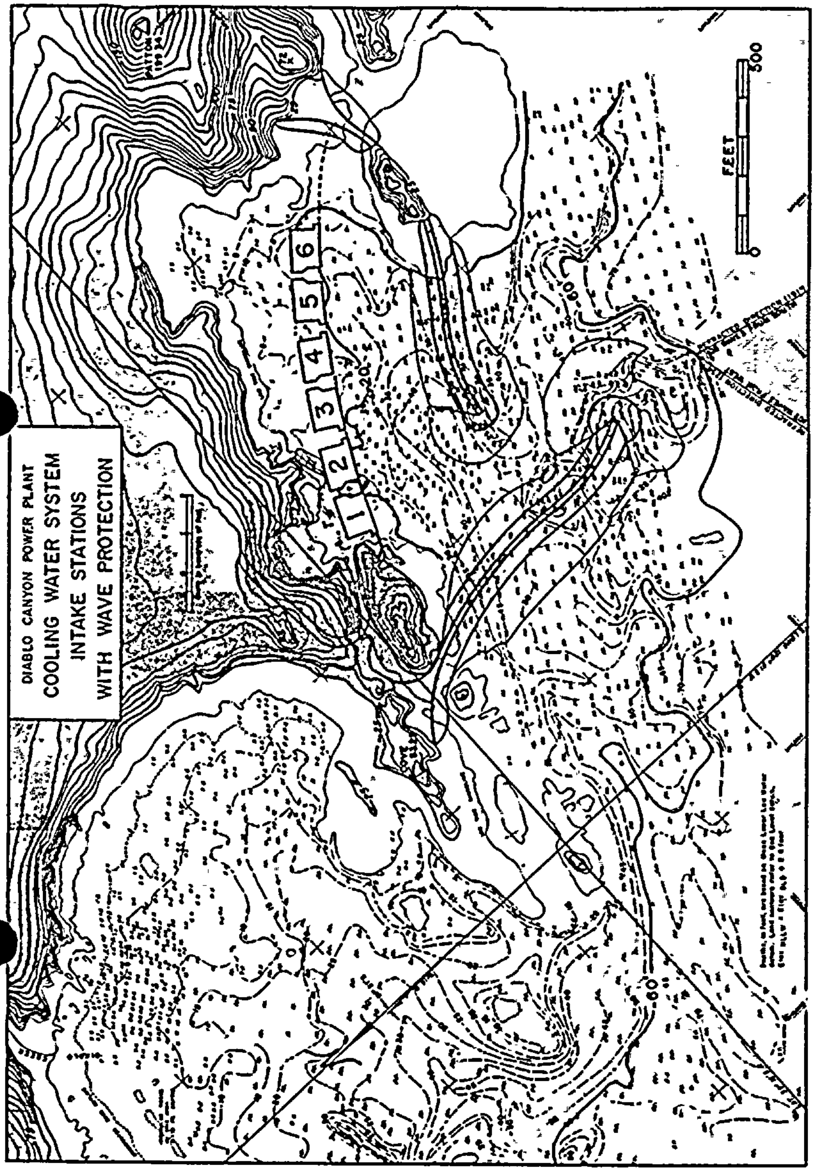


Fig. 5 Breakwater-Sheltered Intake Basin Concept

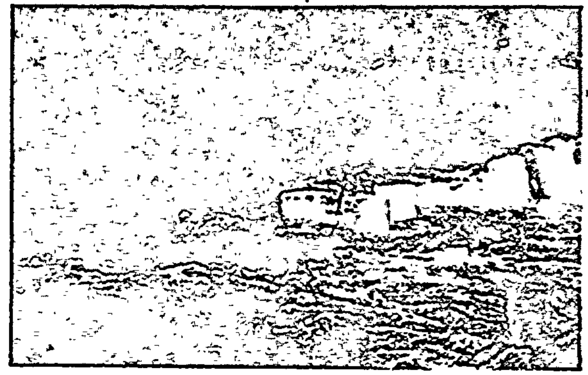
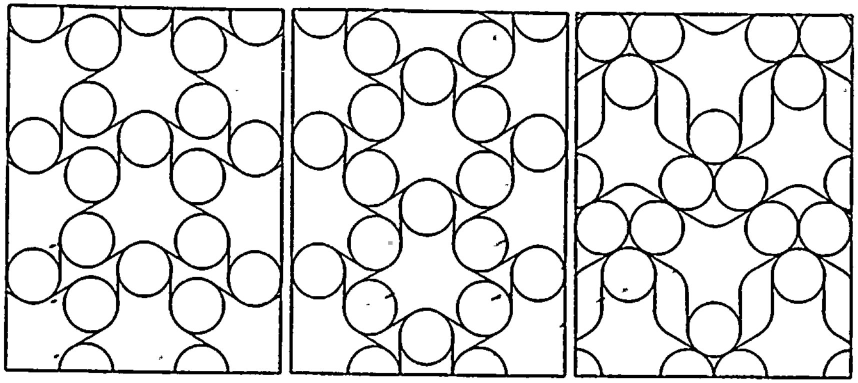


Fig. 14 Diver Directing Placement, East Breakwater

Fig. 15 "B" Stone and 21.5-Ton Tribars, East Breakwater

Fig. 16 Seaward Armor on West Breakwater, 21.5-Ton Tribars

"....expend at least as much effort on submerged placement as on placement above water. Conscientious and meticulous effort, nothing less, will be the required standard of performance, with recognition that results below water will not produce quite so uniform a pattern of placement as is achieved above water, with equivalent effort expended in each region."



A B C

Fig. 13 Geometric Patterns for Tribar Placing

Further, it was required that no Tribar would rest tilted more than 15 degrees from the generally prescribed surface of the armor zone in its location, nor could adjoining legs of two adjoining Tribars stand offset, in the direction of the leg axes, by more than 2 feet in the case of the 21.5-ton Tribars and 2.5 feet if the Tribar weighed 37.1 tons; i.e. less than 30 per cent of the Tribar's height. In the event the contractor did not meet these limits with any Tribar, he was obligated by the contract to re-work the "B" or "E" stone under it that was frustrating compliance. Thus it became of critical financial importance to the contractor to prepare his bedding stone with care. Figures 14, 15 and 16 show results that were achieved above water. In Fig. 14 the diver can be seen who controlled placement of each Tribar. No piece was released from the crane's support until the diver was satisfied he would not have to return later, when owner's diver inspectors made their weekly examinations, to direct a re-setting. At first the diver directed only the underwater placements. Shortly it became apparent to the contractor he would have a better overall control of placing quality if the diver also directed the above-water Tribar placing, and that expansion of duties was made.

Clean Placement of Core.—The owner was adamant, that the water surrounding the construction activities should be kept free of turbidity to the maximum extent possible. The contractor manufactured nets of wire rope with mesh size of about 6 inches, to use as pouches for placing core and "B" or "E" stone. The mesh intersections were fastened by

Table II

Preliminary Comparison of Armor Types, West Breakwater

$H_{01} = 30'$

Type	K^*_d	Unit Wt. #/cu.ft.	Slope	Armor Wt. Tons	Description	Minimum Zone Thickness
Quarrystone	10	160	2:1	32.0	5.4'x13.6'**	14'
			5:2	25.6	5.0'x12.6'	13'
			3:1	21.3	4.7'x11.9'	12.5'
Tribar	20	145	2:1	24.1	7.42' high	7.42'
			5:2	19.3	6.89' high	6.89'
			3:1	16.1	6.50' high	6.50'
Dolos	30	145	3:2	21.5	12.4' high	17.3'
			2:1	16.1	11.3' high	15.7'
			5:2	12.9	10.5' high	14.6'
			3:1	10.7	9.8' high	13.7'

* Converted from experimental values related to H_{33}, with further modifications as deemed appropriate for project conditions.

