Guidance for Flood Risk Analysis and Mapping

Coastal Erosion

November 2015

FEMA

Requirements for the Federal Emergency Management Agency (FEMA) Risk Mapping, Assessment, and Planning (Risk MAP) Program are specified separately by statute, regulation, or FEMA policy (primarily the Standards for Flood Risk Analysis and Mapping). This document provides guidance to support the requirements and recommends approaches for effective and efficient implementation. Alternate approaches that comply with all requirements are acceptable.

For more information, please visit the FEMA Guidelines and Standards for Flood Risk Analysis and Mapping webpage (www.fema.gov/guidelines-and-standards-flood-risk-analysis-and-mapping). Copies of the Standards for Flood Risk Analysis and Mapping policy, related guidance, technical references, and other information about the guidelines and standards development process are all available here. You can also search directly by document title at www.fema.gov/library.

Document History

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1.0 Overview

Erosion processes and consequences of erosion can either be “episodic” or “chronic.” These two descriptors assign a very important temporal component to erosion processes and their results. *Episodic erosion*, also referred to as storm-induced erosion, is predominantly the cross-shore movement of sand and sediment that results from short duration, high intensity meteorological and oceanic storm events. This type of event response results in shoreline adjustment and occurs during a single storm or during a series of closely spaced storm events within a storm season. Shore and backshore profile changes during intense storms can result in dramatic beach and dune erosion, retreat, breaching, or removal of backshore dunes; can cause retreat and collapse of bluff and cliff formations; and can culminate in greater landward encroachment of waves and flooding from the ocean. *Chronic erosion (or accretion)* is associated with slow, long-term processes such as gradual shoreline change associated with: (1) sea-level rise, (2) land subsidence, (3) changes in sediment supply due to watershed modifications, coastal structures, development, and (4) decadal adjustments in rainfall, runoff, and wave climate associated with global warming.

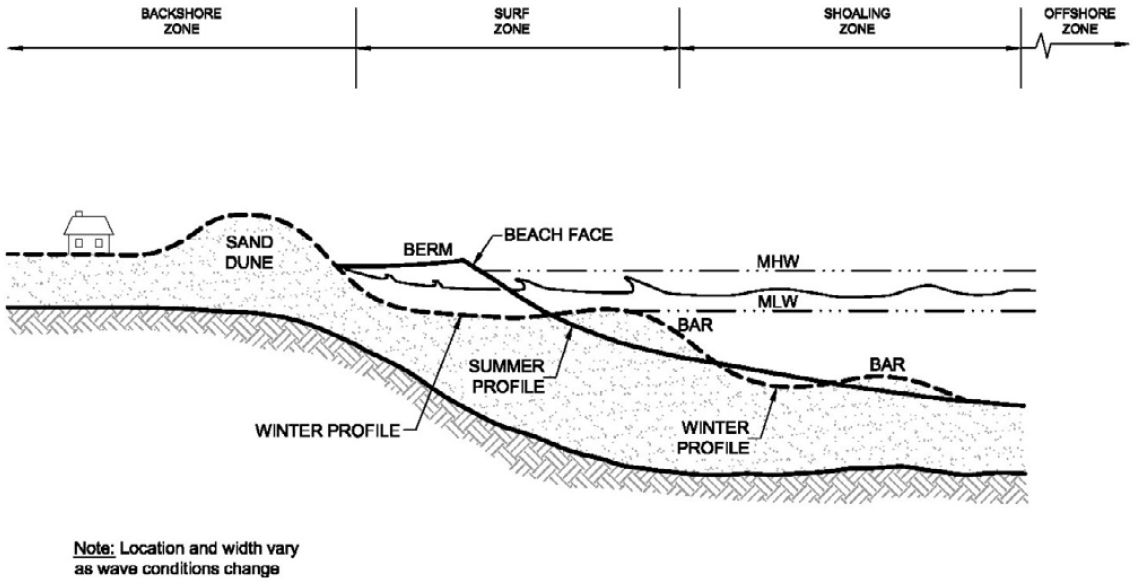
Current FEMA regulations are limited to risks and losses occurring as the direct result of a storm event. The NFIP does not address long-term chronic erosion, but focuses on episodic, flood-related erosion due to coastal storm events.[1] FEMA does not currently map long-term erosion hazard areas as some local or State agencies do. Flood Insurance Rate Maps (FIRMs) indicate risks from flooding hazards in the form of Base Flood Elevations (BFEs) and flood insurance risk zones. Therefore, flood assessment guidelines only include methods for estimating eroded shore and backshore profiles during storm events; the resulting profiles are then used in overland wave propagation, wave runup, and overtopping computations to determine flood risks associated with these events.

Erodible beaches and coastlines undergo typical seasonal changes in profile and location from summer to winter conditions. During winter months, increased total water levels, along with high-energy, steep waves, tend to move sediment offshore. Throughout the summer and early fall, during months of calm seas, the shoreline recovers as sediment moves back onshore. Figure 1-1 provides a sketch of generalized seasonal beach profile changes that occur along sandy shorelines exposed to seasonal high energy wave climates.

To estimate erosion and profile changes for a specific coastal setting, it may be important to consider during which season the potential flooding hazard event will likely occur and the condition of the associated beach profile. Many sandy beaches exhibit significant seasonal changes in their profiles due to seasonal differences in weather and wave climate. Where significant storms occur during the winter it may be appropriate for the Mapping Partner to apply a winter profile.

[1] Discussions of long-term erosion and the potential consequences of chronic erosion are found in materials listed in the reference section of this document and in many of the support documents referenced herein.

Figure 1-1. Typical Coastal Summer and Winter Beach Profiles (after Bascom, 1964)



By their nature, coastlines are extremely complex and dynamic environments. The type and magnitude of coastal erosion are closely related to coastal exposure and beach setting. *Coastal exposure* refers to: (1) whether the coastline and beach are situated on the open coast, e.g., exposed to the undiminished waves, water levels, tides, winds, and currents associated with the open coast, or (2) whether the coastline is located within a sheltered area that is fully or partially protected from the direct action of ocean waves, winds, tides, water levels, and currents. The latter exposure is referred to as *sheltered water*.

Erosion processes resulting from changes in total water level and wave action are similar along the open coast and within sheltered water areas; however, the magnitude, rate, and ultimate beach response may be quite different for sheltered water areas due to dramatic differences in total water-level changes and wave energy during large storms. Sheltered water areas typically have reduced wave energy and smaller runup. Some sheltered water areas found in confined embayments or estuaries may, however, experience higher still water elevations resulting from the combined effects of astronomical tides and fresh water runoff from streams and rivers and modified tidal and surge conditions. Treatment of erosion in sheltered water areas is discussed in Section 2.2.

1.1 Beach and Shoreline Settings

Beach setting refers to localized geomorphic characteristics of the shore and backshore zone related to site-specific geology, profile shape, material composition, and material erodibility; proximity to other dominant features such as coastal inlets, storm outfalls, streams, and creeks; harbors and coastal structures; littoral sediment supply; pocket beaches; and seasonal changes in beach width due to changes in wave direction. Presented here are common coastal shoreline

settings that can be used to describe much of the shorelines of the United States. The main erosion-related factors affecting all coastal profiles during storms events are:

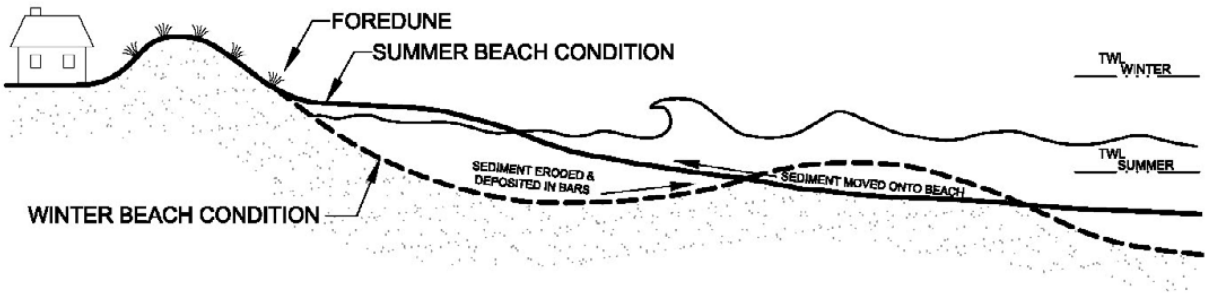
- Antecedent conditions of the beach and backshore (elevation and geometry of the nearshore, foreshore and backshore portions the coastal profile susceptible to erosion) before the occurrence of the specified storm event;
- The magnitude and duration of incipient waves and water levels associated with the 1-percent-annual-chance event
- Grain size, cohesion, and erodibility of materials present along the shoreline

To estimate profile changes for erodible shorelines, Mapping Partners need erosion-assessment methods that account for the unique morphologies of the setting and the general effects of the above processes. At many sites, historical evidence may be available regarding the extent of flooding and erosion resulting from an extreme event comparable to the 1-percent-annual-chance event; if so, erosion treatment giving results consistent with historical records should be applied. In the absence of historical data, various tools and methodologies are available for performing erosion, not all of which are applicable to all settings and the Mapping Partner should select the most appropriate approach.

1.1.1 Sandy Beach Backed by High Sand Dune:

Figure 1.1-1 provides a sketch of a typical beach profile for sandy beaches backed by high sand dunes; these are a common setting in all areas of the country. The profile shape is in a constant state flux, adjusting with the changing wave environment. In this environment the back berm is elevated high enough to prohibit frequent inundation by wave energy, permitting wind swept sediments to form a dune.

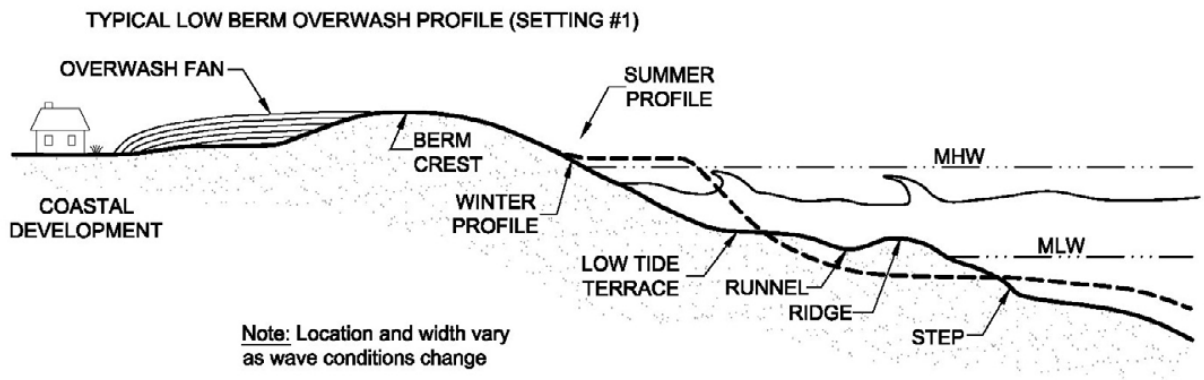
Figure 1.1-1. Sand Beach Backed by High Sand Dune (Beach Setting No. 1) (after Griggs, 1985)



1.1.2 Sandy Beach Backed by Low Sand Dune Berm:

Sandy beaches backed by low sand dunes are also a common setting; where the profile shape is in a constant state flux, adjusting with the changing wave environment. In these coastal settings, however, there is insufficient room or back berm elevation to allow for dune formation.

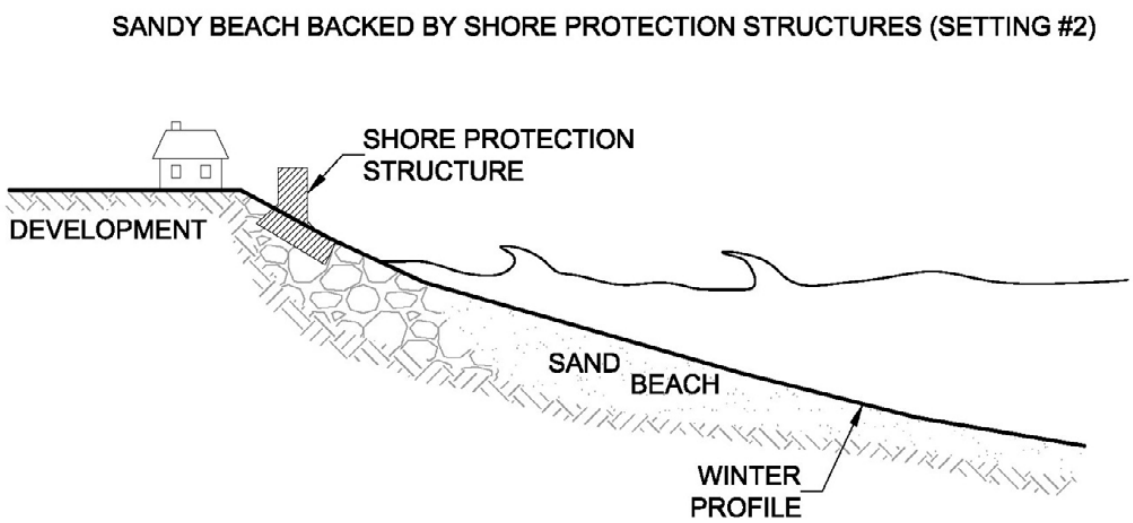
Figure 1.1-2. Sand Beach Backed by Low Sand Berm (Beach Setting No. 1) (after Bascom, 1964)



1.1.3 Sandy Beach Backed by Shore Protection Structure:

These environments are areas where a sandy beach naturally exists but the profile has been altered by human developments.

