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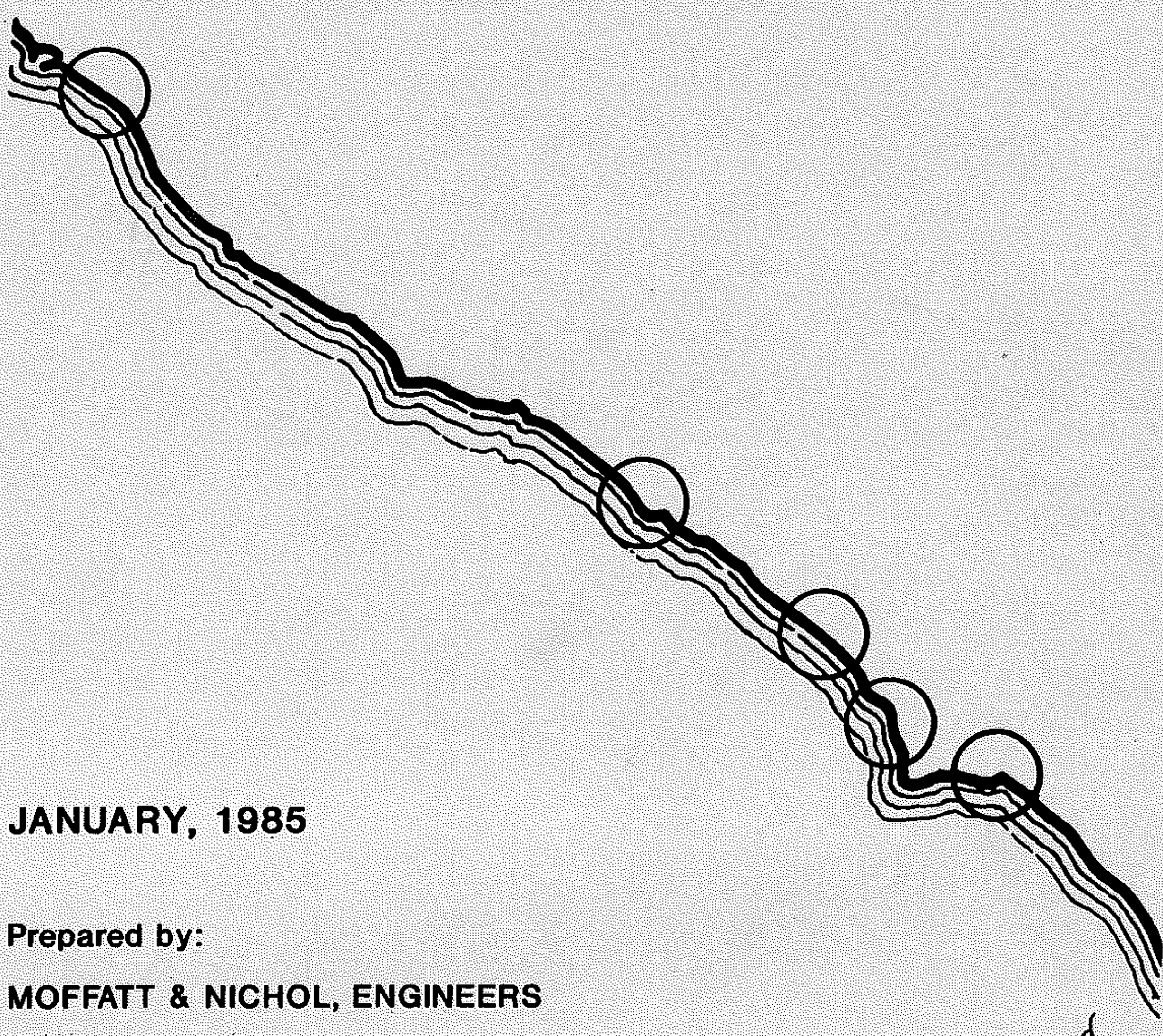
COUNTY OF ORANGE
ENVIRONMENTAL MANAGEMENT AGENCY

COASTAL FLOOD PLAIN DEVELOPMENT ORANGE COUNTY COASTLINE

22 miles of unincorporated Coastline



JANUARY, 1985

Prepared by:

MOFFATT & NICHOL, ENGINEERS

Text Approved on March, 1985 Bx B/S.

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COUNTY OF ORANGE

ENVIRONMENTAL MANAGEMENT AGENCY

REGULATION

COASTAL FLOOD PLAIN DEVELOPMENT INFORMATION

Coastal Flood Plain Development is defined as any structural improvement within the influence area of ocean waves. In Orange County, these coastal areas take the form of sandy beaches, bluffs, and estuaries. Existing development has in many instances proven to be poorly designed to contend with wave influence. In recent years, numerous bulkheads and structures have been damaged or totally destroyed as a result of the devastating influence of storm-generated waves.

New repair or construction of a dwelling or ocean-protective device in a wave-influence area is required by law to conform to special planning and building regulations. The purpose of this notice is to alert you to the potential hazards of improperly designed oceanfront structures, the existence of new regulations, and to provide guidelines on where to obtain assistance regarding regulations for the design, location, and construction of such structures.

The County of Orange has had a Coastal Flood Plain Development Orange County Coastline Report prepared which contains the technical analysis of portions of the County's unincorporated coastline. It establishes design criteria and standards for structures and protective devices for five distinct coastal areas: Sunset Beach, Emerald Bay, South Laguna, Laguna Niguel, and Capistrano Beach.

Individual flood Plain maps also have been prepared from aerial photos of 3,150± feet of the study areas. They show the topography and shoreline characteristics and contain the following information:

1. Elevating contours, i.e., lines that represent a common elevation above mean level.
2. The landward limit of ocean wave influence based on 1984 conditions. This limit is defined as FP-3 or coastal velocity wave influence area.
3. The limit of protective device encroachment onto the beach. This limit is referred to as the Ocean Protective Device String Line (OPDSL). The line has been determined through the public hearing process and is addressed in the Local Coastal Plan for each respective beach community.
4. Breaking wave elevations. These are essential to the proper design of a protective device and other structures built upon pilings along the coast.

When wave damage repairs or new construction is contemplated on oceanfront properties, the following steps should be taken:

1. Visit the Development Processing Center (DPC), 12 Civic Center Plaza, Room G12, Santa Ana or telephone (714) 834-2470.
   a. Review the zoning and overlay maps to see what limitations apply to the property. County staff will assist.
   b. Review the Local Coastal Plan and Zoning Code for additional restrictions applicable to the property. County staff will aid in locating the pertinent sections of these documents.
   c. Obtain a copy of the Coastal Flood Plain Map which includes the property and determine whether an ocean protective device is needed.
2. Select an appropriate consultant familiar with design criteria of the Coastal Flood Plain Development Report and have a preliminary plan prepared for discussion.
3. Contact EMA Planning to arrange a meeting between County staff and the consultant (Land Planning/Coastal Areas, telephone (714) 834-5380. At this time, information regarding required permits (such as Coastal Development, Grading, and Building Permits) and personnel to coordinate with will be given.

The cost of the report is $9.40 ($1 more if mailed) and is available from the cashier at the Development Processing Center (#1 above). The individual sheets are $4.70 and are only available at the Public Counter, Room 225, 12 Civic Center Plaza, Santa Ana.

FGM:lc
7/23/85

MOFFATT & NICHOL, ENGINEERS

May 6, 1985

Mr. Floyd McLellan
Manager, EMA Development Services
Orange County EMA
P.O. Box 4048
Santa Ana, CA 92702/4048

Dear Mr. McLellan:

It was our pleasure to prepare this manual providing technical criteria and guidelines for the review of structures and coastal protective devices at five coastal reaches in Orange County. The text is also intended as a technical design supplement to zoning, land use, specific plans and Local Coastal Programs, although it does not specifically address them. The criteria and considerations for technical design of coastal protective devices are put forward as guidelines and are not necessarily absolute requirements for design methodology. Site-specific design reports and studies prepared by qualified professionals, possibly using more rigorous methodologies, may be appropriate.

Coastal design data are referenced to 1984 conditions and should be periodically updated. Some should be updated because changes occur with time. These are net shoreline position changes and sea level changes with respect to land, including those which may result from the "greenhouse" effect. Design wave and water surface elevation data, including El Nino-Southern Oscillation effects, can be improved as the data set is lengthened. Some of the data will be collected by others and the County can use their results. Regional information concerning design waves, design still water elevations, and relative sea level changes are in this group. Site specific data such as scour depths, bedrock elevations, and net shoreline position changes, however, can probably only be updated through direct efforts by the county. This latter information could, under proper guidance and direction, be partly obtained by interested volunteers. Our experience working with homeowner associations suggests that competent volunteer resources may be readily available.

In this regard, we recommend the County (1) establish a methodology to update and improve the coastal design data sets as new information becomes available, (2) establish a procedure to obtain local information using a combination of trained County staff and beach-resident volunteers, and (3) establish a geotechnical data bank for coastal design review purposes. The implementation of these recommendations would insure an

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improved data set will be available for future design purposes, including beach restoration and maintenance. Data such as limiting scour levels from a major storm like that which occurred on 27 January 1983 are especially valuable for design purposes. Great opportunities to improve the data base will be available in a plan developed to collect data during and immediately following a major storm.

In addition to improvements in the coastal design data section, the section on coastal design criteria should be updated as new experimental and theoretical results became available.

Beach restoration and/or maintenance is an entire aspect of the County of Orange's beaches not addressed in this report on structures and protective devices. It is certainly an important topic, and one that many of the local civic associations have asked about. Beach preservation is associated, however, with the subject of this manual since a wide recreational beach also serves as a protective device for the private and public structures behind it. While some County beaches appear to be stable, others clearly have experienced recent erosion. Erosion could potentially occur on all of them in the future. Planning and engineering guidance to restore and maintain recreational and protective beaches throughout the County will in the future require a detailed and extensive field investigation of coastal processes. This effort could be reduced if the plan for such an investigation was prepared and a low-cost data collection program initiated prior to a comprehensive field measurement effort.

It was our pleasure to work with you and the County on this important project.

Sincerely,

MOFFATT & NICHOL, ENGINEERS

Craig H. Everts

Craig H. Everts

CHE:kg

COASTAL FLOOD PLAIN DEVELOPMENT

Orange County Coastline

FINAL REPORT

Submitted to: County of Orange

Submitted by: Moffatt & Nichol, Engineers
250 W. Wardlow Rd.
Long Beach, California 90807

Date : 29 April 1985

PREFACE

This report addresses an Orange County requirement for technical criteria and standards as a basis for the review of structures and protective devices on private coastal property. Chapter 1 lists specific objectives of the report and provides background information for use in subsequent chapters. Chapter 2 provides design data based on ocean phenomena, including water surface elevation and wave height, for five coastal reaches in Orange County. Chapter 3 identifies the coastal FP-3 zone in which the ocean phenomena must be considered, and the seaward limit beyond which protective devices may not be constructed. Chapter 4 is a step-by-step aid for checking and evaluating plans for structures and protective devices in the FP-3 zone. Design data from Chapter 2 are applied in Chapter 4.

Detailed design of a structure or protective device requires more information than is provided herein, although the references included are adequate to cover most aspects of design.

This report is a complete document that technically stands on its own. It also serves as a working base amenable to periodic updating as practice, new coastal data, and new methods to analyze the data, warrant. The report and accompanying maps, plates and graphs, will enable the County and private property owners of five unincorporated areas of the County to respond more rapidly to coastal needs. Appendices are provided to justify the recommended design data, and to act as a base for the inclusion of a longer data set in the future.

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DL = design life of structure or protective device

E_d = dune crest elevation

E_g = maximum elevation of natural grade beneath residence

E_u = elevations of underside of residence

H_b = design breaking wave elevation

H_o' = deepwater wave height equivalent to observed shallow water wave if unaffected by refraction and friction

H_r = maximum wave runup elevation

H_s = elevation of protective device or structure

H_{sc} = design scour elevation

H_w = design maximum sea surface elevation

K_D = stability coefficient

R = runup distance above stillwater elevation

R' = runup distance above floor

S_r = specific gravity of armor unit

T = wave period

W = stone weight in pounds

W_a = actual setback distance on plans

W_c = set back distance

W_r = unit weight (lb/ft^3)

W_w = unit weight of salt water (64 lb/ft^3)

Y = year past 1984

d_s = design water depth ($H_w - H_{sc}$)

g = acceleration of gravity (32 ft/sec^2)

h_a = extreme astronomical tide distance above datum

h_b = breaking wave height (a vertical distance)

h_e = time-dependent, long-term vertical distance change in sea surface with respect to land

h_o = highest expected sea surface distance above h_a caused by factors other than h_s

h_s = highest expected surge distance above h_a

h_{wc} = correction to design still water elevation resulting from sea level rise relative to land

m = beach or nearshore profile slope (rise/run)

Δh = change in elevation of benchmark in t

Δt = time period

θ = slope of protective device (rise/run)

CHAPTER 1. INTRODUCTION

GEOGRAPHIC COVERAGE. Five Orange county coastal reaches identified in the Local Coastal Program (LCP) are addressed:

(1) Sunset Beach
(2) Emerald Bay
(3) South Laguna
(4) Laguna Niguel
(5) Capistrano Beach

SPECIFIC OBJECTIVES. The three objectives of this report are:

(1) to establish coastal data that can be used in the design of structures and protective devices on private property in the five LCP areas,

(2) to establish the boundaries of coastal areas where the design of structures and protective devices must consider ocean wave forces and flooding by ocean water, and

(3) to establish design criteria and standards for structures and protective devices in the coastal areas using the design data previously determined.

DEFINITIONS. Some terms are used so frequently and have such specific meanings with respect to the coastal requirements of the County of Orange that it is useful to define them at the beginning of the report.

ARTIFICIAL DUNE - A protective device made of sand designed to prevent or reduce damage by waves and/or ocean waters to the structures behind it.

BEACH SLOPE - Inclination of the beach normal to the immediate shoreline (usually rise/run).

BEDROCK - Any material that is more consolidated and less erodable than the beach sand or other soil which lies above it.

BULKHEAD - A protective device designed as a partition to retain or prevent sliding of the land. A secondary purpose is to protect the upland against damage from wave action.

DESIGN BREAKING WAVE ELEVATION - Highest elevation above the Orange County Vertical Datum (OCVD) that would be directly impacted by breaking waves. The upper limit of breaking waves is based on a design wave height and a design water depth condition with a specified design recurrence interval.

DESIGN LIFE AND RECURRANCE INTERVAL - Orange County requires structures and protective devices be designed for a specific minimum life when acted upon by ocean forces with a specific recurrance interval:

(1) DESIGN LIFE, PROTECTIVE DEVICE: The design life of a non-expendable protective device, which must be equal to or greater than 20 years, is the minimum period after construction during which all major components of the device retain their functional and structural design capabilities.

(2) DESIGN LIFE, STRUCTURE: The design life of the foundation of a non-expendable structure, which must be equal to or greater than 30 years, is the minimum period after construction during which all major components of the foundation system retain their functional and structural design capabilities.

(3) RECURRENCE INTERVAL: Time period during which one coastal design event can be expected to occur. The 100-year recurrence interval, which has been chosen to be used for the design of structures and protective devices in Orange County, is the statistical probability that one event that produces a design magnitude value of a coastal phenomenon

will occur in 100 years, or that it has a one percent probability of occurring in a single year. Recurrence interval should not be confused with design life which references an absolute time interval, not a probabilistic value.

DESIGN SCOUR ELEVATION - Lowest elevation, for design purposes, a beach profile will reach at the OPDS Line (see Appendix B).

FP-3 LINE - Landward boundary of the coastal region (FP-3 zone) in which structures must be protected from ocean-related hazards in Orange County.

MLLW - Mean lower low water, the average height of the lower low waters over a 19-year period.

OCVD - Orange County Vertical Datum based on mean sea level as obtained periodically (about every 10 years) through an analysis of 19 years of tide record. This datum is not fixed with respect to the center of the earth, but rises or falls with respect to it as the mean sea surface along the coast of the County of Orange rises or falls. The OCVD is useful in coastal engineering because many design considerations are keyed to mean sea level or mean lower low water (MLLW). MLLW is 2.83 feet lower than OCVD.

OPDSL - The Ocean Protective Device String Line is the seaward limit beyond which the seaward edge of the crest of a protective device may not extend.

PROTECTIVE DEVICE - A seawall, bulkhead, revetment or artificial dune designed to protect a structure located in the FP-3 zone.

REVETMENT - A protective device consisting of a facing of stone, concrete, cast units, etc., built to protect a scarp, embankment or structure against erosion by wave action or currents.

RUNUP - The rush of water up a protective device, beach, bluff face or structure on the impacting of a wave. The amount of runup is the vertical distance above stillwater level reached by the rush of water. The wave runup elevation limit is the highest elevation that will be reached by the rush of water from a breaking wave when that wave occurs during the design wave event with the specified design recurrence interval. The highest elevation subject to wetting by spray from the design wave will be greater than the runup elevation.

SEAWALL - A protective device designed to separate land and water areas, primarily designed to prevent erosion and other damage due to wave action.

STRUCTURE - Habitable dwelling, cabana, garage, deck or restroom, located in the FP-3 zone of Orange County.

CHAPTER 2. COASTAL DESIGN DATA

Two criteria; acceptable structural behavior and acceptable functional behavior, must be satisfied in the design of a structure or protective device in Orange County. These performance criteria are usually defined with respect to extreme conditions that will occur at the site of the proposed structure or device. Structural behavior refers to the ability of the structure or device to survive extreme conditions. To some extent, protection for extremely rare events may be compromised when occasional property damage is more easily borne than a very high initial construction cost. Functional performance refers to meeting the desired effect of the structure or device, i.e., to protect a building from wave impact forces, to prevent flooding by wave uprush, and to prevent soil loss and hence foundation failure under a building. The functional performance criterion also includes the prevention or mitigation of beach sand losses.

Oceanographic and geologic data are required for the design of structures and protective devices subjected to wave and current activity. Six data categories (Fig. 1) to be considered are given below. Data provided are the minimum for design purposes; the project proponent may wish to design to higher criteria.

(1) design still water elevation, H_w, and future elevations considering anticipated long-term changes in sea surface elevations,

(2) extreme anticipated scour elevation, H_{sc}, and changes in that level with time (design water depth is equal to $H_w - H_{sc}$),

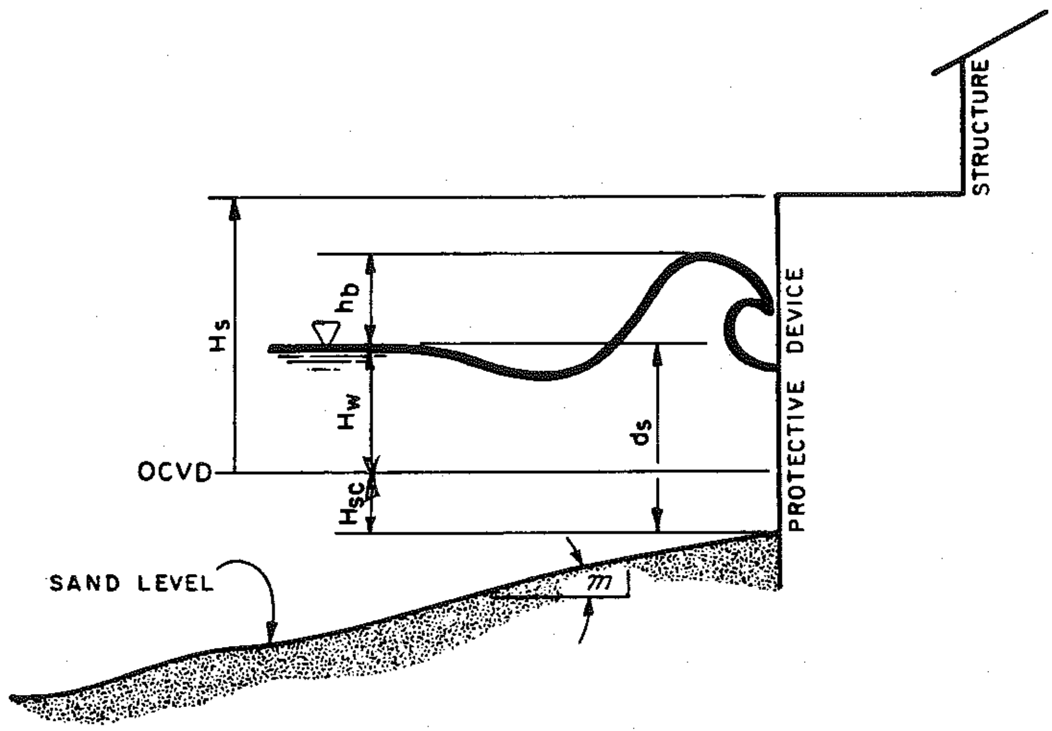
(3) beach or nearshore slope, m,

(4) wave characteristics, including breaking wave height (a distance), h_b; breaking wave elevation, H_b; and wave period, T,

(5) maximum runup elevation, H_r, is equal to H_w + R, where R = runup distance above the stillwater elevation, and

(6) bedrock characteristics

Figure 1 shows some of these data categories in schematic form with H_s = elevation of structure or protective device. A brief definition of each



H_{sc} = DESIGN SCOUR ELEVATION AT OPDSL

H_w = DESIGN STILL WATER ELEVATION

H_s = ELEVATION OF STRUCTURE OR PROTECTIVE DEVICE

d_s = DESIGN WATER DEPTH AT OPDSL

h_b = BREAKING WAVE HEIGHT ABOVE H_w

m = BEACH OR NEARSHORE SLOPE (RISE/RUN)

Figure 1. Definition sketch, coastal design data in Orange County.

data set and its use in design is given in this chapter. The appendices contain more information. Design values for 1984 and changes in these values for subsequent years are illustrated in the following tables and figures:

(1) Figure 2 - Recurrance Interval, Sea Surface Elevation
(2) Table 1 - Design Scour Elevation
(3) Figure 3 - Design Scour Elevation at Capistrano Bay
(4) Table 2 - Beach and Nearshore Slopes
(5) Table 3 - Design Wave Characteristics
(6) Table 4 - Maximum Runup Elevation on a Natural Beach
(7) Table 5 - Bedrock Characteristics

In some cases the values are estimates based on incomplete data. In all cases the data can and should be periodically updated as new data become available. While the values given are considered the best available in 1984, a design based on different data, most notably design scour elevation, may be acceptable if those data can be technically supported.

The data presented in the accompanying tables and figures reflect present conditions and anticipated changes that may occur in the future. For example, as the shoreline retreats (moves landward) in some locations in the future, as based on past trends in shoreline behavior, the maximum scour depth will increase. This will create an increase in design water depth (not design water surface elevation) and a consequent increase in design wave height. The design wave is the largest expected to occur with a 100-yr recurrance interval during the proposed life of the structure.

STILL WATER LEVEL (see Appendix A for additional information).
Still water level is the mean water surface elevation that would exist if waves were absent. It is the base elevation above which waves and uprush occur. Design still water elevation, H_w is one of two parameters that must be established in order to determine the maximum water depth and, hence, the design wave height and runup elevation. The other parameter is minimum bottom elevation, i.e.,

sand scour level, that could occur coincidentally with an abnormally high still water elevation. In a beach environment, the maximum water surface elevation and the minimum bottom elevation can reasonably be expected to occur at the same time during a severe wave storm. For this reason, and because most coastal structures are designed to resist wave forces, the design water surface elevation will be that which would occur during an extreme wave event.

Short and long-term changes in sea surface elevation must be considered. Short-term changes are reversible over the life of a structure or protective device. They are gravity effects caused by astronomical factors, i.e., the position of the earth, moon and sun relative to each other and the distance of the moon and sun from the earth; by weather factors, i.e., storms, which change the barometric pressure and create storm surge by wind stress and wave setup; and climatological factors such as the increased sea surface elevation caused by the El Nino-Southern Oscillation (ENSO) effect. Long-term change, which is assumed irreversable over the life of a structure or protective device, is the slow increase or decrease in the average sea surface elevation relative to the coast. Long-term sea level changes are caused by worldwide changes in the volume and mass of water in the ocean basins, and by changes in the holding capacity of the ocean basins. Long-term sea level changes also occur on a regional scale as a result of land subsidence or uplift relative to the sea surface.

There are several reasons to use field data to establish a design still water elevation. First, there are reasonably long records of water surface elevation within, as well as north and south, of the County of Orange coast. Because the still water maxima exhibit a good correlation between all stations a single maximum elevation can be applied to the entire County of Orange coast. Second: (1) theory is not well developed to predict the perigean spring tide elevation, although the dates of those tides can be accurately forecast; (2) storm surge and wave setup elevations along the County of Orange coast cannot be accurately forecast using theory alone; (3) the water surface rise effect of El Nino at present cannot be accurately forecast because all El Nino events do not have the same effect; and (4) there may be other effects (probably small) that occur during storms which are, at present, not known.

Tsunamis are not considered in the design of small protective devices or structures as herein defined because their effect, while potentially serious, has not been significant on the open County of Orange coast in historic times. The tsunami effect is not additive like the other effects considered because the probability of a tsunami occurring when other extremes occur is very low.

1. Still Water Level Maximum. The maximum recorded water surface elevation in approximately 60 years of record occurred during the 27 January 1983 perigean spring tide and storm event. At that time, the joint probability of many additive mechanisms to raising the water surface was realized. Along the entire, open Orange County coast, the water surface elevation in deep water reached a maximum 5.17 ft above the Orange County Vertical Datum (OCVD) or 8 ft above mean lower low water (MLLW).

Figure 2 shows the recurrance interval for maximum annual still-water surface elevations. The procedure taken, because a relatively long data set (61 years of record at Los Angeles Harbor) is available, was to fit the observed maximum yearly water surface elevation (without wave runup or wave set up) using a Gumbel probability distribution. This approach assumes an extrapolation is applicable, which is justified in the County of Orange case because the extrapolation does not extend to more than twice the period covered by the observed data (Borgman, 1975).

There is a precedent for the H_W = 5.2 ft (OCVD) value. Marine Advisors, Inc. (1965), for example, established a +5.2-ft (OCVD) maximum still water level for the Bolsa Chica shore. They postulated a conservative maximum design value to be an astronomical tide of +7.0 ft (MLLW) plus a storm surge of 1.0 ft. South of the County of Orange, a higher estimate has been made, but for a greater than 1 in

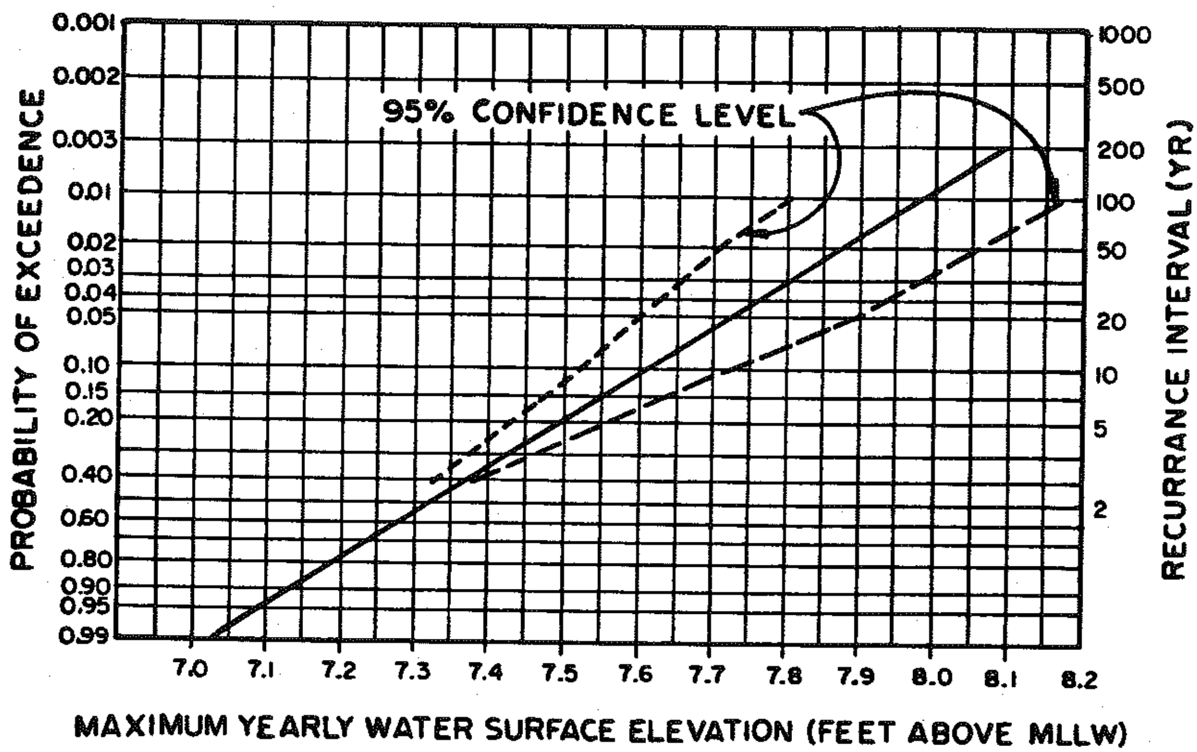


Figure 2. Gumbel distribution of maximum annual sea surface elevation in deep water at Los Angeles Harbor, based on 61 years of data. The 100 year recurrance sea surface elevation is 7.99 ft with a 95 percent confidence that it is between 7.80 and 8.17 ft. (MLLW). The curve was fit by the method of moments.

100 year recurrance interval. Horrer and Joy (1976) in a wave and water surface analysis of the coast at San Onofre, California, estimated the maximum high water elevation to be +9.31 ft (MLLW) coincident with the passage of an inferred intense tropical storm north of the study site. The optimum storm characteristics for barometric surge (+1.2 ft) and wind shear stress and Coriolis surge (+0.78 ft) were assumed to occur simultaneously with a maximum high astronomical tide (+7.0 ft). Horrer and Joy also included an isostatic anomaly of +0.33 ft.

2. Long-Term Change in Still Water Level. In addition to the short-term reversable fluctuations in the sea surface, the effects of a progressive change in sea level must be considered for the life of the structure or protective device. This change must be added to the reversable design elevation of +5.17 ft (OCVD) if sea level is found to be rising relative to land.

The elevation of a structure and/or protective device must be referenced to the OCVD. This is a relatively easy task because a tight network of County of Orange benchmarks (BM's) has been established. The elevation of a BM with respect to the OCVD and its location can be obtained from the County publication titled "Orange County Surveyor, Vertical Control" (Environmental Management Agency). Maps showing the location of all County BM's can be found in a corresponding County publication titled "Orange County Surveyor, Control Maps."

3. Design Still Water Level Calculations. A structure or protective device must be designed for the maximum still water sea surface condition that will occur during its expected life. Design still water elevation, H_w, for a site must be referenced to the BM nearest the project site using the latest level adjustment which will usually be within the last 10 years.

In cases where the sea surface is rising with respect to the BM a correction must be added to the 5.17 ft (OCVD) design still water elevation. This correction provides an estimate of the sea level rise that will occur relative to land at the project site during the design life of a protective device (20 year minimum), or the foundation system of a structure when no protective device is used (30 year minimum). The correction, h_{wc}, obtained from vertical adjustments to the BM given in "Orange County Surveyor, Vertical Control", is

$$h_{wc} = 1.1 \left(\frac{\Delta h}{\Delta t} \times DL\right) \qquad (1)$$

in which:

Δh = change in elevation of the BM nearest the project site over the time period Δt,

Δt = time period as close to the last 10 years as possible, but never less than 6 years, and

DL = design life of structure or protective device.

The corrected design still water elevation, H_w, is therefore

$$H_w = OCVD + H_w + h_{wc} \qquad (2)$$

This is a hindcast procedure. The coefficient 1.1 is used as a time lag adjustment with the assumption the rate of global sea level rise will increase in the future. The h_{wc} will be near zero in most places <u>except</u> Sunset Beach where sea level has been rising relative to land at a recent, but decreasing rate of about 2.5 ft/100 yrs.

<u>EXAMPLE</u>. The design still water elevation for a foundation system is required near the foot of Broadway at Sunset Beach:

Using the County publication "Orange County Surveyor, Vertical Control" the nearest BM is found to be G978. Vertical adjustments since this BM was established are:

Level Year = 1976, 1969

Adjustment MSL elev = 8.023, 8.341

Taking the nearest time interval, Δt, to the last 10 years (1969 to 1976) = 7 yr and calculating the Δh (8.341 - 8.023) the $\Delta h / \Delta t$ is found to be

$$\frac{\Delta h}{\Delta t} = \frac{0.318}{7} = 0.045 \text{ ft/yr}$$

Using a 50 year design life for the foundation system, the added design still water distance is (from Equation 1)

$$h_{wc} = 1.1\ (0.045 \times 50)$$
$$= 2.5 \text{ ft}$$

and the corrected design still water elevation which must be used is (from Equation 2)

$$H_w = 5.17 \text{ ft} + 2.5 \text{ ft}$$
$$= 7.67 \text{ ft (OCVD)}$$

BEACH SCOUR LEVELS (See Appendix B for Additional Information)

The extreme scour elevation, H_{sc}, is required to determine the design water depth and hence design wave height and uprush elevation on a structure or protective device. It is also required to determine the embedment depth of a protective device and the exposed dimension of caissons and piles. Cantilever structures and some piles require a minimum specific length to be buried at all times. Maximum scour level is a required design parameter for structures that require toe stability and for those designed to retain soil behind them without losses from under the structure.

Whenever waves are present, sand moves on beaches. Major movements often occur during wave storms. Waves are more energetic than usual then, and storm-induced, alongshore and offshore-directed currents provide a means to transport the wave-mobilized sand away from or towards the beach. This movement is wholly or partially reversable in that the sand volume lost from the beach during the storm may be partially or completely returned to the beach after the storm. In any given short time period such as a year, the maximum scour limit will usually occur as a result of storm activity. For design purposes the lowest scour elevation estimated at the time of construction must, in the case of a portion of Capistrano Beach, and all of West Street Beach and 1000 Steps Beach, be coupled with long-term trends in scour level. This long-term trend is used to estimate the design scour limit during the life of the structure or protective device.

1. Beach Without Protective Device. Table 1 shows the expected scour depth in the absence of a structure at the Ocean Protective Device String Line (OPDSL) in each LCP area for 1984. At any year, Y, past 1984 the scour elevation at West Street Beach and 1000 Steps Beach can be obtained using the formula given. Figure 3 shows the maximum scour limit without a structure for Capistrano Beach and how that scour limit will likely change in the future. Because the elevation of a portion of Capistrano Beach has been increasing in volume since the 1960's, the lowest scour elevation experienced during storms has become progressively higher along the northeast two-thirds of the reach. The scour elevation along the southeast one-third of the beach is presently decreasing slowly as that shoreline retreats. The scour limit is assumed constant with time at other locations.

2. Beach in Front of Protective Device. A trough will usually form at the toe of the protective device during a storm. The dimensions of the trough will be governed in a complex way by the type of face on the protective device, the type of beach material, and the wave characteristics during the storm. General guidance for estimating the maximum depth of a scour trough below the natural beach scour limit is provided in the Shore Protection Manual (1984) which suggests the scour-trough-depth will be about equal to the height of the maximum unbroken wave that can be supported by the original depth of water at the toe of the protective device.

Clearly the scour depth in beach sand in front of a protective device can be excessive. Consequently, the placement of a rock blanket with

TABLE 1. DESIGN SCOUR ELEVATION

Location	Design Scour Elevation 1984 conditions, ft (OCVD)[1,2,3]	Remarks
Sunset Beach	+11	Continuation of periodic beach replenishment is assumed
Emerald Bay	+1	Stable beach; use bedrock elevation along headlands
South Laguna		
1. Victoria Beach	-3	Scour elevation taken near base of bluffs; beach rotates north or south about center of bay.
2. Aliso Beach	-1	Stable Beach
3. West Street Beach	+1 -0.08 (Y-1984)	Retreating shoreline (1967-1981)
4. 1000 Steps Beach	+1 - 0.08 (Y-1984)	Retreating shoreline (1967-1981)
5. Three Arch Bay	+2	Stable beach; elevation at bedrock near bluffs; based on 27 Jan 83 scour
Laguna Niguel	-1	Bedrock elevation at Dana Strand Beach
Capistrano Beach	See Figure 3	

[1]add 2.83 ft to adjust to MLLW

[2]plus valve indicates elevation is above OCVD, minus valve indicates elevation is below OCVD

[3]unless an adequate rock blanket is provided in front of a protective device this value should not be used

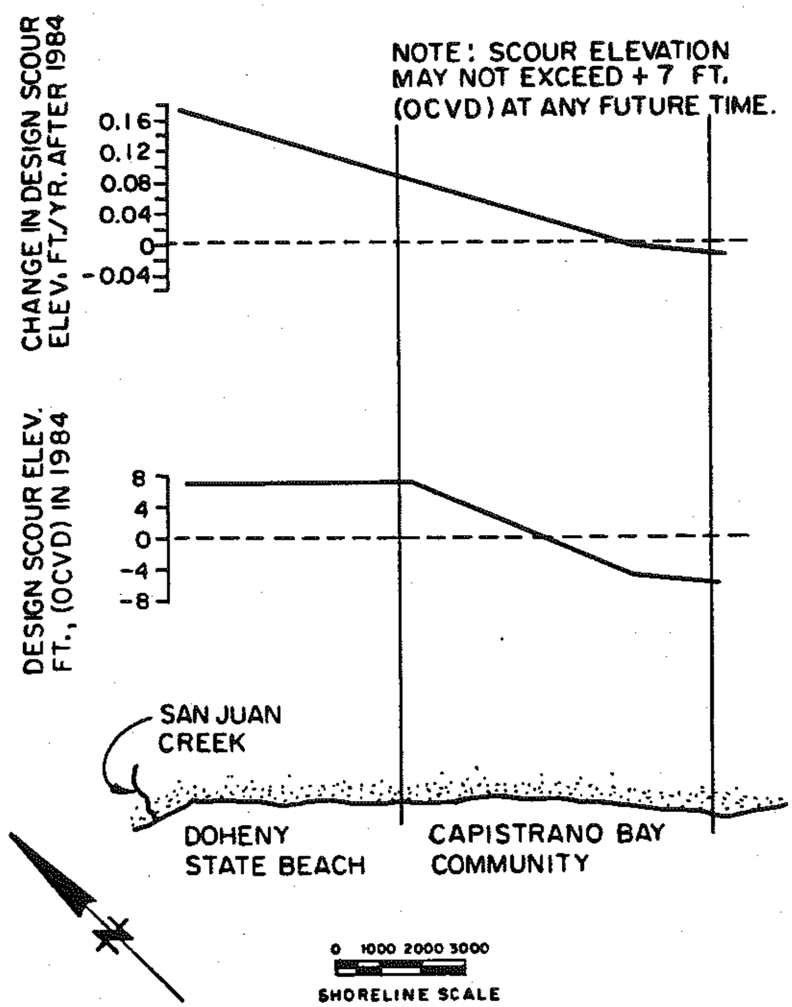


Figure 3. Caspistrano Bay, design scour elevation for 1984, and corrections to the 1984 scour elevations for the period after 1984. Unless an adequate rock blanket is provided in front of a protective device, the scour elevation shown should not be used.

adequate bedding material seaward of the toe of the protective device is highly recommended. This will prevent erosion of the toe and will allow use of the scour limits given in Table 1 and Figure 3. The scour blanket will be discussed in detail in Chapter 4. It can be provided as a toe revetment in front of a seawall or bulkhead, and as excess stone on the face of or at the toe of a revetment.

