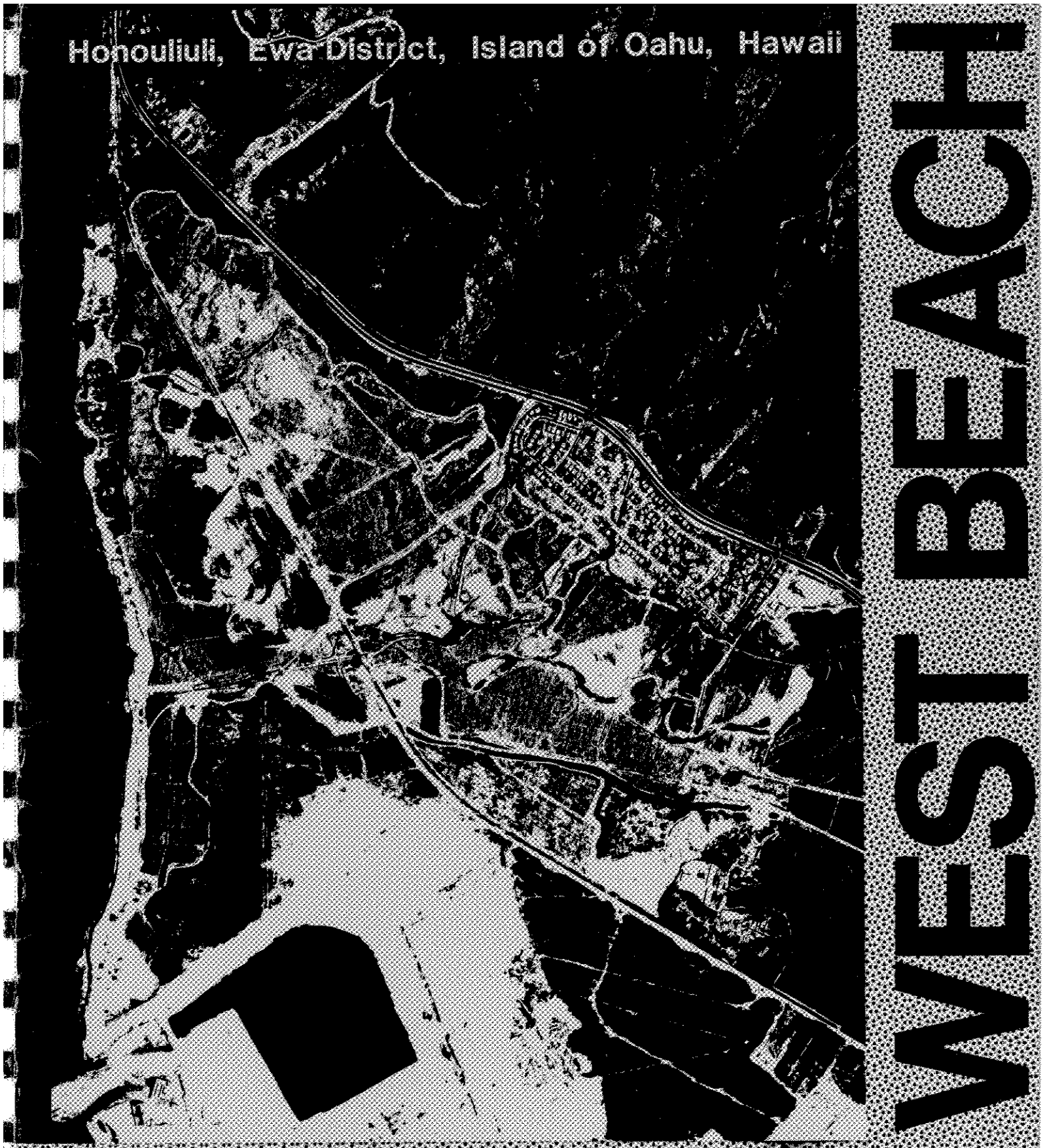


WEST BEACH

APPENDIX II

WEST BEACH

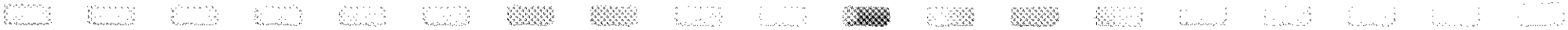
DRAFT SUPPLEMENTAL

ENVIRONMENTAL IMPACT STATEMENT

Honouliuli, Ewa, Oahu, Hawaii

APPENDIX II

TECHNICAL STUDIES



These technical studies have been prepared for the West Beach Supplemental Environmental Impact Statement (EIS) which has been prepared under the Rules & Regulations of the Environmental Quality Commission (Chapter 343, HRS) and NEPA (P.L.91-190). They reflect the current status of condition, forward from the generic EIS prepared in 1979 and accepted in 1980. As such, all comments made by the technical subconsultants are for the current condition of the project site and the proposed project as designed and presented in the EIS.

APPENDIX I - TECHNICAL STUDIES

1. A Report of the Viability of West Beach as a Resort Community and the Estimated Economic Impact; Pannell, Kerr, Forster.

2. Acoustic Evaluation of AICUZ Plan, NAS, Barber's Point, 1984, and Evaluation of HIA Air Craft Noise Impact; Revised Land Use Plans, Proposed West Beach Resort; Darby-Ebisu & Associates, Inc.

3. Air Quality Analysis for West Beach Project Oahu, Hawaii; Root, Barry D.

4. Birds of West Beach; Berger, Andrew J., Ph.D.

5. Botanical Survey of the Proposed West Beach Resort Project; Char, Winona P.

6. Occurrence and Significance of Palaeontological and Archaeological Remains in the West Beach Resorts Development Area, Oahu; Ziegler, Alan C., Ph.D.

7. Traffic Impact Analysis; Community Planning, Inc.

8. West Beach, Oahu: Archaeological Status Report; Barrera, William, Jr.

9. West Beach Project Water Pollution Implications of Project Site Storm Runoff; Dugan, Gordon L. Ph.D.

APPENDIX II - OCEANOGRAPHIC AND MARINA STUDIES:

1. Analysis of Biological Impacts of the Lagoon/Marina Development at West Beach, Oahu, Hawaii; OI Consultants, Inc.

2. Proposed West Beach Marina Hydraulic Model Investigation; U.H. James K.K. Look Laboratory of Oceanographic Engineering, Department of Ocean Engineering; Principal Investigator; Lee, Theodore T.

3. Summary of Technical Input for the West Beach Lagoons and Marina Design Development; Bathen, Karl H.; Dr.

4. Technical Evaluation and Recommendations on Design of Marine Structures; Gerritsen, F.

5. Tsunami and Hurricane Design Criteria and Recorded Wave Data Analysis for West Beach Development Area; Bretschneider, Charles L.

APPENDIX II SECTION 1

ANALYSIS OF BIOLOGICAL IMPACTS OF THE LAGOON/MARINA DEVELOPMENT AT WEST BEACH, OAHU, HAWAII

FINAL REPORT

Prepared for:

West Beach Estates
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July 30, 1984

SUMMARY OF IMPACTS

Impacts of the West Beach project on the marine environment may be due to the following:

1) Physical damage and siltation during construction - Dredging of the wave and discharge channels to the lagoons will directly impact the marine communities living on the substrate to be removed. Adjacent populations may be adversely affected by blasting, if used, or siltation from dredging; the magnitude of these impacts, when considered in the context of the shoreline as a whole, are likely to be minimal. The stretch of shoreline between the Barbers Point Harbor entrance and the Campbell estate is relatively barren, and the intertidal zone is not heavily populated with marine animals (generally between 5 and 20% coral cover). The amount of shoreline directly affected is a small percentage of the total. The surfaces exposed in construction represent potential new habitat for replacements for the populations destroyed. Since the amount of new surface area will be greater than that existing previously, it is likely that populations will eventually reach levels equal to or higher than before construction.

   Siltation during construction should be minimal. Lagoons will be dredged before the entrance channels are formed, and the construction of the channels should result in only very localized elevated suspended solids levels for only a short period of time. Corals, which are the organisms most susceptible to mortality due to siltation, are present but not highly abundant (5 to 20% cover). The amount of local siltation compared to that of the Barbers Point Harbor channel dredging operation, for example, should be small.

2) Freshwater (groundwater) re-distribution - The lagoons and marina, because they will be deep enough to intercept the natural seaward flow of subsurface groundwater, will act as point-source discharges of low salinity, relatively high nutrient, water to the marine environment. The amount of water reaching the ocean will not increase, since the natural flow presently reaches the ocean as a diffuse outflow through the ocean bottom itself. The potential impact stems from the change from a diffuse discharge in an area of high energy (the near-shore and intertidal zone) which results in very rapid mixing and dilution, to a situation in which the groundwater is collected and discharged as a distinct surface layer at a particular point. Such point discharge flows are expected to be relatively small, however, especially compared to that of the adjacent Barbers Point harbor, and will probably have only very localized impacts.

The low salinity, high nutrient water has the potential to stimulate the growth of microalgae within the marina and lagoons and macroalgae on surfaces which it contacts, such as the discharge channel and the nearby ocean bottom. Microalgal growth would only become visible during periods of low flushing; projections of flushing rates for the lagoons and marina suggest no microalgal blooms would occur under any except infrequent low wind, low surf, conditions. Macroalgal growth would be limited to areas of hard substrate (consolidated limestone or coral rubble) which is continuously exposed to this high nutrient water.

3) Runoff and siltation - During periods of heavy rainfall, the capacity of the drainage basin to absorb excess water may be exceeded, and surface flow will result. This flow is presently planned to be discharged into the ocean at two points: the majority of the flow will be channeled into the marina, from whence it will reach the ocean via the marina or the Barbers Point Harbor entrance channel, depending upon which channel alignment is eventually selected and approved; and the balance will be discharged at the shoreline north of the Hawaiian Cultural Center. Until Phase IV is completed, the marina will act as a settling basin and percolation-evaporation pond, collecting runoff during storm events and holding the water while it percolates into the ground or evaporates. Calculations of maximum volume flow during storm events and the retention volume of the marina indicate that the volume of the marina is sufficient to receive all surface flow channeled to it without overflow under most likely rainfall conditions.

After the marina is completed, both the marina and the northern discharge will discharge storm water to the ocean as a surface layer. Depending on the surf conditions, flow volume and sediment load, this flow may persist as a visible surface plume. This intermittent flow is similar to that which occurs in several natural streams along the Waianae coast, which discharge sediment-laden water into the ocean during heavy rainfall. The relative impact of the West Beach discharges will probably be less than those of the natural streams, since much of the West Beach flow will be over either solid, non-eroding surfaces (concrete, asphalt) or well-grassed landscaping areas, and will be channeled in grasses swales or box culverts. The marina will act as a settling basin for the majority of the discharge from the West Beach project area.

4) Increased shoreline access - The West Beach development will open up an area which is now relatively inaccessible to a potentially much greater use by recreational and subsistance fishermen. At present, most fishing in the area is by pole-and-line, and the target species are mostly reef fish. Increased fishing pressure could potentially adversely affect the local stocks of these fish, although projections of impacts for a nearby project suggest the impact of

increased fishing will be small. The presence of the marina with its launching and berthing facilities will probably result in an increase in boat-based fishing activity along this stretch of coastline, with potential negative impacts to fisheries such as the offshore bottomfishes, which appear to be at or near maximum sustainable yield. Increased sport fishing could also be expected.

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INTRODUCTION

The West Beach development planning concept is a resort and residential community encompassing 642 acres located at Honouliuli, Ewa, Oahu, Hawaii. Resort and residential development planned include 4,000 resort units, 1,500 low-density apartment units, and 3,700 medium density apartment units. New recreational facilities planned to service both visitors and residents include an 18-hole golf course, four shoreline lagoons, a beach club, a marina, four public parks, and a Hawaiian cultural center. Support facilities include commercial operations, an elementary school, transit stations, and roadways. The development of this large area of land located at or near the shoreline has the potential to impact the nearshore marine environment. Potential impacts include biota loss due to construction operations (blasting and dredging), increased siltation and turbidity during dredging, redirection of groundwater flow, changes in runoff patterns during storm events, and increased nutrient and suspended solids loading of runoff. The magnitude of the impacts depends on the duration and location of the construction operations and surface runoff discharge points and volumes.

DESCRIPTION OF THE ADJACENT MARINE ENVIRONMENT

Nearshore waters in the West Beach area are classified "A" in Department of Health water quality regulations (State of Hawaii Water Quality Standards, Chapter 54, Title II, November 12, 1982). The water quality conditions of the nearshore area adjacent to the West Beach project have been described (Bienfang and Brock, 1980). The area was found to be a pristine, unperturbed coastal region. Temperature and salinity values were indicative of open, well-flushed coastal areas which are minimally affected by surface runoff. Water clarity was excellent and turbidity levels were low. Other studies have reported variable turbidity levels, depending on surge and tidal conditions (Kimmerer and Durbin, 1975). Dissolved nutrient levels were low and typical of well-flushed coastal areas. Some evidence (elevated nitrate levels and decreased salinity levels) of groundwater intrusions along the coastline was observed.

Recent construction activity immediately to the south of the project site at the Barbers Point Deep Draft harbor has caused changes in the ambient conditions since the 1980 survey. A surface plume of high-turbidity water typically is discharged from the Deep Draft Harbor on a continuous basis. Localized dredging of the entrance channel adds to the levels of turbidity. This plume travels to the north along the coast afronting the project during ebb tide (Bathen, undated data report), and is not completely displaced by non-turbid water during flood tide (Brock, pers. comm.).

The area offshore of the project site between the shoreline and the 18 m depth contour contains seven recognizable benthic biotopes (Figure 1; Bienfang and Brock, 1980). A shallow, low

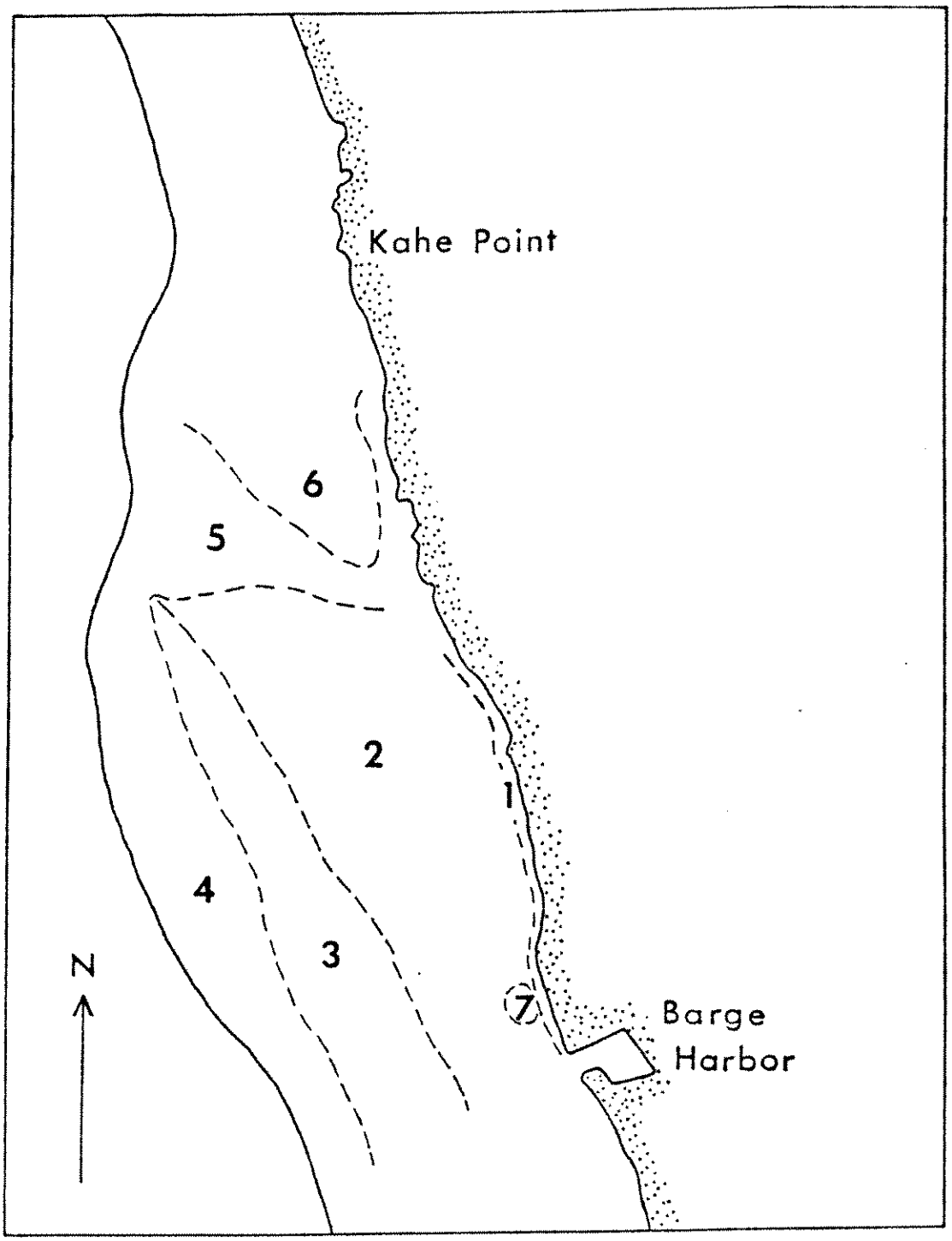

Figure 1. Approximate boundaries (dashed lines) of seven biotopes recognized in the survey at West Beach, Oahu. The 18 m contour is shown on the left (solid line) and represents the seaward limit of the study. From Bienfang and Brock, 1980. The biotopes are:

1 - shallow, low coral coverage biotope;
2 - biotope of high coral coverage;
3 - biotope of large Porites lobata colonies;
4 - deep, low coral coverage biotope;
5 - biotope of sand channels;
6 - biotope of diverse high coral coverage;
7 - biotope of Porites compressa ridges.

(13-18%) coral coverage area is located along the shoreline to a depth of 5 m (approximately 100 m offshore). This zone receives a considerable amount of scouring due to wave activity. The substrate is primarily solid limestone with small scattered corals. The assemblage of fish in this biotope is not particularly diverse; the most common are surgeonfishes and wrasses.

Offshore of this well-scoured area is located an extensive area of high (58%) coral coverage which extend to a distance of 500 to 1,000 m offshore. The substrate is predominantly limestone with scattered pockets of sand and rubble. Fish diversity in this biotope is high; surgeonfishes, wrasses and damselfishes are most abundant. Macroinvertebrates other than corals and macrothalloid algae are scarce.

Further offshore (to a distance of approximately 1,000 m) is located an area of scattered large Porites lobata colonies which grades into an area of deeper water with low (8-12%) coral coverage. The area of large Porites heads extends over a kilometer to the south of the entrance to the Barbers Point Harbor. The substrate of both these biotopes is predominantly limestone, often overlain by a thin veneer of sand and rubble. Fishes are more abundant in the area of large Porites than in any other area. Most species are found in the other biotopes as well, however. Surgeonfishes, wrasses and damselfishes are most abundant. Macrothalloid algae and other macroinvertebrates are not common.

At the northern boundary of the project site, directly offshore of the Alice Kamokila Campbell residence, is located an area of extensive sand channels. The depressions and channel floors are covered with sand and are from 2 to 5 m deep and 1 to 50 m wide. Fish and macroinvertebrates are relatively low in abundance.

North of the sand channel biotope is an area of diverse high (14-48%) coral coverage which is a continuous feature north of the project site (Kimmerer and Durbin, 1975; B-K Dynamics, 1971; Jokiel and Coles, 1974) extending to Kahe Point. This area offshore of the northern portion of the project site has been characterized as having luxuriant coral growth with an associated diverse and abundant fish assemblage (AECOS, 1979). Coral diversity in this area is highest of any area off West Beach. Fish are relatively abundant, and most common are the surgeonfishes and wrasses.

A small area of ridges of mostly dead Porites compressa is located at the southern boundary of the project site.

Access to the shoreline along West Beach is difficult. This limited access reduces fishing activity in the area. Pole fishing is the predominant fishing method along the coast, with reef species as the main catch. Throw-net fishermen visit the raised reef shoreline. Diving conditions off West Beach are

excellent throughout most of the year (AECOS, 1979). The area at the northern boundary of the project and extending to kahe Point has been considered for classification as a marine life conservation district because of the lush and diverse coral growth (State of Hawaii, Div. of Fish and Game, 1977).

ACTIVITIES POTENTIALLY IMPACTING THE MARINE ENVIRONMENT

Impacts of the West Beach project on the marine environment may be due to : 1) physical damage and siltation during construction; 2) freshwater (groundwater) redistribution; 3) runoff and siltation; and 4) increased shoreline access.

1. Physical Damage and Siltation

Actual physical damage to the nearshore marine environment will occur in the construction of the four shoreline bathing lagoons. The lagoons themselves will be located shoreward of the shoreline bench, and construction of them should not entail any direct physical impact on the marine environment. It has been suggested that dredging of the lagoons proceed from the inland slope to the sea, and that break-through to the ocean only occur after the lagoons have been completed (State of Hawaii Department of Land and Natural Resources, 1984). The method of excavation has not yet been established. Alternative methods include ripping of the substrate with bulldozers and removal of material by some type of bucket dredge (dragline or clamshell), or selected blasting to loosen the substrate and subsequent material removal. It has been suggested (W. Godfrey, pers. comm.), based on experience from the adjacent Barbers Point Deep Draft Harbor construction effort that blasting might be the most efficient alternative. Ripping the substrate with bulldozers will not have any direct impact on the nearshore environment; blasting, however, may produce temporary changes in population distributions. Blasting has the potential to scare motile forms, mainly fish, away from the immediate area. This is likely to be a temporary effect, and in all likelihood the fish would return to the site almost immediately after blasting had ceased. Seasonal controls on blasting may be imposed by the National Marine Fisheries Service to protect threatened and endangered whales and marine turtles.

Construction of the channels for inflow and discharge to the lagoons will entail removal of material from the shoreline bench and dredging of a series of shallow (1-foot and 8-foot deep) channels offshore. This activity will kill all attached organisms in the removal area, but will also expose new surfaces to subsequent colonization. The area of newly-exposed surface will be larger than the area removed. The biota of the shoreline bench and nearshore areas to be removed have recently been surveyed (Brock, 1984; attached as Appendix A). The purpose of the survey was to locate, identify and describe the macrobenthic resources in the immediate vicinity of the proposed lagoons. Ten survey sites were established (Figure 2), two affronting each of the four proposed lagoons; additional sites north and south of

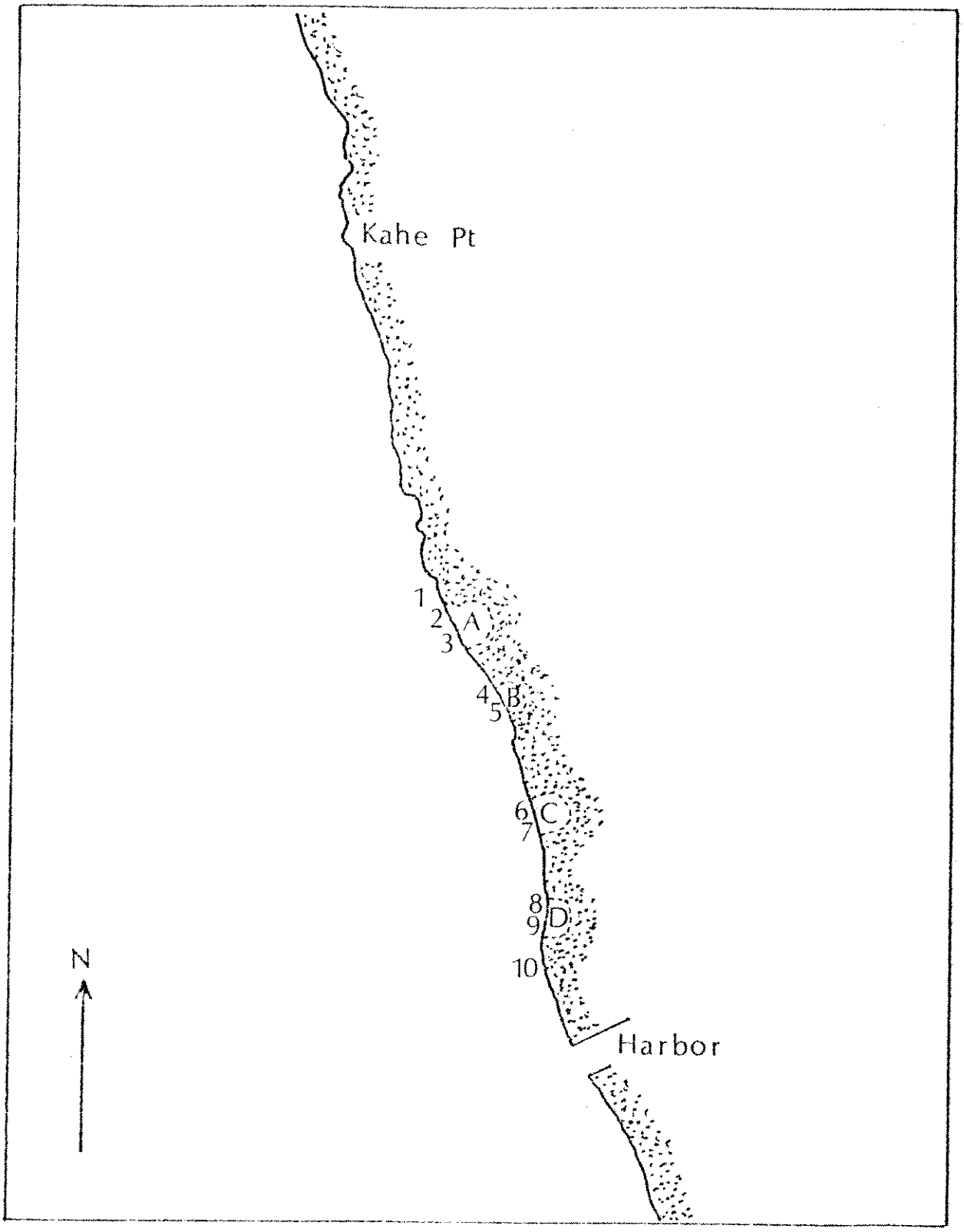

Figure 2. Map showing the approximate locations of the 10 quantitative stations sampling shallow benthic communities adjacent to the four proposed lagoons at West Beach, Oahu. From Brock, 1984.

the lagoons were also established. Quantitative estimates of fish abundance and species composition were made, as well as enumeration of epibenthic invertebrates, corals, and macrothalloid algae. The algal communities on the shoreline bench affronting each lagoon were qualitatively surveyed as well. A summary of the biological characteristics observed at the ten stations is presented in Table 1.

The area encompassed in the survey lies within the shallow low coral coverage biotope (Bienfang and Brock, 1980; see Figure 1). Species found in the subtidal and shallow marine areas were typical of those seen in Hawaiian shoreline areas of similar geology. They include the corals Pocillopora meandrina and Porites lobata, the small sea urchins Echinometra mathaei and E. oblongata, and the snails Drupa morum, D. granulata and D. brunneolabrum. Fifty-five species of macrothalloid algae were observed. A number of fish species were characteristic of these areas, including the surgeonfishes Acanthurus triostegus and A. achilles, the wrasse Thalassoma umbrostigma, and the damselfish Abudefduf imparipennis. There are a number of other diurnally active fish species that are commonly seen not only in this biotope, but in others as well. Cryptic (not easily seen) species present in this zone include the spiny lobster, Panulirus penicillatus, the slipper lobster, Paribaccus antarcticus, the cowry, Cypraea maculifera as well as the aholehole, Kuhlia sandvicensis, and the ala'ihi, Adioryx lactaeoguttatus.

The bottom relief was similar for all stations except Station 4. The bottom at Station 4 provided considerable habitat relief in the form of large broken slabs of limestone. The substrate at the other stations was more uniform and afforded less coverage. The abundances and diversity of fish observed were related to these differences in bottom type: higher abundances and diversity were found at Station 4; no discernible trends in number of species (fish, corals, invertebrates, and algae), coral coverage, individual fish numbers of diversity were observed between any of the other stations.

Three of the sites examined in the more recent study (Brock, 1984) were in the same location as those sampled previously (Bienfang and Brock, 1984). A summary of biological characteristics for the three stations in 1980 and 1984 are given in Table 2. In general, there has been a decrease in fish species, fish diversity, coral species and coral coverage at these three stations over a 54 month period. In one case, the number of individual fish counted increased; this was due, however, to the encounter of a large school of browsing manini (Acanthurus triostegus sandvicensis). Such an event may be considered a rare (chance) event. The number of invertebrate species present in the transect areas increased at every station; however, this is probably an artifact of the semi-quantitative nature of the sampling for these organisms.

The actual area of shelf and bottom to be removed during the construction of the lagoon channels is approximately 2,600 square

Table 1. Summary of the biological findings at the 10 quantitative stations sampling the shallow high energy biotope at West Beach, Oahu in June 1984. Overall means are given at the foot of the table. From Brock, 1984.

	Fish			Corals		Invert.	Algae
Sta. No.	Number Species	Number Indiv.	Diversity (H')	Number Species	Coverage %	Number Species	Number Species
1	14	97	1.68	2	3	8	29
2	14	588	0.70	4	11	5	30
3	12	86	1.77	5	6	9	29
4	24	689	2.32	4	1	9	25
5	17	175	1.74	3	7	8	30
6	11	103	1.15	5	3	9	26
7	15	103	1.59	6	4	4	22
8	10	24	1.97	3	3	6	23
9	11	86	1.39	6	6	5	21
10	12	70	1.59	5	3	7	36
Means	14	202		4	5	6	27

Table 2. Summary of the biological characteristics present at three stations in the West Beach study site 54 months apart. Data for 1979 from Bienfang and Brock (1980); data for 1984 from Brock (1984).

	Station No. 1		Station No. 2		Station No. 10	
	1979	1984	1979	1984	1979	1984
No. Fish Species	26	14	25	14	17	12
No. Fish Individuals	127	97	163	588	112	70
Fish Species Diversity (H')	2.57	1.68	1.93	0.70	1.94	1.59
No. Coral Species	5	2	6	4	6	5
Average Coral Coverage	16	3	17	11	6	3
No. Invert. Species	4	8	4	5	5	7
No. Sea Urchins	29	10	11	10	136	0

meters. Bienfang and Brock (1980) have estimated the extent of the shallow low coral coverage biotope to be from the shoreline to approximately 100 m offshore. The shoreline length between the Barbers Point Harbor and Kahe Point (the minimum extent of this biotope) is 5 km, and thus the area to be removed is only 0.5% of the total such area. Given the similarity in biological populations throughout the biotope, the removal of less than 1% of the area cannot be considered a significant impact.

The construction of the lagoons and the marina, if completed before break-through to the ocean, will cause no increase in turbidity or siltation in the nearshore marine environment. As soon as the lagoons and marina are opened to the sea, however, an initial period of flushing will occur during which fine particles generated by the dredging activity will be carried out into the ocean by the outflow of groundwater and the designed flushing action of the lagoons and marina themselves. The dredging of the channels associated with the lagoons will also generate turbidity plumes. The impact of this turbidity and the associated siltation of suspended particles will occur within a region up- and down-coast from the lagoon sites. The suspended material will impact the resident coral communities through both siltation and decreased light levels due to increased turbidity. It is not possible at this point to estimate the horizontal extent of the plume and the rates of deposition of suspended material at points along the plume path. It is reasonable to expect, however, that the region of highest impact will be immediately adjacent to the dredge sites and that the impact will decrease with distance from the site. Brock (1984) observed many dead but intact coral heads and a layer of fine particulate material up to 1 cm thick at a station approximately 350 - 400 m north of the Barbers Point Deep Draft Harbor, but did not observe any significant mortality of corals or sediment build up at a station 200 m north of the impacted station. It would appear, then, that the expected extent of impacts due to turbidity and siltation would be limited to an area less than 600 m on either side of any particular dredge site, and given the much less extensive dredging to be done for the West Beach lagoon channels, that figure may be an extreme overestimate. The entrance channel to the Deep Draft Harbor is on the order of 150 m in width; the widest lagoon channel is approximately 15 m in width. It is not unreasonable to assume, therefore, that the region of impact due to dredging of the lagoon channels may be similarly decreased, i.e., on the order of 50 to 60 m at most upstream and downstream from the dredge site. Given this assumption as to impact extent, the region actually impacted by dredging and siltation would be approximately 500 m of the 5 km shoreline. It should also be realized that the duration of sedimentation and turbidity as well as the daily dosage is important in the assessment of impact. A relatively short exposure to a specific sediment load may not be fatal to corals while a longer exposure would be. The dredging of the lagoon channels is most certainly going to take less time than the dredging of the Deep Draft Harbor entrance has.

A further factor which may decrease the magnitude of the

dredging impact is the fact that all dredging will occur within a relatively small distance from shore, in an area which is typically well scoured and in which sediment is continuously resuspended. Any sediment which is generated by the dredging operation is likely to remain in suspension and be eventually dispursed over a very wide area or carried offshore. In addition, the near-shore, well scoured region is relatively devoid of corals, having at most less than 20% coral coverage and in some areas less than 5% coral cover. Given all these potentially mitigating conditions, the overall impact of dredging is likely to be small.

2. Freshwater (Groundwater) Re-distribution

The coastal region upon which the West Beach development is located is typified by subsurface seaward flows of high-nutrient, low salinity groundwater. The evidence of this flow has been found in near-shore samples from the ocean which are higher in nitrate and lower in salinity than samples take further offshore (Bienfang and Brock, 1980). Strong flows of brackish water have also been observed within the Barbers Point Harbor (Environmental Consultants, Inc., 1975). Samples taken from borings throughout the project area (Harding Lawson Associates, 1984) showed the presence of a thick (22+ feet) layer of low salinity (<2.5 ppt) water at inland borings, and somewhat higher (3 - 7 ppt) salinities at borings closer to the shore. The highest salinities (16 and 24 ppt) were observed at 22 foot depths in two borings near the shoreline and at the southern end of the project site, near the Deep Draft Harbor. High nitrate levels were generally associated with low salinity levels.

The subsurface flow of low salinity, high nutrient water toward the coast is driven by the hydrostatic head developed further inland. The presence of the lagoons and marina will not tend to increase the rate of this flow, but may change the location at which it enters the ocean. Without the lagoons or marina being present, the groundwater flow would enter the ocean as a diffuse flow over a wide horizontal extent. The lagoons and marina, being located within the depth range of the freshwater layer, may tend to collect what would otherwise be a diffuse flow and become "point" sources for the discharge of this water.

The impact of these point sources of high nutrient water is not likely to be significant to the marine environment. The nearshore area into which they discharge is well mixed due to continuous wave action. Thus any concentrated flow of high nutrient water into the region would be rapidly mixed and diluted to ambient levels. In addition, the long response time of phytoplankton to additions of nutrients makes any significant biolgical response unlikely. Studies of the response of resident phytoplankton to additions of high nutrient water (Bienfang, 1983; Szyper et al, 1984) have shown no measurable change in population levels within two to four days after nutrient addition. Even under conditions of no wind and no surf, tidal flows and ambient currents are sufficiently strong (Bathen,

unpublished data; Divoky et al, 1984) to dilute water from the lagoons and marina sufficiently that no growth response would occur.

The lagoons have been designed with flushing rates sufficiently high that no increases in microalgal populations within them are likely to occur, even under the worst case conditions of no wind or surf. Bathen (1984) states that the flushing rate of the lagoons will be similar to a natural lagoon near the Alice Kamokila Campbell residence and will be between 1 and 24 volumes per day, with an average of 20 volumes per day. Given these high flushing rates, water quality within the lagoons will be similar to that of the adjacent coastal region.

The water quality within the marina has been projected based on considerations of residence times ranging from 2 to 8 days, groundwater input rates, nutrient levels, and phytoplankton growth rates (Bienfang, 1979); water quality projections for the adjacent Barbers Point Harbor have also been made (Environmental Consultants, Inc., 1979). The flushing rate of water within the marina is determined by wind speed and direction and tidal exchange; the residence time of water within the marina has recently been projected to be less than one to 2 days, (Bathen, 1984). Projected phytoplankton standing stocks for this residence time and utilizing conversion of phosphate or nitrate concentrations to chlorophyll ranged from 4.5 to 7.4 ug/l as chlorophyll (Bienfang, 1979). These levels are 10 times higher than those observed in the Barbers Point Harbor (Environmental Consultants, Inc., 1979). The Environmental Consultants, Inc. (1979) projection does not provide quantitative estimates of phytoplankton stocks, but suggests that given similar flushing rates the Harbor would experience increases of up to 4 times in chlorophyll.

These projections, however, require steady-state (constant) conditions which exist over a long period of time. In order for phytoplankton populations to increase, their growth rate must exceed the turn-over rate of the system in which they are growing. Residence times of 3/4 of a day to 2 days give turn-over rates of 0.5 to 1.33 per day. While algal growth rates of 1 to 2 doublings per day have been observed in relatively enclosed bodies and in experimental containers, in natural waters the growth rates are usually on the order of 0.1 to 0.2 doublings per day. Given the observed lag in algal growth response to increased nutrient supplies, it may not be possible for natural populations to response to high nutrient concentrations in any significant manner (i.e., exhibit significant increases in population density) in the face of the more likely high turn-over rates.

No consideration of the effects of grazing, which would significantly decrease phytoplankton stocks, were included in either projection. Given the range of growth rates observed in natural phytoplankton stocks, the uncertainties in the conversion

of nutrient level to chlorophyll, the likelyhood of higher flushing rates than assumed above and the certain presence of grazers, the likelyhood of phytoplankton blooms within the marina which may cause negative water quality impacts is small, except perhaps under unusual circumstances of little or no wind and minimal tidal exchange.

The presence of high nutrient water may induce increased growth of macroalgae, however. The newly-exposed hard surfaces of the lagoons and marina would present attachment sites for macroalgal species. The growth of these species is normally limited by the low levels of nutrients in the nearshore area. Given continuous exposure to an increased level of nutrients due to groundwater influx, it is not unreasonable to expect that higher growth rates of these macroalgae would occur. This is not to say that these hard substrate areas would become overgrown by macroalgae, however. Macroalgae all along the coastline are continuously cropped down by resident fish populations (Brock, 1984), and it is likely that the same process would occur at the new lagoon surfaces. Given the higher production rate of the algae, one could reasonably expect that higher population densities of the grazing fishes might develop.

The presence of new surfaces, increased growth of macroalgae and potentially increased fish populations presents the potential for outbreaks of ciguatera poisoning. Ciguatera, a disease resulting from the ingestion of toxic fish, affects the central nervous system of man. One of the known sources of the disease is a benthic dinoflagellate (Gambierdiscus toxicus) that is associated with benthic algae. The toxin produced by the dinoflagellate is concentrated as it moves up through the food web. Most such outbreaks in Hawaii appear to be associated with new construction in the marine environment. Randall (1958) suggested that the primary agent of the disease (G. toxicus) is an early colonizer on new or denuded surfaces. New surfaces may result from construction activities, storms or the dumping of scrap materials. One outbreak of ciguatera at Pokai Bay about 10 km north of Kahe Point in August, 1978 was coincident with the dredging of a harbor and a small bloom of G. toxicus (Ocean Thermal Corporation, 1983).

It should be emphasized, however, that the relationship between new construction and ciguatera outbreaks is not well established. Many construction projects in marine waters have not had any ciguatera events associated with them; conversely, some ciguatera events have occurred in the absence of construction. From 1900 to 1974, there were 59 incidents of ciguatera affecting 450 people in the Hawaiian Islands; in the 1975-1981 period, 81 outbreaks have occurred, affecting 203 individuals (Sunday Star Bulletin and Advertiser, 9 October, 1983). Fish taken from the Pokai Bay area have caused 15 outbreaks since 1976. No cases have been reported from Kahe Point in the 1976-1981 period. No cases of ciguatera poisoning have been reported in association with the dredging operations at the Barbers Point Deep Draft Harbor (J. Maragos, pers. comm.)

Unless a ban on fishing in the West Beach area is imposed during and after construction, the potential for ciguatera outbreaks exists. The Army Corps of Engineers, in an attempt to monitor the potential for ciguatera outbreaks, has established a program (J. Maragos, pers. comm.) of periodic sampling of benthic algae in the Barbers Point area and examination of these algae for increased levels of Gambierdiscus toxicus, the algal species which produces the ciguatera toxin. This procedure is designed to identify increases in toxicity potential before the toxic material reaches the higher levels of the food chain, and thus allow time to curtail fishing activities and give public warning of potential toxic conditions before they occur. Such a monitoring program may trigger "false alarms", however; preliminary research conducted at the Hawaii Institute of Marine Biology has shown that G. toxicus cultured in the laboratory can be induced to rapidly increase by the simple addition of macrothalloid algal extract. However, these growing cultures appear not to synthesize the toxin. Furthermore, field sampling carried out in the Waikiki, Oahu, area has shown G. toxicus occurring in abundance, yet the presence of toxic fish from these reefs has not been reported (R. York, pers. comm.). The protection to public health afforded by such a monitoring program more than offsets any hardships or conflicts imposed by possible false alarms.

3. Run-off and Siltation

During periods of heavy rainfall, the capacity of the drainage basin to absorb excess water may be exceeded, and surface flow will result, especially with the change in use of the majority of the development area from agriculture to resort/residential, roadway, or golf course. The existing drainage pattern on the property is storm runoff flowing through the old cane fields via unimproved channels and depressions. Surface flow is planned to be discharged into the ocean at two points: the majority of the flow will be channeled into the marina, from where it will reach the ocean directly through a separate marina entrance channel or indirectly via the Barbers Point Deep Draft Harbor entrance channel, depending upon which design is selected; the balance will be discharged at the shoreline north of the Hawaiian Cultural Center.

Until Phase IV is completed, the marina will act as a settling basin and percolation/evaporation pond, collecting runoff during storm events and holding the water while it percolates into the ground or evaporates. Calculations of maximum volume flow during storm events (Dugan, 1984) and the retention volume of the marina (approximately 360 acre-feet, assuming no percolation or evaporation loss) indicate that the volume of the marina would be sufficient to receive all surface flow channeled to it without overflow occuring for any storm of one hour duration, for up to a 10-year storm of 6 hour duration, and for an approximately 3 year storm of 24 hour duration. It is thus likely that during the construction phases of the project

the marina will effectively prevent most if not all storm water runoff from the majority of the project area from reaching the ocean.

After the marina is completed, storm water will be discharged to the ocean as a surface layer. Depending on the surf conditions, flow volume and sediment load, this flow may persist as a visible surface plume. This intermittent flow is similar to that which occurs in several natural streams along the Waianae coast, which discharge sediment-laden water into the ocean during heavy rainfall. The relative impact of the West Beach discharges will probably be less than those of the natural streams, since much of the West Beach flow will be over either solid, non-eroding surfaces (concrete, asphalt) or well-grassed landscaping areas, and will be channeled in grasses swales or box culverts. Dugan (1984) has calculated the volumes of sediment discharged before and after completion of the West Beach project. His figures indicate the concentration of sediment in the runoff water will decrease from the typical undeveloped/agricultural land value of 1,500 mg/l suspended solids to a value of 250 mg/l typical of residential/commercial areas, and that total sediment volume discharged will increase for storms of 1 hour duration (due to increased volume flow), but will decrease compared to natural discharges (due to decreased concentration) for storms of 6 and 24 hour duration.

The discharge of sediment-laden water from the marina will potentially impact only sessile benthic organisms (specifically corals) within some distance from the marina entrance. The coral communities within several kilometers of the marina entrance are characterized by relatively low coverage and low diversity (AECOS, 1979; Bienfang and Brock, 1980; Brock, 1984). Much of the nearshore area is well scoured by wave action, and sedimentation is not likely to cause significant build-up of material or coral mortality. The discharge flow will be diluted by wave action and currents, and sedimentation rates over areas outside the region of wave scour and re-suspension will probably be small. The marina itself will act as a settling basin. Bathen (1984) estimates that half of the suspended material would settle out in the marina. Those particles which, without the precence of the marina, would settle out rapidly and form a potentially harmful deposit on corals, would be removed from the flow before it reached the ocean. The particles still in suspension, then, would tend to stay in suspension longer and be dispursed over a wide area, thus lessening any potential damage to corals.

Discharge from the northern portion of the project area has a higher potential for impact, mainly because the corals in that area are more abundant and diverse than in areas further south (AECOS, 1979; Bienfang and Brock, 1980), and because no mechanism for removal of suspended material before discharge to the ocean will exist. The northern discharge will be composed of flows from areas not impacted by the West Beach development as well as developed areas. Dugan (1984) has estimated that the

amount of suspended material from the northern drainage area will increase after development during storms of 1 hour duration, but will decrease for storms of longer duration. The magnitude of the increase in suspended material discharge is relatively small. While the increase in sediment for a 1 year storm of one hour duration will be on the order of 150% over current conditions, this amount is only 30% of that discharged during a 5 year storm of one hour duration. Such storms (infrequent and of short duration) input sediment to the ocean as a "puff" which may deposit sediment on nearshore corals for only a short period of time. Corals possess a mechanism for removal of deposited sediment which is quite effective and are able to withstand relatively short-duration sediment loading; sediment deposition over a long period of time has much more potential for negative impacts. The corals in the area are naturally exposed to sediment discharge under present undeveloped conditions, and have developed into the abundant and diverse community described. The relatively small increases in sediment deposition on a short-term, sporadic basis will probably not result in any significant changes within the existing population.

4) Increased Shoreline Access

The West Beach development will open up an area which is now relatively inaccessible to potentially much greater use by recreational and subsistance fishermen. At present, most fishing in the area is by pole-and-line, and the target species are mostly reef fish (AECOS, 1979; Bienfang and Brock, 1980; Brock, 1984). Increased fishing pressure could potentially adversely affect the local stocks of these fish, although some recruitment and replacement from surrounding areas would probably occur. An analysis (Atkinson, 1984) of the effects of the proposed OTEC plant at Kahe Point of fish stocks of the area (which included impacts due to plant operation and increased fishing pressure) concluded that natural predation was the factor most heavily controling population levels and that increased fishing pressure would only be of secondary importance in determining population levels. It is likely that this conclusion is also valid for the West Beach area, and that any reasonable increase in fishing effort due to improved access to the shoreline would not significantly change the overall reef-fish populations.

The presence of the marina with its launching and berthing facilities will probably result in an increase in boat-based fishing activity along this stretch of coastline, with potential negative impacts to fisheries such as the offshore bottomfishes, which appear to be at or near maximum sustainable yield (Ralston and Polovina, 1982). Increased sport fishing could also be expected.

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APPENDIX A

Inventory Of The Macrobiota In The Area Affronting The Limestone Beach Shoreline At West Beach, Oahu

July, 1984

Dr. R. E. Brock

Introduction

The need for a survey of the extant marine communities in the shallow waters affronting West Beach was identified with the proposed development of four manmade lagoons to be situated along this shoreline (see Figure 1). The purpose of this survey is to locate, identify, and describe the macrobenthic resources in the immediate vicinity of the proposed lagoon development, as well as comment on potential impacts associated with this development.

There is a very limited amount of credible information concerning the nearshore biota of the West Beach area. The most complete and comprehensive survey was carried out by Bienfang and Brock (1980). The area of study in Bienfang and Brock (1980) covered a 5 km section of coast from the Campbell Industrial Park Barge Harbor northwest to the southernmost point of Kahe Beach Park and seaward to the 18 m isobath. Within this locality lies the area of concern of the present study; it is bordered on the south by the new state harbor and on the north by the Kamakila Campbell Estate. Bienfang and Brock (1980) conducted only three quantitative stations surveying benthic communities in the nearshore region of this 2 km section of coastline. The present study provides greater detail on the macrobenthic communities now present (June, 1984) in this area.

At present, industry borders both to the north and south of West Beach and most of the area is agricultural with little human modification to the shoreline. However, since the time of the Bienfang and Brock (1980) study (conducted in November-December, 1979), the old barge harbor to the south has undergone expansion with considerable dredging activity. Natural storm-generated impacts have likewise occurred. A large storm in January, 1980 affected coral communities in the Kahe Point area (Coles *et al.*, 1982) and Hurricane Iwa in November, 1983 severely affected much of the leeward Oahu coastline. These events may have had considerable impact on the marine communities, thus further justifying the need for the present study.

Methods

The fieldwork which provided the database for this inventory of the macrobiota in the shallow waters affronting West Beach, Oahu was conducted between 15 - 28 June, 1984. The area encompassed in this survey is from the intertidal bench seaward to the 3.5 m isobath at West Beach and is bordered by the newly expanded harbor at Campbell Industrial Park on the south and the southern boundary of the Kamakila Campbell estate on the north (Figure 1).

As noted in the previous West Beach study (Bienfang and Brock, 1980), the quantitative sampling of macrofaunal marine communities presents a number of problems; many of these are related to the scale on which one wishes to quantitatively enumerate organism abundance. Marine communities in the

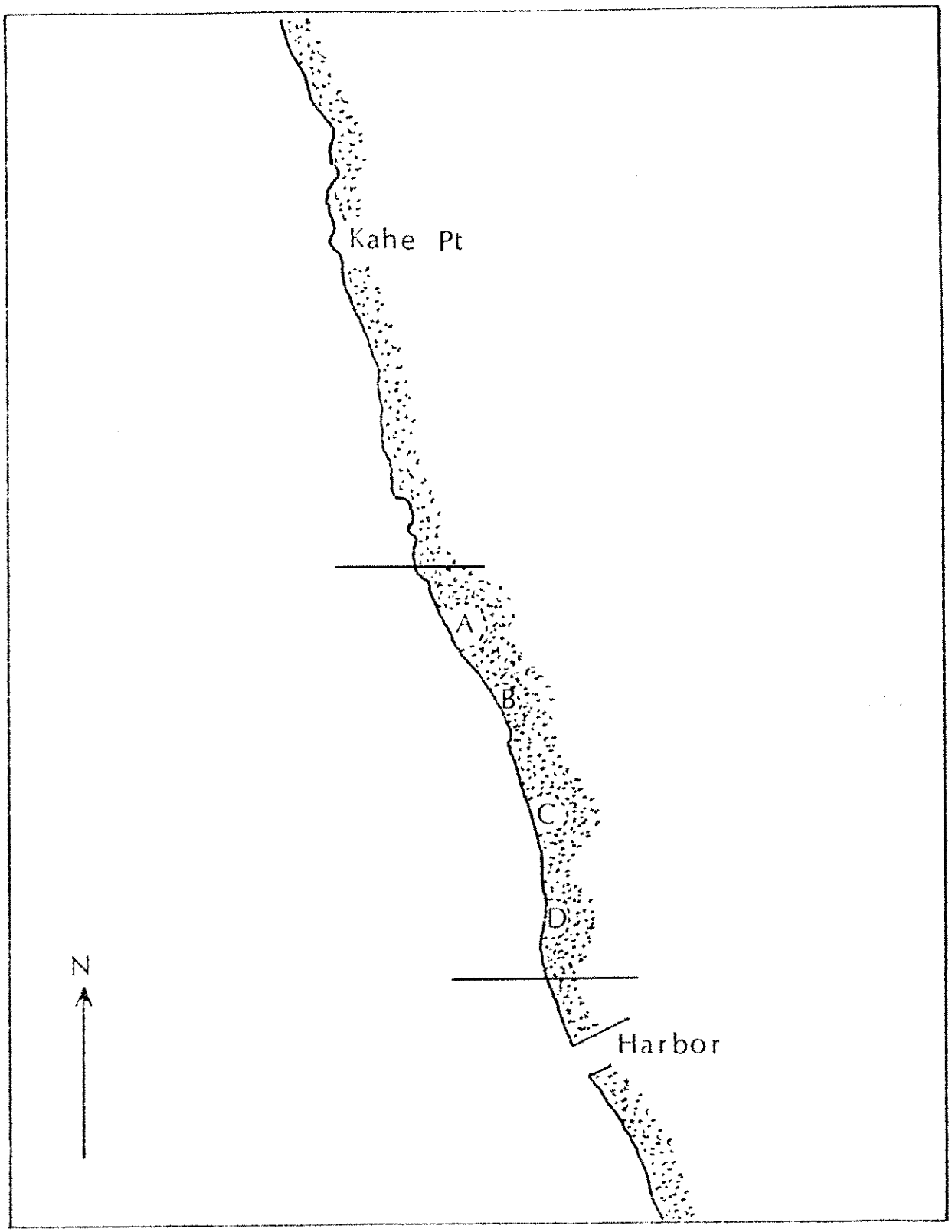

Figure 1. Map showing the coastline at West Beach, Oahu and the approximate location and configuration of the four proposed lagoons (designated by A-D). Also shown are the northern and southern boundaries of this study. (Scale: 1 cm = 250 m)

nearshore region of West Beach may be spatially defined in a range on the order of a few 100^2 cm (such as the community residing in a Pocillopora meandrina coral head) to major biotopes covering many hectares. Recognizing this ecological characteristic, as well as having preliminary inventory information for the area in question, we designed a sampling program that attempted to delineate the major extant communities in the limits of the study area and to quantitatively describe these communities. Thus, a number of techniques were used. The methods here are the same as those used by Bienfang and Brock (1980). Description of these methods is purposely quite explicit so that any subsequent survey seeking to evaluate changes in the area can benefit from the use of the same or comparable techniques.

The study of the West Beach biota by Bienfang and Brock (1980) described the nearshore area (within ~150 m of the shoreline) as being comprised of one major biotope, that is, the shallow low coral coverage biotope. This biotope occurs as a continuous band adjacent to and through the southernmost two-thirds of the West Beach coastline. To obtain an overall perspective on the extent and variation in the major communities in this biotope, a diver equipped with SCUBA swam through much of the area affronting each of the four proposed lagoons. This exercise allowed the qualitative delineation of variation and differences in the major structural elements (e..g, amount of hard substratum, cobble, vertical relief, fish abundance and coral coverage) and marine communities present. Within each of these major areas, quantitative stations were erected and quantitative studies were conducted, including visual enumeration of fish, counts along benthic transect lines, and cover estimates in benthic quadrats. In addition to these quantitative measures, a qualitative reconnaissance was made through the area affronting each of the four proposed lagoons by swimming and noting the presence of species not encountered in the transects.

The location of stations was subjectively chosen as being representative of a given major community. Immediately following site selection, a visual fish census was undertaken to estimate the abundance of fishes. These censuses were conducted over a 20 x 4 m corridor and all fishes within this area to the water's surface were counted. A single diver equipped with SCUBA, transect line, slate and pencil would enter the water, count and note all fishes in the prescribed area [method modified from Brock (1954)]. The 20 m transect line was played out as the census progressed, thereby avoiding any previous underwater activity in the area which could frighten wary fishes.

In addition to frightening fishes, other problems with the visual censis technique include the underestimation of cryptic species such as moray eels (Family Muraenidae) and nocturnal species, e.g., Squirrelfishes (Family Holocentridae), aweoweos (Family Priacanthidae), etc. This problem is compounded in areas of high relief and coral coverage affording numerous shelter sites. Species lists and abundance estimates are more accurate for areas of low relief, although some fishes with cryptic habits

or protective coloration (e.g., the nohus--Family Scorpaenidae; the flatfishes--Family Bothidae) might still be missed. Obviously, the effectiveness of the visual census technique is reduced in turbid water (a problem in the present survey), and species of fishes which move quickly and/or are very numerous may be difficult to count. Additionally, bias related to the experience of the diver conducting counts should be considered in making any comparisons between surveys. In spite of these drawbacks, the visual census technique probably provides the most accurate nondestructive assessment to date of diurnally active fishes.

After the assessment of fishes, an enumeration of epibenthic invertebrates (excluding corals) was undertaken using the same transect line as established for fishes. Exposed invertebrates usually >2 cm in some dimension (without disturbing the substratum) were counted in a 20 x 4 m area. As with the fish census technique, this sampling methodology is quantitative for only a few invertebrate groups, e.g., some echinoderms and holothurians. Most coral reef invertebrates (other than corals) are cryptic or nocturnal in their habits, which makes accurate assessment of them in areas of topographical relief very difficult. This, coupled with the fact that the majority of these cryptic invertebrates are small, necessitates the use of methods beyond the scope of this survey (e.g., Brock and Brock, 1977). Recognizing constraints on time and the scope of this survey, the invertebrate censusing techniques used here attempted only to assess those few macroinvertebrate species that are diurnally exposed.

Exposed sessile benthic forms such as corals and macrothalloid algae were quantitatively surveyed by use of quadrats and the point-intersect method. The point-intersect technique only notes the species of organisms or substratum type directly under a point. Along the previously set fish transect line, 40 such points were assessed (once every 50 cm). These data have been converted to percentages. Quadrat sampling consisted of recording benthic organisms, algae and substratum present as a percent cover in five one m^2 frames placed at 5 m intervals along the transect line established for fish censusing (at 0, 5, 10, 15, and 20 m).

Macrothalloid algae were rarely encountered in the 1 x 1 m quadrats or under one of the 40 points; microalgal species were common but are difficult to identify in the field and thus were ignored. Due to their normally ephemeral nature and apparent small contribution to benthic coverage in this survey, the presence of macrothalloid algal species in the vicinity of each station were noted and visual estimates made on relative coverage by each of the most common species.

On the shoreline limestone bench at West Beach, an abundant algal community exists; subtidally the standing crop is much less due to heavy grazing pressure exerted primarily by surgeonfishes (Family Acanthuridae) and parrotfishes (Family Scaridae). On

this intertidal bench, the algae are protected from these grazing fish due to its emergent nature. This algal community was surveyed in the area of each of the four proposed lagoons. Two stations on the bench affronting each lagoon were selected and inventoried using a relative abundance scale from 1 to 5, where 1 = rare, 3 = common, and 5 = abundant. Diurnally exposed macroinvertebrates are not dominant lifeforms on such bench systems and thus were not quantitatively assessed; however, the entire shoreline of the study area was inspected for macroinvertebrates that are either rare or of commercial importance. None were found.

During the course of the fieldwork, records were kept on observed recreational use patterns in the area while carrying out other field studies. This included counts of shore fishermen, number of fishing poles in use, boats in the vicinity, and the fishing activity from these boats.

Simple methods of data reduction and analysis have been used and are described where met with in the text. Diversity (H') is calculated as described by Pielou (1966) where:

$$H' = - \text{sum}(p_i \ln p_i)$$

where p_i is that proportion of the individuals censused belonging to species i. This is the Shannon-Wiener index.

Results

In total, 10 stations were quantitatively sampled in this survey (see Figure 2). Two stations were sampled affronting each of the four proposed lagoons; thus, Stations 2 and 3 sampled the benthic communities in the vicinity of Lagoon A; Stations 4 and 5--Lagoon B; Stations 6 and 7--Lagoon C; and Stations 8 and 9 sampled communities offshore of Lagoon D. Additionally, one station to the north of Lagoon A (Station 1) and a second to the south of Lagoon D (Station 10) were sampled. Stations 1, 2, and 10 were selected for comparative purposes and are in the approximate locations of the three stations (Numbers 8, 10, and 11) sampled by Bienfang and Brock (1980).

The area encompassed in this study lies within Biotope 1 as defined by Bienfang and Brock (1980). This biotope (termed the shallow coral coverage biotope) parallels the shoreline throughout the entire present study area. It is contiguous from an emergent limestone bench fronting the beach seaward to about 5 m depth (about 100-150 m offshore). This zone receives a considerable amount of scouring due to wave activity. The substratum is primarily solid limestone with small scattered corals in encrusting or prostrate growth forms (e.g. Porites lobata, Montipora verrucosa). In the shallow shoreward portions of the bench, the limestone is pitted with potholes (to 1 m diameter) due to the abrasive action of rocks and stones; more seaward are surge channels (the ridges being in 1 to 2 m of water to 5 m in depth) oriented perpendicular to the shore. From this

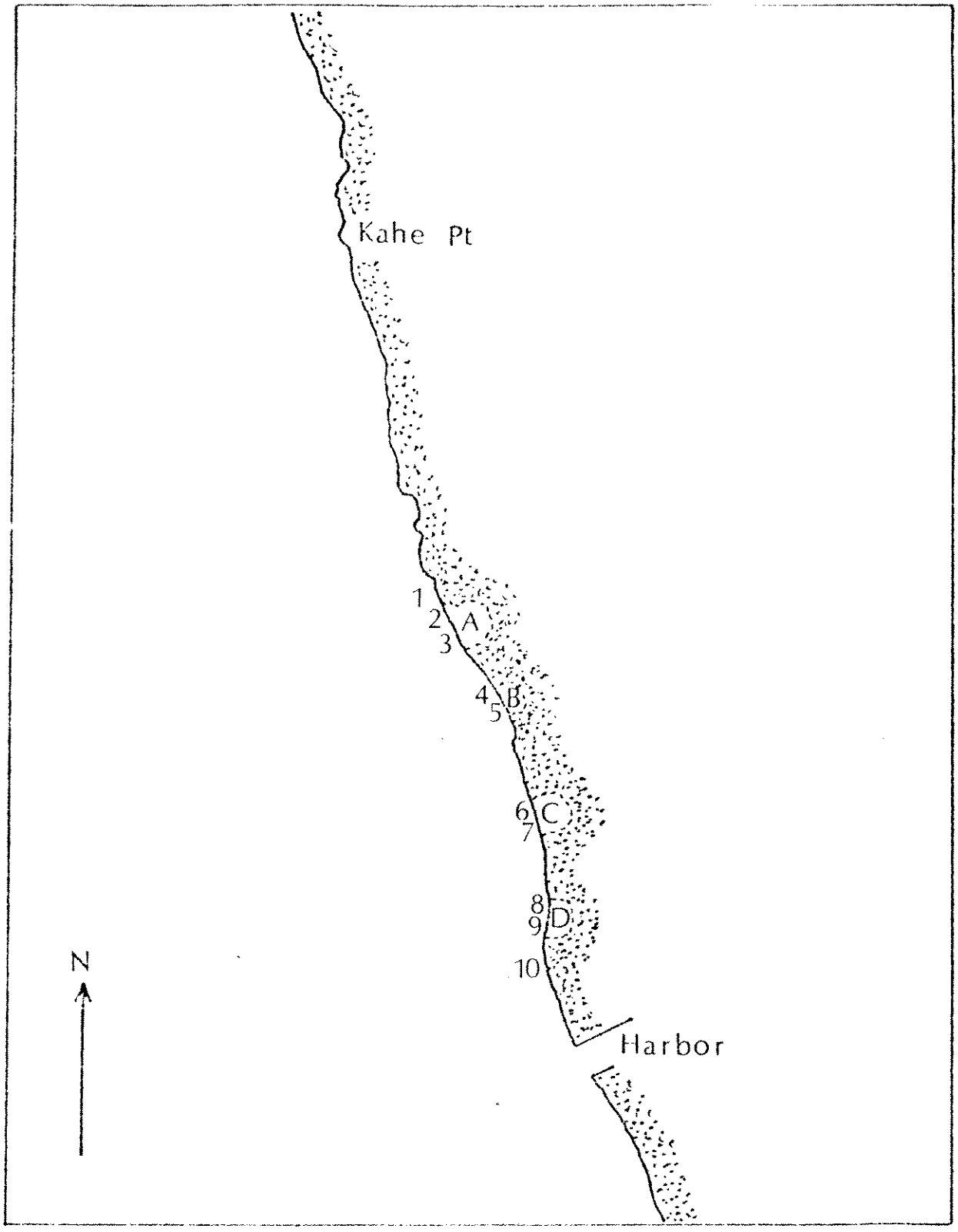

Figure 2. Map showing the approximate locations of the 10 quantitative stations ampling shallow benthic communities adjacent to the four proposed lagoons at West Beach, Oahu. (Scale: 1 cm = 250 m)

point, the substratum slopes seaward to 5-6 m of depth where this high energy biotope merges with the more offshore high coral coverage biotope of Bienfang and Brock (1980).

Species found in the high energy biotope are, in general, those characteristically seen subtidally in any Hawaiian shoreline bench system. These include the corals Pocillopora meandrina and Porites lobata, the small sea urchins Echinometra mathaei and E. oblongata, the snails Drupa morum, D. granulata and D. brunneolabrum. Algal species encountered in this biotope are given in Table 1. A number of fish species are characteristic of this biotope, including the surgeonfishes Acanthurus triostegus (manini), A. achilles (paku'iku'i), the wrasse Thalassoma umbrostigma, and the damselfish, Abudefduf imparipennis. There are a number of other diurnally active fish species that are commonly seen not only in this biotope, but in others as well. Cryptic species present in this zone include the spiny lobster, Panulirus penicillatus, the slipper lobster, Paribaccus antarcticus, the cowry, Cypraea maculifera as well as the aholehole, Kuhlia sandvicensis, and the ala'ihi, Adioryx lactaeoguttatus.

Station 1 is located approximately 250 m north of Station 2 (Figure 3) in 2 m of water about 35 m offshore. The general physiography at this station is a solid limestone bench with scattered potholes (1 to 4 m in diameter) about 2 to 10 m apart. These potholes have a mean depth of 0.5 m. The most important coral species present is Porites lobata; average coverage of all coral species is about 3%. Two species of stony corals were present in the quantitative transect, other species seen nearby include Montipora verrucosa, M. patula, Pavona duerdeni, Porites evermanni and Cyphastrea ocellina. Fourteen species of fishes were recorded in the visual census (Table 2; Appendix I). Most abundant were the surgeonfish, canthurus nigrofuscus and the hinalea, Thallassoma duperreyi. Not censused but seen in the area were the tang, Naso literatus, the butterfly fishes, Chaetodon lunula and C. miliaris, the damselfish, Abudefduf imparipennis and the uhu, Calotomus spinidens. Eight invertebrate species were censused in the 20 x 4 transect (Table 2); in the vicinity were seen the slipper lobster, Paribaccus antarcticus and the cowry, Cypraea maculifera. Algae encountered at this station and their relative abundances are given in Table 3.

Figure 2 gives the general locations of the proposed four lagoons and ten stations along the West Beach coast; Figures 3 through 6 show the detailed bathymetry and more precise locations of the pairs of stations (2 through 9) that are offshore of the four lagoons. Stations 2 and 3 (Figures 2 and 3) sampled benthic communities adjacent the proposed Lagoon A. Station 2 is located about 40 m seaward of the emergent limestone beach in about 5 m of water. The limestone substratum at this station was overlain by occasional large (~2 x 5 m) loose limestone slabs affording some cover; however, only 14 species of fishes were counted (Table 4; Appendix I). Most of the fish censused were in one

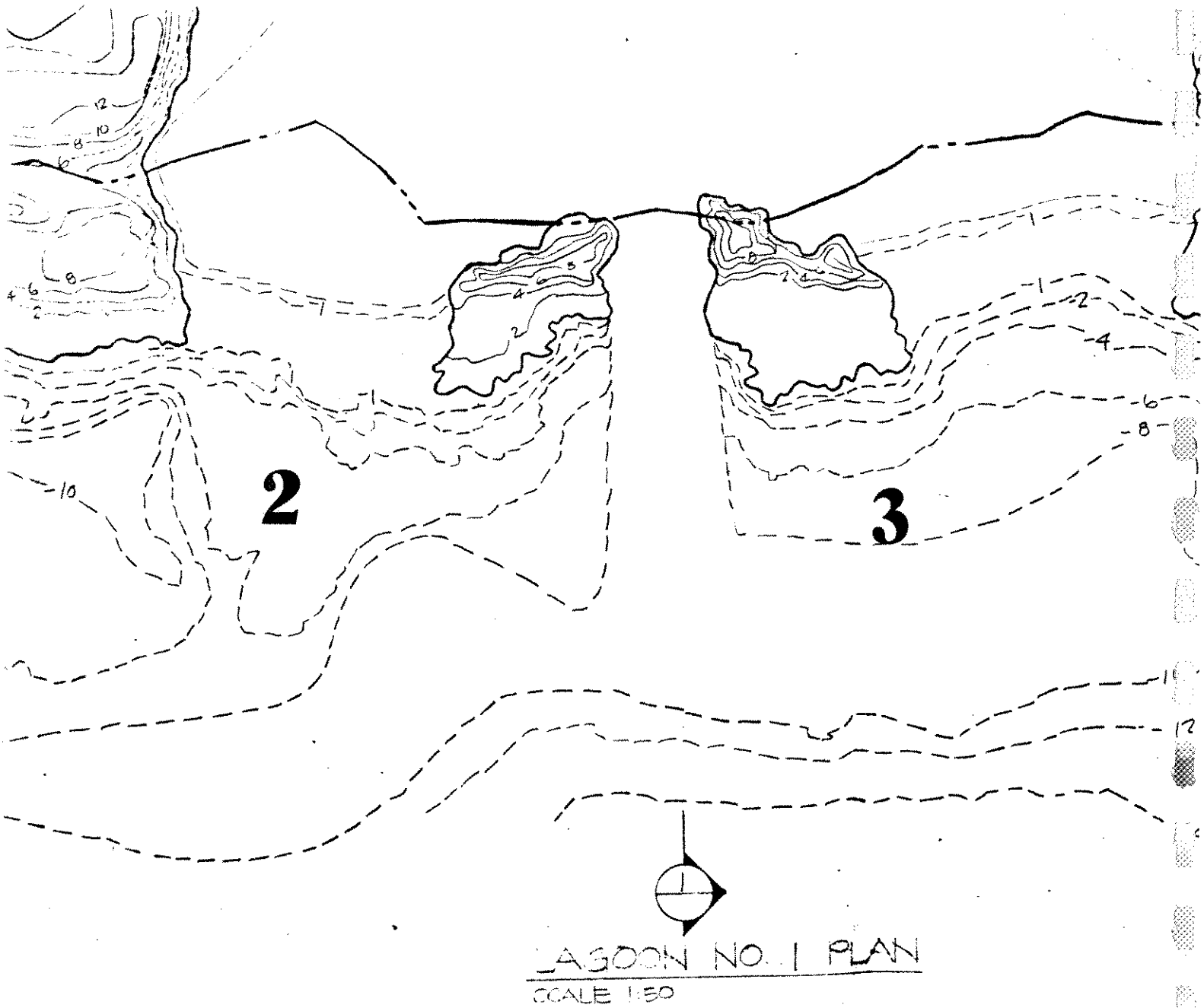

Figure 3. Map of the nearshore bathymetry in the area affronting the northernmost of the four proposed lagoons (Lagoon A, Figure 1) showing the proposed cuts through the existing limestone bench. Given in the figure are the approximate locations of Stations 2 and 3. (Scale: 1:50)

Table 1. Macrothalloid algal species encountered during the June 1994 subtidal survey in the vicinity of the four proposed lagoons at West Beach, Oahu

CYANOPHYTA

Hormothamnion enteromorphoides
Hormothamnion sp.
Lyngbya majuscula
Lyngbya sp. 1
Lyngbya sp. 2
Nostoc spumigena
Oscillatoria sp. 1
Oscillatoria sp. 2

CHLOROPHYTA

Caulerpa racemosa
C. racemosa var.
Desmidiaceae (unid.)
Dictyosphaeria versluysii
Pseudochlorodesmis parva
Strevea anastomosans
Udotea javensis
Valonia ventricosa

PHAEOPHYTA

Dictyota bartayresii
D. friabilis
D. divaricata
Lobophora variegata
Ralfsia pangoensis

RHODOPHYTA

Amansia glomerata
Amphiroa fragilissima
Amphiroa sp.
Antithamnion sp.
Asparagopsis taxiformis
Cladymenia pacifica
Desmia hornemanii
Geliadiales (unid.)
Gelidiella acerosa
Gelidiella sp.
Gelidium pusillum
Gelidiopsis sp.
Griffithsia sp.
Halymenia formosa
Hydrolithon reinboldii
Hypnea sp.
Exophyllum wentii
Gelidium sp.
Jania capillacea
Jania sp.
Laurencia parvipapillata
Lithothamnion sp. 1
Lithothamnion sp. 1
Lithophyllum kotschyanum
Lithophyllum sp. 1
Lithophyllum sp. 2
Martensia fragilis
Mesophyllum mesomorphum
Neogoniolithon frutescens
Peyssonellia rubra
Porolithon sandvicense
Porolithon sp.
Pterocladia capillacea
Sporolithon erythraeum
Tolypiocladia glomerata

Table 2. Summary of the benthic survey conducted at Station 1 at West Beach, Oahu. Results of the 5 m^2 quadrat sampling of the benthic community present (expressed in percent cover) is given in Part A; a 40-point analysis is presented in Part B and counts of invertebrates in Part C. A short summar of the fish census is given in Part D. Water depth 2 m; mean coral coverage 3% (quadrat method).

A. Quadrat Survey

Species	Quadrat Number 1	2	3	4	5
Corals					
Porites lobata	2	1	4	3.5	5
Pocillopora meandrina				0.1	
Anthelia edmondsoni		1		1	0.5
Palythoa tuberculosa			8		
Hard Substratum	98	98	88	95.4	94.5

B. 40-Point Analysis

Species	Percent of Total
Corals	
Porites lobata	10
Anthelia edmondsoni	2.5
Palythoa tuberculosa	2.5
Hard Substratum	85

C. Invertebrate Census (20 x 4 m)

Species	No.
Phyllum Cnidaria	
Anthelia edmondsoni	9 colonies
Palythoa tuberculosa	3 colonies
Phyllum Mollusca	
Drupa granulata	4
D. morum	3
Turbo intercostalis	1
Phylum Echinodermata	
Echinometra mathaei	7
E. oblongata	2
Echinothrix diadema	1

D. Fish Census (20 x 4 m)

14 species
97 individuals

Table 3. Semiquantitative checklist of macrothalloid algal species present at Station 1, West Beach, Oahu. The relative abundance of algae was estimated using a scale of 1 to 5 where 1 = rare, 3 = common, and 5 = abundant.

Species	Relative Abundance
Lyngbya sp. 1	1
Oscillatoria sp. 1	1
Oscillatoria sp. 2	1
Hormothamnion enteromorphoides	1
Hormothamnion sp.	1
Dictyota friabilis	1
D. bartayresii	2
Ralfsia pangoensis	1
Dictyosphaeria versluysii	2
Caulerpa racemosa var.	1
Desmidiaceae (unid.)	1
Pterocladia capillacea	1
Gelidiales (unid.)	3
Gelidium pusillum	2
Gelidiella sp.	3
Martensia fragilis	1
Cladymenia pacifica	1
Gelidiopsis sp.	2
Amansia glomerata	3
Peyssonellia rubra	2
Lithothamnion sp. 1	2
Lithothamnion sp. 2	4
Neogoniolithon frutescens	1
Lithophyllum sp. 1	2
Lithophyllum sp. 2	1
Sporolithon erythraeum	1
Porolithon sandvicense	1
Hydrolithon reinboldii	1
Tolypiocladia glomerata	2

Table 4. Summary of the benthic survey conducted at Station 2 at West Beach, Oahu. Results of the 5 m^2 quadrat sampling of the benthic community present (expressed in percent cover) is given in Part A; a 40-point analysis is presented in Part B and counts of invertebrates in Part C. A short summary of the fish census is given in Part D. Water depth 45 m; mean coral coverage ~ 11% (quadrat method).

A. Quadrat Survey

Species	Quadrat Number 1	2	3	4	5
Corals					
Porites lobata	38	1	8	3	
Pavona varians	1				
Pocillopora meandrina				0.1	
Anthelia edmondsoni	1				
Montipora verrucosa		1	3		
Hard Substratum	60	98	90	96.9	100

B. 40-Point Analysis

Species	Percent of Total
Corals	
Porites lobata	12.5
Montipora verrucosa	2.5
Hard Substratum	85

C. Invertebrate Census (20 x 4 m)

Species	No.
Phyllum Cnidaria	
Anthelia edmondsoni	2 colonies
Phyllum Mollusca	
Drupa morum	2
D. brunneolabrum	5
Phylum Echinodermata	
Echinometra mathaei	9
E. oblongata	1

D. Fish Census (20 x 4 m)

14 species
588 individuals

monospecific school of manini (Acanthurus triostegus sandvicensis, ~500 individuals). Only 88 other fishes were enumerated in the 20 x 4 m census area; most common were the surgeonfishes and kole (Ctenochaetus strigosus) and the hinalea. Porites lobata was the dominant coral with local coverage attaining 30%. Overall, corals showed an average coverage of about 11% based on the quadrat method. Other corals encountered included Montipora verrucosa and Pocillopora meandrina. Motile invertebrate species censused in the 20 x 4 m transect at Station 2 include the drupes, Drupa morum and D. brunneolabrum and the sea urchins, Echinometra mathaei and E. oblongata. Algae present at this station are given in Table 5.

Station 3 is south of Station 2 (Figure 3) and about 40 m offshore in about 3 m of water. The substratum at this station is again solid limestone, with scattered potholes 1 to 2 m in diameter and 0.5 to 1 m in depth. These potholes are spaced 3 to 5 m apart. Twelve species of fishes (86 individuals) were recorded at this station (Table 6; Appendix I). The most common species of fish were the same as those at Station 2 (Acanthurus nigrofuscus, Ctenochactus strigosus, Thalassoma duperreyi). Porites lobata was the dominant coral and coverage by all species averaged 5.5%. Motile invertebrates censused at this station include three species of drupes, at least three species of hermit crabs (Calcinus spp. and Dardanus sp.) as well as two sea urchin species (Echinometra mathaei and Echinometra oblongata) and the sea cucumber, Actinopyga mauritiana. A list of macrothalloid algae present at this station is given in Table 7.

Outside of Stations 2 and 3 but in the general area affronting proposed Lagoon A, a number of other fish and coral species were seen. Fishes seen in the area include the surgeonfishes Acanthurus achilles, A. dussumieri (palani), A. leucopareius and Zebrasoma veliferum, the humuhumu Rhinecanthus rectangulus, the blenny Cirripectus lineopunctatus, the butterflyfishes Chaetodon quadrimaculatus, C. citrinelleus (rare species in Hawaii), C. unimaculatus and Forcipiger flavissimus, the omilu Caranx melampygus, the ala'ihi Adioryx lacteoguttatus, the mempachi Myripristes amaenus, the hinalea Thalassoma ballieui, the damselfish Abudefduf imparipennis, the uhus Calotomus spinidens and Scarus perspicillatus, the kihikihi Zanclus canescens and the puffer Arothron hispidus. In total, 41 species of fishes were found in the area. Coral species present but not recorded in the quantitative survey include Porites evermanni and Montipora verrilli. Overall, coral coverage throughout the area is about 9% which is relatively high for a nearshore high energy habitat; this high coverage may be related to the relatively deep water right up to the emergent limestone bench. Macroinvertebrate species seen in this area included the shrimps Stenopus hispidus and Saron marmoratus, the cones Conus lividus and C. abraeus, the cowry Cypraea maculiferra and the wana Echinothrix diadema. Two stations sampled the supratidal macroalgae present along the exposed limestone bench

Table 5. Semiquantitative checklist of macrothalloid algal species present at Station 2, West Beach, Oahu. The relative abundance of algae was estimated using a scale of 1 to 5 where 1 = rare, 3 = common, and 5 = abundant.

Species	Relative Abundance
Lyngbya sp. 1	1
Lyngbya sp. 2	1
Oscillatoria sp. 2	1
Hormothamnion enteromorphoides	1
Hormothamnion sp.	1
Desmidiaceae (unid.)	1
Ralfsia pangoensis	1
Dictyota bartayresii	2
Caulerpa racemosa	1
C. racemosa var.	1
Struvea anastomosans	1
Udotea javensis	1
Dictyosphaeria versluysii	1
Desmia hornemanii	1
Tolypiocladia glomerata	2
Pterocladia capillacea	1
Gelidiopsis sp.	1
Gelidialea (unid.)	3
Gelidiella sp.	2
Cladymenia pacifica	1
Amansia glomerata	3
Peyssonellia rubra	3
Lithothamnion sp. 1	1
Lithothamnion sp. 2	4
Lithophyllum sp. 2	1
Porolithon sandvicense	1
Hydrolithon reinboldii	2
Mesophyllum mesomorphum	1
Hypnea sp.	1
Antithamnion sp.	1

Table 6. Summary of the benthic survey conducted at Station 3 at West Beach, Oahu. Results of the 5 m^2 quadrat sampling of the benthic community present (expressed in percent cover) is given in Part A; a 40-point analysis is presented in Part B and counts of invertebrates in Part C. A short summary of the fish census is given in Part D. Water depth 3 m; mean coral coverage 5% (quadrat method).

A. Quadrat Survey

Species	Quadrat Number 1	2	3	4	5
Corals					
Porites lobata	10	3	2	3	3.5
Pocillopora meandrina			2.5	1	0.1
Cyphastrea ocellina	0.1				
Pavona duerdeni					2
Anthelia edmondsoni	0.5				
Hard Substratum	89.4	97	95.5	95.9	94.4

B. 40-Point Analysis

Species	Percent of Total
Corals	
Porites lobata	15
Pocillopora meandrina	2.5
Montipora patula	2.5
Hard Substratum	85

C. Invertebrate Census (20 x 4 m)

Species	No.
Phyllum Cnidaria	
Anthelia edmondsoni	6 colonies
Phyllum Mollusca	
Drupa granulata	17
D. morum	7
D. brunneolabrum	6
Phylum Arthropoda	
Calcinus spp.	9
Dardanus sp.	1
Phylum Echinodermata	
Actinopyga mauritiana	1
Echinometra mathaei	14
E. oblongata	3

D. Fish Census (20 x 4 m)

12 species
86 individuals

Table 7. Semiquantitative checklist of macrothalloid algal species present at Station 3, West Beach, Oahu. The relative abundance of algae was estimated using a scale of 1 to 5 where 1 = rare, 3 = common, and 5 = abundant.

Species	Relative Abundance
Nostoc spumigena	1
Oscillatoria sp. 2	1
Lyngbya sp. 1	1
Caulerpa racemosa	1
Dictyosphaeria versyluysii	1
Pseudochlorodesmis parva	1
Udotea javensis	1
Dictyota friabilis	2
Ralfsia pangoensis	2
Martensia fragilis	1
Antithamnion sp.	1
Polypiocladia glomerata	2
Hypnea sp.	1
Geliadiales (unid.)	2
Gelidiopsis sp.	1
Gelidium pusillum	2
Gelidiella sp.	2
Amansia glomerata	3
Halymenia formosa	1
Exophyllum wentii	1
Peyssonellia rubra	2
Asparagopsis taxiformis	1
Lithothamnion sp. 1	1
Lithothamnion sp. 2	4
Porolithon sandvicense	1
Lithophyllum sp. 1	2
Lithophyllum sp. 2	1
Neogoniolithon frutescens	1
Hydrolithon reinboldii	1

Table 8. Macrothalloid algal species and their relative abundances as found at two intertidal stations on the limestone bench affronting proposed Lagoon A at West Beach, Oahu. Relative abundances were estimated using a scale of 1 to 5 where 1 = rare, 3 = common, and 5 = abundance.

Station A-1

a. Bench (0.5 - 1 m above MLLW)

Species	Relative Abundance
Ulva fasciata	1
Dictyosphaera cavernosa	1
Cladophoropsis luxurians	1
Padina japonica	1
Sargassum echinocarpum	1
Acanthophora spicifera	4
Pterocladia capillacea	1
Laurencia nidifica	1
Laurencia sp.	1
Gelidiella acerosa	1
Polysiphonia sp.	1
Porolithon onkodes	1
Amphiroa fragilissima	1

b. Incline (>1 m)

Lyngbya sp. 3	1
Cyanophyta (unid.)	1
Giffordia breviarticulata	1

Station A-2

a. Bench (1/2 - 1 m above MLLW)

Chaetomorpha antennina	1
Cladophoropsis luxurians	1
Dictyosphaeria versluysii	1
Dictyota friabilis	1
Sargassum echinocarpum	1
Pterocladia capillacea	1
Acanthophora spicifera	2
Laurencia nidifica	3
Gelidiella acerosa	1
Amphiroa fragilissima	1

b. Incline (>1 m)

Lyngbya sp. 3	1
Enteromorpha sp.	1
Padina japonica	1
Giffordia breviarticulata	1
Acanthophora spicifera	1

Table 8. Continued.

C. Supratidepool

Calothrix sp.	1
Cyanophyta (unid.)	1
Enteromorpha sp.	1

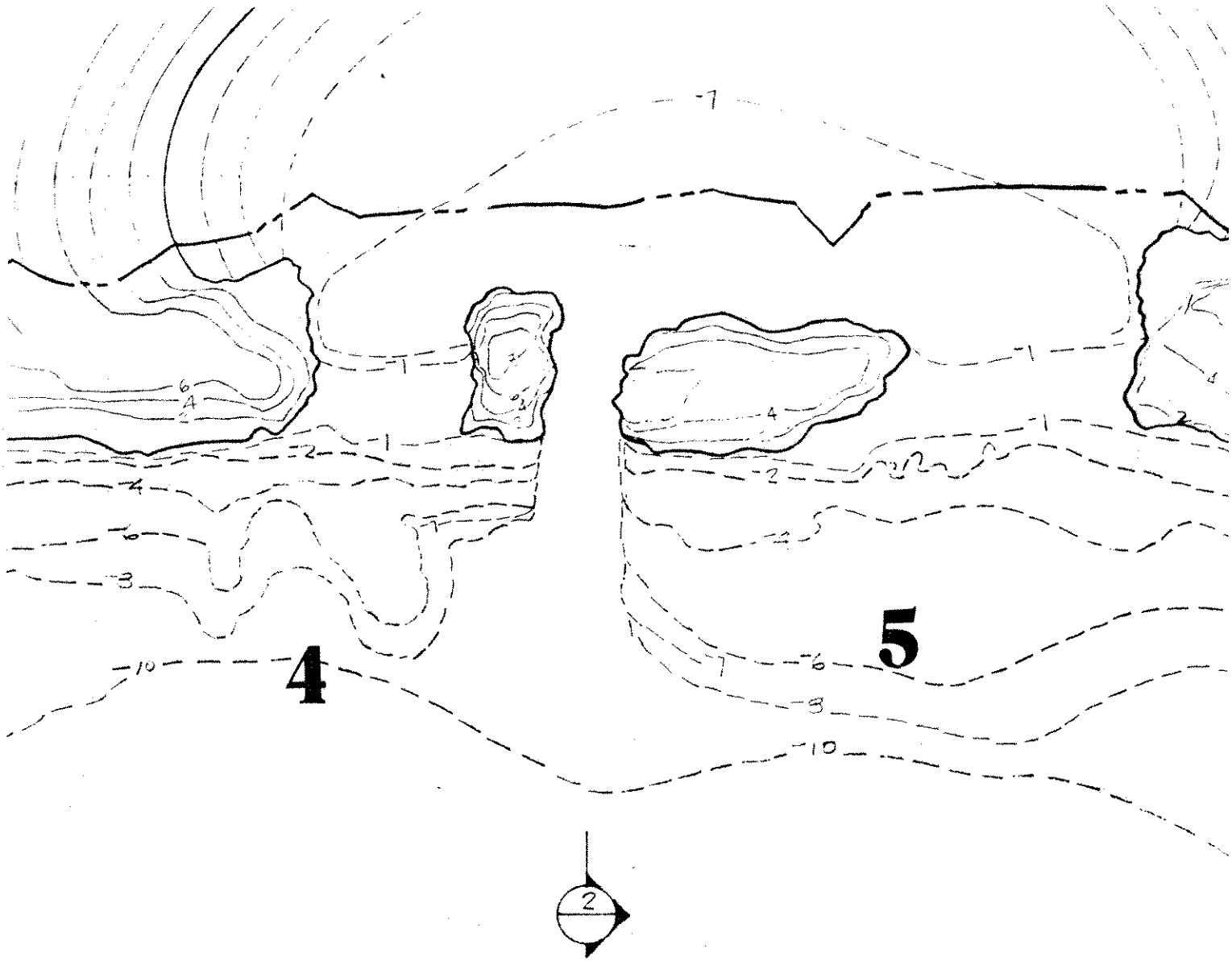

Figure 4. Map of the nearshore bathymetry in the area affronting Lagoon B at West Beach (see also Figure 1) showing the proposed cuts through the existing limestone bench. Given in this figure are the approximate locations of Stations 4 and 5. (Scale: 1:50)

affronting Lagoon A. These species and their relative abundances are given in Table 8.

Stations 4 and 5 sampled the macrobiota present in the shallow waters adjacent proposed Lagoon B. Station 4 was located about 30 m from shore in about 3 m of water (Figure 4). Substratum at this station is solid limestone overlain by large (3 x 5 m) slabs of limestone affording considerable cover. The 20 x 4 m fish transect censused 689 fishes belonging to 24 species (Table 9, Appendix I). Some of the numerically dominant species are of commercial importance. In this group were the palani (Acanthurus dussumieri), the na'ena'e (Acanthurus olivaceus), the manini (Acanthurus triostegus sandvicensis), the kole (Ctenochaetus strigosus), and the aholehole (Kuhlia sandvicensis). Corals were a minor component of the benthos at this station, having an average coverage of about 1% (Table 9). Numerous drupes (Drupa granulata, D. morum), shrimps (Saron marmoratus) and sea urchins (Echinometra mathaei) were present (Table 9). Twenty-five species of algae were found and are given in Table 10.

Station 5 is about 95 m south of Station 4 and about 40 m from shore in 1.5 to 2 m of water (Figure 4). The substratum at this station is again solid limestone with packets or depressions about 2 to 4 m wide and having a depth to about 2 m. These depressions are spaced from 2 to 10 m apart. The dominant coral species is Porites lobata and coverage averages about 7% for all species (Table 11). The soft coral, Palythoa tuberculosa, is common in the area; motile invertebrates present include the ubiquitous drupes and sea urchins (see Table 11). Seventeen speces of fishes (175) individuals were counted in the 20 x 4 m transect (Table 11; Appendix I). The most common species was the hinalea, Thalassoma nigrofuscus. Macroalgae present at this station are given in Table 12.

Three coral species not present in quadrats at either Stations 4 or 5 were seen in the area affronting proposed Lagoon B. These species are Fungia scutaria, Pavona duerdeni, and Porites evermanni; coral coverage over the entire area qualitatively surveyed is about 4%.

Forty-four species of fish were found in the qualitative survey. Species not quantitatively censused include the blenny (Exallis brevis), sharpback puffer (Canthigaster amboinensis), omilu (Caranx melampygus), 'opule (Anampses cuvieri), wrasse (Macropharyngodon geoffroyi), hinalea (Thalassoma ballieui), filefish (Cantherhines dumerilli), weke (Mulloides flavolineatus), noano kea (Parupeneus chryserydros), kumu (Parupeneus porphyreus) and moa (Ostracion meleagris). Two motile invertebrate species were seen: the cone, Conus lividus, and two ula'papa or slipper lobsters (Paribaccus antarticus). Table 13 outlines the species of macrophytic algae and their abundance as found on the emergent limestone bench in the area of proposed Lagoon B.

Table 9. Summary of the benthic survey conducted at Station 4 at West Beach, Oahu. Results of the 5 m^2 quadrat sampling of the benthic community present (expressed in percent cover) is given in Part A; a 40-point analysis is presented in Part B and counts of invertebrates in Part C. A short summary of the fish census is given in Part D. Water depth 3 m; mean coral coverage 1% (quadrat method).

A. Quadrat Survey

Species	Quadrat Number 1	2	3	4	5
Corals					
Pocillopora meandrina	2.5	1	0.1		
Montipora verrucosa		2			
Cyphastrea ocellina		1			
Porites (Synaraea) convexa			1		
Palythoa tuberculosa	1				
Hard Substratum	89.4	97	95.5	95.9	94.4

B. 40-Point Analysis

Species	Percent of Total
Hard Substratum	100

C. Invertebrate Census (20 x 4 m)

Species	No.
Phyllum Cnidaria	
Palythoa tuberculosa	6 colonies
Phyllum Mollusca	
Drupa granulata	15
D. morum	12
D. brunneolabrum	3
Phylum Arthropoda	
Saron marmoratus	21
Phylum Echinodermata	
Echinometra mathaei	31
E. oblongata	3
Heterocentrotus mammillatus	1
Holothuria atra	1

D. Fish Census (20 x 4 m)

24 species
689 individuals

Table 10. Semiquantitative checklist of macrothalloid algal species present at Station 4, West Beach, Oahu. The relative abundance of algae was estimated using a scale of 1 to 5 where 1 = rare, 3 = common, and 5 = abundant.

Species	Relative Abundance
Oscillatoria sp. 1	1
Oscillatoria sp. 2	1
Lyngbya sp. 1	1
Hormothamnion enteromorphoides	1
Hormothamnion sp.	1
Dictyota friabilis	2
Gelidiopsis sp.	1
Cladymenia pacifica	1
Tolypiocladia glomerata	1
Hypnea sp.	1
Pterocladia capillacea	1
Geliadiales (unid.)	3
Gelidium pusillum	2
Gelidiella sp.	2
Martensia fragilis	1
Amphiroa sp.	1
Porolithon sandvicense	
Lithothamnion sp. 1	2
Lithothamnion sp. 2	4
Sporolithon erythraeum	1
Neogoniolithon frutescens	1
Lithophyllum sp. 1	2
Mesophyllum mesomorphum	1
Peyssonellia rubra	1
Hydrolithon reinboldii	2

Table 11. Summary of the benthic survey conducted at Station 5 at West Beach, Oahu. Results of the 5 m^2 quadrat sampling of the benthic community present (expressed in percent cover) is given in Part A; a 40-point analysis is presented in Part B and counts of invertebrates in Part C. A short summary of the fish census is given in Part D. Water depth 2 m; mean coral coverage 7% (quadrat method).

A. Quadrat Survey

Species	Quadrat Number 1	2	3	4	5
Corals					
Porites lobata	3	8	9	4	7
Pocillopora meandrina	0.5				
Montipora verrilli		1	0.5	1	
Palythoa tuberculosa		1			
Hard Substratum	96.5	90	90.5	95	93

B. 40-Point Analysis

Species	Percent of Total
Corals	
Porites lobata	10
Montipora verrilli	2.5
Hard Substratum	87.5

C. Invertebrate Census (20 x 4 m)

Species	No.
Phyllum Cnidaria	
Palythoa tuberculosa	7 colonies
Phyllum Mollusca	
Drupa granulata	7
D. morum	4
D. brunneolabrum	2
Phylum Arthropoda	
Calcinus sp.	1
Phylum Echinodermata	
Echinometra mathaei	9
E. oblongata	4
Actinopyga mauritiana	1

D. Fish Census (20 x 4 m)

17 species
175 individuals

Table 12. Semiquantitative checklist of macrothalloid algal species present at Station 5, West Beach, Oahu. The relative abundance of algae was estimated using a scale of 1 to 5 where 1 = rare, 3 = common, and 5 = abundant.

Species	Relative Abundance
Hormothamnion sp.	1
Oscillatoria sp. 2	2
Lyngbya majuscula	1
Dictyota bartayresii	2
Dictyosphaeria versluysii	1
Valonia ventricosa	1
Gelidiopsis sp.	1
Geliadiales (unid.)	3
Pterocladia capillacea	1
Gelidiella sp.	1
Gelidium pusillum	3
Antithamnion sp.	1
Asparagopsis taxiformis	1
Jania sp.	3
Ralfsia pangoensis	2
Amansia glomerata	2
Peyssonellia rubra	2
Halymenia formosa	1
Martensia fragilis	1
Cladymenia pacifica	1
Exophyllum wentii	1
Amphiroa fragilissima	1
Amphiroa sp.	2
Lithothamnion sp. 1	1
Lithothamnion sp. 2	4
Neogoniolithon frutescens	1
Porolithon sandvicense	2
Hydrolithon reinboldii	1
Lithophyllum sp. 1	1
Sporolithon erythraeum	1

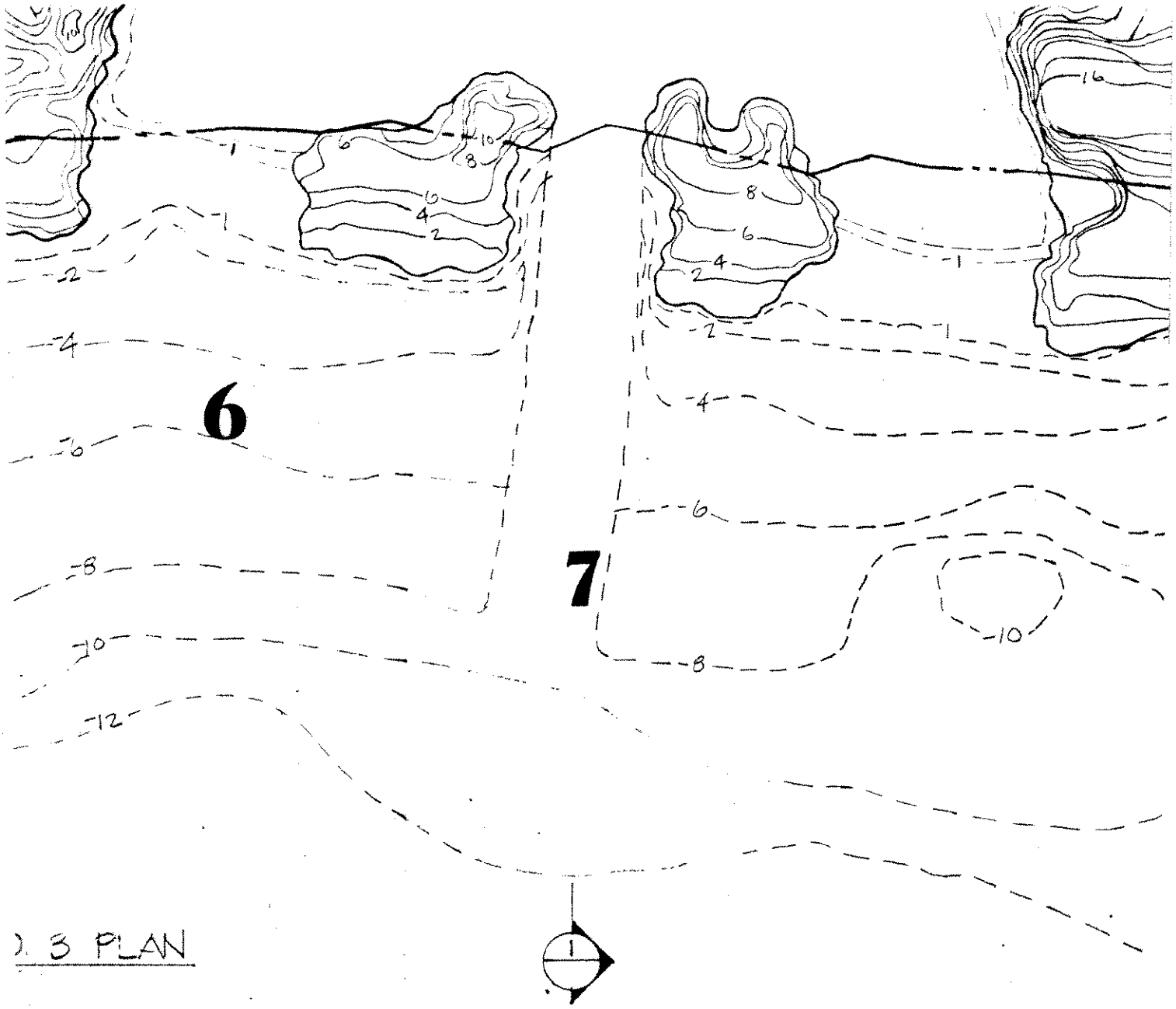

Figure 5. Map of the nearshore bathymetry in the area affronting Lagoon C at West Beach (see also Figure 1) showing the proposed cuts through the existing limestone bench. Given in this figure are the approximate locations of Stations 6 and 7. (Scale: 1:50)

Table 13. Macrothalloid algal species and their relative abundances as found at two intertidal stations on the limestone bench affronting proposed Lagoon B at West Beach, Oahu. Relative abundances were estimated using a scale of 1 to 5 where 1 = rare, 3 = common, and 5 = abundant.

Station B-1

Incline (1/2 - 1 m above MLLW)

Species	Relative Abundance
Ulva fasciata	1
Chaetomorpha antennina	1
Dictyosphaera versluysii	1
Pterocladia capillacea	4
Porolithon onkodes	1

Incline (>1 m)

Calothrix sp.	1
Lyngbya sp. 3	1
Giffordia breviarticulata	1
Ulva fasciata	1
Porolithon onkodes	1

Tidepool (1 m)

Lyngbya sp. 3	1
Giffordia breviarticulata	1
Ulva fasciata	1
Acanthophora spicifera	1

Tidepool (>1 m)

Calothrix sp.	1
Cyanophyta (unid.)	1
Lyngbya sp. 3	1
Ulva fasciata	1
Enteromorpha sp.	2
Padina japonica	1
Acanthophora spicifera	1
Spyridia filamentosa	1
Porolithon onkodes	1
Polysiphonia sp.	1
Hypnea sp.	1
Gelidiella acerosa	1
Laurencia nidifica	1

Table 13 continued.

Station B-2

Bench (1/2 - 1 m above MLLW)

Ulva fasciata	2
Enteromorpha sp.	1
Giffordia breviarticulata	1
Ralfsia pangoensis	1
Acanthophora spicifera	2
Centroceras clavulatum	1
Polysiphonia sp.	1
Laurencia nidifica	1
Porolithon onkodes	1
Pterocladia capillacea	1
Desmia hornemanii	1
Amansia glomerata	1

Tidepool (1 m)

Lyngbya sp. 3	1
Cyanophyta (unid.)	1
Calothrix sp.	1
Enteromorpha sp.	1
Ulva fasciata	1
Padina japonica	1
Giffordia breviarticulata	1
Acanthophora spicifera	1
Gelidiella acerosa	1
Polysiphonia sp.	1

Table 14. Summary of the benthic survey conducted at Station 6 at West Beach, Oahu. Results of the 5 m^2 quadrat sampling of the benthic community present (expressed in percent cover) is given in Part A; a 40-point analysis is presented in Part B and counts of invertebrates in Part C. A short summary of the fish census is given in Part D. Water depth 2 m; mean coral coverage = 3.4% (quadrat method).

A. Quadrat Survey

Species	Quadrat Number 1	2	3	4	5
Corals					
Porites lobata	2	3	4		3
P. evermanni		1.5			
Pocillopora lingulata			0.5		
Pocillopora meandrina	2				
Montipora verrilli		0.5			0.5
Hard Substratum	96	95	95.5	100	94.5

B. 40-Point Analysis

Species	Percent of Total
Corals	
Porites lobata	17.5
Hard Substratum	82.5

C. Invertebrate Census (20 x 4 m)

Species	No.
Phyllum Cnidaria	
Anthelia edmondsoni	4 colonies
Phyllum Mollusca	
Drupa granulata	17
D. morum	3
D. brunneolabrum	1
Conus lividus	1
Conus ebraeus	1
Phylum Echinodermata	
Holothuria atra	1
Echinometra mathaei	23
E. oblongata	2

D. Fish Census (20 x 4 m)

11 species
103 individuals

Stations 6 and 7 quantitatively sampled the extant macrobiota in the area affronting proposed Lagoon C (see Figures 2 and 5). Station 6 was located about 50 m from shore in water about 2 m in depth (Figure 5). The substratum at this station is solid limestone with depressions and small surge channels cut into it. These features are 1 to 2 m wide and usually about 50 cm deep. They are spaced 2 to 4 m apart through the area. In the fish census, 11 species (103 individuals) were noted (Table 14; Appendix I). The dominant species were the hinalea and the surgeonfish. Coral coverage averaged about 3% and Porites lobata was the dominant coral (Table 14). The drupes common to the West Beach area (Drupa granulata, D. morum, and D. brunneolabrum) were censused as were the sea urchins (Echinometra mathaei and E. oblongata) and the sea cucumber, Holothuria atra, in the 20 x 4 m invertebrate survey area. Twenty-six algal species were noted in the area (Table 15).

Station 7 is located about 60 m south of Station 6 and about 60 m from the emergent limestone bench in about 2.5 m of water (Figure 5). This station is characterized by a solid limestone substratum with low (to 50 cm high) flat mounds 1 to 2 m in diameter spaced 2 to 5 m apart. In this habitat, 15 species of fish (103 individuals) were censused (Table 16; Appendix I). The same two dominant species at Station 6 (Thalassoma duperreyi and Acanthurus nigrofuscus) were the most numerous at this station. Porites lobata is the most common coral species and corals (all species covered ~4% of the substratum. Other coral species present include Pocillipora meandrina, Porites compressa, Pocillopora ligulata, Pavona duerdeni and Montipora verrucosa. Invertebrates censused at this station include Drupa granulata, D. morum, Echinometra mathaei and Ophiocoma sp. (Table 16). Twenty-two species of macroalgae were found and are given in Table 17.

In a qualitative survey of the area adjacent Stations 6 and 7, 32 species of fishes were noted. Those not quantitatively censused but present include na'ena'e (Acanthurus olivaceus), koke (Ctenochaetus strigosus), humuhumu'ele'ele (Melichthys niger), butterflyfish (Chaetodon ornatissimus), opelu (Decapturus pinnulatus), wrasse (Labroides phythirophagus), 'olani (Thalassoma purpureum), weke (Mulloides flavolineatus), moa (Ostracion meleagris), mamo (Abdefduf abdominalis), uhu (Scarus perspicillatus), nohu (Scorpaenopsis gibbosa), puffer (Arothron meleagris), and kihikihi (Zanclus canescens). Two additional stony coral species were noted: Montipora verrilli and Cyphaestrea ocellina; the soft coral, Palythoa tuberculosa, was also present. Motile invertebrates present in the area included the sea cucumber, Actinopyga mauritiana (~1/50 m^2), the wana Echinothrix diadema and a yellow nudibranch (Asteronotus cespitosus). On the emergent bench in the area of proposed Lagoon C were found a number of macrothalloid algae; these species and their relative abundances are given in Table 18.

Table 15. Semiquantitative checklist of macrothalloid algal species present at Station 6, West Beach, Oahu. The relative abundance of algae was estimated using a scale of 1 to 5 where 1 = rare, 3 = common, and 5 = abundant.

Species	Relative Abundance
Oscillatoria sp. 2	2
Hormothamnion enteromorphoides	1
Dictoya friabilis	1
D. divaricata	1
Dictyosphaeria versluysii	1
Udotea javensis	1
Caulerpa racemosa	1
Hypnea sp.	1
Desmia hornemanii	1
Laurencia parvipapillata	1
Cladymenia pacifica	1
Hartensia fragilis	1
Hormothamnion sp.	2
Griffithsia sp.	1
Polypiocladia glomerata	2
Amansia glomerata	3
Gelidium pusillum	2
Gelidium sp.	2
Gelidiella acerosa	1
Lithothamnion sp. 1	1
Lithothamnion sp. 2	4
Lithophyllum sp. 1	1
Porolithon sp.	1
Amphiroa fragilissima	1
Sporolithon erythraeum	1
Neogoniolithon frutescens	1

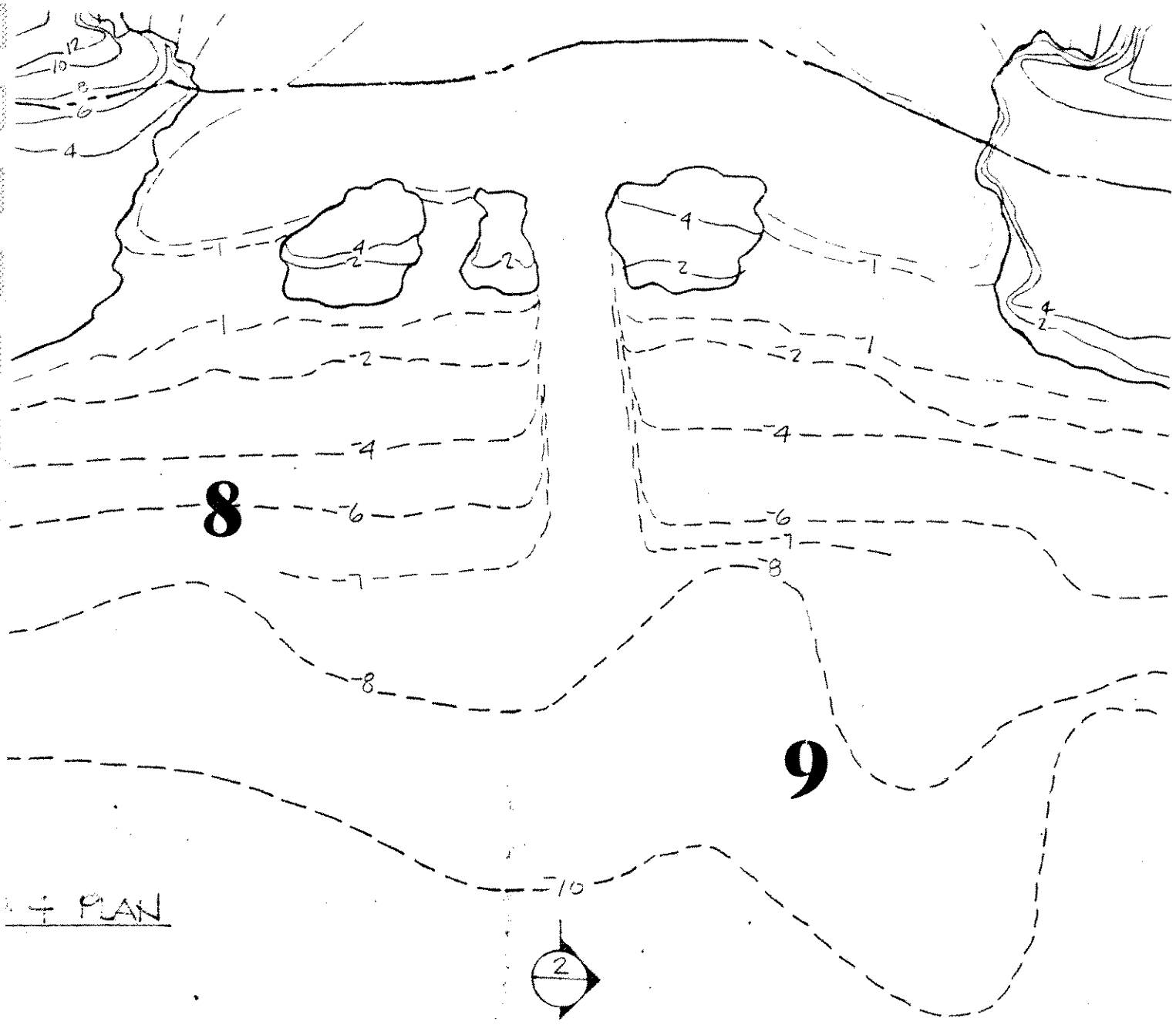

Figure 6. Map of the nearshore bathymetry in the area affronting Lagoon D at West Beach (see also Figure 1) showing the proposed cuts through the existing limestone bench. Given in this figure are the approximate locations of Stations 8 and 9. (Scale 1:50)

Table 16. Summary of the benthic survey conducted at Station 7 at West Beach, Oahu. Results of the 5 m^2 quadrat sampling of the benthic community present (expressed in percent cover) is given in Part A; a 40-point analysis is presented in Part B and counts of invertebrates in Part C. A short summary of the fish census is given in Part D. Water depth 2.5 m; mean coral coverage 3.7% (quadrat method).

A. Quadrat Survey

Species	Quadrat Number 1	2	3	4	5
Corals					
Porites lobata	0.5	3	2	1	2
P. compressa	1				
Pocillopora ligulata			0.5		
Pocillopora meandrina	1				
Pavona duerdeni			0.5		
Montipora verrucosa	1.5	1	0.5		2
Hard Substratum	96	96	96.5	99	94

B. 40-Point Analysis

Species	Percent of Total
Corals	
Porites lobata	5
Montipora verrucosa	5
Pavona duerdeni	2.5
Hard Substratum	87.5

C. Invertebrate Census (20 x 4 m)

Species	No.
Phyllum Mollusca	
Drupa granulata	26
D. morum	3
Phylum Echinodermata	
Echinometra mathaei	1
Ophiocoma sp.	7

D. Fish Census (20 x 4 m)

15 species
103 individuals

Table 17. Semiquantitative checklist of macrothalloid algae encountered at Station 7, West Beach, Oahu. The relative abundance of algae was estimated using a scale of 1 to 5 where 1 = rare, 3 = common, and 5 = abundant.

Species	Relative Abundance
Oscillatoria sp. 2	1
Hormothamnion sp.	2
Dictyota friabilis	1
D. bartayresii	1
Ralfsia pangoensis	1
Dictyosphaeria cavernosa	1
Udotea javensis	1
Gelidiella sp.	3
Gelidiella acerosa	1
Gelidium pusillum	1
Gelidiopsis sp.	1
Amansia glomerata	3
Hartensia fragilis	1
Cladymenia pacifica	1
Desmia hornemanii	1
Tolypiocladia glomerata	2
Porolithon sandvicense	1
Lithothamnion sp. 1	1
Lithothamnion sp. 2	4
Lithophyllum sp. 1	1
Sporolithon erythraeum	1
Neogoniolithon frutescens	1

Table 18. Macrothalloid algal species and their relative abundances as found at two intertidal stations on the limestone bench affronting proposed Lagoon C at West Beach, Oahu. Relative abundances were estimated using a scale of 1 to 5 where 1 = rare, 3 = common, and 5 = abundant.

Station C-1

Bench (1/2 - 1 m above MLLW)

Species	Relative Abundance
Dictyosphaeria versluysii	1
Cladophoropsis luxurians	1
Ulva fasciata	1
Sargassum echinocarpum	1
Acanthophora spicifera	1
Polysiphonia sp.	1
Gelidiella acerosa	1
Gelidiopsis intricata	1
Laurencia nidifica	1
Porolithon onkodes	1

Incline (>1 m)

Lyngbya sp. 3	1
Ulva fasciata	1
Ralfsia pangoensis	1
Acanthophora spicifera	1

Station C-2

Bench (1 m)

Lyngbya sp. 3	1
Ulva fasciata	1
Dictyosphaeria versluysii	1
Cladophoropsis luxurians	1
Padina japonica	1
Sargassum echinocarpum	1
S. obtusifolium	1
Gelidiella acerosa	1
Gelidiopsis intricata	1
Laurencia nidifica	1
Acanthophora spicifera	1
Amphiroa fragilissima	1
Gracilaria coronopifolia	1
Porolithon onkodes	1

Table 10 continued.

Incline (>1 m)

Lyngbya sp. 3	1
Cyanophyta (unid.)	1
Dictyosphaeria versluysii	1
Ralfsia pangoensis	1
Giffordia breviarticulata	1
Acanthophora spicifera	1
Gelidium pusillum	1
Gelidiella acerosa	1

Table 19. Summary of the benthic survey conducted at Station 8 at West Beach, Oahu. Results of the 5 m^2 quadrat sampling of the benthic community present (expressed in percent cover) is given in Part A; a 40-point analysis is presented in Part B and counts of invertebrates in Part C. A short summary of the fish census is given in Part D. Water depth 2 m; mean coral coverage 2.8% (quadrat method).

A. Quadrat Survey

Species	Quadrat Number 1	2	3	4	5
Corals					
Porites lobata	2	1	4	2	2
Montipora verrucosa	1	1	1		
M. verrilli		0.5			
Hard Substratum	97	97.5	95	98	98

B. 40-Point Analysis

Species	Percent of Total
Corals	
Porites lobata	7.5
Hard Substratum	92.5

C. Invertebrate Census (20 x 4 m)

Species	No.
Phylum Mollusca	
Drupa granulata	7
Phylum Arthropoda	
Stenopus hispidus	2
Phylum Echinodermata	
Echinometra mathaei	1
Echinometra diadema	1
Holothuria atra	1
Actinopyga mauritiana	1

D. Fish Census (20 x 4 m)

10 Species
24 Individuals

Quantitative Stations 8 and 9 sampled the area affronting proposed Lagoon D. Station 8 was on the northern side of the proposed lagoon and Station 9 in the southern section (see Figure 6). Station 8 is about 30 m offshore from the emergent limestone bench in 1.5 to 2 m of water. In this area, the substratum is comprised of a hard limestone pavement with little relief and cover present. Ten species of fish (24 individuals) were censused in the 20 x 4 m quantitative transect (Table 19, Appendix I). The most common fish species were the ubiquitous hinalea and surgeonfish. Coral coverage averaged about 3% (all species) with Porites lobata and Montipora verrucosa being the largest contributors. Motile invertebrates counted in the 20 x 4 m area included the drupe, Drupa granulata, the banded shrimp, Stenopus hispidus, the sea urchins--Echinometra mathaei and Echinothrix diadema and the sea cucumbers Holothuria antra and Actinopyga mauritiana (Table 19). Twenty-three species of macroalgae were found at this station; these are given in Table 20 along with their relative abundances.

Station 9 was located about 200 m south of Station 8, approximately 80 m seaward of the emergent limestone bench in ~3 m of water (Figure 6). The limestone substratum at this station is bisected by small surge channels 0.5 to 1.5 m wide, oriented perpendicular to shore. These channels are 0.5 to 1 m deep and are spaced about 2 m apart. In this habitat, 11 species of fish (86 individuals) were censused; Acanthurus nigrofuscus and Thalassoma duperryi were the most abundant species (Table 21; Appendix I). In the area of the quantitative transect Montipora verrucosa was the most common coral species. Porites lobata and Pocillopora meandrina were other species of relative importance. Coral coverage averaged less than 6% in this area. The soft coral, Palythoa tuberculosa, was present at the survey site and exposed motile macroinvertebrates encountered include the drupe, Drupa granulata, the spiny lobster or 'Ula (Panulirus penicillatus), the brittlestar, Ophiocoma sp., and the sea urchin, Echinometra mathaei. Table 22 provides a list of macroalgal species and their relative abundances at this station.

