Chapter 4
Fill Design

4-1. Purpose of Project

Prior to planning and conducting the design of a beach fill project, it should be understood that beach fills are typically constructed for storm and flood protection, erosion mitigation, and enhanced recreation.

a. Storm and flood protection. Designing a beach fill project for storm and flood protection requires a rigorous design approach due to the severe consequences if the project does not perform as required. Some important considerations for storm and flood protection projects are as follows:

(1) Existing and desired level of storm and flood protection. The optimum level of storm and flood protection will be determined based on maximizing the net benefits of the project. Net average annual benefits equal the average annual damages estimated over the future evaluation period (in the absence of a Federal project) minus average annual damages estimated to occur over the same period with the Federal project, minus average annual costs.

(2) Data on historical storm occurrences and effects on the area (including dune breaching, beach erosion, structural damage, and interior flooding) are required to develop erosion excursion damage frequency relations and provide calibration and verification data for shoreline and profile change models.

(3) Value of development, type of construction, proximity to the shoreline, and first floor elevations are required to provide the damage functions for the project area. Typically, dense high-rise development will have a much higher property value than a residential or vacation home development, but the overall risk of damage to a high-rise structure due to foundation undercutting or wave attack will be confined to the lower one or two floors.

b. Mitigation of erosion. Frequently, beach fill projects are located adjacent to an inlet or entrance channel. Occasionally, investigations have identified the need and justification to mitigate erosion attributed to the adjacent Federal navigation project. Such erosion is seldom 100 percent of the total erosion of the adjacent shoreline. Other factors also contribute to the problem such as natural inlet or entrance effects, relative sea level rise, natural wave focusing, other close-by structures, etc. In the interest of providing a comprehensive and complete solution for the total erosion problem, the mitigation effort is incorporated into the beach fill project.

c. Enlargement of recreational area. Due to long-term average erosion and/or storm erosion, the area of beach available for recreational purposes will generally not be sufficient to satisfy the existing or projected future demand. In areas where recreational beach use and its support activities are important to the local economy of the coastal community, a storm protection beach fill project will provide additional recreational beach area incidental to its construction. Recreational benefits based on increased public use or enhanced values of a recreational day incidental to the beach fill can be used in project justification. Present policy limits the extent to which some benefits may be used in optimizing project design.

4-2. Project Design

a. Design selection. In 1985, more than 95 beach fill projects had been constructed by the Corps, protecting over 210 km (130 miles) of U.S. coastline. Unfortunately, few of these projects were monitored to any significant degree to provide guidance for the design of other projects. Due to the geographical and climatological differences between areas along the coast, extrapolating the design of one project to another location may or may not be successful. However, based on the limited monitoring that has been conducted and analysis of some of the projects, certain general guidelines have been developed for the selection and evaluation of project design. According to experience, the design of a beach fill project should include the following considerations:

(1) Level of protection. The primary function of beach nourishment projects is to maintain a beach at a specific location to provide protection to upland areas against storm flooding and waves. A variety of design parameters can be selected to provide an optimum level of protection for each fill site. Consequently, selection of design parameters should be made on the basis of accurate up-to-date information on the project beach, and environmental factors such as wave climate and littoral currents. The principal design parameters of a beach fill project are tidal characteristics; wind and wave climate; storm characteristics and frequency of occurrence; shoreline change history; sediment characteristics; sediment budget; borrow material availability and suitability; and environmental considerations.

(2) Placement techniques. The placement of fill material on a project area is conducted by two primary methods; land-hauling material from a nearby upland source; or direct pumping of sand through a pipeline from subaqueous sources using a dredge (*Shore Protection Manual* 1984).

Two basic types of dredging techniques used are hydraulic pipeline dredges and hopper dredges (with and without pump-out capabilities). The choice of placement method depends upon the location of suitable borrow sources and availability of equipment. Each method differs in its capabilities for the area of fill placement. Fill material brought to the site by truck is limited in its placement to dry beach or water's edge. Pumping and barging materials provides the ability to place materials in the nearshore zones. Design requirements should consider available placement methods.

(3) Placement position. The placement of fill material within the beach system should be determined by the protection requirements and method of placement. Figure 4-1 illustrates the various cross-shore placement locations most widely used within the profile and includes (Smith and Jackson 1990):

(a) Dune only - reinforcement of existing dune or creation of a new dune in the back-beach areas. This technique is intended to provide the beach system with a reserve of sand against erosive storm events and to prevent wave energy from reaching upland property.

(b) Dune and berm - intended to reinforce the dune and widen the berm to withstand erosion due to the storm of record plus an additional width to prevent waves from overtopping and reaching upland property.

(c) Berm only - designed to add volume and widen the beach by translating the wave-breaking zone seaward. Berm height is usually retained at the same height as the natural berm height. For recreational purposes, the increased width of the beach will provide additional area for recreational use.

(d) Profile fill - method is designed to increase the volume of sand throughout the entire profile. Placement usually occurs at an active zone in the profile and relies on natural processes to distribute the fill over the entire profile. Theoretically, this method should produce a profile shape already in equilibrium with the energy environment.

(e) Nearshore berm - intended to simulate natural storm bar formation by creating an artificial shore-parallel storm bar to dissipate storm wave energy before impacting the inshore beach. During prolonged low-energy conditions, much of the artificial bar material may be moved onshore, nourishing the berm and nearshore.

Longshore placement considerations include the creation of a feeder beach. This is performed by stockpiling fill material at the updrift end of the areas intended to receive

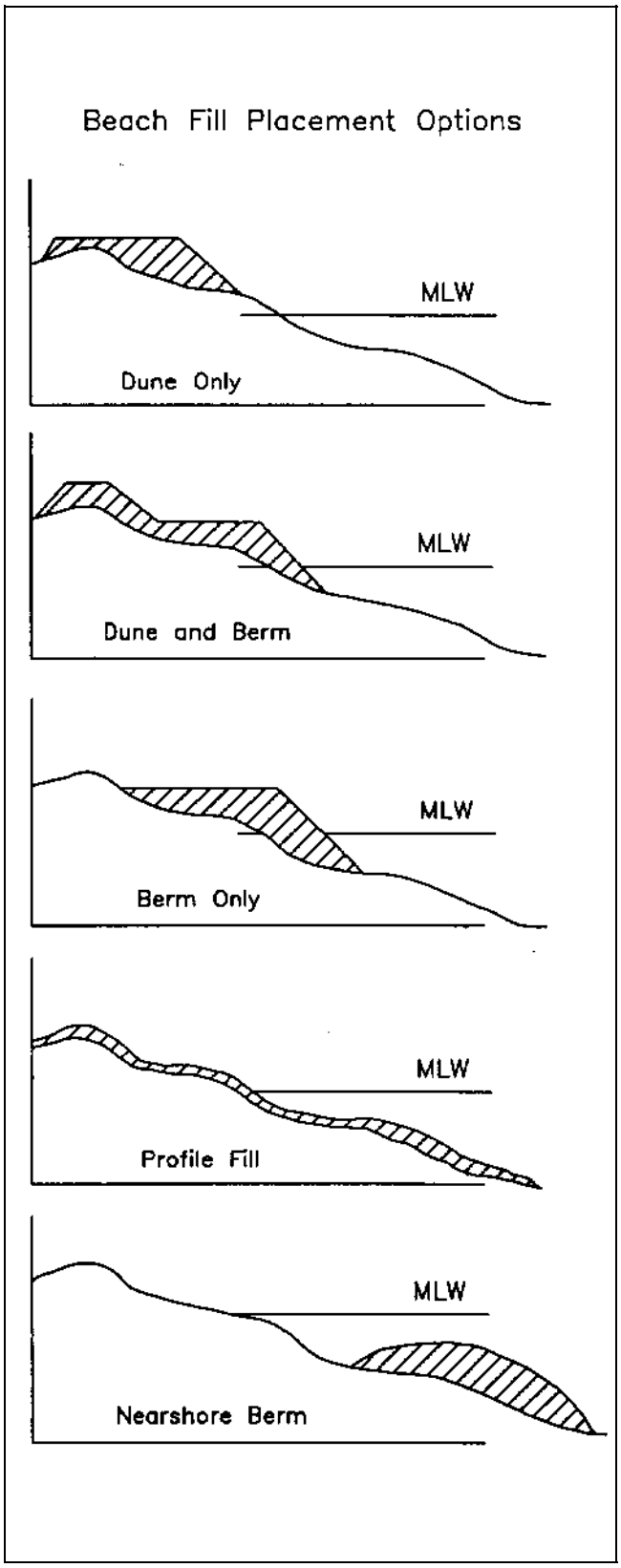


Figure 4-1. Various fill placement zones within the beach profile system

the fill, allowing longshore transport processes to distribute the fill into the fill site. Feeder beaches generally work well in areas that are presently serving as a source of sediment for the downdrift beach or in areas that presently experience a deficit in the supply of littoral material. Examples would be those areas that are downdrift from inlets or other man-made structures that form a littoral barrier. Benefits from a feeder beach are generally limited to the shorelines immediately adjacent to the stockpile. As the stockpiled material spreads under the influence of oblique waves, the orientation of the feeder beach shoreline approaches that of the adjacent beach, resulting in longshore transport out of the feeder area equal to the transport along the adjacent area. Eventually, the shoreline orientation in the feeder beach area will return to its original configuration, at which time the transport out of the feeder beach will be less than that of the adjacent area. Therefore, depending upon the length of the project, feeder beaches should not be expected to satisfy the nourishment requirements for the entire project as these needs may best be satisfied through direct placement.

(4) Combined placement. The profiles illustrated in Figure 4-1 are intended to present ideal fill placement and, in reality, would hardly ever be achieved. In most cases it may be beneficial to use a combined method of placing fill in both the upper and lower portions of the beach profile. Such operations could provide the advantage of limiting shoreline protrusion by the fill since some of it would be placed in the offshore part of the profile. This could reduce initial losses typically experienced during most fill projects by reducing longshore transport out of the project area. A disadvantage of this method is that two placement operations must be used to perform placement in both onshore and offshore portions of the profile.

(5) Construction template. The construction template defines the shape of the fill profile at the time of fill placement. The construction profile of the offshore slope should be on the order of 1:20 to 1:30 from the low water datum to intersection with the existing bottom. Construction profiles are out of equilibrium with the prevailing coastal processes and are expected to reshape themselves, starting almost immediately after placement, as they are influenced by the existing energy conditions. As the reshaping process occurs, much of the material placed on the constructed berm and foreshore will be moved by waves and currents into the nearshore zone. The volume of fill material must allow for this offshore readjustment toward a more natural shaped profile. However, during placement, the fill should be continually monitored to determine actual foreshore slopes. Attempts should not be made to strictly adhere to the construction template foreshore slope in the nearshore region. Adjustments should be made to the construction berm width to allow for differences that occur between assumed and actual slopes. This will prevent unnecessary time expended while attempting to mold the profile in a dynamic region of the profile affected by waves.

(a) If possible, construction berm elevations should be designed to be the same or slightly less than the natural berm crest elevation. The intent is for the construction berm to erode, distributing the material throughout the entire profile, resulting in a naturally shaped profile. Restricting the construction berm height to the natural berm height will prevent scarping of the fill material as it undergoes readjustment. Scarps can pose a threat to humans and also present a problem for nesting sea turtles.

(6) Design template. The design profile is the shape the fill material is expected to achieve after being worked by waves over the first few months to a year after fill placement. The design profile may be based on the pre-fill profile shape if the fill material is similar to the original native beach material. In such cases, the beach profile after nourishment should be the same as before nourishment, except translated seaward. The seaward translation forms a parallelogram similar to that illustrated in Figure 4-2 and can be shown to have an area with the base equal to the berm height (B) plus the depth of closure (H) and a length (Y) represented by the distance that the beach is extended seaward. Using the profile translation dimensions, an equation presented in the *Shore Protection Manual* (1984) to determine the fill volume per unit length of shoreline (assuming 1 m) required to achieve a profile configuration with a specific berm height out to some distance can be utilized and defined as:

$$V = (B + H)\ Y \tag{4-1}$$

where

B = desired berm height (m)
H = depth of closure (m)
Y = desired distance of seaward translation (m)

For example, the volume of borrow material with the same sediment characteristics as the native beach to build a beach with a berm height of 1.5 m (5 ft) to a width of 50 m (164 ft), in an area where the depth of closure is approximately 9 m (30 ft) would be 525 m^3/m (5,650 ft^3/ft) of shoreline. This method of determining fill volumes required to achieve the design profile can be used only if the fill material is similar to that of native beach sediments.