** Limiting length, when ratio of length to least dimension limited to 2.5.

The comparisons rather conclusively ruled out any further considerations of quarrystone armoring for West Breakwater. A similar procedure for East Breakwater brought the same conclusion. The stones would simply be dimensionally too large to transport on highways or by rail, due to vehicle width or clearance limitations, and would probably force the contractor to use cranes at least a class larger for placing pieces of those weights at the distances that structure slopes and depths would require.

Except to those who were familiar with the proceedings of the Tenth International Conference on Coastal Engineering, held at Tokyo in 1966, American engineers were generally not aware of the South African armor piece called "Dolos". Then, during the early phases of the Diablo design work in 1969, an impressive article by the inventor and donor of the Dolos, E. M. Merrifield, M(SA)I.C.E., appeared in "Civil Engineering". Detailed interviews the writer had in South Africa with Mr. Merrifield, with consulting engineers in that country who had designed and inspected construction of Dolos-armored breakwaters, with contractors who had built them and with agencies who owned them, and visits to all existing Dolos-armored breakwaters in South Africa, save one, led to the decision an alternate design for Diablo should be prepared for comparative bidding, using Dolos armoring. That decision was not approved, however, until after hydraulic model studies of the Tribar armor were nearly finished. Rather than delay taking of bids, the owner decided similar model studies for the Dolos-armored alternate would be made after bidding but before award, if submitted proposals favored the Dolos. This was not the outcome, so the Dolos design for Diablo was not subjected to model verification.

Acceptability of Overtopping.—One extreme had been identified, namely trying to build and operate the intake system without wave suppression. The opposite extreme seemed to be building breakwaters so high the 1% wave would not overtop. Armor model studies done in 1966 by Vanoni and Raichlen (5), had recorded run-up, showing variations with both relative depth and relative wave height, for Tribars laid on a 3:1 slope. That work was the basis for preliminary estimates of run-up and related breakwater heights at Diablo. The no-overtop crest for the 30 foot West Breakwater design wave was estimated to be at +38 feet, if the armor sloped at 2:1. For East Breakwater, with the design wave of 28.7 feet and T = 11 sec, similar preliminary estimates put the runup at 1.2 times the wave height, because depths of the toe at the 2:1 slope under design conditions were at the wave's breaking threshold. The resultant preliminary description for a no-overtop East Breakwater became +42 feet. It was known from the earlier concept studies that breakwaters at the intake cove with crests at +33 feet would contain on the order of 90% more material than structures on the same alignments with crests at +20. Given the unique capability of the very large rotating parts of the cooling water pumps, to minimize or nullify surges usually developed when suction levels fluctuate quickly, the owner considered it unjustified to invest in a completely sheltered intake basin. It was decided heavy overtopping, perhaps as much as 9 feet depth over a crest at elevation +20 feet at each breakwater, should be investigated.

Relocation of Pump Stations.—It was also decided the pump stations should be relocated along approximately the -20 feet sea floor contour, as shown on Fig. 5. That decision could save the substantially high cost of underwater drilling, blasting and subaqueous excavation of rather long approach channels, from the -20 contour to the niches. Perhaps more significantly, the relocations virtually avoided risk to sea life that underwater blasting sometimes causes.

Revising Wave Durations Diffraction Model.—The earlier diffraction analyses, on which the height-duration data for wave motion at Pumping Station 1 had been based, did not include estimates of the added turbulence overtopping waves could contribute to the basin. Therefore recommendations were adopted, that a model of the basin should be built and operated to give information on that missing aspect, and generally to verify also the suitability of the earlier estimates insofar as non-overtopping waves were involved.

An undistorted model of the intake basin was built at 1:100 scale ratio, at the University of California's Richmond Field Station. Professor Robert L. Wiegel, F. ASCE, undertook the work under contract to the writer. Model construction and operation was carried out by employees of Pacific Gas and Electric Company.

Monochromatic waves were directed toward the model from three directions, with 5 periods from each direction. Figure 6 shows a 12-second wave from SW attacking the model.

road fill had escaped the preventive mesh, it was removed, by jetting and by picking it away.

The Cap.—"Sonotube" paper cylindrical forms, 12 inches in diameter, were set vertically within the confines of the side forms for the cap, on an approximation of an equilateral spacing of 10 feet. Spacing was altered from 10 feet when necessary, or tubes were allowed to depart slightly from plumb, to fit their lower ends into appropriate voids recesses between the stones. Wads of degradable material, rags or burlap or straw, were put in the bottoms and around their exterior contact lines with the stones, to prevent concrete entering the void or the tube. Then the mesh was re-laid and a very dry mix of concrete was placed over it, less than a foot thick. When that starter layer, actually acting as a bottom form, had gained sufficient strength, the cap was poured to its full depth, a minimum 7 feet. Vertical joints in the cap block were not allowed to be spaced less than 20 feet apart, to provide that no individual monolith would weigh less than 200 tons. Vertical movement between adjacent sections, to accommodate to possible variations in consolidation of the breakwater underneath, was made possible by the nature of the shear key formed at each joint. In plan view, a trapezoidal groove in one block matched its negative in the adjoining one. The trapezoidal tongue and groove were designed 18 inches thick with base dimension 13 feet and 45° convergence. It permits vertical movements between monoliths but resists, as a keyway, horizontal displacements at the joint. The contractor was not allowed to build the cap until after all Tribar armor was acceptably in place. Further, where part of any upper-rank Tribar extended beyond the prescribed face of the cap, it was required that bond or interlock between such Tribar and the cap be prevented. The concern was that consolidation down-slope of the Tribar armor should not leave individual pieces behind, locked away from the matrix by bond or other restraint.

Placement of Tribars.—Three different geometric patterns have been used for placing single-layer Tribar armorings. They are illustrated by Fig. 13. Pattern A might be called the "Reversing Files" placement. It was used in the Diablo model tests. A theoretically perfect pack in this pattern (a "100%" pack) would use a gross area per Tribar equalling $5.0079D^2$, where D is the diameter of any cylindrical element of the Tribar. Pattern B, called the "Hexagonal Modular" placement, also uses $5.0079D^2$ units of area. It was specified for construction, because all pieces are lowered to placement with the same orientation, minimizing confusion during construction operations and giving a double "heel-in" downslope for each piece. The third pattern, C, called "Reversing Ranks", occupies more gross area per piece, $5.6329D^2$. Using it would reduce Tribar requirements 11 per cent, but they would interlock less effectively, could rotate out of pattern more easily, and for those reasons it was not favored.

Specifications for Tribar placement were rigorous. They required putting the pieces, as nearly as practicable, in the "Hexagonal Modular" pattern shown at B in Fig. 13. The specifications obligated the contractor to:

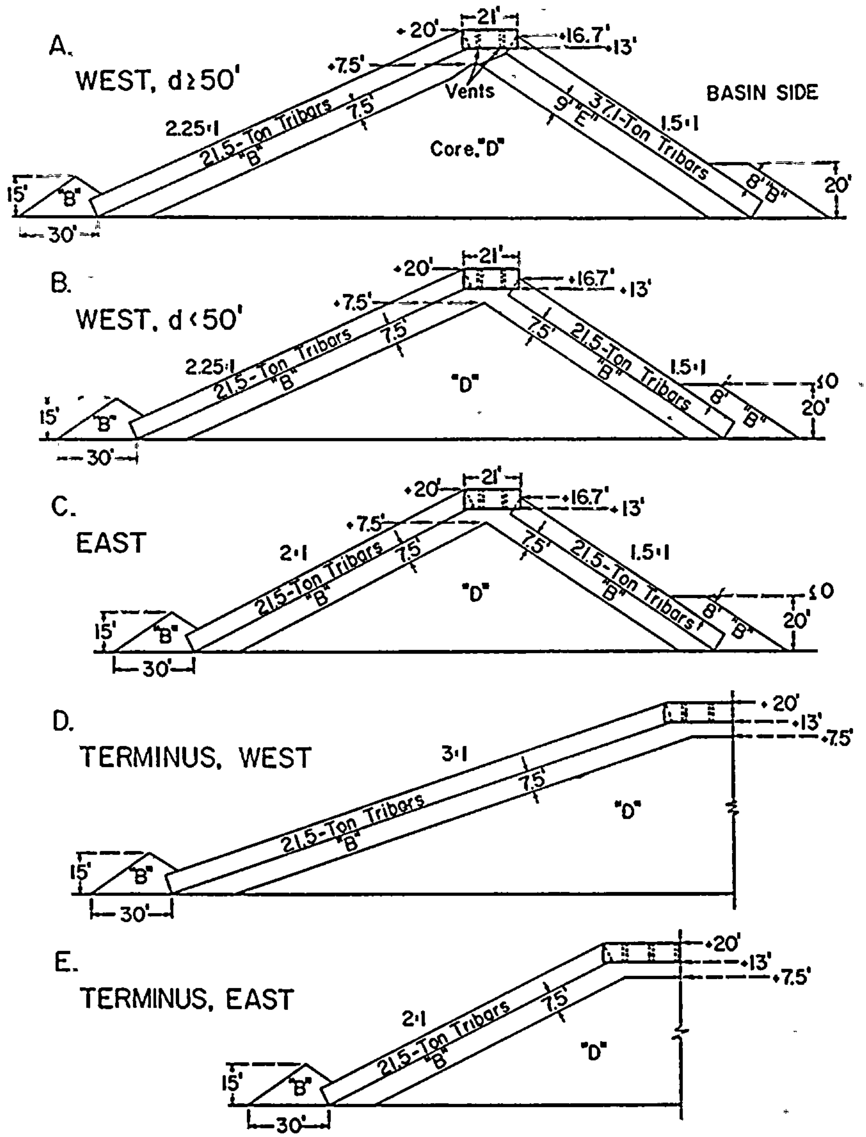


Fig. 12 Constructed Cross-Sections

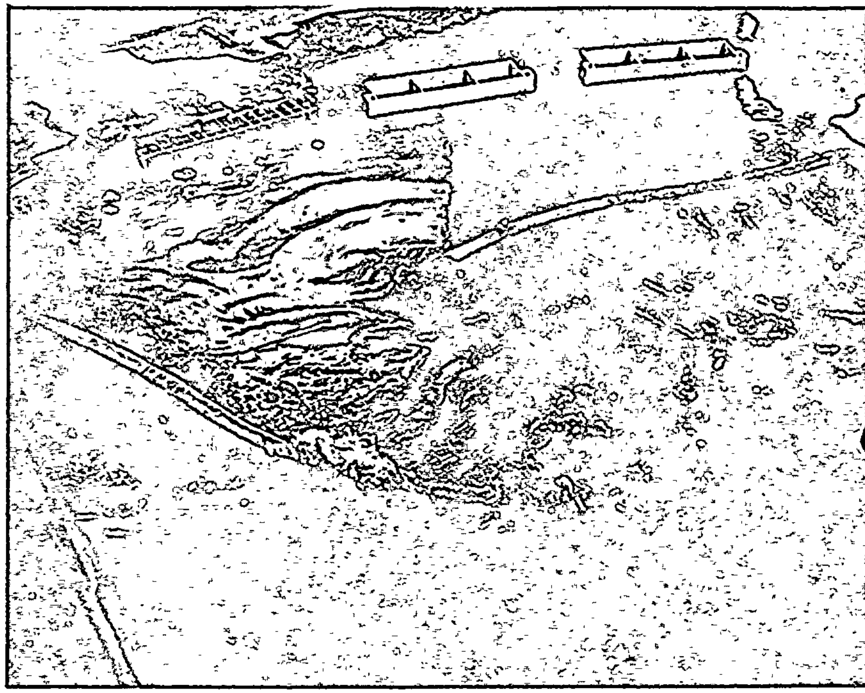


Fig. 6 Southwest Origin Wave in 1:100 Model T_p=12 sec.

Comparison of Results.—Wave motion was recorded at several locations within the model basin and compared with the heights of the waves at the 60 foot depth curve just seaward of the basin entrance. Curves were derived from those recordings, generally fitted by considering the least reduction in wave height found among several runs for each period, to refine the height:duration estimates from the preliminary studies. It was not surprising that the model-related duration data showed a slightly bigger highest wave at Pump Station 1, because the preliminary studies had not added the uncertain effect of aggravated heights inside the basin caused by re-forming of waves, from the cascading water overtopping the breakwaters. That the number of hours when H_{33} would be 3 feet or higher was so little different in magnitude from the preliminary estimates, 200 hours compared with 150, was very gratifying. Fig. 7 shows both duration curves. The area under the original distribution curve is hatched by lines running downward from left to right, while under the model-related curve they run upward from left to right.

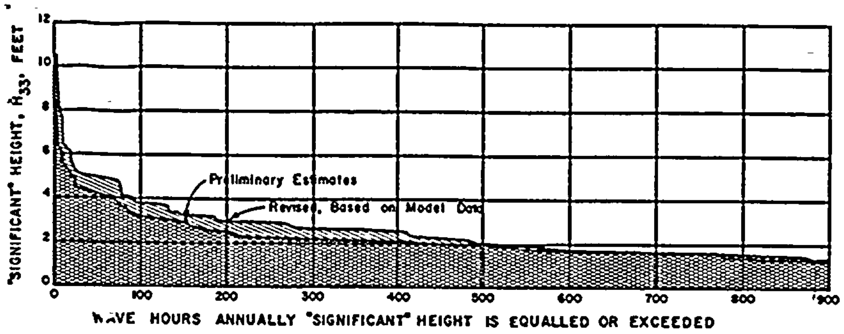


Fig. 7 Wave Height-Duration, Model vs. Preliminary Computations

BREAKWATER AND CROSS-SECTIONS

Preliminary Design for Modelling.—Figure 8 shows the preliminary design cross-section for the deepest location along West Breakwater. The seaward armoring to be tested, 21.5-ton Tribars in single layer, fitted arrangement, was sized by application of prior experience (1) with the piece and discussions with its inventor, Robert Q. Palmer, F. ASCE. Figure 9 shows the Tribar's proportions. In Fig. 10 the reader can see and appreciate the dimensional size of the 21.5-ton armor piece. The right half of Fig. 10 shows forms for these smaller Tribars. The 21.5-ton pieces are 7.17' high; larger ones 37.1 tons each, and 8.58' high, subsequently were also used on West Breakwater's basinward side. The Tribars were bedded on a very open, porous layer of "B" stone. The "B" stone was specified by volume definition rather than by weight per piece. Doing so has logic, it seems, because dimensional size is really the important parameter. The minimum size piece was set at 10 per cent of the volume of the Tribar it would underlie. The median piece of the "B" stone was specified to be twice the volume of the minimum, and the maximum size would be twice the median. The seaward armor slope was selected at 2.25:1, and its nominal surface

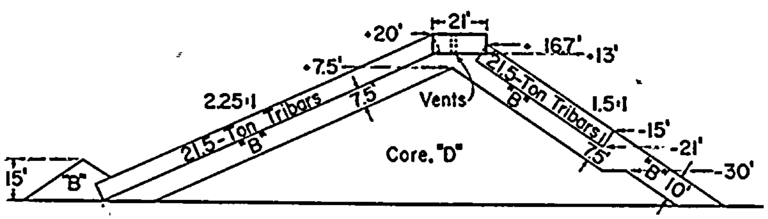


Fig. 8 Preliminary Deep Cross-Section, For Tests

stability improvement to be achieved by flattening. Reflection on what the model displayed, and logic, argues for steepening back slopes of heavily overtopped breakwaters. Doing so increases the available depth of water cushion, to dissipate the plunging jet of water before it attacks the elements on the armored surface. In this context, an ultimate solution could be a vertical back face, if foundation stabilization were conservatively feasible. That is a suspect idea, however, when one considers the possibilities of constructing such a back wall in an ocean environment where heavy overtopping is the expected thing. Another way to go is clearly impractical. That would be a horizontal back slope, where plunging of the overtopping jet would never develop. As in virtually all engineering decisions, something between unusable extremes has to be adopted. In the Diablo situation the judgment was made that a 1.5 to 1 back slope would be the appropriate compromise.