In the qualitative survey of the area affronting Lagoon D, two coral species not noted in the quantitative survey were seen: Pocillopora ligulata and Porites evermanni. In total, 32 species of fish were encountered in the area; species not noted in the quantitative census include: palani (Acanthurus dussumieri), pualu (Acanthurus mata and A. xanthopterus), kole (Ctenochaeus strigosus), humuhumu-ele'ele (Melichthys niger), humuhumu (Rhinecanthus rectangulus), butterflyfishes (Chaetodon lunula and C. quadrimaculatus), po'o paa (Cirrhites pinnulatus), ala'ihi (Adioryx lactaeoguttatus), 'opule (Anampses cuvieri), lolo (Caris gaimardi), kumu (Parupeneus porphyreus), mano (Abudefduf abdominalis), aweoweo (Priacanthus cruentatus), and puffer (Arothron meleagris). One macroinvertebrate species, the ula'papa (Paribaccus antarcticus) was also seen in this general inventory.

Table 20. Semiquantitative checklist of macrothalloid algae encountered at Station 8, West Beach, Oahu. The relative abundance of algae was estimated using a scale of 1 to 5 where 1 = rare, 3 = common, and 5 = abundant.

Species	Relative Abundance
Hormothamnion enteromophoides	2
Hormothamnion sp.	2
Oscillatoria sp. 2	1
Lyngbya sp. 2	1
Dictyota friabilis	1
Ralfsia pangoensis	1
Cladophora sp.	1
Tolypiocladia glomerata	2
Martensia fragilis	1
Gelidiella acerosa	1
Gelidiella sp.	3
Gelidium pusillum	2
Gelidiopsis sp.	2
Amansia glomerata	1
Desmia hornemanii	1
Jania sp.	2
Lithothamnion sp. 1	2
Lithothamnion sp. 2	3
Lithophyllum sp. 1	2
Lithophyllum sp. 2	1
Porolithon sandvicense	1
Hydrolithon reinboldii	2
Neogoniolithon frutescens	1

Table 21. Summary of the benthic survey conducted at Station 9 at West Beach, Oahu. Results of the 5 m^2 quadrat sampling of the benthic community present (expressed in percent cover) is given in Part A; a 40-point analysis is presented in Part B and counts of invertebrates in Part C. A short summary of the fish census is given in Part D. Water depth 3 m; mean coral coverage ~ 5.6% (quadrat method).

A. Quadrat Survey

Species	Quadrat Number 1	2	3	4	5
Corals					
Porites lobata	2	2		3	2.5
Pocillopora meandrina		1	1.5	1.5	0.5
Montipora verrucosa	3	4	3	0.5	0.5
M. patula			1		
Pavona duerdeni		0.5			
Cyphastrea ocellina	0.5				
Hard Substratum	94.5	92.5	94.5	94	96.5

B. 40-Point Analysis

Species	Percent of Total
Corals	
Porites lobata	15
Pocillopora meandrina	2.5
Montipora verrucosa	5
Hard Substratum	75
Sand	2.5

C. Invertebrate Census (20 x 4 m)

Species	No.
Phylum Cnidaria	
Palythoa tuberculosa	4 colonies
Phylum Mollusca	
Drupa granulata	14
Phylum Arthropoda	
Panulirus penicillatus	1
Phylum Echinodermata	
Ophiocoma sp.	24
Echinometra mathaei	3

D. Fish Census (20 x 4 m)

11 Species
86 Individuals

Table 22. Semiquantitative checklist of macrothalloid algae encountered at Station 9, West Beach, Oahu. The relative abundance of algae was estimated using a scale of 1 to 5 where 1 = rare, 3 = common, and 5 = abundant.

Species	Relative Abundance
Oscillatoria sp. 2	2
Hormothamnion sp.	4
Lyngbya sp. 2	1
Udotea javensis	1
Dictyota friabilis	1
Ralfsia pangoensis	2
Antithamnion sp.	1
Tolypiocladia glomerata	3
Cladymenia pacifica	1
Martensia fragilis	2
Amansia glomerata	3
Gelidium pusillum	2
Gelidiella sp.	2
Lithophyllum sp. 1	2
Mesophyllum mesmorphum	1
Hydrolithon reinboldii	2
Lithothamnion sp. 1	2
Lithothamnion sp. 2	5
Lithophyllum sp. 2	1
Porolithon sandvicense	1
Amphiroa sp.	1

Table 23. Macrothalloid algal species and their relative abundances as found at two intertidal stations on the limestone bench affronting proposed Lagoon D at West Beach, Oahu. Relative abundances were estimated using a scale of 1 to 5 where 1 = rare, 3 = common, and 5 = abundant.

Station D-1

Bench (1 m above MLLW)

Species	Relative Abundance
Lyngbya sp. 3	1
Ulva fasciata	1
Dictyosphaeria versluysii	1
Sargassum echinocarpum	1
Porolithon onkodes	1
Polysiphonia sp.	1
Gelidium pusillum	1
Gelidiopsis intricata	1
Pterocladia capillacea	1

Incline (1/2 - 1 m)

Lyngbya sp. 3	1
Ulva fasciata	1
Sargassum echinocarpum	1
Turbinaria ornata	1
Acanthophora spicifera	1
Porolithon onkodes	1
Pterocladia capillacea	1
Gelidium pusillum	1
Polysiphonia sp.	1
Gelidiella ps.	1
Laurencia nidifica	1

Incline (>1 m)

Lyngbya sp. 3	2
Ulva fasciata	1
Enteromorpha sp.	1
Dictyosphaeria versluysii	1
Turbinaria ornata	1
Giffordia breviarticulata	1
Laurencia nidifica	1
Acanthophora spicifera	1

Tidepools (1 m)

Calothrix sp.	1
Cyanophyta (unid.)	1
Lyngbya sp. 1	1
Acanthophora spicifera	1
Gelidium pusillum	1

Table 23 continued.

Station D-2

Bench (1/2 - 1 m above MLLW)

Dictyosphaeria versluysii	1
Cladophoropsis luxurians	1
Valonia aegagropila	1
Ulva fasciata	1
U. reticulata	1
Sargassum echinocarpum	1
Ralfsia pangoensis	1
Gelidiella acerosa	3
Gelidiopsis intricata	2
Laurencia nidifica	1
Laurencia sp.	1
Acanthophora spicifera	1
Porolithon onkodes	1

Incline (1 - 2 m)

Cyanophyta (unid.)	1
Dictyosphaeria versluysii	1
Ulva fasciata	1
Giffordia breviarticulata	1
Ralfsia pangoensis	1
Acanthophora spicifera	1
Gelidiella acerosa	1
Gelidiopsis intricata	1

Tidepools (1 m)

Oscillatoria sp. 1	1
Dictyosphaeria versluysii	1
Ulva reticulata	1
Enteromorpha sp.	1
Sargassum echinocarpum	1
Sargassum obtusifolium	1
Ralfsia pangoensis	1
Turbinaria ornata	1
Gelidiella acerosa	1
Gelidiopsis intricata	1
Acanthophora spicifera	1
Gelidiella sp.	1
Gelidium pusillum	1

Table 23 continued.

Supratidepool (>2 m)

Calothrix sp.	1
Cyanophyta (unid.)	1
Oscillatoria sp. 1	1
Entermorpha sp.	1
Spyridia filamentosa	1
Acanthophora spicifera	1
Gelidiella acerosa	1

Table 24. Summary of the benthic survey conducted at Station 10 at West Beach, Oahu. Results of the 5 m^2 quadrat sampling of the benthic community present (expressed in percent cover) is given in Part A; a 40-point analysis is presented in Part B and counts of invertebrates in Part C. A short summary of the fish census is given in Part D. Water depth 2.5 m; mean coral coverage ~ 3% (quadrat method).

A. Quadrat Survey

Species	Quadrat Number 1	2	3	4	5
Corals					
Porites lobata		1		2	2.5
Pocillopora meandrina	3		2		
Montipora verrucosa	3				0.5
M. verrilli					0.5
Pavona varians					0.5
Hard Substratum	94	99	100	96	96

B. 40-Point Analysis

Species	Percent of Total
Corals	
Porites lobata	12.5
Pocillopora meandrina	2.5
Hard Substratum	77.5
Sand	7.5

C. Invertebrate Census (20 x 4 m)

Species	No.
Phylum Mollusca	
Drupa granulata	9
D. morum	1
D. brunneolabrum	3
Hexabranchus sanguineus	1
Phylum Arthropoda	
Calcinus sp.	3
Trizopagurus strigatus	1
Phylum Echinodermata	
Actinopyga mauritiana	3

D. Fish Census (20 x 4 m)

12 Species
70 Individuals

Macroalgae were surveyed at two stations on the limestone bench affronting Lagoon D. The results of this inventory are given in Table 23. These data show a number of species as being present, however, none are dominating the substratum.

Station 10 (Figure 2) is located about 190 m south of Station 9, approximately 20 m seaward of the emergent limestone bench in about 2.5 m of water. The solid limestone substratum has scattered holes and pockets 0.5 to 3 m in diameter spaced 1 to 4 m apart. The results of the quantitative survey at Station 10 are summarized in Table 24. Twelve species of fish (70 individuals) were censused at this station in the 20 x 4 m transect area (Table 24; Appendix I). The most abundant of these fishes were the surgeonfish, the hinalea, and the damselfish. Five coral species were present with an average coverage of ~3%; Porites lobata and Pocillopora meandrina were the two species with highest coverage. Seven motile macroinvertebrates were met with in the 20 x 4 m census area, the most common being drupes, hermit crabs (Calcinus sp.) and sea cucumbers. Thirty-six species of macroalgae were found at this station and are given with their relative abundances in Table 25. General qualitative observations on the macrobiota in the vicinity of Station 10 were not carried out due to poor water clarity at the time of sampling.

Activity and/or use patterns were observed in the West Beach study area on three days. The data are given in Table 26. Only one individual (a pole fisherman) interviewed on 15 June would discuss his catch, which was comprised of two small (~0.5 kg) Calotomus spinidens. His fishing effort occupied five hours using two shore-casting poles. Pole fishermen in the area appeared to have an average fishing trip of about six hours in duration. The akule netfisherman on 18 June (Table 26) apparently made two sets seaward of the study area in the course of the afternoon and appeared to catch few fish. These data (collected during weekdays) show a reasonable level of use of the area; weekend fishing pressure would probably be considerably greater.

Discussion

In general, a small change in topography and structural relief (on a scale of a few meters) will result in a localized change in the composition of marine benthic communities. However, if one considers communities on a larger scale (hundreds of meters to kilometers), one sees a mosaic of habitat patches which are usually smaller than this but display a degree of regularity and repetitiveness, hence predictability to most any nearshore geographical area. Thus, one can think of benthic communities as being comprised of a series of patches that form a continuum or biotope (e.g., areas of coral, of rubble, of sand, of scoured hard bottom etc.).

Table 25. Semiquantitative checklist of macrothalloid algae encountered at Station 10, West Beach, Oahu. The relative abundance of algae was estimated using a scale of 1 to 5 where 1 = rare, 3 = common, and 5 = abundant.

Species	Relative Abundance
Hormothamnion sp.	2
Oscillatoria sp. 2	2
Lyngbya sp. 1	1
Dictyosphaeria versluysii	2
Udotea javensis	1
Caulerpa racemosa var.	1
Dictyota friabilis	2
D. divaricata	1
Lobophora variegata	1
Ralfsia pangoensis	2
Desmidiaceae (unid.)	2
Gelidiella sp.	2
Hypnea sp.	1
Tolypiocladia glomerata	1
Asparagopsis taxiformis	1
Pterocladia capillacea	1
Gelidium pusillum	2
Geliadiales (unid.)	1
Amansia glomerata	3
Laurencia parvipapillata	1
Jania capillacea	3
Jania sp.	3
Lithothamnion sp. 1	2
Lithothamnion sp. 2	4
Porolithon sandvicense	1
Neogoniolithon frutescens	1
Amphiroa fragilissima	1
Sporolithon erythraeum	2
Hydrolithon reinboldii	1
Lithophyllum kotschyanum	1
Lithophyllum sp.	1
Peyssonellia rubra	1

Table 26. Summary of observed beach use and fishing pressure in the West Beach, Oahu study area on three occasions in June, 1984.

Date	Activity	Number of Individuals Involved
15 June 1984	shorecasting	5 individuals, 11 poles
	gleaning	3 individuals
	walking	3 individuals
18 June 1984	shorecasting	10 individuals, 9 poles
	gleaning	2 individuals
	akule boat and spotter plane	1 boat, 5 individuals (note: just seaward of the study area)
	dive boat	1 boat, 3 individuals
28 June 1984	shorecasting	9 individuals, 11 poles
	throwmetting	2 individuals
	gleaning	4 individuals
	walking	3 individuals

This survey has been carried out entirely in one biotope as recognized by Dienfang and Brock (1980), the shallow low coral coverage biotope. This biotope has been delimited by both physical and biological characters. Physical parameters include water depth and relative exposure to wave action (both are correlated), as well as structure of the substratum (e.g., relative proportions of sand, limestone, etc.) on which the biological components reside. Biological attributes of importance in determining this biotope are (1) relative coral coverage and (2) species dominance in the coral community as well as (3) the abundance and diversity of fishes and invertebrates present.

Substratum is an important parameter governing the structure, diversity and density of organisms in any benthic community. Areas of considerable structural relief (habitat heterogeneity) will harbor a more diverse and greater standing crop of fishes and invertebrates (Brock, 1954; McVey, 1971; Risk, 1972; Environmental Consultants, Inc, 1975; Brock et al., 1979). Sand flats are areas typically of low relief; the opposite extreme is areas comprised of an underlying limestone platform supporting a mature coral community which adds structural relief (shelter) to the habitat. In general, corals, a major structural element of this third dimension, cannot become established in areas of shifting substratum such as sand or loose rubble (Edmondson, 1928; Maragos, 1972) and need areas of hard bottom to survive. Thus the texture of the substratum is an important component in any ecological consideration of benthic communities.

An important modifier of this structural component is water motion; if too severe, water motion may cause scouring or burial (by sand) of benthic communities, leading to a lower diversity. A certain level of water motion is, however, beneficial, and, in fact, required for coral growth (Jokiel, 1978). Water motion carries food and nutrients, as well as disperses larvae for the sessile components of the benthic community. The overwhelming structural force in the shallow low coral coverage biotope at West Beach is water motion.

This nearshore area is exposed to storm waves coming from the south and west (B-K Dynamics, Inc., 1971; CDOC, 1972; Hawaiian Electric Co., 1976). Several of the environmental assessments for the area suggest that the paucity of macroinvertebrates, corals and fish, in shallow waters (<2 m) on hard bottom may be indirectly due to wave activity causing a scouring of the shallow hard substratum.

The storms of January 1980 (partially documented in Coles et al, 1982) and November 1982 (Hurricane Iwa) did extensive damage to benthic (coral) communities on the west coast of Oahu (Kahe Point to West Beach) to depths of 20 m. Waves generated by the 1980 event attained heights of 7 m and did extensive damage to coral communities at depths >20 m along the west coast of Oahu (Dollar, 1982). The breakup of these coral communities probably

Table 27. Summary of the biological findings at the 10 quantitative stations sampling the shallow high energy biotope at West Beach, Oahu in June 1984. Overall means are given at the foot of the table.

	Fish			Corals		Invert.	Algae
Sta. No.	Number Species	Number Indiv.	Diversity (H')	Number Species	Coverage %	Number Species	Number Species
1	14	97	1.68	2	3	8	29
2	14	588	0.70	4	11	5	30
3	12	86	1.77	5	6	9	29
4	24	689	2.32	4	1	9	25
5	17	175	1.74	3	7	8	30
6	11	103	1.15	5	3	9	26
7	15	103	1.59	6	4	4	22
8	10	24	1.97	3	3	6	23
9	11	86	1.39	6	6	5	21
10	12	70	1.59	5	3	7	36
Means	14	202		4	5	6	27

resulted in a decrease in available habitat space, thus affecting fish recruitment to the area, diversity and abundance.

The physiography of the substratum (i.e., relief) is similar for all stations (excluding Station 4) that sampled the shallow high energy biotope. The substratum at Station 4 provided considerable habitat space in the form of large (2 x 5 m) broken slabs of limestone for diurnally cryptic fish species (aholehole and mempachi). The substratum at other stations afforded less cover; related to this were fewer fish species and individuals. Within the shallow biotope of low coral coverage, the physiography of the stations showed a remarkable degree of similarity thus making comparisons possible. Table 27 provides a summary of biological attributes of all 10 stations. Comparison among these stations show no discernible trends insofar as the number of species (fish, corals, invertebrates, and algae), coral coverage, individual fish numbers or diversity are concerned.

At the time of field sampling, a gradient of water clarity existed with low transparency present in the southern part of the study area (Stations 8, 9, and 10) but improving in a northerly (upcoast) direction. This gradient is probably related to construction activities associated with the expansion of the Barber's Point Harbor south of the study site. Shoreline fishing activities appeared to parallel the water clarity gradient; few fishermen were seen south of Station 6. The clustering of fishing activities in the more northerly part of the West Beach study site may be a reflection of the greater fish abundance in the vicinity of Station 4.

Sixty-nine species of fishes were seen in this survey; 46 (66%) of these were encountered in the 20 x 4 m census areas and 23 species were seen in the vicinity of these stations (Appendix I). Other studies conducted in the Barber's Point--Kahe Point area have recorded more than 100 species of fishes occurring in waters <20 m in depth. McCain and Peck (1973) and Coles and McCain (1973) found more than 180 species at Kahe Point and Bienfang and Brock (1980) noted 105 species in a survey of the West Beach area. In the present survey, 12 species of corals were found in the quantitative survey. This represents about 33% of the known shallow Hawaiian coral fauna (Maragos, 1977), however, many species contributed little to the coverage present. Porites lobata, a common Hawaiian coral, accounted for most of the coverage throughout the study area.

Only a few studies have attempted a thorough assessment of the macroinvertebrates resident to any area in the Barber's-Kahe Point region. Problems of identification and/or inappropriate sampling procedures have plagued every attempt at such assessment; only some echinoderms and the corals are relatively easily identified in the field and are large enough to be reasonably sampled by the techniques used in most studies.

The present study has certainly underestimated resident invertebrates due to their size and cryptic habits. The majority

of the invertebrate species on Hawaiian and other coral reefs are small cryptic forms. In coralline habitats, abundance values are frequently on the order of 60,000 to 200,000 organisms per m^2 (Brock and Brock, 1977) and their quantitative assessment requires specialized techniques. Recognizing the fact that these small cryptic and infaunal forms have not been sampled in the present survey (beyond the scope of this study), we can turn our attention to the larger, more diurnally exposed species. Other than corals, these invertebrates are mostly in the phylum Echinodermata and due to their nonconcealing habits, only the sea urchins were probably quantitatively assessed.

Four sea urchin species were encountered in the 20 x 4 m quantitative survey at the ten stations. These species are Echinometra mathaei, E. oblongata, Echinothrix diadema and Heterocentratus mamillatus. Sea urchins ranged in density from 0. to 0.4 individuals per m^2 and over all stations had an average density of 1 per 7 m^2. The most ubiquitous species, Echinometra mathaei, appeared in every station but Station 10.

In terms of species numbers, algae appear to have a relatively even distribution throughout the stations (Table 27). Many macroalgae tend to be very ephemeral (i.e., they recruit rapidly to available substratum, grow, reproduce, and disappear) and they are grazed and browsed upon by a number of fishes and invertebrates. The small quantities of algae coupled with numerous herbivorous fishes (the most abundant trophic group at all stations) seen in the field suggests that the low standing crop of algae is indeed real and is kept down by the grazing activities of fishes. Algae, however, were the major macrobiotic component present on the emergent limestone bench along the West Beach study area. This is probably related to the inability of fish and other large grazers to effectively browse in this habitat. Nothing is known about the stability and/or succession of this community, but it is expected to be quite ephemeral. Thus in a static survey, such as the present one, the algae are a trivial component and not much environmentally pertinent information may be gleaned from them. However, in a study incorporating dynamic aspects of benthic communities (e.g., production and turnover rates), algae are indeed important; such an in-depth approach is beyond the scope of the present concern.

Three stations surveyed by Bienfang and Brock (1980) in November-December 1979 are located in the confines of the present study; three stations in the present effort (Stations 1, 2, and 10) attempted to resample the old Bienfang and Brock stations (Stations 8, 10, and 11). A comparative summary is given in Table 28. In general, there has been a decline in the number of fish species, fish species diversity, coral species and coral coverage at these stations over the 5½ month period. In one case at Station 2 (Table 28), the number of individual fish counted increased between the two samplings. This increase was due to the encounter of a large roaming school of browsing manini (Acanthurus triostegus sandvicensis) estimated at 500 individuals

Table 23. Summary of the biological characteristics present at three stations in the West Beach study site 54 months apart. Data for 1979 from Bienfang and Brock (1980).

	Station No. 1		Station No. 2		Station No. 10	
	1979	1984	1979	1984	1979	1984
No. Fish Species	26	14	25	14	17	12
No. Fish Individuals	127	97	163	588	112	70
Fish Species Diversity (H')	2.57	1.68	1.93	0.70	1.94	1.59
No. Coral Species	5	2	6	4	6	5
Average Coral Coverage	15	3	17	11	6	3
No. Invert. Species	4	8	4	5	5	7
No. Sea Urchins	29	10	11	10	136	0

crossing the transect line at the time of sampling. Such an encounter may be considered a rare (chance) event. The number of invertebrate species present in the 20 x 4 m transect area increased at every station between the two sampling periods. Reasons for this increase could be many, but perhaps the most logical is that the increase is meaningless because the invertebrate sampling technique is probably only quantitative for sea urchins and they all decreased in abundance over this period.

The apparent decline in these benthic communities may be due to natural events or to man's activities in the area. The two large storms in 1980 and 1982 must have affected these shallow water communities; as yet, they may not have recovered. Dollar (1982) notes that infrequent intense storms (such as the 1980 and 1982 events) may effectively wipe out existing coral zonation patterns and return a reef area to an early low diversity successional stage which may take years to recover. Alternatively, the dredging activities connected with the development of the Barber's Point Harbor and concurrent increased turbidity could induce some of the observed changes. Many dead but intact Pocillipora meandrina coral heads were seen at Station 10 in June 1984. Also present at this station was a layer of fine particulate material up to 1 cm in thickness carpeting much of the substratum. Another explanation for the observed decrease in fish may be related to increased use and fishing pressure in the vicinity of the study site. Bienfang and Brock (1980) reported 24 individuals using the West Beach shoreline over six days of observation. Their study site was larger by about one-third but encompassed the present area of concern. During three days of observation in the present survey, 41 individuals were seen using the shoreline resource (Table 26). All of the above explanations have probably contributed to the observed decline in nearshore communities at West Beach. At present, there is no way to delineate the importance of each impact separately (whether it be storm surf, turbidity due to silt generation, or fishing pressure) to the communities in question.

The resurvey data suggest that natural events and human activities in the vicinity of West Beach have led to an apparent degradation of nearshore benthic communities. Bienfang and Brock (1980) found little evidence of negative impacts other than those associated with increased beach use and fishing pressure. These authors found that the resident fish communities were diverse and species of noncommercial importance were well represented. They presented information suggesting that the abundance of commercially important fish species had declined in the area, presumably due to fishing pressure. In 1979-80, the coral communities were considered to be well developed and diverse (Bienfang and Brock, 1980). Coral coverage and species numbers have since declined in those shallow areas adjacent to the shoreline.

Bienfang and Brock (1980, page 104) noted, "...that future shifts in hinterland use (urbanization, industrialization, etc.) or extensive coastal change (dredging, walling, infilling, etc.)

could have large and far-reaching effects to all these communities (affronting West Beach). Such changes would be most likely to occur if development of coastal areas is done without consideration for and sensitivity to the possible impacts to the nearshore marine communities. In this regard, the continuous generation of silt-laden runoff would be an issue of principal concern..." The data presented herein indicate that some degradation has already occurred in the communities affronting the four proposed lagoon sites prior to any development in the area. Intervening events since the Bienfang and Brock (1980) survey must have been sufficiently great to result in the changes now apparent in these shallow communities. This suggests that these communities are susceptible to storm (high energy) impacts, to dredging (turbidity generation now occurring over a kilometer away), and fishing pressure. The shallow benthic communities at West Beach are relatively diverse. Recent impacts appear to be altering them; however, they may be among some of the best examples of extant leeward bench/shallow subtidal communities on Oahu today. Given the apparent change and degradation that has occurred, the generation of further impacts (e.g., dredging and increased turbidity) may result in further degradation to these communities.

It is recommended that more complete documentation through a resurvey of all the Bienfang and Brock (1980) sites, and the area along the alignment of the deep draft harbor entrance channel, be undertaken (1) to better delineate the magnitude of changes in the benthos at West Beach and (2) to determine which of and to what degree the major impact mechanisms (turbidity, fishing pressure, and/or storm waves) are responsible for those changes.

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Appendix I. List of all fish species censused in the 10 quantitative 20 x 4 m stations as well as those encountered in the near vicinity of these stations at West Beach, Oahu. Numbers in body of the table represent counts of individual fishes. Totals and a diversity index (H') are given at the end of the appendix.

SPECIES	STATION NUMBER									
	1	2	3	4	5	6	7	8	9	10
ACANTHURIDAE										
Acanthurus achilles				25	1					
A. dussumieri				108			1			
A. leucopareius				21						
A. mata										
A. nigrofuscus	47	25	25	33	60	21	36	6	39	36
A. olivaceus				52						
A. triostegus sandvicensis		500		51		1		2	1	1
A. xanthopterus										
Ctenochaetus strigosus	10	27	15	53	4					
Naso literatus		2	2		2					1
Zebrasoma flavescens				5						
Z. veliferum										
AULOSTOMIDAE										
Aulostomus chinensis										1
BALISTIDAE										
Melichthys niger		1	1							
Rhinecanthus rectangulus										
BLENNIDAE										
Cirripectus lineopunctatus					1		1			
Exallias brevis										
Runula goslinei					2					1
CANTHIGASTERIDAE										
Canthigaster ambionensis			2							
C. jactator	1	2	4	1	1	1	1		3	1
CHAETODONTIDAE										
Chaetodon fremblii	1		1							
C. lunula				1		1				1
C. ephippium				1						
C. miliaris				1		2	1			
C. multicinctus	1			1			1	1	3	1
C. ornatissimus		2			2					
C. quadrimaculatus						1	1			

Appendix I continued.

SPECIES	STATION NUMBER									
	1	2	3	4	5	6	7	8	9	10
C. citrinellcus										
C. unimaculatus										
Forcipiger flavissimus										
CIRRHITIDAE										
Cirrhitus pinnulatus			1	1			1			
Paracirrhites arcatus									1	
KYPHOSIDAE										
Kyphosus cinerascens	2									
CARANGIDAE										
Decapturus pinnulatus										
Caranx melampygus										
HOLOCENTRIDAE										
Adioryx lacteoguttatus										
Myripristis amaneus				~70						
KUHLIIDAE										
Kuhlia sandvicensis				~200						
LABRIDAE										
Anampses cuvieri										
Coris gaimardi				4	3					
Gomphosus varius	2				3		1			
Labroides phthirophagus		1		1						
Macropharyngodon geoffroyi										
Stethojulus balteatus	3					1	2			
Thalassoma ballieui										
T. duperryi	21	17	28	31	68	68	43	7	32	13
T. umbrostigma	3		1		10		1	1		
MONACANTHIDAE										
Cantherhines dumerili										
Pervagor spilosona		1	1	3	1		2	1	1	

Appendix I continued.

SPECIES	STATION NUMBER 1	2	3	4	5	6	7	8	9	10
MULLIDAE										
Mulloidichthys vanicolensis				23						
Mulloides flavolineatus										
Parupeneus bifasciatus				1						
P. chryserydros		1								
P. multifasciatus	1	4	6	3	1			1	2	
P. porphyreus										
MURAENIDAE										
Gymnothorax eurostus							1			
G. petelli				1						
G. steindachneri							1			
OSTRACIONTIDAE										
Ostracion meleagris	1	1					1			
POMACENTRIDAE										
Abudefduf abdominalis										
A. imparipinnis				1	3	4	3	3	1	8
Stegastes fasciolatus	3	4	5		9	2	8		3	4
PRIACANTHIDAE										
Priacanthus cruentatus										
SCARIDAE										
Calotomus spinidens										
Scarus perspicillatus										
SCORPAENIDAE										
Scorpaenopsis gibbosa										
TETRAODONTIDAE										
Arothron hispidus										
A. meleagris										

Appendix f continued.

SPECIES	STATION NUMBER									
	1	2	3	4	5	6	7	8	9	10
ZANCLIDAE										
Zanclus canescens				2						
TOTAL SPECIES	14	14	12	24	17	11	15	10	11	12
TOTAL INDIVIDUALS	97	588	86	639	175	103	103	24	86	70
DIVERSITY (H')	1.68	0.70	1.77	2.32	1.74	1.15	1.59	1.97	1.39	1.59

APPENDIX II SECTION 2

UNIVERSITY OF HAWAII

JAMES K. K. LOOK LABORATORY OF OCEANOGRAPHIC ENGINEERING

DEPARTMENT OF OCEAN ENGINEERING

TECHNICAL REPORT NO. 59

U HAWAII-LOOK LAB - 85-1

PROPOSED WEST BEACH MARINA

HYDRAULIC MODEL INVESTIGATION

BY

Theodore T. Lee

January 1985

PREPARED UNDER

Contract No. 805-C-02400
West Beach Estates
Honolulu, Hawaii

Citation of Trade Names

Does Not Constitute an Official Endorsement or

Approval of the Use of Such Commercial Products

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FOREWARD AND ACKNOWLEDGEMENTS

The investigation reported herein was authorized on April 1, 1984 by Contract No. 805-C-02400 with West Beach Estates, Honolulu, Hawaii. The project duration was originally from April 1 - November 30, 1984. It was later extended to January 31, 1985 per letter from Mr. William W. Blaisdell to Dr. Theodore T. Lee, Principal Investigator at J.K.K. Look Laboratory of Oceanographic Engineering.

The Federal Commercial Deep Draft Harbor now under construction at Barbers Point is in conformance with House Document No. 93 by the 89th Congress, 1st Session authorized by the Rivers and Harbors Act of 1965.

The U.S. Army District Engineer, Pacific Ocean Division, contracted with the University of Hawaii to do a model study of the harbor in 1967. Although not a part of the authorized harbor, an adjacent marina was included in the model study to investigate the feasibility of small craft facilities in the area.

The present model investigation was to verify and refine design concepts of a proposed marina north of the deep draft harbor. Another important purpose of the model tests was to investigate any adverse effects the West Beach Marina might have on the commercial deep draft harbor. In addition, the effectiveness of modern wave absorbers were investigated.

The problems with which the model study was concerned were of interest to both the U.S. Army Engineer Division, Pacific Ocean, and the State of Hawaii Department of Transportation, Harbors Division. Therefore, representatives from these two organizations were invited to visit Look Laboratory key personnel to discuss with them the design, construction, and operation of the models, and to witness the tests being conducted and data being analyzed. Similarly, the key personnel from West Beach Estates and Alfred A. Yee Division and their guests visited the project several times. Their comments were considered very valuable. These visitors provided information valuable to the design and conduct of the study.

Mr. William R. Godfrey, Alfred A. Yee Division, Leo A. Daly, acting as Technical Representative to West Beach Estates, provided useful consultations during the entire study period. Professor Frans Gerritsen also provided helpful technical consultations. They reviewed the executive summary and

the manuscripts of the final report. Dr. C.L. Bretschneider provided wave information for the project.

This 10-month study was conducted under direct supervision of Dr. Theodore T. Lee and general direction of Dr. Hans-Jurgen Krock, Acting Director of the J.K.K. Look Laboratory of Oceanographic Engineering. Dr. Krock also acted as Associate Investigator and managed the project during the Principal Investigator's absence. He also reviewed the Executive Summary and manuscripts of this final report.

Mr. Robert Q. Palmer was a special consultant working on the construction phase of the project for three months beginning April 1, 1984. His immense contribution is greatly appreciated.

The graduate assistants who were involved in the various phases of the study include the following: Dr. Tarzen Su was responsible for planning, test operations, and data analyses. He made significant contributions to the project. Mr. Gary Mocke participated in model construction, data collection, model tests and data analysis and report preparation. Mr. Eiji Nakazaki prepared the computer programs for both three-dimensional and two-dimensional model data acquisition and analyses. Mr. Jose Andres was involved particularly in the preparation of computer programs and data analysis of the two-dimensional tests. Mr. Ghafar M. Osmanzai assisted in construction, and data collection and analyses. Mr. Massood Siah and Mr. Mark Kirschenbaum assisted in the construction phase of the project. Two undergraduate student helpers (Messrs. Anthony Yeung and Kim Hung Fung) also assisted in this phase.

Technical support was provided by Messrs. Henry F. Ho (Laboratory Technician), Skip Wagner (Electronic Technician) and Roland W.C. Char (Casual Technician). Their valuable assistance is appreciated.

Mr. Robert Q. Palmer provided valuable information on Igloos, a perforated concrete unit designed by the Nippon Tetrapod Company to absorb wave energy. He later also witnessed the tests of the marina with Igloo walls.

I wish also to acknowledge Mrs. Edith Katada and her student assistants (Shari Kakiuchi, Phyllis Kong, and Susan Yanabu) for editing and typing, respectively, the manuscripts of the report.

CONVERSION FACTORS

BRITISH TO METRIC UNITS OF MEASUREMENT

British units of measurement used in this report can be converted to metric units as follows:

	Multiply	By	To Obtain
Length	inches	2.54	centimeters
	feet	0.3048	meters
	miles	1.609344	kilometers
Area	square feet	0.092903	square meters
	square miles	2.58999	square kilometers
	acres	4046.9	square meters
Volume	cubic inches	16.3871	cubic centimeters
Weight	tons (2,000 lbs.)	0.907185	metric tons
Density	pounds per cubic feet	16.0185	kilograms per cubic meter
Velocity	feet per second	0.3048	meters per second

EXECUTIVE SUMMARY

HYDRAULIC MODEL INVESTIGATIONS - WEST BEACH MARINA

I. Description of Test Program

Hydraulic model investigations were conducted at the J.K.K. Look Laboratory of Oceanographic Engineering in Honolulu. Dr. Theodore T. Lee served as Principal Investigator.

Two types of model studies were conducted:

(1) a three dimensional undistorted model, scale 1:100, covering the proposed marina, the adjacent Barbers Point Deep Draft Harbor, Barge Harbor, and the adjoining ocean area.

(2) a two-dimensional undistorted model in the wave flume, scale 1:10, for evaluating alternative marina boundary designs for wave energy absorption.

The objective of the three-dimensional model was to test the wave characteristics of three proposed entrance design concepts for the West Beach Marina. (Figs. 3, 4 and 5)

The objective of the two-dimensional studies in the wave flume was to evaluate the wave reflection coefficients of various structures: a sloping (1:10) spending beach, the vertical walls in the marina's berthing areas, and Igloo wave absorbers. Igloos are a specific type of vertical wave absorbers used in harbor basins. In the case of the West Beach Marina they are being considered for the entrance area as an alternative to the spending beach and as wave absorber in front of vertical walls in the marina basin.

The model test results will be used to select the best design for the marina with (i) minimum wave action within the marina boundaries; (ii) minimum adverse effect on the Barbers Point Deep Draft Harbor; (iii) acceptable navigation conditions for vessels entering the marina or the Deep Draft Harbor.

Seven test plans were studied as follows:

Plan 0 - The Deep Draft Harbor without the marina.

Plan I - Marina entrance perpendicular to the harbor channel with spending beach and without Igloos.

Plan II - Marina with entrance perpendicular to the harbor channel with Igloo wave absorbers.

Plan III - Marina with entrance parallel to the harbor channel with spending beach.

Plan IV - Marina with entrance parallel to the harbor channel with Igloo wave absorbers.

Plan V - Marina with separate entrance and spending beach.
Plan VI - Marina with separate entrance and Igloo wave absorbers.

Test conditions in the three-dimensional model included wave with periods of 6, 8, 12, 14, 15 and 20 seconds and heights of 12, 18 and 36 feet. Wave directions included S45°W, S22.5°W, S67.5°W and W. The wave generators were located approximately at the 150 ft. depth contour. The test water levels were +3.0 ft. for moderate or +4.0 ft. for storm conditions.

In the two-dimensional model wave periods were 6, 12, 15 and 20 seconds with heights ranging from 1 to 4 ft. In this model the effect of a section of floating marina pier on the wave reflection coefficient against a vertical wall was also investigated.

The results of these hydraulic model studies led to the following conclusions.

II. Conclusions

1. Impact on Wave Conditions in the Deep Draft Harbor

It was found during the test of the harbor without the marina (Plan 0) that under certain conditions (e.g. H = 18 ft., T = 6 sec., Direction = S45°W) wave conditions at the berthing site in the Deep Draft Harbor exceeded a wave height of 4 ft. Under these conditions the maximum wave heights in the barge harbor near the entrance exceeded 10 feet.

Both Plans I and II, involving a perpendicular marina entrance, have a moderating influence on the wave heights in the deep draft harbor. Under the conditions listed above, waves at the berthing sites are reduced to below 4 ft; the wave height in the barge harbor is reduced from about 10 ft to about 4 ft.

Plans III and IV, involving a parallel entrance, also have a moderating effect on the deep draft waves but to a somewhat lesser degree than Plans I and II. Plans V and VI, with a separate marina entrance, do not have any significant effect on the wave conditions in the Deep Draft Harbor.

It is concluded that from a wave impact point of view, Plans I and II are superior to the other plans. There is no adverse impact on the wave conditions in the deep draft harbor. On the contrary improvements may be expected. Plans III and IV, although less effective than Plans I and II, have acceptable wave conditions. Plans V and VI appear to be less acceptable with respect to wave conditions than the other plans.

2. Wave Conditions in Marina

For Plans I and II wave heights inside the marina basin are below the maximum allowable limit of 2.0 ft under all tested conditions. For Plans III and IV, wave heights exceed this level in a small number of test conditions. The same is observed for Plans V and VI.

It may be concluded that Plans I and II are better than the other plans.

A comparison between the three-dimensional test results of the conditions without and with Igloos at Stations 4 and 5 shows only minor differences between the two solutions. Based on the two-dimensional testing it is likely that due to the scale effects, under prototype conditions, the spending beach solution will be better because of its lower reflection coefficient, especially for longer period waves.

3. Wave Conditions in Entrance Channel

Wave conditions in the entrance channel are important because they signify the navigation hazards when entering the marina under adverse wave conditions. In Plans I and II the deep draft harbor channel allows relatively safe entering of yachts because its greater depth induces breaking of only very large waves which occur infrequently.

During certain test conditions (18 foot wave height, 14 second period and S22.5°W wave direction) in Plans I and II wave heights in the vicinity of the perpendicular entrance may reach or exceed 10 feet. Such conditions make entering difficult, but these plans provide sufficient maneuvering space to allow safe entering. It should be added that such high conditions are uncommon and that under such circumstances craft should not normally be attempting to enter or exit the marina.

In Plans III and IV, under extreme wave conditions, the wave heights outside the entrance are higher than in Plans I and II. However, the parallel channel allows more room for maneuvering outside the marina entrance.

Inside the entrance the wave conditions are also higher than in Plans I and II, which make maneuvering difficult, although the maneuvering area in the configuration tested is somewhat larger than in Plans I and II.

In Plans V and VI the most serious conditions will occur in the entrance channel offshore, where waves of 18 ft and higher were observed to break. The

model test had no gage installed in that section so that no direct measurements are available. The separate entrance test with Igloos (Plan VI) gave slightly higher wave conditions than the test with a spending beach (Plan V). From a navigational safety point of view the separate entrance alternative appears to be the least desirable.

4. Solutions With and Without Igloos

The differences between the test results for solutions with and without Igloos are small.

In the berthing areas the wave heights seem to be slightly less for the Igloo solution than for vertical walls.

In the entrance, the spending beach should be more effective in view of the results of the two-dimensional tests.

5. Two-Dimensional Test Results

The two-dimensional tests in the wave flume, scale 1:10, provided adequate results regarding the reflection coefficients of alternative marina boundary conditions.

In comparing the Igloo solution with a vertical wall the Igloos appeared effective for a wave period of 6 seconds but less effective for the higher wave periods of 12, 15 and 20 seconds. Measured reflection coefficients for a 6 second wave are:

1:10 Spending Beach	:	0.35
Igloo Wall	:	0.65
Vertical Wall	:	0.78

III. Summary of Model Test Results and Observations

The following table gives the qualitative results of the evaluation of the three major alternative West Beach Marina entrance configurations with respect to wave conditions and navigation.

Factor	Perpendicular Entrance	Parallel Entrance	Separate Entrance
Harbor Wave Conditions	Best	Good	Worst
Marina Wave Conditions	Best	Good	Acceptable
Navigation	Best	Good	Worst

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STUDY OF PROPOSED WEST BEACH MARINA

Hydraulic Model Investigations
(Including Barbers Point Deep Draft Harbor, Barge Harbor and Wave Absorbers)

PART I: INTRODUCTION

DESCRIPTION OF WEST BEACH MARINA, BARBERS POINT DEEP DRAFT HARBOR, AND BARGE HARBOR

I.1 The study is concerned primarily with the proposed West Beach Marina, which is adjacent to the Barbers Point Deep Draft Harbor and Barge Harbor now under construction. A brief description of each of these units follows.

I.1.1 West Beach Marina Development

West Beach Estates plans to develop a resort complex on a large tract of land located in Honŏuliúli in the Ewa District of S.W. Oahu, State of Hawaii. The complex (Fig. 1) consists of residential units, resort hotels, and recreational facilities including a marina (which is the subject of this study) and a series of four lagoons, a park, a golf course and other urban support facilities. The resort development is scheduled to be completed in the next decade to meet growing tourist needs and residential requirements on Oahu.

I.1.2 The West Beach Marina project site is shown in Fig. 2. The proposed marina was designed by the Alfred A. Yee Division of Leo A. Daly with design criteria developed by Dr. Frans Gerritsen, Consultant.

I.1.3 The marina basin will provide space for 350 to 500 boats with a depth of 15 feet below MLLW. In the entrance area, wave absorbers, a spending beach, a short jetty, and a boat ramp are provided for three different design concepts, i.e. (a) a perpendicular entrance to the entrance channel for the Deep Draft Harbor, (b) parallel channel with the Deep Draft Harbor channel, and (c) separate entrance channel from that of the Deep Draft Harbor. Details of the designs are shown in Figures 3, 4, and 5.

II.2 Barbers Point Deep Draft Harbor

I.2.1 The Barbers Point Deep Draft Harbor (Fig. 6) is located on the leeward coast of the Island of Oahu, about sixteen miles from Honolulu,

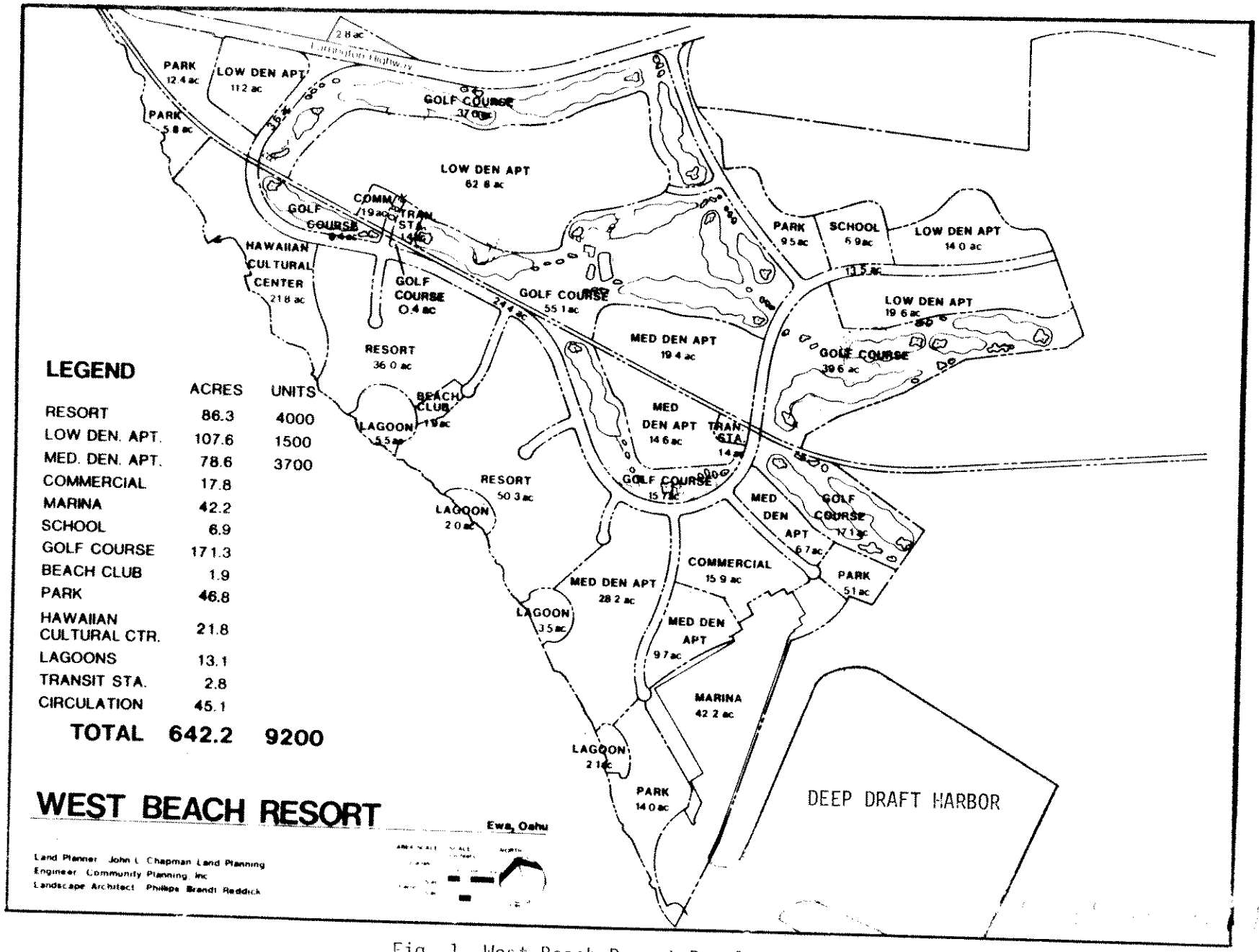


Fig. 1 West Beach Resort Development

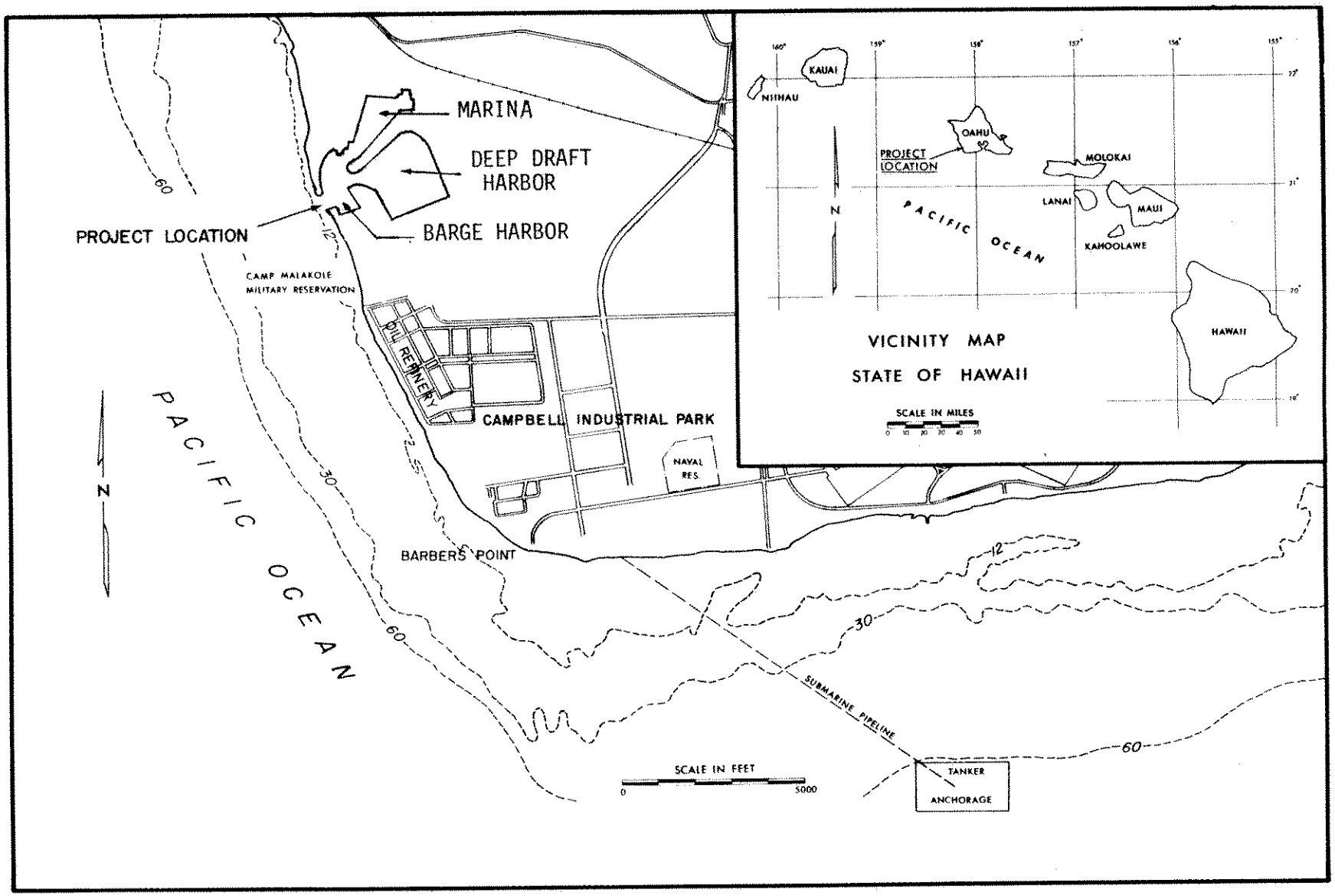


Fig. 2 West Beach Marina (Location Map)

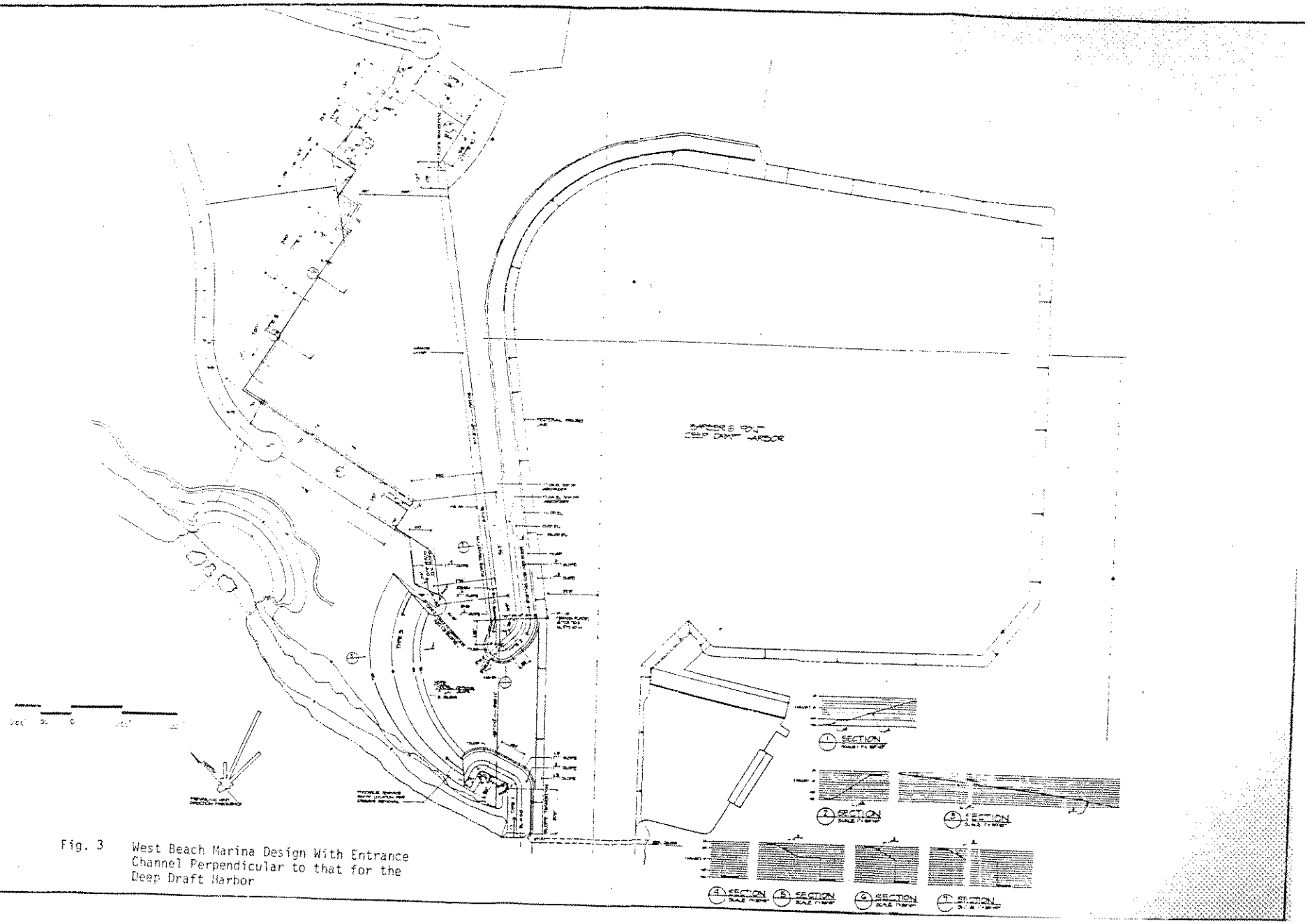


Fig. 3 West Beach Marina Design With Entrance Channel Perpendicular to that for the Deep Draft Harbor

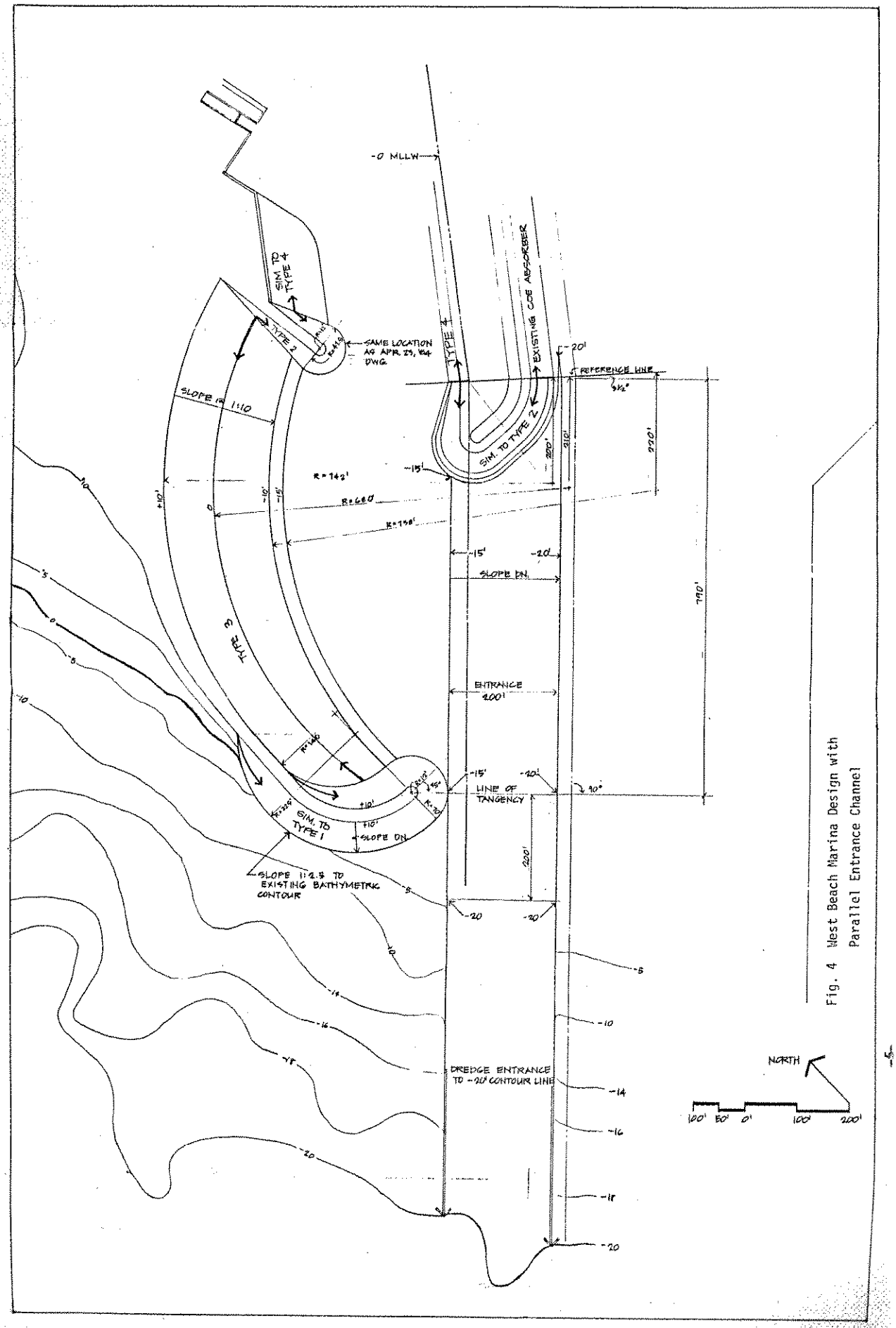


Fig. 4 West Beach Marina Design with Parallel Entrance Channel

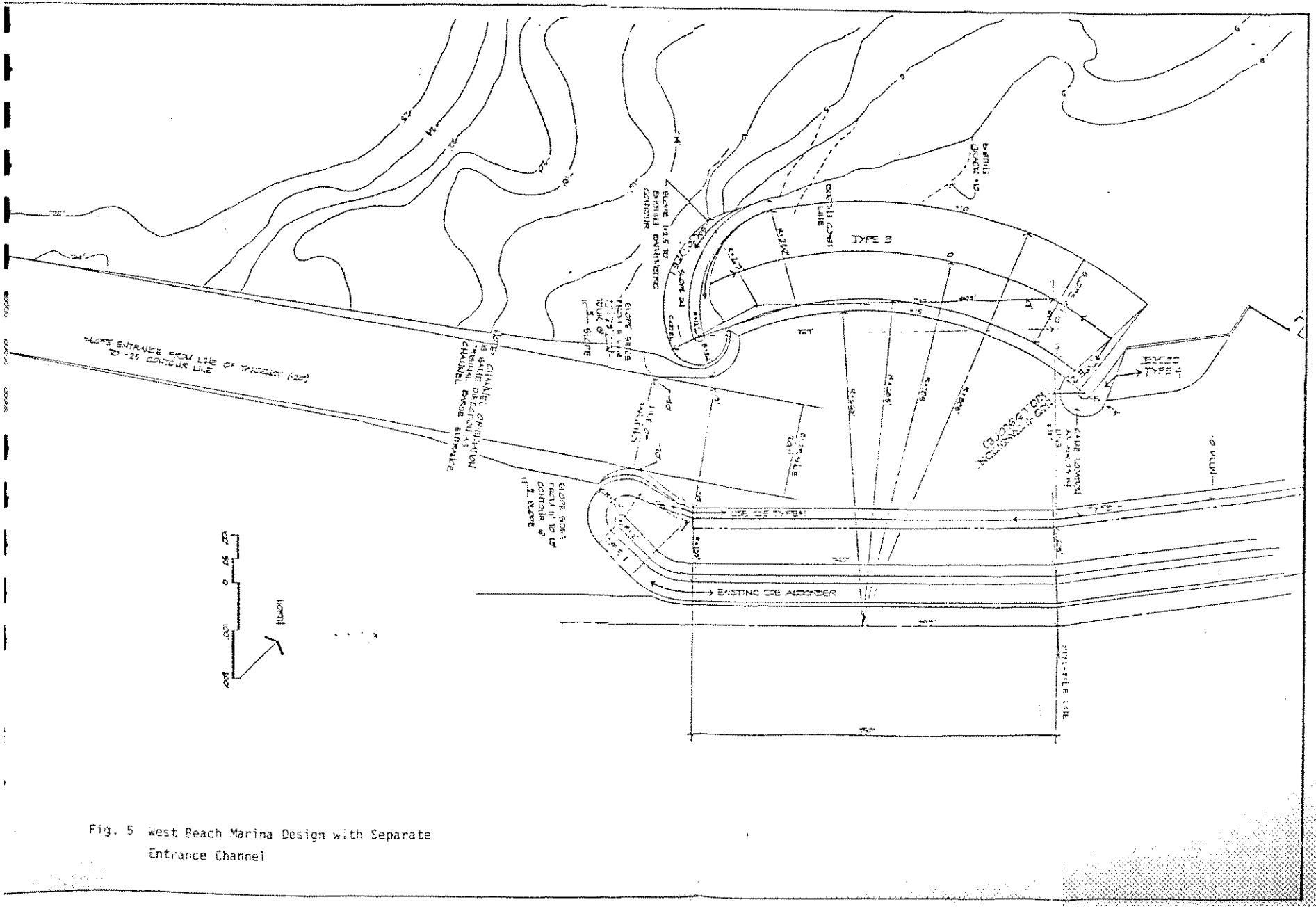


Fig. 5 West Beach Marina Design with Separate Entrance Channel

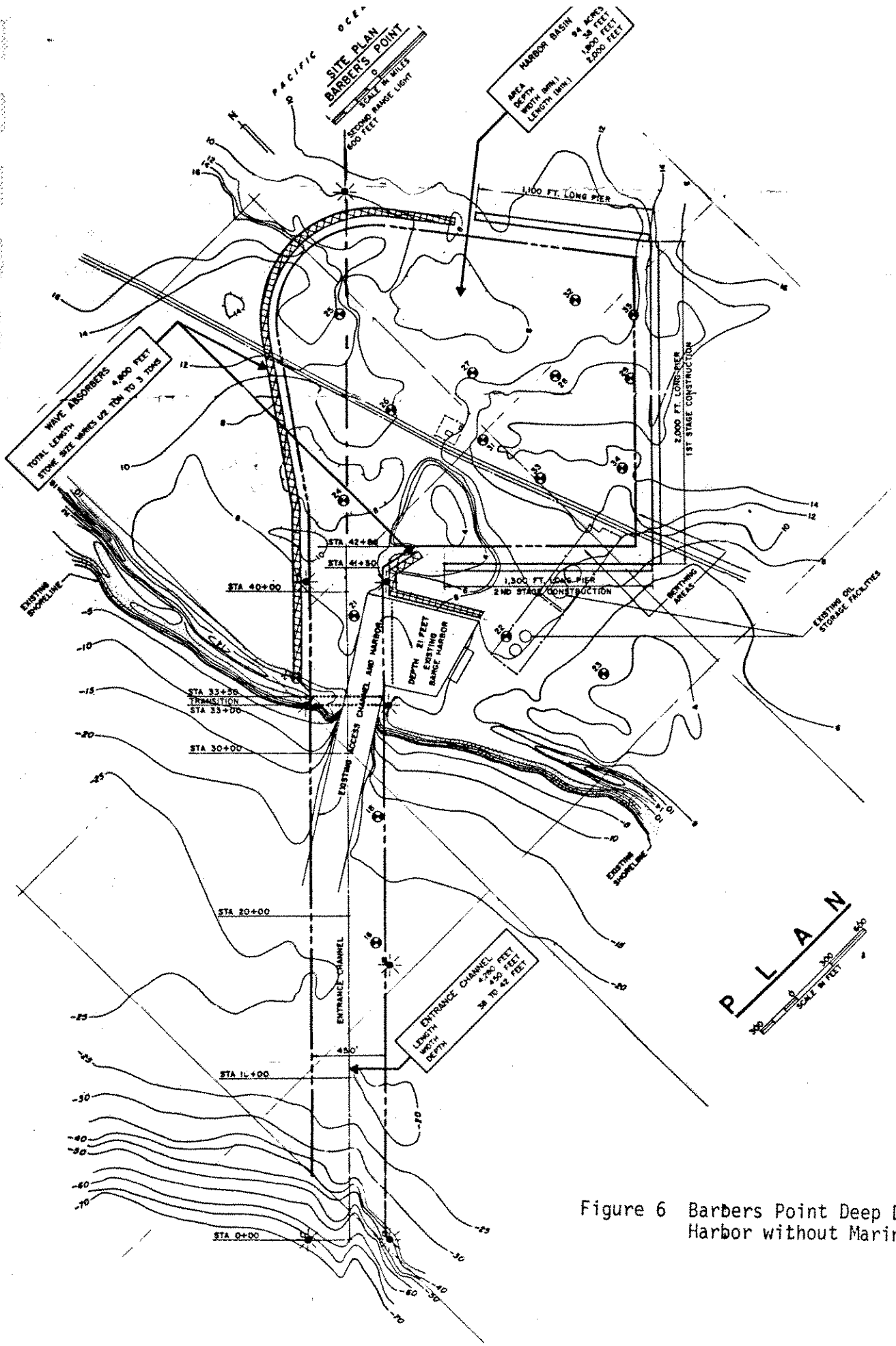


Figure 6 Barbers Point Deep Draft Harbor without Marina

and is a few miles up the coast from the southwest corner of the island (Fig. 2). The Harbor is on the Ewa Plain, an ancient calcareous reef exposed by sea level subsidence. Almost eleven million cubic yards of coral line limestone was excavated to create the commercial harbor. In some areas the material was so hard that heavy blasting was required before even the largest construction equipment could remove it. Thus, the cost of excavation was high, especially offshore. The material has economic value and expected to be used over a period of more than 10 to 17 years.

I.2.2 The harbor consists of a straight offshore channel (3100 ft. long, 450 ft. wide and 42 ft. deep MLLW), an inshore basin 38 ft. deep having an area of 90 acres, and rubble-mound wave absorbers totaling 1320 ft. in length along portions of the inner shoreline of the harbor basin. The harbor berthing areas are designed to accommodate 7 to 11 ships including containers, tankers or bulk carriers and barges.

THE PROBLEM

I.3 The project site is exposed to severe wave conditions. The problem was to design a marina which could provide a shelter for small boats. The marina should not adversely affect the wave conditions in the Barbers Point Deep Draft Harbor.

DESCRIPTION OF THE MODEL STUDY

I.4.1 Goal

The goal of the proposed hydraulic model study is to verify proposed design concepts of the West Beach Marina. The test results will be used to select the best design with

(a) minimum and acceptable wave action within the marina boundaries, and
(b) minimum adverse effect on the Barbers Point Deep Draft Harbor, and
(c) acceptable navigation.

The information should be adequate to determine the most feasible layout considered safe for maneuvering, berthing, and mooring of the pleasure boats in the marina and of commercial ships in the larger deep draft harbor.

I.4.2 Scope of Work

The scope of work includes the following tasks:

(1) To investigate wave characteristics at the marina entrance and within its boundaries for various design concepts. Wave action in the deep draft harbor will also be measured to determine the effect of the marina.

(2) To determine the most desirable plan form for the marina.

(3) To determine the effects of floating piers on the damping of waves.

(4) To evaluate the effects of wave absorbers for the perimeter walls of the marina. Studies in the new three-dimensional wave basin and the two-dimensional wave flume will be involved.

I.4.3 Maximum Wave Height Criteria (Harrenstien and Palmer, 1970)

Based on the U.S. Army Engineer, Honolulu District, the wave height criteria in the Barbers Point Deep Draft Harbor and in the West Beach Marina are desired as follows:

(a) The maximum-tolerable wave height and the desired-maximum limit for the Deep Draft Harbor berthing areas are 4.5 ft. and 2.5 ft. respectively.

(b) The maximum-tolerable wave height and the desired-maximum limit for the Marina berthing areas are 2.0 ft. and 1.0 ft. respectively.

It should be pointed out that the above criteria are developed for normal and moderate waves up to 20 ft tested in the previous model studies. Storm waves of 36 ft were tested in this model study but were not tested previously.

I.4.4 Major Parameters

The major parameters simulated in the 1:100 fixed-bed scale model include:

(a) Marina-harbor plans.

(b) Test wave conditions.

(c) Still-water elevations.

(d) Wave absorbers in the Marina.

(e) Damping effects of floating piers in the Marina.

A brief description of each parameter is given below.

(a) Marina-Harbor Plans

A total of seven plans of the marina-harbor complex was tested.

Plan 0.

This base plan (Fig. 7(a)) consisted of only the Barbers Point Deep Draft Harbor and the Barge Harbor without the West Beach Marina. During the tests the channel entrance to the Marina in the model was blocked off and the separate entrance channel and widened channel for the Marina was filled and blocked. Wave gage locations are shown in Fig. 7. It should be noted that the gage 1 position is changeable with wave directions.

Plan I.

This plan (Fig. 7(b)) consists of both the Deep Draft Harbor and the Marina. The Marina uses the same approach channel as the Deep Draft Harbor. The entrance to the Marina is a perpendicular opening in the side of that approach channel. This plan was tested without Igloos as wave absorbers. Wave gage locations are shown in Fig. 7(b). Gage 1 is changeable with wave directions.

Plan II.

This plan (Fig. 7(b)) is identical to PLAN I except that Igloos were used at locations as shown. It should be pointed out that tests for this configuration were only made for (a) the wave height in the Marina exceeding the desirable maximum wave height criteria according to measurements in Plan I and (b) for the critical wave conditions. Wave gage locations are the same as Plan I (see Fig. 7(b)).

Plan III.

This plan (Fig. 7(c)) consists of a parallel marina entrance channel which is parallel to the entrance channel for the Deep Draft Harbor. The Marina entrance was modified to permit the small boats entering the Marina with minimum or no interference to the large ships navigating into the Deep Draft Harbor. No Igloos were used as wave absorbers for this Plan. Wave gage locations are shown in Fig. 7(c). Again, gage 1 locations are changeable with wave directions.

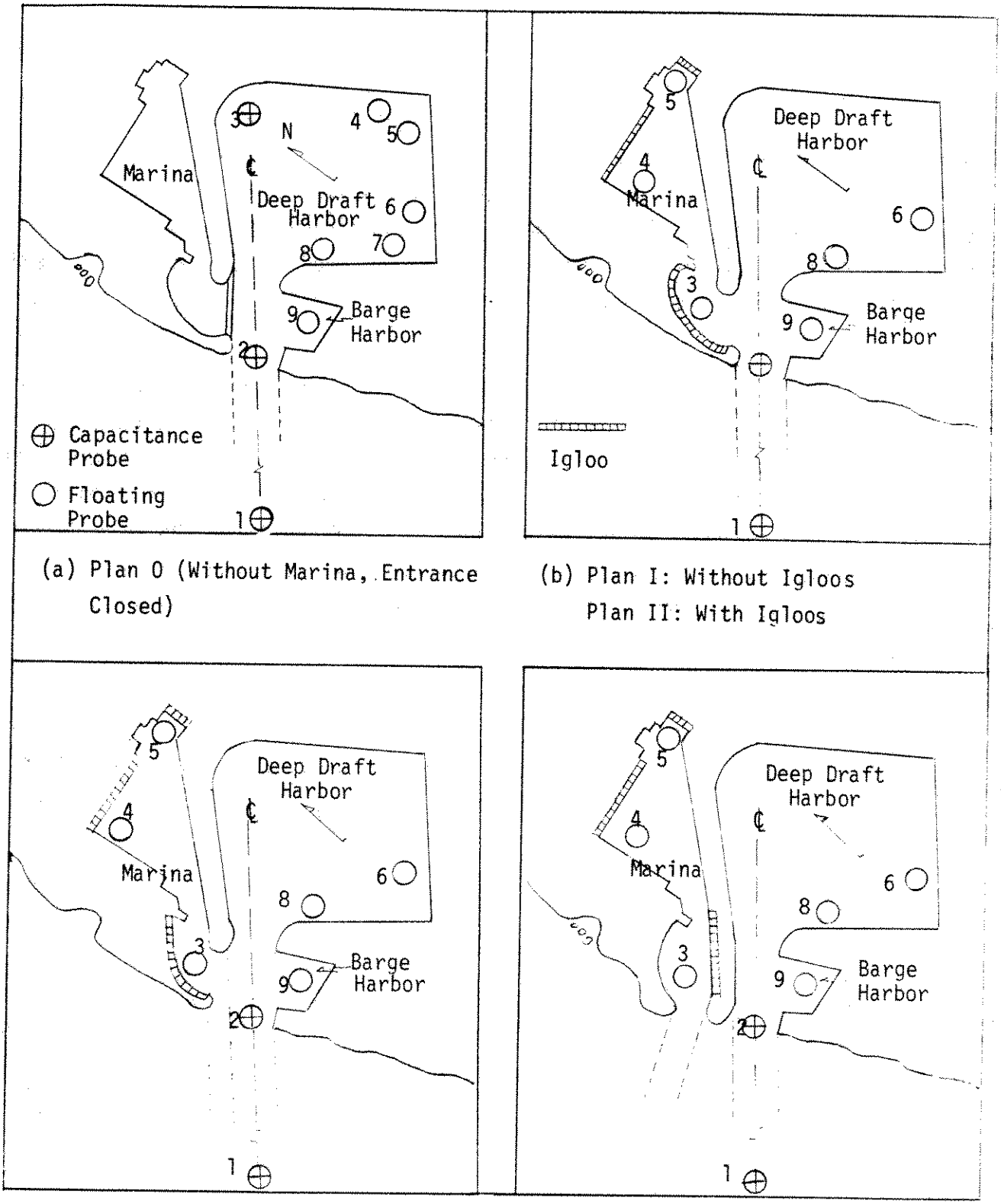


Fig. 7 Test Plans and Gage Locations for the Deep Draft Harbor and Marina Complex

Plan IV.

The layout of this plan was identical as Plan III. However, the Igloos were provided as wave absorbers as shown in Fig. 7(c). Wave gage locations are the same as Plan III. It should be pointed out that tests for this configuration were only made for (a) the wave height in the Marina exceeding the desirable maximum wave height criteria according to measurements in Plan III and (b) for the critical wave conditions.

Plan V.

This plan (Fig. 7(d)) consisted of separate entrances and entrance channels for both the Marina and the Deep Draft Harbor. The Marina channel at the entrance was 20 ft. deep and then sloped down to where it intersected the -25 ft. contour. The Deep Draft entrance channel remained unchanged. This configuration resulted in negligible influence of the Marina on the Deep Draft Harbor operations, particularly on ship maneuvering. There were no Igloos provided for this test plan. Wave gage locations are shown in Fig.7(d) but the position of gage 1 is changeable with wave directions as simulated in the model.

Plan VI.

This plan (Fig. 7(d)) is identical to Plan V except that Igloos were provided at locations as shown in Fig. 7(d). Wave gage locations are the same as Plan V.

(b) Test Wave Conditions

The test wave conditions should provide the necessary information to evaluate the various marina plans. Table 1 shows the test matrix with a total of 154 tests. The tests include normal, moderate and storm wave conditions. For these test wave conditions, the refraction diagrams are available, therefore, the wave generator positions were readily determined. In summary, the test wave conditions included wave periods from 6 to 20 seconds and heights of 12, 18 and 36 ft. respectively and wave directions including S45W, S22.5W, S67.5W and W.

Table 1

Total Test Numbers Based on Test Wave Conditions
And Marina-Harbor Test Plans

Wave Direction From Deep Water	Wave Direction At 100' Depth (Generator)	Wave Height (ft)	Wave Period (sec)	Marina-Harbor Test Plans 0	I	II	III	IV	V	VI
I. STORM CONDITIONS SWL = +4' MLLW:										
W	$\phi = 258^{\circ}$	36	15	✓	✓	✓	✓	✓	✓	✓
S67.5° W	$\phi = 246^{\circ}$	36	15	✓	✓	✓	✓	✓	✓	✓
S45° W	$\phi = 225^{\circ}$	36	15	✓	✓	✓	✓	✓	✓	✓
S22.5° W	$\phi = 216^{\circ}$	36	14	✓	✓	✓	✓	✓	✓	✓
II. NORMAL AND MODERATE CONDITIONS SWL = +3.0 MLLW:										
W	$\phi = 262^{\circ}$	12	12	✓	✓	✓	✓	✓	✓	✓
	$\phi = 258^{\circ}$	12	15	✓	✓	✓	✓	✓	✓	✓
	$\phi = 253^{\circ}$	12	20	✓	✓	✓	✓	✓	✓	✓
	$\phi = 262^{\circ}$	18	12	✓	✓	✓	✓	✓	✓	✓
	$\phi = 258^{\circ}$	18	15	✓	✓	✓	✓	✓	✓	✓
	$\phi = 253^{\circ}$	18	20	✓	✓	✓	✓	✓	✓	✓
S67.5° W	$\phi = 246^{\circ}$	12	15	✓	✓	✓	✓	✓	✓	✓
	$\phi = 246^{\circ}$	18	15	✓	✓	✓	✓	✓	✓	✓
S45° W	$\phi = 225^{\circ}$	12	6	✓	✓	✓	✓	✓	✓	✓
	$\phi = 225^{\circ}$	12	8	✓	✓	✓	✓	✓	✓	✓
	$\phi = 225^{\circ}$	12	12	✓	✓	✓	✓	✓	✓	✓
	$\phi = 225^{\circ}$	12	15	✓	✓	✓	✓	✓	✓	✓
	$\phi = 225^{\circ}$	18	6	✓	✓	✓	✓	✓	✓	✓
	$\phi = 225^{\circ}$	18	8	✓	✓	✓	✓	✓	✓	✓
	$\phi = 225^{\circ}$	18	12	✓	✓	✓	✓	✓	✓	✓
	$\phi = 225^{\circ}$	18	15	✓	✓	✓	✓	✓	✓	✓
S22.5° W	$\phi = 216^{\circ}$	12	14	✓	✓	✓	✓	✓	✓	✓
	$\phi = 216^{\circ}$	18	14	✓	✓	✓	✓	✓	✓	✓

TOTAL TESTS = 154 tests

(c) Still-Water Elevations

For normal and moderate wave conditions the still water elevation was set at +3.0 above MLLW.

For storm wave conditions, the still water elevation was set at +4' above MLLW.

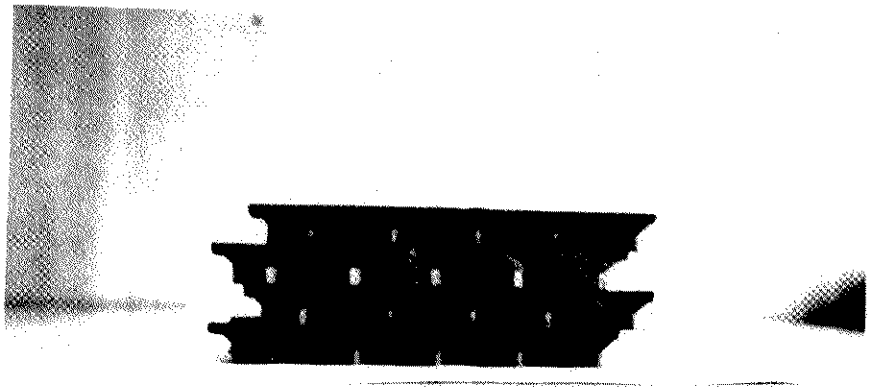
(d) Wave Absorbers in the Marina

The basic design of the Marina consists of a 1:10 slope spending beach and 1:1.5 rubble mound wave absorbers near the entrance and inside of the Marina. The objective is to reduce the wave height in the passage area before the boat is maneuvered into the berthing area.

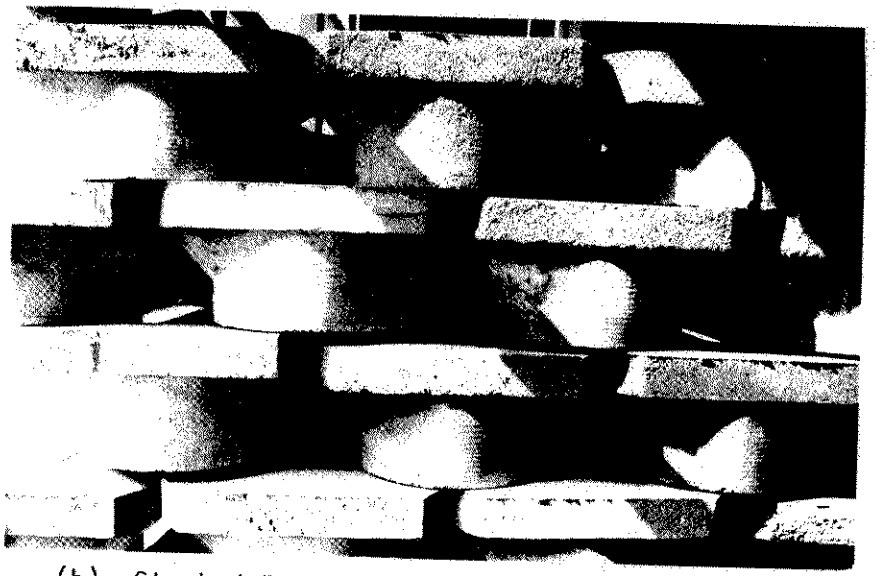
Consideration is given to using Igloo wave absorbers (Fig. 8) in the passage area and critical berthing areas in the Marina (Figs. 7(b), 7(c) and 7(d)). This would save 3.28 acres of space for other purposes and also save 5,200 cu yds of excavation (Fig.9). It was expected that the Igloos would reduce more wave height than conventional sloped wave absorbers for short period waves. It was estimated that a total of 382 Igloos (Type ML34) would be required. It should be pointed out that there will be costs for leasing of Igloo forms, royalty, shipping and handling of forms (round trip between Tokyo and Honolulu, and technical advisory services from Nippon Tetrapod Company). A previous quotation by Tekmarina Inc. (Sonu, 1982) for the Milwaukee Harbor project involving 111 Igloos totaled $70,000.

(e) Damping Effects of Floating Piers in the Marina

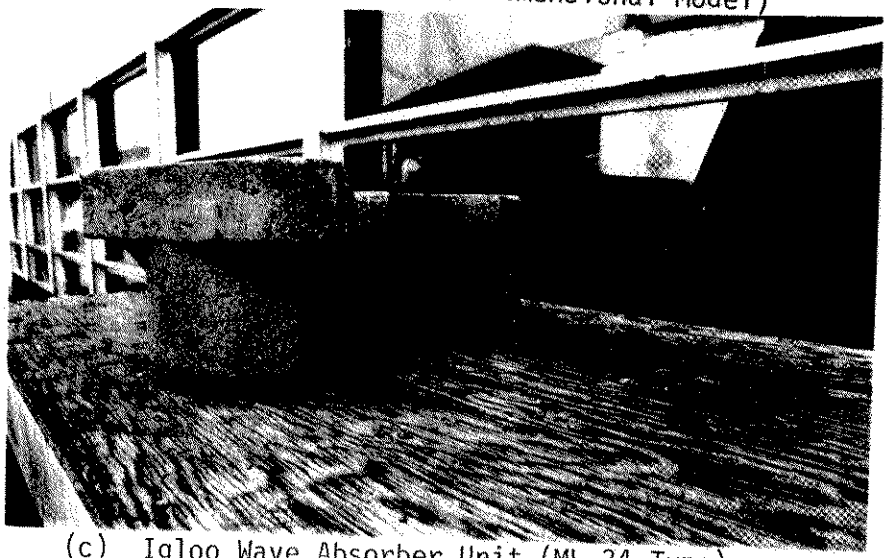
The tests of the damping of waves in the Marina by floating piers was considered not practical at the 1:100 model. Therefore, these damping effect tests were conducted in the 1:10 scale wave absorber tests in the large wave flume.



(a) Stacked Igloo Wave Absorber Units
(1:100 Scale Three-Dimensional Model)



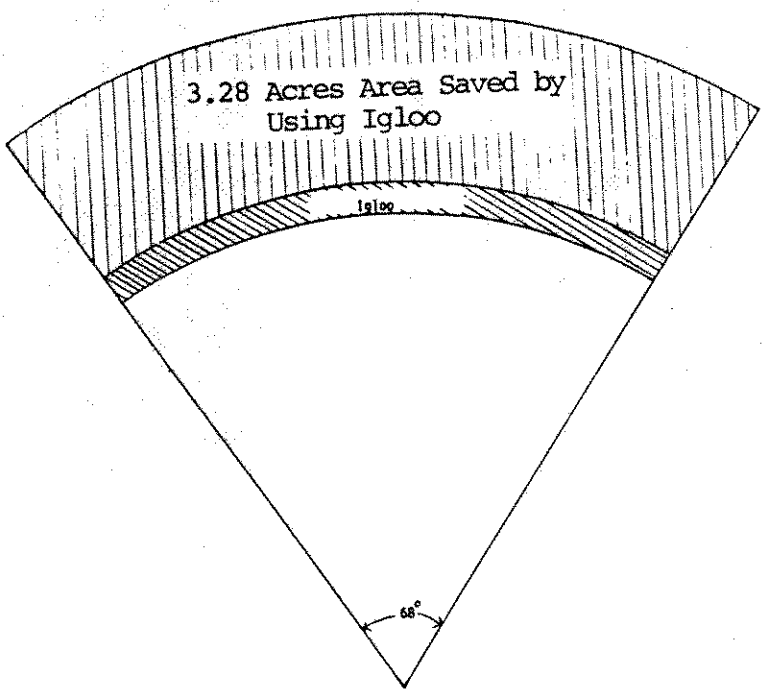
(b) Stacked Igloo Wave Absorber Units
(1:10 Scale Two-Dimensional Model)



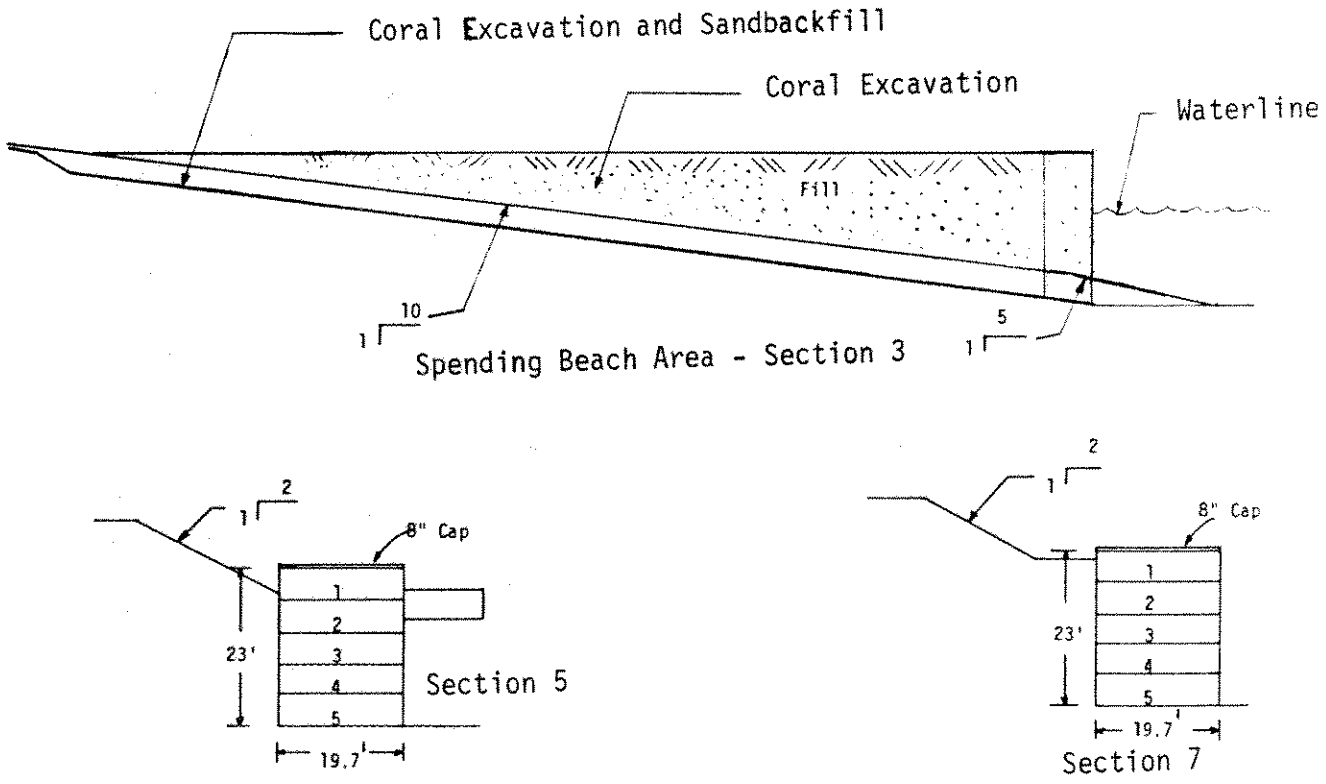
(c) Igloo Wave Absorber Unit (ML-34 Type)
(1:10 Scale Two-Dimensional Model)

(d) Stacked Igloo Wave Absorver Units Installed
for 1:10 Scale Two-Dimensional Tests

Fig. 8 Igloo Wave Absorbers for Both Three-Dimensional and Two-Dimensional Model



(a) Plan View Igloo Wave Absorbor



(b) Section Views

Fig. 9 Igloo Wave Absorbers vs. 1:10 slope spending Beach and other Rubble-mound Wave Absorbers at the Marina Entrance Passage Area

PART II: THE MODEL

DESIGN OF THE MODEL

II.1.1. Three Dimensional Model

The design of the model conformed to accepted engineering practices. References on model technology can be found in the American Society of Civil Engineers paper "Hydraulic Models," by Stevens , et.al. (1942), and in the U.S. Army, Corps of Engineers, Coastal Engineering Research Center special report No. 5, "Coastal Hydraulic Models," by Hudson, et.al. (1979). Basically, the model was designed and operated according to the Froude model law with due consideration of wave attenuation and scale effects in shallow water which will be discussed later.

The selection of the linear scale for the West Beach Marina/Deep Draft Harbor/Barge Harbor model (Fig. 10) was based on several considerations, the most important of which are:

(a) the depth of water required in the model to prevent excessive bottom friction effects, which may not be in similitude with prototype conditions if model-to-prototype scale is too small;

(b) available space in the laboratory model basin to cover the necessary in-shore shelter and offshore deep water areas;

(c) the absolute size of the model waves. Because of the largest storm waves simulated in the model is 36 ft, the depth where the wave is generated is located at -150 ft with a desirable smooth transition to the prototype bottom counter line of -80 ft.;

(d) the efficiency of model operation with wave absorbers along the basin walls to eliminate or reduce undesirable reflections;

(e) the characteristics of wave generation positions and wave measuring gages and recording system;

(f) the scale of 1:100 selected was the same as the scale model for the Barbers Point Deep Draft Harbor previously conducted in 1970, consequently, direct comparisons can be made; and

(g) the cost of model construction.

A geometrically undistorted hydraulic model was desirable to ensure accurate reproduction of wave systems. Based on the above considerations a linear scale of 1:100 model to prototype was selected.

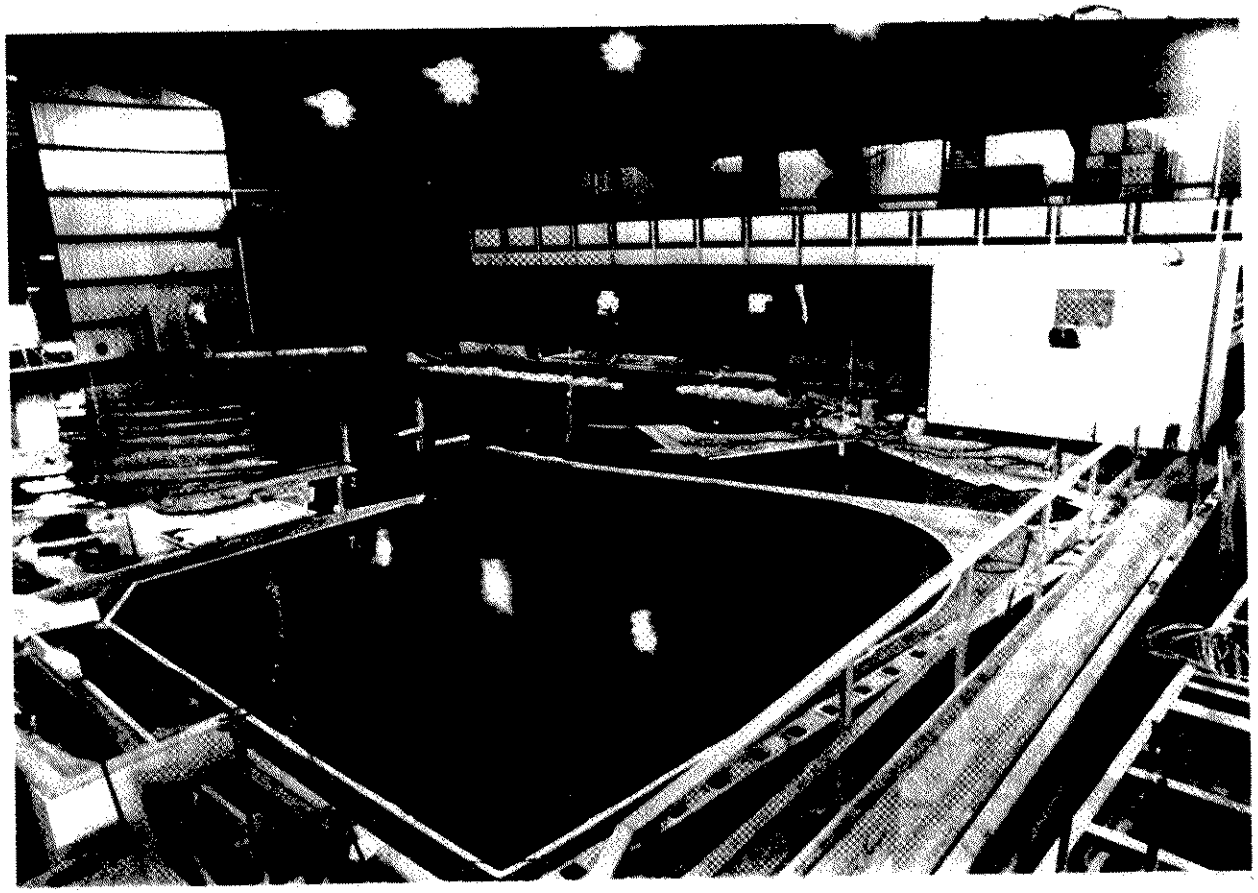


Fig. 10 Barber Point Deep Draft Harbor, Marina, and Barge Harbor Under Wave Action. (Wave Generation in the Three-Dimensional Model from W Direction)

The scale relations used for the design of the subject three-dimensional model are as follows:

Characteristic	Dimensions*	Model: Prototype scale Relation
Length	L	L_r = 1:100
Area	L^2	$A_r = L_r^2$ = 1:10,000
Volume	L^3	$V = L_r^3$ = 1,000,000
Time	T	$T_r = L_r^{1/2}$ = 1:10
Velocity	L/T	$V_r = L_r^{1/2}$ = 1:10

* Dimensions are in terms of length (L) and time (T).

Wave heights were measured at selected locations in the model. For the Barbers Point Deep Draft Harbor (Plan 0) a total of nine gages was used, namely, one near the wave generator measuring incident waves, one at the entrance to the Harbor, six (6) inside the harbor along the perimeter with wave absorbers and at the berthing area and one in the Barge Harbor . For the Marina tests (Plans I, II, III, IV, V, VI) a total of 8 available gages were used, namely, one incident wave gage, one at the Harbor's entrance, two in the berthing area of the Harbor, one at the entrance passage area of the Marina, and two (2) in the berthing area of the Marina. The gage locations for each case are shown in Fig.7. The wave data were recorded and analyzed with a Hewlett Packard computer system which will be described in Part III.

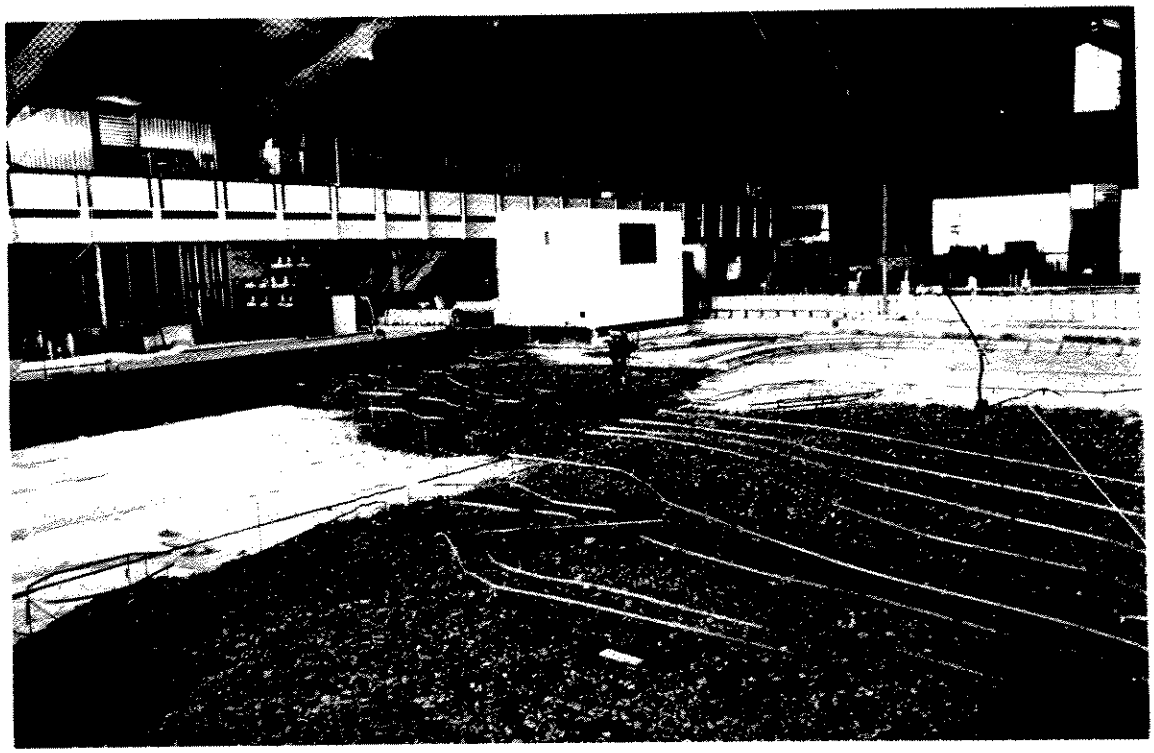
II.1.2. Two Dimensional Model

The two-dimensional model was conducted in the large wave flume with a linear scale of 1:10 so as to reduce scale effect on wave absorbers tests. It was also a geometrically undistorted hydraulic model. The rock size was selected based on the criteria by Harrenstien and Palmer (1970), and Hudson et.al. (1979).

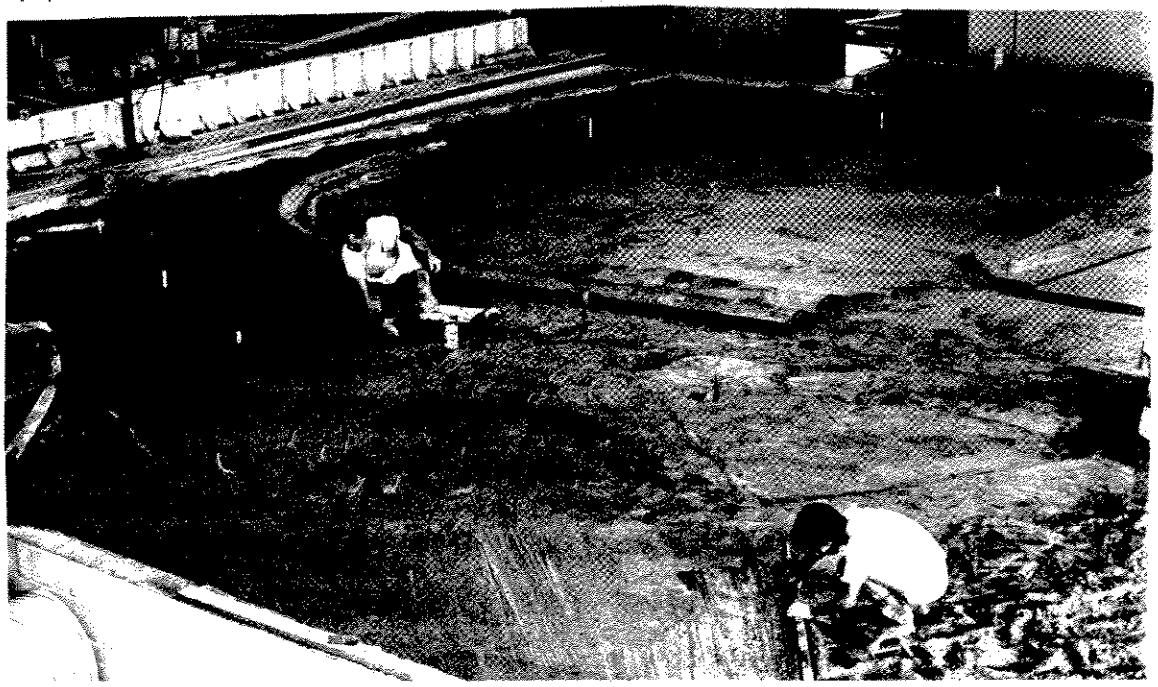
DESCRIPTION OF MODEL AND APPURTENANCES

II.2.1 Three Dimensional Model

The fixed-bed model was molded of fill and concrete (Fig. 11). Construction was accomplished to reproduce the entire Deep Draft Harbor, Marina and Barge Harbor areas, and sufficient offshore area and coastline distance to permit generation of waves from all directions between S67.5°W and W. The contoured offshore area extended to a depth of 80 ft below MLLW, with a sloping transition between 80 and 150 ft depth contour.



(a) Contour Line with Back Fills of the Model



(b) Concrete Work for the Deep Draft Harbor and the Marina

Fig. 11 The Three-Dimensional Model Under Contruction

The total area of the model was about 5500 Sq. ft.

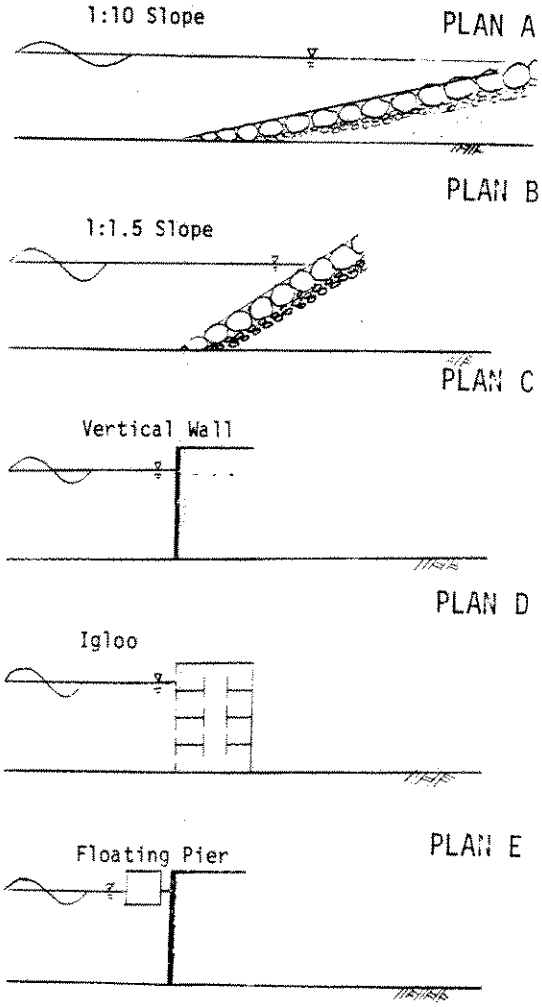
The model was referred to the Mean Low Low Water datum (MLLW) and the local prototype grid system was used for horizontal control. All elevations and tide-levels cited in this report are in ft. referred to MLLW.

The model waves were generated to scale using appropriate sections and orientations of the maximum 60-ft-long wave generator. (See Fig. 12) The tests were carried out with regular waves.

II.2.2. Two-Dimensional Model

The purpose of the two-dimensional model was: (a) to determine the reflection coefficients of five different configurations for West Beach Marina; (b) to determine the effectiveness of Igloo wave absorber structure in reducing reflection coefficients; and (c) to determine the effectiveness of the floating pier in reducing the reflection coefficients. The tests were carried out with regular waves.

Test configurations are:



PLAN A: This plan simulates 1:10 scale spending beach of 1 to 10 slope without Igloo. Rock sizes were 2" to 2-½" aggregates to simulate prototype sizes. The bottom layer in the model was ¼"-3/8" sizes.

PLAN B: This plan simulates 1:10 scale wave absorbers of 1" to 1 1/2" slope with sizes being the same as Plan A.

PLAN C: This plan simulates solid vertical wall. That can be accomplished by reversing solid sides of Igloo toward waves.

PLAN D: This plan simulates the Igloo for Plans A, B and C.

PLAN E: This plan simulates the effect of floating pier in front of the solid wall structure.

Fig. 13 Test Configurations for the Two-Dimensional Model

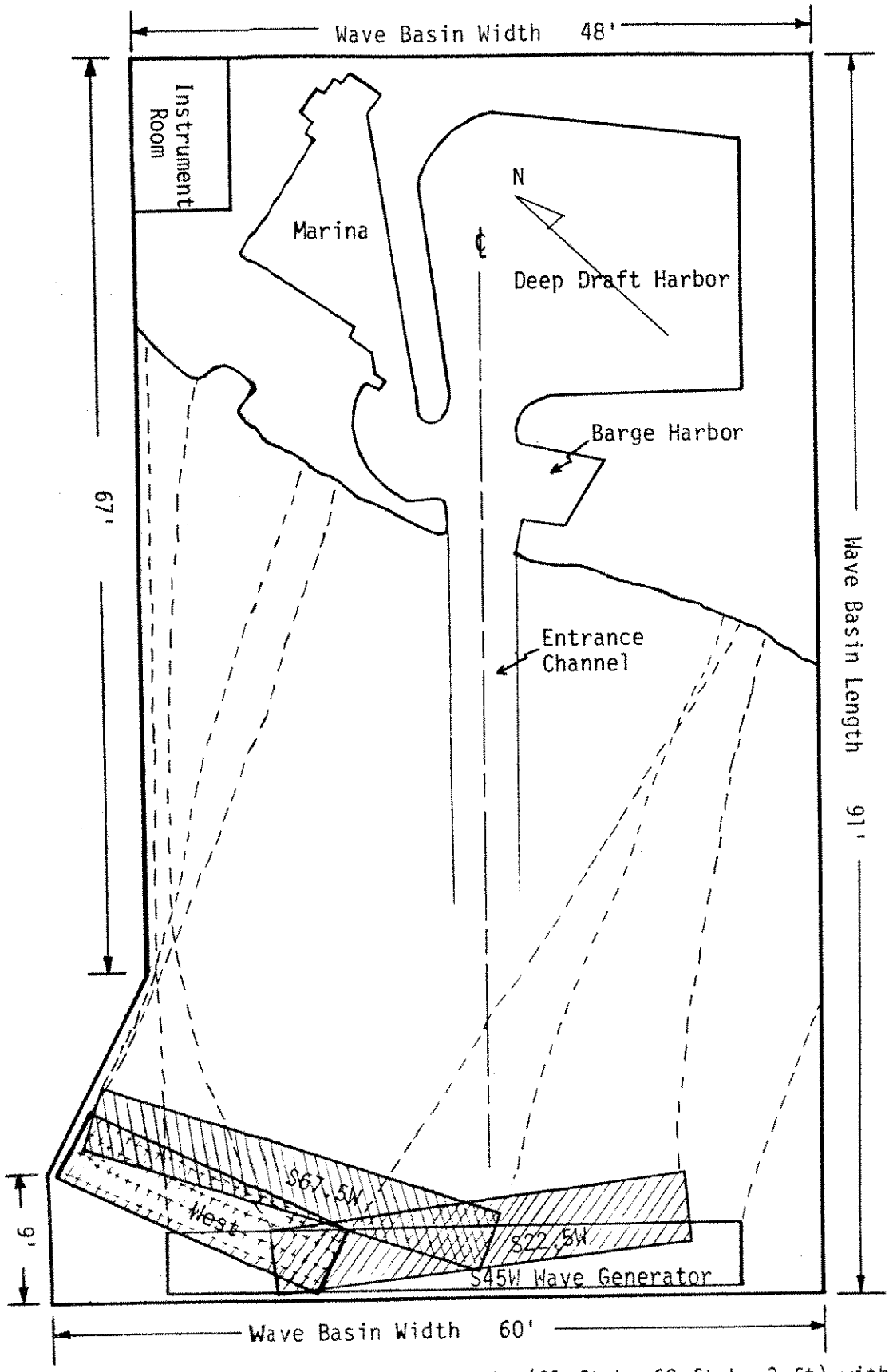


Fig. 12 Three-Dimensional Model Basin (91 ft by 60 ft by 3 ft) with Deep Draft Harbor/Marina/Barge Harbor Layout and Wave Generator Positions Simulating Wave Directions

PART III: TEST PROCEDURES AND FACILITIES

TEST PROCEDURES

III.1.1. Selection of Test Waves

The test waves were selected based on the following considerations:

(a) information furnished by the Alfred A. Yee Division with technical consultations by Drs. F. Gerritsen, C.L. Bretschneider and Look Laboratory researchers;

(b) refraction diagrams that were readily available;

(c) short-term wave measurements near the project site; and

(d) the hindcasts by the Marine Advisors (1964).

The final selection is shown in Table 1 in Part I.

III.1.2. Still-Water Levels Selection

For moderate wave conditions, the still-water level was selected to be +3 ft above MLLW. For storm wave conditions the level was selected as +4 above MLLW. (See Part I)

III.1.3. Test Procedures for the Three Dimensional Model

Prior to formal wave data collection in the model repeatability tests were conducted and some degree of calibration of the model with previous model test results conducted in 1970 was obtained. These results will be covered in Data Quality Assurance Program (Part V and Appendix D).

After the preliminary tests, formal tests were conducted. Waves were measured with two type wave gages - floating type and capacitance type. The former was mostly used in shallow water to eliminate surface tension and wetting effects. The measurements were recorded in a unique data acquisition system (See Fig. 14). A total of 4,000 data points with a time-interval of 1/25 sec. were collected per run. This is equivalent to 80 to 270 waves of 20 sec. to 6 sec. respectively. The record showed the time for the waves to reach the harbor entrance and/or the time to reflect to the wave generator. A study was made on the use of different sections of wave data to obtain maximum and mean wave heights and mean periods. As a result, a data section containing 2,000 to 3,000 data points (20 to 70 waves) was selected, which provided reasonable results. (See Part V and Appendix D)

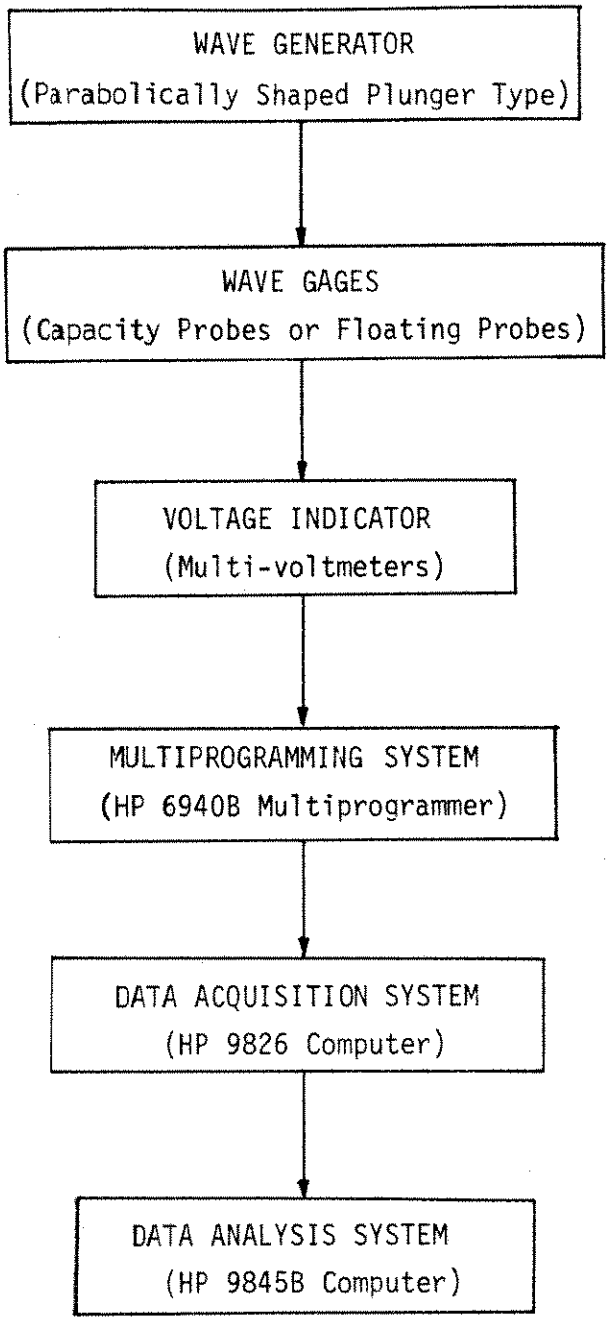


Figure 14 Diagram Showing the Wave Data Acquisition and Analysis System

III.1.4. Wave attenuation in small Scale models

The wave attenuation scale correction factor is a function of water depth, wave period, water viscosity, distance of wave travel and channel width. After a detailed study and a comparison with Barbers Point Deep Draft Harbor model study in 1970, it was decided that a factor of 1.1 for correction of wave heights in Deep Draft Harbor and Barge Harbor and 1.4 for wave height of the Marina where water depth is much shallower are appropriate. Detailed analysis are shown in Appendix A.

III.1.5. Modelling the Wave Absorbers

Reference was made to the reports by Harrenstien and Palmer (1970) and Hudson et.al. (1979). The modelling techniques are described in detail in Appendix B.

III.1.6. Test Procedures for Two-Dimensional Model

Test procedures for the two-dimensional model involved wave generator calibration, gage locations determination for different test waves which is very important to determine wave reflection coefficient based on two wave spectral analysis (Goda and Suzuki, 1976). A detailed description of the test program is given in Appendix C.

TEST FACILITIES

III.2.1. Three-Dimensional Model

The major test facilities included the 60 ft by 91 ft by 3 ft wave basin (Fig.12), wave generator, wave gages, and wave data acquisition and analysis system.

The wave basin is made of concrete with the area sufficient to cover the 1:100 scale model after minor modifications.

The wave generator had a vertical-motion type plunger which was adjustable to reproduce the prototype waves to scale. The regular type wave generator incorporated reciprocating speed and displacement controls which were infinitely variable within the required ranges and therefore were capable of producing a wide range of wave heights and periods. The wave generator was mounted on retractable casters which facilitated changing position and direction. (Fig. 12)

The wave gages used involved both floater and capacitance types for appropriate locations. (See Fig. 7)

The wave data acquisition and analysis system are shown in the block diagrams as shown in Fig. 14.

III.2.2. Two Dimensional Model

The test facilities included wave flume, wave generator, wave gages, and wave data acquisition and analysis system. The wave flume is 180 ft ling by 4 ft wide, and 4 ft high (Fig. 15).

The wave generator is also a plunger type operated with a hydraulic power system.

The wave gages and wave data acquisition and analysis system were the same as the three-dimensional model.

PART IV: TESTS AND TEST RESULTS

THREE DIMENSIONAL MODEL

IV.1.1. Type of Tests

A total of seven test plans was tested and they were described in PARTS I and II. PLAN 0 represented existing condition while PLANS I thru VI represented the proposed marina designs with three different entrances, with and without Igloo wave absorbers.