(a) The profile translation method assumes that the profile will be reshaped by the prevailing coastal processes and form the fill material into the existing equilibrium

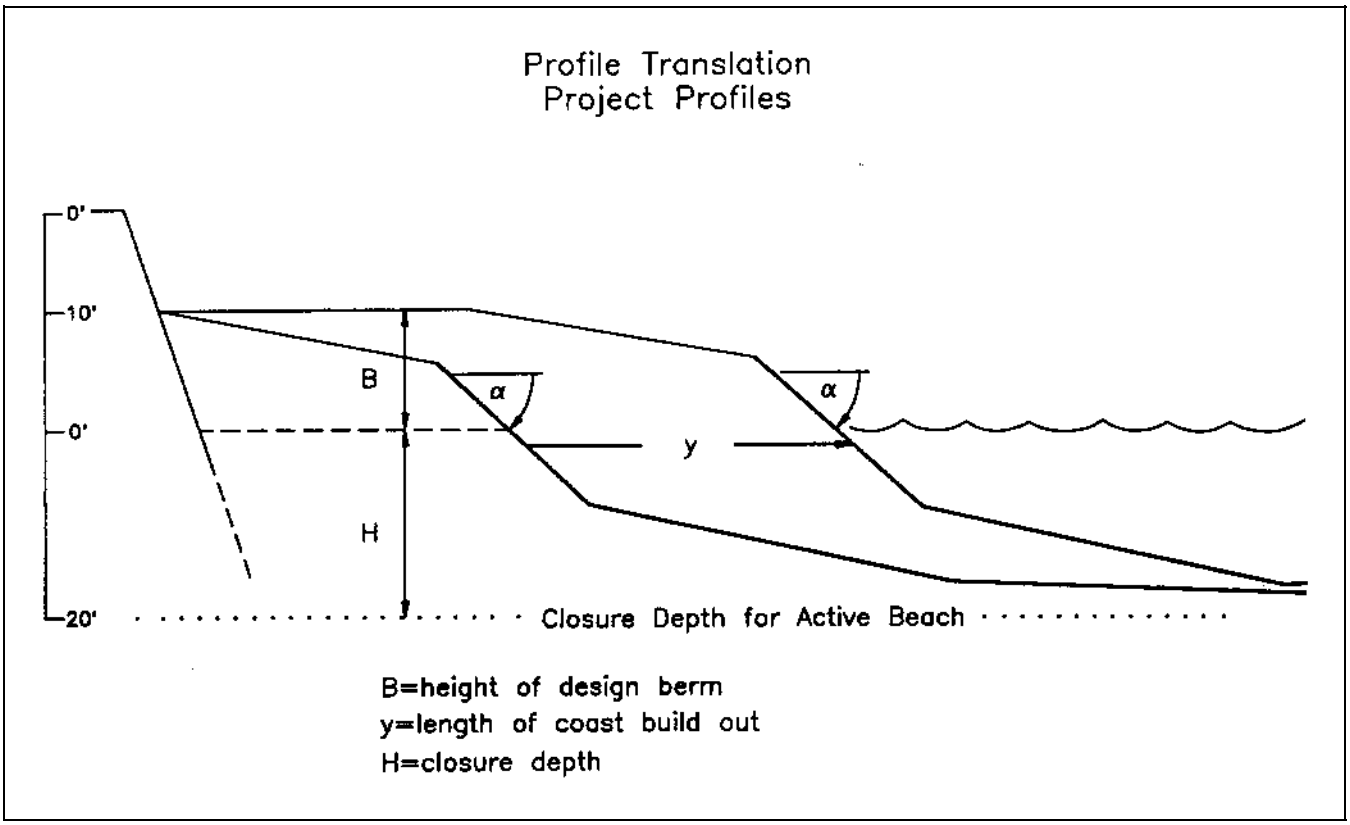


Figure 4-2. Native beach profile versus minimum design beach profile using fill similar to native fill. Design profile is same shape translated seaward

profile shape out to depth of closure with conservation of volume. As the fill profile readjusts, a large percentage of material from the visible beach is relocated to the nearshore portion of the profile. Casual observers may conclude that most of the material is lost from the system. In reality, what appears to be lost from the visible beach is in the nearshore area, within the project's limits.

(b) Figure 4-3 presents the various profiles used for determining fill volume requirements. A design profile based on the "equilibrium profile" concept is generally used for computing fill requirements. The initial slope of the foreshore of a beach design profile is typically on the order of 1:10 to 1:15 above the high-water datum. Existing and historic profile slopes will guide the estimation of these naturally shaped slopes. The depth of closure, which is the depth to which the beach fill material or native beach material can be mobilized under design storm conditions, should be determined and considered in the calculation of the beach fill quantity. Guidance for determining the depth of closure is presented in Section 2-2.

(7) Overfill addition to construction template. The beach fill material must be analyzed sufficiently to evaluate its performance as compared to the native beach material. This is done by determining the overfill ratio (R_A) as presented in Section 3-3. Beach fill material that is similar in grain size distribution to the native beach material can be expected to evolve to a profile slope similar to the native beach profile slope. Beach fill material which contains significant quantities of material finer than the native beach material will lose a significant percentage of the material to winnowing and sorting as the fill material grain size distribution adjusts to be closer to the native material grain size distribution. Therefore larger amounts of fine material will have to be placed on a beach to achieve the desired design profile. Introduction of finer than native material will result in flatter slopes, and coarser material will result in steeper slopes. Consider the example from the overfill ratio, presented in Section 3-2, where the borrow material was finer than the native beach sediments. It was determined that 2.40 units of borrow material would be required to produce 1.0 unit of stable native-like material. Using the profile translation method (Equation 4-1), the volume of borrow material required to achieve the dimensions used in the example above would be (2.40 x 525) 1,260 m^3/m (13,560 ft^3/ft) of shoreline.

(8) Advanced fill addition to construction template. Background erosion rates due to longshore transport, aeolian

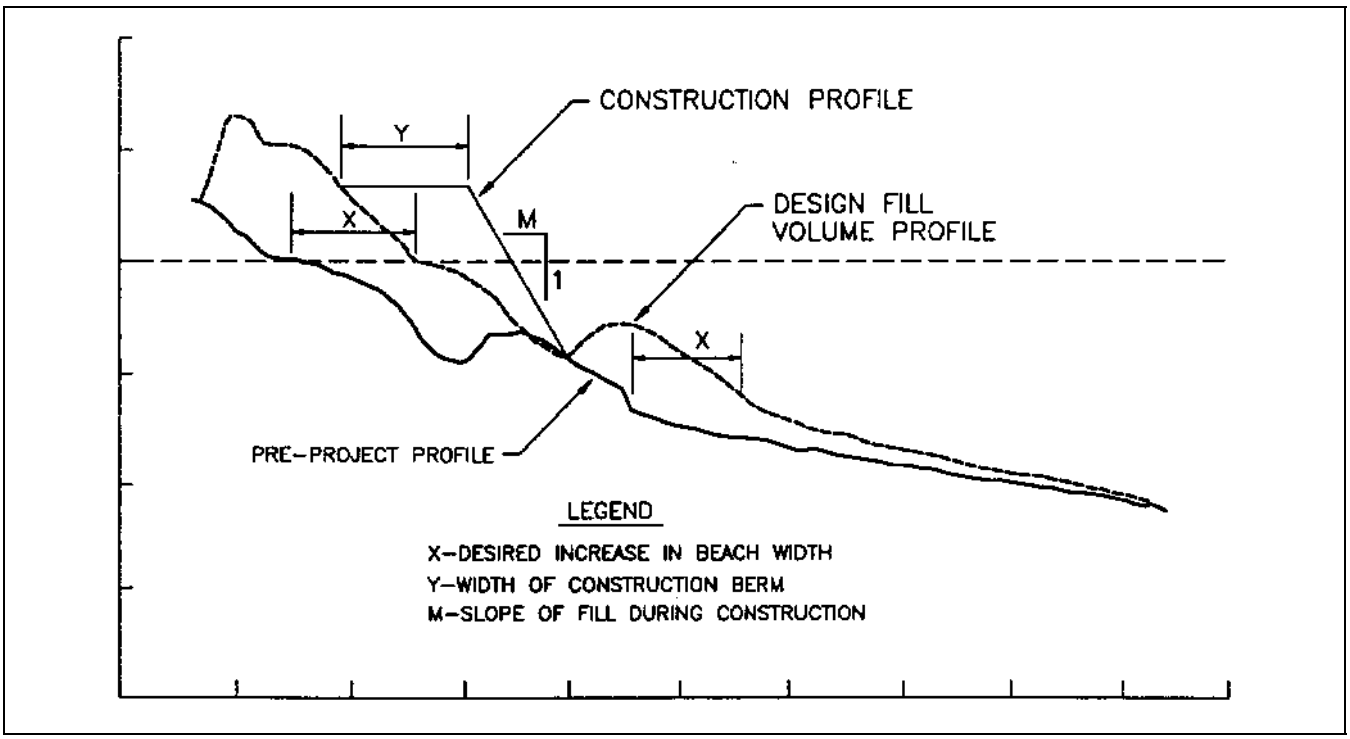


Figure 4-3. Comparison of construction, design, and pre-fill profiles used in determining fill volume requirements

transport, offshore transport, etc., should be determined as accurately as possible to develop an accurate estimate of periodic nourishment requirements for a project. These rates can be estimated from historical shoreline change analyses and sediment budget calculations, provided that coastal structures are not present to modify the natural erosion rates. Careful selection of the analysis period or location will be required if such structures exist. Also to be considered are losses of fill material associated with shoreline perturbations created by the fill itself, causing a loss of fill at each end of the fill area. Additional discussion on advanced fill requirements is provided in Section 4-4.

b. Analytical design method. The concept of "equilibrium profile" is used extensively in the analysis of the response of a beach to long-term or extreme wave conditions and is discussed in detail in Section 2-6. The overall equilibrium profile shape has been found to be governed primarily by sediment size characteristics (Dean 1991). Based on studies of beaches in many environments, Bruun (1954) and Dean (1976, 1977) have shown that many ocean beach profiles exhibit a concave shape such that the depth varies as the two-thirds power of distance offshore along the submerged portions as defined by Equation 2-3. Based on this relationship, Dean (1991) has developed analytical techniques for predicting beach profile response to beach nourishment that can be useful in the planning and design of a beach fill project.

(1) The methods presented by Dean (1991) determine the volume of fill of arbitrary size required to produce a desired width of subaerial beach per unit length of shoreline after the profiles have reached equilibrium. Based on the equilibrium profile equation presented above, the volume of fill required to translate the beach a given distance seaward can be calculated. In his approach, Dean (1991) defines three basic types of nourished profiles:

(a) An intersecting profile, where the profile after nourishment intersects the native profile landward of the depth of closure.

(b) A nonintersecting profile, where the nourished profile does not intersect the native profile before the closure depth.

(c) A nonintersecting submerged profile, where the nourished beach does not intersect the native profile and no subaerial beach exists after equilibrium.

