# FEMA Coastal Flood Hazard Analysis and Mapping

# PHASE 1 SUMMARY REPORT

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Prepared for

FEMA

A Joint Project by

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## Acronyms

| | |
|---|---|
| 1-D | one-dimensional |
| 2-D | two-dimensional |
| ACES | Automated Coastal Engineering System |
| ADCIRC | Advanced Circulation Model for Coastal Ocean Hydrodynamics |
| BFE | Base Flood Elevation |
| CCM | Coastal Construction Manual |
| CDIP | Coastal Data Information Program |
| CEM | Coastal Engineering Manual |
| CERC | Coastal Engineering Research Center |
| CFR | Code of Federal Regulations |
| CHAMP | Coastal Hazard Analysis Modeling Program |
| CHL | Coastal Hydraulics Laboratory |
| DFIRM | Digital Flood Insurance Rate Map |
| DHI | Danish Hydraulic Institute |
| EBE | Event-Based Erosion |
| EST | Empirical Simulation Technique |
| FEMA | Federal Emergency Management Agency |
| FIRM | Flood Insurance Rate Map |
| FIS | Flood Insurance Study |
| FNWC | Fleet Numerical Weather Center |
| *G&S* | FEMA *Guidelines and Specifications for Flood Hazard Mapping Partners Appendix D: Guidance for Coastal Flooding Analyses and Mapping* |
| GIS | Geographic Information Systems |
| GROW | Global Re-analysis of Ocean Waves |
| HURDAT | digital file of storm data for all identified tropical storms in the North Atlantic Ocean |
| IAHR | International Association of Hydraulic Engineering and Research |
| JPM | Joint Probability Method |
| LIDAR | Airborne Light Detection and Ranging |
| NFIP | National Flood Insurance Program |
| nhc | Northwest Hydraulics Consultants |
| NOAA | National Oceanic and Atmospheric Administration |

| | |
|---|---|
| NOS | National Ocean Service |
| NTHMP | U.S. National Tsunami Hazard Mitigation Program |
| PC | Pacific Coast |
| PFD | Primary Frontal Dune |
| PTHA | Probabilistic Tsunami Hazard Analysis |
| PWA | Philip Williams & Associates |
| SC | Study Contractor |
| SEM | Spectral Energy Model |
| SF | Square Feet |
| SFHA | Special Flood Hazard Area |
| SPM | Shore Protection Manual |
| TWG | Technical Working Group |
| USACE | U.S. Army Corps of Engineers |
| USGS | U.S. Geological Survey |
| WHAFIS | Wave Height Analysis for Flood Insurance Studies |
| WIS | Wave Information Studies |

## Listing of the Technical Working Group

The following individuals (listed alphabetically) participated in the Technical Working Groups to prepare Focused Studies, attend workshops and prepare reporting for the Phase One Summary Report.

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# 1 EXECUTIVE SUMMARY

## 1.1 PURPOSE OF STUDY

The Federal Emergency Management Agency (FEMA) is responsible for preparing Federal Insurance Rate Maps (FIRMs) that delineate hazard zones and Base Flood Elevations in coastal areas of the United States. These areas are among the most densely populated and economically important areas in the nation. Coastal areas are subject to a variety of natural processes that result in significant hazards to public safety and property along the nation's coastlines, including extreme conditions of storm surge flooding, waves, erosion, rainfall, and wind. The purpose of this study is to evaluate existing FEMA procedures for delineating coastal flood hazard areas in three major coastal regions of the United States (Atlantic, Gulf, and Pacific) and to develop recommended new guidelines and procedures in one of these areas (Pacific).

This project was authorized cooperatively by FEMA Headquarters, FEMA Region IX, and FEMA Region X in October 2003. The project is managed by Les Sakumoto, Project Officer for FEMA Region IX. Northwest Hydraulic Consultants, Inc. is the lead consultant and manager of the Technical Working Group. This Phase 1 Summary Report provides a brief background on the project approach; describes the process for evaluating existing guidelines; and summarizes the recommendations for the Pacific, Atlantic, and Gulf Coasts. Appendices to this report include information on the Technical Working Group, key references, and Focused Studies on 11 categories of technical topics.

## 1.2 PROJECT CONTEXT AND GOAL

Approximately 50 percent of the population of the United States resides on or near the coast (less than 50 miles from the coastline). More than 3,000 communities are located in this 12,000-mile-long coastal zone, which is covered by approximately 7,400 existing FIRM panels. Much of this inventory of coastal FIRMs is more than 20 years old. Faced with maintenance of the present inventory and creation of new FIRM panels, FEMA began an ambitious plan for Map Modernization in 1997. Congress approved a FY 2003 budget that included a significant increase for funding the Map Modernization Plan, and FEMA has placed a high priority on coastal flood hazard mapping.

In considering the needs of Map Modernization in coastal areas, FEMA recognized the need for a comprehensive review of procedures that will be used to identify coastal flood hazards. This review is needed to consider advances in coastal flood hazard assessment and mapping that might be accomplished based on the current state-of-the-art in scientific understanding of coastal processes, new technology and numerical modeling techniques, improved and expanded data, and modern mapping techniques.

The goal of this project is to incorporate recent advances in the sciences and in coastal engineering into a recommended approach for improved coastal flood hazard mapping, based on an understanding of local and regional coastal processes.

## 1.3 DESCRIPTION OF NEEDS BY GEOGRAPHIC REGION

Guidelines and Specifications for Flood Hazard Partners Appendix D: Guidance for Coastal Flooding Analyses and Mapping (*G&S*) for the Atlantic Coast, Gulf Coast, and Great Lakes have been assembled from elements developed over the course of many years; however, no comprehensive assessment has been done to evaluate their effectiveness in hazard mapping for the Atlantic and Gulf Coasts. During this time, the Pacific Coast was recognized as a special case because of differences in coastal processes (e.g., tsunamis, El Niño) and geomorphic characteristics, but no FEMA guidance was established specifically for this coast.

### 1.3.1 Pacific Coast

The present *G&S* do not address the Pacific Coast as noted in Section D.4, "No FEMA guidance documents have been published for Pacific Ocean coastal flood studies. Guidance is to be developed based on existing methodologies recommend by FEMA coastal states for coastal analyses in the Pacific Ocean." The existing guidelines focus on storm types and coastal processes that are relevant to the open coast settings of the Atlantic and Gulf Coasts. The Pacific Coast is subject to storm types, wave conditions, and coastal processes that differ from those in other coastal regions of the country. Therefore, much of the existing guidance is not directly transferable to the analysis of Pacific Coast coastal flood hazards. An assessment of the existing guidance is needed to determine which portions may be transferred or modified for use on the Pacific Coast and what new procedures are needed. In general, the FIRMs for the Pacific Coast of the United States are more than 20 years old. These maps require comprehensive updating to adequately define hazard zones in some of the most densely populated and fastest growing areas of the United States.

### 1.3.2 Atlantic and Gulf Coasts

The procedures in the existing guidelines can benefit from a comprehensive review considering more recent experience and new technology. Modified or new procedures may be needed to incorporate experience from previous studies and appeals, information on actual damages, and post-storm verification data. In addition, the basis of existing procedures should be reviewed with an improved understanding of ocean and coastal processes from recent research and data. The existing procedures include little guidance on analysis of storm meteorology, storm surge, or wave setup. The existing guidance also may need expansion to address flood hazards in coastal areas not directly exposed to ocean swell and storm seas (e.g., bays and estuaries, referred to as Sheltered Waters in this report)

### 1.3.3 Other Areas

The review and update of the guidelines are intended to facilitate consistent and accurate mapping of coastal flood hazards in the Map Modernization Plan. Because of the unique coastal processes in Alaska, Hawaii, the Great Lakes, Caribbean islands, and Pacific islands, the project focuses on guidelines for the oceanic coastlines of the conterminous United States. It is anticipated that many of the identified procedures will be transferable to these other areas but that additional work will be required to address unique physical characteristics and processes in each of these regions.

## 1.4 PROJECT APPROACH AND SCHEDULE

The project approach includes two key elements to ensure that the project can be completed rapidly and effectively: (1) assembling a team of technical experts (Technical Working Group, or TWG) with experience in various coastal processes and their effects in different geographic regions of the country and (2) conducting the project in two phases—Phase 1 to evaluate the existing guidelines for all three coasts and Phase 2 to develop proposed new draft guidelines for the Pacific Coast.

The TWG is comprised of coastal experts from private industry, academic and research institutions, federal agencies (National Oceanic and Atmospheric Administration, U.S. Army Corps of Engineers, and U.S. Geological Survey), Flood Insurance Study (FIS) contractors, map coordination contractors, and FEMA Headquarters and regional engineers. The TWG includes members from all three coastal regions of the United States and from Europe. This group was organized to implement a collaborative approach to identify the needs and priorities for improved coastal flood hazard mapping procedures, consider potential alternatives, and develop recommendations.

The phased approach to the project allows updated, modified, and new procedures to be developed first for the Pacific Coast, where none are currently specified. Some of these procedures will be applicable with slight modification to study elements for the Atlantic and Gulf Coasts or to specific areas on these coasts. This approach provides an efficient use of new *G&S* developed for the Pacific Coast.

A thorough evaluation of the guidelines must be completed on a schedule that allows coastal mapping to proceed according to the Map Modernization Plan. Needed guideline improvements must be prioritized to maintain this schedule. Phase 1 was initiated in October 2003, and a final report is scheduled for June 2004.

During Phase 2, a draft set of *G&S* for the Pacific Coast will be produced, along with associated backup information and reports. The draft guidelines are scheduled for delivery to FEMA in September 2004. A final draft set of Pacific Coast guidelines is anticipated in October 2004. This schedule will allow coastal flood insurance studies to proceed with new draft guidance in fiscal year (FY) 2004/2005. This schedule requires an intensive work effort to complete a comprehensive review of existing procedures, make necessary modifications to existing procedures, develop new methods, and prepare *G&S*. This effort involves approximately 20 organizations and active participation of more than 50 individuals.

## 1.5 PHASE 1 TASKS

The approach for the assessment phase of the project (Phase 1) was to examine all technical areas of the coastal flood hazard mapping process. Initial tasks focused on a review of the existing guidelines and the needs and priorities for their improvement. Under these tasks, coastal experts from the TWG reviewed existing guideline methodologies for the ocean and coastal processes analyzed in flood insurance studies (e.g., storm meteorology, storm surge, wave setup, wave transformation, wave runup, and overtopping) and evaluated their applicability for each coastline. Case studies were prepared to demonstrate application of guideline methodologies in previous coastal flood insurance studies on each coast, and representative studies were prepared to demonstrate application of guideline procedures to particular coastal processes.

An international literature search was conducted to identify sources of information on existing and evolving coastal engineering practices and to identify pertinent scientific research that may be useful in developing new guidelines. The international experience of several TWG members was used during this task to provide the project with information, techniques, and practices from around the world.

The initial tasks described above served as the basis for reporting and discussion at Workshop 1, held in Sacramento, California, on December 2–4, 2003. The workshop was attended by 38 members of the TWG from across the country. The workshop agenda included:

- review of existing guidelines and practices;
- technical presentations on the state of the science in coastal processes;
- workshop sessions to identify needs, priorities, and potential guideline improvements by coastal geographic areas and coastal processes; and
- summary sessions to list and prioritize needed guideline improvements.

The primary result of Workshop 1 was a list of 53 technical topics for consideration in updating the guidelines. Each item also included an initial assessment of the time and data required to develop improved procedures. This assessment resulted in categorizing each topic as "Critical," "Important," "Available," or "Helpful." "Critical" and "Important" topics were considered the highest priorities for development of new or improved procedures, and were subdivided into topics that could likely be addressed in the 6-month time frame of the project ("Critical") and those that would require longer term development by FEMA ("Important"). "Available" topics were considered areas where existing data or methodologies were readily available for updating or creating guidelines. "Helpful" topics were considered valuable but lower priority. These priority classes were assigned by the TWG for each topic on the Atlantic and Gulf Coasts, Pacific Coast, and in Sheltered Waters (Non-Open Coast).

The results of Workshop 1 were used to formulate Focused Studies that organized the 53 technical topics into 11 categories according to coastal processes and coastal flood hazard mapping procedures. Each of these 11 categories became the subject of a Focused Study:

1) Storm Meteorology
2) Stillwater Elevations
3) Storm Wave Characteristics
4) Wave Transformation
5) Wave Setup
6) Wave Runup and Overtopping
7) Event-Based Erosion
8) Coastal Structures
9) Tsunami
10) Sheltered Waters
11) Hazard Zones

These Focused Studies are included in the Appendices to this report.

The focused studies were conducted by groups of individuals from the TWG, each coordinated by a Focused Study leader. This organization allowed the 11 Focused Studies to be completed simultaneously and rapidly. Preliminary drafts of the Focused Studies were presented at Workshop 2 on February 23–26, 2004, and subsequently were refined by the study groups.

The Focused Studies contain recommendations on the approach for updating the guidelines on three coasts (Pacific, Atlantic, Gulf). These recommendations include further studies and guideline development work that vary in complexity, level of effort, and time requirements. The level of effort required to complete the recommendations for "Critical" and "Available" items identified in Workshop 2 significantly exceeded the available time and budget for Phase 2 (Pacific Coast guidelines). Therefore, in March, the project team engaged in a significant effort to develop options for limiting the scope and cost of Phase 2 work while retaining the most important topics and a balance among the 11 technical categories. The selected option defers some recommendations for future development in the National Flood Insurance Program (NFIP) but maintains the target of producing reliable guidelines for coastal studies on the Pacific Coast in FY 2004/2005.

## 1.6 SUMMARY OF PHASE 1 FINDINGS

A complete list of topics and recommendations developed by the TWG during Workshops 1 and 2 is provided in Table 2 and the Focus Studies in the Appendices. The following are a few of the key findings from the Phase 1 activities:

- Procedures are needed to compute the 1% annual chance flood elevation where 1% stillwater levels do not necessarily coincide with 1% wave conditions (e.g., the Pacific Coast and sheltered waters along all three coasts).
- Procedures to better represent wave setup are needed on all coasts
- Procedures should be developed to use regional databases and wave transformation models to develop wave spectra at the surf zone.
- Methods are needed to evaluate the amount of wave dissipation due to propagation over muddy or flat nearshore areas.
- Procedures to quantify the effects of wave setup and event-based erosion in a variety of geomorphic settings are needed.
- On the Atlantic Coast, a review of the 540 square-foot erosion criterion is needed considering new data; on the Pacific Coast, a similar geometric method is needed based on Pacific Coast data.
- A probabilistic method for tsunami hazard assessment and methods for combining tsunami hazards with other coastal hazards are needed.
- Updates and amplification of existing guidelines for wave runup and overtopping and associated hazard zones are needed. Improved methodology for wave overwash is needed.
- Some coastal processes, such as surge, wave transformation, and tsunamis, are best analyzed at a regional scale rather than in flood studies of individual communities.
- Sheltered waters (non-open coast areas) require specialized guidance because of their unique hydrodynamic and geomorphic characteristics compared to the open coast. For example, new

methods for calculating fetch-limited wind waves should be evaluated and incorporated in guidelines, to the extent appropriate.

## 1.7 RECOMMENDED APPROACH FOR PHASE 2

Recommended approaches to address these and other needs are included in Sections 4 and 5 of this report. A portion of these recommendations will be implemented in Phase 2 to prepare guidelines for the Pacific Coast. The guidelines developed in Phase 2 will be designed to address the following general requirements:

- consideration of geomorphic settings and their relationship to required analysis, including clear distinction between the open coast and sheltered water settings;
- development of alternative procedures for defining the 1% percent annual chance flood elevation where 1% stillwater and 1% wave conditions do not necessarily coincide, and consistency in their application to multiple analyses in a coastal study; and
- identification of analyses that may best be accomplished at regional scale (e.g., tsunami analysis, wave transformation), and the appropriate input to local analyses and hazard mapping.

Phase 2 includes limited case studies in the following areas to develop and test new procedures and to develop simple models designed specifically for use in FEMA flood insurance studies:

- Storm Meteorology – testing to develop procedures for 1% flood elevation determination based on wave and water level combinations in open coast and sheltered waters settings
- Stillwater Elevations – testing for procedures to extract surge data from tide gage data; development of a simplified surge model for the Pacific Coast
- Wave Characteristics – case study to develop wind field and other input data specifications and methods for application of spectral models
- Wave Transformation – testing of wave transformation models
- Wave Setup – testing of Boussinesq models; development and testing of new setup model
- Runup and Overtopping – runup model testing combined with 1% flood elevation testing in Storm Meteorology
- Event-Based Erosion – testing of geometric models and procedures

A case study is also recommended by the TWG to develop a probabilistic methodology that considers both near-field and far-field sources of tsunamis. This case study will be accomplished outside the scope of the current project because of the highly specialized nature of the required analyses. This case study is expected to be accomplished through interagency cooperation among FEMA, the National Oceanic and Atmospheric Administration, and the U.S. Geological Survey, with assistance from private consultants and research institutions, such as the University of Southern California.

Some “Critical” and “Important” topics were identified for the Pacific Coast that will not be addressed in Phase 2 because of limited time and resources. The Focused Studies provide background on these topics, and Section 4 of this report provides a brief summary that can be used for planning of future guidance development by FEMA.

No additional work will be performed for the Atlantic and Gulf Coasts in this project. Section 5 of this report provides a brief summary of recommendations that can be used for planning future guidance development by FEMA. In addition, some Pacific Coast guidelines to be developed in Phase 2 may be applicable to analyses on the Atlantic and Gulf Coasts with little or no modification. The applicability of Pacific Coast guidelines in specific technical categories is identified in Section 5. The Focused Studies also provide reference information that may be useful to study contractors as a supplement to the existing guidelines.

The project approach has relied heavily on the collaboration of Technical Working Group members to meet a compressed schedule. This collaboration and interaction is a significant successful work product of the project, and is gratefully acknowledged.

# 2 INTRODUCTION

This section describes the project and its role in the FEMA Map Modernization Plan. It describes the need for a comprehensive review and update of coastal flood hazard analyses and mapping and provides a brief description of the overall project approach.

## 2.1 PROJECT DESCRIPTION

### 2.1.1 Overview – Map Modernization Plan and Coastal Flood Hazards

Federal law mandates FEMA to compile and update flood hazard maps for more than 19,000 communities nationwide. Because flood hazard conditions change over time due to natural and human-induced changes, FEMA has an ongoing program to update flood maps for floodprone communities. Over time, the needs for flood map updates have increased while federal funding to accomplish this has been limited. Therefore, a significant portion of the present flood map inventory is out of date, while newer communities may not have been mapped yet. To reverse this trend, FEMA prepared a Map Modernization Plan with the goal to upgrade the 100,000-panel national flood map inventory which includes both riverine and coastal areas. To accomplish this goal Congress approved a FY 2003 budget that included a significant increase for funding the Map Modernization Plan. FEMA plans to meet the Map Modernization goals by:

- Developing up-to-date flood hazard data for all floodprone areas, including coastlines nationwide, to support sound floodplain management and prudent flood insurance decisions;
- Providing the maps and data in digital format to improve the efficiency and precision with which mapping program customers can use this information;
- Fully integrating FEMA's community and state partners into the mapping process to build on local knowledge and efforts;
- Improving processes to make it faster to create and update the maps; and
- Improving customer services to speed processing of flood map orders and raise public awareness of flood hazards.

Approximately 50% of the population of the United States lives within 50 miles of the coast. There are more than 3,000 communities along 12,000 miles of coastline, and approximately 7,400 Flood Insurance Rate Map (FIRM) panels covering these coastal communities. Therefore, performance of coastal flood insurance studies and preparing updates to coastal flood hazard mapping are key elements in meeting Map Modernization goals for a large portion of the nation's population. The coastal flood insurance studies and updates to FEMA's new digital mapping format (DFIRM) require application of consistent, scientifically based analysis and mapping procedures. In considering the needs of Map Modernization in coastal areas, FEMA recognized the need for a comprehensive review of procedures that will be used to assess coastal flood hazards. This review is needed to consider advances in coastal flood hazard mapping that can be accomplished based on the current state-of-the-art in scientific understanding of coastal processes, new technology and numerical modeling techniques, improved and expanded data, and modern mapping techniques.

Existing procedures for coastal flood hazard analysis and mapping are described in Appendix D of *Guidelines and Specifications for Flood Hazard Mapping Partners* (FEMA, 2003). This project includes a comprehensive review of these procedures (referred to as guidelines or *G&S* in this report), resulting in a recommended approach for updates to Appendix D. The existing guidelines were written for the Atlantic Coast, Gulf Coast, and Great Lakes areas of the United States. There are currently no guidelines specifically for the Pacific Coast. The project, therefore, also includes preparation of new guidelines for the Pacific Coast.

### 2.1.2 Pacific Coast – Description of Needs

In general, the FIRMs for the Pacific Coast of the United States are more than 20 years old. These maps require comprehensive updating to adequately define hazard zones in some of the most densely populated and fastest growing areas of the United States. The existing guidelines focus on storm types and coastal processes that are relevant to the open coast settings of the Atlantic and Gulf Coasts. The Pacific Coast is subject to different storm types, wave conditions, and coastal processes than other coastal regions of the country. Therefore, much of the existing guidance is not directly transferable to the analysis of Pacific Coast flood hazards.

### 2.1.3 Atlantic and Gulf Coasts – Description of Needs

On the Atlantic and Gulf Coasts, the existing guidelines were developed over an extended period of time, and applied in flood insurance studies in a variety of geomorphic settings. The procedures included in the existing guidelines can benefit from a comprehensive review with more recent experience and new technology. Modified or new procedures may be needed to incorporate experience from previous studies and appeals, information on actual damages, post-storm flood hazard verification data, and new knowledge and technology. In addition, there is a need to review the existing guidelines and their basis in physical processes. An improved understanding of these ocean and coastal processes, based on recent research and data, may allow the analysis procedures in the guidelines to be linked more directly and accurately to these processes. Most recent coastal flood insurance studies have focused on updating the mapping based on analysis of local wave effects at the shoreline. The existing procedures provide little guidance on analysis of storm meteorology, storm surge, or wave setup. New and expanded guidance or regional analyses may be needed to update these areas. The existing guidance may also need expansion to address flood hazards in protected coastal areas (e.g., sheltered bays and estuaries).

### 2.1.4 Purpose Statement and Project Authorization

FEMA is responsible for preparing Federal Insurance Rate Maps (FIRMs) that delineate hazard zones in coastal areas of the United States. These areas are among the most densely populated and economically important areas of the nation. Coastal areas are subject to a variety of natural processes that result in significant hazards to public safety and property, including conditions of extreme rainfall, wind, waves, surge, and erosion. The purpose of this project is to evaluate existing FEMA procedures for delineation of coastal flood hazard areas in three major coastal regions of the United States (Atlantic, Gulf, and Pacific), and to develop recommended new guidelines and procedures in one of these areas (Pacific).

This project was authorized cooperatively by FEMA Headquarters, FEMA Region IX, and FEMA Region X in October 2003. The project is managed by Les Sakumoto, Project Officer for FEMA Region IX. Northwest Hydraulic Consultants, Inc. is the lead consultant and manager of the Technical Working Group.

### 2.1.5 Phase 1 Summary Report

This report was prepared to summarize the first phase of the project. The report provides a brief background on the project approach, describes the process pursued by the TWG to complete the evaluation of existing guidelines and recommend an approach to update them, and summarizes the recommendations for the Pacific, Atlantic, and Gulf Coasts. Appendices to this report include information on the TWG, Key References, and Focused Studies on 11 categories of technical topics.

## 2.2 PROJECT APPROACH

### 2.2.1 Scope – Pacific, Atlantic, Gulf Coasts

The scope of the project includes the three major coastlines (Atlantic, Gulf, and Pacific) of the conterminous United States. The evaluation of existing guidelines and development of procedures is expected to also have applicability in Alaska, Hawaii, and other Pacific and Caribbean islands. However, these areas are subject to unique coastal processes that cannot be adequately addressed in the timeframe of the project. Future development of procedures specific to these areas will be required, drawing on project results for the Pacific Coast.

The project approach includes two key elements to ensure that the project can be completed rapidly and effectively:

1) Assembling a team of technical experts with experience in various coastal processes and their effects in different geographic regions of the country; and
2) Conducting the project in two phases to first evaluate the existing guidelines for all three coasts, and then develop proposed new draft guidelines for the Pacific Coast.

### 2.2.2 Technical Working Group – A Collaborative Approach

The process of evaluating and developing guidelines for coastal flood hazard delineation requires a combination of high technical knowledge, practical experience, and familiarity with FEMA regulations and procedures. Few individuals or organizations possess the capabilities to address the range of technical challenges associated with the diverse processes affecting the three major coastal regions. Yet a comprehensive set of guidelines is highly desirable to ensure consistency in hazard mapping and flood insurance administration.

The project approach therefore relies on collaboration among a team of technical experts and experienced floodplain management professionals from across the country. This team of experts is referred to as the Technical Working Group (TWG), and includes members from: FEMA Headquarters and FEMA Regions I, II, III, IV, VI, IX, and X; National Oceanic and Atmospheric Administration (NOAA), U.S. Geological Survey (USGS), and the U.S. Army Corps of Engineers (USACE); FEMA FIS contractors; coastal engineering and scientific experts from consulting organizations, universities, and institutes; international experts; and floodplain management professionals. The TWG provides a forum for building consensus on the technical issues, provides high-level review of existing guidelines and new procedures, and also provides a connection to a pool of additional technical resources through various organizations.

### 2.2.3 Phased Approach

A phased approach was adopted for the project. The first phase of the work included:

- Reviewing existing procedures and identifying needs as they pertain to the Pacific, Atlantic, and Gulf Coasts;
- Prioritizing issues and identifying additional studies required;
- Conducting Focused Studies to address specific hazard analysis and delineation issues;
- Preparing recommendations to FEMA for: (1) updating guidelines for the Atlantic and Gulf Coasts, and (2) producing guidelines applicable to the Pacific Coast.

This report and the attached appendices are the primary deliverables for Phase 1.

In the second phase, the TWG will focus on procedures specifically needed to assess coastal flooding processes on the Pacific Coast, while identifying procedures that may also be applicable in other regions. For this phase, TWG members will draw upon technical resources available from within their organizations to:

- Perform technical studies to improve existing or develop new assessment and mapping procedures specifically for the Pacific Coast; and
- Produce new coastal flood hazard mapping draft Guidelines and Specifications for the Pacific Coast.

The primary deliverable from Phase 2 will be a set of draft Guidelines and Specifications for Coastal Flood Hazard Mapping on the Pacific Coast. Detailed guidelines development or modification for the Atlantic and Gulf Coasts are not included in this project. However, it is anticipated that much of the work done during the Phase 1 assessment of existing guidelines and during the Phase 2 development of the Pacific Coast guidelines will be informative during the development of flood insurance studies on the Atlantic and Gulf coasts.

The phased approach ensures consistency in the technical basis for updating and developing new guidelines across all three regions, and allows new procedures that are developed for the Pacific Coast to potentially be applied in updates for other areas. The results of this project will assist FEMA to prepare updates of guidelines for the Atlantic and Gulf Coasts, if undertaken in the future. Figure 1 illustrates the key steps and flow of work in each phase.

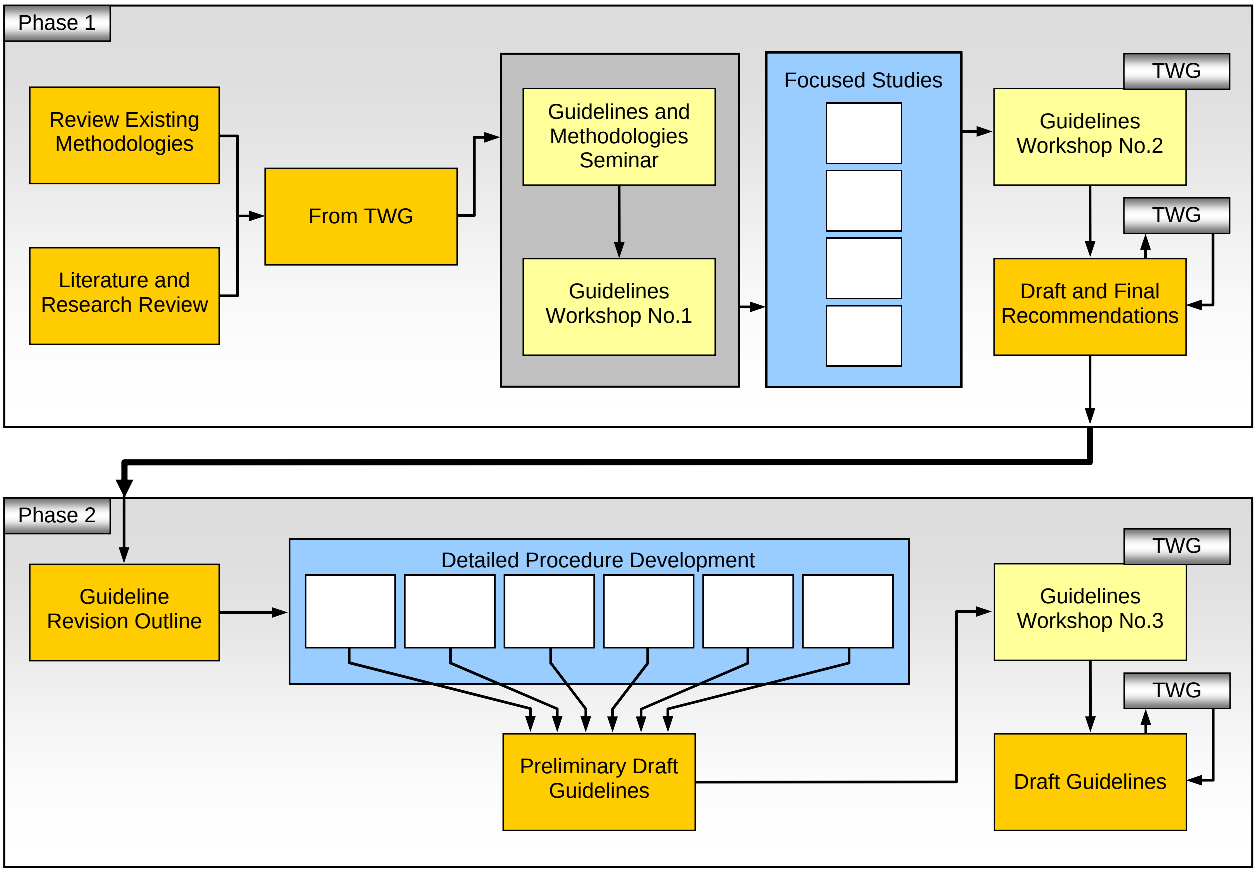


**Figure 1. Project Approach**

### 2.2.4 Objectives and Project Schedule

The objectives of the project are tied to the needs of Map Modernization – a comprehensive review of existing guidelines is needed, as well as development of technical procedures and methodologies to improve the efficiency and reliability of coastal flood hazard mapping. Coastal flood hazard mapping combines the analysis of a series of complex physical processes with FEMA mapping standards for the National Flood Insurance Program (NFIP). A review of all subjects that influence coastal flood hazard zone delineations is therefore an extremely broad and ambitious task.

At the same time, the evaluation and preparation of the guidelines must respect the schedule for Map Modernization and the need to conduct coastal flood insurance studies in FY 2004/2005. For these reasons, the objectives of the project are to make significant improvements in coastal FIS guidance by October 2004. This necessarily results in prioritization of needed improvements to ensure that they can be accomplished within this ambitious schedule.

Figure 2 shows the schedule for the project, including key milestones for Phase 1 and Phase 2.

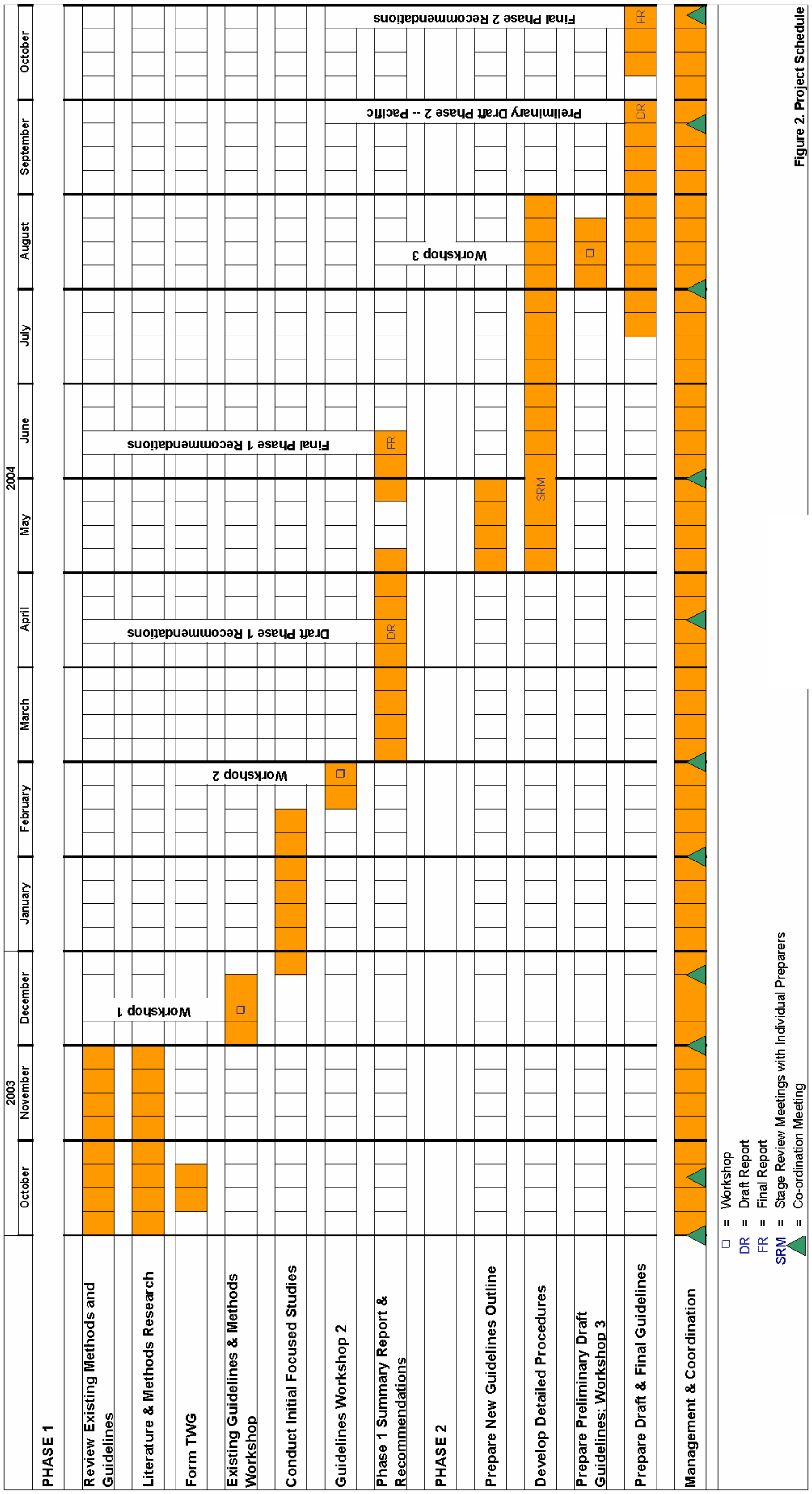


**FIGURE 2 PROJECT SCHEDULE**

# 3 PHASE 1

The purpose of Phase 1 is to establish guidance for updating the *G&S* based on the recommendations from a diverse group of scientists, coastal engineers, and floodplain managers. This section describes the activities of the TWG which evaluated technical issues for coastal flood hazard analyses and mapping and developed priorities for addressing these issues. This information will be used for developing the Phase II scope of this project, which is the development of *G&S* for the Pacific Coast.