Figure 1.1-3. Sand Beach Backed by Shore Protection Structures (Beach Setting No. 2)

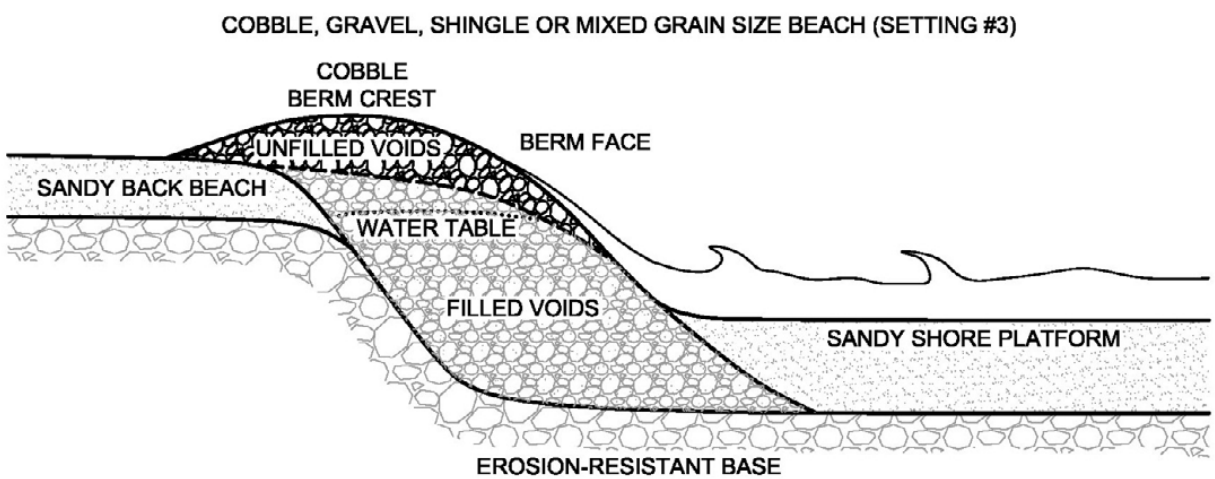


1.1.4 Mixed Grain Size Beach

Beaches armored with cobbles, gravel, or other coarse sediments develop in two distinct coastal environments. Often, these mixed-sediment beaches are prevalent in areas with slowly eroding bluffs that provide coarse sediment to the coastal system. In particular, mixed-sediment beaches are typically found along the shores of relatively sheltered bodies of water, where development of sandy beaches is inhibited by the absence of significant wind and wave action and by limited amounts of erodible sand. The other environment in which mixed-sediment beaches develop is one in which the coastline is exposed to high energy wave action, and, as a result, the finer sediments are winnowed away. Consideration of the wind and wave action to which the beach is exposed is necessary to determine whether the cobbles and gravel will provide a protective armoring against the 1-percent-annual-chance event, or whether wave action will exert sufficient force to erode them away.

Mixed-sediment beaches can vary significantly in overall morphology and sediment size distribution (i.e., size fractionation). These characteristics make it difficult to identify a "typical" mixed-sediment beach profile in either fair-weather or post-storm conditions. Figure 1.1-4 provides one example of a mixed-sediment profile, but the composition and spatial relationships of the various sediment types can vary significantly from beach to beach. Historical profile data, therefore, are essential for the assessment of event-based erosion in mixed-sediment systems.

Figure 1.1-4. Cobble, Gravel, Shingle, or Mixed Grain Sized Beach and Berms

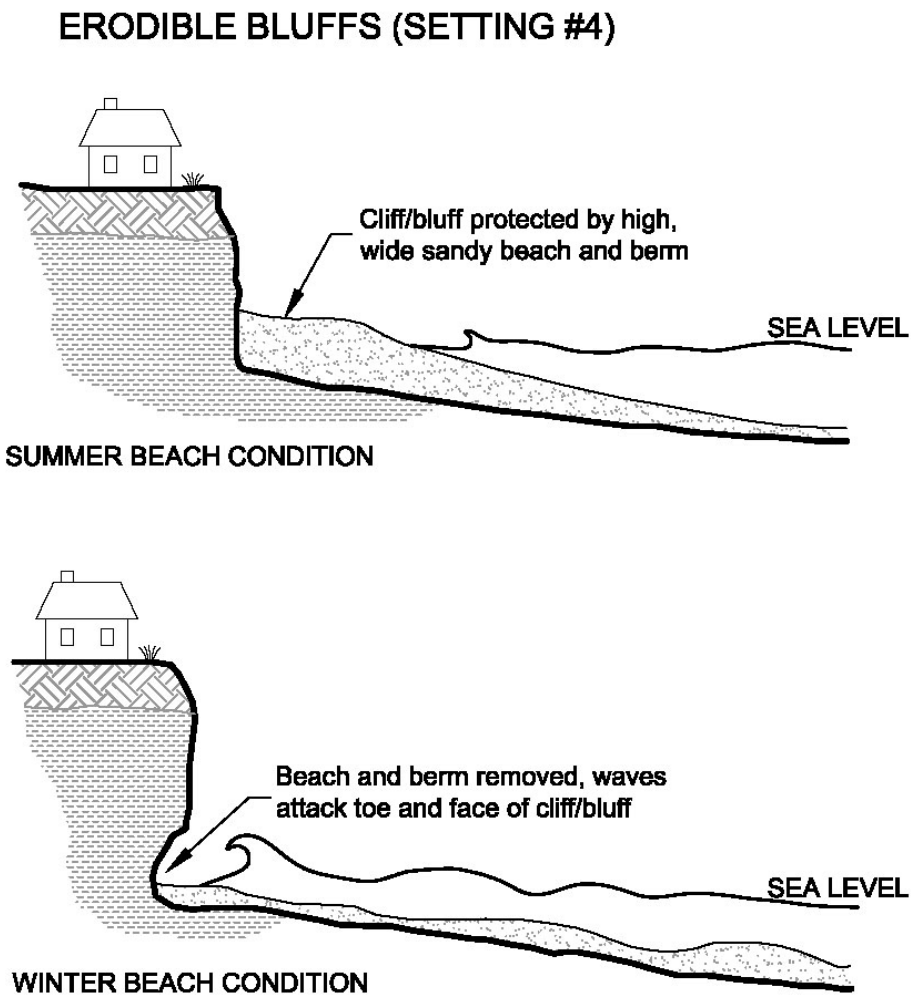


1.1.5 Erodible Bluffs

Found along sections of the US Atlantic, Pacific and U.S. island territory are coasts that have narrow to nonexistent beaches backed by high, steep, erodible coastal bluffs and cliffs. The geomorphic evolution of this bluff-type shoreline is significantly different from that of the sandy beaches backed by either dunes or low-lying berms. A thin sand lens often overlies a rocky beach or bedrock platform fronting the bluff. These thin deposits of sand are removed during each winter storm season. If storm water levels reach sufficient elevations to intersect the toe of the bluff, storm waves can directly impinge upon the bluff face, causing bluff toe erosion (Figure

1.1-5). If enough material is eroded from the toe during a storm, the upper portion of the bluff can fail, resulting in bluff retreat. It should be noted that significant bluff failure may not occur during all storm events. However, if the bluff materials are erodible, toe erosion and bluff failure are possible during individual storm events.

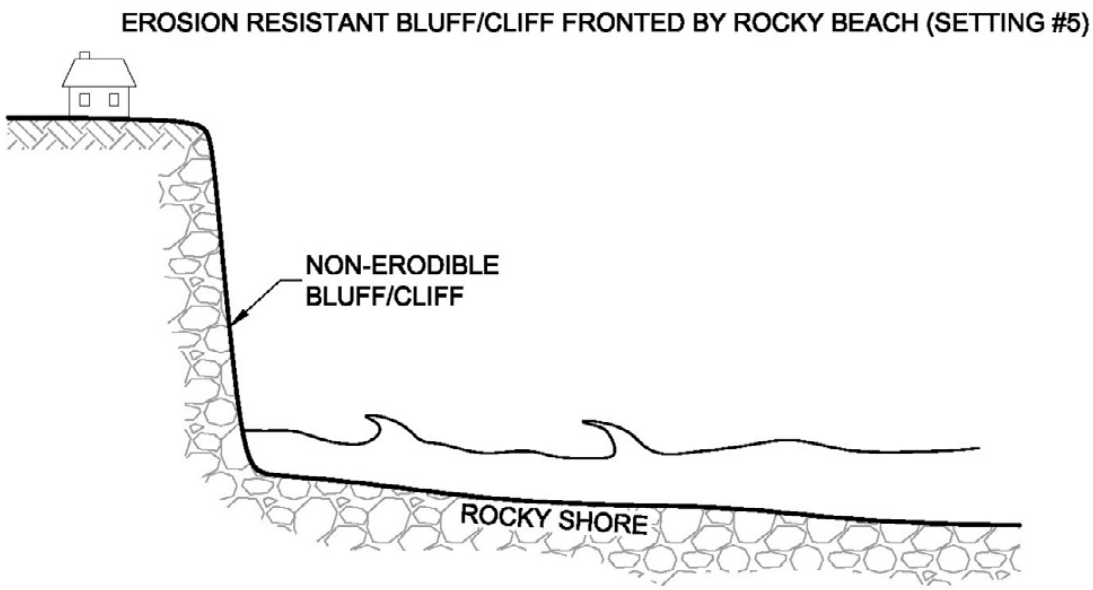
Figure 1.1-5. Erodible Coastal Bluffs (after Griggs, 1985)



1.1.6 Erosion Resistant Bluffs and Cliffs

Erosion-resistant bluffs and cliffs are often fronted by rock terraces, rocky beaches, or narrow rock platforms capped with thin layers of sand or gravel. Once the thin sand cap is eroded from the rocky beach, this beach setting is stable.

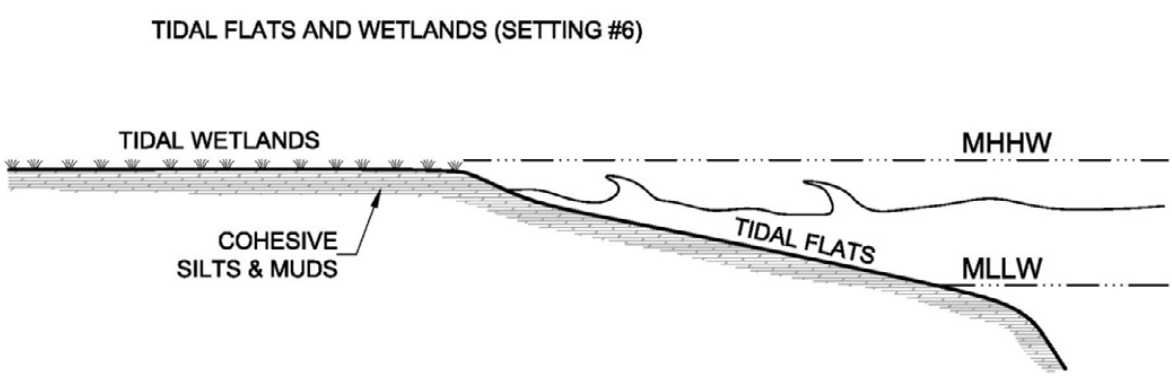
Figure 1.1-6. Non-Erodible Coastal Bluffs and Cliffs



1.1.7 Tidal Flats and Wetlands

Tidal flats and wetlands are low-gradient coastal features, usually comprised of fine cohesive silts and clay. These shorelines generally exist in sheltered water environments where there is little wave energy. Sedimentation processes are typically depositional. Over time, these coastal landforms may become capped with wetland vegetation and detrital deposits, and sand or debris from overland wave propagation during storm events.

Figure 1.1-7. Sheltered Waters (tidal flats and wetlands or other reduced-energy basins) (Beach Setting No. 6)



1.1.8 Artificial Beach

Where construction has placed sediment on the shoreline or structures have been built to reduce the amount of wave energy reaching the shoreline enabling sediment to accumulate. A sample of a large waterfront project that included construction of offshore breakwaters and beach nourishment is presented in Figure 1.1-8. The pocket beaches have been very stable for the last three decades. However, event driven erosion can still alter beach slope and depth for artificial beaches, which in turn influences the flood risks for individual storms. Therefore, storm erosion modeling should be performed when evaluating flood hazards at artificial beaches. However, if the modeling shows the beaches are stable during storm events, then the Mapping Partner can rely on engineering judgment to determine if all of the events in the composite storm database need to be simulated.

Figure 1.1-8. Artificial Beach with Accretional Deposits: Lake Forest Park, North of Chicago, Illinois (Beach Setting 7)

1.1.9 Eroding Sand Bank

Sandy glacial outwash deposits are present in some riverine areas, sheltered water areas, and within the Great Lakes Basin. Wave attack erodes the sand toe during severe storms at high water levels and the bank recedes. The amount of retreat is typically small for individual storm events and detailed numerical modeling may not be required when evaluating wave runup. However, the Mapping Partner should review historical shoreline change rates within the county and make a site specific assessment.