The type of nourished profile is dependent upon inequalities between the fill and native beach sediments. A requirement for intersecting nourished profiles is for the fill material to be coarser than the native beach sediments. Nonintersecting and submerged nourished profiles require that the fill material be equal to or finer than the native material. If a

large enough volume of fill is placed, it is possible for a nonintersecting nourished profile to occur when the fill material is coarser than the native. Intersecting and nonintersecting profiles are determined by the following inequalities:

$$Y\left(\frac{A_N}{H}\right)^{3/2} + \left(\frac{A_N}{A_F}\right)^{3/2} < 1 \quad (\textit{intersecting}) \tag{4-2}$$

$$Y\left(\frac{A_N}{H}\right)^{3/2} + \left(\frac{A_N}{A_F}\right)^{3/2} > 1 \quad (\textit{nonintersecting}) \tag{4-3}$$

where

A_N = value of the A parameter for the native sediments (from the equilibrium profile equation)
A_F = value of the A parameter for the fill material
H = depth of closure

(2) Considering the conditions described by Equations 4-2 and 4-3, the volume of fill required to produce a desired width of subaerial beach per unit shoreline can be determined. For the rare case when the fill material is similar to the native beach sediments (i.e. $A_F \simeq A_N$), the volume (V) of fill required could be determined using Equation 4-1. For circumstances where the fill material is finer than the native sediments (i.e. $A_F \langle A_N$), a critical volume of fill is required for any subaerial beach to form after equilibrium and is defined as:

$$V = \frac{3}{5} H^{5/2} \left(\frac{1}{A_N}\right)^{3/2} \left(\frac{A_N}{A_F}\right)^{3/2} \left(\frac{A_N}{A_F} - 1\right) \tag{4-4}$$

If the amount of fill placed is less than that determined using Equation 4-4, the nourished profile will be submerged. The volume (V) required for satisfying conditions for a nonintersecting nourished profile, where the fill material is finer than the native material and with a subaerial beach after equilibrium, is:

$$V = YB + \frac{3}{5}H^{5/2}\left[\left(\frac{Y}{H^{3/2}} + \left(\frac{1}{A_F}\right)^{3/2}\right)^{5/3} A_N - \left(\frac{1}{A_F}\right)^{3/2}\right] \tag{4-5}$$

Similarly, if the fill material is finer than the native material, a larger quantity of fill material will be required to widen the beach. The volume (V) of fill required to achieve design dimensions satisfying the conditions of an intersecting nourished profile is given by:

$$V = BY + \frac{A_N Y^{5/3}}{\left[1 - \left(\frac{A_N}{A_F}\right)^{3/2}\right]^{2/3}} \tag{4-6}$$

where the fill material is coarser than the native material (i.e. $A_F > A_N$). The coarser the fill material relative to the native material, the steeper the nourished profile and the beach fill will intersect the native beach closer to shore resulting in significantly smaller quantities of fill material to achieve a desired beach width.

(a) The following steps represent the use of Dean's method to determine the volume of fill material necessary to produce a nourished beach (after equilibrium) assuming a 1-m-wide length of shoreline with a berm height (B) of 1.5 m, out to a width of (Y) 50.0 m, assuming a closure depth (H) of 9.0 m, an A_N of 0.1, and an A_F of 0.09. First, it must be determined if the selected fill material would produce an intersecting or nonintersecting nourished profile by determining which inequality from Equations 4-2 and 4-3 is satisfied. Substituting the given values produces:

$$50.0\ (0.10 / 9.0)^{3/2} + (0.10 / 0.09)^{3/2} = 1.2 > 1$$

which satisfies Equation 4-3, indicating a nonintersecting profile. Because a subaerial beach is required, Equation 4-4 allows the determination of a minimum placed volume of fill to prevent the occurrence of a submerged nourished profile and is as follows:

$$V = 3/5\ (9.0)^{5/2}\ (1.0/0.10)^{3/2}\ (0.10/0.09)^{3/2}\ (0.10/0.09 - 1)$$
$$V = (145.8)\ (31.6)\ (1.2)\ (0.11)$$
$$V = 608.2$$

Results from Equation 4-4 state that a volume greater than 608.2 m^3/m (6,546 ft^3/ft) of shoreline is required to produce a subaerial nourished beach. However, the volume of fill material required to produce a nourished beach with the above dimensions at equilibrium is represented by Equation 4-5 and would be:

$$V = 75.0 + 145.8\ [\ (\ 1.9 + 37.0)^{5/3}\ 0.1 - 37.0\ ]$$
$$V = 75.0 + 145.8\ \ (45.2 - 37.0)$$
$$V = 1270.6$$

To achieve the nourished beach dimensions given above after equilibrium would require a volume of 1,270.6 m^3/m (13,675 ft^3/ft) of shoreline for the project area. If the fill and native beach sediment characteristics produce values of A_N and A_F satisfying the condition represented by

Equation 4-2 (intersecting profile, fill coarser than native), then the volume required to achieve the example dimensions would be determined using Equation 4-6.

(3) The amount of fill needed to construct a beach fill must be adjusted for the percent of clay and silt in the borrow material. Such fill behavior is illustrated in Figure 4-4. However, it should be noted that while the end result is qualitatively the same as the profile translation method used with the overfill ratio concept discussed previously, the underlying theory is different. In the overfill ratio case it is assumed that the native beach material is stable for a given wave environment and any finer material will tend to be winnowed from the fill and washed away, until the remaining larger fraction of the beach fill material resembles the native beach fill in size distribution and profile slope. In the grain-size-dependant equilibrium slope theory of Dean, it is assumed that the beach fill material will reach an equilibrium slope which is a function of the fill grain size. In reality the situation is much more complex than either of the theories, with the beach fill reaching an equilibrium with the local wave climate by adjusting in slope and winnowing fines from the upper beach (Figure 4-5). Therefore, it will be useful to compare the results of volume estimates using both techniques.

c. Physical models. As discussed by Hudson (1979) and Fowler and Smith (1987), physical models have been used as a tool in the design of coastal projects. These models are generally fixed bed, movable bed, or a combination of the two. Either two-dimensional or three-dimensional investigations can be conducted. For beach fill projects, three-dimensional movable bed models are required to simulate motions in the onshore-offshore and alongshore directions as well as vertically within the water column. The primary problem associated with physical modeling of movable beds results from effects caused by reducing the length scales, and scaling of both sediment size and fluid properties. Difficulties in simulating the relevant properties in model and prototypes result in “scale effects.” These scale effects are not completely understood and limit the accuracy of the results of movable bed modeling. However, if the limitations imposed by scale effects are taken into consideration, useful and sufficiently accurate information can be obtained for design purposes. To date, only limited applications of these models have been made and all cases involved the use of stabilizing structures such as groins or breakwaters. Some examples of investigations are presented by Hudson (1979).

(1) One of the advantages of physical modeling is the ability to conduct detailed and controlled studies of the complex hydrodynamic interactions in the coastal zone. This makes them especially useful for examining complex geometries of groins, breakwaters, and artificial headlands.

(2) In order to obtain meaningful and reliable engineering data from a beach fill project, extensive calibration and verification data should be available from the site of interest.

(3) A major disadvantage of physical models, in addition to the difficulties in determining proper scaling relationships, is the high cost and time required to conduct the investigation.

d. Numerical models. In recent years, numerous numerical models have been developed to predict shoreline and beach evolution (Kraus 1989). These models can be applied to the formulation and design of beach fill projects. With proper application, these models can be used to efficiently evaluate the performance of alternative project designs and to evaluate the effects of design constraints on project performance. A major advantage of the numerical modeling approach is that once the model is set up and calibrated, design changes can be evaluated efficiently. Kraus (1989) discusses the types of models available and their capabilities as summarized in the following paragraphs.

(1) Profile change/beach erosion models. These models calculate the response of the design profile and dune to storm surges and storm waves, providing estimates of the amount of erosion to the berm and upper beach and transfer of sand from the upper beach to below the water line in response to design storm conditions (Kriebel 1982; Vellinga 1983; Kriebel and Dean 1985; Larson 1988; Scheffner 1988, 1989; Larson and Kraus 1989). Generally, these models simulate the response of only one profile at a time and only in the onshore-offshore direction; longshore transport is neglected. Typical inputs to these types of numerical models include:

(a) Native beach profiles.

(b) Major storm processes and beach response (for model calibration).

(c) Time history of storm waves, water levels (storm surge), tide, and wind.

(d) One grain size characterizing the native beach sediments.

(e) Initial equilibrated design template placed on typical native profile.