## 3.1 FORMATION OF TECHNICAL WORKING GROUP (TWG)

The TWG was formed early in the project, primarily by considering the range of physical processes and analyses that comprise coastal flood hazard analysis for FEMA, and identifying key resources to address these subjects. Expertise was required in a broad range of coastal processes, and experience was required in application of FEMA procedures. The TWG is comprised of about 40 individuals that provide this range of expertise and experience, drawing from sources at:

- FEMA Headquarters and FEMA Regions I, II ,III, IV, VI, IX, and X
- NOAA Pacific Marine Environmental Laboratory
- USACE
- USGS
- FEMA Map Coordination Contractors and National Service Provider
- FEMA FIS Contractors in California, Oregon, Washington, Florida, North Carolina, Mississippi, and Massachusetts
- University of Florida, University of California, University of Southern California, and Oregon State University, and Scripps Institute of Oceanography
- Coastal Experts from Denmark and England

The TWG continues to grow as new technical requirements and resources are identified. Preliminary scoping for Phase 2 efforts expands the TWG with additional members from the United States, as well as coastal engineering expertise from Australia, Japan, and New Zealand.

## 3.2 INITIAL STUDIES

The initial tasks for the project included a review of the existing *G&S* and a literature and practice search. These tasks included an initial assessment of the existing guidelines, organized in a set of 11 technical categories. The 11 categories were selected to represent ocean processes, coastal processes, and mapping procedures that are considered in coastal flood insurance studies. They can be placed in an order that generally coincides with a progression in the coastal study analysis from the open ocean toward the coastline, the effects of the processes at the coastline, and the delineation of flood hazard zones. These categories include:

1) Storm Meteorology
2) Stillwater

3) Storm Wave Characteristics
4) Wave Transformation
5) Wave Setup
6) Wave Runup and Overtopping
7) Event-Based Erosion
8) Coastal Structures
9) Tsunami
10) Sheltered Waters
11) Hazard Zones

These categories have been used through the course of Phase 1 to organize discussion and technical topics, prepare detailed studies, and formulate recommendations. These 11 categories were defined to break down the determination of coastal flood hazard mapping into a number of smaller, more tractable physical processes. The ordering corresponds to the issues as they would be considered in a typical mapping analysis starting from the offshore forcing conditions and moving shoreward. Storm Wave Meteorology defines the wind and wave conditions offshore. Stillwater determines the water depth and Storm Wave Characteristics define the character of the waves. Wave Transformation brings the offshore waves to the nearshore and Wave Setup is the increase in the mean water level due to the presence of the waves. Wave Runup and Overtopping can then be determined from the wave and water level information (and beach profile information). Event-Based Erosion is the adjustment of the beach and shoreline to large events. Tsunami is a Pacific Coast mechanism that may have a significant influence on flood zone mapping. Sheltered Waters relate the above processes to semi-enclosed bodies of water. Hazard Zone provides guidance for the application of the above analyses to the determination of coastal flood hazard maps.

The initial assessment of the existing guidelines was supplemented by a set of case studies and representative studies. Case studies were compiled for specific sites on the Atlantic, Gulf, and Pacific Coasts. These case studies were used to illustrate the application of existing guidelines and practices to problems in coastal flood hazard analysis. The representative studies were used to focus on specific processes or application of specific procedures. The literature search compiled a list of national and international references, and specific references were made available to the TWG.

These materials were provided to TWG members and were the subject of presentations at Workshop 1, held in Sacramento on 2-4 December 2003 (Workshop 1 Binder, nhc 2003). This workshop focused on the needs and priorities for updating the existing guidelines on the Atlantic and Gulf Coasts and for preparing new guidelines for the Pacific Coast. The workshop included plenary sessions for presentations on the existing guidelines, case studies, representative studies, and selected technical topics (e.g., storm surge modeling, wave setup implications, current programs and information on regional wave transformation modeling, recent research on coastal erosion, and state-of-the-art efforts in tsunami modeling and research). Smaller working sessions were organized by geography (Atlantic/Gulf and Pacific Coasts) and by categories of technical topics.

### 3.2.1 Workshop I Prioritization

Table 1 summarizes the topics that were compiled over the course of the three-day workshop, including an initial assessment of priorities. These priorities were categorized considering the project schedule, which allowed approximately six months for development of new guidelines for the Pacific Coast.

Priorities for the Atlantic and Gulf Coasts and Non-Open coasts were also developed using the same categories. Based on this practical consideration, topics were characterized as follows:

- *Critical* – topics that were considered important to improve coastal flood hazard analysis and mapping for the NFIP, that required significant effort to analyze or develop, but could be developed or resolved in six months or less.
- *Important* – topics that were considered important to improve coastal flood hazard analysis and mapping for the NFIP, that required significant effort to analyze or develop, and are likely to require more than six months to be developed or resolved.
- *Available* – topics that could be improved with relatively available data or procedures in less than six months.
- *Helpful* – topics that would be helpful to the NFIP, but were considered less significant or lower priority.

A total of 53 topics were discussed at Workshop 1. As listed in Table 1 significant recommendations from Workshop 1 included the need to:

- Evaluate alternative methodologies for determination of 1% annual chance flood elevations where 1% stillwater elevations do not necessarily coincide with 1% wave conditions, especially for the Pacific Coast and in some sheltered waters
- Consider the use of regional databases and wave transformation models to develop wave spectra at the surf zone
- Develop improved methods for analysis of wave transformation over dissipative bottoms
- Develop a procedure to quantify the effects of wave setup in a variety of geomorphic settings
- Consider updates and application of simple geometric models (e.g., existing "540" criterion) for storm event erosion, as well as potentially feasible of process-based methods and models for estimating erosion
- Consider updates and amplification of existing guidelines for wave runup and overtopping, and for analysis of coastal structures
- Consider the feasibility of frequency-based estimates for tsunami effects, and their combination with other coastal processes and hazards
- Develop procedures for sheltered waters (non-open coasts), considering the unique processes and combinations of processes in these areas in contrast to open coast

## 3.3 WORKSHOP 1 LIST OF TOPICS

**Table 1**
**Workshop 1 List of Topics**

| ID | Category | Topic Description | Atlantic / Gulf | Pacific | Non-Open Coast |
|---|---|---|---|---|---|
| 1 | Wave Characteristics | Definitions of wave types using contemporary terminology (so that everyone is using the same nomenclature): standardize the terms | A | A | |
| 3 | Wave Characteristics | Conversion from Shore Protection Manual to Coastal Engineering Manual | A | A | |
| 4 | Wave Characteristics | Open coast/deep water waves, swell exposure: Use hind-cast databases, select based on evaluation | A | C | |
| 5 | Wave Characteristics | Local seas: use nearshore representation of wind waves rather than offshore wave hindcast | A | C | |
| 6 | Sheltered Waters | Write guidelines for sheltered water methods | H | C | C |
| 7 | Wave Transformation | Evaluate regional models for California | | C | |
| 8 | Wave Transformation | Assess need for regional models (beyond CA); outline methodologies to use | H | C | |
| 9 | Wave Transformation | Propagation over dissipative bottoms/friction (flat, shallow, slopes); evaluate Suhayda methods, etc., and write guidelines | C | H | C |
| 10 | Wave Transformation | Overland wave propagation: review and evaluate new methods to better represent vegetation effects, treatment of elevated pile supported buildings (WHAFIS issue) | I | H | H |
| 11 | Runup, Setup, Overtopping | Review programs, methods, and field data for run-up and over-topping; provide explicit guidance on where models should be applied | H | A | A |
| 12 | Runup, Setup, Overtopping | Review appropriateness of the mean v. higher values for run-up, set-up, and overtopping | H | C | C |
| 13 | Runup, Setup, Overtopping | Develop improved guidance on mapping and determining overtopping volumes | | A | A |
| 14 | Runup, Setup, Overtopping | Review available methods and develop guidance for wavecast debris | H | I | I |
| 15 | Runup, Setup, Overtopping | Tsunamis: Address use of National Tsunami Hazard Mitigation Program products and approaches in the NFIP | H | C | C |
| 16 | Runup, Setup, Overtopping | Tsunamis: Develop method to predict 100-year tsunami events | H | I | |
| 17 | Hazard Zones | Enhance existing guidelines for defining inland limit of VE-zone including the development of a basis for better guidance for heavily over-topped areas | C | C | |
| 18 | Hazard Zones | Investigate the appropriateness of existing VE and AE zone definitions for coastal areas | I | I | |
| 19 | Hazard Zones | Flood risk management of combined coastal and riverine flooding hazards | A | A | |
| 20 | Hazard Zones | Tsunami-structure-debris interaction to define hazard zones | H | I | |
| 21 | Coastal Structures | Failed Coastal Structures: Clarify guidance that when a structure is determined to fail under base flood conditions, the structure is removed, but fill/topography remains and is | A | A | A |

**Table 1**
**Workshop 1 List of Topics**

| ID | Category | Topic Description | Atlantic / Gulf | Pacific | Non-Open Coast |
|---|---|---|---|---|---|
| | | subject to erosion, wave analyses | | | |
| 22 | Coastal Structures | Failed Coastal Structures: Investigate configuration of failed structures | H | H | H |
| 23 | Coastal Structures | Buried Coastal Structures: Add *G&S* language that buried structures are to be evaluated | A | A | A |
| 24 | Coastal Structures | Flood Protection Structures: Review 89-15 and other literature for tsunami failure information/guidance | | A | |
| 25 | Coastal Structures | Flood Protection Structures: Review *G&*S language -- (Study Contractor not required to evaluate all structures) using 89-15 | A | A | A |
| 26 | Coastal Structures | Flood Protection Structures: Review data on (and add to G&S) effects of structures on flood hazards on adjacent properties, flooding/waves behind structures via adjacent properties | H | H | H |
| 27 | Coastal Structures | Coastal Levee vs. Structure Treatment: Review *G&S* and regulations regarding treatment of coastal levees and structures; identify conflicts; clarify *G&S* that evaluations of all "structures" to be per 89-15 | A | A | A |
| 29 | Event - Based Erosion | Tsunami Induced Erosion: Review methods for estimating tsunami-induced erosion and provide recommendations | | I | |
| 30 | Event - Based Erosion | Geometric Erosion Assessment: Review empirical geometric techniques; review pre- and post-event data for CA, OR, WA; review OR setback methodology; develop geometric techniques for Pacific shorelines, including sea cliff, bluff, dunes, beaches | | C | |
| 31 | Event - Based Erosion | Geometric Erosion Assessment: Add/revise *G&S* language regarding bluff erosion in Atlantic/Gulf areas -- better descriptions/discussions are needed | A | | |
| 32 | Event - Based Erosion | Geometric Erosion Assessment: Develop geometric method for bluff erosion in Atlantic/Gulf areas | I | | |
| 33 | Event - Based Erosion | Shingle/Cobble Erosion Assessment: Add *G&S* description/discussion regarding effect of cobble/shingle (including sediment mixtures/layers) on geometric erosion technique | C | C | C |
| 34 | Event - Based Erosion | Shingle/Cobble Erosion Assessment: Develop improved geometric methods which consider cobble/shingle effects | I | I | I |
| 35 | Event - Based Erosion | Guidance for Erosion Assessments in Sheltered Waters: Add *G&S* description/discussion regarding erosion assessments in Sheltered Waters | | | C |
| 36 | Event -Based Erosion | Guidance for Erosion Assessments in Sheltered Waters: Review data and develop geometric methods for determining eroded profile in Sheltered Waters | | | I |
| 37 | Event - Based Erosion | 540 Criteria: Expand database from which 540 sf criterion was determined; review use of median value | I | | |
| 38 | Event - Based Erosion | Physics- or Process-Based Erosion Assessment: Develop assessment procedures that consider temporal and longshore effects/variability | I | I | I |
| 39 | Event - Based Erosion | Primary Frontal Dune Definition: Develop better definition of landward limit of PFD (used for V zone limit); gather and | C | I | I |

**Table 1**
**Workshop 1 List of Topics**

| ID | Category | Topic Description | Atlantic / Gulf | Pacific | Non-Open Coast |
|---|---|---|---|---|---|
| | | evaluate Massachusetts CZM and other approaches | | | |
| 40 | Event - Based Erosion | Document Vertical Erosion Depths; maintain data and make available for use in building performance and insurance tasks (depth-damage functions) | H | H | H |
| 41 | Event - Based Erosion | Long-Term Erosion/Future Conditions: Consider revising *G&S* D.5 language and putting a warning on the FIRM; reference CCM and other reports; discuss implications of study data selection | A | A | A |
| 42 | Event - Based Erosion | Treatment of Nourished Beaches: Ensure clarity in *G&S* that references FEMA policy regarding treatment of nourished beaches | A | A | |
| 43 | Event - Based Erosion | Treatment of Nourished Beaches: No consensus on long-term technical approach for handling this issue; FEMA policy dependent | - | - | - |
| 44 | Wave SetUp | Better define and document; summarize what to consider and how to approach; data requirements | C | C | C |
| 45 | Wave SetUp | Compile example/data sets to perform tests | C | C | C |
| 46 | Wave SetUp | Develop interim method (consider Coastal Engineering Manual, Shore Protection Manual procedures) | C | C | C |
| 47 | Wave SetUp | Develop "ideal method" coupled with storm surge and waves to develop set up | I | I | I |
| 48 | Wave SetUp | Develop procedure for dynamic wave set up | I | I | I |
| 49 | Wave SetUp | Review WRUP™ (available wave run-up program) | A | A | A |
| 50 | Storm Meteorology | Test and recommend storm surge procedures (JPM, EST, Monte Carlo) and identify data sets for each region (e.g., NWS38 and HURDAT for hurricanes; nor'easters; Pacific storms) | I | I | |
| 51 | Storm Meteorology | Guidance on combined probability consideration for all processes; need to define a procedure for determining the 1% annual chance flood elevation | C | C | C |
| 52 | Stillwater | Provide guidance on non-stationary processes (for example, relative sea level change) when establishing current conditions | A | A | A |
| 53 | Stillwater | Identify reliable existing data to compare to existing FEMA flood studies to test performance of surge models | C | | |
| 54 | Stillwater | Develop database for surge versus wave height - develop interim west coast model for surge (possibly ADCIRC) | | C | C |
| 55 | Stillwater | Review the reliability of Pacific tide data to see if surge is embedded in the data sets for the purposes of developing surge factors for regions where there are little or no tide data; provide guidance | | C | C |

Key: C = critical; A = available; I = important; H = helpful

## 3.4 FOCUSED STUDIES

Focused Study groups were established for each of the 11 technical categories developed in Workshop 1. Each Focused Study was assigned a leader and team participants based on experience in the technical areas and in flood hazard mapping. Focused Study teams were comprised of 3 to 9 members depending on the range and complexity of topics identified and the resources needed to complete the Focused Study within the project schedule. The objectives of the Focused Studies included:

- Improved definition of the issues or topics identified at Workshop 1
- Assessment of existing guidelines and procedures related to the topic
- Description of the history and implications of the topic in the NFIP
- Consideration of alternatives and available data for improved guidance
- Recommendation of an approach for updating existing and/or preparing new guidelines
- Preliminary estimation of time required to accomplish the recommended approach

Most of the Focused Studies covered several topics, with varying levels of priority. *Critical* topics were given highest priority for development in the focused studies, followed by *Available*, *Important*, and *Helpful* topics.

The draft Focused Studies were used to guide discussions during Workshop 2, and subsequently modified to reflect those discussions. Summaries of the Focused Studies are the primary technical work products of Phase 1, and are attached as Appendix C to this report. These Focused Studies are intended to:

- Guide development of Phase 2 work on the Pacific Coast
- Serve as a technical resource for preparation of flood insurance studies, especially on the Atlantic and Gulf Coasts
- Serve as a planning tool for future development of guidance on the Atlantic, Gulf, and Pacific Coasts

## 3.5 WORKSHOP 2

Results from the draft Focused Studies were presented at Workshop 2. This workshop was held in Sacramento 23-26 February 2004, and was attended by 40 members of the Technical Working Group. This workshop was used as a forum for discussion of the technical topics in each category and the basis for recommendations developed by each of the Focused Study groups. Table 2 lists the topics and which Focused Study group developed recommendations. The table also identifies related topics so that inter-relationships among topics can be coordinated.

Table 2 shows the compilation of TWG recommendations from Workshop 2. These recommendations were developed with the consensus of the entire TWG. For several topics, case studies were recommended to develop and test new procedures, or to test existing methods in particular settings. The consensus of the group was also used to confirm or adjust the priority classes for each topic, and to carefully state the topic.

Table 2 presents a summary of recommended approaches for each topic and the category under which each topic is applicable as developed at Workshop 2. Due to the number of topic, Table 2 presents a significantly condensed version of the discussions held at Workshop 2. Sections 4 and 5 and the Appendices to this report provide the detailed approaches and background. information for each topic. A key for Table 2 is listed at the end of the table.

The definitions for the Priority Classes assigned to each task by the TWG were given in Section 3.2. These definitions are repeated here for ease of reference.

- *Critical* – topics that were considered important to improve coastal flood hazard analysis and mapping for the NFIP, that required significant effort to analyze or develop, but could be developed or resolved in six months or less.
- *Important* – topics that were considered important to improve coastal flood hazard analysis and mapping for the NFIP, that required significant effort to analyze or develop, and are likely to require more than six months to be developed or resolved.
- *Available* – topics that could be improved with relatively available data or procedures in less than six months.
- *Helpful* – topics that would be helpful to the NFIP, but were considered less significant or lower priority.

**Table 2**
**Workshop 2 Recommendations**

| Topic | Category | Coastal Area | Priority Class | Recommended Approach | Related Topics |
|---|---|---|---|---|---|
| 50 Modeling Procedures | Storm Meteorology | AC<br>GC<br>PC<br>SW | I<br>I<br><br> | Identify and summarize data sources for storm parameters, and compare storm surge statistical methods (EST, JPM, Monte Carlo approaches may all be valuable); prepare guidelines describing the use of each alternative; revisit treatment of storm wind fields and wind stress formulation | 53-55 |
| 51 Combined Probability, Determination of 1% Annual Chance Flood Elevations | Storm Meteorology | AC<br>GC<br>PC<br>SW | C<br>C<br>C<br>C | For each major process combination, prepare Guidelines with recommended methodology and illustrative examples. For wave plus high water perform (2 open/sheltered) case studies for Pacific sites to: (1) implement Wallingford approach; (2) use NOS tide gage data; (3) use NOAA wave buoy data. Develop practical Guidelines from study findings, with examples | All |
| 52 Non-Stationary Processes | Stillwater | AC<br>GC<br>PC<br>SW | A<br>A<br>A<br>A | Identify and summarize data sources for sea level rise and land subsidence and/or uplift; provide basic guidance regarding significance of non-stationarity in flood insurance applications; include guidance on interpretation of historical data. Suggest documentation of projected map impact. | |
| 53 Reliable | Stillwater | AC | C | Develop overview guidance for surge modeling; | 6, 44-48 |

**Table 2**
**Workshop 2 Recommendations**

| Topic | Category | Coastal Area | Priority Class | Recommended Approach | Related Topics |
|---|---|---|---|---|---|
| Surge Data | | GC | C | define procedures to assess accuracy of surge estimates; suggest regional modeling approaches for study economy | |
| | | PC | | | |
| | | SW | | | |
| 54 & 55 Pacific Coast/Sheltered Waters Surge Estimates | Stillwater | AC | | Identify tide gage data sources; develop procedures for surge extraction from tide gage records for FIS use (including test studies); develop simplified numerical modeling method for areas without data (1-D Pacific Surge Model) | 6, 44-48 |
| | | GC | | | |
| | | PC | C | | |
| | | SW | C | | |
| 4 & 5 Swell and Seas | Storm Wave Characteristics | AC | C | WIS database is recommended for use. Clarify extrapolation to 100-year; investigate appropriateness of using either 100-year significant wave height or 20-year maximum; clarify use of equivalent deepwater wave - definition (Topic 1) | 8,9, 51 |
| | | GC | C | | |
| | | PC | C | 1. GROW database is recommended for use in near term for swell and sea. Confirm lack of bias in GROW database. WIS can be used after completion of current revision. CDIP data can be used for model verification. 2. Develop G&S for preparation of input data for wave modification models based on GROW directional spectra. 3. Conduct a study of the available nearshore data for Southern California Bight to assess whether inclusion of the local wind will make a significant change in the high frequency part of the spectrum | 8,9, 51 |
| | | SW | C | Add guidance on use of Coastal Engineering Manual (CEM); conduct a Focused Study to confirm that Shore Protection Manual (SPM) results are similar (validation for previous studies). Conduct a Focused Study and describe procedures for: (1) existing parametric model guidance; (2) enhanced parametric models; (3) spectral energy models | 6, 8, 9, 51 |
| 1 Wave Definitions | Storm Wave Characteristics | AC | A | The recommended approach includes: (1) adopt the CEM "Glossary of Coastal Terminology" and International Association of Hydraulic Engineering and Research "List of Sea State Parameters" (for notations); and (2) clarify the correlation of these terms to the actual guidance and various methodologies to ensure consistency | 4, 5, 50, 51 |
| | | GC | A | | |
| | | PC | A | | |
| | | SW | A | | |
| 10 WHAFIS | Wave Transformation | AC | I ( *C*) | Clarify where WHAFIS, 1-D, and 2-D models are most appropriate. Update WHAFIS and tie back to CHAMP. Minor Effort – code changes for more user friendly program. Moderate Effort – more intense code changes for improvement in accuracy and graphics, add wind direction. Update *G&S* accordingly | 8, 9 |
| | | GC | I ( *C*) | | |
| | | PC | I (*C*) | | |
| | | AC | I | Significant Effort – improve WHAFIS to include combined effects of damping and wind action over | |
| | | GC | I | | |

**Table 2**
**Workshop 2 Recommendations**

| Topic | Category | Coastal Area | Priority Class | Recommended Approach | Related Topics |
|---|---|---|---|---|---|
| | | | | each segment. Include realistic wave breaking model for setup and other processes after developed. | |
| | | PC | H | Evaluate if changes to WHAFIS dissipation criteria are necessary (see topic 9), and incorporate in *G&S* modifications for PC | |
| | | SW | H | Refer to AG, GC, and PC *G&S*<br>Include in PC *G&S* | |
| 7 CDIP CA | Wave Transformation | AC | | | 8 |
| | | GC | | | |
| | | PC | C | Develop interim *G&S* for use of CDIP regional wave models and database (California) | |
| | | PC | I | Expansion of CDIP regional model approach to develop nearshore wave climate database in areas where it is not currently available | |
| | | SW | | | |
| 8 Overall WT | Wave Transformation | AC | H | Refer to PC *G&S* for potential use of regional models | 7, 9, 10 |
| | | GC | H | | |
| | | PC | C | Write *G&S* for Wave Transformations. Tasks:<br>1. Conduct several Focused Studies to assist in writing the Wave Transformations G&S.<br>2. Use available publications to identify a range of methods.<br>3. Develop criteria for level of analysis.<br>4. Include development of guidelines for spatial coverage and wave parameters, and include use of regional models such as CDIP.<br>5. Research available literature to adequately define wave groups, infragravity waves, shallow water spectra, etc. for input into wave setup and runup calculations.<br>6. Evaluate wave transformation models using a selected data set.<br>7. Review available literature and guidance on the range of applicability of contemporary computer models, recommend models for inclusion on the FEMA pre-approved coastal model list, and provide guidance on their application to FEMA FISs.<br>8. Incorporate applicable sections of existing *G&S* for other geographical areas that cover the overland propagation and wave energy dissipation topics. (Topics 9 &10) | 6, 7, 9, 10, 11, 44, 45, 47, 48, 49, 54, 55 |
| | | SW | C | Include in PC *G&S*; reference for AC and GC | |
| 9 Dissipation | Wave Transformation | AC | C | Write *G&S* to include a section on wave energy dissipation over shallow and flat bottoms based on available information.<br>Develop typical ranges for dissipation coefficients for a variety of bed and wave conditions to include | 8, 10 |
| | | GC | C | | |

**Table 2**
**Workshop 2 Recommendations**

| Topic | Category | Coastal Area | Priority Class | Recommended Approach | Related Topics |
|---|---|---|---|---|---|
| | | | | in the *G&S*, based on available information. Provide guidance on calibration if available data not adequate to select coefficients. | |
| | | GC<br>AC | I<br>I | Conduct studies to develop typical ranges for dissipation coefficients for variety of bed and wave conditions to include in the *G&S*.<br>Categorize bed and wave conditions for US coastlines. Revise *G&S* to provide dissipation coefficients on a geographic basis to the extent appropriate; revise *G&S* to adopt Suhayda (1984) method. Provide guidance on calibration of available data not adequate to select coefficients. | |
| | | PC | H (*C*) | Evaluate wave dissipation over marsh and mudflats in the Pacific using available information; provide interim guidance for calculating wave dissipation. | |
| | | PC | H(*I*) | Conduct field data collection to characterize wave dissipation over marsh and mudflats in the Pacific; provide guidance for calculating wave dissipation. | |
| | | SW | C | Include in PC *G&S*; reference for AC and GC | |
| 44&45 Define, Document, Compile Data | Wave Setup | AC<br>GC<br>PC<br>SW | C<br>C<br>C<br>C | The recommended approach for this Topic is the same for all geographic regions: Conduct a thorough examination of all available relevant literature with an emphasis on quality field data sets. These would include experiments conducted especially to investigate wave setup and especially "experiments of opportunity" in major storms including high water marks. Organize data by "settings" identified in the Phase 1 effort. | 11 |
| 46 Interim Method | Wave Setup | AC<br>GC<br>PC<br>SW | C<br>C<br>C<br>C | Several possibilities exist. The "Interim Method" should include consideration of the following: (1) Static and dynamic setup; (2) Irregular waves (implicit in (1) above); (3) Characterization of nearshore bathymetry; (4) A valid wave breaking model; (5) Nonlinearities in $S_{xx}$; and (6) Wave damping where appropriate. An attempt should be made to ensure that the interim method address as many of the settings identified as possible. | 1, 6, 9 |
| 47 Develop Ideal Method - Coupled | Wave Setup | AC<br>GC<br>PC<br>SW | I<br>I<br>I<br>I | The recommended approach for this Topic is the same for all geographic regions. The ideal method would be one in which the storm surge model also incorporates a wave generation model. The wave generation model would predict directional spectra so that the characteristics of the dynamic setup could be calculated directly. It is recommended that this topic be approached as a two phase effort with the first phase evaluating approaches and the second phase pursuing the approach identified. | 9, 10, and many beyond those identified in Table 1 |
| 48 Dynamic | Wave Setup | AC | I | This topic could be incorporated into Topic 47; | 9, 10, and |

**Table 2**
**Workshop 2 Recommendations**

| Topic | Category | Coastal Area | Priority Class | Recommended Approach | Related Topics |
|---|---|---|---|---|---|
| Wave Setup | | GC<br>PC<br>SW | I<br>I<br>I | however, a more realistic approach is to parallel Topic 47 with a first phase to evaluate existing methodologies that could be applied. The results of the first phase would guide the second phase, which would implement the optimal approach identified. It is anticipated that the actual procedures developed would be somewhere between a full physics-based approach which would proceed from a directional spectrum, and the approaches available from Lo and Goda which are either based on somewhat simple calculations or empirical. A probable approach would be one in which the dynamic wave setup is based on parameterized spectra determined as a function of wind fields and continental shelf width of interest. | many beyond those identified in Table 1 |
| 30 Geometric Techniques - PC | Event - Based Erosion | AC<br>GC<br>PC<br>SW | <br><br>C<br> | 1. Select and evaluate existing geometric methods and models.<br>2. Develop guidance for determination of a Most Likely Winter Beach Profile including areas of beach nourishment.<br>3. Evaluate geometric modeling procedures for sand beaches and dunes on PC and test with available data sets.<br>4. Recommend that FEMA expand/support the present USGS/NOAA coastal survey program for the Pacific Coast; update likely winter profiles for various geomorphic settings. | 31, 32, 35, 36, 37 |
| 31 Bluff Erosion - AC/GC/(PC) | Event - Based Erosion | AC<br>GC<br>PC<br>SW | A<br>(*A*)<br>(*A*)<br>(*A*) | Add/revise guidance language to distinguish bluff erosion from other processes with descriptions and examples. | 30, 32, 35-38, 41 |
| 32 Geometric Method for Bluffs - AC/GC/(PC) | Event - Based Erosion | AC<br>GC<br>PC<br>SW | I (*A*)<br>I (*A*)<br>(*A*)<br>(*A*) | 1. Review existing bluff erosion procedures and literature.<br>2. Consider development of geometric procedure for bluff erosion and cliff retreat. | 12, 21, 33, 35, 38, 42 |
| 33 Cobble/ Shingle Effects | Event - Based Erosion | AC<br>GC<br>PC<br>SW | C<br>C<br>C<br>C | 1. Prepare new sections of *G&S* to describe differences between sand dominated beaches and gravel/cobble/shingle beaches found along the north Atlantic, Gulf, Pacific and in Sheltered Waters areas. Provide photos and profile information.<br>2. Gather existing literature on gravel, cobble, and shingle beaches to summarize the existing state of knowledge until specific guidelines can be developed and adopted.<br>3. Review literature on the design and construction of dynamic revetments and cobble berms to provide guidance on beach stability and long term development. | 30, 31, 32, 34, 37 |

**Table 2**
**Workshop 2 Recommendations**

| Topic | Category | Coastal Area | Priority Class | Recommended Approach | Related Topics |
|---|---|---|---|---|---|
| | | | | 4. Examine other possible guidance and available beach and dune data sets for possible clarifications to the 540 SF criterion for sand-dominated beaches versus gravel/cobble/shingle beaches.<br>5. Discuss the limitations of applying geometric models to cobble/shingle beach and dune areas | |
| 34 Cobble/ Shingle - Geometric Method | Event - Based Erosion | AC | I | Develop geometric procedure for cobble/shingle eroded profile. | 12, 21, 33, 35, 38, 42 |
| | | GC | I | | |
| | | PC | I | | |
| | | SW | I | | |
| 35 Erosion – Sheltered Waters | Event - Based Erosion | AC | C | 1. Provide definitions and discussion in *G&S* for sheltered water types of beach morphology, materials, and wave characteristics.<br>2. Provide interim *G&S* based primarily on historical beach profiles and field observations. | 5, 6, 36, 41 |
| | | GC | C | | |
| | | PC | C | | |
| | | SW | C | | |

**Table 2**
**Workshop 2 Recommendations**

| Topic | Category | Coastal Area | Priority Class | Recommended Approach | Related Topics |
|---|---|---|---|---|---|
| 36 Geometric Method – Sheltered Waters | Event - Based Erosion | AC | I | 1. Provide interim *G&S* for the AC and GC based primarily on historical applications of the 540 SF criterion on AC/GC.<br>2. Provide interim *G&S* for the PC based primarily on historical field observations developed on PC.<br>3. Perform pilot studies; refine procedures and describe methods for *G&S*.<br>4. Incorporate event-based models where feasible into final *G&S*.<br>5. Provide guidance on appropriate models for erosion in sheltered waters | 5, 6, 35, 38 |
| | | GC | I | | |
| | | PC | I | | |
| | | SW | I | | |
| 37 Review 540 SF Criterion | Event - Based Erosion | AC | I | 1. Expand database beyond 38 storm events for AC and GC using more recent data.<br>2. Re-evaluate existing data points.<br>3. Consider storm duration in analyses.<br>4. Consider variability of erosion about median at each data point.<br>5. Evaluate geometry of retreat and removal profiles.<br>6. Contingent on 1. through 5., determine whether median erosion trigger should be maintained or revised. | 32, 34, 36 |
| | | GC | I | | |
| | | PC | | | |
| | | SW | | | |
| 38 Process-Based Approach | Event - Based Erosion | AC | I | 1. Further develop and test process-based models using field data and compare with geometric models.<br>2. Develop method to include randomness of storm waves and tides and coincidence in Item 1.<br>3. Provide *G&S* for erosion assessment to coastal bluff fronted by narrow beach.<br>4. As an interim method continue to use the 540 SF Criterion for A/G and GL, and most likely winter beach profile or best documented winter profile for the Pacific Coast. | 30, 31, 32, 35, 36 |
| | | GC | I | | |
| | | PC | I | | |
| | | SW | I | | |
| 39 PFD | Event - Based Erosion | AC | C | Covered in Hazard Zones Topics | |
| | | GC | C | Covered in Hazard Zones Topics | |
| | | PC | I | Covered in Hazard Zones Topics | |
| | | SW | I | Covered in Hazard Zones Topics | |
| 40 Vertical Erosion Depths | Event - Based Erosion | AC | H | Document depths of erosion following storm events and maintain data for depths of erosion and damages to buildings in order to better determine “depth-damage” relationships. As methods and models are coded, calculate and store vertical erosion depths along transects and grids. | 30-36 |
| | | GC | H | | |
| | | PC | H | | |
| | | SW | H | | |
| 41 Long-Term Erosion | Event - Based Erosion | AC | A | 1. Topic considered important to NFIP, but FEMA action on previous work is pending; therefore guidance is best developed by FEMA outside of current project.<br>2. Better risk communication to public - outside of *G&S*. | 30, 31, 32, 35, 36 |
| | | GC | A | | |
| | | PC | A | | |
| | | SW | A | | |