Unless an adequate rock blanket is provided at the toe of a protective device, the maximum scour elevations given in Table 1 and Figure 3 should not be used.

BEACH AND NEARSHORE SLOPE

To determine the maximum wave height and runup distance in front of an existing or proposed structure or protective device, the beach slope, m, must be known. Beach slopes given in Table 2 were obtained from onsite measurements, and from estimates made using historic horizontal ground photographs, most of which were provided by the Los Angeles District, Corps of Engineers. At Sunset Beach the slopes have changed with time, especially the beach slope. Prior to the fill program which begin in the late 1940's, the beach slope was very gradual. Some residents stated they could walk out almost 1000 ft from the property line at low tide. Since artificial nourishment began, the beach has become progressively steeper. The values given in Table 2 are for the present beach.

The nearshore slope values provided in Table 2 from the OCVD line (+2.8 ft, MLLW) to depths of 12, 24 and 36 ft were taken from Corps of Engineers Survey data (Corps of Engineers, 1970), and National Ocean Services charts.

WAVE CHARACTERISTICS (see Appendix C for Additional Information)

Design wave height is used, with other data, to establish the design wave load on a structure or protective device or on elements of a structure or device. It is also used, with the still water elevation, to calculate the design breaking wave height and wave uprush elevation.

TABLE 2. BEACH AND NEARSHORE SLOPES[1]

LOCATION	BEACH SLOPE, ABOVE OCVD,[2]	OCVD to -12 ft.	NEARSHORE SLOPE OCVD to -24 ft.	OCVD to -36 ft.
Sunset Beach	0.2/0.03	0.022 (0.02)[3]	0.012	0.0065
Emerald Bay	0.2/0.02	0.08 (0.02)	0.06	0.05
South Laguna				
1. Victoria Beach	0.2/0.02	0.09 (0.02)	0.05 (18 ft shoal offshore)	0.04
2. Aliso Beach	0.2/0.02	0.10 (0.02)	0.09	0.05
3. West Street Beach	0.2/0.02	0.09 (0.02)	0.08	0.06
4. 1000 Steps Beach	0.2/0.02	0.06 (0.02)	0.04	0.04
5. Three Arch Bay	0.2/0.02	0.06 (0.02)	0.04 (18 ft shoal offshore)	0.04
Laguna Niguel	0.2/0.015	0.023 (0.02)	0.02	0.018
Capistrano Beach	0.2/0.02	0.025 (0.02)	0.015 (many shoals 12-18 ft. offshore)	0.008

[1]rise/run

[2]First value = typical maximum slope; second value = typical minimum slope

[3]maximum slope at design scour limit conditions

Deepwater significant wave heights in excess of 20 feet can be expected to recur during the life of a structure or protective device along the Orange County coast. Examples are 24 ft (27 January 1983) and 27 ft (15-25 September 1939). Island sheltering effects will probably not decrease extreme wave heights east of the islands by more than 20 percent (as was the case of 1939 hurricane waves at Oceanside). Assuming the approach of the extreme waves is normal to shore and diffraction is not a factor, all design waves which act upon protective devices along the County of Orange coast fronted by a sandy beach will be depth dependent. The design breaking wave can be calculated when water depth and slope in front of the structure are known. Because of deep water conditions, waves breaking on rocky headlands may not be depth dependent. Wave characteristics for design purposes are not covered for headlands in this discussion. Wave setup is included in runup calculations.

1. Design Water Depth. Design water depth, d_s, is

$$d_s = H_w - H_{sc} \tag{3}$$

in which H_w = time-dependent maximum still water elevation (5.2 ft OCVD + sea level rise correction, if applicable) taken for the year of construction plus the design life of the structure or protective device; and H_{sc} = time-dependent scour elevation. Scour elevation changes must be considered in the design life of the structure or protective device at West Street Beach, and 1000 Steps Beach (Table 1) and Capistrano Bay (Fig. 3). Sea level rise must be considered, but may not be significant, at all locations.

2. Design Wave Period. A wave period, T, of 22 seconds, the peak spectral period of the 27 January 1983 storm, is taken as the design period to obtain the design breaking wave height. A long period design wave is justified because as wave period increases, the breaking wave height increases for a given water depth. Alternatively, for runup calculations wave periods between 8 and 22 seconds should be considered. Runup is a function of the characteristics of the protective device and the maximum runup distance will probably not occur for the T=22 second wave.

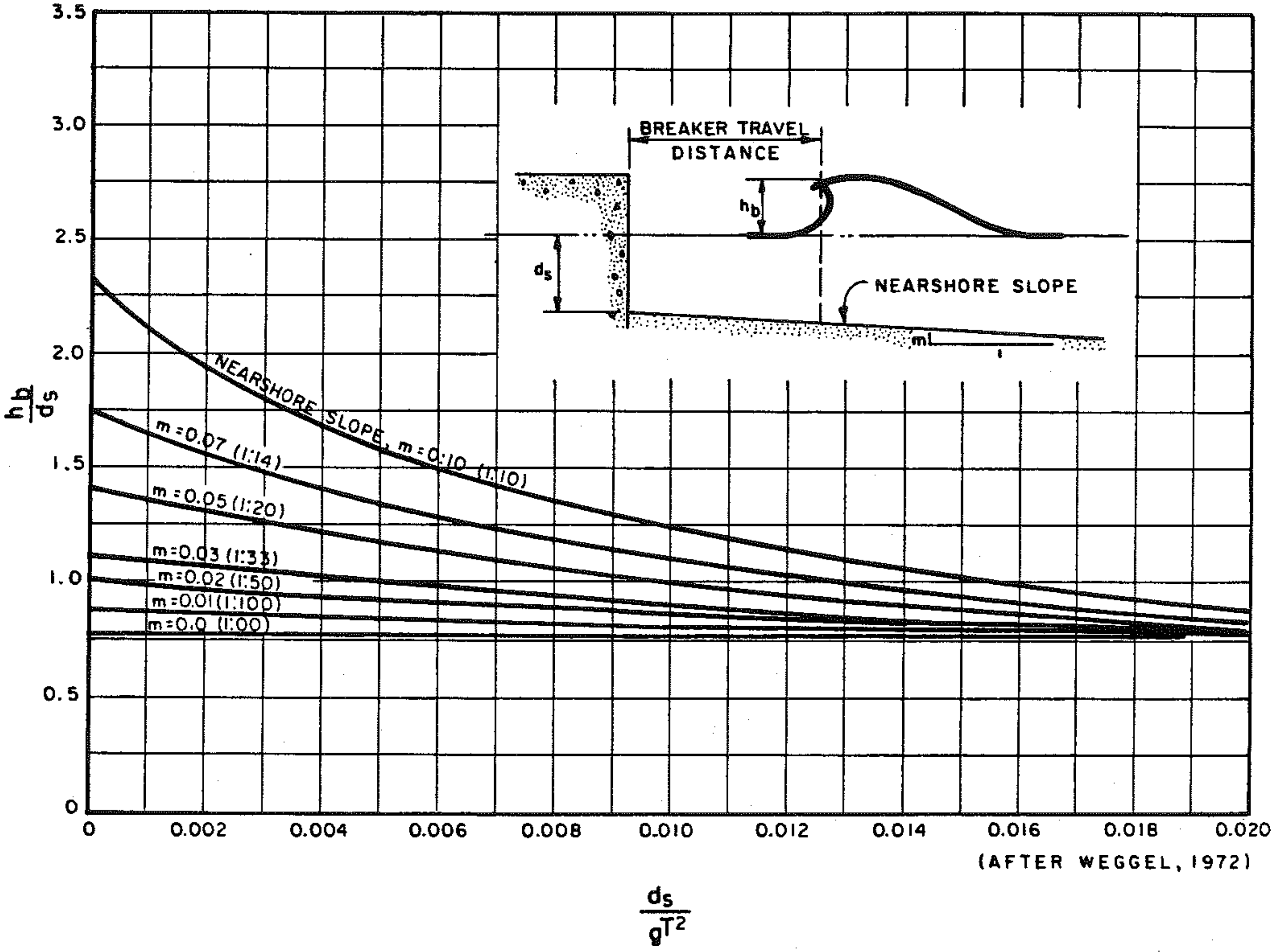


Figure 4. Dimensionless design breaker height versus relative depth structure. (Shore Protection Manual, 1984).

3. Design Breaker Height. Attack by the design breaking wave, h_b, is considered the extreme condition the structure or protective device must survive. Figure 4 can be used to determine h_b, when the slope, m, (Table 2) in front of the toe of the protective device or structure, and d_s, the design water depth is known. The design breaking wave height at each LCP area is provided in Table 3. Values given in this table are for the condition that a scour trough is not formed. However, a scour trough would probably not significantly increase the breaker height. Unless the beach slope was steep, depth-limiting conditions seaward of the trough would allow a breaking wave only slightly higher than h_b on the structure or protective device.

Using a conservative approach based on Goda's (1975) work, the maximum height of a breaking wave, H_b, above the OCVD, is

$$H_b = H_w + h_b \qquad (4)$$

The value of H_b is also given in Table 3.

4. Wave Approach Direction. Table C-1, in Appendix C shows that wave energy from storms can come from the south, west, or north, and that, except in special circumstances, structures and protective devices should be designed for deepwater wave energy from these open-water directions.

DESIGN RUNUP ELEVATION - FLOODING CONSIDERATIONS.

Wave runup is the rush of water above still water elevation up a beach, structure or protective device after a wave has broken. Flooding along the County of Orange coast is often caused by a high stillwater elevation and moderate to severe wave events, which produce high runup elevations. The design runup elevations given in Table 4 for natural beaches is the minimum elevation that would be required of a natural beach or gently-sloping dune to prevent flooding beyond it. Maximum runup on some representative protective devices and bluffs are given on the hazards maps (Chapter 3). Values in Table 4, based on the storm of 27 January 1983, with T=22 sec, were used to establish the FP-3 Line and should not

TABLE 3. DESIGN WAVE CHARACTERISTICS[1]

Location	Design Breaking Wave Height, h_b (ft)[2]	Design Maximum Elevation of Breaking Wave; H_b (ft, OCVD)[3]
Sunset Beach	none[4]	
Emerald Bay	4	9.2
South Laguna		
1. Victoria Beach	8	13.2
2. Aliso Beach	6	11.2
3. West Street Beach	4	9.2
4. 1000 Steps Beach	4	9.2
5. Three Arch Bay	3	8.2
Laguna Niguel	6	11.2
Capistrano Beach	(variable, use Figures 3 and 4)	

[1]using 1984 design still water elevation; 1984 design scour depth; both at the OPDSL

[2]from Figure 4

[3]Assumes H_b is entirely above design still water elevation

[4]Assumes maintenance of wide protective beach

be used as an estimate of the runup elevation on a protective device.

Table 4. RUNUP ELEVATION ON THE NATURAL BEACH[1]

Location	Maximum Runup Elevation ft above OCVD	Remarks
Sunset Beach	15.2	Verified by videotape of event
Emerald Bay	14.2	Verified by observations ± 1 ft
South Laguna	14.2	Verified by observations ± 2 ft
Laguna Niguel	15.2	Verified by photographs ± 1 ft
Capistrano Beach	15.2	Verified by observation and photographs ± 1 ft

[1]based on 27 January 1983 storm data, T = 22 seconds

BEDROCK CHARACTERISTICS

The elevation of the bedrock surface and the geotechnical characteristics of the bedrock material are design considerations when a structure or protective device can be founded on or in the bedrock formation. Bedrock is considered to be any material that is more consolidated and less erodable than the beach sand which lies above it. A seawall, revetment or bulkhead when founded on or within bedrock will resist lateral and vertical wave and soil loads to a greater extent than a structure anchored in sand at the same depth. A structure or protective device correctly tied to bedrock with appropriate filter material will also retain the soil behind it. It should be noted that exposed bedrock can also erode under wave action.

For design purposes, the services of a geotechnical professional is required. Bedrock elevation and the geotechnical characteristics of the material may change greatly from one location to another, even from one coastal lot to the next. General bedrock information is provided in Table 5.

TABLE 5. BEDROCK CHARACTERISTICS

Location	BEDROCK ELEVATION, ft, OCVD	BEDROCK TYPE	REMARKS	REQUIREMENT FOR GEOTECHNICAL DATA FROM SITE
Sunset Beach	First formation: variable, about (-3)	Capistrano Clay (about 2 ft thick)	two horizons, difficult to drive piles through	Yes
	Second formation: variable, about (-7)	Catalina Clay		
Emerald Bay	+1		bedrock location at base of bluffs is unknown but probably near -3 ft (OCVD)	Yes
South Laguna				
Victoria Beach	-3		bedrock location at base of bluffs is unknown but probably near -3 ft (OCVD)	Yes
Aliso Beach	-1		bedrock location at base of bluffs is unknown but probably near -3 ft (OCVD)	Yes
West Street Beach	greater than -1		bedrock location at base of bluffs is unknown but probably lower than -3 ft (OCVD)	Yes
1000 Steps Beach	+1		bedrock location at base of bluffs is unknown but probably near -3 ft (OCVD)	Yes
Three Arch Bay	+2 to +5	Various sedimentary rocks (San Onofre Breccia)		Yes
Laguna Niguel	-1			Yes
Capistrano Bay	variable, -6 to -1	Capistrano Bay Formation	silty clay to siltstone	Yes

CHAPTER 3. COASTAL BOUNDARIES AND COASTAL DESIGN MAPS

Structures in certain areas along the coast of Orange County must be designed to resist wave forces and flooding from the ocean. Protection from ocean phenomena may or may not involve the use of protective devices. The zone in which ocean phenomena must be considered and the seaward allowable limit at which the crest of a protective device may be constructed are defined in this chapter. These locations are also given on a series of Coastal Design maps for the five LCP areas of the County of Orange. The maps may be obtained from The Environmental Management Agency of the County.

BOUNDARIES SHOWN ON MAPS.

1. FP-3 LINE. The Flood Plain-3 boundary must be considered in the design of a structure along the open ocean coast of Orange County. Structures located seaward of the FP-3 boundary, shown as a thick dashed line on the maps, must be protected from wave activity. The boundary is an imaginary vertical plane separating ocean and land regions. On an undeveloped beach or bluff during a storm, the landward limit of wave uprush would be near this line. In most instances the FP-3 line has been located at sites that can be readily identified on the ground. Locations are given for FP-3 boundaries based on shore type:

Stable Beach Areas Backed by Bluffs. At bluff-backed portions of Emerald Bay, Victoria Beach, Aliso Beach, Three Arch Bay, Salt Creek Beach and Dana Strand Beach, where the shoreline is considered stable in 1984 (Appendix B), the FP-3 boundary is located at the design runup limit at an elevation of between 14.2 ft (OCVD) and 15.2 ft (OCVD) on the beach, or backshore, or on the bluffs (Table 4). This location is not based on the existance of past, present and future structures or protective devices. Design runup elevation under natural conditions is based upon the significant design wave height, (highest one-third of all waves during the design event), consequently a few waves during the design event could cause runup to reach an elevation higher than that given on the maps.

It is useful here to explicitly define the location of the base of the bluff as the intersection of a plane created by the fixed, or in-place, bluff face and the sandy beach. Bluff talus, and/or wind-blown beach sand or other alterations of the bluff profile are not considered a part of the bluff face.

Non-Stable Beach Areas Backed By Bluffs. At Thousand Steps Beach and West Street Beach, where the shoreline experienced a net retreat in the period 1967-1981 the design runup elevation will increase slightly with time. However, because the increase will be so slight on the steep bluffs, the location of the FP-3 boundary is considered fixed until at least 1993. It is located at the runup limit for the design storm of 27 January in 1984.

Capistrano Beach. This region is also backed by a natural bluff. However, the railroad grade seaward of the bluff at the present time exceeds the 15.2 ft (OCVD) design runup elevation. At Capistrano Beach the seaward intercept of the railroad grade at elevation 15.2 ft(OCVD), because of its permanence, is designated the FP-3 boundary.

Non-Bluff Areas Adjacent to Bluff Backed Beaches. The creek valleys at Emerald Bay, Aliso Beach and Salt Creek Beach fall under this heading. Using the assumption that no structures are present, the intersection of the maximum calculated runup elevation with the ground surface for the design storm is designated the FP-3 boundary. That location,the runup elevation between the bluffs, does not change in a landward direction. Drag on the runup flow as it moves inland slightly reduces the runup elevation, while flow convergence as it moves inland increases the runup elevation. These are considered to be balanced.

Sunset Beach. Sunset Beach is a special case. The selection of the boundary is complicated because the region from the ocean to the channels at Huntington Harbor lies below the maximum runup elevation on and behind the natural beach (Appendix B, Fig. B2). A second complication is introduced because of the non-permanent nature of the beach. As previously stated, Sunset Beach is now artificially maintained so that beach stability is primarily dependent upon human factors and not nature. The predictability of shoreline change at Sunset Beach based on past natural beach behavior and an understanding of processes responsible for erosion and deposition, therefore, cannot be assumed without assuming human intervention for future maintenance.

The establishment of the FP-3 boundary at Sunset Beach is predicated on the periodic beach nourishment program (or some other solution) continuing into the future. If beach erosion is allowed to progress beyond that which occurs over a 3-7 year replenishment period (Appendix B, Fig. B2) the FP-3 boundary will be relocated in a landward direction.

At Sunset Beach, the FP-3 boundary is designated at the north (South Pacific Avenue) property line of the ocean-facing houses. The highest natural elevation on Sunset Beach is the berm crest or the dune line in front of these houses. Wave runup during a design event will pass between the houses. The flow velocity reaching the parking lot between South Pacific and North Pacific Avenues will have declined such that serious problems caused by flowing water in non-street areas will no longer exist. Flooding will still occur landward of the FP-3 line but damage by currents will be small compared to damage by submersion. The rear of the lots was also selected to aid the legal description of the FP-3 line.

2. OPDS LINE. The Ocean Protective Device String Line, shown as a thin dashed line on the maps, is the seaward limit beyond which the crest of protective devices may not be constructed. Technical criteria were not considered in locating the OPDSL. All design data presented on the maps and in Chapter 2 are referenced to the OPDSL.

COASTAL DESIGN DATA SHOWN ON MAPS

The following data may be used for initial plan check purposes at all sites except Sunset Beach where it does not apply. Data reference 1984 conditions at the OPDSL only.

1. DESIGN BREAKING WAVE ELEVATION. This is the maximum elevation above the OCVD that breaking wave forces will affect a structure or protective device at the OPDS Line in 1984. These data are given in Table 3. Certain conditions may render the 1984 values inaccurate. If sea level is rising relative to land or if the shoreline is experiencing a long-term net retreat, the design breaking wave elevation must be increased. Sea level rise over the life of a structure or protective device must be considered at all locations with special consideration at Sunset Beach. Shore retreat in the period 1967-1981 occurred at the southeast end of Capistrano Beach (Fig.3) and West Street and 1000 Steps Beaches (Table 1). The consequences of a net shore retreat is an increasing scour depth through time and an increase in the breaking wave height.

2. WAVE RUNUP ELEVATION LIMIT ON A SMOOTH VERTICAL WALL. This is the maximum elevation above OCVD that runup from a breaking wave at the OPDSL would reach on a smooth, vertical seawall or bulkhead in 1984. In addition to characteristics of the protective device, maximum runup elevation is a function of water depth in front of the device and the characteristics of the design wave that breaks on the device. As discussed previously, these parameters may vary over the life of a protective device. When that is the case, the value shown on the maps may not be used and procedures discussed in Chapter 4 must be applied.

Runup is the rush of water up the vertical wall on the breaking of a wave. The amount of runup is the vertical distance above stillwater level that the rush of water attains when the wave occurs during the design wave event with the specified design recurrence interval. The highest elevation subject to wetting by spray from the design wave will be greater than the runup elevation.

3. WAVE RUNUP ELEVATION LIMIT ON A ROUGH 1:1.5 (RISE/RUN) SLOPED REVETMENT.

Like the runup values for a smooth vertical wall, this value is given on the maps to assist a plan-checker. A revetment with a 1:1.5 (rise/run) is a relatively common protective device along the Southern California coast. The values for 1984 conditions should be used with caution because design conditions may change with time as previously discussed.

CHAPTER 4. DESIGN CRITERIA FOR PROTECTIVE DEVICES AND PILE OR CAISSON-SUPPORTED STRUCTURES

Previously-established ocean and beach data are integrated into a set of plan-check guidelines in flow-diagram form in this chapter. The guidelines apply to structures and protective devices to be constructed in the FP-3 zone of Orange County. These guidelines are provided to assist the County in checking plans of structures and protective devices to meet the design objectives provided below. Criteria related to earth forces are not addressed and must be established by a geotechnical investigation. A list of the required geotechnical information is included where appropriate.

DESIGN OBJECTIVES

Five design objectives stated below must be considered:

(1) Protect structures from wave impact damage,
(2) Protect structures from erosion of underlying soils,
(3) Protect structures from ballistic damage by floating and other wave-transported debris,
(4) Protect structures from flood damage caused by ocean phenomena, and
(5) Minimize adverse impacts on adjoining property/structures resulting from construction of a protective device or a structure.

Ecological concerns and design appearance on the beach should be considered, but are addressed at the Local Coastal Program level.

FLOW DIAGRAM FOR FP-3 ZONE PLAN CHECK

The following plan-check procedure should be used in the step-by-step order it is shown in the flow diagram of Figure 5. Decision points which require technical input are shown as boxed STEPS. When followed through in its entirety, the appropriate design objectives will be addressed.

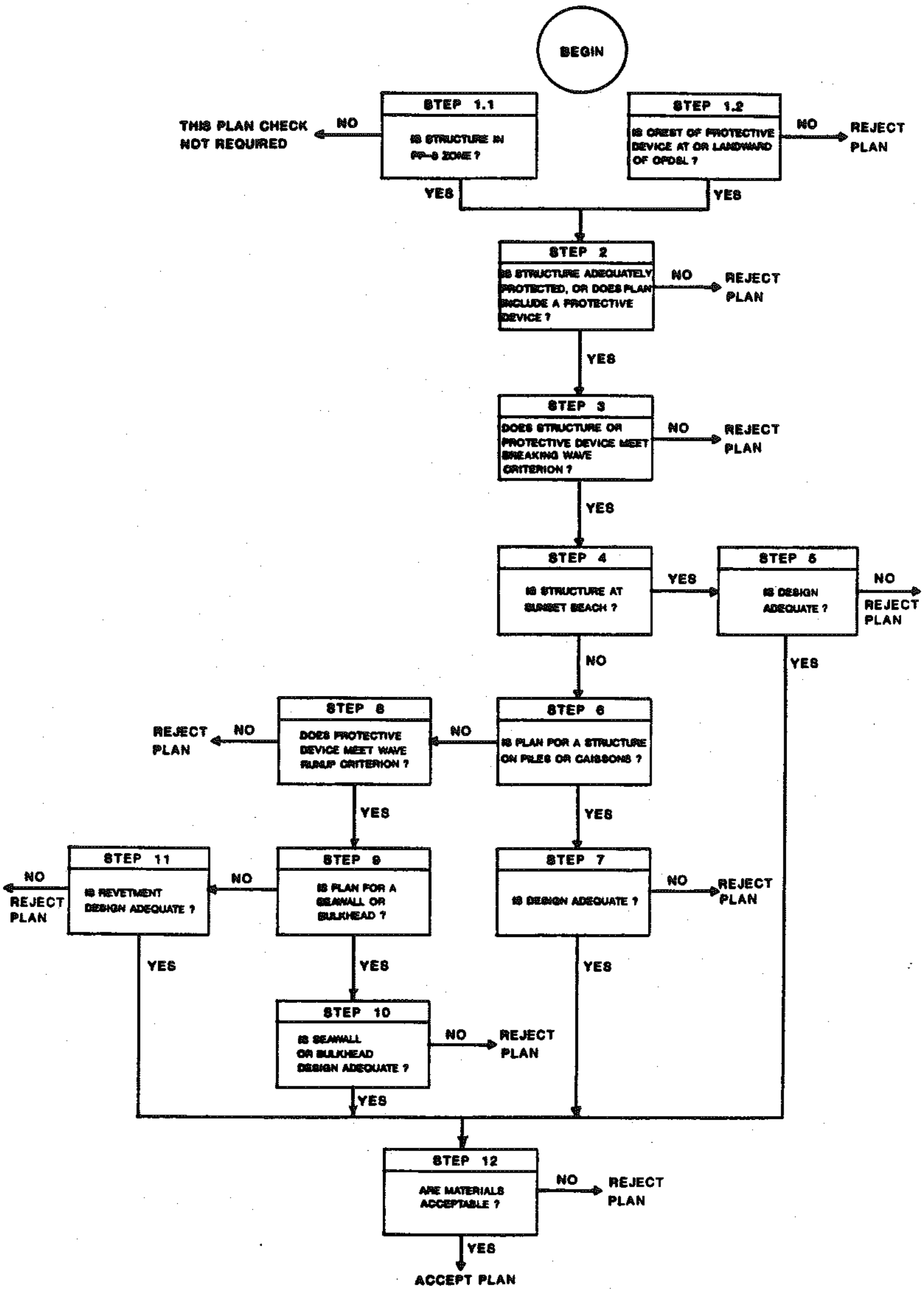


Figure 5. Plan Check Flow diagram.

This plan check procedure is the establishment of design based on an evaluation of the level of protection the structure or protective device provides against design wave conditions. An evaluation is made based on one of three categories:

CATEGORY 1: significant dynamic wave forces will be present above the underside of the structure or over the crest of the protective device,
CATEGORY 2: overtopping by wave runup will occur without significant wave forces, and
CATEGORY 3: no overtopping will occur.

Without a special design to account for wave forces on a structure, structures or protective devices in CATEGORY 1 are considered unacceptable. Overtopping without wave forces is the most common circumstance; CATEGORY 2 encompasses probably 90 percent of the low-lying private structures in Orange County. Structures in this category require floodproofing. No special design for wave phenomena is required of a CATEGORY 3 structure. However, this condition is rare in low-lying oceanfront structures in Orange County.

1. STEP 1. LOCATION OF STRUCTURE OR PROTECTIVE DEVICE. This step is used to determine whether a proposed structure is in the FP-3 zone and, therefore, whether the guidelines in this coastal plan-check procedure are applicable, and, if a protective device is proposed, whether it is properly located.

1.1 IS THE STRUCTURE IN THE FP-3 ZONE?

NO - disregard these coastal design guidelines

YES - GO TO STEP 2 IF A PROTECTIVE DEVICE IS NOT PROPOSED; GO TO STEP 1.2 IF A PROTECTIVE DEVICE IS PROPOSED

1.2 IS THE CREST OF THE PROTECTIVE DEVICE AT OR LANDWARD OF THE OPDSL?

NO - protective device not acceptable

YES - GO TO STEP 2

2. STEP 2. DESIGN SITUATION. In this step the type of proposed structure and/or protective device is identified and the appropriate next step is established:

2.1 Situation A: Proposed structure is supported by piles or caisons without a protective device - GO TO STEP 3

2.2 Situation B: Proposed structure is located behind a protective device - GO TO STEP 3

2.3 Situation C: Proposed structure is located in the FP-3 zone but is not protected by a protective device and is not on piles or caissons - DESIGN IS CONDITIONALLY UNACCEPTABLE

A structure located in the FP-3 zone is considered conditionally unacceptable when it is not adequately supported by piles or caissons or is not protected by a protective device. Such a structure may be susceptable to breaking wave forces, flooding, loss of sand as the beach erodes and other ocean-related phenomena.

2.4 Situation D: The design is for a protective device - GO TO STEP 3

3. STEP 3. DETERMINE THE FUNCTIONAL SUITABILITY OF THE STRUCTURE OR PROTECTIVE DEVICE. The elevation of the breaking design wave with respect to the crest elevation of a protective device or the lowest horizontal supporting member of a structure is addressed in this step.

Figures 6 through 8 illustrate design breaking wave conditions for situations that will be encountered in the County of Orange. The beach profile shown represents the lowest natural beach sand level expected during the design life of the structure or protective device. The design scour level is given for the OPDS Line. If the most seaward piles or caissons (structure) or the crest of a protective device is not located at the OPDS Line, the elevation of the scour level is obtained by projection of the beach slope equal to 1/50 (rise/run) landward from the OPDS Line. This gentle slope is assumed to be the slope that would evolve during a severe storm. If bedrock will be exposed during design conditions (Table 5), the bedrock surface elevation at the site of the piles or caissons or protective device should be used.

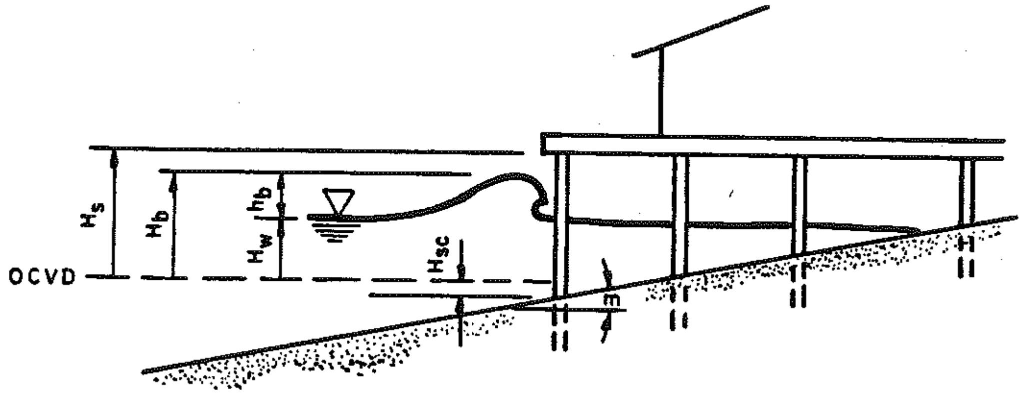
Figures 6-8 are used to determine if the structure or protective device meets its functional (breaking wave) design criterion. One of the following structure or protective device situations must be selected:

3.1 Figure 6. The structure is supported by piles or caissons without an adjacent protective device.

3.2 Figure 7. The seawall or bulkhead is used to protect a structure or bluff.

3.3 Figure 8. The revetment is used to protect a structure or bluff.

The design is considered conditionally unacceptable if $H_s < H_b$. In other words an unacceptable design is such that the crest elevation of the design breaking wave, H_b, is higher than the crest of the protective device, H_s. The design can be made acceptable by designing for wave impact forces. Costs to construct a private dwelling, cabana, garage, deck or restroom for direct wave-impact forces will, in most cases, preclude such a design. This report does not provide information to check a plan that considers wave-impact forces on a principal structure.



H_s = ELEVATION OF STRUCTURE
H_w = STILL WATER ELEVATION
H_b = ELEVATION OF BREAKING WAVE
H_{sc} = SCOUR ELEVATION
m = BEACH SLOPE (RISE/RUN)
h_b = HEIGHT OF BREAKING WAVE

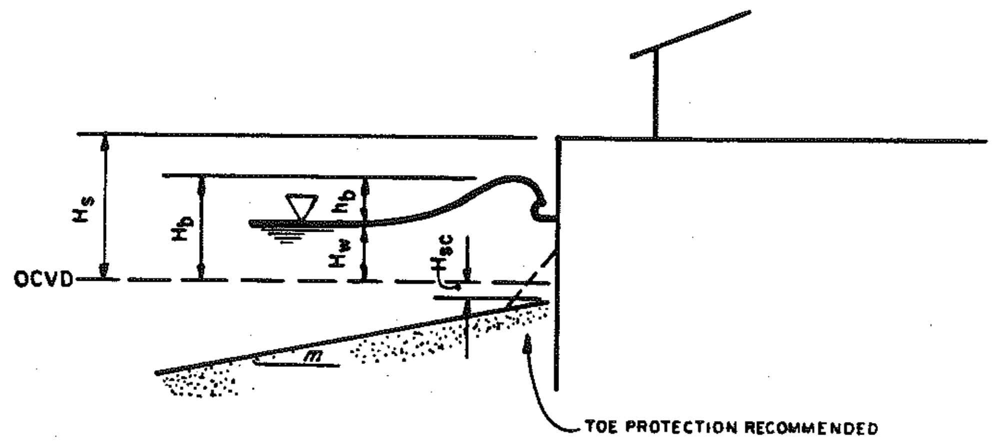
Figure 6. Structure supported by piles or caissons without adjacent protective device (figure not to scale).

STEP 3.1 OBJECTIVE: To determine whether waves will break on structure.

DATA: If H_w and H_{sc} will not change through structure life: use H_s from plans, $H_b = H_w + h_b$ from Table 3 (if $H_{sc} \geq H_w$, $h_b = 0$).
If H_w and/or H_{sc} can be expected to change during the design life of the structure so as to increase the water depth $H_w - H_{sc}$:

(1) Use procedure described in "Long-Term Change in Still Water Level" (Chap. 2) to calculate H_w,
(2) Use Table 1 or Figure 3 to calculate H_{sc},
(3) Use Figure 4 to establish h_b: water depth, $d_s = H_w - H_{sc}$; $T = 22$ sec, $g = 32$ ft/sec^2, $m = 0.02$.
(4) $H_b = H_w + h_b$

PROCEDURE: If $H_s \leq H_b$ DESIGN IS CONDITIONALLY UNACCEPTABLE
If $H_s > H_b$ GO TO STEP 4



H_s = ELEVATION OF WALL CREST
H_w = STILL WATER ELEVATION
H_b = ELEVATION OF BREAKING WAVE
H_{sc} = SCOUR ELEVATION
h_b = HEIGHT OF BREAKING WAVE
m = BEACH SLOPE (RISE/RUN)

Figure 7. Protective device is a vertical wall (seawall or bulkhead). Figure is not to scale.

STEP 3.2 OBJECTIVE: To determine whether waves will break above seawall bulkhead.

DATA: If H_w and H_{sc} will not change through structure life: use H_s from plans, $H_b = H_w + h_b$ from Table 3 (if $H_{sc} \geq H_w$, $h_b = 0$).
If H_w and/or H_{sc} can be expected to change during the design life of the structure so as to increase the water depth, $H_w - H_{sc}$:

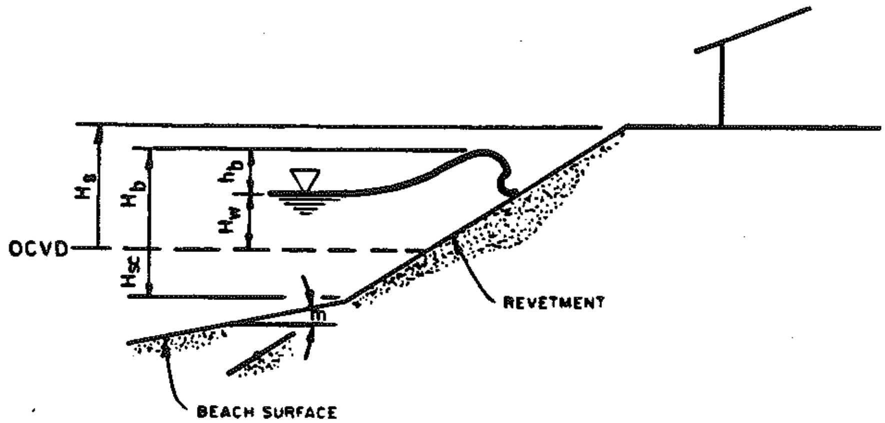
(1) Use procedure described in "Long-Term Change in Still Water Level" (Chapt. 2) to calculate H_w,

(2) Use Table 1 or Figure 3 to calculate H_{sc},

(3) Use Figure 4 to establish h_b: water depth, $d_s = H_w - H_{sc}$; $T = 22$ sec, $g = 32$ ft/sec^2, $m = 0.02$.

(4) $H_b = H_w + h_b$

PROCEDURE: If $H_s \leq H_b$ DESIGN IS CONDITIONALLY UNACCEPTABLE
If $H_s > H_b$ GO TO STEP 4.



H_s = HEIGHT OF REVETMENT
H_w = STILL WATER ELEVATION
H_b = ELEVATION OF BREAKING WAVE
H_{sc} = SCOUR ELEVATION
m = BEACH SLOPE (RISE/RUN)
h_b = HEIGHT OF BREAKING WAVE

Figure 8. Protective device is sloped (usually a revetment). Figure is not to scale.

STEP 3.3 OBJECTIVE: To determine whether waves will break above revetment.

DATA: If H_w and H_{sc} will not change through structure life: use H_s from plans, $H_b = H_w + h_b$ from Table 3 (if $H_{sc} \geq H_w$, $h_b = 0$).
If H_w and/or H_{sc} can be expected to change during the design life of the structure so as to increase the water depth, $H_w - H_{sc}$:

(1) Use procedure described in "Long-Term Change in Still Water Level" (Chapt. 2) to calculate H_w,
(2) Use Table 1 or Figure 3 to calculate H_{sc},
(3) Use Figure 4 to establish h_b: water depth, $d_s = H_w - H_{sc}$, $T = 22$ sec, $g = 32$ ft/sec^2, $m = 0.02$.
(4) $H_b = H_w + h_b$

PROCEDURE: If $H_s \leq H_b$ DESIGN IS CONDITIONALLY UNACCEPTABLE
If $H_s > H_b$ GO TO STEP 4.

4. STEP 4. GEOGRAPHIC LOCATION. Is the structure or protective device located at Sunset Beach?

YES - GO TO STEP 5

NO - GO TO STEP 6

STEP 5. SUNSET BEACH (See Appendix D for more information). Sunset Beach is a special case where ocean-water creates a flooding problem. Flowing water over or through gaps in the artificial dune is the only design consideration. Design guidelines are based on the assumption that the beach will be artificially maintained with a width of at least 150 ft from the ocean-facing private property line. Should the beach retreat to less than that width, the possibility of storm erosion and breaking wave activity at the structures may occur. Then, flood protection as regulated by these design guides will not provide protection from wave impact forces. Design checks must then be modified to reflect the circumstances. This would probably be under temporary emergency conditions.

5.1 Specific Design Objectives. Structures in the FP-3 zone at Sunset Beach must be specifically designed to:

a. prevent flooding within the structure and prevent flood damage to components of the structure, and

b. convey salt water away from the structure.

5.2 Design Considerations.

5.2.a. Foundations. The structure must be supported on piles or caissons with a minimum pile or caisson length of 20 ft.