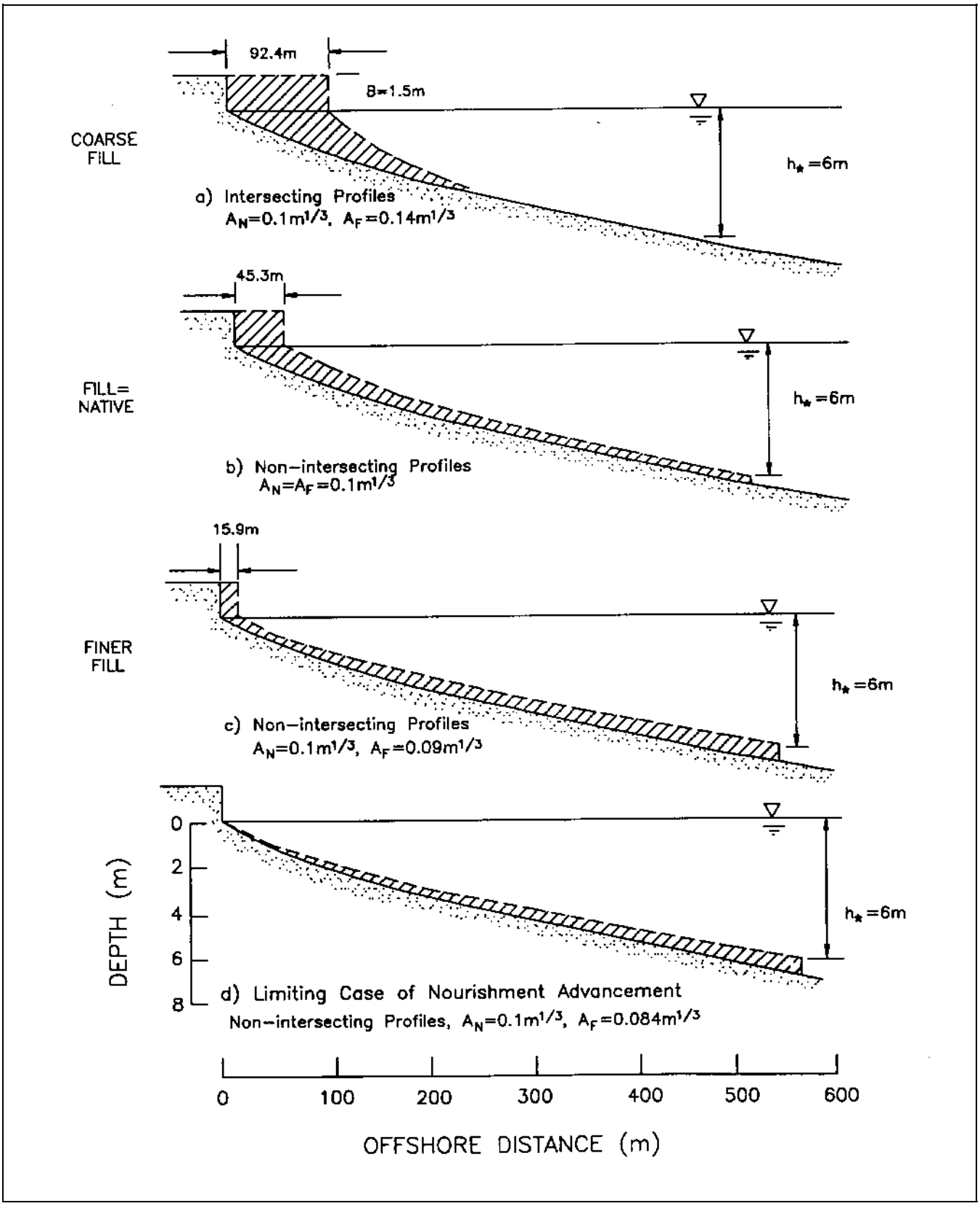


Figure 4-4. Behavior of design profile with varying fill grain size

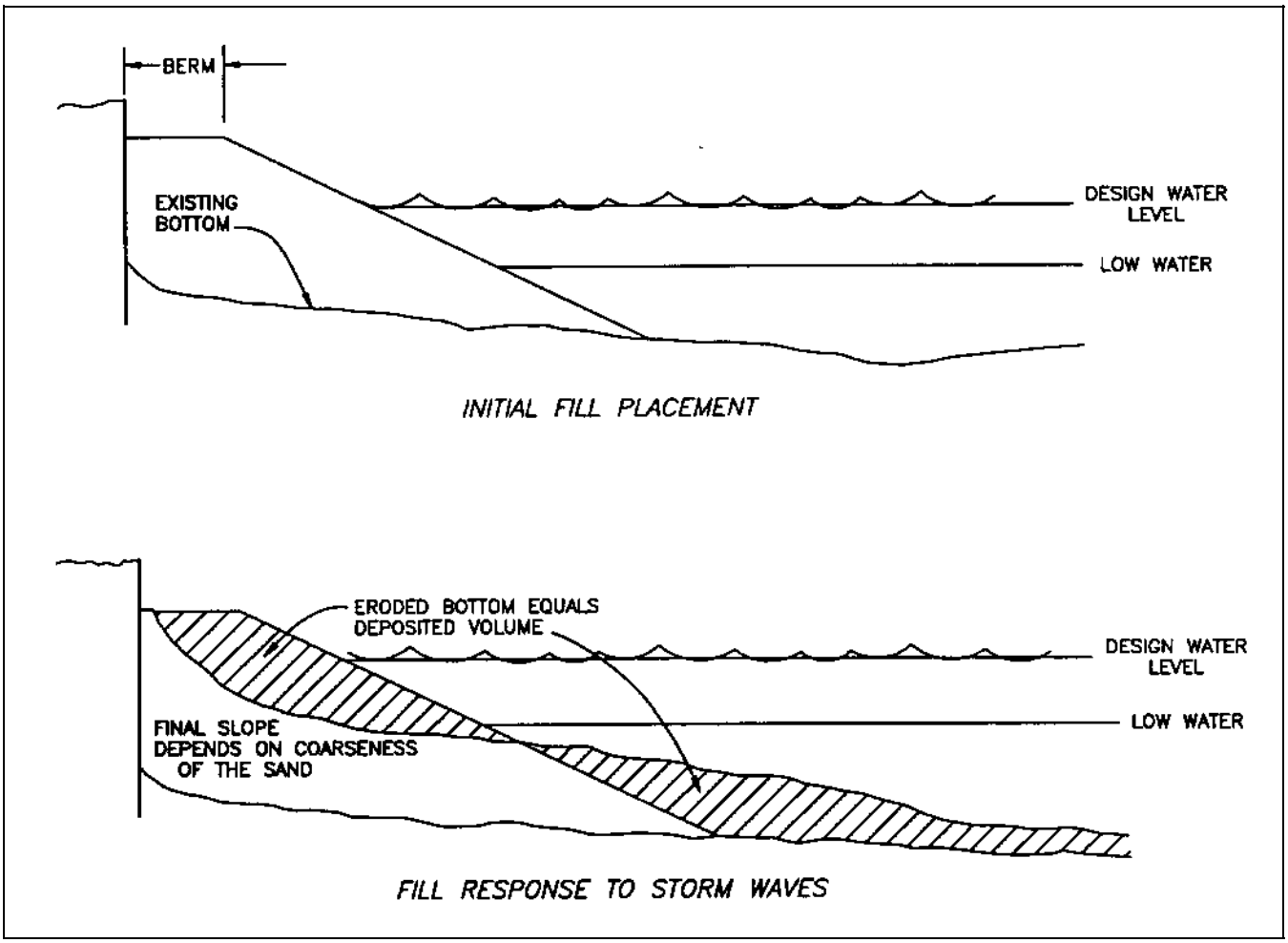


Figure 4-5. A realistic adjustment of beach fill construction profile to storm wave conditions

With proper application, these models can calculate with some reliability beach erosion produced by storms. This capability can be very useful in evaluating the ability of the beach fill design to provide protection to shorefront structures. A number of profile change/beach erosion models are discussed below.

(a) Larson and Kraus (1989) present a recently developed model called SBEACH (Storm-induced BEAch CHange) to predict storm-induced beach erosion which has the capability to reproduce bar and berm formation and movement. The model calculates wave heights at regularly spaced intervals from deep water to the shoreline and then uses separate empirically based relationships to calculate net cross-shore transport rate in four distinct regions along the profile. The model has been calibrated for site-specific projects, and has been found to produce reasonable storm-induced beach erosion results. Use of the model for developing final design parameters is recommended.

(b) Technical Report CERC-87-8 (Birkemeier et al. 1987) evaluated existing theoretical, empirical, and parameterized models which were available prior to SBEACH for predicting beach profile change and dune erosion. Of the models evaluated, two dune erosion models were found to produce reasonable dune erosion estimates. One model (Kriebel 1982) is based on the assumption of uniform energy dissipation in the surf zone and the concept of an equilibrium profile shape which responds to a rising water level by shifting upward and landward. This model was used in the design of a hurricane protection project at Ocean City, MD (Larson and Kraus 1989; Fulford and Grosskopf 1988). A variation of this model can be found in the ACES package (Leenknecht et. al. 1990) and is recommended for preliminary design efforts. The other model (Vellinga 1983) was developed for use on the coast of The Netherlands and is based on a profile shape equation developed from an extensive series of large-scale physical model tests. Both of these models were evaluated using

14 profiles from four different storm events and found to produce comparable results. Of the two, the Kriebel model was found to be less demanding in terms of required input information and therefore easier to consistently apply.

(c) A systematic means of predicting time-dependent erosion of dunes as a function of storm events of known frequency of occurrence is presented in Scheffner (1989). The dune erosion model used in that study is a modified version of the model developed by Kriebel (1982). The technique presented produces dune recession-frequency relationships which can be used for estimating the design life of structures protected by the presence of a berm and dune.

(d) All of the above models require the same basic input data, and provide the same general type of information, although the specific requirements vary from model to model. Basic data required for profile change/beach erosion models include: time series of wave height, period, and direction; time series of water level (combined storm surge and tide); pre-storm profile geometry; and a representative median grain size for the profile. Output includes the post-storm profile geometry from which various contour locations and volume changes can be calculated. Prior to using a model for design it is necessary to calibrate and verify the model using pre- and post-storm profiles bracketing storms of known conditions. A typical application of a model of this type will entail running the model to generate near-shore stage frequency curves and shoreline retreat frequency curves for the without-project profile. In some cases, a damaging wave penetration distance relation will be required for wave damage estimates. A range and number of storm events are run which reflect realistic combinations of various storm parameters descriptive of historic storm events which have impacted in or near the area of interest. An array of beach fill design alternatives is developed and placed on the profile, and the same storm events run through the model for each alternative. For beach fill projects, design alternatives can include variations of berm width, dune height, and addition of a seawall. In some cases, berm height variations may also be of interest if no predominant natural berm elevation is evident or a second storm berm is to be considered. Frequency curves similar to the without-project curves are developed for each alternative. Benefit analyses are then conducted comparing the damages derived from the without-project frequency curves less those derived from the with-project alternative frequency curves, and the project costs over a future 50-year period of analysis expressed in terms of average annual net benefits. The project alternative producing the maximum annual net damage reduction benefits is generally selected as the project plan unless circumstances dictate selection of another alternative.

(2) Shoreline change models. These models are typically one-dimensional in the longshore direction, and calculate the shoreline response to wave action under a wide range of beach, coastal structure, wave, and initial and boundary conditions (Kraus 1983; Kraus and Harikai 1983; Kraus, Hanson, and Harikai 1984; Hanson and Kraus 1986; Hanson 1987; Hansen and Kraus 1989; Gravens and Kraus 1989). The models assume that the profile shape remains constant and the representative shoreline is generally taken to be the mean water line. This type of model is particularly useful in evaluating the effects of structures such as groins or offshore breakwaters on an existing shoreline or beach fill project, the spreading response of a short beach fill beyond project boundaries, and the effectiveness of project fill transition. Shoreline change models include the following:

(a) GENESIS (Hanson and Kraus 1989) is a PC-based program which is capable of simulating long-term (1 to 100 months) spatial changes in longshore transport over beach lengths of 1 to 200 km (<1 to 125 miles). Based on the net volume of sand transported into each computational longshore grid, the program calculates the response of the shoreline to time-varying wave conditions. A wide variety of structures including groins, jetties, and detached breakwaters can be accommodated by the model. Required input parameters for the model include time and spatially varying wave conditions at the seaward boundary, shoreline geometry, structure geometry, and representative median grain size for the beach. Model output includes shoreline position and longshore transport rates for each computational grid. Use of the GENESIS model or the Shoreline Modeling System (SMS) is recommended for developing the final design. The SMS is the GENESIS model combined with a wave transformation model RCPWAVE.

(b) Dean and Yoo (1992) present a shoreline evolution model which is applicable to a range of applications where large perturbations, such as beach fill projects, are placed on a natural beach system. The model can be used to analyze the response of a variety of beach fill geometries to waves. For instance, the model is useful for estimating: the percentage of beach fill material which will remain within a limited beach fill project boundary, with no end structures on a relatively straight coastline at various years in the future; the effects of shore-perpendicular structures placed at the ends of the project; and the effects of nourishing with material more and less transportable than the native. This model is simpler to run than the GENESIS program, and should be useful for conducting planning analyses of beach fill life for a range of geometries. It will not handle the range of geometries of shorelines or structures as GENESIS, and cannot take into account variations in local wave

patterns (except by the recommended use of an ad hoc straightening of shoreline contours, explained in the model documentation). Required model input data include an effective average wave height, depth of closure, effective median grain sizes, berm height, background erosion rates, and initial shoreline and structure geometry. Output includes shoreline geometry as a function of time.

(3) Multi-contour line/schematic three-dimensional models. These models differ from the profile and shoreline change models above in that the assumptions of constant profile shape and constant longshore transport are relaxed. Simplifying assumptions are made to produce schematic 3-D models; for example, to restrict the shape of the profile. Perlin and Dean (1978) extended a version of the "2-contour line model" of Bakker (1968) to an n-contour line model in which depths were restricted to monotonically increase with distance offshore. Due to their complexities, these models are limited in their capabilities and require extensive computational resources. As a result, they are not widely applied in the design of coastal projects.

(4) Fully three-dimensional models. These models calculate waves, currents, sediment transport, and bottom elevation changes point by point in either finite difference or finite element grids placed over the area of interest. However, special expertise, powerful computers, extensive field data collection and extensive verification are required to effectively use these models (Vemulakonda et al. 1988). For these reasons and the high expense involved, applications of these models for prototype design have been limited.

d. Selection of the design tools and procedures. The selection of appropriate methodologies for beach fill planning, analysis, and design will be guided by a number of factors, which include the level of the study effort (Reconnaissance, Feasibility or Design), purpose of the project, and the existing conditions along the study area shoreline. Generally, during the reconnaissance phase of a study, the design tools and procedures are limited to experience, empirical guidance, and analytical methods; particularly since this phase of the study is limited to using available data for the area. However, as numerical models are made more efficient and easy to use, their use is recommended more frequently in this phase of a project study. During the feasibility phase of a project, more advanced tools and procedures such as numerical models are used in addition to experience, empirical guidance, and analytical methods. During the design phase of a project, the final design of the project recommended in the feasibility phase is conducted, which generally warrants the use of the most sophisticated and accurate numerical models, and in specialized cases involving complex structures and wave patterns, physical models.

4-3. Design Considerations

a. Storm surges. Reliable estimates of water level changes under storm conditions are essential for the planning and design of a beach fill project. Previous investigations (Brunn 1962; Edelman 1968, 1972; Dean 1977; Vellinga 1983) have indicated that storm surge is the single most important process in determining the response of a beach to a storm (see EM 1110-2-1414 and EM 1110-2-1412).