**Table 2**
**Workshop 2 Recommendations**

| Topic | Category | Coastal Area | Priority Class | Recommended Approach | Related Topics |
|---|---|---|---|---|---|
| 42/43 Nourished Beaches | Event - Based Erosion | AC | A | Prepare guidance to: (1) Notify FEMA that study area includes beach nourishment project; (2) Conduct research and preliminary analysis to determine whether beach nourishment is likely to have an effect on hazard zone designations and/or BFEs; (3) Provide list of types of information that may be required to assess special cases where beach nourishment may be considered in determining hazard zones and BFEs (as an exception to existing FEMA policy). | |
| | | GC | A | | |
| | | PC | A | | |
| | | SW | | | |
| 21a Failed Structures | Coastal Structures | AC | A | Expand guidance to discuss removal of seawalls, bulkheads, revetments, coastal levees. | 22, 13 |
| | | GC | A | | |
| | | PC | A | | |
| | | SW | A | | |
| 21b1 Failed Structures | Coastal Structures | AC | A | Mention in guidance: removal of the effects of groins, jetties, detached breakwaters on the shoreline. | 22 |
| | | GC | A | | |
| | | PC | A | | |
| | | SW | A | | |
| 21b2 Failed Structures | Coastal Structures | AC | A | Develop specific guidance on how to remove the effects of groins, jetties, detached breakwaters on the shoreline. | 22 |
| | | GC | A | | |
| | | PC | A | | |
| | | SW | A | | |
| 23 Buried Structures | Coastal Structures | AC | A | Mention in guidance: buried structures may exist, should be located and should be considered in analyses. | 22 |
| | | GC | A | | |
| | | PC | A | | |
| | | SW | A | | |
| 25 Flood Protection Structures | Coastal Structures | AC | A | Mention in guidance: detailed TR-89-15 evaluation/certification of coastal structures are not required during FIS, but discuss implications (see Topic 22). | 22, 26, 27 |
| | | GC | A | | |
| | | PC | A | | |
| | | SW | A | | |
| 27a Coastal Levees v. Structures | Coastal Structures | AC | A | Revise *G&S* to differentiate coastal levee requirement from those for other costal flood protection structures; identify conflicts. | |
| | | GC | A | | |
| | | PC | A | | |
| | | SW | A | | |
| 27b Coastal Structure Evaluation Criteria | Coastal Structures | AC | H | Review, revise TR-89-15 evaluation criteria. | 11 |
| | | GC | H | | |
| | | PC | H | | |
| | | SW | H | | |
| 27c Coastal Structure Treatment | Coastal Structures | AC | A | Consider requiring all structures (existing and new) to meet the same evaluation criteria. | 25 |
| | | GC | A | | |
| | | PC | A | | |
| | | SW | A | | |
| 24 Structures - Tsunamis | Coastal Structures | AC | | Review literature and revise guidance for coastal structure evaluation criteria in tsunami-prone areas. | 22 |
| | | GC | | | |
| | | PC | A | | |

**Table 2**
**Workshop 2 Recommendations**

| Topic | Category | Coastal Area | Priority Class | Recommended Approach | Related Topics |
|---|---|---|---|---|---|
| | | SW | | | |
| 22 Failed Structure Configuration | Coastal Structures | AC<br>GC<br>PC<br>SW | H<br>H<br>H<br>H | Review Coastal Engineering Manual (CEM) for treatment of failed structures; revise guidance to include modified Philip Williams & Associates Sandy Point methodology (intact and failed where performance uncertain) and CEM results | 21, 24 |
| 26a Adjacent Properties | Coastal Structures | AC<br>GC<br>PC<br>SW | H<br>H<br>H<br>H | Review literature and develop guidance for evaluating the erosion effects of coastal structures on adjacent properties. | 11, 22 |
| 26b Adjacent Properties | Coastal Structures | AC<br>GC<br>PC<br>SW | H<br>H<br>H<br>H | Review literature and develop guidance for evaluating the hydraulic effects of coastal structures on adjacent properties. | 11, 22 |
| 26c Adjacent Properties | Coastal Structures | AC<br>GC<br>PC<br>SW | H<br>H<br>H<br>H | Deleted | |
| 26d Adjacent Properties | Coastal Structures | AC<br>GC<br>PC<br>SW | H<br>H<br>H<br>H | Develop guidance for evaluating flooding and erosion from adjacent properties. | |
| 26e Minimum Length | Coastal Structures | AC<br>GC<br>PC<br>SW | H<br>H<br>H<br>H | Deleted | 11, 22 |
| 12 Mean v. Higher Value | Runup and Overtopping | AC<br>GC<br>PC<br>SW | H *(C)*<br>H *(C)*<br>C<br>C | 1. Revise guidance to include sandy beach, small dune shore type in runup analyses.<br>2. Review runup distributions for beaches and structures during El Niño, coastal storm and hurricane conditions; review runup damages; evaluate use of R50%, select alternative value if hazard is not properly represented.<br>3. Tsunami runup to be treated by procedures developed specifically for tsunami events.<br>4. Investigate feasibility of interim procedure for modifying the results of RUNUP 2.0. | 11, 16 |
| 11 Methods and Models | Runup and Overtopping | AC<br>GC<br>PC<br>SW | H *(I)*<br>H *(I)*<br>A *(C)*<br>A *(C)* | 1. Evaluate expansion of "Oregon-type" and "CDIP-type" methods as interim Pacific runup method<br>2. Develop test scenarios for side-by-side comparisons of existing runup methods, models (give priority to Pacific and New England scenarios). Will require establishment of probabilities<br>3. Perform comparisons, eliminate methods, models; identify appropriate runup methods, | 4, 5, 7, 8, 12, 16, 44-49 |

**Table 2**
**Workshop 2 Recommendations**

| Topic | Category | Coastal Area | Priority Class | Recommended Approach | Related Topics |
|---|---|---|---|---|---|
| | | | | models by location, morphology and hydraulic conditions. Address uncertainty issues. | |
| 49 WRUP | Runup and Overtopping | AC | A | Evaluate with other runup methods and models in Topic 11. | 11 |
| | | GC | A | | |
| | | PC | A | | |
| | | SW | A | | |
| 13 Overtopping Volumes | Runup and Overtopping | AC | *(A)* | 1. Evaluate existing methods and models for calculating mean overtopping rates<br>2. Determine appropriate procedure for calculating overtopping at structures, remnant dunes, low profile beaches, and barriers<br>3. Revise procedures for overtopping calculations at bluffs.<br>4. Review literature for data on acceptable overtopping rates, revise landward flood hazard zones.<br>5. Review FEMA practice to limit runup elevations to 3 feet above barrier crests. | 11, 12, 14 |
| | | GC | *(A)* | | |
| | | PC | A | | |
| | | SW | A | | |
| 14 Wavecast Debris | Runup and Overtopping | AC | H | 1. Review the literature and quantify the significance of coastal flood damages from drift logs and wave-sprayed stone.<br>2. Review past flood insurance studies that have resulted in methods for defining flood hazards from wave-cast debris, and refine methods where appropriate.<br>3. Incorporate into mapping zones, but don't attempt to specifically map debris (i.e., map the water that carries debris, but not debris itself). | 6, 13, 18, 20, 22 |
| | | GC | H | | |
| | | PC | I | | |
| | | SW | I | | |
| 15 NTHMP | Tsunamis | AG | H | The recommended approach includes: (1) develop digital database; and (2) develop a methodology, including recurrence interval estimation, for use of NTHMP products for NFIP for tsunami hazard zone delineation. (Tasks Go With Topic 16) | 16,20,29 |
| | | GC | H | | |
| | | PC | C | | |
| | | SW | C | | |
| 16 100-year Recurrence | Tsunamis | AG | H | The recommended approach is to perform a comprehensive probabilistic tsunami hazard assessment at a pilot site in California or Oregon or Washington to include: (1) recurrence interval estimate of forcing functions; (2) propagation of tsunamis from Subduction Zone; (3) inundation calculations; (4) probability distributions and integration. Use results to assess whether tsunami condition will govern hazard zone delineation. | 15,20,29 |
| | | GC | H | | |
| | | P | C | | |
| | | SW | C | | |
| 20 Structure-Debris Interaction | Tsunamis | PC | I | Review TR-89-15 for recommendations for impact forces using data for overland flow depths and velocities for the numerical simulations from Item 15 and 16 for one specific locale. (Conditional on Topic 16) Linked to Topic 24. | 15,16 |

**Table 2**
**Workshop 2 Recommendations**

| Topic | Category | Coastal Area | Priority Class | Recommended Approach | Related Topics |
|---|---|---|---|---|---|
| 29 Erosion | Tsunamis | SW | I | Evaluate and integrate USGS erosion data into empirical relationships for the specific locale under study. (Conditional on Topic 16) | |
| 6a Definitions and Classification | Sheltered Waters | AC<br>GC<br>PC<br>SW | H<br>H<br>C<br>C | 1. Review previous sheltered water flood studies, compare methods, geomorphic conditions, unique flood hazards.<br>2. Compile a list of coastal (sheltered water) flood study definitions in *G&S* and prepare definitions for Guidelines.<br>3. Identify and classify Pacific sheltered water physical processes and site characteristics.<br>4. Review classification systems established by others and refine/adapt a system for sheltered water areas. | 1, 5, 9, 10, 11-14, 15-16, 17-19, 20, 21-27, 29, 30, 35-36, 37-43, 44-48, 50-51, 52-55 |
| 6b Historical Information | Sheltered Waters | AC<br>GC<br>PC<br>SW | H<br>H<br>C<br>C | 1. Review previous sheltered water flood studies and document methods used for validating flood study results.<br>2. A summary of the review may include a checklist for results validation.<br>3. Compare results of past flood studies to actual damage and flood observations made by community officials and residents. | 9-10, 11-14, 17-19, 21-22, 24, 30-31, 35-36, 53 |
| 6c Peer Input | All | AC<br>GC<br>PC<br>SW | H<br>H<br>C<br>C | Deleted | All |
| 6d 1% Annual Chance Flood Elevations | Sheltered Waters | AC<br>GC<br>PC<br>SW | H<br>H<br>C<br>C | 1. Review the methods used in previous FEMA-accepted sheltered water flood insurance studies for possible adoption as methods to reference in the new guidelines (Topic 51).<br>2. Evaluate potential need for guidance on joint probability effects considering coastal watersheds.<br>3. Expand discussion of existing guidance on wind data acquisition and analysis and fetch-limited wave forecasting. | 4,5,8-10, 12, 16, 19, 44-48, 50-51, 52-55 |
| 6e Stillwater Elevations and Tidal Currents | Sheltered Waters | AC<br>GC<br>PC<br>SW | H<br>H<br>C<br>C | 1. Review pertinent scientific literature and resource management practices.<br>2. Prepare guidance for the transfer of tide gauge data to ungauged sheltered water bodies.<br>3. Prepare guidance for the estimation and use of tidal datums in flood insurance studies.<br>4. Prepare guidance for the assessment of tidal and nearshore currents and their significance to flood hazards.<br>5. Coordinate guideline development with Wave Setup and Stillwater Focused Study Groups | 44-48, 52-55 |

**Table 2**
**Workshop 2 Recommendations**

| Topic | Category | Coastal Area | Priority Class | Recommended Approach | Related Topics |
|---|---|---|---|---|---|
| 6f Coastal Structures | Sheltered Waters | AC | H | Covered in Topic 21a | 11-14, 17-19, 21-27, 35-36 |
| | | GC | H | | |
| | | PC | C | | |
| | | SW | C | | |
| 6g Hazard Zones | Sheltered Waters | AC | H | Covered in Topic 17 | 13-14, 17-19, 35-36 |
| | | GC | H | | |
| | | PC | C | | |
| | | SW | C | | |
| 6h Inter-relationships | Sheltered Waters | AC | H | Identify and assess interrelationships of new PC *G&S* to other sections of existing *G&S* and other FEMA multi-hazard initiatives. | All |
| | | GC | H | | |
| | | PC | C | | |
| | | SW | C | | |
| 17 VE Zone Limit | Hazard Zones | AC | C | 1. Investigate and develop guidance to better map the BFE transition between PFD and landward hazard zone.<br>2. Establish procedures (hazard identification and mapping) to better utilize VO Zones.<br>3. Establish procedures for identifying and mapping wave overtopping and wave-cast debris hazards.<br>4. Establish improved procedures for establishing the landward limit of the PFD . | 11, 12, 13, & 14 |
| | | GC | C | | |
| | | PC | C | | |
| | | SW | C | | |
| 18 VE/AE Zone Appropriateness | Hazard Zones | AC | I | 1. Investigate and develop Coastal A Zone criteria (wave and erosion damage) and procedures for application within the NFIP.<br>2. Prepare technical bulletins for clarification of proposed revisions to VE Zones, AE Zones, and new VO Zones related to hazard identification and floodplain management.<br>3. Apply new concepts in a case study area.<br>4. Develop an annotated bibliography of related research and papers to support new guidance. | 11, 12, 13, & 14 |
| | | GC | I | | |
| | | PC | I | | |
| | | SW | I | | |
| 19 Combined Coastal/ Riverine | Hazard Zones | AC | A | 1. Review the previous guidance from 1981 for adoption into *G&S.*<br>2. Develop mapping standards to clearly identify this hazard zone. | |
| | | GC | A | | |
| | | PC | A | | |
| | | SW | A | | |

Key:

| | |
|---|---|
| Topic | Topic Number from Table 1 - Workshop 1 List and Subject |
| Category | Major Category from Table 1 - e.g., Stillwater Elevations, Wave Setup, Runup and Overtopping, etc. |
| Geographic Region | AG = Atlantic Coast; GC = Gulf Coast; PC = Pacific Coast; SW = Sheltered Waters |
| Priority Class | Priority Class from Table 1; e.g., H, A, C, I (in parentheses if Focused Study has recommended a change in priority class) |
| Recommended Approach | Brief Description of Recommended Approach |
| Related Topics | Topic Number for Related Topics |

### 3.6 PHASE 2 SCOPING – PACIFIC COAST

A primary objective of the Focused Studies and the recommendations from Workshop 2 was to guide Phase 2 work on the Pacific Coast. Following Workshop 2, the recommended approach for the Pacific Coast was compiled and an estimate of time and budget to accomplish the recommended tasks was developed. This estimate exceeded the available time and budget for the project by 300%. Therefore, options were developed and reviewed with FEMA to prioritize tasks to be included in the Phase 2 work. FEMA made a significant adjustment to the project budget to allow a larger portion of the recommendations to be explored and implemented in Phase 2. The prioritization process attempted to retain significant work in all 11 technical categories to produce a comprehensive set of guidelines for the Pacific Coast.

The selected option includes limited case studies in several areas to develop and test new procedures, and development of simple models designed specifically for use in FEMA flood insurance studies. Model development, case studies, and testing of methods and models are included in the Phase 2 work in the following areas:

- Storm Meteorology – testing to develop procedures for 1% annual chance flood elevation determination based on wave and water level combinations in open coast and sheltered waters settings
- Stillwater Elevations – testing for procedures to extract surge data from tide gage data; development of surge model for the Pacific Coast
- Wave Characteristics – case study to develop wind field and other input data specifications and methods for application of spectral models
- Wave Transformation – testing of wave transformation models
- Wave Setup – testing of Boussinesq models; development and testing of new setup model
- Runup and Overtopping – runup model testing combined with 1% annual chance flood elevation testing in Storm Meteorology
- Event-Based Erosion – testing of geometric models and procedures

A case study is also recommended by the TWG to develop a probabilistic methodology that considers both near-field and far-field sources of tsunamis. This case study will be accomplished outside the scope of the current project due to the highly specialized nature of the required analyses. This case study is expected to be accomplished through inter-agency cooperation between FEMA, NOAA, and USGS, with assistance from private consultants and research institutions such as the University of Southern California.

In addition to the model development, case studies, and testing listed above, Phase 2 work will include evaluation of existing methods and databases as they pertain to coastal flood hazard mapping on the Pacific Coast, and preparation of guidelines in each of the 11 technical categories.

# 4 RECOMMENDATIONS – PACIFIC COAST

This section presents recommendations for the development of *G&S* for the Pacific Coast. The first part of this section discusses the importance of considering both open coast and sheltered waters for Pacific Coast FIS and potential alternatives for the determining the 1% annual chance flood hazard. This is followed by specific recommendations for the Pacific Coast in the 11 technical categories discussed in Section 3.

## 4.1 INTRODUCTION – OBJECTIVES AND NFIP CONSIDERATIONS

A primary objective for these recommendations is to guide work in Phase 2 of the project for the Pacific Coast. For the Pacific Coast, the recommendations are split into recommended Phase 2 work and recommended future development. The work shown in Phase 2 will produce a set of guidelines specifically for the Pacific Coast and facilitate new and updated coastal flood insurance studies for map modernization.

The work in Phase 2 does not include all the recommended Critical, Available, Important, and Helpful topics. The Phase 2 recommendations have been adjusted from the Workshop 2 recommendations taking into consideration available resources and budgetary constraints to maintain the project schedule. These adjustments were made to allow treatment of the full range of technical categories in the guidelines at a significant level of technical detail, considering priorities for needed improvements and relative importance among categories.

Secondary objectives for this section are therefore to recommend future work to further improve and expand the guidelines and to serve as a reference for planning future FEMA technical guidance work. The summaries in this section also provide a concise connection to the appended Focused Studies, which include additional information and references on the topics that were deferred to the future. In addition to new guidelines, these Focused Studies may be valuable references for the NFIP as coastal studies move forward on the Pacific Coast.

## 4.2 GUIDELINES FORMAT AND STUDY PROCESS

On the Pacific Coast, new guidelines will be developed in Phase 2 that can be incorporated by FEMA into Appendix D of the Guidelines and Specifications for Flood Hazard Mapping Partners (FEMA, 2003). This set of guidelines evolved over approximately 20 years and is specifically applicable to the Atlantic and Gulf Coasts. As part of Phase 1, the existing guidelines were reviewed by the project team to determine the potential applicability of this format to new guidelines for the Pacific Coast. Based on this review, the project team feels that the new guidelines would benefit greatly from reorganization and restructuring to address particular aspects of coastal flood hazard analysis and mapping for the Pacific Coast.

Key considerations in the development of a new format for the Pacific Coast guidelines include a few key challenges that may be unique to the Pacific Coast or that may not have been fully developed in the existing guidelines. These include the need to specifically account for potential alternative methods for determining the 1& annual chance flood elevation where 1% stillwater elevations do not necessarily coincide with 1% wave conditions. This issue is particularly important on the Pacific Coast, where this determination is not driven by a single type of event (i.e., hurricanes). In addition, the Pacific Coast guidelines should explicitly account for major differences in physiographic settings and wave climates (e.g., open coast and sheltered waters) considering the differences in the analysie required and the importance of sheltered waters in terms of population centers. The format also should account for the potential advantages of accomplishing some portions of coastal studies at a regional scale, such as wave characteristics analysis, wave transformation, and tsunami studies. Specific recommendations based on the review of the existing guidelines are described briefly below.

The existing guidelines incorporate many references to avoid excessive length. The applicability of specific references for the Pacific Coast should be clarified, updated, and connected to specific situations in coastal flood studies. The existing General Guidance lists 32 publications as references covering a variety of subjects, including 10 references on wave height and runup analysis. The list is not categorized by geographic area, geomorphic setting, or type of analysis. A more structured system for referencing specific methods outside of the guidelines is needed.

The study documentation section (Section D1.2) in the existing guidelines is fairly general and is separated from the specific guidance for major geographic areas. It may be preferable to reorganize the Pacific Coast document to show study documentation requirements near the end or in specific technical sections with specifics on the types of information required for specific situations. The study documentation required should be more specific and clearer.

The Pacific Coast guidelines could benefit from improved flowcharts to illustrate the FIS analysis process, including key decision points. The existing section on study organization and overview includes a flowchart (Figure D-1). Some of the steps that may require computations are not represented in the flowchart (e.g., storm meteorology, stillwater elevations, ocean wave characteristics), although they are discussed later in the text. Some of these are shown as “data requirements.” Figure D-1 shows the overall process, and more detailed flowcharts are used to show specific analyses (e.g., Figure D-4 for erosion assessment), but this structure could be expanded and improved. The flowcharts have little relationship to geomorphic settings, but a table is included showing model types for specific settings. The use of geomorphic settings to characterize the types of analysis that are required and the submittal requirements based on geomorphic settings could clarify the study process and review requirements.

Some processes are not treated comprehensively in the existing guidelines, such as storm meteorology and stillwater elevations, in part because of their regional scale and the need for specialized expertise and resources outside the scope of typical coastal studies. Similarly, the Pacific Coast guidelines must address potential regional studies and their use in local studies.

Specific guidance is not included in the existing guidelines for sheltered waters or for areas subject to combined coastal and riverine flood hazards. These are common geomorphic settings on the Pacific Coast, and should be addressed more specifically.

The existing guidelines are generally organized in the order in which a study is completed, but this approach could be improved, and the relationships between types of analyses (e.g., wave setup and runup and overtopping) should be clarified. Key definitions and a glossary should be included. This may be best done in one or more locations in the document to provide definitions relevant to specific technical analyses in a convenient manner. Examples are included in the existing guidelines for hazard zone mapping. Their use is recommended for the Pacific Coast as well, possibly organized by geomorphic setting.

The following list identifies the key recommendations for the structure and format of new Pacific Coast guidelines:

- Clarify the purpose and organization at the beginning of the document.
- Clearly illustrate the study process with a series of flowcharts, including key decision criteria, and the interrelationships between analyses.
- Define the procedures for selected alternative approaches for determining the 1% annual chance flood elevation, including the connection between different elements of the study analysis using these approaches.
- Indicate analyses that may best be accomplished at regional scale and the information to be derived and used in local studies.
- Provide guidance on procedures and data applicable to specific geomorphic settings, including a specific section on sheltered waters and guidance on combined coastal/riverine flood hazards.
- Provide definitions and key examples.
- Provide improved guidance on study documentation more directly related to the types of analyses and settings included in the study.

## 4.3 OPEN COAST AND SHELTERED WATER SETTINGS

"Sheltered Waters" are water bodies with shorelines that are not subjected to the direct action of undiminished ocean winds and waves. Sheltered Water areas are exposed to similar flood-causing processes as those found along open coastlines, such as high winds, wave setup, runup, and overtopping. Present FEMA *G&S* adequately cover many of the general coastal flood assessment procedures needed to complete flood hazard assessments in Sheltered Waters. However, some aspects of sheltered water flood hazards can not be addressed by the current FEMA Guidelines. For example, wind-generated waves are highly dependent on the shape and orientation of the surrounding terrain to prevailing wind directions. Wave generation and transformation in sheltered waters are usually limited by their open water fetch distance, complex bathymetry and often the presence of in-bay and shoreline coastal structures. These sheltering effects reduce wave energy and flood potential compared to open coast areas.

Other processes, including the effects of terrestrial runoff which modify local tidal and surge hydrology and relatively strong in-bay currents often combine to create tidal and hydrodynamic conditions only found in sheltered waters areas. Bays and estuaries often display significant spatial variability in tidal hydrology. For example, south San Francisco Bay often has a standing tide with nearly twice the tide range of central Bay and an elevated mean tide and high water elevation compared to the open coast. By contrast, north San Francisco Bay, which extends into the Sacramento-San Joaquin Delta area, displays a different, progressively muted tidal range that is affected significantly by local winds and river runoff. Oceanic storm surge can be modified in estuaries and it is not clear whether storm surge is uniformly additive to local tidal datums throughout an estuary, or whether storm surge is amplified or muted within an estuary, or within a given region in a large estuary. On the Pacific Coast similar questions arise during El Niño events regarding how elevated oceanic conditions may or may not affect sheltered water tidal elevations. Wave-cast debris from extreme wave runup and overtopping can be especially problematic, owing to the proximity to sources of such materials in many estuaries. These unique sheltered water flood hazards are not adequately addressed in current FEMA Guidelines.

## 4.4 DEFINE THE 1% ANNUAL CHANCE FLOOD HAZARD (TWO APPROACHES)

The NFIP regulations (44 CFR 59.1) define *base flood* as "the flood having a one percent chance of being equaled or exceeded in any given year." The regulations do not define *base flood elevation*, but the meaning seems clear: the flood *elevation* with a one percent chance of being equaled or exceeded in any given year. Calculating this elevation in coastal areas may be difficult, however, because flood elevation is the net result of several processes (e.g., astronomical tide, storm surge, wave setup, infragravity motions, wave heights, event-based erosion, wave runup), some of which are independent and some of which are related.

### 4.4.1 Two Basic Approaches: Response (Statistical) and Event Selection (Deterministic)

The FEMA *G&S* was drafted initially with a primary focus on open coast Atlantic and Gulf of Mexico flooding, which had the result of reducing the 1% annual chance flood elevation determination to computation of a 1% annual chance stillwater elevation and concurrent wave conditions which typically depend on water depth during the event. (Hurricane and extreme northeaster storm surges are large and may inundate low-lying coastal areas. Wave heights in the inundated areas become depth limited.) The procedure for the Atlantic and Gulf Coasts can be thought of as Response or Statistical, because a large number of storms of varying characteristics are simulated and the 1% annual-chance stillwater elevation is determined from the computed response. The added wave component is also computed by Response Method because the response based waves collapse to a maximum depth limited breaking condition.

The Event Selection method was used in the Great Lakes (Dewberry & Davis, 1991), where the 1% annual chance event was considered to be the 1% annual chance stillwater elevation and the 3-year wave height (or, in the case of Lake Ontario, the half-year wave height). Modified event-based erosion, wave height, and runup procedures were developed by FEMA (2003) for use with the defined 1% event.

Specific guidance for determining the 1% annual chance flood elevation along the Pacific Coast has not been developed. However, a variety of techniques have been used over the years, including a modified event selection method for the Sandy Point (Whatcom County), Washington, Flood Insurance Study (PWA, 2002). The PWA procedure defined three distinct water level and wave condition combinations (events), each with a 1% annual probability of occurrence (Figure 1). Wave runup was calculated using each event, and the event yielding the highest runup was used as the basis for flood hazard mapping.

Other procedures employed in Pacific Coast flood mapping can be collectively referred to as a **response** or **statistical** method. In this method, many combinations of water level and wave height conditions are used as input to wave models, a wave runup-frequency relationship is constructed from the model results, and the 1% annual chance runup elevation is identified from the relationship. Unlike the event selection method, no attempt is made to identify a 1% event; instead, the response of the system dictates the 1% flood elevation.

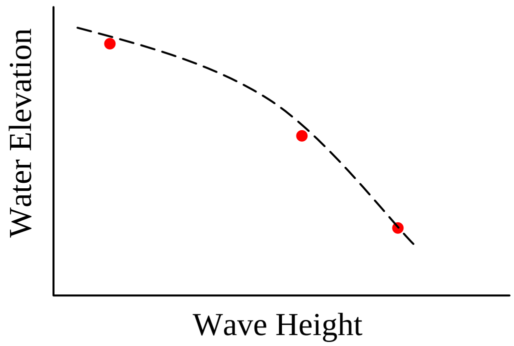


**Figure 3. Multiple water level-wave height combinations (1% events).**

The details of the statistical procedures may vary (e.g., joint probability, coincident time series, Monte Carlo), but each will result in an elevation-frequency distribution from which the 1% elevation is determined (Figure 4). Pacific Coast studies using the response method include the Tetra Tech Southern California Study (1982), and the Ott Water Engineers Northern California Flood Study (1984). More recent reports (1994–2002) detailing the response method have been prepared by the Hydraulic Research Station at Wallingford, and the University of Lancaster, U.K.

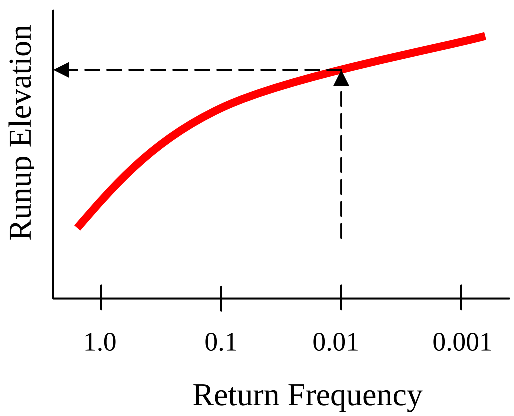


**Figure 4. Runup elevation vs. return frequency.**

### 4.4.2 Implications of Each Method for FEMA Flood Hazard Mapping

For most of the Pacific open coast and of sheltered shorelines on any coast, the event selection method may not be the most appropriate method for two reasons: (1) event specification may be difficult and is not unique (there will not necessarily be a direct correspondence between the 1% annual chance water level and wave conditions), and (2) wave runup will determine the flood elevation for most shorelines, and the maximum wave runup may not necessarily result from the highest water level or the largest waves.

Thus, the response method, although more complicated and time consuming, may yield better results for most Pacific and sheltered coasts. One disadvantage of this method is that revisions to FIRMs will be more difficult to propose and review without a clear specification of events to model. It may be possible to overcome this difficulty (e.g., by working backward from the calculated 1% annual chance runup elevation to one or more water level-wave condition combinations), but this remains to be seen.

#### 4.4.3 Alternatives

Three alternatives are proposed for further study and comparison:

- event selection method (with one or more selected 1% annual chance events),
- response method (using a variety of statistical procedures), and
- hybrid approach (using both methods).

Of these, the hybrid approach requires further elaboration. Such an approach could involve limited use of the response method in a study region—to gain an understanding of the dominant processes/combinations that control the 1% annual chance flood elevation—and concurrent use of the event selection method based on those 1% combinations. In effect, limited use of the response method will help to guide, "calibrate," and extend the applicability of the simpler event selection method.

#### 4.4.4 Proposed Studies for Phase 2

Two study areas are proposed for development, testing, and comparison of the alternative methods listed above: Imperial Beach, California, and Sandy Point, Washington. The latter is a sheltered shoreline where the event selection method has been applied already, but where a 29-year time series of water levels and winds, from which waves can be hindcast, are available for use with the response method and hybrid approach. The former is an open coast shoreline where wave and water level statistics have been compiled and the response method has been applied but where the event selection and hybrid approaches can be applied.

### 4.5 SUMMARY BY TOPIC AREA

#### 4.5.1 Introduction to Technical Category Summaries

The brief subsections that follow provide concise summaries of Focused Study results in the 11 technical categories for the Pacific Coast. The summaries include a brief description of the topics and key issues and a set of recommendations for the Pacific Coast. The recommendations are split into recommended Phase 2 work and recommended future development. The work shown for Phase 2 will produce a set of guidelines specifically for the Pacific Coast and facilitate new and updated coastal flood insurance studies for map modernization.

The work in Phase 2 does not include all the recommended Critical, Important, Available, or Helpful topics. Recommended future development would further improve and expand the guidelines. Future development work is not funded at this time, but these recommendations serve as a reference for planning future FEMA technical guidance work. The following summaries are the direct result of the appended Focused Studies, which include additional discussions, information, and references on the topics.

## STORM METEOROLOGY

### *Topics and Key Issues*

This category covers not only storm meteorology, but also a number of flood frequency issues. Among these are two general methods to determine the 1% annual chance level of some coastal process, characterized as the Event Selection method and the Response-Based method. These terms refer to the manner in which the 1% annual chance coastal flood level is determined. In the Event Selection method, a single 1% offshore storm or wave event is selected with the assumption that if the effects of this single event are followed all the way to the shoreline, they will approximate the true 1% runup. This is a form of the "design storm" concept in the Response-Based method, all significant events are routed from offshore to their runup limits, and only then is the 1% annual chance level determined, based on the entire set of response calculations. The same general approaches apply to processes other than runup. This question is particularly important for the Pacific Coast, where wave effects may be associated with storms at great distance from the coast instead of only with local weather conditions.

There is little guidance in the current *G&S* that is directly transferable to the Pacific Coast regarding event of response methods. For the combination of astronomical tide and storm surge, the study contractor is required to "Describe the method by which the tidal elevation data are convolved with the surge data including tidal constants and tidal records". There is no guidance regarding the combined probability of separate processes such as storm surge and rainfall runoff in a tidal river, and there are no guidelines specifically for the Pacific Coast.

The following Storm Meteorology topic was identified by the TWG:

**Critical –** Topic 51, Combined Probability.

Key issues are:

- The basic flooding mechanism for the Pacific Coast is the combination of waves and high water, where high water is the sum of astronomical tide, storm surge, El Niño, and the static component of wave setup. On the Pacific Coast, the critical combination of these processes is not necessarily associated with a single defined storm type, such as hurricanes is on the Atlantic and Gulf coasts.
- A key issue is whether an Event Selection or a Response-Based method should be applied. The former associates one particular offshore storm or wave event one-to-one with the coastal parameter of interest. The latter considers the effects of a range of offshore conditions, propagating each to the shore, and determining the statistics of the computed responses at the shoreline.
- Candidate methodologies are available for both Event Selection and Response-Based studies including, for example, methods used in the PWA Sandy Point Study and the Tetra Tech Southern California study, as well as the HR Wallingford JOIN-SEA method. These methods require testing before more general guidelines can be written for the Pacific Coast.
- The performance and relative merits of these approaches may differ between open coast sites and sheltered waters. Consequently, it is recommended that case studies be performed in both types of environments to investigate strengths and weaknesses of alternative methods.

- Storm surge, while small on the Pacific Coast, may be addressed by both tide gage analyses and simplified one-dimensional modeling. Appropriate frequency methods will be required to implement the latter, possibly based on Joint Probability Methods (JPM), Empirical Simulation Techniques (EST), or Monte Carlo simulations.
- Tidal rivers subject to riverine flooding are also subject to coastal flooding, which may be entirely independent, or partly correlated. Guidance should be developed to establish the manner in which these processes are integrated in the final mapping (also see Topic 19 of the Hazard Zones Focused Study).
- The astronomical tide often makes a significant contribution to the total stillwater level. Methods to determine the combination of tide and tsunamis, and tide and surge should be established.

### *Recommended Approach*

The recommended approach to these issues includes both the development and verification of methods based partly on the findings of case studies, and the preparation of new guidelines.

Currently available methods include the JPM, EST, and Monte Carlo for storm surge statistics; numerous runup models and methods; and methods for tide and surge combination. The principal problem of the combination of waves and high water has been treated in past studies by PWA and Tetra Tech, and is the subject of the HR Wallingford JOIN-SEA method.

Recommended Approach (Critical Topics)

- Discuss and define methods to determine the 1% annual chance coastal flood level, including consideration of Event Selection and Response-Based methods.
- Document specific methods such as those used in past PWA and Tetra Tech studies, and in the HR Wallingford JOIN-SEA method.
- Perform an Open Coast case study using selected alternative approaches.
- Perform a Sheltered Water case study using selected alternative approaches.
- Based on the above, write draft guidelines on these issues appropriate for Pacific Coast studies.
- Develop guidance for frequency analysis methods for use with Pacific storm surge modeling.
- Develop appropriate methods for the combination of riverine and coastal flood estimates in tidal waters subject to both.
- Develop guidance for the combination of tsunamis and tides.

