FLSGP-T-79-010 C. 2

CIRCULATING COPY
Sea Grant Depository

LOAN COPY ONLY

FLORIDA SEA GRANT COLLEGE

SEAWALL DESIGN ON THE OPEN COAST

by Todd L. Walton, Jr. & William Sensabaugh

NATIONAL SEA GRANT DEPOSITORY
NARRAGANSETT, RI 02882

Report Number 29

June 1979

Marine Research Education Advisory Services
A Sea Grant Program for Florida

DISCLAIMER

The suggested design procedure of seawalls as described in this publication shall be at the sole risk and responsibility of the user with no liability of any nature whatsoever on the part of the authors, the Marine Advisory Program-Sea Grant, or the Florida State University System. It is suggested that for any design a registered professional engineer with experience be consulted.

SEAWALL DESIGN ON THE OPEN COAST

Todd L. Walton, Jr.
and
William Sensabaugh

Todd L. Walton, Jr. is the Coastal Engineering Advisory Specialist, Marine Advisory Program, Coastal Engineering Laboratory, University of Florida, Gainesville, Florida. William Sensabaugh is an engineer at Tetra Tech, Inc., Jacksonville, Florida (formerly with the Dept. of Natural Resources, State of Florida).

Sea Grant Report No. 29
State University System of Florida
Sea Grant College Program

June 1979

TABLE OF CONTENTS

LIST OF FIGURES

INTRODUCTION

Seawall design on the open coast is an often overlooked problem in the State of Florida as well as other areas of the South Atlantic and Gulf Coasts. Escoffier [1] mentions numerous seawall failures and improper design considerations along the Mississippi Gulf Coast which were exposed by hurricanes after the seawalls were built.

In Florida the problem is even more apparent due to its rapidly expanding coastal population. Present seawall design inadequacies were made apparent recently after Hurricane Eloise (September 1975), when repermitting of over 8,000 feet of damaged or destroyed seawalls had to be handled by the Bureau of Beaches and Shores, Florida Department of Natural Resources. Presently, seawalls designed for the open coast are running $150 - $300 per lineal foot. In some areas, the cost of seawall replacement may be higher than the replacement cost of the upland construction it protects.

The basic function of all seawalls is to support, stabilize, and protect upland property and construction against wave action and erosion. The importance of any specific design used must take into consideration the consequences of seawall failure, as well as the initial cost of seawalls. Unfortunately, in past practice, seawall design has been governed by the amount of money the upland owner was willing to pay, with little thought as to whether the structure would endure the critical environment in which it was placed. The attitude contributed to the numerous costly failures as noted after "Eloise".

Presently, the Bureau of Beaches and Shores, Florida Department of Natural Resources requires a permit to build seaward of the coastal construction setback line established in most coastal counties of Florida [2]. In the case of seawalls, permits will be granted conditional to two (2) factors: (1) substantiated need for a seawall; and (2) seawall plans that have been designed by a professional engineer.

The first factor depends to a great extent on location and adjacent construction practices in the area. Due to adverse effects of seawalls in the littoral zone (which will be reviewed in this bulletin), seawall applications for areas of coast where no seawalls presently exist will most probably be turned down. However, if adjacent property owners have seawalls, even though the presently existing seawalls may be ineffective and detrimental to the beach, it is recognized that protection of the upland property of the unseawalled portion of the beach may outweigh the detrimental effects of the additional amount of seawall to the total beach system. In these cases a seawall may be permitted, provided the seawall has been designed properly.

This bulletin is meant to provide both insight into effects which seawalls have on the beach system, and insight into proper design considerations for seawall construction on the open coast. Common types of seawall failures, construction mistakes, and design weak points which consistently show up in Florida seawalls have been considered in an earlier publication, Reference 3.

The present bulletin is not meant to be a design manual, although adequate references have been given which cover aspects of design for shore protective structures in great detail. As most seawalls on the open coast are of the anchored bulkhead type, the main emphasis will be toward anchored seawalls of the type shown in Figure 1.

It is hoped that this bulletin, along with Reference 3, will provide a good starting place for those involved in technical design and construction supervision aspects of seawalls.

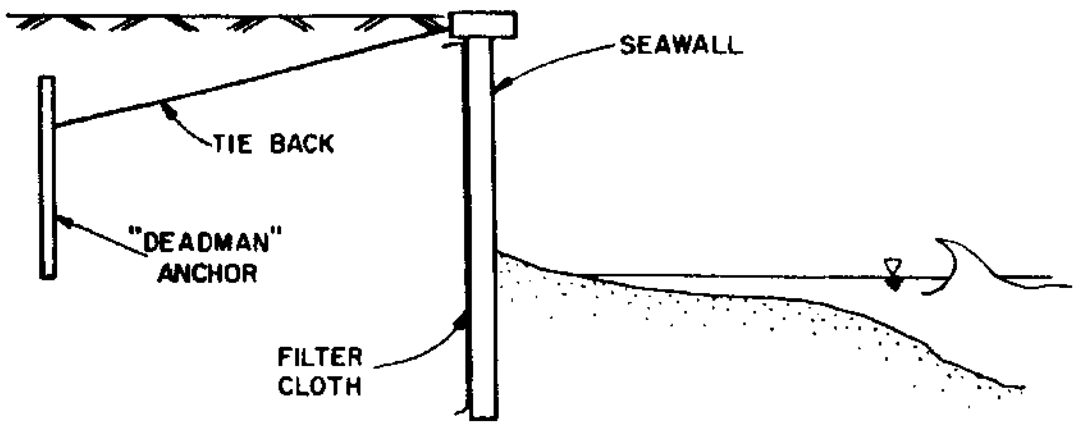

Figure 1. Anchored Seawall (or "Bulkhead")

EFFECTS OF SEAWALLS ON THE BEACH SYSTEM

Seawalls have a detrimental effect on the natural beach profile through a number of related effects, all of which produce scour at the front of the seawall. Scour is defined here as lowering of the beach profile due to the effects of the seawall only.

During storms, seawalls prevent landward erosion of the natural equilibrium beach profile. Typically, the natural profile of the sections of beach without seawalls will erode during storm conditions and establish a submarine bar offshore, which, in turn, provides protection to the beach by "tripping" the higher waves and preventing them from reaching the beach. Any bar built seaward of the seawall, though, must gain its sand through either a corresponding lowering of the portion of profile abutting the seawall or else from the adjacent unseawalled beach. Most likely, a combination of both of these events occurs during any storm.

Scour at seawalls may also be attributed to the fact that waves are expending their energy on a much shorter section of beach when a seawall is present. Water levels in the sand bed in front of the seawall will correspondingly be much higher due to wave reflection and to the fact that water cannot percolate back through the beach as its path is diverted seaward by the seawall. This effect in turn causes higher pore pressures within the sand which "fluidizes" to an extent the sand in front of the seawall and hence erosion takes place.

One other effect of seawall encroachment is the fact that seawalls confine any existing longshore currents across the surf zone. This constricted surf zone causes higher velocities in front of seawalls with corresponding scour of the seabed.

The fact that the natural beach profile is not allowed to erode and build its "defensive" outer bar in a normal manner coupled with higher longshore currents experienced in front of the seawall, cause the whole profile in front of the seawall and adjacent to it to be lower than the storm beach profile without the seawall (See Figure 2). As a consequence, higher waves can reach the seawall and the areas adjacent to it, and thus cause greater beach recessions near the seawall than experienced out of the zone of seawall influence (See Figure 3). Figure 4 shows a seawall at Panama City which survived Hurricane Eloise, but caused detrimental effects on the adjacent property due to the effect discussed above as well as the turbulence and reflected waves caused by the return walls (i.e. walls perpendicular to the main seawall at its ends).