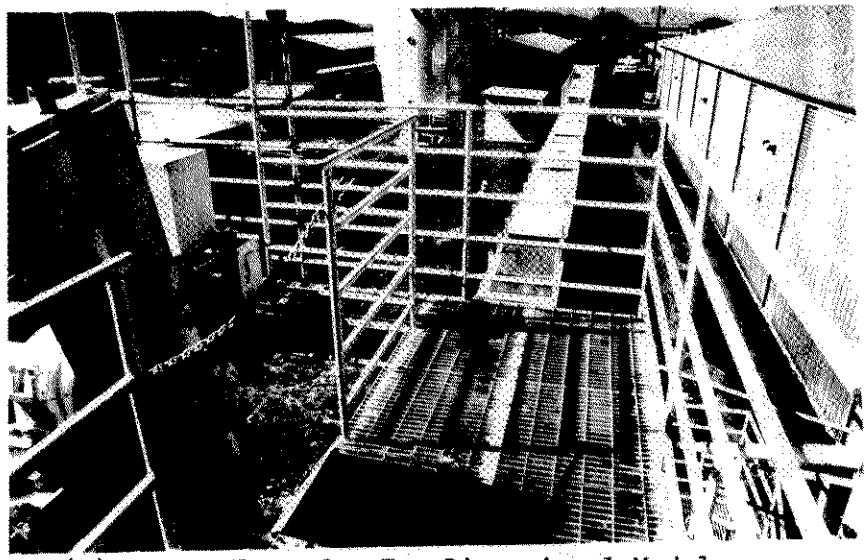
IV.1.2. Test Results

The test results of the seven plans are shown in Table 2 - 8. An analysis of the data has yielded the following results (See Table 9).

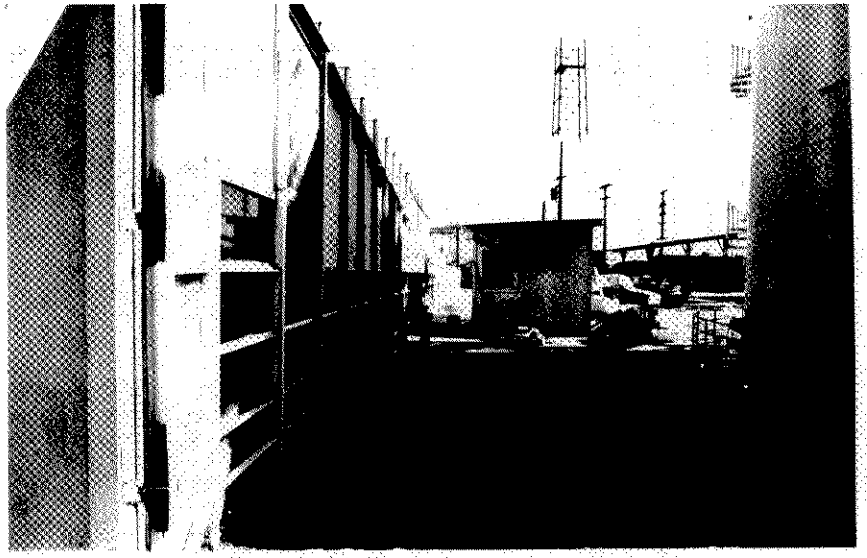
1. Impact on Wave Conditions in Deep Draft Harbor.

During the tests of the harbor without the marina (PLAN 0), 7% of the 154 test wave conditions (e.g. H = 18 ft, T = 6 sec, Direction S 45°W) at the berthing sites in the deep draft harbor exceeded wave height maximum-tolerable max-wave height criteria (4.5 ft) and 29% exceeded maximum desirable criteria (2.5 ft). In other words, 93% and 71% met the tolerable and desirable maximum height criteria respectively.

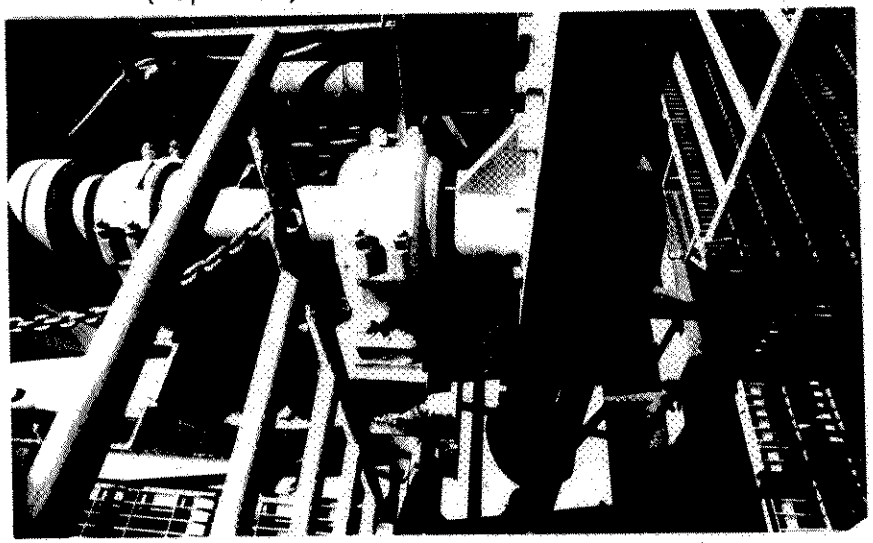
For PLANS I and II, wave conditions in the harbor improved; in PLAN II no waves exceeded the tolerable height criteria. In PLAN I wave height exceeding the desirable criteria reduced to 12% from 29% for PLAN 0. For PLANS I/II 97%/100% meet the tolerable and 88%/80% the desirable criteria, which indicates some improvement in the wave conditions at the harbor berths. For PLANS III and IV, a moderately beneficial effect on the deep draft harbor occurs overall; the percentage of wave conditions of 95%/100% and 66%/50% meet the tolerable and desirable height criteria. Even so, the marina has slightly improved the percentage of tolerable height but the percentages of heights that meet the desirable criteria have lessened. For PLAN V, wave conditions at the harbor berths of 100% and 77% met the



(a) Wave Flume for Two-Dimensional Model (Top View)



(b) Wave Flume for Two-Dimensional Model (Side View)



(c) Wave Generator for Two-Dimensional Model

(d) Wave Gage Arrangement for Two-Dimensional Model

Fig. 15 Wave Flume for Two-Dimensional Model

Table 2

MAXIMUM WAVE HEIGHT (H_{max}) [PLAN 0, without Marina]

INCIDENT WAVE		RESPONSE WAVE HEIGHT (ft)							
			Deep Draft Harbor						Barge Harbor
Gage No. 1		Gage #2	Gage #3	Gage #4	Gage #5	Gage #6	Gage #7	Gage #8	Gage #9
Wave Height (ft)	Wave Period (sec)								
Generator Position S 45° W									
12	6	1.6	2.9	1.0	1.3	0.5	1.6	2.6	0.7
18	6	9.1	7.7	3.5	5.3	4.2	6.8	7.8	10.7
12	8	1.6	0.8	0.5	0.8	1.1	1.3	0.7	1.8
12	8	1.8	1.0	0.8	0.9	0.4	1.8	0.7	1.6
12	12	0.8	1.5	0.8	1.5	1.4	1.9	1.5	4.9
18	12	5.9	3.9	3.0	2.9	3.7	5.2	4.0	10.1
12	15	2.3	2.6	1.5	1.4	0.7	1.6	1.5	2.0
18	15	4.8	5.1	2.9	2.9	1.0	3.2	2.7	2.3
36	15	8.1	4.0	2.8	2.7	1.5	3.3	3.6	5.3
Generator Position S 22.5° W									
12	14	2.3	2.6	1.5	1.4	0.7	1.6	1.5	2.0
18	14	4.8	5.1	2.9	2.9	1.0	3.2	2.7	2.3
36	14	5.1	1.6	1.8	2.0	1.3	2.0	1.3	3.1
Generator Position S 67.5° W									
12	15	2.6	2.0	1.7	1.6	0.4	1.1	1.0	2.4
18	15	2.3	1.5	2.3	2.3	0.3	0.8	0.8	1.5
36	15	7.0	4.6	0.8	1.0	3.1	---	5.0	5.9
Generator Position W									
12	12	3.0	0.8	1.6	2.0	2.1	2.3	1.5	12.1
18	12	2.7	1.9	2.0	3.3	4.9	3.7	2.0	21.6
12	15	3.2	1.3	1.3	1.6	1.0	1.9	0.5	2.5
18	15	3.6	1.0	1.4	2.6	1.4	2.4	0.3	4.2
12	20	2.8	1.7	1.6	1.6	1.8	2.0	2.2	2.5
18	20	2.6	3.8	1.7	1.8	2.9	2.0	2.6	2.3
36	15	3.1	---	2.4	2.5	2.1	1.8	1.1	---

NOTE: Model wave heights have been corrected for attenuation and some deviation of incident wave height simulated. The correction factor for attenuation is 1.1 while the factor for simulation deviation is varied. Wave gage locations are shown in the righthand figure.

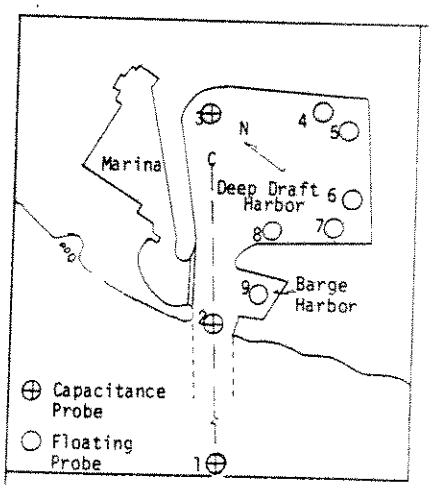


Table 3

Maximum Wave Height (H_{max}) [PLAN 1, Marina with Perpendicular Channel, without Igloos]

INCIDENT WAVE		RESPONSE WAVE HEIGHT (ft)						
			Marina (w/o Igloos)			Deep Draft Harbor		Barge Harbor
Gage No. 1		Gage #2	Gage #3	Gage #4	Gage #5	Gage #6	Gage #8	Gage #9
Wave Height (ft)	Wave Period (sec)							
Generator Position S 45° W								
12	6	4.1	3.0	0.3	neg	1.6	---	0.4
18	6	8.6	5.4	---	neg	2.4	---	3.9
12	8	---	---	---	---	---	---	---
18	8	3.9	4.3	neg	0.3	0.8	---	1.5
12	12	2.4	2.9	0.2	0.3	1.9	---	4.5
18	12	3.0	2.9	0.2	0.3	3.3	---	4.3
12	15	2.9	1.8	neg	0.3	0.9	---	1.0
18	15	9.9	4.8	0.3	0.5	1.6	---	1.5
36	15	4.2	4.0	0.5	0.5	1.0	---	2.1
Generator Position S 22.5° W								
12	14	3.2	4.1	neg	neg	0.2	1.6	0.7
18	14	5.1	10.3	neg	0.3	0.2	2.0	0.9
36	14	7.0	6.8	0.7	1.8	1.8	2.7	5.5
Generator Position S 67.5° W								
12	15	2.4	2.9	neg	0.6	1.0	0.6	2.7
18	15	6.9	5.1	neg	1.9	2.5	1.6	4.3
36	15	8.4	6.4	1.3	1.4	3.4	5.4	6.4
Generator Position W								
12	12	1.9	1.8	neg	0.3	0.7	1.6	3.5
18	12	2.8	5.1	neg	0.8	2.4	2.0	15.9
12	15	2.1	2.3	0.3	0.4	0.4	0.7	0.9
18	15	2.0	2.5	0.5	0.8	0.2	0.6	1.0
12	20	1.2	1.3	0.2	neg	0.3	0.8	0.7
18	20	1.0	0.9	0.1	neg	0.2	1.2	0.4
36	15	3.7	2.9	0.9	0.6	1.3	2.5	1.9

NOTE: Model wave heights have been corrected for attenuation and some deviation of incident wave height simulated. The correction factor for attenuation is 1.1 while the factor for simulation deviation is varied. Wave gage locations are shown in the righthand figure.

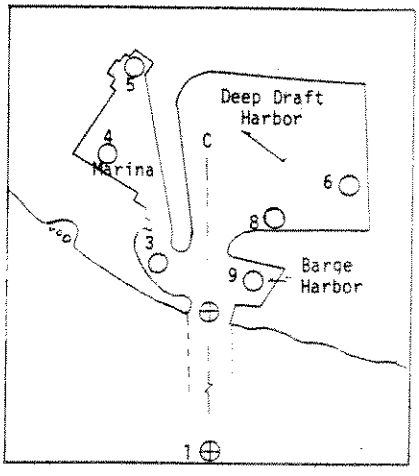


Table 4

Maximum Wave Height (H_{max}) [PLAN II, Marina with Perpendicular Channel with Igloos]

INCIDENT WAVE		RESPONSE WAVE HEIGHT (ft)						
			Marina (w/Igloos)			Deep Draft Harbor		Barge Harbor
Gage No. 1		Gage #2	Gage #3	Gage #4	Gage #5	Gage #6	Gage #8	Gage #9
Wave Height (ft)	Wave Period (sec)							
Generator Position S 45° W								
12	6	3.0	1.9	neg	neg	2.2	2.3	0.6
18	6	6.0	3.0	neg	neg	2.9	3.3	3.9
12	8	1.0	1.8	neg	neg	0.2	0.3	1.5
18	8	0.8	1.5	neg	neg	0.7	0.2	1.5
12	12	2.6	2.8	0.3	0.3	1.3	1.8	4.5
18	12	5.8	3.4	0.3	0.2	1.7	2.7	6.4
12	15	2.3	2.9	0.7	0.4	1.2	1.1	0.8
18	15	---	---	---	---	---	---	---
36	15	5.4	4.9	1.7	0.9	2.6	1.6	2.6
Generator Position S 22.5° W								
12	14	3.3	3.8	neg	0.5	0.2	0.8	1.4
18	14	5.5	6.8	0.3	1.6	0.2	0.6	5.0
36	14	4.6	6.7	0.4	1.5	2.0	2.8	4.9
Generator Position S 67.5° W								
12	15	2.5	4.1	0.3	1.2	1.2	1.2	1.2
18	15	3.8	7.1	0.3	2.0	1.8	1.4	4.3
36	15	4.3	6.3	0.7	1.4	2.0	3.2	5.5
Generator Position W								
12	12	2.7	2.3	neg	neg	1.6	3.7	3.3
18	12	4.0	4.5	0.2	0.2	1.8	3.9	6.0
12	15	2.5	1.9	0.5	0.3	1.0	1.1	1.1
18	15	4.6	1.7	0.5	0.5	1.9	1.3	2.3
12	20	1.0	1.6	0.4	neg	0.6	---	0.7
18	20	0.7	1.5	0.3	neg	0.8	---	0.9
36	15	2.9	3.0	0.6	0.4	1.5	1.5	1.6

NOTE: Model wave heights have been corrected for attenuation and some deviation of incident wave height simulated. The correction factor for attenuation is 1.1 while the factor for simulation deviation is varied. Wave gage locations are shown in the righthand figure.

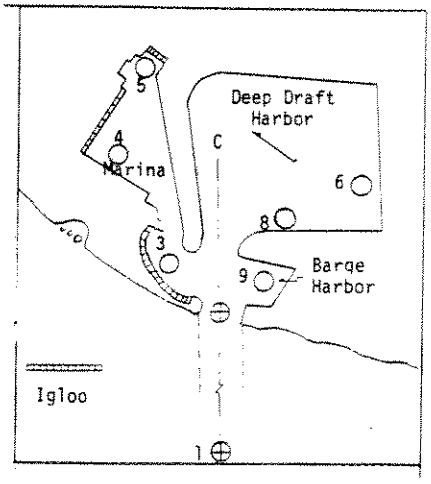


Table 5

MAXIMUM WAVE HEIGHT (H_{max}) [PLAN III, Marina with parallel channel, without Igloos]

INCIDENT WAVE		RESPONSE WAVE HEIGHT (ft)						
			Marina			Deep Draft Harbor		Barge Harbor
Gage No. 1		Gage #2	Gage #3	Gage #4	Gage #5	Gage #6	Gage #8	Gage #9
Wave Height (ft)	Wave Period (sec)							
Generator Position S 45° W								
12	6	3.3	8.0	0.2	0.3	4.5	2.8	0.9
18	6	3.4	1.4	0.6	0.6	6.1	2.7	1.5
12	8	2.9	4.8	neg	0.4	0.6	0.4	2.1
18	8	6.8	8.6	neg	0.8	2.4	1.6	4.0
12	12	3.7	7.2	0.3	1.4	0.9	2.1	2.9
18	12	8.1	9.2	0.4	1.6	3.8	4.0	5.6
12	15	2.7	5.7	0.3	1.2	0.9	1.3	1.5
18	15	3.0	10.9	0.5	1.8	0.8	2.2	2.2
36	15	5.4	9.7	0.4	2.8	2.9	3.0	2.5
Generator Position S 22.5° W								
12	14	3.5	7.4	neg	2.0	0.6	1.3	1.6
18	14	6.9	9.3	0.2	4.6	2.2	3.2	5.1
36	14	5.5	6.8	0.4	2.8	2.4	2.4	5.1
Generator Position S 67.5° W								
12	15	4.0	5.8	0.3	1.7	2.1	2.7	2.0
18	15	6.0	6.0	0.1	1.8	3.6	4.8	1.6
36	15	8.0	11.2	1.7	2.2	3.4	4.0	5.0
Generator Position W								
12	12	3.1	7.5	0.2	1.4	1.1	2.3	2.5
18	12	4.7	8.6	0.3	1.7	1.0	---	4.8
12	15	2.0	4.8	0.2	0.7	0.9	1.1	2.1
18	15	1.8	9.4	0.3	0.7	1.1	---	3.4
12	20	4.1	5.6	1.1	0.5	0.6	1.1	2.1
18	20	6.3	8.4	1.6	0.7	0.9	1.3	2.5
36	15	5.6	5.6	0.2	1.8	1.6	---	3.0

NOTE: Model wave heights have been corrected for attenuation and some deviation of incident wave height simulated. The correction factor for attenuation is 1.1 while the factor for simulation deviation is varied. Wave gage locations are shown in the righthand figure.

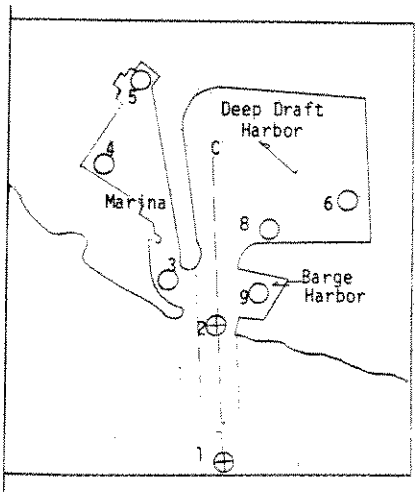


Table 6

MAXIMUM WAVE HEIGHT (H_{max}) [PLAN IV, Marina with parallel channel, with Igloos]

INCIDENT WAVE		RESPONSE WAVE HEIGHT (ft)						
			Marina			Deep Draft Harbor		Barge Harbor
Gage No. 1		Gage #2	Gage #3	Gage #4	Gage #5	Gage #6	Gage #8	Gage #9
Wave Height (ft)	Wave Period (sec)							
Generator Position S 45° W								
12	6	4.9	5.8	0.3	0.2	1.6	0.9	1.0
18	6	11.0	9.3	0.5	0.6	1.7	0.4	4.6
12	8	2.5	3.9	neg	0.3	1.1	0.8	2.7
18	8	2.6	6.6	neg	0.6	1.9	1.1	5.6
12	12	2.9	6.5	0.2	0.9	0.8	1.0	2.4
18	12	3.9	7.5	neg	0.7	3.5	0.9	10.2
12	15	2.5	4.3	0.2	1.0	1.1	1.0	1.3
18	15	2.9	4.5	neg	0.8	1.9	1.1	1.2
36	15	5.4	7.6	0.4	2.0	2.9	3.7	3.7
Generator Position S 22.5° W								
12	14	1.7	8.1	neg	2.3	1.4	1.5	2.3
18	14	5.5	11.3	0.3	3.4	3.7	3.5	2.6
36	14	8.2	7.8	0.2	2.9	3.5	4.5	5.8
Generator Position S 67.5° W								
12	15	4.3	6.3	0.3	2.4	1.9	2.1	1.7
18	15	4.5	6.5	0.2	2.8	4.0	2.1	2.1
36	15	6.4	14.1	0.3	2.2	3.6	3.6	5.9
Generator Position W								
12	12	3.5	8.1	0.5	1.7	1.7	3.0	1.7
18	12	7.8	11.0	0.7	2.2	2.3	7.5	2.1
12	15	1.3	3.4	neg	1.3	0.5	0.9	1.6
18	15	0.8	5.4	0.6	2.4	0.4	0.8	2.4
12	20	4.4	4.9	1.1	0.3	1.1	1.5	1.9
18	20	6.2	6.9	1.4	0.8	1.6	2.2	2.6
36	15	3.2	7.1	---	2.1	1.1	1.6	4.0

NOTE: Model wave heights have been corrected for attenuation and some deviation of incident wave height simulated. The correction factor for attenuation is 1.1 while the factor for simulation deviation is varied. Wave gage locations are shown in the righthand figure.

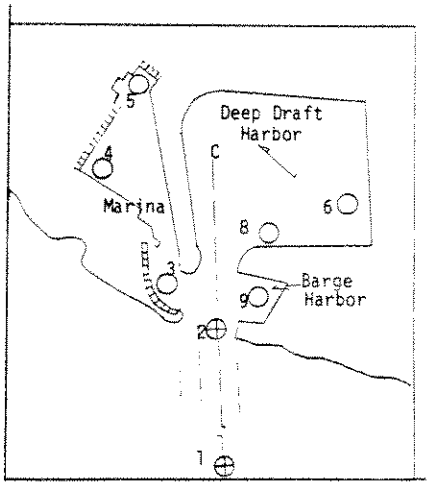


Table 7

MAXIMUM WAVE HEIGHT (H_{max}) [PLAN V, Marina with separate channel, without Igloos]

INCIDENT WAVE		RESPONSE WAVE HEIGHT (ft)						
			Marina			Deep Draft Harbor		Barge Harbor
Gage No. 1		Gage #2	Gage #3	Gage #4	Gage #5	Gage #6	Gage #8	Gage #9
Wave Height (ft)	Wave Period (sec)							
Generator Position S 45° W								
12	6	3.9	5.2	neg	0.7	2.1	2.7	1.7
18	6	5.9	4.5	neg	1.3	2.0	3.8	5.2
12	8	3.9	9.3	neg	0.7	1.1	2.5	2.2
18	8	5.7	14.3	0.4	0.7	1.7	1.5	4.6
12	12	3.3	7.2	neg	0.6	0.9	1.1	3.1
18	12	4.7	9.3	0.4	0.7	0.8	1.2	3.1
12	15	2.4	4.3	0.3	1.1	0.7	1.8	2.0
18	15	7.2	3.3	0.8	1.5	0.9	2.3	3.0
36	15	6.1	6.6	0.3	1.3	2.0	2.8	6.3
Generator Position S 22.5° W								
12	14	1.9	4.5	neg	1.4	1.8	1.8	3.0
18	14	3.3	12.0	0.4	1.3	0.2	4.4	7.0
36	14	3.5	12.6	0.3	2.6	1.3	2.4	4.8
Generator Position S 67.5° W								
12	15	3.5	9.0	neg	1.4	1.1	1.1	3.1
18	15	6.0	13.2	0.4	2.3	1.4	1.7	4.7
36	15	6.3	12.6	0.4	2.0	2.6	2.2	2.4
Generator Position W								
12	12	2.3	7.0	0.3	1.1	0.7	0.7	2.8
18	12	3.7	9.8	0.4	2.8	1.0	0.8	3.8
12	15	1.8	8.5	0.4	1.5	0.8	0.6	1.1
18	15	1.5	12.8	1.9	2.7	0.8	0.3	0.8
12	20	1.6	4.4	0.5	0.3	0.7	1.1	2.2
18	20	2.1	3.4	0.3	0.2	0.5	1.3	3.3
36	15		10.9	1.0	1.9	1.5	2.0	5.4

NOTE: Model wave heights have been corrected for attenuation and some deviation of incident wave height simulated. The correction factor for attenuation is 1.1 while the factor for simulation deviation is varied. Wave gage locations are shown in the righthand figure.

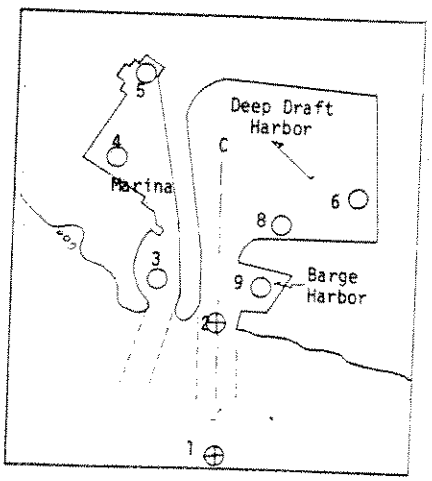


Table 8

MAXIMUM WAVE HEIGHT (H_{max}) [PLAN VI, Marina with separate channel, with Igloos]

INCIDENT WAVE		RESPONSE WAVE HEIGHT (ft)						
			Marina			Deep Draft Harbor		Barge Harbor
Gage No. 1		Gage #2	Gage #3	Gage #4	Gage #5	Gage #6	Gage #8	Gage #9
Wave Height (ft)	Wave Period (sec)							
Generator Position S 45° W								
12	6	2.7	8.4	neg	0.3	1.3	1.9	1.5
18	6	3.6	15.4	neg	0.3	1.8	2.4	3.9
12	8	3.5	7.0	neg	0.3	0.3	0.9	1.6
18	8	6.2	10.8	neg	0.3	2.2	2.2	1.4
12	12	2.6	6.0	neg	1.3	0.9	0.7	5.6
18	12	4.2	13.5	neg	3.2	1.4	2.2	11.0
12	15	3.4	4.2	0.1	1.1	0.9	1.6	1.5
18	15	6.7	2.8	0.1	1.2	1.4	2.6	2.3
36	15	6.0	8.0	0.1	0.7	2.9	2.4	4.0
Generator Position S 22.5° W								
12	14	2.9	7.2	0.3	3.1	1.0	1.1	3.1
18	14	5.9	10.0	0.5	3.1	2.5	1.3	6.4
36	14	3.4	10.5	0.1	3.7	1.5	2.3	4.8
Generator Position S 67.5° W								
12	15	3.4	9.2	0.5	1.8	1.2	1.1	2.2
18	15	7.2	14.1	0.5	3.5	2.0	2.4	1.5
36	14	7.9	15.1	1.2	2.2	2.3	4.7	3.6
Generator Position W								
12	12	2.9	4.0	0.1	1.1	2.1	1.9	3.3
18	12	5.9	4.8	0.1	1.6	4.9	5.2	4.8
12	15	2.5	7.0	0.2	1.9	1.0	1.1	1.3
18	15	2.8	11.3	0.1	2.4	1.1	0.9	1.7
12	20	1.3	4.6	0.6	0.5	1.0	1.3	1.6
18	20	1.4	3.5	0.8	0.5	1.2	1.8	1.5
36	15	5.9	12.9	1.5	5.1	2.0	2.0	4.5

NOTE: Model wave heights have been corrected for attenuation and some deviation of incident wave height simulated. The correction factor for attenuation is 1.1 while the factor for simulation deviation is varied. Wave gage locations are shown in the righthand figure.

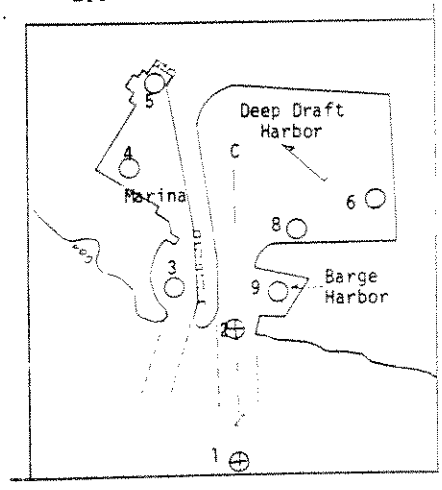


Table 9

Percentage of Measured Waves Meeting the Max. Wave Height Criteria for Different Plans

PLAN	Deep Draft Harbor Berths Meet Max. Wave Height Criteria		Marina Berths Meet Max. Wave Height Criteria		Barge Harbor Meet Max. Wave Height Criteria	
	Tolerable, 4.5 ft	Desirable, 2.5 ft	Tolerable, 2 ft	Desirable, 1.0 ft	Tolerable, 4.5 ft	Desirable, 2.5 ft
0	93%	71%	-	-	71%	71%
I	97%	88%	100%	90%	86%	71%
II	100%	80%	100%	86%	76%	48%
III	95%	66%	91%	66%	78%	64%
IV	100%	50%	86%	68%	77%	59%
V	100%	77%	91%	66%	73%	32%
VI	86%	77%	82%	61%	73%	50%

Deep Draft Harbor Berths and Marina Berths Combined

PLAN	Meet Max. Wave Height Criteria	
	Tolerable, 4.5 ft/2 ft	Desirable, 2.5 ft/1 ft
I, II	99%	83%
III, IV	93%	63%
V, VI	90%	70%

tolerable and desirable criteria respectively which showed that the marina has no adverse effect on the deep draft harbor. Instead, some improvement is shown. For PLAN VI, 86% and 77% of wave conditions met the tolerable and desirable criteria respectively. Overall, there was no significant adverse effect on the harbor. In comparison PLANS I and II are superior to the other plans from a wave impact point of view. PLANS III and IV, although less effective than PLANS I and II, have acceptable wave conditions. PLANS V and VI appear to be less acceptable with respect to wave conditions than other plans.

2. Wave Conditions in Marina.

For PLANS I and II wave heights inside the marina basin (berthing area) are below the maximum tolerable limit of 2.0 ft under all test conditions, and 90% and 86% respectively meet the desirable height criteria of 1.0 ft. For PLANS III and IV, 97%/86% meet the tolerable limit criteria but only 66%/68% meet the desirable criteria as compared with 100%/90% for PLANS I and II. A similar trend is observed for PLANS V and VI.

3. Wave Conditions in Barge Harbor.

Wave conditions in the Barge Harbor are higher than in the Deep Draft Harbor and in the Marina berthing areas. PLANS I and II are relatively better than PLANS III, IV, V and VI. On the average, over all tests only 77% meet the tolerable height criteria and 54% the desirable criteria.

4. Wave Conditions in Entrance Channel.

Wave conditions in the entrance channel are important because they signify the navigation hazards when entering the marina under adverse wave conditions. In PLANS I and II the deep draft harbor channel allows relatively safe entering of yachts because its greater depth induces breaking of only large waves which occur infrequently.

During certain test conditions (e.g. 18 foot wave height, 14 second period and S22°W wave direction) in PLANS I and II, wave heights in the entrance (gage #3) may reach or exceed 10 ft. Such conditions make entering difficult; consequently, these plans should be evaluated in terms of adequate maneuvering space for safe entering. Under most conditions waves in the passage area of the marina are relatively low as compared with other plans; these enable the yachts to enter the berthing area more easily. Furthermore,

it should be noted that large wave conditions in the entrance area are uncommon and that under such circumstances craft should not normally be attempting to enter or exit the marina.

In PLANS III and IV under extreme wave conditions the wave heights outside the entrance are higher than in PLANS I and II. However, the parallel channel allows more room for maneuvering outside the marina entrance.

Inside the entrance the wave conditions are also higher than in PLANS I and II, which make maneuvering difficult, although the maneuvering area in the configuration tested is somewhat larger than in PLANS I and II.

In PLANS V and VI the most serious conditions will occur in the entrance channel offshore where waves of 18 feet or higher were observed to break in the model. The model test had no gage installed in that section so no direct measurements are available. Also the wave heights in the passage area in the outer marina are much higher than those for PLANS I, II, III and IV. The separate entrance test with Igloo wave absorbers (PLAN VI) gave slightly higher wave conditions than the test with a spending beach (1:10 slope). From a navigational safety point of view the separate entrance alternative appears to be the least desirable.

5. Wave Energy Absorber Solutions With and Without Igloos.

To obtain more reliable results for comparisons the tests involved both tests in three-dimensional and two-dimensional models. For three-dimensional model, the comparisons were between PLANS I and II, PLANS III and IV, and PLANS V and VI. As shown in Table 9, the difference between the test results for solutions with and without Igloos are small. However, the results may be subject to scale effects in the three-dimensional model (1:100 scale). In the entrance passage area, the spending beach should be more effective in view of the results of the large scale (1:10) two-dimensional tests.

TWO-DIMENSIONAL MODEL

IV.2.1. Type of Tests

The wave periods and wave heights are selected from the three-dimensional tests (PLANS I and II) (see Table 10).

Table 10

Test Plans of Two-Dimensional Model

For Plans A, B, D (See Figure 13)

6 sec	2 ft	4 ft	(In model:	1.9 sec	0.2 ft	0.4 ft)
12 sec	2 ft	4 ft	(In model:	3.8 sec	0.2 ft	0.4 ft)
15 sec	2 ft	4 ft	(In model:	4.7 sec	0.2 ft	0.4 ft)
20 sec	2 ft	4 ft	(In model:	6.3 sec	0.2 ft	0.4 ft)

For Plans C, D, E (See Figure 13)

6 sec	1 ft	2 ft	(In model:	1.9 sec	0.1 ft	0.2 ft)
12 sec	1 ft	2 ft	(In model:	3.8 sec	0.1 ft	0.2 ft)
15 sec	1 ft	2 ft	(In model:	4.7 sec	0.1 ft	0.2 ft)
20 sec	1 ft	2 ft	(In model:	6.3 sec	0.1 ft	0.2 ft)

The gage spacings were as follows:

Wave Period in Model	Water Depth in Model	Wave Gage Spacing (ft)
1.9	1.8 ft	3.24
3.8	1.8 ft	7.04
4.7	1.8 ft	8.79
6.3	1.8 ft	11.85

The following sketch shows the general gage arrangement:

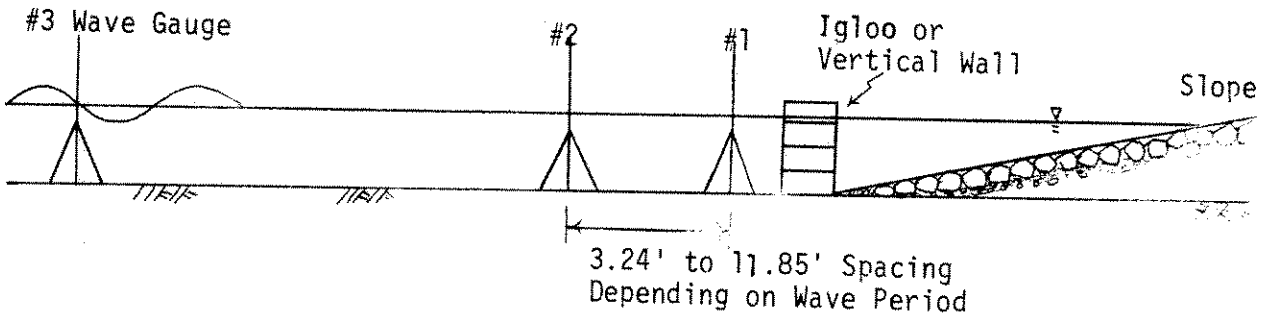
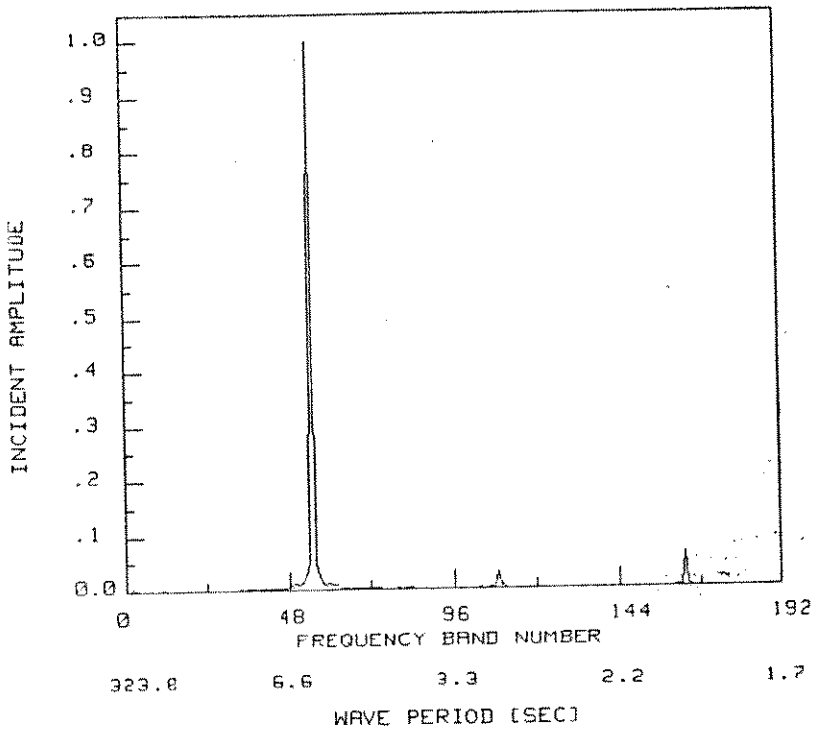


Fig. 16: Wave Gage Locations for the Two-Dimensional Model

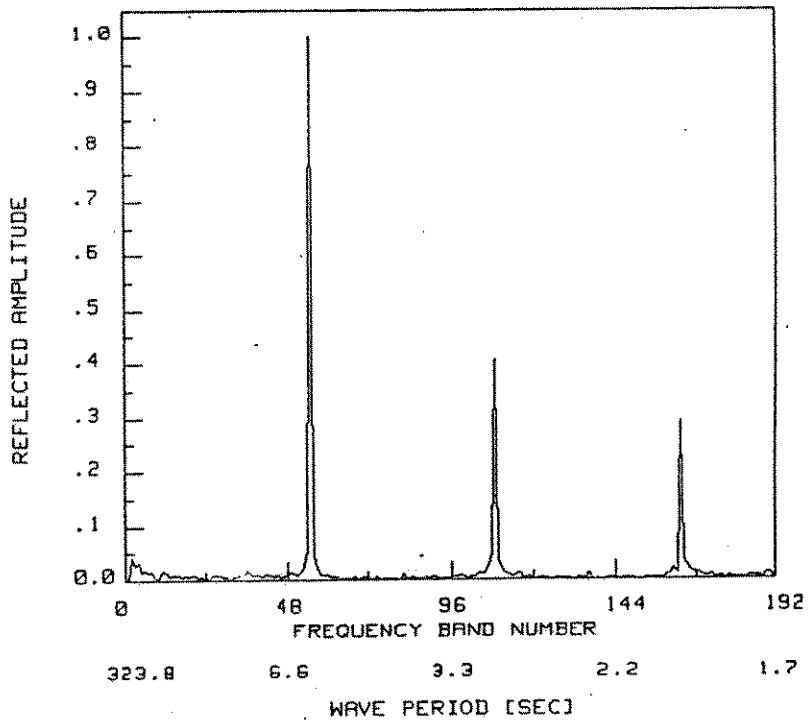
IV.2.2. Test Results

The typical reflection coefficients are determined as shown in Figures 17 through 22. The test results are compared as shown in Table 11.

In comparing the Igloo solution with a vertical wall the Igloos appeared effective for a wave period of 6 seconds and wave height of 2 feet but less effective for the longer wave periods of 12, 15 and 20 seconds. This type of results has been confirmed by Bottin (1976), and Nippon Tetrapod Company's laboratory experiments. Measured reflection coefficients for a 6 second and 2 feet wave in the two dimensional model are shown in Table 12.



VALUES OF Y-COORDINATE HAVE BEEN NORMALIZED BY |MAX.VALUE|
|MAX.VALUE | = .968

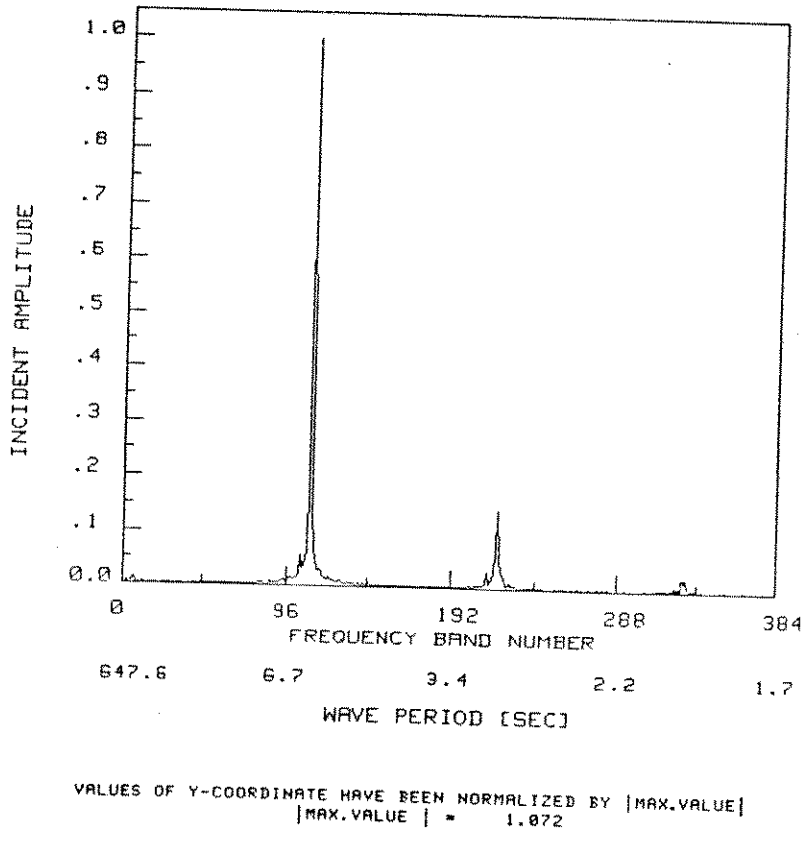


VALUES OF Y-COORDINATE HAVE BEEN NORMALIZED BY |MAX.VALUE|
|MAX.VALUE | =

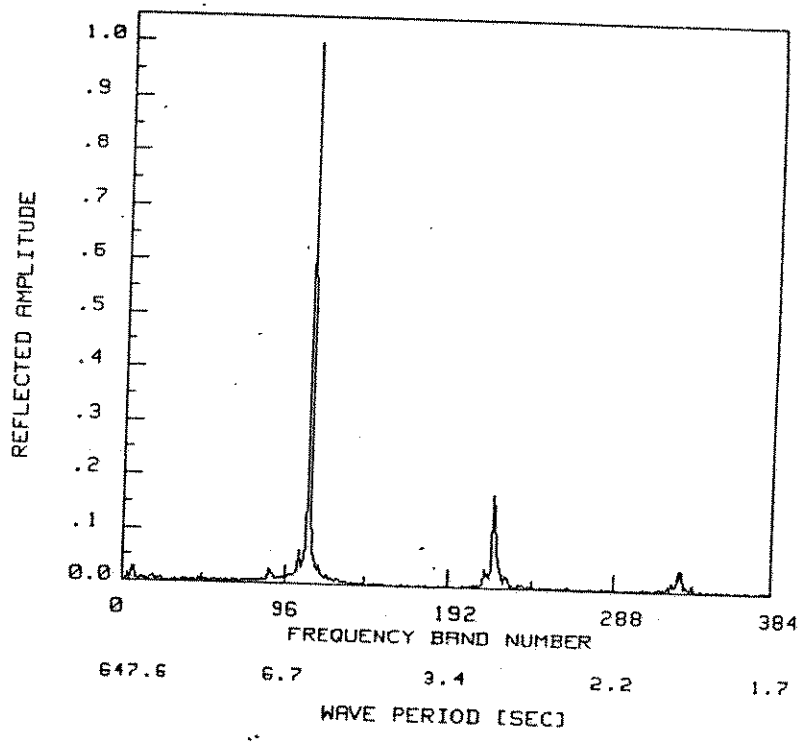
RUN # = C23
RECORDING DATE = 12/18/84
WAVE PERIOD (sec) = 6.0
WAVE HEIGHT (ft) = 2.0
WATER DEPTH (ft) = 18.0
WAVE ABSORVER = SLOPE-1:10

OVERALL COEFFICIENT OF REFLECTION = .345

Fig. 17 Wave Reflection Coefficient for 1:10 Slope Spending Beach
(Wave Period of 6 Seconds and Wave Height of 2 ft)



VALUES OF Y-COORDINATE HAVE BEEN NORMALIZED BY |MAX.VALUE|
|MAX.VALUE | = 1.072

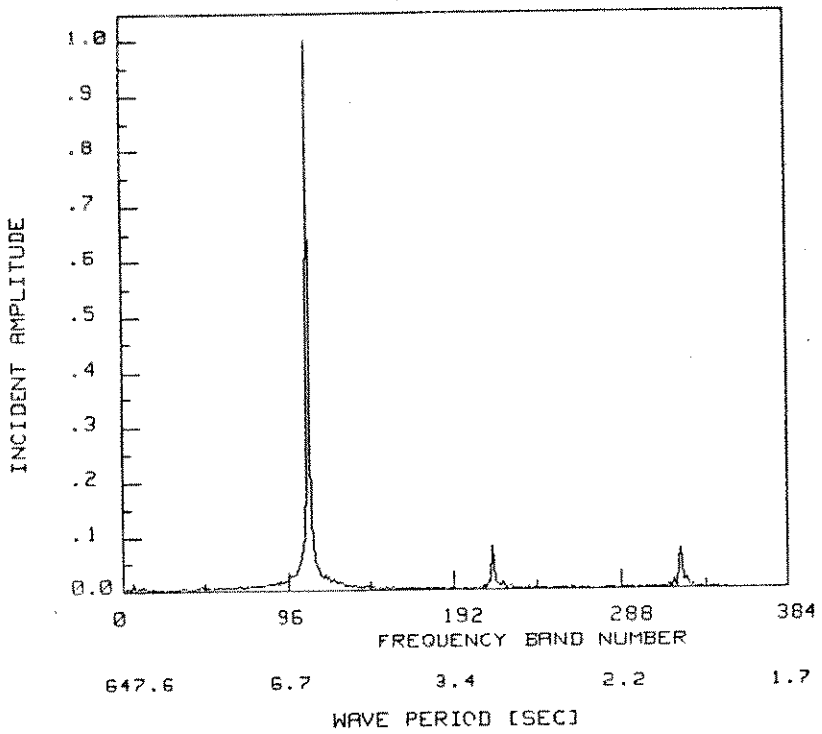


VALUES OF Y-COORDINATE HAVE BEEN NORMALIZED BY |MAX.VALUE|
|MAX.VALUE | = .504

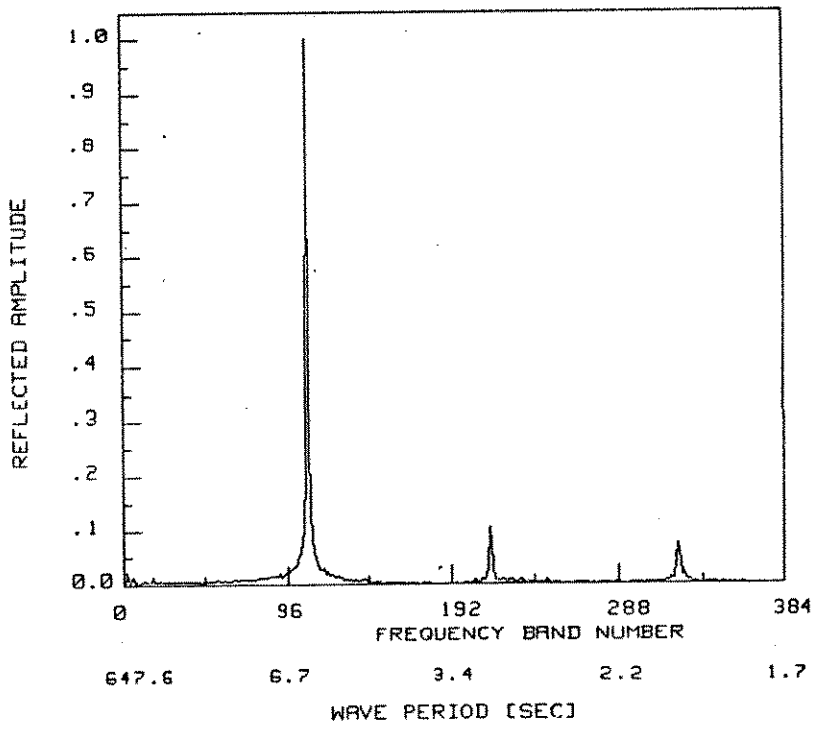
RUN # = F24
RECORDING DATE = 12/21/84
WAVE PERIOD (sec) = 6.0
WAVE HEIGHT (ft) = 2.0
WATER DEPTH (ft) = 18.0
WAVE ABSORVER = IGLOO

OVERALL COEFFICIENT OF REFLECTION = .649

Fig. 18 Wave Reflection Coefficient for Igloo Wall
(Wave Period of 6 Seconds and Wave Height of 2 ft)



VALUES OF Y-COORDINATE HAVE BEEN NORMALIZED BY |MAX.VALUE|
|MAX.VALUE | = .648

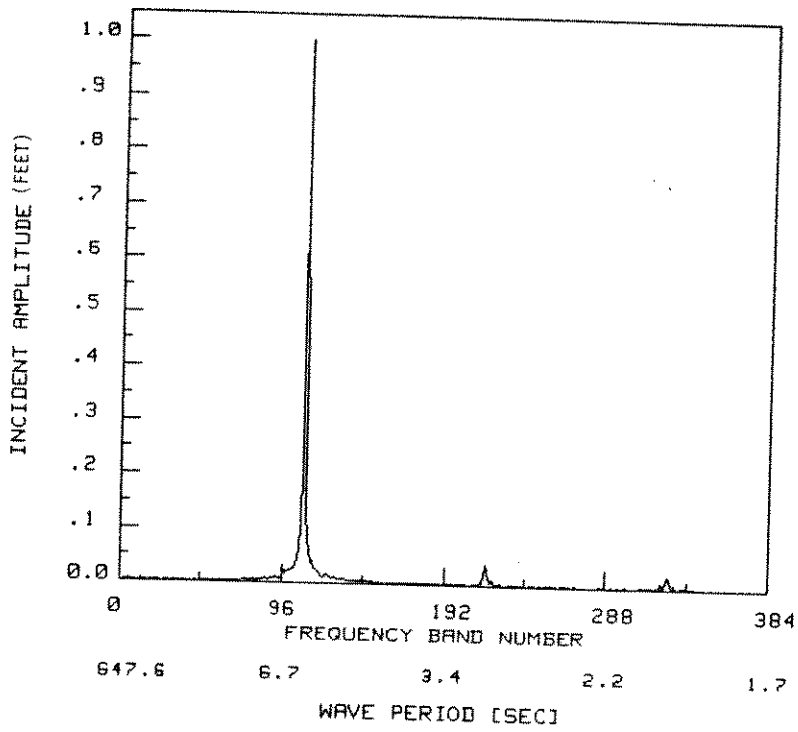


VALUES OF Y-COORDINATE HAVE BEEN NORMALIZED BY |MAX.VALUE|
|MAX.VALUE | = .473

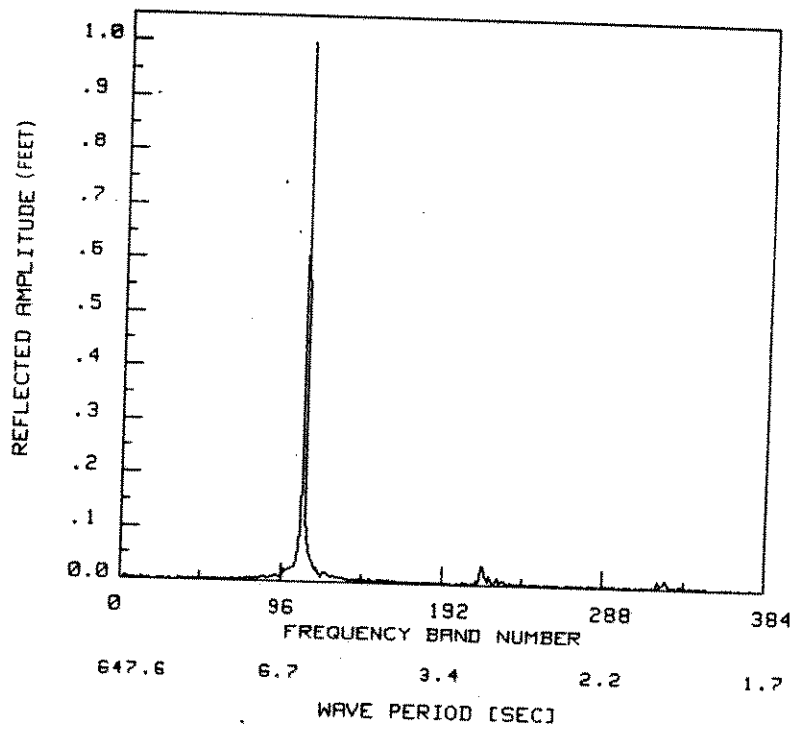
RUN # = C2
RECORDING DATE = 12/21/84
WAVE PERIOD (sec) = 6.0
WAVE HEIGHT (ft) = 2.0
WATER DEPTH (ft) = 18.0
WAVE ABSORVER = VERTICALL-WALL

OVERALL COEFFICIENT OF REFLECTION = .781

Fig. 19 Wave Reflection Coefficient for Vertical Solid Wall
(Wave Period of 6 Seconds and Wave Height of 2 ft)



VALUES OF Y-COORDINATE HAVE BEEN NORMALIZED BY |MAX.VALUE|
|MAX.VALUE | = 1.005 FEET

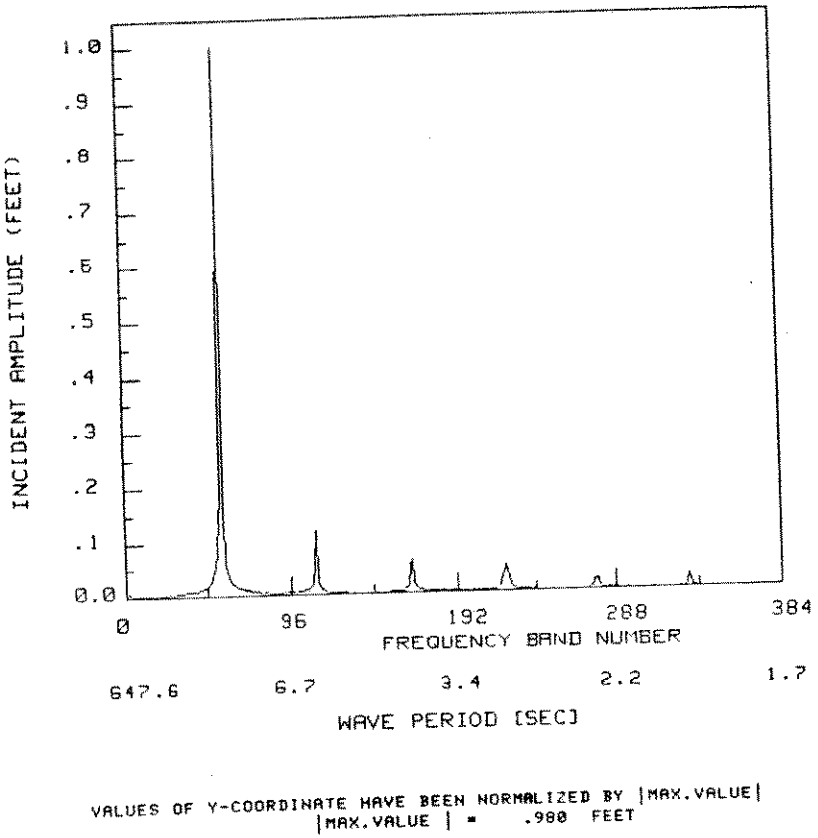


VALUES OF Y-COORDINATE HAVE BEEN NORMALIZED BY |MAX.VALUE|
|MAX VALUE | = .736 FEET

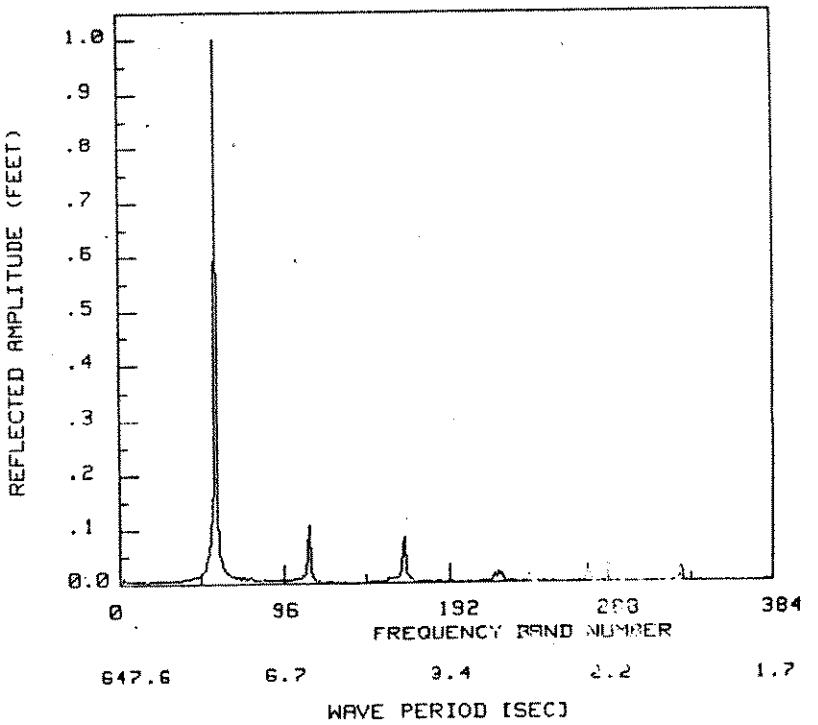
RUN # = E2
RECORDING DATE = 01/03/85
WAVE PERIOD (sec) = 6.0
WAVE HEIGHT (ft) = 2.0
WATER DEPTH (ft) = 18.0
WAVE ABSORVER = VERTICAL-WALL&FLOATING-PIER

OVERALL COEFFICIENT OF REFLECTION = .792

Fig. 20 Wave Reflection Coefficient for Vertical Solid Wall and Floating Pier (Wave Period of 6 Seconds and Wave Height of 2 ft)



VALUES OF Y-COORDINATE HAVE BEEN NORMALIZED BY |MAX.VALUE|
|MAX.VALUE | = .980 FEET

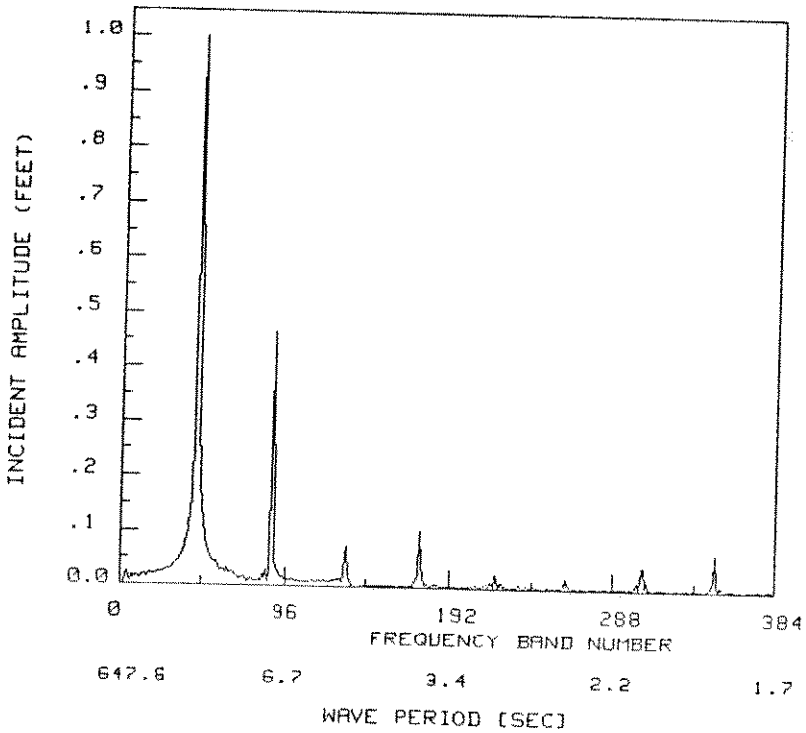


VALUES OF Y-COORDINATE HAVE BEEN NORMALIZED BY |MAX.VALUE|
|MAX.VALUE | = .714 FEET

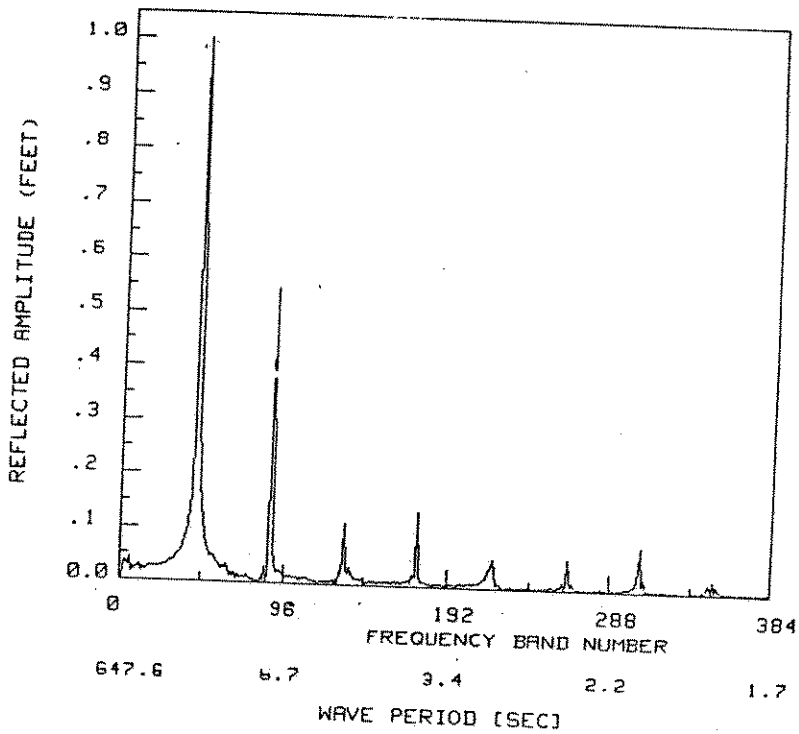
RUN # = D2
RECORDING DATE = 12/21/84
WAVE PERIOD (sec) = 12.0
WAVE HEIGHT (ft) = 2.0
WATER DEPTH (ft) = 18.0
WAVE ABSORVER = IGLOO

OVERALL COEFFICIENT OF REFLECTION = .765

Fig. 21 Wave Reflection Coefficient for Igloo Wall
(Wave Period of 12 Seconds and Wave Height of 2 ft)



VALUES OF Y-COORDINATE HAVE BEEN NORMALIZED BY |MAX.VALUE|
|MAX.VALUE | = .455 FEET



VALUES OF Y-COORDINATE HAVE BEEN NORMALIZED BY |MAX.VALUE|
|MAX.VALUE | = .350 FEET

RUN # = D5
RECORDING DATE = 12/21/84
WAVE PERIOD (sec) = 14.9
WAVE HEIGHT (ft) = 2.0
WATER DEPTH (ft) = 18.0
WAVE ABSORVER = IGLOO

OVERALL COEFFICIENT OF REFLECTION = .883

Fig. 22 Wave Reflection Coefficient for Igloo Wall
(Wave Period of 15 Seconds and Wave Height of 2 ft)

Table 11: Test Results on Wave Reflection Coefficients for Different Wave Absorbers in the Two-Dimensional Model

Wave Absorber	Prototype Incident Wave Height (ft)	Reflection Coefficient Prototype Incident Wave Period (sec) 6	12	15	20
1:10 Slope Spending Beach	1.0	-	-	-	-
Igloo Wall	1.0	0.62	0.74	0.86	0.89
Vertical Solid Wall	1.0	0.76	0.84	0.86	-
Vertical Solid Wall and Floating Pier	1.0	0.75	0.80	0.88	0.93
1:10 Slope Spending Beach	2.0	0.35 1/	0.37 1/	0.76 2/	0.73 2/
Igloo Wall	2.0	0.65	0.77	0.88	-
Vertical Solid Wall	2.0	0.79	0.84	0.91	0.93
Vertical Solid Wall and Floating Pier	2.0	0.79	0.84	-	-
1:10 Slope Spending Beach	3.0	-	-	-	-
Igloo Wall	3.0	0.68	0.78	0.92	0.95
Vertical Solid Wall	3.0	-	-	-	-
Vertical Solid Wall and Floating Pier	3.0	-	-	-	-
1:10 Slope Spending Beach	4.0	0.32 1/	0.50 1/	0.66 2/	0.81 2/
Igloo Wall	4.0	-	-	-	-
Vertical Solid Wall	4.0	-	-	-	-
Vertical Solid Wall and Floating Pier	4.0	-	-	-	-

Note: 1/ = Plunging breakers 2/ = Surging breakers Plunging breakers dissipates more energy

Table 12: Reflection Coefficients Measured for Different Wave Absorbers for 6 second, 2 feet Waves.

Wave Absorbers	Reflection Coefficents
1:10 Slope Spending Beach	0.35
Igloo Wall	0.65
Vertical Solid Wall	0.78
Vertical Solid Wall and one Floating Pier (parallel to wall)	0.79

Note that the floating pier did not reduce the reflection coefficient as expected. However, there will be a series of floating piers in the prototype, and the wave absorbing effect could be larger.

PART V: DATA QUALITY ASSURANCE PROGRAM

Special efforts were devoted to carry out the data quality assurance program as listed below:

(1) Model Construction: The model bottom configuration (contours) was constructed to ensure accurate reproduction of the prototype condition. After concrete was cast, the surface was refinished to avoid undesirable topography deviations.

(2) The wave generator positions were accurately set according to the wave conditions simulated.

(3) Calibration of the wave generator was carefully done for the positions corresponding to wave directions specified.

(4) Calibration of the wave gages was also carefully made.

(5) Repeatability tests were conducted for critical wave conditions in order to ensure good data quality.

(6) The wave heights measured in the small 1:100 scale model are subjected to attenuation as a function of water depth, wave period, water viscosity, distance of wave travel and channel width. Therefore, model wave heights were corrected accordingly. The procedure used was described by Harrenstien and Palmer, 1970.

(7) The wave absorbing characteristics of the Igloos in the 1:100 small scale model do not yield quantitative results. However, qualitative results resulted. The scale effects in the 1:10 scale tests in the large wave tank are small.

(8) To make sure the test results were reasonable, data was analyzed immediately following each test series.

(9) Periodic meetings between the investigators and contractor's technical representative and consultants were held. The technical personnel of the U.S. Army Corps of Engineers and the State Harbors Division were kept informed of the process of the tests and analyses at various stages. Their opinions were appreciated.

(10) The Principal Investigator was charged with the responsibility of the Data Quality Assurance Program.

Detailed information is described in Appendix D.

PART VI: FINDINGS AND CONCLUSIONS

Based on test results of the hydraulic model study it is concluded that (see Table 9):

VI.1.1. Wave Conditions in Deep Draft Harbor, Marina, and Barge Harbor

PLANS I and II - Ninety-nine (99%) of the test wave conditions in the Deep draft harbor and marina combined meet the tolerable maximum heights and about 86% meet the desirable maximum height criteria. Wave conditions in the barge harbor were more severe than in the deep draft harbor.

PLANS III and IV - Ninety-three (93%) of the wave conditions in deep draft harbor and marina meet the tolerable maximum height criteria while 63% meet the desirable maximum height criteria. Wave conditions in the barge harbor were worse than in PLANS I and II.

PLANS V and VI - Ninety (90%) of the wave conditions in the deep draft harbor and the marina meet the tolerable height criteria while 70% meet the desirable maximum height criteria. For the barge harbor the wave conditions were the worst compared with PLANS I and II.

VI.1.2. Wave Conditions in Entrance Channel

Wave conditions in PLANS I and II were better than PLANS III and IV; for PLANS V and VI they are the worst.

VI.1.3. Solutions With and Without Igloo Wave Absorber

The differences between the solutions with and without Igloos are small.

In the berthing areas, the wave heights seem to be slightly less for the Igloo solution than the vertical wall, particularly for wave period less than 6 seconds.

In the entrance area, the spending beach should be more effective in view of the test result of the two-dimensional tests.

In conclusion, the Igloo solution is not effective for wave periods of 12, 15, and 20 seconds.

PART III SUMMARY OF RESULTS AND OBSERVATIONS

The following table gives the qualitative results of the evaluation of the three major alternative West Beach Marina entrance configurations with respect to wave conditions and navigation.

Factor	Perpendicular Entrance	Parallel Entrance	Separate Entrance
Harbor Wave Conditions	Best	Good	Worst
Marina Wave Conditions	Best	Good	Acceptable
Navigation	Best	Good	Worst

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APPENDICES

A. Wave Attenuation as a Scale Effect

B. Modelling of Wave Absorbers

C. Detailed Description of Two-Dimensional Test Program

D. Data Quality Assurance Program

APPENDIX A

WAVE ATTENUATION AS A SCALE EFFECT

Wave attenuation is a phenomenon whereby the height of a wave is attenuated as it propagates. In the Laboratory, inside the harbor this effect is due to viscous damping and hence involves dissipation of wave energy. For most prototype considerations, the relative significance of viscous damping is generally negligible, however, at scale model dimensions the effect may be appreciable. This is particularly true in regard to areas of relatively shallow water depths such as the small craft harbor. Consequently, these small scale attenuation effects must be evaluated. Based on this, the test results can be adjusted accordingly.

In previous studies Keulegan (1950) and Eagleson and Dean (1966) showed that the scale correction factor is a function of water depth, wave period, water viscosity, distance of wave travel, and channel width. The basic equation for wave attenuation (after Keulegan) may be expressed in exponential form in the following manner:

$$\frac{H}{H_o} = e^{-\alpha x} \qquad (1)$$

H = wave height after travel distance x (ft)
(attenuated wave)
H_o = initial wave height (ft)
α = wave attenuation coefficient (1/ft)

$$\text{where } \alpha = k \frac{.2}{CB} \sqrt{\frac{\pi\nu}{T}} \left[\frac{L \sinh 2KD + 2\pi B}{L \sinh 2KD + 4\pi D} \right] \qquad (2)$$

k = experimental coefficient $\simeq$ 1.2 (dimensionless)
C = wave velocity (ft/sec)
B = channel width (ft)
ν = kinematic viscosity @ 75°F = 10^{-5} ft^2/sec
K = wave no. = $2\pi/L$ (1/ft)
D = water depth (ft)

The value of 1.2 for k was based on recommendations by Keulegan. It was decided to carry out computations for T_p = 6 sec, 10 sec, 20 sec, corresponding to periods in the model of: T_m = 0.6 sec, 1 sec, and 2 sec respectively. The

next step is to determine corresponding values of wavelengths for the respective depths and periods. Calculations of wave length are based on linear wave theory.

D (ft)	T(sec)	Wavelength L (ft) 0.6	1.0	2.0
0.1		1.01	1.75	3.56
0.2		1.35	2.43	5.01
0.3		1.55	2.91	6.12
0.4		1.67	3.29	7.02
0.5		1.74	3.59	7.86

T = 0.6 sec
Lo = 1.84 ft

T = 1 sec
Lo = 5.12 ft

T = 2 sec
Lo = 20.48 ft

Substituting values into equation (2) values for α can be obtained:

D (ft)	T(sec)	$(\alpha \times 10^{-3})$ (1/ft) 0.6	1.0	2.0	
0.1		23.92	19.30	13.82	
0.2		8.26	6.99	5.08	Harbor
0.3		4.42	3.93	2.87	
0.4		2.86	2.63	1.94	
0.5		2.09	1.96	1.43	
0.1		25.53	20.41	14.57	
0.2		9.61	7.83	5.62	Marina
0.3		5.72	4.65	3.31	
0.4		4.19	3.30	2.33	
0.5		3.43	2.60	1.78	

These values are plotted in the attached graph (Figure A-1).

Reading off the graph for different water depth corresponding to the channel, the deep draft harbor and the marina:

D = 0.41

T	$(\alpha \times 10^{-3})$
0.6	2.82
1.0	2.64
2.0	1.75

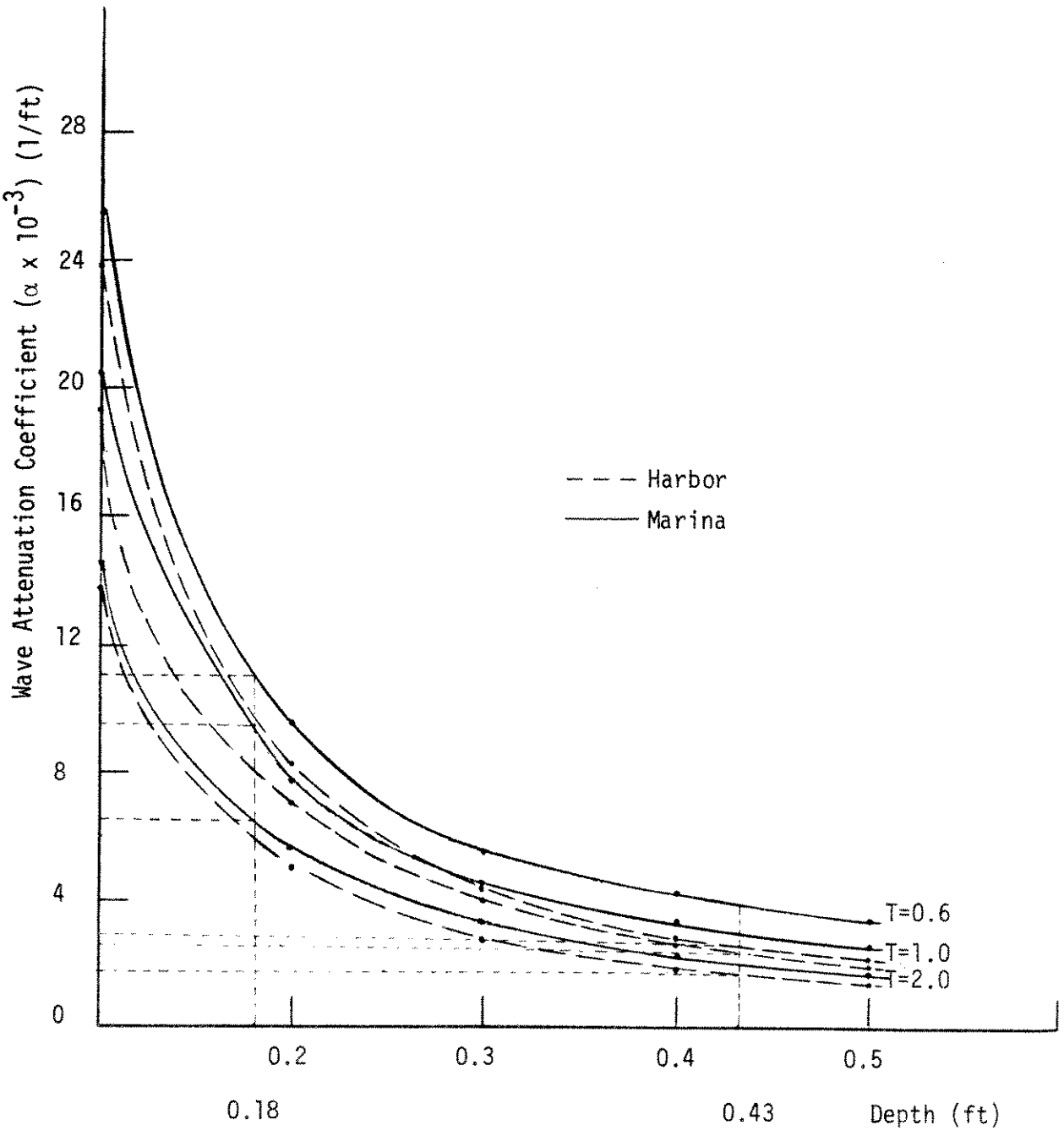

Fig. A-1 Wave Attenuation Coefficient vs. Depth as Function of Wave Period.

D = 0.43

T	$(\alpha \times 10^{-3})$
0.6	2.62
1.0	2.44
2.0	1.60

D = 0.18

T	$(\alpha \times 10^{-3})$
0.6	11.25
1.0	9.0
2.0	6.42

Utilizing the basic equation of wave attenuation the following scale correction factors are obtained:

Harbor: (x_m = 19, or a prototype location 1900 feet within the harbor)

T = 0.6 $\quad \frac{H_o}{H} = e^{(2.62 \times 10^{-3} \times 41)} \; e^{(2.82 \times 10^{-3} \times 19)} = 1.17$

T = 1.0 $\quad \frac{H_o}{H} = e^{(2.44 \times 10^{-3} \times 41)} \; e^{(2.64 \times 10^{-3} \times 19)} = 1.16$

T = 2.0 $\quad \frac{H_o}{H} = e^{(1.60 \times 10^{-3} \times 41)} \; e^{(1.75 \times 10^{-3} \times 19)} = 1.10$

Harbor and Marina (x_m = 12)

T = 0.6 $\quad \frac{H_o}{H} = e^{(2.62 \times 10^{-3} \times 41)} \; e^{(11.25 \times 10^{-3} \times 12)} = 1.27$

T = 1.0 $\quad \frac{H_o}{H} = e^{(2.44 \times 10^{-3} \times 41)} \; e^{(9.0 \times 10^{-3} \times 12)} = 1.23$

T = 2.0 $\quad \frac{H_o}{H} = e^{(1.60 \times 10^{-3} \times 41)} \; e^{(6.42 \times 10^{-3} \times 12)} = 1.15$

X_m= 19 (Marina) instead of 12, for the gage location furthest from the marina entrance

T = 0.6 $\quad \frac{H_o}{H} = 1.38$

T = 1.0 $\quad \frac{H_o}{H} = 1.31$

T = 2.0 $\quad \frac{H_o}{H} = 1.21$

Upon examination of these results it appears that some adjustment of the results obtained is necessary. However this applies far more so to the marina basin rather than for the deep draft harbor. In the final analysis an extremely conservative value of 1.4 as the average gage height multiplying factor for the marina was used. For the deep draft harbor a factor of 1.1 was considered the most realistic.

APPENDIX B

MODELLING OF WAVE ABSORBERS IN 1:100 SCALE MODEL

Selection of Stone Size

For this study reference is made to earlier theoretical and experimental work on wave-energy absorption. An important relevant source of reference is the experimental work carried out in the previous Barbers Point Harbor Study. Studies by the Beach Erosion Board (1949) were carried out for reflections of solitary waves. It was found that for a vertical permeable stone structure backed by an impermeable vertical wall the reflection coefficient decreased with increasing stone size. One of the most extensive series of experiments on wave absorbers, in particular rubble-mound revelments, has been carried out by Lé Mehauté (1965). The primary concern of this report is the problem of wave-absorbers in harbors. Pertaining to a sloping rock wall some of the following observations are noted:

a) The coefficient for roughness and permeability is only approximately determined by available experimental information;

b) The reflection coefficient decreases as the void coefficient increases, and depends only on the structure of the first three layers of stone wave absorber;

c) The size of stones, the slope, and the void coefficient are the most significant factors in wave absorption (the larger stones are better absorbers of wave energy);

d) The reflection coefficient on a scale model of a wave absorber is larger than in the field, therefore, the size of the stones must be increased to avoid these scale effects;

e) The best method for achieving similitude of reflection in a scale model, for the study of harbors agitation, is by experimentally adjusting the wave absorber in a two-dimensional wave tank. A preliminary experimental study at a larger scale will give the value of the reflection coefficient to be duplicated at a desired scale.

The procedure outlined in point e) was carried out in the previous Barbers Point Harbor study. For the three dimensional model study the scale 1:100 would be expected to exhibit scale effects. It was anticipated that stones larger than those prescribed by the model would be required to achieve

similitude in the reflection characteristics of the wave absorber. In the former study 1:33.3 scale tests were (previously) carried out, this scale considered to be large enough so as to accurately simulate the reflection characteristics of the prototype wave absorber. Stones corresponding to scales of 1:70 and 1:140 were tested, the results being compared to those recorded for the 1:33.3 scale tests. Very little difference in reflection characteristics was found between the two scale tests, though the 1:70 stones were associated with a void ratio more similar to that of the prototype. With this consideration in mind and recalling Le Méhauté's recommendation that a larger stone size should give better similitude in small tests, the 1:70 stones were chosen for the 1:100 test.

Acknowledging the results of this study it was deemed appropriate to approach the selection of stone size for modelling in the same manner with this study. Ultimately, the average stone size was chosen so as to be on the average 20% larger than the scaled down 1:100 size. This choice was somewhat arbitrary in that it corresponds at least to some extent with the availability of particular ranges of stone size.

Marina		
Type	Size	
	Model	Prototype
1	0.96"	5.5'
2	0.8"	4.5'
3	0.24"	1.5'
4	0.24"	1.5'

Deep Draft Harbor		
Type	Size	
	Model	Prototype
1	0.68"	4'
2	0.514"	3'
3	0.34"	2'

APPENDIX C

DESCRIPTION OF TWO-DIMENSIONAL TEST PROGRAM

These tests were carried out at a scale of 1:10. They were conducted in a large wave tank situated at the J.K.K. Look Laboratory of Oceanographic Engineering. The tank, having dimensions of 180 ft long x 4 ft wide x 4 ft deep was filled with water to a still water depth of 1.8 feet. The study involved the model construction and testing of four mitigating structures. Monochromatic waves were generated by a parabolically shaped plunger type wave generator.

Measurements:

For this study the following alternative wave mitigating structures were tested:

1. Vertical wall
2. 1:10 slope of randomly placed stone
3. 1:1.5 slope of randomly placed stone
4. IGLOO wave absorber block wall
5. Combination of vertical wall and single floating pier.

Capacitance type wave gages were placed at three stations along the channel, to measure the change in water level during testing. An initial test condition should be added to the above list. This initial test condition also served as the calibration of the wave generator. After drawing up the calibration curves, the various configurations were tested, data taken to measure the reflection response for each set-up.

Each configuration was tested for four different wave periods and at least two of four different wave heights. At the scale 1:10 the prototype

periods are changed by Froude Scaling from 6, 12, 15 and 20 seconds to 1.9, 3.8, 4.7 and 6.3 seconds respectively. In a likewise manner, although the scaling factor is somewhat different, waves of 0.1, 0.2, 0.3 and 0.4 are generated which correspond to waves of 1, 2, 3 and 4 feet respectively in the prototype. It was not deemed necessary to test every combination of wave height and period, reference being made to results of the three-dimensional test data.

Estimation of Reflection Coefficients:

Once the model was constructed the appropriate waves could be generated and measurements were recorded with the minimum of wave gage repositioning. This is as opposed to most historical tests of this nature whereby the technique employed is a tedious and indeed not very accurate test procedure. Previously, the average measured heights for two or three runs were taken, the objective being to determine the maximum (at the antinodes) and the minimum (at the nodes) of the partially standing waves. This would entail moving a wave gage along the flume for each run, the runs having to be stopped before the return of re-reflected waves by the paddle to the structure. Not only was this procedure laborious, but also the test waves were deficient in the spectral components of high frequency due to the wave dispersion unless a special wave generation technique was employed.

However, the method used in the present study, which was initially presented by Goda and Suzuki (1976), overcomes the multiple reflection of waves between a wave paddle and the reflective structure.

Methodology:

Using the method described in the principles of the resolution technique, a computer program was developed to calculate the overall coefficient of reflection. The input to the computer program comprised the following: the water depth, the wave period, the separation between the gages and the surface elevation wave gage measurements. As previously noted, the water depth is fixed at 1.8 feet and the tests are run for four different wave periods. The separation between the wave gages is controlled by constraints in the technique. In short only one gage is moved, this being gage 2 with reference to Figure 16. The other two gages are positioned in such a way that one is immediately in front of the structure being tested while the other is reasonably close to the generating mechanism. Table C-1 shows how the separations between gages 1 and 2 in the model are determined. Table C-2 illustrates the spacing used:

Table C-2 Wave Gage Spacing Selected for Different Wave Periods

Wave Period (sec)	Gage Spacing (ft)
1.9	3.24
3.8	7.04
4.7	8.79
6.3	1.85

Table C-1 Separations Between Gages in Two-Dimensional Model (ft)

DEPTH (Ft)	1.00	1.10	1.20	1.30	1.40	1.50	1.60	1.70	1.80
PERIOD (sec)									
1.00	1.13	1.16	1.18	1.20	1.21	1.23	1.24	1.25	1.25
1.10	1.30	1.33	1.36	1.39	1.41	1.44	1.45	1.47	1.48
1.20	1.48	1.51	1.55	1.58	1.62	1.64	1.67	1.69	1.71
1.30	1.62	1.68	1.71	1.77	1.81	1.85	1.88	1.91	1.94
1.40	1.78	1.84	1.90	1.96	2.00	2.05	2.09	2.13	2.17
1.50	1.93	2.01	2.07	2.14	2.19	2.25	2.30	2.34	2.39
1.60	2.09	2.17	2.25	2.32	2.38	2.44	2.50	2.56	2.60
1.70	2.24	2.33	2.41	2.49	2.57	2.64	2.70	2.76	2.82
1.80	2.39	2.49	2.58	2.67	2.75	2.83	2.90	2.97	3.03
1.90	2.54	2.65	2.75	2.84	2.93	3.02	3.10	3.17	3.24
2.00	2.69	2.81	2.91	3.02	3.11	3.20	3.29	3.37	3.45
2.10	2.84	2.96	3.08	3.19	3.29	3.39	3.49	3.57	3.66
2.20	2.99	3.12	3.24	3.36	3.47	3.58	3.68	3.77	3.86
2.30	3.13	3.27	3.41	3.53	3.65	3.76	3.87	3.97	4.07
2.40	3.28	3.43	3.57	3.70	3.82	3.94	4.06	4.17	4.27
2.50	3.43	3.58	3.73	3.87	4.00	4.13	4.25	4.36	4.47
2.60	3.57	3.74	3.89	4.04	4.18	4.31	4.44	4.56	4.67
2.70	3.72	3.89	4.05	4.20	4.35	4.49	4.62	4.75	4.88
2.80	3.87	4.04	4.21	4.37	4.52	4.67	4.81	4.95	5.07
2.90	4.01	4.20	4.37	4.54	4.70	4.85	5.00	5.14	5.27
3.00	4.16	4.35	4.53	4.71	4.87	5.03	5.18	5.33	5.47
3.10	4.30	4.50	4.69	4.87	5.04	5.21	5.37	5.52	5.67
3.20	4.45	4.65	4.85	5.04	5.22	5.39	5.55	5.71	5.87
3.30	4.59	4.80	5.01	5.20	5.39	5.57	5.74	5.90	6.06
3.40	4.73	4.96	5.17	5.37	5.56	5.75	5.92	6.09	6.26
3.50	4.88	5.11	5.33	5.53	5.73	5.92	6.11	6.29	6.46
3.60	5.02	5.26	5.48	5.70	5.90	6.10	6.29	6.48	6.65
3.70	5.17	5.41	5.64	5.86	6.08	6.28	6.48	6.66	6.85
3.80	5.31	5.56	5.80	6.03	6.25	6.46	6.66	6.85	7.04
3.90	5.45	5.71	5.96	6.19	6.42	6.63	6.84	7.04	7.24
4.00	5.60	5.86	6.12	6.36	6.59	6.81	7.03	7.23	7.43
4.10	5.74	6.01	6.27	6.52	6.76	6.99	7.21	7.42	7.63
4.20	5.88	6.16	6.43	6.69	6.93	7.16	7.39	7.61	7.82
4.30	6.03	6.32	6.59	6.85	7.10	7.34	7.57	7.80	8.02
4.40	6.17	6.47	6.75	7.01	7.27	7.52	7.76	7.99	8.21
4.50	6.31	6.62	6.90	7.18	7.44	7.69	7.94	8.17	8.40
4.60	6.46	6.77	7.06	7.34	7.61	7.87	8.12	8.36	8.60
4.70	6.60	6.92	7.22	7.51	7.78	8.05	8.30	8.55	8.79
4.80	6.74	7.07	7.37	7.67	7.95	8.22	8.48	8.74	8.98
4.90	6.89	7.22	7.53	7.83	8.12	8.40	8.67	8.93	9.18
5.00	7.03	7.37	7.69	8.00	8.29	8.57	8.85	9.11	9.37

For various incident wave conditions the corresponding wave length is computed and all the amplitudes of Fourier componenets were analyzed using the Fast Fourier Transform (FFT) technique. Hence, from this, the amplitude of the frequency is obtained. Finally the overall coefficient of reflection could be computed as the ratio of the reflected to the incident wave energy integrated over the whole range of frequencies.

All tests proceeded smoothly except for the 1:1.5 slope randomly placed stone structure since the structure was not stable enough to resist the incident waves. For the 1:10 slope wave absorber the scale effects prevalent in the three-dimensional 1:100 scale analysis were considered negligible. Ultimately stone corresponding to an average diameter of 2.0 inches was utilized in the structure for the outer layer. Cinder having an average diameter of 0.5" was used as a gravel bedding layer. For the outer layer of the absorber the stone size corresponds to ratio of 1:9, model to prototype. The cinder bedding layer is at a somewhat smaller ratio still. The intention being that the model would perhaps better relate to the prototype at this very slightly distorted scale. This is especially true when one considers that the test stones are not uniform in size, the distorted scale should give a better void ratio comparison.

Principle of Resolution Technique (Goda and Suzuki, 1976)

In reality, there is a multi-wave-reflection system of monochromatic waves in the flume where the experiment is being carried out. Waves initially generated by the wave plunger propagate forward in the flume and are reflected by the test structure. The reflected waves propagate back to the wave plunger and are re-reflected. The re-reflected waves propagate forward again and the process is repeated until the multi-reflected waves are fully attenuated. Then the wave system can be regarded as a superposition of a number of waves

propagating in two opposite directions. It can be demonstrated that the waves propagating in each direction form a single train of progressive waves as a whole, because of the constancy of the wave frequency. If we call the amplitude of incident waves on the structure a_I and that of the reflected waves a_R, these waves can be described in a general form as:

$$\eta_I = a_I \cos(Kx-\omega t-\varepsilon_I) \tag{1}$$

$$\eta_R = a_R \cos(Kx+\omega t-\varepsilon_R) \tag{2}$$

where η_I and η_R are the surface elevations of incident and reflected waves, $K = \frac{2\pi}{L}$ is the wave number with L being the wavelength, $\omega = \frac{2\pi}{T}$ is the angular frequency with T being the wave period, and ε_I and ε_R are the phase angles of incident and reflected waves. In the experiment the surface elevations are recorded at two adjacent locations, namely X_1 and X_2. Thus the observed profiles of composite waves will be:

$$\eta_1 = (\eta_I + \eta_R)_{X_1} = A_1\cos\omega t + B_1\sin\omega t \tag{3}$$

$$\eta_2 = (\eta_I + \eta_R)_{X_2} = A_2\cos\omega t + B_2\sin\omega t \tag{4}$$

where,

$$A_1 = a_I \cos\phi_I + a_R \cos\phi_R \tag{5}$$

$$B_1 = a_I \sin\phi_I - a_R \sin\phi_R \tag{6}$$

$$A_2 = a_I \cos(K\Delta\ell + \phi_I) + a_R \cos(K\Delta\ell + \phi_R) \tag{7}$$

$$B_2 = a_I \sin(K\Delta\ell + \phi_I) - a_R \sin(K\Delta\ell + \phi_R) \tag{8}$$

$$\phi_I = KX_1 + \varepsilon_I \tag{9}$$

$$\phi_R = KX_1 + \varepsilon_R \tag{10}$$

$$\Delta\ell = \text{spacing between two more gages} \tag{11}$$

Equations 3, 4, 5 and 6 together with linear dispersion relationship can be solved to yield the following results:

$$a_I = \frac{1}{2|\sin(K\Delta\ell)|} \quad (A_2 - A_1\cos(K\Delta\ell) - B_1\sin(K\Delta\ell))^2 + (B_2 + A_1\sin(K\Delta\ell) - B_1\cos(K\Delta\ell)) \quad (12)$$

$$a_R = \frac{1}{2|\sin(K\Delta\ell)|} \quad (A_2 - A_1\cos(K\Delta\ell) + B_1\sin(K\Delta\ell))^2 + (B_2 - A_1\sin(K\Delta\ell) - B_1\cos(K\Delta\ell)) \quad (13)$$

The values of A_1, A_2, B_1 and B_2 for the fundamental frequency as well as for higher harmonics are estimated by using a classical Fourier Analysis. Thus, the amplitudes of incident and reflected waves, a_I and a_R are readily estimated by using equations 12 and 13. Equations 12 and 13 show that the amplitude of a_I and a_R diverge around the relative space of $K\Delta\ell = \pi\eta$ with $\eta = 0, 1, 2, \ldots$. During the experiments, the values of separation between gages, $\Delta\ell$, were selected so that $\sin(K\Delta\ell) = 1$, since this is the region where equations 12 and 13 are most stable.

Accuracy and Overall Resolution

An additional advantage of the present technique with respect to the conventional method is its effectiveness in detecting the enhancement of harmonic components of waves through the interaction with the reflective structure. This effect is clearly observable from the results obtained for the case of 1:10 slope where second and higher harmonics are enhanced. The actual generation of harmonics is a direct consequence of the energy radiated by the incoming waves which occurs right before and during the breaking process on the slope. Second and higher harmonics appeared in the incident waves are mainly associated with the finite amplitude of the waves. This effect can be seen from the results since for waves of larger heights of the same period, the energy contained in higher harmonics increases. A minor part of the energy contained in those harmonics may be associated with the re-reflection by the plunger of harmonics previously generated at the reflective structure.

The main source of inaccuracy in the present resolution technique is that coming from the existence of nonlinear harmonic terms in progressive waves. Waves of finite amplitude are accompanied with the harmonics which propagate with the same celerity of the fundamental frequency. For such harmonic components the linear dispersion relationship does not apply and some errors may be evoked in the estimation of a_I and a_R since the degree of resolution is based on the relative phase difference of $K\Delta\ell$.

It should be mentioned at this point that the conventional method of estimating the reflection coefficient with the observed heights of maxima and minima of partial standing wave systems is much more susceptible to the nonlinear effects. The conventional method is only for the fundamental frequency components of small amplitude waves, and therefore, it can not detect other associated components such as the harmonics, but it is affected by them instead.

The accuracy in the measurement of the overall reflection by using the present resolution technique for the case of progressive waves and partially standing waves has been reported to be ± 5% (Goda and Suzuki, 1976). However, most of this inaccuracy is due to the variability of waves in the laboratory flume, and the error involved in the resolution technique itself is considered to be smaller.

APPENDIX D

DATA QUALITY ASSURANCE PROGRAM

Model Construction.

The model bottom contours were carefully layed out with all elevations referred to ± 0 MLLW and then marked with steel strips with a precision survey level. Finally, the model was carefully constructed to ensure accurate reproduction of the prototype bottom condition. After concrete was cast, the surface was refinished to avoid undesirable topography deviations.

Wave Generation Positions.

The wave generator positions were set according to wave conditions simulated. The orientation of the wave generator for each wave direction and period was based on wave refraction diagrams. Special care was taken to set each position the same for various marina-harbor test plans and test conditions.

Wave Generator Calibration.

Wave generators were calibrated for the test conditions so that wave period and height are accurately related to the settings of motor speed and plunger stroke for each condition. The calibration curves for both Three-Dimensional and Two-Dimensional Models are shown in Figs. D-1 and D-2, respectively.

Wave Gage Calibration.

Wave gages of float type and capacitance type were carefully selected for appropriate depths to avoid undesirable errors due to wetting effect of the capacitance gages. This is particularly important for model wave periods shorter than 8 seconds. The automatic wave gage calibration device enabled an accurate and speedy calibration. The orientation of float-type wave gages relative to wave direction was important to obtain accurate results. Also wave heights were measured and compared with capacitance-type probe and float sensor in the barge harbor. The comparison turned out very well. This may be due to the depth of the barge harbor being relatively deep.

Repeatability of Tests.

The repeatability tests were carefully done for critical wave conditions in order to ensure good data quality. The test results were good (see Table D-1).

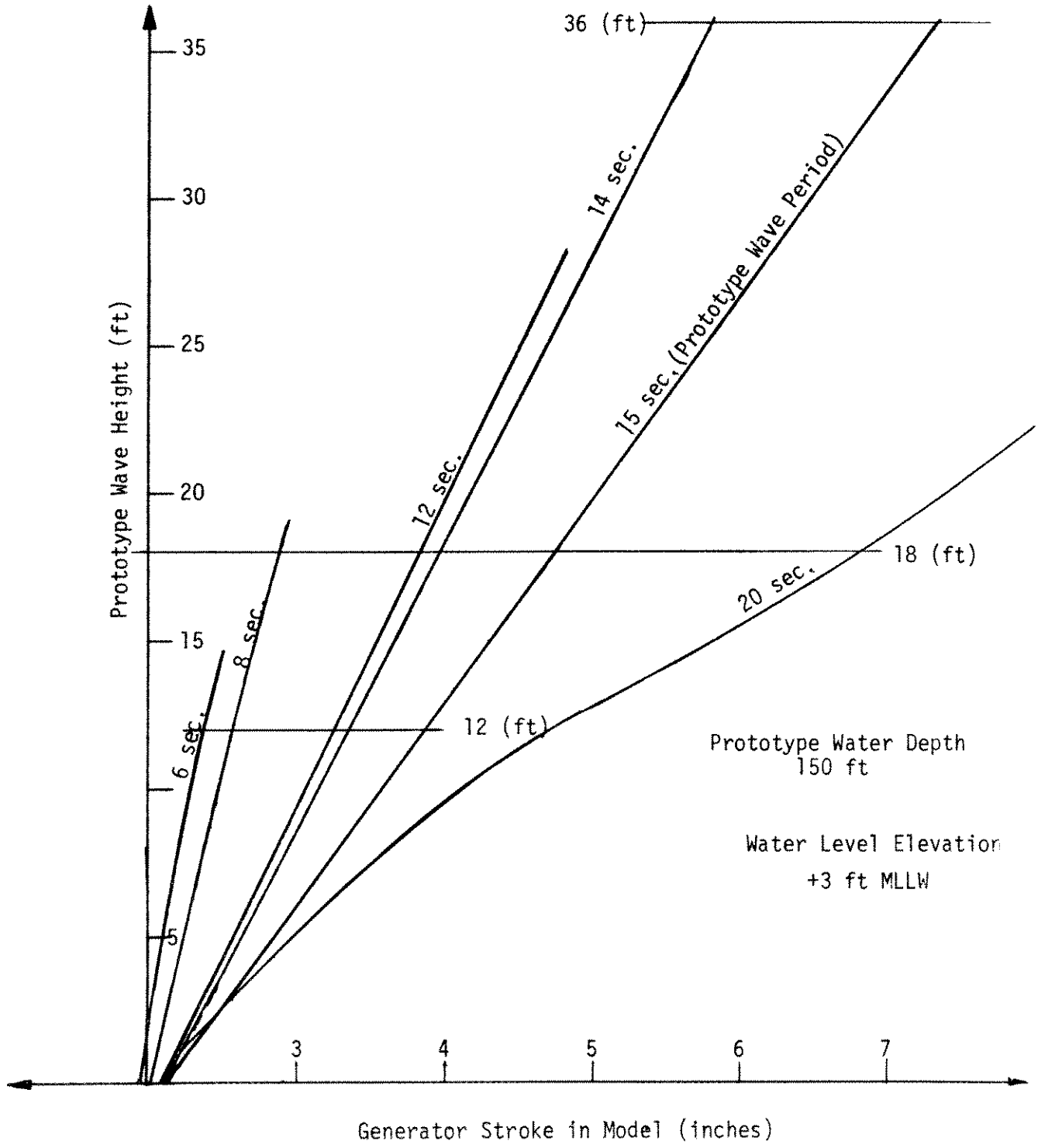

Fig. D-1 Calibration Curve of Wave Generator for the Three-Dimensional Model (1:100).

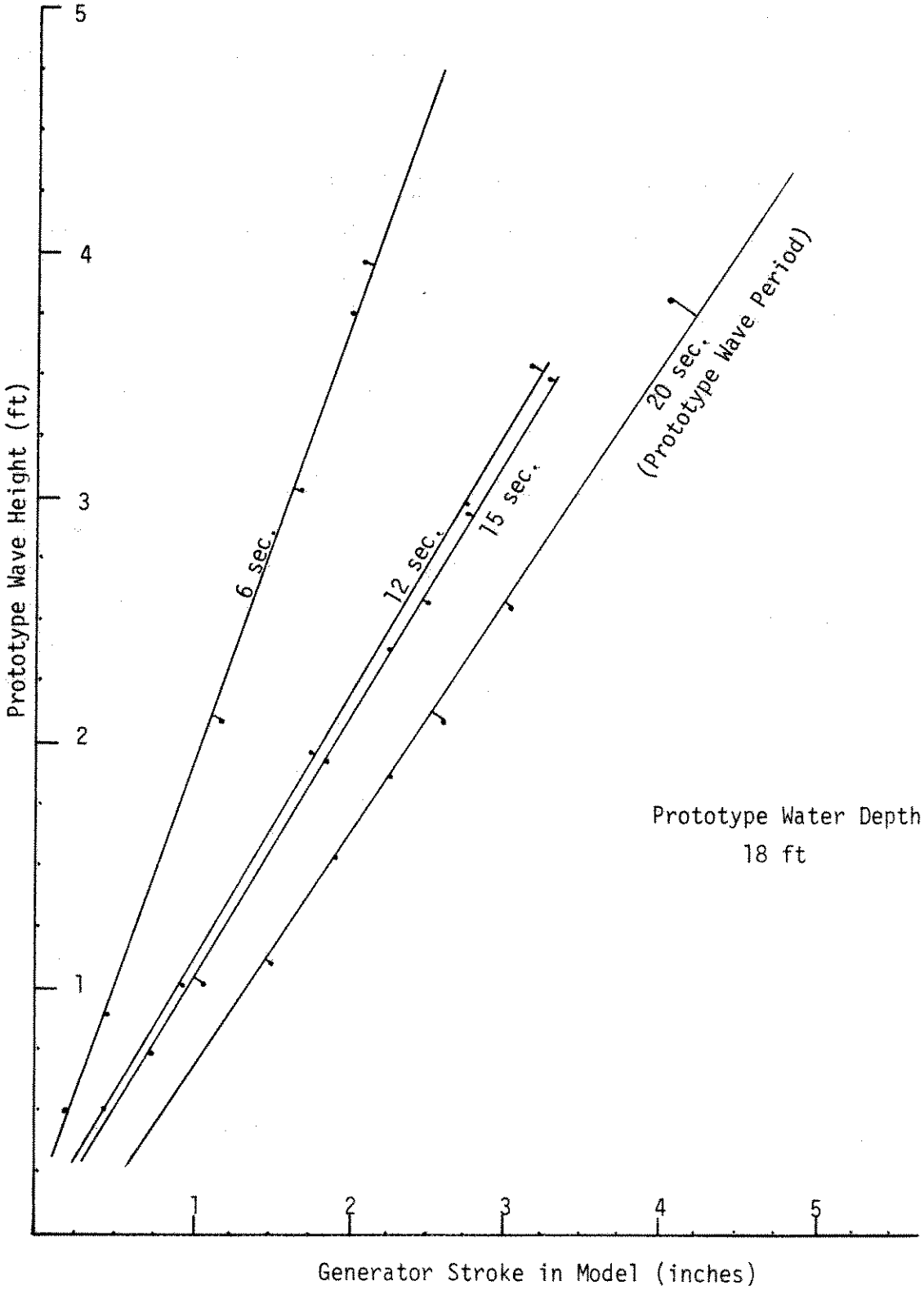

Fig. D-2 Calibration Curve of Wave Generator for the Two-Dimensional Model (1:10)

During these repeatability tests it was found that the rock transition slope from -60 ft MLLW to -150 ft MLLW and the generator functioned well; no undesirable reflection was absorbed. This was perhaps due to the shallow reefs beside the navigation channel providing natural wave absorbers.

Wave Heights in Deep Draft Harbor and Barge Harbor.

It is interesting to note that the waves in both deep draft harbor and barge harbor were amplified for certain wave periods (6 or 12 seconds). The amplification due to resonance phenomena is clearly shown in Fig. D-3. This finding indicates the need to analyze the wave data with sufficient duration of measurements; otherwise the results for very short duration could be misleading.

Selection of Length of Measured Wave Data for Analysis.

A total of approximately 3 minutes of wave data was recorded which is equivalent to 4000 data points of 1/25 sec time intervals. It would be too time consuming to analyze the entire record for the time allowed. Therefore, a special investigation was made to compare the results obtained with 1000-2000, 2000-3000, 3000-4000 data point ranges. A comparison of the results showed the data point range of 2000-3000 was more reasonable and conservative. Such comparisons were made for wave gages 5, 6, 7, 8, 9, where wave heights at the berthing areas of the deep draft harbor and barge harbor were measured. The typical results for gage 5 are shown in Fig. D-4. As indicated in the previous paragraph, the result would not miss the maximum wave height due to harbor oscillation. The data point range of 0-1000 was not considered because the waves have not reached the harbor or have not stabilized.

Comparison of PLAN 0 Tests with Previous Barbers Point Deep Draft Harbor Tests.

Attempt was made to compare PLAN 0 (present tests) with PLAN V-A from the previous Barbers Point Deep Draft Harbor Tests. It was impossible for the following reasons:

(a) test conditions are different
(b) the previous deep draft harbor with marina extended into the Barge Harbor
(c) the present deep draft harbor design has no marina

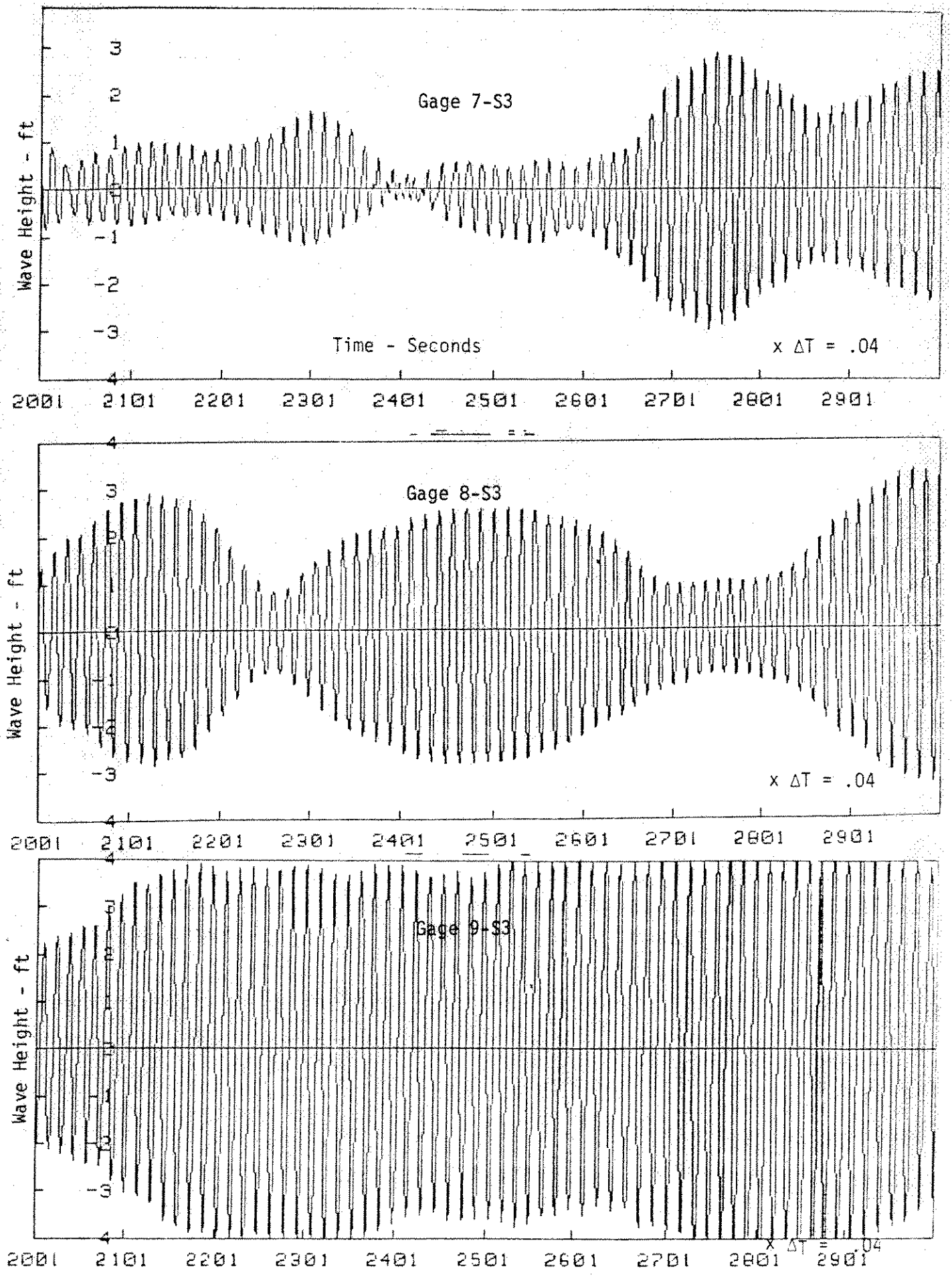

Fig. D-3 Wave Height at Gages 7, 8 and 9 (6 Seconds,18 ft, S45W)

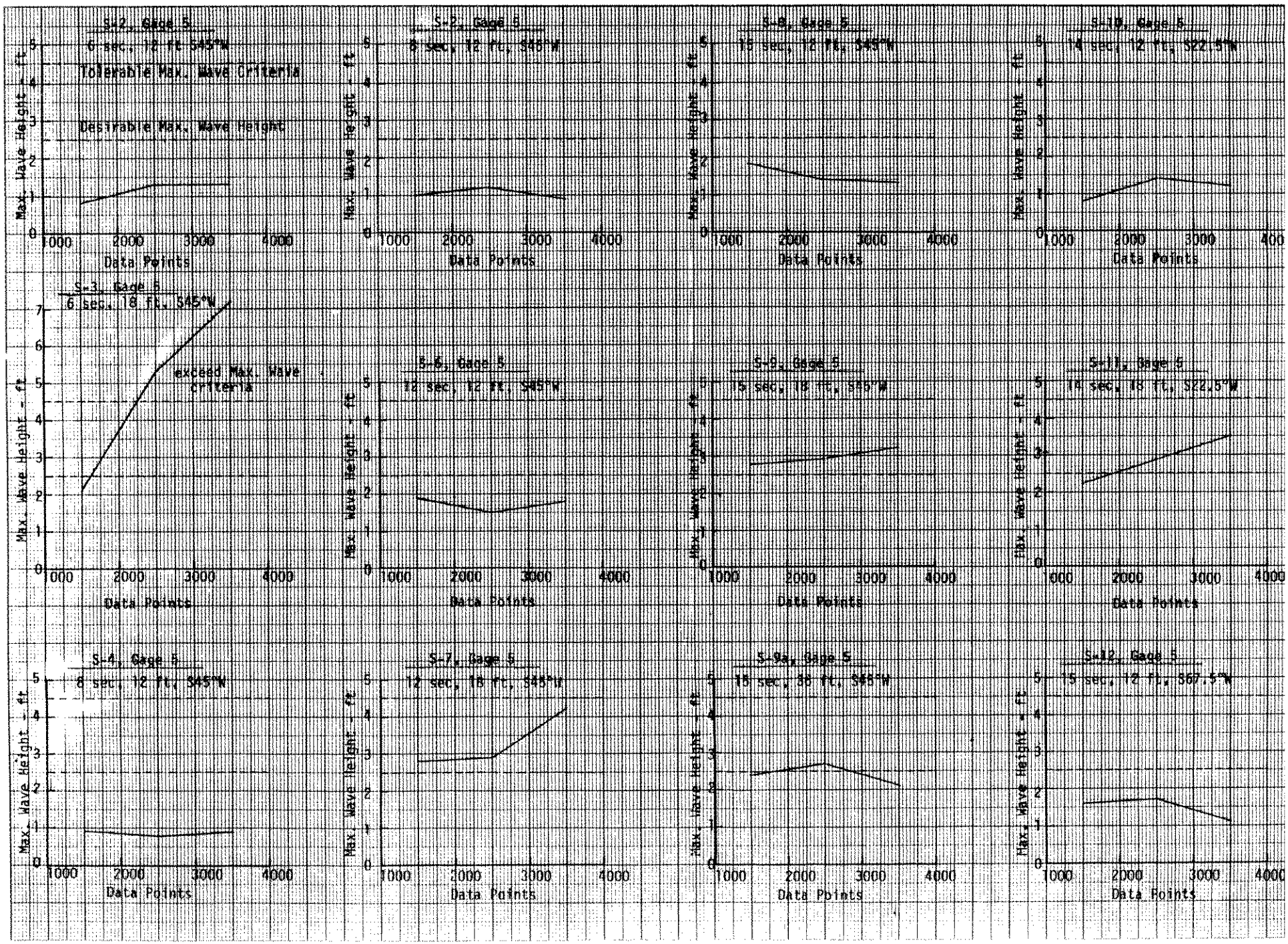

Fig. D- Comparison of Maximum Wave Heights for 1000-2000, 2000-3000 and 3000-4000 Data Ranges

(d) the previous berthing area involved a sloped wall while the present design has vertical walls. However, most of maximum wave heights did not exceed the tolerable limit.

(e) the method of analysis is different, i.e., first 10 waves vs. a longer data record to include effect of harbor oscillation of shorter period.

Wave Attenuation.

The wave heights measured in the small 1:100 scale model are subject to greater attenuation, than the prototype, as a function of water depth, wave period, water viscosity, distance of the wave travel, and channel width. Therefore, model wave heights must be corrected accordingly. The procedure is described in detail in Appendix A. The correction factors for deep draft harbor and marina were selected as 1.1 and 1.4 respectively. This agrees with the calculations for previous tests by Harrenstien and Palmer (1970).

Wave Absorbers Scale Effects.

The sizes of rubble-mound wave absorbers were carefully selected to minimize the scale effects. The procedure is described in detail in Appendix B. The wave characteristics of the Igloo wave absorbers in the 1:100 three-dimensional scale model would not yield quantitative results. However, qualitative results were obtained.

Data Acquisition and Analysis.

To make sure the test results are reasonable, data analysis was made immediately after each test series.

Consultation Meetings.

Periodic meetings between the investigators and the contractor's technical representative and their consultants were held. The technical personnel of the U.S. Army Corps of Engineers Pacific Division and the State Harbor Division were kept informed of the progress of the tests and analyses at various stages. These meetings were very helpful.

Data Quality Assurance Program Administration.

The principal investigator was charged with the responsibility of the data quality assurance program. The final report was reviewed by pertinent personnel.

Long Period Oscillation in the Marina

The long period oscillation phenomenon is quite important particularly if the oscillation period is in resonance with the natural period of the marina or harbor basin.

An effort was made to determine the period of the marina basin oscillation at the berthing areas based on wave time - series. It was found that the long period oscillation depended on the incident wave period, height and direction, as well as on the location in the marina berthing area. In addition, the configuration of the marina entrance had an effect. The results of this evaluation are shown in Table D-1. It was found that Plan V had the least long-period oscillation followed by Plan I and Plan III. The period of oscillation varied from 110 to 120 seconds for gage #4, and 100 to 150 seconds for gage #5 for Plan I. For Plan III, the range was from 90 to 225 seconds for gage #4 and from 90 to 155 seconds for gage #5. For Plan V the range was from 90 to 240 seconds for gage #4 and 120 to 140 seconds for gage #5.

TABLE D-1

Long Period Oscillation Inside the Marina

Direction	Incident Wave		Oscillation Period (seconds)					
	Wave Period (seconds)	Wave Height (ft.)	Plan I		Plan III		Plan V	
			Gage #4	Gage #5	Gage #4	Gage #5	Gage #4	Gage #5
S45W	6	12	N	100	N	120	N	120
	6	18	112	120	N	130	N	140
	8	12	-	-	N	144	N	120
	8	18	120	140	N	144	180	120
	12	12	120	140	N	144	N	N
	12	18	120	140	N	144	120	N
	15	12	120	140	N	150	N	N
	15	18	N	N	N	150	240	N
	15	36	N	120	N	150	240	N
S22.5W	14	12	N	130	N	N	N	N
	14	18	N	130	N	155	N	N
	14	36	N	130	N	120	N	N
S67.5W	15	12	N	N	225	N	N	N
	15	18	N	N	N	150	N	N
	15	36	N	150	180	150	N	N
W	12	12	N	N	N	144	N	N
	12	18	N	N	N	N	N	N
	15	12	N	N	N	150	N	N
	15	18	120	130	90	150	210	N
	20	12	N	130	90	90	N	140
	20	18	N	130	N	160	N	N
	15	36	N	130	165	135	90	N

Note: N - no significant oscillation

SUMMARY OF TECHNICAL INPUT

FOR THE

WEST BEACH LAGOONS AND MARINA DESIGN DEVELOPMENT

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INTRODUCTION.

This technical report was developed by Scientific Environmental Analyses Ltd. (S.E.A. Ltd.) for Herbert K. Horita Realty Inc. (West Beach Estates) and coordinated through Alfred A. Yee Division, Leo A. Daly. The tasks contracted for this technical support are specified in a contract letter dated February 1, 1983 and signed June 6, 1983. The objective of this phase of the work were to developing specifics of the lagoon and marina design for inclusion into the West Beach Development project and to provide E.I.S. supporting information.