The final East Breakwater cross-section was not itself modelled. It was designed by application of the model performance for West Breakwater's deepest cross-section, with adjustments based on accepted theory and practice related to wave motion and armor stability.

REQUIREMENTS FOR CONSTRUCTION

Adopted Cross-Sections.—Figure 12 shows the principal cross-sections adopted for construction. The core material, D stone, was produced by levelling Patton Peak, the hill with summit at elevation +199 feet seen at the extreme right of Figs. 3 and 5. The underlayer "B" and "E" stone was granitic material produced at a quarry about 30 miles away and trucked to the site. The Tribars, unreinforced, were specified to weigh not less than 145 pounds per cubic foot at the time of fabrication. Concrete was specified to be made with air entrainment, using a modified Type II cement, standard compressive strength of 5,000 pounds per square inch or more was required of 90 per cent of all test specimens; Tribars showing either less than 4,500 pounds per square inch compressive strength or weighing less than 140 pounds per cubic foot at time of installation were stipulated for rejection. Cap concrete required the same air entrainment and kind of cement as the Tribars, but its compressive strength requirement was 3,000 pounds per square inch.

The sea floor at the site was free of sediments other than in localized pockets or defiles of the rough rocky terrain, so no filter stone layers under the structure were indicated. Strong emphasis was given to the +7.5 feet elevation of the core summit, to minimum thickness of the bedding stone, classes "B" and "E", and to the surface uniformity of the Tribar armor.

Preservation of Voids.—Much attention was paid in drafting the specifications, and rigid enforcement was required, to prevent any intrusion of finer rock or of concrete into the voids between "B" and "E" stones. The contractor achieved this by spreading chain-link fence fabric over the top of the stone at the level where the concrete cap eventually would be poured, before the contractor spread quarry waste on that plane, for his temporary haul road surface. When the haul road material was removed, the mesh was lifted and the stone below was carefully examined to assure the voids were clear. Where intrusions of haul with the same wave at low tide, -2.5 feet, also overtopped the structure, but with much less depth over the crest and lower velocity. The result was more like a very heavy flow of water down the back slope. Nevertheless, with 10 feet less cushion effect from the still water, these downrushing flows accelerated to the extent of showing turbulent characteristics to about elevation -13 feet, more than 10 feet below still water level.

The 21.5-ton Tribars on the model's 2.25:1 seaward face, the preliminary design configuration, survived the 30-foot design wave very satisfactorily, but damage on the basin side was unacceptably severe. Many Tribars were rolled out of the armor, nearly all at the beginning of the unravelling being in the slope at and below the still water level.

In their earlier collaboration (5), Dr. Raichlen and the writer had seen the vulnerability of pre-cast armor pieces at interfaces with quarrystone armor on seaward slopes. The problem on these back slope interfaces was similar. Looking to see if a mound of "B" stones laid over the interface at -15 feet could diffuse the attack of the plunging jets, a surcharge of that stone class was put over the lower extent of the basin slope, extending 20 feet vertically above the sea floor. This measure only deferred slightly the redevelopment of similar damage. It was decided to investigate the effect on back slope stability to be achieved by increasing the size of its Tribars. Rather than consume time with molding new model pieces the tests of larger armor were carried out by simply reducing the scale ratio from 1:75 to 1:90. The effect was to change the Tribars from modeling a 21.5 tons prototype weight to 37.1 tons $(90/75)^3$. Initial testing of the heavier armor on the back slope was of a cross-section identical with the preliminary design, except for the change in Tribar and "B" stone size. Severe damage again appeared at the -15 feet level, where Tribars and quarrystone-armored zones abutted. It now was clear, that -15 feet was too violently turbulent during overtopping of the deepest part of the breakwater to permit a change to stones of median 4 tons weight. Searching experimentally for a depth at which an interface would be tolerable, additional rows of Tribars were added to bring the interface with stone armor about 20 feet farther down the slope, to elevation -26 feet. The new interface then was 33.5 feet below the high tide still water surface; it had been 25.5 feet below in the rejected plan. An overlay of the bedding stone was placed 13.3 feet thick, on the same 1.5 to 1 slope as the Tribars, from -16 feet elevation to the sea floor. It covered the bottom two rows of Tribars, as it had in the abandoned second trial plan. The results were good. The upper edge of the overlay was planed away level by the attack of the plunging jet, but the interface between the lowest row of Tribars and the stone remained undisturbed.

Finally, a series of tests was run with the basin-side slope at 2:1 instead of 1.5:1. Again the 37.1-ton Tribars were used and the stone overlay on the lowest two rows was included. As compared to the 1.5 to 1 slope, it performed poorly. The amount of damage to the Tribar armor was considered to be repairable, in prototype context, but certainly the larger quantities of materials needed for a 2:1 slope, the greater crane reaching capacity it suggested, and the reduction in cross-section of the entrance it caused were matters of concern, even if there had been

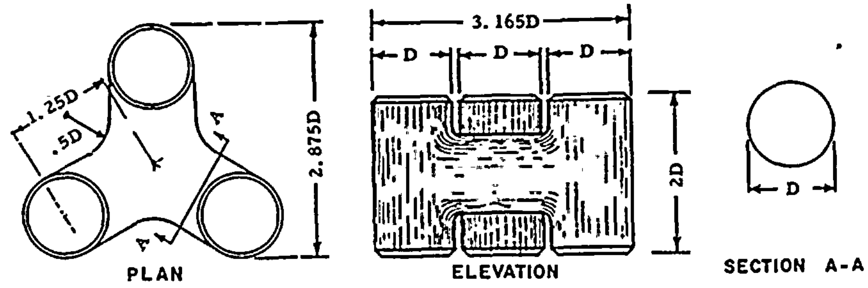


Fig. 9 Tribar Proportions

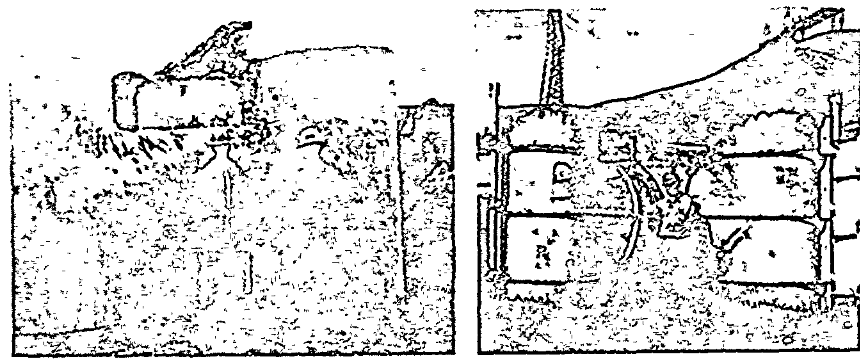


Fig. 10 21.5-Ton Tribar and Steel Form

was flush with the cap surface. The same size tribars were tentatively chosen for the back slope as had been calculated for the seaward. The nominal surface of the back slope armor was kept below the crest of the cap, to avoid impact from rushing overtopping flows. The back slope was tentatively set at 1.5:1, but the surface of the slope armored by Tribars extended only down to elevation -15, not to the bottom.

Concrete Cap.—A parapet to retain the topmost rank of armor pieces was considered necessary, whether overtopping were to be nominal or heavy. Modelling experience repeatedly shows stability problems at slope changes, as well as at interfaces between Tribars and dissimilar armoring. Common sense suggested the overtopping flow should somehow be deflected from impacting against the edge of the top rank of armor on the back slope. The tentative solution was a monolithic concrete cap, which could act as a "book-end" for the seaward blanket of armor and as a skip apron for the back slope.