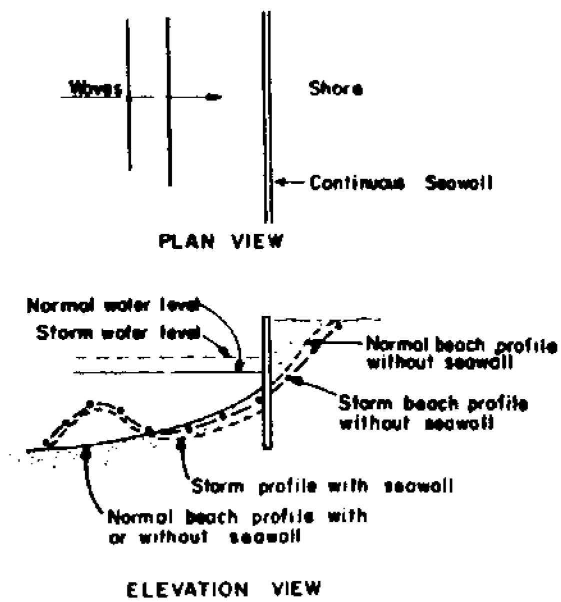

Figure 2. Qualitative Effects of Continuous Seawall on Storm Beach Profile

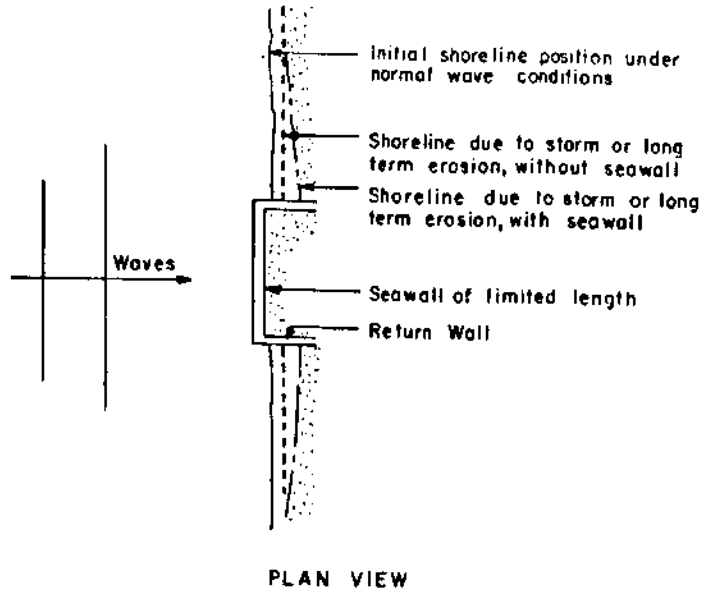

Figure 3. Effect of Seawall of Limited Length on Storm or Long-Term Beach Plan Form

Figure 4. Effects on an Adjacent Beach Caused by a Vertical Faced Seawall (Courtesy of Professor Byron Spangler)

SITE DESIGN CONSIDERATIONS

CALCULATION OF BEACH PROFILE CHANGES DUE TO LONG TERM EFFECTS

As most shore lines in Florida and elsewhere along the open coast are eroding, it is important to consider the long term effects of beach profile modification over the intended life of a seawall. For long term effects, it is reasonable to assume that the structure would have no effect on reducing the erosion of the beach seaward of the seawall. This aspect of design is discussed adequately in Reference 4, and consists of assuming that the foreshore and above water portions of the equilibrium beach profile remain constant in shape over a long period of time and shift landward at a rate equal to the long term annual shoreline recession rate. Figure 5-2 of Reference 4 presents aspects of this design consideration which will not be discussed further here.

CALCULATION OF BEACH PROFILE CHANGE DUE TO STORM EFFECTS

Of most importance to the design of any seawall located on the open coast is the beach profile change brought about by a large storm or hurricane. Although beach profile changes due to storm wave and tide effects on a natural beach are not the same as those occurringon a seawalled beach, it is still a good starting point in design to consider natural beach profile lowering due to storms.

Due to the large beach profile changes associated with extratropical storms and hurricanes, it is important to consider the effects of such a storm in the design of a seawall. Typically, most seawall failures occur during the heavy surf and high tide conditions accompanying storms.

Beach profile changes due to storms is an important aspect of research which has not been looked into thoroughly to date. Various references [5, 6] treat this subject somewhat incompletely, due to a lack of good data from which to make engineering predictions of beach profile changes after storms. The present "crude" approximation for beach profile changes during storms is based on some aspects of both References [5] and [6] and additionally on an extensive set of beach profile data taken along the Panhandle Coast of Florida both before and after Hurricane "Eloise" in September 1975.

The natural beach profile and the "after storm" approximated profile are shown in Figure 5. To obtain the after storm profile, one needs information on the before storm profile and the design storm tide level for the area. Storm surge levels and their associated probabilities can be obtained from either the Flood Insurance Administration (which maintains offices in various locations throughout the United States), or from a county building and zoning department, assuming flood levels have been determined via the Flood Insurance Program for the county.

The "after storm" beach profile is found as per Figure 5 by: (1) establishing the point on the pre-storm beach profile with a water depth equal to $0.5\ \eta_S$ (η_S = design storm suge level); (2) connecting the point so determined with a point on mean sea level (MSL) seaward of the pre-storm shoreline by an amount equal to $4\eta_S$; (3) finding a point on a horizontal line η_S feet above the MSL such that when a line is drawn between it and the new profile point on the MSL, the area eroded between pre and post storm profiles will equal the area accreted.

The "after storm" profile given by this method should provide a "rough" approximation for use in seawall design in areas where dunes exist and the design storm tide does not overtop the dunes, and where considerable building

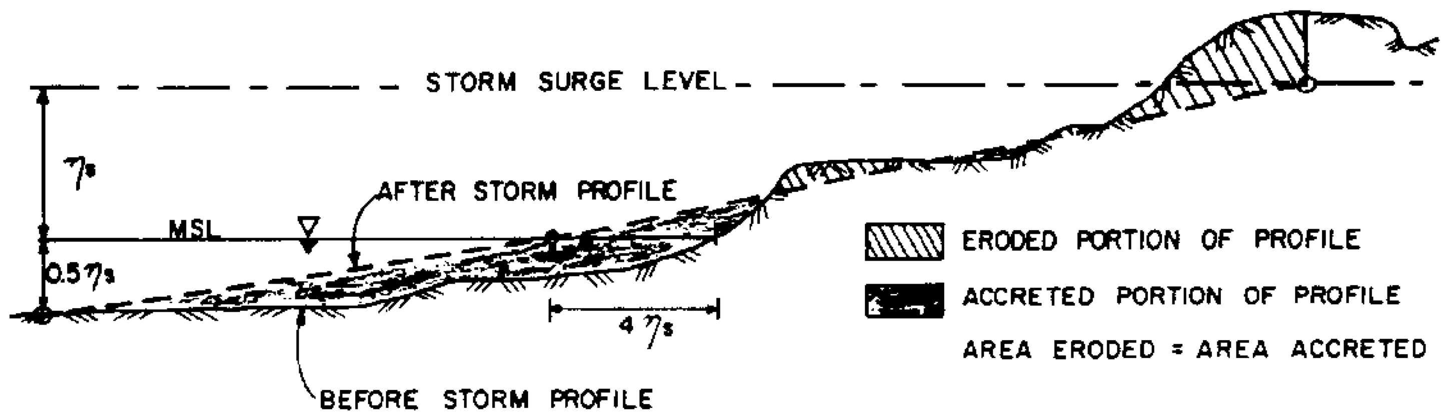

Figure 5. Beach Profile Change Due to Storm

encroachment has not taken place. This method was applied for a number of cases in the Florida Panhandle area and gave reasonable results although in some areas an alternative design method may have to be found.

Any lowering of the beach profile due to the storm effect at the seaward side of the "proposed" seawall location should be considered for design purposes when adjacent areas are relatively unencumbered by seawalls. Although many areas of the after storm beach profile may have a gain of sand, it is recommended for seawall design purposes that no portion of the beach be considered higher after a storm than prior to the storm.

Once this "after storm" profile is established, it becomes necessary to consider the additional effects that placement of a vertical wall will have on the beach profile. These effects will be discussed under the section on scour.

As many areas have been developed heavily along the coast and no longer have dune structures, the above method cannot be used in many instances and an assumption must be made as to the after storm effects based on experience.

SCOUR

Presently, there are few field measurements of scour experienced at seawalls after storms. Typically, by the time post storm damage surveys are completed, the effects of scour at the seawall during storms may well be masked by after storm beach accretion effects.

Presently, Reference [4] recommends that:

> "the maximum depth of a scour trough below the natural bed is 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 structure."