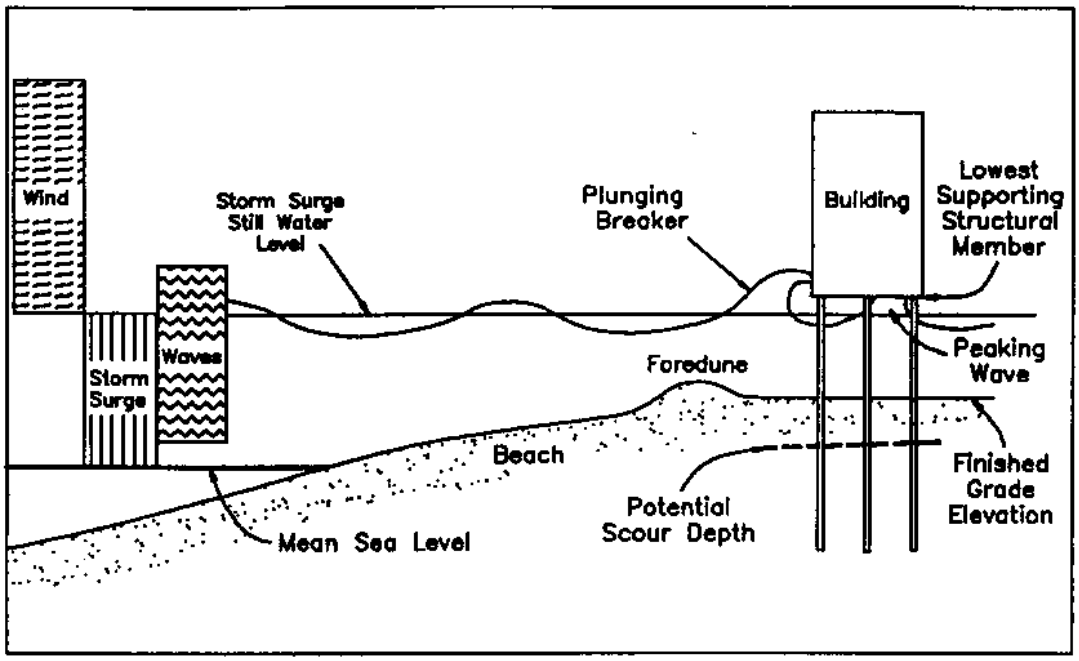
(1) A number of factors are responsible for changing water levels during the passage of a storm. These factors include astronomical tides, direct winds, atmospheric pressure differences, earth's rotation, rainfall, surface waves and associated wave setup, and storm motion. Figure 4-6 illustrates the various components of storm surges.

(2) The most significant effect of the storm surge is that the raised water level exposes the upper parts of the beach to erosion by direct wave attack. The storm surge allows the large waves to pass over offshore bar formations without breaking. When the waves finally break, the remaining width of the surf zone is not sufficient to dissipate the increased energy contained in the storm waves. The remaining energy is spent in erosion of the beach, berm, and dunes. The eroded material is transported offshore where it is deposited on the nearshore bottom to form an offshore bar as previously shown by Figure 2-6. This bar eventually grows large enough to break the incoming waves farther offshore, forcing the waves to spend their energy in the surf zone.

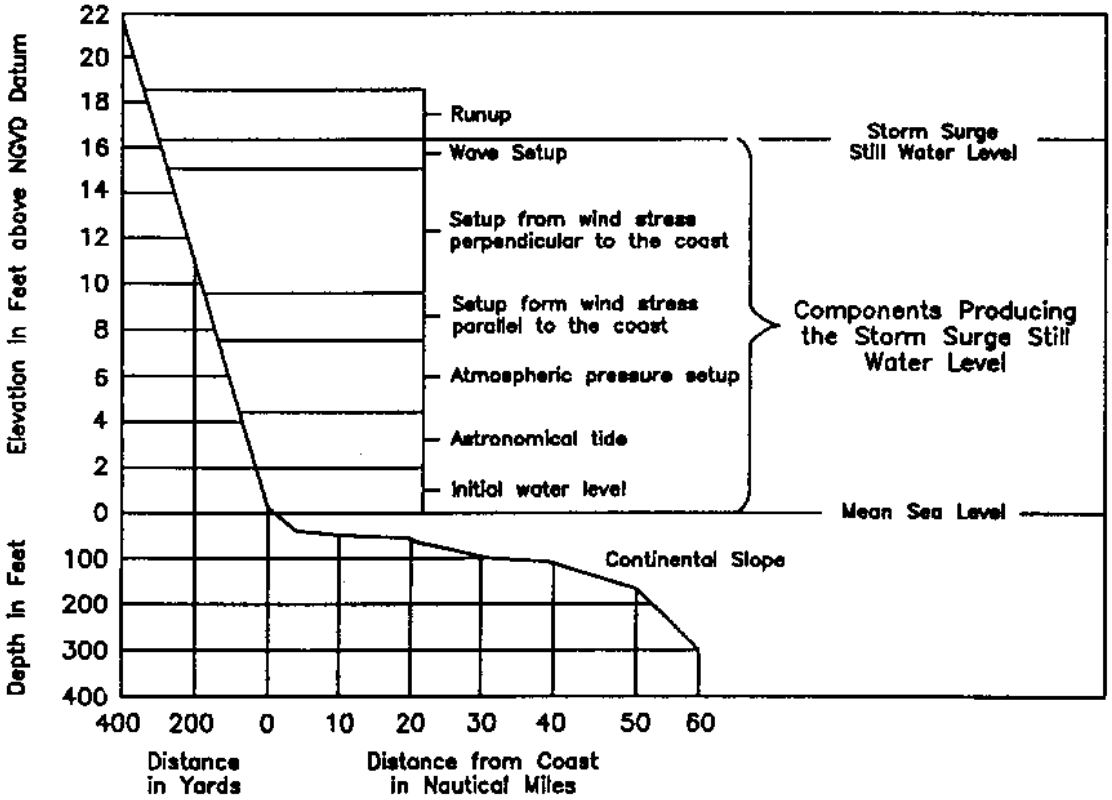
(3) Detailed descriptions of the process of storm surge generation and methods to predict the storm surge are presented in EM 1110-2-1412, Chapter 3, EM 1110-2-1414, and Chapter 3 of the *Shore Protection Manual* (1984).

b. Runup. Vertical height of wave runup above the still-water or storm surge level is important to beach fill design because of the consequences of this runup on the dune and structures behind the beach. Runup during extreme storm events has been observed to overtop and lower the crest height of sand dunes, thereby decreasing the protection provided by the dunes. In situations where the dune crest has been lowered and in areas without dunes, runup can result in direct impact forces on buildings, piers, and boardwalks. These forces can result in significant damage to these structures.

(1) Runup depends on the profile shape and slope, berm height, berm width, and wave number. Modifications of the



Cross–section of the littoral zone showing the three categories of hurricane generated forces and the vertical extent of influence.



Components of the storm surge still water level.

Figure 4-6. Storm surge components and associated water level magnitudes

beach profile, including the berm and dune, during a storm event must be considered. An estimate of the response of the design profile is necessary and determination of runup on this modified profile should be made. Various combinations of these parameters should be evaluated to determine the optimum combination that will limit the runup during storm events to an acceptable level. Determination of the acceptable level will require an evaluation to be made of the proximity of buildings and other structures to the wave runup zone.

(2) In some situations, it may not be possible, for economic or environmental reasons, to design the beach fill to limit the runup as required. The construction of bulkheads or seawalls on the landward side of the beach fill may then be considered.

(3) The procedure for estimating runup will depend upon the beach profile cross section and the wave conditions being considered. Breaking wave conditions are assumed in evaluating runup. Runup can be estimated by the methods described in the ACES Manual (Leenknecht et al. 1990). Holman (1986) examined wave runup maxima on natural beaches. Runup was found to be a function of Irribarren number. Resio (1987) presented a method for estimating maximum runup elevation on a natural sand beach during a storm.

c. Overtopping. A determination of the volume of water that will overtop the proposed height of the beach berm and dune, bulkheads, or seawalls is necessary to determine the interior flooding to be expected. This calculation will determine the amount of protection that will be provided by a proposed plan.

(1) Evaluations of overtopping must consider the overall volume and rate of water contributed to interior flooding during the course of the storm. During the initial stages of a storm, overtopping may be insignificant or nonexistent until the storm surge and wave conditions reach critical heights and the beach and dune profile is eroded. As the storm progresses and the beach is eroded, wave overtopping can be significant. The time-dependent nature of this process must be taken into account in order to evaluate the protection provided by a proposed beach fill plan. In some situations, it may not be possible to design a beach fill plan, even with bulkheads or seawalls included, that will eliminate interior flooding due to wave overtopping (Hanson and Kraus 1989).

(2) As discussed above, overtopping is a time-dependent process. Numerical or physical modeling may be required for an accurate evaluation such as that presented in Hanson and Kraus (1989) and Technical Report CERC-88-1 (Lillycrop, Pope, and Abel 1988). In cases where beach fills are backed by seawalls, Goda (1970) presented a procedure for estimating wave overtopping rates. ACES uses the method developed by Weggel (1976) for calculating wave overtopping rates for monochromatic waves. Ahrens (1977) developed a method for extending Weggel's procedure to irregular waves. Battjes (1974) and Owen (1980) proposed a method for calculating wave overtopping rates over smooth sloped structures. Douglass (1984) evaluated many of these methods and the SPM methods result in lower overtopping rates than the Battjes and Owen methods.

d. Potential dune breaching. The purpose of artificial sand dunes in a beach fill project is to provide a reservoir of sand for nourishing the beach during severe storms and to prevent high water and wave runup and overtopping from damaging backshore areas. During extreme events with storm parameters exceeding the design parameters, breaching of a dune system is possible. During hurricanes, the disappearance of sections of 18- to 30-m-wide dunes has been reported (*Shore Protection Manual* 1984). This can result in extensive coastal flooding, and beach and dune sediments can be swept landward by the water and lost to the dynamic beach system. In some cases, flooding from ocean-side storm surges and waves and return flow will erode enough sand to cut a new tidal inlet through a barrier island.

(1) The potential for dune breaching should be evaluated if dunes are included as part of a storm protection project. Methods for evaluating the potential for dune breaching are presented in Technical Report CERC-87-8 (Birkemeier et al. 1987).

(2) Dunes can be made much more resistant to erosion and breaching if suitable vegetation can be established on the dunes for an adequate length of time to establish an extensive root system. It generally takes 2 to 5 years for beach grass to establish a healthy root system, and up to 10 years before the maximum resistance to erosion and breaching is obtained. An active grass fertilization and maintenance program can greatly enhance the survival and effectiveness of beach grass (Chapter 6, *Shore Protection Manual* (1984)).

e. Longshore transport. A relatively accurate computation of the rate of longshore sediment transport is important in predicting the performance of a beach fill project and in predicting the frequency and quantity of periodic nourishment required. Along a shoreline area where more material is transported from the area than is transported into the area from adjacent shorelines, the frequency and quantities of nourishment will be high. An

example would be a beach section bordered by a jettied inlet on one end and some man-made littoral barrier on the other end. Depending on the wave climate at such a site, longshore losses may result in nourishment costs.

(1) There are a variety of methods available for estimating long-term average transport rates. It is recommended that a number of independent methods be compared to ensure that the estimates are reasonable. One way to predict longshore transport at a site is to adopt the best known rate from a nearby site, with modifications based on local conditions. This should only be done if the wave climate, beach orientation, and sand sizes are very similar, since longshore transport is very sensitive to these factors. Another way to predict longshore transport rates is to compute them from data showing historical changes in the shoreline location. This is especially suitable for beach fill projects, since it is the long-term average shoreline erosion which is of interest. A number of empirical and analytical methods for the calculation of longshore transport based on sediment characteristics and long-term wave data are presented in Chapter 4 of the *Shore Protection Manual* (1984).

f. Seaward limit of significant sediment transport. One of the factors required for fill quantity calculations and a number of analytical and numerical models is the closure depth, or depth of maximum sediment motion during a design level storm. This quantity varies based on wave climate and sediment characteristics. The most accurate method of determining the depth of closure is to examine historical profile measurements that bracket storms approaching design level intensity, which is discussed in Section 2-2. The depth at which no changes in the profile took place can be assumed to be the depth of closure, as previously discussed and illustrated by Figure 2-4. A number of profiles should be examined and the results averaged to obtain an accurate result.