Tasks associated with Topics defined by the TWG to be *Critical* were considered for completion in Phase 2. However, time and budget constraints in Phase 2 do not allow comprehensive treatment of all the *Critical* subtopics. The table below summarizes the tasks selected for completion in Phase 2, and those deferred for future consideration by FEMA.

**Table 3**
**Storm Meteorology Recommendations – Pacific Coast**

| Topic Number | Topic/Subtopic | Timing | Recommended Approach |
|---|---|---|---|
| 51 | General Methods to Determine 1% Coastal Levels | Phase 2 | Define Event Selection and Response-Based methods for both open coast and sheltered waters |
| 51 | Define Specific Methods, Tools, and Data Guidelines for 1% Analysis | Phase 2 | Document specific methods including, for example, the PWA Sandy Point approach, the HR Wallingford JOIN-SEA method, and the FEMA/Tetra Tech 1982 approach. |
| 51 | Open Coast Case Study | Phase 2 | Perform a case study comparing selected methods at a specific open coast site, preferably one for which prior data is available |
| | | Future | Perform a case study with Monte Carlo Method (Wallingford) using multiple variables. The study will take into account wave related variables of swell (height, period and direction) and sea (height) as well as the still water elevation for the open coast. |
| 51 | Sheltered Water Case Study | Phase 2 | Perform a case study comparing methods at a specific sheltered water site, preferably one for which prior data is available. Monte Carlo Methods will be applied for Sheltered Water. |
| 51 | Storm Surge Modeling Frequency Analysis | Future | Test and recommend methods to associate frequency with storm surge for Pacific Coast surge modeling; recommend appropriate data sources |
| 51 | Surge/Riverine Combination | Future | Prepare recommendations for the statistical combination of surge and a riverine runoff profile, with consideration of non-independence of the processes; See also Topic 19 of the Hazard Mapping Focused Study for simple mapping suggestions |
| 51 | Tsunamis and Tide | Future | Develop guidelines for the combination of tsunamis and tide, including a worked hypothetical example |

## STILLWATER

### Topics and Key Issues

The following Stillwater topics were identified by the TWG:

**Critical –** Topics 54 and 55, Surge vs. Wave Height (Pacific Coast Surge Modeling)

**Available –** Topic 52, Non-Stationary Processes

Key issues are:

- Storm surge estimates can be based on an analysis of tide gage data in some regions. This is especially important on the Pacific Coast where storm surge may typically be on the order of only a foot or two, compared with levels of more than 10 feet common on the Atlantic and Gulf Coasts. Consequently, tide gage analysis may be adequate for Pacific Coast stillwater determination wherever gage data are available.
- The *G&S* do not include any significant discussion of appropriate methods for tide gage analyses.
- The *G&S* provide little guidance on the considerations which must go into a storm surge modeling effort, beyond the assumptions implicit in the use of the FEMA storm surge model.
- A simplified 1-D surge model for the Pacific would be a valuable tool. A suitable prototype for such a model is the one used by the Florida Department of Environmental Protection for Florida coastal construction jurisdiction delineation. Such a model is likely to be of sufficient accuracy for estimation of the small Pacific Coast surge levels, and could be applied in areas for which tide gage data is lacking.
- The *G&S* provide little guidance on the matter of non-stationary processes, and how they might affect both the determination of stillwater levels, and the interpretation of historical data used in a FIS.
- The primary non-stationary processes of concern are the relative change of sea level (sea level rise and/or land subsidence), and localized land subsidence associated, for example, with oil and water extraction or tectonic adjustment.
- Owing to improvements in computer technology, future storm surge modeling efforts can be expanded to a regional scope, providing greater uniformity and accuracy in the surge determinations at reduced cost. While this is true for the Pacific Coast, it is particularly pertinent to the Atlantic and Gulf Coasts.

### Recommended Approach

The recommended approach for addressing these issues includes both the development and verification of analytical and modeling methods (tide gage analysis and development of a 1-D surge model), as well as general revision of the *G&S* to provide greater insight for study contractors into the requirements of coastal modeling and data interpretation. Information is available for development of guidance on non-stationary processes, and for development of general storm surge modeling guidance.

Recommended Approach (Critical and Available Topics)

- Provide guidance regarding methods for determining storm surge based on tide gage data.
- Identify data sources for sea level rise, land subsidence, and tides.
- Implement a simplified 1-D storm surge model and prepare guidelines for its use.
- Write general guidelines for Pacific storm surge modeling.
- Write guidelines on how to consider non-stationary processes in a coastal FIS.

Tasks associated with Topics defined by the TWG to be *Critical* or *Available* were considered for completion in Phase 2. However, time and budget constraints in Phase 2 do not allow comprehensive treatment of all the *Critical* and *Available* topics. The table below summarizes the tasks selected for completion in Phase 2, and those deferred for future consideration by FEMA.

**Table 4**
**Stillwater Recommendations – Pacific Coast**

| Topic Number | Topic/Subtopic | Timing | Recommended Approach |
|---|---|---|---|
| 55 | Tide Gage Analysis | Phase 2 | Select and test methods to extract surge estimates from tide gage data in multiple settings. |
| 54 | Tide Gage Analysis Guidelines | Phase 2 | Document procedures for tide gage frequency analysis. |
| 54 | General Considerations for Surge Modeling | Phase 2 | Based on the existing literature, describe the use of surge models and the factors which require consideration in performing a study. |
| 54 | Simplified Storm Surge Model | Phase 2 | Develop a 1-D (bathystrophic) surge model based on the Florida Department of Environmental Protection methodology. Although primarily for Pacific Coast applications, the model may also be useful as an auxiliary tool for the Atlantic and Gulf Coasts. |
| | | Future | Perform testing and example studies of the 1-D surge model and provide expanded Users Manual based on test results. |
| 52 | Non-Stationary Processes | Phase 2 | Write general guidelines for the consideration of non-stationary processes (for example, relative sea level rise, land subsidence), including identification of major data sources. Include guidance on interpretation of historical data. Suggest documentation of projected map impact. |

# STORM WAVE CHARACTERISTICS

## *Topics and Key Issues*

The following Storm Wave Characteristics topics were identified by the TWG:

**Critical –** Topics 4 and 5, Swell and Seas.

**Available –** Topic 1, Wave Definitions.

Key issues are:

- Sources of wave data, need to be identified.
- Two candidate models, until the updated WIS is ready for use, are the Oceanweather Global Re-analysis of Ocean Waves (GROW) model and Fleet Numerical Meteorology and Oceanography Center WAVEWATCH III model.
- Low frequency swell propagation can be accurately modeled from buoy or hindcast sites outside the islands into shore in the Southern California Bight. But an approach is needed to resolve the impact of local seas on the high frequency portion of the spectrum.
- Current *G&S* refers to the Shore Protection Manual (SPM; USACE, 1984) and Automated Coastal Engineering System (ACES; USACE). Update the *G&S* to be consistent with the Coastal Engineering Manual (CEM; USACE, 2003).
- The CEM method is This is a significant deviation from the SPM. Evaluation of CEM Procedures is needed before including CEM procedure in the *G&S*.
- Include in the *G&S* other Empirical Prediction Methods such as the Composite Fetch Method.
- Spectral Energy Models (SEMs) such as SWAN, STWAVE and MIKE OSW, are available. But, SWAN and STWAVE are not included in the FEMA Approved Numerical Models List.
- Comparisons of Empirical Prediction methods and SEMs are needed to continue using Empirical Prediction Methods and for introducing SEMs.
- Definitions are needed in the *G&S* of wave types (sea, swell, and tsunami) in both the time domain and the frequency domain. Two available resources are the CEM and the “List of Sea State Parameters” published by the International Association of Hydraulic Research.
- Specific guidance is needed on how the wave related terms relate to the coastal processes associated with flood studies, methodologies, and models.

## *Recommended Approach*

Storm Wave Characteristics topics were classified by the project team as *Critical* and *Available*. The recommended approach involves revision to the *G&S* using available references and information, and detailed investigations of wave databases and a case study. Topic 5 (Nearshore Representation of Local

Sea for Southern California Bight) is a critical topic, but it is not studied under Phase 2 to accommodate other critical topics from other Focused Studies within the limited resources. Also, this topic can be studied together with regional wave transformation modeling for the Southern California Bight.

Recommended Approach (Critical and Available Topics)

- Recommend use of GROW database for sea and swell. Study the GROW database for one location. Confirm lack of bias and validate data with measured records. Check whether the dataset properly represented extreme events.
- Develop *G&S* for preparation of input data for wave transformation models based on GROW directional spectra.
- Describe the WIS Pacific Coast Database Development and guidance for use in flood insurance studies.
- Conduct a study of the available nearshore data for Southern California Bight to assess whether inclusion of the local wind will make a significant change in the high frequency part of the spectrum.
- Based on results from the study above, adopt one of the three alternatives: 1) assuming no change in wind-induced change in the spectrum, or 2) attempt to model wind-induced changes, or 3) treat changes to the wind wave portion of the spectrum as an independent variable and use joint probability analysis techniques
- Conduct a case study to compare results using CEM procedures to results using SPM procedures for restricted fetch condition is recommended.
- Conduct a Focused Study to compare results from the SEMs and traditional Parametric Models, using restricted fetch methods. Application procedures for the SEMs would be clarified, specifically wind field definition.
- Incorporate and refine the "Glossary of Coastal Terminology" directly from the USACE CEM and from the listings of notations and parameters in the January 1986 publication from the International Association for Hydraulic Research titled, "List of Sea State Parameters.”
- Provide specific guidance on use of wave related definitions for physical processes applicable to coastal flood studies

Tasks associated with Topics defined by the TWG to be *Critical* or *Available* were considered for completion in Phase 2. However, time and budget constraints in Phase 2 do not allow comprehensive treatment of all the *Critical* and *Available* topics. The table below summarizes the tasks selected for completion in Phase 2, and those deferred for future consideration by FEMA.

**Table 5**
**Storm Wave Characteristics Recommendations – Pacific Coast**

| Topic Number | Topic/Subtopic | Timing | Recommended Approach |
|---|---|---|---|
| 4, 5 | Sea and Swell for Pacific Coast | Phase 2 | Review GROW dataset for one location. Check whether the dataset represents extreme events adequately. Confirm lack of bias in the database. Develop *G&S* on use of GROW and steps for developing input data to wave transformation models. Describe the WIS database development and potential use in coastal flood insurance studies. |
| 4, 5 | Nearshore Representation of Local Sea for Southern California Bight | Future | Conduct a study of the available nearshore data for Southern California Bight to assess whether inclusion of the local wind makes a significant change in the high frequency part of the spectrum. Based on the results of the above study, adopt one of the three alternatives: a) assuming no change in wind-induced change in the spectrum, or b) attempt to model wind-induced changes, or c) treat changes to the wind wave portion of the spectrum as an independent variable and use joint probability analysis techniques |
| 4, 5 | Wave Generation in Sheltered Waters | Phase 2 | Compare CEM and SPM procedures using a case study (an existing FIS site) and clarify application of CEM in FEMA studies. Perform a case study to compare SEMs and traditional parametric models using restricted fetch methods. |
| 4, 5 | Wave Generation in Sheltered Waters | Future | Develop application procedure for SEMs including wind field definition based on detailed testing. |
| 1 | Wave Definitions | Phase 2 | Using the compiled glossary of terms and notations (from CHL and IAHR sources), correlate each of key terms with the coastal methodologies and application. Prepare for application for Pacific Coast Guidelines |

# WAVE TRANSFORMATION

## *Topics and Key Issues*

Wave Transformations are important processes that change wave characteristics when waves propagate toward shore. These are addressed as an intermediate step between forcing processes (wave generation) and response processes (wave setup, wave runup, and overtopping) in coastal flood studies.

Wave Transformation receives input from forcing processes (wave generation) and provides output to response processes (wave setup, runup, and overtopping). Coordination with the other Focused Study categories is necessary.

The following Wave Transformation topics were identified by the TWG:

**Critical –** Topic 7, CDIP California; Topic 8, Overall Wave Transformations; Topic 9, Dissipation.

**Helpful –** Topic 10, WHAFIS.

**Important –** Portions of Topic 7, CDIP and Topic 9, Dissipation.

Key issues are:

- Presently, the *G&S* do not include a description of wave transformations. The methods defined in the current *G&S*, (depth limited waves) are biased toward the Atlantic and Gulf Coasts, and are inadequate for the Pacific Coast.
- Flood insurance studies for sites in the Pacific Coast Region have addressed wave transformations with different levels of complexity. The *G&S* should address the selection of methods based on the physical parameters that are encountered in the wave transformation process.
- Wave transformation analysis is required to support wave setup calculations. In particular, methods describing wave breaking and associated momentum transfer are needed.
- Contemporary wave transformation models are available and necessary for use in future studies, but are not currently recognized by FEMA.
- The Coastal Data Information Program (CDIP) currently operates a regional model that hindcasts nearshore waves along the California coast. The model transfer functions are already available to transform deepwater wave spectra to nearshore spectra, but the windwave growth is not included in this model.
- Application of the CDIP wave transformation models in central and northern California is not complete.
- Wave dissipation due to bottom effects is not routinely considered in wave transformation processes. Study contractors need guidance on when and where to apply bottom dissipation mechanisms. Some guidance is available in the current *G&S*; but primarily addresses the Atlantic and Gulf Coasts.

- Overland wave propagation is common during extreme events in the Atlantic and Gulf Coasts, and the WHAFIS 3.0 software, approved by FEMA, is typically used. Overland wave propagation can be significant in some locations in the Pacific Region, but use of WHAFIS for Pacific Coast studies will require modifications to the wind speeds specified based on Atlantic and Gulf Coast conditions.

### *Recommended Approach*

The recommended approach focuses on development of a combination of regional and local wave transformation tools. Considerable effort is required to implement these recommendations. Adequate attention must be devoted to coordination with guidelines development for Storm Wave Characteristics, Wave Setup, and Wave Runup.

Recommended Approach (Critical Topics)

- Write *G&S* for Wave Transformations, based on a review of available literature and experience gained by the application of models and methods.
- Review available literature and guidance on the range of applicability of contemporary computer models, recommend models for inclusion on the list of "Coastal Models Accepted by FEMA for NFIP usage", and provide guidance on their application to FEMA FISs.
- Research available literature on wave groups, infra-gravity waves, and shallow water spectra for input into wave setup and runup calculations.
- Evaluate adequacy of linear wave transformation models and needs to supplement these models. Place emphasis on representation of infragravity waves.
- Use the CDIP regional wave models to create 2 sets of wave transformation coefficients in Southern California: 1) for swell waves and 2) for local wind generated waves.
- Demonstrate the CDIP model skill for predicting nearshore wave conditions during large winter storms using existing buoy data (for the southern, central, and northern California coast).
- Create database, provide user's manual, and develop Fortran and MATLAB codes to assist contractors in using the CDIP model coefficients.
- Incorporate applicable sections of existing *G&S* for other geographical areas that cover the overland propagation and wave energy dissipation topics.
- Summarize available information on wave dissipation over marsh and mudflats in the Pacific. Develop criteria to evaluate importance of wave dissipation. Evaluate if changes to WHAFIS dissipation criteria are necessary.

Recommended Approach (Important or Helpful Topics)

- Apply CDIP regional wave transformation modeling for the California Coast.
- Consider expanding regional wave modeling for Washington and Oregon coasts using CDIP or other programs (e.g., WIS).
- Evaluate any limitations due to the linearity of the transformation models.

- Research on wind wave and swell spectra combination.
- Conduct field data collection for wave dissipation on Pacific Coast
- Develop *G&S* for WHAFIS application for the Pacific Coast

Tasks associated with topics defined by the TWG to be C*ritical* were considered for completion in Phase 2. However, time and budget constraints in Phase 2 do not allow comprehensive treatment of all the C*ritical* topics. *Important* topics cannot be completed within the time frame of the project. Topics characterized as *Helpful* were also deferred for future consideration due to their lower priority. The table below summarizes the tasks selected for completion in Phase 2, and those deferred for future consideration by FEMA.

**Table 6**
**Wave Transformation Recommendations – Pacific Coast**

| Topic Number | Topic/Subtopic | Timing | Recommended Approach |
|---|---|---|---|
| 8 | Wave Transformation with and without Regional Models | Phase 2 | Write *G&S* for Wave Transformations. Tasks: 1) conduct several Focused Studies to inform the Wave Transformations *G&S*; 2) use available publications to identify a range of methods; 3) develop criteria for level of analysis; 4) include development of guidelines for spatial coverage and wave parameters, and include use of regional models such as CDIP; 5) research available literature to adequately define wave groups, infragravity waves, shallow water spectra, etc. for input into wave setup and runup calculations; 6) review available literature and guidance on the range of applicability of contemporary computer models, recommend models for inclusion on the FEMA pre-approved coastal model list, and provide guidance on their application to FEMA FISs; 7) incorporate applicable sections of existing G&S for other geographical areas that cover the overland propagation and wave energy dissipation topics. (Topics 9 &10) |
| | | Future | Evaluate wave transformation models using a selected data set. |
| 7 | California Regional Wave Transformation Models | Phase 2 | Provide CDIP Southern California validation examples and a test case for testing other WT models;<br>Provide guidance and Users Manual on use of CDIP models and model output such as existing model coefficients. |
| | | Future | Use CDIP model to create 2 sets of wave transformation coefficients for southern California, 1) for swell waves and 2) for local wind waves; Expand CDIP for the California Coast. Validate the models for central and northern California; Create database, provide expanded user's manual, and develop Fortran and MATLAB codes to assist contractors in using the CDIP model coefficients.<br>Consider expanding regional wave modeling for Washington and Oregon coasts using CDIP or other programs (e.g., WIS) at the appropriate time and depending on the need, recognizing that regional wave models are more logical in densely populated areas. Individual studies may be performed in sparsely located communities (see Topic 8).<br>Evaluate any limitations due to the linearity of the transformation models.<br>Conduct research on wind wave and swell spectra combination. |
| 9 | Wave Energy Dissipation over Shallow Flat Bottoms | Phase 2 | Evaluate wave dissipation over marsh and mudflats in the Pacific Coast from available information; Develop criteria to evaluate importance of wave dissipation in FISs; Recommend changes to methods and WHAFIS dissipation criteria to the extent feasible. |
| | | Future | Conduct field data collection to characterize wave dissipation over marsh and mudflats and other shallow, dissipative shores in the Pacific; provide expanded guidance for calculating wave dissipation. |
| 10 | Overland Wave Propagation | Future | Evaluate if changes to WHAFIS dissipation criteria are necessary (see Topic 9), and *G&S* modifications for Pacific Coast. |

## WAVE SETUP

### Topics and Key Issues

The following Wave Setup topics were identified by the TWG:

**Critical –** Topics 44 and 45, Define, Document, Compile Data; Topic 46, Interim Method.

**Important –** Topic 47, Develop Ideal Method-Coupled; Topic 48, Dynamic Wave Setup.

Key issues are:

- Under the action of irregular waves, wave setup consists of a static component and a dynamic component. Owing to the long waves that occur on the Pacific Coast, the latter can be quite substantial.
- The setup on the Pacific Coast can be significantly larger than the wind and barometric components during a 1% annual chance event owing, in part, to the narrow continental shelf. Thus, the dominant components will be the astronomical tide and wave setup possibly augmented by an El Niño contribution.
- Dynamic wave setup needs to be addressed. The Pacific Coast may have dynamic wave setup conditions, and the current *G&S* for the Atlantic and Gulf Coasts are based on static.
- Wave setup will require specification of directional wave spectra as input at an offshore location seaward of wave breaking.
- Wave setup is included, to some degree, in wave runup measurements and methods.
- There are two approaches for calculating wave setup: 1) The Boussinesq models which, in principle, can calculate both wave setup and wave runup, and 2) Coupling of more conventional engineering approaches.

### Recommended Approach

It is recommended that methodologies be developed and *G&S* written that address the following: 1) steady and dynamic setup components, 2) irregular waves [implicit in (1) above], 3) characterization of nearshore bathymetry, 4) a valid wave breaking model, 5) nonlinear and directional characteristics of $S_{xx}$, and 6) wave damping where appropriate. An effort should be made to ensure that the interim method address as many of the physiographic settings applicable to the Pacific Coast as possible. A program will be developed which will calculate wave setup using, as input, the wave spectra outside the breaking zone.

Recommended Approach (Critical Topics)

- Prepare definitions applicable to Pacific Coast.
- Based on an intercomparison of Boussinesq models and comparison with data sets, determine whether this type model is appropriate for calculating wave setup and wave runup. If applicable to setup, select one of several Boussinesq models for further application.

- Develop and document an engineering based approach for wave setup modeling along open coasts and in sheltered waters based on methods and procedures available from past studies and literature and for specific types of input data (e.g., wave spectra). Note: This task would be reduced if Boussinesq models are selected.
- Compile potential data sources for testing a new Pacific Coast setup model.
- Develop breaking zone model with particular emphasis on wave setup, proof test, compare with data sets, refine, and write User's Manual. Note: The first portion of this task would be reduced if Boussinesq models are selected.

Recommended Future Development (*Important* Topics)

- Develop Ideal Methodology coupling storm surge and wave models to calculate static wave setup.
- Develop procedure for dynamic wave setup

Tasks associated with Topics defined by the TWG to be *Critical* were considered for completion in Phase 2. However, time and budget constraints in Phase 2 do not allow comprehensive treatment of all the *Critical* topics. *Important* topics cannot be completed within the time frame of the project. The table below summarizes the tasks selected for completion in Phase 2, and those deferred for future consideration by FEMA.

**Table 7**
**Wave Setup Recommendations – Pacific Coast**

| Topic Number | Topic/Subtopic | Timing | Recommended Approach |
|---|---|---|---|
| 44, 45 | Pacific Coast Definitions | Phase 2 | Develop wave setup definitions with emphasis on Pacific Coast applications. |
| 46 | Evaluate Boussinesq Models | Phase 2 | Intercompare at least three Boussinesq models and compare with data. |
| 46 | Develop Engineering Based Approach | Phase 2 | Couple accepted engineering models for calculating wave setup across surf zone. Include procedure for dynamic wave setup. |
| 44, 45 | Compile Data for Testing | Phase 2 | Locate as much quality field data as possible for testing of developed/selected approach(es). |
| 44, 45 | Compile Data for Testing | Future | Locate and compile comprehensive national and international data sources for testing a new Pacific Coast setup model |
| 46 | Develop Breaking Zone Model | Phase 2 | Evaluate candidate breaking zone models that allow specification of non planar profile. |
| 46 | Develop Draft Guidelines and Specifications | Phase 2 | Incorporate findings from above into draft Guidelines and Specifications. |
| 46 | Develop Interim Method | Future | Test Model over a wide range of settings and develop and expand User’s Manual based on test results. |
| 47 | Ideal Model for Static Wave Setup | Future | Couple wave generation and wave setup model, allowing specification of arbitrary tide. |
| 48 | Develop Model for Dynamic Wave Setup | Future | Develop method based on directional and nonlinear spectrum as input. |

## WAVE RUNUP AND OVERTOPPING

### *Topics and Key Issues*

The following Wave Runup and Overtopping topics were identified by the TWG (Note that some of the workshop-assigned priorities and topic details were revised during the Focused Study):

**Critical –** Topic 12, Mean vs. higher value; Topic 11, Methods and models.

**Important –** Topic 14, Wavecast debris.

**Available –** Topic 49, WRUP$^{TM}$; Topic 13 Overtopping volumes.

Key issues are:

- Wave runup and overtopping will control BFEs and flood hazard zones along much of the Pacific Coast, where storm surges are low and where WHAFIS-type analyses yield low wave crest elevations. Wave runup analyses must be undertaken along those shore types analyzed for runup along the Atlantic and Gulf Coasts, plus low-profile beaches and barriers.
- Extreme runup levels tend to occur during El Niño events along the entire Pacific Coast (and possibly during hurricane events for southern California). Infragravity motions are more common and more significant on the Pacific Coast than the Atlantic or Gulf Coasts.
- Runup methodologies need to be tested against Pacific data sets that include El Niño events and infragravity waves. Wave setup may be calculated separately or included in wave runup estimates, but must be considered.
- Mapping the mean runup value may fail to adequately capture wave runup hazards.
- Mapping hazard zones with the mean overtopping rate should be sufficient, provided the thresholds for mapping hazard zones recognize the rates tolerated by buildings and structures.

### *Recommended Approach*

The recommended approach involves a detailed evaluation and testing of available wave runup and overtopping methods and models, using Pacific Coast data sets, in conjunction with testing during other studies, particularly case studies in the Storm Meteorology Group.

<u>Recommended Approach (Critical and Available Topics)</u>

- Evaluation of CDIP-type and Oregon-type methods as interim methods for use until more detailed runup testing and runup calculation procedures are developed.
- Limited testing of the RUNUP 2.0 methodology in conjunction with storm meteorology, wave transformation and wave setup tasks.
- Evaluation of Pacific Coast wave runup data, including consideration of wave runup elevation distributions and associated structural damages. The $R_{50\%}$ runup value will be evaluated with regard

to its ability to capture damaging wave runups. If appropriate, an alternate $R_{x\%}$ value will be recommended and an interim procedure will be developed to adjust RUNUP 2.0 results.

- More comprehensive testing of wave runup methods and models is recommended, along with the identification of appropriate runup calculation procedures for a wide variety of shore types, profile characteristics, and incident water level and wave conditions.
- Evaluate WRUP$^{TM}$ and compare with other models.
- Overtopping methods and data will be evaluated to determine whether NFIP thresholds for mapping landward flood hazard zones are consistent with recent literature on “acceptable” overtopping quantities.
- Update procedures for calculating overtopping and ponding on low bluffs, with gently sloping or adverse slopes.

Recommended Approach (Important Topics)

Review and refine methods for defining flood hazards from wave-cast debris. This task will be undertaken in the hazard zone study.

Tasks associated with Topics defined by the TWG to be *Critical* or *Available* were considered for completion in Phase 2. The *Important* Topic (Topic 14, wavecast debris) will be completed with the Hazard Zone Study within the time frame of the project. Time and budget constraints in Phase 2 do not allow comprehensive treatment of all the *Critical* and *Available* topics. The table below summarizes the tasks selected for completion in Phase 2, and those deferred for future consideration by FEMA.

**Table 8**
**Wave Runup and Overtopping Recommendations – Pacific Coast**

| Topic Number | Topic/Subtopic | Timing | Recommended Approach |
|---|---|---|---|
| Topic number not assigned | Runup on Beaches and Low Barriers | Phase 2 | Revise guidance to call for runup analyses for sandy beach, small dune shore type |
| 12 | Evaluate Use of Mean Runup Value | Phase 2 | Evaluate use of $R_{50\%}$ and select alternate $R_{x\%}$ value (probably between $R_{33\%}$ and $R_{10\%}$) if $R_{50\%}$ understates observed hazard.<br>Develop an Interim procedure to adjust RUNUP2.0.output. |
| 12 | Evaluate Use of Mean Runup Value | Future | Review runup distributions for beaches and structures during El Niño, coastal storm and hurricane conditions; review runup damages. |
| 11 | Wave Setup Component | Phase 2 | Current FEMA methodology includes the wave setup component in the calculated runup height. This procedure should be revisited for its appropriateness along the Pacific, and depending on recommended Pacific methodology (coordinate with Wave Setup study) |
| 11 | Infragravity Motions | Future | Consider effects of infragravity motions, which amplify runup and overtopping, and can be substantial along the Pacific Coast |
| 11 | Wave Setup Component | Phase 2 | Current FEMA methodology includes the wave setup component in the calculated runup height. This procedure should be revisited for its appropriateness along the Pacific, and depending on recommended Pacific methodology (coordinate with Wave Setup study) |
| 11 | Conduct Comparative and Sensitivity Testing of Runup Models and Methods | Phase 2 | Evaluate CDIP-type and Oregon-type methods as interim approaches. Coordinate with case studies in Storm Meteorology, Wave Transformation studies.<br>Test runup methods and models in conjunction with other tests (use common data sets to test wave generation through stillwater level and runup). |
| 11, 49 | Conduct Comparative and Sensitivity Testing of Runup Models and Methods | Future | Identify appropriate runup methods and models by location, morphology and hydraulic conditions.<br>Compare results using simple methods versus numerical models, deterministic (event selection) versus statistical approaches.<br>Write Guidelines on input conditions uncertainty. |
| 13, 14 | Overtopping Rates | Phase 2 | Maintain use of mean overtopping rate (cfs/ft, $m^3/m^3$/s per m)<br>Determine damaging overtopping rates for buildings and evaluate current FEMA hazard zone thresholds.<br>Evaluate FEMA's guidance which limits the runup elevation to 3 feet above a barrier's crest elevation<br>Coordinate with Hazard Zone study. |
| 13 | Overtopping Rates | Future | Overtopping at low profile beaches and barriers, dune remnants, revetments, and vertical walls should be evaluated, including consideration for calculating overtopping and ponding on low bluffs with gently sloping, flat or adverse slopes. |

# EVENT - BASED EROSION

## *Topics and Key Issues*

The following Event - Based Erosion (EBE) topics were identified by the TWG:

**Critical –** Topic 30, Geometric Techniques; Topic 33, Cobble/Shingle Effects; Topic 35, Erosion in Sheltered Waters.

**Available –** Topic 31, Bluff Erosion; Topic 32, Geometric Methods for Bluff Erosion; Topic 41, Long-term Erosion; Topics 42 and 43, Nourished Beaches.

**Important –** Topic 34, Geometric Methods for Cobble/Shingle Beaches; Topic 36, Geometric Methods for Sheltered Waters; Topic 38, Process-Based Methods.

**Helpful –** Topic 40, Document vertical erosion depths.

Key issues are:

- Guidance for evaluating EBE remains unchanged since 1989 and focuses primarily on effects of extreme storms (hurricane or northeasters) along the Atlantic and Gulf Coasts, with a modified approach for the Great Lakes Coasts. Coastal erosion processes and storm characteristics found on the Pacific Coast differ dramatically from those along the Atlantic, Gulf, or Great Lakes.
- FEMA *G&S* can be improved by expanding or adding discussions on potential effects of EBE on runup and base flood elevation.
- The eroded beach profile that exists during the base event is needed in order to calculate the 1% annual chance flood elevation.
- Improved EBE *G&S* and new *G&S* need to embody the same fundamental structure that includes: 1) physiographic and geomorphic setting, 2) sediment characteristics across the active profile, 3) time histories of wave and storm tide characteristics, and 4) local or regional oceanic (El Niño) or topographic (recent tectonic adjustments) characteristics that may affect the study area. Consideration of this common structure will ensure that EBE assessments will be consistent for all applications.
- Guidance for evaluating erosion of cobble/shingle beaches is needed.
- Guidance for evaluating erosion of sandy and non-sandy bluffs and cliffs is needed.
- Guidance for evaluating erosion within sheltered water areas is needed.
- Present *G&S* provide no specific guidance on how to address beach nourishment projects.
- Present *G&S* can be improved by adding discussions of the seasonal effects of littoral as well as off-shore and on-shore sand transport and how those processes may affect beach erosion and seasonal changes in beach profiles that occur along the Pacific Coast

- Existing *G&S* can be improved by better defining "storm-induced erosion" or EBE, and different approaches for assessing beach and back beach profile changes due to erosion on all coasts of the U.S.

- Process based numerical models (1-D and 2-D, steady and unsteady) may provide improved means for assessing EBE. Evaluation of process-based models and comparison of their results with those from geometric methods is recommended

### Recommended Approach

Event Based Erosion topics were classified by the project team as *Critical*, *Available*, *Important* and *Helpful*. Initially, the *G&S* should be updated using available references and information to address topics presently covered in the *G&S*. New *G&S* for the Pacific Coast will include new information and methods for assessing EBE in a variety of settings as discussed in the Focused Studies. New methods will fall into three categories and levels of effort: 1) eroded profiles based on available historical mapping and photographs, 2) profiles based on simplistic empirical methods, and 3) profiles developed from process-based (steady and unsteady) models.

Recommended Approach (Critical and Available Topics)

- Provide interim EBE *G&S* based primarily on historical beach profiles and field observations.

- Develop guidance for determining a "Most Likely Winter Beach Profile" for different settings on PC, including areas of beach nourishment.

- Evaluate and test selected geometric methods for beach and dune erosion applications along the Pacific Coast. Methods should include effects of storm duration and sediment erodibility. Document results.

- Provide discussion of bluff and cliff erosion in different settings to distinguish this type of erosion hazard from other erosion processes; provide examples, figures, and definitions.

- Develop interim approach for assessing bluff and cliff erosion in different settings based on historical profile data.

- Provide discussion of gravel, cobble, and shingle beach and dune erosion in different settings to distinguish this type of erosion hazard from other erosion processes; provide examples, figures and definitions; explain limitations of existing 540 sf Criterion for application to this type of erosion and setting.

- Develop interim approach for assessing gravel, cobble, and shingle beach and dune erosion based on historical beach profile data.

- Provide definitions and discussion of EBE found in sheltered water areas for *G&S*; provide interim *G&S* based on historical beach profiles and field observations.

- Provide language in *G&S* directing study contractors to notify FEMA if their study area includes a beach nourishment area and provide FEMA with a list of information needed to assess special cases where beach nourishment may be considered in determining hazard zones and BFEs (as an exception to existing policy).

Recommended Approach (*Important* Topics)

- Continue to develop and test geometric methods and process-based numerical models for beach and dune erosion applications along the Pacific Coast. Methods should include effects of storm duration and sediment erodibility. Document results and prepare updates for *G&S*.
- Prepare improved *G&S* for assessing bluff and cliff erosion in different settings.
- Evaluate/develop methods (geometric or process-based) for assessing gravel, cobble, and shingle beach and dune erosion.
- Long-term processes are considered important to NFIP, but FEMA action on previous work is pending. Therefore, guidance is best developed by FEMA in the Future
- Perform future pilot EBE study(s) in sheltered waters; refine interim assessment procedures; consider use of process based p-b models; prepare updated *G&S*.
- Develop suite of process based models for general coastal erosion assessments for different settings and material types, including sheltered waters.

Tasks associated with topics defined by the TWG to be *Critical* or *Available* were considered for completion in Phase 2. However, time and budget constraints in Phase 2 do not allow comprehensive treatment of all the *Critical* and *Available* topics. *Important* topics can not be completed within the time frame of the project. The *Helpful* topic was deferred for future consideration due to its lower priority. The table below summarizes the tasks selected for completion in Phase 2, and those deferred for future consideration by FEMA.