This criteria appears to be based on limited field observations of scour at seawalls made prior to 1950. More recently, many laboratory studies have been made of scour [7, 8, 9, 10, 11, 12, 13], but are somewhat open to wide interpretation as to their validity on the open coast, since the vast majority of laboratory tests made do not reasonably account for modeling scale law effects of sediment size. Only one set of tests [9] has been done in a two dimensional modeling basin of appreciable size comparable to prototype conditions, which enables a reasonable comparison of what might happen under breaking wave conditions at a long seawall. The results of this set of tests [9] gives scour at the toe of the wall ranging from $S/H_0 = 0.25 - 2.5$ (S = scour depth, H_0 = deep water wave height). The scour was found to be dependent on the location of the wall in the surf zone in relation to wave breaking and on the wave steepness. Sediment size and density is also an important parameter although only one size of material was used in these large wave tank model tests.

It should be realized that scour conditions for the two dimensional case in the laboratory can only be compared to the real case of an infinitely long seawall. The three dimensional effect in nature will modify the scour pattern, depending on both the length of the wall and location of the wall in relation to the surf width zone. As noted earlier, an encroachment on the surf zone by a seawall will confine the longshore current in the surf zone such that this current may be expected to increase in speed along with a consequent scouring of the bottom.

A theoretical model [14] has been developed for a quasi-three dimensional ase (longshore current effects are included) of a seawall in the surf zone which predicts scour due to the increasing longshore current effect.

From this model an estimate of the scour at the seawall can be made. Based on a number of limiting assumptions as noted in the theoretical model [14], an equation for scour can be derived of the form:

$$\frac{S}{H_b} = 1.60\ (1 - X_s)^{2/5}$$

where S = scour depth at toe of seawall

H_b = breaking wave height

X_s = dimensionless location of seawall in relation to the surf zone (i.e. for the seawall located at the shoreline $X_s = 1.0$)

The above formula has been derived on the basis of perfect reflection at the wall. In the above theoretical three dimensional model as well as the two dimensional laboratory experiments, it can be found that reducing the reflection coefficient of the wall (by either sloping the wall or other means such as placing a rock revetment in front of the wall) will reduce the scour trough.

Again, in the above theoretical case, only an infinitely long seawall has been considered. Shorter walls would be expected to have consequently less scour due to an available supply of sand furnished to the front of the seawall by adjacent properties. This fact may in part explain why most observations of seawall scour in nature are far less than those which might be expected from models or theories. One additional limitation to the model presented in [14] is that zero scour would be experienced if the seawall is located directly at the shoreline when in fact a large scour is experienced here under similar conditions in wave tank testing.

For a seawall having an unprotected toe (i.e. no rock blanket and filter toe protection) it appears reasonable (in light of field and laboratory data) to consider a maximum scour depth equal to the "rule of thumb" expressed earlier and given in Reference [4] after applying considerations for beach profile changes due to long term erosion and short term storm effects. It should be noted that "apparent" seawall scour observed at Panama City after Hurricane "Eloise" hit the coast was considerably less than the maximum predicted by the rule of thumb. Most seawalls with cap elevations less than 10' above grade experienced a maximum of 2-3 feet scour.

Reference [3] discusses the use of toe protection to prevent scour from occurring.

WAVE FORCES

Typical seawall failures in Florida caused by wave forces occur due to a loss of backfill material. When the seawall can no longer transmit the extremely large hydrodynamic forces to the backfill material it fails in shear or moment. Cases of seawall failure in shear or inward bending moment where backfill has been emplaced properly and retained are almost nonexistent. Which implies that good backfill is capable of readily transmitting the high hydrodynamic forces to the soil behind the seawall. Thus, if cohesionless clean sand is used as a backfill material and the fill is graded to the top of the seawall, and precautions are taken to insure that the fill will not be lost (i.e. filter material to retain backfill behind the wall and adequate return wall length), it appears reasonable not to consider wave loadings on the structure but only static water loadings.

Should additional precautions not be taken to insure backfill retention (or if the backfill material is not of a clean cohesionless sand), wave forces must be considered. A reasonable approach to seawall design for hydrodynamic forces is to consider the most severe case of waves breaking on a structure or broken waves on the structure, depending on the height of the wall, the design storm surge level, and the location of the seawall.

Wave forces for design can be based on the methods proposed in Reference [4], section 7.33 and 7.34 on breaking wave and broken wave forces on vertical walls.

SPECIAL DESIGN CONSIDERATIONS

RETURN WALL DESIGN LENGTH

An often occurring mode of failure in seawalls is that the backfill material behind the seawall is lost either through the joints of the seawall face or via flanking of the seawall by wave action with consequent water (and backfill material) drainage around the ends of the seawall. As many seawalls are not designed to take the high wave forces due to broken waves, they depend for their structural integrity on transmitting the dynamic wave forces through the structure into the backfill. When this backfill is lost, the seawall can fail in shear or moment, due to the wave forces. Prevention of backfill loss by use of return walls (often referred to as "wing" walls) long enough so as not to be flanked in a storm is a major concern often not addressed. An example of a return wall which was not long enough to prevent flanking during Hurricane "Eloise" is shown in Figure 6.

The design return wall length must take into consideration recession experienced by the natural beach profile during a storm, as well as an additional length due to the higher beach recession rates experienced near the seawall, as noted earlier in the discussion of scour and shown in Figure 3. No existing design criteria suggest length of return wall to prevent flanking, although measurements of flanking of seawalls in the Panhandle area of Florida (after Hurricane "Eloise") are presented in Figure 7.

Seawall return walls should extend into the dune line far enough to prevent flanking when erosion of the dune occurs. This would entail carrying the return walls to the after storm beach profile contour elevation equal to the seawall cap elevation. Use of the method discussed earlier for approximating after storm beach response can be used for this evaluation. If, for example, a seawall has a crest elevation of +10 feet MSL (mean sea level), the return walls should extend into the dunes a distance, x, shoreward from the 10 foot contour line of the dune; where x = recession of the 10 foot contour line during the design storm and can be estimated by the method discussed earlier on beach profile change due to storm effects.

As seawalls cause anomalous detrimental effects on adjacent unseawalled areas (see Figures 2, 3), it is important to carry the return walls an additional distance, x', beyond the storm recession distance, x, to account for these effects. This distance x' appears to be strongly dependent on the height of the seawall. Using an extensive set of beach elevation data taken along Panama City Beach after Hurricane "Eloise" in September 1975, the distance x' versus length of seawall was plotted for various vertical faced structures where a clear picture of this effect could be seen. This data is presented in Figure 7. From this figure, a "rough" estimate can be made for the distance x'. As an example, consider that a seawall of 300 feet in length is to be built to survive storm conditions similar to "Eloise". From Figure 7 for a seawall length of 300 feet, the distance x' can be conservatively estimated as 45 feet. To this must be added the expected contour recession distance, x, for the storm along with any additional distance necessary to tie in to the existing beach profile at cap elevation.