Figure 1.1-9. Eroding Sand Bank (toe of the bank is protected in distance): Shoreham, Lake Michigan (Beach Setting 8).

1.1.10 Eroding, Cohesive Bank

Due to the high percentage of consolidated glacial sediment in the Great Lakes Basin, eroding cohesive banks are a common geomorphic feature. When the lakebed consists of consolidated glacial sediment (lacustrine clay or glacial till), erosion and lakebed down-cutting is a slow process that is attributed to softening of the surface layer of sediment and erosion due to wave orbital velocities and breaking waves. The banks also erode and retreat landward due to a combination of wave attack at the toe and slope stability factors, such as ground water flows. Typically, bluff recession rates ranges from 1 to 3 feet/year in the Great Lakes Basin. Erosion attributed to any one storm has only minor impacts on the amount of lakebed down-cutting and bluff retreat (Baird, 2011). Therefore, in most cases, the Mapping Partner can ignore erosion processes for eroding cohesive banks when evaluating wave runup and overland wave propagation.

Figure 1.1-10. Eroding Cohesive Bank, Wayne County, Lake Ontario South Shore (Beach Setting 9).

2.0 Storm Induced Erosion Methodology

To estimate profile changes for erodible shorelines, various tools and methodologies are available, not all of which are applicable to all settings. This section discusses a number of available approaches but should not be considered an exhaustive list. The Mapping Partner must have knowledge of the various approaches available and apply an erosion methodology that is appropriate for the unique morphology of the study setting. Some of the methods described herein were developed with region-specific datasets and may be best suited for those respective regions; however, models can be adapted and/or calibrated to be applicable to other geographical regions and coastal settings. The Mapping Partner should use judgement and coordinate with the FEMA Project Officer where appropriate.

Regardless of approach, estimation of coastal erosion during storm events is often aided by the following types of site-specific beach information and data:

1. Summary reports and photos of historic post-storm coastal erosion
2. Aerial photos of study area
3. Local geology and shore and backshore material characteristics
4. Previous Flood Insurance Study (FIS) mapping and reporting
5. Pre and post storm topographic data

There are often numerous online resources available to obtain data for use in erosion analysis for a given study area.

2.1 Sandy Coasts

Measured erosion from significant storms is extremely variable on spatial scales on the order of a quarter mile along the coast (D&D, 1989). Documented effects in hurricanes approximating the local 1-percent-annual-chance event show a wide range of duneface retreat and dune removal possible on US coasts. The crucial initial distinction is whether to expect dune failure or persistence as a flooding barrier.

2.1.1 Empirical Geometric Erosion Models

Storm-induced erosion is a highly complex process affected by the site characteristics as well as storm characteristics and can be computationally intensive to resolve. Geometric erosion models offer a consistent, objective, and simplified approach to performing storm-induced erosion. The models discussed in this section have been developed for sandy shorelines that contain an erodible dune feature.

2.1.1.1 DHL Duneface Retreat

A modified version of a duneface retreat model from the Delft Hydraulics Laboratory (DHL) of the Netherlands (DHL, 1986) has been applied in FEMA flood insurance studies since 1991. The method assumes that the majority of erosion occurs above the Stillwater elevation. The adapted approach eliminates potential problems associated with computation sensitivity to storm wave height and situations dissimilar to the Netherlands coast (Birkemeier et al., 1987; FEMA, November 1988).

For application in FEMA flood studies dune cross-section erosion above 1-percent Stillwater level for 38 events along the US Atlantic and Gulf Coasts, with flood recurrence intervals of 1.25 to 300 years, reveal what size dune is necessary for a durable flooding barrier in storms of specific intensity. The trend of these field data gives a statistical estimate for dune erosion quantity to be expected in a 1-percent-annual-chance event. The data indicate that for erosion to be limited to duneface retreat without breaching, an initial cross-section of 540 square feet is required above the 1-percent Stillwater Elevation (SWEL) and seaward of the rearmost dune crest (D&D, 1989).

Figure 2.1.1-1 introduces terminology for two representative dune types. A frontal dune is a ridge or mound of unconsolidated sandy soil, extending continuously along the shore landward of the sand beach. The dune is defined by relatively steep slopes abutting markedly flatter and lower regions on each side. For example, a barrier island dune has inland flats on the landward side, and the beach or back beach berm on the seaward side. The dune toe is a crucial feature and can be located at the junction between gentle slope seaward and a slope of 1:10 or steeper, marking the front dune face. The rear shoulder, as shown on the mound-type dune Figure 2.1.1-1, is defined by the upper limit of the steep slope on the dune's landward side.

The rear shoulder of mound-type dunes corresponds to the peak of ridge-type dunes. Once erosion reaches those points, the remainder of the dune offers greatly lessened resistance and is highly susceptible to rapid and complete removal during a storm. Figure 2.1.1-1 shows the location of the “frontal dune reservoir,” above the 1-percent-annual-chance SWEL and seaward of the dune peak or rear shoulder. The amount of frontal dune reservoir determines dune integrity under storm-induced erosion.

Figure 2.1.1-1. Dune Features and Present Terminology

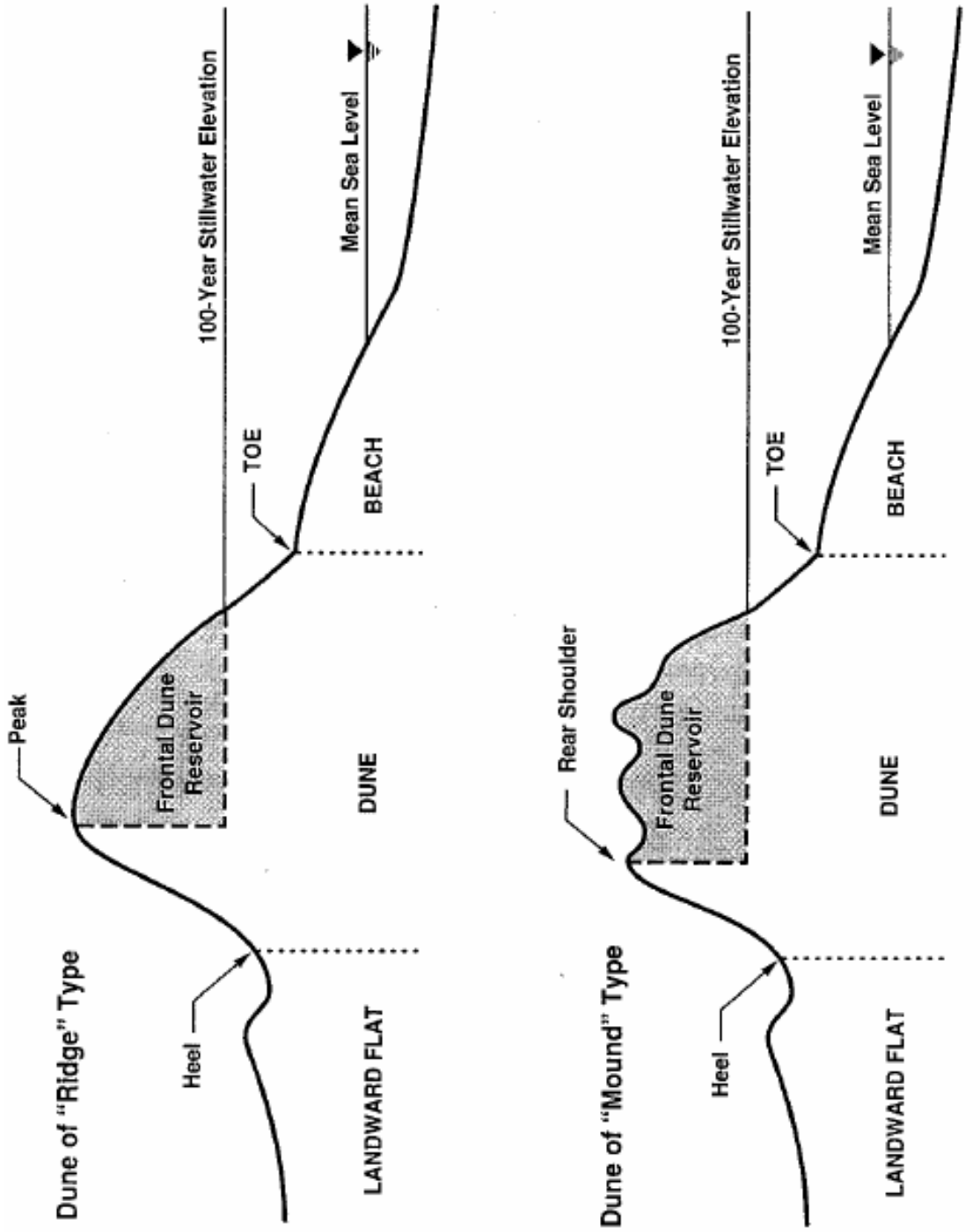
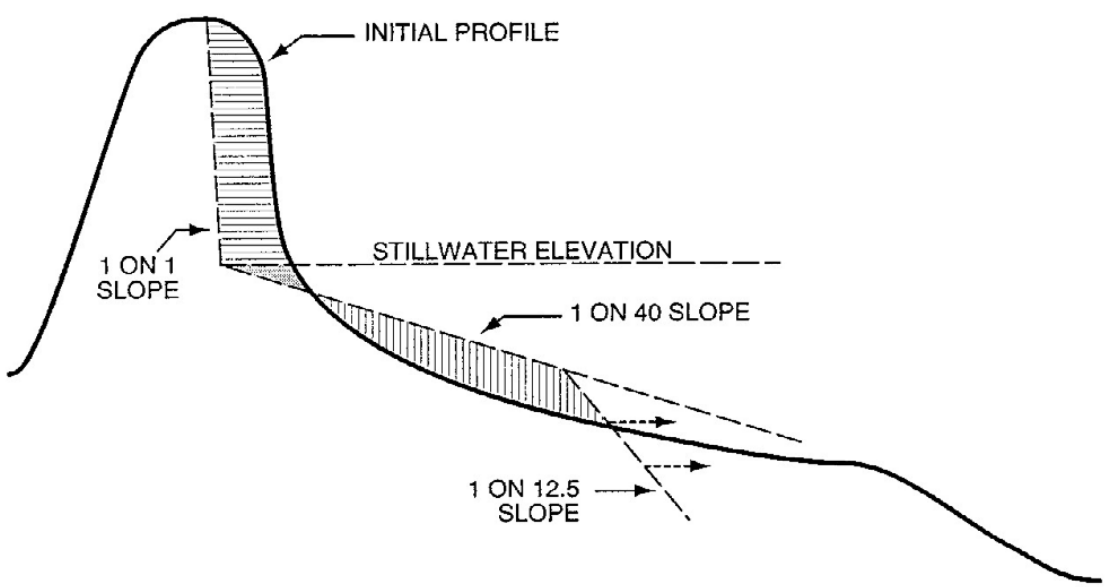


Figure 2.1.1-2 summarizes the treatment cases of duneface retreat. The eroded profile consists of three planar slopes: the uppermost is a retreated duneface slope of 1:1, joining an extensive middle slope of 1:40, which is terminated by a brief segment with a slope of 1:12.5 at the limit to storm deposition. Upper dune erosion is specified to be 540 square feet above the 1-percent-annual-chance (SWEL), including wave set up, and in front of the 1:1 slope. Geometrical construction balances the nearshore deposition with the total dune erosion of somewhat more than 540 square feet by an appropriate seaward extension of the 1:40 slope. The resulting eroded profile is spliced onto the unchanged landward and seaward portions of the pre-storm profile. This procedure gives a complete profile suitable for use with subsequent coastal modeling. During such retreat, the dune remains partially intact and eroded sand is transported in the seaward direction. The post-storm profile provides a balance between sand eroded from the duneface and sand deposited at lower elevations seaward of the dune.

Actual quantities of storm-induced dune erosion are subject to large variations and this procedure presumes a generally representative value for the 1-percent-annual-chance flood condition. Though this empirical, geometric, event based approach has been derived from analysis of data collected at sandy, open coast locations along the US Atlantic and Gulf coasts, it may be but may be applicable, with some calibration, to other geographical regions with similar shoreline settings.

Figure 2.1.1-2. Procedure Giving Eroded Profile in Cases of Duneface Retreat, and Simplification of Dune Retreat Model Developed by Delft Hydraulics Laboratory of the Netherlands.



PROCEDURE:

1 - CONSTRUCT RETREATED DUNEFACE WITH 540 FT^2 EROSION [ ] ABOVE 100-YEAR STILLWATER ELEVATION AND SEAWARD OF 1 ON 1 SLOPE.

2 - DETERMINE ADDITIONAL DUNE EROSION QUANTITY, SHOWN DOTTED, IN WEDGE BETWEEN STILLWATER ELEVATION, 1 ON 40 SLOPE, AND INITIAL PROFILE.