5.2.b. Elevation of Underside of Structure. The minimum vertical distance, $E_u - E_g$, as defined in Figure 9, between the ground elevation, E_g, and the underside of the structure, E_u, is dependent

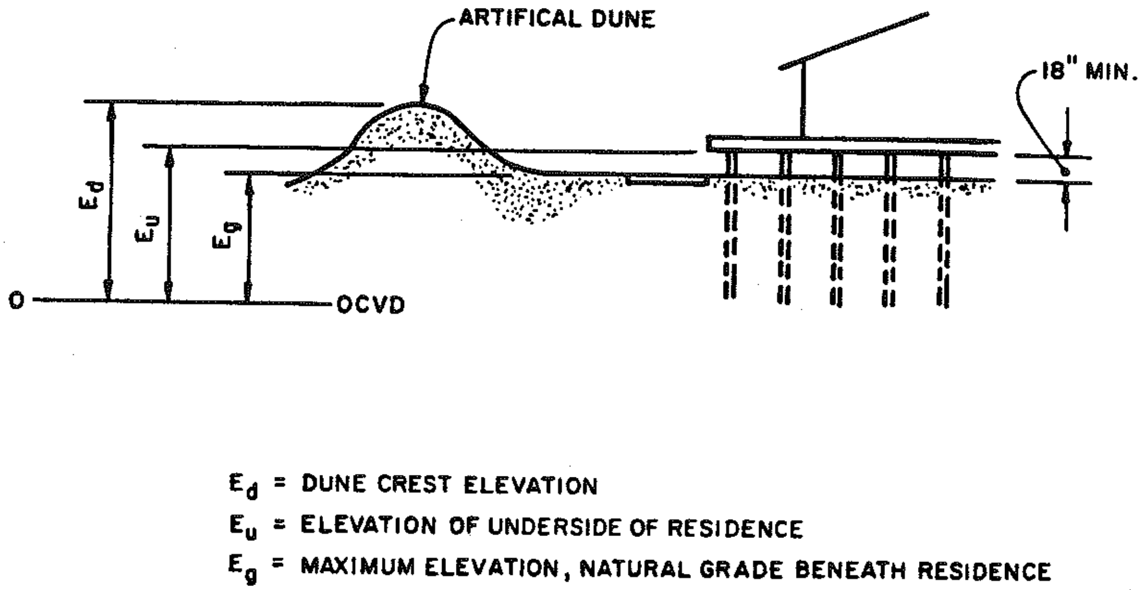


Figure 9. Procedure to calculate distance between ground elevation and underside of residences at Sunset Beach.

STEP 5.2.b OBJECTIVE: To establish the distance between the natural ground elevation and the underside of a structure at Sunset Beach.

REQUIRED VERTICAL CLEARANCES, SUNSET BEACH[1]

E_d in ft (OCVD)	Vertical Clearance $E_u - E_g$, in inches[2]
$\geq$15.2	18
14.2 to 15.2	21
13.2 to 15.1	24
$<$13.2	27

[1]does not apply to decks and garages; natural grade must be similar to that of adjacent property

2natural grade to lowest portion of underside of residence.

upon the elevation of the beach or artificial dune in front of the structure, E_d, and the ground elevation beneath the structure. The minimum is 18 inches to provide a crawl space and to prevent water damage when water moves from adjacent property.

5.2.c. Sideyard Passages. To provide a flow conduit between residences, the minimum unobstructed sideyard distance between adjacent structures will be 6 feet. If the adjacent lot is undeveloped, the dwelling setback from the lot line is 3 feet.

5.2.d. General Guidelines.

(1) Areas under buildings constructed on piles or caissons must be kept clear to allow landward drainage.

(2) When slab or garage parking at the landward end of the building is used, the landward end of the building must be closed for fire protection purposes. This reduces the cross-sectional flow area. The passageway between buildings thus becomes the only drainage conduit. Gates in walkways alongside structures should swing landward so they are parallel and away from flow from the ocean. Gates which open toward the ocean may be difficult or impossible to open during a flood event. Flowing water will either pond behind the gates and accentuate the local flood problem, or the gates and/or adjacent fences will be damaged by the water pressure. This does not apply to chain link, wrought iron or other highly porous gate materials.

(3) The movement of runup-generated water into neighboring properties should be avoided. The ground should slope toward South Pacific Avenue, not toward adjacent properties.

(4) Ice plant cover or other sand retaining landscaping on dunes and adjacent areas is encouraged because it helps stabilize the dune, provides resistance to runup-flow-erosion during severe storms, and creates drag which reduces runup flow across the dune.

5.3 DESIGN ACCEPTABILITY.

IS DESIGN ACCEPTABLE BASED ON DESIGN CONSIDERATIONS?

YES - GO TO STEP 12

NO - REJECT PLAN

6. STEP 6 - TYPE OF STRUCTURE OR PROTECTIVE DEVICE.

Is the plan for a structure on piles or caissons?

YES - GO TO STEP 7

NO - GO TO STEP 8

STEP 7 - DESIGN CONSIDERATIONS, STRUCTURE ON PILES OR CAISSONS.

7.1 Geotechnical Considerations.

7.1.a. Foundation conditions. A soils report should be prepared to address the depth to bedrock, and soil and rock properties to 20 feet below maximum scour depth, or to bedrock. Included should be soil weight, vertical and lateral bearing capacities, active and passive pressures, and soil classification.

7.1.b. Recommended Foundation Type. A recommendation must be given by the geotechnical professional regarding the type of foundation support to use, such as driven piles or drilled piles or caissons.

7.1.c. Pile or Caisson depth. Depth of pile or caisson embedment required to support both vertical and lateral loads at maximum scour depth design conditions must be provided.

7.1.d. Method Recommended to Reach Pile or Caisson Depth. Acceptable or reasonable methods of installing the piles or caissons based on the soil and rock properties must be given.

7.1.e. Method to Penetrate Rock, if Applicable. The report must indicate whether bedrock is soft enough for driven piles (such as steel H-piles) to penetrate, or if easy or hard drilling is required.

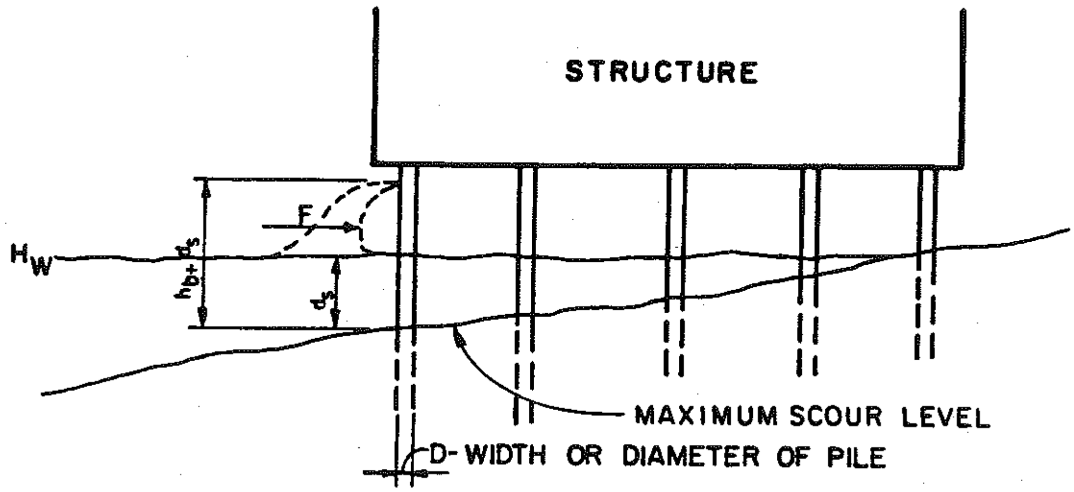
7.1.f. Pile or Caisson Design. Allowable vertical and lateral load capacities for specific pile size and embedment based on soil and rock capacities must be specified.

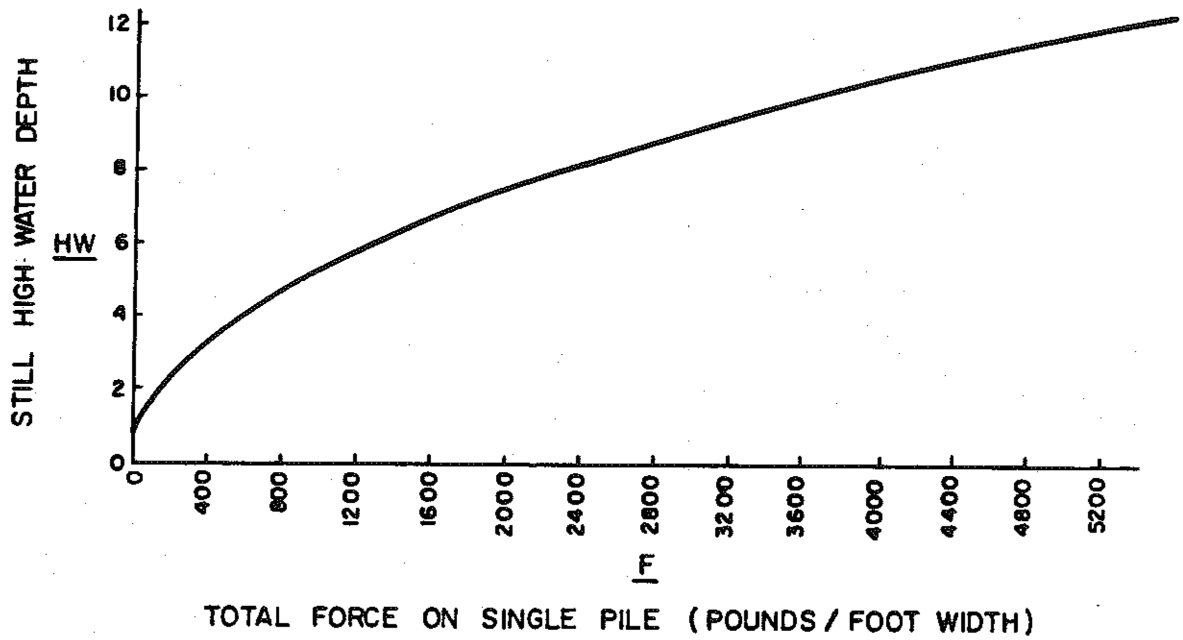
7.1.g. Ground Water Levels. If applicable, ground water levels must be given, and the lag in water level change versus tide should be provided, especially across a wallsection .

7.2 Coastal Design Considerations.

7.2.a. Forces on Individual Piles: The wave force on any individual pile must be determined and the pile designed to resist that load as well as the structural load. This force may be determined using the method given in the SHORE PROTECTION MANUAL (1984), or it can be conservatively determined using the graph in Figure 10, which gives a total force per foot width of the pile based on the total water height from the scour line. This force is applied at a point two thirds of the water height above the sand and distributed as an inverted triangle on the pile.

7.2.b. Design of Total Structure (pile group design): The entire pile group, pile caps and floor system must be designed to resist the total wave load on the pile group, and transmit this load to the soils or bedrock. Depending on the size of the structure, it





H_W = STILL WATER ELEVATION
d_s = WATER DEPTH ($H_W - H_{SC}$)
h_b = HEIGHT OF BREAKING WAVE

Figure 10. Total force per foot width of a pile, F, based on the water depth above the design scour elevation.

is generally assumed that forces on all pilings will occur concurrently. (Note: Earthquake and wind loads must also be checked.) It is likely that wind on the structure and waves on the piles will act concurrently from the same direction.

7.2.c. Protection at Landward End of Structure: If the structure does not extend landward of the highest wave runup, then wave runup must be considered. If site planning allows, the structure should be free-standing without a wall at the shoreward end. Access can be by an expendable causeway or stairs (see Case I, Fig. 11). If this is not practical, then the structure must be designed to withstand uplift pressures adjacent to the end wall; and provisions must be made for venting the pressure from water and trapped air. The floor should be designed to resist an uplift pressure equal to the hydrostatic pressure of a column of water one and one-half times the height the runup would obtain above the floor if, R', the floor was not there and the wall extended vertically. Runup elevation on the wall can be obtained according to procedures in STEP 8.1. The hydrostatic pressure should be applied over an area extending out from the wall a distance equal to the runup height above the floor, R'. In addition, the wall-floor intersection must be vented for a minimum of ten percent of its length, with a maximum vent spacing of eight feet (see Case II, Fig. 11). The rear wall should be evaluated, as appropriate, according to guidelines in STEPS 8 through 11.

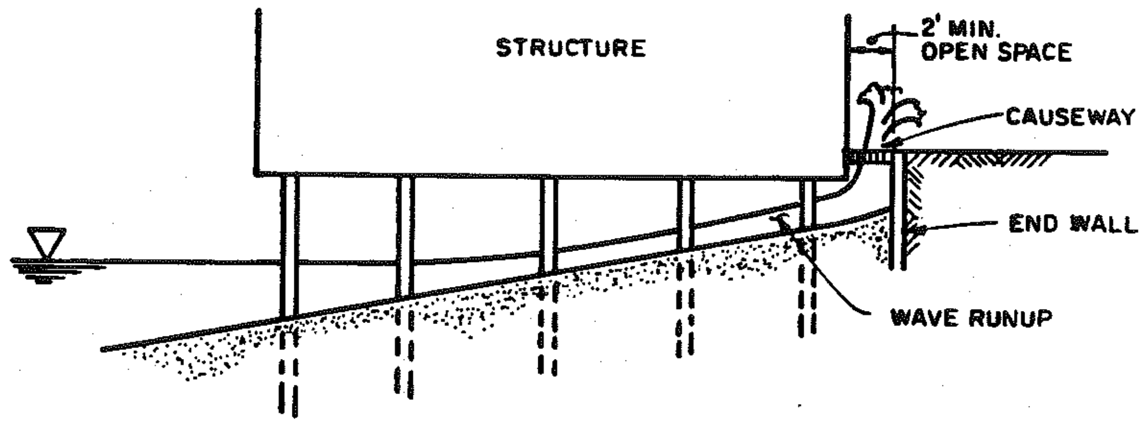
7.2.d. Utilities. Utilities suspended beneath the structure should be above the upper limit of wave action, H_b (Fig. 6). The enclosure of utility lines should be considered in the design.

7.3. Design Acceptability.

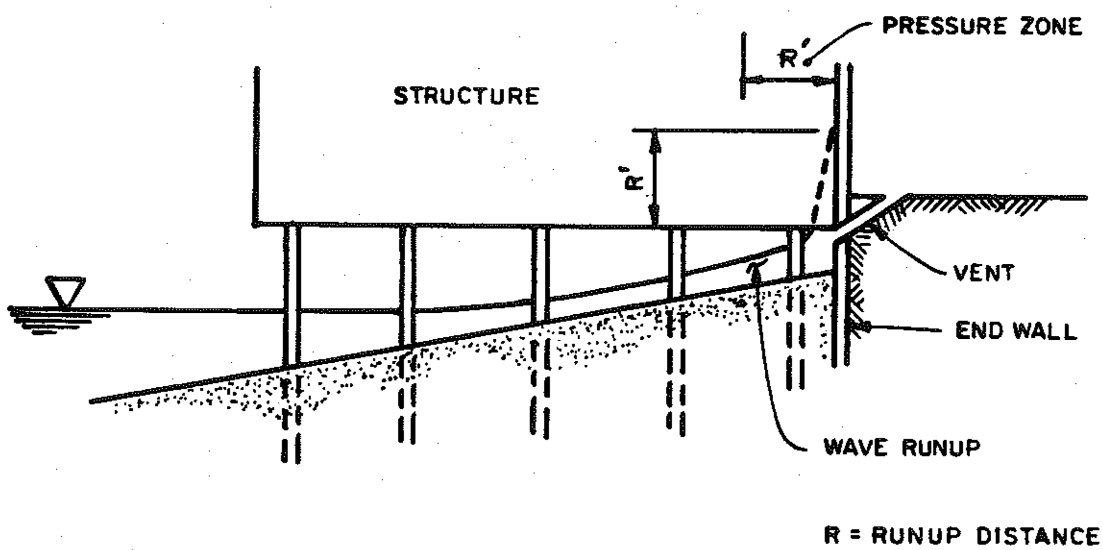
IS DESIGN ACCEPTABLE BASED ON DESIGN CONSIDERATIONS?

YES - GO TO STEP 12

NO - REJECT PLAN



CASE I



CASE 2

Figure 11. Wave runup protection at the landward end of a pile or caisson-supported structure.

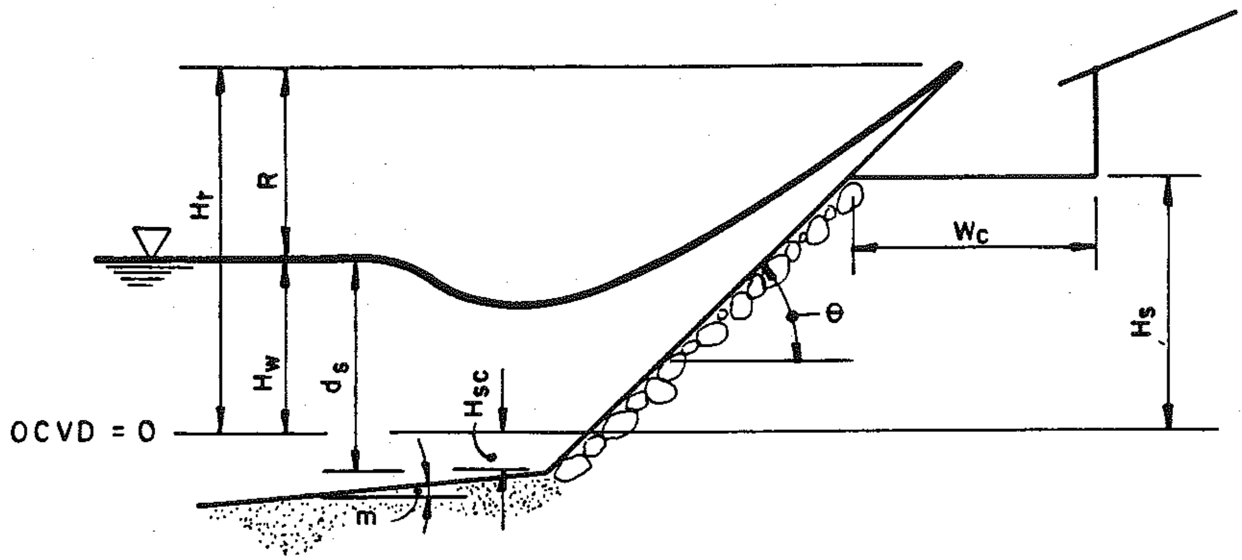
STEP 8 - DETERMINE THE MINIMUM STRUCTURE SETBACK FROM THE PROTECTIVE DEVICE. Although a breaking wave above the top of a protective device is generally considered unacceptable, wave overtopping by runup is acceptable with certain conditions. When the height of runup, H_r (Fig. 12), exceeds the elevation of the protective device, H_s, damage to glass, decking, and possibly the exterior surface of the ocean-facing wall of a structure may occur. As $H_r - H_s$ (the runup elevation above the protective device) increases, spray produced by overtopping becomes more coherent as splash. Damage may be prevented or mitigated by locating the ocean-facing wall of the structure landward of the crest of the protective device. This distance is defined herein as the runup setback distance, w_c, (Fig.12).

In this step the minimum acceptable runup setback distance, w_c, is established for vertical protective devices (seawalls and bulkheads) and sloping protective devices (revetments). The procedure to establish w_c has two parts: (1) calculate the runup elevation on the protective device, and (2) determine the acceptable minimum setback distance.

8.1 Calculate Runup Elevation. Runup elevation depends on the slope, porosity, and roughness of the protective device and wave characteristics impinging on the device. A complete description of the runup phenomena is not available because of the large number of variables involved. Empirical data, mostly from laboratory investigations, must be used to determine H_r. These data are in the form of curves obtained from the SHORE PROTECTION MANUAL (1984). Runup height R is the height above the still water level, H_w, that will be reached by runup flow. The runup elevation, H_r, is

$$H_r = H_w + R. \qquad (5)$$

Input data required to use the runup figures (Figs. 14, 15) are the dimensionless parameters R/H_o', H_o'/gT^2, d_s/H_o' and θ. H_o' is obtained



W_c = SETBACK DISTANCE

H_s = HEIGHT OF PROTECTIVE DEVICE

θ = ANGLE OF PROTECTIVE DEVICE RELATIVE TO HORIZONTAL (Tan θ= RISE/RUN)

d_s = WATER DEPTH

H_w = STILL WATER ELEVATION

R_r = RUNUP ELEVATION

m = BEACH SLOPE ELEVATION

R = RUNUP HEIGHT

H_{sc} = SCOUR ELEVATION

H_r = WAVE RUNUP ELEVATION

Figure 12. Definition sketch, runup and overtopping after a wave has broken on a sloping protective device.

from Figure 13, when h_b is known (H'_o is the unrefracted deepwater wave height). h_b is given in Table 3 for 1984 beach conditions or in Figures 3 and 4. If H_w or H_{sc} (Fig.12) is expected to change during the life of the protective device, Table 3 cannot be used. The procedure described in "Long-Term Change in Still Water Level" (Chapter 2) should be used to calculate, H_w, and Table 1 or Figure 3 should be used to calculate H_{sc}. Figure 4 is then used to calculate h_b where $d_s = H_w + H_{sc}$. If the value of h_b/gT^2 is less than 0.0002 (Fig. 13) use $h_b/H'_o = 3.0$.

Because of the large number of variables involved, including the angle and surface roughness of the protective device and the design beach slope in front of the device, a range of wave periods require consideration. The maximum wave runup elevation must be obtained by trial and error for wave periods between T = 8 sec and T = 22 sec. The period which produces the greatest runup elevation is then designated the design runup wave period (Note, the maximum breaking wave height, which is not dependant on the characteristics of the protective device, is obtained using T = 22 sec).

After calculating the value of H'_o, use Figure 14 to calculate runup on a smooth vertical wall. Empirical data for an Orange County design beach slope, m = 0.02, is unavailable on Figure 14, however, the slope m = 1:10 shown in the figure should yield a conservative runup value. Figure 16 should be used to correct for scale effects by increasing the runup height by k (use tan θ = 0.8). Use Figure 15 for wave runup on a rough, 1:1½ slope revetment. Data on rough, flatter slopes are unavailable, however, flatter rough slopes will yield lower runup values than those calculated using Figure 15. Thus, the use of Figure 15 is conservative for rough slopes of less than a grade of 1:1½. Figure 16 should be used to make scale effects corrections by increasing the runup height by k. Runup elevation is obtained using Equation 5.

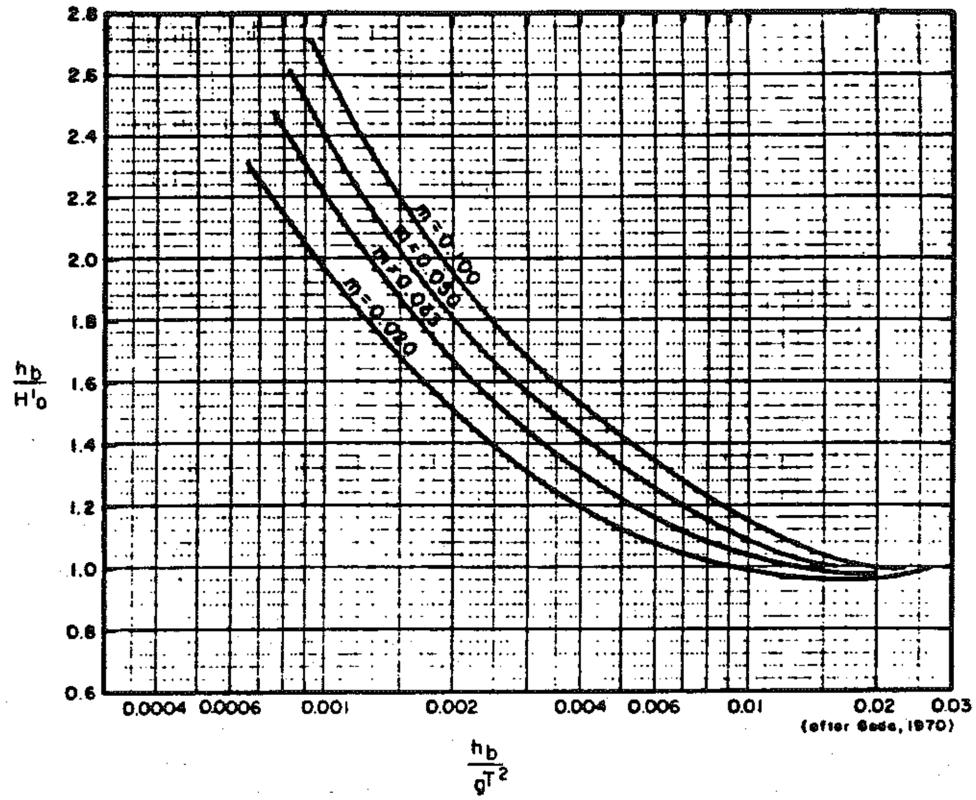


Figure 13. Breaker Height Index, h_b/H_o' Versus h_b/gT^2 (from SHORE PROTECTION MANUAL, 1984)

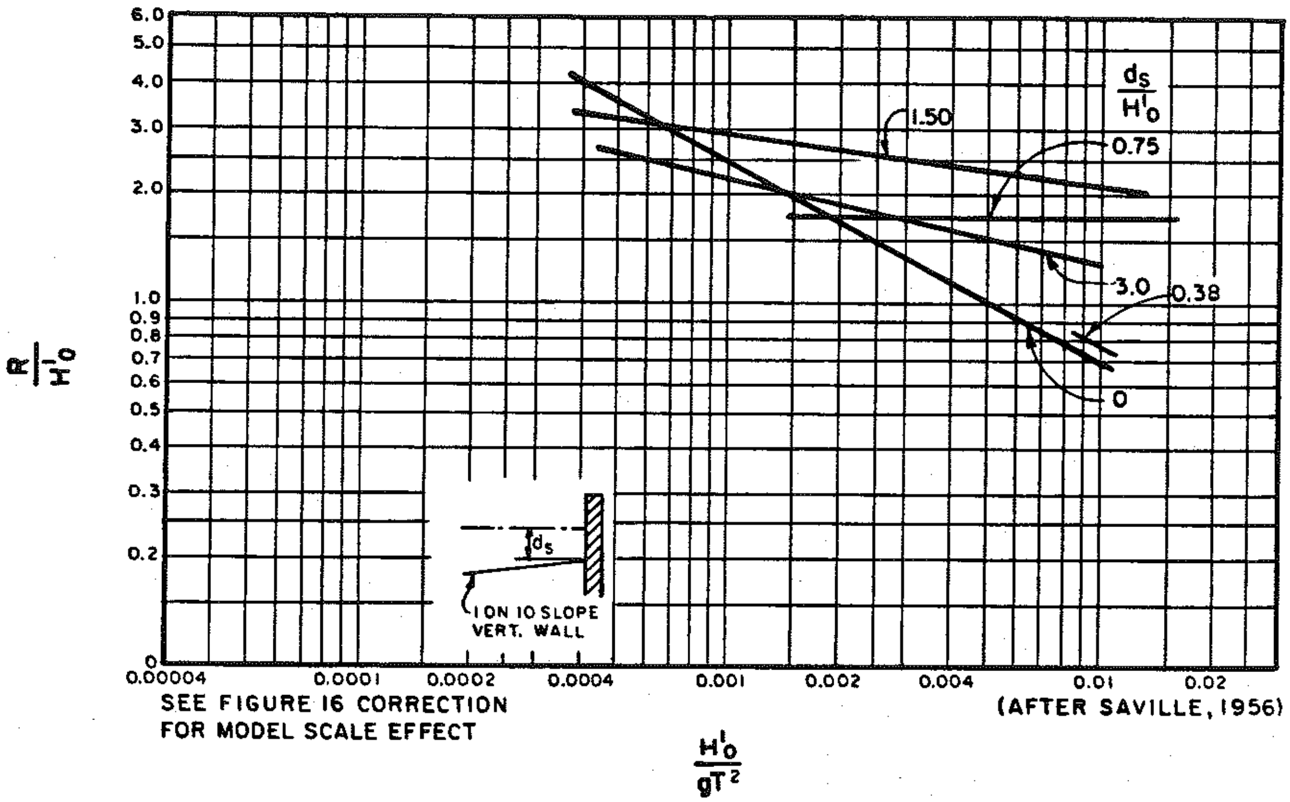


Figure 14, Wave Runup on an Impermeable, Vertical Wall versus H'_o/gT^2 (from SHORE PROTECTION MANUAL, 1984)

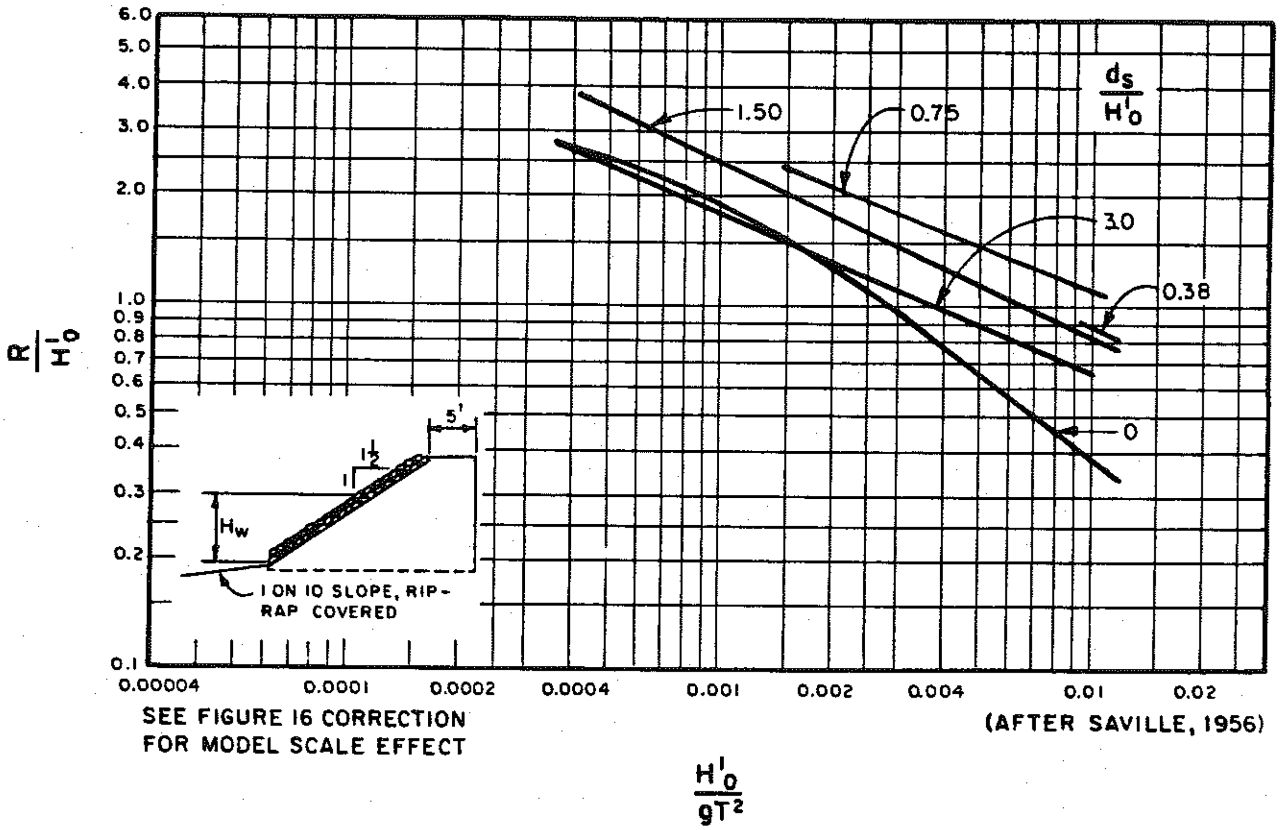


Figure 15. Wave Runup on Impermeable, Riprap, 1:1.5 Slope versus H_o'/gT^2 (from SHORE PROTECTION MANUAL, 1984)

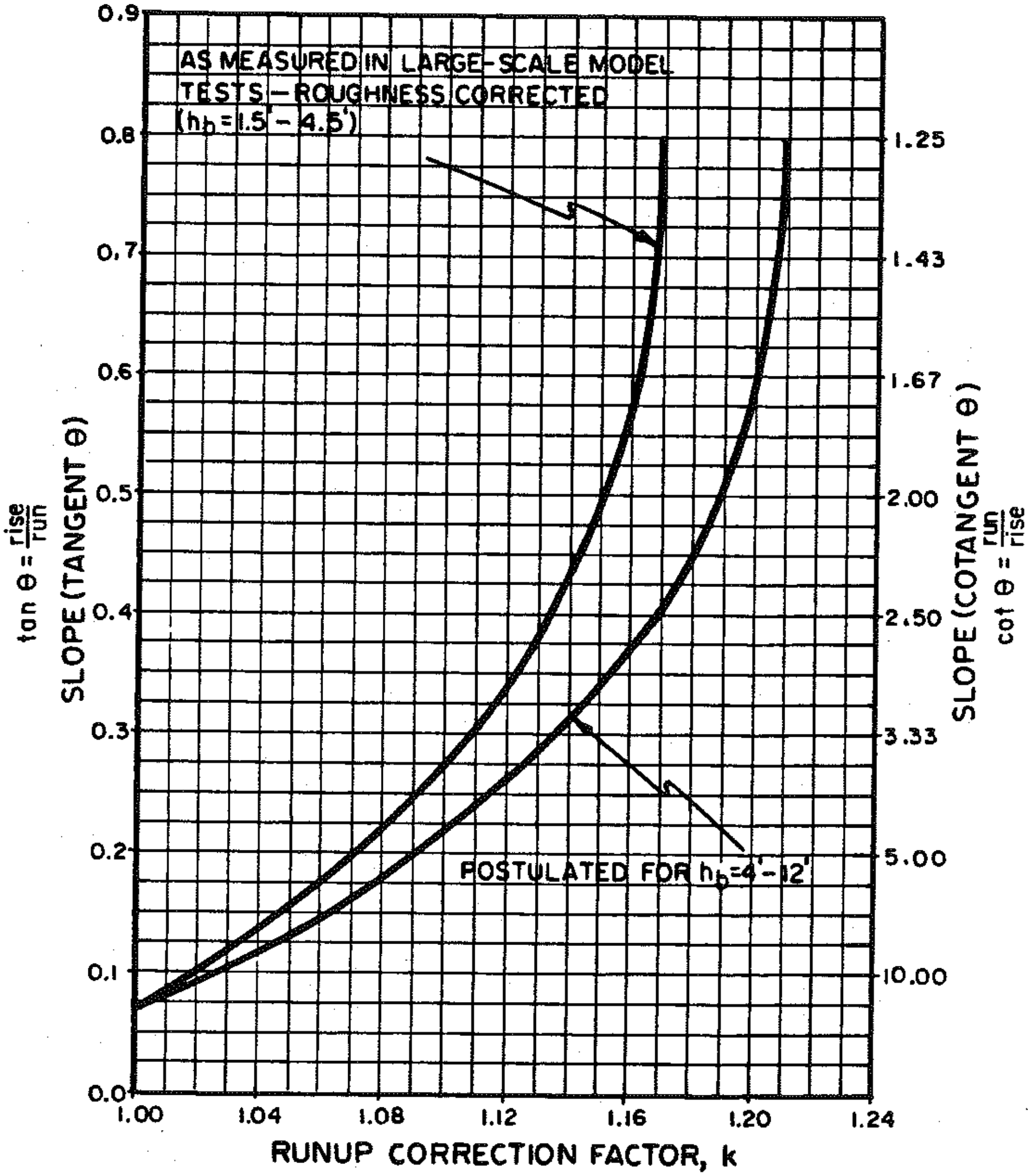


Figure 16. Runup Correction for Scale Effects (from SHORE PROTECTION MANUAL, 1984)

If $H_r < H_s$, no building setback is required

GO TO STEP 9

8.2 Calculate Setback Distance. If $H_r \geq H_s$, a minimum setback distance, w_c, of 10 ft or more is required. Use Figure 17 to determine, w_c. This figure is based on the vertical overtopping distance, $H_r - H_s$, and the assumption that runup flow will be carried horizontally toward the structure by: (1) the horizontal component of runup flow on a sloping protective device, and (2) onshore winds. The horizontal distance traversed by the tip of the uprush watermass is assumed equal to the vertical uprush limit above the top of the protective device.

8.3 Guidelines If Device Is Acceptable. Drainage for seawater that overtops a protective device must be considered. The water surface elevation should be maintained below the sill elevation of any ocean-facing entries. That sill should be at least 6 inches above the deck surface when H_r H_s to prevent water penetration of the structure.

8.4 If Runup Reaches The Structure. If runup flow that overtops a protective device reaches the structure the following situation pertains:

IF $w < w_c$, BUT SHUTTERS OR OTHER MEASURES ARE TAKEN TO PROTECT GLASS AND VULNERABLE MATERIAL ON THE OCEAN WALL OF THE STRUCTURE, USE w_c=10 ft, IF $w <$ 10 ft, DESIGN IS UNACCEPTABLE.

IF $w < w_c$ AND NO WALL OR WINDOW PROTECTION IS PROVIDED, DESIGN IS UNACCEPTABLE.

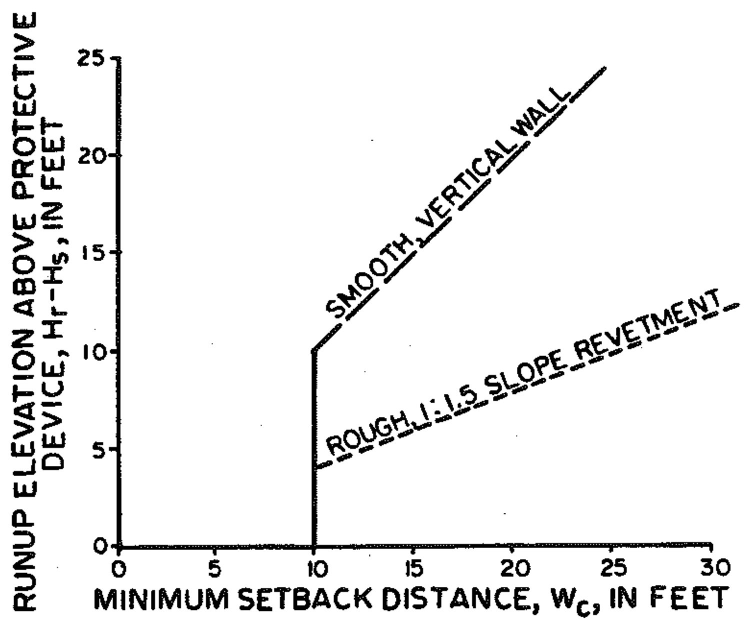


Figure 17. Minimum setback distance when overtopping occurs.