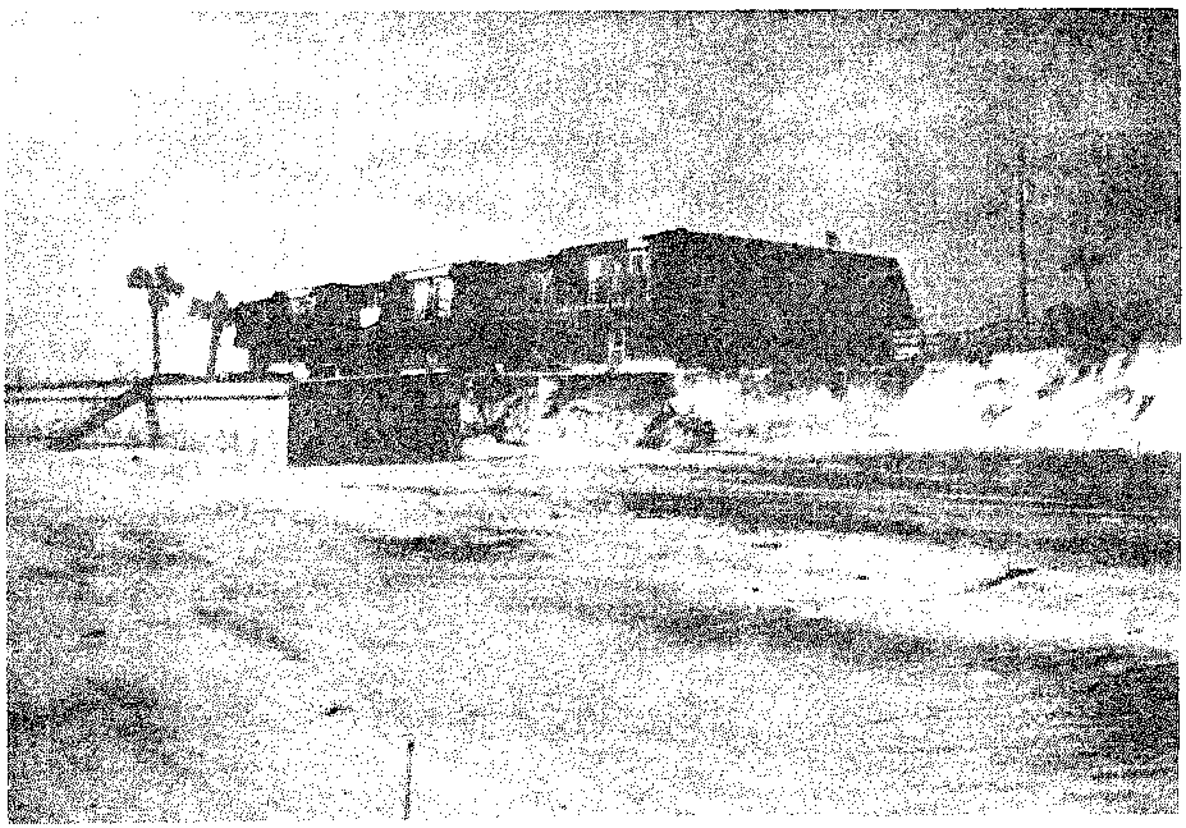

Figure 6. Inadequate Return Wall Length (Courtesy of Professor Byron Spangler)

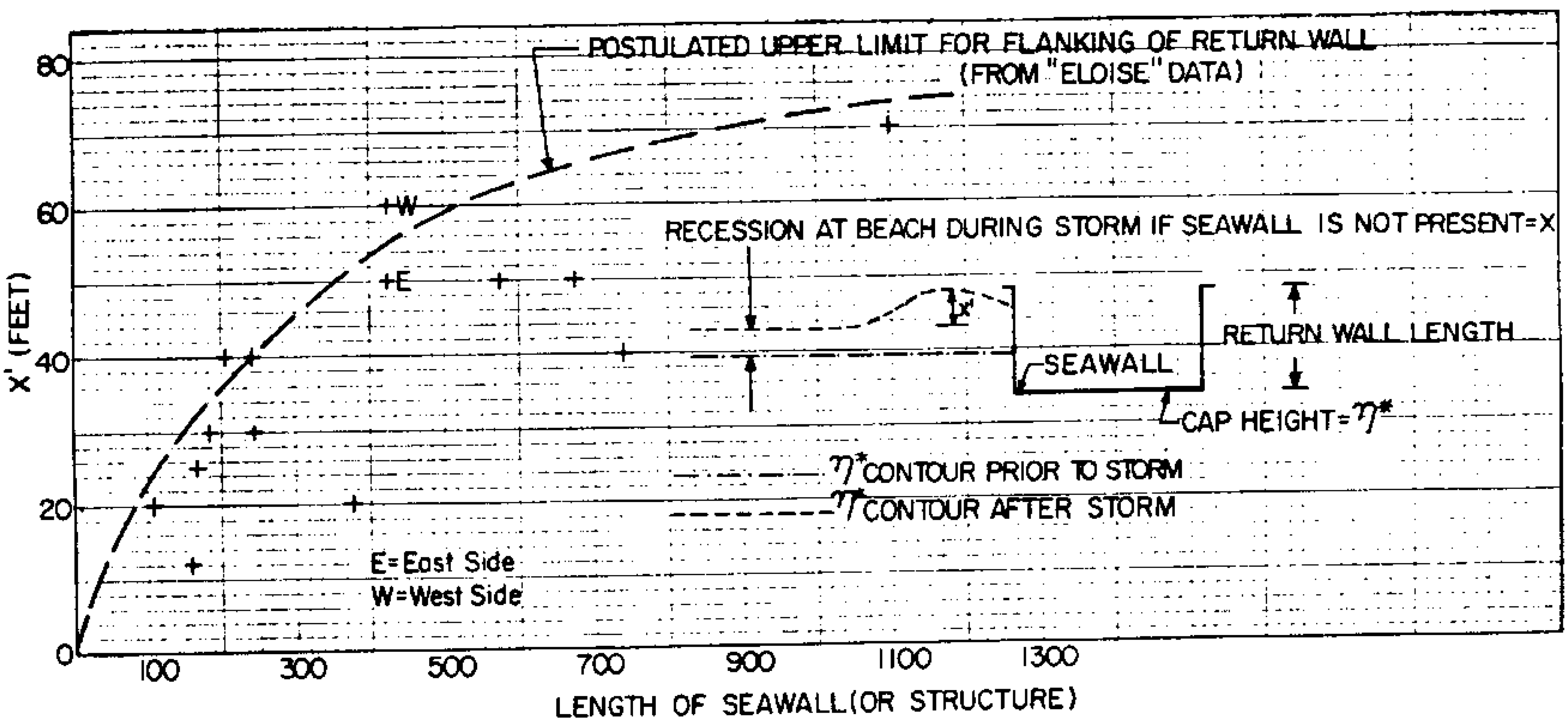

Figure 7. Graph For Use in Design of Return Wall Length

Total return wall design length is shown in Figure 8 for a hypothetical design situation.

In the event that excessive return wall length must be provided for, it may be cheaper to design the seawall on the assumption that the backfill may be lost and to consider wave forces on the front of the wall as a possible failure mechanism.

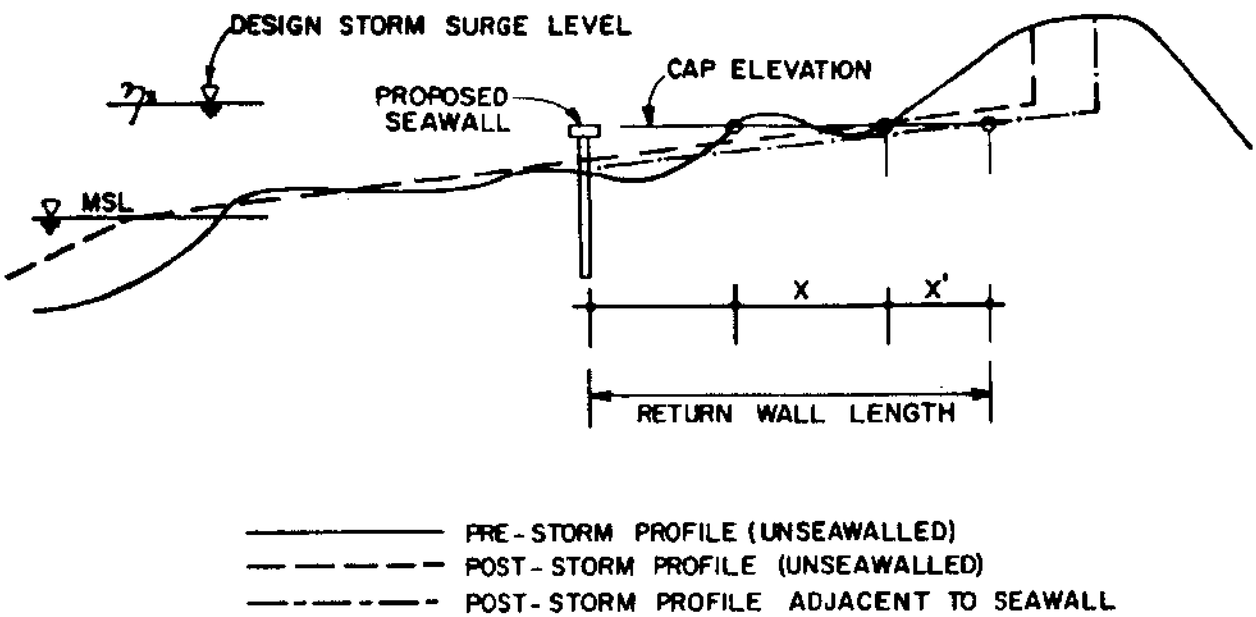

Figure 8. Return Wall Design Length

SUMMARY REVIEW OF DESIGN METHODS

The following discussion has been prepared to assist in the evaluation of the more typical seawalls used along Florida's coastline. Although the discussion addresses the more academic design considerations, it should be re-emphasized that many seawall failures occur because of flanking by wave action, damaged or missing tiebacks, poor construction or inadequate means of relieving excess overtopping rather than design calculations or methods being inaccurate. A good seawall design is meaningless unless the wall is constructed in accordance with the design plans.