This report is a summary of technical guidance given Alfred A. Yee Division/LAD during the period from June 1983 to July 1984. This information, added to other consultant inputs, aided AAYD/LAD in their development of lagoon designs and a marina concept for the West Beach project.

Specifically S.E.A. Ltd. agreed to support the project effort with the following design-technical verification work over a thirty-one week period.

Lagoons:

1. Existing Kamokila Lagoon-profile bathymetry, monitor range of water quality existing, estimate existing rate of exchange, examine biota

in the lagoon, examine sediment character, estimate probable secular changes in the existing lagoon, estimate infiltration.

2. Examine site coastline survey data on substructure, wave activity, sand, basalt and coral resource and character, coastal circulation and meterology, determine infiltration estimates.

3. Aid AAYD specifications and limits for lagoon construction, determine operating limits and monitoring standards, aid in proposing control measures/mechanisms to maintain quality, estimate climatic seasonal and diurnal limits of change, suggest elements of long term management.

4. Aid in developing construction specifications involving armor, beach lining, lagoon bottom and subbottom, construction techniques to minimize construction impact and initial lagoon environmental conditions.

5. Monitor the development by AAYD of specifications and drawings.

6. Provide coordination of results with development of the supplemental environmental impact statement and project permits (not including hearings).

Marina:

1. Review field data on the marina site on subbottom character, infiltration rates, material composition, meterology, coastal currents at entrance, wave conditions, tides, coastal character.

2. Aid in development of the marina configuration and hypsography, entrance channel configuration, slip orientation.

3. Estimate the infiltration rate, marina water quality, flushing and circulation, seiche behavior, wind and wave environments in the marina.

4. Provide assistance in the specifications and selection of construction techniques for the entrance and marina boundries.

5. Aid in the pre-development of a hydraulic model of the marina for circulation-flushing, wave climate and surge verification for the selected design.

6. Monitor the development by AAYD of specifications and drawings.

Other coordinated consulting help was provided for the lagoons (waves and surge environment, coastal protection, littoral drift) and the marina (general design, wave and surge environments, entrance configuration, tsunami). Field work was contracted out by H.K. Horita (subbottom character, infiltration, meterological data, wave data, circulation data, tidal data)

and monitored by S.E.A. Ltd. in support of this work. S.E.A. Ltd. also conducted additional field observations throughout the project duration. It should also be noted that during the duration of the work and in coordinance with the needs of AAYD and the other consultants on the project, several additional analyses were completed beyond those listed above as obligated.

An attempt was made in planning this work to recognize and list the potentially major problem areas that could result from including four coastal lagoons and a marina in the West Beach development. Technical guidance provided earlier for the concepting of the lagoons showed that important site specific data were missing, particularly relating to ground water infiltration rates and quality, and coastal wave climate. Related historical data was also used to complete some analyses and estimates. A subsequent construction guidance phase is projected to include additional site specific lagoon and marina field data. Until that time the results given here must be considered approximate, and conservative as intended.

The results of this report are arranged in the following order:

1. Comments on the available data;
2. List of technical analyses completed covering each contracted subject for the design of and estimating the anticipated environmental conditions in the lagoons and marina.
3. A list of recommendations and comments, in simplified lay terms, pertinent to the development of the lagoons and marina.

4. A list of data sources used for this work.

All detailed technical results were provided to A. A. Yee Division/LAD in technical meetings and discussions over the project duration. The resulting details and recommendations are briefly discussed in this final report.

LIST OF DATA SOURCES

The results and references used in the reports titled Summary Of Technical Input For The West Beach Lagoon And Marina Concept Development, S.E.A. Ltd., April and October 1979, provided the starting point for this work. Additional pertinent material was assembled from the field study reports of Sea Engineering and Harding Lawson Associates, from other project consultants (Gerritsen, Bretschneider, Dugan and Bienfang), from S.E.A. Ltd. field data and from the literature available on several added subjects of interest - primarily ground water characteristics and behavior, biota in the site vicinity, and recent environmental impact information for the intended deep draft Barbers Point Harbor extension.

These past reports contain lists and annodated descriptions of the data sources assembled and reviewed for this work, plus copies of the past reports of other project consultants. These past references and consultant reports were omitted in the bibliography given here for brevity. Twenty two sources were checked, one hundred fifteen reports reviewed and six unpublished data inputs considered.

COMMENTS ON THE DATA

An attempt was made at the start of this work to update past data both with information published since October 1979 and with site specif new field work contracted for the development of the lagoon and marina design effort. Specific emphasis was spent finding recent material pertinent to the sixteen subjects of interest and responsibility given in the table of contents. In addition, new site nearshore bathymetry and land topograghic details for the lagoon area were provided by Community Planning Inc. via A.A. Yee Division/LAD. Lagoon and marina site specific boring data were taken, along with coastal wave data and current data for approximately ten months.

A large amount of material was assembled. In general, the comprehensiveness of the material did allow each subject to be examined in sufficient detail to provide technical guidance for the development of design details and for the identification of key design criteria.

Other inputs to AAYD/LAD influencing the design effort included developer preference, land planning needs, cost and economic considerations, and environmental constraints. In all the resultant designs for the lagoons and marina attempted to meet these needs.

One element of the marina concept, i.e. the entrance configuration, is related to the proposed Barber's Point deep draft harbor. Some specifics of the entrance configuration remain to be coordinated with the Deptartment

of Transportation and therefore the marina entrance may change in the follow on construction guidance phase. The references used for this work have considered the deep draft harbor as existing in the manner given in the harbor E.I.S. and as tested in the physical model underway at this writing.

Estimates of neighboring pollutant sources and feedbacks necessitate using historical estimates for the Barbers Point Harbor or historical observations from other Hawaiian harbors. The eventual performance of the Barbers Point Harbor and the neighboring Conoco refinery waste discharges could, over time, prove to be different than the existing data used.

With the exceptions noted above, sufficient new field data and historical information was available to allow several analyses to be completed. The work helped establish design criteria and selection of the optimum manner in which the lagoons should be constructed. The suggested design constraints developed in this work were based upon conservative limits for lagoon performance. This was to insure, at this early design stage, that the lagoon configurations developed considered all key factors leading to the desired high level of lagoon water quality and asthetics.

SUMMARY OF RESULTS

LAGOONS

Existing Kamokila Lagoons

A detailed examination of three natural coastal lagoons existing at the site was completed. These lagoons include a northern most semi circular 0.97 to 1.16 acre (a function of tidal height) lagoon, and 4.3 ft. average depth. The lagoon is sand lined including a wide sand beach, and has a basalt and coral barrier, with breaks, across the entrance. As such it closely resembles the intended four lagoons to be added along the site coastline and was therefore closely examined to determine how the lagoon successfully maintains its high asthetic quality year-round.

The remaining two existing Kamokila lagoons are south of and adjacent to the northern most lagoon (referred to locally as Paradise Cove). These two are smaller, 0.35 and 0.45 acre (approximately 0.2 and 0.45 million gallons) and shoaler (1.9 ft. and 3.6 ft. average depths). The southern most lagoon is primarily open to the ocean and receives incoming wave activity directly on the sand beach.

The Paradise Cove hypsography shows that a considerable amount of daily flushing occurs via the active introduction of coastal water by incoming waves topping a shoal reef shelf at the entrance. Water exits this lagoon principally via a 7 ft. to 9 ft. deep channel located toward the seaward central area of the lagoon entrance.

A consistent seaward flow of 10 to 25 cm/sec was observed in this channel below the 2 foot depth. Circulation in this lagoon enters both ends and over the shallow 2 foot deep northern third of the entrance and exits via the deeper central channel.

The sand lining the lagoon is graded from coarse cobble and small bolders in the deeper central area to medium coarse sand with a 6° to 12° slope at the water line, to medium and fine sand on the beach berm and back slope. Wave forms in the predominently sand lined bottom indicate active incoming wave activity strikes the beach, sorting the lagoon lining materials for maximum stability.

Field observations taken showed that at incoming 2 foot swell and wind chop the northern most lagoon was flushing approximately 4 times the lagoon volume per day. Incoming waves during the days of observation were found to account for a flushing volume up to 12.7 to 1 over the daily tidal prism. The maximum flushing rate was computed as 23 times/day with approximately 4 foot incoming waves of 8 to 10 second period. Exiting velocities in the central channel varied from 10 to 60 cm/sec (0.2 to 1.2 kts).

Clearly the high quality of the existing lagoons is related to the high quality of incoming coastal water, and active wave and tidal flushing mechanism, the beach stability and the absense of prolific biota even in the presense of nutrient laden infiltrating water (later estimated at 0.1 to 10.0 ft./day local permeability). Later estimates showed that

up to one tenth the lagoon volume of ground water from the surrounding area likely infiltrates daily. This brackish water of approximately 1.0 to 24.0 $^{o}/_{oo}$ mixes with the incoming coastal water and flushes from the lagoon daily. The lagoon salinity exhibits only a slight 1 $^{o}/_{oo}$ drop in salinity again indicating a working and active natrual flushing mechanism.

The developer intent in adding four larger lagoons along the project coastline is to duplicate the performance of the existing lagoons as much as possible. The subsequent design developed intended to accomplish that goal, recognizing that extreme storm events of adversed seasonal conditions may prevail when the lagoons will not be asthetically optimum.

Nearshore Circulation Patterns

The nearshore area is defined here as extending from the coastline seaward to the 100-foot depth. Offshore eddies affecting both the nearshore flow strengths and directions, are believed to exist off Barbers Point. They vary between 2 nm to 20 nm in size, appear to move anticyclonic (clockwise) during November to February, cyclonic (anti-clockwise) during April to September, and move in variable patterns for the remaining months.

At the coastline, tidal flooding flows move west to northwest and ebbing flows south to south southeast. The maximum observed ebb strength was 65 cm/sec. (1.3 kts) and maximum flood strength was 75 cm/sec (1.4 kts). The currents reverse direction with each tidal change and generally range in strength between 10 to 50 cm/sec (0.2-1.0 kts). Currents moving approximately parallel and slightly offshore at speeds of 40 to 60 cm/sec (0.8-1.1 kts) are most often observed.

The maximum current believed to exist in the area offshore of the site shoreline is 1.5 kts during calm wind conditions and 2.0 during Kona storm conditions.

The typical net daily flows move water southward along the coastline during August to April. This net flow is strong (>7nm/day) and directionally consistent during August to January, but directionally variable during February to April. During May to July the net flow moves northwest, and is weak in strength (< 2 1/2 nm/day).

Waves impinging on the coastline also create longshore circulation cells close to the beach. Historical data and the bathymetry in the area suggest a northward movement beginning at about the location of the existing natural coastal ponds, and southward movement southeast of the ponds. The field data indicated a northward movement from late winter to mid summer and a southward transport the remainder of the year. The strengths of either flow would be greatest during periods of significant winter storm activity or summer swell. In general the annual net transport is north-northwest.

Comments. The site nearshore circulation effects both the source water entering the lagoons and the removal of water leaving the lagoons from the area. The lagoons' entrance were located on the coastline in areas that will optimize existing nearshore flows and limit feedback where possible.

The nearshore circulation patterns found off the project coastline in the past suggest that water reaching the lagoon entrance areas arrived primarily from the north and offshore during May to July and September to December, especially during flooding tides. Tradewinds blowing toward offshore would tend to move most, but not all, of the water leaving the Barbers Point harbor toward the west and northwest, angularly away from the coastline during most ebbing tides. During periods of strong ebb tides and Kona or weak winds harbor water and coastal water from the south mixed with offshore water, could reach the southern most lagoon entrance area on the site shoreline. Though harbor water would be diluted between

10:1 to 50:1 with coastal water, the mix could reach this lagoon entrance. This is unavoidable during these conditions. Such conditions may occur 20 to 35 days per year.

Considering all parameters and the yearly patterns, however, the entrance locations for the lagoons is best located at the central or north-central position on the site shoreline. The nearshore drift is southward 3/4 of the year. The harbor is 1/2 mile south and the Kahe Hawaiian Electric cooling water discharge is 1 1/2 miles north. The central area of the site coastline appears to offer the optimum positioning. The coastal bathymetry of nearshore shoal and channel areas also suggests discharging exiting storm runoff water both north and south.

It is also important that no coastal storm drainage or other land originated discharge be allowed in this central shoreline area, and preferably not be allowed (or at least be minimized) in the northern half of the site shoreline.

The nearshore flows parallel the beach, with typical strengths of 0.6 to 1.0 kt. It is believed up to 2.0 kts maximum occur in the area. This current is beyond the capability of most swimmers. If coastal swimming is to be allowed, or small coastal beach facilities provided, they will need natural or manmade limitations to contain swimmers close to shore. In addition people should be kept clear of the lagoon entrance areas, particularly during periods of strong incoming surf for their safety.

Background Water Quality

Data have been taken both north (Kahe and Waianae) and south (Conoco Refinery and Barbers Point) of the site, but few directly off the site coastline. These past data have been applied to either helping establish water quality standards for Hawaiian waters or to solving site specific discharge or environmental problems. Dr. Paul K. Bienfang's 1979 report summarizes the published water quality standards and criteria for both open coastal waters and artificial basins.

Considering all applicable data one can list the probable annual limits of several pertinent water quality parameters likely to be found in the intended lagoons during the year. These properties are:

Temperature (^{o}C)	22.0 - 27.0
Salinity ($^{o}/oo$)	33.50 - 35.10
Density (gm/cm^3)	1.025 - 1.027
Dissolved Oxygen (mg/1)	6.0 - 8.0
Nitrate (micro gm/1)	0.10 - 20.0
Nitrite (micro gm/1)	0.13 - 30.0
Ammonia (micro gm/1)	0.20 - 10.0
Total Nitrogen (mg/1)	0.02 - 0.80
Phosphate (micro gm/1)	0.20 - 0.30
Total Phosphorous (mg/1)	0.003 - 0.065
Secchi Depth (ft)	10 - 60
ph	8.0 - 8.6
B.O.D. (mg/1)	0.4 - 1.8
Total Coliform (col/100 ml)	0 - 5
Suspended Load (ml/m^3)	0.5 - 4.3

These results are most meaningful when compared with the more offshore, supposedly pristine, waters and with the state's water quality standards. A 1970 Oahu Water Quality Program Comparison showed the project offshore waters to be comparable to the coastal water off Diamond Head, Waikiki, and Magic Island. Temperature and salinity variations are typically the same, dissolved oxygen remains annually above 75% saturation, visibility (secchi depth) is high, turbidity below the standards, B.O.D. low, and ph acceptable. Nutrient concentrations, however, are expected to vary seasonally above the acceptable geometric mean but should remain below the "not to exceed 10% of the time" limit for wet open coastal waters. It is not known, however, how much time each year this limit is approached.

The Conoco waste water discharged 1-mile south of the project site does contain toxic or toxic-like substances, in detectable objectional amounts. No field data taken to find these substances (hexane extractable materials, mercury, barium, arsenic, cadmium, lead and chromium) in waters just off the site were taken. It is likely that the southward moving net transport existing in the area approximately six months of the year moves this discharged material south and away from the project area. During strong winter Kona winds and flooding tides this water can flow past the site, offshore a few 100 yards to a mile.

Comment. The coastal waters off the project are, on occasion, the receiving waters for local discharges north and south of the site and the source water for the lagoons. Thus far the coastal water appears to have

remained relatively pristine, indicating an effective net flushing of the area despite these neighboring pressures. Nutrients, particularly nitrogen forms, do presently exceed the state's water quality standards for wet open coastal waters for short periods in winter months primarily as a result of ground water percolation from the coast during months of increased rainfall. In that the lagoons will also be receiving ground water seepage high in biota stimulating nitrate-nitrogen (i.e. up to 10 times the coastal water nutrient level) both the coastal source water and lagoon water are expected to show an increase in nitrogen during winter months when rainfall increases. Daily natural flushing rates will also vary with local wave activity and the prevailing tidal profile. The combination of a small daily tidal change and calm winds and seas, following a recent winter storm, would likely provide insufficient flushing for short periods of time. These periods might extend from a few hours to a day or two. At these times the lagoon water quality would need monitoring of the temperature, salinity, and nutrient levels, to determine if artificial flushing, or restricted use is needed during these unusual conditions.

Site Heat Budget

Determination of the lagoon heat budget concerns itself with estimating the annual gain and loss of heat at the surface of both the coastal waters and lagoon waters. The net heat exchange between the surface of large bodies of water and the atmosphere is the result of five main processes:

a) incoming short wave solar radiation (insolation)

b) short wave radiation reflected from the surface (reflection)

c) the net long wave radiation (back radiation) from the water to the atmosphere

d) heat exchange due to evaporation or condensation

e) the sensible heat exchange.

The usual approach to an analysis of surface heat exchange is to consider only these five processes, and to ignore minor processes such as heat transferred due to precipitation, surface runoff, chemical processes, and energy dissipation by surf and currents. The three processes of insolation, back radiation and evaporation/condensation are considered independent, while the reflected radiation is related directly to insolation, and the sensible heat exchange to evaporation losses.

The relationships that have been developed to date attempting to predict these surface heat exchange components are of an empirical nature, principally because their application requires them to be in terms of readily measurable (and available) physical parameters. In many instances, the equations are merely "best fits" to a relatively small

amount of data. The resulting equation may then only be applicable in the area (or season) where the data was taken.

The pertinent meteorological-oceanographic variables most commonly measured are wind, cloud cover, sea temperature, air temperature, and wet bulb temperature. It is to these five variables that nearly all relationships describing surface heat exchange are adapted. In spite of this limitation, a large number of equation sets for evaluating the five principal heat exchange processes exist. These have been reviewed by Laevastu (1960), who also discussed the general applicability of each equation. As a result he was able to select a set of surface heat exchange relationships which he considered gave the best results. No geographical limitations were set on the equations, which are as follows:

$$Q_i = 0.014 A_n t_d (1-0.0006C^3) \text{ cal/cm}^2\text{/day} \quad \text{(B1}$$

with the condition that when A_n is greater than 75°

$$Q_i = 1.06 t_d (1-0.0006C^3) \text{ cal/cm}^2\text{/day} \quad \text{(B1}$$

where Q_i = incident radiation on sea surface
A_n = noon altitude of sun (deg)
t_d = length of day (min)
C = cloud cover (tenths of sky covered)

$$Q_r = 0.15 Q_i - (0.01 Q_i)^2 \text{ cal/cm}^2\text{/day} \quad \text{(B2}$$

where Q_r = radiation reflected from sea surface

$$Q_b = 1440 \left[\frac{14.38-0.09T_w-0.046U_o}{69.72}\right][1-0.0765C] \text{ cal/cm}^2\text{/day} \quad \text{(B3)}$$

where Q_b = effective back radiation
T_w = temperature of sea surface, °C
U_o = relative humidity, %

$= \frac{100e_a}{e_{sata}}$, where e_a = vapor pressure of the air, mb

e_{sata} = saturated vapor pressure over water at the dry bulb temperature, mb

(Vapor pressures may be found in the Smithsoni Meteorological Tables.)

$$Q_e = \frac{L}{10}[(0.26+0.077W)(0.98e_{satw}-e_a)] \text{ cal/cm}^2\text{/day} \quad (B4)$$

where Q_e = heat exchange due to evaporation

W = wind speed, m/sec

e_{satw} = saturated vapor pressure over water at the temperature of water, mb

L = latent heat of evaporation of sea water

$= 594.5 - 0.51T_w$ cal/cm^3

The quantity in square brackets is the evaporation in mm/cm^2/day.

$$Q_s = 38.9(0.26+0.077W)(T_w-T_a) \text{ cal/cm}^2\text{/day} \quad (B5)$$

where Q_s = sensible heat exchange

T_a = air (dry bulb) temperature, °C

Equation (B5) is derived directly from Bowen's ratio which, at sea level, is as follows:

$$R = 0.66\left(\frac{T_w-T_a}{e_{satw}-e_a}\right)$$

The convention is to consider heat gained by the sea as positive, heat lost as negative. Thus the right hand side of Eqs. (B2)-(B5) would each be preceded by a minus sign. Note also that Eq. (B4) predicts some evaporation even under zero wind conditions. The term 0.26 is necessary as convection will give rise to local air movement even though W=0. It is neglected if W is small and $T_w<T_a$. Also, if $T_w<T_a$, Eq. (B5) becomes:

$$A_s = 3W(T_w-T_a)$$

and heat is gained by the sea. Eqs. (B1)-(B5) (henceforth referred to as "method A") are valid for a daily time basis, giving average values over a 24-hour period if the five variables are representative of daytime averages.

A second set of surface heat exchange equations are those used by Wyrtki (1966) in his study of the net surface heat exchange of the North Pacific Ocean. The equations given in his report, without discussion, are:

$$Q_i - Q_r = Q_{io}(1-r)(1-0.38C^2) \text{ cal/cm}^2\text{/day} \quad (B6)$$

where Q_{io} = some fraction of the solar constant (1.94 cal/cm^2/min) and is a function of latitude and month

r = reflection, also a function of latitude and month

$$Q_b = RS\theta_w^4(0.39-0.05e_a)(1-0.38C^2) + 4RS\theta_w^3(T_w-T_a) \quad (B7)$$

where R = ratio of radiation of sea surface to that of a black body and = 0.97

S = Stefan-Boltzman constant = 1.371×10^{-12} cal/sec/...

θ_w = temperature of sea surface in °K .. cal/sec/cm^2/°K^4

$$Q_e = \rho_a C_D L (q_w - q_a) W \quad (B8)$$

where ρ_a = air density - 1.2×10^{-3} gr/cm^3

C_D = drag coefficient - 1.55×10^{-3}

q_w = specific humitity at sea surface (i.e., the ratio of the mass of water vapor to the mass of moist air in which the water vapor is contained)

q_a = specific humidity of air

$$Q_s = \rho_a C_D C_p (T_w - T_a) W \quad (B9)$$

where C_p = specific heat at constant pressure of air - 0.24 cal/gm/°C

These Eqs. (B6)-(B9) will henceforth be referred to as "method B." This method is somewhat less empirical than method A in that the equations afford a better insight into how the constant parameters affect the surface heat exchange.

In applying methods A and B to computing the net surface heat exchange of the lagoons, and its potential alteration due to incoming coastal and infiltrating water, four of the five input variables (W, C, T_a, T_w, T_{wb}) were obtained from the local weather reports taken at the Barbers Point Naval Air Station. A characteristic mean value of wind speed, cloud cover, air temperature, and wet bulb temperature was computed for each day from ten to fifteen readings of each, taken generally from sunrise to sunset. Any apparently anomalous daily values were discarded and monthly averages were then computed. Monthly averages of sea surface temperatures were obtained from sources cited in the literature survey, considering historical data notations (Wyrtki, 1970) and field data for the West Beach program. No account was taken of the fact that the surface film is about half a degree cooler than the waters

immediately beneath it (Boyum, 1962; Laevastu and Inouye, 1964).

The twelve monthly values of the five variables were used in two independent computer programs involving Eqs. (B1)-(B5) and (B6)-(B9), respectively. In the program for method A, noon altitude of the sun and length of day were taken for the 15th of each month from the Smithsonian Meteorological Tables. In the program for method B, insolation from a cloudless sky (Q_{io}) and reflection were found using Wyrtki's (1966) method. Vapor pressures and relative humidity in both programs were found using air and wet bulb temperatures and the Smithsonian tables. Two of the various different weather conditions tested will be discussed in the results. They are the characteristic monthly values of W, C, T_a, T_{wb}, and the same characteristic weather but with T_w reduced by 0.9°C (1.5°F) simulating a local depressed temperature anomaly.

Given the previously noted water quality values, and a site relative humidity range of 50% to 70%, cloud cover of predominantly 10% to 60%, and wind speed of 10 to 25 kts (mean 12.2 kts) the site annual heat budget was found to be as follows:

Month	Heat Exchanged (+ = gain, - = loss)
June	+92 cal/cm^2/day (maximum)
July	+64 cal/cm^2/day
August	+46 cal/cm^2/day
September	+12 cal/cm^2/day
October	-31 cal/cm^2/day
November	-65 cal/cm^2/day
December	-88 cal/cm^2/day (minimum)
January	-53 cal/cm^2/day
February	-26 cal/cm^2/day

Month	Heat Exchanged (+ = gain, - = loss)
March	+ 2 cal/cm^2/day
April	+53 cal/cm^2/day
May	+74 cal/cm^2/day

The results indicated that heat is gained by the water from March to September and lost from October to February. The slight resultant net gain of +6 cal/cm^2/day is typical for the latitude of Hawaii. This annual gain in heat is transported away from Hawaii in the Pacific ocean currents. Averaging the corresponding surface water evaporation for the above monthly results of net heat exchanged at the surface indicates that approximately 18.8 cm/mo 7.4 inches/mo) of water is expected to evaporate from the surface of the lagoons. This is a normal evaporative loss.

The objective of the second analyses completed was to estimate for a lagoon changes brought about by cool infiltrating waters. The examination of such changes was confined to just those likely to occur as a result of mixing incoming coastal water with infiltrating water.

Considering the incoming water as well mixed the problem is reduced to two dimensions. A relatively simple form of the heat conservation equation could, therefore, be used to predict changes in the hear content in a lagoon. The differential form of the heat conservation equation in two dimensions is given as (Bathen, 1970):

$$\frac{\partial H}{\partial t} = A_h\left[\frac{\partial^2 H}{\partial x^2} + \frac{\partial^2 H}{\partial y^2}\right] - \left[u\frac{\partial H}{\partial x} + v\frac{\partial H}{\partial y}\right] + Q(x,y)$$

where H = total heat content in the mixed layer

t = time, sec

A_h = horizontal mixing coefficient; this varies with latitude and at 20 was assigned a value of 3×10^6 cm^2/sec (Bathen, 1970)

x,y = distances along a horizontal coordinate grid system, cm
x positive northward, y positive eastward

u,v = velocities in the x- and y-directions, cm/sec

$Q(x,y)$ = net surface heat exchange, $cal/cm^2/sec$

The left hand side represents the local (time) of change of heat content, the right hand side the processes causing the change, i.e., horizontal mixing, horizontal advection and the net surface heat exchange with the atmosphere. H is the heat content in the water column, used here in cal/cm^3. The total heat content in the water column was used because near the coast the bottom depth is less than the mixed layer depth. Corrections for shoaling are, therefore, included in H.

A computer program was written to solve the finite difference form of Eq. (C1) numerically. A square grid system was selected capable of covering one of the large intended lagoons. The dimension of an element in the grid system was 610 cm (20 ft.). The location of the grid system was such that the lagoon entrance was placed as the western boundary. Field data and results of the historic survey had shown the predominance of a daily net northward flows following flooding and ebbing tides.

In this case the numerical solution must be convergent to the exact solution and the numerical solution must be stable. Convergence was tested by using a recurrence equation. The rounding error caused by

the difference between the exact numerical solution $H_{i,\,j,\,k}$ and the actual numerical solution $H^*_{i,\,j,\,k}$ must be bounded and decrease with time. By expanding the recurrence equation term by term in a Taylor series the finite difference round off error was found to be proportional to the second and fourth order space derivatives of H, and the second order time derivative of H. In the limit, as Δx, Δy and Δt, go to zero, these derivatives must vanish.

Stability was tested by the Fourier series method. A disturbance was introduced into the solution $H^*_{i,\,j,\,k}$. Stability is insured if this disturbance decreases with time. Substituting the disturbance into the recurrence equation, and simplifying, gave the condition

$$\frac{4A_h\Delta t}{\Delta x^2} + \frac{\Delta t}{\Delta x}\left(|u_{i,j,k}| + |v_{i,j,k}|\right) \leq 1$$

with $A_h = 3\times10^6$ cm/sec and a maximum value of $|u_{i,j,k}| + |v_{i,j,k}| = 120$ cm/sec (2.3 kts)

$$\frac{\Delta t}{\Delta x}\left(\frac{12\times10^6}{\Delta x} + 120\right) \leq 1 \qquad \text{(C2)}$$

A requirement of secondary importance was that the grid size should not be so small as to make the number of calculations excessive. Setting $\Delta x = 610$ cm, the stability requirement became

$$\Delta t(197 + 120) \leq 61{,}000$$

$$\Delta t \leq 192 \text{ sec}$$

Therefore a 3-min time step was selected as the maximum allowable value.

Initial heat storage in the lagoon was calculated by vertically integrating the temperature profile from the surface to the bottom and

converting to cal/cm^3:

$$H = \frac{pC_p \int_0^D T\, dz}{D} \quad cal/cm^3 \qquad \text{(C3)}$$

where p = density of sea water = 1.023 gm/cm^3

C_p = specific heat at constant pressure = 0.954 $cal/gm/^oC$

Both p and C_p were considered constant within the lagoon.

It was necessary to select characteristic current velocities at the lagoon entrance based upon the tidal prism and incoming waves. Velocities were assigned to each grid square along the western boundary of the grid system. In addition the velocities were set equal to zero along the remaining boundaries. Both component velocities in all other grid squares could then be calculated using the continuity equation. The velocities were assumed constant with depth down to the bottom of the lagoon. The maximum value of $|u_{j,j,k}| + |v_{i,j,k}|$ was 80 cm/sec. This occurs in the deep entrance channel.

Following initialization, calculations were started by simulating infiltrating water into the shoreline grid squares assuming complete mixing within the grid ambient waters. Only one discharge rate was considered; a permeability of 5 ft./day (observed range 0.1 to 10 ft./day). After new values of heat content have been calculated for each grid volume using the heat conservation equation, Eq. (C1), the calculated background value was subtracted (note that background values changed because Q (x,y), net surface heat exchange, was not zero).

The change in heat content of selected grid volumes was printed at several time intervals following the start of infiltrating water discharge. Computations were continued until a tendency toward equilibrium was approached. Two conditions were tested; a summer condition +92 cal/cm^2/day, and the slack water case. The intent was that a warm summer layer and zero current velocities, would create a maximum impact condition, whereas the summer positive net surface heat exchange would tend to decrease the impact. The winter condition, Q (x,y) =-88 cal/cm^2/day, and the current conditions discussed above. In this case a minimum local impact results from including circulation and mixing while the negative Q (x,y) would tend to increase the impact.

The results indicated that incoming water will tend to warm approximately 0.47°C in traversing the lagoon during summertime and show approximately no change in winter. Active flushing will prevent any slow above seasonal accumulation of heat in the shallow lagoons. Evaporative losses would tend to increase salinity along the lagoon in the direction of net flow. This anticipated slight gain of 0.1 °/oo to 0.5 °/oo will, however, be offset by ground water infiltration of brackish water. The infiltration will be more significant than the evaporation and is expected, in the limiting case, to lead to diluted surface salinities ranging between 33 °/oo to 34 °/oo. This is compared to incoming water annually averaging 35 °/oo.

Comment. Heat will be gained and lost by the waters flowing through the lagoons in an annual pattern similar to the neighboring coastal

waters. A slight summer gain in heat of about 1/2°C (warming trend) will occur as coastal water flows through the lagoon, along with a slight increase in evaporation when compared to offshore waters. During the winter flushing rates and ground water infiltration will have a more significant effect on the lagoons with the surface waters tending to become slightly less saline than coastal waters. Neither the slight warming or evaporation will be as important as the diluting effect of ground water infiltration.

Site Meterology

The characteristic weather found at the project site is discussed in Bathen 1973. Briefly, air temperatures at the site vary day/night from 77°/60°F winter to 70°/85°F summer. The area is dry with rainfall averaging less than 20 in/year. Data show that most showers occur in the West Beach area during the early pre-dawn hours.

The spring and summer extend from approximately April to November and include the period of strong northeast trade winds from about June to September and the transitional periods just preceding and following. During this season, wind direction ranges from northerly through easterly, and occasionally southerly, but predominately blows from speeds at the lagoon location should average 12.2 kts, but will range seasonally up to 25 or 30 kts. About 30% of the time the local speed exceeds 12 to 15 kts.

The fall and winter is reasonably well-defined by a weakening of the northeast trade winds and the appearance of westerly winds and frontal influences from the north temperate zone. These westerly winds, known locally as Kona storms, are most typically represented by strong winds and high waves from the south to southwestern quadrant. The year-to-year variations in the Kona conditions at the site are considerable. Some winters no Kona storms appear and in other years there may be four to five strong storms. Generally, during the winter months (December to March) winds from the southwestern quadrant should be present 10% to 15% of the time, but Kona winds are expected to occur most often during January.

The historical data record a maximum sustained wind speed in the site area of 40 kts, with gusts to 83 kts, on January 13-14, 1970. In contrast, calm conditions exist 1.5% of the year, usually during the winter months. Kona conditions, from storm conditions to Kona calm or light onshore winds are expected to occur at the lagoon site about a total of thirty days per year.

Comment. The annual mean tradewind streamlines remain essentially orthogonal to the West Beach coastline. This wind blowing directly offshore remains quite consistent year to year. The north-south alligned central part of the site coastline, i.e. the proposed location of the four lagoons, is curved slightly landward. This would help move the water leaving the lagoons away from the site toward the discharge end of each lagoon. The major influence of the predominant tradewinds at the lagoon sites, will be to create a cross lagoon exiting flow which will cause typical surface currents up to 15 cm/sec (0.1 to 0.5 kts) averaging about 10 cm/sec. This will aid overturn and mixing in each lagoon, helping replace bottom waters.

When the wind strength increases, as during storms or early spring periods, dust and debris will blow from the beaches bordering the lagoon and hotel area. Strong winds of 40 kts, for example, will cause cross lagoon current drifts of up to 20 cm/sec (0.4 kts), and small lagoon surface waves up to one-half foot in height. The wind transported particulate material reaching the lagoon surface will be quickly mixed vertically throughout the lagoon waters.

During onshore or southerly Kona winds the surface wind and debris effect on the lagoons will depend upon the height of the buffer zone, or ridge, separating the coastline and lagoons and the amount of loose material this ridge contains. Water leaving each lagoon exit will, in this onshore wind case, have difficulty leaving the discharge area. Depending upon the alignment of the wind to the shoreline, some of the discharged water could recirculate within two to four hours. During such Kona storm conditions this is unavoidable and lagoon operations may necessitate restricting access to the lagoons.

Lagoon Sizing and Hypsography

Each final lagoon shape, size and hypsography (volume distribution) is primarily a decision based on land use and planning needs. The area paralleling the coastline, allocated for the lagoons, dictates the limiting size of each lagoon. Four lagoons were planned sized from 2.0 to 5.5 acres (at msl), varying in shape from semi-circular to oval. The four lagoon concepts developed include long curved beaches with cusp-like ends, deep central areas and shoal entrance barrier islands. The entrance islands are primarily indigenous basalt and sand. Depths of 6 to 7 feet at mean lower low water were choosen. Average depths will be 5.4 to 6.3 feet.

A variety of final lagoon sizes and shapes were specified ranging from a large lagoon with an entrance width of approximately 600 feet and reach of 500 feet, and smaller lagoons with entrance widths of 500 to 530 feet and reaches of 200 to 370 feet. The four lagoons contain approximately 3.5 to 11.3 million gallons at msl. The volume becomes important when considering flushing. Since it is necessary to maximize the daily exchange of water, the lagoons are as shallow as the intended use will allow.

The sizing criteria developed considered maximizing the flushing potential, not restricting the flow into or out of the lagoon nor allowing currents at the lagoon entrance to normally exceed 1.5 kts. Using medium coarse to coarse grade sand on the bottom and entry slopes of 6° to 12° (1:10 to 1:5) the flow must be limited within the lagoon to insure sand is

not excessively transported along the lagoon beach. Thus an entrance for the lagoon includes a shoal shelf area (-1 ft. at msl) on each side of a deep centrally located exit channel. The deep central portions of each lagoon is a constant depth to insure all benthic waters are uniformly flushed. The entrance and exit areas are sufficiently large to directly reach and flush both the lagoon surface and bottom waters.

The lagoons benefit in storm protection from retaining the existing coastal ridge seaward of the lagoon beaches, and retaining much of the existing hard basalt and coral shoreline armored terraces. The selection of the site central shoreline area for locating the lagoons takes advantage of the local bathymetry for a natural 30:1 wave run-up plus aid in flushing using the nearshore circulation patterns. Adjacent nearshore deep channels in the bathymetry will also aid removal of deep exiting water.

Most of the grading around the lagoons, excepting the lagoon beaches, must be directed away from the lagoon waters, particularly those surrounding grounds containing vegetation that receive regular treatment with plant fertilizers, plant control chemicals, or pesticides. Also no direct storm runoff from urban areas, streets, hotels or parking areas should be allowed to reach the lagoons. The results of adding these nutrients (primarily nitrates and phosphate), bacteria, solids and particulate substances, hexane soluble materials, hydrocarbons, oils, tar, waxes and debris would each degrade the aesthetics of the lagoon waters.

Nutrient addition from runoff could, if entering the lagoons, stimulate growth in the phosphate limited case of lagoon waters causing

initial change toward eutrophication. Fecal coliform bacteria tend to decay in salt water and sunlight within a few hours but can be a potential health hazard if concentrations reach 1000 col/100 ml. Hexane soluble hydrocarbons, oils, tar and waxes can coagulate and gather at the line or result in a thin film across the lagoon surface. In this case the aesthetics of the lagoon surface water would become visually objectionable when floatable concentrations of about 1.5 mg/m^2 are reached. Land oriented beach use debris is immediately objectionable if allowed to reach the lagoons.

Comment. Each lagoon size, shape depth distribution and entrance configuration was selected within certain limits to insure optimum potential for flushing, a stable lagoon bottom, beach entry slopes that do not experience sand transport, active flushing of all parts of the lagoon, and non-entry of runoff from the surrounding grounds. Lagoons between 2.0 to 5.5 acres surface area, 6 to 7 ft. deep at mean lower low tide, constant central bottom depth, entry slopes of 10° to 20° and entrance cross-section widths of 400 to 600 ft. were selected. The lagoon shapes chosen also considered the refraction and diffraction of incoming waves and the prevailing tradewind direction to insure no floatable or debris traps are created.

A sand covered beach entry and sand and rock bottom, over a hard sub-bottom was selected. This would help minimize infiltration of ground water and also not provide all sessile biota and optimum substrata for growth.

Tidal Prism

The mean diurnal tidal range at the site coastline is 1.9 feet, with the tide being predominately mixed in character, i.e. two unequal highs and lows per day of 1.3 feet typical change. The greater annual observed tida' changes in the area of approximately 3.0 feet, occur three to four times each year. The time of the tide at the site follows the Honolulu tide by 15 to 18 minutes.

Using an annual daily average tidal change of 2.0 feet, the amount of this tide that can enter the lagoons, i.e. the tidal prism, can be computed. The amount depends upon the lagoon dimensions, the entrance crossectional configuration and smoothness. These factors establish the amount of restriction to the incoming and outgoing flow. Tidal prisms thus ranged from 2.23×10^5 to 6.38×10^5 ft.^{3}.

The computation results show that the average daily flushing velocity in the deep central channel for the above tidal prisms will range from 0.54 ft./sec to 1. 3 ft./sec. The estimated maximum exiting velocities for the lagoons should range between 23 to 48 cm/sec (0.5 to 0.9 kts) during typical tidal and wave conditions. As noted earlier, during extreme storm events water exit velocities at the lagoon exit center could exceed 1.5 kts.

Comment. A tidal rise in the lagoons essentially equivalent to the tidal rise at the coastline is anticipated. During the maximum tide and diurnal conditions the tidal prism increases from an average 52% to 92% of the lagoon volume per day, while during days of minimum tidal change 30% is exchanged per day.

Flushing Criteria

A practical way to evaluate the minimum acceptable rate of flushing for the lagoons was to examine the performance and water quality conditions existing in the Kamokila lagoons along with several other large salt water artificial ponds, tanks or lagoons in Hawaii. Considering the resident time of infiltrated water in local harbors also provides some measure of infiltration rate and flushing vs. tendency toward eutrophication.

Seven manmade or modified salt bodies of water, in addition to the site Kamokila lagoons, were considered and information sought on each. Primary concern was in determining the body volume, turnover rate (flushing) in terms of body volumes per day, the amount of recirculated or new water, the source of this new water and any special treatment given either the new or ercirculated water. The Tables shows a summary of this survey for two California and five Hawaiian cases.

These results, along with several comments received in obtaining this information indicate two flushing criteria exist:

1) For public-oriented marine viewing tanks flushing rates of 4 to 5 times and up to 30 times per day are employed. Five times per day appears to be a working minimum to keep the water suitably clear for unobstructed viewing. Chemical treatment is commonly used to control disease of the tank inhabitants and chlorination, mechanical scrubbing or sand filtering are used to control tank algal growth.
2) In large natural open lagoons flushing rates vary from 2/3 times to 3 times per day. In these cases no treatment is

employed, and in two cases the water flow is restricted to flow one way only.

As noted in the previous section the Kamokila lagoons (i.e. Paradise Cove lagoon) are actively flushed via incoming waves and the daily tidal prism. Flushing rates during field observations ranged from 4 to 23 times per day. Typically rates of 3 to 10 times per day likely exist in these high quality natural lagoons on the project site.

An interesting comparison exists between the Hilton Hawaiian Village lagoon and the Magic Island lagoon. Though the Hilton lagoon flushes more of its volume one way, it draws source water from the partially degraded water inside their coastal beach reef. This lagoon was also initially filled, leaving a silt and clay bottom with fine sand leading to the less than desirable water quality existing in the lagoon today. The nutrient accumulation problem in this case is becoming more severe each year.

In contrast, the Magic Island lagoon is larger, flushes less of its volume daily and is restricted in bottom water exchange yet its water quality is markedly better. This is due to better source water, more aeration due to the wave trap like action of the breaks in the lagoon breakwater, and the clean, medium grade sand bottom over a hard subbottom material. If the Magic Island lagoon water, flushed now 2/3 times per day, were receiving a more active flow throughout its 10-foot depth and if its flushing were increased to three times or better per day, its water quality would improve even more over its present acceptable condition.

Considering all results, including the bioconstraints and

recommendations of consultant Dr. P.K. Bienfang, it is evident that a minimum daily average flushing of 3 times the lagoon volume per day is advisable. Greater flushing rates will provide a greater margin of safety. Rates as high as 5 to 10 times the lagoon volume per day, however, will likely occur ofen depending upon the incoming waves. A trade off of acceptable water quality for the use intended vs. the desire to obrain water quality identical to coastal waters (never better than coastal water) must be made. However, the capabilities of backup pumping equipment might be considered and costed such as to be able to provide cleaning or flushing rates up to 5 times the lagoon volume per day. This capability would insure active flushing if/when it is needed following prolonged calm periods, excessive storm events, or accidental deleterious discharges into the lagoon.

The central shoreline area of the site contains a natural ramplike coral terrace with slopes of 23:1 to 40:1. Thus the shoreline acts as a natural wave runup ramp for incoming surf. Wave flushing of the lagoons is expected to exceed the tidal prism by 10 or more times per day. In the absence of all wave activity for periods longer than 6 hours the flushing would be limited to tidal exchange, i.e. a 1/4 to 1/2 times volume flushing rate.

Thus importance is placed upon comprehensively knowing the low end of the wave spectrum at the site coastline. Figures and Tables included show a summary of historical wave data and information applicable to the site coastline.

The historical results indicate that 341 days of the year waves

and swell greater than one foot in height reach the shoreline. Waves up to 30 feet are predicted for the area during severe storm conditions, but waves between 1 to 6 feet in height predominate during the year. But more important, calm conditions or waves and swell less than 1-foot in height exist, statistically, for 24 days per year. Exactly when these days occur, or how many consecutive hours exist with absolutely flat seas and no incoming swell, is not known for the site area. Interest is usually placed on the upper limit of wave height. Though this is also of interest when considering storm runup and structural strength, it is of no assistance in estimating the lower limiting condition, i.e. when wave induced flushing would be minimal.

Since the summer brings South Pacific swells and wind waves to the south and west coasts of Oahu, and winter brings the north Pacific waves and local wind waves to the coast, the calm periods tend to exist in either late summer when the tradewinds weaken or during mid-winter when Kona winds can bring calm conditions. It is felt, however, that absolutely calm water along the shoreline, without even swell occurring, happens rather rarely, perhaps a few days per year and up to 10 to 20 consecutive hours each occurrence.

The directions of incoming waves will be refracted by the local bathymetry. Considering the range of incoming directions and the offshore bathymetry, it is evident that most waves approach the coastline within 45° of orthogonal to the beach. The nearshore bathymetry also refracts the incoming waves with most waves probably impinging the shoreline within 30° of orthogonal. These conditions are helpful for both wave induced

flushing longshore transports.

SEA Engineering field wave data, as analyzed by Gerritsen, was taken for over 10 months. These results, discussed in Gerritsens report, were considered in the lagoon design and are the basis for the estimates of the anticipated lagoon flushing rates.

Comments. Existing flushing rates in similar local and California salt water lagoons (2/3 to 5 times volume/day) vary as widely as the observed water quality in each lagoon. The past 9 acre Barbers Point harbor, with a single entrance, was believed to flush infiltrated brackish ground water out in approximately 1/2 day. The new deep draft harbor is to flush infiltrated ground water out within 2 to 15 days. In these cases eutrophication is essentially assured. Control of the phytoplankton growth in the West Beach lagoons places lower limits on the flushing rates, with recommendations given keyed to the observed permeability rates, and phosphorus and nitrogen concentrations as reported in the Harding Lawson field report. The follow on pre-construction work will re-examine the site conditions throughout each lagoon site.

The conservative suggested minimum flushing criteria is 3 to 4 times/day, preferably greater. Physical constraints of the entrance configuration, circulation and sand movement in the lagoons, along with keeping the design within reasonable sized limits, suggest flushing rates will range up to 20 times per day. In comparison, the anticipated lower limit is greater than that presently found in the Magic Island lagoon.

Infiltration Injection Rates

Lagoon source water is composed of both incoming coastal sea water and infiltrating brackish ground water. Both will be entering the lagoons continuously, though neither at a day-to-day consistent rate. The incoming coastal water will vary with the coastal conditions, tides and wave activity present. The infiltration will vary with the seasonal rainfall and local storm activity.

A general description of the geology in the project area has been given by Bathen, (1973) and Dr. P. Bienfang (1979). It need not be repeated here. Figures provided show the locations and bore stratification data for seven historical borings taken in 1969 for the West Beach project. These data, Barbers Point core boring results along with project field data taken by Harding Lawson Associates provide the basis for the infiltration permeability and ground water quality estimates.

The data show permeabilities of 0.2 to 6.5 ft/day. Harding Lawson suggested the site rates may range from 0.1 to 10.0 ft./day. This is equivalent to infiltration rates of 0.75 to 74.8 gal/ft.^{2}-day. Historical estimated rates of infiltration for the site area range between 13.2 gal/day/ft.^{2} to 15.5 gal/day/ft.^{2} for the lagoon sides and 1.0 gal/day/ft^2 to 1.8 gal/day/ft.^{2} for the lagoon bottom. The data also show that the lagoon sites are composed primarily of sandy silts and clayey silts and silty sand with densely comented coral fragments. In most cases the lagoon bottoms, prior to lining, is expected to be medium dense to dense materials (i.e. 15 to 300 blows/ft.). Permeability may thus tend to be toward the lower end of the HLA predicted range.

It is estimated that an 5.5 acre lagoon, 8 feet deep (ref. msl) will receive between 0.25 mgd to 0.5 mgd. This water is less saline (6.2 $^{o}/_{oo}$ - 25.5 $^{o}/_{oo}$) than the incoming sea water (34 - 35 $^{o}/_{oo}$) and therefore bouyant. Considering the lagoon volume, in this case 11.25 million gallons, and a non-flushing condition the infiltrating water will comprise approximately 2% to 5% of the lagoon volume. It would comprise 6% to 25% of a twice daily 2-foot tidal prism inflow or up to 1/2 of the daily tidal prism. In general, infiltrating ground water is anticipated to comprise between 1% to 6% of the lagoon volumes for 2.0 to 5.5 acre lagoons. In the presence of a minimum 3 to 4 lagoon volume flushing rate per day, the lagoon waters would contain a daily average of approximately 1/2% to 3% infiltrated water at any time.

Comment. A wide range of hydraulic permeability and infiltration rates exist in the literature for Oahu. The field data taken for the program provide more site specific permeability rates and geological profiles. The result of incoming infiltrating water is expceted to have minimal effect on the lagoon salinity. Dissolved oxygen of 2.8 to 5.1 ppm, BOD_5 less than 2, and total coliform concentration less than 4/100 ml in the infiltration water should not create any problem. Nutrient values are however, significantly higher than coastal waters.

Potential Water Quality Changes

Changes in the lagoon water quality, both of advantage and disadvantage when compared to the quality of the source coastal water, would occur from several processes. These include aeration via incoming waves, infiltration of nitrogen and phosphorus rich ground water, runoff from the beach areas immediately surrounding the lagoons, changes in the heat exchanged at the surface, evaporation, precipitation, nutrient exchange with lagoon lining material, biota uptake of nutrients, changes in organic loading, potential changes in composition of biota, and changes due to human usage of the lagoon water and surrounding beaches.

Incoming water will vary in suspended sand and silt load according to the amount of wave and swell activity existing at the shoreline. Loading would vary, depending upon swell direction and time of year. Suspended loads of approximately 0.5 ml/m^3 for 4.3 ml/m^3 are typical for the area. Suspended sand annually averages 3.0 ml/m^3 during tradewind, southern swell and northwest swell conditions, and silt 2.3 ml/m^3. Portions of the suspended sand may migrate toward the lagoon exits during periods of high incoming waves depending upon particle size and amount of wave activity. Suspended silt sized beach particles (smaller than 50 microns in diameter) could leave the lagoons and enter the coastal waters. The mixing and flushing rate occurring within the lagoons at the time of entry will determine whether the material settles out or remains suspended and subsequently flushes from the lagoons. Suspended load would generally be greatest when wave activity is greatest, as therefore would be flushing and mixing. The quality and sorting of the beach sand and lining materials choosen will establish the amount of suspended load.

Incoming water will be at or near dissolved oxygen saturation. Further aeration apparently occurs in the incoming waves keeping the dissolved oxygen high in incoming waters. Harding Lawson found lagoon water supersaturated in dissolved oxygen (11 mg/l). Oxygen concentrations are expected to range from 6.0 mg/l to 11.0 mg/l.

As discussed in the heat budget section, some heating of lagoon water is anticipated during transit from entrance to exit. This is due to changes in the heat budget from wind shading of the area from surrounding buildings, higher daytime air temperatures on shore, and the shallow character of the lagoons. The amount of heating will depend on the prevailing flushing rate, and may vary seasonally or daily from $0.1^{o}C$ to $0.6^{o}C$. Infiltrating ground water, however, tends to be slightly cooler than coastal water resulting in offsetting a portion of the local heating effect. In all it is estimated the lagoon waters will show a summer warming trend of approximately $1/2^{o}C$ in transiting the lagoons with a flushing rate of 3 to 4 times per day.

Incoming brackish ground water seepage through both the lagoon bottom and sides will bring nutrients to lagoon waters. The brackish infiltrating ground water in the area is expected to contain 1.0 to 7.0 mg/l nitrate (average 2.9 mg/l) phosphate. Coastal sea water contains seasonally between 0.13 to 3.0 micro gm/l nitrate and 0.2 to 0.3 micro gm/l phosphate. The rate per day of incoming ground water is estimated to range between 2% to 5% of the lagoon volume, or 6% to 25% of a typical 2-foot tidal prism. The resulting concentration primarily of nitrogen components for the vertically mix

condition, will vary with the prevailing flushing rate. At the flushing rate of three times per day steady state nitrate concentrations in the lagoon are expected to range from 6 micro gm/l to 50 micro gm/l, and phosphate concentrations from 10 micro gm/l to 30 micro gm/l. These values are several times the related concentrations in the incoming coastal water. At an increased flushing rate of up to 20 times per day these concentrations would drop proportionately lower but still exceed the coastal waters.

Incoming ground water is also less saline than the coastal waters. The result will be dilution of the incoming lagoon sea water. If the ground water enters the lagoon uniformly over each square foot in contact with lagoon sea water, then with a 3 times flushing rate and the anticipated well-mixed condition, the infiltrated brackish waters will be quickly mixed with the lagoon water. Depending upon flushing rate, ground water of 6.5 °/oo to 25 °/oo will comprise a small fraction of the lagoon volume. This could affect lagoon stratification in the absence of mixing by diluting the lagoon surface waters but the effect is minimal, i.e. less than 1 °/oo. If, in the worst winter time case, infiltrated water were confined to a 2-foot tidal prism the salinity in this surface layer would drop 1.5 to 3.0 °/oo.

Comment. Changes in lagoon water quality will result from aeration of incoming water, transport out of suspended beach sand and silt, heating and cooling of lagoon water from the balance of the heat exchanged at the surface and cool infiltrating ground water, nutrient injection in the incoming ground water, and biological uptake and release. The latter effects os assimilation by pelagic phytoplankton and uptake by benthic of sessile

Seiche Behavior

A seiche is a free oscillating motion of the surface of a body of water induced by some impulse or force that previously disturbed the calm surface. It results in a standing oscillation or slow "sloshing" motion, across the length and/or width of the body. The rate of oscillation, or rate of rise and fall of water level at the extreme ends of the water body, is determined by the body dimensions.

Short term changes in the rate of incoming water, changes in wind direction or strength, atmospheric pressure changes, or coastal swell and surges transmitting energy into the lagoons could cause seiches to occur. The surface would oscillate and likely continue for several oscillations since a smooth lined lagoon of regular shape would have limited damping.

Considering the four proposed lagoons with surface areas of 2.0 to 5.5 acres, sloping sides, 8 feet deep at msl, and a given hypsography it is possible to complete the period of the likely seiche activity can be computed and compared to natural periods existing in the coastal area. The objective is to avoid shaping the lagoons so as to induce resonance (amplification) of the lagoon natural period with any coastal phenomena.

The various theories of seiches contend that the free oscillations resulting from a initial disturbance of a basin have periods determined by the horizontal dimensions and depth of the basin. Generally, the equation of horizontal motion (one dimension) and the equation of continuity are used to obtain expressions for calculating free oscillation periods. With the

organisms have been estimated by Bienfang.

Except for the nitrogen and phosphorus nutrient injections in the infiltrating ground water, the remaining water quality changes are expected to be minimal. The potentially high rate of nutrient injection is primarily responsible for specifying a flushing rates of 4 times daily or better. Data need to be taken during the construction phase to better map the exact geologic character of the bed at the lagoon sites, and the rate of ground water infiltration and its chemical properties to determine if any extreme condition needs further consideration. In any case, the lagoon waters mixing and fludhing should be monitored to maintain aesthetically acceptable water quality.

assumption that the water depth is small compared to the wave-length of the oscillation, the vertical accelerations are neglected. As a result, the horizontal velocity is just a function of the horizontal position (x) and time (t). The equations using the surface elevation (η), the horizontal displacement (ξ), gravity (g), horizontal position (x), the variable width of the surface at each cross-section ($b_{(x)}$), variable cross-section area at position x ($S_{(x)}$), and time (t), are:

D3-1 $$\frac{\partial^2 \xi}{\partial t^2} = -g \frac{\partial \eta}{\partial x} \qquad \text{and} \quad \eta = -\frac{1}{b_{(x)}} \frac{\partial}{\partial x}\left[S_{(x)} \xi\right]$$

The solution of these two equations is then obtained, and the result is integrated stepwise by a finite-difference-method to obtain the periods of free oscillations.

The above method of approach, using Equations D3-1, has been used as the basis for this analysis.

A number of complete theoretical approaches to the calculation of seiches with equations D3-1 have been used. The method used here is from Defant (1918) with a correction from Proudman (1925) included to compensate for the Bay opening to the ocean. Defant's method was chosen because it allows a numerical integration by finite differences with any basin shape, is accurate, and provides a means for calculation of the horizontal current velocities. Defant obtained solutions to the equations of motion and continuity (D3-1) by assuming ξ and η to be periodic functions:

D3-2 $\xi = \xi_{o(x)} \cos\left(\frac{2\pi}{T} t - \epsilon\right)$ and $\eta = \eta_{o(x)} \cos\left(\frac{2\pi}{T} t - \epsilon\right)$

where:

ξ_o = horizontal displacement amplitude

η_o = vertical displacement amplitude

T = period

ϵ = phase difference

Substituting these expressions into equations D3-1, considering only small variations along the distance X between cross-sections and small vertical ($\Delta\eta$) and horizontal ($\Delta\xi$) displacements, gives the following expressions:

D3-3 $\Delta\eta_o = \frac{4\pi^2}{g T^2} \xi_o \Delta x$ $\qquad \xi_o = -\frac{1}{S_{(x)}} \int_0^x \eta_o b_{(x)} dx$

These expressions take the following form for computation purposes (Defant, 1961; p. 166):

D3-4 $\eta_2 = \eta_1 + \alpha \frac{\xi_1 + \xi_2}{2}$ vertical displacement at the second cross-section

where:

D3-5 $\alpha = \left(\frac{4\pi^2}{g T^2}\right) \Delta x$

D3-6 $\xi_2 = -\frac{1}{S_2\left(1 + \frac{\alpha V_2}{4 S_2}\right)} \left[q_1 + \left(\eta_1 + \alpha \frac{\xi_1}{4}\right) V_2 \right]$ horizontal displacement at the second cross-section

where:

D3-7 $q_2 = q_1 + \frac{\eta_1 + \eta_2}{2} V_2$

Note: S_N = area of the $N^{\square}$ cross-section

V_N = area of the surface between sections N and $N+1$

Calculation of the periods of free oscillation was accomplished with a computer program using a number of vertical cross-sections of two lagoons and the expressions D3-4 to D3-7. Successive substitutions in the equations (from cross-section 1 to 2) led to a Taylor Series linear numerical integration across each. The assumption was made that the second order terms in the series expansion could be neglected because of the close spacing of the vertical cross-sections. The theoretical results were then corrected for each opening to the ocean by multiplying the results by a Proudman correction factor of $e = 1.098$. The computation was started with an approximate period for a free oscillation in each obtained with the following equation and an estimate of the average depth of each:

D3-8 $$T = \frac{2L}{K\sqrt{gh}}$$

where:

L = oscillation length (M)

K = positive node number (1 to 5)

h = mean depth of the basin (M)

g = gravity (m/sec^2)

The boundary conditions for a free oscillation along the length of the lagoon were that the horizontal displacement, $\Delta\xi$, equals zero at each lagoon end ($x = qL$); for the width of each $\Delta\xi = 0$ along the shoreline and a node is at the mouth of each. Next an initial value for was obtained from equations D3-8 and D3-5. An iteration process across the lagoon was then completed using equations D3-4, D3-6 and D3-7. The process was repeated with a corrected period T, until the boundary conditions were met. The process was continued to obtain the period of free oscillations with up to 5 nodes. Last, the

contribution to current velocities at each cross-section were calculated with the correct period for each mode of oscillation and with the following expression:

D3-9 $$u = -\frac{2\pi}{T}\xi_0 \cos\left[\frac{2\pi}{T}\left(t + \frac{T}{4}\right) - \epsilon\right]$$

The computation results are given as follows:

	Seiche Period - Seconds (at msl) node = 1	
Size (Acres)	Length	Width
5.5	71.8	64.6
2.0	38.3	76.5

The seiche periods above for single node oscillations are for free oscillations occuring along the length and width of the lagoons. Incoming surf beat of 100 to 120 seconds should not be in resonance with the lagoons. However, higher nodal oscillations would have shorter periods, and for oscillations with 2 or 3 nodes it may be possible for incoming storm swells of 12 to 20 sec period to resonate with the smaller lagoons.

The amplitude of the free oscillations are dependent upon the strength of the disturbing force and its period of repetition compared to the natural periods of the lagoon given above. Setting aside the resonance possibility oscillations would be expected to occur in the lagoons with amplitudes similar to those observed in Hawaiian coastal bays. Amplitudes up to 2 1/2 inches have been often observed but typically 1/2-inch to 1 1/2-inch disturbances occur. During Kona storms with heavy south swell free oscillations in the Ala Wai harbor up to 3 inches can occur.

The effect of seiches on the lagoons would be to produce an oscillating current of a few inches per second, maximum in the central lagoon entrance area, toward the bottom. This current would change directions at the lagoon natural

periods given above, but would likely not be observable in the presence of the consistent and stronger net flushing flow.

At the waterline, however, seiche amplitudes of 2 inches will cause the beach waterline to alternately change up and down 4:1 beach slope approximately 7 1/2-inches. In a possibly limiting case of a 20:1 beach slope and a 3-inch amplitude during storm conditions the run-up would be 60 inches at approximately 1/2 to 1 1/4 minutes periodicity.

Comment. Seiches, or oscillations of the lagoon free surface due to a naturally occurring force applied momentarily to upset the lagoon surface equilibrium, will probably occur, and have periods of oscillation of 1/2 to 1/4 minutes. Some small addition to lagoon benthic currents will result along with an oscillating character to the lagoon waterline causing run-up on the beach slope of a few feet.

Extremes Of Temperature, Salinity, Dissolved Oxygen, Currents And Lagoon Surface Waves

It is recognized that the factors causing changes in water properties, currents or lagoon surface waves would not likely all occur at the same time seasonally or diurnally. It is possible, however, to postulate the limiting case and develop an envelop of likely extreme values for each parameter. The following gives the results of that estimate for flushing rates of 1, 3, and 10 times the lagoon volume/day:

PARAMETER	FLUSHING RATE-VOL/DAY	MINIMUM	MAXIMUM
Temperature	1X	21.0°C (Feb.)	27.4°C(Aug./Sept.
	3X	22.5°C (Feb.)	26.8°C(Aug./Sept.
Salinity	1X	31.80 °/oo	35.40 °/oo
	3X	33.40 °/oo	35.60 °/oo
Dissolved Oxygen	1X	5.0 mg/l	7.5 mg/l
	3X	6.0 mg/l	11.0 mg/l
Mean Entrance-Exit Flow **	1X	0.1 ft./sec.	0.4 ft./sec.
	3X	1.2 ft./sec.	1.6 ft./sec.
	10X	3.0 ft./sec.	4.0 ft./sec.
Lagoon Surface Wind Waves *		Calm-Flat	0.7-1.1 ft. in hg 1.5-2.0 sec. Peri

* Under conditions of 30 kt. winds along the lagoon length

** Reference-1 kt. = 1.7 ft./sec.

Comment. The only extreme value of serious concern is the entrance exit flow from the lagoon. Values above 2.5 ft./sec. (1.5 kts) are

concernably high. The shallower the lagoon the less total volume must flux through, but flushing rates are in terms of lagoon volumes/day. The recommendation to consider flushing rates at or above 3 to 4 time /day stems from the problem of nutrient rich ground water infiltration. If, for example the infiltration rate used in this work is high by a factor of 2 and the nitrat concentration high by a factor of 3 (i.e. taking the lower estimates in the literature) the flushing rate could be reduced by a factor of 2 to 3, thus reducing flushing to approximately 1 lagoon volume/day (considering just conservation of nutrient loading, not biological changes.) If this were the case these lower flushing rates would ease the velocity of lagoon exiting water.

In the final design it is recommended that the shoal entrance shelf extending across most of the entrance of each lagoon, contain the optional feature that would allow the addition of "resistance blocks" of permanent material to be added to each shelf area as each lagoon ultimately dictates. The criteria to add such blocks to resist incoming waves would depend upon the infiltration rate ultimately experienced in each lagoon. With this design feature concernably high exiting flow velocities may be controllable, still providing sufficient flushing of infiltrating ground water.

Small Scaled Current Patterns in the Lagoons

The scale considered here refers to changes in the net flow pattern from entrance to exit that result from the lagoon hypsography. Small scaled current patterns will occur as the flow moves along curved portions of the beaches, or around shore cusps. Eddies of varying size will occur downstream of each obstruction to the general flow particularly along the seaward edges of each lagoon. This will be further modified by the cross lagoon wind drift. Specifically what current pattern exists in each lagoon will depend upon the prevailing winds, tidal profile, incoming waves and lagoon hypsography. Typical currents in the lagoon are expected to range from 0 to 48 cm/sec, averaging daily 17-34 cm/sec. The flow will be in over the entrance shoal areas along the beach line converging at the lagoon central area and exiting out the central channel, principally subsurface.

Comment. Care was taken in shaping and designing each final lagoon configuration. This is necessary to insure that stagnant areas will not exist that will harbor degraded water, or floating debris that reaches the lagoon surface. Incoming waves will refract uniformly into each lagoon and generally strike the beach orthogonally, some slight seasonal shifting of sand at the waterline is anticipated. The strength of the net flow, general lagoon shape, predominent wind directions, lagoon surface waves and potential infiltration stratification were considered in developing the specific final design and shape.

Lagoon Lining.

The lagoon lining material was selected considering anticipated incoming waves, sediment permeability and movement, asthetics and the conditions existing in the Kamokila lagoons. A sub-bottom of graded gravel packed for minimum porosity is to underlie each lagoon. Clean medium to medium coarse sand, 1/4 to 1/2 mm sized is proposed for the beaches surrounding the lagoons and coarse to very coarse sand (1/2 mm to 2mm sized) for pond lining over the hard substrata. In the deep central portion of each lagoon adjacent to the entrance area and exit channel small cobble (10 mm to 250 mm) should be used.

Debris from recreational use of the land surrounding the lagoons should also be controlled and not allowed, where possible, to reach the lagoon waters. Once in the water these materials can become water saturated, sink and become buried in the lagoon lining material. Cleaning them from the lagoon would be more laborous than original control on land. The surrounding land will also likely contain decorative vegetation of many types; a source for leaves, plant fibers and seeds. If blown into the lagoon waters they could contribute to the overall organic loading in the lagoon sediments.

Comment. Medium to medium coarse sand will line approximately 20% of each lagoon bottom and be distributed along the water line. Very coarse sand and cobble will line approximately 25% of the deeper portion of each lagoon adjacent to the entrance area. The balance will be coarse sand, graded and washed, over a hard, packed, substrata.

Bacteriological Injection

Bacteriological data is usually limited to coliform bacteria. The coastal waters off the lagoon site areas are historically very low in the coliform concentration. Total coliform (including fecal coliform) concentrations of 2 colonies/100 ml have been found during winter months offshore following storm conditions. The neighboring discharge sources of the Conoco refinery area, the streams north of the project site in the Kahe and Maile area, and the proposed new Barbers Point Harbor are each potential sources of enteric bacteria, nutrients, floatables and toxic material. The most likely source, however, for coliform bacteria reaching the lagoons will be the vegetation and grounds surrounding the lagoons. Fecal coliform and pathogenic bacteria can also enter the water directly through human usage. The coliform concentrations found both in the ground water and in the Kamokila Lagoon were considered negligible (i.e. less than 4 col/100 ml).

Comment. Considering the typical coliform decay rates in Hawaiian waters (90% in 30 to 45 minutes) and the apparent lack of major fecal or pathogenic bacteria sources in the area bacteriological (coliform) problems should be minimal.

Coastal Neighboring, Pollutant Sources and Feedback

Objectional water quality or toxic materials could enter the central coastal area, site of the lagoon intakes, and be drawn into the lagoons. As noted earlier, the offshore and coastal currents would be expected to bring water to the entrance area from the north to northwest for about half the year and from the south quadrant for the remaining half. The nearest site of concern for the source of objectionable water would be the Barbers Point Harbor and the West Beach Marina. During ebbing tides and offshore blowing tradewinds harbor water would tend to move twice daily northwest, toward and somewhat away from the site coastline. Also, during seasonal periods of significant incoming southerly or southwesterly swell and sea, statistically a minimum of 302 hours/year, or 13 days/year, the wave induced longshore transport would move the ebbing harbor water northward along the coastline. It would entrain surrounding open coastal water and become diluted in its objectional concentrations, but reach the lagoon intake areas within 1 to 3 hours. Primarily the small 2.2 acre southern most proposed lagoon would be effected.

Exactly what concentrations of what materials of concern will exist in this mix of harbor and coastal water cannot be said definitively at this point without specific knowledge of the new Barbers Point Harbor water quality. Estimates were made by Dugan (1984) of the storm data for the exiting water quality and the rates runoff for 1 and 24 hour durations, 1, 5, 10, 25, 50 and 100 year events. He used nitrogen and phosphorus concentrations of 0.6 and 0.5 mg/l respectively. The phosphorus is 2 to 5 times greater than lagoon infiltrating ground water and, along with suspended solids of 250 mg/l, could impose severe loading on lagoon waters

if allowed to reach the lagoon entrances. The southern most lagoon may recieve this stress during storm periods along with other pollutant loading reaulting from the intended commercial use of the Barbers Point harbor, i.e. the addition of sulphides, bacteria, oils, tars, floatables, hexane solubles, particulate material, trace metals and toxic materials.

Dugans results show severe storm runoff loading could occur for storms over 6 and 24 hour duration, recurring 50 and 5 years respectfully. The routing of storm drainage from the project coastal waters to the Barbers Point Harbor of the proposed West Beach Marina will add seasonally to the loading in these neighboring waters. When this occurs and flushes out of the Harbor-Marina area it will likely be of concern during times when the coastal drift is northwest, or during winter Kona storm periods when rainfall can be significant and much of Oahu's coastal waters receive large amounts of storm runoff containing fine suspended silt.

If the project storm drainage exists the coastline north, at the project northern boundary, discharged storm runoff could reach the lagoon intake areas within 2 to 4 hours, especially during conditions of northwest swell and sea, statistically 973 hours/year, or 41 days/year. In this case dillution with coastal water should be greater than a similar discharge from the south.

Comment. Coastal storm water discharged at the site nothern boundary and/or Barbers Point Harbor exiting water can reach the lagoon intake

area during specific conditions of sea and swell directions and/or net drift of the nearshore coastal waters. It is estimated that these conditions could exist between 13 to 41 days per year. The severity of this condition will depend upon the concentrations of undesirable materials in either neighboring sources. This subject is yet resolved in the proposed Barbers Point deep draft harbor case. In the case of storms, however, severe stress can occur at several conditions below the 24 hour 100 year storm. The historical data clearly show island wide prolific sediment discharge occurs during severe winter storms, as throughout the 1978 to 1979 anamalously wet winter. The coastal waters almost uniformly degrade for a few days until dispersed and flushed away from the shore by open coastal water transports. During these periods it may be wise to devise methods to segrate from the coastal waters or use standby pumping to accomplish the required aeriation filtering and/or chemical control of biostimulation.

MARINA

Marina Hypsography

Various marina configurations were considered during the development of the West Beach design effort, each change attempting to meet updated land planning and development cost needs. Marinas sized from a few 100 slips to over a thousand slips were sketched. The design marina was selected as a 33.9 acre approximately triangular facility 2900 feet long overall by 950 feet wide at the widest point. The main 2050 foot long marina portion is associated with an 870 foot long spending beach area, connected to either the Barbers Point deep draft harbor via a 400 to 450 foot wide channel or directly to the ocean via a 200 foot wide channel. These three configurations were used for the specific design marina dimensions and entrance shapes to complete the technical analyses of probable circulation patterns, flushing, seiche conditions, heat budget and infiltration. The potential coastal and environmental impact of the facility was also of interest. Depths in the marina were taken as 15 feet (mean lower low water) in the entrance channel and throughout the remainder of the marina. The average is expected to be 16 feet deep at mean sea level (m.s.l.).

The hypsographic computations considered the boundries of the marina to be vertical throughout. An incidental inaccuracy is introduced in the sloping sea walls along the entrance channel. As such, the final marina would be 33.9 acres, with a minimum overall volume at mllw of 820,844 cu. yds. (165.8 million gallons). The marina portion of the facility

is 551,122 cu. yds. (111.3 million gallons) in volume and remains the same for each entrance configuration.

The narrowest restriction in the facility is the 330 foot wide channel between the marina portion and a cresent shaped spending beach across from the opening to the Barbers Point harbor channel or the ocean. At this point a 4500 ft^2 (approximately at msl) vertical crossection exits. The marina entrance is proposed to either connect directly with or adjacent to the main entrance channel for the adjacent 90-acre Barber's Point deep draft harbor or possess a separate channel directly to the ocean. As such, the specific entrance configuration is an item of final agreement between the developer and those state and federal agencies with jurisdiction over the deep draft harbor.

All environment analyses considered the marina portion of the facility is the same for each entrance concept. The land surrounding the marina would be, in part, land altered during construction of the West Beach development and, the remaining eastern part bordering the deep draft harbor. It is assumed in the subsequent work that all grading around the marina will be directed away from the facility. Two exceptions are recognized for the direct routing of storm runoff drains to the marina and tsunami protection grading at the marina borders.

The marina final hypsography will be verified or (change if the marina concept is altered) following completion of the physical model tests

underway. Marina use, circulation, flushing, seiche behavior, and entrance configuration might, for example, require to be changed. The maximum sized marina surface area and mean depth will, however, not change. Therefore, the hypsographic results summarized above, were used for flushing computations, and the design shape used as for seiche computations. Unlike the lagoon concept development, where flushing and lagoon asthetics established to meet development needs. The hypsography will in turn then establish the environmental conditions that will occur in the marina.

Tidal Prisms.

The tidal prism is that amount of water that enters the marina during each tidal change. It is usually given as a fraction, or part, of the basin water depth or volume at some defined tidal reference level. The geometry and smoothness of the basin and its connection to the ocean determine whether a restriction to the incoming and outgoing tidal flow will exist or the marina will have a free unimpeded tidal exchange. In computing the tidal prism a mean lower low water (mllw) reference was used (i.e., marina depth at mllw of 15 feet).

The diurnal tidal range at the project coastline generally varies from 1.3 feet to 2.9 feet (mean of 1.9 feet). The tide is predominately mixed-semidiurnal in character throughout the year, i.e. two unequal highs and lows per day. Annually observed greater tidal changes in the area, i.e. diurnal changes of approximately 3.8 feet, occur a few times each year. However, a conservative average daily tidal

exchange of 2.0, feet considered constant throughout the year, was chosen for this work. The time of this tide at the site would follow the Honolulu tide by 10 to 13 minutes during the year.

The tidal prism computation results show the marina tide is impeded a negligible amount for an 4500 square foot cross-sectional entrance channel. The entrance configuration is constructed to absorb incoming swell and surf, being wider and possible deeper will further insure the tidal prism is not restricted.

The tidal prism for a typical 2-foot tidal change is 14.8×10^6 gallons, or approximately 13% of the marina volume. The exchange occuring over a single 12-hour, 25 minute tidal change will depend upon the prevailing semidiurnal tidal profile. With a 2 foot semi-diurnal tide a typical daily volume change of 27% of the marina volume would occur. During spring tides, this daily change could reach 33%. This does not however consider the efficiency of the exchange process nor consider the volume of infiltrating water, storm runoff or the evaporation - precipitation balance. Flushing involves some knowledge of mixing within the confined boundries of the marina.

Infiltration of Ground Water. Ground water from the surrounding aquifier will infiltrate the marina sides and bottom. The rate of entry of this brackish water will depend upon the amount and head of ground water existing in the area, changing locally season-to-season with rainfall and the variation of hydraulic permeability of the soils.

The geology of the area indicates the marina will lie entirely in the seaward most coastal zone containing an approximately 200 feet thick coral aquifier base. Beneath this brackish aquifier exists an aquiclude zone of alluvium and marine sediments above the basal aquifier. Bore data from seven holes taken in 1969 for the West Beach project and four new holes at the marina site by Harding Lawson Associates are available, though the seven historical holes lie well out of the area proposed for the marina. However, the project data plus data taken for the proposed Barbers Point deep draft harbor just to the southeast can be and were used to estimate the geology in the marina area and the hydraulic permeability.

Just northwest of the marina the geological stratification has been shown to include a 6-inch to 18-inch soils layer, a coral sand layer, and dense structured coral layer overlying open-structured coral to the 10-foot depth below mean sea level (msl). The dense coral has typically the lower hydraulic permeability, i.e. lower than the coral sand or open structure coral. Harding Lawson estimates permeability rates of 0.1 to 10 ft/day throughout the West Beach site. In the marina area, 15 feet below msl, principally medium dense to dense sediments exist (i.e. 20 to 108 blows/ft) of grey-white to light brown cemented, saturated, sediments. This material surprisingly exhibited, however, a rapid pump off rate indicating a high permeability. The sediments must be quite inhomogenous or fractured from spot to spot.

The existing neighboring Barbers Point harbor is approximately 38 feet deep. It apparently cuts vertically downward through the coral sand, dense-structured coral, open-structured coral, and possibly some alluvial silt and sand, in that order. As such, it breaks several more permeable materials than the marina excavation might, were the marina depth limited to 16 feet below msl. Thus, as in the lagoon case, infiltration rates, as justified by historical data and HLA field data are expected to range between 0.1 to 10.0 ft/day permeability, but may tend toward the lower end of this range. The highest permeability found by HLA was approximately 48.6 gal/day/ft^2 in the marina area.

Using the highest infiltration rate, it is estimated that the overall 33.9 acre surface area marina 16 feet deep (msl) could receive between 4.3 mgd to 5.0 mgd of infiltrating ground water. This water will be less saline (25.5 $^{o}/oo$ observed) than the incoming sea water (approximately 34.8 $^{o}/oo$) and therefore bouyant. Considering the overall marina volume at msl, in this case 165.8 million gallons, and a non-flushing condition the infiltrating water will comprise approximately 2.6% to 3.0% of the marina volume. As such, it could comprise approximately 21% of a typical diurnal 2-foot tidal prism classifying the marina as a wet-embayment.

The above estimates assume the local foothills and mountains receive less than 20 inches of rainfall per year, typical for the Waianae district. This area averages approximately 100 mgd rainfall dispersed as 15 mgd in overland stream runoff to the coast, 75 mgd evapotranspiration,

and up to 10 mgd ground water seepage. Thus, a marina infiltration rate of 4.3 to 5.0 mgd is conservative since the neighboring deep draft harbor and coastal waters should receive a major portion of infiltrating water from the surrounding lands. This conservative rate could, however, be exceeded during periods of significant storm activity.

The incoming brackish ground water may vary in salinity from 6 °/oo to 26 °/oo. In that it is less saline than the incoming tidal flow, the marina may tend to become slightly stratified, with a lighter, shallow, less saline surface layer existing in the marina throughout the year, similar to the experience found in the neighboring Barbers Point harbor.

The infiltrating water will add to the net amount of water leaving the marina daily. This assymetry in inflow/outflow should add to the water daily exiting the marina slightly increasing the outgoing tidal prism.

If for example, a 2-foot diurnal tidal prism is considered, and a 4 mgd infiltration rate (evaporation approximately equaling precipitation), the outgoing mean flow at the marina entrance over a 6-hour period for varying semi-diurnal tidal changes would be 1.1 cm/sec. Peak velocities might therefore reach 3 cm/sec (0.06 knots) in the entrance channel during ebbing tides. Offshore blowing winds might tend to increase the exiting slow surface slightly. If the stratification were to limit the

outflow to a bouyant 3 to 4 foot thick layer at the surface, the exiting flows could increase to perhaps 0.1 knot. It appears that, therefore, infiltrating ground water is not expected to cause any difficult flow conditions in the marina.

Incoming storm runoff, to be routed into the marina at three locations, like infiltration will add substantial nutrient loading to the marina waters. The storm runoff, however, could also add a substantial flow to the marina. Dugan (1984) has estimated that the project's east drainage area could deliver 375.9 acre-ft/event to the marina during 24 hours for the 100 year recurrance interval, i.e. the most severe storm. This is almost equivalent to three quarters of the total marina volume, exiting the marina entrance in 24 hours; capable of producing a maximum outgoing flow of 17 to 25 cm/sec in a 3 to 4 foot surface layer. This could cause navigational difficulties for small boats using the marina channel.

Marina Heat Budget

Incoming solar radiation will annually add heat to the water in the marina while heat will be lost via evaporation and back radiation. A small amount of heat will also be exchanged at the surface via sensible heat. The net month-to-month result, or heat budget, is responsible for the annual cycle of heating and cooling of the marina waters. This seasonal change in turn effects stratification in the marina, thus also circulation. It is necessary to estimate the proposed marina heat budget to properly examine each of the above subjects as each is pertinent to the conditions anticipated in the marina environment.

The site exists at the foot of the Waianae range on the southern end of the Waianae coastline. This area tends to be dry, open to the tradewinds, and under less cloud cover than most of Oahu. Each component of the local heat budget was computed using mean monthly meterological data and the method outline previously for the lagoons. The maximum gain in incoming heat, occurring between April and June, was found to be +580 cal/in^2/day. The rate of heat loss, most significant during the months of February, October, and December, is up to -420 cal/in^2/day.

In general, the marina waters are expected to gain heat from March to September and lose heat from October to February. Annually the area is expected to gain approximately 50 cal/in^2/day. As a result the marina waters are expected to be approximately 0.1°C warmer than the coastal waters if the maximum gain is considered distributed over the entire marina depth. If confined to a surface buoyant layer the

temperature may be 0.4°C greater. This would serve to promote a slight stratification in the marina. A balance in heat gained and lost will thus be reached within the marina via slight changes in evaporation and back radiation and through exchange of the marina waters with coastal water.

An estimate of the evaporation - precipitation balance at the surface of the marina was also computed. Approximately 90 in/year are expected to evaporate from the surface and 20 in/year to precipitate on the surface. This gives a monthly average net evaporation rate of 5.8 inches per month, or in terms of volume, approximately 5.34×10^6 gallons per month. Evaporation increases salinity, and the water evaporated is replaced with combined incoming coastal water and infiltrating ground water. This replacement, however, will be very small, i.e. less than 1% of the daily tidal prism. The more significant effect will be in the stratification induced by diurnal heating at the surface and infiltrating brackish ground water.

Stratification and Water Quality.

Incoming solar radiation, the balance of precipitation and evaporation, and infiltrating ground water will act to alter the temperature and salinity of the water in the marina. The first two act at the surface and the latter through the sides and bottom of the marina. In the absence of vigorous mixing or overturn within the marina, i.e. that occurring from an active circulation and wave action, stratification will likely develop.

A warm, less saline surface layer will form and overlie a cooler, more saline subsurface layer. The salinity of the lower layer would be maintained by exchange with water in the deeper incoming tidal flows. The amount of stratification in the surface layer would vary diurnally with daily heating and cooling, but variations seasonally with infiltrating ground water, rainfall changes and heat budget changes will be more significant.

It has been previously noted that the marina waters are expected to be a least 0.1°C warmer annually than the adjacent coastal waters. If this heating effect is limited to the upper few feet of the water column, the temperature stratification may be up to 0.4°C. In addition diurnal heating and cooling could result in up to 0.5°C additional change, peaking during mid-day hours. Overall the increase in temperature toward the surface may annually average just under 1°C.

Infiltrating ground water is expected to range in salinity

between 6 to 26 °/oo. The estimated typical daily infiltration rate was given earlier at approximately 5 mgd. If vigorous mixing were to occur throughout the water column, of 20% infiltrating ground water, the marina salinity would drop 0.4 °/oo. If in the worst case 6 °/oo water were to enter the marina, primarily from the sides, and mix with the surface 2-foot layer the salinity in this layer would drop approximately 5 °/oo. If this same process occurred but with 20 °/oo infiltrating ground water, the surface layer salinity would drop 2.5 °/oo.

A weak but consistent surface stratification is thus very likely to be evident in the marina annually; being most evident during winter storms. In comparison the existing Barbers Point harbor presently demonstrates a measurable stratification. Wind blowing across the marina surface will likely thus induce different circulation patterns, one in a shallow surface layer a few feet thick and a second in the subsurface layer just beneath. Such two (and three) layered systems have been observed for example in Pearl Harbor. In the case of the marina, density differences top to bottom in the water column are expected to exist in the marina throughout the year due to approximately a 0.1°C to 1.0°C and 0.4 °/oo to 5.0 °/oo vertical gragient of temperature and salinity.

The incoming water from infiltrating ground water will be high in nutrients, particularly in nitrogen. Periodic incoming storm runoff will be low in salinity and high in phosphorus and suspended solids. In the limiting case given by Dugan (1984) for suspended solids of 127.7 tons/24 hour 100 year storm the solids entering the marina would either settle

to the bottom or remain suspended in the water for subsequent flushing from the marina. Given typical particle size distribution in suspended load the settleable portion would be distributed approximately within a few hundred feet of the discharge point and would accumulate to a few millimeters thick; i.e. typically 500 ft and 2 to 3 mm/event.

Incoming brackish ground water is expected to contain 1.0 to 8.5 mg/l total nitrogen, 0.05 to 0.25 mg/l total phosphorous and 260 to 330 mg/l bicarbonate. Storm runoff water has been estimated by Dugan (1984) to contain 0.6 mg/l total nitrogen and 0.57 mg/l of total phosphorus. In comparison, the coastal waters in the area have been found to contain 0.15 to 0.38 mg/l of total nitrogen and 0.003 to 0.065 of total phosphorous, i.e. generally comparatively low levels of these nutrients. Thus the mean ratios of ocean water to storm runoff to infiltrating ground water would be for total nitrogen 1:3:25 and total phosphorus 1:17:3. Infiltrating water will be the prime source for nitrogen and storm runoff of phosphorus for the marina waters.

If 5 mgd of infiltrating brackish ground water and the remaining marina background (coastal) water are considered well mixed upon reaching the marina entrance, the exiting water quality, in the limiting case (not including storm runoff) would contain approximately 0.4 mg/l total nitrogen, 0.04 mg/l total phosphate, 12.2 mg/l bicarbonate, and have a temperature 0.2°C warmer than 0.4 °/oo lower than coastal waters. The nitrogen is about twice coastal waters with little difference in phosphorus from coastal water. However, during storm periods and

attendant storm runoff with high phosphorus concentrations, the above ambient nitrogen load and an added phosphorus loads could cause both nitrogen and phosphorus levels in water exiting the marina to be 2 to 3 times typical coastal water concentrations, for the duration of the storm event. In the presence of the stratification induced by a buoyant discharge the surface layer water exiting the marina daily will likely contain the highest nutrient concentrations.

In general nutrient concentrations in the marina will be greater than coastal waters and during storms greater than lagoon waters. The marina will receive a greater portion of ground water infiltration per square foot of marina surface area than the lagoons (i.e. the marina is deeper) plus potentially receive periodic injection of storm runoff. In addition, the marina use is exploitive and its hypsography restricts flushing more than the in the case of the coastal lagoons.

The dissolved oxygen concentrations expected in the marina will principally be a function of the marina water temperature oxygen uptake by the marina phytoplankton community, and replacement of oxygen via vertical and horizontal mixing of new water with resident water. Dissolved oxygen concentrations in the coastal waters is expected to seasonally range from 6.0 to 7.5 mg/l and for infiltrating ground water from 2.8 to 4.8 ms/l. Within the marina, if circulation and mixing is sluggish toward the bottom, dissolved oxygen concentration may be depressed to 3 to 4 mg/l. In extreme cases within the marina sediments, dissolved oxygen may drop to 1 to 2 mg/l as has been observed in Pearl Harbor and the Ala Wai Canal.

Circulation In The Marina.

Circulation in the marina will be established by several factors. These include the marina hypsography and orientation to the prevailing winds, the daily tidal exchange, and the stratification within the marinas.

The marina is on the eastern most end of the project border. It extends approximately 3300 feet longitudinally landward at approximately a 40° angle to the coastline. The landward most 2050 foot portion of the marina is approximately triangular, 950 foot across at its widest point. As such the prevailing tradewinds will blow in the seaward direction angularly across the marina. The flow at the surface will thus tend to move toward the downwind (west) corner of the marina (as will much of the floating debris). Overturn will be aided by this cross marina seaward flow.

It has been shown in a preceding section that the marina waters should typically exhibit some weak vertical stratification throughout the year. This will be due primarily to heating at the surface and to infiltrating brackish ground water with salinities between 6 $^{o}/oo$ to 26 $^{o}/oo$. A warmer, less saline surface layer varying in thickness throughout the marina should be evident at all times of the year. The separation of this surface layer from cooler, more saline subsurface waters will depend upon the degree of mixing existing in the marina from day-to-day. This mixing will be induced by the wind, wind waves in the basin, obstructions to the flow (as from slips and pilings), harbor use by small craft, and circulation patterns set up within the marina.

The basin shape and orientation to the prevailing winds will be the principal factors in establishing the overall horizontal patterns of flow within the marina. The prevailing offshore blowing trades strike the 2050 foot long border of the marina with the Barbers Point harbor blowing almost across the 950 foot width of the marina. A weak southwest cross marina net daily flow at the surface is expected that will tend to transport buoyant surface water angularly across the marina. The thickness of the less surface layer should be greater on the western side of the marina, with the surface layer thickness increasing toward the entrance of the marinas. Some downward sinking of water on the west side would be compensated for by a weak vertical motion upward along the eastern side of the marina. The seaward and across basin wind drift in the shallow surface layer would be compensated by a much weaker eastward moving subsurface flow.

This two layer wind induced circulation in a varying stratified case will cause a very slow mixing and overturn of the marina waters. An analysis of expected circulation rates, amount of stratification differences within the marina, and tidal flows was made to estimate the degree of mixing and exchange of resident water with incoming coastal water and infiltrating water. It was estimated that the residence time of marina portion of the facility, without inducting flushing artifically, would be between two to five days. These overturn rates are 2 to 3 times slower than the maximum theoretical overturn rates computed by comparing the marina volume and tidal prism.

During flooding tides water entering the marina via the entrance channel (all entrance concepts) should exhibit subsurface movement upwind into the basin, particularly along the Barbers Point harbor seawall. During periods of weak tradewinds or Kona southerly winds this flow will likely even be evident at the surface, though weak, unless the basin is receiving a significant volume of fresh water storm runoff. Flooding tidal flows into the basin may be up to 0.3 kts, the stronger flows found in the subsurface layer and toward the entrance.

A typical flooding current profile would show weak incoming flow surface and an increase in strength in the subsurface layer. This flow will move around obstructions and corners in varying eddy patterns. During ebbing tides the exiting flow throughout the water column should be seaward but be strongest and first to change seaward in the surface layer. The maximum out-going flows could reach 0.4 kts during strong tidal changes or periods of significant storm runoff.

Both incoming and outgoing flows will tend to diminish in strength further into the marina. Care was taken in shaping the marina to avoid creating stagnant areas or debris traps, particularly on the western (downwind) side and landward most portion of the basin.

Flushing Schemes and Limits.

Flushing criteria for the previous lagoon concepting work was established with the intent of providing as optimum a quality of water for recreational contact use as possible or practical, given the source coastal water quality and anticipated lagoon environmental conditions. Flushing rates in terms of lagoon volumes per day are obtainable. The necessity to dilute infiltrating nutrient laden ground water to control biostimulation and produce lagoon stability were used to establish a minimum desirable flushing rate. The marina criteria is not as stringent and is established principally by marina hypsography, stratification, circulation and the prevailing tidal prism.

Flushing of the marina under typical conditions would be accomplished through the mixing of entering water with resident water and the subsequent removal of this mix via the daily tidal changes and the net daily outflow from infiltrating ground water. The efficiency of this process is judged by the difference in water quality of the entering and exiting water. Given these data the effectiveness of the mixing and exchange processes can be evaluated.

The marina is considered to typically receive up to an additional 5 mgd of infiltrating ground water. The ground water will be the least saline and most nutrient laden water entering the marina. The marina entering coastal waters will be the most saline and low in nutrient concentrations. Within the marina a varying mix of these source

waters will exist, acted upon by the resident biota, further altering the marina water quality.

The tidal prism for a 2.0 foot tidal change in the overall marina is 22.1 million gallons. In the minimum case of one two-foot diurnal tidal change occurring in 24 hours and 50 minutes 13% of the marina volume would enter and leave daily, carrying an additional 3% of the total volume in a net ground water exiting flow. Thus this minimum volume leaving daily is approximately 26.5 million gallons, representing 16% of the overall marina volume. This volume is increased to 53 mgd (32%) in the case of two semi-diurnal tides. If completely new water were exchanged each tidal cycle the marina overturn would occur in about two days. In practice the process of mixing and flushing is not, however, that efficient.

Considering the anticipated stratification and circulation within the marina a slow, three to five day, turnover may be expected. Experience from other bays and harbors in Hawaii in the presence of similar phenomena, particularly in partially stratified cases, shows similar turnover rates for shallow flat bottom basins with an unhibited subsurface flow. The incoming infiltrating ground water will add to the daily net outflow. Though the maximum theoretical turnover volumes can sum to one third of the marina volume in practice between 6% to 11% of the total marina volume is expected to be turned over daily. The most active exchange will occur in the surface layer and least active in the bottom waters. The surface 2 to 3 feet might be flushed daily while the bottom waters will take several days.

Winds and Waves in the Marina.

Tradewinds at the site are expected to vary in strength annually from 5 to 20 knots, averaging 12.2 knots. The predominant tradewind direction is from 070° true. Trades are expected to occur up to 90% of the summer days and 50% of the winter days. Periods of calm occur 1.5% of the year. Kona conditions of weak winds and Kona storms with winds from the south and west quadrant may occur a few days each year. Storm wind speeds generally reach 30 knots. The strongest wind, reported as observed at the site on January .3-14, 1979, was 40 knots sustained, gusting to 83 knots.

The trades predominate throughout the year and will thus be responsible for producing the typical wave climate found within the marina. Surf, swell and storm surge entering the marina will of course be the far more significant wave activity that could cause damage within the basin if not removed at the marina entrance by the proposed spending beach. The subject of sea state conditions at the marina entrance is discussed by Gerritson (1984). The tradewinds blowing across the marina will, however, induce circulation, as has been discussed earlier, plus establish a wind chop within the marina with reasonably consistent wave activity.

It is estimated that under typical tradewind conditions small 0.1 to 0.3 foot waves with 1.5 to 2.0 second periods are expected in the marina. During extreme conditions of strong trade or 40 kt Kona winds the fetch across the marina could allow waves up to 1.3 feet in height to develop. In this case the western side and seaward most portions of the

marina will have the greatest wave activity from tradewinds. This side will also tend to accumulate most of the surface debris and floatables reaching the harbor from use of marina and surrounding grounds. During Kona storms the landward-most berths of the marina will experience the greatest wave activity.

Seiche Behavior.

A seiche is an oscillating behavior of the free surface of the marina. Though all the causes of such oscillations, that can occur both along and across the marina, are not fully understood, it is known that only a small amount of energy is needed to produce and maintain a standing oscillation in a body of water. Weak periodic variations in wind direction or strength, rapid barometric pressure changes, entrance surge and surf conditions or incoming storm runoff will produce seiches in the marina, particularly if these variations are of a periodicity corresponding to one of the natural periods of the marina as determined by its shape and depth.

The proposed marina portion of the facility is basically triangular in configuration with an entrance channel off one apex. The overall marina surface will thus oscillate both along the 2920 foot seawall direction and across the 950 ft marina. The method used to numerically compute the marina free oscillations was presented in the previous discussion on lagoon seiching. The computation was completed for the marina considering 1 to 5 nodes of oscillation.

If the marina, with its narrow entrance channel, tends to perform as a closed basin these oscillations will have a maximum period single nodal oscillation of between 4.2 to 4.4 minutes (mllw to mhhw) along the marina length, and 1.3 to 1.4 minutes across the marina width. Amplitudes of vertical motion for the single node closed basin case may be up to a few inches, similar to those seiche oscillations presently observed in the

Ala Wai Harbor, Keehi Lagoon and Kewalo Basin. In the entrance channel cross channel oscillations with periods of 20 to 21 seconds may develop. A possibility exists in this case for the channel to co-oscillate with any storm surge with 18 to 24 seconds periodicity.

In the more significant case, the marina may oscillate as an open basin, that is open to the ocean via a direct entrance channel. The single node (at the entrance) case for an open basin gives marina oscillations with periods from 9.4 to 10.1 minutes. The semi-diurnal and diurnal tides would likely not co-oscillate with the marina in this case since marina length is too short. However, the local periods for some tsunamis approach these periods, and severe problems of seiching in the marina could occur during tsunamis. Further, it has been reported that the configuration of the Barbers Point deep draft harbor could produce harbor oscillations in a "pumping mode" (single entrance node) with a maximum period of 13.3 minutes. These neighboring basins could share seiching energy producing undesirable marina oscillations if the entrance spending beach is ineffective.

The periods for multinodal cases become less critical as do the amplitudes of these oscillations. Using the single node case and assuming a maximum disturbance of 6 cm the maximum horizontal current velocities resulting from an oscillation along the length of the marina would be 1.4 cm/sec toward the bottom central area of the marina. This small re-reversing flow at the bottom of the marina, with a 5 minute periodicity, is significant only to help induce mixing and overturn within the marina. It poses no navigation concern.

Once the marina begins free oscillations, the oscillation, or "sloshing" will slowly decay at a rate given by the boundry friction within the marina. A comparative analysis made with data from Kaneohe Bay indicates the marina oscillations should decay approximately 20% in amplitude each successive oscillation. That is a 6 cm disturbance should decay to 1.5 cm in height within 60 minutes.

Coastal Effects.

The neighboring coastal waters will be influenced by water daily exiting the Barbers Point deep draft harbor. Approximately 1/4 of this exiting water may be from the marina. The volume of water discharged each day from the marina will depend upon several conditions prevailing at the time. These include the tidal prism occurring within the 24-hour period, the rate of ground water infiltration for that season of the year, the net result of evaporative losses versus direct precipitation on the surface, and the prevailing conditions of storm drainage, if any, into the marina. The two most significant of these factors are the infiltrating water (rich in nitrogen) and storm runoff (rich in phosphorus).

The "low" extreme condition would be a day of weak tidal changes and calm-flat coastal sea state, occurring during the driest season with evaporation exceeding precipitation, a weak rate of ground water infiltration and no storm drainage. Under these conditions approximately 23 to 28 mgd would leave the marina, flowing into the coastal receiving waters. The "high" extreme could occur during a spring tide, occurring during the wet season with a high ground water infiltration rate and precipitation exceeding evaporation. Under these conditions approximately 55 to 64 mgd would leave the marina daily. If the latter conditions also occurred during a severe Kona storm, i.e. when the marina was receiving storm runoff from a 100-year storm with a 24-hour duration, the volume leaving the marina exit could reach 155 mgd during the day of the storm.

Both the quality and volume of water leaving the marina effects coastal conditions at and around the marina entrance. Estimates of the quality of the exiting water were presented earlier. In any case the volume of water being discharged will leave the marina in two daily ebbing tides, i.e. mixed semi-diurnal conditions prevailing in Hawaii during the year. Thus between 12 to 14 million gallons may leave the marina entrance in any typical 6-hour 25-minute ebbing tidal period, increasing to 80 million gallons during a severe storm. Averaged exit velocities for these flows from the marina entrance over the entire 15-foot deep water column would be between 0.02 to 0.1 kts. Exiting in just a 3 foot deep buoyant surface layer the flows would be 0.07 to 0.4 kts. Only in the latter case of a severe winter storm might marina exiting water noticably influence coastal conditions around the Barbers Point harbor entrance.

The slightly buoyant water leaving the marina would tend to remain in the coastal surface layer. During spring and summer months, flooding tides, or periods of onshore southerly surf or Kona winds, the discharged water from the marina would tend to flow northward along the project coast, and reach the southern most lagoon intake area within 1 1/2 to 2 hours. Dilutions at that time might reach 5 to 20:1. A portion of this degraded (nutrient rich) water might be drawn into this lagoon. In this case this small southern most lagoon would likely show a greater biostimulation than the remaining lagoons. The prevailing coastal circulation is the important factor in this case and altering the marina channel alignment would have no substantial change on this condition.

The longshore transport may in turn also be affected by the marina discharged water, particularly in the immediate area of the marina exit. If, in a limiting Kona wind case, the combined marine-harbor daily exiting water stays within 500 feet of the shore (i.e. 0 to 10 foot depth), it would add approximately 15% to the longshore transport. This flow would tend to follow the existing flow from tidal currents and coastal surf set-up. During periods of slack tides and calm sea states the exiting water would move radially outward from the marina entrance. During prevailing trades the marina-harbor discharge water would tend to move in the south-southwest to west-northwest quadrant.

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WEST BEACH PROJECT

TECHNICAL EVALUATION AND RECOMMENDATIONS ON DESIGN OF MARINE STRUCTURES

by

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0.0 SUMMARY, CONCLUSIONS AND RECOMMENDATIONS

0.1 Summary

This report deals with ocean engineering analysis of design criteria and technical recommendations for lagoons and marina of the West Beach Project.

The scope of work is detailed in the Introduction (Section 1.0) of the report.

Areas of study efforts are wave conditions and design considerations (Section 2.0), wave run-up on open coasts and lagoons (Section 3.0), littoral drift and sediment transport (Section 4.0), design aspects of lagoons in planform and profile (Section 5.0), marina design considerations (Section 6.0) and project planning aspects (Section 7.0).

References are given in Section 8.0 and acknowledgements in Section 9.0.

Regarding design wave conditions two different sets of situations must be distinguished:

* operational conditions,
* extreme conditions.

Discussions are provided on both sets of design conditions.

As basis for wave analysis, wave data collected by Sea Engineering Inc. in 1981 and 1982 were used as input.

For the evaluation of wave run-up and lagoon dynamics special consideration must be given to nearshore wave dynamics

as they affect wave set-up, surf beat, and wave group behavior.

Although the latter phenomena find themselves in the middle of scientific interest and investigations, design information on these aspects is still inadequate and therefore estimates have to be made instead.

Evaluation of the amount of littoral drift, of the effect of littoral drift on lagoon regime and of the effect of lagoons on littoral drift and on adjacent shorelines are difficult to make because of the uneven distribution of sand over the nearshore seabottom. Nevertheless estimates have been provided regarding the gross rate of littoral drift and sedimentation and erosion rates of the lagoons.

From a design point of view recommendations have been provided with respect to coastal elevations and set-back criteria, on lagoons planforms and profiles and on various marina entrance design alternatives.

Finally the author participated in project planning and coordination and attended meetings with the Corps of Engineers in Honolulu, and with the State Harbors Division.

0.2 Conclusions

The observed wave data provide valuable insight into the yearly wave climate. There is a distinct difference between summer and winter conditions.

The long period swells from northwesterly directions during the winter months provide the dominant design aspects

for operational conditions.

Waves above a certain height create unsafe swimming conditions for the average swimmer.

To obtain an idea of how often such conditions would occur, it was assumed that a wave height of 2.5 ft. or higher in the lagoon would create unsafe conditions for the average swimmer. The corresponding wave height outside of the entrance is estimated at 7.5 ft. Waves equal to or higher than 7.5 ft. are expected to occur on the average only a few days per year during the winter months. During the wave observations in 1981 and 1982 this critical level was exceeded only one time.

Extreme design conditions are of interest because of stability of coastal and harbor structures.

Determination of the extreme design conditions can be done in different ways, e.g. by selecting a design frequency or by carrying out an optimum design analysis. For the latter information on initial cost and repair cost of structures are required.

For this stage of conceptual design it is assumed that a design frequency of 1/100 per year will be appropriate.

Wave observations carried out for this project were of too short duration (less than one year) to serve as a basis of long term trends.

Based on other sources the 1/100 per year wave height for this area is estimated at 35 ft.

Bretschneider (1984) gives for the West Beach Design Hurricane a wave height of 34.7 ft (significant wave height) and a period of 12.1 sec. (significant period). For Hurricane Iwa calculated values are H_s = 41.0 - 42.7 ft. and T_s = 14.2 - 14.7 sec.

Values of T_s = 14.5 sec. and T_s = 16 sec. have been used in wave run-up calculations.

Extreme wave heights as in Hurricane Iwa will give the largest wave set-up and storm surge effect on tide level.

Extreme wave conditions do not necessarily generate the severest wave attack on coastal structures because of energy dissipation due to bottom friction and breaking.

The severest waves attacking coastal structures may be significantly less in height than the extreme waves associated with a hurricane and may occur more frequently.

The effects of wave groupiness and irregular waves on wave run-up are considerable. Estimates have been made for those effects in the calculations. The values of wave run-up obtained are therefore considerably larger than the values given in Bretschneider (1984).

Estimated values for littoral drift, sedimentation and erosion rates are orders of magnitude only, indicative of the processes in qualitative sense.

Order of magnitude for sedimentation and erosion of lagoon beaches are estimated to be the same. For any given year, however, erosion may exceed sedimentation and visa versa.

0.3 Recommendations

1. Recommendations are given regarding sand size, beach slope, and planform of lagoons. (Plates I-IV). It is emphasized that at any given time, actual slope and planform may deviate considerably from calculated values. A considerable margin should be provided in set-back guidelines.

2. It is recommended, that the walkway which encircles the lagoons should have an elevation of no less than 13 ft. above M.S.L. Minor overtopping during extreme conditions may still occur.

   It is recommended that the set-back distance between walkway and 6 ft. beach contour should be no less than 100 ft. to allow for dynamic adjustments in beach profile and planform as a result of varying wave conditions.

3. Under extreme conditions existing coast elevations will be overtopped by wave run-up. An increase in coastal elevation and/or an adequate set-back distance from the shoreline must be provided to prevent damage to structures in the coastal zone near the shoreline.

   Based on wave run-up calculations the following recommendations are made:

   a) Typically the southern section of the project shoreline can best be protected by a 50 ft. wide coastal berm at elevation +16 ft. (M.S.L.) (See Figure 3.3)

b) Typically the northern section of the project shoreline is best protected by a berm of varying elevation of a distance of 250 ft. from the shoreline (Figure 3.3)

From a wave run-up point of view this berm elevation may vary between +12 and +16 ft. However to provide adequate protection against tsunami run-up, a minimum elevation of +15 ft is recommended (Bretschneider, 1984) The +15 ft barrier against tsunami run-up may also be located further inland, however, if infrequent inundations of the areas near the shoreline are accepted in the overall planning concept.

4. Because of wave spray accompanying wave run-up during high waves it is recommended that floor levels of buildings near the coast be no lower than 21 ft.

5. Plans for alternative solutions for the marina entrance are presented (Plates V through VII)

The solution with the joint use of the deep draft harbor channel (Plate V) is considered the most attractive one from both economic and environmental points of view. This solution provides an all-weather type of marina, which is considered a strong advantage.

In case the joint use of the deep draft harbor entrance channel would be selected the installation of a warning system is recommended. This can be a joint effort between Coast Guard, U.S. Army

Corps of Engineers, State Harbors Division and Developer.

The joint use of the deep draft harbor entrance channel by both commercial and recreational vessels is advantageous from an environmental point of view for the following reasons:

(1) no loss of potentially productive reef area for boat channel;

(2) maintenance dredging offshore limited to deep draft channel.

On the other side of the environmental evaluation the water quality in a marina with direct connection with the ocean may be less degraded than the water quality in a marina connected with the deep draft harbor.

However, it must be realized that the marina has its own sources of pollution (ships, run-off) which are not affected by the entrance channel connection.

6. In view of the degree of accuracy present in current computational techniques for the stability and dynamics of lagoons it is recommended one lagoon be constructed before construction of the other lagoons, to conduct a measurement program on wave conditions and beach behavior over a period of time (e.g. one year) to verify some of the theoretical concepts used in the analysis of sandy beach behavior and of the amount of artificial nourishment required.

Results of this study can then be used in the final design of the other lagoons.

At present it may be prudent engineering practice to expect that on the average sand loss from the lagoons will occur and that therefore artificial nourishment will be necessary to compensate for the loss of sand.

7. Recommendations on water quality, flushing, and long wave response of lagoons and marina are provided in K. Bathen (1984).

2.0 WAVE CONDITIONS AND DESIGN CONSIDERATIONS AT PROJECT SITE

2.1 Available wave data

In 1981 and 1982 Sea Engineering, Inc. carried out wave observations at an offshore locations at project site. Reference is made to Figure 1.1 where the station for wave observation is shown. The recorded wave data include 798 34-minute wave data sets. The recording instrument was a pressure sensor at a depth of 43 ft. The data are of good quality and include an analysis of the distribution of wave energy over various spectral periods (Sea Engineering, 1983). Some aspects of the wave data are analysed in Bretschneider (1984).

Additional wave data are presented in Look Laboratory Report #8 (1970), which include statistical hindcast wave data for the Barbers Point area. These data were used for the deep draft harbor model study and originate from the Honolulu District, U.S. Army Corps of Engineers. These data are included in Bretschneider (1984), Section III, (Appendix I), Tables A and B.

Frequency distributions for significant wave height and wave period obtained from those tables are presented in Figures 2.1 and 2.2.

The Sea Engineering Inc. data have been measured at a depth of 43'; therefore direct comparison between wave height distributions of the Corps data and of the Sea

Engineering data is not possible. The impression is that there is a fair agreement between these two types of data as far as wave height is concerned.

Regarding wave period a direct comparison is possible. (see Figure 2.2). The two distributions are somewhat different. In both distributions the highest frequency is in the period-class of 11-13 sec.

Observed wave periods are more frequent in the 9-11 sec. class and less frequent in the higher period classes above T_s = 13 sec., compared to the hind cast data.

The deep water data from the Corps of Engineers' Tables were analyzed according to wave direction. The results are shown in Figure 2.3. The dominant directions are NNW, NW, SSW and S. In the Sea Engineering data wave direction was not measured.

Using data from other sources an estimate for the 100 year wave height for storms can be made. The estimated value is H_s = 35 ft (10.7 m). This value corresponds with Bretschneider's recommended design wave height of 34.7 ft. (Bretschneider, 1984). Bretschneider recommends for a hurricance a design period of 12.1 sec., although for Hurricane Iwa calculated periods are 14.2 and 14.7 sec.

For the characteristic period in waves two different parameters are often used. They are the significant wave period (Ts) and the peak period (Tp).

The significant wave period is defined as the mean

period of the one-third highest waves of a record. The peak period is defined as $Tp = \frac{1}{fo}$, when fo is the frequency at which the spectral energy density has its highest value.

From the available data a histogram was constructed for the ratio $(\frac{Tp}{Ts})$. The results are presented in Figure 2.4. The highest number of occurrences is found at Tp/Ts = 1.15. This corresponds approximately with Bretschneider's analysis.

2.2 Selection of design wave conditions for swimming lagoons

In defining design wave conditions two separate types of conditions must be considered:

(1) operating conditions, and

(2) extreme conditions for design

The first refer to safe conditions for swimming and bathing, the second to safety of structures.

In order to obtain a criterion for operating conditions an assumption must be made as to what condition constitutes a limit of safety.

A point of consideration is that swimming and bathing will be done inside the lagoons; those beaches are protected from extreme high waves by shallow depths at their entrances. Potential danger to swimmers includes two aspects:

a. direct wave impact

b. wave induced return currents through deep part of entrance.

In this section we will limit ourselves to (a). For this discussion we are assuming that the limit of safety is reached when wave heights on the beach exceed 2.5 ft. The corresponding wave height outside of the lagoon entrance depends very much on the water level. At regular high tide the wave height outside the entrance is estimated at 3x the wave height inside, leading to a wave height outside of the entrance of 7.5 ft. (2.3 m). Waves equal to or higher than 7.5 ft. occur only a few days per year. During the wave observations of 1981 and 1982 this level was exceeded only one time.

In a typical year these conditions will occur during the winter months. Because of the dominantly shallow entrance the effect of wave period on wave height inside will be small.

2.3 Test conditions for the hydraulic model of the marina

The data from section 2.1 also provide information on what conditions to use for testing of various design alternatives in the hydraulic model of the marina.

Figure 2.3 shows that dominant wave directions are NW, WNW, SSW and S. However waves from W, WSW and SW do occur, although less frequently.

Observations show that for a specific wave height the variation in wave period may be considerable. The testing program should express this variability.

Extreme conditions are presented by waves such as

produced by Hurricane Iwa. Waves with Hs = 10.70 m (35 ft) and Ts = 12-15 sec. provide the extreme design conditions for the project and should be included in the model testing.

2.4 Wave Transformation in the Coastal Zone

Wind waves and swells when approaching the coastline from deep water are subject to transformations, in which the following processes play a part:

* shoaling
* refraction
* diffraction
* energy dissipation due to bottom friction
* energy dissipation due to breaking
* wave set-up
* change in form of energy spectrum.

The mathematical formulations for the dissipation process are described in a recent paper by the author.

In this paper some simplifications have been introduced: parallel bottom contour lines are assumed and diffraction and stream refraction are neglected.

Reference is made to this paper for further information on the mathematical formulation.

The model was applied to West Beach. For the two-dimensional formulation, traverses T1 through T6 were used as basis for the calculations (see Figure 2.5).

The calculations were carried out for the following sets of wave conditions at the measuring station:

(1) Hs = 2.0 m; Ts = 12 sec.; Dir SSW, SW

(2) Hs = 3.5 m; Ts = 14 sec.; Dir NW

(3) Hs = 7.0 m ; Ts = 16 sec.; Dir NW

(4) Hs = 10.0 m; Ts = 12 sec.; Dir SSW, SW

Results of Calculations

The calculated wave heights are expressed in H_{rms}-values, which is quite customary when energy considerations are involved. The results are shown in Tables 2.1 through 2.6 in which the following values are listed:

* Wave height at breaking (m), column 2
* Depth at breaking (m), column 3
* Angle of wave approach at breaking, (o), column 4
* Wave height at 10 ft. contour (m), column 5
* Wave direction (relative to normal) at 10 ft. contour (column 6)
* Wave set-up at d=o contour (m), (column 7).

The method of parallel contours gives rise to difficulties in interpretation where the depth contours make a bend. This is the case between traverses T5 and T6, which cross each other at the shoreline. In the nearshore region (d = 10 ft) the results for T5 and T6 can be averaged to obtain expected values.

The results of the calculations, as tested in tables 2.1 through 2.6 reveal the following trends (excluding harbor entrance):

(1) Regardless of wave height and wave direction, waves at the 10 ft. contour are always highest in T_1 and lowest in T_6.

(2) Even with extremely high waves in deep water (Hs = 10 m at d = 43 ft), energy dissipation and refraction reduces the significant wave height to not more than 2.6 m (H_{rms} = 1.85 m) near the coastline.

(3) Deep water waves of low height dissipate much less energy. For Hs = 2.0 m (at d = 43 ft) the wave height will be the same near the shoreline, for directions SW and SSW and periods of 12 sec.

(4) High waves generate a considerable higher wave set-up than low waves. The ratio of $\bar{\eta}/H_{rms(43')}$ at the shoreline in T_1 ranges from 0.073 to 0.127, with an average value of 0.1. The wave set-up values are taken at the original shoreline (d=0'); the theoretical maximum values $\bar{\eta}_{max}$ will therefore be somewhat higher.

Mean wave set-up values are dependent on wave steepness. This is confirmed by the results of the calculations. Reference is made to Gerritsen (1981).

(5) Results of calculations indicate that the ratio of shallow water wave height and deep water wave height decreases as the wave height increases. A graphical presentation is given in Figure 2.6.

(6) As a final result lines of equal wave height (H_{rms} - values) are sketched in Figure 2.5 for the 3.00 m, 2.00 m and 1.00 m wave height contour; waves are from the N.W., $H_{rms}(100')$ = 3.75 m and T_s = 14.0 sec.

Table 2.1

Wave Direction: S.S.W.

H_s (43') = 2.00 m (Traverse #3)

H_{rms} (43') = 1.42 m (Traverse #3)

H_{rms} (100') = 1.40 m (All traverses)

T_s = 12 sec.

TRAVERSE #	$H_{rms/h_b} = 0.4$			H_{rms} (3m)	α (3m)	WAVE SET-UP AT d = 0
	$H_{rms(b)}$ (m)	h_b (m)	α_b (o)	(m)	(o)	(m)
(1)	(2)	(3)	(4)	(5)	(6)	(7)
1	1.49	3.73	14.16	1.44	12.93	0.17
2	1.41	3.56	13.81	1.33	12.85	0.17
3	1.32	3.32	13.35	1.27	12.85	0.16
4	1.18	3.07	16.78	1.18	16.78	0.15
5	1.12	2.68	15.70	1.11	16.77	0.14
6	0.86	2.08	17.12	0.84	20.83	0.10

Wave height attenuation and wave set-up for waves from S.S.W.

H_s = 2.00 m; T_s = 12 sec.

Table 2.2

Wave Direction: S.W.

H_s (43') = 2.00 m (Traverse #3)

H_{rms} (43') = 1.42 m (Traverse #3)

H_{rms} (100') = 1.40 m (All traverses)

T_s = 12 sec.

TRAVERSE #	$H_{rms/h_b} = 0.4$			H_{rms} (3m)	α (3m)	WAVE SET-UP AT d = 0
	$H_{rms}(b)$ (m)	h_b (m)	α_b (o)	(m)	(o)	(m)
(1)	(2)	(3)	(4)	(5)	(6)	(7)
1	1.51	3.73	5.97	1.46	5.46	0.18
2	1.43	3.56	3.83	1.35	5.43	0.17
3	1.34	3.32	5.63	1.28	5.43	0.16
4	1.23	3.07	10.51	1.23	10.51	0.16
5	1.15	3.06	10.51	1.15	10.51	0.15
6	1.03	2.43	15.48	1.03	17.37	0.13

Wave height attenuation and wave set-up for waves from S.W.

H_s = 2.00 m; T_s = 12 sec.

Table 2.3

Wave Direction: S.S.W.

H_s (43') = 10.00 m (Traverse #3)

H_{rms} (43') = 7.07 m (Traverse #3)

H_{rms} (100') = 11.50 m (All traverses)

T_s = 12 sec.