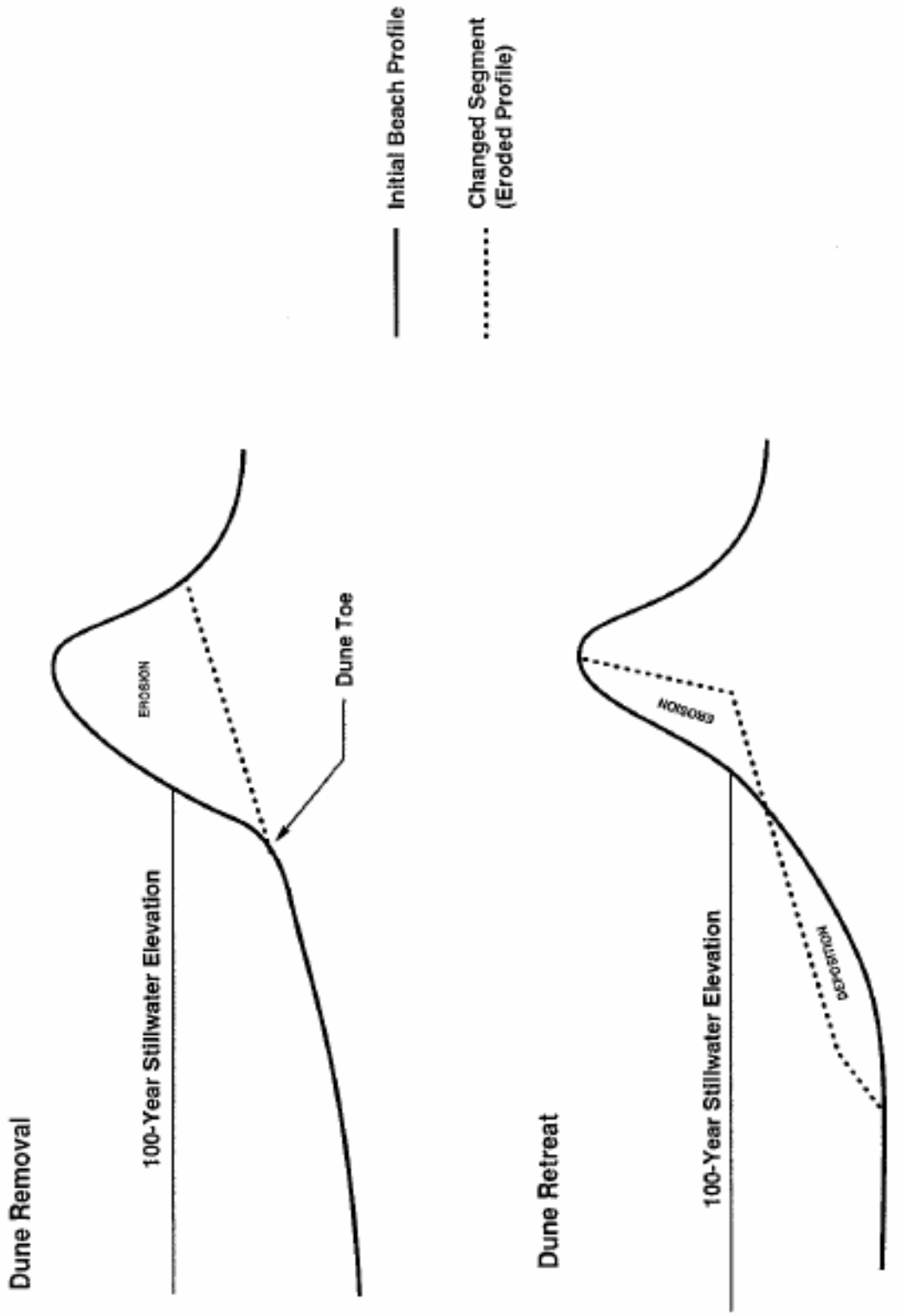
3 - BALANCE TOTAL DUNE EROSION WITH POSTULATED DEPOSITION [ ] BY APPROPRIATE PLACEMENT OF 1 ON 12.5 SLOPE AS LIMIT TO DEPOSITION.

2.1.1.2 Duneface Removal

Duneface removal is perform when the dune reservoir is less than that cross-sectional area determined to be require for some remnant of the dune to remain as a flood barrier, commonly 540 square feet. When the reservoir volume is below that amount the dune should be removed. Construction of the dune removal profile is simple: the profile is modified with a 1:50 seaward-dipping from the backside (landward) of the dune through the dune toe. This treatment simply removes the major vertical projection of the dune from the transect.

Construction of a removal profile focuses on the usually distinct dune toe. The dune toe is taken to be the junction between the relatively steep slope of the front dune face and the notably flatter seaward region of the beach or the back-beach berm (including any minor foredunes). If a clear slope break is not apparent on a given coastal transect, its location may be taken at the typical elevation of definite dune toes on nearby transects within the study area. Alternatively, the dune toe may be set at the local 10-percent SWEL, which has been shown to be an adequate approximation along the Atlantic and Gulf coasts. Figure 2.1.1-3 provides schematic sketches of the different geometries of dune erosion arising in coastal flood hazard assessments.

Figure 1.1.1-3. Schematic Cases of Eroded Dune Geometries with Planar Slopes



2.1.1.3 Finalize Erosion Assessment

The dune reservoir cross-sectional value of 540 square feet, defined here as the threshold between dune removal and dune retreat, is a median value derived from profile data sourced from multiple locations along the US Atlantic and Gulf coasts. This threshold value may not be wholly accurate for any given study location as site characteristics as well as storm characteristics are always unique. Validating the present erosion assessment for a specific site by means of available evidence is advisable.

At many sites, historical evidence may be available regarding the extent of flooding, erosion, and damage in an extreme event comparable to the local 1-percent-annual-chance flood. In these instances, an erosion treatment providing results more consistent with historical records may be selected as appropriate. That choice may be relatively clear-cut given potential differences in expected erosion and inland flood penetration for duneface retreat versus dune removal. Where available historical evidence is not definitive, the decision between retreat and removal on a given transect should be based solely on size of the frontal dune reservoir. Present procedures for erosion assessment are highly simplified, but provide a consistent, unbiased estimation and a level of detail appropriate to coastal flood map projects.

Finally, the dissipative effect of wide sand beaches that shelter dunes from the full storm impact erosive waves can alter the likelihood of dune retreat or removal. If the existing slope between mean level and the 1-percent-annual-chance SWEL is 1:50 or gentler, overestimation of erosion is possible during the 1-percent-annual-chance flood; therefore, the Mapping Partner should examine this carefully. This effect and other variables, such as sand size, dune vegetation, and actual storm characteristics at a specific site, make thorough comparison of estimated erosion to documented historical effects in extreme storms necessary.

2.1.1.4 K&D

The K&D model, developed by Kriebel and Dean (1993), is an equilibrium profile erosion model that has traditionally been applied to sandy beaches backed by dunes in California, with successful test applications in Oregon and Washington. The model considers the total water level, storm duration, breaking wave height, D_{50} of the beach material, and profile characteristics (beach face slope and surf zone profile) to determine the maximum beach erosion potential for a particular storm event. Conservation of sand volume between the erosion of the dunes and the offshore deposition is maintained.

The K&D model was developed for four different beach profiles: (1) a square berm, (2) a sloping backshore, (3) a sand beach backed by high dunes (15 to 50 feet high), and (4) a sand beach backed by a low berm with a wide backshore. Therefore, the K&D model is applicable to a wide variety of beach conditions and settings. For the purposes of these guidelines, we only consider sand beach backed by sandy high dunes; the solution to estimate maximum erosion potential, R_∞, for this setting is as follows:

- Maximum erosion potential for a beach backed by a low sand berm:

$$R_\infty = \frac{S\left(W_b - h_b / m\right)}{B + h_b - S/2} \tag{2-1}$$

Figure 2.1.1-4. Definition Sketch for K&D Geometric Model

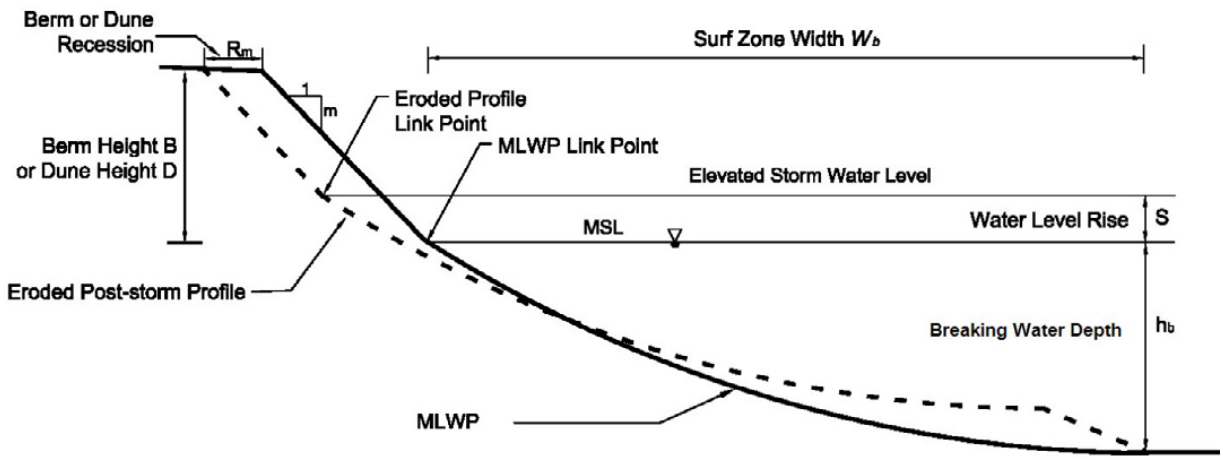
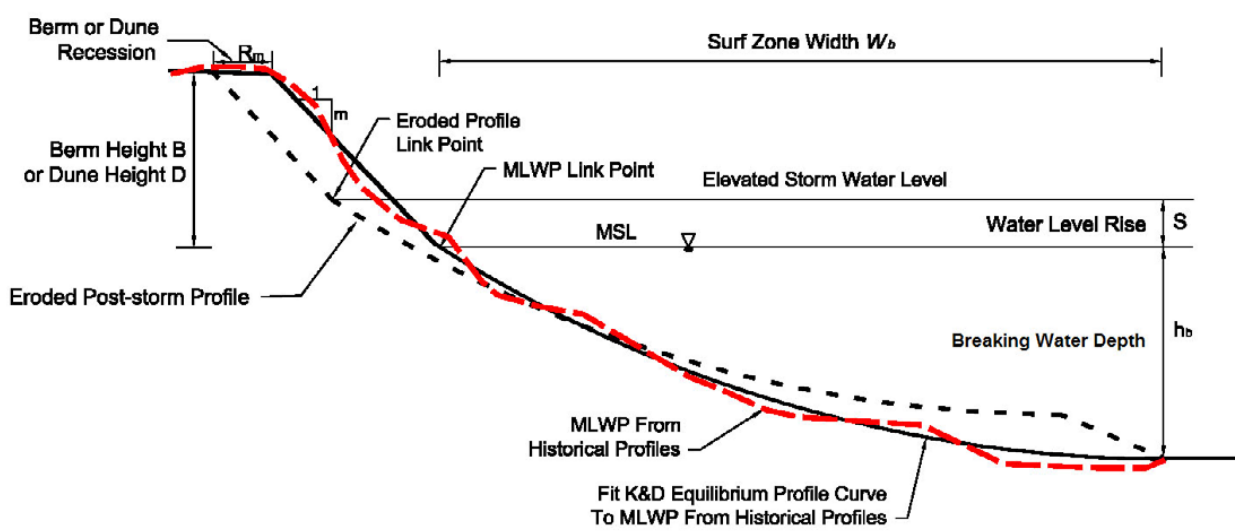


Figure 2.1.1-5. Sketch for K&D Geometric Model for Case Where Historical Beach Profile Data Are Available to Prepare the MLWP



- Maximum erosion potential for a beach backed by a high sand dune:

$$R_{\infty} = \frac{S\left(W_b - h_b / m\right)}{D + h_b - S/2} \tag{2-2}$$

where S is the water-level rise representing the sum of the peak storm surge (wind effects and barometric pressure effects) and the wave setup, h_b is the breaking water depth, W_b is the surf zone width, m is the slope of the foreshore fronting face, and B and D are the berm and dune heights above the prevailing water level, respectively. Equation 2-1 estimates the maximum recession potential, assuming that the storm event lasts indefinitely. The actual storm-induced

recession (R_m), which depends strongly on the duration of each storm event, must be multiplied by a storm duration recession reduction factor, α. For backshore profiles that are not well approximated by the analytical solutions given in Kriebel and Dean (1993), a conservation of sand volume equation (i.e., a simple balance of cuts and fills) may be solved numerically. Further discussion of this computational procedure is provided in the following guidelines.

In the event that the total water level is higher than the crest of the dune, the K&D model may no longer be applied and the profile must be adjusted for overtopping. When the K&D model is applied to estimate the storm-induced erosion, various model input parameters are required. The calculation of storm-induced erosion, using the K&D convolution method, is delineated as follows:

Acquire Wave and Water-level Data:

1. Obtain hindcasted wave data and measured historical water levels necessary to define the oceanographic conditions including waves and water levels for 10-20 largest storm events for every hindcasted year.

2. Acquire historical beach profiles to establish the Most Likely Winter Profile (MLWP) (i.e., pre-storm beach profile conditions). MLWP is discussed in Section 3.