Typical seawalls on Florida's beaches are of relatively low flexibility (reinforced concrete), and are constructed in relatively homogenous clean sand. Furthermore, typical walls are jetted into place and backfilled with clean sand. This does not imply that all seawalls built on Florida's coast are of this type or that these assumptions may automatically be assured for all walls. However, the above limitations apply to the vast majority of seawalls designed on the open coast of Florida and therefore these assumptions will be used in a later design example.

Anchored Bulkhead Design. A number of different analytical approaches have been proposed for investigating stability and moments in anchored retaining walls. These approaches include:

1. Danish Rules
2. Fixed Earth Support
   a. Equivalent Beam
   b. Graphical
3. Free Earth Support
4. Braced Cut

The Danish Rules involve empirically based procedures [15, 16] developed in the early 1900's by the Danish Society of Engineers. Although many existing bulkheads have been successfully constructed based on these rules, research has shown significant inconsistencies in the assumed lateral pressure distribution. In view of recent test results, the use of the Danish Rules appears unjustified.

The Fixed Earth Support, Equivalent Beam method, is based on the assumption that the bulkhead deflections are such that the elastic line of the bulkhead will reverse its curvature at a point of contraflexure [16, 17]. This allows consideration of a bulkhead as two beams, pinned at the point of contraflexure. Since this procedure tacitly assumes no deflection of the toe of the sheet pile, the depth of the imbedment required for conventional seawalls is generally greater than that computed on the free earth assumption. It should be noted, however, that this procedure assumes a rather simplistic soil pressure distribution and predicts lower bending moments than those based on a free earth assumption.

The Fixed Earth Support, Graphical Method [16], is also based on the assumption that there is no deflection about the toe of the bulkhead. This method, however, involves an extremely time-consuming graphical trial and error method of determining the required imbedment.

The Free-Earth Support method of analysis is perhaps the oldest and most conservative design procedure [16, 17, 18]. It assumes a Coulomb type soil pressure distribution with no deflection at the anchor point. Analysis by this procedure yields a minimum depth of imbedment required to stabilize the wall against toe-out failure and the maximum bending moment is computed on the basis of this imbedment. It should be noted that this procedure does not assume that any inherent advantage is gained by imbedding a bulkhead deeper than required for stability against toe-out failure. It also assumes that slight yield of the wall will take place at the anchor point such that backfill loading on the wall will assume an active pressure distribution.

Model tests by Rowe [18] established a relationship between the flexibility of a bulkhead and the bending moment within the bulkhead. Moment reduction curves are available showing the reduction to the bending moment, as calculated by the fixed earth support procedure, based on a flexibility parameter. An inspection of the curves however, shows that, for reinforced concrete, the moment reduction is generally negligible unless the bulkhead is extremely long or thin.

An anchored bulkhead with a rigid support, such as a very short length of anchor rod tied to an unyielding support, should be treated by the "Braced Cut" Method [17], which gives a much greater total pressure than the Coulomb value and deviates from the above methods in that the soil pressure behind the wall does not have the typical "hydrostatic" type distribution of active pressure.

The method to be used must, in the final analysis, be based on the design engineers' judgement and evaluation of site conditions. However, for the typical Florida seawalls discussed above, it appears that the Free Earth Support approach is the most reasonable open coast design approach for reinforced concrete seawalls having low flexibility. In most cases of reinforced concrete design, the flexibility of the design seawall will be so low that moment reduction cannot be allowed.

The first step in seawall design after determining the site and design conditions of after storm profile and scour, is to calculate the minimum depth of imbedment of the sheet pile. Figure 9 provides a design chart for imbedment depth determination for critical conditions most likely to occur on the open coast (i.e. saturated fill behind wall, receeding water level in front of wall, water head on wall), which would occur shortly after a storm subsides and the storm surge receeds. Note that surcharge loads have an important bearing on the stability of seawalls. Consequently, all structures and loads that may surcharge the soil upland of the wall should be considered in the wall design and additonal structures should not be constructed unless it is determined that the wall can withstand the additional surcharge load. No surcharge effects are accounted for in the present chart.

When a differential head is maintained across a seawall, percolation under the wall results. The resulting seepage pressures tend to reduce the effective unit weight of passive soils and increase the effective unit weight of active soils. If the differential head is small (order of 1 foot) the effect of the unit weights is generally negligible. If the differential head is large, however, the effects of seepage can be significant. These effects are considered in the chart given.

ANCHORAGE DESIGN

The most common types of anchorage used in Florida seawalls are variations of the "deadman" anchor or sheet pile anchor types. Any of the anchorage types shown in Figure 10 are suitable, if properly designed. Where insufficient room exists behind the seawall design location for construction work and/or placement of typical anchorage systems, one of the methods of anchoring shown in type (d) or (e) may have to be considered.

In any design it is most desirable to locate the anchor out of the zone of potential active failure. For the anchorage to develop full passive soil resistance, (i.e. maximum holding power), it must be even further back as shown in Figure 11. Where anchorage must be closer to the seawall that the zone dictated by Figure 11, a reduction in allowable resistance of the anchor system must be made as discussed in References [15], [18].

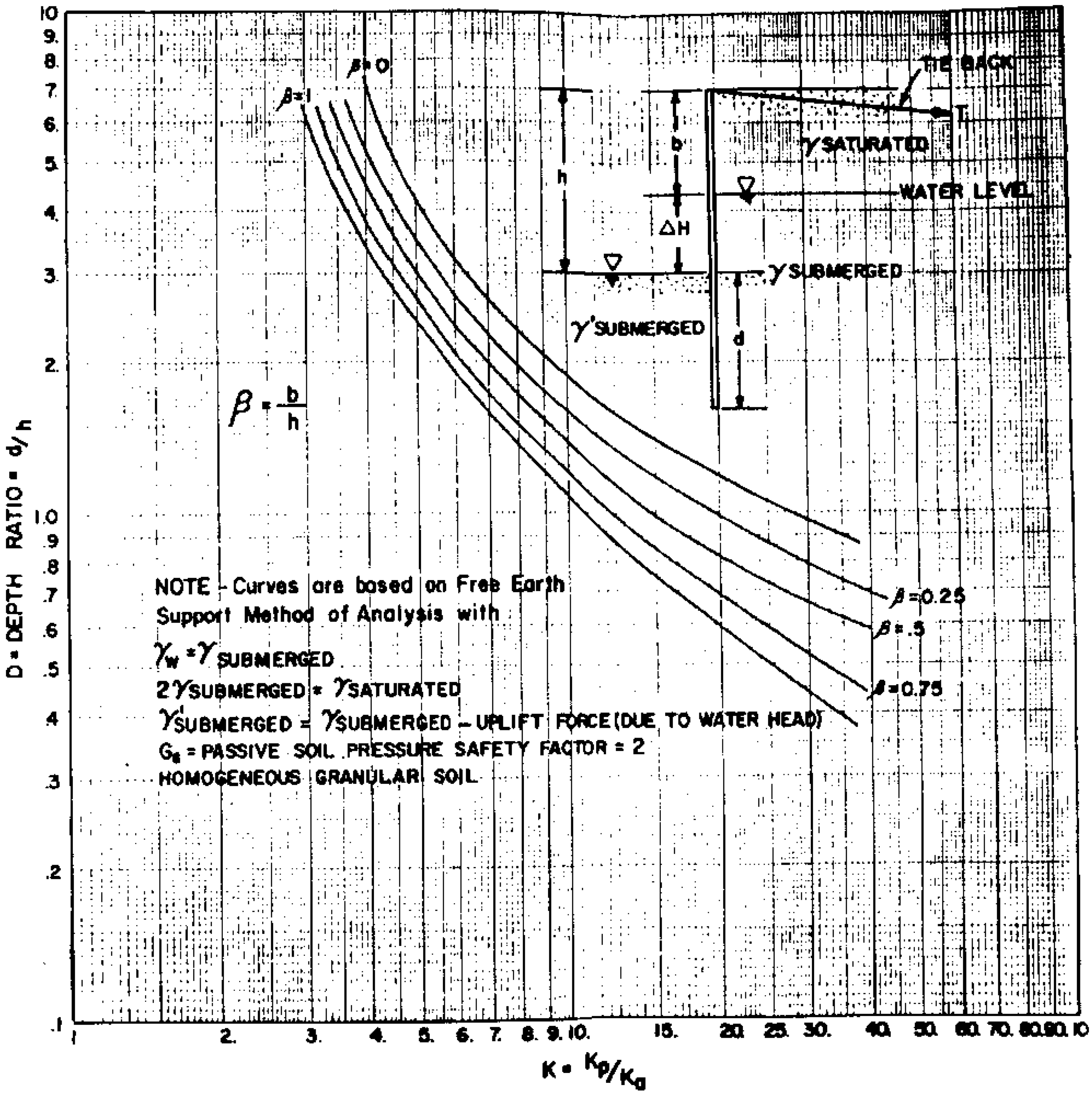

Figure 9. Minimum Depth of Sheet Pile Wall to Prevent Failure Under Critical Conditions