TRAVERSE #	$H_{rms/h_b} = 0.4$			H_{rms} (3m)	α (3m)	WAVE SET-UP AT d = 0
	$H_{rms(b)}$ (m)	h_b (m)	α_b (o)	(m)	(o)	(m)
(1)	(2)	(3)	(4)	(5)	(6)	(7)
1	2.74	5.29	17.46	1.83	13.79	0.61
2	2.22	5.54	17.79	1.62	13.73	0.60
3	2.09	5.20	17.28	1.53	13.72	0.58
4	1.75	4.46	20.87	1.42	16.62	0.46
5	1.61	4.05	19.98	1.31	17.61	0.45
6	1.16	2.78	20.46	1.16	21.42	0.29

Wave height attenuation and wave set-up for waves from S.S.W.

H_s = 10.0m; T_s = 12 sec.

Table 2.4

Wave Direction: .S.W.

H_s (43') = 10.00 m (Traverse #3)

H_{rms} (43') = 7.07 m (Traverse #3)

H_{rms} (100') = 9.00 m (All traverses)

T_s = 12 sec.

TRAVERSE #	$H_{rms/h_b} = 0.4$			H_{rms} (3m)	α (3m)	WAVE SET-UP AT d = 0
	$H_{rms(b)}$ (m)	h_b (m)	α_b (o)	(m)	(o)	(m)
(1)	(2)	(3)	(4)	(5)	(6)	(7)
1	2.17	5.29	7.32	1.85	5.80	0.60
2	2.28	5.70	7.54	1.64	5.78	0.60
3	2.17	5.45	7.39	1.57	5.78	0.57
4	1.83	4.57	13.19	1.46	11.08	0.50
5	1.77	4.43	13.02	1.34	11.08	0.49
6	1.19	3.06	18.00	1.19	18.00	0.35

Wave height attenuation and wave set-up for waves from S.W.

H_s = 10.00 m; T_s = 12 sec.

Table 2.5

Wave Direction: N.W.

H_s (43') = 3.50 m (Traverse #3)

H_{rms} (43') = 2.48 m (Traverse #3)

H_{rms} (100') = 3.75 m (All traverses)

T_s = 14 sec.

TRAVERSE #	$H_{rms/h_b} = 0.4$			H_{rms} (3m)	α (3m)	WAVE SET-UP AT d = 0
	$H_{rms(b)}$ (m)	h_b (m)	α_b (o)	(m)	(o)	(m)
(1)	(2)	(3)	(4)	(5)	(6)	(7)
1	1.87	4.66	24.67	1.65	20.06	0.22
2	1.82	4.55	24.30	1.47	19.95	0.21
3	1.69	4.20	23.33	1.39	19.95	0.20
4	1.69	4.22	20.89	1.38	17.87	0.23
5	1.47	3.67	19.53	1.27	17.88	0.22
6	1.27	3.17	12.04	1.23	11.84	0.27

Wave height attenuation and wave set-up for waves from -

H_s = 3.50m; T_s = 14 sec.

Table 2.6

Wave Direction: N.W.

H_s (43') = 7.00 m (Traverse #3)

H_{rms} (43') = 4.95 m (Traverse #3)

H_{rms} (100') = 8.30 m (All traverses)

T_s = 16 sec.

TRAVERSE #	$H_{rms/h_b} = 0.4$			H_{rms} (3m)	α (3m)	WAVE SET-UP AT d = 0
	$H_{rms(b)}$ (m)	h_b (m)	α_b (o)	(m)	(o)	(m)
(1)	(2)	(3)	(4)	(5)	(6)	(7)
1	2.15	5.29	26.04	1.82	19.99	0.36
2	2.23	5.54	26.55	1.61	19.88	0.35
3	2.09	5.20	25.70	1.51	19.86	0.33
4	1.94	4.80	22.14	1.50	17.92	0.40
5	2.05	5.20	23.03	1.39	17.92	0.40
6	-	-	-	1.36	11.99	0.51

Wave height attenuation and wave set-up for waves from N.W.

H_s = 7.00m; T_s = 16 sec.

3.0 WAVE RUN-UP

3.1 Wave run-up on the open coast

A very important design aspect of the West Beach Project is the wave run-up against shores and beaches during storms and periods of high swell.

Wave run-up is defined as the vertical distance between the highest point on a slope or structure that is reached by the waves and the mean water level at the toe of the structure.

Wave run-up is related to the following parameters.

(a) beach slope or slope of coastal structure, $m = \tan \alpha$ (α is angle with horizontal plane),

(b) wave steepness (H / L_o or H_o / L_o),

(c) depth of water at the toe (d_b),

(d) roughness of slope (expressed by wave run-up reduction coefficient, r),

(e) permeability of slope,

(f) presence of berm,

(g) form of the wave spectrum (expressed in spectral width parameter ε),

(h) dynamics of surf beat,

(i) long period surges in mean water level due to low pressure fields, or sudden changes in atmospheric pressure,

(j) composition of slope.

The available wave data provide information on wave height and wave period for a period of less than one year.

Wave characteristics for extreme conditions must be obtained from other sources including hindcasting of waves during storms and hurricanes.

Information on surfbeat dynamics must also be estimated from other sources.

Calculations of wave run-up

Several methods are available to calculate wave run-up for regular and irregular waves:

(1) Using graphs of Shore Protection Manual (1973)

(2) Using Hunt's empirical formula:

$$R = \sqrt{HLo} \tan\alpha \qquad (3.1)$$

in which H represents the wave height, L_o the wave length in deep water and α the slope angle (with the horizontal plane), in combination with calculating the breaker depth from:

$$\frac{d_b}{H_o} = 0.39 \left(\frac{Ho}{Lo}\right)^{-1/3} \qquad (3.2)$$

which formulation is recommended in Shijf (1972) in combination with Saville's compositie slope method. Although the validity of (3.2) is only approximately correct it seems to give fair results in combination with Saville's method.

(3) Use of Hunt's formula, using breaker height H_b determined at the breaking point, and use of composite slope method.

(4) In irregular waves, Hunt's formula and

(a) use of H_{rms} (root mean square wave height) at breaking point and correction factor for irregular waves;

(b) use of H_{max} at toe of structure or shore barrier.

The method of calculating wave run-up is an iterative method. From the point of breaking and from an estimated value of the wave run-up the composite slope angle α is determined from:

$$\tan \alpha = \frac{R + d_b}{\Delta x} \qquad (3.3)$$

where R is the estimated wave run-up and d_b the breaker depth. Δx is obtained from the offshore profile.

The obtained value of R is compared with the estimated value and a correction is made to account for the difference.

In a few iterations the estimated and calculated run-up will be approximately the same.

In the following, results will be presented using methods 2 and 4. However control calculations have been carried out for the other methods to verify the methods used.

The formulas and graphs used for the calculations have been obtained from laboraotry experiments with regular (deterministic) waves.A corrective scale factor is usually applied, but in our calculations it is assumed to

be included in the factor for irregular waves.

In reality waves show irregular behavior, displaying a distribution of wave height and wave period, a distribution of energy in the form of a wave spectrum, and the occurrence of groups of high and low waves. The long wave associated with the group behavior is called surf beat.

In addition to wave run-up values, obtained for regular waves estimates must be made for the effects of irregular waves.

Wave run-up is calculated using a modified form of the Hunt's equation:
(Schijf, 1972);

$$R(n) = C(n)\varepsilon \sqrt{H_S\, g\, \hat{T}^2}\, \tan\alpha \qquad (3.4)$$

Hunt's formula (3.7) can be written in the same manner, as follows:

$$R = 0.4 \sqrt{H_0 g\, T^2}\, \tan\alpha \qquad (3.1a)$$

In expression (3.4) Hs is the significant wave height and $\hat{T}$ the spectral period at which the wave spectrum has its highest energy density. (Ref. Schijf, 1972) The coefficient $C_{(n)}$ depends on the spectral width parameter ε and the percentage (n) that the wave run-up is allowed to be exceeded. In engineering practice a level of 2% is often used.

The relationship $C_{(2)}\varepsilon = f(\varepsilon)$ is given in Schijf (1972) and is included as Figure 3.1. For the spectral width

parameter an (arbitrary) value of 0.4 is assumed for all calculations.

The ratio R(n)/R can be obtained from the above formulations.

The effects of the dynamics of surf beats and of low pressure fields must be estimated additionally.

A note of interest is that due to Hurricane Iwa the debris-line on the landside of the Barbers Point Harbor basin under construction, was found at a consistent elevation of +7 ft. (personal communication Gary Wible, Resident Engineer, U.S. Army Corps of Engineers). This may be a confirmation of the considerably increased mean waterlevel, associated with this hurricane.

Results of Calculations

Calculations have been carried out for profiles $(\frac{1}{1})$, $(\frac{2}{2})$, $(\frac{1}{2})$, $(\frac{2}{2})$ shown in Figure 3.2 (planform) and in Figure 3.3 (profiles)

Wave run-up characteristics are dominated by the following features:

(a) wave heights at the coast increase from North to South,

(b) slope steepness decreases in the same direction.

These two features have opposite effects on wave run-up. Profile $(\frac{2}{1})$ is at close proximity to a depression in the seabottom which provides a steep slope. Profile $(\frac{1}{1})$ also has a steep slope between -5 ft. and +6 ft. (M.S.L.)

The results of the wave run-up calculations are summarized in the following table. The design water level is +4 ft (M.S.L.)

Table 3.1

Wave run-up at selected profiles, using different methods

Profile	Method	H(ft)	T(sec)	tanα	Run-up elevation (ft)	Remarks
$(\frac{1}{1})$	(4)-a)	3.8	16	0.147	24	$H=H_o=H_{rms}$; Profile extended
$(\frac{1}{1})$	(4-b)	8.8	16	0.05	11.5	$H=H_{max}$; profile with berm
$(\frac{2}{1})$	(2)	8	14.5	0.11	25	$H=H_{rms}$; profile extended
$(\frac{2}{1})$	(4-a)	5.8	16	0.125	26	$H=H_{rms}$; profile extended
$(\frac{2}{1})$	(4-b)	12.8	16	0.069	15	$H=H_{max}$; profile with berm
$(\frac{1}{2})$	(4-b)	12.8	16	0.056	13	$H=H_{max}$; existing profile
$(\frac{2}{2})$	(2)	10	14.5	0.044	15	$H=H_o=H_{rms}$; extended profi

The calculations have been carried out for the existing profile, for an extended profile or a profile with a berm.

It may be noted that for profiles $(\frac{1}{1})$ and $(\frac{2}{1})$ wave run-up will be excessively high for the extended profile so that a berm set back a certain distance from the shoreline will become an attractive solution.

Based on the run-up calculations the following recommendations are made: (see Figure 3.3)

a) The southern section of the project shoreline, characterized by profiles ($\frac{1}{2}$) and ($\frac{2}{2}$) should have a 50' wide berm at an elevation of 16 ft. (M.S.L.).

b) The Northern part of the coastline is best protected with a berm structure respectively at elevation +16 ft (profile ($\frac{2}{1}$)) and at elevation +12 ft (profile ($\frac{1}{1}$). The berms are located at a distance of 250 ft. from the shoreline.

However, in view of the possibilities of tsunami inundation it is recommended that the minimum elevation of the berm be +15 ft. (see Bretschneider, 1984). The 15 ft. barrier may also be somewhat further inland if (infrequent) inundation of coastal lands is accepted.

In the calculations for run-up various aspects of irregular and dynamic wave behavior are incorporated. Reference is made to section 3.1, items (a) through (j). The following considerations apply:

re (a): considered in formula.

re (b): considered in formula

re (c): a correction of +1 ft. may be used to account for extra wave set-up,

re (d): a reduction of -1 ft may be feasible due to rough slope,

re (e): no reduction applied,

re (f): no low berm included in calculations,

re (g): a correction factor of 1.77 was calculated based on $\varepsilon = 0.4$.

re (h): a correction of +(1-2) ft. may be applied

re (i): a correction of +(2-3) ft. is possible during a storm

re (j): incorporated in formula.

The effects of the above phenomena will not all occur at the same time so that not all corrections must be applied simultaneously for each case. For the estimates of items c, d, h, and i a correction of +3 ft. has been applied to the calculations.

It is furthermore assumed that the effects of short waves riding on top of long waves (estimated by Bretschneider (1984) at 17%) are included in the factor 1.77 applied under (g).

In conclusion it may be observed that wave run-up is a determining parameter for setting design elevations on the open coast. The criteria exceed those of tsunami run-up.

The situation is different inside the marina basin, where wave run-up is small and where the design water level condition is dominated by tsunamis.

An area of transition is the entrance to the marina.

Run-up conditions will be evaluated from the model experiments presently under way at the I.K.K. Look Laboratory

3.2 Wave run-up on lagoon beaches

Wave run-up on lagoon beaches is somewhat different

from the open coast for the following reasons:

(1) wave height is reduced due to shallow entrance configuration;

(2) beach slopes consist of porous sand;

(3) waterlevel may become higher due to oscillations and additional wave set-up.

(4) average wave period is reduced due to wave breaking on entrance sill.

In the process of calculations, first an estimate is made about the wave conditions in the lagoon. After that the wave run-up is calculated using Hunt's formula. In this calculation the maximum wave height, H_{max}, rather than the root mean square wave height, H_{rms}, will be used, but without correction for the form of the wave spectrum, (method 4-b).

The design wave height is based on the design water level. For this an extreme elevation of 6 ft above M.S.L. is used. This includes the effects of extra wave set up and surf beat oscillations.

Wave energy enters into the lagoon over the wide and shallow sections (elevation - 1.5 ft. (M.S.L.)) and through the narrow gap in the middle.

It is assumed that the waves passing over the shallow sections are the dominant waves; they will be used as basis for the calculations.

With a sill of -1.5 ft. and a water level of +6 ft.,

water depth over the sill is 7.5 ft.

The maximum wave height over the sill is then calculated by:

$$H_{max} = \gamma_b \, h_b$$

with $\gamma_b = 0.8$, and $h_b = 7.5$ ft.,

$$H_{max} = 0.8 \times 7.5 \text{ ft.} = 6.0 \text{ ft.} = 1.83 \text{ m}$$

For the calculation of wave set-up a reduced wave period of T = 10 sec. is assumed which gives $L_o = 156$ m = 512 ft.
For the calculation of run-up, tan α is estimated at tan $\alpha = 0.1$ and the effects of porosity by r = 0.9

$$R = 0.9 \sqrt{1.83 \times 156} \times 0.1 = 1.52 \text{ m}$$

$$= 5.0 \text{ ft.}$$

The run-up elevation is then

$$Z = 6 + 5 = 11.0 \text{ ft.}$$

During high wave conditions and an exceptionally high waterlevel the berm may be raised to an elevation of +10 ft. and a profile similar to Figure 3.4 may be developing:

The assumed value of tan $\alpha = 0.1$ seems therefore justified.

It is to be noted that corrections of the type (a) through (j) of section 3.1 are not necessary here, because they have already been incorporated in the design water level considerations.

The above calculations may be expected to have an inaccuracy of 10-20%, so that elevations of 13 ft may be reached.

The elevation of the beach walk should therefore be no less than +13 ft. and may be occasionally overtopped during extreme conditions. It is recommended that the distance between beach walk and 6 ft contour line should be not less than 100 ft.

3.3 Surfbeat

One of the characteristics of irregular waves is that waves travel in groups, in which a group of higher waves is followed by a group of lower ones.

This group behavior of the waves induces a long wave phenomena called surf beat. The height of such waves are approximately 1/10 of the deep water wave height, whereas the period may be of the order of 7-8 times the dominant wave period.

In the surf zone the wave group effect is reinforced by the varying mass transport due to breaking, increasing the height of the surf beat wave.

Unfortunately no measurements are availble at West Beach to document these phenomena; however it is expected that they have a distinct effect on the surfrone processes such as wave set-up, wave run-up, excitation of lagoons and longshore currents. In the absence of measurements estimates must be made as to the effect of the long period phenomena such as surfbeat on the shore dynamics. In section 3.1 some of these effects are listed and estimates for the effect on run-up are given.

Stationary maximum wave set-up is estimated at 0.16 of the deep water wave height (Gerritsen, 1981). During severe storms (H = 20 ft) this will amount to about 3 ft. Due to the dynamical aspects of the problem, it is estimated that during sets of high waves the wave set-up may be increased to double its mean value, which is 6 ft. With a high tide of 3 ft (including some meteorological effects) the mean sea level of the coast could reach values of 9 ft (MSL). Such will significantly increase the maximum level of wave run-up, as discussed in the previous sections.

The period of the surf beat type oscillations is important for the excitation of oscillations in the lagoons. With a wave period of 14 sec. it may be expected that surf beat oscillations have a period of 98-112 seconds (Gerritsen, 1984). Shorter or longer ocean waves will naturally have different surf beat periods. The induced surf beat wave height in the surface is estimated at 1-2 ft. for average conditions and 3-4 ft for storm conditions. It is important to compare the surfbeat period with the resonance periods of the lagoon. Dr. K. Bathen calculated the resonant periods to be in the range of 47-82 sec. There exists the possibility of resonance during storms. Such resonance will increase wave run-up and may induce unsafe swimming. Surf beat oscillations in the lagoon may then be of the order 1-2 ft.

4.0 LITTORAL DRIFT AND SEDIMENT TRANSPORT

The phenomena which we are concerned about are:

a) Wave induced sediment transport in the surf zone;

b) Wave and current induced sediment transport outside surf zone;

c) Sediment transport into and out of lagoons.

For the West Beach project an estimation of actual sediment movements is particularly difficult because of the following reasons:

(1) only part of the seabottom and coastal zone is covered by sediment

(2) no directional information is available to calculate alongshore energy fluxes.

The methodologies to calculate the sediment transport are either based on wave energy concept or on a shearstress induced transport.

The first method is documented in the Shore Protection Manual (1973), the second in a paper by Byker (1967). For the problem at hand (sedimentation in lagoon) the first method is preferred.

Calculations based on this method lead to an average gross transport rate (see Trindade, 1982) of 500-1000 m^3/day. The fact that only part of the seabottom is covered with sand gives a reduction to 1/2 to 1/3 (estimate) of the transport rate. This leads to an average alongshore

transport of 160 - 500 m^3 / day or an equivalent of 60,000 - 180,000 m^3 per year.

Assuming that 10% of this transport enters each of the lagoon entrances gives a sedimentation rate of 6,000-18,000 m^3/year. It is furthermore estimated that the rate of erosion for the lagoons is of the order 5,000 - 10,000 m^3/year, which is of the same order of magnitude as the sedimentation rate.

It is to be emphasized that the above figures are gross-estimates and only give an order of magnitude estimate of possible conditions.

To obtain the best information on sand movement along the reef and its impact on the lagoons the construction of one lagoon executed at an early date which is then monitored for a period of approximately one year could provide detailed information on design for the other lagoons.

At this stage in design it is considered prudent practice to assume that over a period of a year or several years the sand losses from the lagoons will exceed the sedimentation rates, so that artificial nourishment may be required.

5.0 CONCEPTUAL DESIGN OF LAGOONS

5.1 General Considerations

The West Beach Project has as one of its major features the development of a number (4) of lagoons lined with sandy beaches. The lagoon concept evolved from elongated basins connected with the ocean by channels or ducts to provide the necessary flushing to semi-circular or elliptically shaped areas in which the beaches are shaped and maintained by natural wave action.

The latter are modelled after the existing natural lagoons of the adjacent Campbell estate to the north of the project.

For the lagoons the general design criteria may be defined as follows:

(a) Beaches must be naturally stable but may fluctuate in slope and planform due to the dynamics of ocean waves and nearshore processes.

(b) Flushing must be adequate to maintain clean water.

(c) Entrance structures must be stable to withstand wave action under all conditions.

(d) Swimming and bathing must be safe under common conditions; unsafe conditions may occur only infrequently during conditions of high swell.

In this report we will discuss aspects of items a, c, and d. The aspects of flushing and water quality will be dealt with in Dr. K. Bathen's report.

5.2 Stability of lagoon beaches

The stability problem has three important aspects.

* expected beach slope
* expected plan form
* expected changes in beach slope and plan form as a result of changing wave and tide conditions.

Beach slope

In order to make an estimate of the prevalent beach slope the two dominant parameters are: wave characteristics and sediment characteristics.

It has been known from observations on many beaches that the slope of a beach varies depending on the characteristics of the beach sediments and the degree of wave exposure. (Gerritsen, 1978)

As far as the characteristics of the sediment are concerned, various factors play a part but the most significant one is the grain size of the beach material.

Figure 4.1 shows the relationship between beach slope and sand size at mid-tide for a number of Hawaiian beaches. In this figure three curves are drawn: For exposed beaches, for moderately protected beaches and for protected beaches. These curves are taken from Bruun (1976) (original source Bascom (1951)) and have reference to mainland beaches; they show that on the whole Hawaiian beaches fit the characteristics of mainland beaches. Wave exposure is a not sharply defined term; it indicates, however, that generally high

exposure is characterized by high waves as well as high period.

The beach slope is usually taken as an average slope between the M.L.W. and M.H.W. water lines. In some instances this slopes extends itself seaward to the breaking point.

Regarding the slope of beaches, some beaches in exposed areas are characterized by one or more offshore bars, on which the waves break. In less exposed beaches this is usually not the case, although some minor bar(s) may be present.

The beaches of the West Beach lagoons may be considered to be in the category moderately exposed beaches, and therefore the middle curve of Figure 4.1 may be applicable.

In the design of the beaches we have a choice to select the diameter. It is preferable to select a coarse beach sand with average diameter of 1/2-1 mm and an estimated beach slope of 1:5 to 1:6.

The fairly steep beach slope induces a wave run-up, which is higher than for a beach of lesser steepness. However the coarse sand is much less likely to go into suspension in the breaking waves.

If a coarse sand is selected, the loss of sand from the beach by wave induced currents is consequently also of lesser magnitude.

Due to the shape of the plan-form the beaches at the two corners of the semi-circular or elliptical bay are less

exposed and the beach slope may be expected to be somewhat reduced in these areas. Furthermore some sorting of beach material will take place whereby the finer particles will move to the more protected sections.

For design purposes it is assumed that opposite the middle of the gap the beach slope will be 1:6, ranging from -3 to +6 ft. (M.S.L.) In the more protected areas on the side the beach slope will be reduced to an estimated value of 1:8.

In nature most natural beaches conform to a parabolic cross-sectional shape. For design purposes the proposed beaches have a 1:12 slope below -3 ft. to -6 ft. Below this level a slope of 1:50 to -8 ft. is assumed. (See plate 1). The above gives a schematized representation of the parabolic beach profile.

Due to the impact of wave action a sorting of beach sediment also will take place along the beach profile. The largest particles are found in the vicinity of the plunging point of the breaking wave.

Plan form

In order to evaluate the expected plan form of the beaches, conditions at the entrance must be specified.

The philosophy of entrance design involves in general the creation of two wide and shallow entrance areas, in which waves break at all times and by which wave induced mass transport enters the lagoon. A relatively deep and narrow

section provides the opening, through which the wave induced flow can return to the ocean.

With an assumed entrance profile wave energy flux entering the lagoon can be estimated.

A methodology has been developed to calculate equilibrium plan forms under given wave impact and has been applied to the West Beach lagoons (Gerritsen and Dayananda, 1984). In this method the following assumptions are made:

1) Waves approach the coastline at right angles. With small apertures (as in the case under study) the effect of wave direction on plan form is assumed to be small.
2) The diffraction of the waves inside the lagoon is calculated using the numerical method of La Combe.
3) The wave pattern before reaching the beach slope, is also obtained by calculating the wave patterns from the propagation of energy flux (Huygens' method)
4) On the beach slope wave refraction is taken into consideration.
5) Equilibrium plan-form conditions were then obtained by calculating radiation stress components parallel and perpendicular to the coast and the resultant gradients in water level.

An equilibrium shape is obtained when the resultant longshore velocity equals zero. Results of the various calculation for the lagoons #1-4 are given in plates I through IV. For details of the methodlogy reference is made

to Gerritsen and Dayanada (1984).

Beach dynamics

The calculated beach slopes and plan form are subject to change due to changes in wave characteristics and water level conditions, such as:

(1) During high swell conditions, the water level outside the lagoon will be considerably increased (2-3 ft) due to wave set-up.

This changes the ratios between energy fluxes through the shallow portions and deep portions of the gap and thereby influence the shape of the plan form.

A distinct wave direction difference from the normal (perpendicular) one will also affect the shape of the beach making it asymmetrical rather than symmetrical.

(2) During those conditions more energy will pass through the gap and the beaches will tend to erode, depositing the material in deeper sections.

(3) During high swell conditions wave group effects will be considerable thereby inducing a long period oscillation of the water level in the lagoon and on the beach. Due to this effect wave run-up will be increased which will affect the level of the berm.

(4) Changes in beach profile and planforms will also be induced due to the effects of hurricanes, such as Hurricane Iwa in November 1982.

(5) In view of the above wave and beach dynamics it will be necessary to leave sufficient space between the proposed average beach profile and any structures. This includes the proposed walkway along the beach.

A minimum distance of 100 ft. between the walkway and the +6 ft. beach contour is recommended.

5.3 Entrance Structures

It is assumed that the local barrier of beach rock, that runs along the entire project, is of such quality that it will withstand wave action at the entrance.

Further studies on the quality of this rock are necessary to evaluate whether or not this rock has the assumed structural integrity.

At this point in time it is furthermore assumed that no further protective works are necessary to restrict the maximum wave action entering the lagoon.

In this respect it will be necessary to take a closer look at the conditions that are assumed to constitute safe swimming conditions and their frequency of occurrence.

Such analysis will then reveal whether or not it will be necessary to design protective structures outside the lagoon entrance to limit maximum wave energy inside the lagoons.

5.4 Safe swimming conditions

Safe swimming conditions are characterized by the following criteria:

(1) wave heights not to exceed a maximum wave height

(2) wave induced currents not to exceed maximum values

(3) return flow through gap not to exceed maximum values

(4) long period water level fluctuations not to exceed certain magnitude. The following criteria are suggested:

* Wave heights inside lagoons not to exceed 2.5 ft.
* Wave induced currents not higher than 1.5 kn in shallow entrance area.
* Wave induced return flow through middle gap not to exceed 1.5 knots
* Long period water level fluctuations not to exceed 0.5-1 ft.

Lagoons will be designed in such a manner that under operational conditions the above criteria will be met. During extreme weather and/or wave conditions these limits may be exceeded but timely warnings should be given to advise against swimming under those conditions.

A special study is presently underway to evaluate waves and currents during winter conditions at the existing Kamokila Lagoons. This study will provide valuable additional information for the detailed designing of the West Beach lagoons.

It must be noted that long period oscillations induced by surf beat and by wave group effect on breaking induce strong velocities in entrance cross-section. These velocities also depend on the depths of the entrance sections.

6.0 MARINA DESIGN CONSIDERATIONS

6.1 General Considerations

The proposed marina at West Beach covers approximately 37 acres and is intended to provide private slips for sail and power boats, slips for commercial boats for charter fishing and dinner cruises, a public boat launching ramp and support facilities such as fuel dock, pump out facilities, car parking and some modest repair facilities.

This report does not deal with the general planning aspects of the marina but limits itself to the hydraulic and navigational aspects of the problem.

It also does not include considerations about flushing and resonance problems. The latter will be discussed in a separate report by Dr. K. Bathen.

The Marina is approximately 2900 ft. long (including 900 ft. of entrance channel) and is 900 ft. wide at its widest point. The basin will be 15 ft. deep below MLLW and will have a minimum channel width of approximately 200 ft.

From a navigational point of view design requirements differ for the marina basin and entrance channel. In the entrance channel allowable wave heights are higher than in the basin.

Specific elements to be considered in the design are:

- Configuration, size and orientation of the basin, adequate to meet project requirements;

- An entrance channel that provides safe navigation conditions to the marina
- Measures to prevent wave action from entering into the basin;
- Measures to reduce wave agitation in basin;
- Measures to prevent serious sedimentation of entrance channel.

A short discussion of several design considerations is given in this report.

6.2 Marina entrance

In this section we will discuss entrance location and special measures required for dissipating wave energy.

Conceptually there are three possible locations for the entrance to the marina:

- entrance inside deep draft harbor,
- separate traffic lanes in an enlarged combined entrance channel for both commercial and recreational navigation
- separate entrances for deep draft harbor and marina

The solution to be selected must meet the following requirements:

- safe manuevering for incoming and outgoing boats (adequate width of shipping lanes)
- adequate orientation with respect to prevailing winds;
- absence of strong circulatory flow patterns (eddies) which adversely affect navigation and promote sedimentation in entrance;

- absence of strong wave reflection and confused seas;
- adequate vision for both traffic entering and leaving marina and in areas where commercial and recreational traffic lanes interfere;
- minimum adverse effect on the littoral drift and consequently on the stability of adjacent shorelines.

The alternative design solutions for the marina entrance are being tested in a hydraulic model in the J.K.K. Look Laboratory of the University of Hawaii.

Wave conditions in marina entrance

It is of interest, both with respect to marina design and navigational concerns to make an estimate of the wave height at the marina entrance. For the joint channel solution the marina entrance is defined as the area represented by measuring station #3 in the deep draft harbor model study. (Figure #6.1)

Separate Channel. For the separate channel a location at the crossing of the axis of the approach channel and the original shoreline defines the entrance area.

The ratio between the nearshore wave height (at 10 ft. depth) and the deepwater wave height was calculated in section 2.2. Reference is made to Figure 2.6, which refers to traverse #1.

When a separate entrance of 25 ft. depth is dredged wave energy will decrease along the axis of the channel in shoreward direction due to channel refraction.

Refined calculations could be made to determine wave conditions at the marina entrance (as defined above), but in view of the model studies presently being conducted at the J.K.K. Look Laboratory, such calculations were not felt necessary.

Nevertheless an estimate has been made of the reduction in wave height in the channel; such estimate is presented in Figures 6.2a and b. In Fig. 6.2-a the ratio between the estimated wave height at the entrance and the deep water wave height is plotted against the deep water wave height.

It is estimated that the wave height at the entrance is 50-75% of the deep water wave height which ratio varies with wave height. In Fig. 6.2-b the estimated wave height at the entrance is plotted against the deep water wave height.

Joint use channel or parallel channel. Model studies carried out previously give information on wave height in station #3 (Figure 6.1) These data may be used to obtain insight into wave height conditions at the entrance to the channel (Look Lab Report #8, 1970)

Diagrams similar to Figure 6.2-b may be drawn to determine the wave height at Station #3 (H_3) versus the wave height in deep water (H_o).

It appears that both the wave period and the wave direction affect this ratio.

The results are plotted in Figures 6.3 through 6.5.

Although the data are for selected conditions and some

trends seem not well defined, the following general conclusion can be drawn.

1. For the directions W-WNW-NW waves in the entrance are generally higher than for the more southerly directions.
2. There is a dependency on both wave height and wave period.
3. For wave heights in deep water of 14 ft. and a wave period of 8 seconds the wave height at station #3 reaches its highest value (of the date plotted), of 9 ft. (Figure 6.3).
4. Waves larger than 20 ft. were not tested in the model so that no information on the wave heights in Station 3 is available for those conditions. It is estimated that during a hurricane (H_s = 35 ft) waves in the entrance may reach values of above 10 ft.

   The model study presently underway will provide additional information on this condition.

The observation that waves from the direction S-SW do not generate the highest waves in the entrance channel is not directly obvious, because the channel has an orientation toward S.W.

Apparently under waves from the S.W. wave refraction will divert much wave energy away from the channel leading to lower waves in the entrance area.

Entrance inside deep draft harbor

This solution utilizes the 450 ft. wide and 42 ft.

deep entrance channel to the deep draft harbor and thereby provides an all weather safe access route to the marina. (plate V) This includes storm conditions and thus is an important advantage.

The nearby small boat harbors at Pokai Bay and at Waianae do not provide all-weather access.

Using the existing deep draft harbor channel will furthermore have a minimum adverse impact on the coastal environment outside the entrance; there is no adverse impact on littoral drift conditions.

A solution that involves the marina entrance inside the deep draft harbor must be designed in such a way that it maximizes wave energy dissipation. One of the ways to do this is to provide a so-called spending beach (slope 1:10 or less) arrangement. The proposed solution (Plate V) has such an arrangement. Such a solution is believed to also have a favorable effect on the wave environment of the deep draft harbor.

A marina entrance inside the deep draft harbor may be located near the entrance or further inside the harbor basin. A location close to the ocean has two distinct advantages:

(1) excitation of long waves will be smaller;

(2) no interferences between boats entering the marina and commercial ships in the process of berthing.

The disadvantage of this solution is the joint use

of the entrance channel by both commercial and recreational vessels and the possibility of a collision.

It is recognized that the U.S. Army Corps of Engineers did not design the deep draft harbor entrance channel for joint use by commercial and recreational vessels.

However, expected commercial traffic densities are low and a joint use of the existing channel could therefore be considered.

A proper warning system should then be installed to secure collision free traffic.

Separate traffic lanes in combined entrances

This solution has the advantage of separating the traffic flows for the marina and the deep draft harbor, potentially reducing the risk of collision.

This solution is more expensive and invites more energy into the marina given the same width of entrance. A reduction of the entrance width to the marina may be necessary.

The impacts of this solution on the deep draft harbor will be studied in the hydraulic model. This option may expose the existing barge harbor basin to more direct wave attack from NW waves. The favorable effects on reducing wave height in the deep draft harbor may not be as significant as in the first solution with a joint entrance channel.

Separate entrances

A separate entrance is likely to improve the water quality in the marina because there is no direct connection between the two basins. Water in the commercial harbor is likely to have a lower standard because of pollution associated with activities in this basin.

The major advantage is the complete separation of the two different flows of navigational traffic so that danger of collision is at a minimum.

For a separate marina entrance a depth of 25 ft. below MLLW was selected in order to reduce the number of days that the entrance becomes unnavigable because of waves breaking in the channel.

Even with a 25 ft. depth of the channel such condition is expected to occur several days per year.

In the marina basin waves are reduced in height and a depth of 15 ft. below M.LL.W is therefore adequate.

When waves break in the marina entrance channel conditions are hazardous for entering vessels. Under those circumstances yachts could still use the deep draft harbor as a harbor of refuge, but such use of the deep draft harbor channel is undesirable because of safety and other reasons, (e.g. lack of adequate docking facilities for yachts in big harbor).

A yacht owner may make the wrong decision by trying to use the marina channel despite its unsafe condition due to breaking waves, and may thereby get himself into a hazardous situation.

An example of such an occurrence is the recent accident involving the tour boat "Pearl Kai", in the entrance to Kewalo Basin on June 19, 1984, whereby the captain tried to enter the harbor under unsafe conditions.

Furthermore, a separate harbor entrance will have a greater adverse impact on the offshore environment because of necessary dredging and of impact to the shoreline and offshore bottom in the immediate vicinity of the entrance.

Sedimentation of the channel, which is likely to occur, will require regular maintenance dredging.

6.3 Marina basin

The size, configuration and orientation of the marina basin were determined by planning considerations of external circumstances, such as the required distances from deep draft harbor hazard zones.

Depending on boat size the available harbor will provide space for 350 to 500 boats, making the marina one of average size.

In the design of boat slip arrangements, attention must be paid to the following aspects:

- predominant winds and short period "chop";
- wave absorption devices;
- avoidance of debris traps;
- flushing of debris;
- adequate manuevering space;
- adequate depth (15 ft. below MLLW)

With respect to wave absorption the long breakwater facing the south side of the harbor will be constructed as a wave absorber. In order to avoid wave reflection from the east end of the basin a facing of igloo-blocks may be appropriate.

The other walls are conceived as vertical sheet pile walls; the docks and piers will be of the floating type.

6.4 Other hydraulic considerations

Other hydraulic considerations that may affect the efficiency of the marina are:

* response to long period and short period waves
* adequate flushing;
* sedimentaiton traps (for run off)

The long period waves of interest are tsunami waves, weather induced oscillations (hurricanes) and surf beat.

The harbor's own frequencies are away from the expected excitation frequencies of the ocean so that no special problems are anticipated (see K. Bathen, 1984).

The location of the marina entrance near the ocean boundary suggests that major interaction phenomena between the two harbor basins are unlikely.

Aspects of flushing and density flows are also discussed in Dr. K. Bathen's report.

6.5 Stability of structures

Tentative figures for the weight of rock for protective structures may be obtained by using the Hudson formula (S.P.M. 1973)

$$W_r = \frac{\gamma_r H^3}{K_D (S_r - 1)^3 \cot\alpha} \qquad (6.1)$$

where

W_r = weight of armor rock

H_s = significant wave height

K_D = stability coefficient, based on type of armor material (assumed $K_D = 2.3$)

S_r = specific gravity

γ_r, γ_w specific weight of rock and water

α = slope of breakwater

For basalt a specific gravity of 2.8 is assumed. If lower densities have to be used, weight and block sizes have to be adjusted according to formula (6.1).

The following types of basalt structures were analyzed:

(1) Shore revetment or breakwater:

$H = 4.5$ m, $\gamma_r = 2800$ kg/m^3, $\cot\alpha = 3$.

Required rock size: $W_r = 6.3$ ton. $D_r = 5$-6 ft.

(2) Entrance to marina

$H = 3.7$ m, $\gamma_r = 2800$ kg/m^3, $\cot\alpha = 3$.

Required rock size $W_r = 3.5$ ton. $D_r = 4$-5 ft.

(This corresponds to Corps of Engineers rock size of 3-4 tons)

(3) Spending beach

$H_s = 1.83$ m, $\gamma_r = 2800$ kg/m^3, $\cot\alpha = 10$.

Required rock size $W_r = 128$ kg (280 lbs); $D_r = 1.5$ ft.

(4) Marina basin

$H_s = 1.0$ m $\gamma_r = 2800$ kg/m^3, $\cot\alpha = 1.5$.

Required rock size: $W_r = 138$ kg (300 lbs); $D_r = 1.5$ ft.

The above listed rock sizes are only a first approximation because no account has been given to wave period. Corrections must therefore be made for the following:

(1) wave height data, obtained from the model study;

(2) evaluation of the effect of wave period on rock size;

(3) evaluation of value of K_D coefficient with respect to wave period.

It is furthermore to be noted that the applicability of the Hudson formula is somewhat out of experimental range for the small slope angle (1:10) of the spending beach. This also needs further evaluation in the hydraulic model.

7.0 PROJECT PLANNING AND COORDINATION, ADDITIONAL ASPECTS OF MARINA DESIGN

During the course of this study meetings were held with Mr. William R. Godfrey, AIA, of Alfred A. Yee, Division Leo A. Daly, regarding development of planning aspects on lagoons and marina.

During these meetings consultancy services were provided on technical design aspects of lagoons and marina, such as:

- Construction drawings (preliminary) for lagoons;
- Set back lines for walkways and buildings;
- required elevation's for coastal zone to protect against tsunamis and wave run-up;
- Various alternative entrance designs for marina;
- Basin configuration;

Engineering drawings were prepared by the Alfred A. Yee Division, Leo A. Daly office in various plans. In addition, three meetings were attended to discuss alternative plans with Corps of Engineers and Hawaii State Harbors Division.

Furthermore consultancy services were provided regarding the hydraulic model study at J.K.K. Look Laboratory and several meetings were attended to discuss scope and conditions of the hydraulic model study, including the required test conditions.

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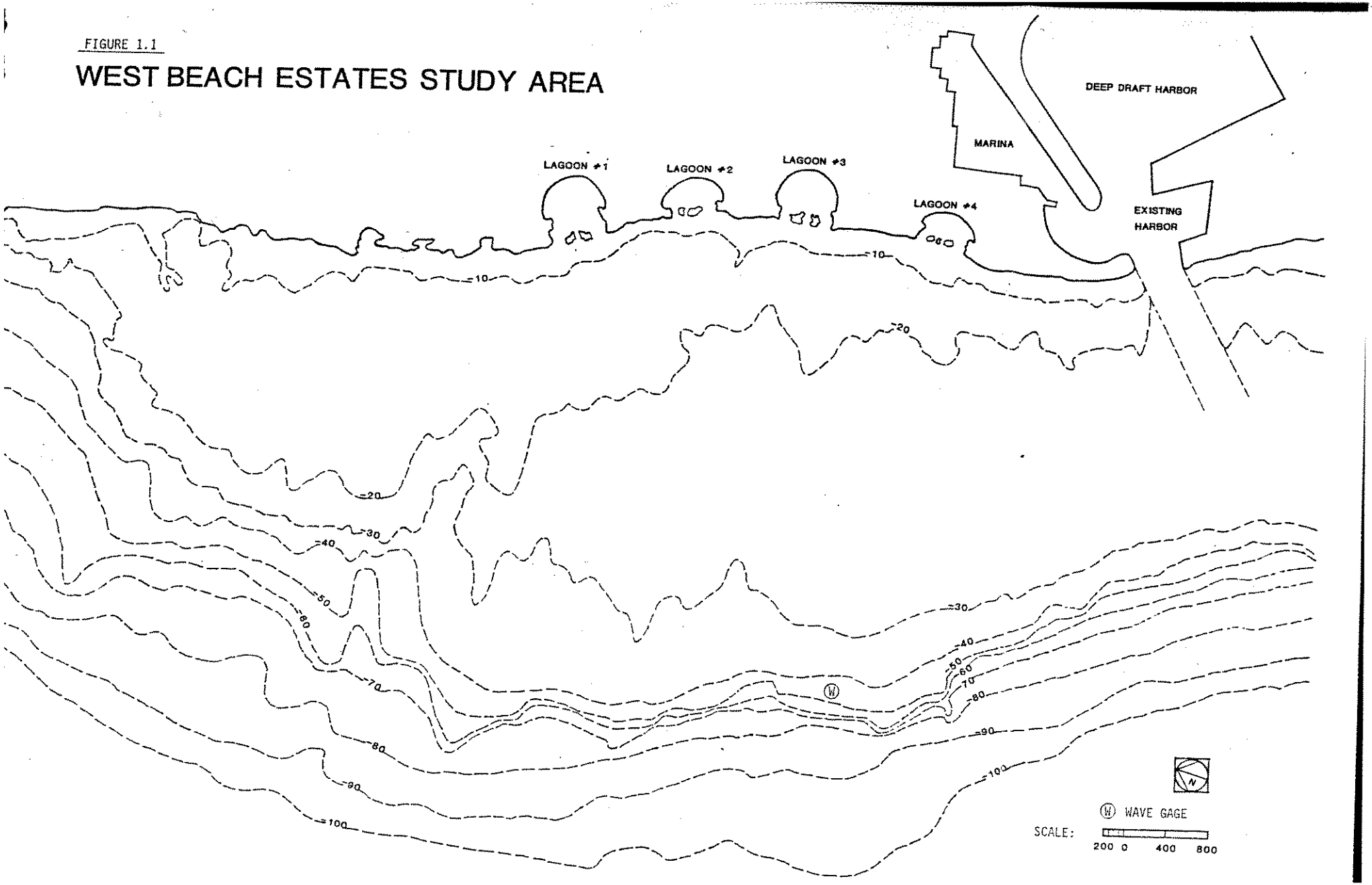
9. ACKNOWLEDGEMENTS

Valuable contributions to this study were provided by the Office of Alfred A. Yee Division A Daly through Mr. William R. Godfrey, A.I.A., in terms of coordination of efforts, developing of engineering concepts and drawings and reviewing of manuscript.

Furthermore close cooperation was maintained with Drs. K. Bathen and C.L. Bretschneider during the course of this study.

FIGURE 1.1

WEST BEACH ESTATES STUDY AREA



DISTRIBUTION OF Hs
DEEP WATER OBSERVATIONS
FROM HINDCAST DATA

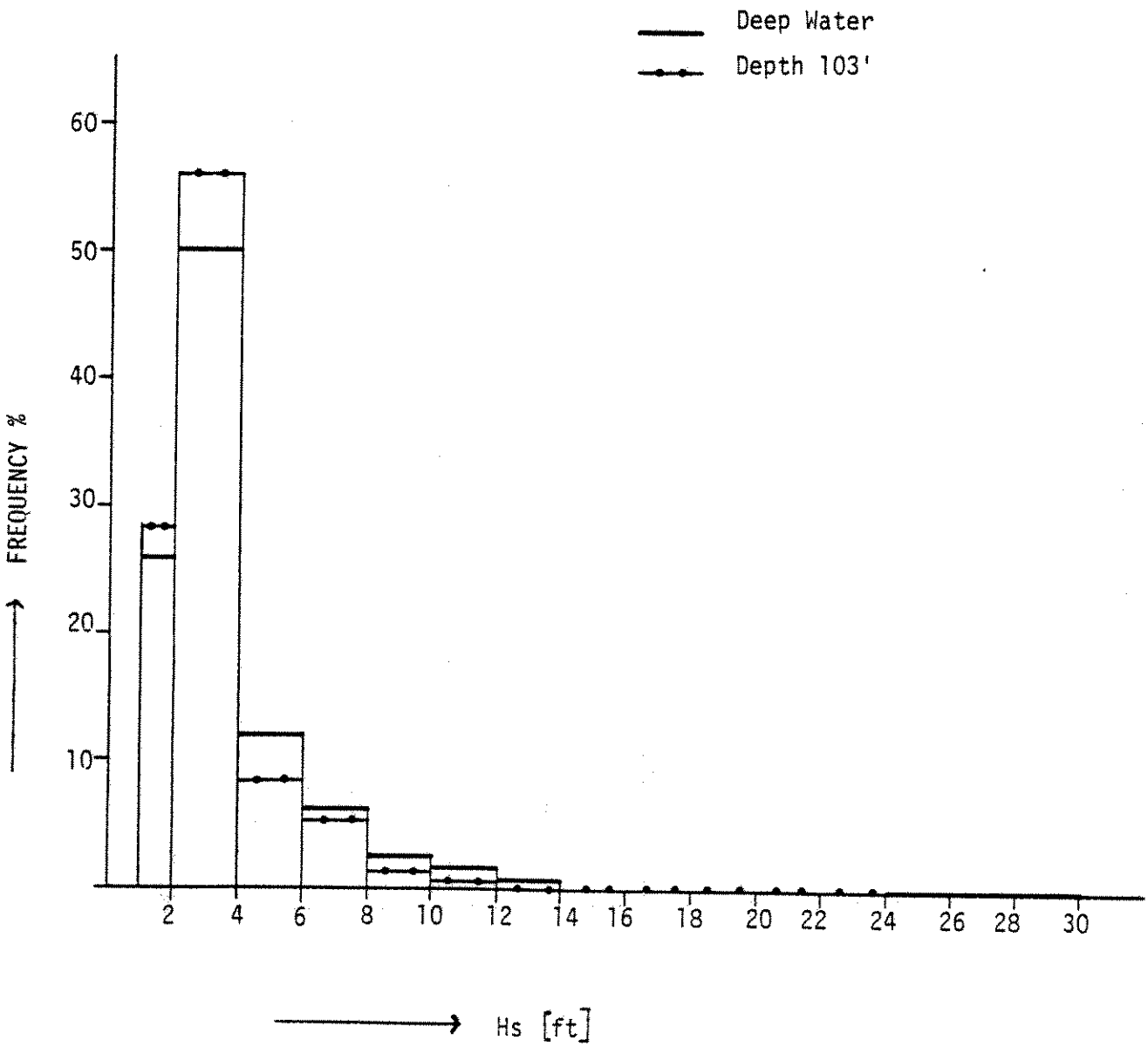


FIGURE 2.1

DISTRIBUTION OF Ts
DEEP WATER OBSERVATIONS

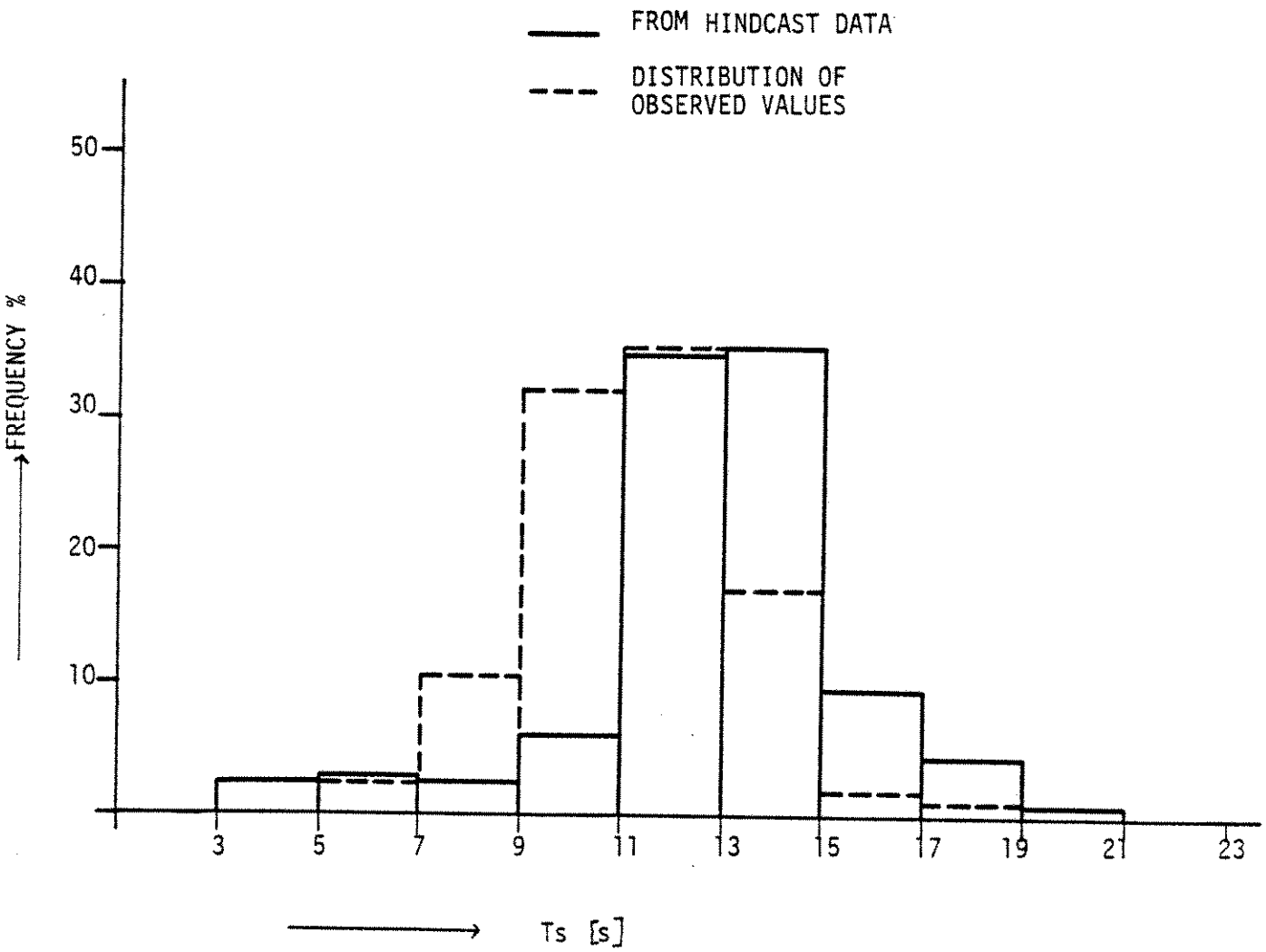


FIGURE 2.2

DISTRIBUTION OF WAVE DIRECTION
DEEP WATER OBSERVATIONS

(FROM HINDCAST DATA)

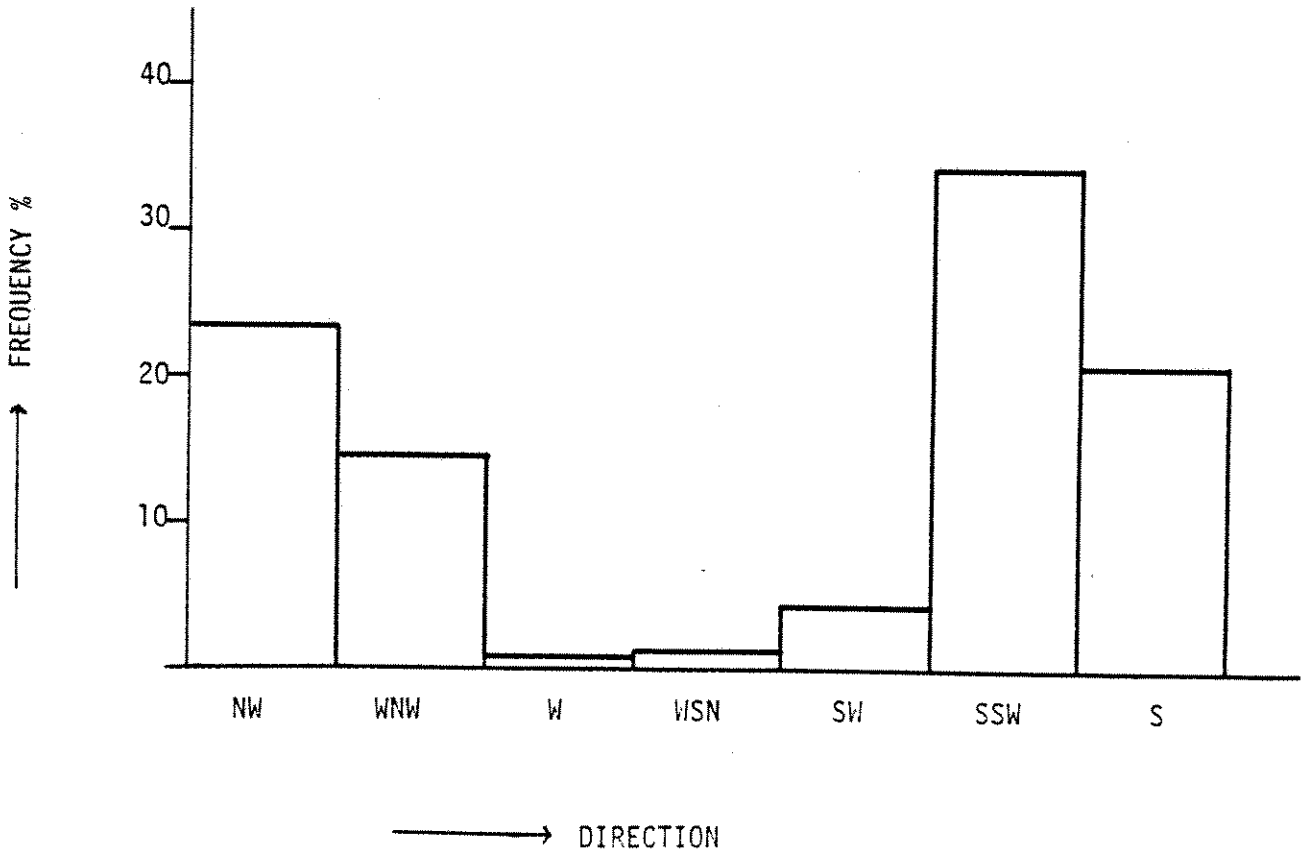


FIGURE 2.3

RELATION BETWEEN Tp AND Ts

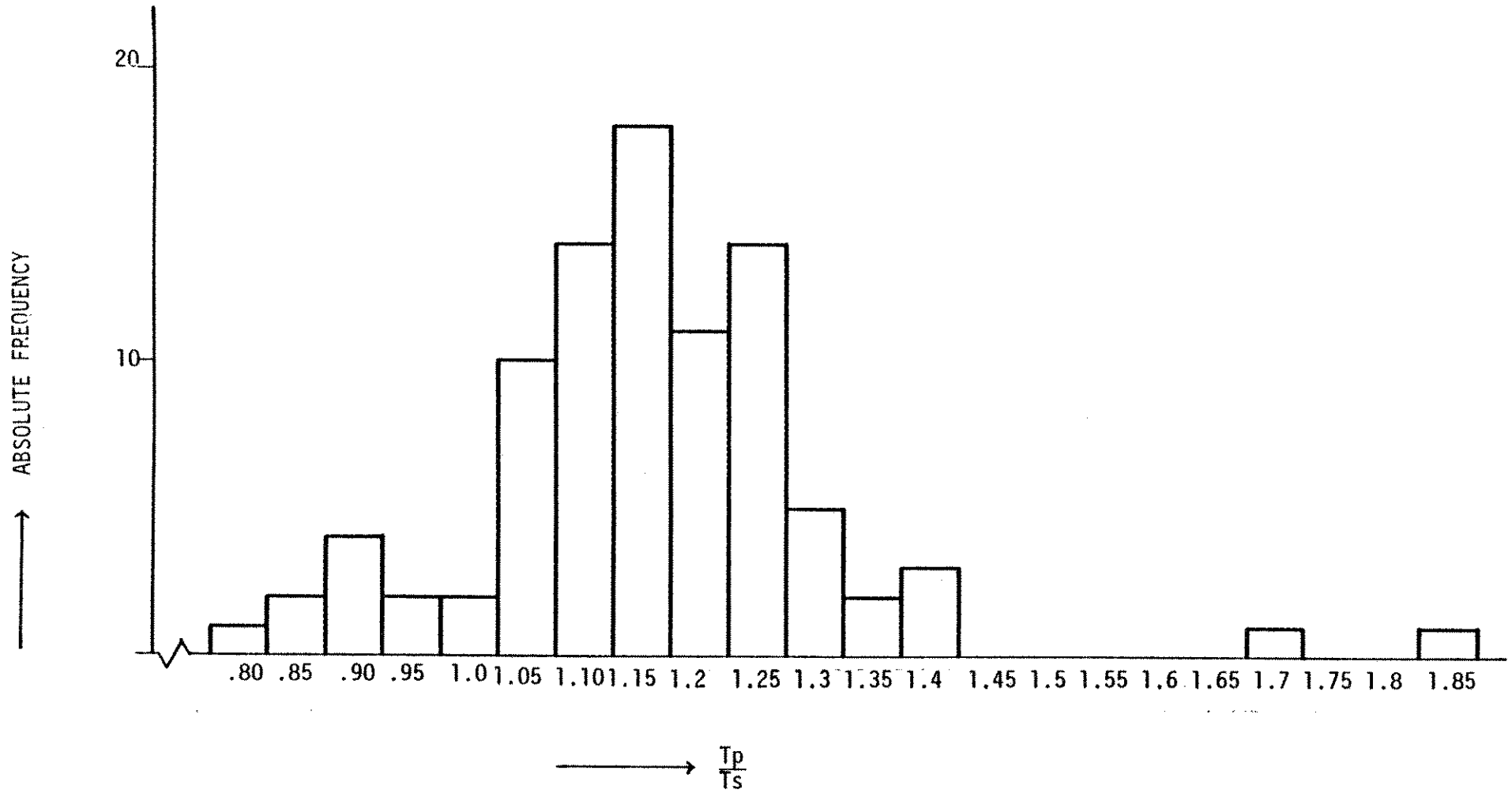


FIGURE 2.4

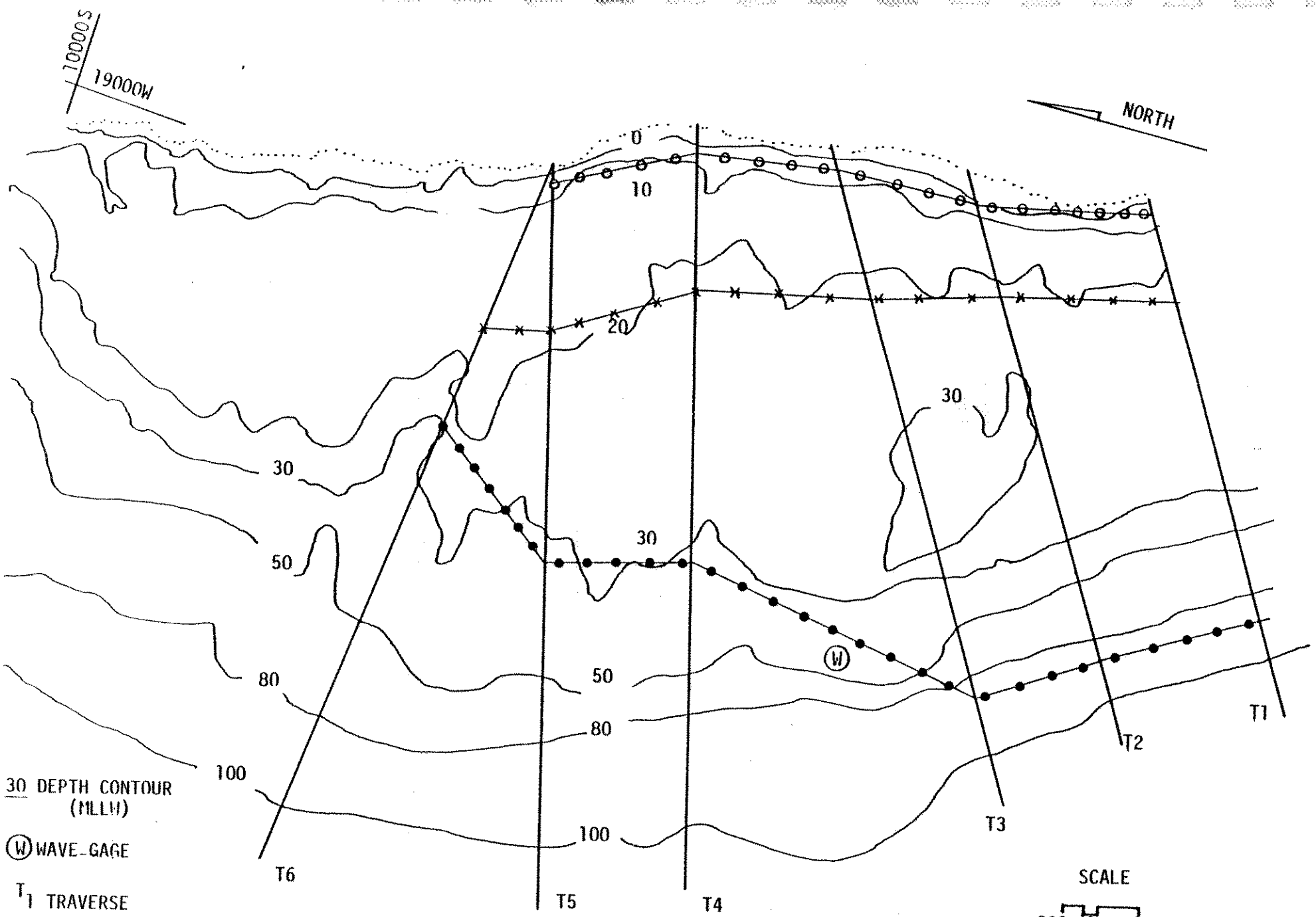


FIGURE 2.5

LINES OF EQUAL WAVE HEIGHT AT WEST BEACH PROJECT AREA

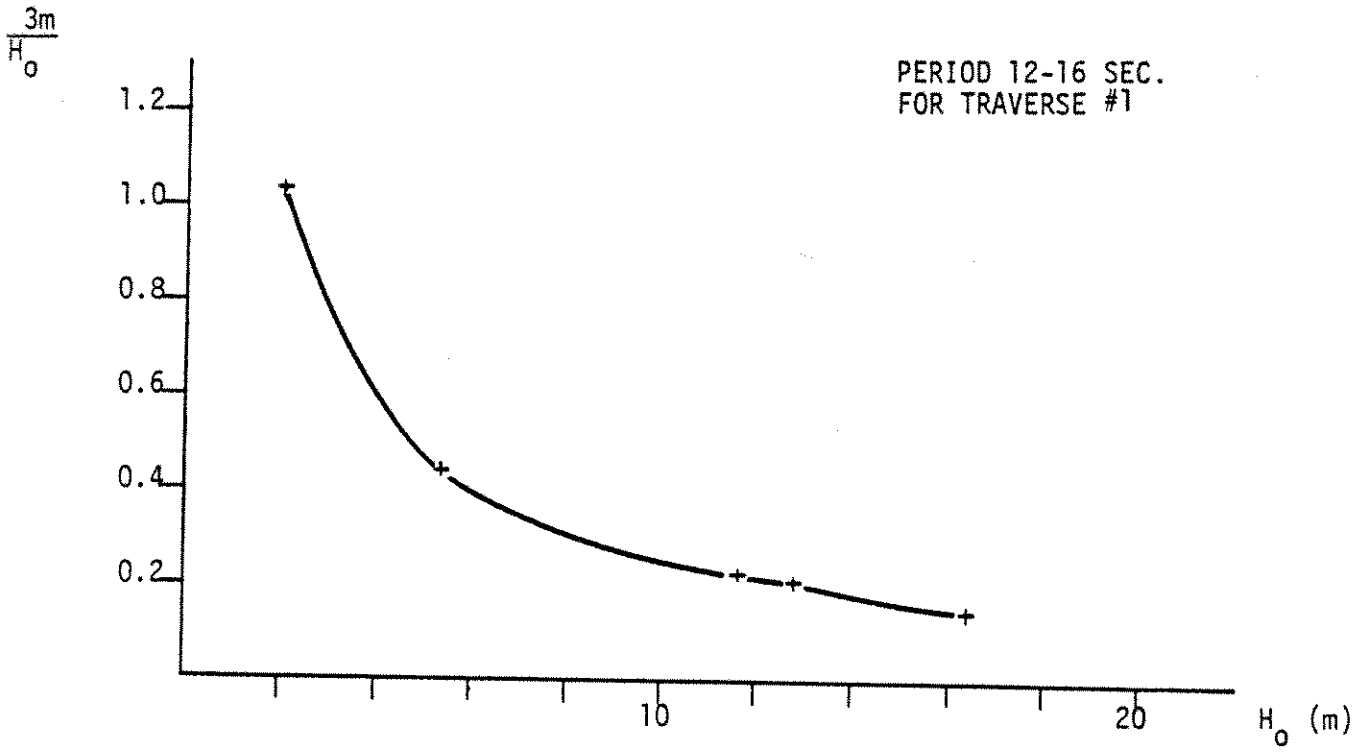


FIGURE 2.6

RATIO OF WAVE HEIGHT IN SHALLOW WATER (3m = 10ft) AND IN DEEP WATER

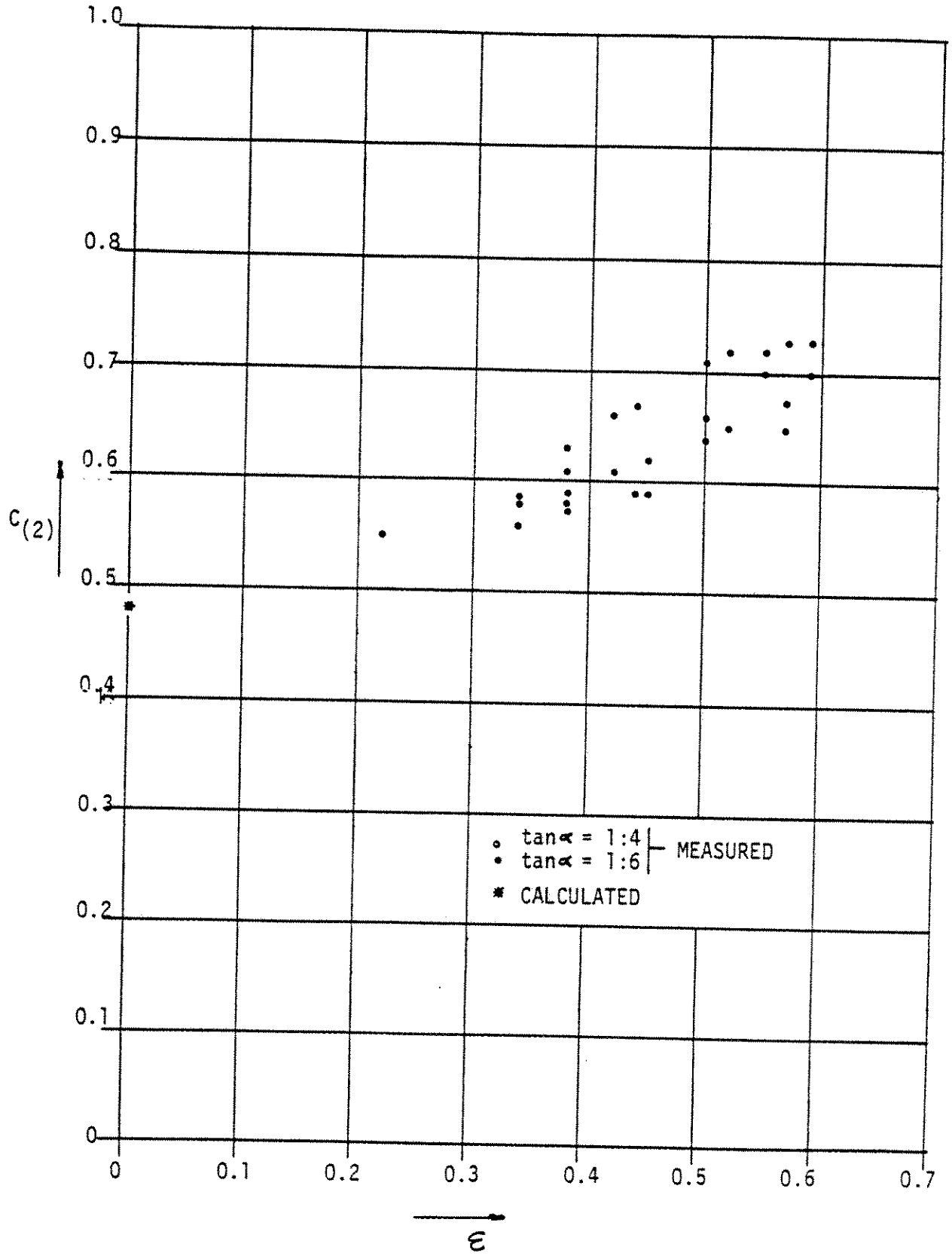


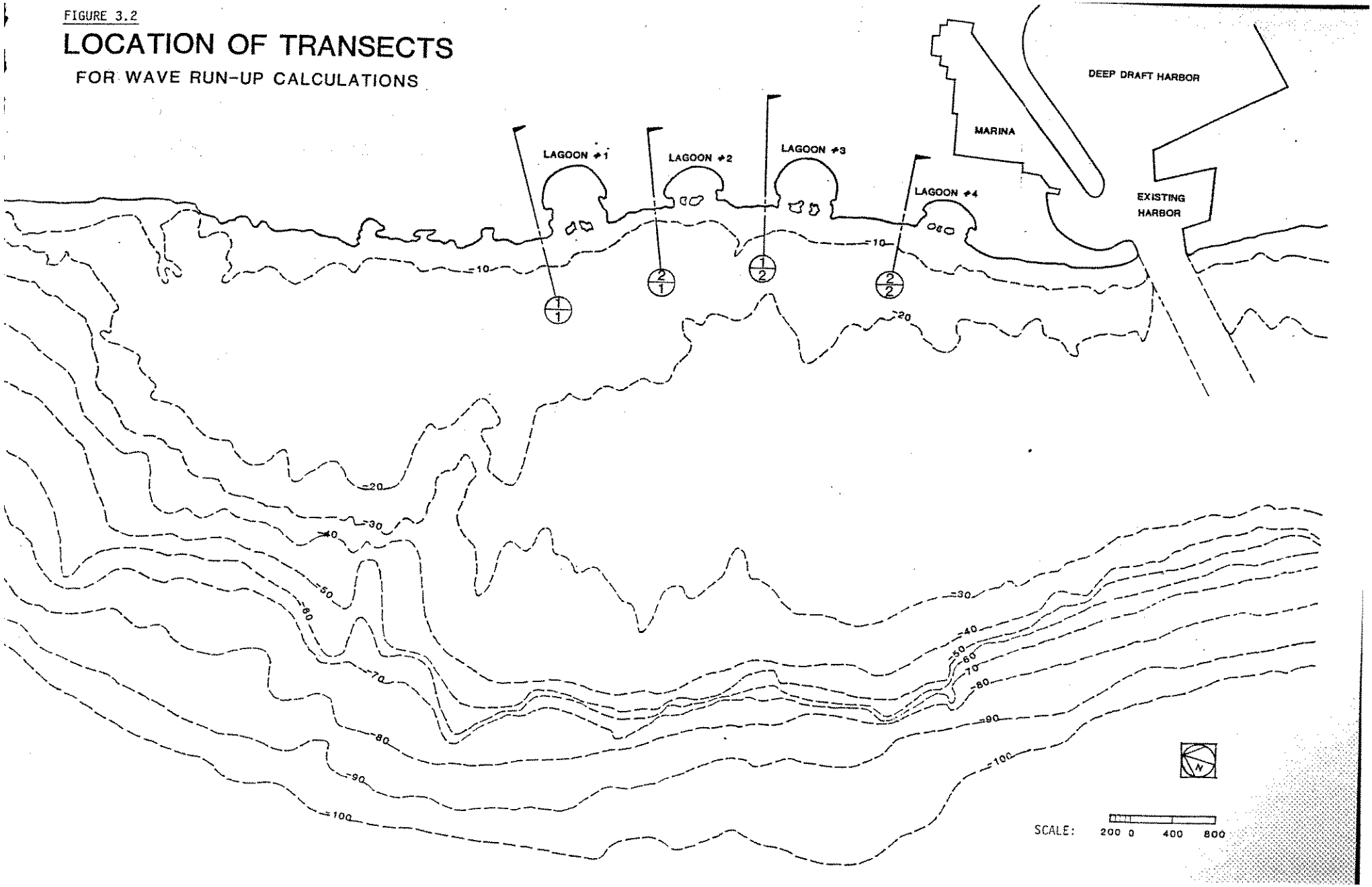
FIGURE 3.1

DEPENDENCY OF $C_{(2)}$ ON SPECTRUM WIDTH

FIGURE 3.2

LOCATION OF TRANSECTS

FOR WAVE RUN-UP CALCULATIONS



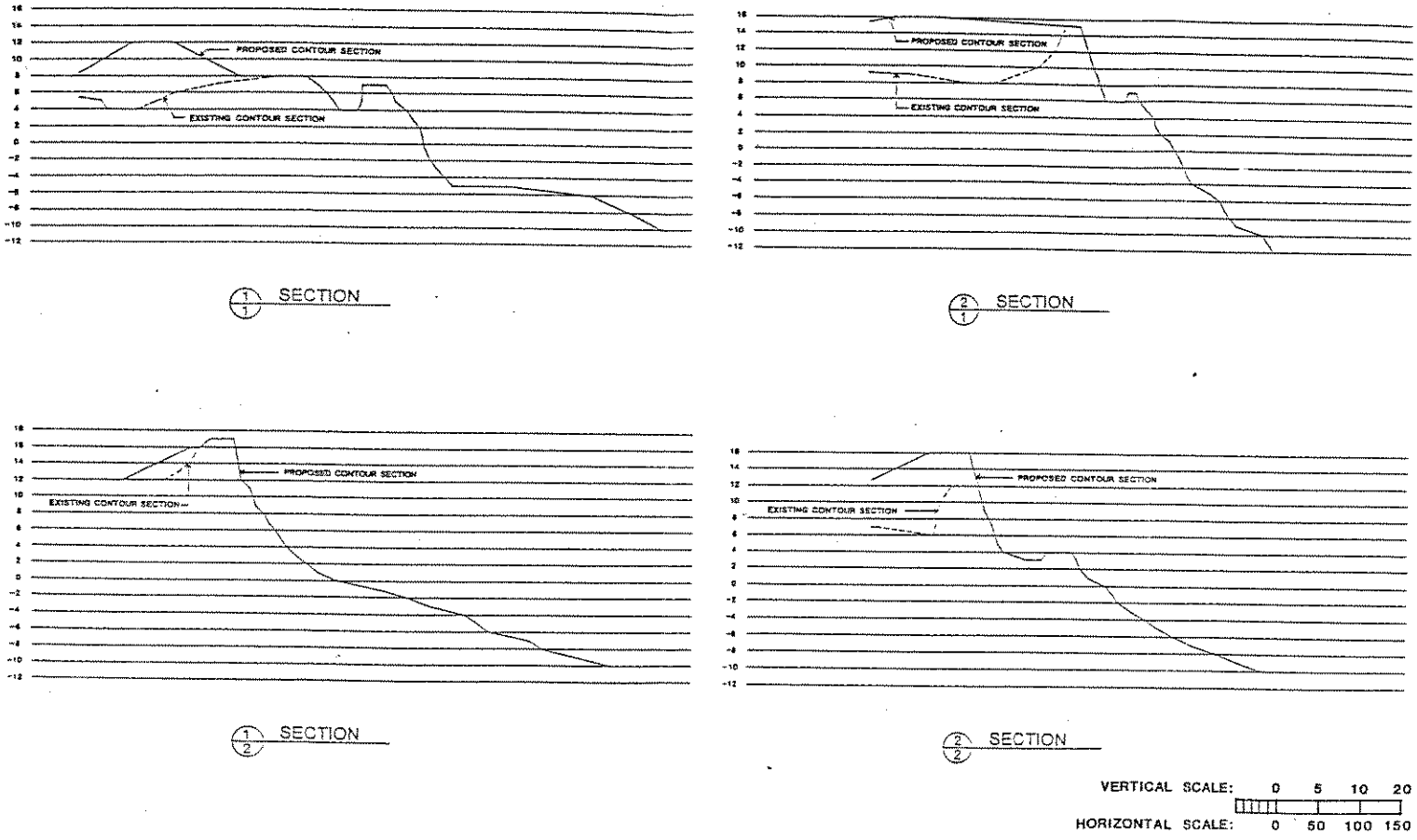


Figure 3.3

PROFILES. ACCORDING TO TRANSECTS 1-1 THRU 2-2.

BEACH PROFILE IN LAGOONS

DURING HIGH SWELL CONDITIONS

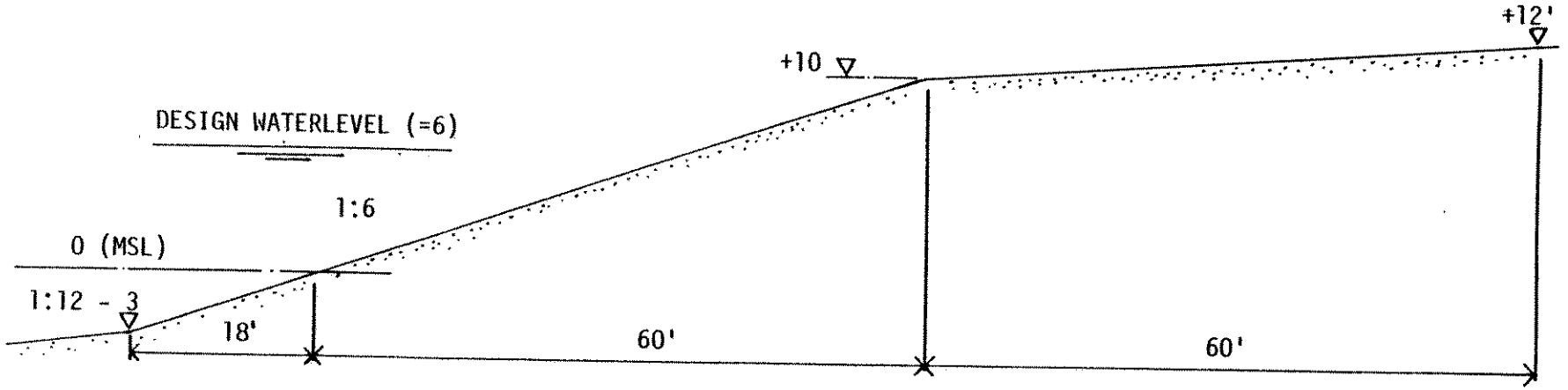
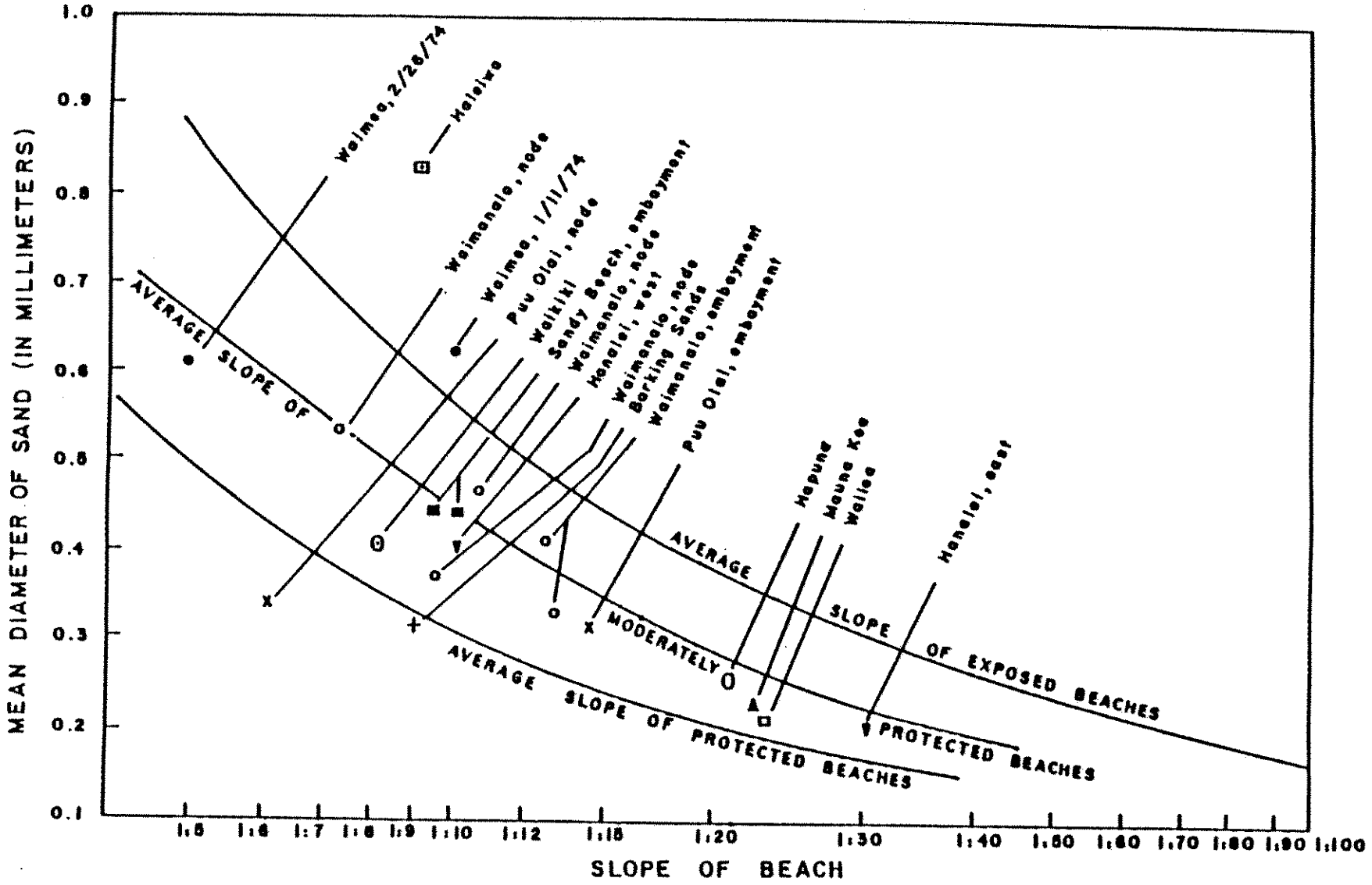


FIGURE 3.4



Relationship between beach slope and sand size at mid-tide for Hawaiian beaches

FIGURE 4.1

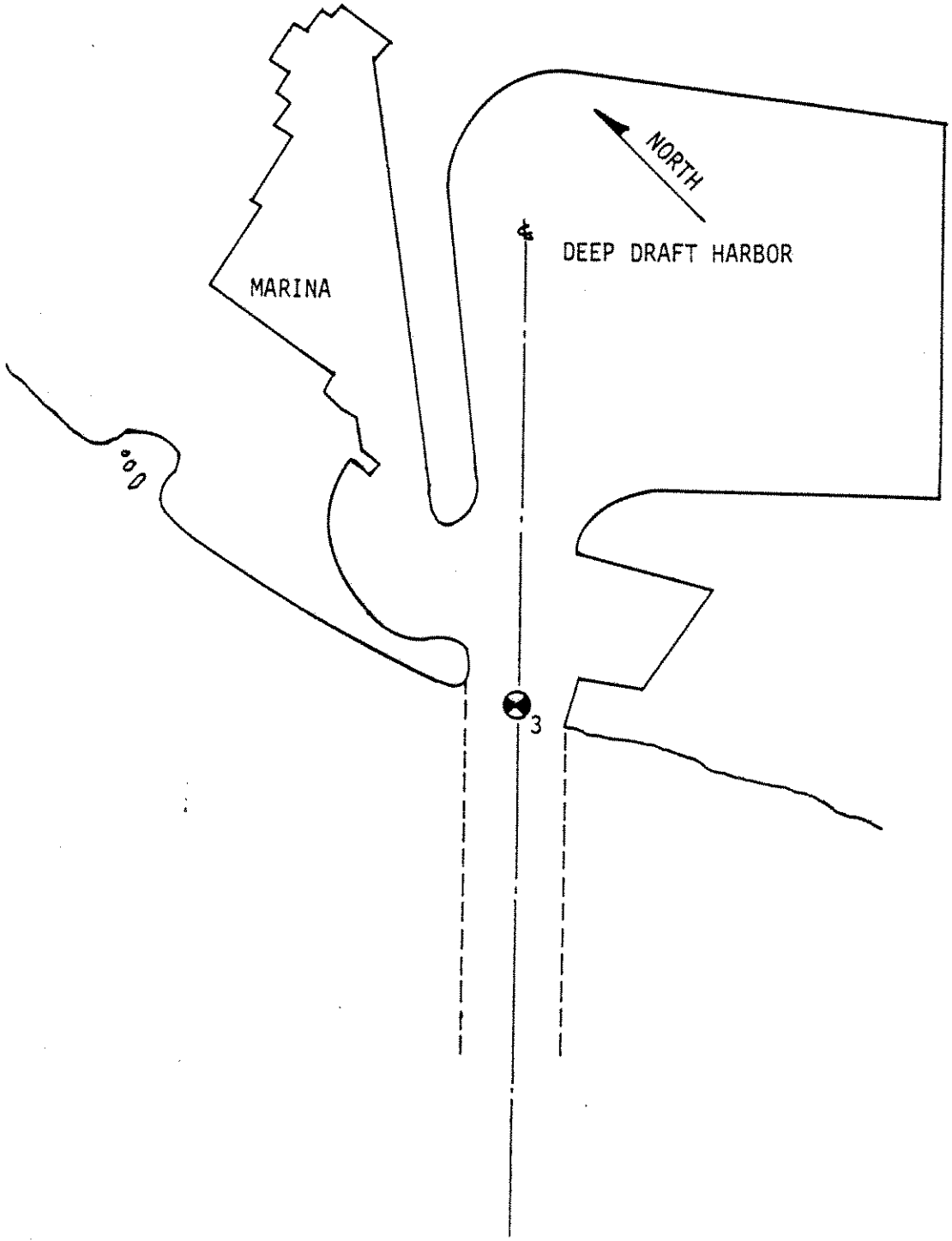


FIGURE 6.1
Location of test station #3

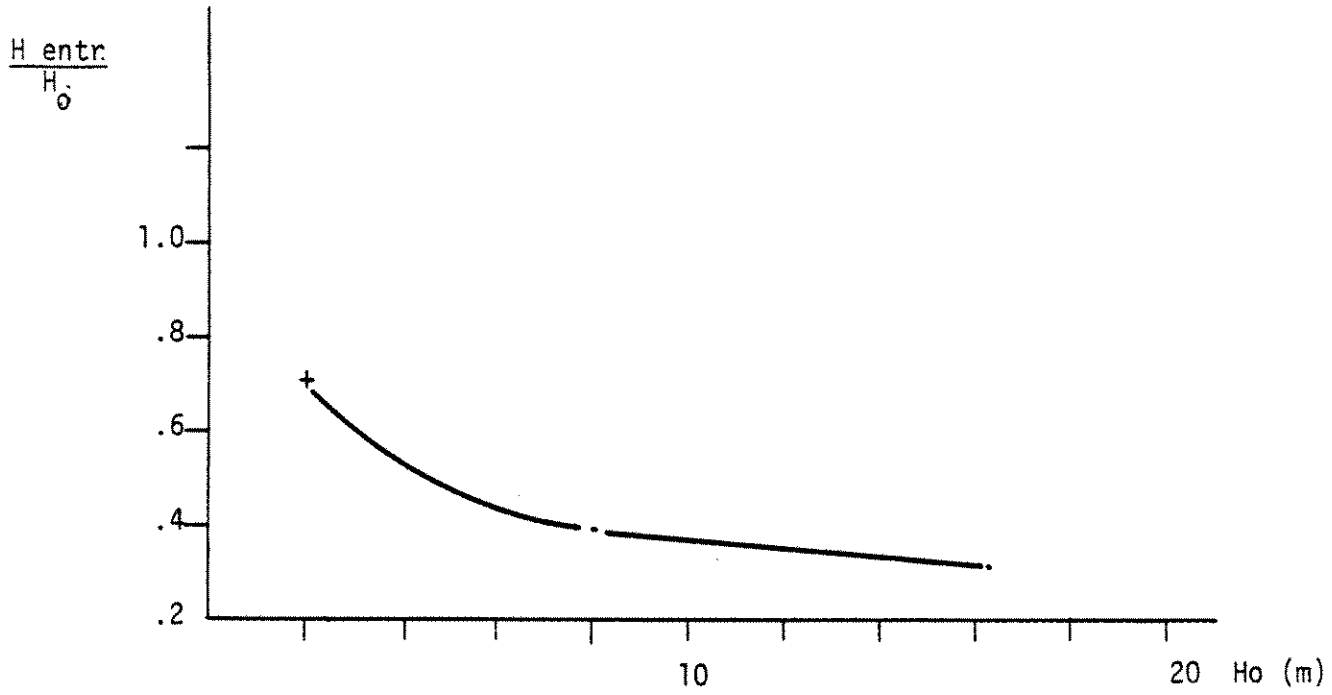


FIGURE 6.2-a

ESTIMATED RATIO OF WAVE HEIGHT AT MARINA ENTRANCE AND IN DEEP WATER

SEPARATE ENTRANCE CHANNEL

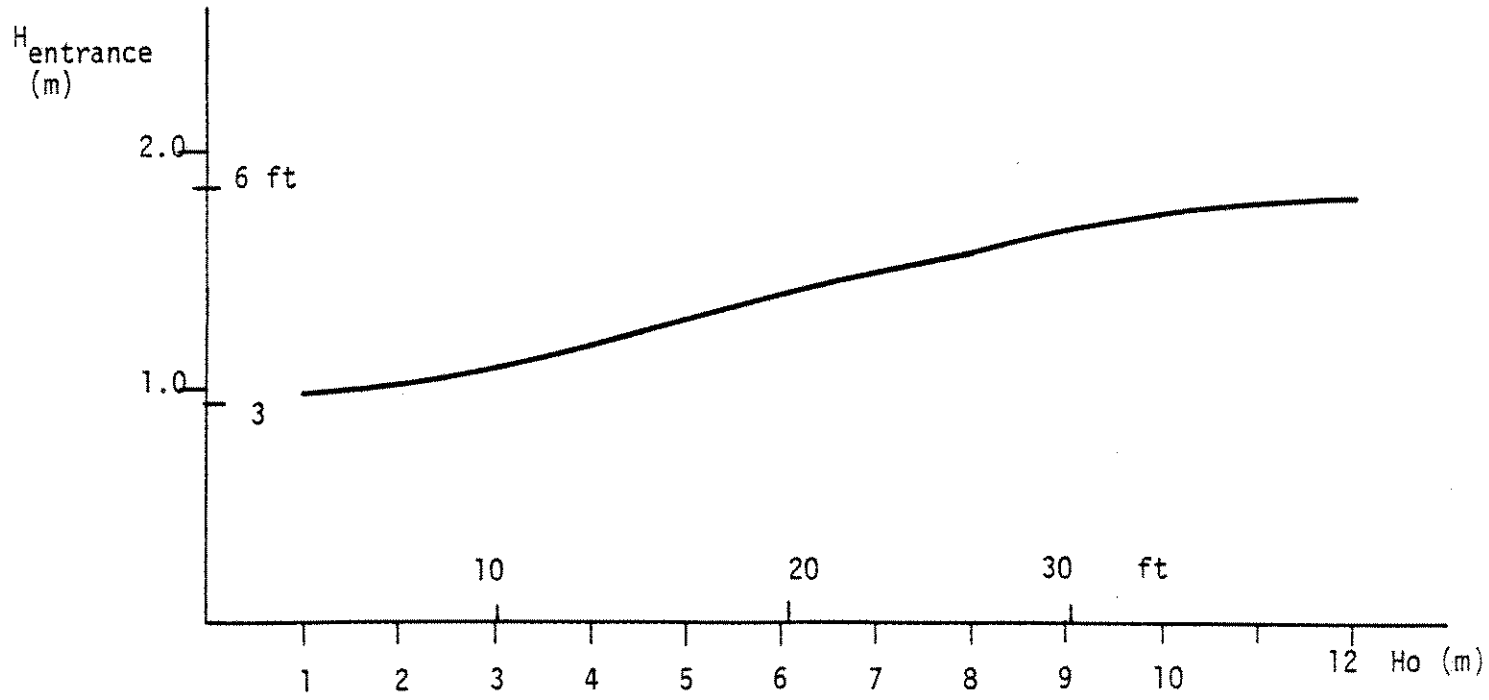
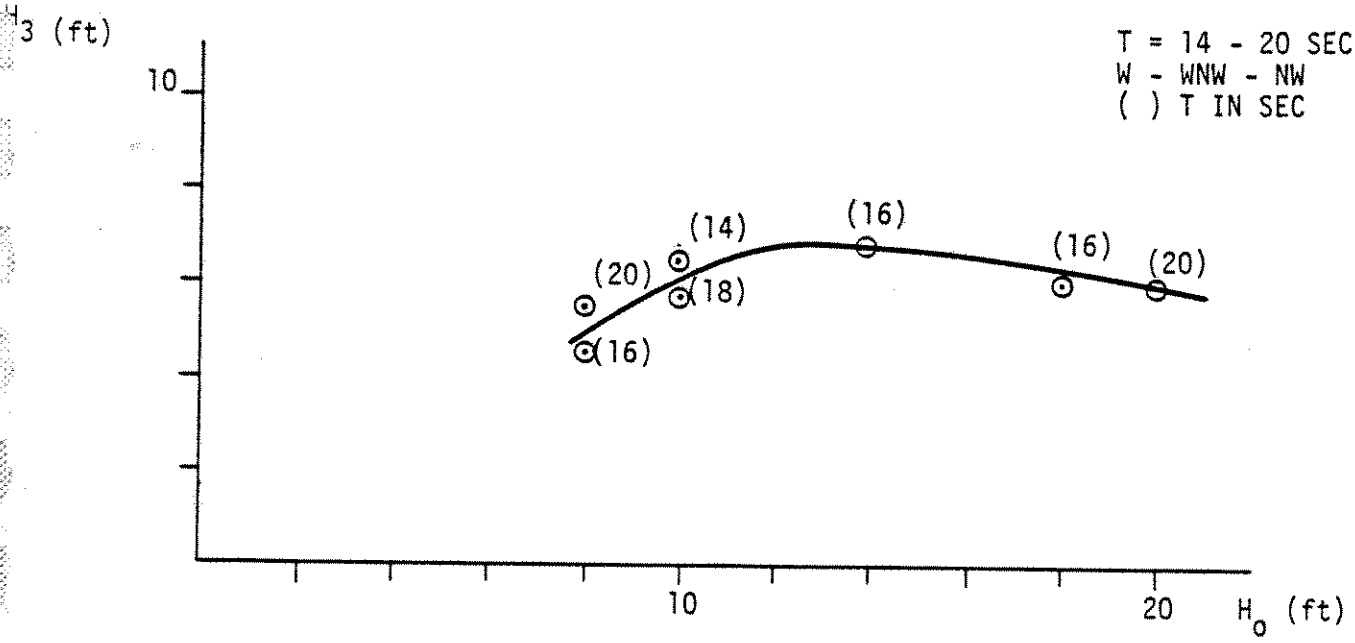


FIGURE 6.2-b

ESTIMATED WAVE HEIGHT AT MARINA ENTRANCE-SEPARATE ENTRANCE CHANNEL



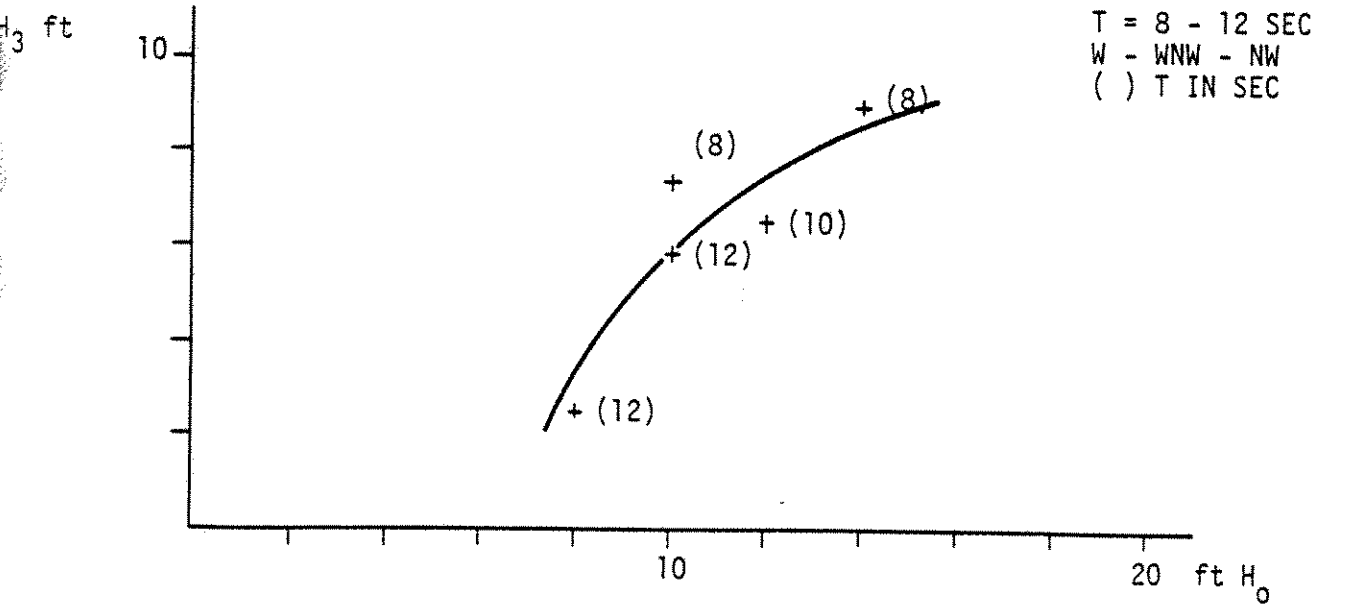
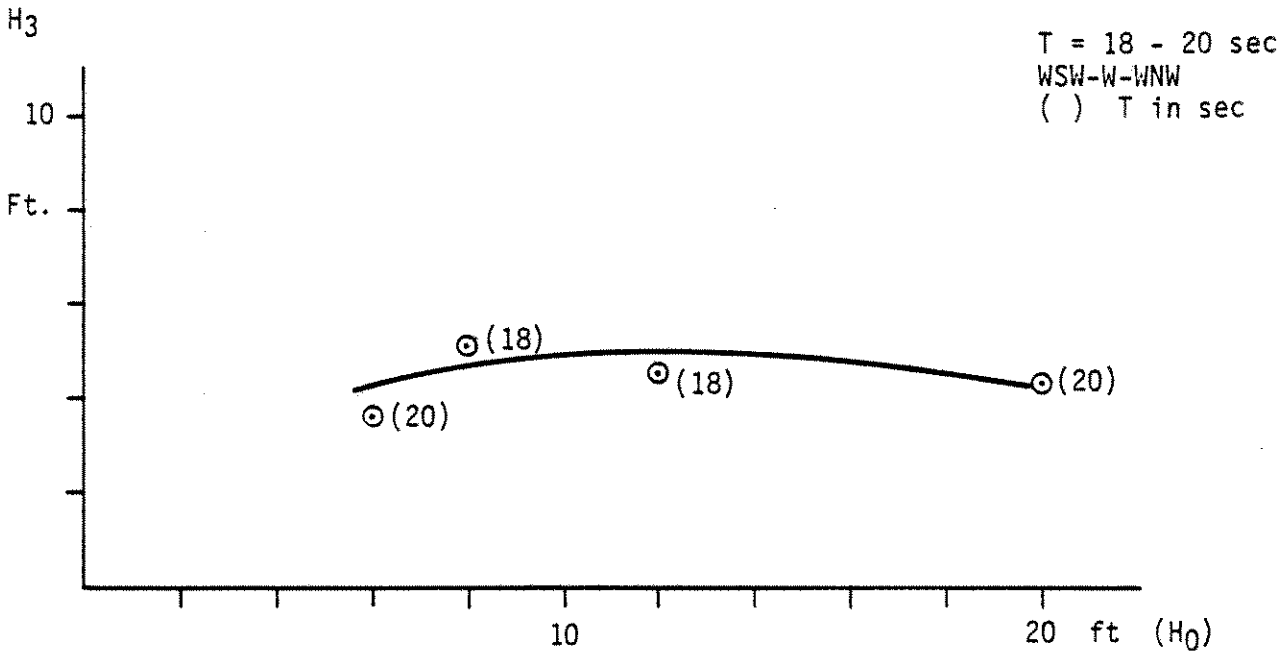


FIGURE 6.3

WAVE HEIGHT IN ENTRANCE CHANNEL OF DEEP DRAFT HARBOR (STA #3)
DIRECTION W - WNW - NW

(SOURCE : HYDRAULIC MODEL STUDY)



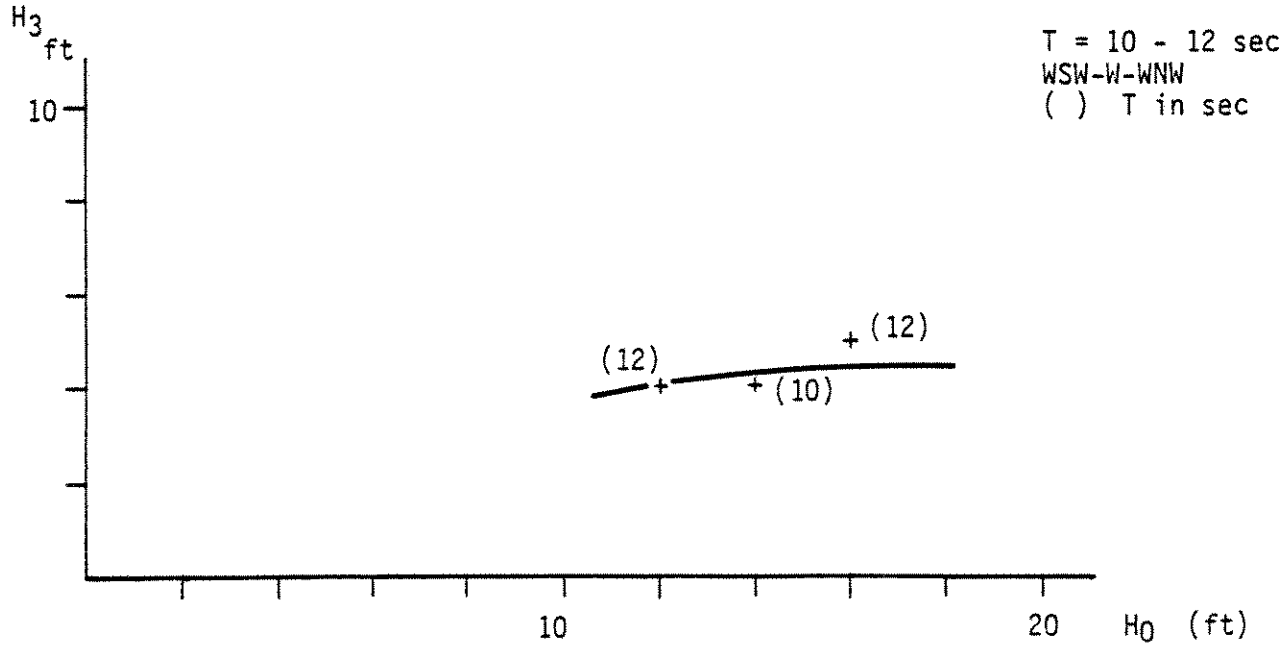


Figure 6.4

WAVE HEIGHT IN ENTRANCE CHANNEL OF DEEP DRAFT HARBOR (STA #3)
DIRECTIONS WSW-W-WNW.