3. Seek historical pre- and post-storm profiles to validate the application of the simple K&D geometric models.

Quantify Peak Storm Conditions of a Selected Storm Event:

The Total Water Level (TWL) should account for storm surge, wave setup, wave runup, and any increase induced by El Niño events. The peak storm conditions are used to determine the TWL (S), water depth of breaking wave (h_b), and the surf zone width (W_b) needed by the K&D geometric model. The Mean Sea Level (MSL) water depth can be used as a representative water depth to calculate wave transformation. The procedures are listed as follows:

1. Determine the breaking water depth (h_b) and the surf zone width (W_b), based on the MLWP and the selected wave event.
2. Estimate the wave setup and runup.
3. Calculate the storm surge induced by wind effects and barometric pressure effects, if applicable.
4. Estimate the increase in water level induced by the El Niño Southern Oscillation (ENSO) events, if applicable.
5. Determine the total water level (S) induced by the storm (see Figures 2.1.1-4 and 2.1.1-5).

Calculate Storm-induced Beach Erosion:

1. Calculate the maximum beach erosion potential R_∞ using Equation 2-1, if the subject beach is backed by sand berms with height B.

2. Calculate the maximum beach erosion potential R_∞ using Equation 2-2, if the subject beach is backed by sand dunes with height D.
3. Calculate the time scale (T_S) from Equation 2-6.
4. Determine the storm duration (T_D), and compute the storm duration recession reduction factor, α, from Figure 2.1.1-8 for the given value of T_D/T_S.
5. Multiply the maximum recession potential (R_∞) by the storm duration recession reduction factor to estimate the storm-induced beach erosion and recession distance (R_m).

Prepare Eroded Post-Storm Beach Profile:

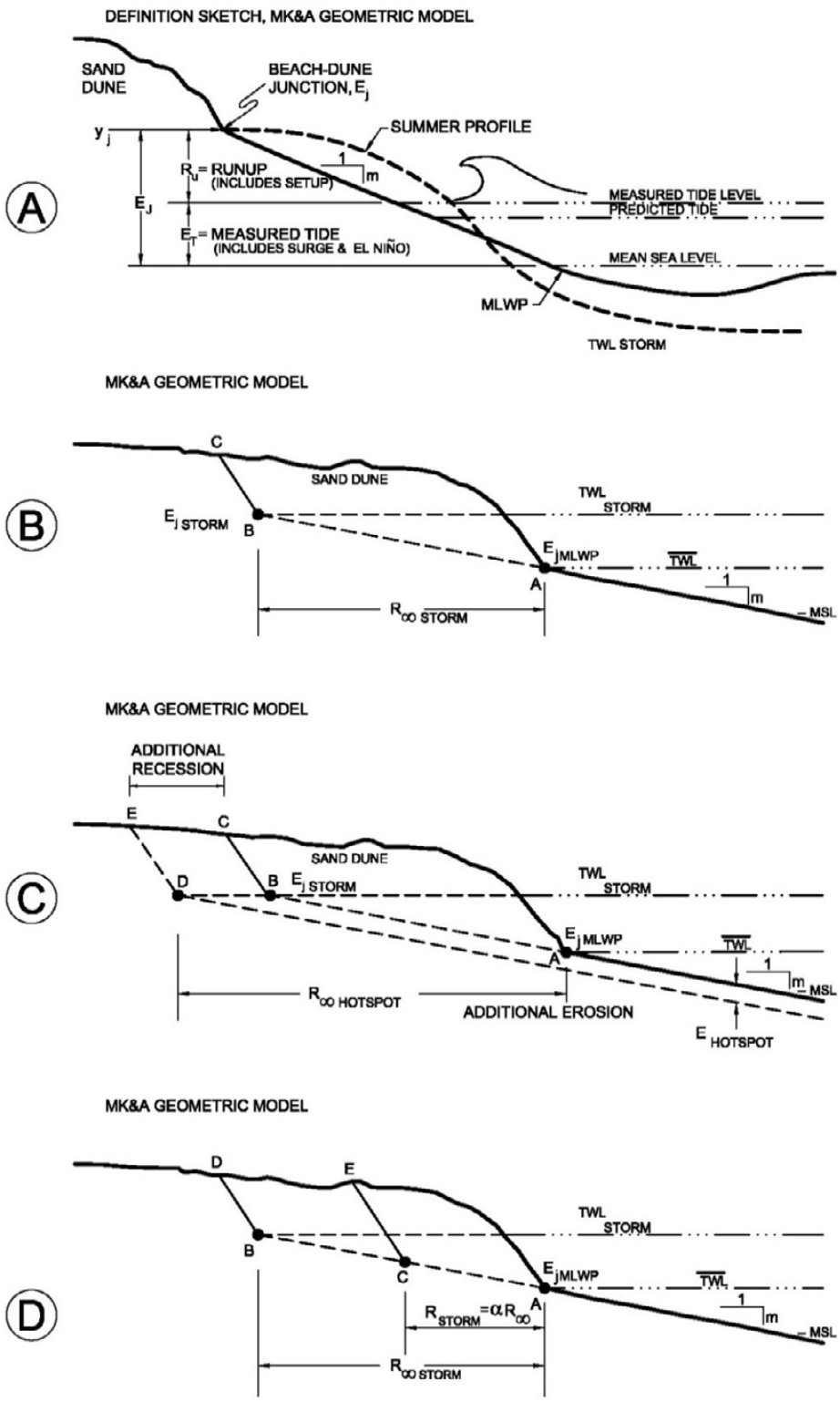
1. Set back the upper foreshore profile above the elevated storm wave level (see Figures 2.1.1-4 and 2.1.1-5) landward by the calculated berm or dune recession distance R_m with the same fronting-face slope (m).
2. Place the new link point between the upper foreshore section and the surf zone section at the elevated storm water level.
3. Shift the surf zone section of the MLWP below the MSL landwards and upwards to the link point (see dashed curve below the MSL line in Figures 2.1.1-4 and 2.1.1-5).
4. The adjusted profile from Steps 1 to 3 produces the “eroded storm profile” for a specified location and beach profile. Mapping Partners should perform these steps for all beach profiles needed to describe the spatial adjustments to the beach and dune system being evaluated.
5. Document results and assumptions.

2.1.1.5 MK&A

The MK&A model was developed by Komar et al. (1999) to estimate foredune erosion and further modified by McDougal and MacArthur (2004) to provide estimates of beach profile recession due to large storm events. The erosion potential is determined entirely by the change in the total water level and the beach slope, and is very sensitive to the slope. The MK&A model was developed and tested for the Oregon and Washington coast where dunes are high and overtopping is unlikely during a storm event.

The model is based on the underlying assumptions of an MLWP and the characteristic shape of shoreline recession that will result during a large wave and water-level event. The shoreline recession profile is characterized by the beach face slope, m, the beach-dune juncture elevation, E_j, and cross-shore location of the beach-dune juncture, y_j. These are shown in Figure 2.1.1-6. The juncture elevation is taken to occur at the maximum extent of the total runup plus the measured tide. The measured tide includes all processes that influence the water surface elevation such as surge and El Niño. The total water level (TWL) is the sum of the stillwater level (SWL) plus wave setup and runup. The sum of the astronomical tide, coastal processes due to El Niño, and surge is the still water level (SWL) and is typically obtained from measurements. Wave setup and runup are calculated using methods described in detailed guidance units for wave setup and runup.

Figure 2.1.1-6. Definition Sketches for Terms and Dimensions Required by the Modified Komar & Allan Geometric Model (after Komar et al., 2002, and McDougal and MacArthur, 2004)



2.1.1.5.1 Summary of the MK&A Geometric Modeling Approach for Sand Beaches Backed by Sandy Berms and Dune

Figure 2.1.1-6 shows the sequence of key activities and computational considerations required to determine storm-induced beach profile changes for sandy beaches backed by low sand berm or high sand dune using the MK&A model approach.

Develop Data:

1. Obtain wave and water-level data necessary to define the waves and water levels for the 10-20 largest storms each year.
2. Determine existing shoreline location and conditions.
3. Define reaches alongshore in which wave, beach, and backshore conditions are nearly uniform. Data and calculations must be conducted for at least each subreach.
4. Obtain beach profile data required to establish the MLWP or the annual winter wave and water-level conditions to develop an MLWP for each subreach.
5. Determine median sand diameter, D_{50}, on the beach face for each subreach.
6. Obtain historical beach profile data required to estimate the magnitude of local hot spot erosion and site-specific beach lowering with each subreach being evaluated within the study area.
7. Seek historical data for use in validating results from the application of the simple geometric models.

Determine Beach Recession for Each Storm Event (refer to Figure 2.1.1-6 for illustration of terms):

1. Estimate the MLWP for each cross-shore profile
2. Determine static setup and/or TWL as required for the geometric recession model to calculate the potential recession for the storm, $R_{\infty\, storm}$.
3. Determine storm duration recession reduction factor for the storm, α (Figure 2.1.1-8).
4. Determine duration limited recession for storm, R_{storm}, and if the berm/dune is breached, modify beach and berm/dune profile to account for breaching or local hot spot erosion if necessary.
5. If runup is different on the modified profile, re-compute runup.
6. If runup results in overtopping, then compute overtopping. Save the maximum overtopping value. Also compute the overtopping volume as V = integral $Q\ dt$ over duration of storm.
7. For each year, save conditions corresponding to the largest annual TWL storm event: TWL, Q, V, α, H, T, D, γ, R_{storm} , etc.

Mapping Partners should use measured beach profile data wherever possible: (1) to aid in estimating the MLWP, and (2) to determine, calibrate, and validate the eroded beach profile for

a specified storm event. The eroded beach profile estimated for a particular storm event is the profile required for computing runup and overtopping associated for that event.

2.1.1.6 MK&A Process

The first step for determining eroded beach profiles is to estimate the MLWP for each cross-shore profile. When using the MK&A method, the upper profile is characterized by the beach face slope in the swash zone, *m* and the beach-dune juncture elevation and cross-shore location, $E_{j\ MLWP}$ and $y_{j\ MLWP}$ as shown in Figure 2.1.1-6. Because both the elevation and location of the juncture may be associated with different magnitudes of TWL events, the notation $()_{MLWP}$ is used to denote the MLWP case. The juncture elevation in the MK&A model is taken to occur at the maximum extent of the still water plus the total runup. The measured tide includes all processes that influence the water surface elevation such as the astronomical tide, surge, and El Niño. The runup is defined to include wave setup. The beach face slope is determined in the swash zone at high water levels. For the MLWP, *m* and $E_{j\ MLWP}$ are determined from beach profile measurements following a significant storm or at the end of the winter season, or they may be determined from typical winter wave and water-level conditions (as explained below).

The MLWP should be determined from profile data immediately following a significant storm or series of winter storms. Profiles taken during the summer and fall should not be used. Profiles measured later in the winter season are preferred as they should represent the maximum beach response due to the seasonal cycle. If appropriate post-storm or late winter profiles are not available the Mapping Partner should estimate them; a process to estimate this is described in Section 3.3.

2.1.1.6.1 $E_{j\ MLWP}$ from Wave and Water Levels

Given the difficulty in identifying a single value to select for $E_{j\ MLWP}$ based on beach profile data alone, it may be possible to supplement the estimate with information about the waves and water levels that are typically responsible for producing the dune-beach juncture elevation, E_j . The juncture elevation can be estimated for the typical winter wave conditions as:

$$E_j = (R + E_T)_{\text{winter storm average}} \tag{2-3}$$

where the runup includes the setup and the tide includes surge and El Niño (see Figure 2.1.1-6A). In Equation 2-3, E_j represents the average of the sum of *R* and E_T from 10 to 20 largest storms per year, averaged over the storm duration for the entire wave data record.