Stability of Monolithic Caps.—Truly massive monoliths on breakwaters have been bodily moved off their bases during extreme storms. Impressive examples are the monoliths on the breakwater at Wick, Scotland. Gaillard (1) quotes reports by Thomas Stevenson of those blocks weighing 1,500 short tons and 2,900 short tons that were tipped completely off the breakwater by storm waves in 1872 and 1877, respectively. The writer believes that air trapped by upwelling water in voids beneath such monoliths, compressed by the upwelling and impacting seas, stores the necessary energy for sustained lifting of anything resting on such an air cushion. Then the moving wave adds motive power for horizontal displacement. To prevent such air cushions under the Diablo cap, vent stacks through it were specified at appropriate intervals. Strict provisions for an open, highly porous zone of stones under the cap, separating the cap from the less open core material, was called for to provide for easy relief of air.

Bedding Stone.—The "B" stone that was also specified for bedding under the Tribars suited this purpose well. Idealized as a sphere, the diameter of the ~~minimum~~ median "B" stone is practically the diameter of the circle that can be inscribed within the Tribar's 3 legs. The minimum stone, as a sphere, would have a diameter close to that of the Tribar's leg. The specified mixture produces a voids ratio of about 47 per cent, a characteristic that enhances free movement of water into the voids and provides a place to absorb temporarily the uprushing volume of water that otherwise would run farther up the structure. Further, it drains rapidly again during drawdown, so the transient reservoir of voids between stones is empty in time to fill again with the next wave.

Core.—Under the "B" stone the core was specified to be made with run-of-quarry material, from chips and spalls as small as a quarter inch to pieces as large as 15 cubic feet, but requiring that 80 per cent of the mixture, by weight, should vary between 4 inches and 18 inches nominal diameter.

Breakwater Model.—Professor Fredric Raichlen, M. ASCE, conducted model tests for the writer on the armor's stability. He used a flume that could be tilted longitudinally to the critical slopes to create the very high waves of the overtopping features of the tests. The reader is referred to his short description of the tests in ref. (4). His complete report is reference (3), but it is out of print. Fig. 11 shows four photographs of the design wave in his model, 15 seconds period, 30 feet high at +7.5 feet tide, going over the crest of the model and plunging against the basin-side slope during runs when the linear scale ratio of the model was 1:75. The overtopping flow scales 9 feet deep on the +20 feet crest block. The horizontal velocity was high enough that the overtopping flow leapt free past the crest block, much as one would see a partially ventilated nappe under the flow over a weir.

When the falling jet plunged through the surface of the water in the basin it actually pushed the level of the relatively still water down the armored slope, and the turbulence beyond gave the impression of a hydraulic jump. Figure 11 shows the penetration of bubbles associated with the plunging jet, extending more than 25 feet below the still water level to approximately elevation -18 feet. Experimental runs

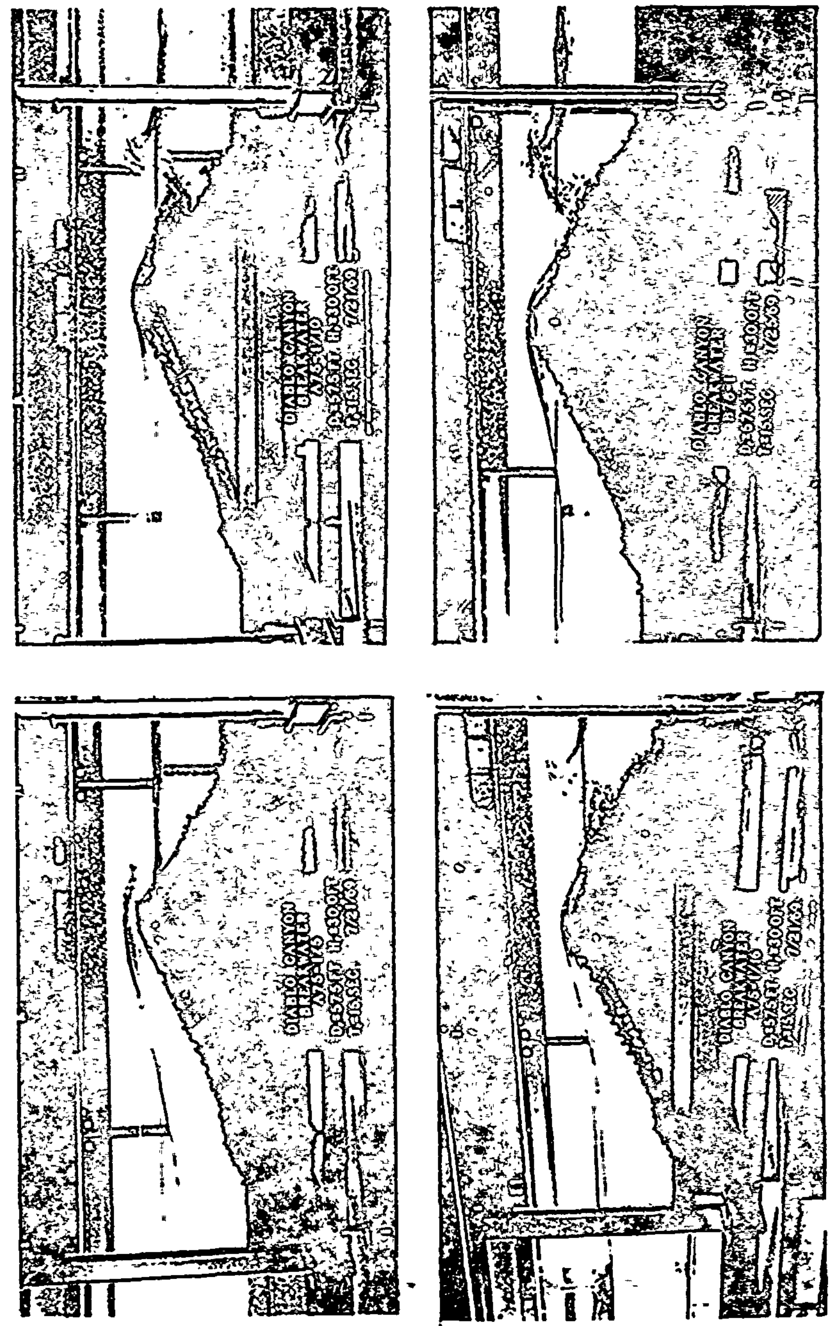


Fig. 11 Trajectory and Plunge of Design Waves, 9' Overtop

SAFETY IMPLICATIONS OF DAMAGED BREAKWATERS

The breakwaters at the Diablo Canyon Nuclear Power Plant are not designated as safety-related structures. The principal function of the breakwaters was to create a sufficiently quiescent sea condition to allow construction of the intake structure. As an added benefit, the breakwaters have reduced wave action at the intake structure, resulting in reduced maintenance of the intake structure and associated equipment.

During the week of January 26, 1981, the west breakwater was damaged by storm waves over about 150 feet of its approximately 1000 foot length, starting at the center of the terminal cone and working landward. Thus about 8 percent of the total of 1800 feet making up the east and west breakwaters has been damaged. An inquiry was made as to whether this minor damage has any significance from the standpoint of safety or plant operation.

The only safety-related items located in the intake structure are the auxiliary saltwater pumps together with their associated piping and equipment. The pumps are installed in the intake structure in watertight compartments which will accommodate any combination of sea waves, extreme tides, storm waves, storm surge, and tsunami effects up to elevation +30 feet, referenced to Mean Lower Low Water (MLLW). These compartments are located in the central section of the structure between the two sets of main circulating water pumps. This location is set back 80 feet from the front wall of the structure. Ventilation air for the auxiliary saltwater pumps is provided through air intakes located in the walls of two concrete vent stacks each with plan dimensions of 10 feet by 9 feet. The bottom of the air intake louvers is at elevation +30 feet (MLLW). This arrangement is shown in the attached Figure 1.

Section 2.4 of the Diablo Canyon Units 1 and 2 FSAR contains a discussion of the analyses performed to demonstrate the capability of the intake structure to provide the necessary protection for the safety-related features located therein for an unlikely combination of extreme high tides, storm waves, storm surge, and tsunami effects. The analyses included consideration of damaged breakwaters. Appendices to Section 2.4 contain reports from consultants retained in connection with this work.