Source: Hydraulic Model Study

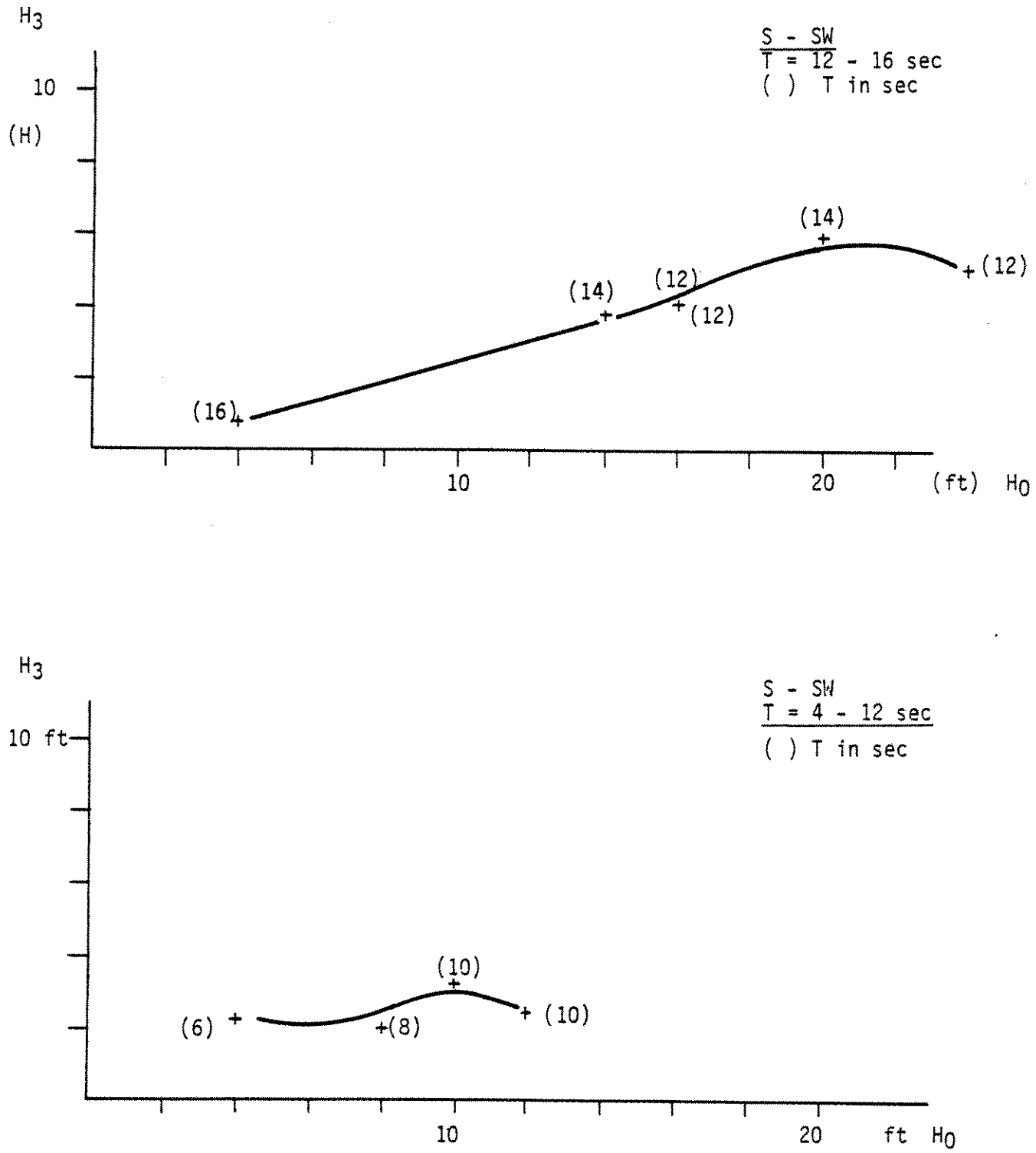


Figure 6.5

WAVE HEIGHT IN ENTRANCE CHANNEL OF DEEP DRAFT HARBOR (STA #3)
DIRECTIONS S-SW

Source: Hydraulic Model Study

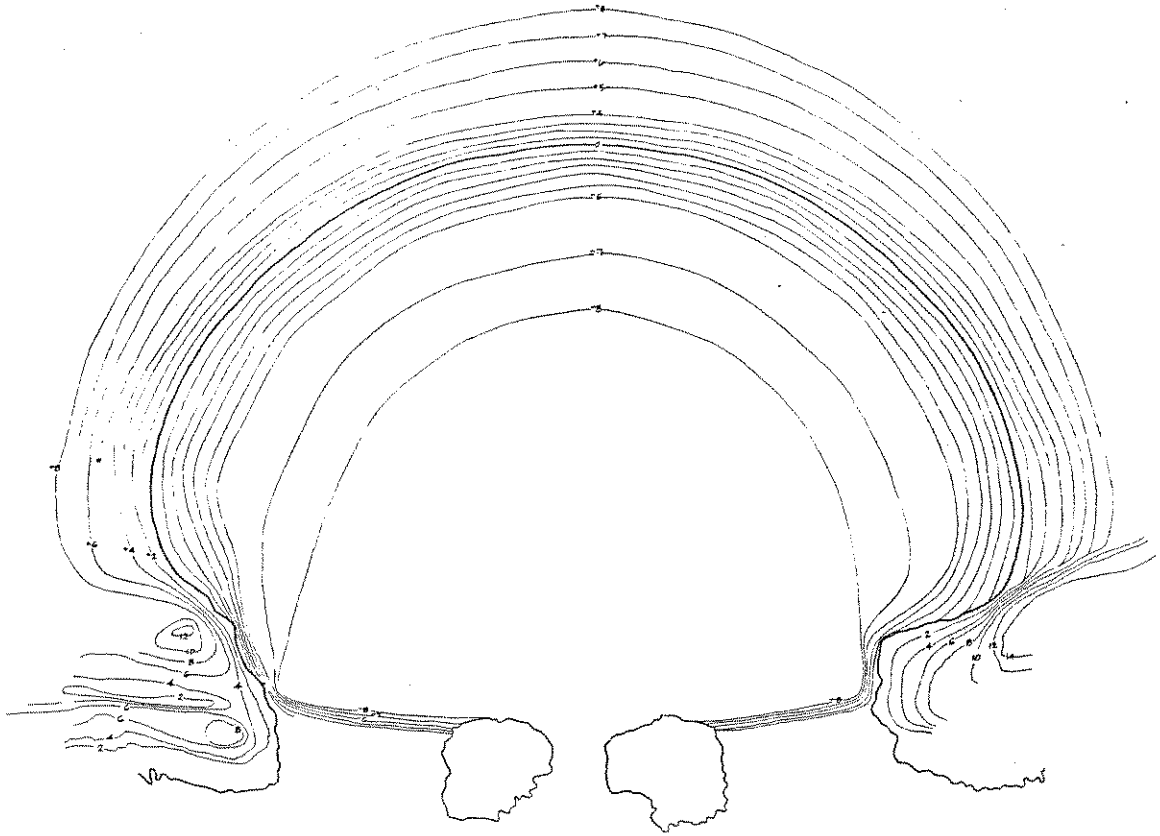
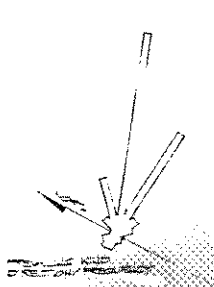
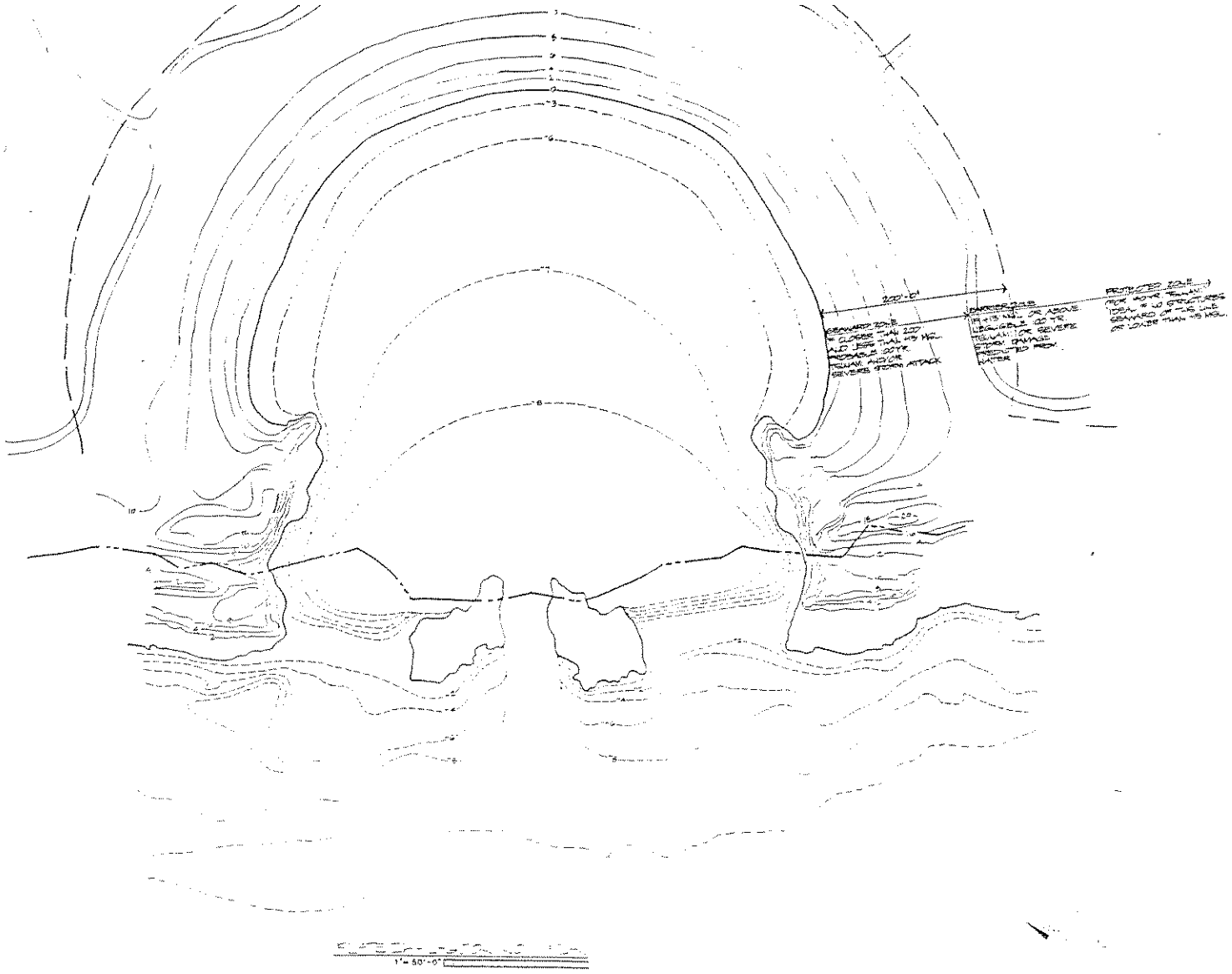


PLATE I -LAGOON NO. 1 NATURAL CONFIGURATION PLAN

1" = 50'-0"



ALFRED A. YEE



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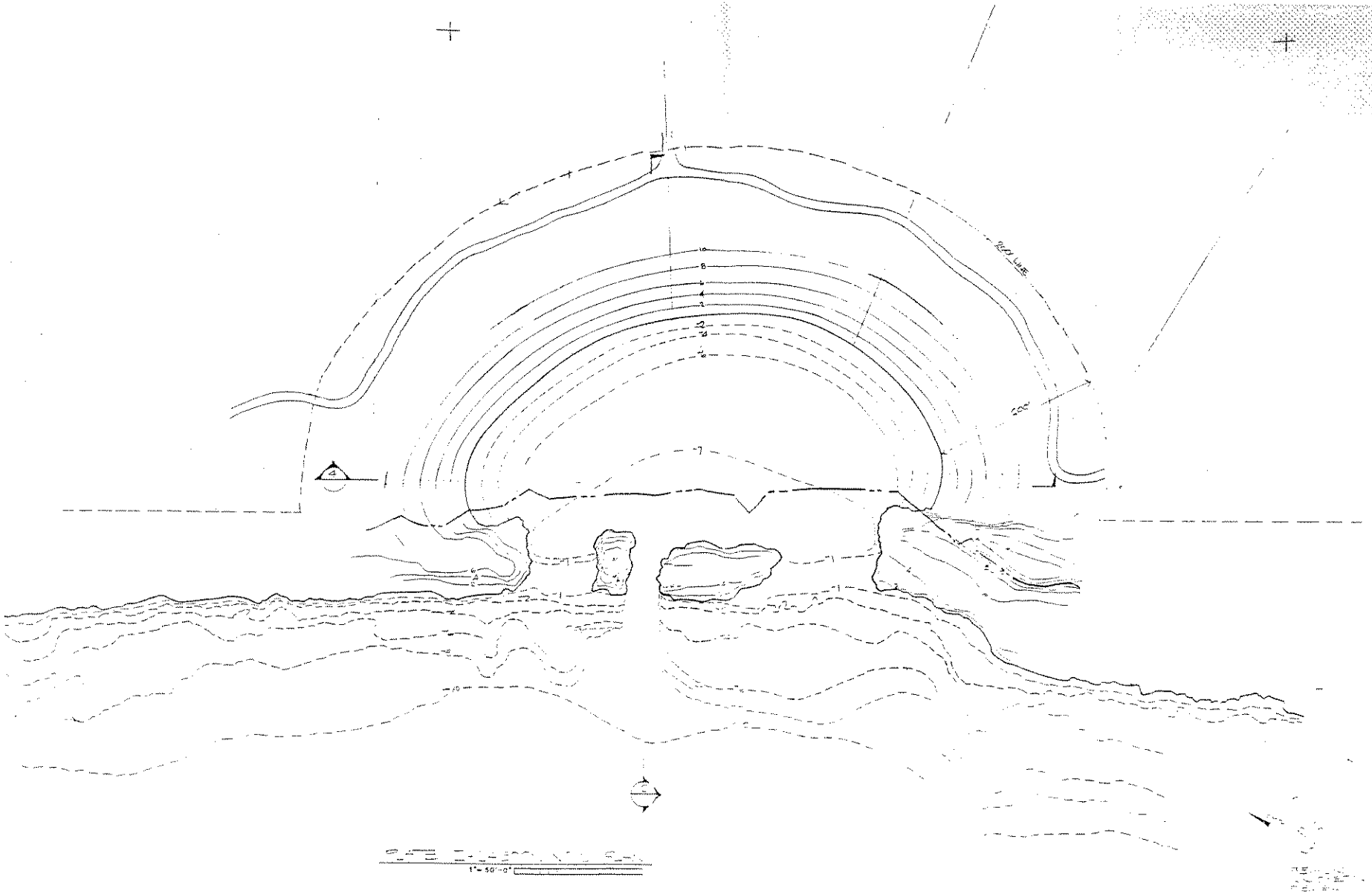

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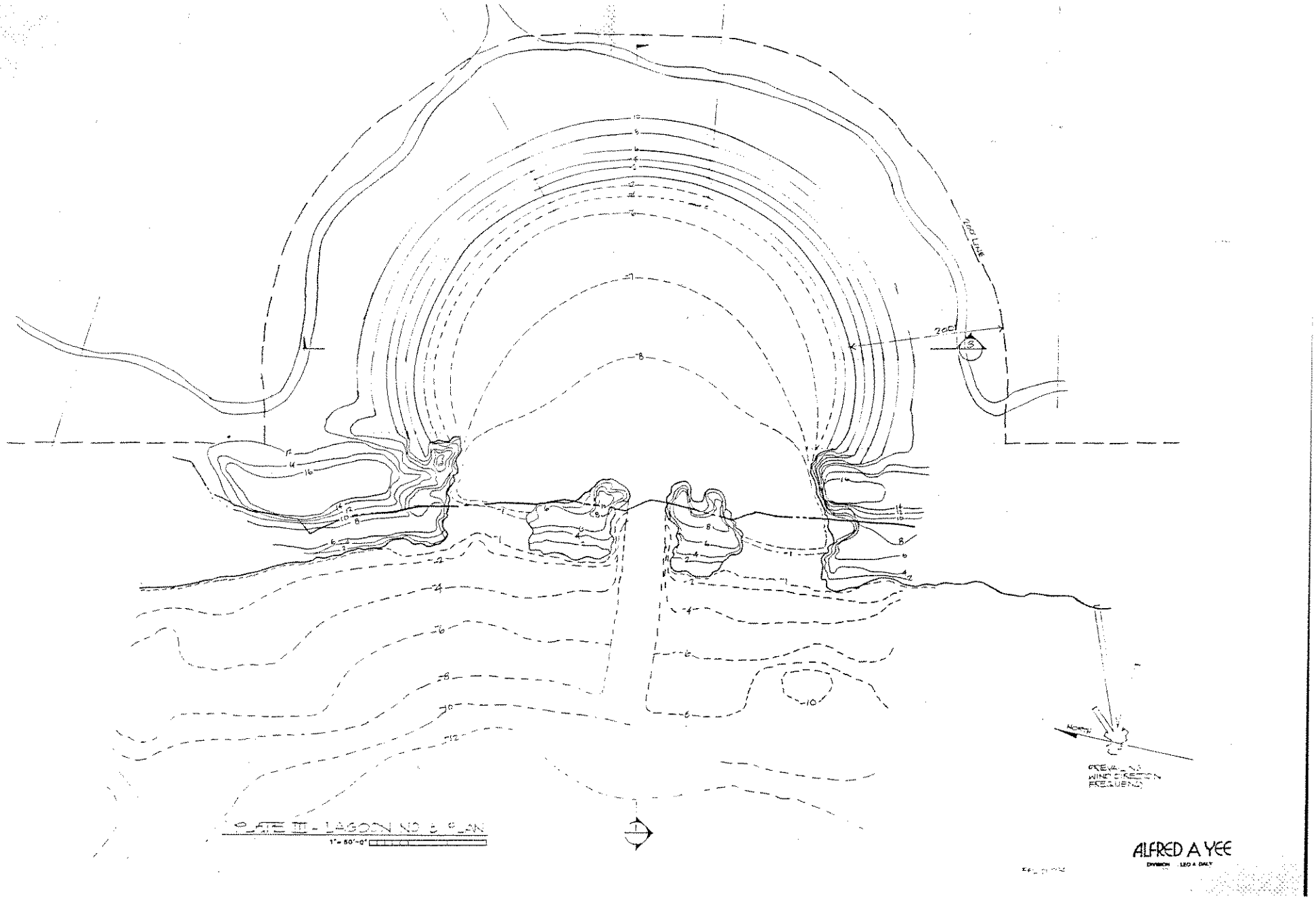
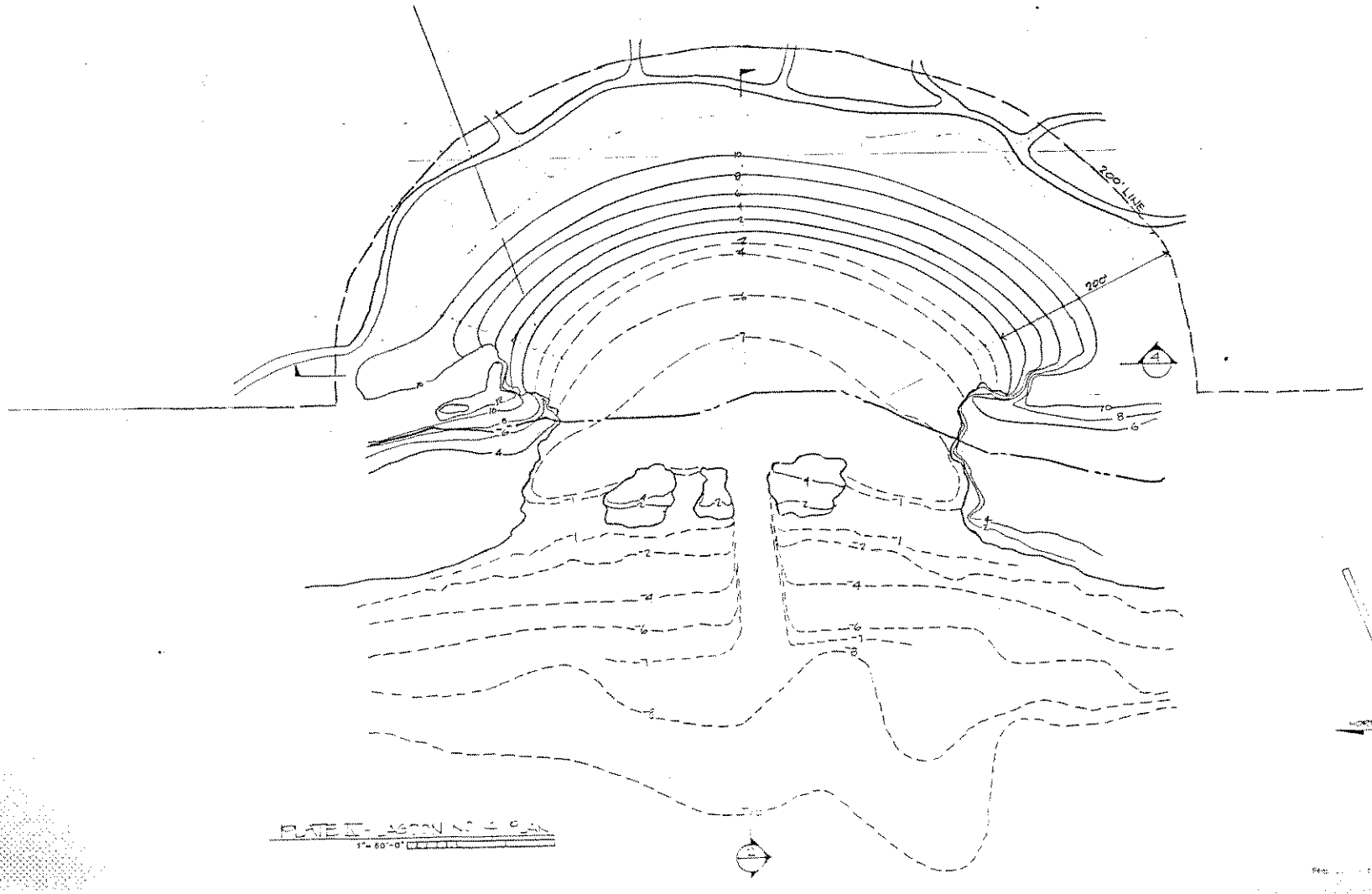
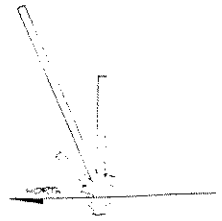

GATE III - LAGOON NO 2 PLAN
1" = 50'-0"
ALFRED A YEE
DIVISION - LEO A DALY





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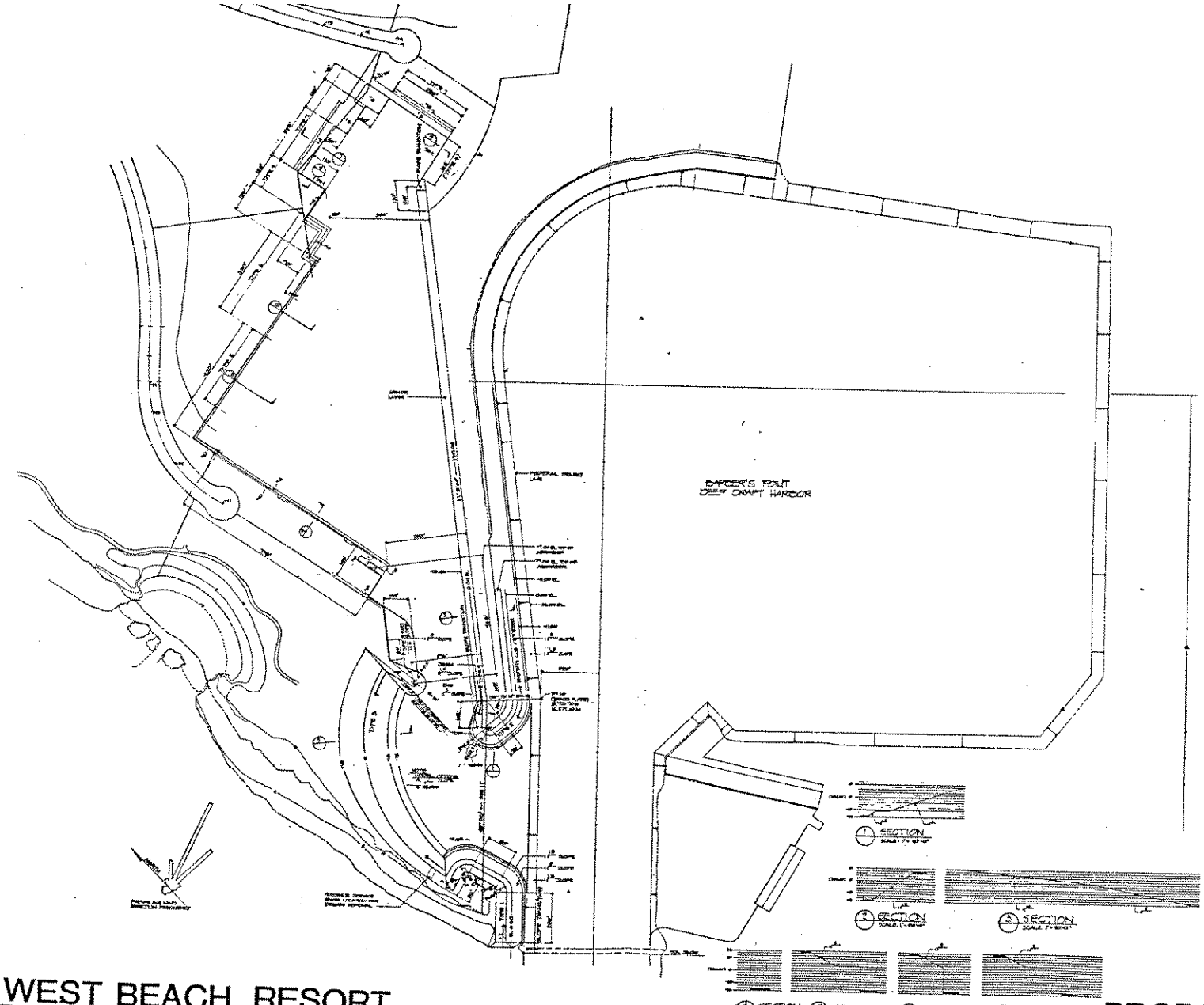
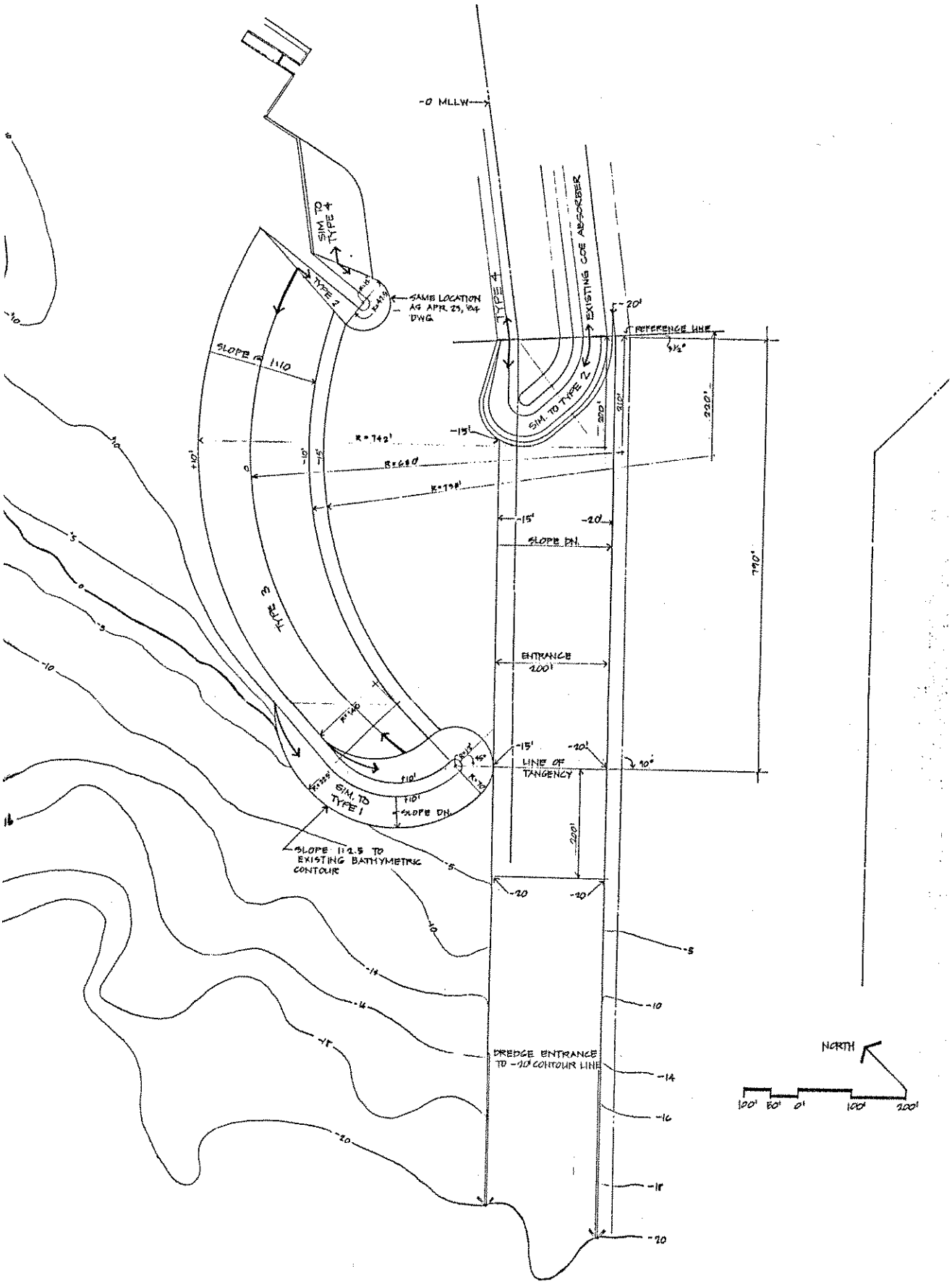


WEST BEACH RESORT

PROPOSED MARINA

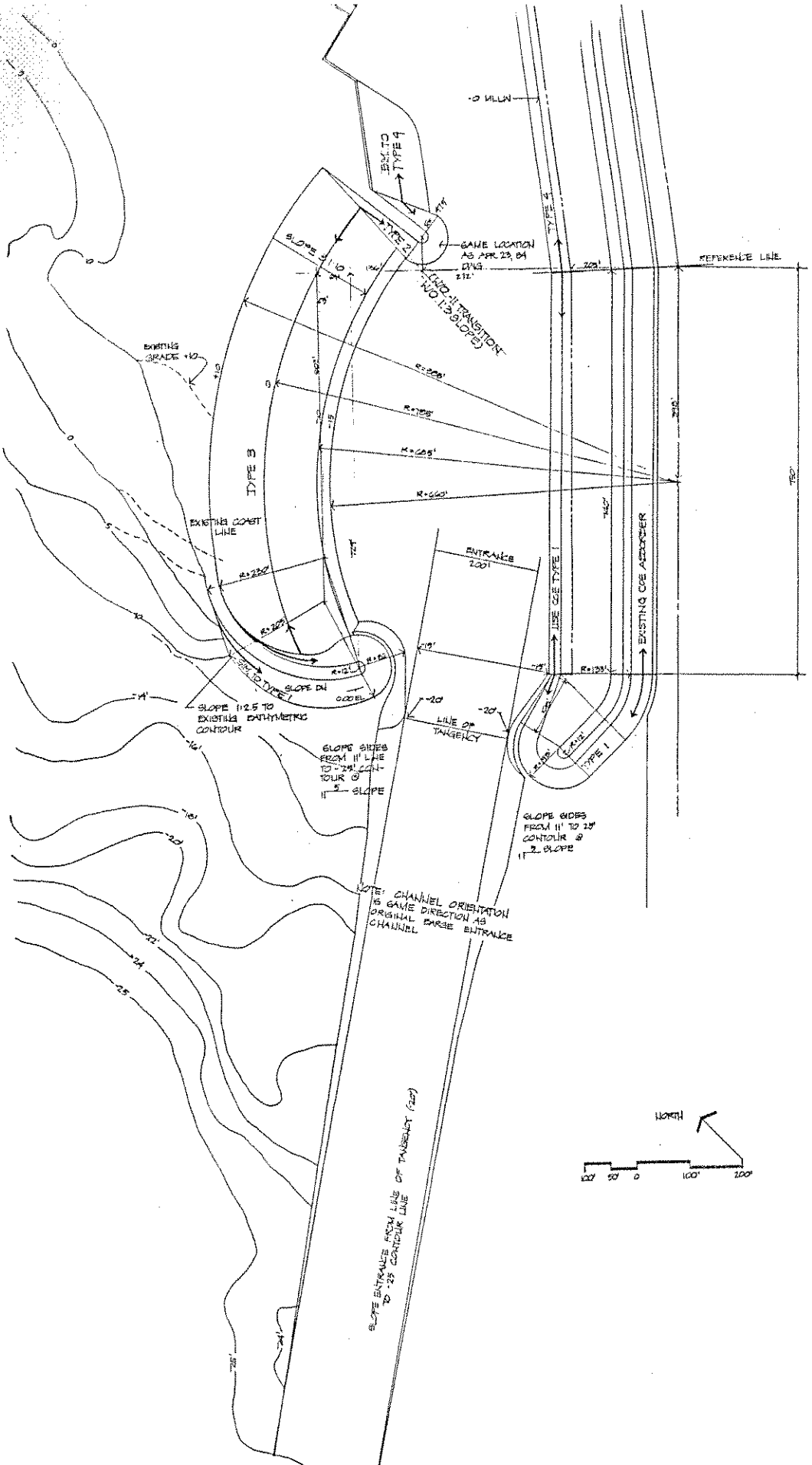
PLATE V



WEST BEACH RESORT

PARALLEL CHANNEL OPTION

PLATE VI



WEST BEACH RESORT — SEPARATE CHANNEL OPTION

PLATE VII

CHARLES L. BRETSCHNEIDER & ASSOCIATES, Ltd.
OCEANOGRAPHIC CONSULTANTS
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TSUNAMI AND HURRICANE DESIGN CRITERIA

AND

RECORDED WAVE DATA ANALYSIS

FOR

WEST BEACH DEVELOPMENT AREA

by

Charles L. Bretschneider

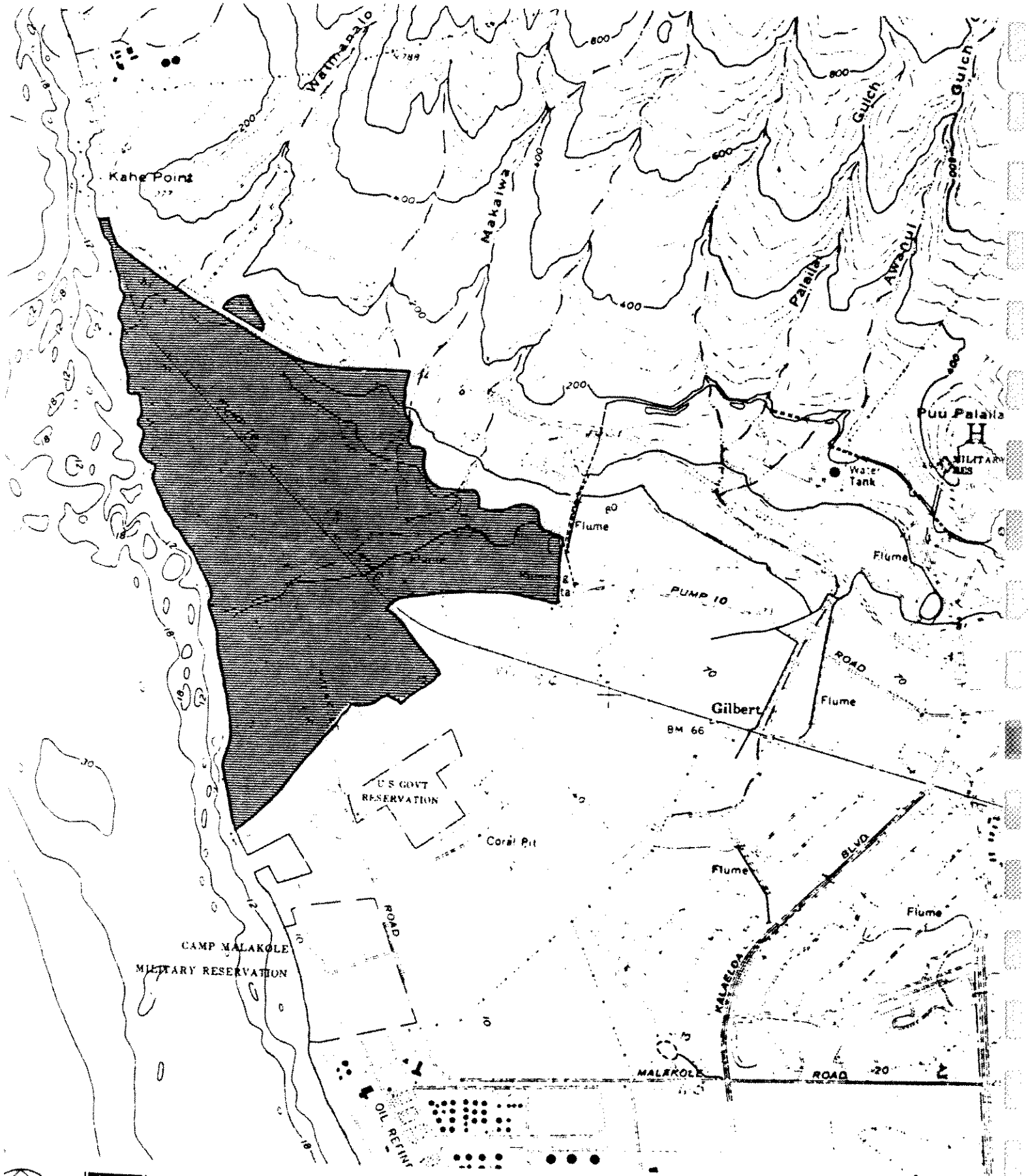
Prepared For

Herbert K. Horita Realty, Inc.
Honolulu, Hawaii

TECHNICAL REPORT NO. 9

Report In Three Sections

March 1984



VEST BEACH RESORT

PETITION AREA

EXHIBIT B

CHARLES L. BRETSCHNEIDER & ASSOCIATES, Ltd.
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TSUNAMI AND HURRICANE DESIGN CRITERIA AND RECORDED WAVE DATA ANALYSIS FOR WEST BEACH DEVELOPMENT AREA

CONCLUSION AND RECOMMENDATIONS

1. INTRODUCTION

A detailed report of 129 pages and appendices was prepared after a very detailed study and analysis of the West Beach area was made. The latest techniques developed by Dr. Charles L. Bretschneider, prior and during the course of the study, were used as the basis of the report. In addition to various references and sources of data information, these main references and sources of data were used in the above report:

1.1. Manual for Determining Tsunami Run-Up Profiles on Coastal Areas of Hawaii, prepared by M&E, Inc. for U.S. Army Corps of Engineers POD, August 1978. The above report is based in part on published theory of Bretschneider and Wybro (1976).

1.2. Hurricane Iwa (23 November 1982), U.S. Weather Service, Honolulu, weather charts, weather data and publications; and BRET MODEL-X hurricane wind and pressure fields (1981, UNIDO, United Nations report), and BRET 2-D wave forecasting relationships (1980, DOE Department of Energy, OTEC reports, Look Lab, University of Hawaii).

1.3. SEA ENGINEERING, INC. (1983), oceanographic data collection for coastal engineering analysis and design, West Beach, Oahu, Hawaii, 11 pp plus 19 pages data, #1 thru #9 data sets, 798 - 34-minute wave records, from June 22, 1981 to Jan. 9, 1982 and May 3-10, 1982.

2. GENERAL STATEMENT

The above study has been the most comprehensive study made in the State of Hawaii using the very latest available theories and developments, supplemented by actual recorded wave data from four main source areas:

(1) trade winds

(2) kona storms

(3) southern swells originating in the Southern Hemisphere, and

(4) west-west-northwest-north swells originating in the North Pacific and the Gulf of Alaska.

3. SUMMARY, CONCLUSION AND RECOMMENDATIONS: TSUNAMI INUNDATION ELEVATIONS

3.1. The tsunami inundation elevations 200 feet inland from the coastline were calculated to be 8.2 feet and 9.0 feet MSL for the 100-year and 140-year return period, respectively. The 100-year recurrence interval corresponds to a one percent chance of occurring in any one year. The 140-year correspondes to 0.7 percent change and also the average of maximum of record of 140 years.

3.2 The above values of 8.2 and 9.0 feet corresponds to the then existing terrain roughnesses at the time when tsunami run-up actually occurred and assumed MSL conditions. If extreme high

tide also occurred 2 feet above MSL, the above elevations would be 10.2 and 11.0 feet respectively for the 100-year and 140-year return period.

3.3. If the terrain roughness is decreased by development, then additional run-up elevation height would result. Therefore, it is conservatively estimated (by calculation) that extreme tsunami run-up elevations would be 13.1 feet and 15.1 feet respectively for the 100-year and 140-year recurrence intervals.

3.4. The tsunami elevations also vary from location to location and could be lower than the extreme conditions. It is estimated that the variance in the ± 90 percent confidence limits are ± 2.0 feet. Therefore the mean tsunami elevations will be 11.1 feet and 13.1 feet respectively for the 100-year and the 140-year recurrence intervals.

3.5. Recommendations: In view of the above discussion it is recommended that the tsunami design elevations be used as follows:

11 ± 2.0 = 9 to 13 feet MSL for the 100-year return period, and

13 ± 2.0 = 11 to 15 feet MSL for the 140-year return period,

where it is understood that 13 and 15 feet in the above values are intended for those areas where the existing terrain roughness will be smoothed, and 9 to 11 feet can be used where no change has been made nor will be made to existing terrain roughness. The average values of 11 and 13 feet can be used where only small changes in terrain roughness will be made.

3.6. Tsunami Hydrographs. The 100-year and 140-year tsunami hydrographs for the entrance to Barbers Point West Beach Marina were determined by amplification and Froude scaling of the reconstruction 1960 Honolulu tsunami hydrograph. The amplification factor was determined from the ratio of the 100-year (and the 140-year) tsunami inundation limits between West Beach Marina and Honolulu in accordance with the Manual given previously in Section 1.h. Two types of hydrographs are given for each of the return periods, 100-year and 140-year as follows:

(1) Non-sinusoidal hydrograph in similitude with the reconstructed 1960 Honolulu tsunami hydrograph.

(2) A sinusoidal repeating hydrograph having a fundamental period and amplitude corresponding to the maximum of the non-sinusoidal hydrograph of (1) above.

4. SUMMARY CONCLUSIONS AND RECOMMENDATIONS FOR DESIGN HURRICANE

4.1. Two extreme storm conditions have changed maximum design wave conditions for the south coast of Oahu, namely the 7-11 Jan 1980 Kona Storm and Hurricane Iwa, 23 Nov 1982. These two storms have been analyzed in sufficient detail for the West Beach area to arrive at a design hurricane. The maximum value of the deep water significant wave height of 34.7 feet, which by coincidence corresponds to approximately the 100-year maximum significant wave as based on U.S. Army Corps of Engineers detailed hindcasts of non-tropical extreme storms, including kona storms.

Maximum significant wave heights for Hurricane Iwa (1982) were between 41 and 43 feet off Kauai corresponding to the 300-to-350-

year return period and off West Beach of about 34 to 35 feet corresponding to approximately the 100-year return period.

4.2. There was a squall line and a pressure jump in the barograph for Hurricane Iwa (1982). This pressure jump caused a bore type wave which caused water elevations of 17 to 19 feet above 11 to 13 feet MSL land at the Waianae Coast. These elevations were considerably higher than could be calculated by standard wave run-up procedures for concave slope.

4.3. Using storm tide of 3.2 feet and 0.8 feet MSL tide (+ 4.0 ft MLLW) for the design hurricane maximum wave run-up calculations for West Beach area were 10.8 ft MSL for straight slope and 12.7 feet concave slope, which for all practical purposes can be assumed to be 11 ft MSL and 13 ft MSL. The above values of wave run-up are compatible to the 11 ft MSL and 13 ft MSL tsunami wave run-up mean values for the 100-year and 140-year recurrence intervals.

4.4. In the same method of reasoning as given for the ± 90 percent confidence limits for tsunamis, it is concluded that the design hurricane wave run-up will be 11 ± 2.0 ft MSL and 13 ± 2.0 ft MSL.

4.5. Therefore, it is recommended that the design hurricane wave run-up be established at the same elevations as the design tsunami wave run-up recommendations given in Section 3.5.

5. SUMMARY CONCLUSIONS AND RECOMMENDATIONS BASED ON WAVE RECORDED DATA ANALYSIS OFF WEST BEACH

5.1. Sea Engineering, Inc. (1983) recorded 798 34-minute wave data sets (See 1.3). These data represent one of the most comprehensive

recorded data sets to date in Hawaii, and first off West Beach area. However, the instrumentation was installed after the 1980 Kona Storm and was retrieved prior to the 1982 Hurricane Iwa. Nevertheless very much useful information on types of waves off West Beach were recorded.

5.2. In particular long period swells appeared in almost all wave records, swells from the south and swells from the west-northwest-north. There were many wave records for which short period maximum wave heights were superimposed on the long period swells. Perhaps this is one of the reasons why observed wave run-up elevations often exceed wave run-up calculated by standard techniques.

5.3. Recommendation. It is recommended that wave run-up for long period swells off West Beach be increased in elevation height because short period maximum wave heights are superimposed on the swells. Based on the analysis of potential wave run-up a suggested method was given, which in effect recommends that run-up calculations be increased by the suggested ratio of 1.17. This would be in addition to all other corrections and adjustments in wave run-up such as scale effects and concave slopes.

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TSUNAMI AND HURRICANE DESIGN CRITERIA AND RECORDED WAVE DATA ANALYSIS FOR WEST BEACH DEVELOPMENT AREA

GENERAL SUMMARY

The report consists of three independent sections: I - Tsunami Study, II - Hurricane Study, and III - Wave Data Analysis. Each of the above three sections include a separate summary, an introduction, a detailed text with technical discussion, and the results and interpretations of the calculations.

The purpose of this General Summary is to put into proper perspective and interpretations of the overall results of the three sections as given in the three separate summaries.

I - Tsunami Study. The Tsunami Run-Up and Inundation limits were established for the 100-year and 140-year recurrence intervals. The basic reference and procedures used for the determinations are given in the manual by the U.S. Army Corps of Engineers, P.O.D. (1978) Manual for Determining Tsunami Run-Up Profiles on Coastal Areas of Hawaii, called the Tsunami Run-Up Manual.

The tsunami elevations and inundation limits depend on the initial tsunami, the roughness of the coastal terrain and the slopes or elevations of the coastal terrain at the time/place of the statistical tsunami data incorporated in the above manual. It is estimated by calculations for the

100-year and 140-year recurrence intervals that the tsunami elevations two hundred feet inland from the coastline would be 8.2 and 9.0 feet respectively. Where the land elevations are greater than the above tsunami height limits, less than 200 feet inland from the coastline, that distance would be at the inland limits of 8.2 feet and 9.0 feet.

The tsunami height also depends on the tide and the present or future planned roughness. Therefore, it is conservatively estimated that the 100-year recurrence interval tsunami elevations will be 10.2 feet and 13.1 feet respectively for roughness elements corresponding to Manning's n = 0.05 (very rugged) and Manning's n = 0.0175 (smooth, fine sand or mud flats). The corresponding values for the 140-year recurrence interval tsunami elevations are conservatively estimated at 11.0 feet and 14.1 feet for n = 0.05 and n = 0.0175 respectively. The energy grade lines at the coastline are E.G.L. = 15.9 feet and 17.3 feet respectively for the 100-year and 140-year recurrence intervals.

In view of the fact that the original tsunami run-up data varies from location to location in the same general area, it seems appropriate to assign some type of confidence limits to the results. For example, in wave spectra statistics the 90 percent confidence limit is generally used, and for 20 to 40 degrees of freedom, the upper and lower 90 percent fiducial confidence limits are about 1.5 and 0.6 respectively. Since the spectral energy is proportional to the square of the wave height, it can be inferred that the corresponding fiducial confidence limits for wave height would be the square root of 1.5 and 0.6, or 1.22 and 0.77 respectively. Thus it can be assumed that the tsunami heights can also vary by approximately ± 22 percent of 8.2 and 9.0 feet, or about ± 2.0 feet from location to location within the West Beach Development Area.

Thus for the 100-year and 140-year recurrence interval the 90 percent fiducial confidence limits for maximum tsunami run-up for very rugged terrain (n = .05) would be at 12.2 and 15.1 feet MSL respectively. The corresponding values for Manning's n = .0175 would be 13.0 feet and 16.1 feet respectively.

In view of the above conservative assumption it is recommended that the upper limits for tsunami inundation at West Beach Development Area be established at 13 feet for the 100-year recurrence interval and 15 feet for the 140-year recurrence interval.

In addition to the above tsunami inundation limits also are given a design and an alternate design hydrograph for the 100-year recurrence interval. Tables are given for both the 100-year and 140-year recurrence intervals. The tsunami hydrographs are required for input time history water elevations at the channel entrance to the West Beach Marina. The tsunami hydrographs are based on the reconstructed 1960 tsunami hydrograph at Honolulu tide gage. The 100-year recurrence interval inundation calculations using the Tsunami Run-Up Manual are essentially in agreement with the maximum of the reconstructed 1960 tsunami hydrograph. Therefore, it is assumed that the reconstructed 1960 tsunami hydrograph corresponds approximately to the 100-year recurrence interval for Honolulu. An amplification factor was applied to the reconstructed 1960 Honolulu hydrograph to determine the 100-year West Beach hydrograph. Froude scaling was used to determine the 140-year West Beach hydrograph from the 100-year West Beach hydrograph.

The amplification factor for the 100-year recurrence interval was determined from the ratio of the inundation limits (200 feet inland from the coastline) between West Beach and Honolulu. The inundation limits were determined by use of the Tsunami Run-Up Manual. The amplification factor was determined to be $R_1 = 1.62$ for the 100-year recurrence interval. There is no Froude

scaling between the reconstructed 1960 Honolulu hydrograph and the West Beach 100-year hydrograph, only the amplification factor $R_1 = 1.62$ is applied.

The Froude scaling of the 100-year hydrograph to the 140-year hydrograph has a length scale ratio between the 140-year and 100-year inundation limits 200 feet inland from the coastline and is found to be $\lambda = 1.099$ and the time scale $\tau = \sqrt{\lambda} = 1.05$.

The alternate tsunami hydrograph is simply a repeated sinusoidal hydrograph using the maximum amplitude equal to one-half the vertical distance between the maximum crest elevation and the average of the minimum trough elevations preceding and following the maximum crest elevation. The 100-year and the 140-year alternate hydrographs have amplitude of 8.1 feet and 8.9 feet respectively. The corresponding primary periods are 40 minutes and 42 minutes respectively.

Depending on the scope of effort required, it would be possible to reconstruct the entire 1960 Honolulu tsunami record, apply the given amplification factor, and determine a longer tsunami hydrograph for the West Beach Marina Channel entrance for the 100-year recurrence interval. Froude scaling can then be used to determine the corresponding 140-year hydrograph for the channel entrance.

II - Hurricane Study. The Hurricane Study represents the most recent developments and methods for determining model hurricanes. The BRET MODEL-X is the most probable model hurricane for determining the hurricane pressure and wind field. The BRET 2-D wave model has been coupled with the BRET MODEL-X hurricane from which one can determine deep water significant wave heights, period and direction and the Bretschneider (1959) three-parameter wave spectrum. The BRET 2-D wave model when coupled with the appropriate

hurricane model (Rankin Vortex Model for small diameter hurricane) has been verified by wave data from Hurricane Camille (1969). A universal equation for determining the maximum wind speed in a hurricane has been developed from data based on central pressure and maximum winds from 319 Western North Pacific cyclones (12-year period from 1971-1982) and also data on forward speed and radius of maximum wind from 122 U.S. East Coast and Gulf Coast hurricanes. The equation when applied to Hurricane Iwa (1982) has verified the maximum wind of 80 knots reported by RECON aircraft.

The Rankin Vortex number is used as a guide for the selection of the appropriate hurricane model. For example, the Hydromet Rankin Vortex Model applies to Hurricane Camille (1969) of the Gulf of Mexico. The BRET MODEL-X hurricane applies to Hurricane Iwa (1982).

The BRET MODEL-X hurricane parameters were used in the gradient wind equations to calculate the maximum gradient winds at various radial distances from the hurricane and for various forward speeds of translation and were correlated with the adjusted wind speeds to the 10-meter level measured at Barking Sands, Lihue, Barbers Point, and Honolulu, and finally establishing a radius of maximum wind of approximately 50 nautical miles.

Allowing a five percent reduction in wind speed due to coastal roughness, it was determined that the ratio of the 10-meter level over water wind speed to the gradient wind speed was 0.78 approximately. Thus, the 10-meter elevation wind speed is correlated with averages of 1-minute average wind speed, which has been designated as V_{10}.

The BRET 2-D wave model was previously correlated with V_{10} for both tropical and extra-tropical cyclones. Refined calibration of the BRET 2-D wave model was made by use of data from Hurricane Eloise (1975).

The BRET 2-D wave model was then applied to Hurricane Iwa (1982) to determine maximum wave conditions from near the center of the hurricane to a radius of about 200 nautical miles. Maximum values of significant wave height for Hurricane Iwa were from H_s = 26.8 feet for stationary hurricane, and for the moving hurricane H_s = 41, 41, 42.7 and 44.6 feet respectively for forward speeds of 19, 20, 25 and 30 knots. The maximum RECON wind speed was taken as 80.8 knots for all cases. The corresponding overwater 10-meter level wind speeds were 55, 63.1, 62.9, 62.7 and 62.5 knots respectively for the stationary hurricane (forward speed = 0) and 19, 20, 25 and 30 knots.

The maximum conditions for Hurricane Iwa most probably occurred for forward speeds of 20 or 25 knots, corresponding to maximum significant wave heights between 41 and 43 feet.

A thorough review of previous hurricane wave hindcasts and also because of the closeness of Hurricane Raymond (1983) and the results of Hurricane Iwa, an estimate of a Design Hurricane was made for West Beach Area. The parameters finally selected were latitude ϕ = 20.5°, radius of maximum wind (R_c = 32) R = 30 nautical miles, central pressure P_o = 964 mbs, same as for Hurricane Iwa and P_∞ = 1012, 2 mbs greater than Hurricane Iwa. Various forward speeds, V_F = 0, 12, 15 and 18 knots were tested, and V_F = 15 knots was retained as the most probable value for the Design Hurricane most likely corresponding to the 100-year recurrence interval.

Significant waves and wave spectra for maximum conditions were calculated for radial distance from near the center of the design hurricane to radial distance of 180 nautical miles. The maximum significant wave height was 34.7 feet, corresponding to the overwater wind speed, V_{10} = 65.3 knots.

Previous calculations were made for the January 7-11, 1980 Kona Storm off Sand Island, for which the hindcast maximum significant wave height was 29 feet.

Assuming that the Design Hurricane corresponds approximately to the 100-year recurrence interval, then the Kona Storm significant wave height of 29 feet is probably between the 25-year and the 50-year recurrence interval, and the recurrence interval for Hurricane Iwa exceeds 200 to 250 or more years.

A summary of West Beach Design hurricane wind and wave data are given in Table I together with other past hindcasts made by Bretschneider. Details of wave spectra are given in the text.

Wave Run-Up calculations were made for both the Waianae Coast and West Beach using the conditions set forth for Hurricane Iwa. Refined standard formulations were used for smooth slope and 25% increase for concave slopes. For T = 14.5 seconds and total tide of 4 feet MLLW, maximum run-up calculated was 7.6 x 1.25 = 9.6 feet + 3.2 feet tide and storm tide = 12.8 feet MSL for West Beach. Similar calculations made for the upper Waianae Coast resulted in total maximum run-up of 12.5 feet MSL. In the U.S. Army Corps of Engineers P.O.D. (1983) Post Disaster Report Hurricane Iwa (1982), it is reported that observed high water elevations 200 feet inland from the coastline were 17 and 19 feet MSL. Thus, conservative standard wave run-up calculations do not account for the full extent of flooding by Hurricane Iwa (1982). The land elevations corresponding to 17 and 19 feet above are approximately 11 and 13 feet respectively. Thus, there is six feet water depth not accounted for.

There was something quite different associated with Hurricane Iwa that caused the excessive run-up and which could be associated with any future hurricane in Hawaii. That is, a squall line passed over Hawaii during the peak of the storm. There was a pressure jump of 3.2 mbs at Honolulu (See Storm Data, November 1982, NOAA, pp 4-8).

TABLE I

SUMMARY OF SIGNIFICANT WAVES AND WINDS FOR WEST BEACH DESIGN HURRICANE AND VARIOUS OTHER HURRICANE AND KONA STORM HINDCASTS BY C.L. BRETSCHNEIDER

Condition	R N.Miles	V_F Knots	V_{10} Knots	V_{MAX} Knots	$H_{1/3}$ Ft	T_s Sec	f_o^{-1} Sec	H_B Ft	d_B Ft	S_w Ft	Note
West Beach Design Hurricane	30	15	65.3	83.7	34.7	12.1	13.8	29.3	37.5	2.5	(1)
Hurricane Iwa (1982)	50	20	62.9	80.6	41.0	14.2	16.2	36.4	46.6	2.8	(2)
Hurricane Iwa (1982)	50	25	62.7	80.4	42.7	14.7	16.9	38.2	49.0	2.9	(2)
Hurricane Nina (1957)	--	--	50.0	---	25.5	13.0	14.0	25.0	32.0	1.6	(3)
OTEC 100-Year Keahole Point	12	10	61.6	69.5	26.7	9.8	11.3	21.4	27.4	2.0	(4)
OTEC 100-Year Keahole Point	12	10	66.3	74.8	25.7	9.2	10.5	20.0	25.6	2.0	(4)
Kona Storm January 1980	--	--	42.0	---	29.0	13.5	15.4	27.9	35.8	1.8	(5)

NOTES:

(1) Latest methods and information used based on Note (2) below. BRET MODEL-X hurricane wind MODEL-X, BRET 2-D wave model

(2) Considerable analysis (at least first approximation) BRET MODEL-X wind model overwater V_{10} wind speeds calibrated for land station measurements.

(3) Wilson's space time wind field method used.

(4) Hydromet Rankin Vortex wind model, BRET 2-D wave model (in view of Hurricane Iwa Note (2) and results Note (1), Keahole OTEC should be upgraded).

(5) BRET 2-D (modified 1-D, one-dimensional wave model).

R = radius of maximum winds; V_F = forward speed; V_{10} = 10-meter elevation overwater winds; V_{MAX} = MAX RECON or gradient wind; $H_{1/3}$ = significant wave height; T_s = significant period; f_o^{-1} = modal period of spectrum; H_B and d_B = breaking wave height and depth; S_w = wave set-up.

Similar pressure jumps have been associated with squalls. See Abraham (1961, ch. 31) where he summarized a number of squall events, did theory, and conducted model studies to show that this was a resonance phenomenon. For example, in 1954 a squall line over Southern Lake Michigan, having a pressure jump of 0.1 feet water head depression (equivalent to 3.0 mbs), was followed by a wave which reached a velocity of $\sqrt{gd}$, created large waves (several as a matter of fact) which, when arriving at Michigan City reached 5 to 6 feet and when it struck the Chicago breakwater with surges 3 to 8 feet, it caused the drowning of 7 persons (See Weigel, 1964, page 109).

Now returning to Hurricane Iwa, there would have had to be a surge, say 2 feet for a bore type wave Froude number $F = 2$, $V = 2\sqrt{gh}$ to have been associated with the squall line to have caused the water to rise 6 feet above the land elevations, for example, if it had struck a vertical structure; or with $F = 1$, say a surge of about 3 to 3.5 feet for $F = 1$ to do the same.

In case of the Waianae Coast, eyewitnesses saw such types of waves. In one case a driver and a car were engulfed by the wave and pushed off the road. Many large stones of coral were brought up from the ocean side of the beaches and transported far inland. Large slabs were moved and rounded coral blocks as large as a foot to a foot-and-a-half in diameter were transported inland.

The point here is that such a surge is possible and is relevant to the present study and should be given some consideration for West Beach area. The fact that the Corps of Engineers (1983) report does not show any damage or high run-up for West Beach does not mean high run-up did not occur. No one was there to study the area. Tsunami run-up at West Beach is predicted higher than for Honolulu and Waikiki, which had large run-up damage during Hurricane Iwa.

In regard to the 100-year Design Hurricane such high water levels will not be so large as those observed for Hurricane Iwa (1982) but the run-up will be greater than that calculated by conservative standard methods.

Presently only an estimate can be made because considerable additional work would be required.

It would probably be reasonable to assume that the 100-year wave run-up for West Beach would be approximately 15 feet or about the same as governed by the tsunami flood determinations.

III - Wave Data Analysis. Actual wave recorded data is essential for all present and future projects. Wave recorded data approximately 4400 feet offshore in 45 feet MSL water depth were made by Sea Engineering, Inc. (1981-1982). A total of 798 34-minute long records were obtained. The data included waves from

(a) northern swell;

(b) Kona storms; and,

(c) southern swell

as well as short period tradewind waves superimposed thereon. From the data records, it appears that long period swells are very frequent with T = 20 to 22 seconds or more. Such long period swells originate from the Southern Hemisphere winter storms, from early June through late August or September. Fourteen to 16 second swells arrive from the Northern Hemisphere winter months, early October through April or May.

Kona storms during the winter months in Hawaii would generate waves upon the swells from the south or southwesterly directions. Tropical storms during the hurricane months would generate large wave superimposed on the swells from the Southern Hemisphere. The combination of summertime hurricane waves in

Hawaiian waters and southern swells from a severe Southern Hemisphere winter storm would represent a situation to be considered. The combination of winter Kona storm waves with winter swells from the north or northwest should also be considered.

Thus, the run-up problem becomes quite complex. It would appear from the data analysis (Section 3) that one of the worst combinations for maximum wave run-up would be local Kona waves superimposed on 20 to 22 period southern swells. The problem to be addressed is, "How do the long period swells interact with the Kona waves? Does the swell when in shallow water depth become near-breaking actual breaks at much greater heights when they are riding with the Kona waves?" From the data analysis, this would seem to be the case--that the wave will break at greater heights and therefore the corresponding run-up would also be greater.

Probability distributions for either a narrow spectrum or a broader spectrum would not solve the problem of wave run-up. This type of superposition or combinations would result in a double-peaked spectrum wherein the two types of waves would be separated into two independent spectra. It would seem that the only way to really solve the problem is by conducting hydraulic model tests for the above-type wave combinations.

Wave run-up calculations depend strongly on the wave period. Wave periods from southern swell, for example, are longer than for the Design Hurricane but the swell heights are less, and the probability of large breaking waves is less. However, with the superposition of large secondary local Kona waves, for example, can pose a problem of concern. The combination of the two sets of wave trains can create conditions in wave run-up similar to the breaking conditions for hurricane waves. There are two differences:

(1) First, the wave period will be longer tending to give larger run-up; and,

(2) Second, there will be little storm tide as compared with a hurricane.

Thus, a larger run-up will be from an elevation 2 to 3 feet lower. These are compensating factors. Again, it is reasonable to assume that the maximum run-up from MSL for the complex wave systems discussed above will be essentially the same as governed by the design tsunami. However, the probability of occurrences may be different. Considerable additional research and model testing is required for the West Beach complex wave systems.

Based on the data analysis three highly correlated equations, based on group data analysis, have been determined, which can be used to predict the design wave period, T_{hx}, of the design maximum wave height, H_x, including one standard deviation corresponding to the product variance between group data and original data. The final form of the equation are as follows:

$$1)\quad \frac{T_{hx}}{T_{1/3}} = 0.74930\sqrt{\frac{H_x}{H_{1/3}}}\left[1 + 0.0218\,\frac{gT_{1/3}^2}{10^3 H_{1/3}}\right] \quad [1 \pm 0.13]$$

$$2)\quad \frac{T_{hx}}{T_p} = 0.65181\sqrt{\frac{H_x}{H_{1/3}}}\left[1 + 0.0320\,\frac{gT_{1/3}^2}{10^3 H_{1/3}}\right] \quad [1 \pm 0.18]$$

$$3)\quad \frac{T_{1/3}}{T_p} = 0.86984\left[1 + 0.0155\,\frac{gT_p^2}{10^3 H_{1/3}}\right] \quad [1 \pm 0.14]$$

H_x = maximum wave height, $H_{1/3}$ = significant wave height, T_{hx} = period of maximum wave, $T_{1/3}$ = significant period, T_p = modal period of spectrum.

In final conclusion, it is recommended that the flood limit be established at 13 feet and 15 feet MSL respectively for the 100-year and 140-year recurrence interval. These limits would seem to satisfy all three sections of the West Beach Study, at least approximately, until further recommended studies are made.

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WEST BEACH AREA AND MARINA STUDY:

DESIGN TSUNAMI, HURRICANES

AND

WAVE DATA ANALYSIS

This report is presented in the following three sections:

Section I: Tsunami Run-Up and Inundation Limits for the West Beach Development Area, and

Tsunami Hydrographs at the Entrance to the West Beach Marina.

Section II: Hurricane Winds, Waves, Wave Spectra, and Inverted Barometer Pressure Tide, and Unrefracted Breaking Waves and Wave Set-Up Directed Perpendicular to the Coastline of the West Beach Development Area.

Section III: An Analysis of the Recorded Wave Data Off West Beach Development Area, with Special Emphasis on Potential Wave Run-Up.

Each of the above three sections has its own technical division, summary and conclusion and list of references.

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SECTION I

TSUNAMI STUDY

SUMMARY

A. The Tsunami Run-Up study for the 100-year and the 140-year recurrence intervals for the West Beach Development Area (See Fig. 1 and Exhibit B) is based upon the latest theory, techniques and available historical data on tsunami run-up. The methods and procedures used are those methods originally proposed by Bretschneider and Wybro (1976), Tsunami Inundation Predictions. The above reference methods have been incorporated in the U.S. Army Corps of Engineers, P.O.D. Honolulu (1978), Manual for Determining Tsunami Run-Up Profiles on Coastal Areas of Hawaii. Also incorporated in the above manual are statistical methods for determining recurrence intervals and corresponding run-up elevation 200 feet inland from the coastline. The statistical methods, based on the analysis of past historical data on tsunami run-up in Hawaii were developed by Houston, Carver and Marble (1977), Tsunami-Wave Frequency of Occurrence for the Hawaiian Islands. The above statistical work was done by Computor at the U.S. Army Corps of Engineers, Waterways Experiment Station, Vicksburg, Miss.

Fig. 2 shows the schematic approach to calculate tsunami run-up and inundation limit. Fig. 3 gives the Tsunami Run-up 200 feet inland from the coast, assuming that the land is sufficiently low so that the water

reaches 200 feet inland from the coasts. Figs. 4 and 5 give the results of the calculations for the 100-year and the 140-year recurrence intervals. The run-up elevation intersects the land topography at different locations depending on the roughness of the terrain. For the existing roughness the inundation limit is 160 feet and 170 feet inland from the coast for the 100-year and 140-year recurrence intervals. The inundation limit increases as the roughness decreases. The limit for smooth slope of zero roughness will be that where the energy grade line elevation at the coastline is equal to the land elevation inland from the coastline. The energy grade line at the coast is 15.9 and 17.3 feet respectively for the 100-year and 140-year recurrence interval (See Figs. 4 and 5).

B. Tsunami hydrographs to be used at the entrance to the West Beach Marina have been determined based on the 1960 tsunami recorded tide at the Honolulu tide gage: as reported by Hwang and Lee (1979) Tsunamis (See Fig. 6) and utilized by Houston, Carver and Marble (1977). The 100-year and 140-year tsunami run-up values determined for the West Beach Development Area were correlated with the corresponding run-up elevation determined for Honolulu, using the U.S. Army Engineers Manual (1978). This correlation was used to determine the amplification factors R_1 and R_2 for the 100-year and 140-year tsunami run-up elevations respectively. The run-up for Honolulu for the 100-year recurrence interval is 5.1 feet corresponding to an estimate of about 5.0 feet for the 1960 tsunami. The estimate of 5.0 feet at the Honolulu tide gage for the 1960 tsunami is based on an extrapolation of the tide record for missing data where the tide elevation exceeds the tide gage limit. The 1960 tsunami hydrograph was reconstructed to include estimates for the missing data.

Since the 100-year tsunami run-up at Honolulu is essentially the same as that for the estimate of the 1960 tsunami tide, it is assumed that the reconstructed 1960 tsunami hydrograph is approximately the 100-year tsunami hydrograph at Honolulu.

The Froude scaling law can be used to predict the 100- and 140-year tsunami hydrograph for the entrance to the West Beach Marina, but first one must take into account the amplification factor between the West Beach Marina entrance and the Honolulu reconstructed tide record. For the 100-year tsunami hydrograph the time scale should be the same such that $\lambda = 1$ and $\tau = \sqrt{\lambda} = 1$ and the amplification factor $R_1 = 1.62$. For the 140-year hydrograph $R_2 = 1.78$, $\lambda = 1.78/1.62 = 1.0988$, and $\tau = \sqrt{1.098} = 1.05$. The predicted 100-year tsunami hydrograph is shown in Fig. 7. Tabulations for both the 100-year and 140-year recurrence intervals are given in Table 6.

An alternate design tsunami hydrograph, assuming a sinusoidal wave equivalent to the primary wave of the predicted 100-year tsunami hydrograph, is shown in Fig. 8. This alternate tsunami hydrograph may give somewhat different results in the Marina. The sinusoidal wave (hydrograph) is much easier to generate in the Hydraulic Laboratory. Both the predicted (non-sinusoidal) and the sinusoidal hydrographs can be programmed for Computor. A comparison can be made for the sinusoidal hydrograph between the computer results and the hydraulic laboratory results.

The design hydrograph given here, in fact, represents only a portion of tsunami period, and there are other periods that should be considered (See the Appendix of this Section).

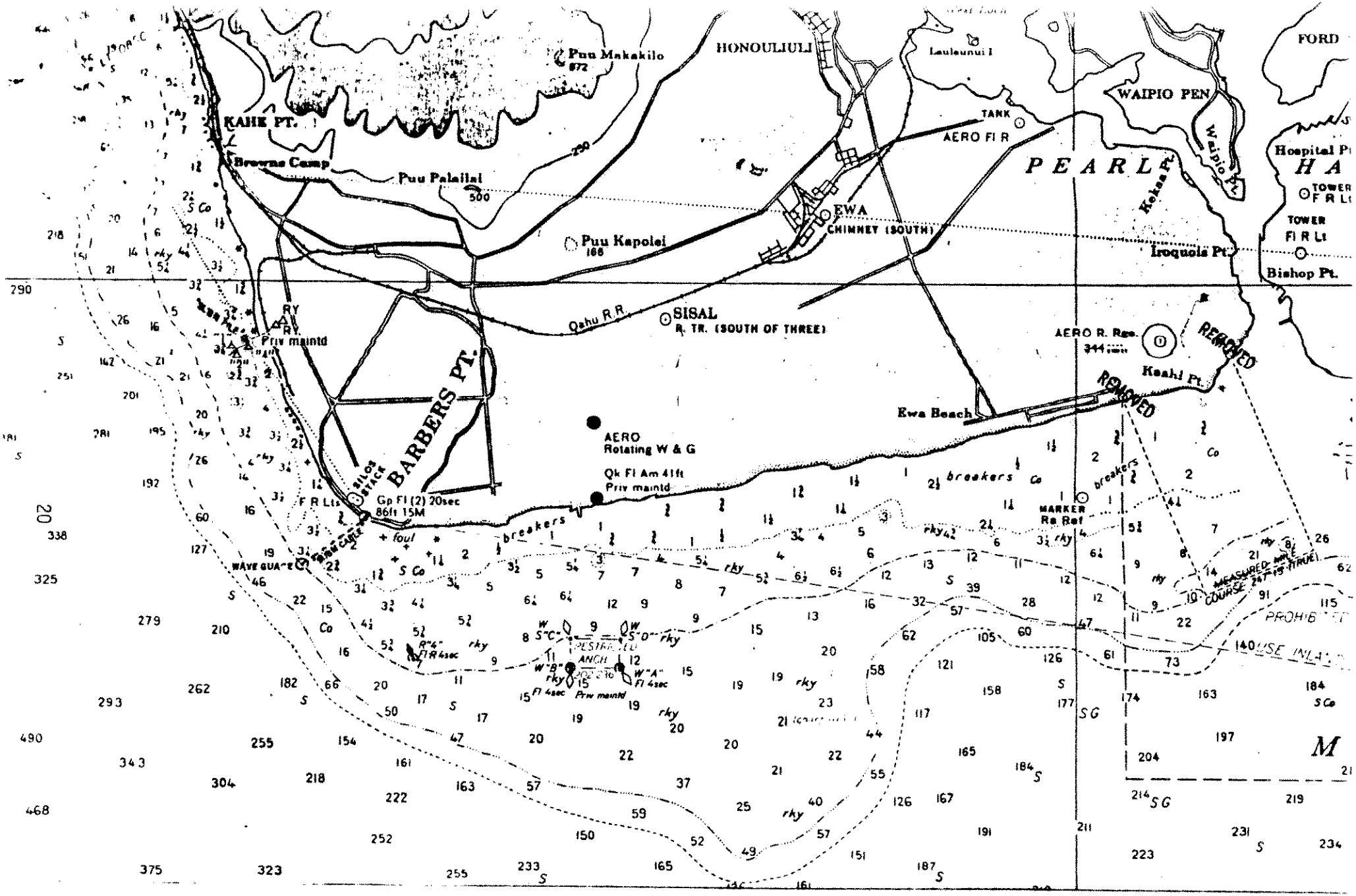


Figure 1. LOCATION MAP WEST BEACH AREA (BARBERS PT TO KAHE PT)

TSUNAMI RUN-UP AND INUNDATIONS FOR THE WEST BEACH DEVELOPMENT AREA

I. INTRODUCTION

In the past, most tsunami research has been directed more-or-less toward four general categories:

(1) source area and mechanisms of tsunami wave generation;

(2) propagation of tsunamis from source area to land areas or coastlines;

(3) harbor surging and beach run-up; and,

(4) historical data collection and statistical interpretations.

Very little consideration has been given to the nature of the flooding problem as a consequence of tsunami transformation into a flood wave at the top of the beach, dune or at the coastline and its propagation over previously dry bed or coastal terrain. All tsunami statistics are based on data that were compiled for the then existing topographical conditions at time of tsunami arrival. In case of Hawaii, this covers up to 140 years of tsunami data at Hilo, for example. If the pre-existing topographical conditions, elevations and/or roughness are changed as a result of urban and industrial development and, if the same tsunami event occurred, then the consequences of flooding will be entirely different than those which occurred from past history.

Since coastal regions are choice locations for urban and industrial development, there is a need to study the effect of bed roughness on potential tsunami flooding, because there can be a potentially dangerous risk in loss of life and property if the terrain elevation and/or roughness is decreased and the proper friction factors are not considered in obtaining the design

elevations. In addition, economical design evaluations depend on terrain elevation and bed roughness.

A literature search of friction factors has been performed, and much of the information based on canal, channel and river flow can be used. However, there is a lack of data on friction factors for typical Hawaiian terrain subjected to potential tsunami flooding. Some of this gap has been filled by determining friction factors using five-anemometer level wind profile measurements over the typical terrain, since it is obvious that one cannot use water current profiles. Detail technical reports and pertinent publication of the study are listed in the references.

2. DETERMINATION OF TSUNAMI ELEVATIONS, GROUND ELEVATIONS AND ENERGY GRADE ELEVATIONS 200 FEET INLAND FROM THE COASTLINE

The report by the U.S. Army Corps of Engineers (Reference 1) can be used to determine tsunami elevations above mean sea level. The location of these elevations are considered to be 200 feet inland from the coastline, except where the land elevations exceed the tsunami elevation. In this case, the tsunami elevation location will be somewhere between the coastline and 200 feet inland from the coastline where the two elevations coincide.

There are two coefficients A and B which must be determined from the appropriate plates given in the U.S. Army Corps of Engineers report (Ref. 1). The location of the particular site of interest can be located on the appropriate plate, and fine interpolations are required for accuracy. Once the coefficients A and B have been determined from Reference 2, then the following equations are used to determine the tsunami elevations to be used as the starting point for the calculations:

$$1) \quad H = -B - A \log_{10} F$$

2) $h_s = H - d_g$ (MSL)

3) $E \cdot G \cdot L = d_g + \sqrt{3}\, h_s$ (MSL) (non-bore)

In the above equations

H = elevation of the maximum tsunami (wave crest above mean sea level (MSL) 200 feet inland from the coast, or as otherwise limited by the elevation of the land as mentioned above.)

F = frequency per year of occurrence

F = 1/100 for the 100-year recurrence interval, F = 1/50 for the 50-year, F = 1/200 for the 200-year recurrence interval, etc.

h_s = the water surface depth is equal to the difference between the tsunami elevation and the ground elevation.

d_g = the ground elevation measured from mean sea level (MSL)

E·G·L = the energy grade line (MSL)

The energy grade line decreases from the coastline in proportion to the energy dissipation by friction and turbulence which are interrelated, and when E.G.L. = d_g, the inland extent of flooding is reached.

Since the tsunami elevations given in the U.S. Army Corps of Engineers report (Ref. 1) are for a location 200 feet inland from the coastline, both the tsunami depth of water and the energy grade line at the coastline will be greater, but the water surface elevation itself will be lower depending on the elevation of the land 200 feet inland from the coast. Thus, the origin for calculations begin at 200 feet inland from the coastline seaward to the coastline and inland to the point of zero water depth.

Example calculation for tsunami elevation 200 feet inland from the coastline: Given A = 22, B = 23 and F = 1/100 and d_g = 12 feet.

From Equation 1:

$$H = -23 + 22 \log 100 = 21 \text{ feet}$$

From Equation 2:

$$h_s = 21 - 12 = 9 \text{ feet, and}$$

From Equation 3:

$$\text{E.G.L.} = 12 + \sqrt{3}\ 9 = 27.6 \text{ feet}$$

This means that if a vertical wall were placed at this location to stop the tsunami, the water surface elevation would rise to 27.6 feet above mean sea level or 27.6 - 12 = 15.6 feet above ground elevation, d_g.

3. DETERMINATION OF TSUNAMI ELEVATIONS, GROUND ELEVATIONS AND ENERGY GRADE ELEVATIONS SEAWARD AND INLAND FROM 200 FEET INLAND FROM THE COASTLINE

The previous calculation shows how to determine the values for location 200 feet inland from the coastline. Bretschneider and Wybro (1976) have given equations for calculating the water surface elevations for the tsunami elevations as the flood wave progresses inland from the coastline, taking bottom conditions and friction into account. This is a one-dimensional steady state or quasisteady state and hence is only a good approximation, assuming all other conditions are properly selected, including the friction factors.

The basic steady state one-dimensional equation for tsunami inundation given by Bretschneider and Wybro (1976) is:

$$4) \quad \frac{dh}{dx} = -\left[\tan\theta + \frac{n^2 g F^2 h^{-1/3}}{(1.486)^2}\right]\left[1 + 0.5F^2\right]^{-1}$$

where

dh/dx = the slope of the water surface

$\tan\theta$ = the slope of the ground (assumed to be small)

n = Manning's "n", the friction factor

g = acceleration of gravity

$F = \overline{V}/\sqrt{gh}$ the Froude no.

For the special case of a flat bottom $\theta = 0$ and the solution of Eq. 4 becomes

$$5) \quad \frac{h}{h_o} = \left[1 - \frac{x}{x_R} \right]^{3/4}$$

$$6) \quad x_R = \frac{3}{4} h_o^{4/3} \frac{(1.486)^2}{g} \left(\frac{1 + 0.5\ F^2}{n^2\ F^2} \right)$$

The exact solution of Eq. 4 for $\tan\theta \neq 0$ is given by:

$$7) \quad x = \frac{3a}{b} \left[-\frac{1}{3}h + \frac{1}{2} \left(\frac{c}{b}\right) h^{2/3}\ h^{1/3} + \left(\frac{c}{b}\right)^3 \ln \left(1 + \frac{b}{c} h^{1/3}\right) \right] + \text{const}$$

where

a = $\tan$ $\neq 0$

$$b = \frac{n^2 g F^2 h^{-1/3}}{(1.486)^2}$$

$c = (1 + 0.5\ F^2)$ and $F = \overline{V}/\sqrt{gh}$

The const in Eq. 7 is evaluated where $h = 0$ (i.e. d_g = E.G.L.) and is given by

$$8) \quad x_R = \text{const} = \frac{3}{4} \left[\frac{h_o}{F^2 n^2} \right] \left[\frac{(1.486)^2}{g} \right] \left[1 + 0.5\ F^2 \right]$$

Generally, the problem will require a numerical solution because the friction factor and the bottom slope changes with distance inland, although

sometimes one can use large increments of bottom of constant depth or constant slope if the roughness does not change.

4. NUMERICAL EQUATIONS

For solution by numerical methods, Equation 4 can be written in discrete form as

$$9)\quad h_2 = h_1 - \left[\tan\theta + \frac{n^2 g\, F^2 h_1^{-1/3}}{(1.486)^2}\right]\left[\frac{F^2}{2} + 1\right]^{-1} \Delta x$$

where $\frac{dh}{dx}$ was approximated by $\frac{h_2 - h_1}{\Delta x}$ and h by h_1.

This form as proposed by Bretschneider and Wybro is suitable for determining the depth, h_2, at an incremental distance Δx inland.

The numerical solution of Equation 4 was given in an inverse form by M&E Pacific, Inc. by approximating the depth of the incremental segment, h, with the average depth, h_A, instead of h_1, and solving for Δx is given as follows:

$$10)\quad \Delta x = \frac{\Delta h \left[\frac{F^2}{2} + 1\right]}{\tan\theta + \frac{n^2 g\, F^2\, h_A^{-1/3}}{(1.486)^2}}$$

where

Δx = an increment of the horizontal distance in feet

Δh = incremental difference of the tsunami wave depth in feet

h_A = average tsunami wave depth in the incremental distance x, in feet

$$\left(h_A = h_1 - \frac{\Delta h}{2}\right) = \frac{1}{2}(h_1 + h_2)$$

It has been found that Equation 10 is more suitable than the equation for numerical calculation of the steady state tsunami flood wave problem.

5. NON-BORE TSUNAMI FLOOD WAVE OVER A PREVIOUSLY DRY BED

The limiting non-bore is given for the Froude number $F \cong 1$, and using $g = 32.16$ ft/sec^2, Equation 10 becomes:

$$11) \quad \Delta x = \frac{1.5\ \Delta h}{\tan\theta + 14.564 n^2 h_A^{-1/3}}$$

Example calculations, using Equation 11 are shown in Figure 2.

For accuracy in calculations it is desirable to keep $\Delta h \overline{\overline{<}} 1.0$ ft when $h_A \overline{\overline{>}} 2.0$ ft and use $\Delta h = 0.5$ ft when $h_A \overline{\overline{<}} 2.0$ ft and even sometimes use $\Delta h = 0.1$ to 0.2 ft when $h_A \overline{\overline{<}} 0.5$ ft. These conditions can usually be met when $\tan\theta + 14.5654 n^2 h_A^{-1/3}$ is such that $\Delta x \overline{\overline{<}} 50$ to 75 feet for the upper limit, generally associated with low $\tan\theta$ and large Manning "n"; and $\Delta x \overline{\overline{>}}$ 5 to 10 feet for the lower limit, generally associated with large $\tan\theta$ and large Manning "n". Thus the controlling part of Equation 11 is as follows:

$$12) \quad \tan\theta + 14.654 n^2 h_A^{-1/3}$$

For a flat bottom $\tan\theta = 0$ and the tsunami flood wave profile is governed by the bed roughness in terms of Manning's "n" and the water depth of the tsunami as given by h_A. The rougher the terrain, the greater the loss of energy and the shorter the extent of flooding. For a very smooth bottom slope, say n = 0, the tsunami flood wave will continue inland unchanged over a flat bed until it arrives at a barrier slope $\tan\theta > 0$ or a vertical wall of elevation equal to or greater in height than $\sqrt{3}\ h_A$, and will return

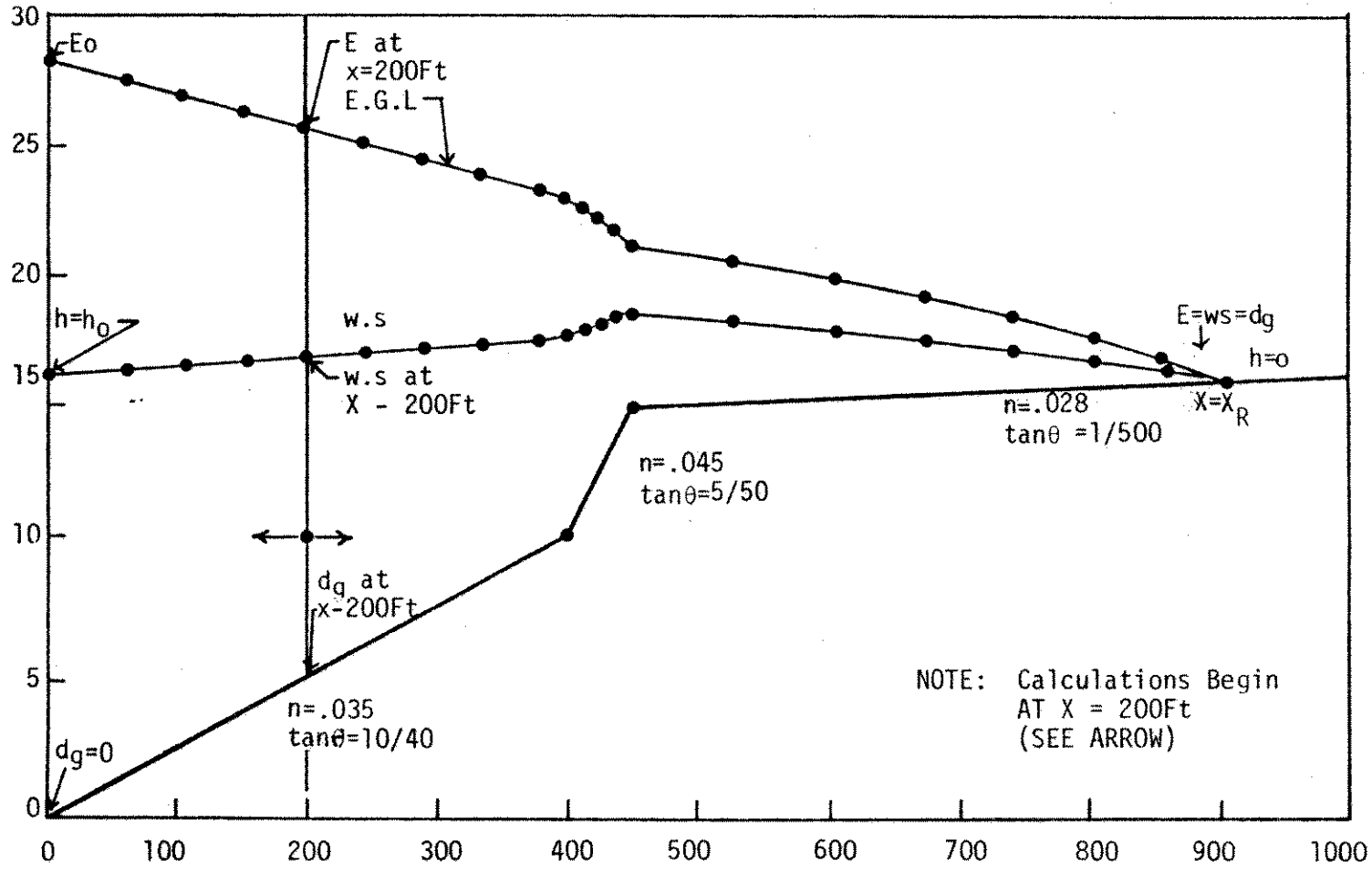


FIGURE 2. TYPICAL EXAMPLE ENERGY GRADE LINE, WATER SURFACE AND GROUND ELEVATIONS

shoreward as a reflected or partially reflected wave. If the tsunami is of sufficient duration, the water surface elevation will reach steady state coinciding with the energy grade line given by $\sqrt{3}\ h_A$ above the ground elevation. At the coastline the tsunami EGL is $\sqrt{3}\ h_o$, where h_o is the water depth above MSL at the coastline.

The term tan θ is very important in Equation 11 or 12 because it can be on the order of or greater than $14.564n^2h_A^{-1/3}$ for typically steep Hawaii coastal areas. Typical values of $14.564n^2h_A^{-1/3}$ are given in Table 1 for various values of Manning's "n" and h_A, and can be compared with values tan θ = 0, 1/500, 1/100, 1/50, 1/10, 1/5, 1/3 for example, covering a wide range of Hawaii coastline subject to tsunami flood wave and inundation. It is questionable whether Equations 11 or 12 will give reliable results for tan θ = 1/5 and 1/3 or larger.

The case of composite profiles, with varying land conditions, can be computed using numerical procedures by introducing the changes at the appropriate locations. Since each iteration in the integration process is independent, changes to n, tan θ, and Δh or Δx can be made in any segment. This is illustrated in Figure 2 for three slopes having different Manning's "n".

There will be instances, in the solution of Equation 13, when the Δx computed will exceed the horizontal distance to a point of significance, such as a change in ground slope of Manning's friction factor. For these instances, a solution by trial and error must be made; i.e., an estimation of Δh or h_A is made and Δx is computed and compared against the distance to the point of significance. Fortunately, the trial and error procedure is quite simple and a solution is usually made in the first estimation. The first estimation is made from the analysis of the original relationship

TABLE 1

Ranges in Typical Values of $14.56n^2h_A^{-1/3}$

Typical Values of $14.56n^2h_A^{-1/3}$

h_A	Values of Mannings "n"						
	.025	.030	.035	.040	.045	.050	.055
0	0	0	0	0	0	0	0
2	.0072	.0104	.0142	.0185	.0234	.0289	.0350
4	.0057	.0083	.0112	.0147	.0186	.0229	.0277
6	.0050	.0072	.0098	.0128	.0162	.0200	.0242
8	.0046	.0066	.0089	.0116	.0147	.0182	.0220
10	.0042	.0061	.0083	.0108	.0137	.0169	.0204
12	.0040	.0057	.0078	.0102	.0129	.0159	.0192
14	.0038	.0054	.0074	.0097	.0122	.0151	.0183

NOTE: Typical values of $\tan\theta$ = 0,1/500,1/100,1/50,1/10,1/5,1/3, etc.

between Δx and Δh, and usually only a simple interpolation or extrapolation is all that is required; the last partial increment of Δx can also be extended graphically to an intersection point of the MSL energy grade line, water surface elevation (MSL) and the ground elevation (MSL), all of which must coincide at the distance X_R, the extent of horizontal flooding. This can be done when the last value of h_A is less than Δh.

6. TSUNAMI FLOODING ELEVATIONS, ENERGY GRADE LINE AND GROUND ELEVATIONS FOR WEST BEACH DEVELOPMENT AREA

The location map for the study area is shown in Fig. 1. The procedures outlined in the previous sections were used to calculate the flooding elevations and energy grade line from 200 feet inland from the coastline to the coastline and landward to the point where the water surface (W.S.) and the energy grade line (E.G.L.) intersects with the ground elevation (d_g) MSL.

Plates 3, 22, 23, 25 and 26 of (Ref. 1) were used for West Beach Area and later for Honolulu and Waikiki areas, and are included in the Appendix of Section I-A of their report.

Plates 23 and 26, respectively were used to determine the A and B coefficients used in Eq. 11, for numerical integration. Values of Manning's "n" were estimated from Table 2 and also from recent experiments over typical Hawaiian terrain (Ref. 4). The area of interest falls within the ranges of Stations 92 and 95 shown in Plate 3 of the Appendix to Section I-A. The results of the Coefficients A and B and the predicted Tsunami elevations for the Stations 91.5 to 94.5 are given in Table 3 for the 100-year and 140-year recurrence intervals.

TABLE 2

Suggested Values of Manning's n for
Various Coastal Terrain Conditions

n	Conditions
0.015 - 0.025	Very smooth (mud flats, ice, well maintained concrete paved ways, beaches of fine sand)
0.025 - 0.030	Smooth (dried earth, coarse sand beaches, badly maintained concrete paved ways, very thin lawn grass up to 1 cm high)
0.030 - 0.035	Average for developed areas (lawn grass up to 5 cm high, gravel, presence of some buildings houses, and other obstructions)
0.035 - 0.045	Open coast, relatively smooth and open area (grass up to 10 cm, sparse population of trees*, sparse bush, even bottom)
0.045 - 0.055	Moderately rough open coastal areas (thick grass, uneven bottom consisting of large rocks, coral, etc., presence of trees with low foliage, brush, lava rock, etc.)
0.055 - 0.070	Unusually rough costal areas (dense brush, dense tree population with exposed roots, coarse lava rock formations)

* Trees with high foliage such that only trunks are exposed to flow

TABLE 3

A&B COEFFICIENTS, 100- AND 140-YEAR PREDICTED TSUNAMI

Elevations 200 feet inland from the Coastline*

(WEST BEACH AREA)

Oahu Station No.	91.5	92.0	92.5	93.0	93.5	94.0	94.5
Coefficient A	5.5	6.8	6.0	5.5	5.1	5.0	6.0
Coefficient B	3.5	5.0	4.0	2.9	2.0	1.9	2.8
100-Year H ft MSL	7.5	8.6	8.0	8.1	8.2	8.1	9.2
140-year H ft MSL	8.3	9.6	8.9	8.9	8.9	8.8	10.1

* The above elevations have a variation of ± 22 percent.

A plot of the tsunami elevations 200 feet inland from the coastline given in Table 3 is shown in Fig. 3. The traverses shown in Fig. 3 are essentially the same for West Beach Area from the coastline to the intersection of W.S., E.G.L. and ground elevation d_g (MSL). Other transverse elevations beyond d_g vary in height.

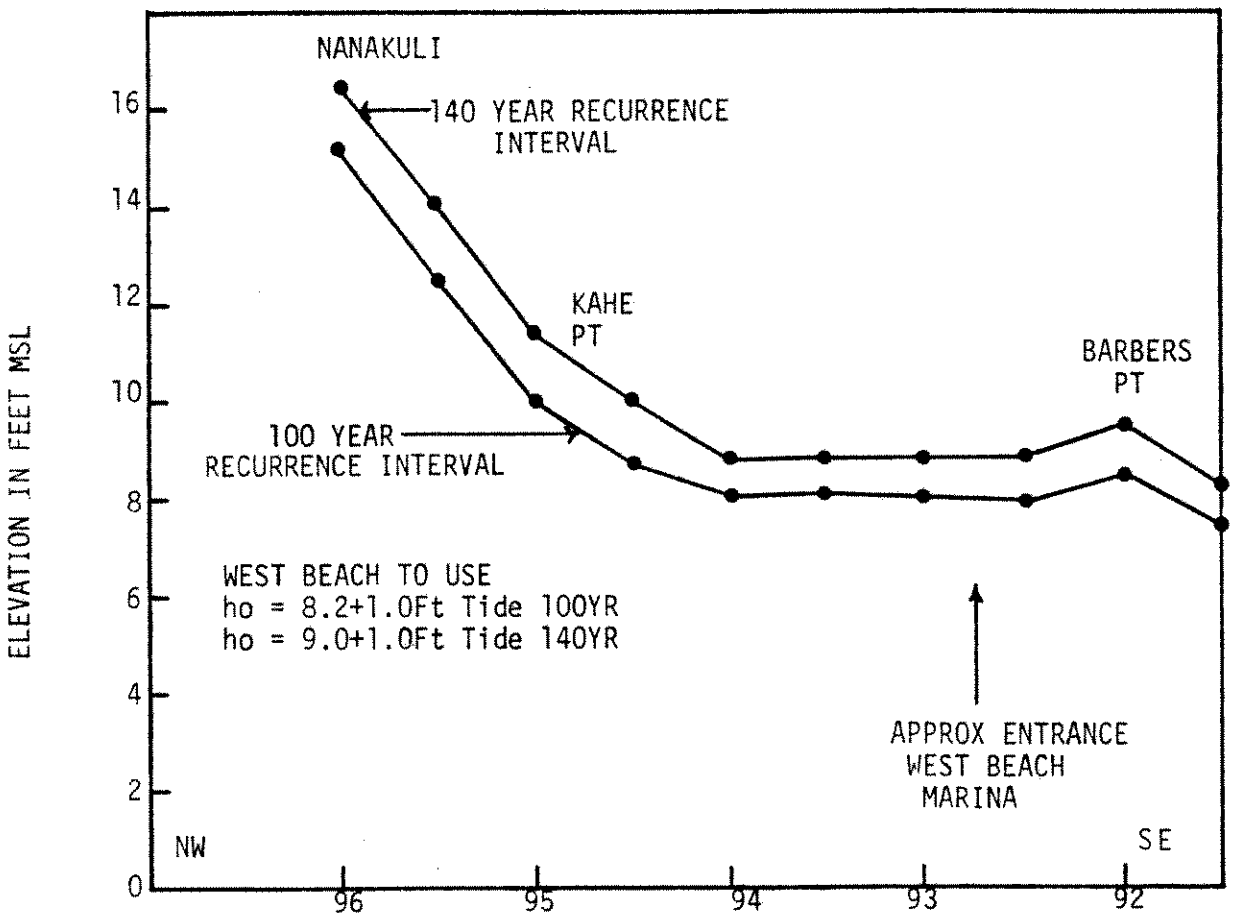
The elevation of the land was determined from topographical charts. The water surface depth hs is equal to the difference between tsunami elevation and ground elevation, whence

$$13) \quad hs = H - d_g \text{ MSL}$$

The energy grade line elevation above mean sea level (MSL) for the non-bore tsunami is obtained from Eq. 3 previously given.

$$3) \quad \text{E.G.L.} = d_g + \sqrt{3}\, hs \text{ (MSL)}$$

The results of the calculations are shown in Figs. 4 and 5 respectively for the 100-year and 140-year tsunami recurrence intervals respectively.



STATION LOCATION

FIG 3 TSUNAMI ELEVATIONS 200FT INLAND FROM THE COASTLINE (WEST BEACH AREA)

STATION LOCATION FROM PLATE 3
U.S. ARMY CORPS OF ENGINEERS (SEE. APPENDIX)
(REF. 1)

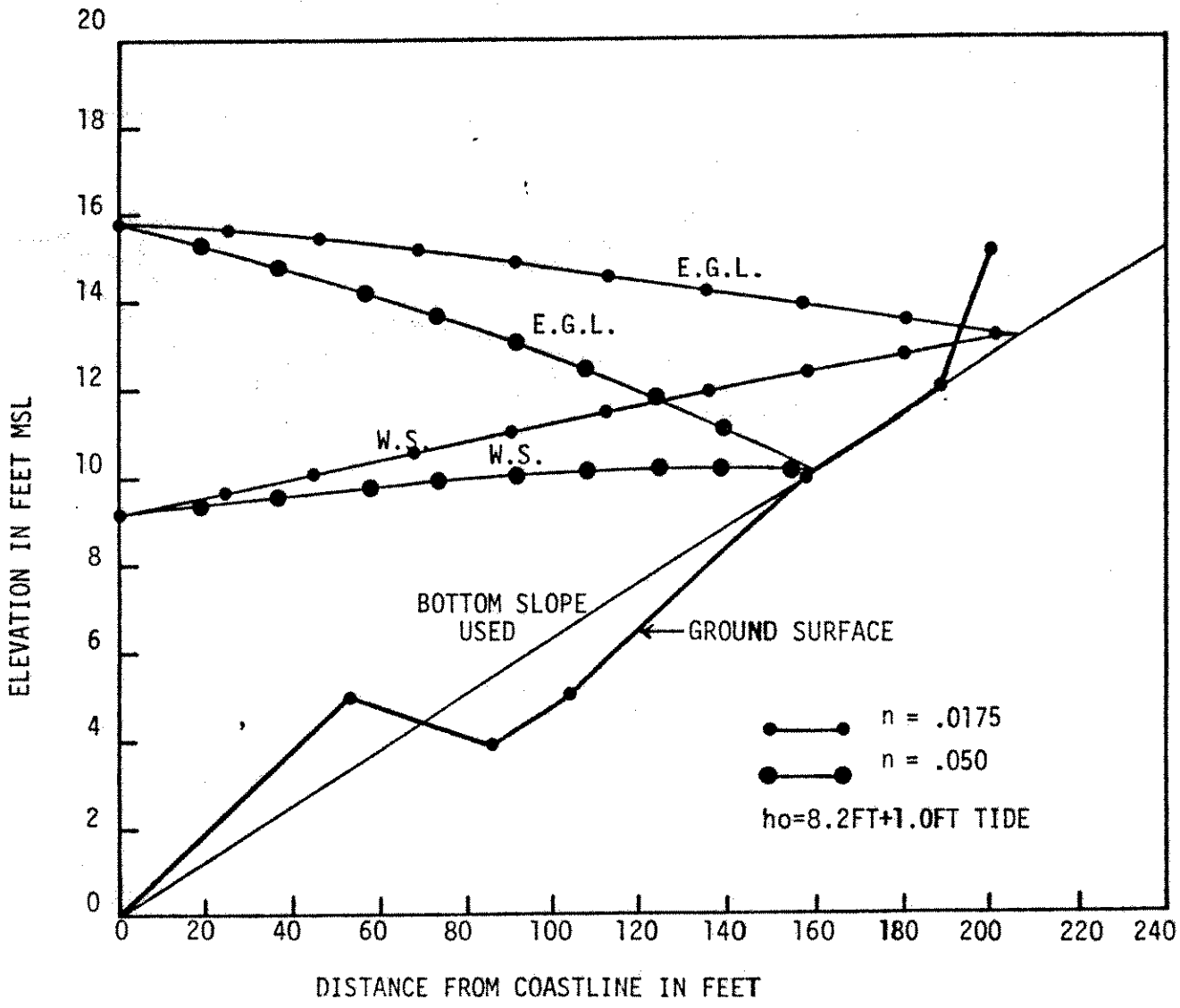


FIG.4 TSUNAMI WATER SURAFACE (W.S.) AND ENERGY GRADE LINE (E.G.L.)
FOR WEST BEACH AREA
(ho= 9.2 FT) FOR 100-YEAR RECURRENCE INTERVAL

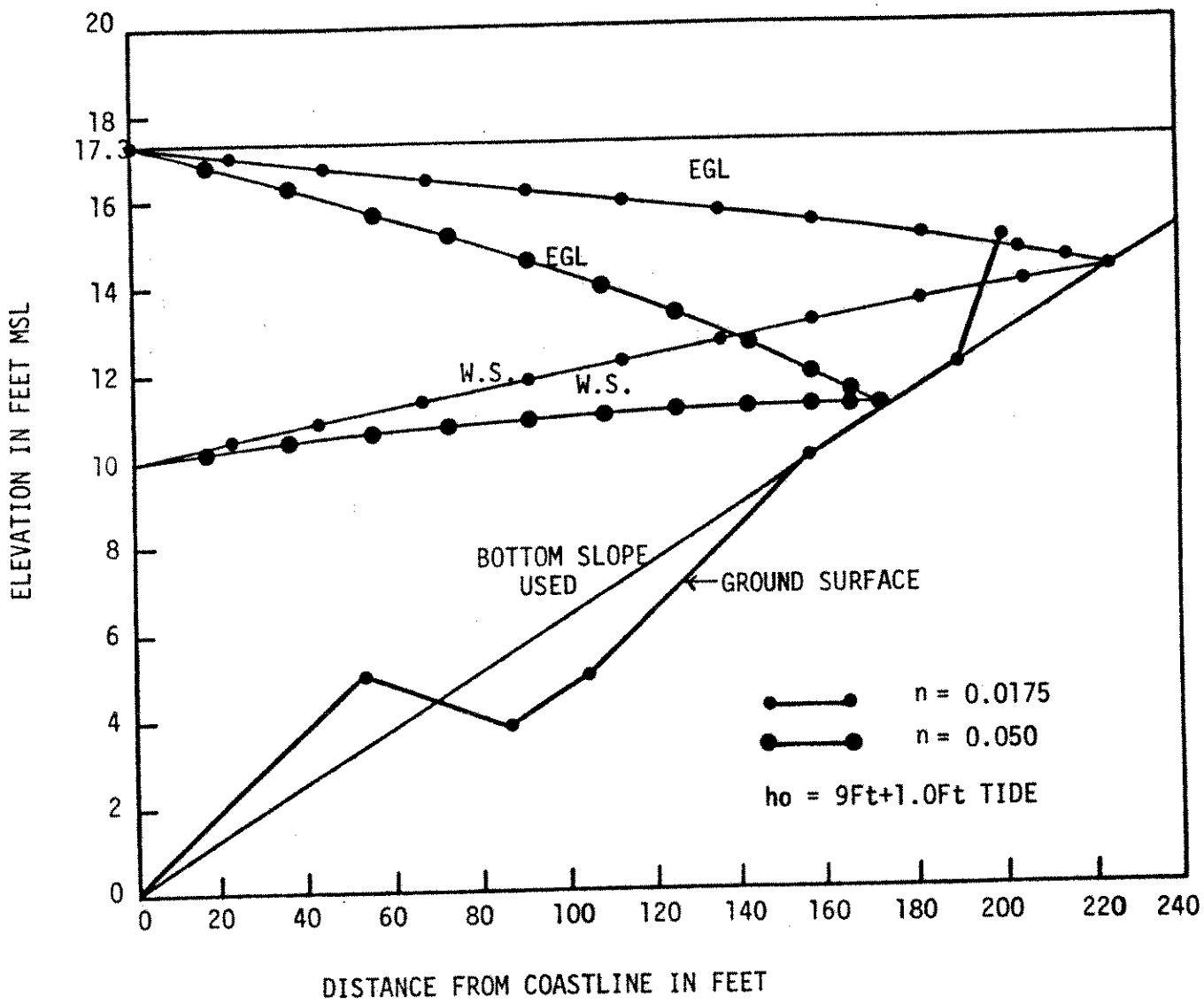


FIG. 5 TSUNAMI WATER SURFACE (W.S.) AND ENERGY GRADE LINE (E.G.L.) FOR WEST BEACH AREA (ho= 10FT) FOR 140-YEAR RECURRENCE INTERVAL

When the roughness is changed from what was used in Figs. 4 and 5, then the water elevations and energy grade line change will also change. Increasing the roughness will decrease the tsunami flooding. Decreasing the roughness will increase the flooding. The energy grade line at the coastline, E.G.L. = 15.9 and 17.3 feet MSL respectively for the 100-year and 140-year recurrence intervals, as shown in Figs. 4 and 5.

7. A COMPARISON OF PREDICTED TSUNAMI ELEVATIONS AT WEST BEACH WITH WAIKIKI AREA

Calculations have been made for tsunami elevations for three locations: San Souci, Kuhio, and Ala Moana beaches. The results of the calculations are given in Table 4.

TABLE 4

A AND B COEFFICIENTS, 100-YEAR AND 140-YEAR PREDICTED TSUNAMI ELEVATIONS 200 FEET INLAND FROM THE COAST (WAIKIKI AREA)*

Station No.	63	64	65	93
Location	San Souci Beach	Kuhio Beach	Ala Moana Beach	West Beach
A-Coeff.	7.1	3.9	2.4	5.5
B-Coeff.	4.5	1.1	0.9	2.9
Tsunami Elevations (ft)				
100-Year	9.7	6.7	3.9	8.1
140-Year	10.7	7.3	4.3	8.9

*The above tsunami elevations have a variation of ± 22 percent.

For the 100-year recurrence interval the maximum difference in run-up at West Beach is about 1.4 ft higher than at Kuhio Beach and about 1.6 ft lower than at San Souci Beach.

8. CONFIDENCE CONSIDERATIONS ON TSUNAMI INUNDATIONS

The tsunami height depends on the tide and the present or future planned roughness. Therefore, it is conservatively estimated that the 100-year recurrence interval tsunami elevations will be 10.2 feet and 13.1 feet, averages respectively, for roughness elements corresponding to Manning's n = 0.05 (very rugged) and Manning's n = 0.0175 (smooth, fine sand or mud flats). The corresponding values for the 140-year recurrence interval tsunami elevations are conservatively estimated at 11.0 and 14.1 feet for n = 0.05 and n = 0.0175 respectively. The energy grade lines at the coast-line are E.G.L. = 15.9 feet and 17.3 feet respectively for the 100-year and 140-year recurrence intervals.

In view of the fact that the original tsunami run-up data are averages and vary from location to location in the same general area, it seems appropriate to assign some type of confidence limits to the results. For example, in wave spectra statistics the 90 percent confidence limit is generally used, and for 20 to 40 degrees of freedom, the upper and lower 90 percent fiducial confidence limits are about 1.5 and 0.6 respectively. Since the spectral energy is proportional to the square of the wave height, it can be inferred that the corresponding fiducial confidence limits for wave height would be the square root of 1.5 and 0.6, or 1.22 and 0.77 respectively. Thus it can be assumed that the tsunami heights can also vary by approximately ± 22 percent of 8.2 and 9.0 feet, or about ± 2.0 feet from location to location within the West Beach Development Area.

Thus for the 100-year and 140-year recurrence interval the 90 percent fiducial confidence limits for maximum tsunami run-up for very rugged terrain (n = .05) would be at 12.2 and 15.1 feet MSL respectively. The corresponding values for Manning's n = .0175 would be 13.0 feet and 16.1 feet respectively.

In view of the above conservative assumption it is recommended that the upper limits for tsunami inundation at West Beach Development Area be established at 13 feet for the 100-year recurrence interval and 15 feet for the 140-year recurrence interval.

9. DESIGN TSUNAMI HYDROGRAPH FOR WEST BEACH PROPOSED MARINA

In order to determine the time history and discharge into and out of the West Beach Channel entrance to the Marina, and the water level rise and fall within the Marina, it is necessary to predict the design tsunami hydrograph. This information is necessary for a later study, where it is also necessary to have the geometry of the Channel and the Marina in order to perform the hydraulic calculations.

The procedure used for obtaining the 100-year design hydrograph and an alternate design hydrograph is discussed subsequently.

First, a literature search was conducted to gather existing hydrographs for the important tsunamis that occurred in the past. Refs. 2 and 5 give the important data on tsunami hydrographs. The 1960 tsunami resulted in the highest run-up values for historical tsunami for the area of interest. The Honolulu tide gage is the closest to the West Beach Development area.

Fig. 6 reproduced from Ref. 2 gives a comparison between the U.S. Army Corps of Engineers Waterways Experiment Station (W.E.S.) numerical model

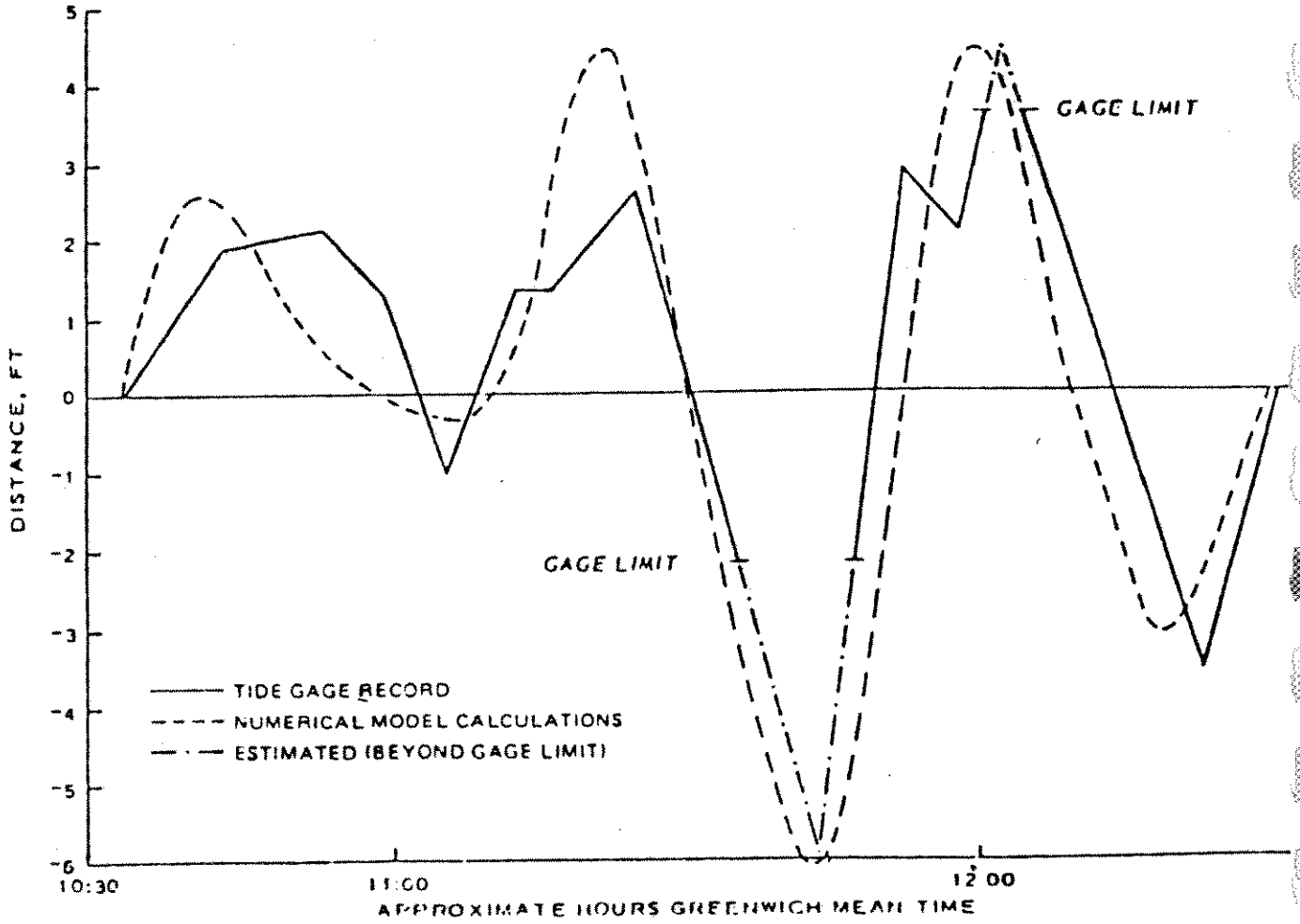


FIGURE 6. COMPARISON AT HONOLULU, HAWAII FOR 1960 TSUNAMI
(copy of Fig. 5 of "Tsunamis" NSF (1979) ref. 5)

prediction and the staff gage at Honolulu. The tsunami record is not a continuous trace and has missing data beyond the gage limit. Elevations have been read off Fig. 6 and are summarized in Table 5.

The Honolulu Record of the 1960 tsunami can be used to determine a model design hydrograph for the entrance to the West Beach Marina. However, it must first be noted that the maximum height was not recorded and was perhaps 4 to 4.5 feet off the scale of the gage. The numerical model (Refs. 2 and 5) estimate is that the 1960 tsunami was about 4.3 or 4.5 feet off scale. The primary period of the tsunami was between 39 and 41 minutes.

It is important to determine the predicted tsunami elevations at Honolulu for the 100-year (also the 140-year if needed) recurrence interval for comparison with those values already determined for West Beach Development Area. The A and B coefficients must be determined for Honolulu as was done for the West Beach Development Area. A summary of these calculations are given in Table 6.

The information given in Tables 5 and 6 can be used to convert the 1960 Honolulu hydrograph to a 100-year design hydrograph for the entrance to the West Beach Marina.

The 100-year Honolulu predicted tsunami is 5.1 feet given in Table 6 as compared with the 1960 tsunami (extrapolated) of about 5.2 feet. Thus the scale factor is $\lambda \cong 1.0$. However there is an amplification factor $R_1 = 1.62$. Froude scale law applies to time $\tau = \sqrt{\lambda} = 1.0$ for the 100-year tsunami. The amplification factor $R_1 = 1.62$ was used to convert the water surface elevation from Honolulu to West Beach Marina, but the time scale remains unchanged for the 100-year tsunami.

TABLE 5 THE 1960 TSUNAMI ELEVATIONS FOR HONOLULU GAGE (OBTAINED FROM FIG 6)

TIME	TIDE GAGE	EST. BEYOND GAGE	TIME	TIDE GAGE	NUMERICAL MODEL TIME	CALCI
					10:30	0
10:30	0		10:30	0	10:35	1.8
10:35	0.6		10:42	1.95	:40	2.5
:40	1.5		10:53	2.05	:45	2.1
:45	1.95		10:59	1.25	:50	1.0
:50	2.05		11:05	-1.00	:55	0.3
:55	1.90		11:12	1.35	11:00	-0.1
11:00	1.00		11:16	1.35	:05	-0.3
:05	-1.00		11:25	-2.50	:10	0
:10	0.50		11:36	-2.15	:15	1.9
:15	1.33		11:37	MSG	:20	4.50
:20	1.95		11:47	MSG	:25	3.35
:25	2.50		11:47.5	-2.15	:30	0
:30	0.15		11:52.5	2.90	:35	-3.0
:35	-2.0		11:58	2.05	:40	-5.4
:40		-4.2	12:01	3.55	:45	-5.9
:45	0	-4.5	12:01.5	MSG	:50	-2.9
:50	2		12:04	MSG	:55	2.0
:55	2.5		12:04.5	3.55	12:00	4.3
12:00	3.3		12:22.5	-2.50	:05	2.3
:05	3.6		12:30	0	:10	0.3
:10	1.3				:15	-2.35
:15	-0.7				:20	-3.0
:20	-2.55				:25	-1.5
:25	-2.45				:30	0
:30	0					

TABLE 6

WEST BEACH TSUNAMI
CORRELATION WITH 1960 HONOLULU TSUNAMI

Station No.	Honolulu Harbor Entrance			
	66	67	68	69
A-Coeff.	2.4	2.5	2.8	2.1
B-Coeff.	1.0	0.9	0.5	0.2
100 YEAR H(ft)	3.8	4.1	5.1*	4.0
140 YEAR H(ft)	4.2	4.5	5.5*	4.3

1960 Tsunami Honolulu Range (4.5 to - 5.9) = 10.4 FT

1/2 (RANGE) = 5.2* FT = HEIGHT OF WATER

NOTE: 1960 Tsunami H = 5.2 FT is essentially the same as 100 YR Tsunami = 5.1 FT (Use H - 5.15 FT)

Station No.	WEST BEACH MARINA ENTRANCE			
	92.5	93	93.5	94
A-Coeff.	6.0	5.5	5.1	5.0
B-Coeff.	4.0	2.9	2.0	1.9
100 YEAR H(ft)	8.0	8.1	8.2	8.1
140 YEAR H(ft)	8.9	8.9	8.9	8.8

AMPLIFICATION FACTOR FOR WEST BEACH MARINA

100 YR R_1 = 8.1 ÷ 5.15 = 1.57
140 YR R_2 = 8.9 ÷ 5.15 = 1.73

SCALE FACTOR

100 YR $\lambda_1 = R_1/R_1 = 1.0$ = $\sqrt{\lambda} = 1.0$
140 YR $\lambda_2 = R_2/R_1 = 1.1$ = $\sqrt{\lambda} = 1.05$

The 140-year tsunami can be scaled as follows:

$$\lambda = R_1/R_2 = 1.78/1.62 = 1.0988$$

$$\tau = \sqrt{\lambda} = 1.05$$

The Froude scaled 100-year tsunami hydrograph is given in Table 7 together with the 140-year hydrograph. Figure 7 shows the 100-year tsunami hydrograph only. The 140-year hydrograph given in Table 7 would be similar to the 100-year tsunami given in Fig. 7.

Depending on the scope of effort required, it would be possible to reconstruct the entire 1960 Honolulu Tsunami Record (See Fig. 6 of Appendix to Section I), and then repeat similar calculations just previously given: in this manner all frequencies would be included.

9.1 Alternate Design Tsunami Hydrograph. It is noted in the 1960 Honolulu Tsunami Hydrograph (Fig. 6) that there is a primary period between 39 and 41 minutes, approximately. Using T = 40 minutes for the primary period and an amplitude equal to 1/2 the total range between maximum and minimum ranges one can obtain from Table 6 A = 1/2 (4.5 + 5.9) = 5.2 feet and T = 2 x 20 = 40 minutes for the Honolulu Hydrograph. From Table 6 H = 5.1 feet for the 100-year tsunami, and the sinusoidal 100-year tsunami will be given by the following equation:

$$\eta = 1.62\ (5)\ \sin \frac{2\pi t}{T}$$

or

$$\eta = 8.1 \sin \left(\frac{360^\circ}{40} t \right) \qquad \text{(100-year)}$$

The 140-year tsunami hydrograph would scale A = 8.1 λ = 8.1 (1.0958) = 8.9 feet. And T = 40 $\sqrt{\lambda}$ = 42 minutes.

Thus

$$\eta = 8.9 \sin \left(\frac{360^\circ}{42} t \right) \qquad \text{(140-year)}$$

TABLE 7 WEST BEACH TSUNAMI HYDROGRAPH BASED ON 1960 HONOLULU MEASURED TSUNAMI

Honolulu 1960 Tsunami			100 YR		140 YR	
Time HST	t MIN	EL FEET	t MIN	EL FEET	t MIN	EL FEET
10:30	0	0	0	0	0	0
:35	5	1.8	5	2.92	5.25	3.20
:40	10	2.5	10	4.05	10.50	4.45
:45	15	2.1	15	3.40	15.75	3.74
:50	20	1.0	20	1.62	21.00	1.78
:55	25	0.3	25	0.49	26.25	.53
11:00	30	-0.1	30	- .16	31.50	-.18
:05	35	-0.3	35	- .49	36.75	-.53
:10	40	0	40	0	42.0	0
:15	45	1.9	45	3.08	47.25	3.38
:20	50	4.5*	50	7.29	52.50	8.01
:25	55	3.35	55	5.43	57.75	6.00
:30	60	0	60	0	63.00	0
:35	65	-3.0	65	-4.86	68.25	-5.34
:40	70	-5.4	70	-8.75	73.50	-9.61
:45	75	-5.9*	75	-9.56	78.75	-10.50
:50	80	-2.9	80	-4.70	84.00	-5.16
:55	85	2.0	85	3.24	89.25	3.56
12:00	90	4.3	90	6.97	94.50	7.65
:05	95	2.3	95	3.73	99.75	4.09
:10	100	0.3	100	0.49	105.00	.53
:15	105	-2.35	105	-3.81	110.25	-4.18
:20	110	-3.0	110	-4.86	115.50	-5.34
:25	115	-1.5	115	-2.43	120.75	-2.64
12:30	120	0	120	0	126.00	0

*NOTE: Amplitude, $A = \frac{1}{2}(4.5 + 5.9) = 5.2$ feet

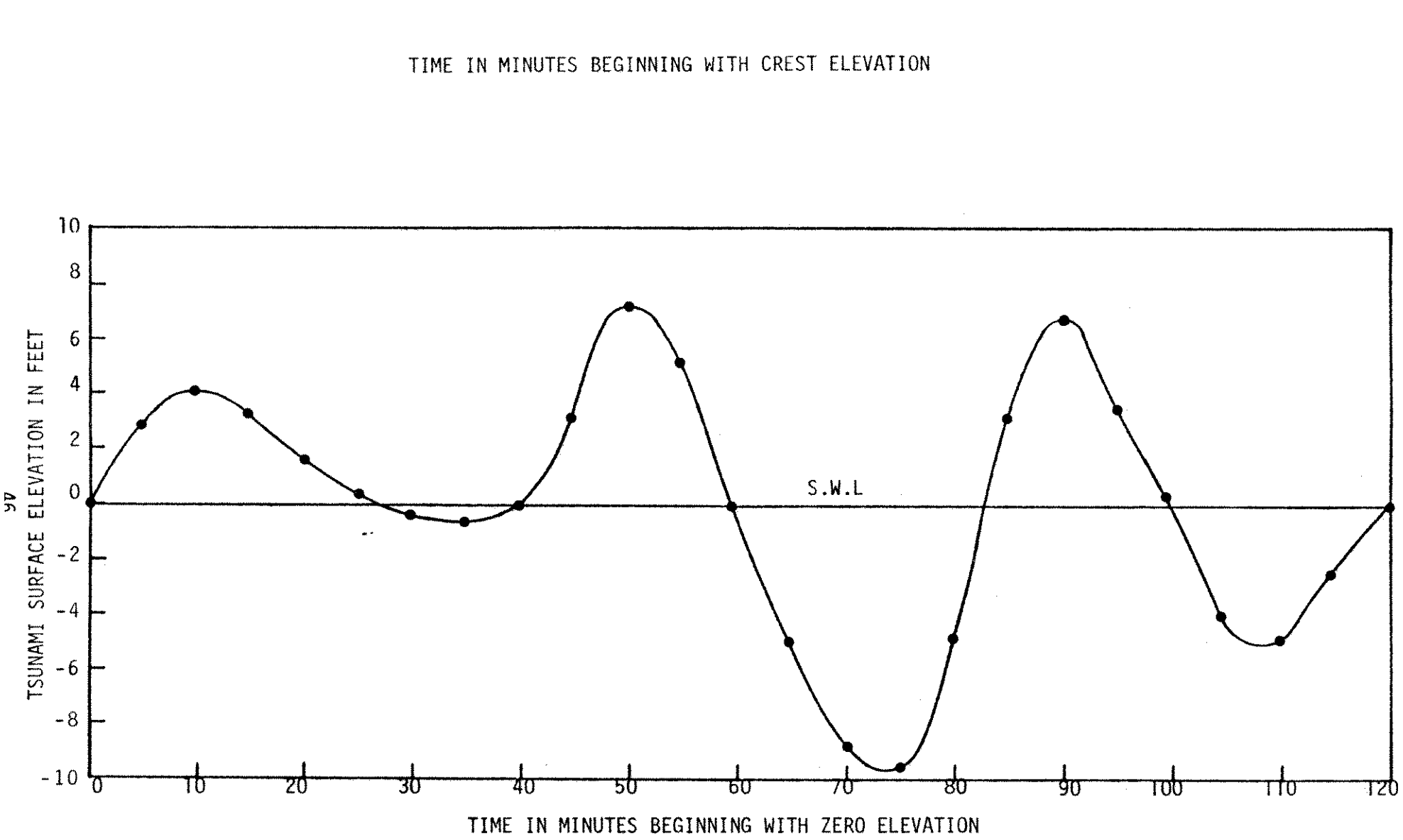


FIG. 7 DESIGN TSUNAMI SCALED FOR 100-YEAR RECURRENCE INTERVAL
WEST BEACH MARINA ENTRANCE
SCALED FROM 1960 HONOLULU RECORD

The results of the sinusoidal alternate tsunami hydrographs for the 100-year and 140-year recurrence intervals are given in Table 8. The 100-year alternate tsunami hydrograph is also shown in Figure 8. the 140-year alternate hydrograph from Table 8 would look similar to Figure 8.

The alternate hydrograph would be easier to use in the hydraulic laboratory, as well as for numerical calculations. It may be difficult to reproduce the hydrograph (Fig. 7) for the non-periodic tsunami in the hydraulic laboratory.