<u>STEP 8.2 OBJECTIVE.</u> To determine whether minimum acceptable structure setback distance, W_c, from top of protective device when $H_r > H_s$, is used.

DATA: W_c from above curve (use 1:1½ curve for all revetments)
W_a = actual setback distance from plans

<u>PROCEDURE</u> If $W_a \geq W_c$: GO TO STEP 9
If $W_a < W_c$: DESIGN MAY BE UNACCEPTABLE
(See Step 8.4, "RUNUP REACHES STRUCTURE")

8.5 Design Acceptability.

IS DESIGN ACCEPTABLE?

YES - GO TO STEP 9

NO - REJECT PLAN

9. STEP 9. TYPE OF PROTECTIVE DEVICE.

Is the plan for a seawall or bulkhead?

YES - GO TO STEP 10

NO - GO TO STEP 11

STEP 10 - DESIGN CONSIDERATIONS FOR SEAWALLS AND BULKHEADS.

10.1 General Considerations. The purpose of a seawall or bulkhead is to support, stabilize and protect the property and any structures behind the device. Because many types of seawalls and bulkheads, including gravity walls, cantilever walls, anchored walls, double walls, and composite walls, may provide adequate protection, and because the designs vary so greatly, general design considerations adequate for plan check purposes only are presented in this step. Actual design will require a more detailed analysis.

A seawall or bulkhead will in most cases transmit hydrodynamic forces produced by waves to the soil behind it; the soil must be compacted and retained. Most seawall and bulkhead failures in Orange County have occurred because backfill material was lost and the wall failed in shear or inward-bending moment. Seawall failures are less likely to occur where backfill is properly placed, compacted and retained. Another mode of failure is due to inadequate design of tie backs when scour occurs at the toe of a cantilever or tiedback wall.

10.2. Special Design Considerations. A plan check should consider the following aspects of failure or damage:

10.2.a. excessive lateral loads caused by earth, water and surcharge on the landward side of the device

10.2.b. losses of soil behind the device and

10.2.c. wave forces on the device

10.3 Determination of Acceptable Structural Stresses.

10.3.a. Structural Requirements. Normally, one of the following structural systems is used:

(1) A cantilever wall founded in bedrock.

(2) A gravity wall founded on bedrock.

(3) A vertically spanned wall supported by bedrock at the bottom (either by penetration into bedrock; or, by bearing on bedrock, with a rubble deposit on bedrock providing lateral support at the bottom) and by a beam at (or near) the top. The beam shall span horizontally along the width of the protective device, and shall be horizontally supported by cables or ties extending to "deadman" anchors at the proper spacing. (Easements granted by the adjacent property owners, which permit the installation of the anchors, must be acquired prior to the issuance of permits for walls designed with this system.)

(4) An alternative system submitted, for approval, by the civil engineer.

10.3.b. Design Provisions.

(1) The wall shall be designed to resist all hydrostatic and hydrodynamic pressures acting on each side of the wall, together with earth pressure, surcharge and any other loading conditions.

(2) Each condition of load, caused by the processes of sand erosion and accretion, shall be considered and the wall designed for the worst possible load combination.

(3) Acceptable structural stresses shall be based on wind induced levels for service load stresses with an allowable increase of 133%. For load factor design use Load Factor = 1.4 (See Paragraph 4).

(4) Design Criteria for Deadman Anchorage:

(a) Allowable value of anchor pull = ultimate value/2, factor of safety of 2 against failure.

(b) Soil within passive wedge of anchorage shall be compacted to no less than 90% of maximum unit weight (ASTM D698 Test).

(c) Tie rod is designed for allowable anchor pull. Tie rod connections to wall and anchorage are designed for 1.2 times allowable anchor pull at normal working stressees.

(d) The rod connection to anchorage is made at the location of the resultant earth pressures active on the vertical face of the anchorage.

(e) Drilled rock anchor shall be tested and to not less than 150% of design load.

(f) Design should provide for corrosion allowance and/or protection.

(5) The toe embedment of the wall system and the design of the anchorage should provide a minimum load (Safety) factor of 2.0.

10.4 Determination of Acceptable Structural Stresses.

The wave induced forces on the structures are quite complex, particularly in relation to their recurrence interval. The maximum wave forces that are being discussed relate to high tide conditions and a maximum scour condition at the site, thus allowing for the maximum depth for which a wave can break. This is further complicated because the maximum forces on a particular structure may not occur at a maximum depth. Since many of the wave induced forces are depth limited, (i.e., would break or cause a maximum wave force to occur when a given depth of water over a bottom condition would occur), it is likely that in many cases the maximum wave would break at something less than a maximum credible recurrence interval. A 100 year event would be the maximum breaking wave, but because of the depth limiting factor at the site, almost the same forces could be induced by a 15, 20 or 25 year event. Because of this likelihood of occurrence the reoccurrence interval for this type of wave force on structures is more comparable to wind induced forces normally considered in building codes. Therefore, in considering factors of safety for structural materials, the normally applied increases in service load stresses or comparable load factors for ultimate strength design comparable to wind induced stresses should be utilized. This also applies on structures without protective devices and structures which support live loads and earth loads.

10.5 Lateral Loads.

The soil-structure interaction is so highly structure-type-dependent that soil stresses can only be determined accurately by knowing the structure type and soil properties at the site. Because soil properties generally vary over short distances, it is advisable to sample the soil at each construction site. The soil report should recommend imbedment depth for conditions likely to occur as a storm subsides, including saturated fill behind the wall and receding water in front of the wall. Percolation under the wall must be considered if the differential head across the walls is greater than about one foot. Seepage pressures in this situation may be significant. The filter system should be designed to prevent piping. Flow through the filter is bi-directional and within the intertidal region flow may reverse direction approximately four times per day. The type of anchorage must also be specified in a soils report. Care should be taken to insure the anchoring system does not settle after construction.

10.6 Wave Forces.

Wave forces considered on a vertical wall are shown in Figure 18. Earth pressures on the back of the wall should be considered and discussed in a geotechnical report. Wave pressures are greatest at the top of a wall, while earth pressures are normally greatest at the bottom. Consequently, the engineer should consider the difference in these pressures particularly at the top of the wall and ensure that the restraining elements at the top of the wall are capable of taking the maximum wave forces without the aid of the soil. If a wall is cantilevered above the earth backing, the wave forces must obviously be considered as the prime element for the wall. Wave forces, for simplicity, may be assumed as a triangular distribution over the affected height with the centroid of the total wave force acting at 2/3 the distance above the scour line on the water side and the wave crest elevation.

10.7 Earthquake Loads.

Anchored bulkheads and gravity walls have been known to suffer damage in earthquakes. Among others, failures resulted from increased lateral stresses behind the walls, a reduction in water pressure outside the walls (where the wall is constructed in relatively deep water), and a loss of strength of backfill materials. The geotechnical report should discuss any increase in soil pressure due to seismically-induced motion.

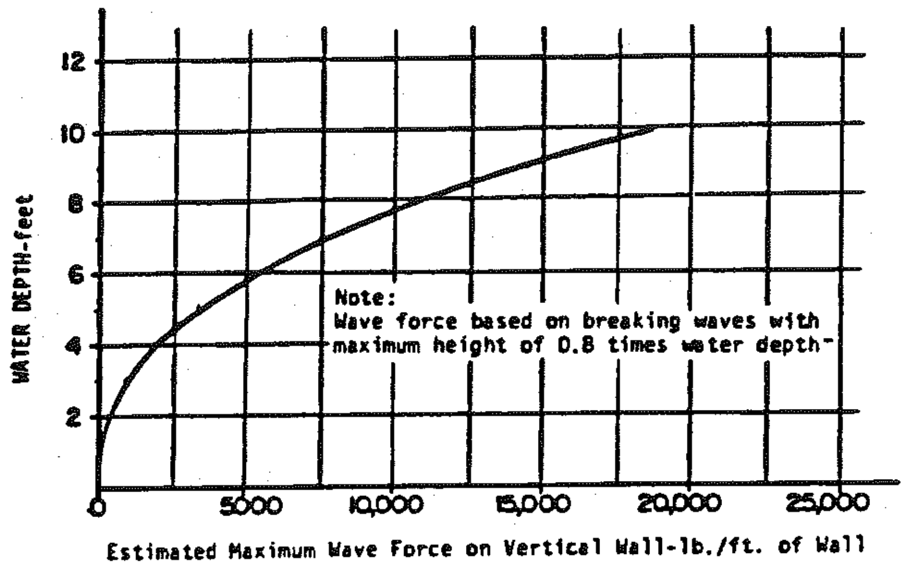


Figure 18. Estimated maximum wave force on a vertical wall (from Doyle and Fox, 1984).

10.8 Placement of Backfill.

Cohesionless clean sand or similar material should be used as backfill. Backfill material should be well compacted and graded to the top of the wall. The backfill should be retained by appropriate filter material to prevent loss through weep holes or other voids. If significant water is expected over the top of the wall (STEP 8) erosion protection should be considered for the horizontal area behind the wall.

10.9 Toe Scour. Where a large increase in scour depth can be anticipated in front of a wall, toe protection is highly recommended. The elevation of the toe protection should be at least 5 ft above the natural maximum scour level. If toe protection in unconsolidated material is not provided, an added scour depth equal to the height of the breaking wave, h_b, must be added to the natural scour depth. This is the depth that must be used in establishing wall stability.

Placement of a rock blanket (toe protection) at the toe of a protective device is usually the easiest and most economical method to prevent a scour trough from forming. The surface layer of rock or armor units must be of sufficient size, density, and durability that they will not be significantly moved or damaged during a design storm. The surface rock or armor units must be placed on bedding. Bedding should be composed of one or more layers of material coarser than beach sand and finer than the surface rock. A geotextile filter or a rock filter is acceptable. The purpose of the filter is to prevent the toe protection rock from moving into the underlying beach sand. Design of the toe revetment should be in accordance with guidelines prescribed in STEP 11, REVETMENTS.

10.10 Wall Depth.

To prevent backfill soil leakage resulting from piping, the wall must be made impermeable to sand to at least 2 ft below the design scour level (Table 1, Fig. 3) if toe protection is used. If toe protection is not used, a scour elevation below the natural local scour depth equal to the breaking wave height must be used. If the beach scour level is bedrock, the design must ensure the wall is properly sealed from backfill losses at the bedrock surface.

10.11 Requirement for Filter Behind Wall.

Even small cracks and holes in an exposed wall section can become conduits for the seaward flow of large quantities of backfill material. Because it is difficult to ensure such cracks and holes don't develop during construction or after construction, a filter behind the wall is recommended. That filter should extend the length and depth of the inside of the wall, and possibly be carried shoreward at least 2 feet at the base of the wall when a geotextile filter is used. When a gravel filter is installed, the thickness should be such that backfill material will be retained. To control ground water levels, it may be necessary to provide weepholes at 10 to 15 feet on center to allow any water that overtops the wall, or ground water from behind the wall, to drain through the wall, unless the wall itself is designed for the equivalent fluid pressure generated by the submerged earth and the water. This particular point should be discussed in the geotechnical report.

10.12 Requirement for Wing Walls.

Backfill material will be lost if a seawall or bulkhead is not correctly connected to adjacent walls or if a wing wall ("return wall") is not provided when adjacent walls do not exist. Seawall or bulkhead design guidelines also apply to the wing wall.

The primary additional design requirement is that the wing wall or walls be long enough to prevent flanking during any design event for the life of the device. Criteria to establish the length of the wing wall must be established on a site by site basis. The storm profile of the upper beach must be projected landward to beyond the region where the profile will be changed by wave activity. This will be a time-changing profile if the shoreline is retreating (Table 1, Figure 3). A wing wall distance 10-ft beyond the estimated landward limit of profile change is recommended to account for scour that could result from wave reflection at the wing wall. This is especially important where more than 100 lineal feet of adjacent property is unprotected. Wing walls must be used adjacent to property on which the structure is founded on piles or caissons without a protective device.

In a landward direction, the scour depth will become progressively less than the depth at the OPDSL. Scour caused by the wing wall must be considered if toe protection is not used. The design wall elevation to prevent breaking waves from passing above the wall should be determined based on the procedure given in STEP 3.

10.13 Seaward Returns on Vertical Walls.

Returns at the top of vertical, and near-vertical, walls can be used to deflect upward-directed, wave-produced flow back toward the ocean. These devices, either incorporated as an integral part of the wall or adequately fastened to the wall, should have a seaward angle with the vertical of 10-20 degrees. Around fifteen degrees is standard practice. Less than a 10 degree angle usually does not significantly decrease the runup over and to the landward side of the wall. A return with an angle much greater than 20 degrees is subject to large upward forces, which must be accounted for in the design. The vertical dimension of the return is usually 2-4 ft, and it is usually located near or at the top of the wall.

10.14 Guides to Reduce Adverse Effects.

Protective devices may create a short-term beach scour seaward of the wall and on adjacent beaches. For this reason the following recommendations are provided to minimize the potential adverse effects of protective devices:

10.14.a. Seawalls, bulkheads and revetments should be constructed in line with adjacent protective devices. This eliminates the possibility of reflected waves from wing walls.

10.14.b. Seawalls, bulkheads and revetments should be tied to adjacent protective devices whenever possible to prevent the loss of backfill and to present a relatively uniform face to breaking waves.

10.14.c. A toe revetment should be provided to an elevation at least 5 ft. above the natural scour depth. This revetment will reduce some wave reflection and decrease scour in front of the device.

10.14.d. To reduce wave energy being reflected at an angle toward adjacent beaches and to reduce the scour it could produce, toe revetments are recommended, where possible, on wing walls where adjacent property is not fronted by a protective device, i.e., the structure is on piles.

10.15 Design Acceptability.

IS DESIGN ACCEPTABLE BASED ON DESIGN CONSIDERATIONS?

YES - GO TO STEP 12 (STEP 11 FOR TOE REVETMENTS)
NO - REJECT PLAN

STEP 11 - DESIGN CONSIDERATIONS FOR REVETMENTS. The rock revetment is the most common type used to protect private property in Southern California. Its design is emphasized in this step. The rock revetment can adjust and settle somewhat after construction without causing structural failure. It also allows for relief of hydrostatic uplift pressure generated by wave action. An underlying filter allows for pressure relief over the entire foundation area. The design of other types of revetments is addressed in the SHORE PROTECTION MANUAL (1984).

A rock revetment as shown in Figure 19 is composed of one or more layers of random-shaped and random-placed stones protected with a cover layer of selected quarrystone. The head (upper portion of the slope) of a rubble device normally sustains more extensive and frequent damage than the trunk (lower portion). Rock revetments are flexible, so damage from waves which exceed the design wave is progressive. The displacement of several armor stones will usually not result in the complete loss of protection.

11.1 Slope. Cover (armor) layer slopes steeper than 1:1½ (rise/run) are not recommended. Slopes of 1:2 are recommended where the revetment is constructed on sand. A steeper slope, but not steeper than 1:1½ can be used where other conditions require it.

11.2 Cover Layer Design. The empirical formula (from SHORE PROTECTION MANUAL, 1984) expressed in stone weight, W, for armor units of nearly uniform size required to withstand a design wave on a slope is

$$W = \frac{W_r \; h_b{}^3}{K_D \; (S_r - 1)^3 \cot \theta} \qquad (6)$$

in which:

W = weight in pounds (W for a two-quarrystone thick cover layer can range from 1.25 W to 0.75 W),

W_r = unit weight (saturated surface dry) in lb/ft^3,

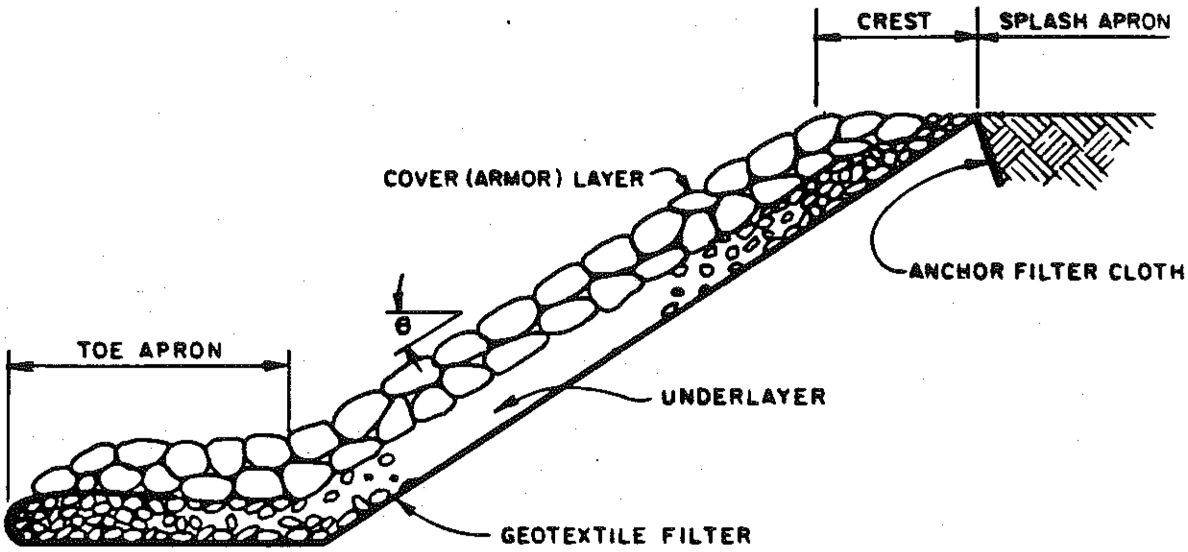


Figure 19. Idealized section of a typical Southern California revetment.

h_b= breaking wave height as given in Table 3, or as modified according to guidelines in Figure 8,

S_r= specific gravity of armor unit relative to water (S_r=W_r/W_w; W_w= unit weight of salt water, 64 lb/ft^3),

K_D= stability coefficient as shown in Table 6 (from SHORE PROTECTION MANUAL, 1984) that varies with the slope of the armor units, roughness of armor-unit surface, sharpness of edges, and interlocking of units, among others, and

θ = cover-unit slope (angle from the horizontal in degress).

Major overtopping is assumed not to occur when using Equation 6. This equation is presented as a guide for plan checking. Experience and engineering judgement are required to establish K_D (Table 6), and in the design of all protective devices, including revetments. The SHORE PROTECTION MANUAL also presents a procedure to calculate the weight of the 50 percent size for graded riprap armor stone (Eq. 7-117, 1984).

11.3 Thickness of Cover Layer. A two-unit armor layer thickness is recommended. If a one-unit layer is used, special care must be exercised in the placement of the single armor layer to ensure an adequate cover with good interlock is obtained. The long axis of each stone should be placed perpendicular to the slope.

11.4 Underlayer and Filter Design. The primary function of a revetment is to retain and protect the soil behind the revetment. This is accomplished by providing a filter that allows the passage of water, but retains the fine material. Armor stone is placed over the filter to hold it in place against the forces due to waves. Underlayer stone is placed between the filter and armor stone to keep the smaller filter material from washing out between the voids in the armor layers.

Table 6. Suggested K_D Values for use in determining armor unit weight (from Shore Protection Manual, 1984).

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

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

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

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

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

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

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

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

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

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

Two types of systems are commonly constructed. The most common in practice today is the use of a geotechnical fabric filter overlain by either armor stone or an underlayer stone and then armor stone. The second type is the use of several graded layers of stone successively larger in size that properly transitions between the in-situ soils and the armor layers. Both types of structures are acceptable, but the former is preferable. In many cases, a quarry run material placed in sufficient thickness can underlie up to two-ton stone. The engineer must show that the quarry material has sufficient thickness and material of a wide enough gradation that a graded layer can develop after winnowing has occurred. There must be at least two layers of the largest stone in the gradation.

Up to one-ton stone can be placed directly over the filter material, although it is desirable to have at least a 6 to 12 inch layer of gravel over the geotechnical fabric to prevent tearing the material during placing.

Underlayers should be two layers thick and should weigh 1/10 to 1/15 of the weight of the adjacent layer of overlying material. This applies to structures that use a geotechnical fabric and to those that are graded stone layers.

11.5 Toe of Revetment. It is usually difficult to construct the toe of a revetment situated on sand below the maximum design scour level if that level is below the water table. Consequently, the toe of a revetment warrants special protection. The basic principle is to design the toe to be deep enough as to have an apron that can settle into a scour pocket to prevent the revetment from being undermined. Some general guidelines for toe design are given below. Not all guidelines are required. The application depends on design. The intent is to provide a toe that can be constructed and will provide adequate protection.

11.5.a. Cover for the toe apron should be an extension of the adjacent revetment slope such that it can settle to the maximum anticipated scour depth plus a minimum of 3 feet.

11.5.b. Where possible, the toe apron should be extended below the design scour limit (Table 1) a distance equal to the design wave height (Table 4), but no less than 3 ft.

11.5.c. Cover thickness of the buried apron should be a minimum of two armor stones thick.

11.5.d. Use of a geotextile filter fabric is recommended for use beneath the revetment and toe apron. The fabric should: (1) stop about 3 ft from the outer edge of the apron to protect it from being undermined, or (2) be extended beyond the edge of the apron and folded back over the bedding layer and some of the cover stone, then buried in cover stone and sand.

11.6 <u>Crest Width.</u> As a general rule for a revetment subject to overtopping, the minimum crest width should equal the combined widths of three armor units (Fig. 19).

11.7 <u>Prevention of Soil Loss Behind Revetment.</u> An overtopped revetment may become unstabilized because:

(1) Sediment behind the revetment is eroded by runup water.

(2) Soil supporting the top of the revetment is lost and the device fails from the top down.

(3) Water in the soil behind and beneath the revetment produces a drainage problem.

The solution to soil loss behind the revetment is to armor the bank behind the device with a splash apron. This apron could be a concrete or asphalt deck which drains water away from the structure or a filter blanket covered by a bedding layer.

11.8 Guides to Reduce Adverse Effects. See STEP 10.14.

11.9 Design Acceptability.

IS DESIGN ACCEPTABLE BASED ON DESIGN CONSIDERATIONS?

YES - GO TO STEP 12
NO - REJECT PLAN

12 STEP 12 - MATERIALS. The most commonly used materials in construction in the FP-3 zone of Orange County are wood, concrete, steel and stone. Synthetic fabrics are commonly used as a filter material. This step includes guidelines provided to assist in evaluating the materials used for the foundation of structures (STEPS 5 and 7), seawalls and bulkheads (STEP 10) and revetments (STEP 11). A good reference for construction materials is the Corps of Engineers, Coastal Engineering Research Center, Special Report No. 10.

12.1 WOOD.

12.1.a. Materials. Recommendations for the use of timber piles in foundations may be found in publications of the American Wood Preservers Institute (AWPI). The principal woods used for piling are southern pine and coastal Douglas fir with a few other less frequently used woods such as red pine, lodgepole pine, western larch, and oak.

12.1.b. Potential Problems. Softwood lumber, timber, and piling used in or near salt water environments should be properly treated for protection against insects, decay, and marine organisms. Under optimum conditions, wood so protected will have its useful life extended to about four or five times that of untreated wood. Practically, this represents a life expectancy of approximately 20 years when in contact with sea water and at least 30 years in other locations. Untreated wood should be used only for temporary structures and facilities.

Timber structures are subject to damage or destruction by fire. To date, no completely effective chemical treatment has been developed which will eliminate this problem.

12.1.c. <u>Guides-Preservative Treatment</u>

(1) Tables 7 and 8 indicate the amount of preservative to be retained in various woods for various conditions. The net retentions in the tables are minimum penetration requirements.

(2) Oil type preservatives afford protection against weathering and checking as well as against decay and are recommended for the treatment of sawed wood that is to be used in contact with the ground.

(3) Waterborne preservatives are recommended where cleanliness, freedom from odor, or paintability is required.

(4) The dual treatment, (Table 7) is recommended for submerged salt water conditions in Orange County since both Limnoria and Toredo are known to be present.

12.1.3. <u>Other Treatments.</u>

(1) Marine borer damage to wood piles can be prevented and the life of in-place piles can be economically extended almost indefinitely, by means of some kind of barrier jacket. Piles could be jacketed in concrete, wrapped with metal sheathing such as aluminum, copper, cupro-nickel alloys or in sheets of 30 mil polyvinyl chloride.

(2) Pile cut-offs, except fender piles, should be protected by drilling 3/4-inch holes about 1-inch apart through the untreated portion to a depth of 1½-inches, and the holes filled with creosote or

Table 7. Preservative retention for treatment of wood piles (From AWPA-C3).

Application		Types of Preservative	Retention by SPECIES (pcf): Southern Pine	Douglas Fir Pacific Coast	Oak	Red Pine Ponderosa Pine Jack Pine	Lodgepole Pine Western Larch Interior Douglas Fir
Foundation, Land and Fresh Water		Creosote[1]	12.0	17.0	6.0	12.0	17.0
		Pentachlorophenol	.6	.85	.3	.6	.85
		CCA or ACA	.8	1.0	NR	.8	1.0
Marine (Submerged)	Single	Creosote[1]	20.0	20.0	10.0		
		CCA or ACA	2.5	2.5	NR		
	Dual	Creosote[1]	20.0	20.0			
		CCA or ACA	1.0	1.0			

[1] Includes Creosote-Coal Tar

Table 8. Preservative retention for timber treatment (From AWPA-C2).

Application	Types of Preservative	Retention by SPECIES (pcf): Southern Pine, Coast Douglas Fir, Western Hemlock, Larch	Softwoods: Interior Douglas Fir, Ponderosa Pine, Redwood, Jack Pine, Lodgepole Pine, White Pine, Red Pine, Sugar Pine	Oak	Gum
Submerged	Creosote[1]	25.0	25.0	10.0	12.0
	CCA or ACA	2.5	2.5	NR	NR
Splash Zone	Creosote[1]	12.0	8.0	5.0[2]	5.0[2]
	Pentachlorophenol	.6	.4	.35	.4
	ACC	NR	.25	.5	.5
	CCA or ACA	.6	.6	.4	.4
Above Ground	Creosote[1]		8.0	5.0	6.0
	Pentachlorophenol		.4	.3	.3
	ACC, ACA, CCA		2.5	2.5	2.5
	CZC		.45	.45	.45
	FCAP		.25	.25	.25
Soil Contract	Creosote[1]	10.0	10.0	.35	.4
	Pentachlorophenol	.5	.5	.5	.5
	ACC	.5	.5	.4	.4
	CCA or ACA	.4	.4		
Dual Treatment	Creosote[1]	20.0			
	CCA or ACA	1.5			

[1]Includes Creosote-Coal Tar
[2]6 pcf for members under 5 in (16 mm) thick.

liberally coated with creosote where drilling is not practical. A layer of Irish felt should be placed on top and covered by a 15 mil layer of polyethylene before placing the pile cap.

(3) Insofar as practical, wood pieces should be trimmed, bored and counterbored before pressure treatment. When pretreated wood is cut in the field it is essential that the exposed wood be generously mopped with the same preservatives.

(4) For some fire protection, the most commonly used precaution is to adopt timber sizes having a minimum dimension of four inches for main framing members. Heavy timber members will retain their structural integrity throughout long periods of fire exposure because of their size and the slow rate at which charning penetrates inward from the wood surface. Piling can be most easily protected against fire by concrete encasement, if economically feasible.

(5) Piling located completely below grade and below the water table may be installed without a preservative treatment.

12.1.3. Sizes and Shapes.

(1) Wood sheet piles used for groins, bulkheads, and subterranean cutoff walls should be beveled at the bottom on one side and edges. Sheet piles should not be driven more than a meter except in soft ground. If deeper penetration is required, excavate the area along the line of piles so that the piles need to be driven only a meter to final tip elevation. Members should be sized according to the loads and conditions to be resisted by the sheeting. Tongue and groove sheeting consists of planks milled so that on one edge there is a projecting tongue and on the opposite edge a groove into which the tongue of the adjoining plank is fitted when driven. When large load carrying capacity wood sheet piles are required Wakefield sheet piling is used and up of three layers of planks are spiked or bolted together to form a sheet pile. The middle plank of the three should project

beyond the edges of the planks on each side to form a tongue on one edge and a groove on the other.

(2) Steel hardware used in wooden coastal structures, such as bolts, spikes, driftpins, rods, wire rope, chain, plates, and shapes should be galvanized in accordance with ASTM A123, A153, A336, or A525 as applicable. Sizing of all steel hardware for use in marine work should be very conservative, with consideration given to exposure conditions and abrasion. Stainless steel such as type 304 and, if economically feasible, Monel is more resistant to the marine corrosion environment than any of these materials but also much more expensive.

12.1.f. References.

(a) West Coast Lumber Standard Grading Rules No. 16 published by the West Coast Lumber Inspection Bureau for Douglas Fir, Western Hemlock, Western Red Cedar, White Fir, and Sitka Spruce.

(b) Grading rules for southern pine published by the Southern Pine Inspection Bureau.

(c) National Design Specifications for Stress Grade Lumber and Its Fastenings, published by the National Lumber Manufacturers Association.

(d) American Society for Testing and Materials (ASTM) Standard D25 for Round Timber Piles.

(e) Standards for pressure treating softwood lumber and timber, published by the American Wood Preservers Bureau. (AWPB).

(f) American Wood-Preservers Association (AWPA) standards.

(g) Construction Materials for Coastal Structures, Special Report No. 10, dated February 1983, prepared by Moffatt and Nichol Engineers for the U.S. Army Corps of Engineers Coastal Engineering Research Center at Fort Belvoir, Virginia.

12.2 <u>CONCRETE</u>:

12.2.a. <u>Materials.</u>

(1) The durability of Portland cement concrete, defined as its ability to resist weathering action, abrasion, or any other processes of deterioration, is a major factor in its excellence as a coastal construction material. Proper selection of materials is important in achieving a concrete mix suitable for waterfront work.

(2) Cement for concrete used in soil containing from 0.10 to 0.20 percent water soluble sulfate (as SO_4) or used in water containing from 150 to 2,000 parts per million (ppm) SO_4 should be Type II in accordance with ASTM C150. In environments where the water soluble sulfate exceeds 0.20 percent or the sulfate solution contains from 2,000 to 10,000 ppm, Type V cement should be used.

(3) Water for mixing concrete should meet the requirements of ASTM C94. Water used in construction in prestressed concrete work should be free from oil and contain not more than 650 ppm of chlorides as Cl, nor more than 1300 ppm of sulfate as SO_4.

(4) Aggregates should have clean, hard and uncoated particles and comply with ASTM C33. Potentially reactive aggregates should not be used in concrete exposed to seawater or alkali environments. Tests for potentially reactive aggregates should comply with ASTM C227.

(5) Air entraining admixtures are recommended to improve both the workability and durability of concrete. Chemical admixtures for reducing under-cement ratio resulting in improved quality and strength of concrete and improved workability during placing should conform to ASTM 494.

(6) Water-cement ratios should be kept low even though strength requirements may be met with a higher value. A maximum water-cement ratio of 0.45 should be used for structures exposed to seawater and 0.50 for all other structures. In any case, a minimum cement content of 6 sacks per cubic yard of concrete is recommended to achieve a dense mixture for waterfront work.

12.2.b. Potential Problems.

(1) For marine usage, wherein the concrete is either submerged continually or periodically, or in close proximity to water, specific measures should be used to produce the strength and density required to provide a structure of the desired life span. When the proper concrete mix is correctly mixed and placed in properly designed structural members, a life of 50 years is commonly attainable.

(2) Spalling, cracking, and abrasion are potential problems which could affect the life and structural integrity of the concrete.

12.2.c. Concrete Design Considerations.

(1) Strength of concrete for various applications should have the following minimum compressive strength at 28 days:

Prestressed Concrete Piles and precast units..........................	= 5000psi
Marine Structures..............	= 3500 psi
Other Structures................	= 3000 psi

(2) Prestressing strands should conform to ASTM A416, Grade 270. Reinforcing steel should conform to ASTM A615, Grade 40 or 60.

(3) Spalling of concrete is generally caused by corrosion of the reinforcing steel which in turn, is due largely to exposure to seawater or inadequate embedment of reinforcing steel. A minimum cover of 3 inches in precast, prestressed units and 3 inches over

reinforcing steel and use of a low-permeability, air-entrained concrete is recommended to provide insurance against spalling. Recently developed fusion-bonded epoxy coated reinforcing could be used in an extreme case of a high alkaline and chloride contaminated environment.

(4) Abrasion resistance of concrete is affected primarily by compressive strength, aggregate properties, finishing methods, and curing, with compressive strength being the most important factor. For areas subject to high abrasion, such as structures subject to the abrasive action of abrasive materials carried by wave action, a minimum compressive strength of 4000 pounds per square inch is recommended.

(5) Cracking is the most common problem attendant with the use of concrete. Cracking may be caused by excessive structural loading, shrinkage during the curing stage, and thermal expansion. Such cracks may not affect the load capacity of the member involved but they could allow water to enter and contact the reinforcing steel. This condition may cause rust to form and eventually lead to spalling of the concrete. The decision of whether a crack should be repaired to restore structural integrity or merely sealed is dependent on the nature of the structure and the cause of the crack. If the structural stresses which caused the crack have been relieved by its occurence, the structural integrity can be restored. Epoxy-resin restores the structure to its original strength. In the case of working cracks such as those caused by foundation movement or cracks which open and close from temperature changes, the only satisfactory solution is to seal them with a flexible water resistant material such as epoxy, asphalt or coal tar material.

12.2.d. <u>References</u>.

(1) American Society for Testing and Materials (ASTM) Standard A416 for uncoated seven-wire stress-relieved strand for prestressed concrete, A615 for deformed bars for concrete reinforcing,

C33 for aggregates, C94 for ready-mixed concrete, C150 for cement, C227 for potential alkali reactivity of cement aggregate combinations, and C494 for chemical admistures.

(2) Manual of Standard Practice for Detailing Reinforced Concrete Structures published by the American Concrete Institute (ACI 315).

(3) Building Code Requirements for Reinforced Concrete published by the American Concrete Institute (ACI 318).

(4) Manual for Quality Control for Plants and Production of precast Concrete Products published by the Prestressed Concrete Institute (MNL-116).

(5) Construction Materials for Coastal Structures, Special Report No. 10, dated February 1983, prepared by Moffatt and Nichol, Engineers for the U.S. Army Corps of Engineers Coastal Engineering Research Center at Fort Belvoir, Virginia.

12.3 STEEL

12.3.a. Material.

Steel is an alloy of iron and carbon with the carbon content under 2 percent. Manganese is added to improve strength and toughness and copper, usually less than 0.2 percent, is added to increase corrosion resistance. Steel is relatively inexpensive, strong, and available in various shapes and sizes for marine applications Steel structures in marine environments suitable for marine applications are indicated in ASTM specifications included in enclosed references.

12.3.b. Potential Problems.

Corrosion rates of steel in sea water or in a marine atmosphere with high chloride ion content as contained in sodium chloride, and relatively high oxygen content will corrode at a rate of 8 to 11 mills per year, (0.008 in to 0.011 in). For a long term exposure this is a serious loss of material. Polluted salt water will result in increased corrosion rates. Marine organisms attached to steel will increase localized corrosion. Abrasion resulting from winds or waves and their ability to transport particles of sand, dirt etc. can cause severe loss of steel. When coupled with dissimilar metals, intense corrosion results.

12.3.c. Use.

Because steel is readily available in all shapes and sizes and can be bolted, riveted or welded, it has a wide variety of uses in marine environments. To ensure a satisfactory steel performance it can be protected with properly applied coatings. In buried or submerged structures, cathodic protection can be used to protect the steel. Coatings or concrete have proven to be effective against abrasion in many uses. Steel is used in foundations as H piles or pipe piles, as sheet piles for bulkheads and retaining walls, and wire mesh gabions.

12.3.d. References.

American Society for Testing Materials (ASTM) Standard A36-77 for structural steel, A131-78 for structural steel for ships, A328-75a for steel sheet piling, A573 for structural carbon steel plates of improved toughness, A69077 for high-strength, low-alloy steel H piles and sheet piling for use in marine environments, A710-79 for low-carbor age-hardening nickel-copper-chromium-columbiam alloy steels.

12.4 GEOTEXTILE FILTERS.

12.4.a. Material.

The most common use of plastics in coastal construction is as a geotextile filter. The plastic material must have good chemical stability and resistance to environmental deterioration, and it must lend itself to fabric construction. Fabrics made of synthetic polymers of polyvinylidene chloride, polypropylene, polyethylene, polyester and polyamide have proven to be successfull for geotextile use. Fabric construction is of three general types: woven, nonwoven, and combination fabrics. Woven fabrics are manufactured by weaving; the yarns cross at right angles and overlap one over the other. Filters of woven fabrics are made using a variety of yarns. Monofilament yarns use a single filament of a polymer. Multi filament cloths, which are yarns containing many fine filaments, have a slightly more irregular pore system and are generally thicker than monofilaments. Mono-multifilament combination fabric contains monofilament yarn in one direction and multifilament yarn in the other. Nonwoven fabrics include all materials which are not woven or knitted. They consist of discrete fibers, which may have a preferred orientation or may be placed in a random manner and do not form a regular or simple pattern as do wovens. Nonwoven fabrics are composed of either continuous filament or staple filament fibers. Continuous filaments are extruded, drawn and laid in the fabric as one continuous fiber. Staple filaments are cut to length before being laid in the fabric. The engineering properties of nonwoven fabrics are controlled by the fiber type, the geometric relationships of the fibers, and the methods of bonding. Four methods of bonding are neede punched, heat bonded, resin bonded, and combination bonded.

12.4.b. Potential Problems.

Selection of a geotextile for a filter should be based on the filtering and physical properties as well as the chemical properties of the fabric consistant with the site specific requirements. Ultraviolet radiation (sunlight) will cause degradation of synthetics, and even with ultraviolet stabilizers added to the synthetics, which significantly prolong structural life when fabric is exposed to ultraviolet, excessive sun light will cause rapid (1 to 2 years) deterioration.

In placement, geotextile filters must be laid loosely, but free of wrinkles, creases and folds. Overlaps of adjoining sheets should be 18 inches and staggered for installations in the dry. For under water applications, the overlaps should be 3 feet. Material placement on top of a fabric must be carefully controlled with no material dropped more than 1 foot onto the fabric.

12.4.c. Use.

Geotextiles are used in construction as filters, material separators and reinforcement for soils. The most common use is as filters which permit the passage of water through the fabric but not soil or sand particles. A geotextile filter is a permeable fabric constructed of synthetic fibers designed to prevent piping (prevent soil from passing through it) and remain permeable to water without significant head loss or without permitting the development of excessive hydrostatic pressure.

Woven geotextile filters have been used beneath stone and interlocking concrete block revetments, linings for the interior of vertical seawalls to permit water passage through weepholes, joints, king pile and panel, jetting foundations and most soil stabilizing conditions.

12.4.d. References.

(1) U.S. Army Corps of Engineers 1977 C.E. Guide Specifications.