2.1.1.6.2 MK&A Model for Estimating Beach Profile Changes

The recession in the MK&A model due to $E_{j\ \text{Storm}}$ is calculated as the recession in excess of the MLWP. The maximum potential recession is given by:

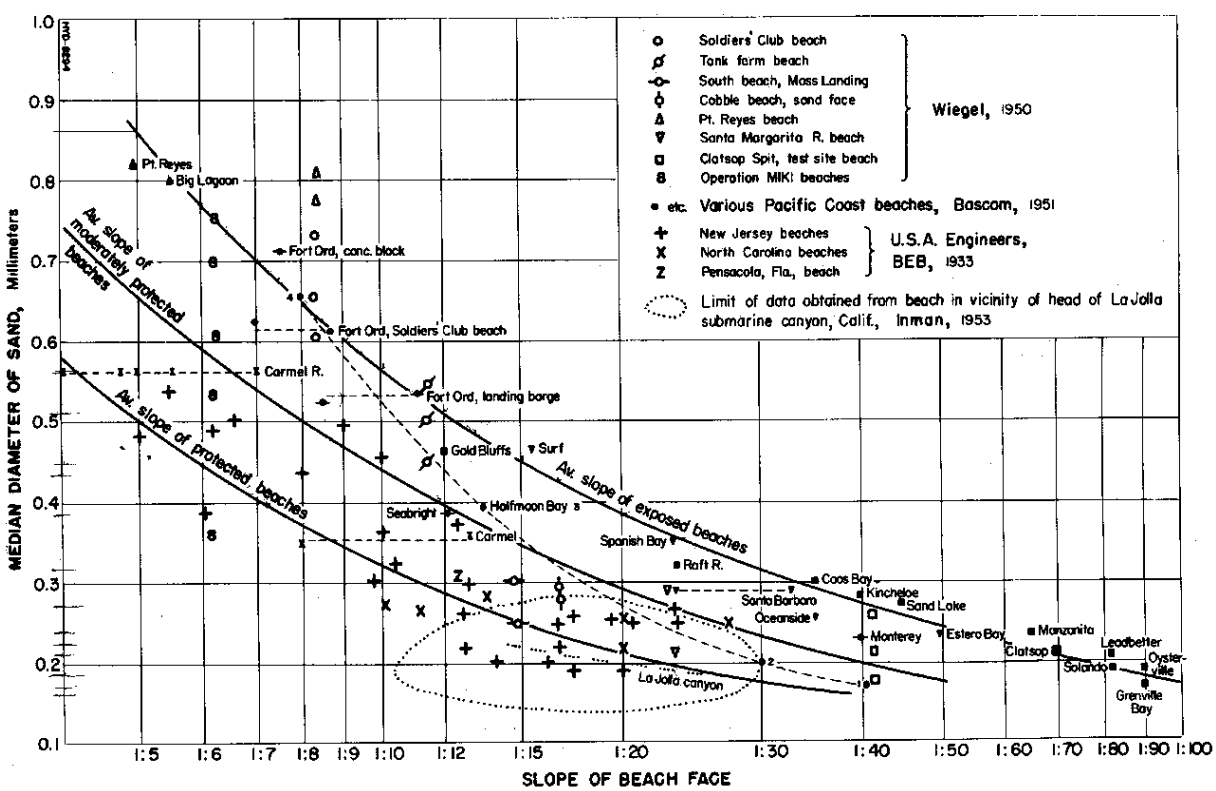
$$R_{\infty Storm} = \frac{E_{jStorm} - E_{jMLWP}}{m} \tag{2-4}$$

where E_{jStorm} and E_{jMLWP} correspond to beach-dune juncture elevations evaluated at the storm conditions and for the MLWP.

The cross-shore location of the juncture point, y_j is the initial location for the MLWP and may change with time (Figure 2.1.1-6). This can be in response to chronic erosion, sea-level changes, or other long-term effects. It may be necessary to adjust y_j for the MLWP if the time between the MLWP determination and the analysis of the recession is significant or if chronic shoreline position changes are significant.

Bascom (1964), Wiegel (1964), and others have shown that there are strong correlations between the beach face slope, m, and the median diameter of the beach sands as shown in Figure 2.1.1-7. These types of relationships can be used to estimate the beach face slope. The user should select the curve that best matches the coastal exposure, beach material characteristics, and settings represented by the curves prepared by the original authors. Open coasts along Oregon and Washington experience beach slopes approximately two times as steep as one would estimate using Wiegel's regional relationship as shown in Figure 2.1.1-7, or approximately 1:25-30 (v:h). Mapping Partners should check estimated slope values from Figure 2.1.1-7 with observed data. It is recommended that regional relationships similar to these be developed and tested for the different coastal exposures and settings found in California, Oregon, and Washington for estimating winter beach face slope.

Figure 2.1.1-7. Relationships Between Beach Slope and Median Diameter of Beach Sands (from Wiegel, 1964)



If a beach consists of a thin layer of sand capping a wave-cut terrace or other erosion-resistant materials, then the MLWP occurs at the location and profile of the erosion-resistant layer.

Following Komar et al. (2002), where an adjustment was allowed for hot spots, the recession may be written as:

$$R_{\infty HotSpot} = \frac{E_{jStorm} - E_{jMLWP} + E_{HotSpot}}{m} \tag{2-5}$$

where $E_{HotSpot}$ is the localized lowering of the profile due to shoreline recession during a significant storm event due to local hot spots. Effects of site-specific hot spots and the amount of local beach lowering at that location is estimated from seasonal monitoring data from past large storm events.

2.1.1.7 Dune Overtopping with the MK&A Model

When overtopping occurs, the dune profile is adjusted by extending the MLWP slope *m* to the backside of the dune. Relationships like those shown in Figure 2.1.1-7 by Wiegel (1964) can be used to estimate the ultimate beach face slope following significant dune breaching. If this approach is used, Mapping Partners should check the reliability of their results with observed information and data.

2.1.1.8 Time Dependency of Profile Response (Within the MK&A and K&D Models)

The time scale for the beach profile was estimated from numerical model results to be:

$$T_s = C_1 \frac{H_b^{3/2}}{g^{1/2} A^3} \left(1 + \frac{h_b}{B} + \frac{m W_b}{h_b} \right)^{-1} \tag{2-6}$$

in which T_s is the time scale, C_1 is an empirical constant (=320), H_b is the breaker height, h_b is the breaker depth, g is the acceleration due to gravity, B is the berm elevation, m is the beach face slope, W_b is the surf zone width, and A is the beach profile parameter that defines an equilibrium profile according to Equation 2-7.

$$\text{h = A y2/3} \tag{2-7}$$

The beach profile parameter, A, depends primarily upon sediment grain size, D_{50}. Table 3-1 summarizes A over a range of sediment sizes. The values in Table 3-1 are well approximated by the equations:

$$A = \begin{cases} 0.06551 \mathrm{n}(D_{50}) + 0.208 & (\mathrm{m}^{1/3}) \\ 0.099731 \mathrm{n}(D_{50}) + 0.309 & (\mathrm{ft}^{1/3}) \end{cases} \tag{2-8}$$

in which D_{50} is the sand diameter in mm and A is in $m^{1/3}$ or $ft^{1/3}$. Table 2.1-2 gives estimates of the time scale for several representative conditions. It is seen that typical times are on the order of 10 to 100 hours. As the surf zone width increases, the response time also increases. Properties that increase the surf zone width include larger wave height, smaller sand size, and a milder slope. The response time also increases as the berm height increases. The longer profile response time associated with larger wave heights has the interesting result that the largest

wave height may not yield the largest recession because it takes longer for the larger waves to achieve the maximum potential recession. Consider the first two waves in Table 2.1-2, which only differ in wave height and the associated breaker depths. Assuming the period in both cases is 13 seconds and the storm duration is 24 hours and employing methods discussed below, the 10-foot wave height has a recession of 70 feet and the 20-foot wave has a recession of 55 feet.

Table 2.1-1. Equilibrium Beach Profile Coefficients (Dean and Dalrymple, 2002)

D_{50} (mm)	A ($m^{1/3}$)	A ($ft^{1/3}$)
0.1	0.063	0.0936
0.2	0.100	0.1486
0.3	0.125	0.1857
0.4	0.145	0.2155
0.5	0.161	0.2392
0.6	0.173	0.2571
0.7	0.185	0.2749
0.8	0.194	0.2883
0.9	0.202	0.2987
1.0	0.210	0.3120

Table 2.1-2. Estimates of the Beach Profile Time Response

H_b (ft)	h_b (ft)	D_{50} (mm)	A ($ft^{1/3}$)	m	B (ft)	W_b (ft)	T_s (hrs)
10	13	0.2	0.1486	0.05	10	801	28
20	25	0.2	0.1486	0.05	10	2267	53
30	38	0.2	0.1486	0.05	10	4164	77
20	25	0.2	0.1486	0.05	1	2267	14
20	25	0.2	0.1486	0.05	10	2267	53
20	25	0.2	0.1486	0.05	20	2267	64
20	25	0.2	0.1486	0.01	10	2267	96
20	25	0.2	0.1486	0.02	10	2267	80
20	25	0.2	0.1486	0.10	10	2267	34
20	25	0.1	0.0936	0.05	10	4533	138
20	25	0.2	0.1486	0.05	10	2267	53
20	25	0.5	0.2392	0.05	10	1110	18

The beach profile response is determined by a convolution integral. It is assumed that the time dependency of the storm hydrograph may be approximated as:

$$f(t) = \sin^2\left(\pi \frac{t}{T_D}\right) \quad \text{for } 0 < t < T_D \tag{2-9}$$

where t is time from the start of the storm and T_D is the storm duration. The convolution integral is:

$$R(t) = \frac{R_\infty}{T_S} \int_0^t f(\tau) e^{-(t-\tau)/T_S} d\tau \tag{2-10}$$

which integrates to:

$$\frac{R(t)}{R_\infty} = \frac{1}{2}\left\{1 - \frac{\beta^2}{1+\beta^2}\exp\left(-t/T_S\right) - \frac{1}{1+\beta^2}\left[\cos\left(2\pi t/T_D\right) + \beta \sin\left(2\pi t/T_D\right)\right]\right\} \tag{2-11}$$

where $\beta = 2\pi T_S / T_D$ and R_∞ is the maximum potential recession that would occur if the storm duration was infinite as yielded by Equations 2-4 and 2-5 (Figure 2.1.1-6D) for the MK&A method. If the storm duration is long with respect to the profile time scale, then a significant portion of the maximum potential shoreline response will occur. As the ratio of T_S / T_D decreases, less of the maximum shoreline change will be realized. The time of the maximum recession is determined by setting the derivative of Equation 2-11 equal to zero and solving for the time. This yields:

$$\exp\left(\frac{-t_m}{Ts}\right) = \cos\left(\frac{2\pi t_m}{T_D}\right) - \frac{T_D}{2\pi T_S}\sin\left(\frac{2\pi t_m}{T_D}\right) \tag{2-12}$$

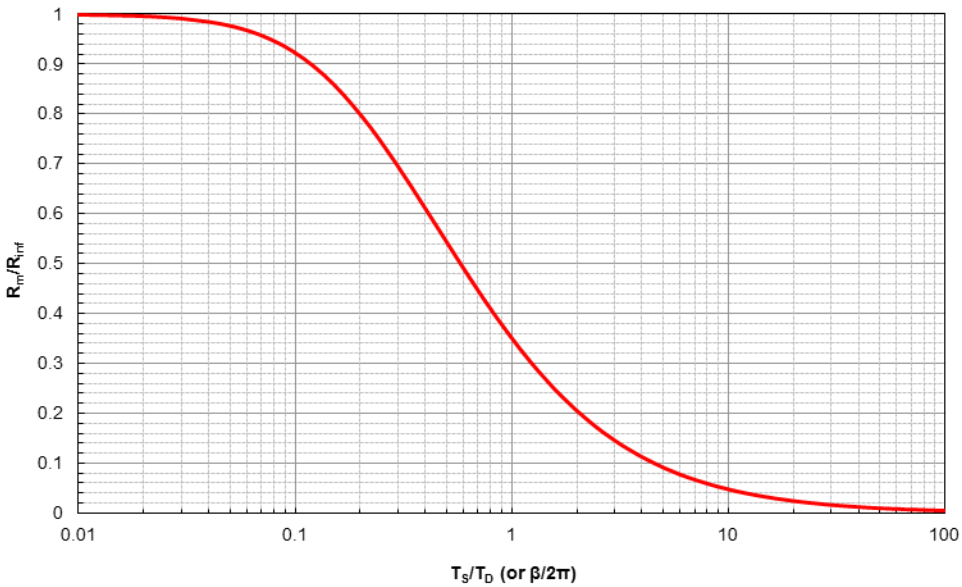
in which t_m is the time that the maximum occurs with respect to the start of the storm. Unfortunately, this is a transcendental equation and must be solved by approximation or numerical methods. The maximum recession that occurs as the result of a single storm or duration limited response is:

$$\alpha = \frac{R_m}{R_\infty} = \frac{1}{2}\left[1 - \cos\left(2\pi \frac{t_m}{T_D}\right)\right] \tag{2-13}$$

where α is the storm duration recession reduction factor, R_m is the maximum recession that occurs for the given storm duration that occurs at time t_m. Figure 2.1.1-8 gives the solution to Equation 2-13 in graphical form. Therefore, duration limited recession is:

$$Rm = \alpha R\infty \tag{2-14}$$

Figure 2.1.1-8. Storm Duration Recession Reduction Factor



2.1.1.9 Multiple Storm Responses

Unless there is site-specific information or guidance for using multiple storms, it is recommended that a single storm analysis be used. If multiple storms are to be considered, then the cumulative recession may be estimated by summing the contribution of each storm to the recession beyond the previous profile. McDougal and MacArthur (2004b) discuss methods for conducting cumulative recession analyses in their report entitled *EBE MLWP Discussion.* Before initiating a seasonal response investigation, Mapping Partners should check with the FEMA Project Officer to confirm that this level of analysis is necessary and that there are sufficient historical data to confirm the results.

2.1.2 Processes Based Erosion Models

Process-based erosion models, such as those discussed in this section, may be suitable for application in coastal settings outside of those for which they were developed, however, this should be done with caution. With the consent of the FEMA Project Officer, the Mapping Partner may investigate the applicability of process-based models, but must ensure that model assumptions and limitations are consistent with their use in the study area. Because process-based models generally require more sophisticated input data and more computational effort than the geometric models, the Mapping Partner should select a model that is consistent with the level of effort to be applied in the overall study. Model selection must be made in close coordination with the FEMA Project Officer. Where used, results from process-based models should be evaluated against historical data to ensure that the results are reasonable.