(1) A limited field study may be warranted to sample variations in nearshore sediment characteristics, with interpretations as described in Chapter 4 of the *Shore Protection Manual* (1984). This method uses distinct variations in sediment characteristics (grain size, color, carbonate content) as one proceeds seaward to delineate the normal seaward limit of motion.

g. Design constraints. During the planning and design of a beach fill project, a number of design constraints may be encountered, including, but not limited to, the following:

(1) Line-of-sight interference due to dune or bulkhead construction. A tradeoff may have to be made between the heights that are acceptable to local interests and those required for storm and flood protection. Design analysis should clearly identify the consequences of such a decision.

(2) Availability of borrow material within an economical distance from the project area that is suitable from an engineering standpoint and is acceptable to local interests from an aesthetic point of view. The primary local concerns will include grain size being too large for comfortable beach recreation, stones mixed into beach fill, and fine silt and clay deposits which are periodically uncovered by wave action. Importation of sand material may have to be considered as well as means of extending the residence time of the fill material.

(3) Provision of adequate public beach access that does not compromise the protection provided by the project. This is especially important when the project incorporates vegetated dunes as part of the protection, since the vegetation must be protected from foot traffic while allowing public access. Design of dune walkovers and vehicle accesses may be required.

(4) Objections from local interests to disruptions in recreational activities and public safety concerns during initial construction and periodic nourishment activities.

(5) Objections to wind-blown sand from the beach and dune on shore-front property.

(6) Objections from environmental groups or agencies. Dredging windows will likely be dictated by environmental concerns, and frequently project design must accommodate the preservation of critical habitat areas.

4-4. Design Elements

a. Berm elevation and width. Natural beach berms are formed by the deposit of material by wave action. The height of a berm is related to water level fluctuations, normal foreshore and nearshore slopes, and wave climate. Some beaches have a lower berm, which is formed by the uprush of normal wave action during the ordinary range of water level fluctuations. A higher berm, or storm berm, may also exist, which is formed by wave action during higher water levels.

(1) The degree of protection to the backshore depends greatly on the width and height of the berm. If a beach fill is placed to a height lower than the natural berm crest, a ridge will form along the crest of the berm and high water may overtop the berm crest causing ponding and temporary flooding of the backshore area. To prevent this flooding, the berm elevation should be designed to equal the elevation of the natural berm. If additional protection is required to

prevent significant wave runup and overtopping of the backshore, multiple berms can be designed to some storm berm elevation. Techniques to estimate the height and width of the berm for design purposes include evaluating the performance of the natural berm during past storm events, comparing the beach profile at the site with beach profiles at sites of similar exposure characteristics (waves and tides) and similar size beach material, and calculating the wave runup and overtopping rates for various berm geometries using the techniques described in Section 4-3. If multiple berm configurations are included, the seaward berm should be constructed to the natural berm height, to prevent scarping, and the landward berm(s) to some storm berm elevation.

(2) If the purpose of the fill is to restore an eroded beach to protect backshore improvements from major storm damage, the width may be determined as the protective width which has been lost during storms of record. It is recommended that this width be compared to the width calculated by the use of beach erosion/profile change models as described below.

(3) Where the beach fill serves as a stockpile to be periodically replenished, the berm should be wide enough to accommodate the recession expected during the intervals between nourishment operations. This can be estimated from long-term erosion rates if the fill material is similar to the native material, or it can be calculated based on longshore transport rates adjusted for the difference in grain size between the native and the fill material or use of numerical models such as GENESIS.

(4) The recommended design approach to determine berm dimensions for storm and flood protection projects involves the application of a profile response model, such as SBEACH, to evaluate the effects of storms on various berm configurations. Using this approach, a large number of berm widths and dune elevations may be tested with a range of storm parameters ranging from conditions representing 5- to 500-year return period events. This provides a more quantitative analysis of the performance of the proposed berm. The relative effectiveness of berm elevation versus berm width for a given volume of sand can also be tested.

b. Wave adjusted design profile. The initial slope of any beach fill will typically be steeper than that of the natural profile over which it is placed due to the limitations of placement techniques. The subsequent behavior of the fill depends on the characteristics of the fill material and the nature of the wave climate. Design profiles are generally used for computing fill requirements since wave action will shape the profile into an equilibrium shape which depends on the grain size distribution, water level, and the wave climate (Figure 4-7). Often the design profile is assumed to be identical in shape to the pre-fill profile if grain size of the fill material is similar to that of the native beach material (Delft Hydraulics Laboratory 1986). If the fill

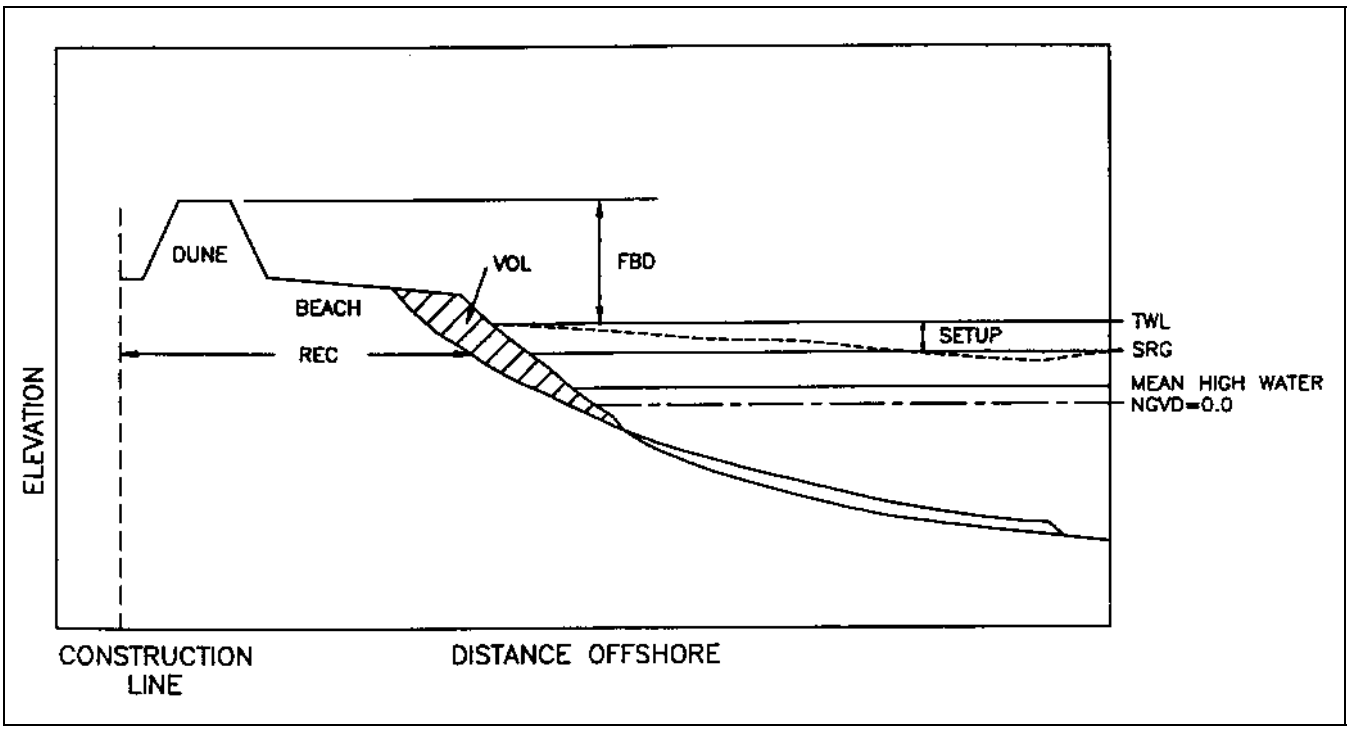


Figure 4-7. Schematic showing construction profile response related to water level

material is finer than the native material, it is generally assumed that wave sorting and winnowing will cause the loss and offshore transport of the finer fraction of the fill material. Therefore the fill quantity must be increased to account for the loss of fines, as discussed above in Section 4-2. An alternative methodology is to assume that the finer fill will reach a flatter equilibrium slope than the native beach material (Dean 1991), which will again require the placement of larger quantities of fill material than if the fill material were the same size as the native material. The required placement volume from each of the approaches can be determined and compared as part of the design process.

c. Dune dimensions. Sand dunes may be an important protective feature in a beach fill project, as they prevent storm tides and the associated wave runup and overtopping from directly damaging oceanfront structures and flooding interior areas.

(1) To be most effective, the crest height of the dune should be at or above the limit of wave runup for the design storm in the project area and should be able to withstand the design storm event without eroding completely. The numerical models discussed in Section 4-2 should be used to evaluate the response of dunes to storm conditions. However, project economic feasibility may limit the dune crest elevation and width. As a result, overtopping of the dune will be anticipated. The effects of the overtopping on inland flooding should be evaluated to determine the acceptable dune elevation. A method for selecting optimum dune height is presented by Ulrich (1993).

(2) Sand dunes also serve as stockpiles to feed the beach. During a storm, the initial attack of storm waves is on the beach berm fronting the dune followed by waves attacking the dune when the berm is eroded or overtopped. If the duration of the wave attack is long enough, the waves can erode the dune, lowering the dune crest. Typically, significant amounts of sand eroded from the berm and dune are transported directly offshore and deposited in a bar formation as shown by Figure 4-8 (Stauble and Grosskopf 1993).

This process helps to dissipate incident wave energy during a storm. Ideally, the amount of sand provided should be sufficient to allow storm bar formation without total erosion of the dune. Profile response models such as SBEACH are the best method of evaluating the dune/berm/bar response to storm conditions. Model calibration and verification will be especially important if accurate simulation of the dune erosion with vegetated dunes is required, since this will vary greatly with the age, health and extent of root mass of the vegetation. Volumes of sand eroded from beaches and dune systems have been estimated to be as great as 1 m^3/m (23 yd^3/ft) of beach (Everts 1973). A discussion of beach and dune erosion during severe storms is presented in Hughes and Chiu (1981). Use of storm response profiles to design emergency fills is described by Grosskopf and Behnke (1993) and is presented in Figure 4-9.

d. Transitions at boundaries. Termination of the beach fill section at the project boundaries can be accomplished using hard structures, usually referred to as terminal structures, which include groins, jetties, and breakwaters or by filling transition zones at the terminal ends of the beach fill. Careful attention should be paid to the termination of the beach fill since it may be important to provide the desired level of protection to the project boundary, yet expensive to provide transition fill onto adjacent beaches. While often approached as an afterthought to the project design, the beach-fill boundaries deserve careful design consideration.

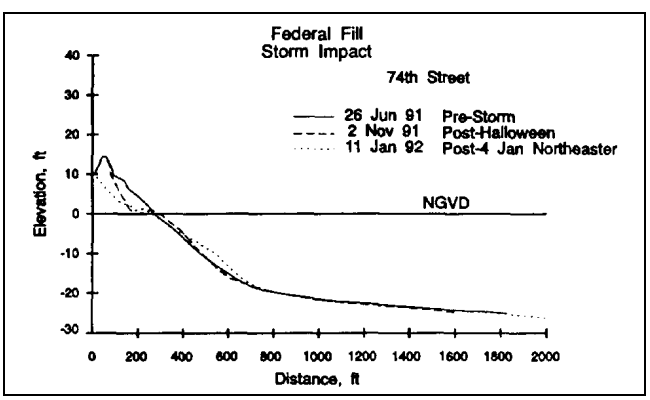


Figure 4-8. Representative pre- and post-storm profile readjustment from a site at Ocean City, MD (from Stauble and Grosskopf (1993))

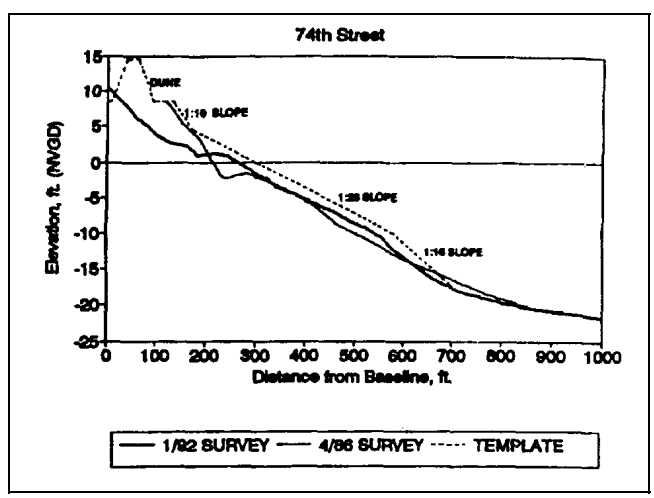


Figure 4-9. Storm response profile used for the design of an emergency fill

(1) Hard structures will allow an abrupt termination of the beach fill section. However, these structures are costly and can interfere with the natural longshore transport of sediment along the shoreline. This interference could result in adverse effects along the unrestored beach and subsequent objections by adjacent property owners. Techniques to minimize effects on adjacent beach areas are presented in EM 1110-2-1617.