TABLE 8 WEST BEACH ALTERNATE TSUNAMI HYDROGRAPH BASED ON PRIMARY SINUSOIDAL TSUNAMI HONOLULU 1960

100 YR A = 8.1 ft		140 YR A = 8.9 ft	
t MIN	EL FEET	t MIN	EL FEET
0	0	0	0
2	2.50	2.1	2.75
4	4.76	4.2	5.23
6	6.55	6.3	7.20
8	7.70	8.4	8.46
10	8.10	10.5	8.90
12	7.70	12.6	8.46
14	6.55	14.7	7.20
16	4.76	16.8	5.23
18	2.50	18.9	2.75
20	0	21.0	0
22	-2.50	22.1	-2.75
24	-4.76	24.2	-5.23
26	-6.55	26.3	-7.20
28	-7.70	28.4	-8.46
30	-8.10	31.5	-8.90
32	-7.70	32.6	-8.46
34	-6.55	34.7	-7.20
36	-4.76	38.8	-5.23
38	-2.50	39.9	-2.75
40	0	42.0	0
REPEAT	REPEAT	REPEAT	REPEAT

$$\eta = A \sin \frac{2\pi\tau}{T} = A \sin \left(360° \frac{\tau}{T}\right)$$

	A(ft)	T(min.)
100 YR	8.1	40
140 YR	8.9	42

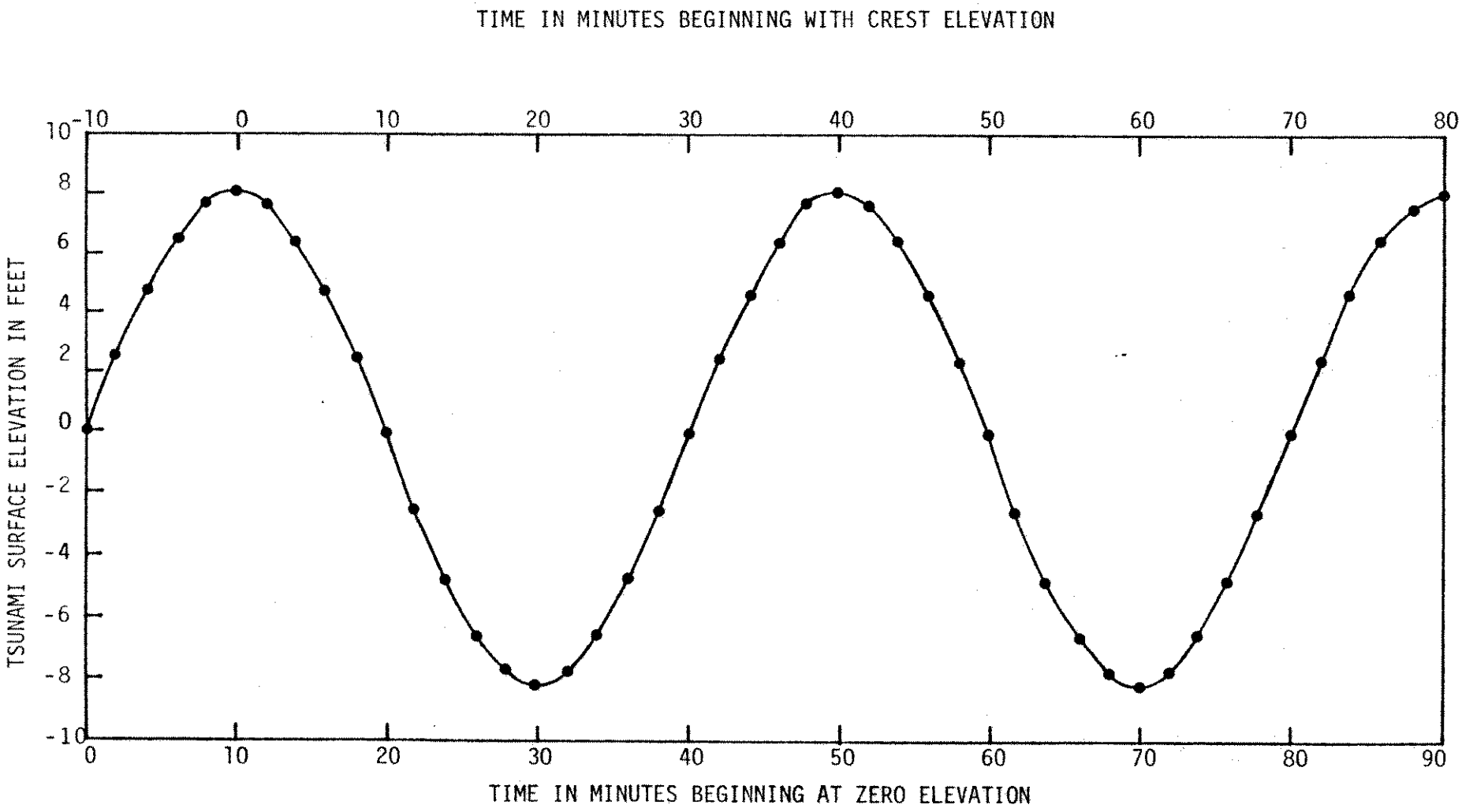


FIG. 8 ALTERNATE SINUSOIDAL DESIGN TSUNAMI 100-YEAR RECURRENCE INTERVAL
WEST BEACH MARINA ENTRANCE

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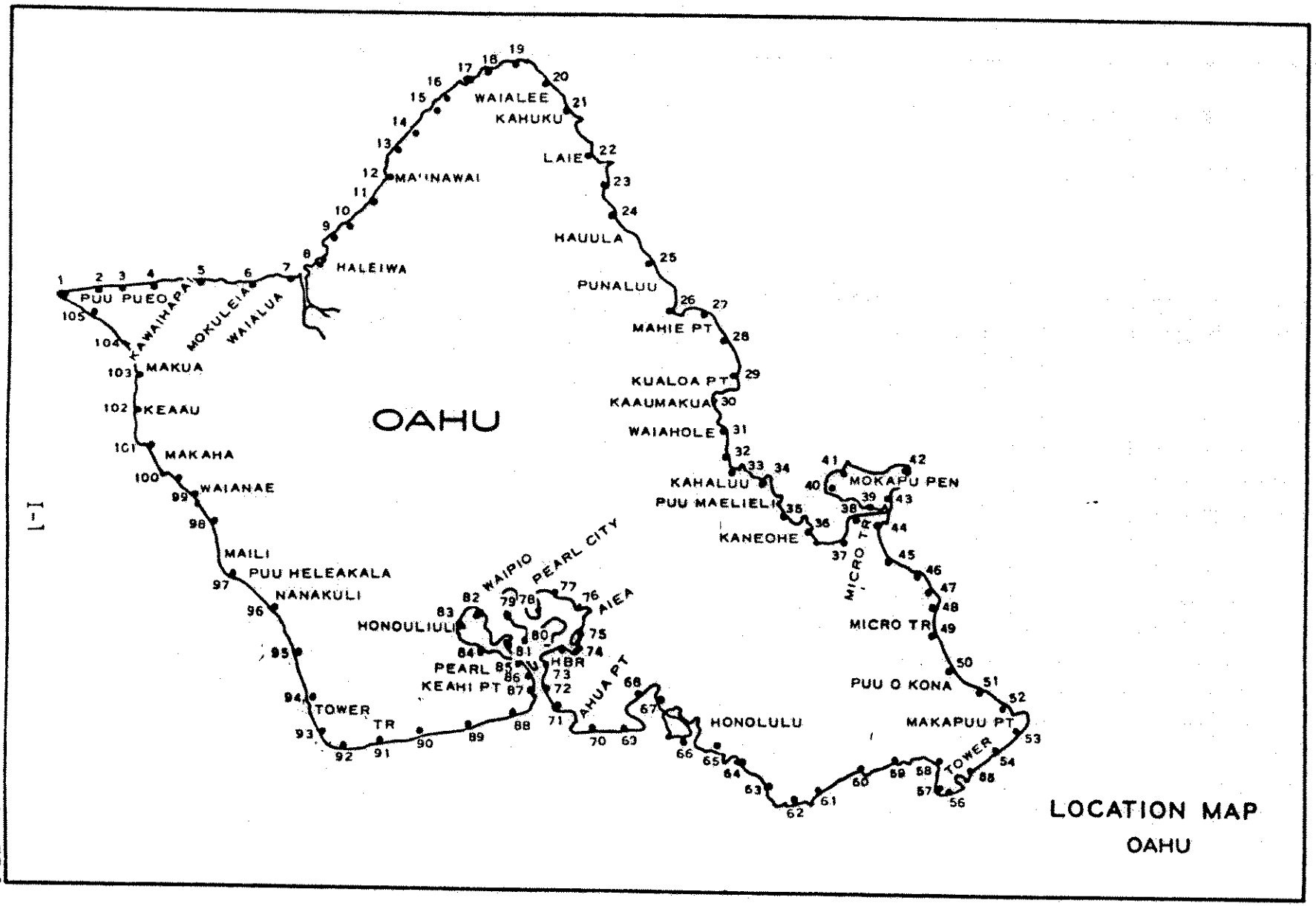
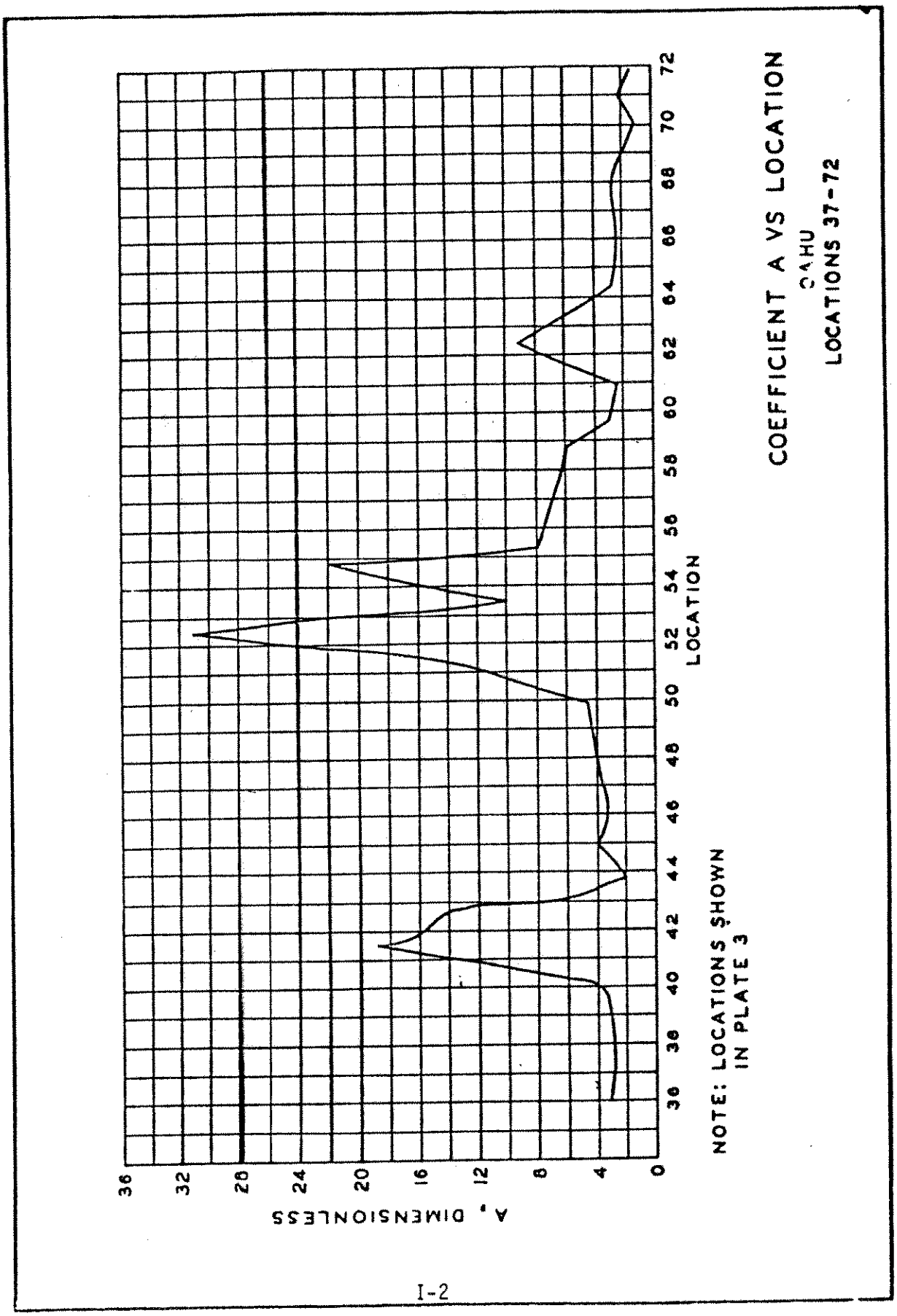

OAHU
PUU PUEO
KAWAIHAPAI
MOKULEIA
WAIALUA
HALEIWA
MA'INAWAI
WAIALEE
KAHUKU
LAIE
HAUULA
PUNALUU
MAHIE PT
KUALOA PT
KAAUMAKUA
WAIAHOLE
KAHALUU
PUU MAELIELI
KANEOHE
MOKAPU PEN
MICRO TR
MAKUA
KEAAU
MAKAHA
WAIANAE
MAILI
PUU HELEAKALA
NANAKULI
WAIPIO
PEARL CITY
AIEA
HONOULIULI
PEARL HBR
KEAHI PT
AHUA PT
HONOLULU
PUU O KONA
MAKAPUU PT
TOWER
TR
LOCATION MAP
OAHU

I-1

PLATE 3


COEFFICIENT A VS LOCATION
OAHU
LOCATIONS 37-72
A, DIMENSIONLESS
LOCATION
0
4
8
12
16
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24
28
32
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30
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42
44
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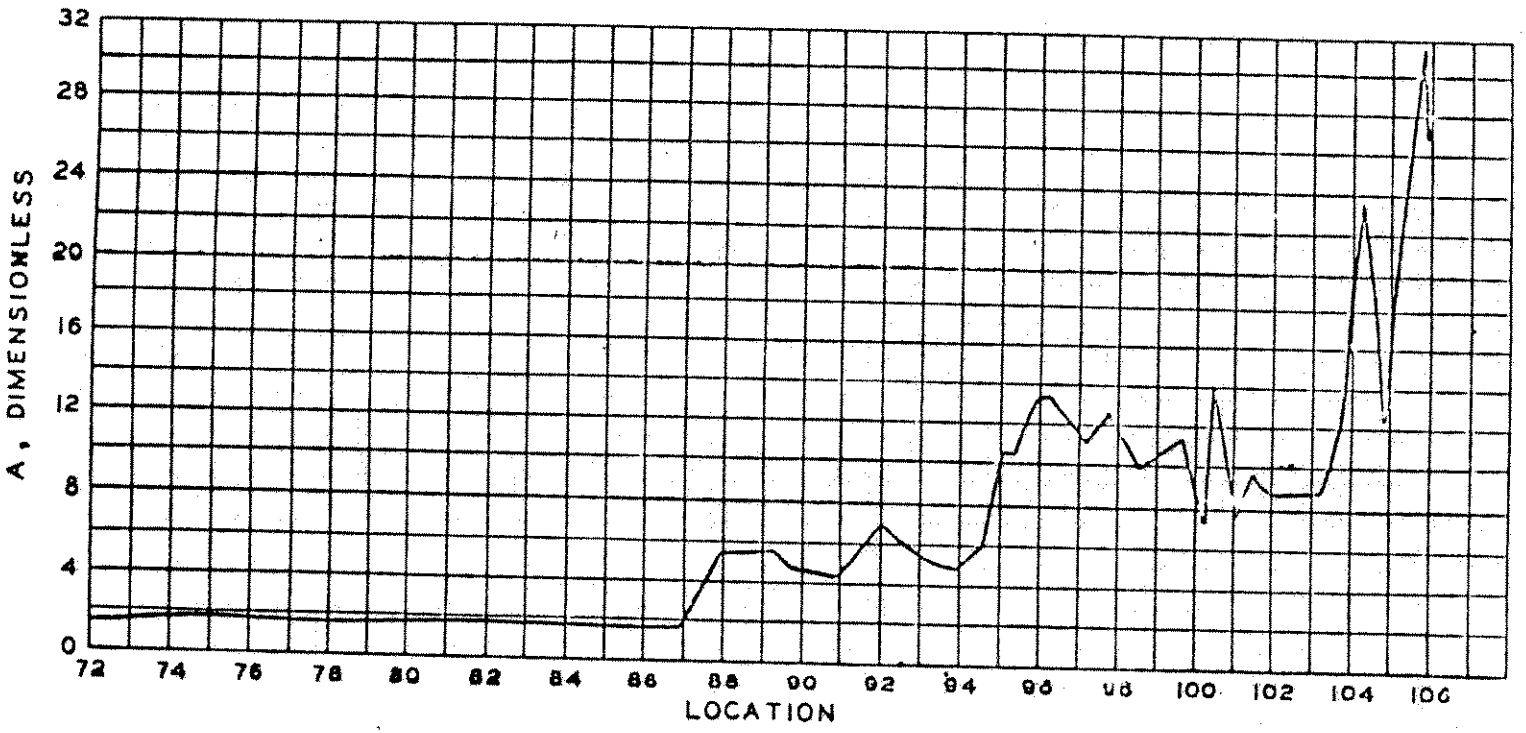
NOTE: LOCATIONS SHOWN
IN PLATE 3

PLATE 22



NOTE: LOCATIONS SHOWN
IN PLATE 3

COEFFICIENT A VS LOCATION
OAHU
LOCATIONS 73-105

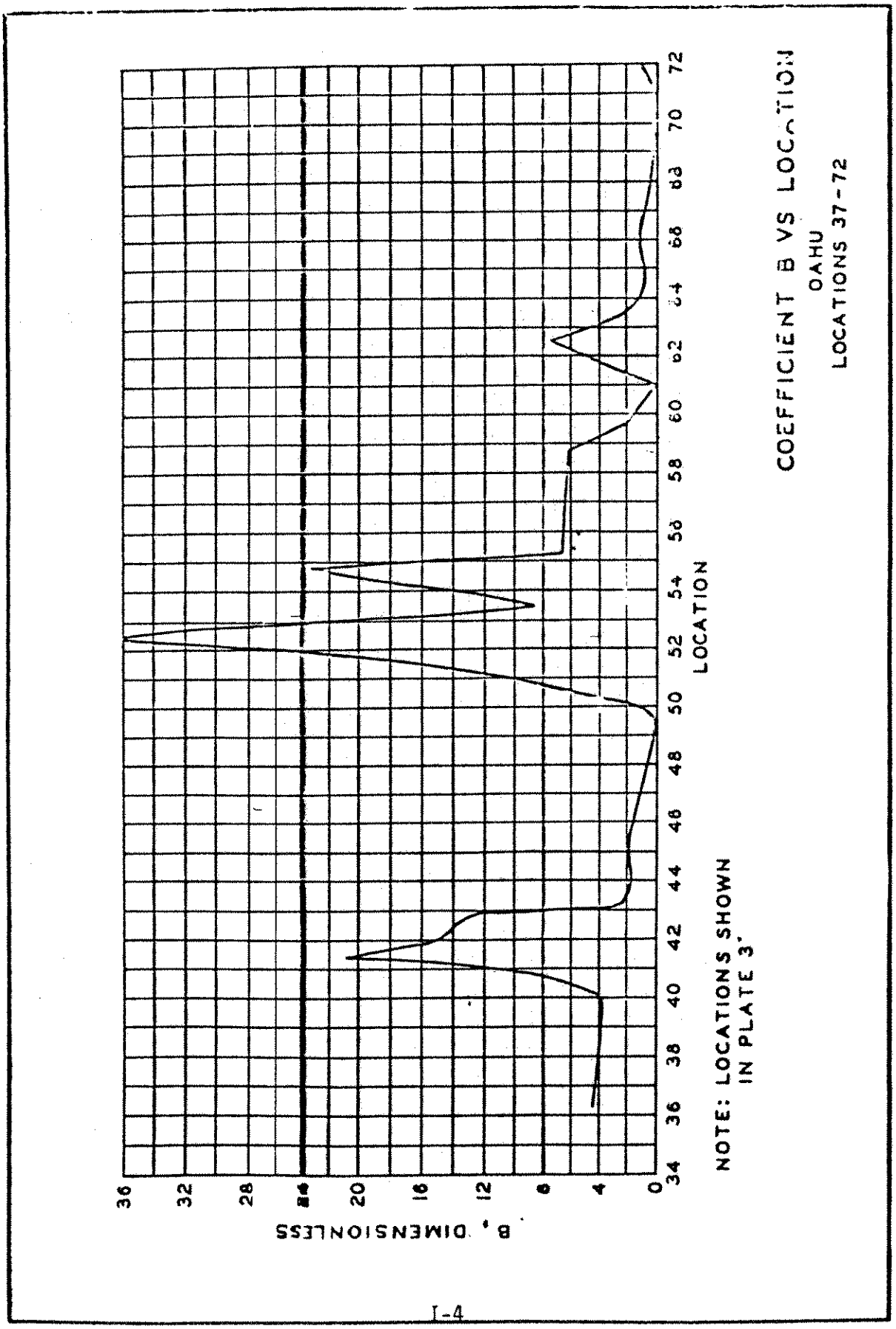
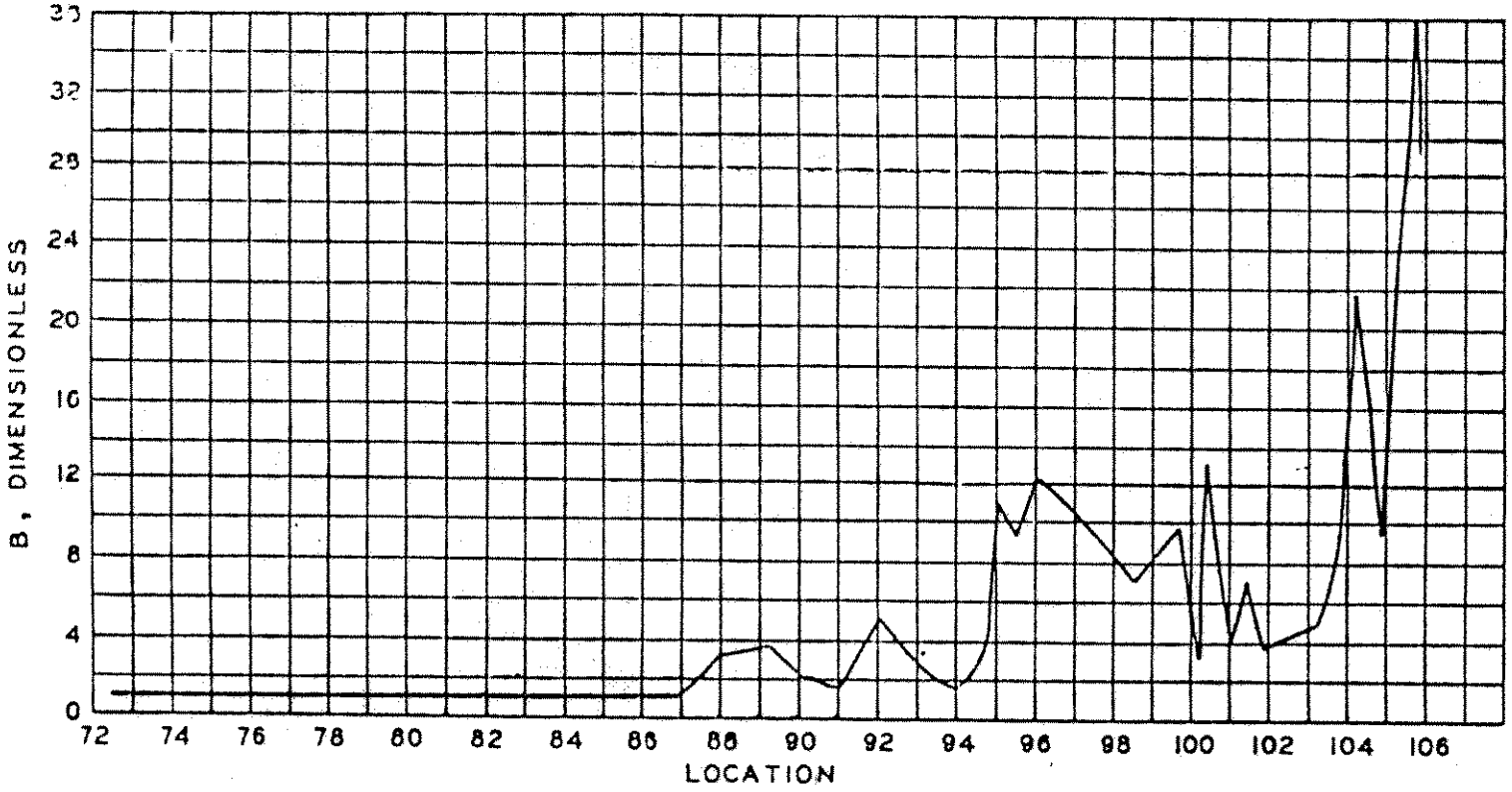

COEFFICIENT B VS LOCATION
OAHU
LOCATIONS 37-72
B, DIMENSIONLESS
LOCATION
NOTE: LOCATIONS SHOWN IN PLATE 3.

PLATE 25



NOTE: LOCATIONS SHOWN IN PLATE 3

COEFFICIENT B VS LOCATION
OAHU
LOCATIONS 73-105

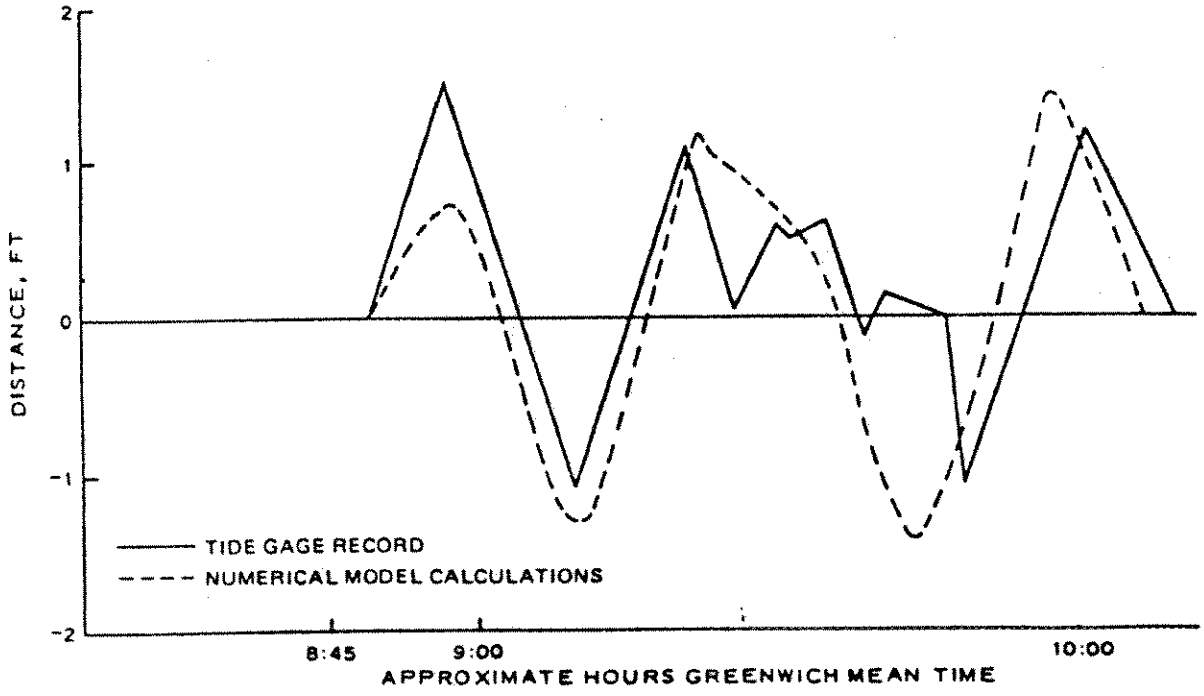


FIGURE 4 COMPARISON AT HONOLULU, HAWAII, FOR 1964 TSUNAMI

NOTES: BY C. L. BRETSCHNEIDER

(1) Grid lines put in by C.L.B.

(2) Primary period $T \tilde{=} 40$ minutes

(3) $A = \frac{1}{2}(4.5 + 5.9) = 5.2$ ft

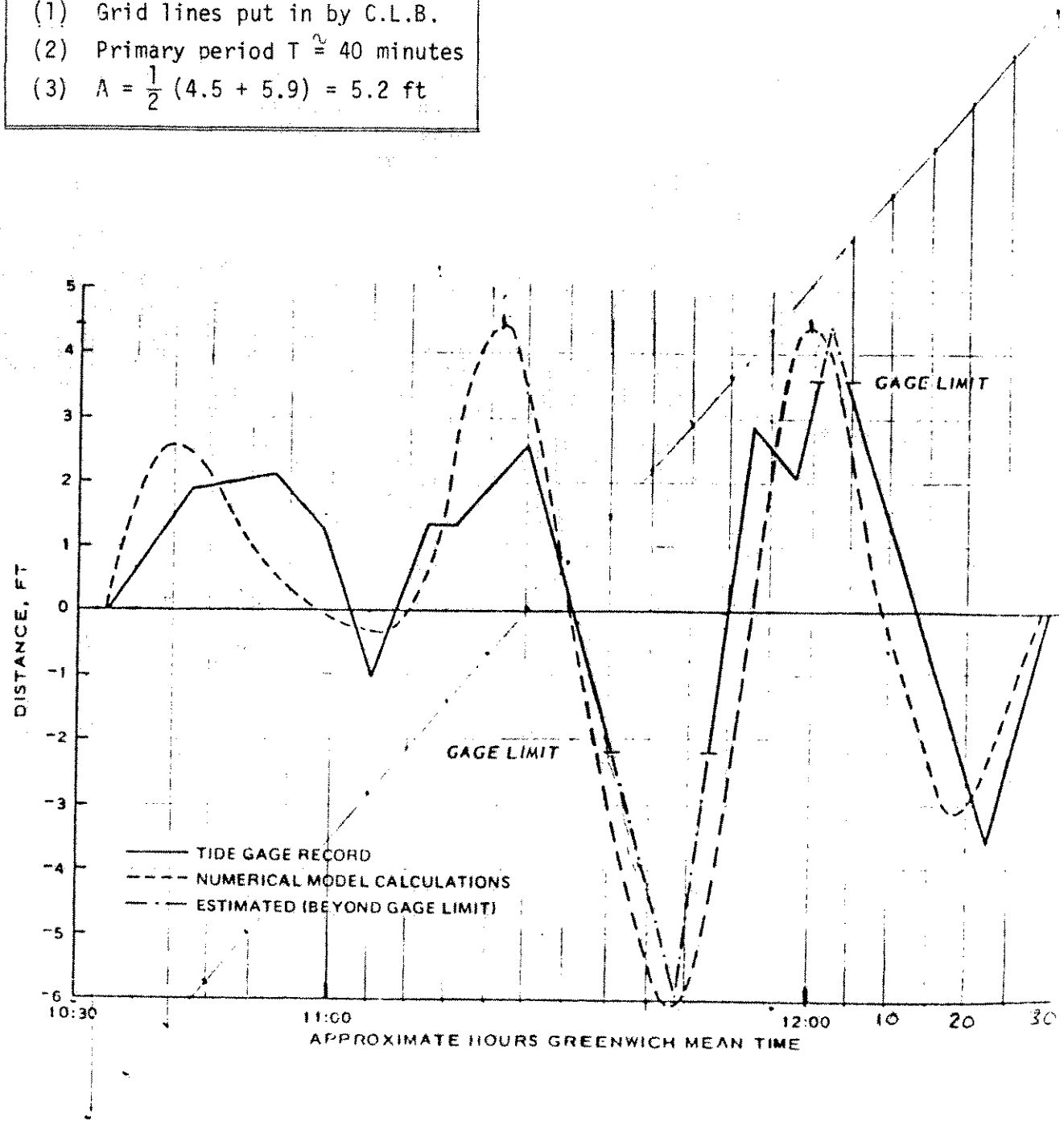


FIGURE 5 COMPARISON AT HONOLULU, HAWAII, FOR 1960 TSUNAMI

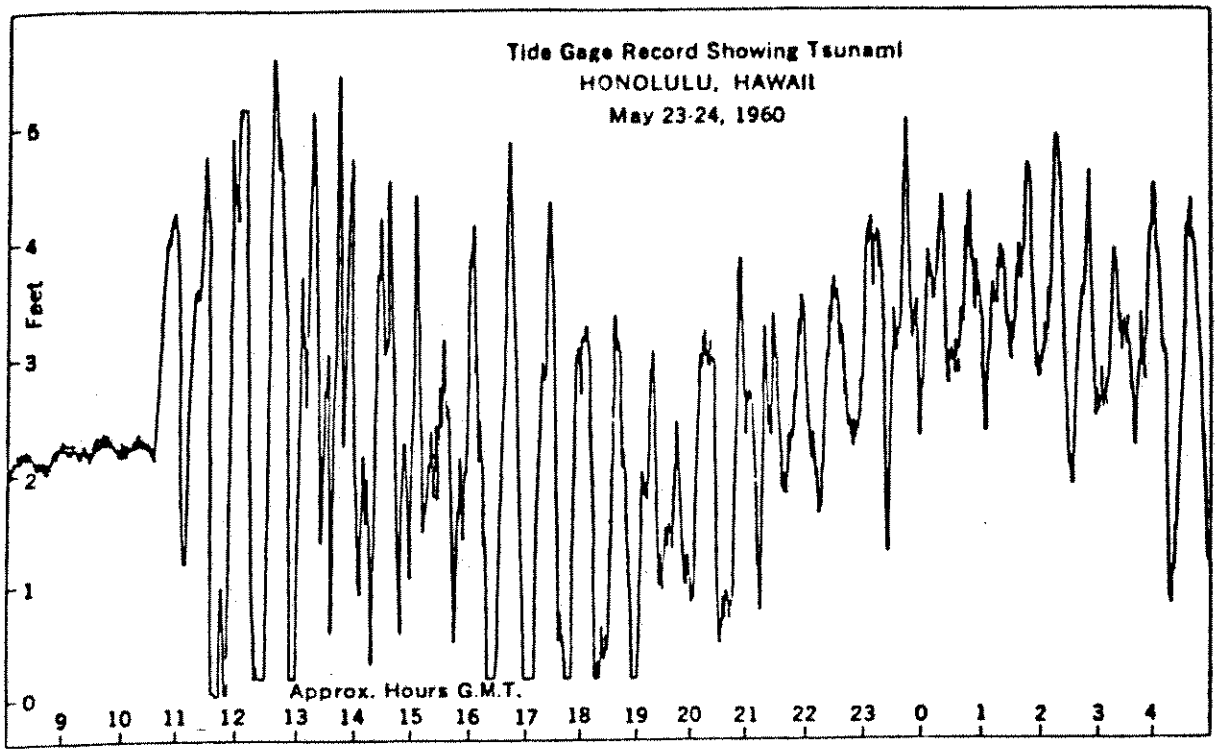


Figure 6. 1960 TSUNAMI TIDE RECORD AT HONOLULU

(from Symons and Zelter, 1960)

Extracted from U.S. Army Corps of Engineers (1973) Shore Protection Manual Errata, pp 3-75.

APPENDIX

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2. Choice of Hurricane Model
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SECTION II

HURRICANE STUDY

SUMMARY

The hurricane study was quite extensive because of very recent developments just prior to and after Hurricane Iwa (1982). The original intent of the hurricane study was changed because of the events associated with Hurricane Iwa (1982). A literature review was also conducted for previous studies of hurricane waves conducted by Bretschneider and also for the January 7-11, 1980 Kona Storm. Table I summarizes the findings, including the results for Hurricane Iwa, the 1980 Kona Storm and the design hurricane for West Beach, Oahu.

The main improvements for the methods and techniques used in the hindcasts include:

(a) the development of a new wind equation for the determination of maximum wind, V_{max} based on data from 319 Western North Pacific tropical cyclones and 122 U.S. East Coast and Gulf of Mexico hurricanes, and verified by the analysis of Hurricane Iwa (1982); and,

(b) an extensive evaluation of hurricane wind models, including the development of the BRET-MODEL-X hurricane which was verified for Hurricane Iwa (1982) for a previous study (See Bretschneider

TABLE I

SUMMARY OF SIGNIFICANT WAVES AND WINDS FOR WEST BEACH DESIGN HURRICANE AND VARIOUS OTHER HURRICANE AND KONA STORM HINDCASTS BY C.L. BRETSCHNEIDER

Condition	R N.Miles	V_F Knots	V_{10} Knots	V_{MAX} Knots	$H_{1/3}$ Ft	T_s Sec	f_o^{-1} Sec	H_B Ft	d_B Ft	S_w Ft	Note
West Beach Design Hurricane	30	15	65.3	83.7	34.7	12.1	13.8	29.3	37.5	2.5	(1)
Hurricane Iwa (1982)	50	20	62.9	80.6	41.0	14.2	16.2	36.4	46.6	2.8	(2)
Hurricane Iwa (1982)	50	25	62.7	80.4	42.7	14.7	16.9	38.2	49.0	2.9	(2)
Hurricane Nina (1957)	--	--	50.0	---	25.5	13.0	14.0	25.0	32.0	1.6	(3)
OTEC 100-Year Keahole Point	12	10	61.6	69.5	26.7	9.8	11.3	21.4	27.4	2.0	(4)
OTEC 100-Year Keahole Point	12	10	66.3	74.8	25.7	9.2	10.5	20.0	25.6	2.0	(4)
Kona Storm January 1980	--	--	42.0	---	29.0	13.5	15.4	27.9	35.8	1.8	(5)

NOTES:

(1) Latest methods and information used based on Note (2) below. BRET MODEL-X hurricane wind MODEL-X, BRET 2-D wave model

(2) •Considerable analysis (at least first approximation) BRET MODEL-X wind model overwater V_{10} wind speeds calibrated for land station measurements.

(3) Wilson's space time wind field method used.

(4) Hydromet Rankin Vortex wind model, BRET 2-D wave model (in view of Hurricane Iwa Note (2) and results Note (1), Keahole OTEC should be upgraded).

(5) BRET 2-D (modified 1-D, one-dimensional wave model).

R = radius of maximum winds; V_F = forward speed; V_{10} = 10-meter elevation overwater winds; V_{MAX} = MAX RECON or gradient wind; $H_{1/3}$ = significant wave height; T_s = significant period; f_o^{-1} = modal period of spectrum; H_B and d_B = breaking wave height and depth; S_w = wave set-up.

(NOTE: Table I also included in General Summary)

1981 and 1984). Thus, the West Beach Development Area hurricane study is based on the most up-to-date information and methods of calculations.

Hurricane Iwa (1982) was selected for an independent detailed analysis (See Figures 1, 2 and 3; also developed for other studies). The design hurricane for West Beach is similar to Hurricane Iwa, is somewhat smaller in diameter, and has moved past West Beach to give maximum conditions of breaking waves and wave set-up on the beach. The design hurricane is close to the 100-year return period, whereas Hurricane Iwa (1982) was considerably greater.

Hindcast data for design hurricane are given in Table II. Wave spectra are given only for the design hurricane as part of Table II. The wind and wave fields given in Table II are only for the right quadrant of the hurricane, for which maximum conditions are for winds and waves directed toward the coast. Winds and waves directed away from the coast (or in the left side of the hurricane) are lesser in magnitude and are not included. There was no need to include the winds and waves directed away from the coast.

During the investigation of Hurricane Iwa (1982) on other studies, it was found that the hurricane was associated with a squall line, a rapid increase in wind speed and change in direction. With the passage of the squall line, there was an associated pressure jump in the barograph. Calculations were made for maximized wave run-up on a previous study using the hindcast data for Hurricane Iwa (1982). It was found that the maximum calculated run-up was considerably lower than the observed and reported high water elevations. It was concluded that the pressure jump was the cause of a large bore type wave (or waves).

This finding led to further calculations on wave run-up for the West Beach area and is further discussed later in this Section.

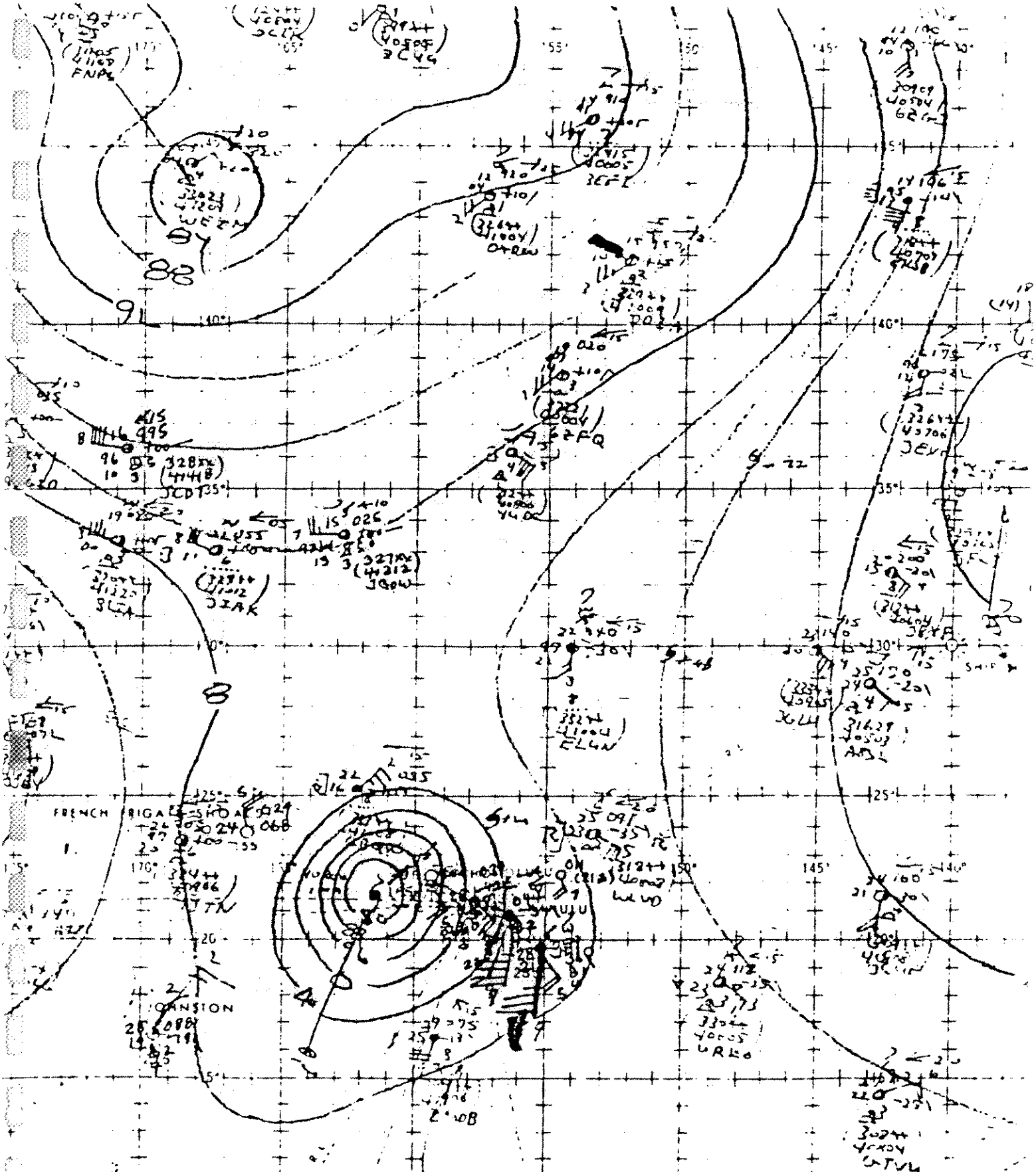
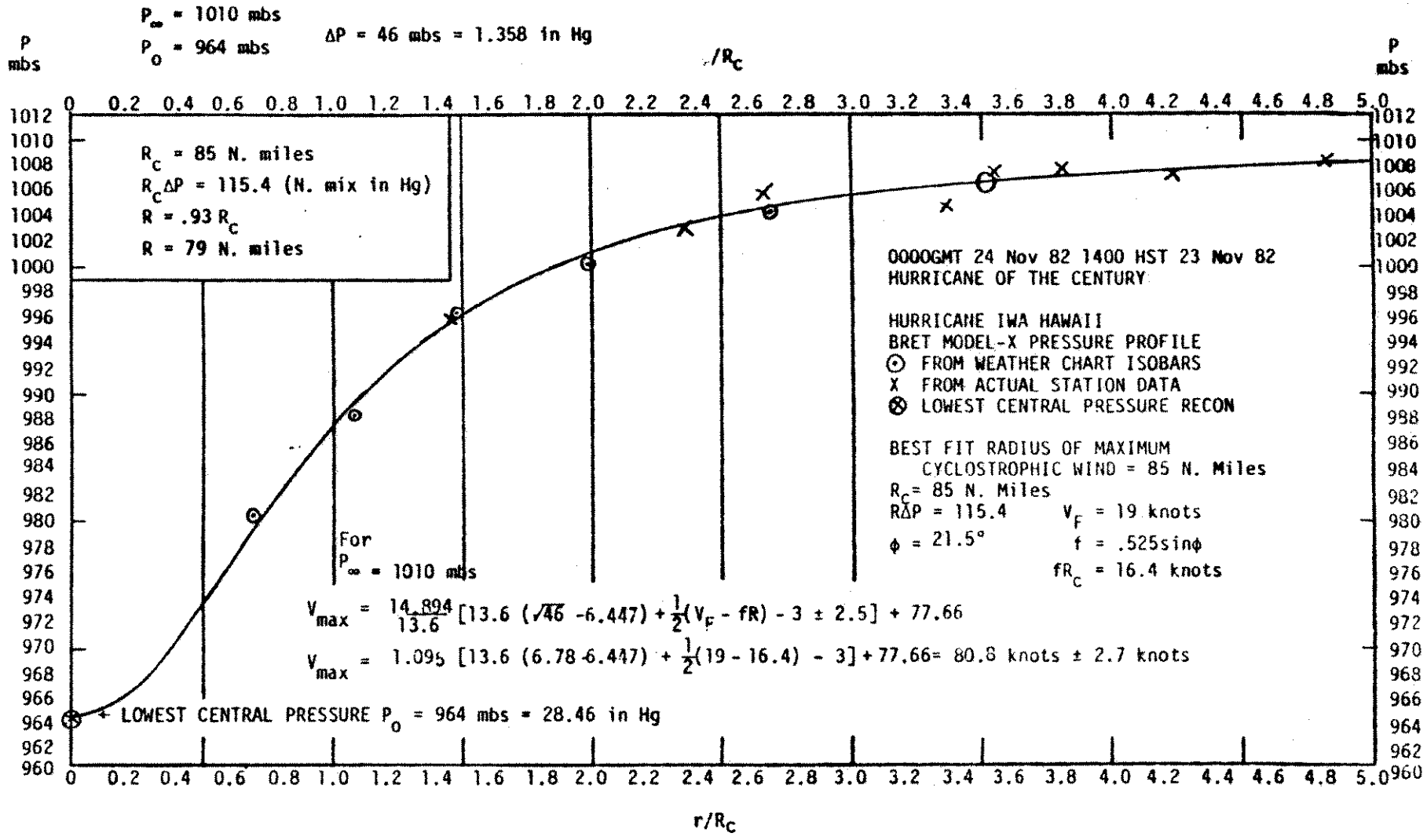


Figure 1. HURRICANE IWA, November 23, 1982, 2)M HST



PRESSURE PROFILE HURRICANE OF THE CENTURY U.S.A. IWA HAWAII

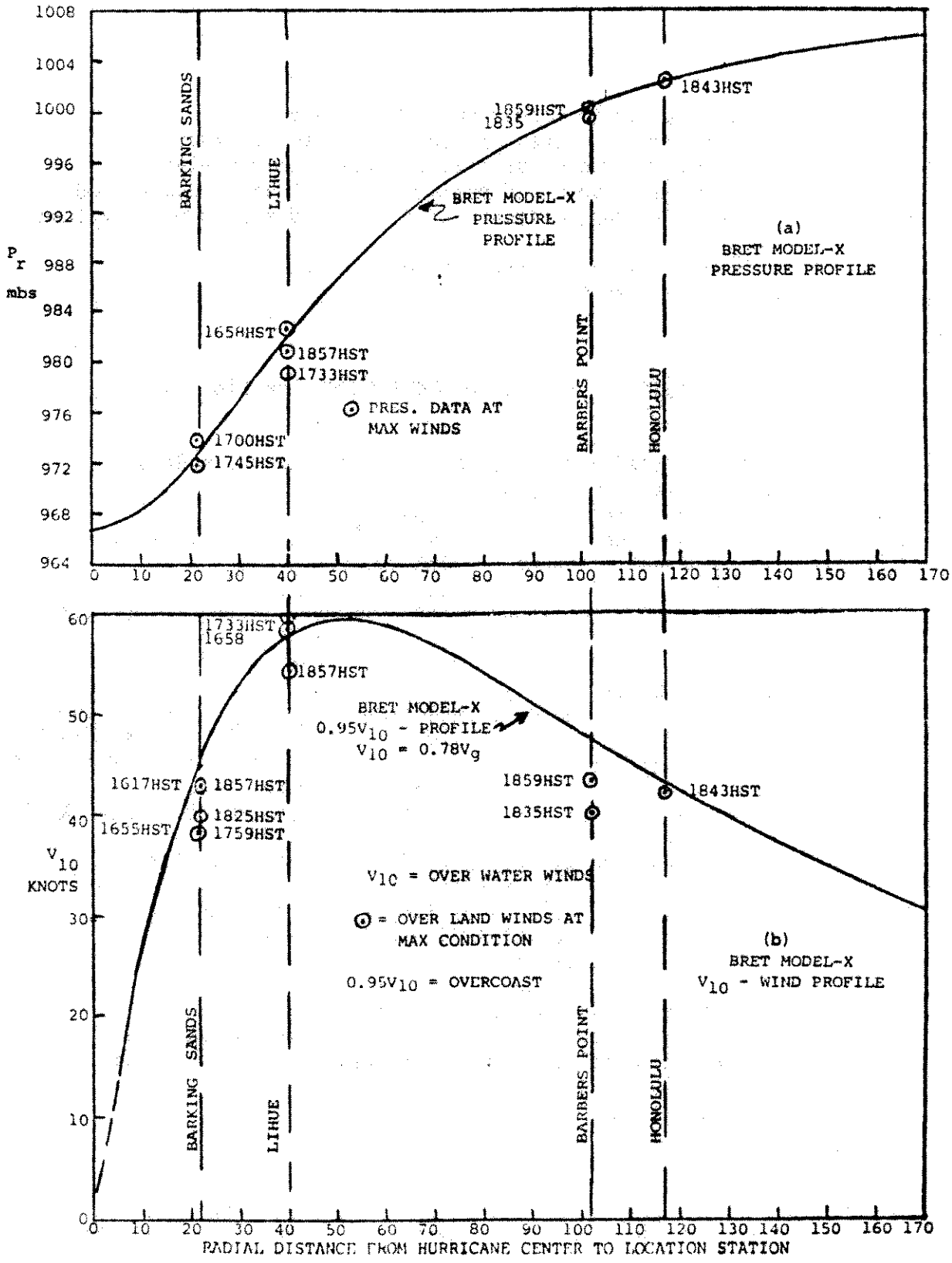


FIG. 3 PRELIMINARY MODEL-X HURRICANE AND SELECTED DATA

SYMBOLS USED IN TABLES I, II AND III

ϕ	=	20.5° lat. center location of hurricane
V_F	=	forward speed of translation (V_F = 0 and 15 knots)
R_C	=	32 n. miles, radius of maximum cyclostrophic wind
R	≅	30 n. miles, radius of maximum gradient and surface wind
P_n	=	P_∞ = 1012 mbs, pressure at infinite distance from hurricane center
P_o	=	964 mbs, central pressure of hurricane
ΔP_o	=	$P_\infty - P_o$ = 48 mbs

DEFINITION OF SYMBOLS PRINTOUT DATA (See Fig. 1)

r	=	radial distance from center of hurricane (n. miles)
A°	=	angle r makes with (+X) axis
X,Y	=	coordinates (n. miles) for U_{rs}, H_s, etc.
U_{rs}	=	vector surface wind velocity (knots) at (r, A°; or X,Y)
U_{xs}, U_{ys}	=	components of surface wind velocity, U_{rs}
β_u	=	angle of U_{rs} with respect to (+X) - axis
H_s	=	significant wave height (feet) (vector) at (r, A°, or X,Y)
H_x, H_y	=	components of significant wave height
β_h	=	angle of H_s with respect to (+X) - axis
$\beta_u - \beta_H$	=	angle H_s makes with U_{rs}
$T(f_o)$	=	f_o^{-1} = modal period of spectrum, (sec^{-1})
T_s	=	0.877 f_o^{-1} ≅ significant wave period (sec) (Wiegel, 1964)
H_b, d_B	=	non-refracted breaking wave and depth of breaking (Munk, 1949)
S_w	=	wave set-up on beach due to H_b, d_B (Munk, 1949)
$S(f_o)$	=	max. energy density (ft^2sec) of the BRETSCHNEIDER TWO-PARAMETER WAVE SPECTRUM
S(max)	=	max. energy density (ft^2sec) for the BRETSCHNEIDER THREE-PARAMETER WAVE SPECTRUM
S(f)	=	energy density (ft^2sec) vs $f(H_{zs})$ or sec^{-1} for the BRETSCHNEIDER THREE-PARAMETER WAVE SPECTRUM
f	=	frequency (hrs) or (sec^{-1})
T_f	=	f^{-1} period for f, (sec)
G	=	amplification factor, $S(max)/S(f_o)$
RH_o	=	ρ = linear regression coefficient between Rayleigh distribution of wave height and Rayleigh distribution of wave length (or T^2)
A^4	=	a parameter of the 3-parameter spectrum.

For G, ρ and A^4 see Bretschneider, et al. (1980) OTEC Report. Correction of 2π is missing in the expression for G. Limits G = 1 to 3/2. A^4 = 1 to 9/4 (ρ = 0 to 1).

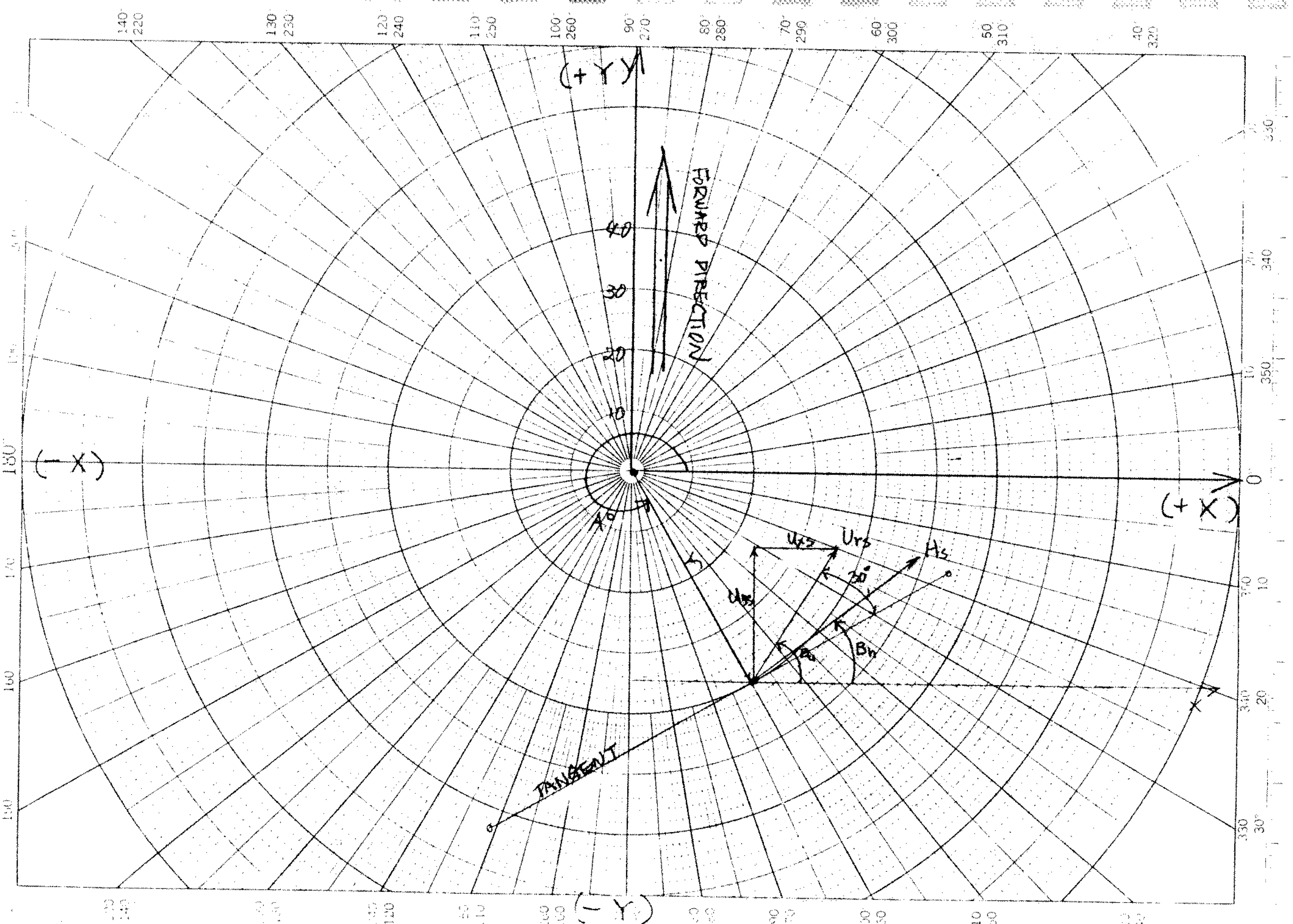


Figure 4 DEFINITION OF COORDINATE SYSTEM FOR HURRICANE

TABLE II

WEST BEACH DESIGN HURRICANE, V_F = 0
BRET HURRICANE WIND MODEL-X and WAVE MODEL-2D

Lat.= 20.5 deg.; Rc= 32 n.mi.; Pn= 1012 mbs; Po= 48 mbs; Vf= 0 knts

r N.M.	A DEG.	Hs FEET	X N.M.	Y N.M.	Hx FEET	Hy FEET	Uxs KNOTS	Uys KNOTS	Urs KNOTS	Tfo SEC.	Ts SEC.
10.0	290	22.0	3.4	-9.4	16.7	14.4	21.8	26.0	33.9	14.6	12.8
15.0	310	24.1	9.6	-11.5	17.6	16.5	15.6	42.9	45.7	12.8	11.2
20.0	310	25.5	12.9	-15.3	19.0	17.0	18.2	50.0	53.2	12.2	10.7
25.0	310	26.4	16.1	-19.2	20.2	17.0	19.6	53.8	57.3	12.0	10.6
30.0	310	26.8	19.3	-23.0	21.0	16.7	20.1	55.1	58.7	12.0	10.6
35.0	310	26.8	22.5	-26.8	21.4	16.1	19.9	54.8	58.3	12.1	10.6
40.0	310	26.5	25.7	-30.6	21.5	15.5	19.4	53.3	56.8	12.1	10.7
45.0	310	25.9	28.9	-34.5	21.4	14.7	18.7	51.3	54.6	12.2	10.7
50.0	310	25.2	32.1	-38.3	21.0	13.9	17.8	49.0	52.1	12.3	10.7
60.0	310	23.4	38.6	-46.0	19.9	12.3	16.0	44.0	46.9	12.3	10.8
70.0	300	19.9	35.0	-60.6	17.1	10.1	20.9	36.2	41.8	11.6	10.2
80.0	300	17.8	40.0	-69.3	15.5	8.9	18.7	32.3	37.3	11.4	10.0
90.0	300	15.9	45.0	-77.9	13.9	7.8	16.6	28.8	33.3	11.2	9.8
100.0	300	13.6	50.0	-86.6	11.8	6.8	14.9	25.8	29.8	10.5	9.2
110.0	290	11.3	37.6	-103.4	9.9	5.5	17.1	20.4	26.6	9.7	8.5
120.0	290	9.9	41.0	-112.8	8.7	4.8	15.4	18.3	23.9	9.4	8.2
130.0	290	8.7	44.5	-122.2	7.6	4.2	13.8	16.5	21.5	9.1	8.0
140.0	290	7.3	47.9	-131.6	6.4	3.6	12.4	14.8	19.3	8.5	7.4
150.0	290	6.1	51.3	-141.0	5.2	3.2	11.2	13.3	17.4	7.8	6.8
160.0	280	5.1	27.8	-157.6	4.5	2.4	12.0	10.1	15.7	7.2	6.3
170.0	280	4.4	29.5	-167.4	3.8	2.1	10.9	9.1	14.2	6.9	6.0
180.0	280	3.7	31.3	-177.3	3.3	1.8	9.8	8.2	12.8	6.5	5.7

r N.M.	Urs KNOTS	Bu DEG	Hs FEET	Bh DEG	Ts SEC.	Hb FEET	dB FEET	Sw
10.0	33.9	50	22.0	41	12.8	22.4	28.7	1.313
15.0	45.7	70	24.1	43	11.2	21.8	27.9	1.609
20.0	53.2	70	25.5	42	10.7	22.0	28.2	1.792
25.0	57.3	70	26.4	40	10.6	22.3	28.5	1.897
30.0	58.7	70	26.8	38	10.6	22.5	28.8	1.940
35.0	58.3	70	26.8	37	10.6	22.6	28.9	1.935
40.0	56.8	70	26.5	36	10.7	22.5	28.8	1.895
45.0	54.6	70	25.9	35	10.7	22.2	28.4	1.834
50.0	52.1	70	25.2	33	10.7	21.8	27.9	1.758
60.0	46.9	70	23.4	32	10.8	20.8	26.7	1.587
70.0	41.8	60	19.9	31	10.2	18.0	23.0	1.333
80.0	37.3	60	17.8	30	10.0	16.5	21.1	1.166
90.0	33.3	60	15.9	29	9.8	15.1	19.3	1.017
100.0	29.8	60	13.6	30	9.2	13.1	16.8	.856
110.0	26.6	50	11.3	29	8.5	11.0	14.1	.706
120.0	23.9	50	9.9	29	8.2	9.8	12.6	.606
130.0	21.5	50	8.7	29	8.0	8.8	11.3	.521
140.0	19.3	50	7.3	29	7.4	7.5	9.6	.435
150.0	17.4	50	6.1	32	6.8	6.3	8.0	.358
160.0	15.7	40	5.1	28	6.3	5.3	6.7	.296
170.0	14.2	40	4.4	29	6.0	4.6	5.9	.251
180.0	12.8	40	3.7	29	5.7	4.0	5.1	.210

TABLE II cont'd WEST BEACH DESIGN HURRICANE, V_F = 15 KNOTS
BRET HURRICANE WIND MODEL-X and WAVE MODEL 2-D

Lat.= 20.5 deg.; Rc= 32 n.mi.; Pn= 1012 mbs; Po= 48 mbs; Vf= 15 knts

r N.M.	A DEG.	Hs FEET	X N.M.	Y N.M.	Hx FEET	Hy FEET	Uxs KNOTS	Uys KNOTS	Urs KNOTS	Tfo SEC.	Ts SEC.
10.0	300	32.0	5.0	-8.7	15.3	28.1	16.9	36.8	40.5	17.7	15.5
15.0	310	33.3	9.6	-11.5	17.6	28.3	15.6	49.9	52.2	15.3	13.4
20.0	310	33.8	12.9	-15.3	19.0	27.9	18.2	57.0	59.8	14.3	12.5
25.0	310	34.3	16.1	-19.2	20.2	27.7	19.6	60.8	63.9	13.9	12.2
30.0	310	34.7	19.3	-23.0	21.0	27.6	20.1	62.1	65.3	13.8	12.1
35.0	310	34.7	22.5	-26.8	21.4	27.3	19.9	61.7	64.9	13.9	12.2
40.0	310	34.5	25.7	-30.6	21.5	27.0	19.4	60.3	63.3	14.0	12.3
45.0	310	34.1	28.9	-34.5	21.4	26.6	18.7	58.3	61.2	14.2	12.4
50.0	310	33.5	32.1	-38.3	21.0	26.1	17.8	55.9	58.7	14.3	12.6
60.0	310	32.0	38.6	-46.0	19.9	25.1	16.0	51.0	53.5	14.6	12.8
70.0	300	27.6	35.0	-60.6	17.1	21.7	20.9	43.7	48.4	13.8	12.1
80.0	300	25.7	40.0	-69.3	15.5	20.5	18.7	39.7	43.9	13.6	12.1
90.0	300	23.9	45.0	-77.9	13.9	19.5	16.6	36.2	39.9	13.9	12.2
100.0	300	21.4	50.0	-86.6	11.8	17.9	14.9	33.2	36.3	13.5	11.8
110.0	300	18.4	55.0	-95.3	9.4	15.8	13.3	30.4	33.2	12.6	11.1
120.0	290	16.5	41.0	-112.8	8.7	14.0	15.4	26.3	30.5	12.2	10.7
130.0	290	15.3	44.5	-122.2	7.6	13.2	13.8	24.4	28.1	12.2	10.7
140.0	290	13.7	47.9	-131.6	6.4	12.1	12.4	22.7	25.9	11.7	10.3
150.0	290	12.0	51.3	-141.0	5.2	10.9	11.2	21.2	24.0	11.2	9.8
160.0	10	10.7	157.6	27.8	-2.4	10.4	-10.1	19.9	22.3	10.7	9.4
170.0	10	9.9	167.4	29.5	-2.1	9.7	-9.1	18.7	20.8	10.6	9.3
180.0	10	9.1	177.3	31.3	-1.8	8.9	-8.2	17.6	19.4	10.5	9.2

r N.M.	Urs KNOTS	Bu DEG	Hs FEET	Bh DEG	Ts SEC.	Hb FEET	dB FEET	Sw
10.0	40.5	65	32.0	61	15.5	32.7	41.8	1.897
15.0	52.2	73	33.3	58	13.4	30.5	39.0	2.199
20.0	59.8	72	33.8	56	12.5	29.4	37.6	2.349
25.0	63.9	72	34.3	54	12.2	29.2	37.4	2.443
30.0	65.3	72	34.7	53	12.1	29.3	37.5	2.484
35.0	64.9	72	34.7	52	12.2	29.4	37.7	2.481
40.0	63.3	72	34.5	51	12.3	29.5	37.8	2.446
45.0	61.2	72	34.1	51	12.4	29.5	37.7	2.390
50.0	58.7	72	33.5	51	12.6	29.3	37.5	2.317
60.0	53.5	73	32.0	52	12.8	28.9	36.9	2.154
70.0	48.4	64	27.6	52	12.1	25.2	32.2	1.840
80.0	43.9	65	25.7	53	12.1	24.0	30.7	1.668
90.0	39.9	65	23.9	55	12.2	23.0	29.4	1.513
100.0	36.3	66	21.4	57	11.8	20.9	26.7	1.330
110.0	33.2	66	18.4	59	11.1	18.1	23.1	1.137
120.0	30.5	60	16.5	58	10.7	16.5	21.1	1.004
130.0	28.1	61	15.3	60	10.7	15.6	19.9	.908
140.0	25.9	61	13.7	62	10.3	14.2	18.1	.800
150.0	24.0	62	12.0	64	9.8	12.6	16.1	.700
160.0	22.3	117	10.7	103	9.4	11.3	14.5	.617
170.0	20.8	116	9.9	102	9.3	10.7	13.7	.557
180.0	19.4	115	9.1	101	9.2	10.0	12.8	.500

TABLE II cont'd

r n.m.	Us ft/sec	Hs ft	Tfo sec	S(fo)	S(max)	G	RHO	A4
10.0	40.5	31.99	17.67	11961	11379	1.00	0.0000	1.25

(f/fo)	S(f/fo)	f Hzs	S(f) ft.sec	Tf sec
.2	0.0000	.011	0.0	77.46
.4	.0000	.023	.0	38.73
.6	.0000	.034	.0	25.82
.8	.0284	.045	128.1	19.37
1.0	.2375	.057	1073.1	15.49
1.2	.3056	.068	1381.0	12.91
1.4	.2328	.079	1052.1	11.07
1.6	.1521	.091	687.3	9.68
1.8	.0960	.102	433.8	8.61
2.0	.0610	.113	275.7	7.75
2.2	.0396	.125	179.0	7.04
2.4	.0264	.136	119.1	6.46
2.6	.0180	.147	81.3	5.96
2.8	.0126	.159	56.9	5.53
3.0	.0090	.170	40.6	5.16
3.2	.0066	.181	29.6	4.84
3.4	.0049	.192	22.0	4.56
3.6	.0037	.204	16.6	4.30
3.8	.0028	.215	12.7	4.08
4.0	.0022	.226	9.8	3.87

r n.m.	Us ft/sec	Hs ft	Tfo sec	S(fo)	S(max)	G	RHO	A4
15.0	52.2	33.28	15.31	10076	11312	1.12	.3149	1.65

(f/fo)	S(f/fo)	f Hzs	S(f) ft.sec	Tf sec
.2	0.0000	.013	0.0	67.13
.4	.0000	.026	.0	33.57
.6	.0002	.039	1.0	22.38
.8	.1707	.052	723.5	16.78
1.0	.4020	.065	1704.0	13.43
1.2	.2925	.078	1239.8	11.19
1.4	.1638	.091	694.2	9.59
1.6	.0888	.105	376.3	8.39
1.8	.0496	.118	210.1	7.46
2.0	.0289	.131	122.6	6.71
2.2	.0176	.144	74.8	6.10
2.4	.0112	.157	47.5	5.59
2.6	.0074	.170	31.3	5.16
2.8	.0050	.183	21.3	4.80
3.0	.0035	.196	14.9	4.48
3.2	.0025	.209	10.6	4.20
3.4	.0018	.222	7.8	3.95
3.6	.0014	.235	5.8	3.73
3.8	.0010	.248	4.4	3.53
4.0	.0008	.261	3.4	3.36

TABLE II cont'd

r n.m.	Us ft/sec	Hs ft	Tfo sec	S(fo)	S(max)	G	RHO	A4
20.0	59.8	33.75	14.27	9575	11908	1.24	.5421	1.90

(f/fo)	S(f/fo)	f Hzs	S(f) ft.sec	Tf sec
.2	0.0000	.014	0.0	62.55
.4	.0000	.028	.0	31.28
.6	.0001	.042	.3	20.85
.8	.1641	.056	666.7	15.64
1.0	.4454	.070	1809.5	12.51
1.2	.3076	.084	1249.7	10.43
1.4	.1571	.098	638.3	8.94
1.6	.0777	.112	315.7	7.82
1.8	.0400	.126	162.5	6.95
2.0	.0218	.140	88.5	6.26
2.2	.0126	.154	51.0	5.69
2.4	.0076	.168	30.9	5.21
2.6	.0048	.182	19.6	4.81
2.8	.0032	.196	12.9	4.47
3.0	.0021	.210	8.7	4.17
3.2	.0015	.224	6.1	3.91
3.4	.0011	.238	4.4	3.68
3.6	.0008	.252	3.2	3.48
3.8	.0006	.266	2.4	3.29
4.0	.0004	.280	1.8	3.13

r n.m.	Us ft/sec	Hs ft	Tfo sec	S(fo)	S(max)	G	RHO	A4
25.0	63.9	34.32	13.91	9630	12410	1.29	.6220	1.97

(f/fo)	S(f/fo)	f Hzs	S(f) ft.sec	Tf sec
.2	0.0000	.014	0.0	60.98
.4	.0000	.029	.0	30.49
.6	.0001	.043	.2	20.33
.8	.1622	.058	664.1	15.25
1.0	.4615	.072	1889.5	12.20
1.2	.3127	.086	1280.2	10.16
1.4	.1541	.101	630.8	8.71
1.6	.0733	.115	300.2	7.62
1.8	.0364	.129	149.1	6.78
2.0	.0192	.144	78.7	6.10
2.2	.0108	.158	44.1	5.54
2.4	.0064	.173	26.1	5.08
2.6	.0039	.187	16.2	4.69
2.8	.0025	.201	10.4	4.36
3.0	.0017	.216	7.0	4.07
3.2	.0012	.230	4.8	3.81
3.4	.0008	.244	3.4	3.59
3.6	.0006	.259	2.5	3.39
3.8	.0004	.273	1.8	3.21
4.0	.0003	.288	1.4	3.05

TABLE II cont'd

r	Us	Hs	Tfo	S(fo)	S(max)	G	RHO	A4
n.m.	ft/sec	ft	sec					
30.0	65.3	34.67	13.84	9779	12717	1.30	.6429	1.99

(f/fo)	S(f/fo)	f Hzs	S(f) ft.sec	Tf sec
.2	0.0000	.014	0.0	60.71
.4	.0000	.029	.0	30.35
.6	.0001	.043	.2	20.24
.8	.1617	.058	672.8	15.18
1.0	.4657	.072	1937.2	12.14
1.2	.3139	.087	1305.9	10.12
1.4	.1532	.101	637.3	8.67
1.6	.0722	.116	300.2	7.59
1.8	.0355	.130	147.6	6.75
2.0	.0186	.144	77.2	6.07
2.2	.0103	.159	42.9	5.52
2.4	.0061	.173	25.2	5.06
2.6	.0037	.188	15.5	4.67
2.8	.0024	.202	10.0	4.34
3.0	.0016	.217	6.6	4.05
3.2	.0011	.231	4.5	3.79
3.4	.0008	.246	3.2	3.57
3.6	.0006	.260	2.3	3.37
3.8	.0004	.274	1.7	3.20
4.0	.0003	.289	1.3	3.04

r	Us	Hs	Tfo	S(fo)	S(max)	G	RHO	A4
n.m.	ft/sec	ft	sec					
35.0	64.9	34.72	13.91	9861	12774	1.30	.6341	1.98

(f/fo)	S(f/fo)	f Hzs	S(f) ft.sec	Tf sec
.2	0.0000	.014	0.0	61.01
.4	.0000	.029	.0	30.50
.6	.0001	.043	.2	20.34
.8	.1619	.058	679.1	15.25
1.0	.4639	.072	1945.5	12.20
1.2	.3134	.086	1314.3	10.17
1.4	.1536	.101	644.0	8.72
1.6	.0727	.115	304.7	7.63
1.8	.0359	.129	150.5	6.78
2.0	.0188	.144	79.0	6.10
2.2	.0105	.158	44.0	5.55
2.4	.0062	.173	25.9	5.08
2.6	.0038	.187	16.0	4.69
2.8	.0025	.201	10.3	4.36
3.0	.0016	.216	6.9	4.07
3.2	.0011	.230	4.7	3.81
3.4	.0008	.244	3.3	3.59
3.6	.0006	.259	2.4	3.39
3.8	.0004	.273	1.8	3.21
4.0	.0003	.288	1.3	3.05

TABLE II cont'd

r n.m.	Us ft/sec	Hs ft	Tfo sec	S(fo)	S(max)	G	RHO	A4
40.0	63.3	34.51	14.04	9836	12594	1.28	.6072	1.96

(f/fo)	S(f/fo)	f Hzs	S(f) ft.sec	Tf sec
.2	0.0000	.014	0.0	61.56
.4	.0000	.028	.0	30.78
.6	.0001	.043	.2	20.52
.8	.1625	.057	679.5	15.39
1.0	.4585	.071	1916.9	12.31
1.2	.3118	.085	1303.3	10.26
1.4	.1546	.100	646.5	8.79
1.6	.0741	.114	310.0	7.70
1.8	.0371	.128	155.0	6.84
2.0	.0197	.142	82.3	6.16
2.2	.0111	.157	46.4	5.60
2.4	.0066	.171	27.6	5.13
2.6	.0041	.185	17.2	4.74
2.8	.0027	.199	11.1	4.40
3.0	.0018	.214	7.5	4.10
3.2	.0012	.228	5.2	3.85
3.4	.0009	.242	3.7	3.62
3.6	.0006	.256	2.7	3.42
3.8	.0005	.271	2.0	3.24
4.0	.0004	.285	1.5	3.08

r n.m.	Us ft/sec	Hs ft	Tfo sec	S(fo)	S(max)	G	RHO	A4
45.0	61.2	34.10	14.19	9719	12225	1.26	.5673	1.92

(f/fo)	S(f/fo)	f Hzs	S(f) ft.sec	Tf sec
.2	0.0000	.014	0.0	62.24
.4	.0000	.028	.0	31.12
.6	.0001	.042	.3	20.75
.8	.1635	.056	674.7	15.56
1.0	.4505	.070	1858.9	12.45
1.2	.3092	.085	1276.1	10.37
1.4	.1562	.099	644.5	8.89
1.6	.0763	.113	315.0	7.78
1.8	.0389	.127	160.4	6.92
2.0	.0210	.141	86.6	6.22
2.2	.0120	.155	49.5	5.66
2.4	.0072	.169	29.8	5.19
2.6	.0045	.183	18.7	4.79
2.8	.0030	.197	12.2	4.45
3.0	.0020	.211	8.3	4.15
3.2	.0014	.225	5.8	3.89
3.4	.0010	.240	4.1	3.66
3.6	.0007	.254	3.0	3.46
3.8	.0005	.268	2.2	3.28
4.0	.0004	.282	1.7	3.11

TABLE II cont'd

r n.m.	Us ft/sec	Hs ft	Tfo sec	S(fo)	S(max)	G	RHO	A4
50.0	58.7	33.49	14.34	9482	11668	1.23	.5164	1.87

(f/fo)	S(f/fo)	f Hzs	S(f) ft.sec	Tf sec
.2	0.0000	.014	0.0	62.88
.4	.0000	.028	.0	31.44
.6	.0001	.042	.3	20.96
.8	.1647	.056	662.3	15.72
1.0	.4407	.070	1771.8	12.58
1.2	.3061	.084	1230.6	10.48
1.4	.1580	.098	635.1	8.98
1.6	.0790	.112	317.5	7.86
1.8	.0411	.126	165.1	6.99
2.0	.0226	.139	90.7	6.29
2.2	.0131	.153	52.7	5.72
2.4	.0080	.167	32.1	5.24
2.6	.0051	.181	20.4	4.84
2.8	.0034	.195	13.5	4.49
3.0	.0023	.209	9.2	4.19
3.2	.0016	.223	6.4	3.93
3.4	.0012	.237	4.6	3.70
3.6	.0008	.251	3.4	3.49
3.8	.0006	.265	2.5	3.31
4.0	.0005	.279	1.9	3.14

r n.m.	Us ft/sec	Hs ft	Tfo sec	S(fo)	S(max)	G	RHO	A4
60.0	53.5	32.03	14.64	8891	10352	1.16	.3966	1.74

(f/fo)	S(f/fo)	f Hzs	S(f) ft.sec	Tf sec
.2	0.0000	.014	0.0	64.18
.4	.0000	.027	.0	32.09
.6	.0002	.041	.6	21.39
.8	.1681	.055	631.2	16.05
1.0	.4170	.068	1565.3	12.84
1.2	.2979	.082	1118.5	10.70
1.4	.1618	.096	607.3	9.17
1.6	.0851	.109	319.4	8.02
1.8	.0463	.123	173.8	7.13
2.0	.0265	.137	99.3	6.42
2.2	.0159	.150	59.5	5.83
2.4	.0099	.164	37.3	5.35
2.6	.0065	.178	24.3	4.94
2.8	.0044	.191	16.3	4.58
3.0	.0030	.205	11.3	4.28
3.2	.0021	.219	8.0	4.01
3.4	.0016	.232	5.8	3.78
3.6	.0012	.246	4.3	3.57
3.8	.0009	.260	3.3	3.38
4.0	.0007	.273	2.5	3.21

TABLE II cont'd

r n.m.	Us ft/sec	Hs ft	Tfo sec	S(fo)	S(max)	G	RHO	A4
70.0	48.4	27.64	13.80	6256	7131	1.14	.3494	1.69

(f/fo)	S(f/fo)	f Hzs	S(f) ft.sec	Tf sec
.2	0.0000	.014	0.0	60.53
.4	.0000	.029	.0	30.26
.6	.0002	.043	.5	20.18
.8	.1696	.058	447.0	15.13
1.0	.4082	.072	1075.9	12.11
1.2	.2948	.087	777.0	10.09
1.4	.1630	.101	429.6	8.65
1.6	.0873	.116	230.0	7.57
1.8	.0482	.130	127.1	6.73
2.0	.0279	.145	73.5	6.05
2.2	.0169	.159	44.6	5.50
2.4	.0107	.174	28.2	5.04
2.6	.0070	.188	18.5	4.66
2.8	.0047	.203	12.5	4.32
3.0	.0033	.217	8.7	4.04
3.2	.0024	.232	6.2	3.78
3.4	.0017	.246	4.5	3.56
3.6	.0013	.261	3.4	3.36
3.8	.0010	.275	2.5	3.19
4.0	.0007	.290	1.9	3.03

r n.m.	Us ft/sec	Hs ft	Tfo sec	S(fo)	S(max)	G	RHO	A4
80.0	43.9	25.71	13.84	5465	5892	1.08	.2194	1.53

(f/fo)	S(f/fo)	f Hzs	S(f) ft.sec	Tf sec
.2	0.0000	.014	0.0	60.68
.4	.0000	.029	.0	30.34
.6	.0004	.043	.9	20.23
.8	.1738	.058	397.4	15.17
1.0	.3861	.072	882.7	12.14
1.2	.2864	.087	654.8	10.11
1.4	.1656	.101	378.6	8.67
1.6	.0925	.116	211.5	7.58
1.8	.0530	.130	121.1	6.74
2.0	.0315	.145	72.1	6.07
2.2	.0196	.159	44.7	5.52
2.4	.0126	.173	28.8	5.06
2.6	.0084	.188	19.2	4.67
2.8	.0058	.202	13.2	4.33
3.0	.0041	.217	9.3	4.05
3.2	.0029	.231	6.7	3.79
3.4	.0021	.246	4.9	3.57
3.6	.0016	.260	3.7	3.37
3.8	.0012	.275	2.8	3.19
4.0	.0009	.289	2.1	3.03

TABLE II cont'd

r n.m.	Us ft/sec	Hs ft	Tfo sec	S(fo)	S(max)	G	RHO	A4
90.0	39.9	23.95	13.89	4817	4925	1.02	.0745	1.35

(f/fo)	S(f/fo)	f Hzs	S(f) ft.sec	Tf sec
.2	0.0000	.014	0.0	60.92
.4	.0000	.029	.0	30.46
.6	.0008	.043	1.5	20.31
.8	.1784	.058	355.4	15.23
1.0	.3662	.072	729.4	12.18
1.2	.2783	.086	554.4	10.15
1.4	.1674	.101	333.4	8.70
1.6	.0969	.115	193.0	7.61
1.8	.0571	.130	113.8	6.77
2.0	.0348	.144	69.4	6.09
2.2	.0220	.158	43.8	5.54
2.4	.0144	.173	28.7	5.08
2.6	.0097	.187	19.3	4.69
2.8	.0067	.202	13.4	4.35
3.0	.0048	.216	9.5	4.06
3.2	.0035	.230	6.9	3.81
3.4	.0026	.245	5.1	3.58
3.6	.0019	.259	3.8	3.38
3.8	.0015	.274	2.9	3.21
4.0	.0011	.288	2.3	3.05

r n.m.	Us ft/sec	Hs ft	Tfo sec	S(fo)	S(max)	G	RHO	A4
100.0	36.3	21.39	13.46	3764	3742	1.00	0.0000	1.25

(f/fo)	S(f/fo)	f Hzs	S(f) ft.sec	Tf sec
.2	0.0000	.015	0.0	59.02
.4	.0000	.030	.0	29.51
.6	.0013	.045	2.0	19.67
.8	.1893	.059	291.6	14.75
1.0	.3603	.074	554.8	11.80
1.2	.2723	.089	419.3	9.84
1.4	.1651	.104	254.3	8.43
1.6	.0966	.119	148.7	7.38
1.8	.0575	.134	88.5	6.56
2.0	.0353	.149	54.4	5.90
2.2	.0225	.163	34.6	5.37
2.4	.0148	.178	22.7	4.92
2.6	.0100	.193	15.4	4.54
2.8	.0069	.208	10.7	4.22
3.0	.0049	.223	7.6	3.93
3.2	.0036	.238	5.5	3.69
3.4	.0027	.253	4.1	3.47
3.6	.0020	.267	3.1	3.28
3.8	.0015	.282	2.4	3.11
4.0	.0012	.297	1.8	2.95

TABLE II cont'd

r n.m.	Us ft/sec	Hs ft	Tfo sec	S(fo)	S(max)	G	RHO	A4
110.0	33.2	18.42	12.62	2633	2591	1.00	0.0000	1.25

(f/fo)	S(f/fo)	f Hzs	S(f) ft.sec	Tf sec
.2	0.0000	.016	0.0	55.34
.4	.0000	.032	.0	27.67
.6	.0018	.048	1.9	18.45
.8	.2056	.063	220.2	13.84
1.0	.3633	.079	389.1	11.07
1.2	.2673	.095	286.3	9.22
1.4	.1603	.111	171.6	7.91
1.6	.0932	.127	99.8	6.92
1.8	.0553	.143	59.2	6.15
2.0	.0339	.158	36.3	5.53
2.2	.0216	.174	23.1	5.03
2.4	.0142	.190	15.2	4.61
2.6	.0096	.206	10.3	4.26
2.8	.0067	.222	7.1	3.95
3.0	.0047	.238	5.1	3.69
3.2	.0034	.254	3.7	3.46
3.4	.0025	.269	2.7	3.26
3.6	.0019	.285	2.1	3.07
3.8	.0015	.301	1.6	2.91
4.0	.0011	.317	1.2	2.77

r n.m.	Us ft/sec	Hs ft	Tfo sec	S(fo)	S(max)	G	RHO	A4
120.0	30.5	16.52	12.23	2086	2016	1.00	0.0000	1.25

(f/fo)	S(f/fo)	f Hzs	S(f) ft.sec	Tf sec
.2	0.0000	.016	0.0	53.62
.4	.0000	.033	.0	26.81
.6	.0038	.049	3.2	17.87
.8	.2436	.065	203.2	13.41
1.0	.3674	.082	306.5	10.72
1.2	.2554	.098	213.0	8.94
1.4	.1494	.114	124.7	7.66
1.6	.0859	.131	71.7	6.70
1.8	.0506	.147	42.2	5.96
2.0	.0310	.164	25.8	5.36
2.2	.0196	.180	16.4	4.87
2.4	.0129	.196	10.7	4.47
2.6	.0087	.213	7.3	4.12
2.8	.0060	.229	5.0	3.83
3.0	.0043	.245	3.6	3.57
3.2	.0031	.262	2.6	3.35
3.4	.0023	.278	1.9	3.15
3.6	.0017	.294	1.5	2.98
3.8	.0013	.311	1.1	2.82
4.0	.0010	.327	.9	2.68

TABLE II cont'd

r	Us	Hs	Tfo	S(fo)	S(max)	G	RHO	A4
n.m.	ft/sec	ft	sec					
130.0	28.1	15.28	12.17	1864	1775	1.00	0.0000	1.25

(f/fo)	S(f/fo)	f Hzs	S(f) ft.sec	Tf sec
.2	0.0000	.016	0.0	53.37
.4	.0000	.033	.0	26.68
.6	.0000	.049	.0	17.79
.8	.0284	.066	20.1	13.34
1.0	.2375	.082	168.6	10.67
1.2	.3056	.099	217.0	8.89
1.4	.2328	.115	165.3	7.62
1.6	.1521	.131	108.0	6.67
1.8	.0960	.148	68.2	5.93
2.0	.0610	.164	43.3	5.34
2.2	.0396	.181	28.1	4.85
2.4	.0264	.197	18.7	4.45
2.6	.0180	.214	12.8	4.11
2.8	.0126	.230	8.9	3.81
3.0	.0090	.246	6.4	3.56
3.2	.0066	.263	4.7	3.34
3.4	.0049	.279	3.5	3.14
3.6	.0037	.296	2.6	2.96
3.8	.0028	.312	2.0	2.81
4.0	.0022	.329	1.5	2.67

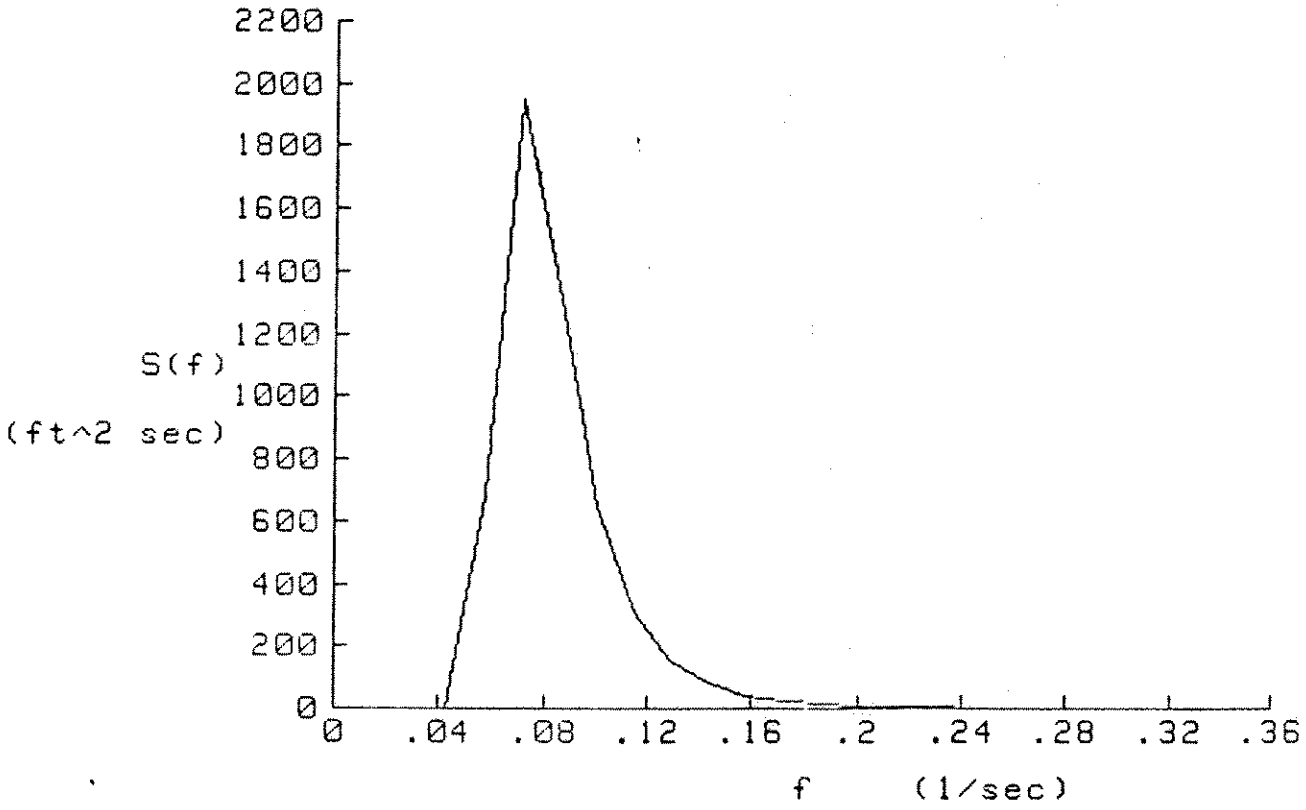


Figure II-1. WEST BEACH DESIGN SPECTRUM

MAX H = 34.7 feet

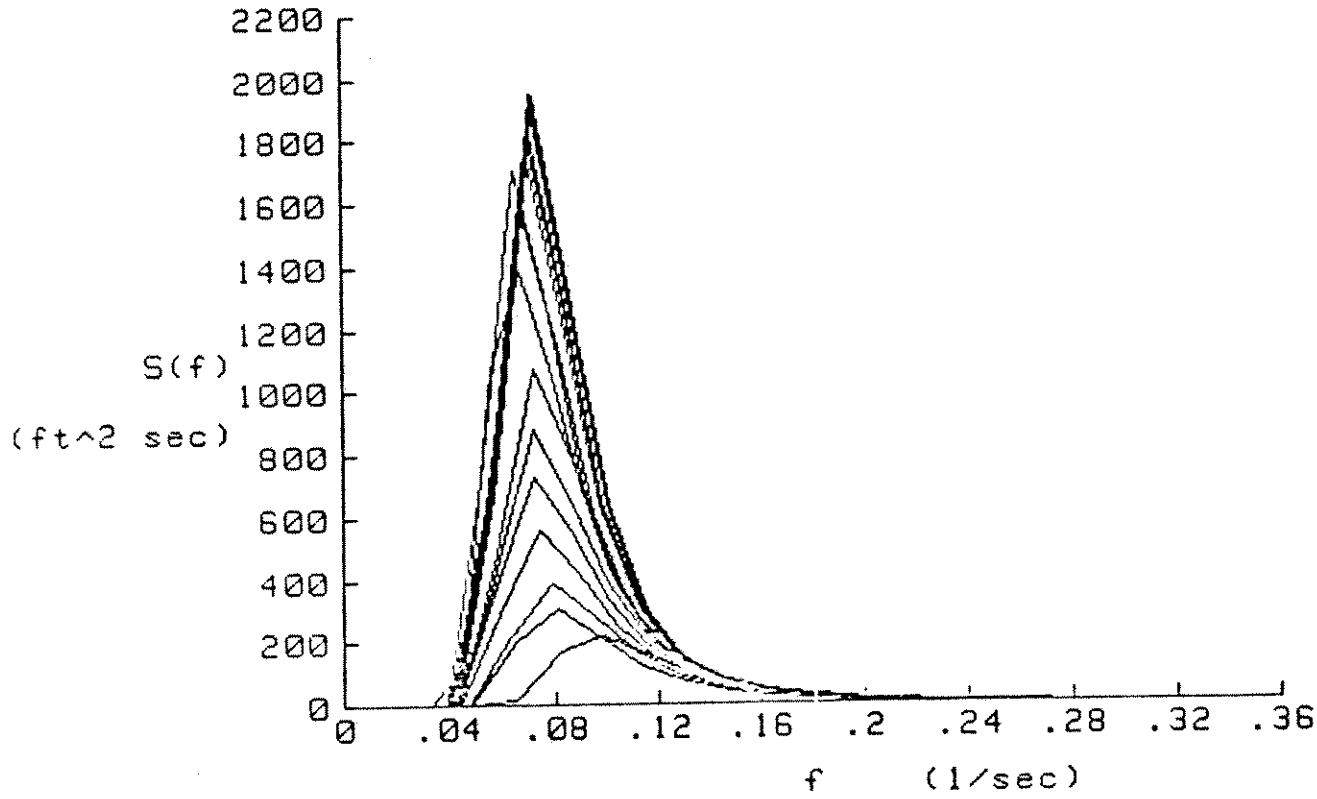


Figure II-2. FAMILY OF WAVE SPECTRA FOR WEST BEACH AREA

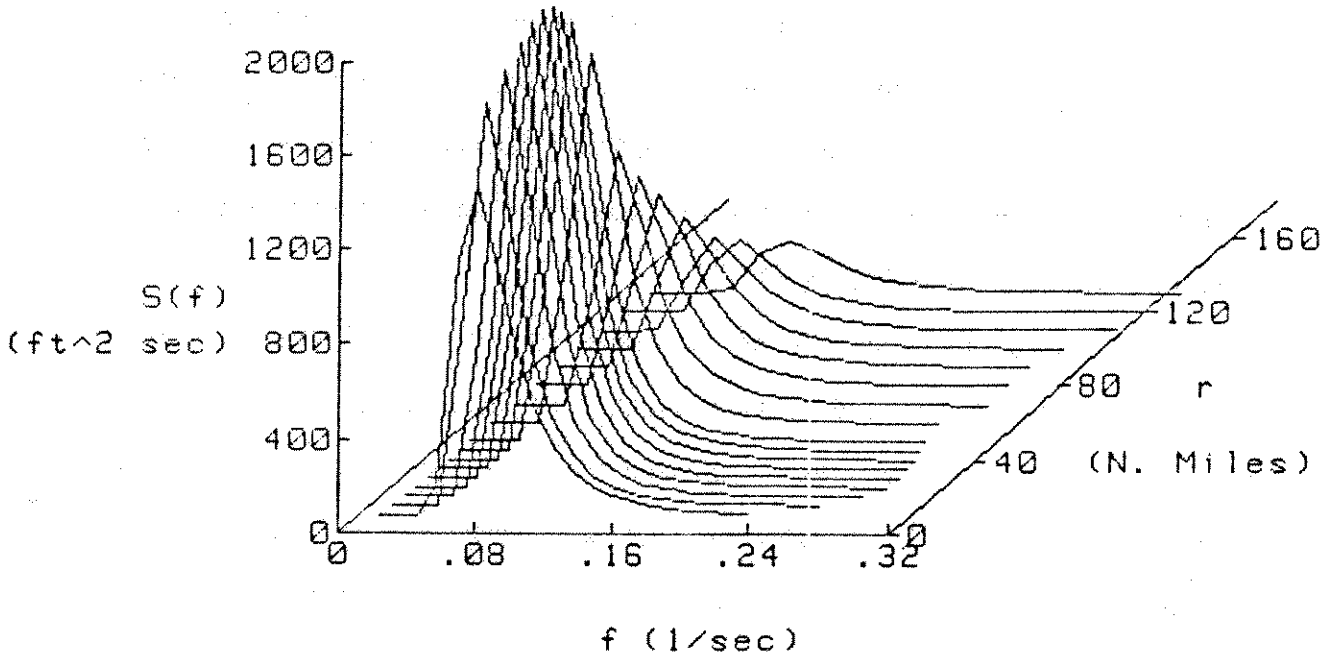


Figure II-3. TWO-DIMENSIONAL PLOT OF WAVE SPECTRA IN DISTANCE FROM CENTER OF DESIGN HURRICANE

METHODS FOR CALCULATING HURRICANE WINDS, WAVES AND WAVE SPECTRA AND BREAKING WAVES, WAVE RUN-UP AND WAVE SET-UP

1. INTRODUCTION

The hurricane study is based upon an extensive evaluation of Hurricane Iwa (1982), Fig. 1, and a verification of the BRET-MODEL-X hurricane pressure profile, from which is determined the hurricane wind field over open water. Wind recorded data, where available have been used for verification of the wind model. The BRET-Hurricane Wave MODEL-2D has been verified by recorded data from the Gulf of Mexico during Hurricane Camille (1969) and is also calibrated for the three parameter Bretschneider wave spectrum with wave spectra from Hurricane Eloise (1975).

Based on (a) the analysis of maximum wind and central pressures measured from RECON aircraft for 319 Western North Pacific tropical cyclones and typhoons and (b) the analysis of forward speed of translation and radius of maximum wind for 122 U.S. East Coast and Gulf of Mexico Bretschneider (1983) developed a highly correlated statistical equation for determining the maximum wind in a hurricane having specified parameters: (a) ϕ, latitude (b) R_c, the radius of maximum cyclostrophic wind (c) P_o, the central pressure (d) P_∞, the terminal pressure at infinite distance from the hurricane center, and (e) V_F the forward speed of translation of the hurricane. The prediction equation is as follows:

1) $$V_{MAX} = [A\,(\sqrt{\Delta P_o} - \langle \sqrt{\Delta P_o} \rangle) + 77.66] + \frac{1}{2}\frac{A}{K}\,(V_F - fR - 6 \pm 5)$$

V_{MAX} = maximum wind speed in knots, estimated by RECON aircraft.

The terms in the brackets are based on the analysis of 319 Western North Pacific cyclones, and is independent on the choice of model. The

terms in parenthesis is based on the analysis of 122 U.S. East Coast and Gulf Coasts, where 6 knots and ± 5 knots are the mean of the values and standard deviation respectively of V_F - fR.

V_F = forward speed of hurricane (knots)

fR_C = 0.525 R_C sin ϕ (knots), where ϕ is the latitude

f = Coriolis parameter

ΔP_o = $P_\infty - P_o$, (mbs = millibars pressure)

P_o = centra pressure (mbs)

P_∞ = terminal or theoretical pressure at infinite distance from center of hurricane

2) $A = 0.209941\ P_\infty - 197.1453$

3) $\langle \sqrt{\Delta P_o} \rangle = 0.0872376\ P_\infty - 81.633189 \quad (\text{mbs})^{1/2}$

K = constant depending on the choice of model and is found in the following equation:

4) $U_R = K' \sqrt{\text{MAX } \frac{rdp}{dr}}$

5) $U_R = K \sqrt{\Delta P_o}$ (knots)

6) $\Delta P_o = \text{MAX } \frac{rdp}{dr}$

K' = 18.7 to 19.2 for all models, depending on air density ρ_a.

The constant K depends on air density and choice of model, for example, K = 13.22 to 13.62 for the BRET-MODEL-X, 11.60 to 11.95 for Fujita Model, and 11.34 to 11.68 for the Hydromet Rankin Vortex Model.

Since A and $\langle \sqrt{\Delta P_o} \rangle$ (Eqs. 2 and 3) must be the same for all models, there can be only one model which is best for a particular set of hurricane parameters.

There are methods that can be used to determine the best model. When the pressure profile is given, then that is all that is required for the determination of the appropriate hurricane model.

When only certain parameters are given, then a guide is available for the selection of the model.

2. CHOICE OF HURRICANE MODEL

A guide to the choice of the hurricane model is required when the pressure profile is not given. The choice of the model is not dependent on the main part of Equation 1 as given in the brackets. The remaining part of the equation is of lesser importance for V_{max}, but does depend on the choice of the model as reflected in the proper value of K. However the importance of the choice of the model is required to obtain the entire pressure profile which is used with the gradient wind equations. Whereas Eq. 1 can only be used to obtain V_{max}, and the appropriate value of the maximum gradient wind and the true radius of maximum gradient wind, which will be less than the radius of maximum cyclostrophic wind, except for very small radius of maximum wind. Once the radius of maximum wind is determined and the maximum gradient wind is determined, then, and the appropriate model is selected. Then it is possible to determine the model hurricane wind field.

A non-dimensional parameter, which we will call the Rankin Vortex number, is as follows:

$$7) \quad N_R = \frac{fR}{U_R}$$

When the hurricane is at low latitude, ϕ, has a small radius, R, and a large cyclostrophic wind, U_R, (large ΔP_o) then N_{R_C} is very small, and the Rankin Vortex model applies. When N_{R_C} becomes larger then some other model

is more applicable. For example, when N_{R_C} = 0.018 found for Hurricane Camille (1969) for the Gulf of Mexico the Rankin Vortex model applies, and when N_{R_C} = 0.22 to 0.15 found for Hurricane Iwa (1982) then the BRET-MODEL-X applies. T. Uji (1975) investigated Typhoon Vera (1959) assuming FUJITA's 1962 model, (for Vera N_R = 0.15) and found reasonable results. FUJITA's MODEL is the same as BRET-MODEL with b = 1/2 compared with BRET-MODEL-X with b = 1. Bretschneider (1982) analyzed 6 Indian Seas cyclones to determine R_C the radius of maximum wind, MAX $[\frac{rdp}{dr}]$ and P_{R_C}, the pressure at the radius of maximum cyclostrophic wind. The BRET-MODEL-X pressure profiles fitted with high correlation the data pressure profiles as follows:

N_{R_C}	0.199	.0617	.0516	.215	0.114	.0720
r	.9995	.9993	.9990	.9928	0.9890	.9824

r is the linear regression coefficient between data pressures and theoretical pressures for BRET-MODEL-X.

Based on the six cyclones above, Typhoon Vera, Hurricane Iwa and Hurricane Camille, the following guide is given:

Hydromet RANKIN VORTEX MODEL	$0 \leq N_C \leq 0.05$
FUJITA MODEL (BRET MODEL, b = 1/2)	$.05 \leq N_C \leq 0.10$
BRET-MODEL-X (b = 1)	$.10 \leq N_C \leq .22$ or more

A transition from the Hydromet Rankin Vortex Model to Fujita and Bret-Models might be made by Hollands (1980) Modification of the Rankin Vortex Model.

The general form of Bret Model (using b < 1.0) can also be used as a transition toward the lower N_{R_C} numbers to meet Hollands (1980) method.

3. EXAMPLE VERIFICATION OF EQUATION 1

3.1 HURRICANE IWA (1982).

Based on the pressure profile data analysis for 0000 GMT, 24 Nov 1982 (2 p.m., 23 Nov 1982), it was found that

$$R_C = 85 \text{ n. miles} \quad (R = 79 \text{ n. miles})$$

$$P_R = 987 \text{ mbs at } R_C$$

$$\Delta P_O = \text{MAX}\left[\frac{rdp}{dr}\right] = 46 \text{ mbs}$$

$$\phi = 21.5°$$

$$N_{R_C} = \frac{.525\ R_C \sin 21.5°}{11.7\ \sqrt{\Delta P_O}} = 0.21$$

Thus BRET-MODEL-X applies and the central pressure is obtained from

$$P_O = P_{R_C} - \frac{1}{2}\Delta P_O = 964 \text{ mbs}$$

$$P_\infty = P_O + \Delta P_O = 1010 \text{ mbs}$$

RECON aircraft reported a measure minimum pressure estimated to be 964 mbs, one verification.

$$P_\infty = P_{R_C} + \frac{1}{2}\left(\text{MAX}\ \frac{rdP}{dr}\right) = 1010 \text{ mbs, another verification.}$$

The above information R_C, P_∞, and P_O are sufficient to determine the complete pressure profile from P_O = 964 mbs at r = 0 to P_∞ = 1010 for $r \to \infty$, where r is the radial distance measured from the center of the hurricane. A comparison is made between the theoretical pressure profile from BRET-MODEL-X hurricane and the recorded and weather chart isobar data for 6 p.m. HST 23 Nov 1982. The comparison shown in Fig. 2 shows a very high correlation between the data and the theoretical pressure profile.

Eq. 1 can be used to predict V_{MAX} for Hurricane Iwa, using the following measured parameters for 2 p.m. HST 23 Nov 1982:

Latitude ϕ = 21.5°

Radius of Maximum Cyclostrophic Wind, R_C = 85 n. miles

Central Pressure P_0 = 964 mbs

Terminal Pressure P_∞ = 1010 mbs

Forward Speed V_F = 19 knots

$\Delta P_0 = P_\infty - P_0 = \text{MAX}\left[r \frac{dp}{dr}\right]$ = 46 mbs

From Eq. 2, A = 14.894

From Eq. 3, $< \sqrt{\Delta P_0} >$ = 6.447 mbs $^{1/2}$

For BRET-MODEL-X, K = 13.6

Using Eq. 4 and V_F = 19 knots,

$1/2\ (V_F - fR) - 3 = -1.678$ knots.

Thus $V_{MAX} = 14.894\ (\sqrt{46} - 6.447) + 77.66 - 1.838 \pm 2.5 = 80.8$ knots $\pm$ 2.7 knots.

RECON aircraft measurements were estimated for V_{MAX} = 80 knots, which is to the closest $\pm$ 2.5 knots. Thus, the verification is excellent.

During the next three to six hours the hurricane traveled forward with increasing forward speed, from V_F = 19 knots reaching 32 knots at 8 p.m. HST Nov 23. The radius of maximum wind also decreases to about 50 nautical miles and the central pressure increases such that ΔP_0 became smaller. V_{MAX} = 80 knots remained the same during this time. Using the reverse of Equation 1, we can calculate the required ΔP_0 for these conditions, and we find the following for R_C = 52 ($R \cong 50$) n. miles and for V_F = 15, 20, 25 and 30 knots that ΔP_0 = 45.2, 42.7, 40.4 and 38.1 mbs respectively, and for ϕ = 22°50' the corresponding Rankin Vortex number ranged from N_{R_C} = 0.13 to 0.15. Thus the BRET-MODEL-X still applies. Fig. 3(a) shows the BRET MODEL pressure profile for ΔP_0 = 43 mbs. The corresponding wind profile is shown in Fig. 3(b).

At 8 p.m., 23 Nov 82, Hurricane Iwa probably exceeded its critical forward speed for maximum wave generation after which the waves will be lesser in height (See Bretschneider, 1972).

3.2 HURRICANE CAMILLE (1969).

Hurricane Camille (1969), Gulf of Mexico. Based on the parameters for Hurricane Camille (1969) given a report by Schwerdt, Ho and Watkins (1979), one finds the following:

Latitude ϕ = 30.3°

P_o = 90.8 KPa = 908 mbs

P_∞ = P_w = 100.8 KPa - 1008 mbs

$R_c \simeq R$ = 8 N. Miles

V_F = T (kt) = 16 knots

$V_{10} = V_x = .886 V_{MAX}$ (for Gulf of Mexico)

ΔP_o = 1008 - 908 = 100 mbs ($\sqrt{\Delta P_o}$ = 10 $mbs^{1/2}$)

From Eqs. 2 and 3 for P_∞ = 1008 mbs, calculate

A = 14.475, and

$< \sqrt{\Delta P_o} >$ = 6.272 $mbs^{1/2}$

Calculate

$1/2 (V_F - fR_c) - 3 = 1/2 (16 - 2.12) - 3 = 3.94$ knots

From Equation 1

$$V_{MAX} = \frac{14.475}{13.6} [13.6 (10 - 6.272) + 3.94 \pm 2.5] = 77.66 = 135.8 \text{ knots}$$

For the Gulf of Mexico $V_x = .886\ V_{MAX}$, whence

$V_{10} = .886\ V_{MAX} = 120.3 \pm 2.6$ knots

From Schwerdt, Ho, and Watkins, page 89, one finds

$V_x = V_{10} = 121$ knots

which is the same as V_{10} = 120.3 ± 2.6 = 121 ± 2.5 knots.

4. DETERMINATION AND VERIFICATION OF SURFACE WINDS

Once the model hurricane has been determined, then the next step is to use the gradient wind equation to determine the gradient winds and finally the surface wind. The surface winds are defined as the 10-meter elevation overwater winds, required for wave forecasting and storm surge calculation. A reduction coefficient is required such that V_{10} (max) = C_f V_{MAX}, which is applied at the true radius of maximum wind [R = R_g at V_g (max)]. All of this is taken into accont by using the gradient wind and correlating the gradient winds with measured or recorded surface winds. This is all done by theory, or better, by computer.

For Hurricane Iwa (1982), it has been determined that the BRET-MODEL-X hurricane is the best choice to begin with.

First, it is necessary to correct the recorded wind speeds at the various stations to the standard anemometer level of 10 meters (33 feet). For example, the anemometer heights above the ground for the following stations: Honolulu, h = 25 feet, Barbers Point h = 12 feet, Barking Sands h = 14 feet, and Lihue h = 20 feet. The overwater wind speeds at 10 meter height were determined by using the 1/7 power law and allowing a 5 percent correction for coastal frictional affects.

The results were correlated with the corrected measured wind speeds and it was determined that V_{10} = 0.78 V_g, where V_g is the gradient wind speed for the BRET-MODEL-X. Excellent agreement is then found for the corrected wind speeds at Honolulu, Barbers Point and Lihue. The predicted overwater wind speeds at 10 meters corrected to overland wind speeds for the above four stations are shown in Fig. 4. The corresponding pressure profile is given

in Fig. 3. The overwater wind speeds are about 5% greater than the corrected overland wind speeds. The differences between the BRET-MODEL-X overwater wind speeds and the corrected overland stations are due to the frictional effects over the coastline to the land stations, as well as the accuracy of the 1/7 power law.

V_{10}, the ten meter level overwater wind speed is required for the generation of hurricane waves, breaking wave heights and depths, wave set-up and wind-driven storm surge. The inverted barometer effect, storm surge component is determined from the pressure profile accompanying the wind model.

The BRET 2-D wave forecasting model coupled with the wind model determines the wave fields, including the two- and three-parameter wave spectrum of Bretschneider, all of which have been calibrated using data from Hurricane Eloise (1975) in the Gulf of Mexico.

The design hurricane was selected as a compromise between Hurricane Iwa (1982) and Hurricane Raymond (1983) in Hawaii. It is believed that this design hurricane is more realistic of the most probable 100-year hurricane for West Beach area.

Table I (also included in the General Summary) gives a summary of hurricane design criteria, including those previously determined for various locations in the State of Hawaii. Also included are the results of a previous study for the severe winter Kona Storm of Jan. 7-11, 1980 for the Sand Island area. The results of the calculation for the design hurricane are given in Table II. Hindcasts for Hurricane Iwa are given in Table III.