2.1.2.1 SBEACH

The SBEACH model was developed by the United States Army Corps of Engineers (USACE) as a tool for simulating the performance of beach fill design and erosion from short-term events. The model was developed with significant reliance on empirical data derived from large wave tank tests as well as field data collected from US Atlantic and Gulf Coastal sites. The model has been applied to numerous field case studies on the Atlantic and Gulf coasts, and to a lesser degree in the Great Lakes where environments closely fit the conditions for which the model was developed and calibrated. However, several less-successful experiences using SBEACH, EBEACH, and COSMOS have occurred on the coasts of California (Noble Consultants, 1994) and Oregon (Komar et al., 1999; Komar, 2004).

2.1.2.2 CSHORE

The CSHORE model (Johnson 2012) has been found to estimate bluff and dune erosion during storms reasonably well (Baird 2013). Melby (2012) found agreement between measured and predicted erosion along open coast beaches was not good.

2.1.2.3 COSMOS

The COSMOS, developed by Nairn and Southgate (1993), and Southgate and Nairn (1993) model utilizes the 'energentics' approach where sediment transport is dependent mean velocity currents in both the bed and suspended boundary layers.

2.2 Erosion Assessment in Vicinity of Coastal Structures

The erosion assessment procedures that the Mapping Partner must complete for coastal structures are dependent on an assessment of the structure and the most likely resultant profile of the structure (intact, partially failed, or completely failed), as well as other factors. Complete guidance for the assessment of coastal structures can be found in the detailed guidance units for Coastal Structures.

If a coastal structure is determined to remain intact but will be inundated by the 1-percent-annual-chance flood, the ground profile landward of the structure (including any primary frontal dune (PFD) identified) must be evaluated for storm-induced erosion. Because the structure remains intact, application of standard erosion procedures for an open coast setting may not be appropriate. However, some amount of erosion, both seaward and immediately landward of the structure may be appropriate; the Mapping Partner should exercise professional judgement in constructing the eroded profile.

If the assessment of a coastal structure indicates a failed profile is more appropriate, the Mapping Partner must determine whether the structure will completely or partially fail during the base flood. When failure will be complete, the Mapping Partner shall remove the structure entirely from the analysis transect. The remaining soil profile should be altered to achieve its likely slope immediately after structure failure. Information on slopes behind failed structures is limited. These slopes may vary from 1:100 (v:h) for unconsolidated sands to 1:1 or steeper for consolidated material landward of the failed structure. The post-failure slope for this analysis should be in the range of 1:1 to 1:1.5. Note that the post-failure slope may not necessarily match the long-term stable slope, but will serve as the basis for subsequent site-specific, event-based, erosion, wave height, wave runup, and wave overtopping analyses. If the Mapping

Partner determines that the structure will partially fail, storm-induced erosion must still be evaluated both seaward and landward of the structure. Historical post-storm surveys and/or photographs of analogous structure failures, where available, may be the best indicator of the failed profile.

2.3 Mixed/Coarse Sediment Systems

Beaches armored with cobbles, gravel, or other coarse sediments develop in two distinct coastal environments. Mixed-sediment beaches are generally found where a bluff is slowly eroding landward and contributing a consistent source of course sediment to the system, or areas in which the shoreline is exposed to high-energy waves that prevent fine sediment from depositing.

The Mapping Partner should review historical shoreline change data and/or collect field observations for the mixed sediment beaches to evaluate if they erode significantly during individual storm events. If these beaches are stable, no erosion modeling is required. Conversely, if the beach is dynamic and responds significantly to storm events, the erosion potential should be considered for the response evaluation of individual storms. Storm induced erosion should be based on post storm profile data, where available. If storm induced response is unknown, some amount of erosion should be considered by the Mapping Partner.

2.4 Beaches Backed by Erodible Bluffs or Cliffs

1. Where bluffs or cliffs are vulnerable to significant wave energy, erosion at the toe of the feature will generally result in a failure of the upper face and a landward retreat. Once the Mapping Partner has determined that a bluff or cliff is susceptible to erosion it is important to investigate the coastal setting and history of episodic and chronic bluff erosion for the study area. The following is a general process for erosion analysis and assessing wave hazards (note this procedure is not currently recommended for application along the Great Lakes): Obtain reliable beach and bluff profile data (surveyed cross-shore profiles or Light Detection and Ranging (LiDAR) data) for existing conditions. Try to obtain these data near the end of the winter season (for example, in March or April).
2. Determine whether bluff erosion and failure monitoring data are available for the study area. Obtain and examine that information to determine the magnitude of episodic toe erosion and bluff retreat.
3. Estimate top-of-bluff elevations and compare to potential significant storm TWL and whether the bluff is subject to overtopping or frequent wave attack or toe erosion.
4. Perform a site inspection to confirm general historical information related to episodic erosion or overtopping hazards associated with the site. Determine relative erodibility of the bluff materials using standard geologic/geotechnical field procedures (Sunamura 1983; USACE-LAD, 2003; and Williams et al., 2004).
5. If potential damage to structures or public safety are determined not to be significant, the Mapping Partner shall document those results and whether further analyses are recommended.

6. If further analysis of bluff erosion or overtopping is not recommended, or the site is determined to be non-eroding, assume that the bluff or cliff is non-eroding during large events.
7. Perform all further runup and overtopping analyses on the surveyed existing winter conditions beach and bluff profiles for the site.
8. Document results, and summarize the data, methods used, and assumptions associated with the analyses.

2.4.1 Detailed Bluff Erosion Analyses

Given wave and TWL characteristics and the erodibility of buff materials, the statistical bluff failure model estimates bluff toe erosion induced by impinging waves and predicts random episodic bluff failures for varying storm conditions. A semi-empirical formulation developed by Sunamura (1982, 1983) is used to quantify the short-term bluff toe erosion rate as a function of the intensity of impinging waves and the site-specific erosion resistance of bluff materials:

$$X = \sum_{i=j}^{N} X_i = \sum_{i=j}^{N} k\left(C + \ln\frac{\rho g H_i}{S_c}\right)\Delta t_i \qquad (2\text{-}15)$$

where X is the accumulated bluff toe erosion depth from N waves acting on the bluff toe, X_i is the individual erosion by the i-th wave with height of H_i and duration of Δt_i, S_c is the compressive strength of the bluff material, ρ is the density of water, g is the gravitational acceleration, C is a non-dimensional constant, k is a constant with dimensions of length over time [L/T], and subscript j is the group number of the critical wave height H_j to initiate the toe erosion, which is given by $H_j = S_c e^{-c} / \rho g$.

This procedure requires regional and site-specific data. A statistical Monte Carlo simulation approach is used to characterize the correlation between bluff toe erosion and bluff failure for temporally varying wave conditions. If the cumulative depth of the bluff toe notch induced by storm waves exceeds a locally determined threshold value for triggering a bluff failure, the individual upper bluff retreat is determined by a randomly selected retreat value from an historic database for the site. The threshold value is empirically determined from historical bluff failure events. It may vary from one coastal bluff region to another.

The methodology may be applied in any situation where undermining of the bluff toe triggers upper bluff block failure; however, substantial field data are required to determine several of the required parameters and for proper calibration of the bluff failure model. Therefore, if the Mapping Partner determines that a detailed bluff erosion study is necessary, he/she must provide the following field data, at a minimum:

1. The type of rock formation and/or bluff soil materials from which stability and the erosion-resistant force of the bluff material can be quantified.
2. Field measurements of bluff toe erosion in response to cumulative wave energy associated with past storm events for determining and calibrating empirical coefficients required by the Sunamura formula used by the model.

3. Historical data of upper bluff failures, indicating approximate horizontal length and transverse width of bluff top land loss during past storm events for formulating the probability distribution of the severity of bluff failure.

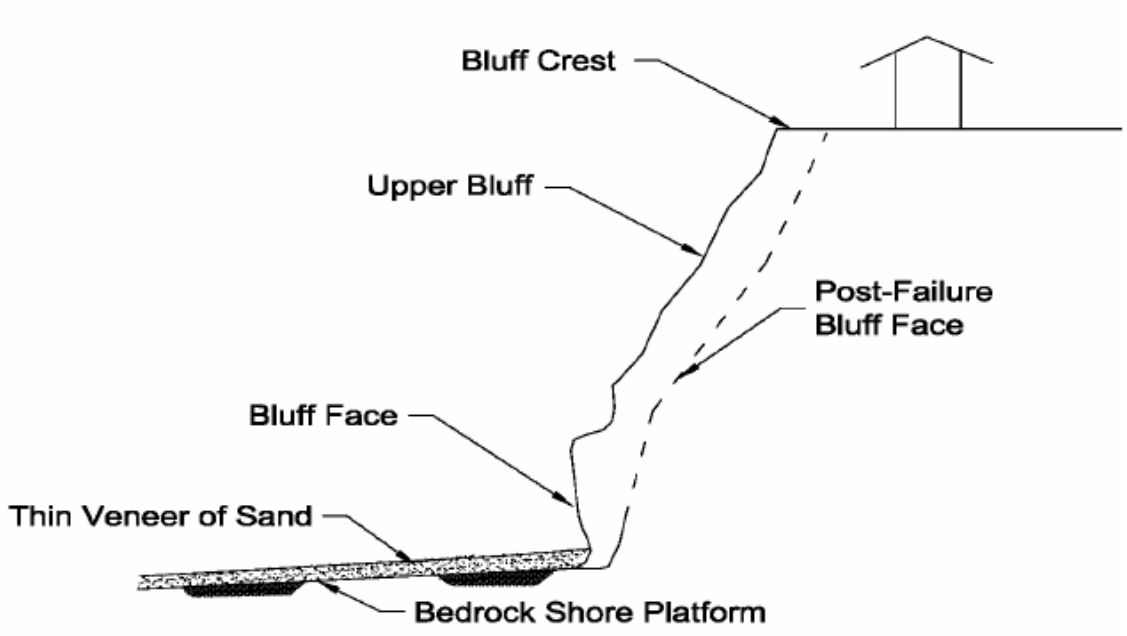
Two sets of field data are required to establish and calibrate the wave-induced toe erosion and to establish the statistical representation of upper bluff failure events. To calibrate the toe erosion produced by the Sunamura model, the depth of the toe erosion shall be measured before and after significant storm events and correlated to the cumulative wave energy during those events at the bluff toe. At least two full years of data are required to capture seasonal variability of toe erosion, and up to five years of data may be needed to calibrate the correlation between the impinging waves and the resultant toe erosion. Longer monitoring periods are desirable and will include more storm events and more cumulative wave energy statistics, and thus result in higher accuracy in model calibration.

To assemble the representative statistics of episodic bluff failure, adequate observations of upper bluff failures are required. At least two to five years of monitoring data are required to provide a reasonable representation of the size distribution of the failures. The larger the database, the less uncertainty there is in the predicting upper slope failure. There are no known analytical methods for forecasting bluff toe erosion and failure; therefore, a statistical approach is the only means of forecasting bluff failure and retreat due to the random temporal wave action during large storms. To capture any seasonal variability, at least 5 years of data are required, and to ensure a statistically valid database, up to 10 years of data may be needed if failures are uncommon. This is likely to limit the applicability of this approach for traditional FEMA coastal flood studies, unless these data are readily available at the beginning of the project. If it is determined that data are available and that the application of the statistical bluff failure model is necessary, use the following procedures:

2.4.1.1 Characterize Fronting Beach Conditions

The Mapping Partner shall perform the steps 1 through 6 listed at the front of this section, and assess whether the potential damage to the bluff-top developments resulting from bluff failure is highly probable, or not. If a subject bluff is fronted by either a sand berm or dune with a sufficient width to separate the bluff from direct wave impingement during the winter months, the storm-induced erosion to the berm and dune should be applied. If the protective sand berm or dune is typically removed during the winter months, the eroded condition should be used as the winter beach profile and the bluff failure model should then be subsequently used. A sketch of a typical erodible bluff fronted by a rock platform capped with a thin sand layer is shown in Figure 2.4.1-1.

Figure 2.4.1-1. Typical Erodible Bluff Profile Fronted by Narrow Sand-Capped Beach



2.4.1.2 Application of the Bluff Failure Model

The following are procedures for applying the statistical bluff failure model:

1. Collect field data for each setting and subreach along the study area.
    a) Measure bluff toe erosion (notching) from wave attack during at least two separate periods.
    b) Conduct field probing to determine the bedrock layer across the beach area.
    c) Determine the intersection of the bluff toe and bedrock layer and the cross-shore slope.
2. Assemble historical upper bluff failure events.
    a) Determine bluff failure characteristics in terms of retreat distance.
    b) Formulate the cumulative probability distribution of the magnitude of various bluff failure events.
    c) Determine the threshold value of toe notch depth when the upper bluff failure event is triggered (see USACE, 2003; Williams et al., 2004).
3. Calibrate Sunamura's empirical equation.
    a) Determine the wave conditions during storm events within the two separate historical wave and erosion periods.
    b) Estimate the temporal histogram of breaking wave heights at the bluff base for each of the selected periods with synchronized tide levels.
    c) Determine the bluff resistance force for the type of bluff material at the site.

d) Calibrate Sunamura's empirical equation (i.e., Equation 2-15) from the cumulative toe erosion measured in these two separate periods and quantify the total impinging wave energy during each period from hindcast data.