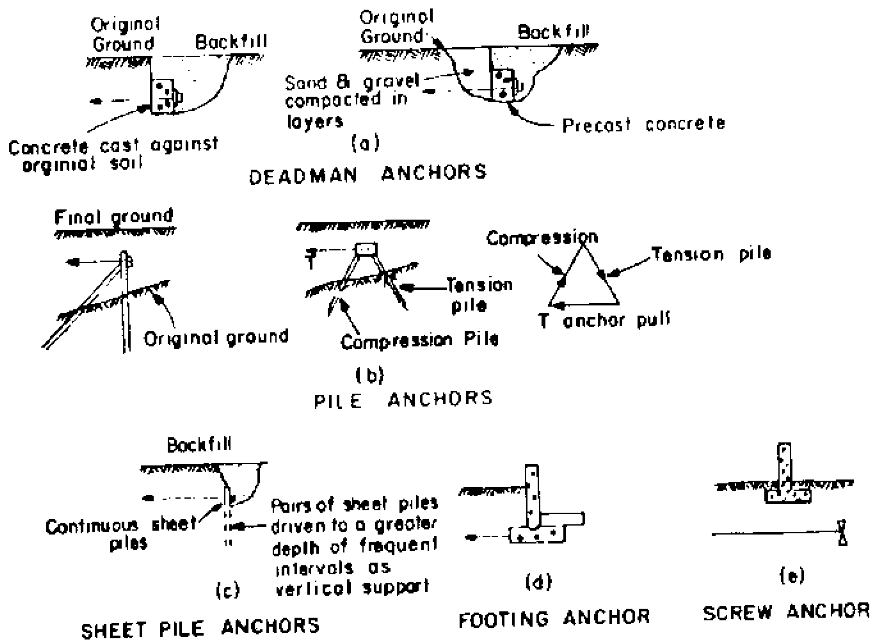

Figure 10. Types of Anchoring Systems

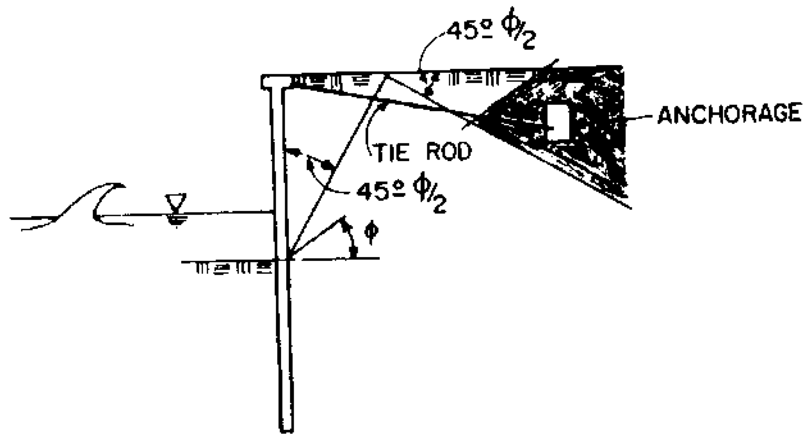

Figure 11. Location of Anchorage for Full Passive Resistance

A number of adequate design methods exist for anchorage design (15, 16, 17, 18, 19, 20), and should be compiled with in design of the anchorage system. Each method depends on the type of anchorage used and the soil conditions for which it can be properly applied. Care must be exercised to see that the anchorage system does not settle after construction. Additionally, the soil within the passive wedge of the anchorage should be compacted to at least 90 percent of maximum density unless the deadman is forced against firm natural soil. Reference [1] states that if tiebacks are used they should be designed as both tension and compression members.

EXAMPLE DESIGN FOR SEAWALL

Seawall is to be located as shown in Figure 12, and have a cap elevation of +10 MSL and be 200 feet in length.

(1) Consider effects of long term "historical" shoreline changes: In the present case it will be assumed that the shoreline has been stable for the past 50 years and therefore expected long term equilibrium profile changes are negligible. Where these changes are not negligible, long term equilibrium profile changes as discussed earlier should be considered.

(2) Consider effects of design storm level for short term profile modification:

Consider for the present case a design storm surge of 11 feet MSL. Figure 12 shows the before and after storm profiles for the seawall site. The "after storm" profile has been constructed as discussed previously. At the proposed seawall location a profile lowering of 1.7 feet should be accounted for in design. Also note in Figure 12, a recession of the 10 foot (cap elevation) contour a distance of 34 feet.

(3) Consider additional effects of scour due to vertical faced seawall depth of water at seawall face = 3' = d_s
slope of beach - 1:15; from Reference [4] for an assumed storm wave period of 8 seconds $\frac{H_b}{d_s} = 1.6$; therefore breaking wave H_b at seawall face = $1.6\ d_s = 1.6(3') = 4.8'$. An additional scour of 3 feet will be considered (note this assumption is unconservative according to Reference 4). The reduction in scour from the Reference 4 approach is considered a benefit of having a good dune system in the area. The use of a rock and filter toe scour protection properly designed in accordance with Reference [3] will help to prevent scour from occuring. In the present case no seawall toe protection will be considered and the scour will be taken into consideration in design.

(4) Design length of return walls

length necessary to tie in to 10' contour	= 10'
additional length due to storm effects, X	= 34'
additional length due to effects of seawall, X'	= 35'
total length of design return walls	79'

If the return walls can be tied into an upland structure to prevent loss of backfill this length may not be necessary. The primary purpose of the return walls is to hold the backfill to prevent the primary wall from failure due to storm wave action.

(5) Design calculations (See Figure 13)
(Footnotes to go with calculations are numbered along with an asterick and are referred to in the notes for Design Calculations section).