American Society for Testing Materials Standard

(2) D 1682-64 Breaking Loads and Elongation of Textile fabrics,
(3) D 1683-68 Seam Breaking Strength, D751-73 Puncture Strength,
(4) D 3884-80 for abrasion resistance.

12.5 STONE.

Stone refers to individual blocks, masses, or fragments that have been broken or quarried from bedrock exposures, or obtained from boulders and cobbles in alluvium, that are intended for commercial use.

12.5.a. Stone Classification. Stone classification is based mainly on composition and texture:

(1) granite: medium to coarse-grained igneous stones that consist mainly of feldspar and quartz

(2) basalt and related stone: any dense, fine-grained, dark gray or black volcanic stone

(3) carbonate stone: limestone, dolomite or marble

(4) sandstone: a sedimentary rock composed mainly of particles 0.01 inch to 0.25 inches in diameter cemented together with a silica or calcite and are well suited for marine exposure

(5) miscellaneous types of stone: such as shist, greenstone, conglomerate, serpentine, shale, etc.

12.5.b. Potential Problems.

Most of the varieties of stone normally used in coastal structures are very durable. Periodic wetting and drying will result in leaching action in a porous stone. Leaching water not chemically combined may remove cementing material and weaken the stone. Calcareous stones are subject to decomposition by chemical attack, usually by acids.

12.5.c. Uses.

Stones are used in the construction of breakwaters, jetties, groins, sea walls and revetments. Stones are also used as a fill material in caissons. Graded stones ranging from sand to bedding or core material is used as underlayer stone, rip rap, and armor stone. A high density stone is necessary if used in a coastal structure. The design size of armor stone is a function of density, slope, and wave height. Placing stone should begin at the bottom of a section and continue in a manner as to produce a graded mass of material with maximum interlocking and minimum voids. The armor layer should be firmly seated on the underlying stone. Generally, stone should be placed with the longest axis perpendicular to the slope of the structure face.

12.5.d. Placement Methods.

(1) Stone should be placed by equipment and methods suitable for handling materials of the size specified. Placement of the stone should begin at the bottom of the section and should continue in a manner so as to produce a graded mass of material with maximum interlocking and minimum voids. In general, the larger stones should be placed so that vertical joints are broken with the long axis of the stone set approximately normal to the structure slope and pointing inward toward the center of the structure section.

(2) The method used in placement of filter, bedding, or core material should be such that the soft and organic materials on the bottom are displaced outward toward the extreme outside toes of the required sections of the structure and in the direction of the construction. The stone should be handled and placed in such a manner as to minimize segregation and provide a well-graded mass.

(3) In areas where the stone is to be placed on geotextile filter cloth, care should be taken so as not to rupture the cloth and

the stone should not be dropped from a height greater than about 1 foot.

(4) Underlayer stone should be placed to a full zone thickness in one operation in a manner to avoid displacing the underlying material or placing undue impact force on underlying materials and supporting subsoils. The underlayer stone should be placed in a manner to produce a resultant graded mass of stone with minimum voids. Rearranging of individual stones may be required to achieve this result.

12.5.e. <u>References</u>:

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APPENDIX A. DESIGN STILL WATER LEVEL

The design maximum sea surface elevation, H_w, is considered to be

$$H_w (t) = h_a + h_s + h_o + h_e(t) \tag{A1}$$

in which h_a = extreme expected astronomical tide distance above OCVD, based on a 100 yr recurrance interval; h_s = highest expected surge distance above the extreme astronomical tide distance; h_o = other factors, such as the El Nino - Southern Oscillation (ENSO) effect that increases the sea surface distance above the extreme astronomical tide distance; and h_e = time-dependent, long-term change in the sea surface datum with respect to the land. It is this long-term sea surface rise or fall that necessitates a time dependency in H_w. H_w does not include wave runup or wave setup. Wave setup is a superelevation of the water surface over normal surge elevation due to onshore mass transport of the water by wave action alone. It is usually included in empirical runup data. An important factor in establishing H_w comes in the determination of probabilities of whether h_a, h_s, and h_o will occur simultaneously during the design life of a structure or protective device in the County of Orange.

Because of the long record of data available, the approach taken to establish H_w was to use the maximum recorded water surface elevation as the design elevation. Causes of the extreme elevation, however, are discussed and show the selected H_w is a reasonable design value. Conditions on 27 January 1983 produced the highest water surface elevation of record. On that date, the earth, moon and sun were aligned (syzygy position) and the moon was in its closest monthly orbital position to the earth (perigee position). The resultant perigean spring tide effect, an astronomical phenomenon, was amongst the highest of the year. Added to that superelevated water surface situation was the El Nino - Southern Oscillation effect which existed in the region throughout 1983. Additionally, the storm that occurred on 27 January was responsible for an increase in the water surface elevation by (1) an inverse barometric effect which allowed the water surface to rise above normal elevations because of reduced atmospheric pressure, and (2) wind setup which is the vertical rise of the still water level on a windward shore caused by wind shear stress on the water surface.

Astronomical Tide Maximum. Tides along the County of Orange coast are semi-diurnal, i.e., a complete cycle takes 12.4 hr, with a diurnal inequality. Water surface elevations used in this Appendix are referenced to a mean lower low water (MLLW) vertical datum as shown in Table A1 for a primary tide station at Los Angeles (Outer Harbor) and intermediate tide stations in the County of Orange.

TABLE A1. TIDE DATA (in feet above MLLW)[1]

	San Clemente	Newport Beach	Los Angeles
Highest tide observed[2]	not measured	7.86	7.96
Mean highest high water (MHHW)	5.3	5.4	5.5
Mean high water (MHW)	3.7	3.7	3.8
Mean sea level (MSL)	2.7	2.8	2.9

[1]Subtract 2.83 ft to convert elevation to OCVD.
[2]27 January 1983

A perigean spring tide is the largest astronomical tide that can occur. The perigean spring tide which occurred coincidentally with the storm on 27 January 1983 produced the highest water surface elevation of 61 years of record at Los Angeles. At San Diego, well inside San Diego Bay where the MHHW elevation is 5.9 ft, or 0.4-ft higher than at Los Angeles, the 27 January 1983 storm also produced the highest tide of record, +8.35 ft (MLLW). The length of the analysed San Diego record is 58 years. The water surface elevation above MHHW at San Diego, Newport Beach and Los Angeles was almost the same at 2.45 ft. This suggests the perigean-spring-tide, the ENSO effect, and the storm-produced water surface elevation increase along the coast of the County of Orange was near constant at about 2.45 ft above MHHW (Table A1).

The second highest water surface elevation of record at Los Angeles, Newport Beach, and San Diego occurred on 8 August 1983 and almost reached the January levels. Again, it occurred at the time of a wave storm and a perigean spring tide. At Los Angeles it reached +7.87 ft (MLLW) at 2036 hr; at Newport Beach it reached +7.82 ft (MLLW) and at San Diego it reached +8.34 ft (MLLW) at about the same time. This further supports the contention that extreme tides along the County of Orange coast will be similar, and that extreme tides of between +7.8 to +8.0 ft (MLLW) in the County can occur at any season and at a frequency greater than previously anticipated.

Perigean spring tides as large as the 27 January 1983 event occurred many times in the last 50 years and can be expected to occur many times in the next 50 years. Table A2 shows the dates when perigean spring tides will occur in the period 1985 to 2000. Between 1920 and 1985 perigean spring tides occurred at an average rate of 4 times per year. While the prediction of the date of occurrance of perigean spring tides is accepted, an accurate prediction of the perigean spring tide level at a specific site is not possible. At present the only accurate way to establish such elevations is by field measurement.

As noted, at Los Angeles, Newport Beach and San Diego the two largest maximum water surface elevations have occurred at the time of a perigean spring tide. About two-thirds of the yearly maximum water surface elevations also occurred at the time of perigean spring tides (Fig. A1). In many of the cases shown, the yearly maximum water surface elevation was caused by a storm-enhanced water surface superimposed on the perigean spring tide.

Wood (1976) noted the coincidence of strong onshore winds, barometric low pressure systems, and perigean spring tides appears to exceed the normal probability distribution, considering strong onshore winds could occur at a far greater number other than coincident with perigean spring tides. He does not, however, establish the reasons for the relationship and notes that perigean spring tides, without supporting onshore winds,

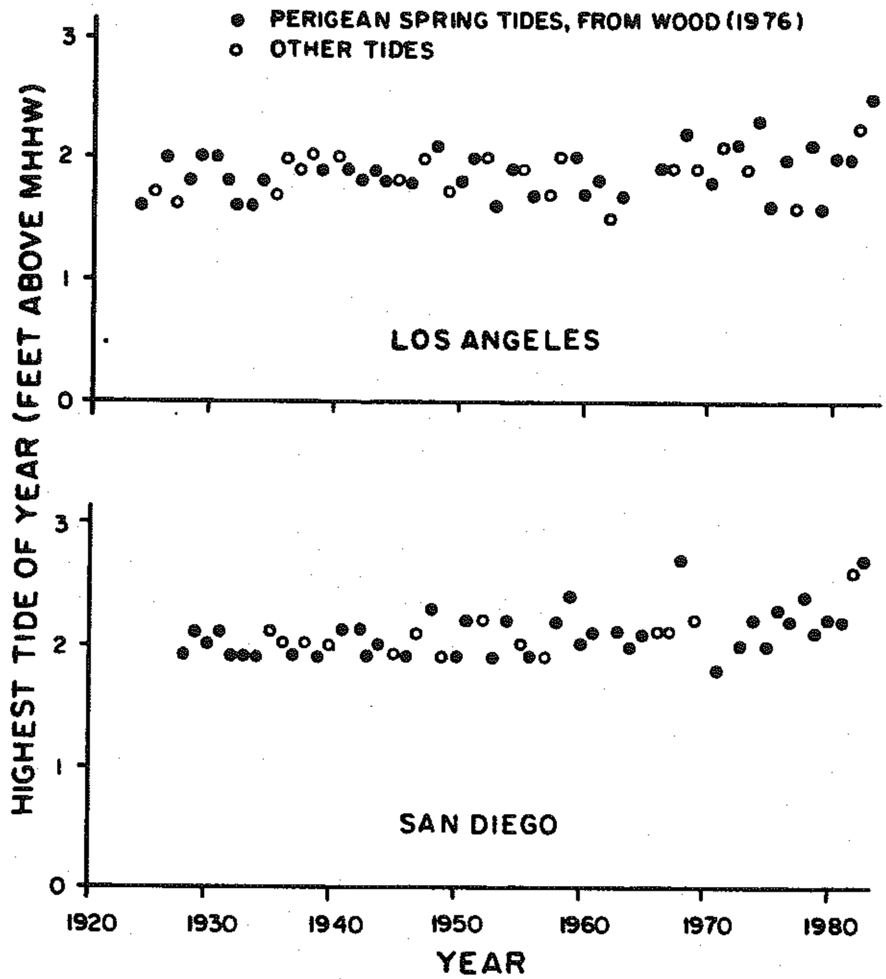

Figure A1. Yearly maximum water surface elevations at Los Angeles and San Diego tide gages showing the influence of perigean spring tides.

usually do not result in coastal flooding. This is evident when the 24 September 1939 hurricane, which created the largest-ever-observed waves offshore of Southern California, is considered. The high water level produced by that event was not even the highest of the month, much less the yearly maximum shown in Figure A1. Had that storm occurred at the time of a perigean spring tide or even a spring tide, coastal flooding would, no doubt, have been severe.

TABLE A2. Dates of Perigean Spring Tides, 1985–1999[1]

1985	Apr 5	1986	May 23	1987	July 10
	May 4		Jun 21		Aug 9
	Oct 13		Dec 1		
	Nov 12		Dec 30		
	Dec 11				
1988	Jan 18	1989	Mar 7	1990	Apr 24
	Feb 17		Apr 5		May 24
	Aug 27		Oct 14		Dec 2
	Sep 25		Nov 12		Dec 31
1991	Jun 12	1992	Jan 19	1993	Feb 6
	Jul 11		Feb 18		Mar 8
	Dec 21		Jul 29		Apr 6
			Aug 27		Sep 15
					Oct 15
1994	Mar 27	1995	May 14	1996	Jan 20
	Apr 25		Jun 12		Jun 30
	Nov 3		Dec 21		Jul 30
	Dec 2				
1997	Feb 7	1998	Mar 27	1999	May 15
	Mar 8		Apr 26		Jun 13
	Aug 18		Oct 5		Nov 22
	Sep 16		Nov 3		Dec 22

[1] from Wood (1976)

In the one-third of the cases where the yearly water surface maximum did not occur at the time of a perigean spring tide (Fig. A1) it overwhelmingly (86 percent of the time) occurred in a 67 day period between 30 November and 4 February. This coincides with a period when wave storms are prevalent. The other 14 percent of the yearly maxima (3 cases) occurred in a 27 day period between 30 June and 26 July.

The highest astronomical tide expected at the time of an extreme wave event is thus assumed to be a perigean spring tide. Because of the clear relationship between extreme high water events caused by perigean spring tides and events which produce strong onshore winds, the most extreme of which occurred on 27 January 1983, the establishment of the maximum design water surface elevation along the County of Orange coast during the life of a structure or protective device could occur again when strong onshore winds and a perigean spring tide coincide. A method to accurately separate the storm surge component of the total extreme high water elevation from the perigean spring tide component has not, as yet, been perfected.

Storm Surge. Storm surge estimates are an important part of forecasting extreme water levels. Storm surge is caused by wave transport, horizontal shear stress on the water surface and inequalities of air pressures on the windward side of gravity waves produced by a wind blowing over it. A decrease in atmospheric pressure also heightens the water surface elevation. Wind shear induces a surface current in the direction of the wind. The current is impeded in shallow water so the water surface in the downwind, nearshore region is raised. The reason storm surge heights can be so high on the Atlantic Ocean and Gulf of Mexico coasts of the United States is that Continental Shelf widths and, thus, shallow water fetches, are greater than those of the County of Orange and the characteristics of the storms there promote higher surges.

Storm surge maxima calculated entirely from theoretical considerations require verification. Most theoretical methods apply to surges caused by hurricanes because the hurricane wind field is better known and therefore is of a somewhat simpler, but still complex structure. No criteria have been established for storm surge caused by extratropical storms along the County of Orange coast. A major reason is that surge, as Tetra Tech noted (Lee, et al, no date) in their FEMA work, is of small importance along this coast. Tetra Tech (Lee et al, no date) calculated a peak surge for the September 1939 hurricane of 1.17 ft. Because of the narrow Southern California shelf, they believe the storm surge was caused mainly by the inverse barometric effect with wind stress playing a negligable part in surge generation. Storm surge is included in the design still water elevation provided in this report.

Bechtel, (1966) in a feasibility investigation for siting a nuclear power and desalting plant in the offshore vicinity of Bolsa Chica, concluded a storm surge value of 1.0 ft was reasonable. They based their findings partly on tide gage recordings.

Seasonal Variations in Water Surface Elevation. Mean sea level exhibits a seasonal cyclicity as shown in Figure A2. This average 0.5 ft variation, which is caused partly by heating and cooling of ocean waters, generally peaks in August and September. Water expands when warmed and contracts when cooled, reaching a minimum, usually in April. An astronomical effect is also involved.

Also, every 4 to 5 years a 0.4 to 0.6 ft increase (and decrease) in sea surface elevation occurs, i.e., the range of the seasonal cycle increases. This monthly average peak (and depression), caused by astronomical effects, occurred in the 1982-83 period of perigean spring tides.

El Nino - Southern Oscillation Effect. ENSO is a dislocation of the world's largest weather system that disrupts the prevailing winds of the Pacific Ocean basin. The low pressure system that is usually centered in the equatorial region north of Australia, and which is responsible for the east to west-directed trade winds, is reduced or absent. This in turn reduces or eliminates the trade winds, and allows warm water to move east in low latitudes of the Pacific Ocean. The result can be a reduction in south-flowing currents along the County of Orange coast and a consequent increase in water temperature. Thermal expansion of surface waters and the absence of the

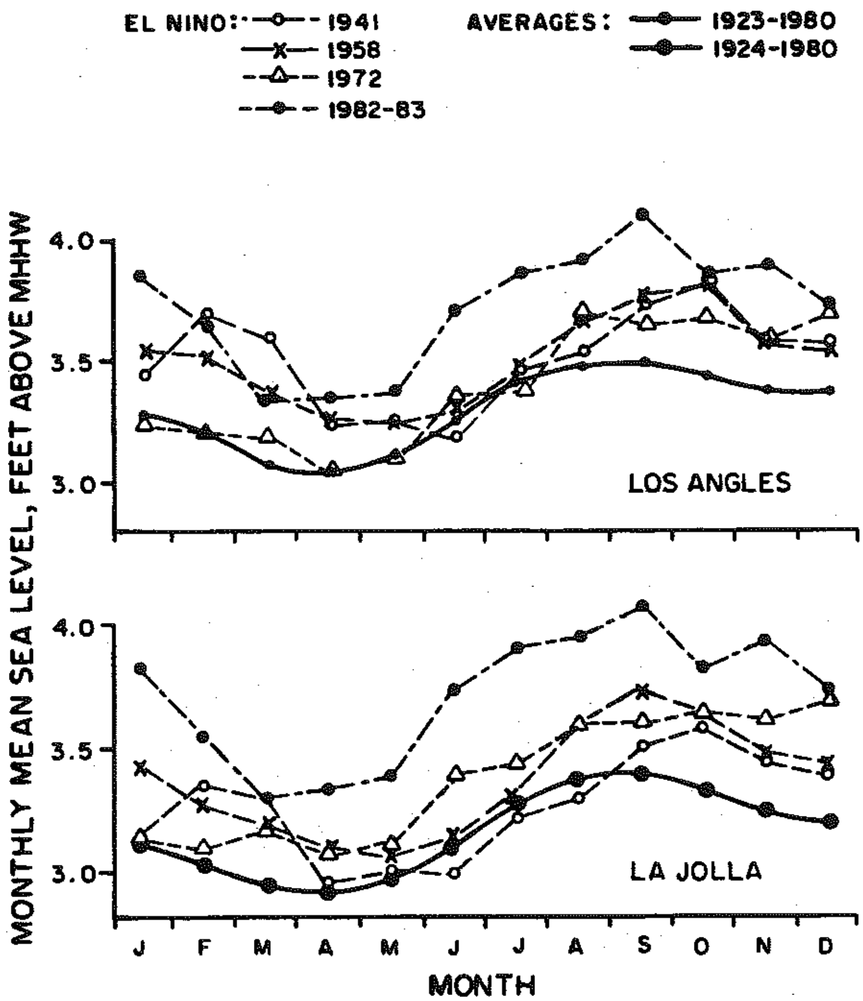

Figure A2. Sea level variations, by month, showing influence of El Nino events.

equational zonal current produces a slight water surface rise. This reached its peak in 1982-1983. However, severe ENSO events have been recorded along the southern California coast since 1891. On average they have appeared every 13 years (0.08 of the time) but the interval between occurrances has varied from 34 to 6 years. The effect of ENSO is difficult to separate from other processes that cause the water surface over large regions to rise or fall.

Tide gage records illustrate the effect of ENSO. Figure A3 shows the average yearly change in sea level during some years El Nino had an effect. The water surface elevation difference between the yearly average of all years, and of ENSO years is somewhat above +0.2 ft for ENSO years. Between 1940 and 1980 this 0.2 ft rise in the El Nino years occurred along the entire U.S. West Coast at about the same magnitude. Interestingly, at the same time water surface elevations were also above the average on the East and Gulf coasts, but at only a slightly higher than average elevation.

Figure A2 shows the difference between the average monthly water surface elevation at Los Angeles and La Jolla and the water surface elevation, by month, when the ENSO effect was extant. While the difference shown is the result of ENSO and other unknown effects, it can be assumed solely the result of the ENSO effect if storm surge averages equal to zero over all years of record. The ENSO effect was most pronounced during the storm season between August and the following April, so it must be considered in a determination of maximum water surface elevation during the predominant storm season. Because the record at La Jolla is similar to that at Los Angeles, a reasonable assumption is that the ENSO sea surface rise effect will be similar and non-varying in an alongshore direction in the County of Orange. The maximum difference in the water surface between the long-term average and the elevation which occurred at times of ENSO was 0.7 ft (summer 1982), and the average is about 0.4 ft during the August-April storm season. Thus, the probability that the ENSO effect will occur during the storm season is 0.08 (1 in 13 years) and the increased water surface elevation is about 0.4 ft. The ENSO effect had the

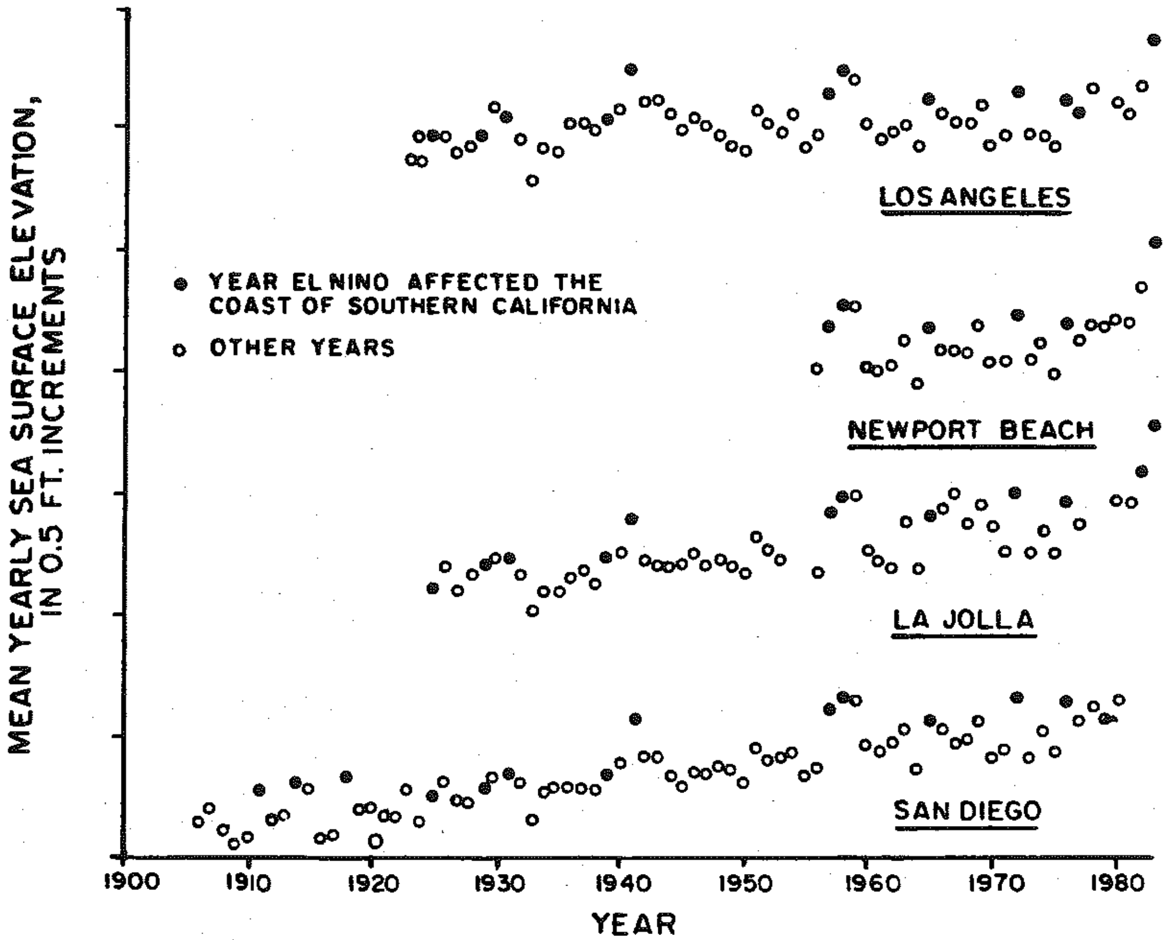

Figure A3. Sea Surface elevation by year showing the El Nino effect.

greatest influence on sea surface elevation in 1982-1983 (Fig. A2).

Tsunamis. Earthquakes, landslides, and volcanic eruptions at the Pacific Ocean periphery or within the ocean basin may create very long period gravity waves called tsunamis. They cross the ocean as low amplitude waves, but are often greatly amplified by shoaling, convergence and resonance when they reach land. Diffraction behind an island or islands may also amplify the wave in certain areas. While they are a significant concern in northern California, their measured effect along the County of Orange coast has been much less.

It is important to distinguish between tsunami waves from remote and local origins. Great destruction is usually the consequence of local events of great magnitude. There is a lack of history of such events off the County of Orange coast. There is also no indication that the County of Orange coast is particularily sensitive to tsunamis generated great distances away (conversely, Hilo, Hawaii, and Crescent City, California, are two locations that are highly sensitive).

Table A3 shows the maximum increase or decrease from the normal tide level of the low water trough to the adjacent high water crest for the La Jolla and Los Angeles tide gages for five great tsunamis from remote sources. The maximum rise above the existing stillwater elevation was only about 2.5 ft at Los Angeles in 1960. Since tsunamis are not related to astronomical tides or onshore wind events, and because they affect the coast for a very short time (average is about 24 hr), their relatively infrequent incidence (once per 4.5 years) along the County of Orange Coast is highly unlikely to occur at the peak of a perigean spring tide and/or associated with a severe wave storm. They are, consequently, not considered in the establishment of the maximum design water surface elevation for protective devices in the five LCP areas.

TABLE A3. Maximum Tsunami Water Effect[1]

Tsunami Event	Range in feet La Jolla	Range in feet Los Angeles
1946	1.4	2.5
1952	0.8	2.0
1957	2.0	2.1
1960	3.3	5.0
1964	2.2	3.2

[1]range = vertical distance from low water trough to adjacent high water crest at tide station listed

Long-Term Sea Level Changes. Water surface fluctuations caused by storms and astronomical tides occur on the order of days or hours and other effects such as those caused by the El Nino - Southern Oscillation event occur on a time scale of months. These are reversable fluctuations in the sense that the water surface returns sometime after the event to its prior elevation. There is also a long-term change in the average elevation of the sea surface, upon which the shorter reversible fluctuations are perturbations that might have to be considered.

Because the coastal design objective is to produce a structure or protective device that will function well for its intended life, usually 20-50 years for a device designed to protect a private shorefront building, the design parameters should be those that evidence extremes throughout the life of the structure. A gradual sea level rise will increase the design extreme water surface elevation throughout that life. In the design of structures, then, there must be a time-varying component to the design stillwater elevation.

A recent rise in sea level with respect to the land has been documented for many areas of the world using, among other evidence, long-term tide gage records (Hicks et. al, 1983). Causes of relative sea level change are both global and regional in nature. Global variations result from changes in the volume of contained water as well as the volume of the ocean basins. Water volume varies directly with thermal expansion (Etkins and Epstein, 1982) and indirectly with the volume of ice above sea level (Donn et. al., 1962). Regional causes of relative sea level change include tectonic adjustments, land subsidence, and long-term changes in atmospheric pressure, temperature, and wind patterns. These account for the alongshore variations in sea level change rates shown in Figure A3. The average recent global rise rate is probably 0.003 to 0.004 ft/yr.

Figure A4 illustrates relative sea level changes using tide data collected during the same years at three sites along the coast, including Newport Bay in the County of Orange. The purpose in presenting these data is to show the alongshore (regional) difference in sea level change rates and to show the large variations that can exist when different periods are selected to obtain an average sea level change rate. For the periods 1956 through 1980 and 1956 through 1983 the sea level change rate varied in a consistant manner from La Jolla to Los Angeles Harbor. The addition of the years 1981, 1982 and 1983 to the 1956 through 1980 data set increased the empirically-derived rate of sea level rise relative to land by 0.0053 ft/yr, and in effect more than doubled the rate of the previous 24 year period. This empathizes, for example, the effect of the 1982-83 ENSO event (Fig. A2) on the yearly sea level elevation, and the effect of a few, high water elevation years on the longer record. The rates shown for the years 1956 through 1980 are more representative of the last three decades of sea level change in Southern California.

Figure A4 indicates the recent rise rate (1956-1980) in the southern half of the County of Orange is about 0.0006 ft/yr (or about 0.06 ft/century) which is too small a rate, if it continues into the future, to consider in design. Because the global rise rate has been estimated at 0.004 ft/yr, the coast at Newport Beach must have been rising at approximately 0.0034 ft/yr, thereby almost keeping up with the global rate of rise of the sea surface. In the 1956 through 1980 period, the general uplift rate at the Los Angeles tide gage exceeded the global

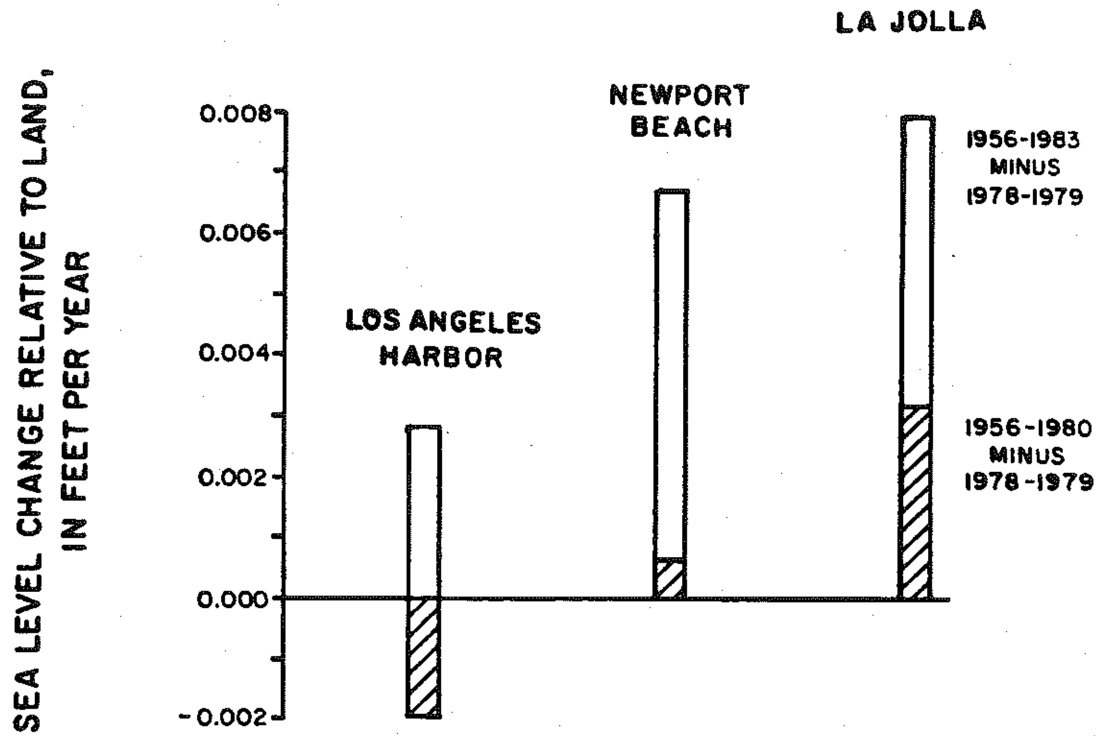

Figure A4. Sea level change rates at three Southern California tide stations. Variations in rates in an alongshore direction and through time are indicated. Data are based on yearly average sea surface data from primary tide gages at the locations shown.

sea level rise rate. This produced a net relative fall of the sea surface relative to land.

Sunset Beach lies along a reach of coast that has been subsiding. This is shown in Figure A5 which is based on frequent leveling by County of Orange surveyors. Note for the period approximately 1969-1983 the rate of subsidance (and hence the approximate sea level rise rate) at Sunset Beach averaged 0.025 ft/yr or 40 times the rate along the south county shoreline. This rate was rapid enough in this past 15 years to warrant it being considered in the design of shore protection devices. As shown in Figure A5, however, the subsidance rate has declined significantly through 1983 with respect to the rate that was measured up to 1976. This suggests the future subsidance rate may be less. Relative sea level changes should be monitored and considered in the future, especially at Sunset Beach.

Two general procedures exist to forecast the relative (with respect to the shoreline) sea level change rate at a specific location. The first is to extrapolate future rates from past rates. That would mean using a future value of perhaps 0.0006 ft/yr for the southern portion of the County of Orange coast as interpolated from Figure A4 between Los Angeles and La Jolla and a rate of 0.025 ft/yr for Sunset Beach (Fig. A5). The assumption implicit in using these rates is that they will not vary in the future. Because they are based on field data that will continue to be collected, those rates can be updated as needed. On the negative side, if the relative sea level change rate increases greatly in the future, a structure or protective device designed for a 50 or 100 year life using extrapolated rates will not be adequate without modification toward the end of the projected life of the structure.

A second procedure to forecast relative sea level change rates considers the mechanisms that cause the changes. The forecast is predicated on knowing how the causitive mechanism will vary in the future. This sea level change forecast method usually considers only the global component of sea level change. At least three groups

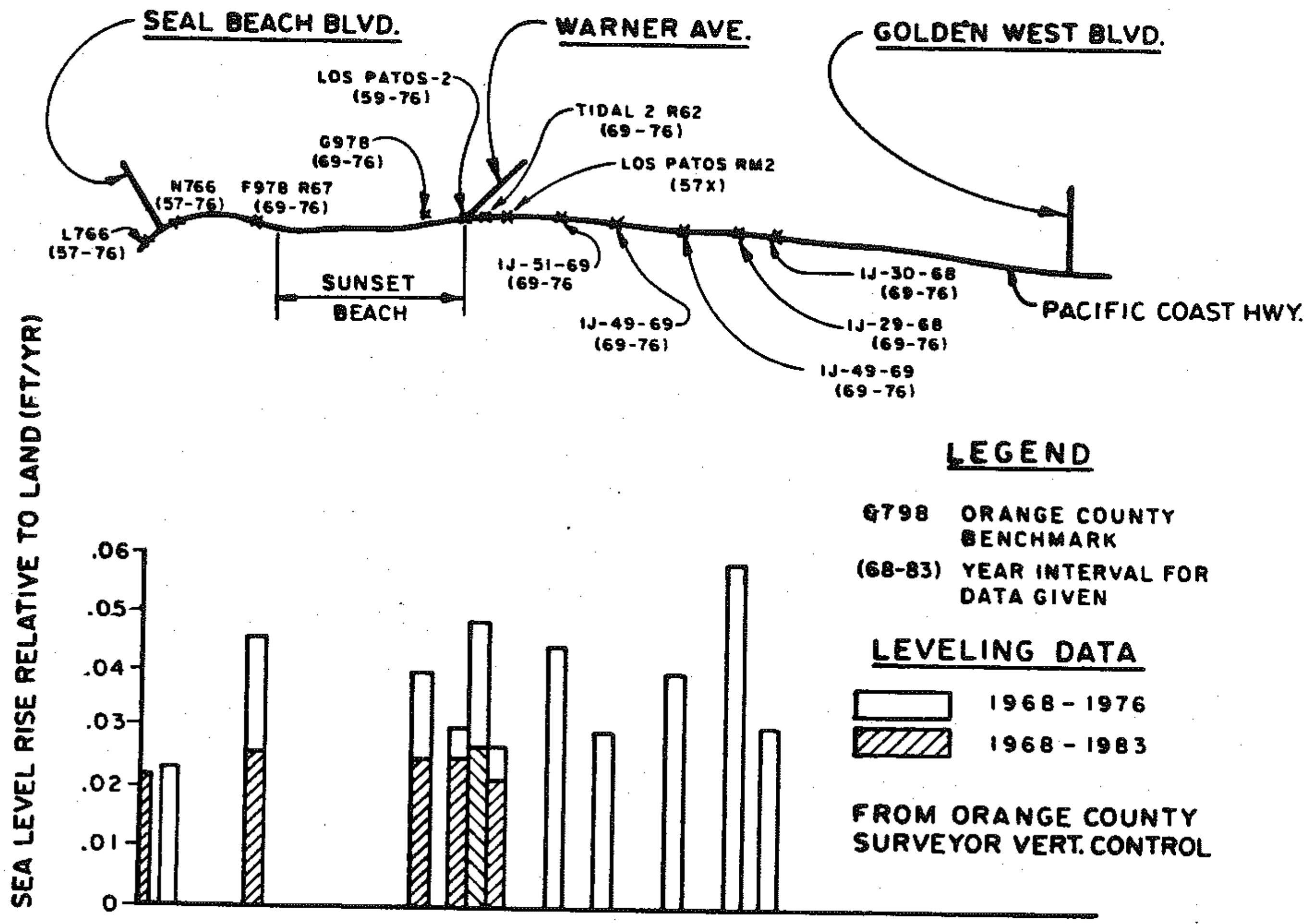

Figure A5. Sea level rise relative to land in the region that includes Sunset Beach.

(Environmental Protection Agency, 1983; National Research Council, 1982; and National Science Foundation, 1983) have concluded that a significant warming of the earth's atmosphere cannot be avoided in the next century. The groups differ in the effects of this warming, but not on its cause. An increase in atmospheric CO_2 due to fossil fuel consumption is creating a "greenhouse effect", that is, the entrappment of heat near the earth's surface. The present estimate is that the CO_2 content in the atmosphere will double from pre-industrial levels between 2050 and 2080. Present estimates are that the global average temperature will increase 2-3°C, with a larger increase at the poles. These temperature increases, which will likely cause a melting of some polar ice and a warming of surface ocean waters, because of the inertia of the oceanic response, may follow the CO_2 increase with a lag of 10-20 years. The lag in sea level rise with respect to the CO_2 increase will likely be even greater.

Present models to predict global sea level rise as a result of atmospheric warming have limitations that result in a wide range of forecasted sea level elevations. Most studies agree, however, that sea level will rise at a substantially greater rate in the next century than it has in the past century. The Environmental Protection Agency (Hoffman et. al., 1983) estimates a global rise of between 4.8 ft and 7 ft by 2100 AD is most likely, but states that an estimate of a low of 1.9 ft to a high of 11 ft cannot be discounted. The National Research Council, in a more conservative approach, has estimated a global sea level rise of 2.3 ft in the next 100 years.

A method using past sea level change rates is recommended. It is included in Chapter 1 of this report. Because sea level changes can be updated frequently using Orange County Surveyor information, this is considered the most accurate for design purposes.

APPENDIX B. DESIGN SCOUR ELEVATION

Design scour elevation is a function of location on the beach. Therefore, if the beach profile is changing position with time, the design (lowest) scour elevation at a fixed place will also change with time as illustrated in Figure B1. In general, seasonal changes in the beach profile will be much greater than the net yearly change that occurs over the period of many years. This, in consanance with the projected life of the structure, should be considered in its design.

This appendix treats each Local Coastal Program (LCP) area separately. Extreme scour elevation is given for the Ocean Protective Device String Line (OPDSL), which is the general seaward limit of improvements (decks, patios, etc.) along Sunset Beach, Emerald Bay and Capistrano Bay where homes are located on or near beach sand. In areas of bluffs and cliffs, i.e., South Laguna and Laguna Niguel, the OPDSL is generally at the base of the bluff or cliff. Shoreline change trends at each LCP area are given based on changes that occurred during 1967-1981 period.