4. Calibrate bluff retreat model by simulating past bluff failure events.
    a) Assemble historical wave characteristics at the bluff base and synchronize with measured tide levels.
    b) Determine the probability distribution of wave characteristics at the bluff base.
    c) Apply the Monte Carlo sampling technique to randomly select the histogram of wave characteristics at the bluff base.
    d) Estimate the cumulative notch depth at the bluff toe using the calibrated toe erosion equation (Equation 2-15).
    e) Apply the same Monte Carlo sampling technique to randomly select a bluff failure event if the accumulative notch depth is deeper than the prescribed threshold value deduced from Step 2. Assemble historical bluff failure events.
    f) Perform multiple simulations for a required long-term duration (e.g., 10 years) until a statistical representation regarding the occurrence of bluff failure is achieved.
    g) Derive the statistical mean and other pertinent properties, such as the exceeding probability of a cumulative bluff retreat distance at the end the modeled duration.
    h) Compare results with observed data from the site and adjust coefficients as necessary.
5. Apply calibrated model for 1% annual storm event.
    a) Determine winter profiles for fronting beach conditions and elevation of bluff-beach intercept.
    b) Apply calibrated model for entire 1% annual storm.
    c) Determine amount of toe erosion and bluff crest line recession for the 1% storm.
    d) Use this adjusted profile for all further runup and overtopping analyses associated with the 1% annual storm.
6. Document results, data, and assumptions.

2.5 Estimating Beach Profiles for Beaches Backed by Erosion-Resistant Bluffs or Cliffs

Erosion-resistant bluffs and cliffs are often fronted by rock terraces, rocky beaches, or narrow rock platforms capped with thin layers of sand or gravel. Once the thin sand cap is eroded from the rocky beach, this beach setting is stable. Therefore, Mapping Partners shall assume the sand cap is removed from the beach profile before a significant storm event and use the adjusted rocky beach profile along with measured profiles for the non-erodible bluffs or cliffs for all subsequent runup and overtopping computations. All assumptions, methods, data resources, and results should be well documented.

2.6 Tidal Mudflats and Wetlands

Mapping Partners may assume that tidal mudflats and wetland profiles do not erode over the time-scale of a single storm event. Mapping Partners should compare existing tidal flat and wetland profiles with recent post-storm profiles to verify this assumption.

3.0 Miscellaneous Components

3.1 Estimating Grain Size

To successfully solve the time convolution component of the K&D equilibrium profile model, the study contractor must estimate the sediment grain size. This characterizes the beach and dunes capacity to withstand wave attack and erosion. The preferred approach is to measure D_{50} from field samples taken from a particular analysis site. In the absence of detailed field data on the lakebed geology, an analysis of profile geometry can be used to estimate lakebed substrate type and transitions from mobile sand and gravel deposits to hard bottom (e.g. bedrock) or consolidated sediment (e.g. glacial till). The equilibrium beach concept has been used extensively to describe profile shapes over nearshore regions with a wide variation in sediment characteristics. Analyses of many beaches (e.g. Dean 1977) have indicated the applicably of a simple expression for the subaqueous profile:

$$d = Ax^{2/3} \tag{3-1}$$

where d is the water depth, A is a shape parameter, and x is a cross-shore coordinate, positive offshore with the origin at the still-water shoreline. Dean (1991) provided the theoretical basis for the concave profile shape, Equation (3-1), based on the assumptions of linear saturated waves and uniform energy dissipation. Applicability, therefore, is limited to the active surf zone. Available profile data can be used to determine the optimal shape parameter through an error minimization. Consider a single transect comprised of equally spaced discrete points extending from the still water shoreline to the edge of the surf zone. An analysis minimizing the root-mean-squared error between data and the analytical equilibrium beach yields an estimate for the shape parameter

$$A = \frac{\overline{d_i x_i^{2/3}}}{\overline{x_i^{4/3}}} \tag{3-2}$$

where the over-line depicts averaging across all points in the surf zone.

In general, it is noted that smaller sand sizes are associated with mildly sloping beaches and a smaller shape parameter. Empirical relations between the shape parameter and sediment characteristics have been developed, and the most widely-cited expressions indirectly relate A to the sediment size through the fall velocity w_f. Dean (1991), for instance, proposed

$$A = 0.067 w_f^{0.44} \tag{3-3}$$

where the units for A and w_f are $m^{1/3}$ and *cm/s* respectively. On the other hand, Kriebel et al. (1991) proposed

$$A = 2.25(w_f^2/g)^{1/3} \tag{3-4}$$

which is valid for any units. The difference between the two formulas for *A* is less than 30 percent for sands with w_f = 1–10 cm/s.

Equations that relate the fall speed of natural sediments and grain size are written as explicit expressions for w_f and are not, in general, easily inverted. For example, one widely-used expression due to Soulsby (1997) is given as

$$w_f = \frac{\nu}{d}\left(\sqrt{10.36^2 + 1.049 d^3 g \frac{s-1}{\nu^2}} - 10.36\right) \tag{3-5}$$

where v is the kinematic viscosity, *d* is the grain diameter, *g* is the acceleration of gravity, and *s* is the sediment specific gravity. Equation (3-5) is readily solved for the fall velocity with a given sediment diameter. Solving the inverse relation, however, requires an iterative method for determining *d*.

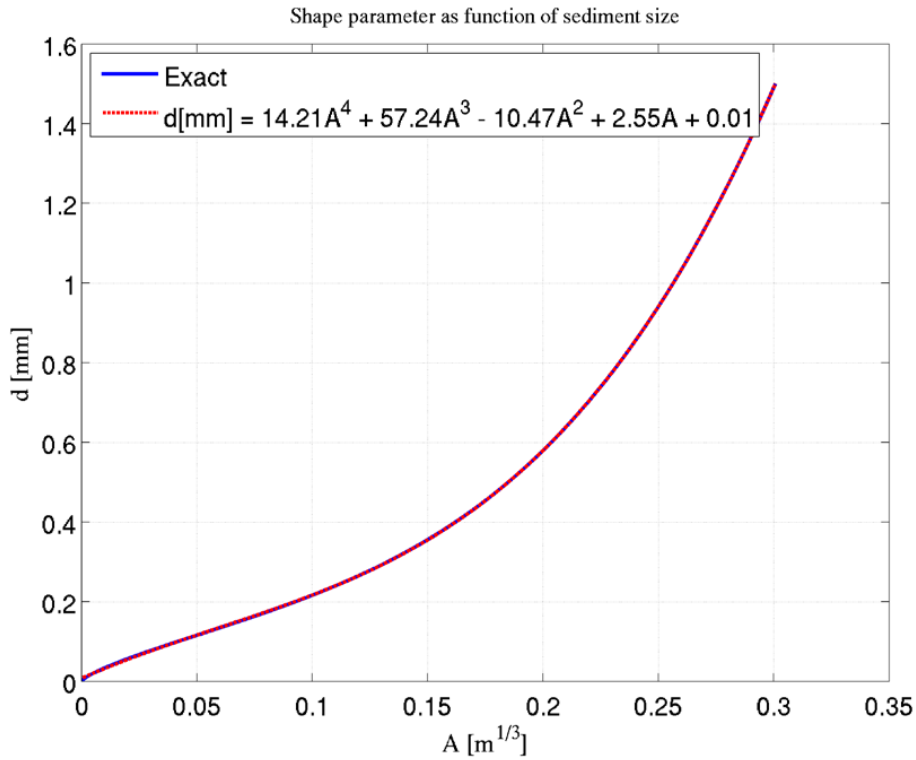
Simplified Approach

A practical and accurate method for grain size determination can be developed by approximating the relations with a fitted curve. Figure 3.1-1 depicts the exact relationship of *A* and sediment size, making use of (3-4) and an iterative solution of (3-5). Also shown is an explicit empirical polynomial curve for sediment size

$$d_{mm} = 14.21A^4 + 57.24A^3 - 10.47A^2 + 2.25A + 0.01 \tag{3-6}$$

where d_{mm} is the sediment diameter with units of *mm*. Equation (3-6) is easily applied to determine the characteristic sediment grain size when an optimized shape parameter is determined from measured data. No significant error is introduced by using the provided empirical relationship, but the application should be limited to $A < 0.3\ m^{1/3}$ to remain within the fitted domain.

Figure 3.1-1. Exact and approximate relationship between shape parameter A and characteristic sediment size (solid blue line and dashed red line are equal)



3.2 Beach Morphology Change in Response to Lake Level Cycles

When investigating the individual flood response for storm events, the Mapping Partner should investigate the degree of profile change that has occurred historically due to fluctuating lake levels. Due to a general lack of mobile coarse grained sediment (sand and gravel) for cohesive and bedrock shorelines, these changes in beach width are not anticipated for most of the bank/bluff sites.

For sites where the data collection campaign was conducted at a lake level that is similar to the level required in modeling the historical storms for the response-based approach, the data can likely be used without modification. However, if the lake level for the historical storm is significantly different from the conditions during the data collection, it may be necessary to modify the bathymetry and beach conditions on the profile before subsequent analyses.

For cases where lake level changes are significant, it is advisable to consider alterations to the bathymetry and beach volume used for model initialization. It is therefore advised to use methods based on simple mass balance relationship if changes to the beach and lake bottom position are required. The Bruun Rule (Bruun 1962), for instance, could be used to estimate the lake level difference from the data collection period to the actual storm being simulated.

3.3 Determining MLWP for K&D and MK&A Models

First, determine existing shoreline location and conditions. Then, establish representative reaches within the shoreline area being analyzed that are similar in coastal morphology (average offshore/nearshore bathymetry, wave exposure, onshore beach slope, beach materials, etc.). This may consist of only one typical reach or several different typical reaches for the shoreline area being analyzed. Following are procedures for establishing the MLWP for a sandy beach backed by either a low sand berm or a high sand dune for application with the K&D geometric model.

1. **Procedure for a Study Site Without Previously Surveyed Historical Profiles**

    1. Determine existing shoreline location and conditions.
    2. Always use measured historical post-storm winter beach profile data when available to establish the MLWP. However, if there are no historical post-storm winter beach profile data, conduct a basic wading survey from the crest of the berm or dune to the approximate mean low low water (MLLW) line (see National Oceanic and Atmospheric Administration [NOAA] tidal datum) following a series of winter storms in March or April to prepare a surveyed beach profile from the berm crest to approximately MLLW.
    3. Collect sediment samples, preferably in late March or early April to determine the median sand diameter (D_{50}) for use in Equations 2-7 and 2-8.
    4. Determine the MSL from NOAA's tidal datum, and identify the MSL location across the beach profile. This location divides the beach profile into an upper foreshore berm/dune section and the surf zone section.
    5. Plot the measured upper foreshore profile section above the MSL line based on the basic wading survey, site photographs, and available historical information (see Figure 2.1.1-4).
    6. Determine the berm or dune height (*B* or *D*) above the MSL line and foreshore slope (*m*) from the estimated upper foreshore section (see Figure 2.1.1-4).
    7. Approximate the surf zone section of the MLWP from Kriebel and Dean's equilibrium beach profile, based on the measured D_{50} and the application of Table 3-2 and Equations. 2-7 and 2-8.
    8. Assemble the entire MLWP based on the surveyed upper foreshore and surf zone sections linked at the MSL, as illustrated in Figure 2.1.1-4.
    9. Document data sources, assumptions, and conversions.

2. **Procedure for a Study Site With Previously Surveyed Historical Profiles:**

    1. Determine existing shoreline location and conditions.
    2. Select a representative surveyed winter profile (see Figure 2.1.1-5) from historical post-storm beach profile data, which was surveyed during the end of the winter season (March-April) and represents the typical winter beach profile conditions.

3. Determine the MSL from NOAA's tidal datum, and identify the MSL location across the beach profile. This location divides the beach profile into an upper foreshore berm/dune section and the surf zone section.
4. Determine the berm or dune height (*B* or *D*) above the MSL line and compute the foreshore slope (*m*) through linear curve fitting to the upper foreshore section.
5. Determine the surf zone section of the MLWP by curve-fitting Kriebel and Dean's equilibrium profile from Equations. 2-7 and 2-8 to the surf zone section of the surveyed beach profile below the MSL line (see Figure 2.1.1-5).
6. Assemble the entire MLWP based on the approximated upper foreshore and surf zone sections linked at the MSL, as illustrated in Figure 2.1.1-5.
7. After the MLWP is defined, determine the K&D model input parameters including the berm or dune height (*B* or *D*) and the foreshore slope (*m*) (see Figure 2.1.1-4 and Figure 2.1.1-5).

4.0 Sheltered Waters

Sheltered waters refers to bays, sounds, estuaries, fjords, and other water bodies that are hydraulically connected to open coast waters during flood conditions. Sheltered water shorelines can be exposed to the same types of flood-producing processes as open coastlines (i.e., high winds, storm surge, wave generation and overland propagation, wave runup, and wave overtopping). However, sheltering implies the inland propagation of open coast flooding and a modification of these processes by land masses or other obstructions. In many cases, sheltered water shorelines are subject only to locally generated waves. For some geometries, the distinction between open coast and sheltered waters may not be evident. In such cases, the Mapping Partner should discuss the most appropriate approach with the FEMA Project Officer.