**Table 9**
**Event Based Erosion Recommendations – Pacific Coast**

| Topic Number | Topic/Subtopic | Timing | Recommended Approach |
|---|---|---|---|
| 30 | Geometric Methods for Assessing Erosion | Phase 2 | Evaluate geometric methods and models. Develop *G&S* for determining most likely Pacific winter beach profile, including beach nourishment areas. Evaluate geometric modeling procedures for sand beaches and dunes on PC and test with available data sets. At a minimum, prepare interim *G&S* methods based on historical beach profiles and field observations. |
| 31, 32 | Bluff and Cliff Erosion | Phase 2 | Review available literature and reporting; provide language and descriptions to PC *G&S* to distinguish bluff and cliff erosion from other processes; provide figures and examples. Review existing bluff erosion procedures and international literature. Discuss interim approach for estimating bluff and cliff erosion based on historical profile data. |
| | | Future | Develop geometric procedures for bluff and cliff erosion and retreat. Consider development and use of process-based numerical/statistical modeling methods for future inclusion in the NFIP program. |
| 33, 34 | Gravel, Cobble, and Shingle Beach and Dune Erosion | Phase 2 | Provide discussion of gravel, cobble, and shingle beach and dune erosion in different settings to distinguish this type of erosion hazard from other erosion processes. Provide examples, figures and definitions. Discuss a simplified interim approach for cobble/shingle beaches based on historical beach profiles. |
| | | Future | Explain limitations of existing 540 Criterion for application to this type of erosion and setting. Discuss simplified *interim approach* for assessing gravel, cobble and shingle beach and dune erosion based on historical beach profile data. Develop geometric procedures for gravel, cobble and shingle beach erosion.<br>Consider development and use of process-based numerical/statistical modeling methods for future inclusion in the NFIP program. |
| 35, 36 | *G&S* in Sheltered Water areas | Phase 2 | Provide definitions and discussion of EBE found in sheltered water areas for *G&S;* provide interim *G&S* based on historical beach profiles and field observations |
| | | Future | Perform future pilot EBE study(s) in sheltered waters; refine interim assessment procedures; consider use of process-based models; prepare updated *G&S* |
| 38 | Physics/Process Based Methods | Phase 2 | Discuss difference between simplified geometric methods and Processed-Based models. |
| | | Future | Develop suite of Processed-Based models for general coastal erosion assessments for different settings and material types, including sheltered waters and overwash |
| 40 | Document vertical depths of erosion | Future | Document depths of erosion following storm events and maintain data for depths of erosion and damages to buildings in order to better determine “depth-damage” relationships. |
| 41 | Long-term Erosion | Future | This topic is considered important to NFIP, but FEMA action on previous work is pending. Therefore, guidance is best developed by FEMA in the future. |
| 42, 43 | Nourished Beaches | Phase 2 | Provide language in *G&S* directing study contractors to notify FEMA if their study area includes a beach nourishment project and provide FEMA with a list of information needed to assess special cases where beach nourishment may be considered in determining hazard zones and |

**Table 9**
**Event Based Erosion Recommendations – Pacific Coast**

| Topic Number | Topic/Subtopic | Timing | Recommended Approach |
|---|---|---|---|
| | | | BFEs (exception to existing FEMA policy). |

## COASTAL STRUCTURES

### *Key Topics and Issues*

The following Coastal Structures topics were identified by the TWG:

**Available –** Topic 21, Failed Structures; Topic 23, Buried Structures; Topic 25, Flood Protection Structures; Topic 27, Coastal Levees.

**Important –** Topic 24, Structures-Tsunamis.

**Helpful –** Topic 22, Failed Structure Configuration; Topic 26, Adjacent Properties.

Key issues are:

- Coastal structures can modify flood levels, wave effects, and topography landward, seaward, and adjacent to the structures, and must be considered during the mapping of coastal flood hazards. Two scenarios are commonly encountered: 1) Structures and their effects are analyzed during Flood Insurance Studies, and 2) Structures frequently serve as the basis for revisions to FIRMs. Treatment of structures in these two cases should be consistent.
- FEMA *G&S* can be improved by expanding or adding discussions on coastal structure failure, buried structures, and the effects of structures.
- The effects of structures can be divided into two categories; effects on erosion and effects on flood conditions. Two scenarios are important for each: 1) The effects of structures on adjacent properties, and 2) The effects on property immediately landward and seaward of a structure.
- Guidance for evaluating coastal structures has been largely unchanged since publication of the USACE report CERC TR 89-15 in 1989. The evaluation criteria and guidance need to be reviewed considering more recent publications and information. Revisions may or may not be warranted.
- Guidance needs to clearly state that study contractors are not required to use CERC TR 89-15.
- Guidance on the evaluation of coastal structures in tsunami-prone areas is needed.
- FEMA *G&S* call for structure "removal" from subsequent flood hazard analyses in the event that a structure fails (i.e., does not survive the base flood event), but guidance on uncertified structure removal should be expanded and revised. More importantly, the configuration of a failed structure can affect wave runup and overtopping calculations. A method to address uncertified structures, used in a recent Pacific Coast flood study (by PWA), has been modified by the Focused Study and is recommended for use.
- Coastal structures and levees are sometimes treated differently, and those differences should be justified or eliminated. The *G&S* should address coastal levees.
- FEMA *G&S* were written primarily considering seawalls, bulkheads, revetments, and do not address the effects of other structure types (e.g., jetties, groins, breakwaters). While treatment of these other structures is needed, it is deemed a lower priority than revising the guidance related to seawalls, bulkheads, revetments, and levees.

### *Recommended Approach*

The recommended approach involves making revisions to the *G&S* using available references and information. The effort will be modest by comparison with some of the other Focused Study topics.

Recommended Approach (Available Topics)

- Buried structures and failed structure configurations (including progressive collapse of revetments).
- Treatment of failed ("removed") structures for wave height and runup analyses.
- Investigation of structure effects on erosion and flood hazards.
- Consistency in treatment of coastal structures and coastal levees.
- Work with Tsunami Group to develop guidance for evaluating structures in tsunami-prone areas.

Recommended Approach (Helpful Topics)

- Revision/update of CERC TR 89-15 coastal structure evaluation criteria.
- Development of minimum structure characteristics necessary to receive mapping credit during Flood Insurance Studies and flood map revisions.
- Revision of guidance to consider coastal jetties, groins and breakwaters.

Tasks associated with Topics defined by the TWG to be *Available* were considered for completion in Phase 2. However, time and budget constraints in Phase 2 do not allow comprehensive treatment of all the *Available* topics. Topic 26, characterized as *Helpful*, was deferred for future consideration due to its lower priority. However Topic 22, which is also characterized as *Helpful*, was included for completion in Phase 2 because the topic has been a significant one in past FIS work in the Pacific Coast. The table below summarizes the tasks selected for completion in Phase 2, and those deferred for future consideration by FEMA.

**Table 10**
**Coastal Structures Recommendations – Pacific Coast**

| Topic Number | Topic/Subtopic | Timing | Recommended Approach |
|---|---|---|---|
| 21a, 21b.1, 23 | Failed and Buried Structures | Phase 2 | Revise guidance to better describe buried structures and failed structure configurations (including progressive failure of revetments). |
| 22a, 22b | Wave Effects Analyses at Failed Structures | Phase 2 | Using modified PWA method, write guidance for mapping runup and overtopping at uncertified (or failed) coastal structures. |
| 25 | Flood protection Structures | Phase 2 | Mention in guidance, detailed TR 89-15 evaluation/certification of coastal structures are not required during FIS, but discuss implications |
| 26a, 26b, 26d | Effects of Structures on Erosion, Flood Hazards | Phase 2 | Investigate effects of structures on erosion and flood hazards; develop guidance for incorporation into flood hazard mapping. |
| 27a | Coastal Levees and Structures | Phase 2 | Identify and resolve inconsistencies in treatment of coastal levees and coastal structures |
| 24 | Tsunami-prone Structures | Future | Investigate historical data on structure failure/success during tsunamis; develop evaluation criteria for tsunami-prone structures. |
| 27b, 27c | Structure Evaluation Criteria | Future | Review CERC TR 89-15 considering more recent data on structure stability and failure; revise structure evaluation criteria for existing and new structures. |
| 21b.2 | Jetties, Groins, Breakwaters | Future | Develop criteria/guidance for evaluating failure of other structure types, and the effects of these failures on mapped flood hazards |
| 26e | Minimum Structure Dimensions | Future | Determine minimum structure dimensions necessary to receive mapping credit during FIS and revisions to FIRMs |

## TSUNAMI

### *Topics and Key Issues*

The following Tsunami topics were identified by the TWG:

**Critical –** Topic 15, National Tsunami Hazard Mapping Program (NTHMP); Topic 16, 100-year recurrence.

**Important –** Topic 20, Structure-Debris Interaction; Topic 29, Erosion.

Key issues are:

- NOAA tsunami inundation maps presently show the maximum credible tsunami inundation limits. Since a return period was not assigned to NOAA maps, the actuarial needs of NFIP are not served by NOAA maps. Another drawback of the NOAA maps in California is that only nearfield events are considered and farfield events are not. However, NOAA maps can be a part of FEMA's multi-hazard mapping efforts.
- NOAA maps are useful, but FIS studies require consideration of 1% annual chance flood.
- Present NOAA procedures do not account for farfield events; only nearfield events are considered.
- The NTHMP has identified sources of Tsunami risks for Southern and Central California (local and distant earthquakes, and coseismic or aseismic subaerial and subaqueous slides), Northern California to Northern Washington (Cascadia Subduction Zone Earthquakes, coseismic or aseismic subaerial and subaqueous slides), Puget Sound (local earthquakes and, coseismic or aseismic subaerial and subaqueous slides and from delta failures). The issue is to determine which of these sources will contribute significantly to the 1% annual chance base flood elevation required for Flood Insurance Maps. Some of these sources may produce infrequent tsunamis with small runup elevations and may not be considered for the NFIP.
- Past FEMA Tsunami Mapping methods were developed by Houston and Garcia (1978). The limitations of their methods are: 1) only farfield events from Alaska and South America are considered and potential rupture of Cascadia Subduction Zone had not been recognized at that time; 2) the computational boundary is a vertical wall at the shoreline; and 3) faults are modeled as a simple, rapid uplift of the ocean floor. Improved methods have been developed since the 1970s and 1980s when the Houston and Garcia procedures were applied first along the Pacific Coast. FEMA needs reliable methods that will utilize state-of-the-art long wave propagation models and geophysics based probabilistic procedures to define the magnitude and probability of the forcing function for such rare events.
- FEMA needs a method that recognizes hazards from multiple tsunami sources, utilizes the knowledge available within the tsunami community in terms of source identification; geophysics based probabilistic assessments, and propagation modeling. Tsunami anomalies in tide records, where available, may be used in modeling and verification of results.

- High velocities are associated with tsunamis. Current mapping practices call for the statistical combination of tsunami runup frequency curves and storm wave runup frequency curves. A new methodology is needed to depict the hazards associated with high velocity tsunami waves propagating landward from the coastline.

- Methods for calculating debris impact loads on structures are needed. Such methods may lead to development of *G&S* for assessing the performance and survivability of coastal structures during a 1% annual chance event tsunami.

- Little is known about the physics of tsunami induced erosion. Post-tsunami observations show that tsunami induced erosion damages can be severe. Therefore, procedures for estimating likely changes in beach and back beach profiles are needed in order to determine tsunami runup elevations.

### *Recommended Approach*

It is recommended that a Probabilistic Tsunami Hazard Assessment (PTHA) methodology be developed for NFIP purposes. The procedure will be based on an integrated, interdisciplinary, and highly focused six-month pilot study to define the tsunami hazards in a specific locale in Washington, Oregon, or California by carefully examining the NTHMP and NFIP methods and tools. The pilot study will combine recommendations from both Critical Topics 15 and 16. Topics 20 and 29 require longer-term fundamental research and are recommended for future consideration.

Recommended Approach (Critical Topics)

The recommended work will focus on Topics 15 and 16:

- Develop geologic and geophysical digital database.

- Develop a methodology suitable for NFIP tsunami hazard zone delineations, including recurrence interval estimation. The methods are likely to use existing NTHMP products and procedures.

- Conduct a six-month pilot study to develop procedures for defining tsunami hazards along the Washington, Oregon, or California coast

Recommended Approach (Important Topics)

- Estimate impact forces on typical coastal structures using overland flow depths and velocities from the numerical tsunami simulations performed above for one coastal location.

- Examine available USGS post-tsunami erosion data. Attempt to develop a simplified empirical relationship for approximating changes in beach profiles during a 1% annual chance tsunami for the specific locale under study.

Unlike the other ten work categories detailed in the Phase 1 Report, some of the tsunami research and development tasks recommended here are being considered for completion under an interagency agreement between FEMA and NOAA. This applies primarily to Topics 15 and 16. Therefore, the majority of recommended tasks associated with Topics 15 and 16 are shown below as future tasks along with Topics 20 and 29, below.

**Table 11**
**Tsunami Recommendations – Pacific Coast**

| Topic Number | Topic/Subtopic | Timing | Recommended Approach |
|---|---|---|---|
| No Topic No assigned | Prepare General Procedures for Pacific Coast *G&S* | Phase 2 | Prepare guidance for use of information and hazard mapping work products produced by NOAA under Topic numbers 15 and 16, below. Include these procedures in the general *G&S* for the Pacific Coast. |
| 15 | Address Use of NTHMP Program Products and Approaches | Future | Develop digital database.<br>Develop method suitable for NFIP tsunami hazard zone delineations, including recurrence interval estimation. |
| 16 | Develop Method to Predict 100-year Tsunami Event | Future | Perform comprehensive pilot study at a selected site in California or Oregon or Washington to develop and test numerical methods for:<br>1) Improve recurrence interval estimating procedures for farfield and nearfield sources by increasing the coverage and quality of the historic and prehistoric tsunami records and develop probability distributions for both tsunamigenic earthquake and landslide sources.<br>2) Estimate the 1 percent chance tsunami<br>3) Test procedures for propagating tsunamis from Alaska, Chile, and Cascadia Subduction Zone to the Pacific Coast. Verify model predictions with tidal records, if available<br>4) Calculate runup and inundation elevations<br>5) Calculate combined probability distribution of tsunami runup and storm wave generated runup (if data are available). |
| 20 | Tsunami-Structure–Debris Interaction To Define Hazard Zones | Future | Estimate impact forces on typical coastal structures using overland flow depths and velocities from the numerical tsunami simulations performed above for one coastal location. |
| 29 | Review Methods of Tsunami Induced Erosion | Future | Examine available USGS post-tsunami erosion data. Attempt to develop a simplified empirical relationship for approximating changes in beach profiles during a 1% annual chance tsunami for the specific locale under study. |

## SHELTERED WATERS

### Topics and Key Issues

The following Sheltered Waters topics were identified by the TWG:

**Critical –** Topic 6a, Definitions and Classification; Topic 6b, Historical Information; Topic 6d, 1% Annual Chance Flood Event; Topic 6e, Stillwater Elevations and Tidal Currents, Topic 6f, Coastal Structures (covered in 21a); Topic 6g, Hazard Zones (covered in 17); Topic 6h, Inter-Relationships.

Key issues are:

- Sheltered Waters (SW) are water bodies with shorelines that are not subjected to the direct action of undiminished ocean waves. Although similar processes contribute to flooding in sheltered water shorelines as along open coastlines, such as wave setup, runup, and overtopping, there are several aspects of sheltered water flood hazards not addressed in current *G&S*.
- Wave generation and transformation in SWs are typically limited by an open water fetch distance, complex bathymetry, and often by the presence of structures. A sheltering effect typically reduces wave energy and flood potential compared to open coast areas. However, wave runup and overtopping along SW shorelines may present additional hazards from wave-cast debris and backshore flooding.
- Wave-cast debris from extreme wave runup and overtopping can be especially problematic, owing to the proximity to fluvial sources of such materials in many estuaries.
- SW areas often have unique flood hazards, due to the effects of fluvial drainages and modified tidal and surge hydrology, and relatively strong tidal currents.
- Other unique flood-related characteristics include the complex geometry of embayments, non-coincidence of peak storm surge with peak winds, shallow water and restricted wind fetches for wave growth, and non-sandy shoreline types with special erosion and scour hazards.
- New guidelines are needed to inform and guide Mapping Partners in the preparation of coastal flood insurance studies and flood hazard maps in sheltered water areas of the coastal floodplain.

### Recommended Approach

Sheltered waters topics were classified by the project team as *Critical* to the Pacific studies and applicable to all coasts. The recommended approach involves revisions to the *G&S* that will: 1) better define, provide examples, and classify SWs and associated physical processes that contribute to flooding; 2) expand existing guidance for SW areas using available references and information; 3) discuss river-tidal joint probability issues, 4) develop linkages between SW and other sections of the *G&S* and, 5) seek FEMA approval for methods used by Mapping Partners in recent Pacific Ocean sheltered water flood studies.

Recommended Approach (Critical Topics)

- Provide definitions, examples, and develop a classification method and general approach conducting SW studies versus open coast studies. This will serve as a framework and approach for Mapping Partners to follow when conducting coastal flood hazard assessments.
- Prepare general guidance for documenting and using high water marks to reconstruct historic flood conditions to validate flood study results.
- Prepare guidance specific to defining the 1% annual chance flood event, including consideration of the combined effects of riverine and tidal flooding.
- Expand guidance on wind data acquisition and analysis and on fetch-limited wave forecasting in SWs.
- Prepare guidance for estimating stillwater elevations in ungauged SWs bodies and evaluating the effects of tidal and riverine currents on wave propagation in SWs.
- Prepare guidelines that comply with other related FEMA Map Modernization objectives and multi-hazard planning initiatives.

Tasks associated with Topics defined by the TWG to be *Critical* were considered for completion in Phase 2. However, time and budget constraints in Phase 2 do not allow comprehensive treatment of all the *Critical* topics. The table below summarizes the tasks selected for completion in Phase 2, and those deferred for future consideration by FEMA.

In addition to the specific tasks listed in the table, the Sheltered Waters Phase 2 effort will involve collaboration and coordination with other study groups as indicated below:

- Work with the Storm Meteorology group to develop guidance for combined probability considerations for defining the 1% annual chance flood event in sheltered water areas (Topic 51).
- Work with the Stillwater group to develop general guidance for storm surge evaluation in sheltered waters using tide gage analysis and 1-D surge model (Topic 54 and 55).
- Work with the Wave Characteristics group to develop guidance on application of CEM and SPM methods, and to evaluate application of Spectral Energy Models and Empirical Prediction Methods in sheltered waters (Topics 4 and 5).
- Work with the Wave Transformation group to develop guidance on wave transformation (Topic 8), wave propagation over dissipative bottoms (Topic 9) and overland wave propagation (Topic 10) in SWs.
- Work with the Wave Setup group to develop guidance for defining wave setup in sheltered water settings (Topics 44, 45, 46).
- Work with the Event-Based Erosion group to develop guidance for erosion assessments in cobble/shingle materials (Topic 33) and general guidance for erosion assessments in sheltered water areas (Topic 35).

- Work with the Runup and Overtopping group to develop guidance for using mean versus higher runup heights (Topic 12) and estimating overtopping volumes for backshore hazard mapping along sheltered waters (Topic 13).

- Work with the Hazard Zones group to develop guidance for considering wave-cast debris (Topic 17) and mapping flood hazards from combined coastal-riverine flood areas (Topic 19).

**Table 12**
**Sheltered Waters Recommendations – Pacific Coast**

| Topic Number | Topic/Subtopic | Timing | Recommended Approach |
|---|---|---|---|
| 6a | Definitions and Classification | Phase 2 | Provide definitions, examples, and develop a classification method based on SW physical processes and site characteristics that can be used during SW flood hazard studies. |
| 6b | Flood Event Reconstruction | Phase 2 | Review previous SW flood studies and document methods used for validating flood study results. Prepare general guidance for documenting and using high water marks to reconstruct historic flood conditions. |
| 6d | Combined Tidal-Riverine 1% Annual Chance Event Assessment | Phase 2 | Prepare guidance for defining the 1% annual chance flood event involving riverine and tidal flooding and expand guidance on wind data acquisition and analysis and fetch-limited wave forecasting. |
| 6e | Stillwater Estimation | Phase 2 | Prepare guidance for estimating stillwater elevations in ungauged sheltered waters bodies and evaluating the effects of tidal and riverine currents. |
| 6h | Hazard Mitigation Coordination | Future | Prepare general guidance for Mapping Partners to coordinate the preparation of coastal studies with other hazard mitigation activities. |
| 6h | Focused Study Coordination | Phase 2 | Collaborate/coordinate with other study groups to address “Critical” sheltered waters topics found in other Focused Studies. |
| | PC Guidelines | Phase 2 | Prepare general *G&S* section for assessing sheltered water areas on the Pacific Coast. |

# HAZARD ZONES

## *Topics and Key Issues*

The following Hazard Zones topics were identified by the TWG:

**Critical –** Topic 17, VE Zone Limit.

**Available –** Topic 19, Combined Probabilities and Mapping for Areas Subject to Both Coastal and Riverine Flood Sources.

**Important –** Topic 18, VE/AE Zone Appropriateness; Topic 39, PFD Definition.

Key issues are:

- The existing definition of the primary frontal dune (PFD) is included in 44 CFR Section 59.1 of the NFIP regulations, and is based on "where there is a distinct change from a relatively steep slope to a relatively mild slope" in the land surface. The definition does not provide a quantitative method for defining the landward limit of the PFD, yet it has significant influence on hazard zone delineation The PFD definition and delineation also has implications for floodplain management, since dune areas within a VE Zone are protected under 44 CFR subsection 60.3(e)(7) of the NFIP regulations.

- Coastal high hazard zones are defined in 44 CFR Section 59.1 of the NFIP regulations to include the area up to the landward limit of the PFD along open coasts. In practice, this definition frequently dominates the determination of the VE Zone boundary. An improved definition or quantitative methodology is needed to improve consistency in hazard zone delineation. This issue is most applicable on the Atlantic and Gulf coasts where dunes are common, but also affects some areas of the Pacific Coast.

- The use of the PFD definition for VE Zone mapping may cause areas that are subject to significantly different levels of flood risk to be mapped in a single VE Zone. The seaward portion may be subject to inundation by active coastal processes during the base flood (erosion, wave height, wave runup, and wave overtopping), and the landward portion included solely on the basis of the PFD limit defined by topography.

- Transitions in the Base Flood Elevations (BFEs) are frequently abrupt where the PFD definition is used to establish a VE Zone limit, and the AE zone behind the PFD has a much lower computed BFE. Improved procedures are needed to accurately relate mapped BFEs to flood risk.

- The VE Zone limits are based on a breaking wave height of 3 feet or more and runup depths of 3 feet or more. The basis for these criteria is not clear, and they may underestimate areas subject to significant damage by coastal processes.

- The wave overtopping criteria presently used in VE Zone hazard mapping require expansion and review to evaluate threshold rates, extent of the mapped zones, and potential for use of VO Zones to more accurately reflect actual hazards landward of overtopped dunes, coastal ridges, and shore protection structures.

- Mapping procedures do not presently consider wave-cast debris (logs, stones, etc.), but these hazards are significant on the Pacific Coast. New procedures may be needed to identify areas subject to significant damages.

- Coastal SFHAs on the Pacific Coast are generally narrow and dominated by wave runup. Therefore, the distinction between seaward portions of AE Zones (that can be subject to severe coastal hazards) and more landward portions (that are subject to lesser flood and erosion hazards) is not deemed to be as significant an issue as on the Atlantic and Gulf Coasts. However, a nationwide review is needed to assess the feasibility of subdivision of the coastal AE Zone SFHA.

- A methodology is needed for determining and mapping flood hazard areas where coastal flooding intersects and combines with a riverine flood profile. Previous FEMA guidance should be reviewed for this purpose.

### *Recommended Approach*

Hazard zone topics were classified by the Technical Working Group as *Critical*, *Important* and *Available*, and applicable to all coasts. The recommended approach to preparing *G&S* for the Pacific Coast has the purpose of clarifying existing guidance on coastal high hazard zones, describing FIRM hazard zone delineation using results from coastal analyses, expanding upon examples to include Pacific Coast typical conditions, and revising guidance using available references and information.

Recommended Approach (Critical and Available Topics)

- Establish improved procedures for establishing the landward limit of the PFD, and develop guidance to better map the BFE transition between PFD dominated VE Zones and landward SFHA hazard zones.

- Establish procedures (hazard identification and mapping) to better utilize VO Zones for areas subject to severe wave overtopping at dune ridges and coastal protection structures.

- Establish procedures for identifying and mapping coastal high hazard zones for wave overtopping and wave-cast debris hazards in SFHAs with historically significant damages from this unique hazard.

- Review the previous 1981 FEMA guidance and new guidance on how to conduct the assessment and mapping of combined coastal-riverine areas for adoption into the *G&S*.

Recommended Approach (Important Topics)

- Investigate and develop coastal A Zone criteria

- Prepare technical bulletins for clarification of proposed revisions to VE Zones, AE Zones, and new criteria for VO Zones related to hazard identification, mapping, and floodplain management.

- Develop new *G&S* examples of wave transect hazard mapping specifically for the expected conditions along the Pacific Coast and sheltered waters.

Tasks associated with Topics defined by the TWG to be *Critical* and *Available* were considered for completion in Phase 2. However, time and budget constraints in Phase 2 do not allow comprehensive treatment of all the *Critical* topics. *Important* topics cannot be completed within the time frame of the project (although a limited number of mapping examples can be developed during Phase 2). The table below summarizes the tasks selected for completion in Phase 2, and those deferred for future consideration by FEMA.

**Table 13**
**Hazard Zones Recommendations – Pacific Coast**

| Topic Number | Topic/Subtopic | Timing | Recommended Approach |
|---|---|---|---|
| 17 | Primary Frontal Dune VE Zone | Phase 2 | Develop guidance to better map the BFE transition between PFD dominated VE Zones and landward SFHA hazard zones |
| 17 | Guidance on VO Zone Mapping | Phase 2 | Establish procedures (hazard identification and mapping) to better utilize VO Zones for areas outside established VE Zones. |
| 17 | VE Zone Mapping Options & Criteria | Phase 2 | Establish procedures for identifying and mapping wave overtopping and wave-cast debris hazard zones based on historical significance of hazard. |
| 17, 39 | VE Zone Limit and PFD Definition | Future | Establish improved procedures for establishing the landward limit of the PFD; test procedures in a case study |
| 19 | Combined Coastal-Riverine Zones | Phase 2 | Review the previous 1981 FEMA or revised/new guidance on how to conduct the assessment and mapping of combined coastal-riverine areas for adoption into *G&S*. |
| Topic number not assigned | Hazard Zone Mapping Examples | Phase 2 and Future | Develop new hazard zone mapping examples in *G&S* specifically for the Pacific Coast. |
| 18 | Hazard Zones and Technical Bulletins | Future | Investigate and develop coastal A Zone criteria. Prepare technical bulletins for clarification of proposed revisions to VE Zones, AE Zones, and new VO Zones related to hazard identification and floodplain management. Develop an annotated bibliography of related research and apply new concepts in a case study. |

## 4.6 SUMMARY OF TOPICS AND RECOMMENDATIONS – PACIFIC COAST

For easy reference, all of the Pacific Coast categories have been combined in one table, as follows.

**Table 14**
**Summary of Pacific Coast Recommendations**

| Topic Number | Topic/Subtopic | Timing | Recommended Approach |
|---|---|---|---|
| **STORM METEOROLOGY RECOMMENDATIONS – PACIFIC COAST** | | | |
| 51 | General Methods to Determine 1% Annual Chance Coastal Levels | Phase 2 | Define Event Selection and Response-Based methods for both open coast and sheltered waters |
| 51 | Define Specific Methods, Tools, and Data Guidelines for 1% Annual Chance Analysis | Phase 2 | Document specific methods including, for example, the PWA Sandy Point approach, the HR Wallingford JOIN-SEA method, and the FEMA/Tetra Tech 1982 approach. |
| 51 | Open Coast Case Study | Phase 2 | Perform a case study comparing selected methods at a specific open coast site, preferably one for which prior data is available |
| | | Future | Perform a case study with Monte Carlo Method (Wallingford) using multiple variables. The study will take into account wave related variables of swell (height, period and direction) and sea (height) as well as the still water elevation for the open coast. |
| 51 | Sheltered Water Case Study | Phase 2 | Perform a case study comparing methods at a specific sheltered water site, preferably one for which prior data is available. Monte Carlo Methods will be applied for Sheltered Water. |
| 51 | Storm Surge Modeling Frequency Analysis | Future | Test and recommend methods to associate frequency with storm surge for Pacific Coast surge modeling; recommend appropriate data sources |
| 51 | Surge/Riverine Combination | Future | Prepare recommendations for the statistical combination of surge and a riverine runoff profile, with consideration of non-independence of the processes; See also Topic 19 of the Hazard Mapping Focused Study for simple mapping suggestions |
| 51 | Tsunamis and Tide | Future | Develop guidelines for the combination of tsunamis and tide, including a worked hypothetical example |
| **STILLWATER RECOMMENDATIONS – PACIFIC COAST** | | | |
| 55 | Tide Gage Analysis | Phase 2 | Select and test methods to extract surge estimates from tide gage data in multiple settings. |
| 54 | Tide Gage Analysis Guidelines | Phase 2 | Document procedures for tide gage frequency analysis. |
| 54 | General Considerations for Surge Modeling | Phase 2 | Based on the existing literature, describe the use of surge models and the factors which require consideration in performing a study. |

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| 54 | Simplified Storm Surge Model | Phase 2 | Develop a 1-D (bathystrophic) surge model based on the Florida Department of Environmental Protection methodology. Although primarily for Pacific Coast applications, the model may also be useful as an auxiliary tool for the Atlantic and Gulf coasts. |
| | | Future | Perform testing and example studies of the 1-D surge model and provide expanded Users Manual based on test results. |
| 52 | Non-Stationary Processes | Phase 2 | Write general guidelines for the consideration of non-stationary processes (for example, relative sea level rise, land subsidence), including identification of major data sources. Include guidance on interpretation of historical data. Suggest documentation of projected map impact. |
| **STORM WAVE CHARACTERISTICS RECOMMENDATIONS – PACIFIC COAST** | | | |
| 4, 5 | Sea and Swell for Pacific Coast | Phase 2 | Review GROW dataset for one location. Check whether the dataset represents extreme events adequately. Confirm lack of bias in the database. Develop *G&S* on use of GROW and steps for developing input data to wave transformation models. Describe the WIS database development and potential use in coastal flood insurance studies. |
| 4, 5 | Nearshore Representation of Local Sea for Southern California Bight | Future | Conduct a study of the available nearshore data for Southern California Bight to assess whether inclusion of the local wind makes a significant change in the high frequency part of the spectrum. Based on the results of the above study, adopt one of the three alternatives: a) assuming no change in wind-induced change in the spectrum, or b) attempt to model wind-induced changes, or c) treat changes to the wind wave portion of the spectrum as an independent variable and use joint probability analysis techniques |
| 4, 5 | Wave Generation in Sheltered Waters | Phase 2 | Compare CEM and SPM procedures using a case study (an existing FIS site) and clarify application of CEM in FEMA studies. Perform a case study to compare SEMs and traditional parametric models using restricted fetch methods. |
| 4, 5 | Wave Generation in Sheltered Waters | Future | Develop application procedure for SEMs including wind field definition based on detailed testing. |
| 1 | Wave Definitions | Phase 2 | Using the compiled glossary of terms and notations (from CHL and IAHR sources), correlate each of key terms with the coastal methodologies and application. Prepare for application for Pacific Coast Guidelines |
| **WAVE TRANSFORMATION RECOMMENDATIONS – PACIFIC COAST** | | | |
| 8 | Wave Transformation with and without Regional Models | Phase 2 | Write *G&S* for Wave Transformations. Tasks: 1) conduct several Focused Studies to inform the Wave Transformations *G&S*; 2) use available publications to identify a range of methods; 3) develop criteria for level of analysis; 4) include development of guidelines for spatial coverage and wave parameters, and include use of regional models such as CDIP; 5) research available literature to adequately define wave groups, infragravity waves, shallow water spectra, etc. for input into wave setup and runup calculations; 6) review available literature and guidance on the range of applicability of contemporary computer models, recommend models for inclusion on the FEMA pre-approved coastal model list, and provide guidance on their application to FEMA FISs; 7) incorporate applicable sections of existing *G&S* for other geographical areas that cover the overland propagation and wave energy dissipation topics. (Topics 9 &10) |
| | | Future | Evaluate wave transformation models using a selected data set. |

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| 7 | California Regional Wave Transformation Models | Phase 2 | Provide CDIP Southern California validation examples and a test case for testing other WT models;<br>Provide guidance and Users Manual on use of CDIP models and model output such as existing model coefficients. |
| | | Future | Use CDIP model to create 2 sets of wave transformation coefficients for southern California, 1) for swell waves and 2) for local wind waves; Expand CDIP for the California Coast. Validate the models for central and northern California; Create database, provide expanded user's manual, and develop Fortran and MATLAB codes to assist contractors in using the CDIP model coefficients.<br>Consider expanding regional wave modeling for Washington and Oregon coasts using CDIP or other programs (e.g., WIS) at the appropriate time and depending on the need, recognizing that regional wave models are more logical in densely populated areas. Individual studies may be performed in sparsely located communities (see Topic 8).<br>Evaluate any limitations due to the linearity of the transformation models.<br>Conduct research on wind wave and swell spectra combination. |
| 9 | Wave Energy Dissipation over Shallow Flat Bottoms | Phase 2 | Evaluate wave dissipation over marsh and mudflats in the Pacific Coast from available information; Develop criteria to evaluate importance of wave dissipation in FISs; Recommend changes to methods and WHAFIS dissipation criteria to the extent feasible. |
| | | Future | Conduct field data collection to characterize wave dissipation over marsh and mudflats and other shallow, dissipative shores in the Pacific;<br>provide expanded guidance for calculating wave dissipation. |
| 10 | Overland Wave Propagation | Future | Evaluate if changes to WHAFIS dissipation criteria are necessary (see Topic 9), and *G&S* modifications for Pacific Coast. |
| **WAVE SETUP RECOMMENDATIONS – PACIFIC COAST** | | | |
| 44, 45 | Pacific Coast Definitions | Phase 2 | Develop wave setup definitions with emphasis on Pacific Coast applications. |
| 46 | Evaluate Boussinesq Models | Phase 2 | Intercompare at least three Boussinesq models and compare with data. |
| 46 | Develop Engineering Based Approach | Phase 2 | Couple accepted engineering models for calculating wave setup across surf zone. Include procedure for dynamic wave setup. |
| 44, 45 | Compile Data for Testing | Phase 2 | Locate as much quality field data as possible for testing of developed/selected approach(es). |
| 44, 45 | Compile Data for Testing | Future | Locate and compile comprehensive national and international data sources for testing a new Pacific Coast setup model |
| 46 | Develop Breaking Zone Model | Phase 2 | Evaluate candidate breaking zone models that allow specification of non planar profile. |
| 46 | Develop Draft Guidelines and Specifications | Phase 2 | Incorporate findings from above into draft Guidelines and Specifications. |
| 46 | Develop Interim Method | Future | Test Model over a wide range of settings and develop and expand User's Manual based on test results. |