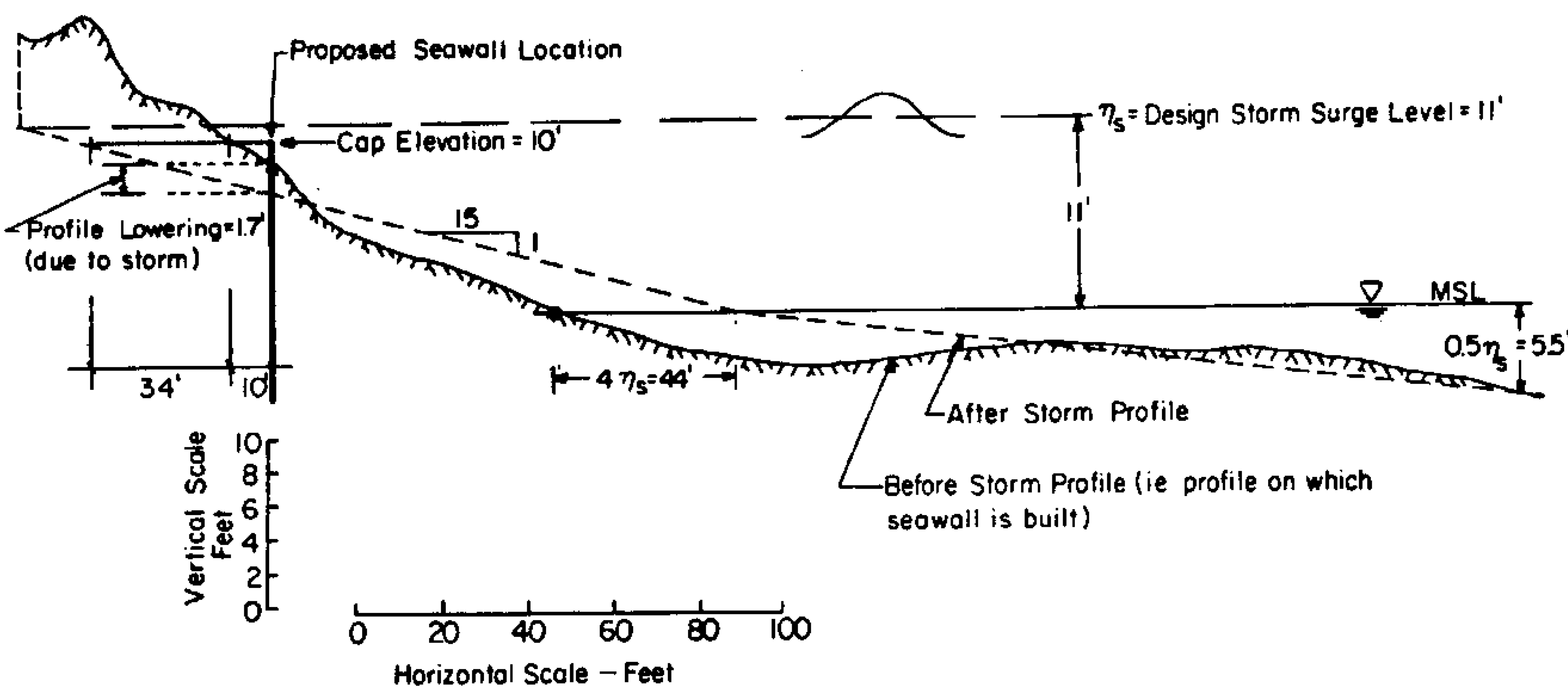

Figure 12. Proposed Seawall Location and Design Profile Assumptions

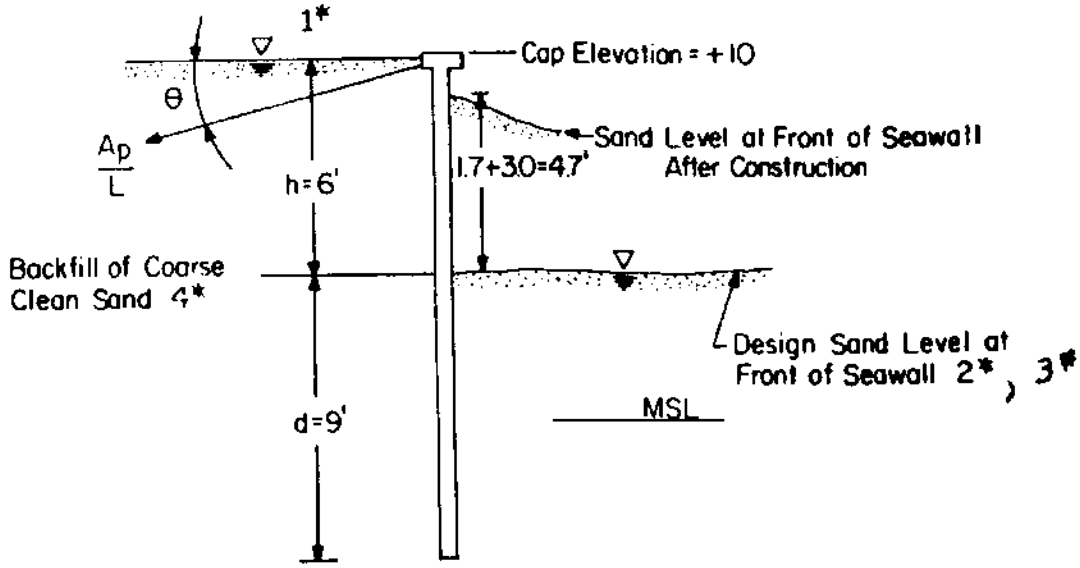

Figure 13. Design Assumptions

The design method used is the Free Earth Support Method due to the use of reinforced concrete design having low wall flexibility. No surcharge loading is assumed in this design, although, if a substantial structure such as a house is to be built adjacent to the seawall, additional loadings must be considered on the wall [15, 16, 17, 18].

<u>List of Notations</u>

γ_w = weight of seawater 64#/ft^3

γ_{sub} = submerged unit weight of backfill (= 64#/ft^3, assumed from Reference [20] for clean sand)

γ_{sat} = saturated unit weight of backfill (= 128#/ft^3, assumed from Reference [20] for clean sand)

K_a = active pressure coefficient 6*(.27 for 35° assumed from Reference [20] for clean sand)

K_p = passive pressure coefficient 6*(3.70 for = 35° assumed from Reference [20] for clean sand)

d = depth of imbedment

b = height of saturated fill

ΔH = water table difference between front and back of wall

$h = b + \Delta H$

G_s = factor of safety for passive pressure distribution (= 2.0, 7*)

Θ = vertical angle of tie rods

$(\frac{A_p}{L})_h$ = horizontal anchor pull per l.f. of wall (#/l.f.)
where A_p = anchor pull
L = length of wall

-minimum depth of pile embedment-
from Figure 9:

$$K = \frac{K_p}{K_a} = \frac{3.70}{0.27} = 13.7$$

$$\beta = \frac{b}{h} = 0$$

$$D = \frac{d}{h} \simeq 1.5$$

h = 6' therefore d = 9'

Thus, for given design conditions seawall should go 9' + 13.7' below existing profile for a reasonable conservative design, i.e. total slab length = h + d = 6' + 9' = 15'.

-Anchor Pull-

Summing horizontal forces acting on the wall

$$\Sigma F_H = 0 = (\frac{A_p}{L})_h + \frac{K_p}{G_s}\ \frac{d^2}{2}\ \gamma'_{sub} - K_a\ \gamma_{sub}\ \frac{(\Delta H + d)^2}{2} - \gamma_w\ (\frac{\Delta H^2}{2} + \frac{\Delta Hd}{2})$$

$(\frac{A_p}{L})_h$ = 1527#/l.f. of seawall

-Point of Zero Shear-

Solve for point of zero shear which is also point of maximum moment. First locate whether point of zero shear is above or below sand level.

$(\frac{A_p}{L})_h - K_a\ \gamma_{sub}\ \frac{\Delta H^2}{2} - \gamma_w\ \frac{\Delta H^2}{2}$ = Shear at design sand level = 64#/ft.

therefore point of maximum moment is above the sand level at a distance X from top of wall. In this problem it is conservative to estimate it at sand level.

-Maximum Bending Moment in Wall-

Maximum bending moment estimated at sand level is:

$$M_{max} = (\frac{A_p}{L})_h\ X - K_a\ \gamma_{sub}\ \frac{\Delta H^3}{6} - \gamma_w\ \frac{\Delta H^3}{6}$$

$$M_{max} = \simeq 6500\ \frac{\text{ft. - \#}}{\text{ft. of wall}} = 6.54\ \frac{\text{Kip - feet}}{\text{ft. of wall}}$$

Actual structural design for the seawall will not be carried further although if reinforced concrete is used, it is recommended that walls no less than 8" thick be considered with at least a 3" concrete covering of all reinforcing steel.

Although design details have not been addressed, it should be noted that the seawall cap must have adequate dimensions and reinforcement to:

(a) withstand the bending moment resulting from the loads on the tie-back rods and sheet piles,

(b) securely hold the sheet pile in place.

In addition, adequate development length must be provided for tie-back rods to prevent pull-out, and tie-back rods must be protected against corrosion. Methods for protection of tie-back rods from corrosion include coating the rods with bitumastic and wrapping with burlap, or encasement in concrete.

NOTES FOR DESIGN CALCULATIONS

1* Water level behind wall is assumed to be at top of cap. In any storm in which waves break on the seawall, a significant amount of over-topping will probably occur. Additionally, heavy rains often occur during a storm and may saturate the soil behind the wall. Drainage provided for in the seawall may not be sufficient to relieve excess hydrostatic pressure behind the wall.

2* Design sand level on the seaward side of the seawall takes into account profile lowering due to storm wave effects on adjacent unseawalled beach which carries over onto seawalled portion of beach and also scour due to breaking waves on the vertical structure.
The difference in original sand level and design sand level is:

1.7'	profile lowering due to design storm
+3.0'	scour at vertical faced structure
=4.7'	total difference

3* The most severe loading conditions for seawalls generally occur some time after the maximum storm surge when the backfill is still at or near saturation but the water level at the exposed face of the wall has dropped.