Several conservative assumptions were adopted in performing the analyses discussed above. These include:

1) An 11 foot resultant displacement postulated due to movement on the Hosgri fault.

2) A vertical component of displacement taken as two-thirds of the resultant displacement. Thus a 7.33 foot vertical displacement was used in determining the tsunami wave potential of the Hosgri fault.

3) Short period storm wave peaks were assumed to occur simultaneously with long period tsunami waves, extreme tides, and storm surge. These components were algebraically summed to estimate a maximum water elevation.

4) The fully reflected storm wave component (referred to in the report as the transitory wave component) at the intake structure was based on a reflection coefficient assuming a vertical smooth wall of infinite height.

   This assumption would be appropriate if one assumes a smooth, vertical surface at the vent stack location in water of infinite depth with the reflecting surface of infinite length and height thus reflecting the incident wave with 100% efficiency. In reality, the reflecting surface largely consists of the natural sea cliff, which is neither smooth, vertical (actual slope is 2 to 1), nor of infinite length and height, and which is set back from the vent stack location from 100 to 150 feet (see Figures 1 and 2). The algebraic summation of incident and reflected waves as done in the analyses is extremely conservative and results in calculated values for storm wave height which are excessive by perhaps a factor of two.

5) The most extreme case considered assumed that the breakwaters did not exist. This assumption is not credible.

The breakwaters are designed for, and have until now sustained severe wave-induced dynamic forces including overtopping for approximately 10 years without significant damage. The construction employed, involving heavy quarry stone and precast concrete interlocking armor pieces (tribars), has substantial capability to withstand both wave-induced and seismic forces.

Nevertheless, at the request of the NRC, sensitivity studies on combined wave runup were performed for various assumed seismically damaged breakwater configurations. These configurations included breakwater side slopes slumped to 1 to 4, 1 to 5, and 1 to 6, as well as totally destroyed (i.e. non-existent) breakwaters. The breakwater side slope utilized for the analysis of the undamaged breakwater was 1 to 2 while the as-built slopes are 1 to 2.25 and 1 to 2 for the windward sides of the west and east structures, respectively, and 1 to 1.5 for the leeward sides of both breakwaters.

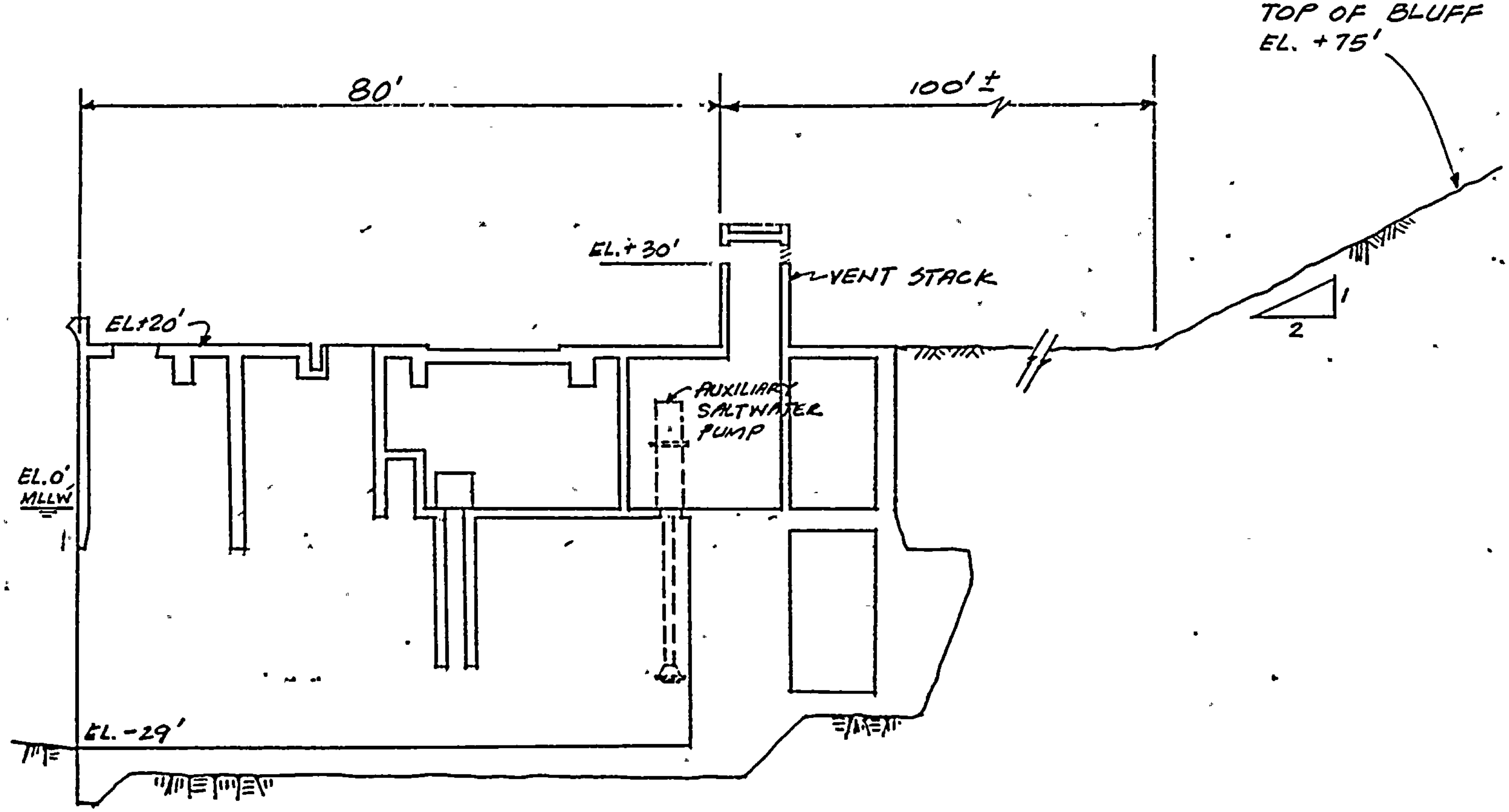
Table 1 of Earthquake Generated Water Waves at the Diablo Canyon Power Plant (Part 2), March 1975 by Tetra-Tech, Incorporated, notes that the water level caused by the unlikely combination of sea waves, extreme tides, storm waves, storm surge, and tsunami effects does not exceed +30 feet until the breakwater slumps to slopes of 1 to 5 or 1 to 6. Even then the water level is only +30.8 feet and +32.1 feet, respectively. For the non-credible case of no breakwater, the maximum water elevation is +44.3 feet.

In considering Table 1 (attached for reference convenience), three points are worth noting. First the tsunami wave component is relatively insensitive to breakwater damage, i.e. it only varies by 0.7 feet across all configurations assumed. Second, and more significantly, the 18 foot storm wave height shown for the "without breakwater" case represents the significant (i.e. average height of the highest one third of the waves in a wave train) storm wave for the Pacific Coast at this location. This significant wave height is used as the incident storm wave for all breakwater cases analyzed. If one boks at the statistics of occurrence of waves at Diablo Cove (see Figure 3) it is apparent that waves of this height range (18.0 - 19.9 feet) occur about 9 out of 13,600 wave-hours per year, or less than one tenth of one percent of the time. It follows then that higher waves, such as the so called 1% wave which would attain 1.67 times the significant wave height, or 30 feet, has an even smaller occurrence probability. Third, the assumption of a fully reflected storm wave is an inappropriate assumption given the actual physical arrangement of the intake structure and the shoreline topography. Finally the total duration of the prolonged water elevation listed in Table 1 is about 200 seconds or 3.3 minutes. This "prolonged" elevation is well within the design basis of the auxiliary saltwater system. It is not until the "transitory" component of the storm wave, lasting a fraction of 200 seconds, is added that the design basis is exceeded and then significantly only for the non-credible case of no breakwater.