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| 47 | Ideal Model for Static Wave Setup | Future | Couple wave generation and wave setup model, allowing specification of arbitrary tide. |
| 48 | Develop Model for Dynamic Wave Setup | Future | Develop method based on directional and nonlinear spectrum as input. |
| **WAVE RUNUP AND OVERTOPPING RECOMMENDATIONS – PACIFIC COAST** | | | |
| Topic number not assigned | Runup on Beaches and Low Barriers | Phase 2 | Revise guidance to call for runup analyses for sandy beach, small dune shore type |
| 12 | Evaluate Use of Mean Runup Value | Phase 2 | Evaluate use of $R_{50\%}$ and select alternate $R_{x\%}$ value (probably between $R_{33\%}$ and $R_{10\%}$) if $R_{50\%}$ understates observed hazard.<br>Develop an Interim procedure to adjust RUNUP2.0.output. |
| 12 | Evaluate Use of Mean Runup Value | Future | Review runup distributions for beaches and structures during El Niño, coastal storm, and hurricane conditions; review runup damages. |
| 11 | Wave Setup Component | Phase 2 | Current FEMA methodology includes the wave setup component in the calculated runup height. This procedure should be revisited for its appropriateness along the Pacific, and depending on recommended Pacific methodology (coordinate with Wave Setup study) |
| 11 | Infragravity Motions | Future | Consider effects of infragravity motions, which amplify runup and overtopping, and can be substantial along the Pacific Coast |
| 11 | Wave Setup Component | Phase 2 | Current FEMA methodology includes the wave setup component in the calculated runup height. This procedure should be revisited for its appropriateness along the Pacific, and depending on recommended Pacific methodology (coordinate with Wave Setup study) |
| 11 | Conduct Comparative and Sensitivity Testing of Runup Models and Methods | Phase 2 | Evaluate CDIP-type and Oregon-type methods as interim approaches.<br>Coordinate with case studies in Storm Meteorology, Wave Transformation studies.<br>Test runup methods and models in conjunction with other tests (use common data sets to test wave generation through stillwater level and runup). |
| 11, 49 | Conduct Comparative and Sensitivity Testing of Runup Models and Methods | Future | Identify appropriate runup methods and models by location, morphology and hydraulic conditions.<br>Compare results using simple methods versus numerical models, deterministic (event selection) versus statistical approaches.<br>Write Guidelines on input conditions uncertainty. |
| 13, 14 | Overtopping Rates | Phase 2 | Maintain use of mean overtopping rate (cfs/ft, $m^3$/ per m)<br>Determine damaging overtopping rates for buildings and evaluate current FEMA hazard zone thresholds.<br>Evaluate FEMA's guidance which limits the runup elevation to 3 feet above a barrier's crest elevation<br>Coordinate with Hazard Zone study. |
| 13 | Overtopping Rates | Future | Overtopping at low profile beaches and barriers, dune remnants, revetments, and vertical walls should be evaluated, including consideration for calculating overtopping and ponding on low bluffs with gently sloping, flat or adverse slopes. |
| **EVENT BASED EROSION RECOMMENDATIONS – PACIFIC COAST** | | | |
| 30 | Geometric | Phase 2 | Evaluate geometric methods and models. Develop *G&S* for determining most |

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| | Methods for Assessing Erosion | | likely Pacific winter beach profile, including beach nourishment areas. Evaluate geometric modeling procedures for sand beaches and dunes on PC and test with available data sets. At a minimum, prepare interim *G&S* methods based on historical beach profiles and field observations. |
| 31, 32 | Bluff and Cliff Erosion | Phase 2 | Review available literature and reporting; provide language and descriptions to PC *G&S* to distinguish bluff and cliff erosion from other processes; provide figures and examples. Review existing bluff erosion procedures and international literature. Discuss interim approach for estimating bluff and cliff erosion based on historical profile data. |
| | | Future | Develop geometric procedures for bluff and cliff erosion and retreat. Consider development and use of process-based numerical/statistical modeling methods for future inclusion in the NFIP program. |
| 33, 34 | Gravel, Cobble, and Shingle Beach and Dune Erosion | Phase 2 | Provide discussion of gravel, cobble and shingle beach and dune erosion in different settings to distinguish this type of erosion hazard from other erosion processes. Provide examples, figures, and definitions. Discuss a simplified interim approach for cobble/shingle beaches based on historical beach profiles. |
| | | Future | Explain limitations of existing 540 Criterion for application to this type of erosion and setting. Discuss simplified *interim approach* for assessing gravel, cobble, and shingle beach and dune erosion based on historical beach profile data. Develop geometric procedures for gravel, cobble, and shingle beach erosion.<br>Consider development and use of process-based numerical/statistical modeling methods for future inclusion in the NFIP program. |
| 35,36 | *G&S* in Sheltered Water areas | Phase 2 | Provide definitions and discussion of EBE found in sheltered water areas for *G&S;* provide interim *G&S* based on historical beach profiles and field observations |
| | | Future | Perform future pilot EBE study(s) in sheltered-waters; refine interim assessment procedures; consider use of process-based models; prepare updated *G&S* |
| 38 | Physics/Process Based Methods | Phase 2 | Discuss difference between simplified geometric methods and Processed Based models. |
| | | Future | Develop suite of processed-based models for general coastal erosion assessments for different settings and material types, including sheltered waters and overwash |
| 40 | Document vertical depths of erosion | Future | Document depths of erosion following storm events and maintain data for depths of erosion and damages to buildings in order to better determine “depth-damage” relationships. |
| 41 | Long-term Erosion | Future | This topic is considered important to NFIP, but FEMA action on previous work is pending. Therefore, guidance is best developed by FEMA in the future. |
| 42, 43 | Nourished Beaches | Phase 2 | Provide language in *G&S* directing study contractors to notify FEMA if their study area includes a beach nourishment project and provide FEMA with a list of information needed to assess special cases where beach nourishment may be considered in determining hazard zones and BFEs (exception to existing FEMA policy). |
| **COASTAL STRUCTURES RECOMMENDATIONS – PACIFIC COAST** | | | |
| 21a, 21b.1, 23 | Failed and Buried Structures | Phase 2 | Revise guidance to better describe buried structures and failed structure configurations (including progressive failure of revetments). |
| 22a, 22b | Wave Effects Analyses at | Phase 2 | Using modified PWA method, write guidance for mapping runup and overtopping at uncertified (or failed) coastal structures. |

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| | Failed Structures | | |
| 25 | Flood protection Structures | Phase 2 | Mention in guidance, detailed TR 89-15 evaluation/certification of coastal structures are not required during FIS, but discuss implications |
| 26a, 26b, 26d | Effects of Structures on Erosion, Flood Hazards | Phase 2 | Investigate effects of structures on erosion and flood hazards; develop guidance for incorporation into flood hazard mapping. |
| 27a | Coastal Levees and Structures | Phase 2 | Identify and resolve inconsistencies in treatment of coastal levees and coastal structures |
| 24 | Tsunami-prone Structures | Future | Investigate historical data on structure failure/success during tsunamis; develop evaluation criteria for tsunami-prone structures. |
| 27b, 27c | Structure Evaluation Criteria | Future | Review CERC TR 89-15 considering more recent data on structure stability and failure; revise structure evaluation criteria for existing and new structures. |
| 21b.2 | Jetties, Groins, Breakwaters | Future | Develop criteria/guidance for evaluating failure of other structure types, and the effects of these failures on mapped flood hazards |
| 26e | Minimum Structure Dimensions | Future | Determine minimum structure dimensions necessary to receive mapping credit during FIS and revisions to FIRMs |
| **TSUNAMI RECOMMENDATIONS – PACIFIC COAST** | | | |
| No Topic No assigned | Prepare General Procedures for Pacific Coast *G&S* | Phase 2 | Prepare guidance for use of information and hazard mapping work products produced by NOAA under Topic numbers 15 and 16, below. Include these procedures in the general *G&S* for the Pacific Coast. |
| 15 | Address Use of NTHMP Program Products and Approaches | Future | Develop digital database.<br>Develop method suitable for NFIP tsunami hazard zone delineations, including recurrence interval estimation. |
| 16 | Develop Method to Predict 100-Year Tsunami Event | Future | Perform comprehensive pilot study at a selected site in California, Oregon, or Washington to develop and test numerical methods for:<br>1) Improve recurrence interval estimating procedures for farfield and nearfield sources by increasing the coverage and quality of the historic and prehistoric tsunami records and develop probability distributions for both tsunamigenic earthquake and landslide sources.<br>2) Estimate the 1% annual chance tsunami<br>3) Test procedures for propagating tsunamis from Alaska, Chile, and Cascadia Subduction Zone to the Pacific Coast. Verify model predictions with tidal records, if available<br>4) Calculate runup and inundation elevations<br>5) Calculate combined probability distribution of tsunami runup and storm wave generated runup (if data are available). |
| 20 | Tsunami-Structure–Debris Interaction To Define Hazard Zones | Future | Estimate impact forces on typical coastal structures using overland flow depths and velocities from the numerical tsunami simulations performed above for one coastal location. |
| 29 | Review | Future | Examine available USGS post-tsunami erosion data. Attempt to develop a |

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| | Methods of Tsunami Induced Erosion | | simplified empirical relationship for approximating changes in beach profiles during a 1% annual chance tsunami for the specific locale under study. |
| **SHELTERED WATERS RECOMMENDATIONS – PACIFIC COAST** | | | |
| 6a | Definitions and Classification | Phase 2 | Provide definitions, examples, and develop a classification method based on SW physical processes and site characteristics that can be used during SW flood hazard studies. |
| 6b | Flood Event Reconstruction | Phase 2 | Review previous SW flood studies and document methods used for validating flood study results. Prepare general guidance for documenting and using high water marks to reconstruct historic flood conditions. |
| 6d | Combined Tidal-Riverine 1% Annual Chance Event Assessment | Phase 2 | Prepare guidance for defining the 1% annual chance flood event involving riverine and tidal flooding and expand guidance on wind data acquisition and analysis and fetch-limited wave forecasting. |
| 6e | Stillwater Estimation | Phase 2 | Prepare guidance for estimating stillwater elevations in ungauged sheltered waters bodies and evaluating the effects of tidal and riverine currents. |
| 6h | Hazard Mitigation Coordination | Future | Prepare general guidance for Mapping Partners to coordinate the preparation of coastal studies with other hazard mitigation activities. |
| 6h | Focused Study Coordination | Phase 2 | Collaborate/coordinate with other study groups to address "Critical" sheltered waters topics found in other Focused Studies. |
| | PC Guidelines | Phase 2 | Prepare general *G&S* section for assessing sheltered water areas on the Pacific Coast. |
| **HAZARD ZONES RECOMMENDATIONS – PACIFIC COAST** | | | |
| 17 | Primary Frontal Dune VE Zone | Phase 2 | Develop guidance to better map the BFE transition between PFD dominated VE Zones and landward SFHA hazard zones |
| 17 | Guidance on VO Zone Mapping | Phase 2 | Establish procedures (hazard identification and mapping) to better utilize VO Zones for areas outside established VE Zones. |
| 17 | VE Zone Mapping Options and Criteria | Phase 2 | Establish procedures for identifying and mapping wave overtopping and wave-cast debris hazard zones based on historical significance of hazard. |
| 17, 39 | VE Zone Limit and PFD Definition | Future | Establish improved procedures for establishing the landward limit of the PFD; test procedures in a case study |
| 19 | Combined Coastal-Riverine Zones | Phase 2 | Review the previous 1981 FEMA or revised/new guidance on how to conduct the assessment and mapping of combined coastal-riverine areas for adoption into *G&S*. |
| Topic number not assigned | Hazard Zone Mapping Examples | Phase 2 and Future | Develop new hazard zone mapping examples in *G&S* specifically for the Pacific Coast. |
| 18 | Hazard Zones and Technical Bulletins | Future | Investigate and develop coastal A Zone criteria. Prepare technical bulletins for clarification of proposed revisions to VE Zones, AE Zones, and new VO Zones related to hazard identification and floodplain management. Develop an annotated bibliography of related research and apply new concepts in a case study. |

# 5 RECOMMENDATIONS – ATLANTIC AND GULF COASTS

## 5.1 INTRODUCTION – OBJECTIVES AND NFIP CONSIDERATIONS

This section of the report presents a brief discussion on the need for guidelines to address both open coast and sheltered waters settings. Specific recommendations for the Atlantic and Gulf Coasts are summarized by technical category. These summaries are very brief descriptions of the results of the Focused Studies. The reader should refer to the appendices for a more thorough treatment of the topics for the Atlantic and Gulf Coasts.

The objectives for these recommendations are to guide future development of updates to the guidelines on the Atlantic and Gulf Coasts, indicate the potential applicability of Phase 2 work on the Pacific Coast to procedures for the Atlantic and Gulf Coasts, and provide a reference for the NFIP and map modernization until the existing guidelines, procedures, and regional studies are formally updated.

## 5.2 OPEN COAST AND SHELTERED WATER SETTINGS

"Sheltered Waters" are water bodies with shorelines that are not subjected to the direct action of undiminished ocean winds and waves. Sheltered Water areas are exposed to similar flood-causing processes as those found along open coastlines, such as high winds, wave setup, runup and overtopping. Present FEMA *G&S* adequately cover many of the general coastal flood assessment procedures needed to complete flood hazard assessments in Sheltered Waters. However, some aspects of sheltered water flood hazards can not be addressed by the current FEMA Guidelines. For example, wind-generated waves are highly dependent on the shape and orientation of the surrounding terrain to prevailing wind directions. Wave generation and transformation in sheltered waters are usually limited by their open water fetch distance, complex bathymetry and often the presence of in-bay and shoreline coastal structures. These sheltering effects reduce wave energy and flood potential compared to open coast areas.

Other processes, including the effects of terrestrial runoff which modify local tidal and surge hydrology and relatively strong in-bay currents often combine to create tidal and hydrodynamic conditions only found in sheltered waters areas. Bays and estuaries often display significant spatial variability in tidal hydrology. For example, south San Francisco Bay often has a standing tide with nearly twice the tide range of central Bay and an elevated mean tide and high water elevation compared to the open coast. In contrast the north bay which extends into the Sacramento-San Joaquin Delta area displays a progressively muted tidal range and lower elevated mean tide resulting from combined effects of complex tidal hydraulics, residual currents, local winds and river runoff. Oceanic storm surge can be modified in estuaries and it isn't clear whether storm surge is uniformly additive to local tidal datums throughout an estuary, or whether storm surge is amplified or muted within an estuary, or within a given region within a large estuary. However, this depends on local conditions and must be evaluated with appropriate methods.

On the Atlantic coast similar questions arise during hurricane events versus local storm events regarding how storm and oceanic conditions may or may not affect sheltered water tidal elevations. Atlantic Coast sheltered waters (such as the sounds behind North Carolina's Outer Banks, Chesapeake Bay, Delaware Bay, and other smaller water bodies) may experience significant wind setup in these shallow areas followed by a sudden calming of the wind resulting in long wave seiching within the sound. Similar seiching effects are experienced in the Great Lakes. Other important flood-related characteristics include the complex geometry of the embayments, lack of coincident peak storm surge with peak winds and waves, shallow water and restricted wind fetches for wave growth, and non-sandy shoreline types with special erosion and scour hazards.  Wave-cast debris from extreme wave runup and overtopping can be especially problematic, owing to the proximity to sources of such materials in many estuaries. These sheltered water flood hazards are not adequately addressed in current FEMA Guidelines.

## 5.3 DEFINE THE 1% ANNUAL CHANCE FLOOD HAZARD (TWO APPROACHES)

The issues of computing the wave conditions and still water levels during a 1% annual chance event has been discussed in Section 4.3, Open Coast and Sheltered Water Settings. For the open coasts of the Atlantic and the Gulf , the *G&S* assumes that during a hurricane event the 1% annual chance wave (which becomes depth limited in shallow water) will occur simultaneously with 1% annual chance water level. In some sheltered waters along the Atlantic and Gulf Coasts, the 1% annual chance wave and 1% annual chance water level may not occur simultaneously, primarily due to hurricane track relative to the configuration of the sheltered water body. Because the hydrometeorological setting of the Atlantic and Gulf Sheltered Water is similar to the Pacific Coast in terms of statistical correlation between water levels and waves, two basic approaches for extreme event definition, the Event Selection and the Response method, described in Section 4.4 of this report will be applicable. The *G&S*  does not have specific guidance detailing the 1% annual chance event issues for Sheltered Waters. Hence, the *G&S* developed for the Pacific Coast will be useful for Atlantic and Gulf Sheltered Waters.

## 5.4 INTRODUCTION TO TECHNICAL CATEGORY SUMMARIES

The subsections that follow provide concise summaries of Focused Study results in the 11 technical categories for the Atlantic and Gulf Coasts.  The summaries include a summary of existing *G&S*, a brief description of the topics, and key issues and a set of recommendations for the Atlantic and Gulf Coasts. Phase 2 of this project does not include further work on development of guidelines for the Atlantic and Gulf Coasts.  The recommendations therefore include a discussion of available methods, the potential applicability of guidelines to be developed in Phase 2 for the Pacific Coast, and recommended future development.

The following summaries are the direct result of the appended Focused Studies, which include additional discussion, information, and references on the topics. These Focused Studies provide an additional reference for the NFIP and map modernization until the existing guidelines, procedures, and regional studies are formally updated.

## STORM METEOROLOGY

### *Overview of Existing Guidelines*

This category covers not only storm meteorology, but also a number of flood frequency issues. Among these are two general methods to determine the 1% annual chance level of some coastal process, characterized as the Event Selection method and the Response-Based method. These terms refer to the manner in which the 1% annual chance coastal flood level is determined. In the Event Selection method, a single 1% annual chance offshore storm or wave event, which is followed to shore and on to its runup level, is selected with the assumption that the runup level would approximate the true 1% annual chance runup. In the Response-Based method, all significant events are routed from offshore to their runup limits, and only then is the 1% annual chance level determined, based on the entire set of response calculations. The same general approaches apply to processes other than runup.

For the Atlantic and Gulf Coasts, the question of method is less important than on the Pacific Coast, because the primary wave effects are associated with limit height breakers during local, intense hurricanes; consequently, the existing guidelines are quite limited. The Study Contractor is instructed to adopt the "controlling" wave for level mapping. There is little specific guidance on the selection of wave parameters for wave setup and runup determinations. In many places, the guidelines refer to the need to choose a parameter - deepwater wave height, for example, which is somehow "associated with" another process such as the 1% annual chance stillwater level. It is generally not clear from the guidelines how this is to be done, and the matter is left to the study contractor's judgment with the injunction that the assumptions be documented. Section D.2.2.6, for example, refers to "the meteorology of storms expected to provide approximate realizations of the 1-percent-annual-chance-flood" and suggests that such storms would be useful in "assessing wave characteristics likely associated with" that flood. Subsequently, it is suggested that "the 1-percent-annual-chance flood is likely associated with central pressure deficits having exceedance probabilities between 5 and 10 percent" with the implication that wave height and period estimated from hurricane formulas using pressures in this range would be appropriate.

Another important storm meteorology issue is the manner in which frequency is attached to storm surge calculations. The accepted approaches are all Response-Based, with a large number of storms of varying characteristics being simulated and the 1% annual chance level determined from an analysis of the computed response. An example of an Event Selection method, not commonly used in recent years, is the simulation of one particular storm (a design storm) chosen somehow to approximate 1% conditions. The basic approach discussed in the guidelines is the Joint Probability Method, which considers the total rate of occurrence of storms defined by multiple parameters with individual probabilities. The Atlantic and Gulf Coast guidelines suggest the approach originally developed by NOAA, with the required hurricane data taken from NOAA publications such as NWS 38. The newer Empirical Simulation Technique (EST) has been applied in recent studies both for the USACE and for FEMA, but is not considered in the current guidelines.

There is little additional guidance on storm meteorology in the current guidelines. The Study Contractor is required to "Describe the method by which the tidal elevation data are convoluted with the surge data including tidal constants and tidal records" for the combination of astronomic tide and storm surge. There is no guidance for the combined probability of separate processes such as storm surge and rainfall runoff in a tidal river, and there are no guidelines specifically for the Pacific Coast.

### *Topics and Key Issues*

The following Storm Meterology topics were identified by the TWG:

**Critical –** Topic 51, Combined Probability.

**Important –** Topic 50, Modeling Procedures.

Key issues are:

- Storm surge frequency analysis can be performed using Joint Probability, Monte Carlo, or the newer EST methods. These alternatives should be compared and evaluated using a common data set and a single storm surge model.
- The adequacy of NWS 38 as a data source for new storm surge studies should be reviewed, both from the standpoint of additional years of data since its publication, and also for its use of a coast-referenced coordinate system.
- Although not as critical as on the Pacific Coast, it is important to establish what offshore wave conditions should be selected for determination of such flood-enhancing mechanisms as setup and runup.
- Astronomical tide often makes a significant contribution to the total stillwater level. The methods by which tide and surge can be combined depend on their relative magnitudes and the degree to which they may interact physically. Guidelines should be developed for techniques to perform this combination.
- The manner in which flood levels are determined in tidal zones that are subject to both riverine and coastal flooding has been neglected in the existing guidelines. Methods to determine the joint result range from simple addition of rates to complex hydrologic modeling. See also Topic 19 of the Hazard Mapping Focused Study.
- Improved observations during recent years indicate that past assumptions regarding hurricane wind fields may require improvement.
- Similarly, improved determinations of wind stress under extreme wind conditions suggest that improvement of wind stress formulations used in surge modeling may be warranted.

### *Recommended Approach*

The recommended approach to these issues includes both the development and verification of methods, and the preparation of new and revised guidelines.

Currently Available Methods, Information, and Guidelines

Currently available Atlantic/Gulf methods include the Joint Probability, Monte Carlo, and EST methods for storm surge statistics; numerous runup models; methods for tide and surge combination summarized in the FEMA Surge Model documentation; and the Monte Carlo method adopted by the Florida Department of Environmental Protection.

Applicability of Pacific Coast Guidelines

The topics treated under Storm Meteorology have a different emphasis on the Atlantic and Gulf Coasts than on the Pacific Coast. For the Atlantic and Gulf Coasts, the primary concern is with the storm data and frequency methods used in storm surge modeling. The primary problem for the Pacific Coast is determination of the 1% annual chance flood elevation (base flood elevation) resulting from the combination of waves with tide, surge, and setup. Guidelines will be developed for the Pacific open coast based on the Event Selection Method and Response-Based Method. These methods will also be utilized to develop guidelines for determination of base flood elevation in the sheltered waters of the Pacific Coast. Sheltered waters in both the Pacific and the Atlantic and Gulf Coasts are characterized by possible non-coincidence of extreme stillwater level and extreme wave conditions. Because of this similarity, the procedures for the Pacific Coast sheltered waters, or part thereof, may be applicable to the Atlantic and Gulf Coasts. The following tasks undertaken in Phase 2 will develop procedures that may be applicable on the Atlantic and Gulf Coasts:

- Perform a sheltered water case study utilizing the Event Selection and Response-Based Methods.
- Provide guidance regarding the combination of surge and tide using convolution and FL-DEP methods. The convolution method will be applicable where surge and tide combine approximately linearly, or where one of the two processes dominates the other. The FL-DEP method does not require the assumption of linear combination and will likely apply on relatively steep open coasts.

Recommended Future Development

- Provide guidance regarding the combination of surge and tide in settings where two-dimensional surge modeling is warranted
- Develop guidance for the combined effects of riverine and coastal flooding
- Compare and evaluate storm surge frequency methods including Joint Probability Method, Monte Carlo, and Empirical Simulation Technique
- Evaluate storm parameter data sources and statistics
- Review wind field formulations for hurricanes, northeasters, and other storms
- Review wind stress formulations to reflect improved recent observations

**Table 15**
**STORM METEOROLOGY RECOMMENDATIONS – ATLANTIC AND GULF COASTS**

| Topic Number | Topic/Subtopic | Recommended Approach (Future Work) |
|---|---|---|
| 51 | Tide and Surge Combination | Develop guidelines for the combination of surge and tide, including examples drawn from past studies (with consideration of FEMA surge studies, ADCIRC/EST, and the FL-DEP Monte Carlo method) |
| 51 | Surge/Riverine Combination | Prepare recommendations for the statistical combination of surge and a riverine runoff profile, with consideration of non-independence of the processes; see also Topic 19 of the Hazard Mapping Focused Study for simple mapping suggestions |
| 50 | Storm Surge Frequency Aanalysis | Apply/Compare methodologies (JPM, EST, Monte Carlo) using a common hydrodynamic model and storm data set |
| 50 | Storm parameters for surge modeling | Review and evaluate available sources of storm parameters used in storm surge modeling, including NWS 38, HURDAT, and other databases |
| 50 | Storm Wind Fields | Review best available data regarding wind fields and compare with fields used in storm surge models; recommend the most appropriate models for FIS use (tropical storms, northeasters) |
| 50 | Wind Stress Formulation | Review best available data for wind stress and compare with formulations used in storm surge models; recommend the most appropriate formulation for FIS use |

## STILLWATER

### *Overview of Existing Guidelines*

For the Atlantic and Gulf Coasts, the primary difficulty with stillwater is the determination of storm surge and static wave setup, plus the contribution of astronomical tide. Existing FEMA guidelines are relatively brief–consisting primarily of checklists and requirements for data submission and documentation during a study. The material concerned with general surge modeling is contained in Section D.1.2.4, Hydrodynamic Storm Surge Model. Additional storm surge guidance is contained in Section D1.2.5, Storm Surge Model Calibration and Verification, which consists of two paragraphs on verification procedures and required backup documentation; Section D1.4.1, [Intermediate Data Submission] Before Storm Surge Model Calibration Runs, a list of eight items to be submitted for review prior to proceeding with model runs; and Section D1.4.2, Before Operational Storm Surge Runs, a checklist of seven items to be submitted for review prior to performing the main statistical simulation set of runs. There is some additional material of a general nature in Section D-2.2 dealing with Data Requirements.

The available guidelines are generally based on the use of the FEMA storm surge model, although brief mention is made of the Stone and Webster Northeaster Model and the possible stillwater elevation determination by statistical analysis of available tide gage records, provided the recorded tide gage records include 20 years or more of data. Section D.2.2 also states that “use of synthetic computer models for storm surge assessments are suggested for use and application over tide gage data, where tide gage data is limited and complex shorelines are present which cause appreciable variation in flood elevations for a community.”

### *Topics and Key Issues*

The following Stillwater topics were identified by the TWG:

**Critical –** Topic 53, Identify Reliable Existing Data to Compare to Existing FEMA Flood Studies to Test Performance of Surge Models.

**Available –** Topic 52, Provide Guidance on Non-stationary Processes [i.e., sea level change] when establishing current conditions.

Key issues are:

- Storm surge estimates can be based on an analysis of tide gage data in some regions.
- The FEMA coastal guidelines do not include any significant discussion of appropriate methods for tide gage analysis.
- The guidelines provide little guidance regarding the considerations that must be made for storm surge modeling, beyond the assumptions implicit in the use of the FEMA storm surge model.
- The availability of many new surge models and supporting tools for grid development and maintenance suggests the need for more detailed guidance regarding models and modeling practice.

- In some areas of the Atlantic and Gulf Coasts a simplified 1-D surge model would be a valuable tool. A suitable prototype for such a model is the one used by the Florida Department of Environmental Protection for Florida coastal construction jurisdictional delineations.
- The FEMA guidelines provide little guidance on the matter of non-stationary processes, and how they might affect both the determination of stillwater levels, and the interpretation of historical data used in a FIS.
- The primary non-stationary processes of concern are the relative change of sea level (sea level rise and/or land subsidence), and localized land subsidence associated, for example, with oil and water extraction or tectonic adjustment.
- Owing to improvements in computer technology, future storm surge modeling efforts can be expanded to a regional scope, providing greater uniformity and accuracy in the surge determinations, at reduced cost.
- An important question is how well FEMA coastal surge estimates will agree with experience. Model calibration in any particular study is difficult owing to uncertainties in both historical storm characteristics and levels of flooding.
- It should be possible to perform a global "calibration" through a statistical evaluation of the performance of the FEMA methodology along all major coastlines.

### *Recommended Approach*

The recommended approach for addressing these issues includes both the development and verification of analytical and modeling methods (tide gage analysis and bathystrophic surge modeling), as well as general revision of the *G&S* to provide greater insight for Study Contractors regarding the requirements of coastal modeling and data interpretation.

#### Currently Available Methods, Information and Guidelines

Information is available for development of guidance on non-stationary processes, and for development of general storm surge modeling guidance.

#### Applicability of Pacific Coast Guidelines

The Stillwater topics are generally applicable to both the Atlantic/Gulf and Pacific Coasts. The differences are primarily matters of emphasis, not physics. In particular, storm surge is generally small on the Pacific Coast in comparison with the Atlantic/Gulf. Despite this, the work for one coast will be applicable to the other. Therefore, results from the following Phase 2 work proposed for the Pacific should provide improved guidance for the Atlantic and Gulf Coasts.

- Provide guidance regarding methods for determination of storm surge based on tide gage data.
- Write general guidelines for storm surge modeling
- Implement a simplified 1-D storm surge model with guidelines for its use
- Write guidelines for consideration of non-stationary processes in a FIS

Recommended Future Development

- Develop global methods to evaluate surge model performance
- Develop guidelines for large scale regional surge modeling

**Table 16**
**Stillwater Recommendations – Atlantic and Gulf Coasts**

| Topic Number | Topic/Subtopic | Recommended Approach (Future Work) |
|---|---|---|
| 53 | General Considerations for Surge Modeling | Based on the existing literature, describe the use of surge models applicable to Atlantic and Gulf Coasts and the factors that require consideration in performing a study. |
| 53 | Surge Modeling Global Calibration | Develop statistical procedures to assess the performance of the FEMA surge models through the consideration of global experience on all coasts. |
| 53 | Regional Surge Modeling | Develop guidance for large scale regional surge modeling. |

## STORM WAVE CHARACTERISTICS

### *Overview of Existing Guidelines*

Existing FEMA guidelines provide three approaches for estimating storm wave characteristics: (1) wave data from offshore wave buoys, (2) wave data from hindcasts or numerical modeling based on historical records, (3) wave data from specific calculations based on assumed storm meteorology. For the second approach the USACE Wave Information System (WIS) hindcasts are used and these are specified at some specific (average) water depth. Mapping Partners convert such wave information into an equivalent condition at some other water depth for appropriate treatment of flood effects. For the third approach, the Shore Protection Manual (SPM) and ACES V1.7 are recommended for hurricanes and extratropical storms, respectively. The current approaches are generally adequate since the "controlling" wave height (1.6 times the significant wave height) will invariably be the limiting breaking wave at the original shoreline for WHAFIS application. However, wave setup calculations are sensitive to deep water conditions for which more accurate determinations may be necessary.

### *Topics and Key Issues*

The following Storm Wave Characteristics topics were identified by the TWG:

**Critical –** Topics 4 and 5, Sea and Swell for Open Atlantic/Gulf Coasts.

**Available –** Topic 5, Wave Generation in Sheltered Water; Topic 1, Wave Definitions.

Key issues are:

- Workshop 2 considered whether the WIS database is adequate for Atlantic and Gulf or alternative databases are necessary. The Technical Working Group determined that WIS, which was updated recently, is adequate for wave data estimation for Atlantic and Gulf Coast. Use of other available databases, such as Oceanweather's Global Re-analysis of Ocean Waves (GROW) model, is not necessary. Additionally, swell data are not important for hurricane conditions.
- Instructions are needed on the appropriate use of the WIS database–such as whether to use 100-year significant wave height or the 20-year maximum wave height in WHAFIS modeling.
- Clarification is needed on the use of equivalent deep water wave height for runup computations.
- For wave generation in sheltered waters with restricted fetch, SPM and ACES are used. The wind speed inputs into SPM or ACES are 60 mph for northeaster-dominated areas and 80 mph for hurricane-dominated areas. The appropriateness of these wind conditions should be analyzed based on more recent information.
- The Coastal Engineering Manual (CEM) has officially replaced SPM; however, CEM procedures for restricted fetch need to be evaluated before accepting the procedures for the guidelines.
- Definitions are needed in the *G&S* of waves in both the time domain and the frequency domain. Two available resources are: CEM and the International Association of Hydraulic Research publication entitled "List of Sea State Parameters".

- Specific guidance is needed on how the wave-related terms apply to the coastal processes associated with flood studies, methodologies, and models.

### *Recommended Approach*

The recommended approach is to wait until the completion of Phase 2 work for the Pacific Coast for Topic 5 (Wave Generation in Sheltered Water) before undertaking any revision to the *G&S* for the Atlantic and Gulf Coasts. The remaining critical and available topics can be revised using available references and information. The effort will be small in comparison to the storm wave characteristics efforts for the Pacific Coast.

#### Currently Available Methods, Information, and Guidelines

The updated WIS database is available and recommended for use for both the Atlantic and Gulf open coasts.

#### Applicability of Pacific Coast Guidelines

The following Pacific Coast work on Topic 5 (Sheltered Waters) will be directly applicable to the Atlantic and Gulf coasts:

- The recommendations from the Pacific Coast case study, which will compare results using CEM procedures to results using SPM procedures for a restricted-fetch Pacific Coast site, can be adopted for the Atlantic and Gulf Coast guidelines.
- The recommendations from the case study, which will compare results from the Spectral Energy Models (SEMs) and traditional Parametric Models using restricted fetch methods, can be adopted for the Atlantic and Gulf. The study will clarify application procedures for the SEMs, specifically wind field definition.

#### Recommended Future Development

- The WIS database is recommended for use. Investigate the appropriateness of using either the 100-year significant wave height or the 20-year maximum wave height while modeling WHAFIS.
- Clarify use of equivalent deep water wave conditions.
- Clarify statistical methodologies for determination of the 1% annual chance event.
- Develop guidelines on sheltered water based on Pacific Coast guidelines.
- Incorporate standard wave related definitions from USACE CEM and 1986 International Association for Hydraulic Research (IAHR) publication, "List of Sea State Parameters."
- Provide specific guidance on use of wave related definitions for physical processes applicable to coastal flood studies.