(2) In addition, longshore transport may result in end losses to a beach fill project since the fill is essentially a perturbation to the shoreline, which is out of equilibrium with the natural shoreline geometry, and the longshore forces tend to restore equilibrium by spreading the sand in the alongshore direction. In this case the local transport rate may be significantly higher than the average rate over the project as a whole, due to the different local shoreline orientation to the waves and may be a dominant process in fill loss. Dean and Yoo (1993) have found that sediment transport in the vicinity of beach fill projects occurs in three different phases (Figure 4-10). Their approach, discussed briefly in Section 4-2, provides a method for estimating the percentage of beach fill material which will remain on a relatively straight coastline with no end structures for various renourishment intervals. The approach provides an equation for beach planform evolution which combines sand conservation with sediment transport processes and is expressed in terms of the fraction of sediment remaining in the placement area as a function of time:

$$M(t) = \frac{1}{J\sqrt{\pi}}\left(e^{-J^2}-1\right) + erf(J) \tag{4-7}$$

where

$$J = \frac{L_f}{2\sqrt{Gt}} \tag{4-8}$$

$erf(J)$ = an error function of J
t = time (seconds)
L_f = length of the fill

and

$$G = \frac{\left(0.1h_b^{5/2}\ g^{1/2}\right)}{(H + B)} \tag{4-9}$$

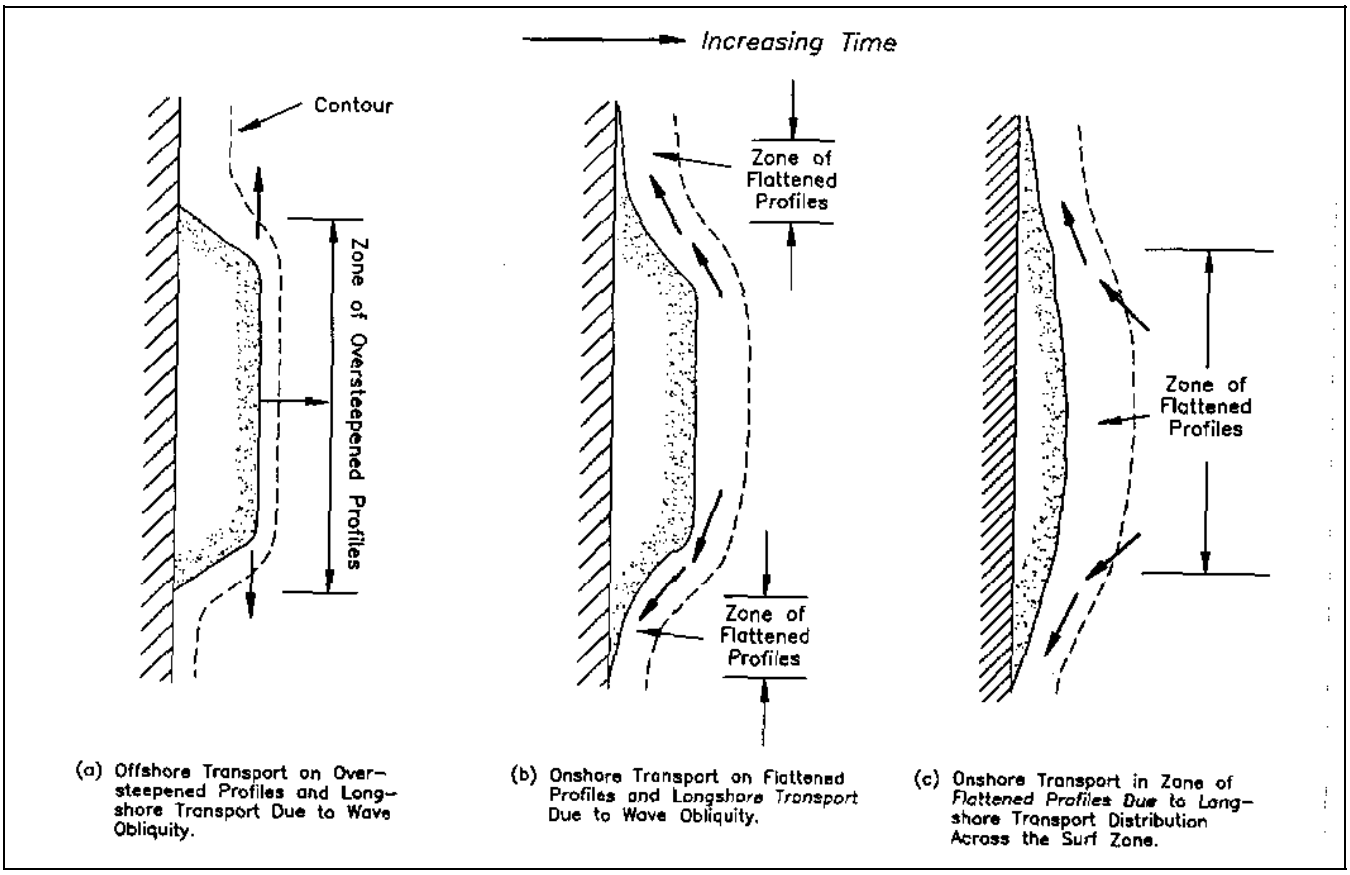


Figure 4-10. Different phases of sediment transport in the vicinity of beach nourishment projects (from Dean and Yoo (1993))

where the variables have been previously defined. *Erf(J)* can be obtained from various mathematical texts containing error functions and tables. An approximate relationship can be used when $1/J$ is less than 1.0 (causing *M(t)* to be between 1.0 and 0.5) and is expressed as:

$$M(t) = 1 - \frac{1}{J\sqrt{\pi}} \tag{4-10}$$

(a) Consider a beach fill project to be placed on a straight beach with no end structures and a placement length of 10,000 m (33,000 ft) with a breaker height (h_b) of 1.0 m (3.3 ft), closure depth (H) of 6.0 m (19.6 ft), a berm height (B) of 1.5 m (4.9 ft), and a renourishment interval (t) of 5 years. The first step is to determine if Equation 4-10 is valid by using Equations 4-8 and 4-9 to calculate the value of $1/J$. In this case, $G = 0.042$, $J = 1.96$, and $1/J = 0.51$, satisfying the conditions validating Equation 4-10. Using Equation 4-10, $M(t) = 0.71$, meaning that after 5 years, 71 percent of the fill material will remain within the fill length boundaries.

(b) Mitigation of fill end losses can be accomplished by extending the lateral ends of fill and tapering the fill ends at a smooth angle in relation to the pre-fill shoreline. Tapering the fill ends will decrease the perturbation effects and minimize the angle of shoreline orientation in relation to approaching wave direction. Optimum tapering lengths of fill end boundaries can be determined with the aid of shoreline response models such as GENESIS.

(3) Filled transition sections are subject to accelerated losses due to the difference in orientation of the transition section with respect to the natural shoreline. These losses are a result of the larger angle between the transition section shoreline and the nearshore wave crest in the project area. Increases in longshore transport rates occur along these sections since littoral transport is directly related to this angle. Alternative combinations of transition angle and length should be evaluated to determine the most cost-effective design for the project. Procedures for determining longshore transport rates along beach segments with varied transition angles are presented in Chapter 4, Section V,3 of the *Shore Protection Manual* (1984). The shoreline response model of Dean (1991) provides an alternative method of estimating the response of a beach fill termination to long-term average wave conditions. More complex beach fill transitions can be analyzed using the GENESIS shoreline response model (Hanson and Kraus 1993).

(4) Costs of the transition sections over the project life should be compared to compartmenting the beach fill material with groins or jetties and the most cost-effective approach should be selected. However, environmental concerns, land ownership constraints, or other factors may determine the selection of the optimum transition.

e. Ancillary structures. In the majority of cases, renourishment of the beach fill section is required to maintain the project dimensions during the project life. Structures such as groins, jetties, and offshore breakwaters can be incorporated in a beach fill project to reduce the nourishment requirements.

(1) During the plan formulation phase of the project, alternative plans should be evaluated to determine if the incorporation of structures and reduction in periodic nourishment requirements is more cost-effective than designing the beach fill project without structures.

(2) Techniques for design of a groin system to reduce beach fill losses and offshore breakwater design are presented in EM 1110-2-1617 and the *Shore Protection Manual* (1984). Additional guidance for the design of offshore breakwaters for shoreline stabilization is presented in Dally and Pope (1986) and Chasten et al. (1993). Numerical shoreline change models such as GENESIS and that of Dean (1991) are available to comparatively evaluate the performance of beach fills with and without structures such as groins and breakwaters. Hanson and Kraus (1989) present details of the use and applicability of the GENESIS numerical shoreline change model.

f. Advanced nourishment requirements. The majority of beach fill projects include nourishment to maintain the dimensions of the beach fill that were selected for construction. These beach dimensions and the resulting project performance are factored into the economic analysis of the project. In order to ensure that these design dimensions are maintained until the first periodic nourishment event occurs, advanced nourishment of the beach fill is usually incorporated during the initial beach fill operation. Advanced nourishment usually consists of placing an additional amount of beach fill (Figure 4-11) to offset the expected losses from the time of completion of the project to the first scheduled nourishment event.

(1) Historical erosion rates can be used to estimate the expected annual losses. Adjustment of the historical erosion losses may be necessary to account for differences between the beach fill and the native beach material on which the historical losses are based. Both the overfill factor R_A and the renourishment factor R_J may have to be used. In the absence of historical erosion rate data, the net longshore sediment transport rate in the project area can be used to estimate the expected annual loss of beach fill from the project. Another source of annual fill loss that must be

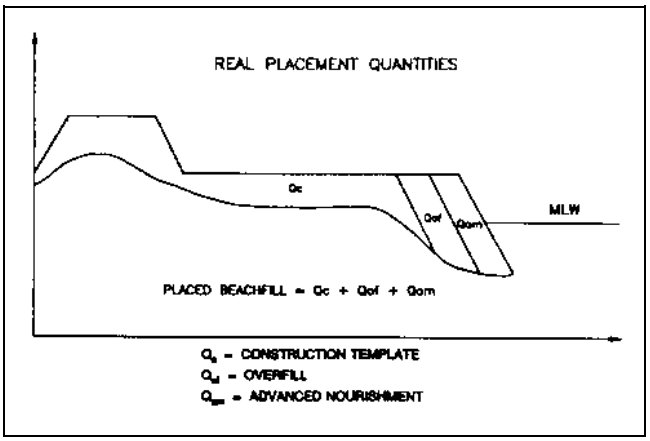


Figure 4-11. Advanced fill volume in addition to the designed placement volume is sometimes necessary to achieve the total construction volume

considered is end losses due to shoreline perturbations caused by the fill area itself (Dean and Yoo 1993). If erosion rates have been significantly influenced by existing structures such as bulkheads, revetments, seawalls, and groins, or a substantially coarser fill material is proposed, numerical models such as GENESIS may be required to estimate future beach fill losses. It should be noted that potential littoral transport rates in the project area may exceed existing transport rates due to the presence of man-made structures or nearshore hard bottoms. The sediment transport potential will remain constant following fill construction. However, availability of the additional material could result in higher rates of transport out of the project area than previously estimated, which could result in higher nourishment requirements than that indicated by historical shoreline behavior.