The NRC concluded in Safety Evaluation Report Supplement 5, that seismically-induced damage to the breakwater is not expected to exceed that represented by the 1 to 4 side slope assumed in the analysis discussed above. In fact, the most recent NRC calculation yields a drop in the breakwater crest of less than 3 feet (SER Supplement 7). This equates to a slope of 1 to 2.2. Breakwater damage, whether caused by seismic or wave forces, or a combination of the two, would not result in a configuration for which the safety features of the intake facility are not qualified.

We are continuing to monitor the behavior of the breakwaters and, as weather permits, will perform an underwater survey to define accurately the extent of the damage and to establish the failure mechanism. No final determination regarding remedial action is expected until the failure mechanism is known. This work may require several months.

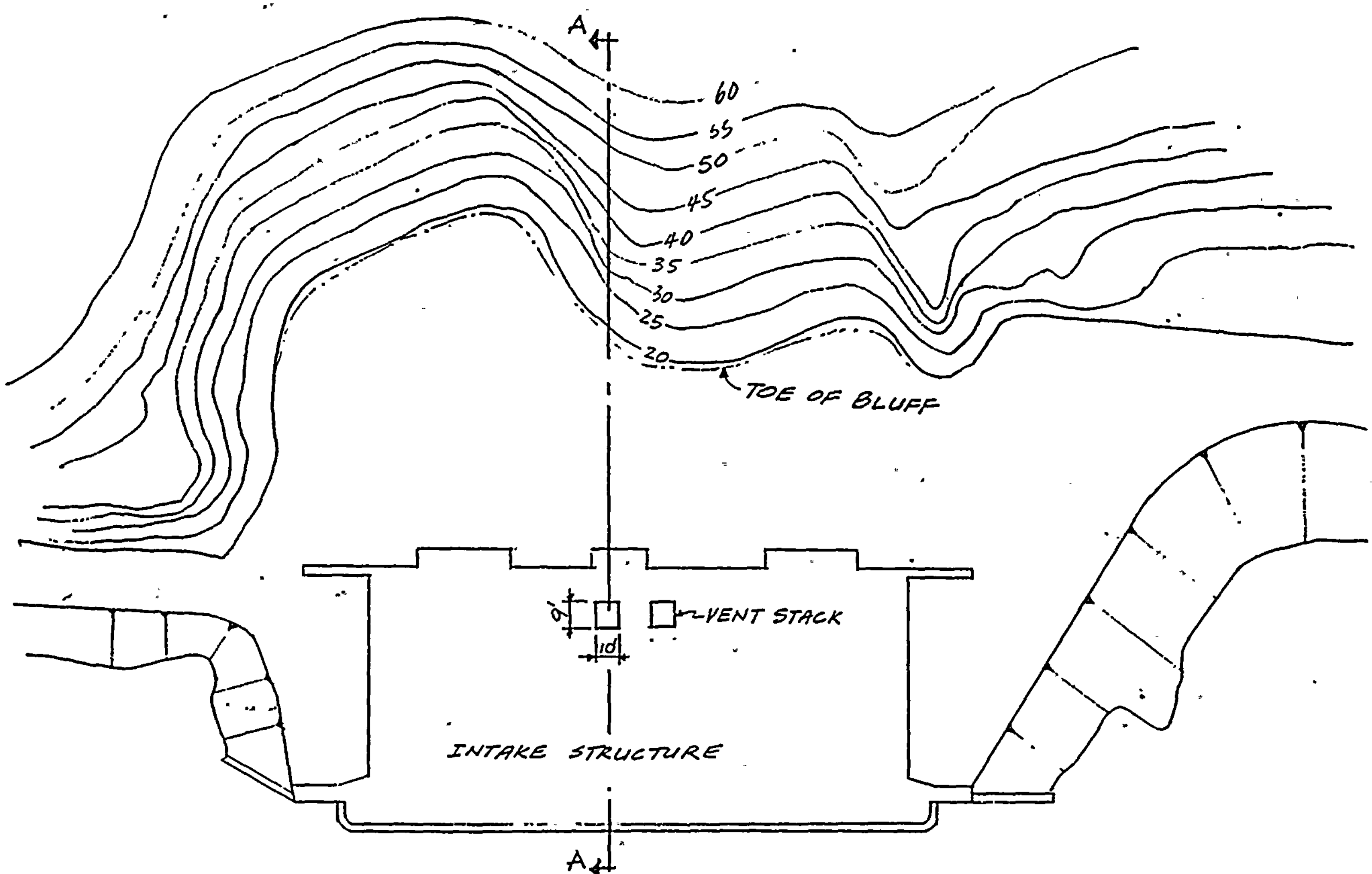
In summary, had the Diablo Canyon Units been operating, the recent storm wave damage to the breakwaters would not have affected plant safety or operability. No credible amount of breakwater damage, whether from wave action or seismic activity, would adversely affect plant safety.