Figure B1 shows the scour elevation referenced to the OCVD from which the design wave is calculated. It is the elevation that would exist near the OPDSL if the protective device were not there. The figure also shows the profile elevation at the base of a vertical wall. This is the scour elevation that must be considered in evaluating the stability of the device when no toe protection is provided.

Sunset Beach

Sunset Beach lies in the lower portion of a large crenulate-shaped bay (hooked shape) in the southeast lee of the Palos Verdes Peninsula. In the Twentieth Century the fate of the beach has been dominated by three human activities that reduced the supply of sand and one that increased it.

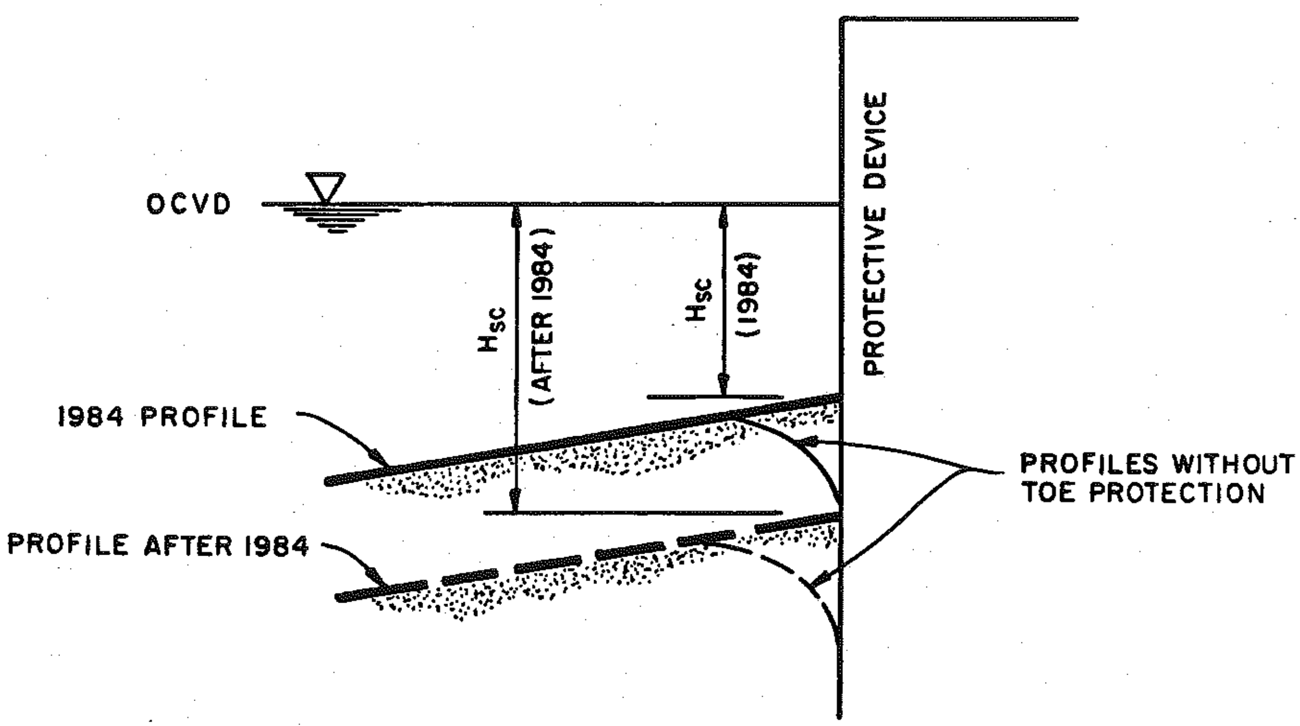

Figure B1. Design Scour Elevation, H_{sc}, after 1984 when toe protection is used (thick solid and thick dashed lines) and when toe protection is not used (thinner lines).

Construction of the Los Angeles-Long Beach Breakwater in the early to mid-1900's has partially shielded Sunset Beach from waves approaching from the west and southwest, and thereby caused a reduction in the alongshore movement of sand to Sunset Beach from the Los Angeles and San Gabriel River entrances. Simultaneously, a yet unquantified decrease in the volume of river-borne sand reaching the coast occurred. Sand entrapment by dams, sand and gravel extraction in river channels, and river flow regulation produced this progressive and continuing decline in sand supplied from upland sources. A complete or near-complete loss of littoral sand from updrift Seal Beach resulted when jetties were constructed at the entrance to Anaheim Bay by the Navy in 1944.

In the late 1940's the U.S. Army Corps of Engineers began artificially supplying sand to Surfside Colony and sometimes Sunset Beach. Most of this sand subsequently moved in a net south-easterly direction, thereby maintaining beaches at least as far as Huntington Beach and probably to Newport Beach. The health of the beach and the well-being of the homes of Sunset Beach are, at present, dependent upon periodic artificial nourishment.

Historic Shoreline Behavior. The shoreline probably reached its most landward limit sometime prior to construction of the jetties at Anaheim Bay in the early 1940's. A timber sheetpile bulkhead was constructed near the present private property line after development of Sunset Beach began in 1904. This suggests the property line was located in an exposed position at that time. The present beach varies in width between 100 and 300 ft as the result of beach replenishment. Figure B2 shows the volume of sediment placed on the beach and the year it was placed. It also shows the average rate of loss of that material between fill sequences. Since the Anaheim Bay jetties were constructed, 12,850,000 yd^3 (or an average of 330,000 yd^3/yr) of sand has been artificially added southeast of the Bay. The volume rate required has been increasing (Fig B2) probably because of shoreface adjustments. An analysis of the beachfill program will be required to determine when a dynamic equilibrium loss rate will be achieved. The

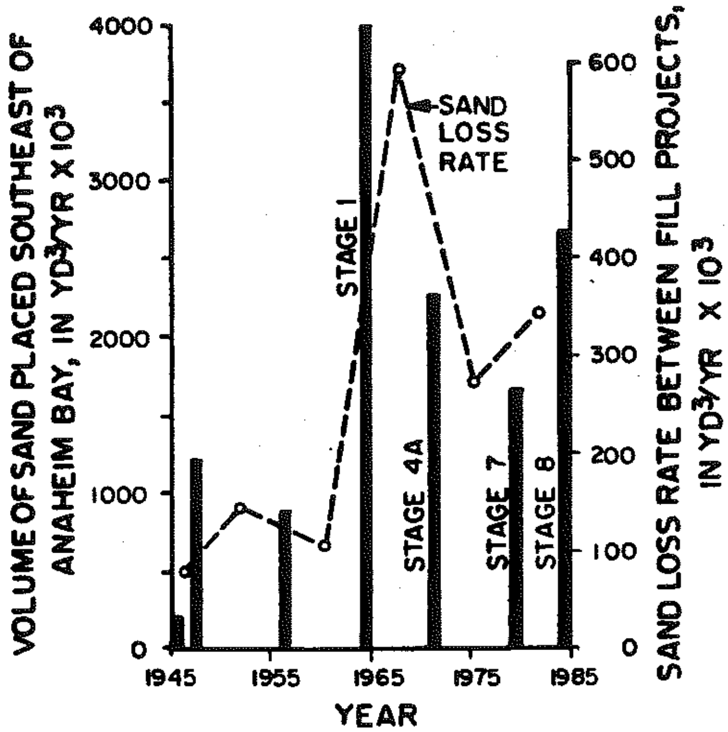

Figure B2. Sand volumes artificially placed on beaches southeast of the Anaheim Bay jetties since 1945 (from R. Clancy, Corps of Engineers, 1984).

average recent 300,000 to 600,000 yd^3/yr loss of placed material represents movement in an alongshore and possibly an offshore direction. Some offshore movements undoubtedly occurred in the initial years after replenishment began as the shoreface trended toward a position in equilibrium with the waves. Quite likely, however, most of the losses were the result of sand moving along the shore to the southeast. This sand moves south from Surfside Colony and subsequently past Sunset Beach. Present day offshore losses are unknown, but are probably quite a bit smaller than the volume moved in an alongshore direction..

Some general comments can be made concerning beach behavior at Sunset Beach:

(1) Scarps, where they occur, tend to be formed along the length of Sunset Beach. Scarps are vertical cuts made in the foreshore usually as a result of storms when storm waves do not greatly overtop the berm crest (which is typically at about +11 to +13 ft, OCVD).

(2) Scarps are more common after the beach is artificially filled because the beach face (foreshore) under such conditions, is steeper. A steep beachface will erode faster than a more gentle beachface, all other conditions being equal.

(3) The berm is typically sloped from the berm crest to the artificial dune toe in front of the first row of residences.. This reverse berm slope may be accentuated by the movement of artificial fill placed closer to the Anaheim Bay entrance.

(4) Seawater flow in the depression behind the berm crest is caused by wave overtopping and water ponding. Flow to relieve the ponded water is reported to be typically directed parallel to shore and to the southeast. This suggests the berm surface slopes not only toward land but also slightly toward the southeast.

(5) Flooding of streets only occurs when there is a high tide (not necessarily a perigean spring tide) accompanied by high storm waves. The storm waves produce wave setup and wave runup which cause the flooding.

(6) Shoreline change rates vary in an alongshore direction. The largest alongshore changes are at Surfside Colony, where the southeast jetty at Anaheim Bay acts as a headland and also reflects waves to the southeast. Shoreline changes downdrift (the direction in which most sand moves parallel to shore) of a headland are such that, without the addition of artificial fill material, a hook-shaped bay will form with its maximum indentation near the headland. This situation is most noticable at Surfside Colony. Wave reflections from the jetty enhance the longshore sediment transport rate to the southeast. The accelerated rate occurs as far as the bulge that often forms near Anderson Street. Further downdrift at Sunset Beach, major alongshore variations in transport are less and shoreline changes vary less from place to place.

Maximum Scour Level Sand loss from natural causes, and accelerated by human modifications of the littoral system, reached a maximum in the 1930's. At the bulkhead (property line) in Sunset Beach the water depth was reportedly three feet at high tide. This would place the sand elevation around 0 ft (OCVD). Alongshore variations in this depth are unknown, but were probably small, since various sources have reported 0 ft (OCVD) in the 1930's at different locations between Anderson Street and Warner (then Los Patos) Avenue.

Sunset Beach, under present littoral conditions, is dependent upon artificial beach fill for its recreational and protective benefits. Design considerations herein assume the beach nourishment program will continue at a rate and schedule equal to those of the past (Fig. B2). If the fill program that has been in effect since the 1940's is ended, and no other beach stabilization measures implimented, the sand at Sunset Beach would be lost at a rate of 200,000 to 500,000 yd^3/yr. The shoreline would retreat an average 20 to 50 ft/yr, and the 1930's scour limit at the property line would be exceeded within 10 years at some sites.

Emerald Bay

An 1885 United States Coast and Geodetic Survey (USC & GS) chart shows the Emerald Bay shoreline about 60 ft landward of the 1967 shoreline shown in Figure B3. The shoreline positions in Figure B3 are based on an analysis of aerial photographs taken in the period 1967-1981. As shown, the history of beach behavior at Emerald Bay has been one of stability or a slight progradation (movement seaward). Fluctuations in the position of the shoreline based on the aerial photograph analysis and observations of long-time residents indicates:

(1) shoreline position has fluctuated (normal to shore) as much as 150 ft in the period of a year,

(2) depending upon wave approach direction, the shoreline can shift toward the northwest or southeast headlands of Emerald Bay. These shifts (1973-northwest; 1981-southeast) may also accompany losses or gains of sand to or from the nearshore region,

(3) the shoreline sometimes reaches the bluffs near the headlands as a result of storms,

(4) sand in the central portion of Emerald Bay is not lost completely during severe storms, although much of it is carried seaward but apparently not lost around the headlands.

For design purposes, based on the 27 January 1983 storm which is considered by residents as the most severe to affect this beach, the lowest scour level is considered +1 ft OCVD at the base of the bluffs in the center (Fig. B3) of the Bay and at bedrock toward each of the headlands. Scour to bedrock has not been observed in the center of the Bay.

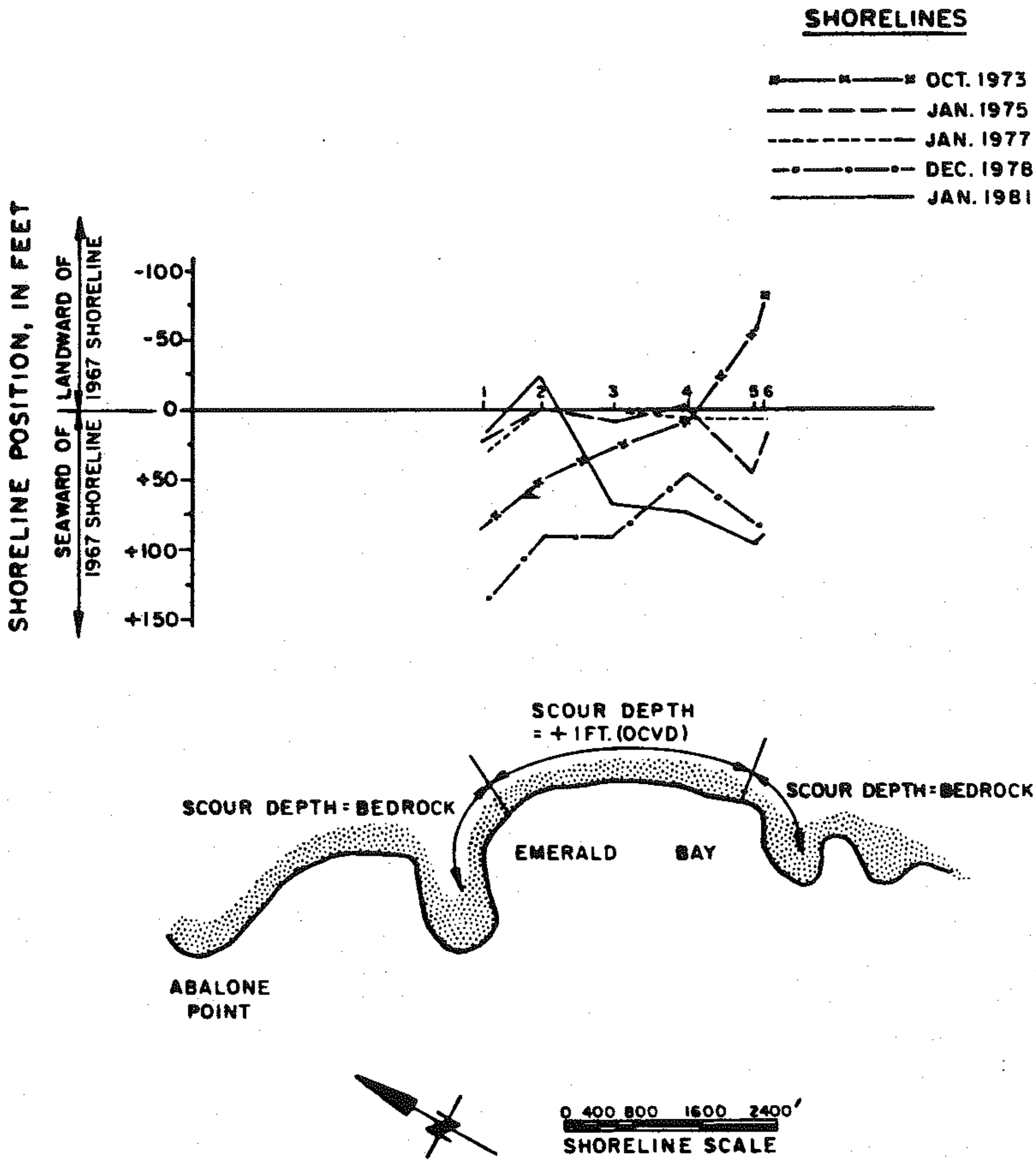

Figure B3. Shoreline position changes at Emerald Bay in the period 1967-1981. The zero-position or reference shoreline is the 1967 shoreline.

South Laguna

This bluff-backed reach of coast includes, from north to south, Victoria Beach, Aliso Beach, West Street Beach, 1000 Steps Beach, and Three Arch Bay. Aliso Beach, the longest of the South Laguna pocket beaches, is the only beach in this reach with a land source of sand (Aliso Creek) other than the backing bluffs.

Victoria Beach. In 1885 the shoreline in this 2100-ft long pocket beach was about 50 ft landward of the location of the 1981 shoreline. Nineteen-twenty photos of this beach show much less sand than existed in 1981. Shorelines in intervening years (1926, 1934, 1957, 1971) were all within 80 ft of each other in the central portion of the beach. Shoreline position changes in Figure B4 show fluctuations over the period of a few years can exceed 100 ft at the ends and about 50 ft in the center of Victoria Beach. The beach rotates about the middle, depending upon preceeding wave conditions, with a short-term, maximum shift landward or seaward at the bounding headlands of up to 150 ft.

For design purposes Victoria Beach can be considered stable, with significant fluctuations in shoreline position which rotate about the center of the embayment. The design scour depth is, therefore, considered non-varying with time (1984 conditions). Based on reported seasonal shoreline variations, reported scour depths after the 27 January 1983 storm, and the recent failure of "protective" walls near the center of the bay, the recommended design scour elevation is -3 ft (OCVD).

Aliso Beach. This 3600-ft long beach is cut by Aliso Creek near the center of the reach. It has been slightly progradational to stable in historic times at the northwest (Treasure Island) end (Fig. B4). A large fillet often forms at the southeast end after the creek has discharged sand to the shore (Fig. B5). This was the situation in 1959 and 1982 (chart data) and in 1981 (Fig. B5, based on an aerial photograph analysis). Shore-normal fluctuations on the southeast beach

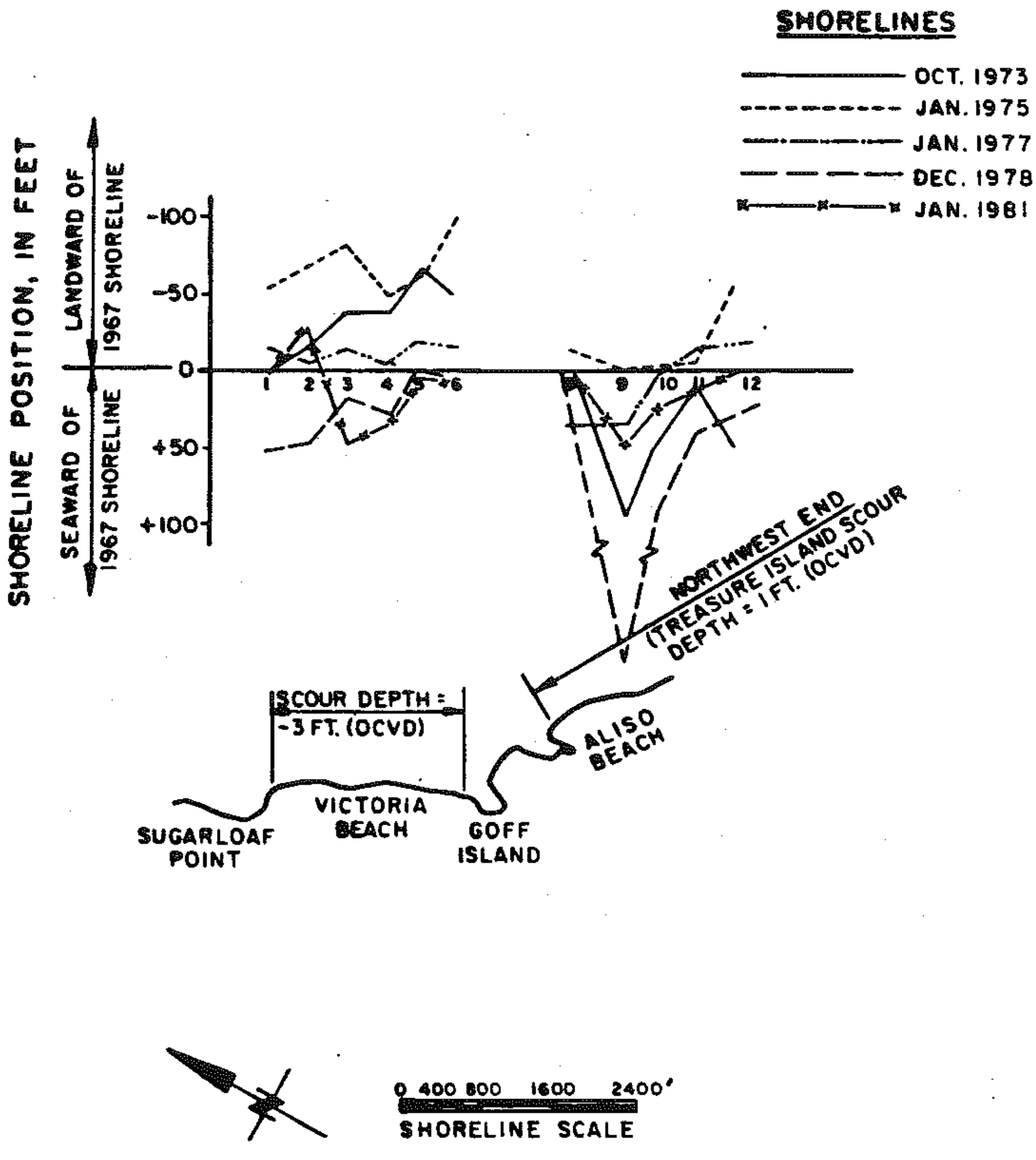

Figure B4. Shoreline position changes at Victoria Beach in the period 1967-1981. The zero-position or reference shoreline is the 1967 shoreline.

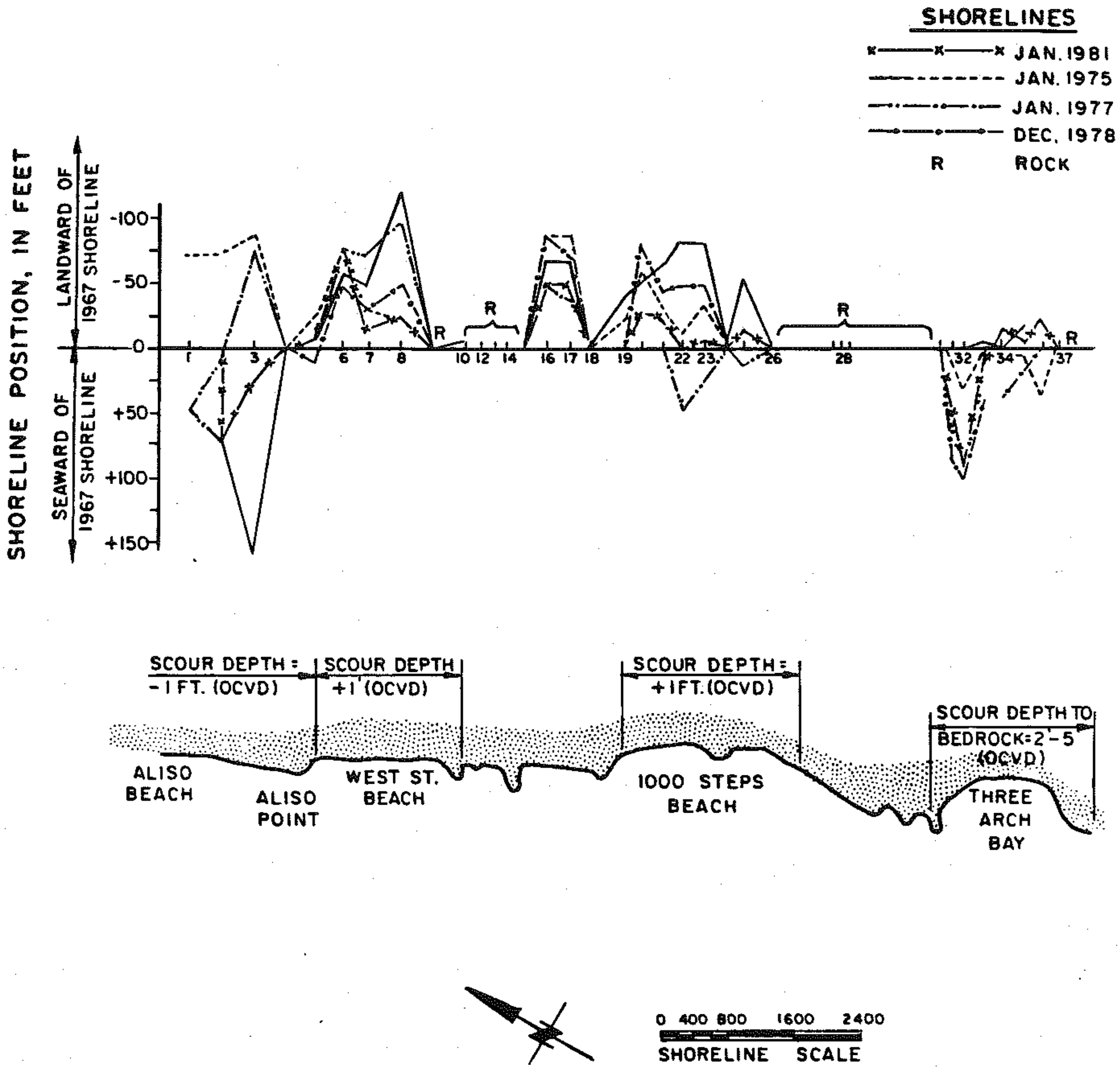

Figure B5. Shoreline position changes at Capistrano Beach in the period 1967-1981. The zero position is the reference shoreline of 1967.

were about 250 ft between 1967 and 1981. In 1971 the southeast beach had retreated to its 1885 position. In the 1920's the sand volume of this beach was less than it was in 1981.

Aliso Beach is considered a stable beach for design purposes, but one in which large fluctuations in shoreline position occur southeast of Aliso Creek. These fluctuations cannot be anticipated. Therefore, a sand level minimum of -1 ft (OCVD) should be used along the entire length of Aliso Beach. It should be noted also that Aliso Creek, in flood, often runs northwest and parallel to and against the bluffs.

West Street Beach. This beach system from northwest to southeast is composed of Camel Point Beach, Laguna Royale Beach and West Street Beach. It was widest in 1959. Oscillations have been large, i.e., to the northwest in 1934 and 1959, and to the southeast in 1982 and the maximum shoreline retreat occurred on 27 January 1983. In the period 1967-1981, the shoreline gradually retreated a maximum of about 100 ft on the southeast end and an average 60 ft (Fig. B5) for the entire beach. The maximum scour elevation is considered to be +1 ft (OCVD), which is above the bedrock level. This elevation can be expected to decline at a rate of 0.08 ft/yr (4 ft/yr shoreline retreat times an assumed 1:50 beach slope) after 1984 if the retreat experienced in the 1967-1981 period continues in the future.

1000 Steps Beach. This comprises two connected-beach systems separated by a small headland. The southeast beach is Paradise Cove Beach; the one on the north is 1000 Steps Beach. The total reach is 2400-ft long.

Historically this beach was widest in 1959. The 1982 beach had, on average, retreated 30 ft. Width fluctuations over the period of a year are less than 100 ft. Figure B5 shows this beach retreated an average 4 ft/yr in the period 1967-1981 with the largest retreat near the center of each embayment.

The 27 January 1983 storm has been reported as the most serious in local memory (including the September 1939 storm). In 1983, the scour level at the base of the bluffs dropped to a minimum of +1 ft (OCVD), which is taken as the 1984 design level. Not all the sand above bedrock was lost at this time and no bedrock was exposed in the central portions of the embayments. With an approximate average shoreline retreat rate of 4 ft/yr, for each year after 1984 the scour level should be decreased by 0.08 ft/yr. This is the same design scour situation that was recommended for West Street Beach.

Three Arch Bay. Three Arch Bay, like Emerald Bay, is a deeper pocket beach than West Street or 1000 Steps Beaches, but like the latter beaches it does not have a river source of sand. Also, like the Emerald Bay beach, the beach at Three Arch Bay has been a stable feature since at least 1934. Maximum beach width, based on a comparison of charts, occurred in 1959. Figure B5 shows that small alongshore shifts in the shoreline occur, i.e., to the northwest in the years 1977, 1978, and 1981 when compared to the 1967 and 1973 shoreline positions.

Sand was scoured to its lowest level during the 27 January 1983 storm when waves were breaking at the base of the bluffs. Rock was visible at the north and south ends of the Bay after the storm. In some places the bluffs have been notched near an elevation of +5 ft OCVD by wave activity. Bedrock at +2 to +5 ft (OCVD) at the base of the bluffs should be considered the design scour elevation. The elevation of bedrock at the base of the bluffs begins, known, and before a design to protect the bluffs is in process, bedrock depth should be delineated.

Laguna Niguel Beaches.

Salt Creek Beach on the northwest, which receives sand from Salt Creek, and Dana Strand Beach on the southwest, constitute this stable to slightly accretional beach system (Fig. B6). Yearly fluctuations exceed 200 ft near the outlet of Salt Creek and at the southeast end of Salt Creek Beach. At that location a minor headland separates Salt Creek Beach from Dana Strand Beach. It is reasonable to assume Salt Creek, like Aliso Creek, supplies sand in sufficient quantities to maintain this combined 8500-ft long, relatively straight beach system.

The worst storms of memory at Dana Strand Beach occurred on 27 January 1983 and 28 February 1983. Both occurred at the time of perigean spring tides. Wave runup during the storms overtopped a 12.3 ft (OCVD) revetment and eroded a vegetated slope above the revetment (runup to at least +15 ft OCVD) at Dana Strand Beach. Scour to bedrock at about -1 ft (OCVD) is a typical winter condition at Dana Strand Beach. The revetment there is founded on bedrock. Bedrock elevation at Salt Creek Beach is unknown. The design scour elevations is taken at -1 ft (OCVD) at both locations.

Capistrano Bay

History. This reach lies within a crenulate-shaped (hook-shaped) bay downdrift or downcoast of a headland. Dana Point was the headland prior to harbor construction, and presently the east end of the south breakwater of Dana Point Harbor is the controlling headland. Thus, harbor construction moved the apparent location of the headland a mile or so to the south. The effect of the harbor structures has, however, not yet been quantified. The alongshore movement of sand southeast and away from Doheny Beach and the San Juan Creek outlet may have been slowed because the beaches are now shadowed from waves which approach from the west and northwest (similar to the situation at Sunset Beach). The time to reach a new equilibrium bay shape will probably be large. In fact, some evidence exists that the natural bay before the harbor was constructed was also not in complete equilibrium. Had it been, there would have been a sandy beach extending to Dana Point. Bluffs south of Dana Point were eroding to bring the bay toward an equilibrium shape.

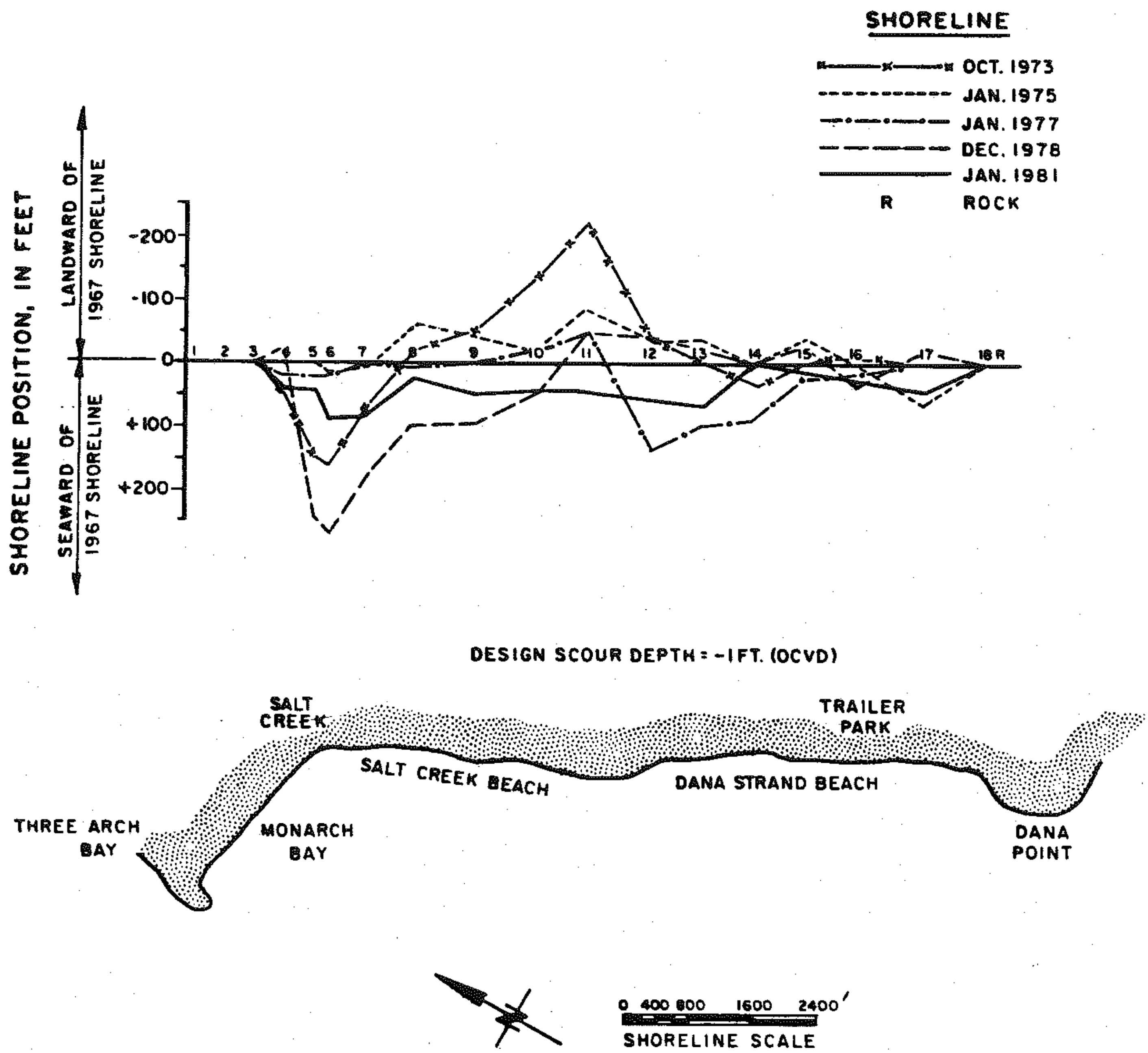

Figure B6. Shoreline position changes at Laguna Niguel for the period 1967-1981. The zero-position on reference shoreline is the 1967 shoreline.

San Juan Creek is a major contributor of sediment. The flood of 1916 is considered the maximum flood within living memory. Periods before 1960 probably produced more sand than between about 1960 and the present in which an average of about 130,000 yr^3/yr reached the coast. Excluding that which entered naturally or by dredging and placement on the beaches from San Juan Creek sources, a recorded 680,000 yd^3 from upland sources was placed on Doheny State Beach in 1966.

Waves and wave-created currents cause sediment to be moved both parallel and normal to shore. The extent and direction of such movement is dependent upon the magnitude of the waves (wave heights and periods) and the approach direction of the waves. Since 1978 wave energy reaching the coast from the south and southwest has increased. This means more of the total volume of sand moved parallel to shore has moved to the north and northwest. Consequently, progradation of the shoreline north of the zero shoreline change location on Figure B7 since 1978 may be caused to some extent by changes in yearly wave climate. Large changes occur in the development of a crenulate-shaped bay when wave conditions change over a long time period.

The volume of sediment moved offshore is unknown, but it may have been significant since 1978. In a longer term perspective, there is evidence that in some areas of Southern California sand moves very slowly in a net onshore direction. Based on existing knowledge, the net losses of sand from Capistrano Bay beaches will probably be small when averaged for a long term in the future unless the sea level rise relative to the shoreline increases.

An analysis of the effects of sea level rise at Capistrano Bay north of the inflection point was made. It indicates a sea level rise of 0.004 ft/yr (present rise rate which, however, may increase in the future) has caused an average shore retreat of 0.7 ft/yr, or an apparent (and insignificant) sand loss of 10,000 yd^3/yr at the Capistrano Bay Community in the past.

Historic Shoreline Behavior. Figure B7 shows shoreline changes that have occurred on the Capistrano Bay Community beach and or Doheny State Beach between 1967 and 1984. The maximum shoreline progradation (movement seaward) along the northwest portion of the reach occurred just after Dana Point Harbor was constructed (between 1967 and 1973). Progradation since 1973 has generally continued, and the overall reach of progradation has expanded to the southeast (Fig. B7). Prior to the early 1960's this beach was slightly erosional. An erosion problem had existed at the northwest end of the Capistrano Bay Community and was most severe in the 1940's and 1950's.

The seaward shoreline bulge that is presently so predominant near the northwest end of the Capistrano Bay Community is also found on a 1934 USC & GS chart. The bulge does not show predominantly on a subsequent chart in 1959. The zone of curvature of this bulge on Figure B7 appears to be slowly migrating to the southeast.

Based upon an analysis of shoreline changes shown in Figure B7, an average of about 100,000 yd^3/yr has been deposited between San Juan Creek and the zero shoreline-change location since 1967. This is about 75 percent of the estimated recent yearly discharge from San Juan Creek.