(2) The frequency of future nourishment can be determined from economic considerations, which include construction mobilization costs and economics of beach fill scale, and local interest considerations, which generally do not wish to be faced with the disruptions of construction if the beach is used for recreation. The Ocean City, MD, beach fill project used a 4-year advanced nourishment and renourishment schedule (Fulford and Grosskopf 1988). The renourishment interval should take into consideration the higher losses expected with advancing the shoreline further seaward. Shoreline advance causes perturbations in the natural shoreline geometry, resulting in end losses and the increased risk of severe storm losses inherent during longer intervals. Actual intervals will be determined by the occurrence or lack of storm events as well as other climatic factors.

(3) The advanced nourishment volume is added to the placement volume to obtain the total construction volume for the project, and the construction template is designed to ensure that the total volume is placed on the beach.

g. Construction template. Construction slopes are seldom the same as design slopes because of the working limitations of equipment used to place and shape the fill, and because the selective sorting of the fill by waves and currents will naturally shape the profile to an environmentally equilibrated form after placement. Two construction approaches are generally used. One is to overbuild the upper part of the beach and the other approach is to create an initial construction profile that extends significantly offshore.

(1) The “overbuilding” method places the required fill volume onshore in a construction template with the beach berm at the design elevation but with a berm width greater than the design berm width and fill slope that is steeper than the equilibrium slope on the seaward side (Figure 4-12). Dimensions of the construction template berm width are generally adjusted to provide the required construction volume. Part of the fill is often placed underwater, as determined by the fill’s berm width and seaward slope. Readjustment of the fill sediments into a more equilibrated profile shape is accomplished almost entirely by waves and currents that erode and redistribute the placed fill. Scarping is one problem that may be encountered in the overbuilding approach. Steep scarps may develop at the toe of the fill as waves begin the readjustment, and these scarps may make access to the beach difficult. Scarping can be controlled by decreasing the berm elevation for the construction profile while extending the berm width seaward, or they can be mechanically smoothed as part of the construction contract or regular beach maintenance and cleaning.

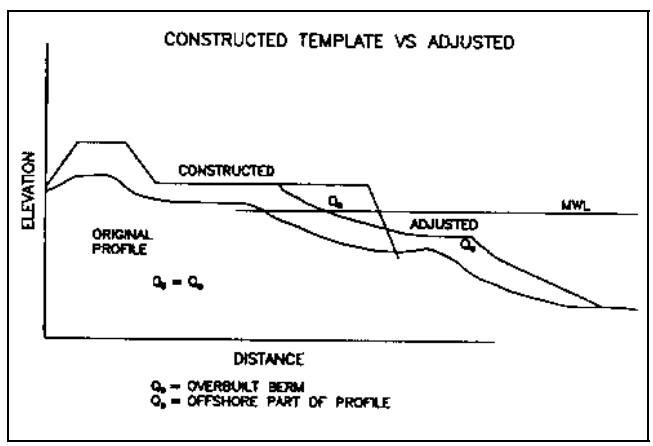


Figure 4-12. The overbuilding method of fill placement used to achieve required fill volume onshore, creating a wider berm and steeper slope

(2) The second approach is to initially place more of the fill offshore. Redistribution of the sediment across the profile by waves and currents will still take place after construction to equilibrate the profile shape, but much of the reworking will occur offshore of the fill rather than onshore. This method consists of specifying the total construction volume required in the profile, and a general placement profile, but doesn't require precise placement to construction template grade lines. Care must be taken to ensure that the volume is not placed too low on the profile where it will not be effective in storm protection. Using this construction approach, the contractor's payment is dependent on the amount of material placed on the beach to the offshore depth where the design slope meets the existing bottom. This approach provides the contractor with an incentive to minimize his fill losses.

(3) Both construction approaches result in an onshore fill section that is placed to a desired berm elevation and width and has steep initial slopes. This onshore fill eventually adjusts to a natural slope and narrows the berm, leaving the impression that much of the fill has been lost, although it has only moved offshore to reestablish a stable profile. During the planning and design of a beach fill project, a public involvement program should be conducted to educate the public regarding the natural and expected evolution of a beach fill. More protection may be realized during the evolution process by having more material higher up on the beach.

h. Volume of fill. Following the determination of the dimensions of the beach and dune for storm protection, the total quantity of fill required can be determined. The primary considerations are: (1) the design profile for the area, (2) characteristics of the borrow material compared to the native beach material, (3) the required overfill ratio (R_A); and (4) advanced nourishment and overbuilding requirements. The first consideration involves determining the design profile and required berm width, as discussed previously, and then determining the volume of fill required to achieve that profile. The volume of fill required can then be calculated using the translation method or Dean's method presented in Section 4-2. It is recommended that both methods be used and compared. When comparing these two methods, the translation method must be adjusted using the overfill ratio (R_A) to account for possible differences between the fill and native beach materials. If an artificial dune is included in the design, the volume of material required to achieve the design dune dimensions must be added to the design profile volume to determine the construction profile volume. An advanced nourishment fill volume may be added to the placement volume to obtain the total construction volume, as shown in Figure 4-11. The total construction volume for a beach fill project can be represented as:

$$V_T = (Q_c + Q_{of} + Q_{am}) L_s \tag{4-11}$$

where

V_T = total of placed fill material (cubic meters (cubic feet))
Q_d = quantity from design template (cubic meters/meter (cubic feet/feet))
Q_{of} = quantity from overfill adjustment (cubic meters/meter (cubic feet/feet))
Q_a = quantity from advanced nourishment (cubic meters/meter (cubic feet/feet))
L_s = length of shoreline reach (meters (feet))

The overfill adjustment is only necessary when determining volumes for the design nourished profile using the translation method.

(1) If the project area exhibits longshore variability, it may be necessary to subdivide the project area into segments or reaches. All volume determinations should be applied to each reach of shoreline considering the design conditions for each reach. Equation 4-11 can then be applied for each reach of shoreline and the volumes summed to determine the total construction volume.

i. Periodic nourishment. Following the initial beach fill placement, periodic nourishment of the beach will be required to maintain the project dimensions.

(1) The need for renourishment will be determined by the long-term average shoreline retreat or longshore transport rate calculations. It should be recognized by the designer that year-to-year erosion rates can vary greatly from the long-term average rates, and may be significantly influenced by the occurrence of major storms. Therefore, while an average nourishment interval can be estimated, the actual required interval will vary depending on beach conditions dictated by previous climatic conditions. If renourishment is being considered as part of the project design, the advanced nourishment quantities should be included in the total construction fill volume. Project monitoring is important in order to account for unusual conditions during the project life. These facts should be communicated to managers involved with the project and to the public. Provisions to accomplish the required monitoring and initiation of nourishment preparations should be addressed in the project O&M Manual.

(2) The renourishment factor proposed by James (1975) can be used to determine the volume of renourishment which will be required if a borrow source is selected that is texturally different from the native beach sand. With this approach, different sediment sizes will have different residence times within the dynamic beach system. Coarse particles will generally pass more slowly through the system than finer sizes. The overfill ratio R_A should be determined and applied to the periodic nourishment material to determine the total construction volume.

(3) A monitoring program should be conducted which includes periodic surveys of the beach fill area. Analysis of these surveys will indicate when nourishment is required to maintain or reestablish project dimensions and the actual loss rate of the beach fill from the project. These surveys will allow a more accurate evaluation of the future periodic nourishment requirements.

(4) Increases in the quantity of periodic nourishment required due to projected historic relative sea level rise should be considered during the design phase of a project. Typical beach fill projects with a 50-year performance evaluation period should therefore consider these effects on periodic nourishment quantities. Sources capable of providing periodic nourishment during the projected performance period should be identified.

(5) Consideration should also be given to bypassing sand across tidal inlets from accreted areas at updrift jetties and from ebb and flood deltas at inlets. Likewise, back-passing of sand from a terminal downdrift jetty to an updrift beach fill project should be evaluated as an efficient sand recycling measure. Different types of sand transfer systems can be seen in Figure 4-13 and are discussed in EM 1110-2-1616. The effect of these measures on adjacent beaches must be evaluated.

(6) Some designers look on the advanced nourishment as a buffer for the design section against long-term erosion and more frequent storm events, the idea being to ensure that the full design section is available during extreme storm events. In such cases, more frequent storm events are used to design the nourishment quantity and interval. For instance, a 3-year interval might be sized to withstand a 5-year average return period stage event or some other event having more or less certainty of occurrence during the interval.

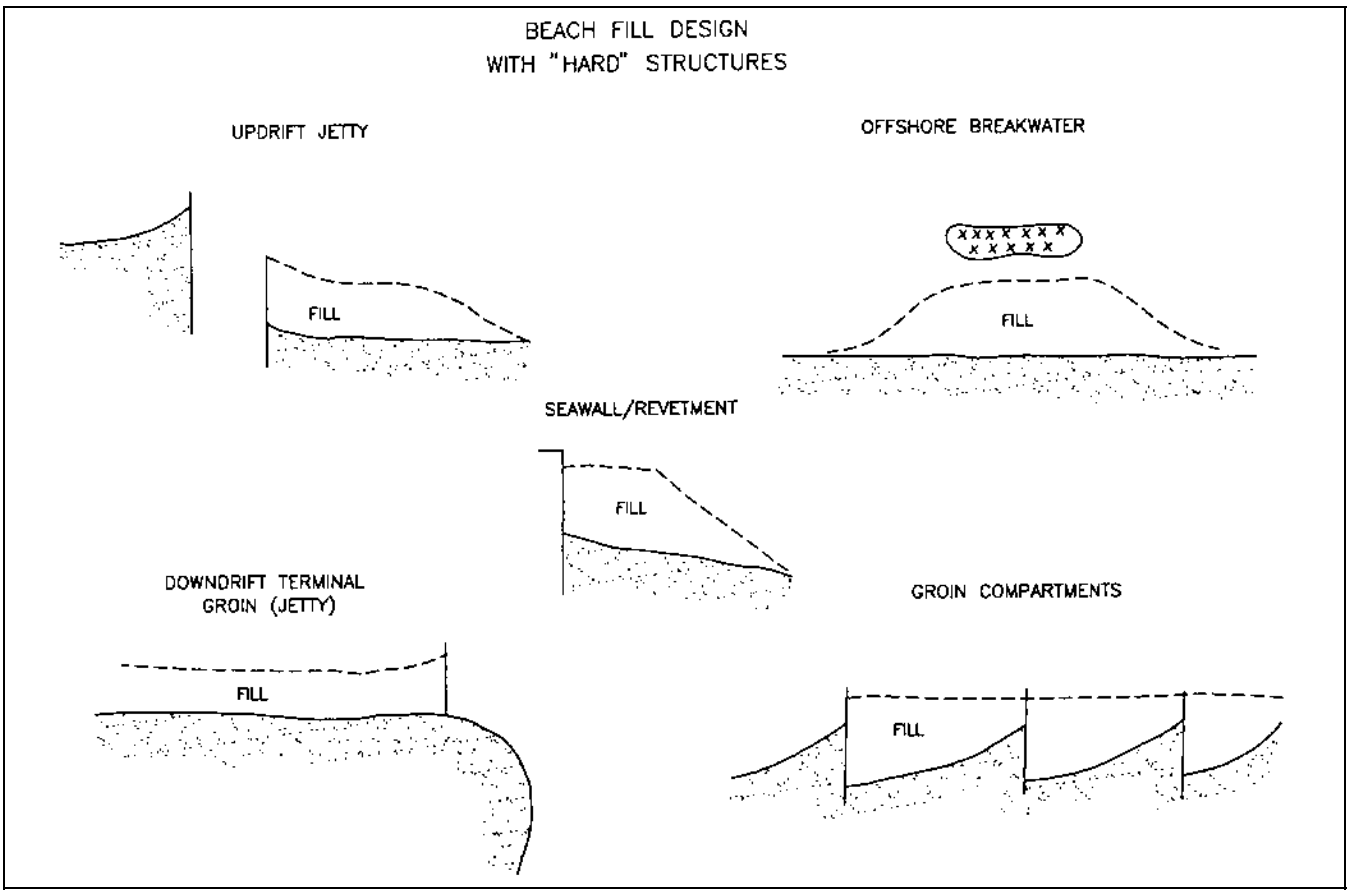


Figure 4-13. Types of littoral barriers where sand transfer systems have been successfully employed