SECTION A-A

NO SCALE

FIGURE 1

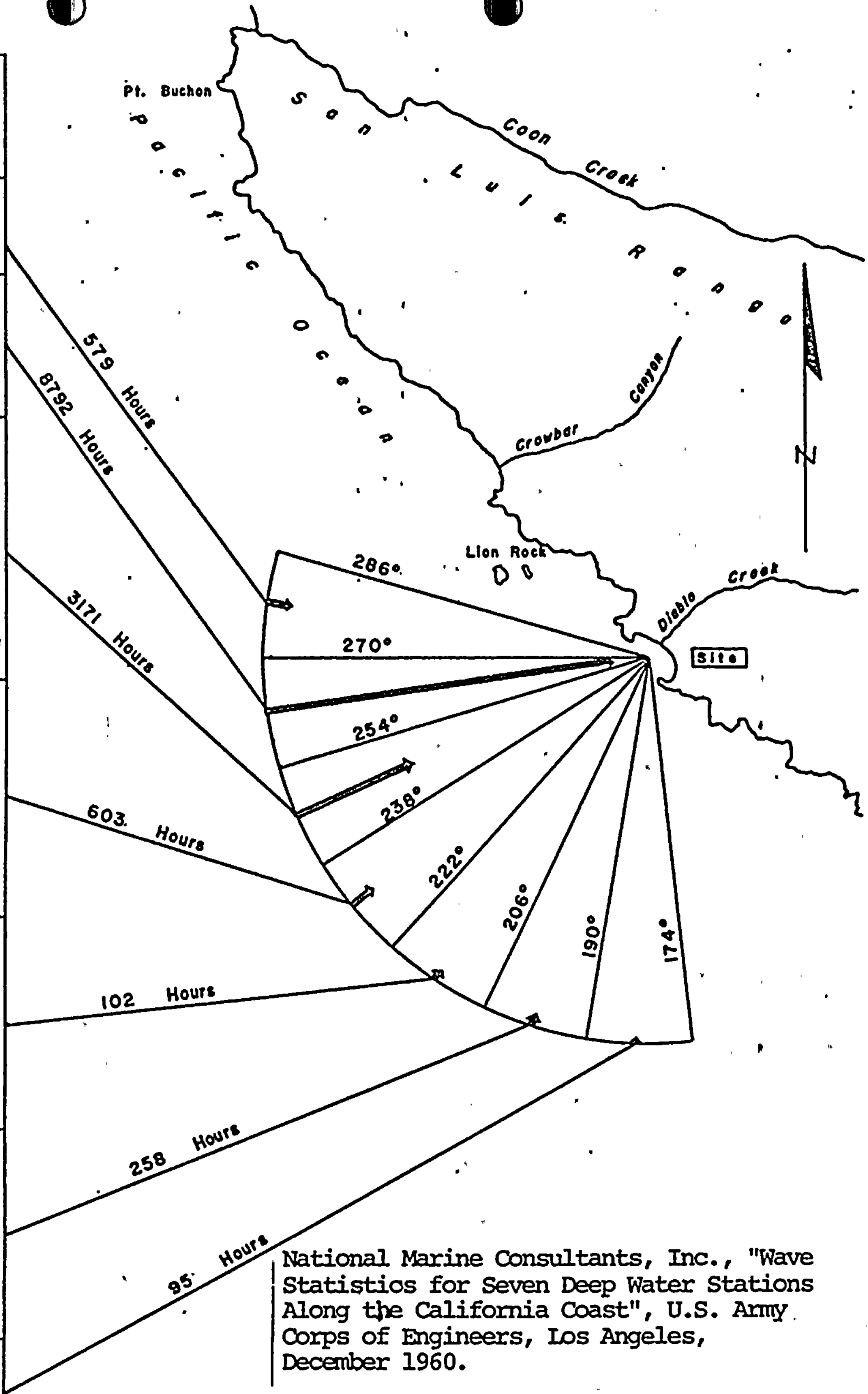


OCEAN

PLAN VIEW
NO SCALE

FIGURE 2

Hours Per Year When	Highest Third of Waves Averaged
371	0-1.9'
167	2'-3.9'
41	4'-5.9'
579	
2743	0-1.9'
4319	2'-3.9'
1661	4'-5.9'
60	6'-7.9'
9	8'-9.9'
8792	
38	0-1.9'
1359	2'-3.9'
887	4'-5.9'
477	6'-7.9'
216	8'-9.9'
92	10'-11.9'
50	12'-13.9'
38	14'-15.9'
5	16'-17.9'
9	18'-19.9'
3171	
4	0-1.9'
317	2'-3.9'
161	4'-5.9'
45	6'-7.9'
68	8'-9.9'
00	10'-11.9'
2	12'-13.9'
4	14'-15.9'
2	16'-17.9'
603	
18	0-1.9'
54	2'-3.9'
9	4'-5.9'
14	6'-7.9'
5	8'-9.9'
0	10'-11.9'
2	12'-13.9'
102	
26	2'-3.9'
117	4'-5.9'
38	6'-7.9'
48	8'-9.9'
21	10'-11.9'
4	12'-13.9'
4	14'-15.9'
258	
77	2'-3.9'
16	4'-5.9'
2	6'-7.9'
95	
13 600 Wave-hrs./Yr.	



National Marine Consultants, Inc., "Wave Statistics for Seven Deep Water Stations Along the California Coast", U.S. Army Corps of Engineers, Los Angeles, December 1960.

OCCURRENCE OF WAVES AT DIABLO COVE

SAN LUIS OBISPO COUNTY, CALIFORNIA

Average Annual Conditions At 10-Fathom Line

Figure 3

TABLE 1

EXTREMES OF RUNUP AND DRAWDOWN AT THE INTAKE STRUCTURE, DIABLO CANYON (SANTA MARIA BASIN FAULT EARTHQUAKE)

Agent Producing Water Level Change	WITH BREAKWATER: Maximum Runup, R(ft) Above MLLW Datum		WITH BREAKWATER: Maximum Drawdown, D(ft) Below MLLW Datum		WITHOUT BREAKWATER: Maximum Runup, R(ft) Above MLLW Datum		WITHOUT BREAKWATER: Maximum Drawdown D(ft) Below MLLW Datum	
	Prolonged	Transitory	Prolonged	Transitory	Prolonged	Transitory	Prolonged	Transitory
(a) Astron. Tide	+ 5.3		- 1.0		+ 5.3		- 1.0	
(b) Tsunami	+ 9.2		0		+ 8.5		0	
(c) Meteor. Tide	+ 1.0		- 0.5		+ 1.0		- 0.5	
(d) Storm Wave (1 yr)								
Standing Wave Set-up	+ 3.15		+ 4.09		+ 11.52		11.59	
Wave Height		9		6.66		18		14.49
Prolonged Water Elevation	+ 18.65		+ 2.59		+ 26.32		+ 10.09	
Wave Height Crest		+ 9				18		
Wave Height Trough				-6.66				-14.49
Max. Water Elevation	+27.65		- 4.07		+44.32		- 4.40	

TABLE 1 (Continued)

	BREAKWATER SLUMP CONDITION					
	1 on 4 Slope		1 on 5 Slope		1 on 6 Slope	
Agent Producing Water Level Change	Maximum Runup, R (ft) Above MLLW Datum		Maximum Runup, D (ft) Above MLLW Datum		Maximum Runup, R (ft) Above MLLW Datum	
	Prolonged	Transitory	Prolonged	Transitory	Prolonged	Transitory
(a) Astron. Tide	+5.3		+5.3		+5.3	
(b) Tsunami	+8.7		+8.7		+8.7	
(c) Meteor. Tide	+1.0		+1.0		+1.0	
(d) Storm Wave (1 yr)						
Standing Wave Set-up	+3.56		+4.59		+5.05	
Wave Height		9.9		11.20		12.02
Prolonged Water Elevation	+18.56		+19.59		+20.05	
Wave Height Crest		+9.9		+11.20		+12.02
Wave Height Trough						
Max. Water Elevation	+28.46		+30.79		+32.07	

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PACIFIC GAS AND ELECTRIC COMPANY

GENERAL COMPUTATION SHEET

SHEET NO. 8 OF ____ SH

JOB / FILE NO. ____

LOCATION West Beach

SUBJECT Wave records @ entrance to breakwater

MADE BY ____ DATE ____ CHECKED BY ____ APPROVED BY ____

Date	Time	Period	Hs
1-28-81	14:00		15
		17	12
		15	6
		17	10
		15	10
		17	15
		15	10
		17	15
		15	15
		12	8
		17	18
		15	15
		15	10
		17	8
		15	8
		14	6
		17	10
	14:16:50		8
		12	6
		12	6
		15	8
		15	4
		16	10
		15	8
		15	6
		17	8
		15	8
		15	8
		13	10
		15	8
		17	10
		13	8
		10	4
		15	8

Reference 3

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PACIFIC GAS AND ELECTRIC COMPANY

GENERAL COMPUTATION SHEET

SHEET NO. 2 OF ___ SH

JOB / FILE NO.

LOCATION West Bre...

SUBJECT Wave records @ entrance to breakwa...

MADE BY ___ DATE ___ CHECKED BY ___ APPROVED BY ___

Date	Time	Period	Hs
1-28-81	14:30:40		10
		14	6
		15	8
		14	10
		15	8
		13	6
		13	6
		12	8
		14	10
		14	8
		12	10
		13	8
		15	8
		15	10
		16	15
		13	12
		16	15
		15	15
		15	8
	14:44:15		8
		17	8
		15	8
		17	10
		15	15
		17	15
		15	10
		17	6
		13	6
		15	~~8~~ 6
		15	4
		13	8
		18	8
		13	8

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PACIFIC GAS AND ELECTRIC COMPANY

GENERAL COMPUTATION SHEET

SHEET NO. 3 OF ____

JOB / FILE NO. ____

LOCATION West Berk...

SUBJECT Wave records @ entrance to breakwa...

MADE BY ____ DATE ____ CHECKED BY ____ APPROVED BY ____

Date	Time	Period	Hs
1-28-81	14:46:10		15
		15	15
		13	15
		15	18
		13	8
		15	15
		15	10
		13	8
		14	10
		12	8
		15	6
	14:55:45		15
		10	8
		13	10-
		19	8
		[illegible]	[illegible]

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PACIFIC GAS AND ELECTRIC COMPA

GENERAL COMPUTATION SHEET

SHEET NO. 4 OF ___ S

JOB / FILE NO. ___

LOCATION West Brw.

SUBJECT Wave records @ entrance to breakwa

MADE BY ___ DATE ___ CHECKED BY ___ APPROVED BY ___

Date	Time	Period	Hs
1-28-81	15:00		15
		15	15
		14	15
		15	18
		15	10
		16	10
		14	8
		16	8
		15	6
		13	6
		12	8
		13	6
		13	4
		15	8
		14	8
		15	6
	15:13:30		6
		16	8
		17	~~8~~ 6
		15	6
		16	10
		15	10
		14	15
		17	10
		15	8
		17	6
		17	10
		16	8