4* This analysis is based on the assumption that the backfill behind the wall, for a minimum distance equal to the height of the wall, consists of relatively homogenous, clean sand. The magnitude and distribution of active soil pressures would be different for backfill not comprised on clean sand. The use of material, other than clean sand, for backfill is discouraging because:

a. Silty sand, silt or clay does not drain as freely as clean sand.
b. The use of a filter media will not stop the loss of fines if a silty backfill is used.
c. If the backfill contains an appreciable amount of clay, the lateral earth pressure may not remain at an active state and may regain values near that of earth pressure at rest. Refer to References [15, 16].

5* When large head differentials exist between the seawall front face and the assumed water level behind the wall, the upward seepage pressure in fron of the seawall should be taken into consideration. This upward seepage pressure effectively reduces the weight of the soil in front of the wall. An approximation to the weight reduction can be given as:

$\Delta\gamma = \frac{20\Delta h}{D}$ (Reference [17]) where $\Delta\gamma$= weight reduction in lbs/ft^3
Δh= water level difference in feet

For the case considered, making a first assumption that Δh= D the effective buoyant weight of the soil in front of the wall becomes $73\#/ft^3 - 20\#/ft^3 = 53\#/ft^3$.

6* Active and Passive Pressure Coefficients can be found in Reference [20]. They depend on both fill slope angle, coefficient of internal friction for soil, and wall friction angle. It is felt best to ignore the effects of any wall friction for open coast seawall design since critical conditions will occur for a wet (hence lubricated) seawall.

7* Reference [16] recommends a value of 2.0 for non-cohesive soils. This factor will in effect increase the depth of imbedment to account for the uncertainty in passive pressure development and distribution in front of the wall. Often a factor of safety of 1.0 is used for analysis of D and then the embedment depth D is effectively increased by 20-70%. This depth of embedment increase is for the same effect as the factor of safety of 2.0. Alternatively in the design given, $G_S = 1.0$ can be used to find D and then the given D is increased by 70%. Either of these methods is equivalent.

8* In cases where the backfill cannot be lost (i.e. adequate return wall length has been provided) design for wave forces should not be necessary provided the backfill is of clean noncohesive sand and extends to the top of the wall. In cases where these assumptions cannot be met, it is necessary to design for the worst wave conditions to be expected at the seawall site. If breaking waves might be experienced on the seawall during the storm for which the design water level has been chosen, then breaking wave conditions should provide the critical design criteria for seawall overtopping and/or shear failure. Reference [4] should prove invaluable for wave force calculations in design.

REFERENCES

1. Escoffier, F. "Design and Performance of Sea Walls in Mississippi Sound," Proceedings of Second Conference of Coastal Engineering. 1951, pp. 257-67.

2. Purpura, J. A., and Sensabaugh, W. M. "Coastal Construction Setback Line," Marine Advisory Program, Publication SG-74-002. University of Florida, Gainesville, Florida 1974.

3. Seawall and Revetment Effectiveness, Cost and Construction, C. Collier, May 1975, Marine Advisory Program, Publication SG-6. University of Florida, Gainesville, Florida.

4. U.S. Army Coastal Engineering Research Center: Shore Protection Manual, U.S. Government Printing Office, 1973, pp. 5-5,5-7.

5. Edelman, T. "Dune Erosion During Storm Conditions," Proc. 11th Conference on Coastal Engineering, American Society of Civil Engineers. New York, 1968, 719.

6. Vallianos, L. Beach Fill Planning - Brunswick County, North Carolina, in Proc. 14th Int. Conf. on Coastal Engineering, American Society of Civil Engineers, New York, 1975, 1350.

7. Russell, R. C. H., and Inglis, C. "The Influence of a Vertical Wall on a Beach in Front of It," Proceedings of the Minnesota International Hydraulics Convention, Minneapolis, Minnesota, 1953, pp. 221-226.

8. Sawagari, T. "Scouring Due to Wave Action at the Toe of a Permeable Coastal Structure," Proceedings of the Tenth Conference on Coastal Engineering, 1966, pp. 1036-1047.

9. Sato, S., Tanaka, N., and Irie, I., "Study on Scouring at the Foot of Coastal Structures," Proceedings of the Eleventh Conference on Coastal Engineering, 1968, pp. 579-598.

10. Herbich, J. B., and Ko, S. C. "Scour of Sand Beaches in Front of Seawalls," Proceedings of the Eleventh Conference on Coastal Engineering, 1968, pp. 622-643.

11. Kadib, Abdel-Latif. "Beach Profile as Affected by Vertical Walls," Beach Erosion Board, U.S. Army Corps of Engineers Tech Memo No. 134, 1963.

12. Chesnutt, C. B., and Schiller, R. E., Jr. "Scour of Simulated Gulf Coast Sand Beaches Due to Wave Action in Front of Sea Walls and Dune Barriers," COE Report No. 139, TAMU-SG-71-207, May, 1971.

13. Herbich, J. B., Murphy, H. D., and Van Weele, B. "Scour of Flat Sand Beaches Due to Wave Action in Front of Sea Walls," Proceedings of the Santa Barbara Specialty Conference on Coastal Engineering, 1965, pp. 705-726.

14. "The Effect of Vertical Seawalls on Longshore Currents," D. F. Jones, Ph.D. Thesis, University of Florida, 1975.

15. U.S. Steel: Steel Sheet Piling Design Manual, April 1972.

16. Tschebotarioff, Gregory P. "Foundations, Retaining and Earth Structures," 2nd Edition, McGraw Hill.

17. Leonards, G. A. "Foundation Engineering," McGraw Hill Book Co., Inc. New York, 1962.

18. Terzagi, Karl: ANCHORED BULKHEADS, Trans. ASCE 1954.

19. "Foundation Engineering," Peck, Hanson, Thornburn - Second Edition, John Wiley & Sons, Inc., 1974.

20. Department of the Navy, Naval Facilities Engineering Command: DESIGN MANUAL - SOIL MECHANICS, FOUNDATIONS, and EARTH STRUCTURES, DM-7, March 1971.

21. "Prestressed Concrete Sheetpiles for Bulkheads and Retaining Walls," by W. E. Dean, Civil Engineering Magazine, April 1960.

22. "Concrete Shore Protection," Portland Cement Association, Skokie, Illinois, 1955.

23. Staff Report on Design of Reinforced Concrete Seawalls, Bureau of Beaches and Shores, Dept. of Natural Resources, State of Florida, Tallahassee, 1976.

24. American Concrete Institute: BUILDING CODE REQUIREMENTS FOR REINFORCED CONCRETE (ACI 318-71), 1971.

25. Variance Permit File, Bureau of Beaches and Shores, Department of Natural Resources, State of Florida, Tallahassee.

26. "The Effect of Waves on Rubble-Mound Structures," F. Raichlen, Annual Review of Fluid Mechanics, Vol. 7, 1975.

The Florida Sea Grant College is supported by award of the Office of Sea Grant, National Oceanic and Atmospheric Administration, U.S. Department of Commerce, grant number 04-8-M01-76, under provisions of the National Sea Grant College and Programs Act of 1966. The Florida Sea Grant College was initiated in 1972 with three major components: applied marine research, education, and advisory services.

> This public document was promulgated at a cost of $515.58 or $.17¢cents per copy, to provide current information on seawall design and construction along the open coast. Cost does not include postage and handling.

State University System of Florida Sea Grant Reports are published by the Marine Advisory Program which functions as a component of the Florida Cooperative Extension Service, John T. Woeste, Dean, in conducting Cooperative Extension work in Agriculture, Home Economics, and Marine Sciences, State of Florida, U.S. Department of Agriculture, U.S. Department of Commerce, and Boards of County Commissioners, cooperating. Printed and distributed in furtherance of the Acts of Congress of May 8 and June 14, 1914.

6/3M/79

Copies of this and other Florida Sea Grant and Marine Advisory Program publications are available from:

Marine Advisory Program
G022 McCarty Hall
University of Florida
Gainesville, FL 32611

DEPOSITORY
CAMPUS
RI 02882

RECEIVED
NATIONAL SEA GRANT DEPOSITORY
DATE AUG 3 1 1979