**Table 17**
**Storm Wave Characteristics Recommendations – Atlantic and Gulf Coasts**

| Topic Number | Topic/Subtopic | Recommended Approach (Future Work) |
|---|---|---|
| 4,5 | Sea and Swell for Open Atlantic and Gulf Coasts | Investigate the appropriateness of using either the 100-year significant wave height or the 20-year maximum wave height while modeling WHAFIS. Clarify use of equivalent deep water wave condition. Clarify extrapolation to 100-year |
| 5 | Wave Generation in Sheltered Water | Develop Guidelines on Sheltered Water based on Pacific Coast *G&S*. |
| 1 | Wave Definitions | Incorporate and refine the "Glossary of Coastal Terminology" directly from the USACE CEM.<br><br>Incorporate and refine the five listings of notations and parameters in the 1986 International Association for Hydraulic Research publication, "List of Sea State Parameters."<br><br>Provide specific guidance on how wave related terms in the USACE and IAHR sources relate to each other and how they should be applied relative to the following: (1) FEMA guidance for coastal flood studies, (2) physical processes that are directly associated with FEMA coastal hazard assessments and flood mapping, and (3) required coastal hazard study methodologies<br><br>Prepare an application for Atlantic and Gulf Coast Guidelines |

## WAVE TRANSFORMATION

### *Overview of Existing Guidelines*

Wave Transformations are addressed in of the FEMA G&S in terms of overland travel (Sections D.2.6 - 2.6.4) and application of the WHAFIS model. This treatment is one-dimensional (defined by a profile), and limited to shallow water breaking and dissipation processes. Dissipation due to propagation over shallow areas and marsh plants is included. However, wave dissipation due to muddy bottoms has not been included in WHAFIS. Wave refraction, diffraction and shoaling are not addressed, except in passing references such as on page D-70: "Where land shelter or wave refraction may result in reduced incident waves, it is appropriate to specify an initial significant wave height for the transect." The emphasis of the *G&S* is on depth-limited, shallow water propagation and dissipation, which is logical because these are important issues in the Atlantic and Gulf Coasts.

### *Topics and Key Issues*

The following Wave Transformation topics were identified by the TWG:

**Critical –** Topic 9, Wave Energy Dissipation Over Shallow, Flat Bottoms.

**Important –** Topic 10, Overland Wave Propagation; Candidate Improvements to WHAFIS.

**Helpful –** Topic 8, Wave Transformation With and Without Regional Models.

Key issues are:

- Wave Transformations are important processes that change wave characteristics when propagating toward shore, generally from deep to shallow water, and are addressed as an intermediate step between forcing processes (wave generation) and response processes (wave setup, wave runup, and overtopping) in coastal flood studies.

- Wave dissipation caused by bottom effects are not routinely considered in wave transformation processes. Effects of wave energy dissipation in shallow water can result in reduced wave heights in certain shorelines. Ignoring wave dissipation may lead to overestimates of flood hazard risk for shorefront development. Study Contractors need guidance on when and where to apply bottom dissipation mechanisms. Some guidance is available in the current *G&S*.

- Overland wave propagation is common during extreme events in the Atlantic and Gulf Coasts. FEMA-approved WHAFIS 3.0 is presently applied in FISs. Potential improvements to WHAFIS have been identified (see Topic 10).

- The emphasis of the *G&S* on depth limited shallow water propagation and dissipation may be logical for the Atlantic and Gulf Coasts. However, it will be preferable to cross-reference new Pacific Coast Wave Transformation guidelines because the Atlantic and Gulf Coast methods may not be appropriate for all sites, including sheltered waters.

### *Recommended Approach*

The recommended approach to the Wave Transformation focuses on improvement of wave dissipation and propagation modeling in Atlantic and Gulf Coast settings.

Applicability of Pacific Coast Guidelines

Pacific Coast work will be applicable to the Atlantic and Gulf for Topics 8 and 9:

- While focused on the Pacific Coast, the guidance on wave transformation will also be useful for flood studies on the Atlantic and Gulf Coasts, especially since wave transformation methods are not discussed elsewhere in the *G&S*. The wave transformation methods to be recommended are general approaches applicable to all water bodies, and hence can be used for Atlantic and Gulf Coasts, as well as sheltered waters. Guidance on the appropriate methods for a range of site conditions will also be provided.
- Guidance will also be developed for wave dissipation over shallow flats and marshes, which should complement existing guidance.

Recommended Future Development

- Write *G&S* to include a section on wave energy dissipation over shallow and flat bottoms.
- Develop typical ranges for dissipation coefficients for a variety of bed and wave conditions to be included in the *G&S*.
- Categorize bed and wave conditions for U.S. coastlines. Revise *G&S* to provide dissipation coefficients on a geographic basis; revise *G&S* to adopt the Suhayda (1984) or other appropriate method.
- Develop improvement to WHAFIS model

**Table 18**
**Wave Transformation Recommendations – Atlantic and Gulf Coasts**

| Topic Number | Topic/Subtopic | Recommended Approach (Future Work) |
|---|---|---|
| 9 | Wave Energy Dissipation over Shallow Flat Bottoms | Write *G&S* to include a section on wave energy dissipation over shallow and flat bottoms;<br>Develop typical ranges for dissipation coefficients for variety of bed and wave conditions to include in the *G&S*.<br>Categorize bed and wave conditions for US coastlines. Revise *G&S* to provide dissipation coefficients on a geographic basis; revise *G&S* to adopt Suhayda (1984) method. |
| 10 | Overland Wave Propagation, Candidate Improvements to WHAFIS | Evaluate new methods to better represent vegetation effects, treatment of elevated pile supported buildings<br>Minor Effort – WHAFIS code changes for more user friendly program<br>Moderate Effort – more intense code changes for improvement in accuracy and graphics (in WHAFIS)<br>Significant Effort - Revise WHAFIS to consider combined effects of damping and wind action over each segment. |
| 8 | Overall Wave Transformation with and | Cross reference Pacific Coast guidelines, and emulate important topics for Atlantic and Gulf Coasts. |

| | | |
|---|---|---|
| | without Regional Models | |

## WAVE SETUP

### *Overview of Existing Guidelines*

FEMA *G&S* are based on the 1984 USACE SPM. These results have been developed from laboratory tests and wave theory and are applicable for beaches of uniform slope, although some guidance is given for non-planar beach profiles. The guidance applies to the static wave setup at the shoreline, but does not address dynamic wave setup. The *G&S* mention setup across reefs, but do not provide specific guidance. The *G&S* also do not provide guidance on settings such as flooded barrier island and areas with dissipative (e.g., muddy) bottoms.

### *Topics and Key Issues*

Table 2 lists the topics identified by the Technical Working Group for Wave Setup for the Atlantic and Gulf Coasts.

**Critical –** Topic 44, Better Define and Document; Topic 45, Compile Example Data and Perform Tests; Topic 46, Develop Interim Method.

**Important –** Topic 47, Develop Ideal Method; Topic 48, Develop Procedure for Dynamic Wave Setup.

Key issues are:

- Under the action of irregular waves, wave setup consists of a static component and a dynamic component, both of which can be substantial and are relevant to erosion and other storm-induced hazards.  The dynamic component is not considered in the present guidance.
- The Atlantic and Gulf Coasts include a broad range of physiographic settings and procedures are needed for each setting.
- Considerations of inland excursion of static and dynamic setup, and wave setup variation over flooded inland areas have been a challenge in some flood studies.
- Wave setup has not been treated uniformly in previous flooding studies on the Atlantic and Gulf (A&G) Coasts. It is estimated that approximately 40% of previous studies on the A&G coasts have included wave setup in specification of the 1% annual chance storm surge. Wave setup can comprise up to approximately 50% of the total 1% surge elevation in locations with narrow continental shelves such as southeast Florida.
- Ideally, wave setup will require specification of directional wave spectra as input at an offshore location seaward of wave breaking.
- Wave setup is included, to some degree, in wave runup measurements and methods. It will be necessary to separate these terms to avoid double counting of setup.
- There are two approaches for calculating wave setup: (1) The Boussinesq models which, in principle, can calculate both wave setup and wave runup, and (2) Coupling of more conventional engineering-based models.

### Recommended Approach

The recommended approach is generally similar to that for the Pacific Coast with the exception of specification of the input wave characteristics. Because the wind-induced setup plays a more dominant role on the Atlantic and Gulf Coasts, it is necessary to utilize a model that incorporates a wind field. This same wind field could be used to generate waves. The method and *G&S* should include the same elements as for the Pacific Coast. Interaction with other Focused Study groups will be essential throughout the effort.

Currently Available Methods, Information and Guidelines

The general technology includes theory, a great deal of laboratory data, but very little quality field data– are available. Challenges include selecting the most appropriate approach (Boussinesq or engineering-based models). Current guidance is based on a depth-limited wave at the shoreline. Current guidance, which is based on SPM procedures, should be retained until new methods are developed.

Applicability of Pacific Coast Guidelines

It is estimated that 60% of the work accomplished for the Pacific Coast will be applicable to Atlantic and Gulf Coasts. As noted, the principal difference will be in the specification of the wave characteristics upon which the setup will be based. In particular, the items that will be directly applicable are:

- Intercomparison of Boussinesq models and comparison with data sets. Select Boussinesq or engineering-based approach.
- Develop and document engineering-based approach for wave setup modeling along open coasts and in sheltered waters. With the exception of wave input, this item will be identical.
- Compile potential data sources for testing.
- Develop breaking zone model with particular emphasis on wave setup, proof test, compare with data sets, refine, and write draft User’s Manual.

Recommended Future Development

The Atlantic and Gulf Coasts will benefit by the methods developed for the Pacific Coast and overall insights gained in Phase 2 on related coastal processes such as wave runup. However, additional work on Topics 44, 45, and 46 will be required to formulate guidance for Atlantic and Gulf Coast physiographic settings.

For the ideal method, which would couple storm surge and wave setup in a single methodology, the following additional tasks need to be undertaken:

- Develop “Ideal Methodology” coupling storm surge and waves to calculate static wave setup
- Develop modeling procedure for dynamic wave setup based on wave spectra

**Table 19**
**Wave Setup Recommendations – Atlantic and Gulf Coasts**

| Topic Number | Topic/Subtopic | Recommended Approach (Future Work) |
|---|---|---|
| 44 | A&G Coast Definitions | Develop wave setup definitions with emphasis on A&G coast applications. |
| 45 | Compile Data for Testing | Locate as much quality field data as possible for testing of developed/selected approach(es). |
| 46 | Develop Engineering Based Approach | Couple accepted engineering models for calculating wave setup across surf zone. Include procedure for dynamic wave setup. |
| 46 | Evaluate Boussinesq Models | Intercompare at least three Boussinesq models and compare with data. |
| 46 | Develop Breaking Zone Model | Evaluate candidate breaking zone models that allow specification of non-planar profile |
| 47 | Ideal Model for Static Wave Setup | Couple wave generation and wave setup model, allowing specification of arbitrary tide. |
| 48 | Develop Model for Dynamic Wave Setup | Develop method based on directional and nonlinear spectrum as input. |

## WAVE RUNUP AND OVERTOPPING

### *Overview of Existing Guidelines*

Existing Guidance in Section D.2 calls for the use of the FEMA RUNUP 2.0 model, except for the case of vertical/near-vertical barriers, where SPM methods are recommended. Section D.2 overtopping methods are based on Owen (1980) and Goda (1985).

### *Topics and Key Issues*

The following Wave Runup and Overtopping topics were identified by the TWG:

**Critical –** Topic 12, Use of Mean vs. Higher Values for Runup and Overtopping.

**Available –** Topic 13, Overtopping Volumes; Topic 49, WRUP$^{TM}$.

**Important –** Topic 11, Review Methods and Models.

**Helpful –** Topic 14, Wavecast Debris.

Key issues are:

- Runup tends not to control BFEs along the Atlantic and Gulf Coasts, except in New England and in bluff areas (wave height and primary frontal dune criteria tend to control VE zone designations and BFEs in low-lying and dune-backed areas).
- Many effective Flood Insurance Studies were completed using the FEMA early runup model, RUNUP 1.0. Substantial differences between the results of RUNUP 1.0 and 2.0 can exist, but the magnitude and significance of these differences is currently unknown (few comparative studies have been performed).

### *Recommended Approach*

The recommended approach involves: 1) comparing RUNUP 1.0 and 2.0 results; 2) evaluating the use of $R_{50\%;}$ 3) adjusting RUNUP 2.0 results, where appropriate; 4) testing runup methods and models (first priority is New England); and 5) evaluating overtopping and revising hazard zones.

<u>Currently Available Methods, Information and Guidelines</u>

Updated runup and overtopping methods, models and data exist.

<u>Applicability of Pacific Coast Guidelines</u>

Much of the Pacific Coast Phase 2 work will be applicable to the Atlantic and Gulf Coasts. However, many tasks need to be repeated for the specific physiographic and hydrodynamic settings of the Atlantic and Gulf Coasts. The applicable Phase 2 tasks are:

- The evaluation of the R50% value on the Pacific Coast might also be applicable to Atlantic and Gulf Coasts, but only approximate consistency between the coasts is expected. The relative importance of infragravity motions and dynamic wave setup on different coasts will preclude transferring Pacific Rx% results (and adjustments to RUNUP 2.0) without additional testing on the Atlantic and Gulf.

- Overtopping calculations, threshold rates, and mapping methods are expected to generally transfer to the Atlantic and Gulf Coasts.
- RUNUP 2.0 has been used extensively along the Atlantic and Gulf Coasts already, and any updated guidance developed from the Pacific Coast work should serve to improve guidance in Section D.2.

Recommended Future Development

- Perform detailed comparisons of wave runup and mapping using RUNUP 1.0 and 2.0. Determine whether to adjust prior studies using RUNUP 1.0 or to restudy using RUNUP 2.0 (or other methods).
- Analyze Atlantic and Gulf runup distributions, and compare with Pacific results for transfer of appropriate Rx% level and any adjustments to RUNUP 2.0 results.
- Conduct more comprehensive testing of wave runup methods and models, and identify appropriate runup calculation procedures for a wide variety of shore types, profile characteristics, and incident water level and wave conditions (same as Pacific).
- Update procedures for calculating overtopping and ponding on low bluffs, with gently sloping or adverse slopes (same as Pacific).

**Table 20**
**Wave Runup and Overtopping Recommendations – Atlantic and Gulf Coasts**

| Topic Number | Topic/Subtopic | Recommended Approach (Future Work) |
|---|---|---|
| No Topic number assigned. | Revise Guidance to Reflect Current FEMA Practice | Revise guidance to describe use of ACES for runup and overtopping calculations (ACES is based on more recent procedures than SPM or RUNUP 2.0).<br>Revise guidance to clarify use of *equivalent* deepwater wave conditions with RUNUP 2.0 |
| 12 | RUNUP 1.0 vs. 2.0 | Perform detailed comparisons of wave runup using RUNUP 1.0 and 2.0.<br>Determine whether to adjust prior RUNUP 1.0 studies or to restudy using RUNUP 2.0 (or other methods). |
| 12 | Evaluate Use of Mean Runup Value | Review runup distributions and damages for Atlantic/Gulf beaches and structures, compare against Pacific.<br>Evaluate use of $R_{50\%}$ and select alternate $R_{x\%}$ value (probably between $R_{33\%}$ and $R_{10\%}$) if $R_{50\%}$ understates observed hazard. |
| No topic number assigned. | Wave Setup Component | Treatment of wave setup component (in FEMA's current wave runup procedure) to be coordinated with Wave Setup study. |
| 11, 49 | Conduct Comparative and Sensitivity Testing of Runup Models and Methods | Compare results using simple methods versus numerical models, deterministic (event selection) versus statistical approaches.<br>Test runup methods and models – priority to be given to testing in New England region.<br>Identify appropriate runup methods and models by location, morphology and hydraulic conditions |
| 13, 14 | Guidance for Overtopping and Wave Cost Debris | Maintain use of mean overtopping rate (cfs/ft, $m^3$/ per m)<br>Evaluate recent data and methods<br>Apply Pacific results relative to damaging overtopping rates and FEMA hazard zone thresholds<br>Evaluate wave-cast debris coincidence with overtopping<br>Coordinate with Hazard Zone study |

# EVENT BASED EROSION

## *Overview of Existing Guidelines*

FEMA guidelines (Appendix D) have not been updated since 1989 and focus primarily on the effects of extreme hurricanes and northeasters. They do not provide specific guidance for assessing event-based erosion (storm-induced erosion) in sheltered waters, or non-sandy beach and coastal dune areas; and provide only a simplified empirically based geometric relationship (the 540 Criterion) for erosion assessments along the Atlantic and Gulf open coasts. Existing event-based erosion (EBE) procedures do not account for beach materials with different erodibilities, for storms with different durations, or for dune overwash processes.

## *Topics and Key Issues*

Table 2 lists the topics identified as necessary to improve current guidelines and/or develop new guidelines related to event-based erosion.

**Critical –** Topic 33, Add Discussions to *G&S* Regarding Limitations of Geometric Methods for Cobble/Shingle Beaches; Topic 35, Add Discussions to *G&S* Regarding Erosion Assessments in Sheltered Water Areas.

**Available –** Topic 31, Add Discussions to *G&S* Regarding Bluff Erosion; Topic 32, Develop Geometric Method for Bluff Erosion; Topic 41, Discuss Long-term Erosion/Future Conditions; Topics 42 and 43, Treatment of Nourished Beaches.

**Important –** Topic 34, Develop Geometric Methods for Cobble/Shingle Beaches; Topic 36, Review Data and Develop Geometric Methods for Sheltered Water Areas; Topic 37, Expand Database and Re-evaluate Aspects of 540 Criterion; Topic 38, Assess and Develop Process-Based Methods.

**Helpful –** Topic 39, "Primary Frontal Dune Definition," was moved to the Hazard Zones Focused Study; Topic 40 Documentation of Observed Vertical Erosion Depths for "Depth-Damage" Assessments).

Key issues are:

- Guidance for evaluating EBE remains unchanged since 1989 and focuses primarily on effects of extreme storms (hurricane or northeasters) along the Atlantic and Gulf Ccoasts, with a modified approach for the Great Lakes Coasts.
- Beach material properties, coastal erosion processes, and storm characteristics found along the north Atlantic Coast may differ significantly from those along the south Atlantic, Gulf, or Great Lakes.
- The main erosion related factors affecting beach profiles are: (1) the forcing processes that include the duration and time histories of the wave characteristics, water levels, and runup; and (2) the response elements that include the physiographic setting and the beach and dune/bluff characteristics, including material erodibility.

- Refinement to Atlantic and Gulf Coast *G&S* and new *G&S* should have the same fundamental structure as the Pacific Coast *G&S* to be developed that includes: (1) physiographic and geomorphic setting, (2) sediment characteristics across the active profile, (3) the effects of time histories of storm wave and tide characteristics, and (4) local or regional oceanic or topographic characteristics that may affect the study area. Consideration of this common structure will ensure that event-based erosion assessments will be consistent for all applications.
- The eroded beach profile that exists during the base event is needed to calculate the 1% annual chance flood elevation. Present guidelines do not specifically account for event duration, different beach materials, or dune overwash processes.
- Existing *G&S* can be improved by better defining "storm induced erosion" or event-based erosion and discussing different approaches for assessing beach and back beach profile changes caused by erosion on all coasts of the United States.
- Process-based numerical models (1-D and 2-D, steady and unsteady) may provide improved means for assessing event-based erosion in the future. Reliable numerical procedures are not presently available for general applications in Flood Insurance Studies.
- Guidance for evaluating erosion of cobble/shingle beaches is needed.
- Guidance for evaluating erosion of sandy and non-sandy bluffs and cliffs is needed.
- Guidance for evaluating erosion within sheltered water areas is needed.
- Present *G&S* provide no specific guidance on how to address beach nourishment projects.
- The 540 Criterion is based on limited data from which the erosion-frequency relationship and median value trigger for dune removal were developed. Those data and criteria may need updating.

### *Recommended Approach*

Initially, the *G&*S should be updated using more current and available reference materials and information to address topics presently covered in the *G&S*. Future *G&S* for the Atlantic and Gulf Coasts should be expanded to include new information and improved alternative methods discussed or referenced in the Focused Studies. New methods being developed for the Pacific Coast may provide additional insight and useful information in the following three categories and levels of effort: (1) developing eroded profiles based on available historical mapping, LIDAR data, and photographs, (2) profiles based on simplistic empirical methods (other than the 540 Criterion), and (3) discussions of future methods to develop profiles using process-based (steady and unsteady) models.

<u>Currently Available Methods, Information, and Guidelines</u>

More recent information (than is provided in the present *G&S*) on Event Based Erosion processes and evaluation procedures are available. See appended Event-Based Erosion Focused Study for discussions of sheltered water areas, cobble/shingle beach processes, insights on process-base modeling methods, and discussions on erosion processes for different physiographic settings.

Applicability of Pacific Coast Guidelines

Approaches and insights adopted from Pacific Coast Phase 2 work on the following topics may be helpful to the Atlantic and Gulf Coasts:

- Simplified geometric models (their basis and limitations).
- Interim approach for assessing bluff and cliff erosion
- Interim approach for assessing gravel, cobble and shingle beach and dune erosion
- Interim methods for erosion assessments in sheltered water areas
- Guidance on information needed to assess special cases of beach nourishment (as an exception to existing FEMA policy).

Recommended Future Development

- Provide discussion of gravel, cobble, and shingle beaches, and dune erosion in different settings to distinguish this type of erosion hazard from other erosion processes; provide examples, figures and definitions; explain limitations of existing 540 Criterion for application to this type of erosion and beach material characteristics
- Develop new methods and *G&S* for sheltered water areas
- Describe bluff and cliff erosion; explain limitations of existing 540 Criterion for application to this type of erosion process; develop methods for assessing bluff and cliff erosion in different coastal settings
- Evaluate whether nourished beaches affect hazard zone delineations and BFEs
- Develop methods (geometric or process-based) for assessing gravel, cobble, and shingle beach and dune erosion
- Expand data sets and review erosion-frequency relationship and median value trigger for dune removal upon which the 540 Criterion is based
- Develop suite of process-based models for general coastal erosion assessments in different settings, including dune overwash processes

**Table 21**
**Event Based Erosion Recommendations – Atlantic and Gulf Coasts**

| Topic Number | Topic/Subtopic | Recommended Approach (Future work) |
|---|---|---|
| 33, 34 | Gravel, cobble, and shingle beach and dune erosion | Review available literature and reporting; improved *G&S* language and descriptions for Atlantic and Gulf coasts to distinguish gravel, cobble and shingle beach and dune erosion from other processes; provide figures and examples.<br>(1) Perform case studies to test and develop new geometric methods for cobble beaches, (2) Test process based methods, (3) Develop new *G&S*. |
| 35, 36 | *G&S* in Sheltered Water areas | Improve *G&S* with definitions and discussion of characteristics of sheltered water areas and the types of morphology, material types and wave characteristics unique to sheltered water areas. Recommend interim G&S based on historical beach profiles and field observations.<br>(1) Conduct pilot studies, (2) Test process-based methods, (3) Develop new *G&S* for sheltered water areas |
| 31, 32 | Bluff and Cliff Erosion | Review available literature and reporting; improve *G&S* language and descriptions for Atlantic and Gulf Coasts to distinguish bluff and cliff erosion from other processes; provide figures and examples.<br>(1) Review existing bluff erosion procedures and international literature, (2) Develop geometric procedures for bluff and cliff erosion and retreat, (3) Consider development and use of process-based numerical/statistical modeling methods for future inclusion in the NFIP program. |
| 41 | Long - Term Erosion | This topic is considered important to NFIP, but FEMA action on previous work is pending. Therefore, guidance is best developed by FEMA in the future. |
| 42, 43 | Nourished Beaches | Recommend modifying *G&S* to direct Study Contractors to follow a procedure to notify FEMA that the study area includes beach nourishment project. Provide FEMA with a list of information needed to assess special cases where beach nourishment may be considered in determining hazard zones and BFEs (exception to existing FEMA policy).<br>Conduct research and case studies to determine whether beach nourishment is likely to have an effect on hazard zone designations of BFEs. |
| 37 | Clarify Applicability and Limitations of 540 Criterion | Clarify limitations of 540 Criterion regarding its application to different types of coastal settings and material types. Discuss limitations of geometric methods versus process-based methods.<br>For the 540 Criterion: (1) Expand data base, (2) Define erosion area-frequency relationship, (3) Review use of median value trigger for dune removal. |
| 38 | Physics and Process-Based Methods | Describe differences and advantages between "geometric" and "process-based" EBE methods. Interim methods: continue to use 540 Criterion for Atlantic and Gulf Coasts where applicable; use most documented post-storm beach and dune profiles for areas where 540 is not applicable.<br>(1) Further develop and test process-based models; (2) Develop method to include randomness of storm wave heights and tides and their coincident occurrence; (3) Develop and test process-based methods and prepare *G&S* for process-based erosion assessment of (a) coastal bluffs fronted by narrow beaches and (b) sandy and non-sandy beaches and dunes, including dune overwash. |
| 40 | Document Vertical Depths of Erosion | Document depths of erosion following storm events and maintain data for depths of erosion and damages to buildings in order to better determine "depth-damage" relationships. |

## COASTAL STRUCTURES

### *Overview of Existing Guidelines*

Existing Guidance in Section D.2 calls for the evaluation of structures to determine whether they will survive the 1% annual chanceflood event; the guidance references CERC TR 89-15 for evaluation criteria, but states study contractors should consider available documentation and performance information (i.e., use engineering judgment) as well.

### *Topics and Key Issues*

The following Coastal Structures topics were identified by the TWG:

**Available –** Topic 25, Review *G&S* language regarding 89-15; add new procedure for flood hazard modeling in the presence of coastal structures; Topic 21, Clarify guidance for dealing with failed structures during base flood; Topic 23, Add *G&S* language that buried structures are to be evaluated; Topic 27, Review and clarify *G&S* and regulations regarding treatment of coastal levees and structures; Topic 24, Review 89-15 and other literature for tsunami failure information and guidance – of some importance on South Atlantic and Gulf Coasts.

**Helpful –** Topic 22, Investigate configuration of failed structures; Topic 26, Review data on, and add to *G&S*, effects of structures on flood hazards on adjacent properties, flooding/waves behind structures via adjacent properties; and a portion of Topic 27, Review and revise TR-89-15 evaluation criteria.

Key issues are:

- Coastal structures can modify flood levels, wave effects, and topography, both landward of, seaward of, and adjacent to the structures, and must be considered during the mapping of coastal flood hazards. Two scenarios are commonly encountered: structures and their effects are analyzed during Flood Insurance Studies; and structures frequently serve as the basis for revisions to FIRMs.
- FEMA *G&S* can be improved by expanding or adding discussions on coastal structure failure, buried structures, and the effects of structures.
- The effects of structures can be divided into two categories: effects on erosion and effects on flood conditions. Two scenarios are important for each: (1) the effects of structures on adjacent properties; and (2) the effects on property immediately landward (and seaward) of a structure.
- Guidance for evaluating coastal structures has been largely unchanged since publication of the USACE report CERC TR 89-15 in 1989. The evaluation criteria need to be reviewed considering more recent information. Revisions may or may not be warranted.
- Guidance needs to clearly state that study contractors are not required to use CERC TR 89-15.
- Guidance on the evaluation of coastal structures in tsunami-prone areas is needed.

- FEMA *G&S* call for structure "removal" from subsequent flood hazard analyses in the event that a structure fails (i.e., does not survive the base flood event), but guidance on uncertified structure removal should be expanded and revised. More importantly, the configuration of a failed structure can affect wave runup and overtopping calculations. A method to address uncertified structures, used in a recent Pacific Coast flood study (by PWA), has been modified by the Focus Study and is recommended for use.

- Coastal structures and levees are sometimes treated differently, and those differences should be justified or eliminated. The *G&S* should address coastal levees.

- FEMA *G&S* were written primarily considering seawalls, bulkheads, revetments, and do not address the effects of other structures types (e.g., jetties, groins, breakwaters). While treatment of these other structures is needed, it is deemed a lower priority than revising the guidance related to seawalls, bulkheads, revetments and levees.

### Recommended Approach

The recommended approach is to revise the *G&S* using available references and information. The effort will be modest by comparison with some of the other Focus Study topics.

Currently Available Methods, Information and Guidelines

Updated information on coastal structure evaluation and criteria are available. See Coastal Structures Focused Study report.

Applicability of Pacific Coast Guidelines

Pacific coast work will be directly applicable to the Atlantic and Gulf coasts on five topics:

- Buried structures and failed structure configurations (including progressive collapse of revetments).
- Treatment of failed ("removed") structures for wave height and runup analyses.
- Investigation of structure effects on erosion and flood hazards.
- Consistency in treatment of coastal structures and coastal levees.
- Evaluating structures in tsunami-prone areas.

Recommended Future Development

- Revise/update CERC TR 89-15 coastal structure evaluation criteria.
- In addition to the current structural criteria, develop minimum structure dimensions (e.g., length, return wall length) necessary to receive mapping credit during Flood Insurance Studies and flood map revisions.
- Revise guidance to consider jetties, groins and breakwaters.

**Table 22**
**Coastal Structures Recommendations – Atlantic and Gulf Coasts**

| Topic Number | Topic/Subtopic | Recommended Approach (Future work) |
|---|---|---|
| 26 | Jetties, Groins, Breakwaters | Develop criteria/guidance for evaluating failure of other structure types, and the effects of these failures on mapped flood hazards |
| 26 | Minimum Structure Dimensions | Determine minimum structure dimensions necessary to receive mapping credit during FIS and revisions to FIRMs |
| 27 | Structure Evaluation Criteria | Review CERC TR 89-15 considering more recent data on structure stability and failure; revise structure evaluation criteria. |

## SHELTERED WATERS

### *Overview of Existing Guidelines*

Appendix D.1 through D.2 of the existing *G&S* are generally written to provide guidance for coastal flood studies along the open coasts of the Atlantic Ocean and Gulf of Mexico. Several references to sheltered water areas are made in these *G&S*, but detailed guidance is not provided. *G&S* for the Great Lakes regions are provided in Appendix D.3, but may not be applicable for general application to smaller shelter water areas with limited fetch.

### *Topics and Key Issues*

The following Sheltered Waters topics were identified by the TWG:

**Critical –** Topic 6a, Definitions and classifications; Topic 6b, Prepare guidance for developing validation data from historic events; Topic 6d, Define 1% annual chance flood event in SW; Topic 6e, Guidance for estimating Stillwater elevations; Topic 6h, Coordinate/integrate SW guidelines with other Focused Studies and other Map Mod objectives.

Key issues are:

- The existing *G&S* are generally written to provide guidance for coastal flood studies along the open coasts of the Atlantic Ocean and Gulf of Mexico. Several references to sheltered water areas are made in these guidelines, but detailed guidance is not provided.
- Sheltered waters are water bodies with shorelines that are not subjected to the direct action of undiminished ocean waves. Although similar processes contribute to flooding along sheltered water shorelines as along open coastlines, such as wave setup, runup and overtopping, there are several aspects of sheltered water flood hazards not addressed in the current *G&S*. Additional guidance is needed.
- Wave generation and transformation in SW are typically limited by an open water fetch distance, complex bathymetry and often the presence of structures. A sheltering effect typically reduces wave energy and flood potential compared to open coast areas; however, wave runup and overtopping along SW shorelines may present additional hazards from wave-cast debris and backshore flooding.
- Wave-cast debris from extreme wave runup and overtopping can be especially problematic, owing to the proximity to fluvial sources of such materials in many estuaries.
- SW areas often have unique flood hazards due to the effects of fluvial drainages, modified tidal and surge hydrology, and relatively strong tidal currents.
- Other unique flood-related characteristics include the complex geometry of the embayments, non-coincidence of peak storm surge with peak winds, shallow water and restricted wind fetches for wave growth, and non-sandy shoreline types with special erosion and scour hazards.
- Appendix D.2.2.7 states the “analysis of restricted fetches” in “sheltered coastal sites” is addressed in the existing guidelines and the ACES software is referred to; however, more specific guidance is needed on how to apply this software to fetch-limited conditions.

- Appendix D.2.5.5 addresses wave runup and overtopping on shoreline barriers where overtopping flows discharge across landward-dipping or level backshore slopes to a "bay, river, or backwater". These situations are prevalent in SW areas. Additional guidance is needed.

- Appendix D.1.2.4 states "Methods by which barriers, inlets and rivers have been treated" are required in documentation of the hydrodynamic storm surge model. However, no guidance is provided for methods to consider modeling for sheltered waters.

- New guidelines are needed to inform and guide Mapping Partners in the preparation of coastal flood insurance studies and flood hazard maps in sheltered water areas of the coastal floodplain.

### *Recommended Approach*

The recommended approach is identical to that for the Pacific Coast. A separate section on Sheltered Waters is recommended for the Pacific Coast *G&S* as well as the Atlantic and Gulf Coast Guidelines to direct Mapping Partners to pertinent guidance found elsewhere in the *G&S* and readily available literature. This section will also provide specific new information and guidance for assessing flood hazards in Sheltered Waters.

Currently Available Methods, Information and Guidelines

- Many FEMA-approved coastal flood insurance studies have been completed in sheltered waters located along the Atlantic and Gulf Coasts.

- The USACE has published a guide for local officials for use in planning shoreline erosion management and mitigation projects in sheltered waters.

- Other information describing the physical setting, physical processes and coastal flood hazards in sheltered waters along the Atlantic and Gulf Coasts is available on the Internet and through other public sources. See appended Focused Study on Sheltered Waters for discussions of key coastal flooding assessment topics, known procedures, and recommended sources of information.

Applicability of Pacific Coast Guidelines

Work completed for the Pacific Coast will be applicable to the Atlantic and Gulf Coasts on three topics:

- Provide general definitions, examples, and develop a classification method and general approach for conducting sheltered water studies versus open coast studies. This will serve as a framework and generalized approach for Mapping Partners to follow when conducting coastal flood hazard assessments.

- Prepare general guidance for documenting and using high water marks to reconstruct historic flood conditions to validate flood study results.

- Prepare guidelines that comply with other related FEMA Map Modernization objectives and multi-hazard planning initiatives.

The Phase 2 Sheltered Waters work for the Pacific Coast *G&S* will involve collaboration and coordination with other Focused Study groups on related sheltered water "Critical" topics listed in the summary table for the Atlantic and Gulf Coasts. Technical references, some data, and general procedures should be applicable to Atlantic and Gulf Sheltered Water areas.