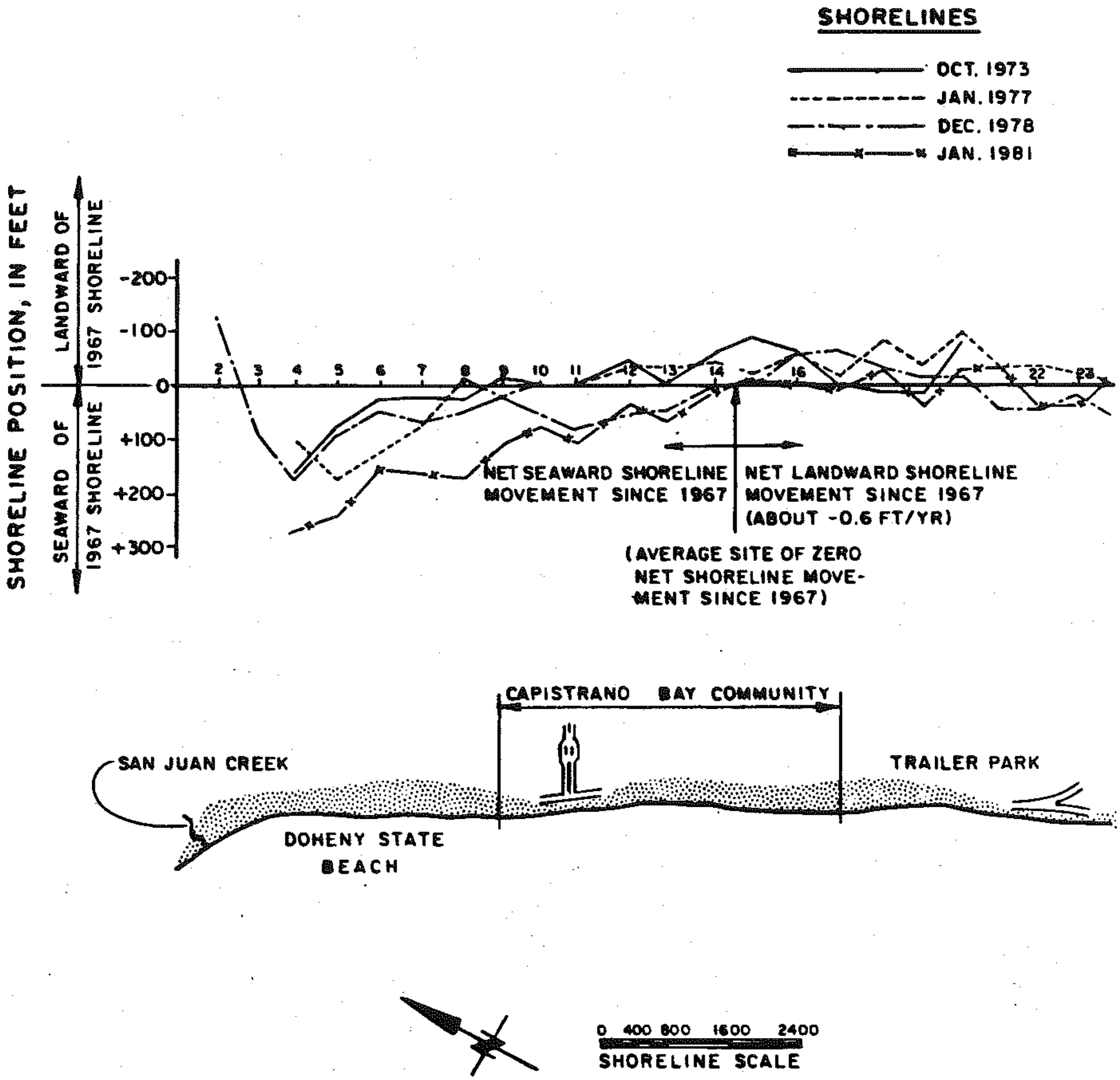

Figure B7. Shoreline position changes at Capistrano Beach in the period 1967–1981. The zero position is the refrence shoreline of 1967.

APPENDIX C. DESIGN WAVE CHARACTERISTICS

Wave characteristics for design purposes are usually determined for deepwater sites and propagated through shallowing water to the site of the proposed structure. Along sandy shorelines of the County of Orange, waves that reach devices designed to protect private property are depth-dependent. That is, the height of the extreme wave that will stand without breaking on, or in front of, a protective device is limited by the water depth. Deepwater waves are considered in order to provide a more complete treatment of wave characteristics, and to justify the depth-limited criterion used in this report. The wave characteristics provided are those that would break at the property line in water depths obtained from data presented in Appendixes A and B.

Wave height is required to establish wave forces on the structure, uprush elevation, and other needed design data. Because water depth in front of a structure varies, the wave height at the structure also varies. The nature of the wave may also change during a tidal cycle from non-breaking to breaking, to possibly only runup if the wave breaks in front of the structure. The wave data presented for design purposes are for a depth-limited wave that would break on the structure.

<u>Deepwater Waves</u>. Recent advances in the study of surface waves has emphasized the usefulness of an analysis of wave energy spectra in estimating wave conditions for design purposes (U.S. Army, Corps of Engineers, 1984). Wave energy taken as a function of frequency (the inverse of wave period) provides more information because the processes that transform wave energy are sensitive to wave period. Spectral approaches consider this. From a practical standpoint most procedures for coastal design use a single design wave height and wave period. However, there is a relationship between an application of spectral wave parameters and the simpler design height and period. Significant wave height (the average of the highest one-third of all waves), which is a height parameter often used, corresponds to an

energy-band wave height parameter. The peak spectral period (the inverse of the dominant frequency of the wave energy spectrum) is comparable to the significant wave period in many situations (U.S. Army Corps of Engineers, 1984).

Historic charts that show the distribution of meterological conditions at a given time have been used to calculate wave characteristics for major storms off the southern California coast. Because of difficulty and cost, the actual measurement of wave climate in deep water has not been done along this coast. Prior to the 1983 winter storms, hindcasts by Marine Advisors (1960) for the period 1900 to 1958, and the Department of Navigation and Ocean Development (1977) for the period 1951 to 1974, were used in the selection of design waves. In 1983, the highest waves since 1939 occurred. Walker et.al. (1984) show that the recurrence interval for the deepwater significant wave height increased about 25 percent when the 1900-1957 (Marine Advisors, 1960) hindcasts are compared to 1900-1983 hindcast waves (Fig. C1).

Islands offshore of the County of Orange block some portions of the directional spectrum of storm waves. For 13 major storms which occurred between 1900 and 1957, Marine Advisors (1960) calculated the ratio of wave height landward of the islands (after island filtering) to wave height seaward of the islands. In the Oceanside area they calculated the ratio to be 1:4 to 1:1.2. The latter value is for 24-25 September 1939, in which a tropical storm caused the severest waves that occurred between 1900 and 1957 from Pt. Fermin to San Diego. Island sheltering along the County of Orange coast is probably on the order of that calculated for Oceanside. For storms along the coast of southern California, the highest waves have arrived at shore generally in a five to nine hour period (Marine Advisors, 1960).

The 15-25 September 1939 storm was the most severe that has occurred since 1900 from the standpoint of wave height. The maximum wind of that tropical storm at the Los Angeles-Long Beach Outer Harbor was 50 knots. Wave heights observed at the harbor ranged from 12 to 40 ft. Swell heights were estimated at 30 ft by people ashore. Ships in the

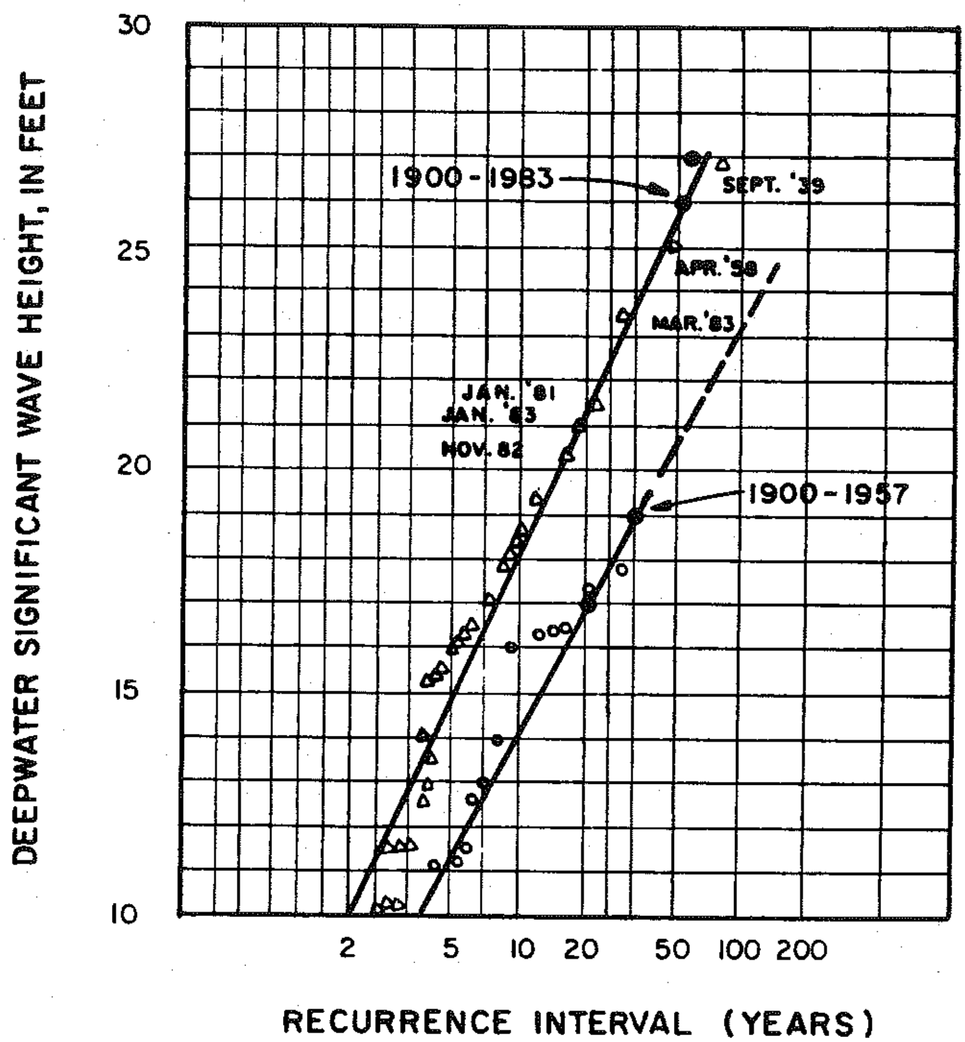

Figure C1. Recurrence interval for deepwater significant wave height for the periods 1900-1957, and 1900-1983, showing the increase in height vs recurrence interval for the latter period (from Walker et al, 1984).

Catalina Channel reported 45-ft high waves. Damage to the Los Angeles-Long Beach Harbor breakwater occurred for the second time. The first time it was damaged was in April 1930 as a result of southern hemisphere swell. It was damaged a third time on 27 January 1983. The significant deepwater wave height of the 1939 storm was 27 ft.

Seymour, et.al. (1984) have calculated deepwater wave approach directions for a site 50-mi west of Los Angeles. For 42 storms which produced hindcast wave heights of over 10 ft between 1900 and 1983, they found the wave approach direction to be as given in Table C1.

TABLE C1. DEEPWATER WAVE APPROACH DIRECTION

Approach Direction	Percent of Storms
South (160° - 220°)[1]	26
West (250° - 290°)	52
North (320° - 350°)	22

[1]includes four presumed hurricanes.

Seymour et. al. (1984) note that the 22 percent of storm waves that came out of the northwest was unexpectedly low. The northwest track has characteristically been assumed to dominate the wave climate off southern California. During those years when the ENSO effect exists, large waves out of the west may reach the County of Orange coast from a semi-permanent low, north of Hawaii. Waves out of the north are unlikely then because Bering Sea storms are held to the Aleutians. Tropical storms which approach out of the south develop as surface water temperatures rise. Because ENSO events cause an increase in water temperature, severe waves from the south are more likely when the ENSO condition prevails. Seymour et al (1984) found hurricanes (severe tropical storms) associated with four strong ENSO events, (1911, 1925, 1957, 1982). Two strong ENSO events did not produce major storms (1918, 1941). Of storms out of the south, Seymour et al

(1984) found 73 percent were associated with the ENSO phenomenon, including three of the four fall hurricanes which occurred. Fifty percent of the storms out of the west were associated with ENSO events. No storms out of the north showed this association. Thus, 58 percent of the major non-northern storms were associated with an ENSO event. This is significant because of the ten largest storm events since 1900, eight were from the west and two from the south. None were from the north. The relationship between storms and ENSO events appears significant. The effect of an ENSO event on water surface elevations is most pronounced between August and February, the storm season.

Storms from January to March 1983 differed significantly from previous storms. Maximum deepwater significant wave heights varied between 13 and 24 feet for eight storms. Two of the most significant storms produced waves with exceptionally long periods of 22 seconds (Seymour, et.al., 1984). The largest storm of the winter occurred on 27 January 1983. The energy density of that storm, the third largest calculated, was slightly less than that of the 1939 (September) hurricane out of the south, and slightly less than a storm out of the west which occurred in April 1958. For design purposes, Seymour, et.al. (1984) suggest the 1983 storm year might be expected to occur with a recurrance interval of 25-30 years.

The recurrance interval for wave heights of 24 ft (27 January 1983) to 27 ft (15-25 September 1939) is 50-100 years (Fig. C1). Island filtering effects will probably not decrease extremal wave heights by more than 20 percent (as was the case of the 1939 hurricane waves at Oceanside). Assuming the approach of the extreme wave is normal to shore and diffraction is not a factor, all extreme waves which act upon protective devices along the County of Orange coast fronted by a beach will be depth-dependent.

APPENDIX D. SUNSET BEACH FLOODING

Flooding of coastal residences at Sunset Beach is caused by a combination of a high still water level elevation and high wave runup elevation. Damages occur when water enters residences by flowing or seeping through floors, walls or glass areas. Sediment carried by wave runup may directly cause damage when sand is packed under a house and the subflooring is lifted, or indirectly when wood systems are weakened by long exposure to wet sand. Damage caused by the direct impact of waves has not been a recent problem at Sunset Beach and is not considered under the flood heading.

CHARACTERISTICS OF FLOOD EVENTS. Flooding at Sunset Beach occurs with some regularity. The following storms are remembered by residents as having an effect on the beach and some resulting flood problems:

15-25 September 1939: This hurricane produced the largest deepwater waves of record off Orange County. It destroyed the boardwalk, carried beach sand away, and dropped the sand level to approximately 0 ft (OCVD) against the bulkhead. Most, if not all, homes were then built on piles. Those homes that were lost were constructed at a low enough elevation to be directly affected by wave action. This storm is remembered as the second most serious storm (after the 27 January 1983 storm) in memory. Flooding was not as great in September 1939 as it was 27 January 1983.

1963: Several homes in Surfside Colony were destroyed during a storm in 1963.

18 February 1969: This storm caused flooding up to 1.5 ft deep between 5th and 11th Streets. It produced breaking wave heights at the shore in excess of 6 ft. The wave period was long, which residents refer to as "ground swell." It was not a time of perigean spring tides.

21 December 1976: This storm produced very heavy surf and caused flooding up to a reported 3-ft depth on Pacific Coast Highway at Anderson Street. Breaking waves were reported to be 7 to 8-ft high and the tide was about +4.2 ft (OCVD).

8 January 1978: This was a perigean spring tide without large waves. Water filled the depression landward of the natural berm crest, but caused no street flooding.

30 November 1982: Gale force winds of 40-50 mph produced large waves which, accompanied by +3.2 to 3.9 ft (OCVD) tides and 3 inches of rain, caused the worst flooding that had occurred in seven years. Flooding was most serious between 10th and 19th streets where no artificial dune existed.

29 December 1982. This flood event moved water onto Pacific Coast Highway at 11th Street from the Bay. The cause was a perigean spring tide, surge caused by onshore winds, and wave runup resulting from high waves.

27 January 1983: This storm is considered to be the worst from a flooding perspective since Sunset Beach was developed. Some minor isolated structural damage may also have occurred. It was one of a set of four storms affecting the Southern California coast between 22 and 29 January. The 27 January event occurred at the peak perigean spring tide when the ENSO (El Nino) effect had also raised the ocean surface 0.5 to 0.7 ft above normal. With wind and wave surge, this storm produced the maximum recorded still water elevation (+5.17 ft, OCVD) at Los Angeles Harbor. Water surface elevations at an artificial dune in front of South Pacific Avenue homes were greatest one hour before to two hours after high tide the morning of the 27th. Flow to and across the dune and through street-end breaches in the dune was reported at a maximum of 1.5 ft deep and strongest between Warner Avenue and 16th Street. Flow velocities at the street ends probably reached the order of 10 ft/sec. Pacific Coast Highway

and intermediate streets were flooded to 2 ft depths. Bayview Street on Sunset Island was under 0.5 - 1.0 ft of water from flooded channels in Huntington Harbor. Most of this floodwater probably came from freshwater land runoff and saltwater contributions through Anaheim Bay. Overwash directly from the beach was probably of secondary importance.

Flow over and through low spots in the artificial dune and especially through street-end openings carried much beach sand landward. Some homes were filled with sand to 0.5 to 2 ft. In some places homes on piles had a similar build-up of beach sand under them. Pacific Coast Highway was closed for almost two months between Seal Beach Blvd. and Golden West St. in order to clear sand from the roadway.

The largest waves occurred about one hour past the time of predicted high tide. Breaking waves offshore were reported to be 8 to 15-ft high. Waves 16-ft high were reported at the Seal Beach Pier. The wave period was very long at 22 seconds. Four to five breaker lines were reported offshore.

As shown in Figure D1, in May, 1984, the artificial dune (called a berm by beachfront residents) had a crest elevation average of +13 to +16 ft (OCVD). Street-end breaches in the dune at the time of the storm were at an elevation of approximately 12 ft (OCVD). The elevation of the berm at the toe of the dune near Anderson Street, and of the natural berm farther east was about +11 ft (OCVD). The natural berm dips landward from its crest near the ocean with an elevation of +12 to +13 ft (OCVD). This is between 100 and 250 ft seaward of the artificial dune.

28 February 1983: Sand was deposited on Pacific Coast Highway in Huntington Beach as a result of this storm.

2 March 1983: This storm, which did not occur during high tides, produced very large waves. At the end of the Huntington

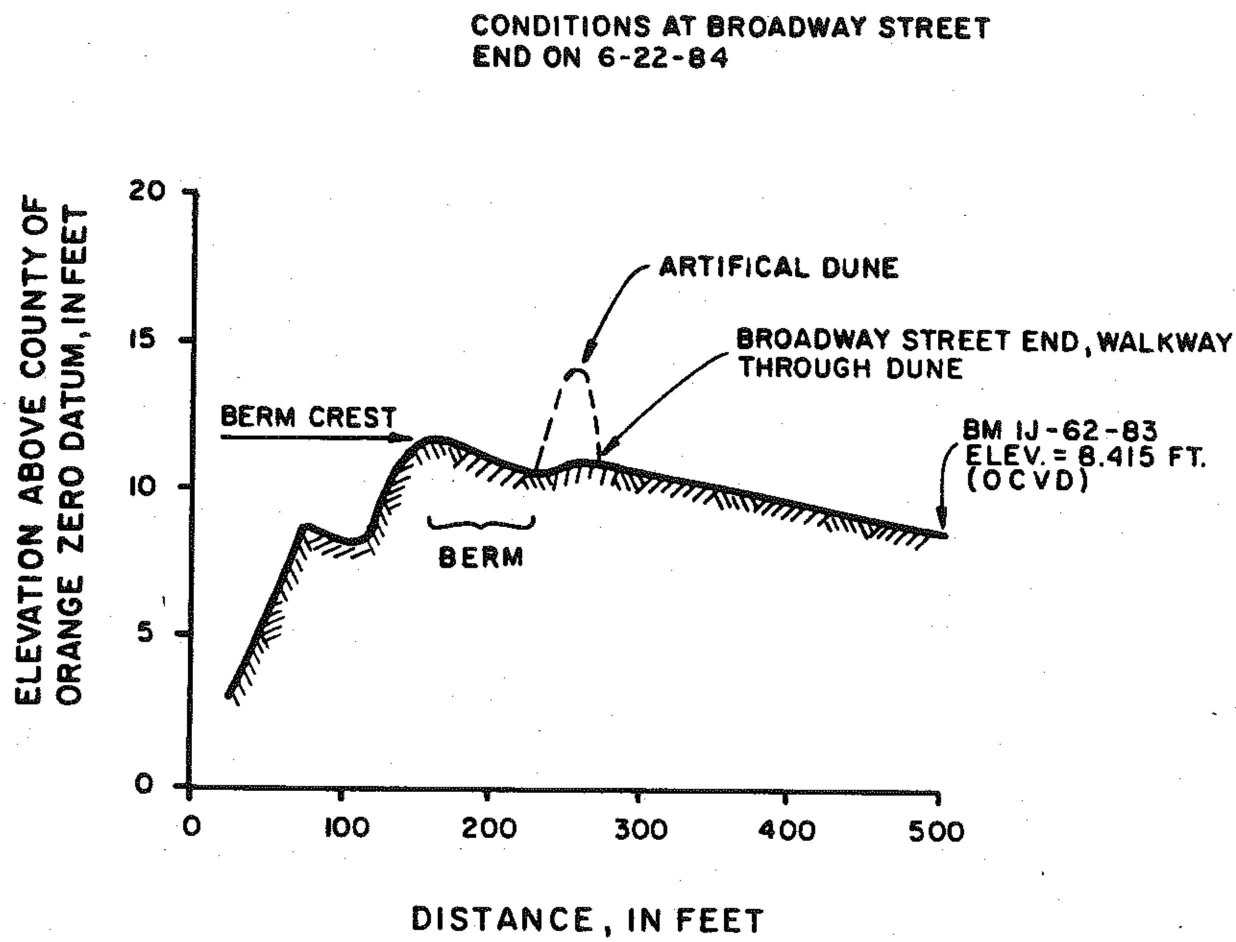

Figure D1. Cross-section across beach from North Pacific Avenue to foreshore. Sunset Beach, 22 June 1984.

Beach Pier breaking waves 14-ft high were reported. Wave heights at oil-production islands off the north coast of the County of Orange were greater than 15 ft.

8 August 1983: This hurricane-produced storm, which occurred at the time of a perigean spring tide, caused flooding on Pacific Coast Highway.

7 September 1983: Another perigean spring tide event, this storm carried about 1 ft of water through streets between 4th and 8th streets.

MAXIMUM RUNUP ELEVATION. Field data provide the best estimate of the maximum wave runup elevation during a design event. Observations of residents and videotapes of the runup, waves, and flow characteristics of the landward-directed current are available and are presented here for the flood of record (27 January 1983) at Sunset Beach. The maximum runup elevation was about 15.2 ft (OCVD) on a typical profile similar to that shown in Figure D1. Wave runup calculations using accepted empirical relationships, such as those provided in the Shore Protection Manual (1984), yield maximum runup results that are generally too large by at least two feet. The reason is that empirical data are unavailable for a beach with a compound slope and the sequences of slopes that exist at Sunset Beach. A theoretical approach to predict runup on Sunset Beach is also unavailable.

MITIGATION OF THE FLOOD PROBLEM. Flood problems can be mitigated by preventing water or sediment from penetrating the living area or damaging the foundation, joists or subflooring of structures. The best protection is provided when the natural protective beach is maintained at the appropriate width, elevation, and slope, or an artificial dune is employed with sufficient elevation such that runup flow does not overtop it and reach the structure under design conditions and with a sufficient sand volume that storm erosion will not destory it. Even the complete blockage of wave runup in front of a residence, however, may not preclude flooding if the neighboring properties are not adequately protected.

In many cases it will be impossible to completely eliminate overtopping because of limits placed on the elevation of the artificial dune. Acceptable overtopping values are then dependent upon the elevation and grade on which the structure is founded, and upon the characteristics of the structure. For this reason, structures on caissons or piles are recommended for Sunset Beach. Structures on piles or caissons allow a partial or completely uninhibited landward flow of sea water. When some overtopping must be accommodated, adequate drainage is mandatory. Flood problems caused by the accumulation of water at the structure usually occur because of inadequate landward drainage at such a rate that the water surface elevation is everywhere maintained below the underside of the structure.

GENERAL COMMENTS ON AN ARTIFICIAL DUNE. Flooding of residences at Sunset Beach along South Pacific Avenue and further inland can be prevented or reduced by maintaining a continuous (no street-end openings) artificial dune and a return dune or wall at both ends of the beach beyond which the dune is not maintained, i.e., Surfside Colony and southeast of Warner Avenue. This dune exists, with openings, today, but at an elevation (+13.7 to +15.2, OCVD) that will, in most cases, not completely prevent overtopping, flooding and sediment movement landward during a storm such as occurred on 27 January 1983. The dune elevation design tradeoff is, of course, the amount of flooding that is acceptable (based on a specified dune elevation), and the view which is restricted as the dune elevation increases. Residences on piles or caissons will provide a less restricted view for a specified dune elevation than residences on grade. With the subsidence occurring in the Sunset Beach area, a planned increase in dune crest elevation with time for a given measure of protection should be considered. Based on recent relative sea level changes at Sunset Beach (Fig. A5), our increase in crest elevation would not be warranted on a frequency greater than once per 20 years.

The dune should be continuous. That is, street end access for emergency vehicles should, if used, be limited and consist of vehicle ramps over the dune or access through the dune with a mechanism for quickly closing the access breach in a storm. Ramps for pedestrian and vehicle access protect the dune and insure its integrity as a storm protection device. Many communities on the U.S. Atlantic and Gulf of Mexico coasts, and in Australia, require the dune be preserved with such ramps which are usually constructed of treated wood for reasons of aesthetics and economics. If selected street ends are allowed to remain open, special cautionary measures should be enacted prior to and during a time of perigean spring tides (Table A2, Appendix A) when the tide is accompanied by a storm. Also, during years when an El Nino (ENSO) event is occurring, special caution should be exercised because in those circumstances storms occur on a much greater than average frequency.

If the beach nourishment program in effect in 1984 is halted or delayed, beach erosion that threatens homes in Sunset Beach should be anticipated. This erosion, if it continues long enough, will destroy the artificial dune and homes will be open to wave attack as well as to flooding. Protection will then be obtained only through the use of seawalls, bulkheads or revetments. Homes constructed on piles above the breaking wave elevation will sustain less damage than homes on slabs or on joists resting on shallow foundations in the sand. In time, however, if the beach continues to retreat, the homes on piles will also be lost.

LEGEND

1. FP-3 -----

FP-3 LINE - Heavy dashed line on map. Structures must be protected from wave activity in the FP-3 zone, located seaward of the FP-3 line. The FP-3 line would be near the landward limit of wave uprush on an undeveloped beach or bluff during a storm.

2. OPDS ———

OPDS LINE - Light dashed line on map. The Ocean Protective Device Stringline is the seaward limit beyond which protective devices may not be constructed. The seaward edge of the crest of a protective device may not extend seaward of the OPDS line.

3. BREAKING WAVE CHARACTERISTICS

NOT APPLICABLE AT SUNSET BEACH.

wave runup elevation limit on a rough 1 on 1.5 (rise/run) sloped revetment*

wave runup elevation limit on a smooth vertical seawall or bulkhead*

design breaking wave elevation*

*Above OCVD at the FP-3 line considering 1984 conditions

4. DEFINITIONS

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REVETMENT - A protective device consisting of a facing of stone, concrete, cast units, etc., built to protect a scarp, embankment, or structure against erosion by wave action or currents.

RUNUP - The rush of water up a protective device, beach, bluff face or structure on the breaking of a wave. The amount of runup is the vertical height above stillwater level that the rush of water reaches. The wave runup elevation limit is the highest elevation that will be reached by the rush of water from a breaking wave when that wave occurs during the design wave event with the specified design recurrence interval. The highest elevation subject to wetting by spray from the design wave will be greater than the runup elevation.

SEAWALL - A protective device that separates land and water areas, primarily designed to prevent erosion and other damage due to wave action.

STRUCTURE - A habitable dwelling, cabana, garage, deck, restroom, etc., located in the FP-3 zone. The design life of the foundation of a structure, which must be equal to or greater than 30 years, is the minimum period after construction during which all major components of the foundation system not protected by a protective device retain their functional and structural design capabilities.

MOFFATT & NICHOL, ENGINEERS
LONG BEACH, CALIFORNIA

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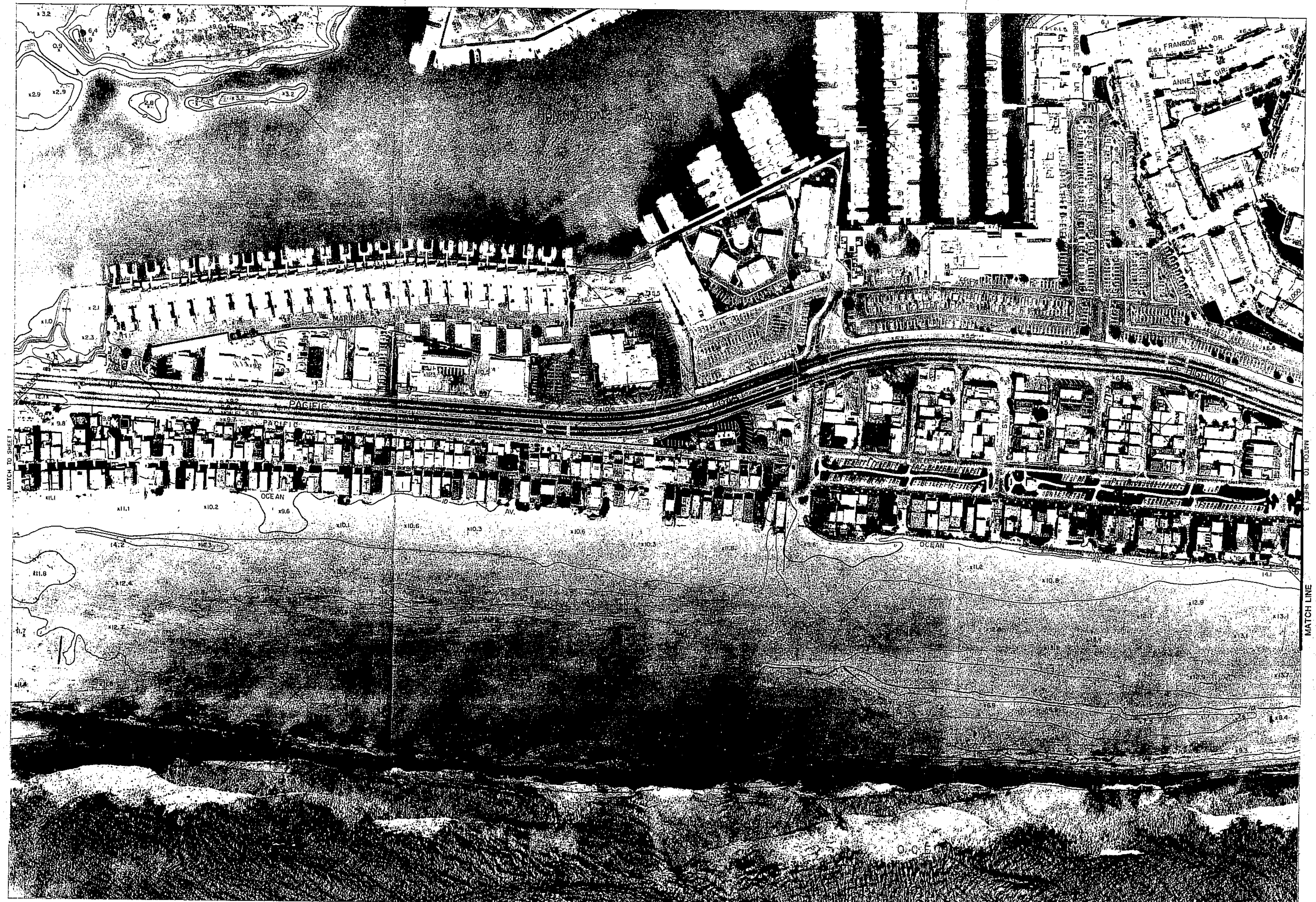

TOPOGRAPHIC MAP SYMBOLS

HORIZONTAL AND VERTICAL CONTROL POINT
HORIZONTAL CONTROL POINT
VERTICAL CONTROL POINT
OBSCURED CONTOURS
DEPRESSION CONTOURS
x123.5 SPOT ELEVATIONS

SHEET INDEX

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ORANGE COUNTY
ENVIRONMENTAL MANAGEMENT AGENCY
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COASTAL FLOOD PLAIN
SUNSET BEACH

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T.F. RATTRAY L.S. 3200

PHOTO DATE: 4/10/84
CONTOUR INTERVAL: 2&5 FT.
SCALE: 1"= 100'

SHEET 2 OF 4 SHEETS

LEGEND

1. FP-3 - - - - -

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OPDS LINE - Light dashed line on map. The Ocean Protective Device Stringline is the seaward limit beyond which protective devices may not be constructed. The seaward edge of the crest of a protective device may not extend seaward of the OPDS line.

3. BREAKING WAVE CHARACTERISTICS

NOT APPLICABLE AT SUNSET BEACH.

- wave runup elevation limit on a rough 1 on 1.5 (rise/run) sloped revetment*
- wave runup elevation limit on a smooth vertical seawall or bulkhead*
- design breaking wave elevation*

*Above OCVD at the FP-3 line considering 1984 conditions

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MOFFATT & NICHOL, ENGINEERS
LONG BEACH, CALIFORNIA

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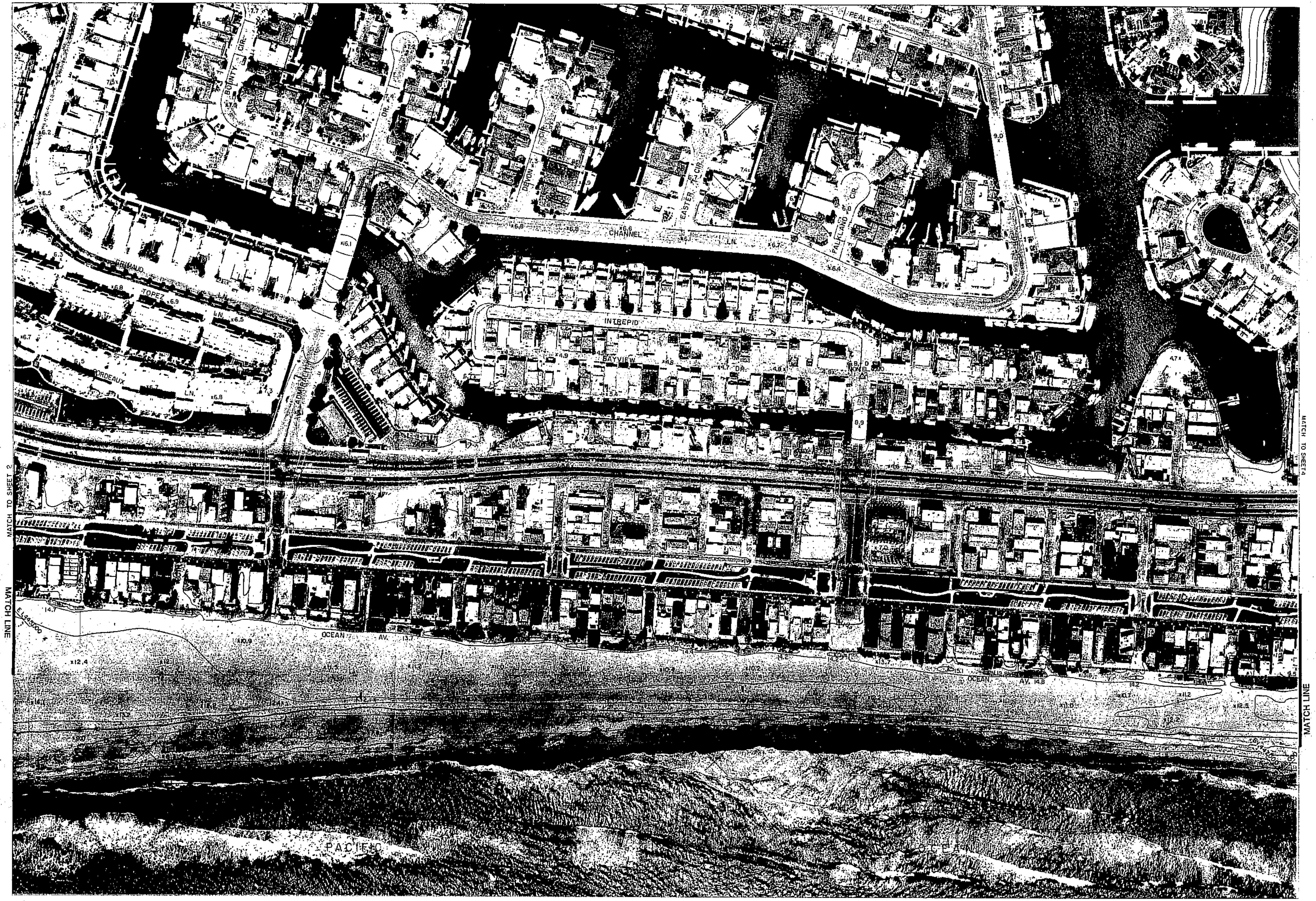

TOPOGRAPHIC MAP SYMBOLS

- HORIZONTAL AND VERTICAL CONTROL POINT
- HORIZONTAL CONTROL POINT
- VERTICAL CONTROL POINT
- OBSCURED CONTOURS
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- x 123.5 SPOT ELEVATIONS

SHEET INDEX

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ORANGE COUNTY
ENVIRONMENTAL MANAGEMENT AGENCY
PUBLIC WORKS COUNTY SURVEYOR DIVISION

COASTAL FLOOD PLAIN
SUNSET BEACH

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T.F. RATTRAY · L.S. 3022

PHOTO DATE: 4/10/84
CONTOUR INTERVAL: 2 & 5 FT.
SCALE: 1" = 100'

SHEET 3 OF 4 SHEETS

2

LEGEND

1. FP-3 -----

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NOT APPLICABLE AT SUNSET BEACH.

wave runup elevation limit on a rough 1 on 1.5 (rise/run) sloped revetment*

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design breaking wave elevation*

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LONG BEACH, CALIFORNIA

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URBAN DESIGN
VAN DELL/SHIOTA PLANNING GROUP

TOPOGRAPHIC MAP SYMBOLS

HORIZONTAL AND VERTICAL CONTROL POINT

HORIZONTAL CONTROL POINT

VERTICAL CONTROL POINT

OBSCURED CONTOURS

DEPRESSION CONTOURS

x123.5 SPOT ELEVATIONS

SHEET INDEX

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ORANGE COUNTY
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COASTAL FLOOD PLAIN
SUNSET BEACH

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T.F. RATTRAY L.S. 3280

PHOTO DATE: 4/12/84
CONTOUR INTERVAL: 2&5 FT.
SCALE: 1" = 100'

SHEET 4 OF 4 SHEETS

LEGEND

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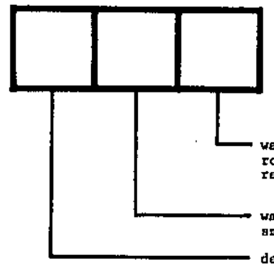

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SHEET INDEX

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PUBLIC WORKS-COUNTY SURVEYOR DIVISION

COASTAL FLOOD PLAIN
EMERALD BAY

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T.F. RATTRAY L.S. 3200

PHOTO DATE: 4/10/84
CONTOUR INTERVAL: 2-5 FT.
SCALE: 1" = 100'

SHEET 1 OF 2 SHEETS

4

LEGEND

1. FP-3 -----

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MOFFATT & NICHOL, ENGINEERS
LONG BEACH, CALIFORNIA

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URBAN DESIGN
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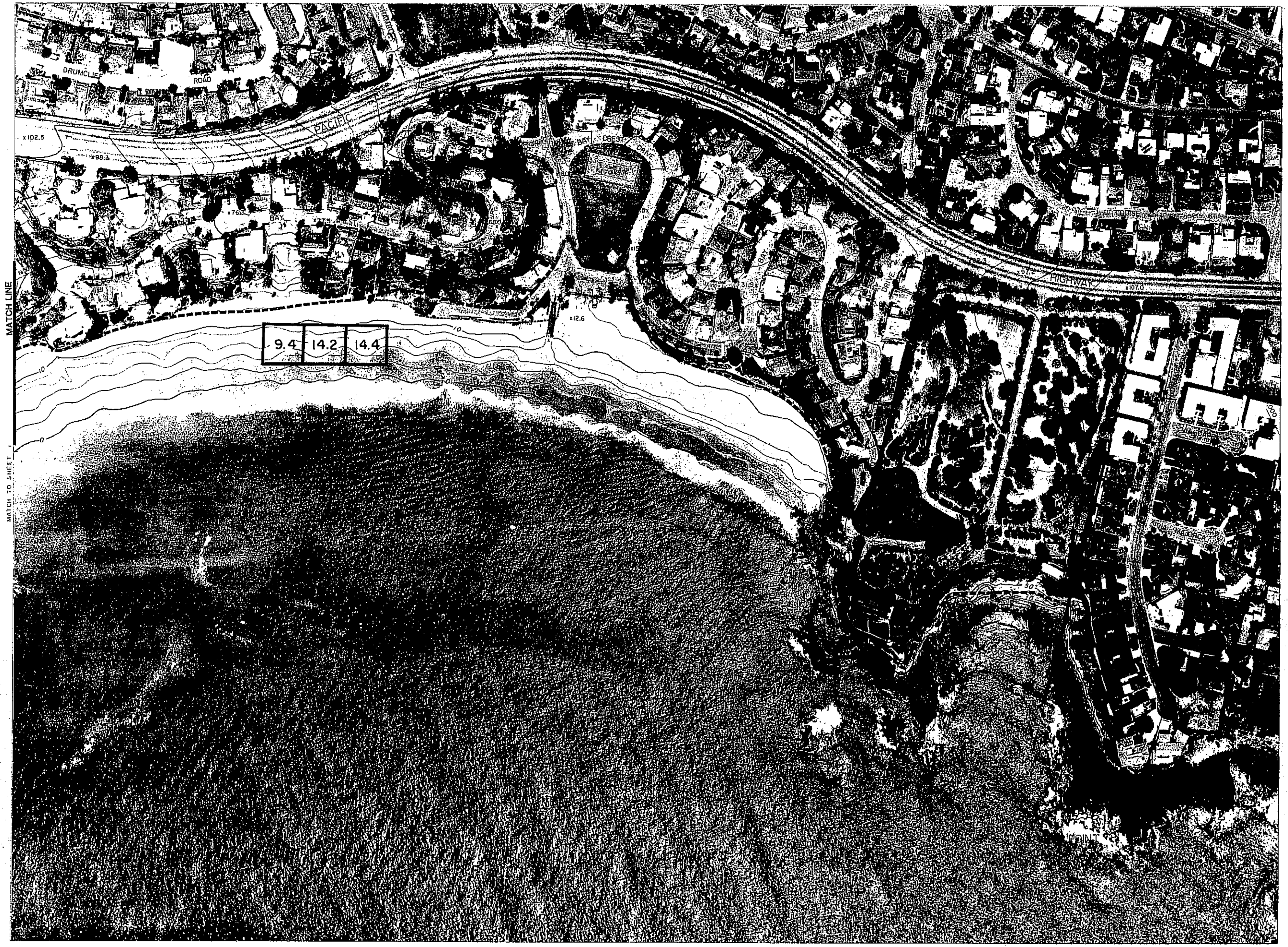
714/474-1412



TOPOGRAPHIC MAP SYMBOLS

HORIZONTAL AND VERTICAL CONTROL POINT

HORIZONTAL CONTROL POINT

VERTICAL CONTROL POINT

OBSCURED CONTOURS

DEPRESSION CONTOURS

x 123.5 SPOT ELEVATIONS

SHEET INDEX

NOTE:
CONTOUR ELEVATIONS ARE BASED ON MEAN SEA LEVEL DATUM, ORANGE COUNTY SURVEYOR 1976 ADJUSTMENT.