TABLE III

HURRICANE IWA, Vmax=80.8 KNOTS, Rc=54 N. MILES
VARIOUS FORWARD SPEED AND CENTRAL PRESSURE

Lat.= 21.5 deg.; Rc= 54 n.mi.; Pn= 1010 mbs; Po= 43.49 mbs; Vf= 0 knts

r N.M.	A DEG.	Hs FEET	X N.M.	Y N.M.	Hx FEET	Hy FEET	Uxs KNOTS	Uys KNOTS	Urs KNOTS	Tfo SEC.	Ts SEC.
10.0	240	10.9	-5.0	-8.7	10.9	0.0	20.2	0.0	20.2	13.4	11.7
20.0	290	24.9	6.8	-18.8	19.2	16.0	23.6	28.1	36.7	15.3	13.4
30.0	310	27.4	19.3	-23.0	20.3	18.5	16.3	44.7	47.6	13.9	12.2
40.0	310	28.9	25.7	-30.6	22.1	18.6	18.2	50.1	53.3	13.5	11.8
50.0	310	29.4	32.1	-38.3	23.2	18.1	18.8	51.7	55.0	13.4	11.8
60.0	310	29.3	38.6	-46.0	23.7	17.3	18.6	51.0	54.3	13.5	11.8
70.0	310	28.6	45.0	-53.6	23.6	16.2	17.8	49.0	52.1	13.6	11.9
80.0	310	26.7	51.4	-61.3	22.1	15.0	16.8	46.3	49.3	13.3	11.7
90.0	300	24.8	45.0	-77.9	21.2	12.9	23.0	39.9	46.1	13.0	11.4
100.0	300	23.2	50.0	-86.6	19.9	11.8	21.4	37.1	42.9	12.9	11.3
110.0	300	21.5	55.0	-95.3	18.6	10.8	19.9	34.4	39.7	12.8	11.2
120.0	300	19.9	60.0	-103.9	17.3	9.8	18.4	31.8	36.8	12.6	11.0
130.0	300	17.8	65.0	-112.6	15.3	8.9	17.0	29.4	34.0	12.0	10.6
140.0	300	15.4	70.0	-121.2	13.1	8.1	15.7	27.2	31.4	11.3	9.9
150.0	290	13.9	51.3	-141.0	12.2	6.7	18.6	22.2	28.9	10.9	9.6
160.0	290	12.6	54.7	-150.4	11.1	6.1	17.2	20.5	26.7	10.7	9.3
170.0	290	11.4	58.1	-159.7	10.0	5.4	15.9	18.9	24.7	10.4	9.1
180.0	290	10.2	61.6	-169.1	8.9	5.0	14.6	17.4	22.8	10.0	8.7
190.0	290	8.7	65.0	-178.5	7.6	4.4	13.5	16.1	21.0	9.3	8.1
200.0	290	7.6	68.4	-187.9	6.4	4.0	12.5	14.9	19.4	8.7	7.6
210.0	280	6.6	36.5	-206.8	5.8	3.1	13.7	11.5	17.9	8.2	7.2
220.0	280	5.9	38.2	-216.7	5.2	2.8	12.7	10.6	16.6	7.9	6.9
230.0	280	5.2	39.9	-226.5	4.6	2.5	11.7	9.8	15.3	7.6	6.7
240.0	280	4.6	41.7	-236.4	4.0	2.2	10.8	9.1	14.1	7.2	6.4

r N.M.	Urs KNOTS	Bu DEG	Hs FEET	Bh DEG	Ts SEC.	Hb FEET	dB FEET	Sw
10.0	20.2	0	10.9	0	11.7	13.2	16.9	.544
20.0	36.7	50	24.9	40	13.4	25.1	32.2	1.500
30.0	47.6	70	27.4	42	12.2	25.1	32.1	1.816
40.0	53.3	70	28.9	40	11.8	25.5	32.6	1.979
50.0	55.0	70	29.4	38	11.8	25.8	33.0	2.037
60.0	54.3	70	29.3	36	11.8	25.8	33.0	2.018
70.0	52.1	70	28.6	34	11.9	25.5	32.6	1.949
80.0	49.3	70	26.7	34	11.7	24.0	30.7	1.802
90.0	46.1	60	24.8	31	11.4	22.5	28.8	1.654
100.0	42.9	60	23.2	31	11.3	21.4	27.4	1.521
110.0	39.7	60	21.5	30	11.2	20.2	25.9	1.391
120.0	36.8	60	19.9	29	11.0	19.0	24.3	1.264
130.0	34.0	60	17.8	30	10.6	17.1	21.9	1.116
140.0	31.4	60	15.4	32	9.9	14.9	19.0	.963
150.0	28.9	50	13.9	29	9.6	13.6	17.4	.859
160.0	26.7	50	12.6	29	9.3	12.6	16.1	.768
170.0	24.7	50	11.4	28	9.1	11.5	14.7	.683
180.0	22.8	50	10.2	29	8.7	10.4	13.3	.602
190.0	21.0	50	8.7	30	8.1	9.0	11.5	.516
200.0	19.4	50	7.6	32	7.6	7.8	10.0	.445
210.0	17.9	40	6.6	28	7.2	6.8	8.8	.385
220.0	16.6	40	5.9	29	6.9	6.2	7.9	.339
230.0	15.3	40	5.2	29	6.7	5.6	7.1	.297
240.0	14.1	40	4.6	29	6.4	4.9	6.3	.258

TABLE III cont'd

Lat.= 22.83 deg.; Rc= 54 n.mi.; Pn= 1010 mbs; Po= 42.96 mbs; Vf= 20 knts

r N.M.	A DEG.	Hs FEET	X N.M.	Y N.M.	Hx FEET	Hy FEET	Uxs KNOTS	Uys KNOTS	Urs KNOTS	Tfo SEC.	Ts SEC.
10.0	330	25.5	8.7	-5.0	0.0	25.5	0.0	28.5	28.5	18.8	16.5
20.0	300	38.8	10.0	-17.3	17.5	34.6	18.1	41.0	44.8	19.4	17.0
30.0	310	40.2	19.3	-23.0	19.9	34.9	16.1	53.2	55.6	17.2	15.1
40.0	310	40.6	25.7	-30.6	21.7	34.4	18.0	58.4	61.2	16.3	14.3
50.0	310	41.0	32.1	-38.3	22.8	34.1	18.6	60.1	62.9	16.2	14.2
60.0	310	41.0	38.6	-46.0	23.2	33.8	18.3	59.3	62.1	16.3	14.3
70.0	310	40.5	45.0	-53.6	23.1	33.3	17.6	57.3	59.9	16.5	14.5
80.0	310	38.3	51.4	-61.3	21.4	31.7	16.6	54.6	57.0	16.2	14.2
90.0	300	35.8	45.0	-77.9	20.7	29.2	22.7	48.9	53.9	15.9	14.0
100.0	300	34.3	50.0	-86.6	19.5	28.2	21.1	46.1	50.7	16.0	14.0
110.0	300	32.8	55.0	-95.3	18.2	27.4	19.5	43.3	47.5	16.1	14.1
120.0	300	31.3	60.0	-103.9	16.9	26.4	18.0	40.8	44.6	16.1	14.2
130.0	300	28.6	65.0	-112.6	14.7	24.5	16.6	38.3	41.8	15.6	13.7
140.0	300	25.7	70.0	-121.2	12.5	22.4	15.3	36.1	39.2	14.9	13.1
150.0	290	23.4	51.3	-141.0	11.7	20.2	18.1	32.0	36.8	14.4	12.6
160.0	290	22.1	54.7	-150.4	10.6	19.4	16.7	30.2	34.5	14.4	12.6
170.0	290	20.8	58.1	-159.7	9.6	18.5	15.4	28.6	32.5	14.3	12.5
180.0	290	19.2	61.6	-169.1	8.3	17.3	14.2	27.1	30.6	14.0	12.3
190.0	290	17.4	65.0	-178.5	7.1	15.9	13.1	25.8	28.9	13.4	11.7
200.0	290	15.8	68.4	-187.9	6.0	14.7	12.0	24.5	27.3	12.9	11.3
210.0	10	14.7	206.8	36.5	-2.9	14.4	-11.1	23.3	25.8	12.7	11.1
220.0	10	13.9	216.7	38.2	-2.6	13.7	-10.2	22.2	24.5	12.7	11.2
230.0	10	13.1	226.5	39.9	-2.4	12.9	-9.4	21.2	23.2	12.8	11.2
240.0	10	12.3	236.4	41.7	-2.1	12.2	-8.7	20.3	22.1	12.8	11.2

r N.M.	Urs KNOTS	Bu DEG	Hs FEET	Bh DEG	Ts SEC.	Hb FEET	dB FEET	Sw
10.0	28.5	90	25.5	90	16.5	29.3	37.6	1.343
20.0	44.8	66	38.8	63	17.0	39.5	50.6	2.307
30.0	55.6	73	40.2	60	15.1	37.3	47.8	2.620
40.0	61.2	73	40.6	58	14.3	36.4	46.6	2.750
50.0	62.9	73	41.0	56	14.2	36.4	46.6	2.800
60.0	62.1	73	41.0	55	14.3	36.5	46.8	2.785
70.0	59.9	73	40.5	55	14.5	36.6	46.8	2.722
80.0	57.0	73	38.3	56	14.2	34.8	44.6	2.548
90.0	53.9	65	35.8	55	14.0	32.9	42.1	2.359
100.0	50.7	65	34.3	55	14.0	32.1	41.0	2.224
110.0	47.5	66	32.8	56	14.1	31.3	40.0	2.092
120.0	44.6	66	31.3	57	14.2	30.4	38.9	1.959
130.0	41.8	67	28.6	59	13.7	28.0	35.8	1.773
140.0	39.2	67	25.7	61	13.1	25.2	32.3	1.584
150.0	36.8	60	23.4	60	12.6	23.1	29.6	1.429
160.0	34.5	61	22.1	61	12.6	22.2	28.5	1.328
170.0	32.5	62	20.8	63	12.5	21.3	27.3	1.230
180.0	30.6	62	19.2	64	12.3	19.9	25.5	1.124
190.0	28.9	63	17.4	66	11.7	18.1	23.2	1.011
200.0	27.3	64	15.8	68	11.3	16.6	21.3	.915
210.0	25.8	115	14.7	101	11.1	15.7	20.0	.841
220.0	24.5	115	13.9	101	11.2	15.1	19.3	.779
230.0	23.2	114	13.1	100	11.2	14.5	18.6	.720
240.0	22.1	113	12.3	100	11.2	14.0	17.9	.661

TABLE III cont'd

Lat.= 22.83 deg.; Rc= 54 n.mi.; Pn= 1010 mbs; Po= 40.36 mbs; Vf= 25 knts

r N.M.	A DEG.	Hs FEET	X N.M.	Y N.M.	Hx FEET	Hy FEET	Uxs KNOTS	Uys KNOTS	Urs KNOTS	Tfo SEC.	Ts SEC.
10.0	330	28.7	8.7	-5.0	0.0	28.7	0.0	29.8	29.8	19.7	17.3
20.0	300	41.1	10.0	-17.3	16.1	37.8	17.4	42.0	45.4	20.2	17.7
30.0	310	41.9	19.3	-23.0	18.5	37.6	15.4	53.5	55.7	17.7	15.5
40.0	310	42.4	25.7	-30.6	20.5	37.2	17.2	58.6	61.0	16.9	14.8
50.0	310	42.7	32.1	-38.3	21.5	36.9	17.8	60.1	62.7	16.7	14.7
60.0	310	42.7	38.6	-46.0	21.9	36.6	17.5	59.4	61.9	16.9	14.8
70.0	310	42.4	45.0	-53.6	21.8	36.3	16.8	57.4	59.8	17.1	15.0
80.0	310	39.3	51.4	-61.3	19.6	34.1	15.9	54.8	57.0	16.6	14.6
90.0	300	37.6	45.0	-77.9	19.5	32.1	21.7	49.4	54.0	16.6	14.5
100.0	300	36.3	50.0	-86.6	18.3	31.3	20.1	46.7	50.9	16.7	14.6
110.0	300	34.9	55.0	-95.3	17.1	30.5	18.6	44.1	47.9	16.9	14.8
120.0	300	33.3	60.0	-103.9	15.7	29.4	17.2	41.6	45.0	16.9	14.8
130.0	300	30.2	65.0	-112.6	13.3	27.1	15.8	39.3	42.3	16.2	14.2
140.0	300	27.3	70.0	-121.2	11.3	24.9	14.6	37.1	39.8	15.5	13.6
150.0	290	25.4	51.3	-141.0	10.9	23.0	17.3	33.3	37.5	15.3	13.4
160.0	290	24.3	54.7	-150.4	9.9	22.2	15.9	31.6	35.4	15.3	13.4
170.0	290	23.0	58.1	-159.7	8.8	21.2	14.6	30.1	33.4	15.3	13.4
180.0	290	21.1	61.6	-169.1	7.5	19.7	13.5	28.6	31.6	14.9	13.0
190.0	290	19.3	65.0	-178.5	6.3	18.2	12.4	27.3	29.9	14.3	12.6
200.0	10	17.9	197.0	34.7	-3.1	17.7	-11.4	26.0	28.4	14.0	12.3
210.0	10	17.0	206.8	36.5	-2.7	16.8	-10.5	24.9	27.0	14.0	12.3
220.0	10	16.2	216.7	38.2	-2.4	16.0	-9.7	23.8	25.7	14.2	12.5
230.0	10	15.4	226.5	39.9	-2.2	15.3	-8.9	22.9	24.5	14.5	12.7
240.0	10	14.5	236.4	41.7	-1.9	14.3	-8.2	22.0	23.5	14.6	12.8

r N.M.	Urs KNOTS	Bu DEG	Hs FEET	Bh DEG	Ts SEC.	Hb FEET	dB FEET	Sw
10.0	29.8	90	28.7	90	17.3	32.7	41.9	1.528
20.0	45.4	68	41.1	67	17.7	42.2	54.0	2.422
30.0	55.7	74	41.9	64	15.5	39.2	50.2	2.711
40.0	61.0	74	42.4	61	14.8	38.4	49.1	2.844
50.0	62.7	74	42.7	60	14.7	38.2	49.0	2.889
60.0	61.9	74	42.7	59	14.8	38.4	49.2	2.875
70.0	59.8	74	42.4	59	15.0	38.6	49.4	2.818
80.0	57.0	74	39.3	60	14.6	36.0	46.1	2.603
90.0	54.0	66	37.6	59	14.5	34.9	44.6	2.456
100.0	50.9	67	36.3	60	14.6	34.2	43.8	2.328
110.0	47.9	67	34.9	61	14.8	33.6	43.0	2.202
120.0	45.0	68	33.3	62	14.8	32.6	41.8	2.065
130.0	42.3	68	30.2	64	14.2	29.7	38.0	1.858
140.0	39.8	69	27.3	66	13.6	27.0	34.6	1.674
150.0	37.5	63	25.4	65	13.4	25.5	32.6	1.540
160.0	35.4	63	24.3	66	13.4	24.7	31.7	1.442
170.0	33.4	64	23.0	67	13.4	23.9	30.6	1.343
180.0	31.6	65	21.1	69	13.0	22.1	28.2	1.222
190.0	29.9	66	19.3	71	12.6	20.3	25.9	1.110
200.0	28.4	114	17.9	100	12.3	19.1	24.4	1.021
210.0	27.0	113	17.0	99	12.3	18.4	23.5	.949
220.0	25.7	112	16.2	99	12.5	18.0	23.0	.884
230.0	24.5	111	15.4	98	12.7	17.7	22.6	.818
240.0	23.5	110	14.5	97	12.8	17.0	21.7	.747

TABLE III cont'd

Lat.= 22.83 deg.; Rc= 54 n.mi.; Pn= 1010 mbs; Po= 37.84 mbs; Vf= 30 knts

r N.M.	A DEG.	Hs FEET	X N.M.	Y N.M.	Hx FEET	Hy FEET	Uxs KNOTS	Uys KNOTS	Urs KNOTS	Tfo SEC.	Ts SEC.
10.0	330	32.1	8.7	-5.0	0.0	32.1	0.0	31.1	31.1	20.5	18.0
20.0	0	44.0	20.0	0.0	-12.3	42.3	-16.6	42.9	46.0	21.3	18.7
30.0	310	43.5	19.3	-23.0	16.9	40.1	14.7	53.9	55.9	18.3	16.0
40.0	310	44.4	25.7	-30.6	19.3	40.0	16.5	58.7	60.9	17.6	15.4
50.0	310	44.5	32.1	-38.3	20.3	39.7	17.0	60.1	62.5	17.4	15.2
60.0	310	44.6	38.6	-46.0	20.7	39.5	16.7	59.4	61.7	17.5	15.3
70.0	310	43.6	45.0	-53.6	20.0	38.7	16.0	57.5	59.7	17.5	15.4
80.0	300	40.7	40.0	-69.3	19.3	35.8	22.1	52.6	57.0	17.1	15.0
90.0	300	39.6	45.0	-77.9	18.3	35.1	20.6	50.0	54.1	17.3	15.1
100.0	300	38.4	50.0	-86.6	17.2	34.3	19.1	47.4	51.1	17.5	15.3
110.0	300	37.3	55.0	-95.3	16.0	33.7	17.7	44.8	48.2	17.7	15.5
120.0	300	34.9	60.0	-103.9	14.2	31.9	16.3	42.4	45.5	17.5	15.3
130.0	300	31.9	65.0	-112.6	12.0	29.5	15.0	40.2	42.9	16.8	14.8
140.0	300	29.1	70.0	-121.2	10.2	27.3	13.8	38.1	40.5	16.2	14.2
150.0	290	27.8	51.3	-141.0	10.1	25.9	16.4	34.6	38.3	16.3	14.3
160.0	290	26.7	54.7	-150.4	9.1	25.1	15.0	33.0	36.2	16.4	14.4
170.0	290	25.0	58.1	-159.7	7.9	23.7	13.8	31.4	34.3	16.2	14.2
180.0	290	23.2	61.6	-169.1	6.7	22.2	12.7	30.0	32.6	15.8	13.9
190.0	290	21.4	65.0	-178.5	5.7	20.6	11.7	28.7	31.0	15.4	13.5
200.0	10	20.4	197.0	34.7	-2.8	20.2	-10.8	27.5	29.6	15.4	13.5
210.0	10	19.5	206.8	36.5	-2.5	19.3	-9.9	26.4	28.2	15.6	13.7
220.0	10	18.8	216.7	38.2	-2.2	18.6	-9.1	25.4	27.0	16.1	14.2
230.0	10	18.0	226.5	39.9	-2.0	17.9	-8.4	24.5	25.9	17.1	15.0
240.0	10	16.9	236.4	41.7	-1.7	16.8	-7.7	23.6	24.8	16.4	14.4

r N.M.	Urs KNOTS	Bu DEG	Hs FEET	Bh DEG	Ts SEC.	Hb FEET	dB FEET	Sw
10.0	31.1	90	32.1	90	18.0	36.3	46.4	1.724
20.0	46.0	111	44.0	106	18.7	45.8	58.6	2.562
30.0	55.9	75	43.5	67	16.0	41.0	52.5	2.794
40.0	60.9	74	44.4	64	15.4	40.5	51.8	2.945
50.0	62.5	74	44.5	63	15.2	40.3	51.6	2.984
60.0	61.7	74	44.6	62	15.3	40.5	51.9	2.972
70.0	59.7	74	43.6	63	15.4	40.0	51.2	2.878
80.0	57.0	67	40.7	62	15.0	37.5	48.0	2.674
90.0	54.1	68	39.6	62	15.1	37.1	47.5	2.561
100.0	51.1	68	38.4	63	15.3	36.6	46.9	2.440
110.0	48.2	68	37.3	65	15.5	36.2	46.4	2.321
120.0	45.5	69	34.9	66	15.3	34.4	44.0	2.149
130.0	42.9	69	31.9	68	14.8	31.6	40.4	1.949
140.0	40.5	70	29.1	70	14.2	29.0	37.1	1.770
150.0	38.3	65	27.8	69	14.3	28.2	36.1	1.661
160.0	36.2	65	26.7	70	14.4	27.6	35.4	1.565
170.0	34.3	66	25.0	72	14.2	26.2	33.5	1.447
180.0	32.6	67	23.2	73	13.9	24.5	31.4	1.329
190.0	31.0	68	21.4	75	13.5	22.8	29.1	1.217
200.0	29.6	111	20.4	98	13.5	22.2	28.4	1.140
210.0	28.2	111	19.5	97	13.7	21.6	27.7	1.064
220.0	27.0	110	18.8	97	14.2	21.6	27.6	.990
230.0	25.9	109	18.0	96	15.0	21.8	27.9	.901
240.0	24.8	108	16.9	96	14.4	20.3	26.0	.849

5. DISCUSSION ON POTENTIAL WAVE RUN-UP

The tsunami wave run-up was considered in Section I of this report. In this section we discussed wave run-up for hurricane conditions associated with a barometric pressure jump and the passage of a squall line. Wave run-up of the complex wave systems recorded off West Beach are analyzed and discussed in Section III.

Hurricane Iwa (1983) is used as an example to discuss wave run-up. A typical bottom profile, off West Beach, is shown in Fig. 5. The total tide above MLLW, including storm tide and wave set-up is taken as 3.2 feet above MSL or 4.0 feet above MLLW.

The calculations are more of an example for demonstration purposes, for which wave run-up is maximized. The results do not necessarily represent design elevations at any particular location.

The calculations are made for all breaking waves governed by T = 14.5 seconds and corresponding breaking depth, not exceeding d_B = 35 to 40 feet.

The breaking wave height and depth are calculated using the criteria for bottom of constant slope as given by Bretschneider (1960). The results will be closer to the Miche (1944) criteria than to the Solitary Wave Theory given by Munk (1949).

The equations are as follow:

1) $H_B/L_b = 1/7 \tanh \frac{2\pi d_B}{L_B}$

2) $L_B = 1.21\, L_A$, and

3) $L_A = Lo \tanh \frac{2\pi d_B}{L_A}$

where

H_B = breaking wave height

d_B = water depth at breaking

L_B = wave length at breaking

Lo = $gT^2/2\pi$. deep water linear wave theory wave length

L_A = linear wave theory wave length

The factor 1.21 given in Equation 2 is within less than 1% satisfying Bernoulli's equation, considering all the other parameters (See Bretschneider, 1960). Given a particular depth d_B and T = 14.5 seconds, calculations can be made for: L_A from Eq. 3, L_B from Eq. 2, and H_B from Eq. 1.

Wave run-up for breaking waves can be calculated from Hunt's (1959) formula:

4) $R = \sqrt{L_o H_B}\,/S \qquad (RS = \sqrt{L_o H_B})$

where

R = run-up elevation

S = cot α = horizontal distance ÷ vertical distance

The composite slope method of Saville (1958) is used. An increase correction of 25% for concave slope, based on model experiments at Delft (See Schijf, 1972), is applied to the run-up calculations.

All formulae and methods given above are based on smooth slope, and no corrections are made for roughness. Corrections for roughness is indeed a different task and is not required for the present demonstrations. Model tests and other roughness considerations for rip rap protection are required.

The results of the calculations are shown in Fig. 5. It is believed that Fig. 5 represents the maximum run-up under the above assumptions and selected wave period. Similar calculations were made for the Waianae Coasts for Hurricane Iwa (1982). It was reported by the U.S. Army Corps of

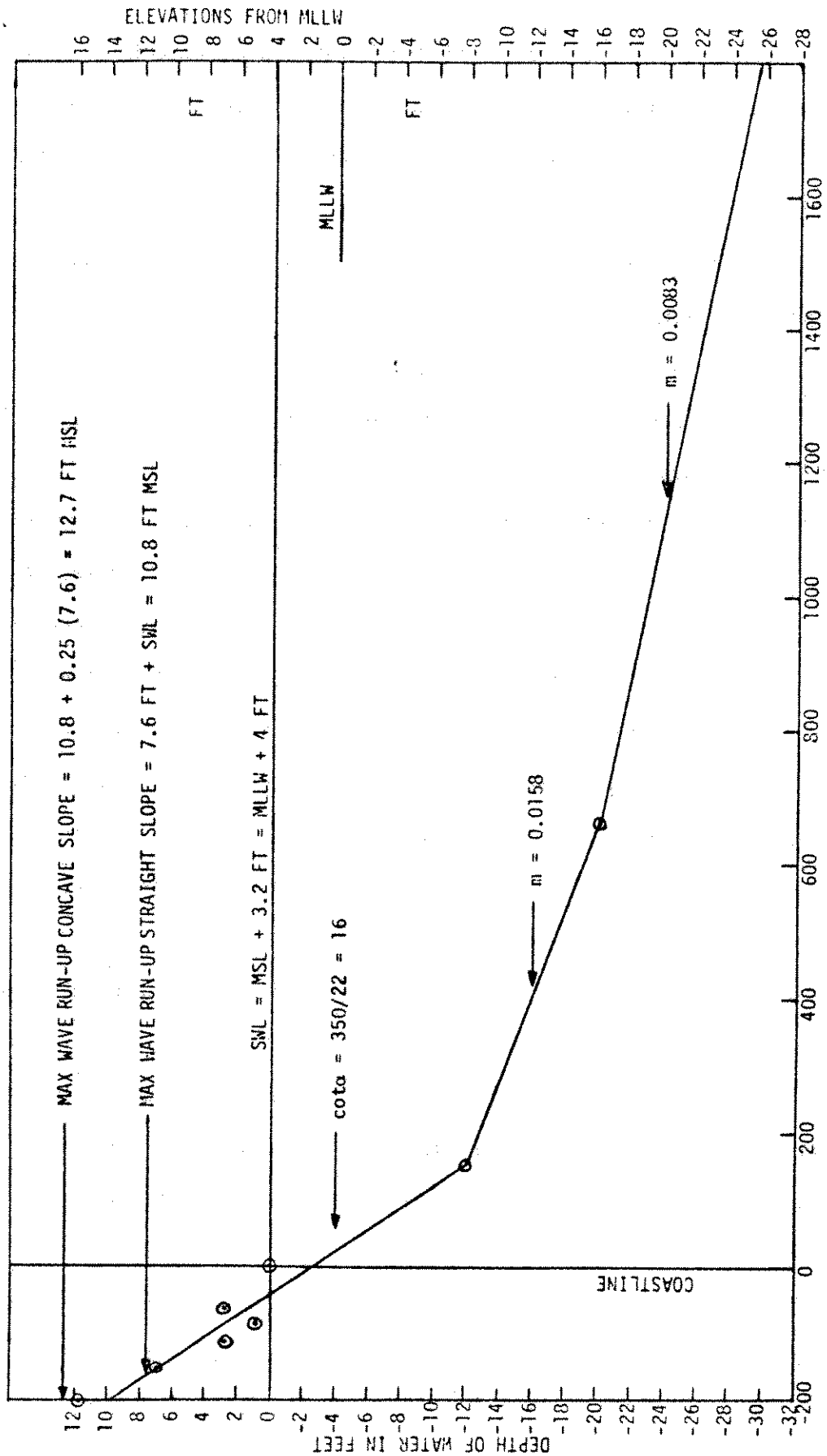


Figure 5. WAVE RUN-UP FOR T = 14.5 SEC. FOR TYPICAL BOTTOM PROFILE OFF WEST BEACH

Engineers (1983) that water levels at the Waianae area reached 17 and 19 feet above mean sea level 200 feet inland from the coastline. The run-up calculations were about 6 feet below the 17 and 19 feet elevations reported by the Corps of Engineers (1983).

There was something quite different associated with Hurricane Iwa that caused the excessive run-up, and which could be associated with any future hurricane in Hawaii. That is, a squall line passed over Hawaii during the peak of the storm. There was a pressure jump of 3.2 mbs at Honolulu (See STORM DATA, Nov. 1982, NOAA, pp 4-8).

Similar pressure jumps have been associated with squalls. See Abraham (1961, ch. 31) where a number of squall events are summarized. Abraham (1961) conducted model studies to show that this was a resonance phenomenon. For example, in 1954 a squall line passed over Southern Lake Michigan, having a pressure jump of 0.1 foot water head (equivalent to 3.0 mbs) was followed by a wave which, when a velocity of $\sqrt{gd}$ was reached, large waves 5 to 6 feet high arrived at Michigan City, and when they struck the Chicago breakwater with surges 3 to 8 feet, it caused the drowning of 7 persons (See Weigel, 1964, page 109).

Now returning to Hurricane Iwa, there would have had to be a surge, say 2 feet for a bore type wave Froude number $F = 2$, $V = 2\sqrt{gh}$ to have been associated with the squall line to have caused the water to rise 6 feet above the land elevations, for example, if it had struck a vertical structure; or with $F = 1$, say a surge of about 3 to 3.5 feet for $F = 1$ to do the same.

In case of the Waianae Coast, eyewitnesses saw such types of waves. In one case a driver and a car was engulfed by the wave and pushed off the road. Many large stones of coral were brought up from the ocean side of

FROM Eq. 1

$$\frac{T_{hx}}{T_{1/3}} = 0.74930\ \sqrt{1.78}\ \left[1+.0218\ \frac{32.16(12.04)^2}{10^3(34.7)}\right]\ [1\pm0.13]$$

$$\frac{T_{hx}}{T_{1/3}} = 1.0026\ [1\pm0.13];\ T_{1/3} = 12.04 \text{ sec., and}$$

$$T_h = 12.07 \pm 1.57 \text{ sec.};\ 10.5 \leq T_n \leq 13.6 \text{ sec.}$$

FROM Eq. 2 for T_p =

$$\frac{T_{hx}}{T_p} = .6518\ \sqrt{1.78}\ \left[1 + 0.0320\ \frac{32.16\ (12.04)^2}{10^3(34.7)}\right]\ [1\pm0.18]$$

$$\frac{T_{hx}}{T_p} = 0.8734\ [1\pm0.18];\ T_p = 13.8 \text{ sec.}$$

$$T_{hx} = 12.05 \pm 2.17 \text{ sec.};\ 9.9 \leq T_{hx} \leq 14.2 \text{ sec.}$$

3. WAVE RUN-UP FOR COMPLEX WAVE TRAINS

A problem of concern is the possibility of greater wave run-up for the complex wave trains consisting of maximum short period wave heights superimposed on the long period swells. Standard methods and techniques are not available for calculating wave run-up for such complex wave trains as obtained off West Beach area.

From the data records it appears that long period swells are very frequent, 20 to 22 seconds or more. Such long period swells originates from the Southern Hemisphere winter storms, from early June through late August or September. 14 to 16 seconds swells arrive from the Northern Hemisphere winter months, early October through April or May.

Local wind waves from the trades can be superimposed on the swell. Kona storms during the winter months in Hawaii would generate waves upon the swells from the South or Southwesterly directions. Tropical storms during the summer months would generate large waves superimposed on the swells from the Southern Hemisphere. The combination of a summer time hurricane waves in Hawaiian waters and southern swells from a severe Southern Hemisphere winter storm would represent a situation to be considered. The combination of winter Kona storm waves with winter swells from the North or Northwest should also be considered. From the data analysis this would seem to be the case, that the wave will break at greater heights, and therefore the corresponding run-up would also be greater.

Probability distributions for either a narrow spectrum or a broader spectrum would not solve the problem of wave run-up, this type of superposition or combinations would result in a double peaked spectrum, wherein the two types of waves would be separated into two independent spectra. It would seem that the only way to really solve the problem is by conducting hydraulic model tests for the above type wave combinations.

4. AN ANALYTICAL POSSIBILITY FOR WAVE RUN-UP OF COMPLEX WAVE TRAINS

It is not known what the directional affects of the complex wave trains are. It will be assumed that the short period maximum waves ride with the long period swells. An analytical possibility is suggested by considering the relative run-ups of $R^*/R_{1/3}$ between the data given in Tables I-A-1 and I-B-1; and also between $R^*/R_{1/3}$ given in Tables II-A-1 and II-B-1. A summary of the overall weighted averages for $R^*/R_{1/3}$ are given below:

TABLE I-A-1 $R^*/R_{1/3} = 1.2781$ N = 421

TABLE I-B-1 $R^*/R_{1/3} = 1.5028$ N = 402

The ratios of the above two numbers: $1.5028 \div 1.278 = 1.176$.

TABLE II-A-1 (For $T \geq 9$ sec.)

$R^*/R_{1/3} = 1.2833$ N = 380

TABLE II-B-1 $R^*/R_{1/3} = 1.4866$ N = 328

The ratios of the above two numbers: $1.4866 \div 1.2833 = 1.158$.

The above ratios are slightly different because of the different methods of grouping and selections of the corresponding data. The overall ratio is about 1.17.

It is suggested that wave run-up based on standard methods of calculations be increased by about 17 percent to account for the fact that short period maximum waves can be superimposed on the long period swells. This would include the breaking wave heights appropriately selected or determined for the long period swell. The above 17 percent is in addition to other corrections to be made, such as corrections for concave slopes, roughness corrections, and model to prototype scale effects.

TABLE 3

SYMBOLS USED FOR STATISTICAL STUDY

N -	Number of Data Averages
< >	Mean Value
S.D.	Standard Deviation from Mean
$H_{1/3}$	Significant Wave Height, Ave. Highest one-third
T_p	Modal Period of Frequency Spectrum $T_p = f_o^{-1}$
f_o	Frequency at Max Peak of Spectrum
$T_{1/3}$	Average Period of Significant Waves
T_z	Average Period of Zero Crossing
H_x	Maximum Wave Height of Wave Record
T_{hx}	Period of Maximum Wave Height
T_x	Maximum Wave Period of Wave Record
H_{tx}	Wave Height of Max. Wave Period
$SR_{1/3}$	$S^{-1}T_{1/3}\sqrt{H_s}$ Related to Run-Up (Hunt's Formula)
S	is Slope for Wave Run-Up ($S = \cot \alpha$)
SR_x	$S^{-1}T_{hx}\sqrt{H_x}$ Related to Run-Up for H_x
SR_{tx}	$S^{-1}T_x\sqrt{H_{xt}}$ Related to Run-Up for T_x
$\frac{R^*}{R_{1/3}}$	$\sqrt{\frac{H_x}{H_{1/3}}}$ Relative Run-Up (See Text)
$\frac{R_x}{R_{1/3}}$	$\frac{T_{hx}\sqrt{H_x}}{T_{1/3}\sqrt{H_{1/3}}}$ Relative Run-Up Max Wave Height
$\frac{R_t}{R_{1/3}}$	$\frac{T_x\sqrt{H_{tx}}}{T_{1/3}\sqrt{H_x}}$ Relative Run-Up Max Wave Period

$S = \cot \alpha =$ horizontal distance ÷ vertical distance

TABLE I-A-1 STATISTICAL ANALYSIS OF RECORDED WAVE DATA (GROUP METHOD I)

SEA. NO.	N S.D.	$\overline{T}_{hx}$ sec	$\overline{H}_x$ cm	$\overline{T}_p$ sec	$\overline{H}_{1/3}$ cm	$\overline{T}_{1/3}$ sec	$\overline{T}_z$ sec	$\overline{T}_x$ sec	$\overline{H}_{tx}$ cm	$SR_{1/3}$ sec cm 1/2	$\langle \frac{R_x}{R_{1/3}} \rangle$	$\langle \frac{R_{tx}}{R_{1/3}} \rangle$	$\langle \frac{R^*}{R_{1/3}} \rangle$
#1	8 S.D.	11.8 1.0	75 26	12.9 2.4	44 11	13.4 1.2	11.3 0.6	19.0 1.5	14 5	75 15	1.345 .106	1.032 .350	1.299 .080
#2	17 S.D.	12.0 2.4	72 16	13.5 2.3	45 10	12.0 1.8	11.8 1.4	21.1 3.1	19 8	81 18	1.259 .183	1.146 .229	1.258 .056
#3	30 S.D.	13.2 0.9	92 22	13.5 1.3	56 12	12.4 0.6	12.0 0.6	20.4 2.7	26 12	92 12	1.367 .126	1.115 .312	1.282 .052
#3	30 S.D.	11.7 0.5	82 18	13.0 1.5	50 9	11.8 0.8	11.7 0.6	20.5 1.7	20 7	83 11	1.280 .098	1.105 .258	1.287 .080
#4	30 S.D.	11.5 0.5	103 29	12.5 2.0	62 17	11.3 1.0	11.3 0.7	20.5 3.2	29 11	89 17	1.312 .139	1.226 .261	1.291 .067
#4	24 S.D.	13.0 0	104 34	13.2 1.3	61 18	12.2 0.9	12.0 0.7	21.4 2.7	23 9	95 18	1.393 .133	1.068 .190	1.303 .074
#4	21 S.D.	15.3 2.0	123 35	15.7 2.8	79 24	14.7 2.3	13.9 1.8	22.1 1.9	37 15	131 39	1.316 .117	1.036 .182	1.254 .066
#4	11* S.D.	14.0 0	109 31	14.4 1.6	69 22	13.5 1.0	13.0 1.0	21.2 1.7	34 19	112 25	1.320 .112	1.073 .140	1.260 .047
#5	20 S.D.	11.0 0.9	101 32	12.8 2.4	60 17	11.4 1.2	11.4 0.9	20.7 2.4	25 9	88 14	1.252 .118	1.160 .213	1.289 .046
#5	8* S.D.	10.0 0	109 42	11.5 2.6	63 22	10.6 0.8	10.8 0.7	20.1 1.6	28 10	83 12	1.246 .109	1.252 .158	1.310 .051
#5	8* S.D.	12.0 0	88 16	14.1 2.3	53 9	12.2 0.9	11.9 0.7	20.5 2.5	21 8	88 12	1.271 .118	1.031 .190	1.283 .051
#5	25 S.D.	13.6 0.9	114 47	14.7 1.8	73 28	13.1 1.2	12.8 1.0	21.6 2.5	29 12	111 28	1.305 .115	1.052 .198	1.247 .074
#5	15* S.D.	13.0 0	106 39	14.1 1.4	66 22	12.5 0.8	12.3 0.6	21.5 2.9	26 10	101 19	1.318 .089	1.071 .211	1.266 .053
#6	24 S.D.	10.0 1.1	93 36	11.7 2.2	55 14	10.4 1.2	10.7 0.9	19.7 2.0	25 11	77 17	1.237 .148	1.276 .358	1.266 .048
#6	14* S.D.	9.2 0.8	81 19	11.3 2.6	50 9	9.9 1.2	10.4 0.8	20.1 1.8	21 10	71 14	1.184 .134	1.313 .334	1.287 .079
#6	10* S.D.	11.0 0	110 47	12.1 1.4	61 17	11.3 0.9	11.2 0.7	19.0 2.2	30 10	87 17	1.310 .142	1.224 .403	1.318 .104
#6	15 S.D.	12.0 0	119 47	12.9 1.3	71 23	11.2 0.8	11.2 0.6	18.7 2.2	32 14	93 22	1.390 .103	1.121 .263	1.283 .066
#6	14 S.D.	13.0 0	173 63	14.3 0.7	102 33	13.3 0.8	12.8 0.8	20.3 1.7	40 11	134 28	1.274 .145	.960 .106	1.301 .095
#6	17 S.D.	14.0 0	138 57	15.1 1.4	83 32	13.2 1.0	12.7 0.8	21.8 2.5	38 22	119 29	1.370 .111	1.066 .294	1.284 .064
#6	12 S.D.	15.6 0.8	182 60	16.3 0.9	117 30	15.4 0.8	14.5 0.5	22.0 1.5	47 27	174 23	1.209 .065	.792 .150	1.262 .076
#6	7* S.D.	15.0 0	196 69	15.9 0.7	118 34	15.2 0.6	14.5 0.5	22.6 1.7	37 19	164 25	1.266 .102	.804 .113	1.280 .084
#7	19 S.D.	12.2 2.3	209 70	14.0 2.2	130 44	12.3 2.1	12.0 1.5	19.7 2.2	47 19	144 51	1.247 .131	.958 .178	1.276 .065
#8	21 S.D.	10.6 1.9	140 57	12.1 1.7	89 38	10.8 0.9	11.0 0.8	19.8 2.2	32 24	98 27	1.274 .191	1.102 .265	1.261 .068
#8	5 S.D.	8.2 1.1	124 52	11.6 1.8	83 36	10.8 1.0	11.0 0.8	19.6 2.1	30 18	98 27	.936 .223	1.042 .174	1.227 .045
#9	16 S.D.	12.6 1.4	124 24	13.4 1.5	76 17	12.6 1.1	12.5 1.0	21.4 5.2	29 14	110 17	1.279 .078	.990 .233	1.281 .045

* Also included in previous set

TABLE I-A-2 STATISTICAL ANALYSIS OF RECORDED WAVE DATA (GROUP METHOD I)

SEA. NO.	N S.D.	$\bar{T}_{hx}$ sec	$\bar{H}_x$ cm	$\bar{H}_{1/3}$ cm	$\bar{T}_{1/3}$ cm	$\langle \frac{T_{hx}}{T_p} \rangle$	$\langle \frac{T_p}{T_{1/3}} \rangle$	$\langle \frac{T_z}{T_{1/3}} \rangle$	$\langle \frac{H_x}{H_{1/3}} \rangle$	$\langle \frac{T_{hx}}{T_{1/3}} \rangle$	$\langle \frac{T_x}{T_{1/3}} \rangle$	$\langle \frac{H_{tx}}{H_{1/3}} \rangle$
#1	8	11.8	75	44	13.4	.946	1.122	.997	1.693	1.037	1.711	.330
	S.D.	1.0	26	11	1.2	.203	.143	.062	.210	.071	.241	.129
#2	17	12.0	72	45	12.0	.888	1.132	.990	1.573	1.000	1.785	.434
	S.D.	2.4	16	10	1.8	.127	.113	.047	.144	.130	.324	.152
#3	30	13.2	92	56	12.4	.984	1.090	.972	1.645	1.065	1.650	.469
	S.D.	0.9	22	12	0.6	.102	.092	.026	.135	.093	.246	.192
#3	30	11.7	82	50	11.8	.913	1.097	.997	1.662	.995	1.747	.410
	S.D.	0.5	18	9	0.8	.087	.096	.037	.209	.054	.170	.181
#4	30	11.5	103	62	11.3	.947	1.106	1.004	1.671	1.025	1.825	.470
	S.D.	0.5	29	17	1.0	.165	.149	.035	.172	.072	.262	.154
#4	24	13.0	104	61	12.2	.996	1.043	.977	1.704	1.069	1.763	.378
	S.D.	0	34	18	0.9	.094	.232	.031	.198	.083	.277	.105
#4	21	15.3	123	79	14.7	.989	1.068	.946	1.578	1.049	1.520	.471
	S.D.	2.0	35	24	2.3	.107	.094	.041	.165	.078	.177	.136
#4	11*	14.0	109	69	13.5	.988	1.068	.965	1.590	1.047	1.579	.474
	S.D.	0	31	22	1.0	.116	.110	.034	.117	.079	.121	.147
#5	20	11.0	101	60	11.4	.887	1.123	1.002	1.662	.972	1.824	.413
	S.D.	0.9	32	17	1.2	.180	.171	.0361	.118	.085	.204	.118
#5	8*	10.0	109	63	10.6	.917	1.023	1.711	.951	1.908	.439	1.246
	S.D.	0	42	22	0.8	.258	.026	.126	.078	.180	.112	.109
#5	8*	12.0	88	53	12.2	.870	1.155	.974	1.654	.990	1.686	.386
	S.D.	0	16	9	0.9	.133	.137	.026	.110	.077	.200	.128
#5	25	13.6	114	73	13.1	.934	1.126	.973	1.559	1.047	1.661	.413
	S.D.	0.9	47	28	1.2	.097	.103	.034	.177	.070	.217	.129
#5	15*	13.0	106	66	12.5	.937	1.123	.983	1.604	1.042	1.724	.400
	S.D.	0	39	22	0.8	.100	.099	.024	.134	.061	.234	.137
#6	24	10.0	93	55	10.4	.878	1.116	1.035	1.664	.960	1.879	.466
	S.D.	1.1	36	14	1.2	.161	.148	.051	.215	.098	.370	.205
(#6)	14*	9.2	81	50	9.9	.849	1.135	1.054	1.604	.936	2.001	.436
	S.D.	0.8	19	9	1.2	.184	.176	.053	.123	.105	.383	.217
#6	10*	11.0	110	61	11.3	.994	1.090	1.008	1.747	.994	1.720	.508
	S.D.	0	47	17	0.9	.079	.103	.034	.289	.079	.277	.193
#6	15	12.0	119	71	11.2	.940	1.158	1.002	1.651	1.077	1.681	.451
	S.D.	0	47	23	0.8	.112	.111	.028	.172	.061	.180	.153
#6	14	13.0	173	102	13.3	.909	1.077	.959	1.700	.978	1.523	.403
	S.D.	0	63	33	0.8	.043	.071	.028	.249	.060	.123	.086
#6	17	14.0	138	83	13.2	.932	1.154	.965	1.653	1.067	1.600	.437
	S.D.	0	57	32	1.0	.080	.142	.020	.163	.081	.324	.139
#6	12	15.6	182	117	15.4	.959	1.054	.943	1.599	1.011	1.427	.389
	S.D.	0.8	60	30	0.8	.047	.038	.040	.193	.060	.107	.191
#6	7*	15.0	196	118	15.2	.947	1.046	.957	1.645	.989	1.486	.301
	S.D.	0	69	34	0.6	.041	.040	.031	.216	.039	.087	.086
#7	19	12.2	209	130	12.3	.874	1.142	.978	1.629	.985	1.609	.360
	S.D.	2.3	70	44	12.1	.116	.093	.049	.171	.089	.184	.068
#8	21	10.6	140	89	10.8	.895	1.114	.923	1.594	.989	1.840	.349
	S.D.	1.9	57	38	0.9	.166	.111	.092	.174	.166	.243	.130
(#8)	5	8.2	124	83	10.8	.722	1.081	1.026	1.508	.772	1.832	.335
	S.D.	1.1	52	36	1.0	.175	.134	.027	.113	.159	.233	.094
#9	16	12.6	124	76	12.6	.948	1.058	.988	1.644	.998	1.644	.395
	S.D.	1.4	24	17	1.1	.076	.085	.020	.115	.055	.388	.207

* Also included in previous set

(#6) and (#8) excluded from Table 1, used to develop the prediction equations ($\bar{T}_{hx}$ < 9.2 seconds)

TABLE I-B-1 STATISTICAL ANALYSIS OF RECORDED WAVE DATA (GROUP METHOD I)

SEA. NO.	N S.D.	$\overline{T}_{hx}$ sec	$\overline{H}_x$ cm	$\overline{T}_p$ sec	$\overline{H}_{1/3}$ cm	$\overline{T}_{1/3}$ sec	$\overline{T}_z$ sec	$\overline{T}_x$ sec	$\overline{H}_{tx}$ cm	$S R_{1/3}$ sec cm 1/2	$\langle \frac{R_x}{R_{1/3}} \rangle$	$\langle \frac{R_{tx}}{R_{1/3}} \rangle$	$\langle \frac{R^*}{R_{1/3}} \rangle$
#1	30	3.97	97	14.0	43	10.5	11.0	21.0	17	78	.507	1.135	1.484
	S.D.	0.57	35	2.1	9	1.2	0.7		5	21	.120	.250	.214
#2	30	4.03	134	8.6	53	7.9	9.0	17.7	18	57	.802	1.331	1.594
	S.D.	0.89	51	2.9	11	0.9	0.8	2.0	6	6	.184	.225	.275
#2	27	4.1	107	14.0	43	10.5	11.0	21.1	18	69	.622	1.309	1.582
	S.D.	0.7	30	3.3	7	1.6	1.1	3.1	7	16	.140	.295	.202
#3	26	4	123	12.7	50	10.4	10.9	20.1	23	74	.601	1.289	1.564
	S.D.	0	46	1.6	14	1.5	1.0	3.1	12	19	.175	.371	.251
#3	24	4	121	11.7	53	9.9	10.6	19.5	22	72	.632	1.242	1.495
	S.D.	0	36	2.7	9	1.6	0.9	2.0	12	13	.172	.283	.168
#4	30	4	119	12.2	52	9.7	10.3	19.6	21	70	.631	1.217	1.511
	S.D.	0	38	2.6	12	1.1	0.8	3.1	10	12	.134	.178	.217
#4	30	4	105	12.6	51	11.0	11.3	20.6	23	79	.521	1.223	1.418
	S.D.	0	27	2.1	8	1.0	0.8	2.9	10	12	.077	.269	.148
#5	30	4	117	13.6	57	11.3	11.4	19.9	26	85	.520	1.192	1.421
	S.D.	0	39	1.9	10	1.6	1.09	2.3	9	14	.137	.265	.208
#6	20	4	104	13.2	54	10.9	11.0	19.4	23	80	.515	1.149	1.387
	S.D.	0	30	1.6	12	1.2	0.9	2.1	8	15	.095	.236	.158
#6	20	4	131	12.0	53	10.1	10.6	20.3	26	74	.643	1.446	1.569
	S.D.	0	46	3.8	14	2.4	1.5	3.0	13	28	.177	.390	.207
#6	40*	4	117	12.6	53	10.5	10.8	19.8	25	77	.579	1.291	1.478
	S.D.	0	41	3.0	13	1.9	1.2	2.6	11	22	.154	.351	.204
#7	19	4	263	13.9	102	10.9	11.1	19.4	46	108	.569	1.213	1.513
	S.D.	0	106	2.4	40	1.6	1.1	2.4	23	21	.103	.220	.143
#7	24*	4.0	246	14.1	111	11.2	11.4	19.6	53	116	.545	1.200	1.488
	S.D.	0.5	113	2.7	40	1.9	1.4	2.4	29	34	.118	.214	.200
#8	15	4	294	11.6	119	10.2	10.8	19.0	49	110	.661	1.227	1.576
	S.D.	0	164	3.1	64	2.0	1.3	1.9	30	44	.242	.302	.250
#8	10*	4	306	13.5	139	11.5	11.6	19.2	57	132	.507	1.061	1.446
	S.D.	0	192	1.4	69	0.7	0.6	1.9	34	38	.086	.156	.183
#8	5*	4	271	7.84	81	7.72	9.22	18.6	32	67	.969	1.560	1.836
	S.D.	0	103	1.76	31	1.21	.51	1.8	6	4	.147	.236	.084
#9	9	4	121	14.7	59	11.7	11.8	21.2	25	90	.493	1.150	1.425
	S.D.	0	25	1.8	6	1.1	0.8	3.2	8	11	.069	.194	.150
#9	13*	3.69	140	14.7	63	11.7	11.9	21.3	27	93	.478	1.171	1.478
	S.D.	.48	40	1.5	8	1.0	0.7	3.0	8	11	.095	.167	.169

* Also included in previous set

TABLE I-B-2 STATISTICAL ANALYSIS OF RECORDED WAVE DATA (GROUP METHOD I)

SEA. NO.	N S.D.	$\bar{T}_{hx}$ sec	$\bar{T}_p$ sec	$\bar{H}_{1/3}$ cm	$\bar{T}_{1/3}$ sec		$\langle \frac{T_p}{T_{1/3}} \rangle$	$\langle \frac{T_z}{T_{1/3}} \rangle$	$\langle \frac{H_x}{H_{1/3}} \rangle$	$\langle \frac{T_{hx}}{T_{1/3}} \rangle$	$\langle \frac{T_x}{T_{1/3}} \rangle$	$\langle \frac{H_{tx}}{H_{1/3}} \rangle$
#1	30	3.97	14.0	43	10.5		1.346	1.056	2.246	.382	2.023	.397
	S.D.	0.57	2.1	9	1.2		.261	.071	.673	.064	.346	.102
#2	30	4.03	8.6	53	7.9		1.070	1.147	2.619	.517	2.257	.348
	S.D.	0.89	2.9	11	0.9		.271	.054	1.058	.121	.213	.093
#2	27	4.10	14.0	43	10.5		1.329	1.050	2.541	.393	2.037	.435
	S.D.	0.69	3.3	7	1.6		.267	.059	.665	.074	.435	.156
#3	26	4	12.7	50	10.4		1.234	1.059	2.513	.393	2.028	.410
	S.D.	0	1.6	14	1.5		.185	.075	.872	.038	.385	.213
#3	24	4	11.7	53	9.9		1.201	1.091	2.261	.418	1.650	.469
	S.D.	0	2.7	9	1.6		.293	.126	.504	.080	.246	.192
#4	30	4	12.2	52	9.7		1.255	1.063	2.349	.390	2.020	.390
	S.D.	0	2.6	12	1.1		.240	.059	.721	.032	.309	.118
#4	30	4	12.6	51	11.0		1.145	1.028	2.034	.367	1.887	.445
	S.D.	0	2.1	8	1.0		.155	.044	.430	.036	.326	.186
#5	30	4	13.6	57	11.3		1.215	1.024	2.060	.353	1.788	.452
	S.D.	0	1.9	10	1.6		.209	.075	.167	.033	.284	.124
#6	20	4	13.2	54	10.9		1.144	1.010	1.948	.373	1.780	.424
	S.D.	0	1.6	12	1.2		.276	.042	.456	.040	.200	.128
#6	20	4	12.0	53	10.1		1.185	1.070	2.500	.417	2.087	.502
	S.D.	0	3.8	14	2.4		.255	.100	.686	.085	.432	.203
#6	40*	4	12.6	53	10.5		1.200	1.040	2.222	.385	1.927	.463
	S.D.	0	3.0	13	1.9		.203	.082	.636	.090	.364	.172
#7	19	4	13.9	102	10.9		1.286	1.033	2.308	.375	1.814	.469
	S.D.	0	2.4	40	1.6		.210	.063	.431	.053	.293	.176
#7	24*	4.0	14.1	111	11.2		1.267	1.023	2.221	.370	1.783	.480
	S.D.	0.5	2.7	40	1.9		.193	.064	.671	.066	.275	.176
#8	15	4	11.6	119	10.2		1.125	1.080	2.539	.350	1.940	.409
	S.D.	0	3.1	64	2.0		.155	.124	.765	.022	.541	.085
#8	10*	4	13.5	139	11.5		1.179	1.015	2.120	.409	1.669	.407
	S.D.	0	1.4	69	0.7		.116	.045	.532	.024	.199	.092
#8	5*	4	7.84	81	7.72		1.016	1.210	3.378	.528	2.482	.413
	S.D.	0	1.76	31	1.21		.178	.130	.303	.021	.620	.080
#9	9	4	14.7	59	11.7		1.267	1.019	2.049	.346	1.821	.421
	S.D.	0	1.8	6	1.1		.205	.052	.439	.033	.206	.142
#9	13*	3.69	14.7	63	11.7		1.261	1.012	2.210	.317	1.814	.427
	S.D.	.48	1.5	8	1.0		.170	.044	.518	.053	.186	.133

* Also included in previous set

TABLE II-A-1 STATISTICAL ANALYSIS OF RECORDED WAVE DATA (GROUP METHOD II)

N S.D.	$\overline{T}_{hx}$ sec	$\overline{H}_x$ cm	$\overline{T}_p$ sec	$\overline{H}_{1/3}$ cm	$\overline{T}_{1/3}$ sec	$\overline{T}_z$ sec	$\overline{T}_x$ sec	$\overline{H}_{tx}$ cm	$SR_{1/3}$ sec cm 1/2	$\langle \frac{R_x}{R_{1/3}} \rangle$	$\langle \frac{R_{tx}}{R_{1/3}} \rangle$	$\langle \frac{R^*}{R_{1/3}} \rangle$
1 S.D.	21 - -	171	21.3	129	21.3	19.3	26	58	245	1.135	.818	1.151
3 S.D.	18 0	135 21	19.7 1.6	83 16	17.1 0.9	15.4 0.5	23.7 1.2	42 8	155 19	1.351 .100	1.003 .256	1.284 .114
5 S.D.	17 0	170 27	17.1 1.1	104 18	15.8 1.2	14.4 1.0	21.8 0.8	55 20	161 23	1.382 .112	1.012 .273	1.282 .061
11 S.D.	16 0	184 91	16.5 1.7	117 54	15.2 1.6	14.2 1.2	21.4 1.4	49 25	163 49	1.334 .195	.914 .196	1.249 .069
24 S.D.	15 0	152 66	15.7 1.4	96 37	14.3 1.1	13.8 1.0	21.8 1.8	40 18	139 35	1.323 .112	.989 .219	1.266 .103
51 S.D.	14 0	133 64	14.7 1.4	81 38	13.2 1.0	12.8 0.9	20.9 2.2	35 19	117 32	1.368 .125	1.028 .217	1.281 .070
85 S.D.	13 0	122 50	13.6 1.2	73 29	12.6 0.9	12.3 0.7	21.1 3.2	29 13	106 24	1.338 .122	1.035 .241	1.296 .118
88 S.D.	12 0	112 55	13.3 1.6	68 34	11.8 0.9	11.7 0.7	20.2 2.4	29 16	95 23	1.312 .128	1.108 .255	1.282 .069
60 S.D.	11 0	108 48	12.3 1.6	66 29	11.0 0.9	11.1 0.7	19.7 2.6	26 10	88 21	1.296 .140	1.139 .298	1.282 .070
34 S.D.	10 0	93 30	11.4 1.9	55 10	10.5 0.8	10.8 0.6	19.9 2.5	21 8	77 13	1.241 .137	1.148 .191	1.298 .067
18 S.D.	9 0	89 38	10.7 2.2	55 22	9.76 .99	10.2 0.7	19.6 2.2	21 9	72 16	1.172 .137	1.248 .265	1.264 .046
23* S.D.	7.04 .88	91 34	11.2 2.8	55 23	9.48 1.35	10.0 1.0	18.9 2.6	22 12	69 21	.971 .204	1.231 .178	1.286 .065
28 S.D.	5 0	106 50	12.8 3.8	62 31	10.1 1.7	10.6 1.2	20.8 4.0	30 25	79 29	.669 .154	1.400 .398	1.320 .117
xxxx	4	SEE TABLE II-B-1 FOR T_{hx} = 4 sec										
21 S.D.	3 0	139 65	15.2 0.9	67 35	11.3 1.6	11.7 1.2	20.2 2.0	28 15	92 30	.392 .079	1.169 .167	1.443 .165
19 S.D.	3 0	133 101	12.1 1.2	55 20	10.3 1.4	10.7 0.8	20.1 3.7	22 8	75 14	.449 .110	1.214 .187	1.498 .184
8 S.D.	3 0	138 40	7.4 1.7	50 8	7.81 0.87	9.2 0.8	18.1 2.2	20 7	55 6	.644 .123	1.460 .162	1.657 .268
48[xx] S.D.	3 0	136 77	12.7 3.0	59 27	10.3 1.9	10.9 1.3	19.8 2.9	25 12	79 25	.456 .132	1.235 .200	1.501 .202
1	3	135	13.4	74	11.0	15.8	71.0	36	95	.368	4.502	1.351
1	2	102	2	33	2.3	5.1	61.0	1	13	1.529	4.617	1.758
1	2	313	10.2	54	10.7	10.9	20.0	27	79	.450	1.322	2.408
19[xxx] S.D.	16.6 0.8	172 71	17.2 1.9	108 43	15.7 1.5	14.4 1.1	21.8 1.5	49 22	161 39	1.350 .159	.954 .218	1.263 .072

* Also included in previous set

TABLE II-A-2 STATISTICAL ANALYSIS OF RECORDED WAVE DATA (GROUP METHOD II)

N S.D.	$\overline{T}_{hx}$ sec	$\overline{H}_x$ cm	$\overline{H}_{1/3}$ cm	$\overline{T}_{1/3}$ sec		$\left\langle \frac{T_p}{\overline{T}_{1/3}} \right\rangle$	$\left\langle \frac{\overline{T}_z}{\overline{T}_{1/3}} \right\rangle$	$\left\langle \frac{H_x}{\overline{H}_{1/3}} \right\rangle$	$\left\langle \frac{T_{hx}}{\overline{T}_{1/3}} \right\rangle$	$\left\langle \frac{T_x}{\overline{T}_{1/3}} \right\rangle$	$\left\langle \frac{H_{tx}}{\overline{H}_{1/3}} \right\rangle$
1 S.D.	21 -	171 -	129 -	21.3		.986	.894	1.326	1.088	1.347	.450
3 S.D.	18 0	135 20	83 16	17.1 0.9		1.153 .094	.901 .027	1.656 .287	1.055 .056	1.387 .106	.525 .194
5 S.D.	17 0	170 27	104 18	15.8 1.2		1.080 .105	.910 .042	1.648 .154	1.071 .081	1.381 .091	.554 .252
11 S.D.	16 0	184 91	117 54	15.2 1.6		.990 .331	.943 .032	1.564 .180	1.067 .130	1.420 .160	.426 .144
24 S.D.	15 0	152 66	96 37	14.3 1.1		1.106 .108	.965 .027	1.569 .161	1.059 .090	1.541 .195	.429 .179
51 S.D.	14 0	133 64	81 38	13.2 1.0		1.116 .112	.967 .030	1.647 .188	1.067 .083	1.589 .205	.428 .138
85 S.D.	13 0	122 50	73 29	12.6 0.9		1.088 .088	.979 .030	1.664 .187	1.035 .071	1.683 .271	.382 .124
88 S.D.	12 0	112 55	68 34	11.8 0.9		1.124 .107	.989 .034	1.648 .179	1.023 .075	1.703 .200	.435 .185
60 S.D.	11 0	108 48	66 29	11.0 0.9		1.119 .116	1.012 .035	1.648 .185	1.006 .081	1.788 .277	.412 .149
34 S.D.	10 0	93 30	55 10	10.5 0.8		1.084 .154	1.027 .036	1.689 .174	.956 .077	1.889 .227	.398 .118
18 S.D.	9 0	89 38	55 22	9.76 .99		1.093 .168	1.050 .055	1.599 .116	.932 .097	2.035 .381	.398 .164
23* S.D.	7.04 .88	91 34	55 23	9.48 1.35		1.173 .207	1.062 .055	1.657 .168	.757 .144	2.008 .268	.394 .140
28 S.D.	5 0	106 50	62 31	10.1 1.7		1.262 .312	1.068 .085	1.743 .300	.510 .084	2.112 .482	.460 .124
xxxx	4	SEE TABLE II-B-2 FOR T_{hx} = 4 sec									
21 S.D.	3 0	139 65	67 35	11.3 1.6		1.367 .201	1.036 .056	2.109 .497	.260 .026	1.816 .276	.433 .138
19 S.D.	3 0	133 101	55 20	10.3 1.4		1.191 .159	1.053 .082	2.276 .588	.297 .045	1.960 .375	.409 .145
8 S.D.	3 0	138 40	50 8	7.81 0.87		.944 .199	1.169 .035	2.807 .921	.388 .042	2.322 .161	.404 .102
48** S.D.	3 0	136 77	59 27	10.3 1.9		1.227 .235	1.064 .079	2.292 .651	.306 .058	1.959 .348	.418 .134
1	3	135	74	11.0		1.219	1.436	1.824	.273	6.455	.486
1	2	102	33	2.3		.870	2.217	3.091	.870	2.652	.303
1	2	313	54	10.7		.953	1.019	5.796	.187	1.869	.500
19** S.D.	16.6 0.8	172 71	108 43	15.7 1.5		1.098 .082	.924 .033	1.506 .346	1.068 .346	1.405 .133	.476 .182

* T_{hx} = 6, 7 & 8 sec

** T_{hx} = 16, 17 & 18 sec

TABLE II-B-1 STATISTICAL ANALYSIS OF RECORDED WAVE DATA (GROUP METHOD II)

N S.D.	$\bar{T}_p$ sec	$\bar{T}_{hx}$ sec	$\bar{H}_x$ cm	$\bar{H}_{1/3}$ cm	$\bar{T}_{1/3}$ sec	$\bar{T}_z$ sec	$\bar{T}_x$ sec	$\bar{H}_{tx}$ cm	$\$R_{1/3}$ sec cm 1/2	$<\frac{R_x}{R_{1/3}}>$	$<\frac{R_{tx}}{R_{1/3}}>$	$<\frac{R^*}{R_{1/3}}>$
1 S.D.	21.5 0	4 0	99 -	39 -	10.3 -	10.9 -	22 -	21 -	64 -	.618 -	1.567 -	1.593 -
5 S.D.	19.6 0	4.40 .55	160 68	67 26	11.7 2.7	12.0 1.4	22.8 1.6	44 19	98 30	.571 .117	1.532 .164	1.563 .333
4 S.D.	18.2 0	4.25 .50	118 24	58 10	13.1 2.8	12.8 1.8	22.8 1.0	34 21	100 27	.478 .119	1.315 .367	1.425 .112
10 S.D.	17.0 0	4.00 .67	131 98	69 40	12.4 2.0	12.5 1.4	21.7 1.2	32 15	102 40	.443 .109	1.221 .196	1.337 .096
28 S.D.	16.0 0	3.89 .57	134 61	69 36	11.3 1.9	11.5 1.3	20.7 1.6	33 24	92 31	.510 .144	1.268 .226	1.416 .170
35 S.D.	15.0 0	4 0	156 92	71 38	11.1 1.2	11.3 0.7	19.3 1.4	36 23	92 26	.543 .096	1.188 .265	1.487 .150
37 S.D.	14.2 0	4 0	143 121	68 47	11.5 1.1	11.6 .8	20.5 2.8	27 22	92 28	.505 .093	1.117 .212	1.456 .242
42 S.D.	13.4 0	4 0	115 32	53 14	10.7 1.2	11.0 0.8	20.8 4.4	23 12	78 16	.562 .124	1.242 .284	1.476 .190
34 S.D.	12.8 0	4 0	128 56	59 26	10.7 1.2	11.1 0.8	19.6 2.7	24 14	81 19	.566 .120	1.166 .352	1.485 .212
29 S.D.	12.1 0	4 0	104 47	49 16	10.2 0.9	10.7 0.6	21.0 2.6	20 9	71 12	.574 .111	1.298 .267	1.444 .187
23 S.D.	11.6 0	4 0	136 64	63 29	9.9 0.9	10.3 0.6	19.0 2.7	25 10	77 18	.601 .122	1.236 .255	1.461 .159
18 S.D.	11.1 0	4 0	144 84	54 26	10.0 1.0	10.4 0.6	19.6 2.0	19 10	72 16	.662 .169	1.142 .197	1.628 .299
16 S.D.	10.00 0.58	4 0	132 49	54 13	9.21 1.01	9.80 0.73	19.3 4.0	22 8	67 12	.686 .136	1.336 .319	1.551 .180
15 S.D.	8.46 .31	4 0	131 61	51 8	8.76 1.02	9.53 .61	18.5 2.3	23 7	64 10	.712 .167	1.377 .201	1.556 .266
31* S.D.	9.25 0.91	4 0	131 55	53 11	8.99 1.02	9.67 0.68	18.9 3.2	23 7	66 11	.699 .145	1.356 .265	1.553 .221

* SUM OF PRECEDING TWO N = 15 + 16 = 31

TABLE II-B-2 STATISTICAL ANALYSIS OF RECORDED WAVE DATA (GROUP METHOD II)

N S.D.	$\bar{T}_p$ sec	$\bar{T}_{hx}$ sec	$\bar{H}_{1/3}$ cm	$\bar{T}_{1/3}$ sec		$\left\langle \frac{T_p}{T_{1/3}} \right\rangle$	$\left\langle \frac{T_z}{T_{1/3}} \right\rangle$	$\left\langle \frac{H_x}{H_{1/3}} \right\rangle$	$\left\langle \frac{T_{hx}}{T_{1/3}} \right\rangle$	$\left\langle \frac{T_x}{T_{1/3}} \right\rangle$	$\left\langle \frac{H_{tx}}{H_{1/3}} \right\rangle$
1 S.D.	21.5	4	39 -	10.3 -		2.087 -	1.058 -	2.540 -	.388 -	2.136 -	.538 -
5 S.D.	19.6 0	4.40 .55	67 26	11.7 2.7		1.749 .374	1.058 .163	2.532 1.124	.392 .100	2.020 .372	.652 .116
4 S.D.	18.2 0	4.25 .50	58 10	13.1 2.8		1.438 .308	.985 .080	2.041 .306	.341 .109	1.796 .381	.578 .340
10 S.D.	17.0 0	4.00 .67	69 40	12.4 2.0		1.404 .214	1.018 .062	1.797 .260	.333 .090	1.783 .290	.479 .131
28 S.D.	16.0 0	3.89 .57	69 36	11.3 1.9		1.460 .258	1.034 .084	2.033 .497	.355 .080	1.890 .386	.474 .160
35 S.D.	15.0 0	4 0	71 38	11.1 1.2		1.367 .161	1.027 .066	2.231 .452	.365 .043	1.753 .231	.462 .169
37 S.D.	14.2 0	4 0	68 47	11.5 1.1		1.244 .124	1.012 .047	2.090 .594	.350 .035	1.793 .276	.397 .119
42 S.D.	13.4 0	4 0	53 14	10.7 1.2		1.273 .155	1.037 .066	2.213 .581	.380 .046	1.976 .469	.432 .182
34 S.D.	12.8 0	4 0	59 26	10.7 1.2		1.207 .142	1.037 .064	2.251 .720	.377 .044	1.845 .358	.403 .124
29 S.D.	12.1 0	4 0	49 16	10.2 0.9		1.198 .115	1.051 .049	2.121 .604	.390 .038	2.072 .307	.402 .126
23 S.D.	11.6 0	4 0	63 29	9.88 .93		1.185 .120	1.046 .054	2.159 .492	.409 .041	1.932 .313	.415 .142
18 S.D.	11.1 0	4 0	54 26	10.0 1.0		1.120 .113	1.048 .048	2.716 1.107	.404 .041	1.881 .486	.347 .102
16 S.D.	10.00 0.58	4 0	54 13	9.21 1.01		1.094 .084	1.069 .052	2.436 .589	.440 .050	2.108 .436	.410 .089
15 S.D.	8.46 .31	4 0	51 8	8.76 1.02		.975 .094	1.095 .064	2.486 .879	.462 .050	2.133 .308	.442 .118
31* S.D.	9.25 .91	4 0	53 11	8.99 1.02		1.036 .107	1.080 .058	2.461 .731	.450 .051	2.120 .374	.428 .099

* SUM OF PRECEDING TWO N = 15 + 16 = 31

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APPENDIX I

SECTION III

Table A Statistical Hindcast Wave Data for Deep Water

Table B Statistical Hindcast Wave Data for Shallow Water

Table A STATISTICAL HINDCAST WAVE DATA FOR DEEP WATER
Estimated Duration and Magnitude of Deepwater
Waves Approaching West Beach* from the
Various Directions

Wave Height ft	Duration, hr/yr, for Wave Periods of 3-5 sec	5-7 sec	7-9 sec	9-11 sec	11-13 sec	13-15 sec	15-17 sec	17-19 sec	19-21 sec	21.5 sec	Total
Northwest (304° - 326°)											
1-2	-	-	26	166	158	9	-	-	-	-	359
2-4	-	-	-	53	245	263	44	-	-	-	605
4-6	-	-	-	-	17	166	170	9	-	-	332
6-8	-	-	-	-	53	78	166	35	-	-	332
8-10	-	-	-	-	9	9	26	79	35	-	158
10-12	-	-	-	-	-	-	4	70	26	-	100
12-14	-	-	-	-	-	-	-	9	-	-	9
14-16	-	-	-	-	-	8	-	-	-	-	8
16-18	-	-	-	-	6	-	3	-	-	-	9
18-20	-	-	-	-	-	-	-	-	-	-	-
20-22	-	-	-	-	-	-	9	6	-	-	15
27.6	-	-	-	-	-	-	-	-	-	10	10
Total	-	-	26	219	488	533	392	208	61	10	1937
West-Northwest (282° - 303°)											
1-2	-	-	26	149	96	9	-	-	-	-	280
2-4	-	-	-	26	96	184	26	-	-	-	332
4-6	-	-	-	26	61	140	96	35	9	-	367
6-8	-	-	-	26	-	9	44	44	-	-	123
8-10	-	-	9	-	-	-	9	26	-	-	44
10-12	-	-	-	-	-	-	-	35	-	-	35
12-14	-	-	-	-	-	-	-	26	-	-	26
Total	-	-	35	227	253	342	175	166	9	-	1207
West (259° - 281°)											
1-2	-	-	-	-	-	-	-	-	-	-	-
2-4	9	26	-	-	-	9	-	-	-	-	44
4-6	-	-	-	-	-	-	-	-	-	-	-
6-8	-	-	-	17	-	-	-	-	-	-	17
8-10	-	-	-	-	-	-	-	-	-	-	-
10-12	-	-	-	-	-	-	-	-	-	-	-
12-14	-	-	-	4	-	-	-	-	-	-	4
20-22	-	-	-	-	-	-	-	-	1	-	1
Total	9	26	-	21	-	9	-	-	1	-	66

*Data from Honolulu District, Corps of Engineers (1970) (For proposed Barber Point Harbor Model Study)

Table A ( ) (cont) STATISTICAL HINDCAST WAVE DATA FOR DEEP WATER
Estimated Duration and Magnitude of Deepwater
Waves Approaching West Beach* from the
Various Directions

Wave Height ft	Duration, hr/yr, for Wave Periods of 3-5 sec	5-7 sec	7-9 sec	9-11 sec	11-13 sec	13-15 sec	15-17 sec	17-19 sec	19-21 sec	21.5 sec	Total
West-Southwest (237° - 258°)											
1-2	26	-	-	-	-	-	-	-	-	-	26
2-4	-	26	-	-	-	-	-	-	-	-	26
4-6	-	-	26	-	-	-	-	-	-	-	26
6-8	-	-	9	-	-	-	-	-	-	-	9
12-14	-	-	-	9	-	-	-	-	-	-	9
16-18	-	-	-	-	2	-	-	-	-	-	2
Total	26	26	35	9	2	-	-	-	-	-	98
Southwest (214° - 236°)											
1-2	35	-	-	-	-	-	-	-	-	-	35
2-4	-	35	-	-	-	96	140	-	-	-	271
4-6	-	-	26	-	-	-	-	-	-	-	26
6-8	-	-	9	-	-	-	-	-	-	-	9
12-14	-	-	-	9	-	-	-	-	-	-	9
16-18	-	-	-	-	2	-	-	-	-	-	2
28-30	-	-	-	-	1	-	-	-	-	-	1
Total	35	35	35	9	3	96	140	-	-	-	353
South-Southwest (192° - 213°)											
1-2	44	-	-	-	885	368	-	-	-	-	1297
2-4	-	61	-	-	359	1060	-	-	-	-	1480
4-6	-	-	26	-	-	-	-	-	-	-	26
6-8	-	-	9	-	-	-	-	-	-	-	9
12-14	-	-	-	2	-	-	-	-	-	-	2
16-18	-	-	-	-	2	-	-	-	-	-	2
24-26	-	-	-	-	-	3	-	-	-	-	3
Total	44	61	35	2	1246	1431	-	-	-	-	2819
South (169° - 191°)											
1-2	88	-	-	-	-	44	-	-	-	-	132
2-4	-	88	-	-	806	394	70	-	-	-	1358
4-6	-	-	44	-	44	105	-	-	-	-	193
6-8	-	-	9	-	-	-	-	-	-	-	9
12-14	-	-	-	9	-	-	-	-	-	-	9
16-18	-	-	-	-	2	-	-	-	-	-	2
Total	88	88	53	9	852	543	70	-	-	-	1703

*Data from Honolulu District, Corps of Engineers (1970) (For proposed Barber Point Harbor Model Study)

Table B STATISTICAL HINDCAST WAVE DATA FOR SHALLOW WATER
Estimated Duration and Magnitude of Shallow-Water
Waves at 103-Depth Approaching West Beach
from the Various Directions

Wave Height ft	Duration, hr/yr, for Wave Periods of 3-5 sec	5-7 sec	7-9 sec	9-11 sec	11-13 sec	13-15 sec	15-17 sec	17-19 sec	19-21 sec	Total
				Approach Direction (S30°W)						
1-2	132	-	-	-	885	412	-	-	-	1429
2-4	-	149	-	-	1209	1559	70	-	-	2987
4-6	-	-	70	-	-	-	-	-	-	70
6-8	-	-	18	-	-	-	-	-	-	18
8-10	-	-	-	-	-	-	-	-	-	-
10-12	-	-	-	11	-	-	-	-	-	11
12-14	-	-	-	-	2	-	-	-	-	2
14-16	-	-	-	-	2	-	-	-	-	2
20	-	-	-	-	-	3	-	-	-	3
Total	132	149	88	11	2098	1974	70	-	-	4522
				Approach Direction (50°W)						
1-2	35	-	-	-	-	-	-	-	-	35
2-4	-	35	-	-	-	96	140	-	-	271
4-6	-	-	26	-	-	-	-	-	-	26
6-8	-	-	9	-	-	-	-	-	-	9
8-10	-	-	-	-	-	-	-	-	-	-
10-12	-	-	-	9	-	-	-	-	-	9
12-14	-	-	-	-	-	-	-	-	-	-
14-16	-	-	-	-	2	-	-	-	-	2
24	-	-	-	-	1	-	-	-	-	1
Total	35	35	35	9	3	96	140	-	-	353
				Approach Direction (S70°W)						
1-2	26	-	-	-	-	-	-	-	-	26
2-4	-	26	-	-	-	-	-	-	-	26
4-6	-	-	26	-	-	-	-	35	9	70
6-8	-	-	9	-	-	-	-	44	-	53
8-10	-	-	-	-	-	-	-	61	-	61
10-12	-	-	-	-	-	-	-	26	-	26
12-14	-	-	-	9	-	-	-	-	-	9
14-16	-	-	-	-	2	-	-	-	-	2
20	-	-	-	-	-	-	-	-	1	1
Total	26	26	35	9	2	-	-	166	10	274

*Data from Honolulu District, Corpos of Engineers (1970) (For proposed Barber Point Harbor Model Study)

Table B (cont) STATISTICAL HINDCAST WAVE DATA FOR SHALLOW WATER
Estimated Duration and Magnitude of Shallow-Water
Waves at 103 ft-Depth Approaching West Beach*
from the Various Directions

Wave Height ft	Duration, hr/yr, for Wave Periods of 3-5 sec	5-7 sec	7-9 sec	9-11 sec	11-13 sec	13-15 sec	15-17 sec	17-19 sec	19-21 sec	Total
					Approach Direction (S86°W)					
1-2	-	-	-	-	-	9	184	-	-	193
2-4	9	26	-	-	-	9	192	9	-	245
4-6	-	-	-	-	-	140	126	35	-	301
6-8	-	-	17	-	-	9	56	149	61	292
8-10	-	-	-	-	-	-	-	-	9	9
10-12	-	-	-	3	-	-	-	-	-	3
12-14	-	-	1	-	-	-	-	6	-	7
14-16	-	-	-	-	-	-	-	-	-	-
16-18	-	-	-	-	-	-	9	-	-	9
18-20	-	-	-	-	-	-	-	-	10	10
Total	9	26	18	3	-	167	567	208	71	1069
					Approach Direction (N78°W)					
1-2	-	-	52	315	254	9	-	-	-	630
2-4	-	-	-	79	358	613	-	-	-	1050
4-6	-	-	-	26	114	78	-	-	-	218
6-8	-	-	-	26	9	9	-	-	-	44
8-10	-	-	9	-	6	8	-	-	-	23
Total	-	-	61	446	741	717	-	-	-	1965

*Data from Honolulu District, Corps of Engineers (1970) (For proposed Barber Point Harbor Model Study)

Using the parameter for X, Y, and Z, and rearranging terms to obtain equations for $T_{hx}/T_{1/3}$, T_{hx}/T_p and $T_{1/3}/T_p$, one obtains the solutions in terms of quadratic equations. The last term of each quadratic equation is small and is under a radical sign. The quadratic equations can be expanded by use of the binominal theorem. The following equations are thus determined:

$$1) \quad \frac{T_{hx}}{T_{1/3}} = 0.74931\sqrt{\frac{H_x}{H_{1/3}}}\left[1 + 0.021820\,\frac{gT_{1/3}^2}{10^3 H_{1/3}}\right]$$

$$2) \quad \frac{T_{hx}}{T_p} = 0.65181\sqrt{\frac{H_x}{H_{1/3}}}\left[1 + 0.031962\,\frac{gT_{1/3}^2}{10^3 H_{1/3}}\right]$$

$$3) \quad \frac{T_{1/3}}{T_p} = 0.86984\left[1 + 0.0155\,\frac{gT_p^2}{10^3 H_{1/3}}\right]$$

It is seen from the above equations that the period of max wave height is directly related to the square root of the ratio of $H_x/H_{1/3}$, with a small correction due to the reciprocal of the wave steepness.

Equations 1, 2, and 3 were used to predict the quantities $T_{hx}/T_{1/3}$, T_{hx}/T_p and $T_{1/3}/T_p$, respectively, using the appropriate data from Table 1. Standard deviations and variances from the means were determined, and the variances found were as follow: Eq. 1, ± 5%; Eq. 2, ± 5% and Eq. 3, 3.2%, for the corresponding weighted averages of the grouped data.

The average variances of the individual data sets from each group set was found from Tables I-A-1 and I-A-2 as follow: $T_{hx}/T_{1/3}$, ± 8.03%; T_{hx}/T_p, ± 12.3%, and $T_{1/3}/T_p$, ± 10.9% .

It then follows that the combined variances are: $1 - [1 \pm 0.05][1 \pm 0.08] = \pm 0.13$ for Eq. 1; $1 - [1 \pm 0.05][1 \pm 0.12] = \pm 0.18$ for Eq. 2; and $1 - [1 \pm 0.03][1 \pm 0.11] = \pm 0.14$ for Eq. 3.

The final forms of eqs. (1-d), (2-d) and (3-d) are as follows:

1) $$\frac{T_{hx}}{T_{1/3}} = 0.74930 \sqrt{\frac{H_x}{H_{1/3}}} \left[1+0.0218 \frac{gT_{1/3}^2}{10^3 H_{1/3}}\right] \left[1\pm 0.13\right]$$

2) $$\frac{T_{hx}}{T_p} = 0.65181 \sqrt{\frac{H_x}{H_{1/3}}} \left[1+0.0320 \frac{gT_{1/3}^2}{10^3 H_{1/3}}\right] \left[1\pm 0.18\right]$$

and

3) $$\frac{T_{1/3}}{T_p} = 0.86984 \left[1+0.0155 \frac{gT_p^2}{10^3 H_{1/3}}\right] \left[1\pm 0.14\right]$$

EXAMPLE I: FOR T_x OF LONG PERIOD SWELL GREATER THAN MEASURED

GIVEN: $T_{1/3}$ = 18 sec., H_x = 8 feet, $H_x/H_{1/3}$ = 1.78

FROM Eq. 1

$$\frac{T_{hx}}{T_{1/3}} = 0.74930 \sqrt{1.78} \left[1+0.028 \frac{32.16(18)^2}{10^3(8)}\right] \left[1\pm 0.13\right]$$

$$\frac{T_{hx}}{T_{1/3}} = 0.99697 \quad [1.0284][1\pm 0.13]$$

$$T_{hx} = 18.5 \text{ sec} \pm 2.4 \text{ sec.}; \quad 16.1 \leq T_{hx} \leq 20.9 \text{ sec.}$$

EXAMPLE II: FOR WEST BEACH DESIGN HURRICANE (DEEP WATER)

GIVEN: $H_{1/3}$ = 34.7 feet; T_p = 13.8 sec.; $H_x/H_{1/3}$ = 1.78

FROM Eq. 3

$$\frac{T_{1/3}}{T_p} = 0.86984 \left[1+0.0155 \frac{32.16(13.8)^2}{10^3(34.7)}\right] = 0.8722$$

$$T_{1/3} = 12.04 \text{ sec.}$$

the beaches and transported far inland. Large slabs were moved and rounded coral blocks as large as a foot to a foot-and-a-half in diameter were transported inland.

The point here is that such a surge is possible and is relevant to the present study and should be given some consideration for West Beach area. The fact that the Corps of Engineers (1983) report does not show any damage or high run-up for West Beach does not mean high run-up did not occur. No one was there to study the area. Tsunami run-up at West Beach is predicted higher than for Honolulu and Waikiki, which had large run-up damage during Hurricane Iwa.

In regard to the 100-year Design Hurricane, such high water levels will not be so large as those observed for Hurricane Iwa (1982) but the run-up will be greater than that calculated by conservative standard methods.

Presently only an estimate can be made because considerable additional work would be required.

It would probably be reasonable to assume that the 100-year wave run-up for West Beach would be approximately 15 feet or about the same as governed by the tsunami flood determinations.

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SECTION III

WAVE DATA ANALYSIS

SUMMARY

Sea Engineering, Inc. (1983) reported 798-34 minute wave data sets offshore West Beach Development area during 1981 and 1982. The total of 798 records were analyzed by Sea Engineering, Inc. (1983). The study included wave records from three distinct wave types: (1) Northern Swell, generated by North Pacific Storms, (2) Kona storm waves from the West and Southwest generated by winds from local low pressure system, and (3) Southern swell from storms in the South Pacific and Indian Oceans off New Zealand and Eastern Australia. Long period swell appears in almost all of the records, and short period local waves appear in almost half of the records of the swell.

Data on currents and tides were also recorded by Sea Engineering (1983), but these data were not analyzed as part of this study.

A SEA Data Model 635-11 digital recording wave and tide gauge, described by Sea Engineering (1983), was used to obtain daily records of wave height and period and tide elevation. The gauge was programmed to record wave data for approximately 34 minutes every 6 hours, and tide elevation continuously – 32 times per hour.

The gauge was located toward the southern end of the study area, and on the bottom in a depth of water approximately 45 feet MSL, and an approximate distance of 4400 feet from the shoreline.

The 798-34 minute wave records should represent an adequate number of samples from the three distinct sources: (a) Northern Swell, (b) Kona Storms (mild) and (c) Southern Swell. The samples do not represent extreme conditions, for example: (1) an extreme southern swell storm, (2) the January 7-11, 1980 Kona Storm which caused some havoc in Kewalo Basin, Sand Island and West Beach area, and (3) Hurricane IWA on November 23, 1982, which caused considerable damage along the south and west shores of Oahu, as well as on Kauai.

In so far as the possibility of recording very large waves is concerned, one might say that it was unfortunate that the wave recording began 1 1/2 years too late for the 1980 Kona Storm and terminated one year too soon for the 1982 IWA Hurricane. On the other hand, based in part on past experience by other investigators, the instrumentation might have been subjected to possible loss or damage.

Sea Engineering (1983) presented a summary of the 798 wave records based on an analysis, including spectrum analysis, including but not limited to the following parameters:

$H_{1/3}$ (cm) = significant wave height (of each record).

T_p (sec) = period of wave spectrum having maximum wave energy.

$T_{1/3}$ (sec) = period of significant wave height

T_z (sec) = zero crossing (mean) wave period of record.

H_x (cm) = H_{max} maximum wave height.

T_{hx} (sec) = period of max wave height.

T_x = MAX period of the record.

H_{tx} = wave height of the corresponding max period (T_x).

Other information given by the Sea Engineering (1983) was not utilized in the further analysis given in the present report.

The present analysis of the data given by Sea Engineering (1983) consists essentially of a statistical nature, wherein are established relationships between $H_{1/3}$, H_x and H_{tx}; $T_{1/3}$, T_p, T_z, T_{hx} and T_x; and corresponding standard deviations. In addition to the above analysis, relative run-up values are given in terms of $T_{1/3}\sqrt{H_{1/3}}$, $T_{hx}\sqrt{H_x}$, and $T_x\sqrt{H_{tx}}$; where $T_{1/3}$ and $H_{1/3}$ are significant wave period and wave height; T_{hx} and H_x are period of maximum wave height and maximum wave height; and T_x and H_{tx} are maximum wave period and wave height of maximum wave period respectively.

Three prediction equations have been determined by a least square regression analysis, which relate, T_{hx}, $T_{1/3}$, T_p and H_x and $H_{1/3}$, where T_p is the modal period of the spectrum and the other terms are same as defined. These prediction equations are as follows:

$$1)\quad \frac{T_{hx}}{T_{1/3}} = 0.74931\sqrt{\frac{H_x}{H_{1/3}}}\left[1 + .0.022\,\frac{gT^2_{1/3}}{10^3\,H_{1/3}}\right]\quad [1 \pm 0.13]$$

$$2)\quad \frac{T_{hx}}{T_p} = 0.65181\sqrt{\frac{H_x}{H_{1/3}}}\left[1 + 0.032\,\frac{gT_p^2}{10^3\,H_{1/3}}\right][1 \pm 0.18]$$

$$3)\quad \frac{T_{1/3}}{T_p} = 0.86984\left[1 + 0.015\,\frac{gT_p^2}{10^3\,H_{1/3}}\right]\qquad [1 \pm 0.14]$$

The above three equations are based on the grouping of wave data given in Table 1, and are determined from equations which have very high regression correlation coefficients. Thus, the equations represent the averages of groups of data, each group of which have independent standard deviations. Predictions were made for the data in Table 1, using the above three equations, and it was found that the standard deviations of the ratios $T_{hx}/T_{1/3}$ and T_{hx}/T_p were 5 percent and for $T_{1/3}/T_p$ was 3.2 percent of the corresponding mean

ratios. The values of ($\pm$ 0.13), ($\pm$ 0.18), and ($\pm$ 0.14) represent the overall product of the variance (S.D. $\pm$ 100), respectively.

There are some similarities between the above equations and the results presented by Wiegel (1964). However, the prediction equations should show an improvement over those of Wiegel (1964) for predicting the wave period T_h of the maximum wave height H_x, because T_h depends on the ratio of $H_x/H_{1/3}$ as well as on either $T_{1/3}$ or T_p.

Wave Hindcast Data. In addition to the data analysis for West Beach records, included in Appendix I of this section are wave hindcast data for Barbers Point Harbor entrance. The hindcast data were extracted directly from the U.S. Army Corps of Engineers (1970) Report on the Model Study of the Proposed Barbers Point Harbor. The hindcasts data were made by Marine Advisors, using the Bretschneider wave forecasting relations, and supplemented with some measurements. Appendix I is included for convenience of reference in using the present report for Model tests of the Proposed West Beach Marina as it affects the Barbers Point Harbor.

Wave Run-up. The measured wave data off West Beach consisted of many wave trains, of which short period maximum waves were superimposed on long period swells. Such conditions must all exist for the hindcast data, although such conditions have never been considered before. Based on the analysis of the recorded data, it is suggested that a factor of 1.17 be applied to wave run-up calculations based on standard techniques. The factor of 1.17 should also be applied to model tests, after scale effects are taken into account.

TABLE 1 CORRELATION DATA FOR DETERMINING WAVE PERIOD, T_{hx} of MAX WAVE HEIGHT, Hx

SEA No.	N No.	H_x cm	T_{nx} sec	$H_{1/3}$ cm	$T_{1/3}$ sec	T_p sec	$\frac{10^3 H_x}{gT^2_{nx}}$	$\frac{10^3 H_{1/3}}{gT^2_{1/3}}$	$\frac{10^3 H_{1/3}}{gT^2_p}$
#1	8	75	11.8	44	13.4	12.9	.550	.250	.265
#2	17	72	12.0	45	12.0	13.5	.510	.319	.252
#3	30	92	13.2	56	12.4	13.5	.539	.372	.314
#3	30	82	11.7	50	11.8	13.0	.611	.366	.302
#4	30	103	11.5	62	11.3	12.5	.795	.495	.405
#4	24	104	13.0	61	12.2	13.2	.628	.418	.357
#4	21	123	15.3	79	14.7	15.7	.536	.373	.327
#4	11	109	14.0	69	13.5	14.4	.567	.386	.340
#5	20	101	11.0	60	11.4	12.8	.852	.471	.374
#5	8	109	10.0	63	10.6	11.5	1.112	.572	.486
#5	8	88	12.0	53	12.2	14.1	.624	.363	.272
#5	25	114	13.6	73	13.1	14.7	.629	.434	.345
#5	15	106	13.0	66	12.5	14.1	.640	.431	.339
#6	24	93	10.0	55	10.4	11.7	.949	.519	.410
#6	10	110	11.0	61	11.3	12.1	.928	.487	.425
#6	15	119	12.0	71	11.2	12.9	.843	.578	.435
#6	14	173	13.0	102	13.3	14.3	1.045	.588	.509
#6	17	138	14.0	83	13.2	15.1	.718	.486	.371
#6	12	182	15.6	117	15.4	16.3	.763	.503	.449
#6	7	196	15.0	118	15.2	15.9	.889	.521	.476
#7	19	209	12.2	130	12.3	14.0	1.433	.877	.677
#8	21	140	10.6	89	10.8	12.1	1.271	.779	.620
#9	16	124	12.6	76	12.6	13.4	.797	.488	.432

N = ΣM 402

MEDIAN REGRESSION EQUATIONS

1) X = 1.781076973 Y-0.077724911; r = 0.9588

2) X = 2.353759 Z-0.150459976; r = 0.9565

3) Y = 1.321669895 Z-0.040890240; r = 0.9799

NOTE: See text and Table I-A-2 for the two sets of data (#6) and (#8) eliminated for determination of the prediction equations.

STATISTICAL ANALYSIS OF RECORDED WAVE DATA

1. INTRODUCTION

In the past many investigators who analyzed wave records considered various techniques such as significant wave method, probability distributions of wave height and period and joint probability distributions, and wave spectral analysis. The literature in full of references on this subject, a list too long to be included. Therefore, only a few references, each of which included many additional references are included at the end of this section. The method of analysis given in this section of the report supplement much of previously published material. A preliminary investigation of the 798 records summarized by Sea Engineering (1983) show that many of the records are somewhat different from those reported in the literature. For example, the records show combinations from more than one origin of generation. There are long period swell records, some quite regular and others influenced by local wave generation. There are local Kona Wind wave records that also include long period swell from distant storms. The records are complicated, and may have considerable affects on wave run-up calculations when using standard wave run-up methods based on regular waves. Wave run-up measurements based on irregular waves and wave spectra might probably be different from some of the West Beach wave records which have combinations of two or more wave spectra having widely dispersed peaks of wave periods. In the present analysis an attempt is made to address the differences in the West Beach Wave Records and those records previously reported in the literatures. Of the

798 wave records, there are 314 wave records for which the period of maximum wave height, T_{hx} = 4 seconds, 49 for T_{hx} = 3 seconds and 28 for T_{hx} = 28 seconds, as shown in Table 2. These are waves superimposed on long period swells for which significant wave period $T_{1/3}$, modal period T_p are both large, and zero crossing wave period T_z are all large compared with T_{hx}. Figure 1 shows the double peaks in the histograph of wave period, one at T_{hx} = 4 sec., and the other at about T_{hx} = 12 to 13 seconds. Figure 2 shows the distribution of the modal period T_p. Thus, the analysis of the data can be made with either or both T_{hx} and T_p. The analysis is based on two methods of grouping discussed, subsequently. (See Table 3 page 116 for symbols.)

GROUP METHOD I (SEA. NO.)

SEA. No. #1 through #9 represent each of the 9 individual periods of sets of records tabulated by Sea Engineering (1983). The results of method I are given in Table I-A-1 through I-B-2. Table I-A-2 is actually a continuation of Table I-A-1, and the arrangement of the results is only a matter of convenience. The second column gives N the number of waves used in the statistical analysis of the data. S.D. below each value of N gives the standard deviations corresponding to the mean values given in all the remaining columns for each group. As can be seen from the above tables there are more than one grouping of the data for the longer periods of sets of records, and in some cases several sets are combined as indicated at the bottom of the table.

Tables I-B-1 and I-B-2 for T_{hx} = 4.0 seconds are similar in construction to Table I-A-1 and I-A-2 respectively.

TABLE 2

NUMBER OF WAVES HAVING SAME PERIOD T_{nx} of MAX WAVE H_{max}

SEA NO. T_{nx} (sec)	#1	#2	#3	#4	#5	#6	#7	#8	#9	1-9
2	1	0	0	0	0	0	0	0	1	2
3	5	11	9	6	4	3	4	3	4	49
4	22	49	50	64	31	41	33	15	9	314
5	4	5	2	6	3	3	5	0	0	28
6	0	3	1	0	2	2	0	0	0	8
7	0	2	0	1	0	1	0	2	0	6
8	0	0	1	1	0	3	4	0	0	9
9	0	2	0	3	1	5	4	3	0	18
10	1	2	5	4	8	6	2	6	2	36
11	2	1	8	21	4	10	11	2	1	60
12	3	4	26	16	8	15	8	5	3	80
13	2	1	17	24	15	13	5	2	6	85
14	0	4	6	11	4	17	6	0	3	51
15	0	1	1	4	6	7	3	1	1	24
16	0	1	1	1	0	3	5	0	0	11
17				1	1	2	1			5
18				3						3
19										0
20										0
21				1						1
2-21										798
Totals	40	86	127	167	87	131	91	39	30	798

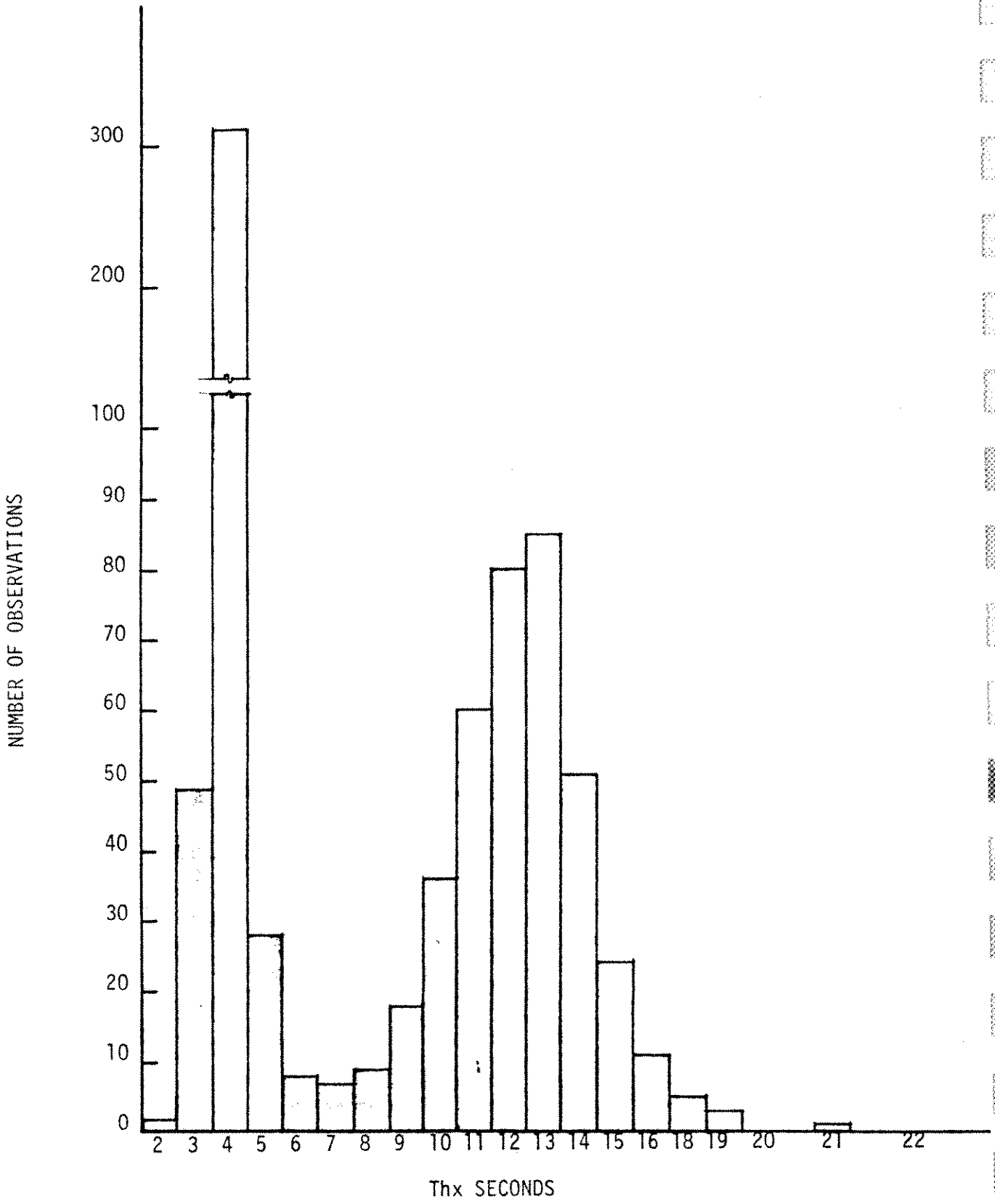

FIG. 1 NO OF OBSERVATIONS FOR PERIOD Thx
OF MAXIMUM WAVE

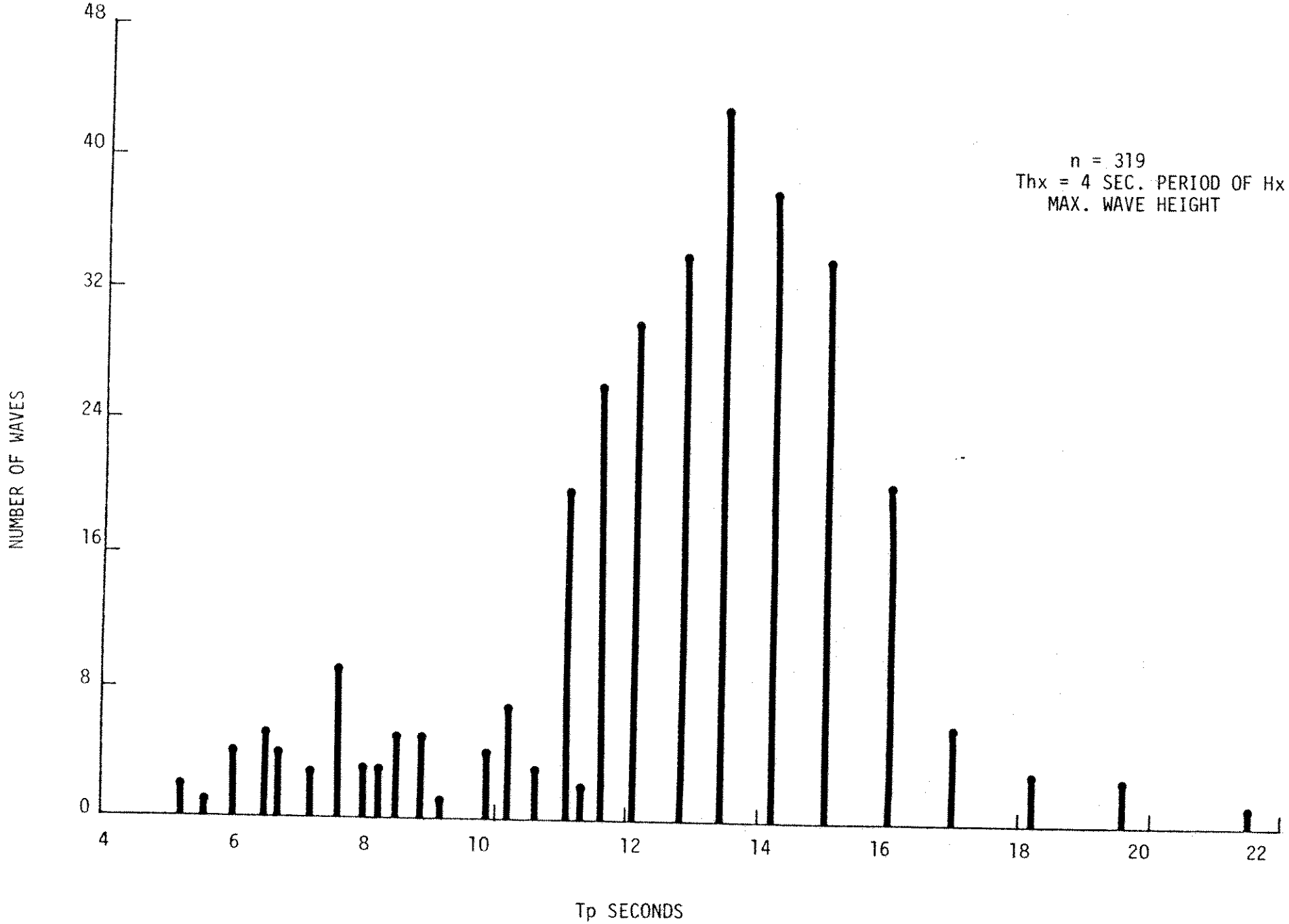

FIGURE 2. NUMBER OF WAVES FOR Tp WITH Thx = 4 SEC

GROUP METHOD II (T_{hx})

This method groups the data according to the period T_{hx} of max H_x; and for T_{hx} = 4 seconds in terms of T_p, the modal period of the wave spectrum. In this method of grouping the data are selected from all the record (SEA. #1 through SEA. #9). The statistical analysis was the same as that given for GROUP METHOD I, and the results are given in Table II-A-1 through II-B-2. A graphical presentation for T_{hx} = 4 seconds is shown in Figure 3, for example, the primary difference in the two grouping methods is that in Method I, the grouping are in terms of the sets of SEA data SEA #1, #2, #3 . . . and #9; where as in Method II the grouping of the data considers all 9 sets of SEA data as one complete set. Graphs similar to Figure 3 could have been prepared relating the various parameters to $T_{1/3}$ or T_{hx}, etc.

However, it is better to develop prediction equations, which can become more useful.

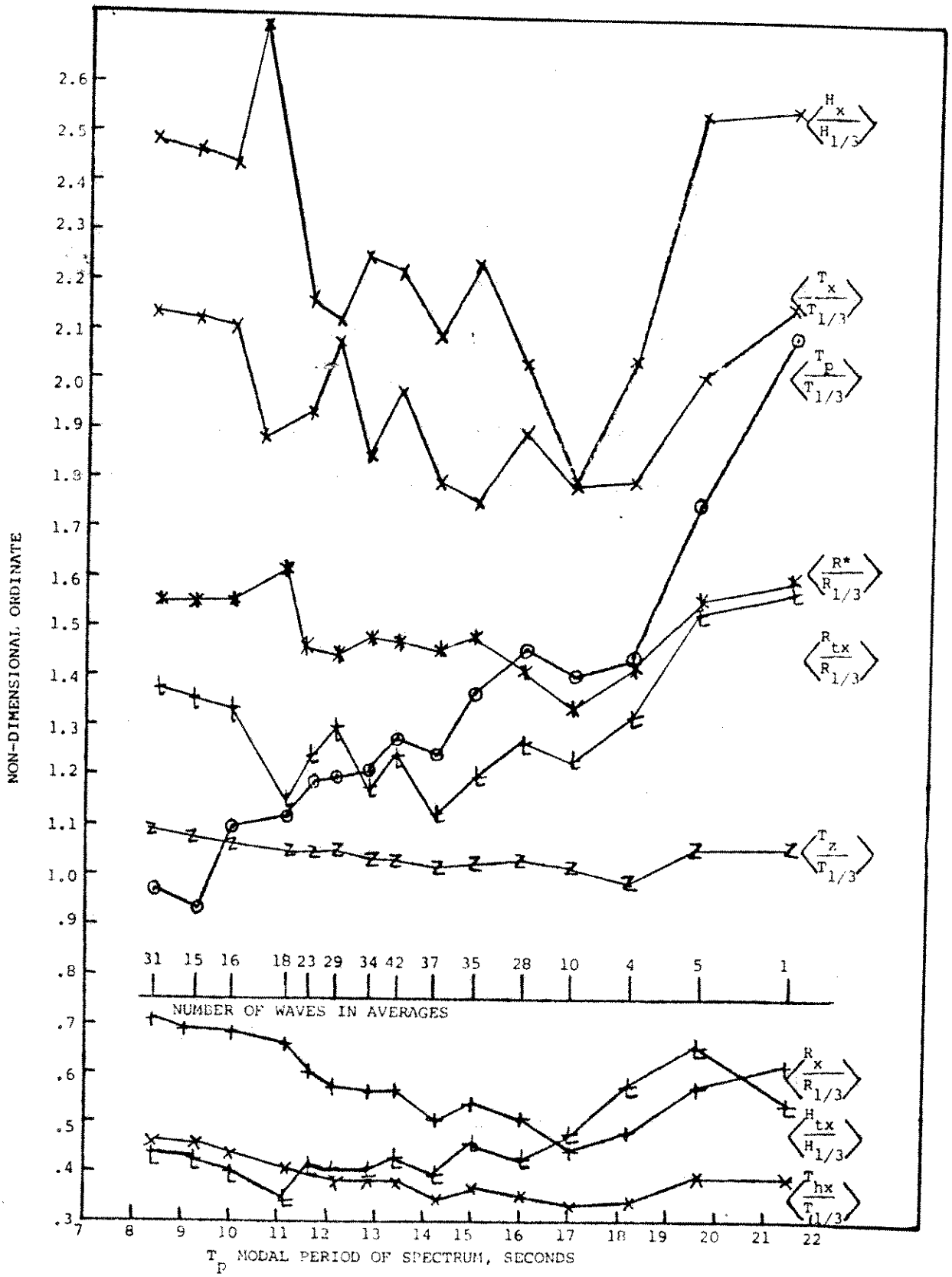

FIG 3 NON-DIMENSIONAL RELATIONS FOR T_{hx} = 4 SECONDS

2. DETERMINATION OF PREDICTION EQUATIONS FOR WAVE PERIOD, T_{hx}, OF MAXIMUM WAVE HEIGHT, H_x

The wave data given in Table 1 is from Group Method I, Table I-A-1, except for the two sets of data for $\overline{T}_{hx}$ = 9.2 and 8.2 seconds. These two sets of data have been eliminated from this present analysis because the averages of T_{hx} include low values of T_{hx}, associated with long period T_p of the spectrum, resulting in complex wave trains.

Numerous approaches were investigated, and it was concluded that the best way to develop prediction equations was in terms of the following parameters

$$X = \frac{10^3 H_x}{gT_{hx}^2} \quad , \quad Y = \frac{10^3 H_{1/3}}{gT_{1/3}^2} \quad , \text{ and } \quad Z = \frac{10^3 H_{1/3}}{gT_p^2}$$

The weighted averages of the above parameters for the grouped data was determined as follows:

$$X = \frac{1}{M} \Sigma\, NX_i \quad , \quad Y = \frac{1}{M} \Sigma\, NY_i \quad , \text{ and } \quad Z = \frac{1}{M} \Sigma\, NZ_i$$

where M = 402 sets of data from 34 minute records from SEA, as given in Table I-A-1, N = number of data of each set of data X_i, Y_i and Z_i are the corresponding values determined from the data of H_x, T_{hx}, $H_{1/3}$, $T_{1/3}$ and T_p of Table 1.

In any linear regression analysis of data there are two regression equations, u on v and v on u as follows:

(a) $$u = r \frac{S_u}{S_v} (v - \overline{v}) + \overline{u}$$

and

$$(b) \qquad v = r \frac{S_v}{S_u} (u - \bar{u}) + \bar{v}$$

where r = regression correlation coefficient

$S_u = \sqrt{\bar{u}^2 - (\bar{u})^2}$ = S.D., standard deviation of u

$S_v = \sqrt{\bar{v}^2 - (\bar{v})^2}$ = S.D., standard deviation of v

$\bar{u}$ and $\bar{v}$ are the mean values of u.

When the regression coefficients r = 1.0, then eq. a is the same as eq. b.

In the present analysis it was found that the regression correlation coefficients were very high. Correlations were made between X and Y, X and Z, and Y and Z.

Since the above three equations are based on the same sets of data, and since the regression coefficients are high, it will be convenient to use the median regression equation, based on the average of the above two regression equations as follows:

$$(c) \quad u = \frac{1+r^2}{2r} \frac{S_u}{S_v} (u-\bar{u}) + \bar{u}$$

The above single equation can be used to predict either u from v or v from u. The mean values and standard deviations of u and v, and the regression coefficient will still be the same. The distributions of S_u and S_v will be slightly altered, but the mean value will not change as long as r is quite large, and this is the case from the present data analysis.

The regression analysis of the data from Table 1, results in three equations, together with regression correlation coefficients as follow:

1-c) X = 1.781077 Y - 0.077725 ; r = 0.9588 (See Fig. 4)

2-c) X = 2.353759 Z - 0.150460 ; r = 0.9565 (See Fig. 5)

3-c) Y = 1.32167 Z - 0.040890 ; r = 0.9799 (See Fig. 6)

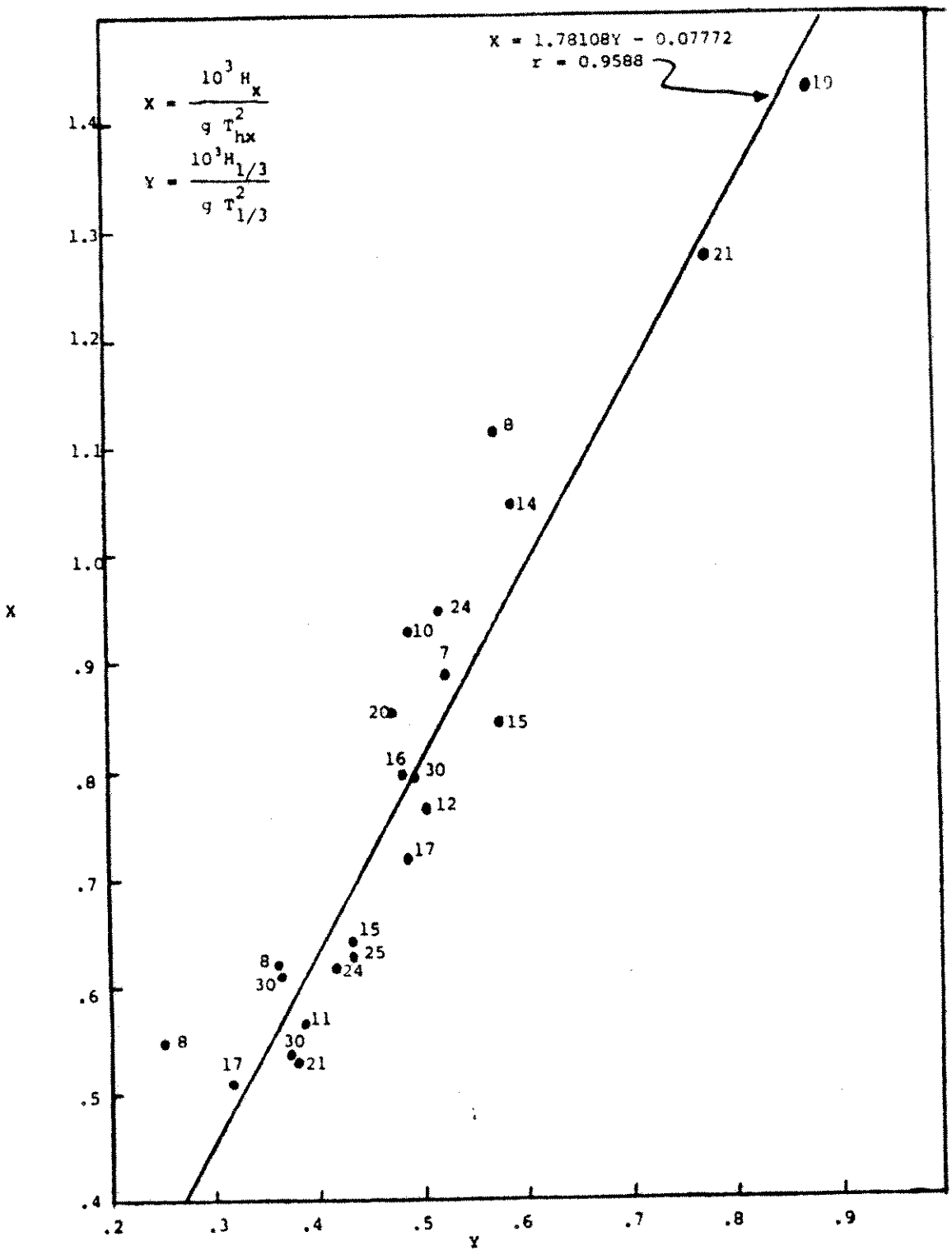

FIG 4 $\frac{10^3 H_x}{g T_{hx}^2}$ vs $\frac{10^3 H_{1/3}}{g T_{1/3}^2}$

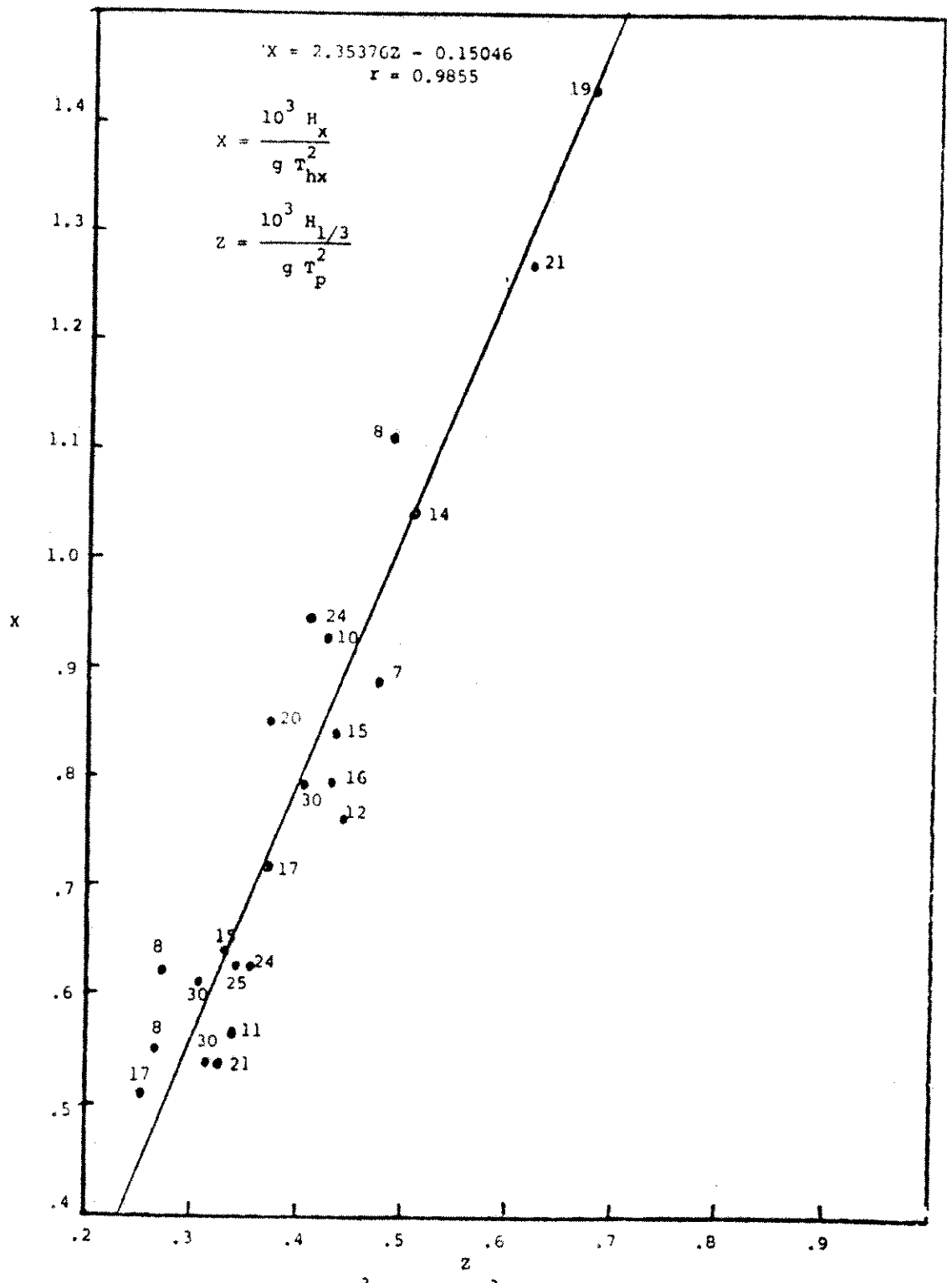

FIG 5 $\frac{10^3 H_x}{g T_{hx}^2}$ vs $\frac{10^3 H_{1/3}}{g T_p^2}$

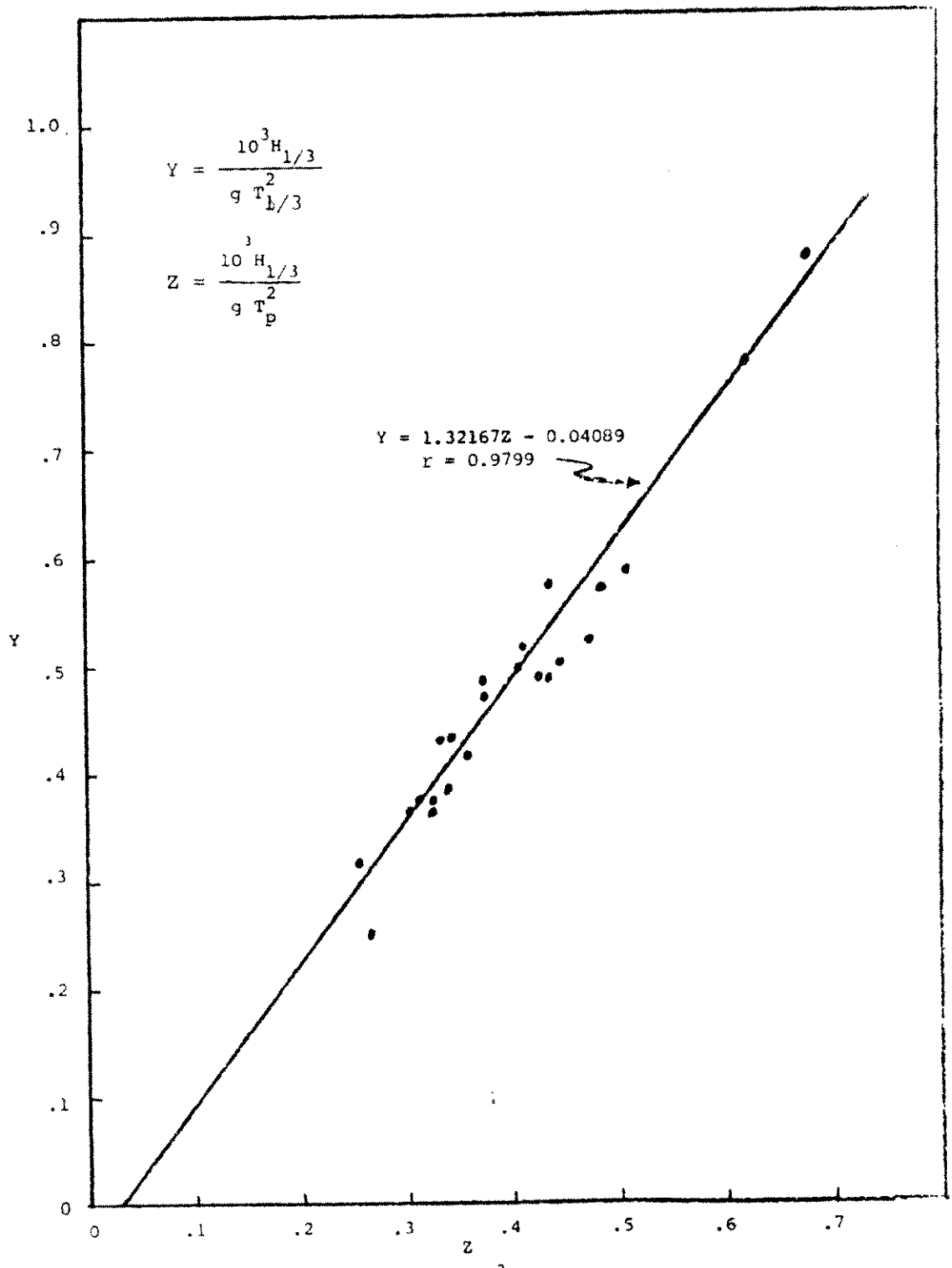

FIG 6 $\frac{10^3 H_{1/3}}{g T_{1/3}^2}$ VS $\frac{10^3 H_{1/3}}{g T_p^2}$