ORANGE COUNTY
ENVIRONMENTAL MANAGEMENT AGENCY
PUBLIC WORKS-COUNTY SURVEYOR DIVISION

COASTAL FLOOD PLAIN
EMERALD BAY

RATTRAY AND ASSOC., INC.
SURVEYING & MAPPING
ESCONDIDO, CA. LOS ANGELES, CA. SANTA ANA, CA.
(619)745-9666 (213)413-4097 (714)979-9452

I CERTIFY THAT THIS MAP MEETS THE AGREED-UPON ACCURACY REQUIREMENTS.

T.F. RATTRAY L.S. 3202

PHOTO DATE: 4/10/84
CONTOUR INTERVAL: 2-5 FT.
SCALE: 1"= 100'

SHEET 2 OF 2 SHEETS

5

LEGEND

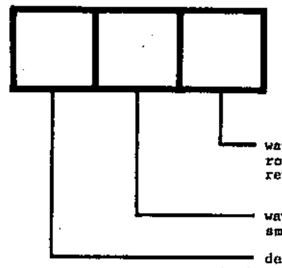
1. FP-3 -----

FP-3 LINE - Heavy dashed line on map. Structures must be protected from wave activity in the FP-3 zone, located seaward of the FP-3 line. The FP-3 line would be near the landward limit of wave uprush on an undeveloped beach or bluff during a storm.

2. OPDS ———

OPDS LINE - Light dashed line on map. The Ocean Protective Device Stringline is the seaward limit beyond which protective devices may not be constructed. The seaward edge of the crest of a protective device may not extend seaward of the OPDS line.

3. BREAKING WAVE CHARACTERISTICS



*Above OCVD at the FP-3 line considering 1984 conditions

4. DEFINITIONS

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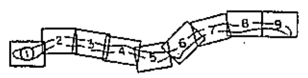
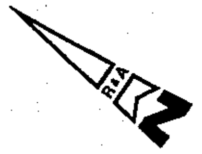
IRVINE, CA 92714
714/474-1412

TOPOGRAPHIC MAP SYMBOLS

- HORIZONTAL AND VERTICAL CONTROL POINT
- HORIZONTAL CONTROL POINT
- VERTICAL CONTROL POINT
- OBSCURED CONTOURS
- DEPRESSION CONTOURS
- 123.5 SPOT ELEVATIONS



SHEET INDEX



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ORANGE COUNTY
ENVIRONMENTAL MANAGEMENT AGENCY
PUBLIC WORKS-COUNTY SURVEYOR DIVISION

COASTAL FLOOD PLAIN
LAGUNA NIGUEL / SOUTH LAGUNA

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SURVEYING & MAPPING
ESCONDIDO, CA. (619)745-9666 — LOS ANGELES, CA. (213)413-4087 — SANTA ANA, CA. (714)879-9450

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T.F. RATTRAY — L.S. 3200

PHOTO DATE: 4/10/84
CONTOUR INTERVAL: 2 & 5 FT.
SCALE: 1" = 100'

SHEET 1 OF 9 SHEETS

LEGEND

1. FP-3 -----

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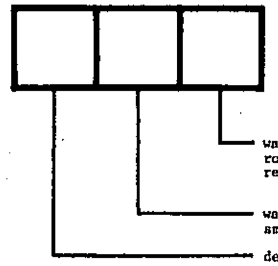
3. BREAKING WAVE CHARACTERISTICS



wave runup elevation limit on a rough 1 on 1.5 (rise/run) sloped revetment*

wave runup elevation limit on a smooth vertical seawall or bulkhead*

design breaking wave elevation*

*Above OCVD at the FP-3 line considering 1984 conditions

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IRVINE, CA 92714
714/474-1412

TOPOGRAPHIC MAP SYMBOLS

HORIZONTAL AND VERTICAL CONTROL POINT

HORIZONTAL CONTROL POINT

VERTICAL CONTROL POINT

OBSCURED CONTOURS

DEPRESSION CONTOURS

SPOT ELEVATIONS

SHEET INDEX

ORANGE COUNTY
ENVIRONMENTAL MANAGEMENT AGENCY
PUBLIC WORKS COUNTY SURVEYOR DIVISION

COASTAL FLOOD PLAIN
LAGUNA NIGUEL / SOUTH LAGUNA

PATTRAY AND ASSOC., INC.
SURVEYING & MAPPING

1" = 100'	2 9

LEGEND

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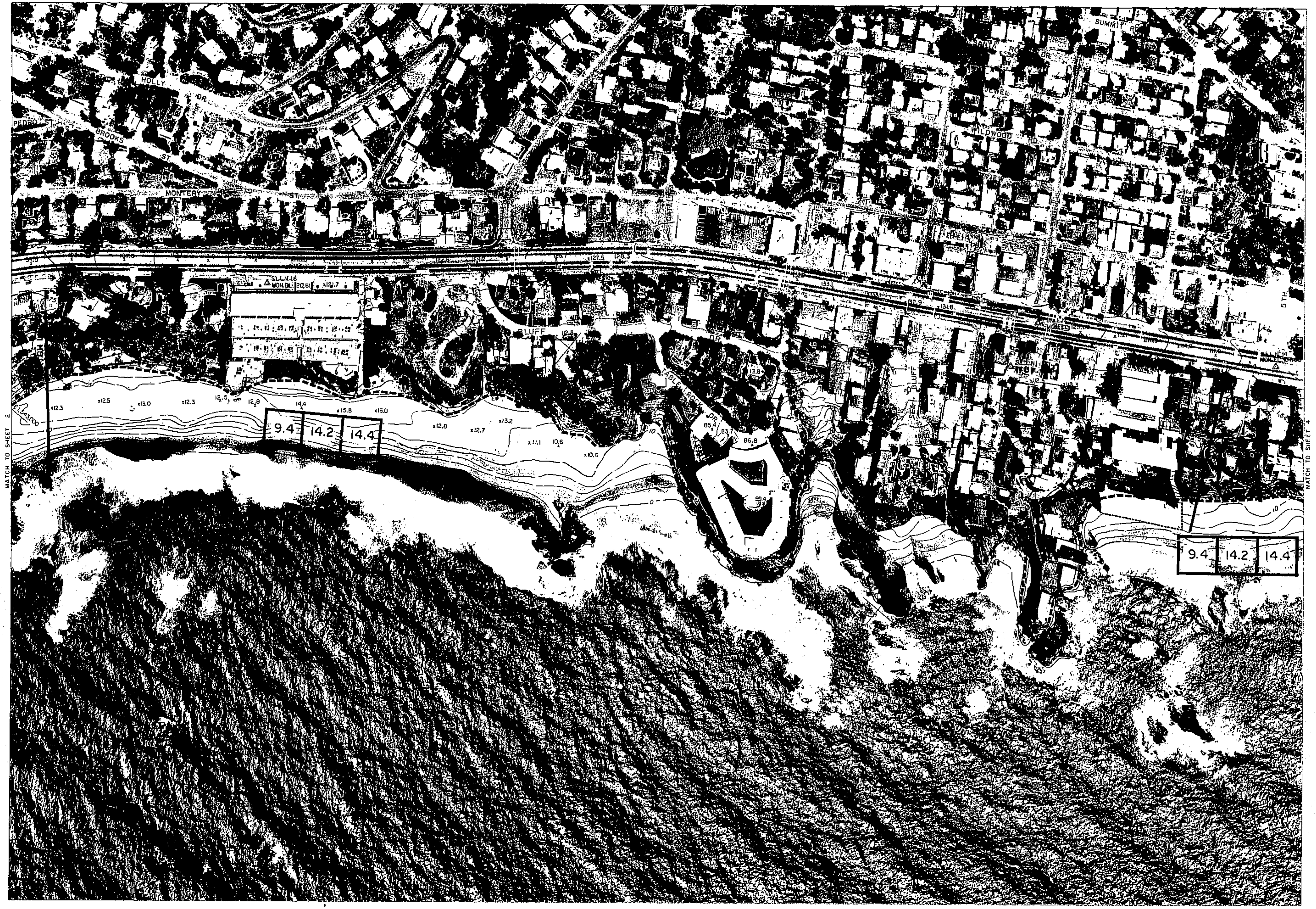
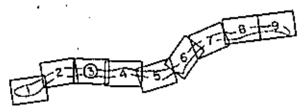


TOPOGRAPHIC MAP SYMBOLS

- HORIZONTAL AND VERTICAL CONTROL POINT
- HORIZONTAL CONTROL POINT
- VERTICAL CONTROL POINT
- OBSCURED CONTOURS
- DEPRESSION CONTOURS
- x123.5 SPOT ELEVATIONS



SHEET INDEX

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ORANGE COUNTY
ENVIRONMENTAL MANAGEMENT AGENCY
PUBLIC WORKS-COUNTY SURVEYOR DIVISION

COASTAL FLOOD PLAIN
LAGUNA NIGUEL / SOUTH LAGUNA

RATTRAY AND ASSOC., INC.
SURVEYING & MAPPING
ESCONDIDO, CA. (619)745-9666 LOS ANGELES, CA. (213)413-4087 SANTA ANA, CA. (714)979-9452

I CERTIFY THAT THIS MAP MEETS THE AGREED UPON ACCURACY REQUIREMENTS.

PHOTO DATE: 4/10/84
CONTOUR INTERVAL: 2.85 FT.
SCALE: 1" = 100'

SHEET 3 OF 9 SHEETS

LEGEND

1. FP-3 -----

FP-3 LINE - Heavy dashed line on map. Structures must be protected from wave activity in the FP-3 zone, located seaward of the FP-3 line. The FP-3 line would be near the landward limit of wave uprush on an undeveloped beach or bluff during a storm.

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wave runup elevation limit on a rough 1 on 1.5 (rise/run) sloped revetment*

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17801 CARTWRIGHT ROAD
IRVINE, CA 92714
714/474-1412

TOPOGRAPHIC MAP SYMBOLS

HORIZONTAL AND VERTICAL CONTROL POINT
HORIZONTAL CONTROL POINT
VERTICAL CONTROL POINT
OBSCURED CONTOURS
DEPRESSION CONTOURS
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SHEET INDEX

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ORANGE COUNTY
ENVIRONMENTAL MANAGEMENT AGENCY
PUBLIC WORKS-COUNTY SURVEYOR DIVISION

COASTAL FLOOD PLAIN
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ESCONDIDO, CA. (619)743-9666 LOS ANGELES, CA. (213)413-4087 SANTA ANA, CA. (714)979-9450

I CERTIFY THAT THIS MAP MEETS THE AGREED-UPON ACCURACY REQUIREMENTS.
T.F. RATTRAY L.S. 3202

PHOTO DATE: 4/10/84
CONTOUR INTERVAL: 2&5 FT.
SCALE: 1" = 100'

SHEET 4 OF 9 SHEETS

C

LEGEND

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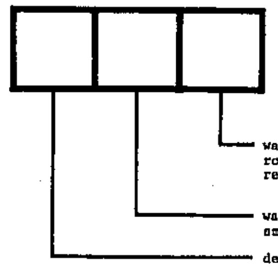

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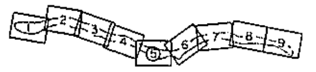

SHEET INDEX

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ENVIRONMENTAL MANAGEMENT AGENCY
PUBLIC WORKS-COUNTY SURVEYOR DIVISION

COASTAL FLOOD PLAIN
LAGUNA NIGUEL / SOUTH LAGUNA

RATTRAY AND ASSOC., INC.
SURVEYING & MAPPING
ESCONDIDO, CA. (619)745-9666 LOS ANGELES, CA. (213)413-4087 SANTA ANA, CA. (714)979-9450

I CERTIFY THAT THIS MAP MEETS THE AGREED-UPON ACCURACY REQUIREMENTS.
T.F. RATTRAY L.S. 3202

PHOTO DATE: 4/10/84
CONTOUR INTERVAL: 2 & 5 FT.
SCALE: 1" = 100'

SHEET 5 OF 9 SHEETS

LEGEND

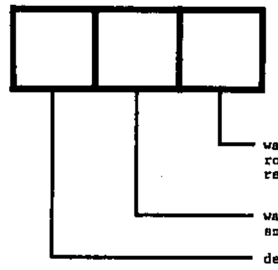
1. FP-3 -----

FP-3 LINE - Heavy dashed line on map. Structures must be protected from wave activity in the FP-3 zone, located seaward of the FP-3 line. The FP-3 line would be near the landward limit of wave uprush on an undeveloped beach or bluff during a storm.

2. OPDS ———

OPDS LINE - Light dashed line on map. The Ocean Protective Device Stringline is the seaward limit beyond which protective devices may not be constructed. The seaward edge of the crest of a protective device may not extend seaward of the OPDS line.

3. BREAKING WAVE CHARACTERISTICS



*Above OCVD at the FP-3 line considering 1984 conditions

4. DEFINITIONS

BULKHEAD - A protective device designed as a partition to retain or prevent sliding of the land. A secondary purpose is to protect the upland against damage from wave action.

DESIGN BREAKING WAVE ELEVATION - Highest elevation above OCVD that would be impacted by wave forces breaking against a vertical wall, if one existed, normal to the direction of wave approach. The upper limit of breaking waves is a measure of wave action based on a design wave and a design water depth condition with a specified design recurrence interval.

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RECURRENCE INTERVAL - Time period during which one coastal design event can be expected to occur. The 100-year recurrence interval, which must be used for the design of structures and protective devices in Orange County, is the statistical probability that one event that produces a limiting value of a coastal phenomenon will occur in 100 years, or that it has a one percent probability of occurring in a single year. Recurrence interval should not be confused with design life which references an absolute time interval not a probabilistic value.

REVETMENT - A protective device consisting of a facing of stone, concrete, cast units, etc., built to protect a scarp, embankment, or structure against erosion by wave action or currents.

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SEAWALL - A protective device that separates land and water areas, primarily designed to prevent erosion and other damage due to wave action.

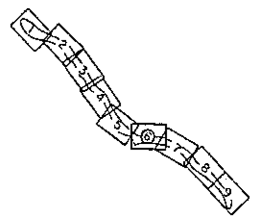
STRUCTURE - A habitable dwelling, cabana, garage, deck, restroom, etc., located in the FP-3 zone. The design life of the foundation of a structure, which must be equal to or greater than 30 years, is the minimum period after construction during which all major components of the foundation system not protected by a protective device retain their functional and structural design capabilities.

MOFFATT & NICHOL, ENGINEERS
LONG BEACH, CALIFORNIA

VSP GROUP
COMMUNITY PLANNING
LANDSCAPE ARCHITECTURE
URBAN DESIGN
VAN DELL/SHIOTA PLANNING GROUP
17801 CARTWRIGHT ROAD
IRVINE, CA 92714
714/474-1412

TOPOGRAPHIC MAP SYMBOLS

HORIZONTAL AND VERTICAL CONTROL POINT
HORIZONTAL CONTROL POINT
VERTICAL CONTROL POINT
OBSCURED CONTOURS
DEPRESSION CONTOURS
123.5 SPOT ELEVATIONS



SHEET INDEX

NOTE:
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ORANGE COUNTY
ENVIRONMENTAL MANAGEMENT AGENCY
PUBLIC WORKS-COUNTY SURVEYOR DIVISION

COASTAL FLOOD PLAIN
LAGUNA NIGUEL / SOUTH LAGUNA

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T.F. RATTRAY L.S. 3200

PHOTO DATE: 4/10/84
CONTOUR INTERVAL: 2.85 FT.
SCALE: 1" = 100'

SHEET 6 OF 9 SHEETS

11

LEGEND

1. FP-3 -----

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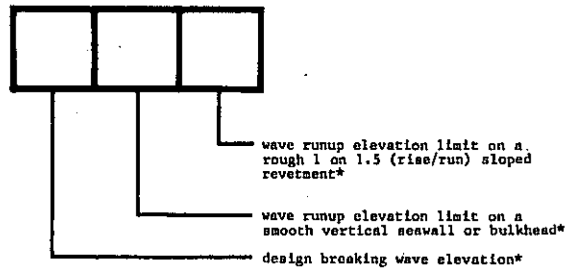

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MOFFATT & NICHOL; ENGINEERS
LONG BEACH, CALIFORNIA

VSP GROUP
COMMUNITY PLANNING
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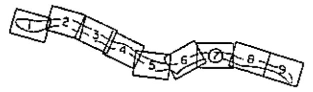
URBAN DESIGN

VAN DELL/SHIOTA PLANNING GROUP
17801 CARTWRIGHT ROAD
IRVINE, CA 92714
714/474-1412

TOPOGRAPHIC MAP SYMBOLS

- HORIZONTAL AND VERTICAL CONTROL POINT
- HORIZONTAL CONTROL POINT
- VERTICAL CONTROL POINT
- OBSCURED CONTOURS
- DEPRESSION CONTOURS
- x123.5 SPOT ELEVATIONS



SHEET INDEX

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ORANGE COUNTY
ENVIRONMENTAL MANAGEMENT AGENCY
PUBLIC WORKS-COUNTY SURVEYOR DIVISION

COASTAL FLOOD PLAIN
LAGUNA NIGUEL / SOUTH LAGUNA

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T.F. RATTRAY — L.S. 3200

PHOTO DATE: 4/10/84
CONTOUR INTERVAL: 2 & 5 FT.
SCALE: 1" = 100'

SHEET 7 OF 9 SHEETS

LEGEND

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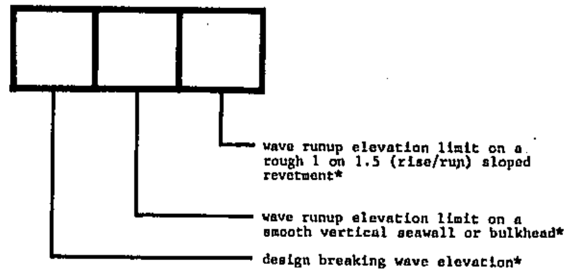

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MOFFATT & NICHOL, ENGINEERS
LONG BEACH, CALIFORNIA

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URBAN DESIGN
VAN DELL/SHIOTA PLANNING GROUP
17801 CARTWRIGHT ROAD
IRVINE, CA 92714
714/474-1412

TOPOGRAPHIC MAP SYMBOLS

HORIZONTAL AND VERTICAL CONTROL POINT
HORIZONTAL CONTROL POINT
VERTICAL CONTROL POINT
OBSCURED CONTOURS
DEPRESSION CONTOURS
x 123.5 SPOT ELEVATIONS

SHEET INDEX

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ORANGE COUNTY
ENVIRONMENTAL MANAGEMENT AGENCY
PUBLIC WORKS-COUNTY SURVEYOR DIVISION

COASTAL FLOOD PLAIN
LAGUNA NIGUEL / SOUTH LAGUNA

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ESCONDIDO, CA. (619)745-9666 | LOS ANGELES, CA. (213)413-4087 | SANTA ANA, CA. (714)979-9452

I CERTIFY THAT THIS MAP MEETS THE AGREED UPON ACCURACY REQUIREMENTS.

PHOTO DATE: 4/10/84
CONTOUR INTERVAL: 2&5 FT.
SCALE: 1" = 100'

SHEET 6 OF 9 SHEETS

LEGEND

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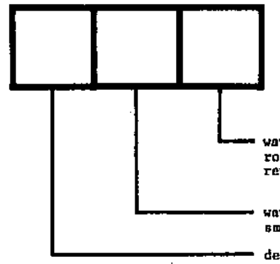

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LONG BEACH, CALIFORNIA

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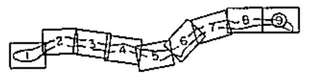
URBAN DESIGN
VAN DELL/SHIOTA PLANNING GROUP
17801 CARTWRIGHT ROAD
IRVINE, CA 92714
714/474-1412

TOPOGRAPHIC MAP SYMBOLS

△ (with dot in circle)	HORIZONTAL AND VERTICAL CONTROL POINT
△	HORIZONTAL CONTROL POINT
⊙	VERTICAL CONTROL POINT
5	OBSCURED CONTOURS
	DEPRESSION CONTOURS
× 123.5	SPOT ELEVATIONS



SHEET INDEX

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ORANGE COUNTY
ENVIRONMENTAL MANAGEMENT AGENCY
PUBLIC WORKS-COUNTY SURVEYOR DIVISION

COASTAL FLOOD PLAIN
LAGUNA NIGUEL / SOUTH LAGUNA

RATTRAY AND ASSOC., INC.
SURVEYING & MAPPING
ESCONDIDO, CA. (619)745-9666 LOS ANGELES, CA. (213)413-4007 SANTA ANA, CA. (714)979-9450

I CERTIFY THAT THIS MAP MEETS THE AGREED-UPON ACCURACY REQUIREMENTS.
I.F. RATTRAY L.S. 3200

PHOTO DATE: 4/10/84
CONTOUR INTERVAL: 2&5 FT.
SCALE: 1" = 100'
SHEET 9 OF 9 SHEETS

LEGEND

1. FP-3 -----

FP-3 LINE - Heavy dashed line on map. Structures must be protected from wave activity in the FP-3 zone, located seaward of the FP-3 line. The FP-3 line would be near the landward limit of wave uprush on an undeveloped beach or bluff during a storm.

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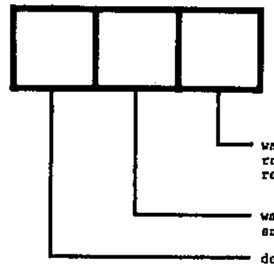

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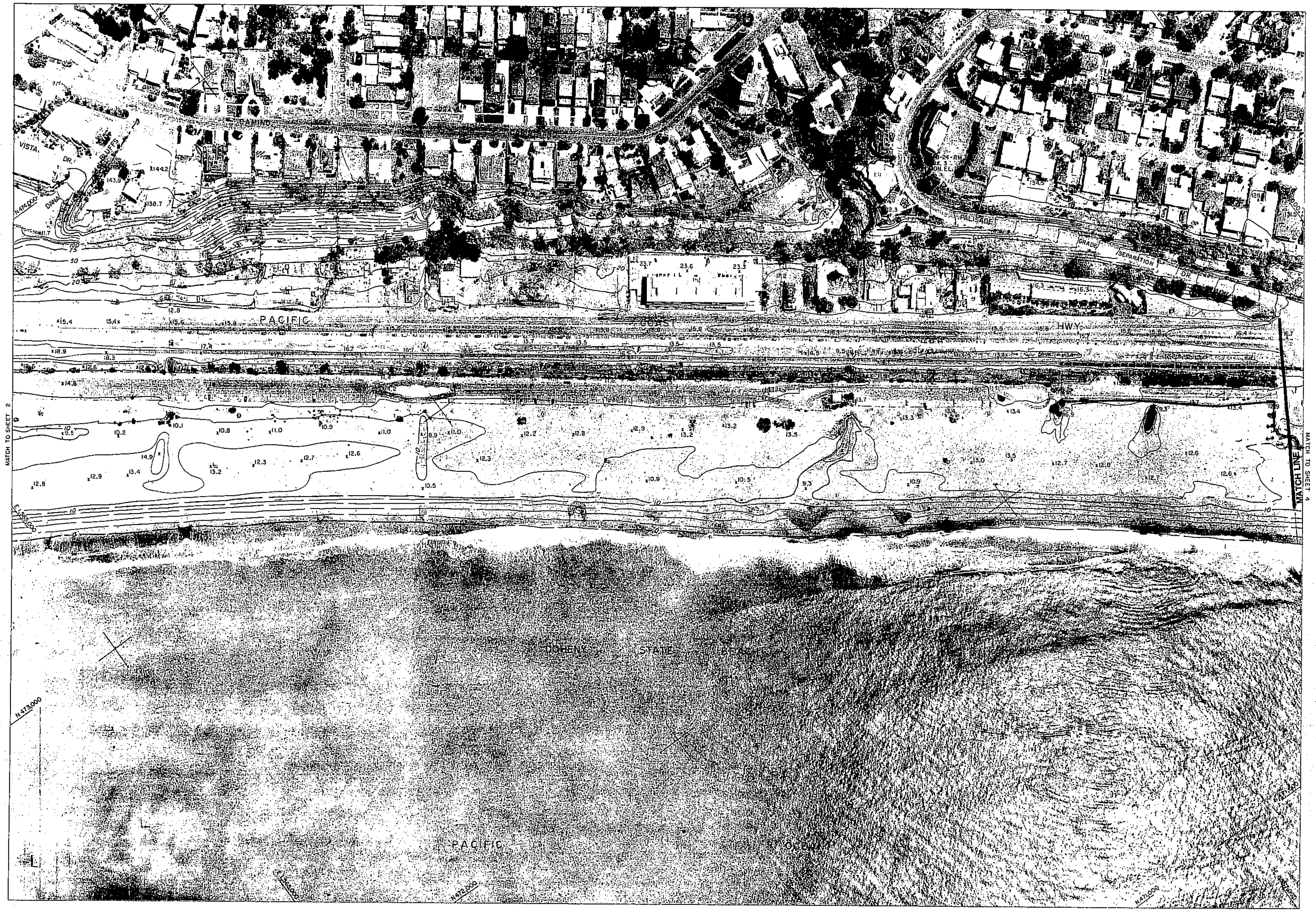

TOPOGRAPHIC MAP SYMBOLS

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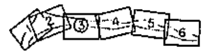
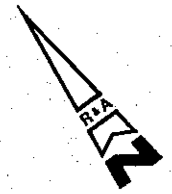


SHEET INDEX



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ORANGE COUNTY
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COASTAL FLOOD PLAIN
CAPISTRANO BEACH

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SURVEYING & MAPPING
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T.F. RATTRAY L.S. 3200

PHOTO DATE: 4/10/84
CONTOUR INTERVAL: 2.5 FT.
SCALE: 1"= 100'

SHEET 3 OF 6 SHEETS

LEGEND

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3. BREAKING WAVE CHARACTERISTICS

wave runup elevation limit on a rough 1 on 1.5 (rise/run) sloped revetment*

wave runup elevation limit on a smooth vertical seawall or bulkhead*

design breaking wave elevation*

*Above OCVD at the FP-3 line considering 1984 conditions

4. DEFINITIONS

BULKHEAD - A protective device designed as a partition to retain or prevent sliding of the land. A secondary purpose is to protect the upland against damage from wave action.

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RECURRENCE INTERVAL - Time period during which one coastal design event can be expected to occur. The 100-year recurrence interval, which must be used for the design of structures and protective devices in Orange County, is the statistical probability that one event that produces a limiting value of a coastal phenomenon will occur in 100 years, or that it has a one percent probability of occurring in a single year. Recurrence interval should not be confused with design life which references an absolute time interval not a probabilistic value.

REVETMENT - A protective device consisting of a facing of stone, concrete, cast units, etc., built to protect a scarp, embankment, or structure against erosion by wave action or currents.

RUNUP - The rush of water up a protective device, beach, bluff face or structure on the breaking of a wave. The amount of runup is the vertical height above stillwater level that the rush of water reaches. The wave runup elevation limit is the highest elevation that will be reached by the rush of water from a breaking wave when that wave occurs during the design wave event with the specified design recurrence interval. The highest elevation subject to wetting by spray from the design wave will be greater than the runup elevation.

SEAWALL - A protective device that separates land and water areas, primarily designed to prevent erosion and other damage due to wave action.

STRUCTURE - A habitable dwelling, cabana, garage, deck, restroom, etc., located in the FP-3 zone. The design life of the foundation of a structure, which must be equal to or greater than 30 years, is the minimum period after construction during which all major components of the foundation system not protected by a protective device retain their functional and structural design capabilities.

MOFFATT & NICHOL, ENGINEERS
LONG BEACH, CALIFORNIA

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COMMUNITY PLANNING
LANDSCAPE ARCHITECTURE
URBAN DESIGN
VAN DELL/SHIOTA PLANNING GROUP
17801 CARTWRIGHT ROAD
IRVINE, CA 92714
714/474-1412

TOPOGRAPHIC MAP SYMBOLS

HORIZONTAL AND VERTICAL CONTROL POINT
HORIZONTAL CONTROL POINT
VERTICAL CONTROL POINT
OBSCURED CONTOURS
DEPRESSION CONTOURS
x 123.5 SPOT ELEVATIONS

SHEET INDEX

NOTE:
CONTOUR ELEVATIONS ARE BASED ON MEAN SEA LEVEL DATUM, ORANGE COUNTY SURVEYOR 1976 ADJUSTMENT.

ORANGE COUNTY
ENVIRONMENTAL MANAGEMENT AGENCY
PUBLIC WORKS-COUNTY SURVEYOR DIVISION

COASTAL FLOOD PLAIN
CAPISTRANO BEACH

RATTRAY AND ASSOC., INC.
SURVEYING & MAPPING
ESCONDIDO, CA. (619)745-9666 — LOS ANGELES, CA. (213)413-4287 — SANTA ANA, CA. (714)979-9450

I CERTIFY THAT THIS MAP MEETS THE AGREED-UPON ACCURACY REQUIREMENTS.
L.S. 3200

PHOTO DATE: 4/12/84
CONTOUR INTERVAL: 2&5 FT.
SCALE: 1" = 100'

SHEET 4 OF 6 SHEETS

LEGEND

1. FP-3 -----

FP-3 LINE - Heavy dashed line on map. Structures must be protected from wave activity in the FP-3 zone, located seaward of the FP-3 line. The FP-3 line would be near the landward limit of wave uprush on an undeveloped beach or bluff during a storm.

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3. BREAKING WAVE CHARACTERISTICS

design breaking wave elevation*	wave runup elevation limit on a smooth vertical seawall or bulkhead*	wave runup elevation limit on a rough 1 on 1.5 (rise/run) sloped revetment*

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TOPOGRAPHIC MAP SYMBOLS

- HORIZONTAL AND VERTICAL CONTROL POINT
- HORIZONTAL CONTROL POINT
- VERTICAL CONTROL POINT
- OBSCURED CONTOURS
- DEPRESSION CONTOURS
- ×123.5 SPOT ELEVATIONS

SHEET INDEX

NOTE:
CONTOUR ELEVATIONS ARE BASED ON MEAN SEA LEVEL DATUM, ORANGE COUNTY SURVEYOR [illegible]

ORANGE COUNTY

COASTAL FLOOD PLAIN
CAPISTRANO BEACH

285

5 6

17

LEGEND

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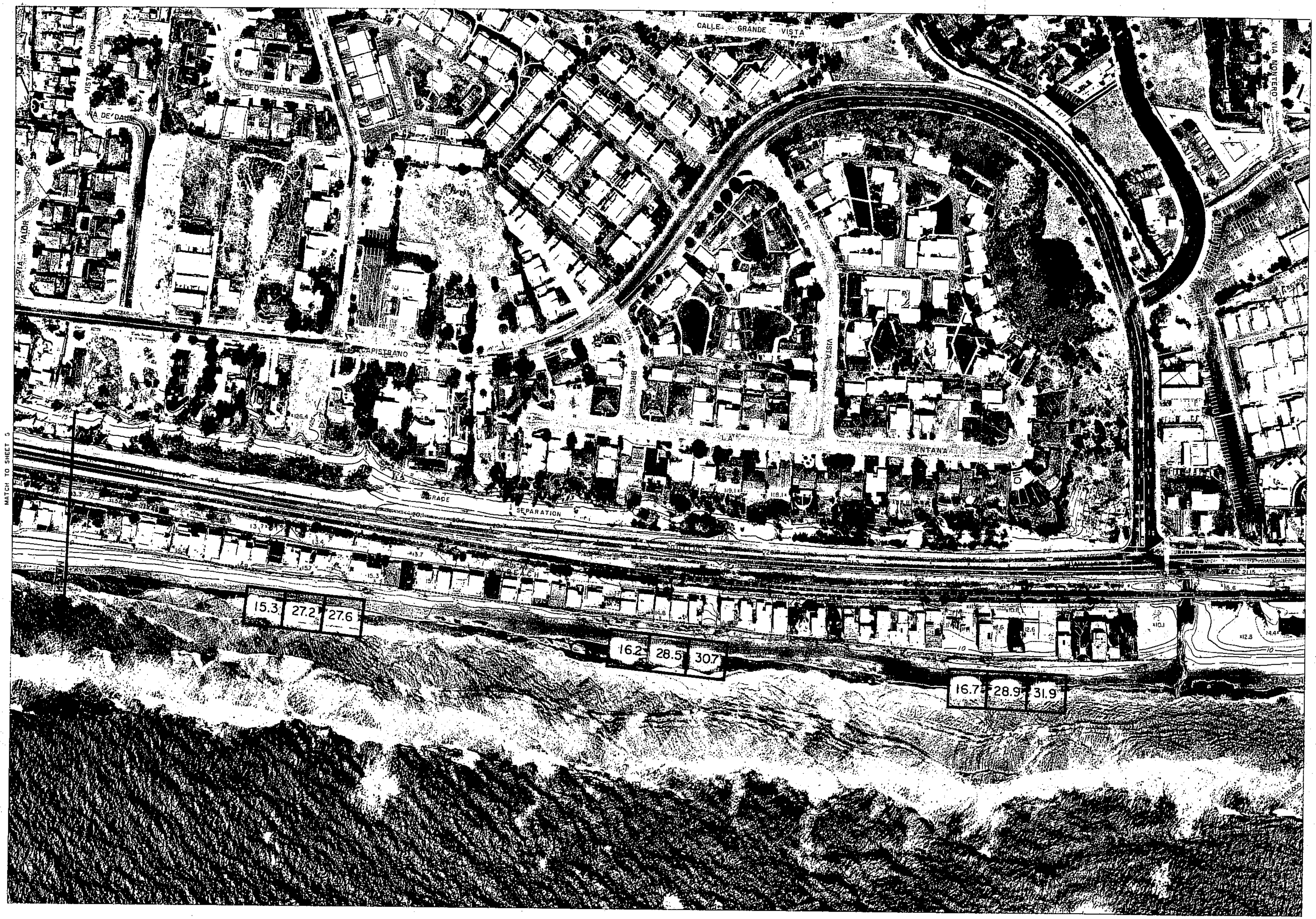

TOPOGRAPHIC MAP SYMBOLS

HORIZONTAL AND VERTICAL CONTROL POINT
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1 2 3 4 5 6

SHEET INDEX

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PHOTO DATE: 4/10/84
CONTOUR INTERVAL: 2&5 FT.
SCALE: 1" = 100'

SHEET 6 OF 6

County of Orange

MEMO

F850-123.2

DATE: May 29, 1985

TO: Distribution List DEPT/DIST:

FROM: Jerry Sterling, Sr. Engineer/Grading Section PHONE NO.: 472-7943

SUBJECT: Final Flood Plain Mapping

Attached are copies of the final Flood Plain Maps (1" = 200') reflect FP-3 Line and the protective device String Line. Please disregard and destroy any copies of the draft mapping in your possession.

Copies of the maps in 1" = 100' scale are available in Central Files should you have need for the larger scale.

Should you have need for additional information do not hesitate to call me at any time.

JDS:jb

Attachment 1" = 200' FP-3 Mapping

Distribution List

Floyd McLellan	Deveopment Services
George Britton	Project Planning
Ken Winter	Project Planning
Ken R. Smith	Transportation/Flood
Vince Rosales	Design
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Jim Williams	EMA/Design
Henry H. Ruley	Building Plan Check
Bill Zaun	Director/Regulations
Donald Cotner	Building Plan Check
Ken E. Smith	Flood Design
✓ Doug Crouse	Subdivision Division
Joe Natsuhara	Public Works Design C