Recommended Future Development

The characteristics and physics of wave runup and overtopping are fundamentally the same on the Atlantic and Gulf Coasts as they are on the Pacific Coast. However, the physical setting, the magnitude, seasonal frequency, and direction of regional storm systems that lead to high stillwater elevations and wave action that combine to generate flood hazards can be very different on the coasts. Several of these coastal differences should be addressed in the remaining two sheltered water topics:

- Prepare guidance specific to defining the 1% annual chance flood event involving dependent and independent joint probability occurrences of riverine and tidal flooding in sheltered water areas and expand guidance on wind data acquisition and analysis and fetch-limited wave forecasting in sheltered waters.
- Prepare guidance for estimating stillwater elevations in ungaged sheltered waters bodies and evaluating the effects of tidal and riverine currents on wave propagation in sheltered waters.

**Table 23**
**Sheltered Waters Recommendations – Atlantic and Gulf Coasts**

| Topic Number | Topic/Subtopic | Recommended Approach (Future work) |
|---|---|---|
| 6a | Definitions and Classification | Provide definitions, examples, and develop a classification method for sheltered water studies. |
| 6b | Flood Event Reconstruction | Prepare general guidance for documenting and using high water marks to reconstruct historic flood conditions. |
| 6d | Combined Tidal-riverine 1% Annual Chance Event Assessment | Prepare guidance specific to defining the 1% annual chance flood event involving riverine and tidal flooding and expand guidance on wind data acquisition and analysis and fetch-limited wave forecasting. |
| 6e | Stillwater Estimation | Prepare guidance for estimating stillwater elevations in ungaged sheltered water bodies and evaluating the effects of tidal and riverine currents. |
| 6h | Hazard Mitigation Coordination | Prepare general guidance for Mapping Partners to coordinate the preparation of coastal studies with other hazard mitigation activities. |
| 6h | Focused Study Coordination | Collaborate/coordinate with other Focused Study groups to address sheltered waters Critical topics found in other Focused Studies. |

## HAZARD ZONES

### *Overview of Existing Guidelines*

FEMA *G&S* (Section D2.7) contains requirements for depicting the results of the hazard analyses on the FIRMs. In Section D.2.7.2, "Identification of Flood Insurance Risk Zones," is an overview of the various hazard zone mapping criteria for zones VE, AE, AO, AH, and X, considering the combined effects of storm-induced erosion, wave height, wave runup, wave overtopping, primary frontal dunes, and coastal flood protection structures. The *G&S* also includes a series of examples that represent common flood hazard zone mapping scenarios based on transects.

### *Topics and Key Issues*

The following Hazard Zones topics were identified by the TWG:

**Critical –** Topic 39, Definition of the primary frontal dune; Topic 17, Several sub-topics related to delineation of VE Zone limits, including BFE transitions, use of VO Zones, wave overtopping, wave-cast debris hazards, and use of the primary frontal dune definition.

**Available –** Topic 19, Determination of combined probabilities and mapping for areas subject to both coastal and riverine flood sources).

**Important –** Topic 18, Several sub-topics related to the appropriateness of existing VE and AE Zones.

Key issues are:

- The definition of primary frontal dune (PFD) is "where there is a distinct change from a relatively steep slope to a relatively mild slope" in 44 CFR 59.1. The definition does not provide a quantitative method for establishing the landward limit of the PFD, yet it has significant influence on hazard zone delineation (see below). The PFD definition and delineation also has implications for floodplain management because dune areas within a VE Zone are protected under 44 CFR 60.3(e)(7).

- Coastal high hazard zones are defined in 44 CFR 59.1 to include the area up to the landward limit of the PFD along open coasts. In practice, this definition frequently dominates the determination of the VE Zone boundary. An improved definition or quantitative methodology is needed to improve consistency in hazard zone delineation.

- The use of the PFD definition for VE Zone mapping may cause areas that are subject to significantly different levels of flood risk to be mapped in a single VE Zone. The seaward portion may be subject to inundation by active coastal processes during the base flood (erosion, wave height, wave runup, and wave overtopping). The landward portion may be subject to a lower level of risk, but is included solely on the basis of the PFD limit defined by topography.

- Transitions in the BFEs are frequently abrupt where the PFD definition is used to establish a VE Zone limit, and the AE Zone behind the PFD has a much lower computed BFE. Improved procedures are needed to accurately relate mapped BFEs to flood risk. Alternative procedures for mapping the transition in BFEs or alternative flood hazard zone delineations may be advisable.

- The wave overtopping criteria presently used in VE Zone hazard mapping require expansion and review to evaluate threshold rates, the extent of the mapped zones, and the potential for use of VO Zones to more accurately reflect actual hazards landward of overtopped dunes, coastal ridges, and shore protection structures. This is particularly applicable to the Northeast Atlantic Coast, where flood hazard zones may be dominated by wave runup and overtopping, and wave-cast debris is a significant hazard.

- Coastal Special Flood Hazard Areas (SFHAs) on the Atlantic and Gulf Coasts may be quite broad with many subdivided hazard zones and BFEs. These areas are subject to significant overland wave propagation (primarily in Mid- to South-Atlantic and entire Gulf Coast). A review is needed to determine the feasibility of subdivision of the coastal AE Zone SFHA into two portions: (1) a seaward portion exposed to direct flood and wave effects from a principal flood source, to be regulated as a Coastal A Zone (similar to VE Zone regulations): and (2) a more landward portion of the AE Zone where wave effects are reduced and VE Zone regulations are not needed.

- A methodology is needed for determining and mapping flood hazard areas where coastal flooding intersects and combines with a riverine flood profile. Previous FEMA guidance should be reviewed for this condition.

Recommended Approach

The overall recommended approach is identical to that for the Pacific Coast – revise the *G&S* using available references and information. There may be some limited use and application of primary frontal dune VE Zone identification and mapping criteria on the Pacific Coast.

Currently Available Information, Methods, and Guidelines

- The Massachusetts Coastal Zone Management (MA CZM) division has developed an improved methodology for automating the identification and mapping of the landward limits of the primary frontal dune VE Zone. This method is available and could be reviewed for potential use in other coastal areas.

- Existing guidance on Coastal A Zones are not available, but other published material helps to establish the need and possible regulatory enforcement options of the Coastal A Zone.

Applicability of Pacific Coast Guidelines

The four main items for Phase 2 work on the Pacific Coast (see recommended approaches in the Hazard Zones Focused Study) are also applicable to the Atlantic and Gulf Coasts. The following items could be based largely on Phase 2 work for the Pacific Coast, with revisions to extend their applicability to the Atlantic and Gulf coasts:

- Establish improved procedures for establishing the landward limit of the PFD, and develop guidance to better map the BFE transition between PFD-dominated VE Zones and landward SFHA hazard zones.

- Establish procedures (hazard identification and mapping) to better utilize VO Zones for severe wave overtopping areas where VE Zones have limited use and application.
- Establish procedures for identifying and mapping hazard zones for wave overtopping and wave-cast debris hazards, primarily a concern in the Northeast Atlantic region.
- Review the 1982 FEMA (Tetra Tech) or revised/new guidance on how to conduct the technical assessment and mapping of combined coastal-riverine areas for adoption into the *G&S* .

Recommended Future Development

- Provide further technical guidance in the *G&S* to clarify the PFD mapping criteria.
- Consider adoption of new quantitative methodologies for identification and mapping (e.g., MA CZM).
- Prepare technical bulletins for clarification of proposed revisions to VE Zones, AE Zones, and new criteria for VO Zones.
- Investigate and develop Coastal A Zone criteria (wave and erosion damage).
- Develop new Coastal A Zone guidance and apply new concepts in a case study area.

**Table 24**
**Hazard Zones Recommendations – Atlantic and Gulf Coasts**

| Topic Number | Topic/Subtopic | Recommended Approach (Future Work) |
|---|---|---|
| 39 | Primary Frontal Dune VE Zone | Prepare an improved and refined definition of the PFD slope transition as revision to NFIP regulations, and provide further technical guidance in *G&S* to clarify the PFD mapping criteria through a case study (e.g., Lewes, DE)<br>Consider adoption of quantitative methodologies and procedure for identification and mapping of the PFD landward limit (heel) slope criteria (e.g., MA CZM use of LIDAR and GIS automated methods) |
| 18 | Coastal A Zone Hazard Zone | Investigate and develop Coastal A Zone criteria (wave and erosion damage) and procedures for application within the NFIP;<br>Develop an annotated bibliography of related research and papers to support new guidance for Coastal A Zones;<br>Apply new concepts in a case study area. |
| 18 | Hazard Zone Technical Bulletins | Prepare technical bulletins for clarification of proposed revisions to VE Zones, AE Zones, and new VO Zones related to hazard identification, Special Flood Hazard Mapping and floodplain management. |
| 19 | Combined Coastal-Riverine Zones | Develop mapping standards to clearly identify this hazard zone. Develop alternate methods for identification of hazard zone. |

## 5.5 SUMMARY OF RECOMMENDATIONS – ATLANTIC AND GULF COASTS

For ease of reference, all of the topics and all of the categories have been combined in the following table.

**Table 25**
**SUMMARY OF ATLANTIC AND GULF COAST RECOMMENDATIONS**

| Topic Number | Topic/Subtopic | Recommended Approach (Future Work) |
|---|---|---|
| | | STORM METEOROLOGY RECOMMENDATIONS – ATLANTIC AND GULF COASTS |
| 51 | Tide and Surge Combination | Develop guidelines for the combination of surge and tide, including examples drawn from past studies (with consideration of FEMA surge studies, ADCIRC/EST, and the FL-DEP Monte Carlo method) |
| 51 | Surge/Riverine Combination | Prepare recommendations for the statistical combination of surge and a riverine runoff profile, with consideration of non-independence of the processes; see also Topic 19 of the Hazard Mapping Focused Study for simple mapping suggestions |
| 50 | Storm Surge Frequency Analysis | Apply/Compare methodologies (JPM, EST, Monte Carlo) using a common hydrodynamic model and storm data set |
| 50 | Storm Parameters for Surge Modeling | Review and evaluate available sources of storm parameters used in storm surge modeling, including NWS 38, HURDAT, and other databases |
| 50 | Storm Wind Fields | Review best available data regarding wind fields and compare with fields used in storm surge models; recommend the most appropriate models for FIS use (tropical storms, northeasters) |
| 50 | Wind Stress Formulation | Review best available data for wind stress and compare with formulations used in storm surge models; recommend the most appropriate formulation for FIS use |
| | | **STILLWATER RECOMMENDATIONS – ATLANTIC AND GULF COASTS** |
| 53 | General Considerations for Surge Modeling | Based on the existing literature, describe the use of surge models applicable to Atlantic and Gulf Coasts and the factors that require consideration in performing a study. |
| 53 | Surge Modeling Global Calibration | Develop statistical procedures to assess the performance of the FEMA surge models through the consideration of global experience on all coasts. |
| 53 | Regional Surge Modeling | Develop guidance for large scale regional surge modeling. |
| | | **STORM WAVE CHARACTERISTICS RECOMMENDATIONS – ATLANTIC AND GULF COASTS** |
| 4, 5 | Sea and Swell for Open Atlantic and Gulf Coasts | Investigate the appropriateness of using either the 100-year significant wave height or the 20-year maximum wave height while modeling WHAFIS. Clarify use of equivalent deep water wave condition. Clarify extrapolation to 100-year |
| 5 | Wave Generation in Sheltered Water | Develop Guidelines on Sheltered Water based on Pacific Coast *G&S*. |
| 1 | Wave Definitions | Incorporate and refine the "Glossary of Coastal Terminology" directly from the USACE CEM.<br>Incorporate and refine the five listings of notations and parameters in the 1986 International Association for Hydraulic Research publication, "List of Sea State Parameters."<br>Provide specific guidance on how wave related terms in the USACE and IAHR sources relate to each other and how they should be applied relative to the following: (1) FEMA guidance for coastal flood studies, (2) physical processes |

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| | | that are directly associated with FEMA coastal hazard assessments and flood mapping, and (3) required coastal hazard study methodologies<br>Prepare an application for Atlantic and Gulf Coast Guidelines |
| **WAVE TRANSFORMATION RECOMMENDATIONS – ATLANTIC AND GULF COASTS** | | |
| 9 | Wave Energy Dissipation over Shallow Flat Bottoms | Write *G&S* to include a section on wave energy dissipation over shallow and flat bottoms;<br>Develop typical ranges for dissipation coefficients for variety of bed and wave conditions to include in the *G&S*.<br>Categorize bed and wave conditions for US coastlines. Revise *G&S* to provide dissipation coefficients on a geographic basis; revise *G&S* to adopt Suhayda (1984) method. |
| 10 | Overland Wave Propagation, Candidate Improvements to WHAFIS | Evaluate new methods to better represent vegetation effects, treatment of elevated pile supported buildings<br>Minor Effort – WHAFIS code changes for more user friendly program<br>Moderate Effort – more intense code changes for improvement in accuracy and graphics (in WHAFIS)<br>Significant Effort - Revise WHAFIS to consider combined effects of damping and wind action over each segment. |
| 8 | Overall Wave Transformation with and without Regional Models | Cross reference Pacific Coast guidelines, and emulate important topics for Atlantic and Gulf Coasts. |
| **WAVE SETUP RECOMMENDATIONS – ATLANTIC AND GULF COASTS** | | |
| 44 | A&G Coast Definitions | Develop wave setup definitions with emphasis on A&G Coast applications. |
| 45 | Compile Data for Testing | Locate as much quality field data as possible for testing of developed/selected approach(es). |
| 46 | Develop Engineering Based Approach | Couple accepted engineering models for calculating wave setup across surf zone. Include procedure for dynamic wave setup. |
| 46 | Evaluate Boussinesq Models | Intercompare at least three Boussinesq models and compare with data. |
| 46 | Develop Breaking Zone Model | Evaluate candidate breaking zone models that allow specification of non-planar profile |
| 47 | Ideal Model for Static Wave Setup | Couple wave generation and wave setup model, allowing specification of arbitrary tide. |
| 48 | Develop Model for Dynamic Wave Setup | Develop method based on directional and nonlinear spectrum as input. |
| **WAVE RUNUP AND OVERTOPPING RECOMMENDATIONS – ATLANTIC AND GULF COASTS** | | |
| No Topic number assigned. | Revise Guidance to Reflect Current FEMA Practice | Revise guidance to describe use of ACES for runup and overtopping calculations (ACES is based on more recent procedures than SPM or RUNUP 2.0).<br>Revise guidance to clarify use of *equivalent* deepwater wave conditions with RUNUP 2.0 |
| 12 | RUNUP 1.0 vs. 2.0 | Perform detailed comparisons of wave runup using RUNUP 1.0 and 2.0. Determine whether to adjust prior RUNUP 1.0 studies or to restudy using RUNUP 2.0 (or other methods). |

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| 12 | Evaluate Use of Mean Runup Value | Review runup distributions and damages for Atlantic/Gulf beaches and structures, compare against Pacific.<br>Evaluate use of $R_{50\%}$ and select alternate $R_{x\%}$ value (probably between $R_{33\%}$ and $R_{10\%}$) if $R_{50\%}$ understates observed hazard. |
| No topic number assigned. | Wave Setup Component | Treatment of wave setup component (in FEMA's current wave runup procedure) to be coordinated with Wave Setup study. |
| 11, 49 | Conduct Comparative and Sensitivity Testing of Runup Models and Methods | Compare results using simple methods versus numerical models, deterministic (event selection) versus statistical approaches.<br>Test runup methods and models – priority to be given to testing in New England region.<br>Identify appropriate runup methods and models by location, morphology and hydraulic conditions |
| 13, 14 | Guidance for Overtopping and Wave Cost Debris | Maintain use of mean overtopping rate (cfs/ft, $m^3$/ per m)<br>Evaluate recent data and methods<br>Apply Pacific results relative to damaging overtopping rates and FEMA hazard zone thresholds<br>Evaluate wave-cast debris coincidence with overtopping<br>Coordinate with Hazard Zone study |
| **EVENT BASED EROSION RECOMMENDATIONS – ATLANTIC AND GULF COASTS** | | |
| 33, 34 | Gravel, Cobble and Shingle Beach & Dune Erosion | Review available literature and reporting; improved *G&S* language and descriptions for Atlantic and Gulf Coasts to distinguish gravel, cobble, and shingle beach and dune erosion from other processes; provide figures, and examples.<br>(1) Perform case studies to test and develop new geometric methods for cobble beaches, (2) Test process based methods, (3) Develop new *G&S*. |
| 35, 36 | *G&S* in Sheltered Water Areas | Improve *G&S* with definitions and discussion of characteristics of sheltered water areas and the types of morphology, material types and wave characteristics unique to sheltered water areas. Recommend interim *G&S* based on historical beach profiles and field observations.<br>(1) Conduct pilot studies, (2) Test process-based methods, (3) Develop new *G&S* for sheltered water areas |
| 31, 32 | Bluff and cliff erosion | Review available literature and reporting; improve *G&S* language and descriptions for Atlantic and Gulf Coasts to distinguish bluff & cliff erosion from other processes; provide figures and examples.<br>(1) Review existing bluff erosion procedures and international literature, (2) Develop geometric procedures for bluff and cliff erosion and retreat, (3) Consider development and use of process-based numerical/statistical modeling methods for future inclusion in the NFIP program. |
| 41 | Long-term erosion | This topic is considered important to NFIP, but FEMA action on previous work is pending. Therefore, guidance is best developed by FEMA in the future. |
| 42, 43 | Nourished Beaches | Recommend modifying *G&S* to direct Study Contractors to follow a procedure to notify FEMA that the study area includes beach nourishment project. Provide FEMA with a list of information needed to assess special cases where beach nourishment may be considered in determining hazard zones and BFEs (exception to existing FEMA policy).<br>Conduct research and case studies to determine whether beach nourishment is likely to have an effect on hazard zone designations of BFEs. |

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| 37 | Clarify Applicability and Limitations of 540 Criterion | Clarify limitations of 540 Criterion regarding its application to different types of coastal settings and material types. Discuss limitations of geometric methods versus process-based methods.<br>For the 540 Criterion: (1) Expand data base, (2) Define erosion area-frequency relationship, (3) Review use of median value trigger for dune removal. |
| 38 | Physics and Process Based Methods | Describe differences and advantages between "geometric" and "process-based" EBE methods. Interim methods: continue to use 540 Criterion for Atlantic and Gulf Coasts where applicable; use most documented post-storm beach and dune profiles for areas where 540 is not applicable.<br>(1) Further develop and test process-based models; (2) Develop method to include randomness of storm wave heights and tides and their coincident occurrence; (3) Develop and test Process-Based methods and prepare *G&S* for Process-Based erosion assessment of (a) coastal bluffs fronted by narrow beaches and (b) sandy and non-sandy beaches and dunes, including dune overwash. |
| 40 | Document Vertical Depths of Erosion | Document depths of erosion following storm events and maintain data for depths of erosion and damages to buildings in order to better determine "depth-damage" relationships. |
| **COASTAL STRUCTURES RECOMMENDATIONS – ATLANTIC AND GULF COASTS** | | |
| 26 | Jetties, Groins, Breakwaters | Develop criteria/guidance for evaluating failure of other structure types, and the effects of these failures on mapped flood hazards |
| 26 | Minimum Structure Dimensions | Determine minimum structure dimensions necessary to receive mapping credit during FIS and revisions to FIRMs |
| 27 | Structure Evaluation Criteria | Review CERC TR 89-15 considering more recent data on structure stability and failure; revise structure evaluation criteria. |
| **SHELTERED WATERS RECOMMENDATIONS – ATLANTIC AND GULF COASTS** | | |
| 6a | Definitions and Classification | Provide definitions, examples, and develop a classification method for sheltered water studies. |
| 6b | Flood Event Reconstruction | Prepare general guidance for documenting and using high water marks to reconstruct historic flood conditions. |
| 6d | Combined Tidal-Riverine 1% Annual Chance Event Assessment | Prepare guidance specific to defining the 1% annual chance flood event involving riverine and tidal flooding and expand guidance on wind data acquisition and analysis and fetch-limited wave forecasting. |
| 6e | Stillwater Estimation | Prepare guidance for estimating stillwater elevations in ungaged sheltered water bodies and evaluating the effects of tidal and riverine currents. |
| 6h | Hazard Mitigation Coordination | Prepare general guidance for Mapping Partners to coordinate the preparation of coastal studies with other hazard mitigation activities. |
| 6h | Focused Study Coordination | Collaborate/coordinate with other Focused Study groups to address sheltered waters Critical topics found in other Focused Studies. |
| **HAZARD ZONES RECOMMENDATIONS – ATLANTIC AND GULF COASTS** | | |
| 39 | Primary Frontal Dune VE Zone | Prepare an improved and refined definition of the PFD slope transition as revision to NFIP regulations, and provide further technical guidance in *G&S* to clarify the PFD mapping criteria through a case study (e.g., Lewes, DE)<br>Consider adoption of quantitative methodologies and procedure for identification and mapping of the PFD landward limit (heel) slope criteria (e.g., MA CZM use of LIDAR and GIS automated methods) |

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| 18 | Coastal A Zone Hazard Zone | Investigate and develop Coastal A Zone criteria (wave and erosion damage) and procedures for application within the NFIP;<br>Develop an annotated bibliography of related research and papers to support new guidance for Coastal A Zones;<br>Apply new concepts in a case study area. |
| 18 | Hazard Zone Technical Bulletins | Prepare technical bulletins for clarification of proposed revisions to VE Zones, AE Zones, and new VO Zones related to hazard identification, Special Flood Hazard Mapping and floodplain management. |
| 19 | Combined Coastal-Riverine Zones | Develop mapping standards to clearly identify this hazard zone. Develop alternate methods for identification of hazard zone. |

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# FEMA Coastal Flood Hazard Analysis and Mapping

## Phase 1 Summary Report
## Appendix

**February 2005**

Prepared for:

# FEMA

A Joint Project by
FEMA Region IX, FEMA Region X, FEMA Headquarters

**FEMA Study Contractor:**

northwest hydraulic consultants, inc.

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**Of**
# APPENDIX

# Introduction to the Phase 1 Focused Study Reports

This Appendix to the *Phase 1 Summary Report* (**nhc**, February 2005) contains eleven (11) Focused Study Reports prepared by the Technical Working Group (TWG) on eleven categories of technical topics pertaining to FEMA Coastal Flood Hazard Assessment and Mapping Guidelines. Goals of the Phase 1 investigations were to evaluate existing FEMA Guidelines for all three coasts and to examine the key technical areas of the current coastal flood hazard mapping process. Initial tasks focused on a review of the existing guidelines and the needs and priorities for their improvement. Under these tasks, coastal experts from the TWG reviewed existing guideline methodologies for the ocean and coastal processes analyzed in flood insurance studies (e.g., storm meteorology, storm surge, wave setup, wave transformation, wave runup, and overtopping) and evaluated their applicability for each coastline. Case studies were prepared to demonstrate application of guideline methodologies in previous coastal flood insurance studies on each coast, and representative studies were prepared to demonstrate application of guideline procedures to particular coastal processes.

An international literature search was conducted to identify sources of information on existing and evolving coastal engineering practices and to identify pertinent scientific research that may be useful in developing new guidelines. The international experience of several TWG members was used during this task to provide the project with information, techniques, and practices from around the world.

The initial tasks described above served as the basis for reporting and discussion at Workshop 1, held in Sacramento, California, on December 2–4, 2003. The workshop was attended by 38 members of the TWG from across the country and Europe. The workshop agenda included:

- review of existing guidelines and practices;
- technical presentations on the state of the science in coastal processes;
- workshop sessions to identify needs, priorities, and potential guideline improvements by coastal geographic areas and coastal processes; and
- Summary sessions to list and prioritize needed guideline improvements.

The primary result of Workshop 1 was a list of 53 technical topics for consideration in updating the current FEMA guidelines. Each item also included an initial assessment of the time and data required to develop improved procedures. This assessment resulted in categorizing each topic as "Critical," "Important," "Available," or "Helpful." "Critical" and "Important" topics were considered the highest priorities for development of new or improved procedures, and were subdivided into topics that could likely be addressed in the 6-month time frame of the project ("Critical") and those that would require longer term development by FEMA ("Important").

"Available" topics were considered areas where existing data or methodologies were readily available for updating or creating guidelines. "Helpful" topics were considered valuable but lower priority. These priority classes were assigned by the TWG for each topic on the Atlantic and Gulf Coasts, Pacific Coast, and in Sheltered Waters (Non-Open Coast).

Results from Workshop 1 were used to formulate *focused studies* that organized the 53 technical topics into 11 categories according to coastal processes and coastal flood hazard mapping procedures. Each of these 11 categories became the subject of a *focused study* and resulted in a stand-a-lone report, including topics on: (1) Storm Meteorology, (2) Stillwater Elevations, (3) Wave Characteristics, (4) Wave Transformation, (5) Wave Setup, (6) Event-Based Erosion, (7) Wave Runup and Overtopping, (8) Coastal Structures, (9) Sheltered Waters, (10) Tsunamis, and (11) Hazard Zones. These eleven Focused Study Reports are included in this Appendix to the Phase 1 Report.

The *focused studies* were conducted by groups of individuals from the TWG, each coordinated by a focused study leader. This organization allowed the 11 *focused studies* to be completed simultaneously and rapidly. Preliminary drafts of the *focused studies* were presented at Workshop 2 on February 23–26, 2004, and subsequently were refined by the study groups and submitted to FEMA in May 2004. These initial drafts of the Phase 1 Summary Report and Focused Study Reports were revised into Final Drafts that were submitted to FEMA in June 2004. Focused Study leaders responded to FEMA review comments, made revisions to the reporting and prepared the Final Phase 1 Summary Report and this Appendix containing the Focused Study Reports.

The *focused studies* contain recommendations on the approach for updating the guidelines on three coasts (Pacific, Atlantic, and Gulf). These recommendations include further studies and guideline development work that vary in complexity, level of effort, and time requirements. The level of effort required to complete the recommendations for "Critical" and "Available" items identified in Workshop 2 significantly exceeded the available time and budget for Phase 2 (development of Pacific Coast guidelines). Therefore, in March 2004 the project team engaged in a significant effort to develop options for limiting the scope and cost of the next phase of work (Phase 2 – development of Draft Pacific Coast Guidelines) while retaining the most important topics and a balance among the 11 technical categories. The selected option deferred some recommendations for future development in the National Flood Insurance Program (NFIP) but maintained the target of producing reliable guidelines for coastal studies on the Pacific Coast in FY 2004/2005.

## SUMMARY OF KEY FINDINGS FROM THE PHASE 1 FOCUSED STUDIES

A complete list of topics and recommendations developed by the TWG during Workshops 1 and 2 is provided in Table 2 of the Phase 1 Summary Report. Following are a few of the key findings from the Phase 1 activities and the completion of the eleven *Focused Studies*:

- Procedures are needed to compute the 1% annual chance flood elevation where 1% stillwater levels do not necessarily coincide with 1% wave conditions (e.g., Pacific Coast and sheltered waters along all three coasts).
- Procedures to better represent wave setup are needed on all coasts.
- Procedures should be developed to use regional databases and wave transformation models to develop wave spectra at the surf zone.
- Methods are needed to evaluate the amount of wave dissipation due to propagation over muddy or flat nearshore areas.
- Procedures to quantify the effects of wave setup and event-based erosion in a variety of geomorphic settings are needed.
- On the Atlantic Coast, a review of the 540 square feet erosion criterion is needed in light of new data; on the Pacific Coast, a similar geometric method is needed based on Pacific Coast data.
- A probabilistic method for tsunami hazard assessment and methods for combining tsunami hazards with other coastal hazards are needed.
- Updates and amplification of existing guidelines for wave runup and overtopping and associated hazard zones are needed. Improved methodology for wave overwash is needed.
- Some coastal processes, such as surge, wave transformation, and tsunamis, are best analyzed at a regional scale rather than in flood studies of individual communities.
- Sheltered waters (non-open coast areas) require specialized guidance because of their unique hydrodynamic and geomorphic characteristics compared to the open coast. For example, new methods for calculating fetch-limited wind waves should be evaluated and incorporated in guidelines, to the extent appropriate.

Recommended approaches to address these and other needs are included in Sections 4 and 5 of the February 2005 Phase 1 Summary Report.

Following are Acknowledgements for those who participated on the Technical Working Group and a listing of selected Key References from each Focused Study Report. Following the Acknowledgements and Key References are the eleven Focused Study Reports discussed in the February 2005 Phase 1 Summary Report.

# ACKNOWLEDGEMENTS OF FOCUSED STUDY TEAM MEMBERS

The following individuals are gratefully acknowledged for their contributions and participation as members of the Technical Working Groups and for the key roles they played as participants and writers on one or more of the eleven Focused Study Teams. These individuals performed the focused studies, participated in technical workshops, and prepared this report.

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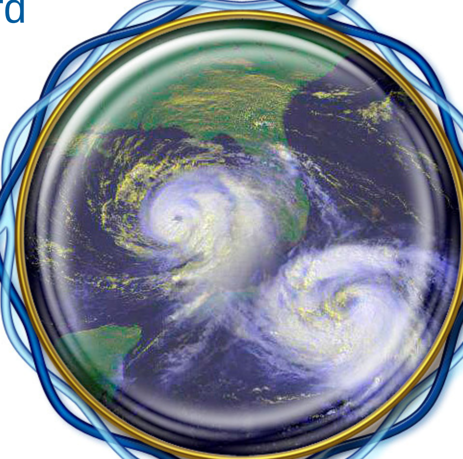
# Storm Meteorology

## FEMA Coastal Flood Hazard Analysis and Mapping Guidelines Focused Study Report

February 2005

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## Figures



## Acronyms

| | |
|---|---|
| 1-D | one-dimensional |
| 2-D | two-dimensional |
| ACES | he significant wave; the significant wave can be determined using the Shore Protection Manual or Automated Coastal Engineering System |
| DEP | State of Florida, Department of Environmental Protection |
| EST | Empirical Simulation Technique |
| FEMA | Federal Emergency Management Agency |
| FIS | Flood Insurance Study |
| FNWC | Fleet Numerical Weather Central |
| HRS | Hydraulic Research Station |
| HURDAT | digital file of storm data for all identified tropical storms in the North Atlantic |
| JPM | Joint Probability Method |
| MSL | Mean Seal Level |
| NGVD | National Geodetic Vertical Datum |
| NOAA | National Oceanic and Atmospheric Administration |
| NOS | National Ocean Survey |
| PDF | probability distribution function |
| PWA | Philip Williams and Associates |
| RT | Return Time |
| WIS | Wave Information Studies |
| WL | water level |

# 1 INTRODUCTION

## 1.1 CATEGORY AND TOPICS

This Focused Study describes a proposed approach for the development of new FEMA Guidelines for two topics:

- The first is the determination of storm meteorology (storm statistics) used in coastal storm surge flood studies.
- The second is the formulation of guidance for estimation of the 100-year flood when two or more flood-forcing mechanisms are important.

The particular topics addressed in this report were identified during Workshop 1 of the project and are described below.

**Storm Meteorology Topics and Priorities**

| Topic Number | Topic | Topic Description | Priority | | |
|---|---|---|---|---|---|
| | | | Atlantic / Gulf Coast | Pacific Coast | Non-Open Coast |
| 50 | Modeling Procedures | Review and recommend storm surge statistical procedures (JPM, EST, Monte Carlo), and identify data sets for hurricanes, nor'easters, and Pacific storms | I | I | -- |
| 51 | Combined Probability, Determinations of 1% Flood Elevations | Develop guidance on combined probability considerations for all processes; define procedure to determine the 100-year flood event | C | C | C |
| Key: C = critical; A = available; I = important; H = helpful | | | | | |

Topic 50 is construed to be an effort to develop guidance regarding the statistical aspects of storm surge modeling, outlining procedures and data sources that are needed to implement procedures such as the Empirical Simulation Technique, Monte Carlo methods, and the Joint Probability Method. These methods are used to attach rates of occurrence to particular storms used in the hydrodynamic simulations, and to derive rates of occurrence of the resulting flood levels. The Joint Probability Method (JPM) has been used generally in past FEMA coastal surge studies, whereas the newer Empirical Simulation Technique (EST) has recently been approved by FEMA and is now coming into use.

Topic 51 is a more general task, extending to all mechanisms of coastal flooding, not just surge, but including, for example, astronomic tide, storm waves, and tsunamis, as well as the combined probability of coastal and riverine elevations in tidal waters. The goal is, given two or more such mechanisms affecting a site, to determine what the 1-percent-annual flood elevation is, as a function of the statistics of the several contributing mechanisms.

### 1.2. STORM METEOROLOGY FOCUSED STUDY GROUP

The Storm Meteorology Focused Study Group was made up of Robert Battalio, Ian Collins, Robert Dean, Darryl Hatheway, Norm Scheffner, and David Divoky, who served as Team Leader.

# 2 CRITICAL TOPICS

## 2.1 TOPIC 51: COMBINED PROBABILITY (FOR ALL GEOGRAPHIC REGIONS)

### 2.1.1 Description of the Topic and Suggested Improvement

The problem addressed in this topic is the determination of the total 1% flood elevation at a particular site that may be affected by multiple flood elevation processes or by processes with multiple components. High water levels accompanied by flooding may be the result of extreme astronomical tide; storm-induced tide; tsunamis; wave setup, runup, and overtopping; or riverine rainfall runoff (in estuaries). These may be affected by seasonal effects (El Niño conditions) and additional long-term factors such as changes in relative sea level (for example).

The goal is to provide guidance for determining the 1% flood event in such cases. Clearly, the total level reached during an extreme flood may be the result of a combination of many influences, each having its own associated probability or rate of occurrence. (Note: Strictly speaking, what we will loosely call "probability" is actually rate of occurrence measured in units of events per year; the 1% flood is the level occurring at an average rate of 0.01 times per year. This distinction between mathematical probability and temporal rate is occasionally quite important.)

The contributing events may be statistically independent or may be correlated in some manner. Furthermore, two (or more) events that do not occur together must still be statistically combined because the total rate of occurrence of a given flood height is influenced by both. Methods to handle the several possible combinations need to be summarized and guidelines developed.

### 2.1.2 Description of Procedures in the Existing Guidelines

***Joint Probability Method***

This Combined Probability topic does not address the combination of two or more processes, but is concerned with the total rate of occurrence of a storm defined by multiple parameters with individual probabilities. The Atlantic and Gulf Coast Guidelines suggest using the approach that was originally developed by the National Oceanic and Atmospheric Administration (NOAA), in which the governing hurricane parameters (i.e., central pressure index, radius to maximum winds, forward speed, and direction of travel) are examined for statistical independence and then the probabilities multiplied to derive the probability of occurrence of a particular storm. The required hurricane data are taken from *Hurricane Climatology for the Atlantic and Gulf Coasts of the United States* (National Weather Service, 1987). This item is discussed below, under Topic 50.

### *Tide and Surge*

The Study Contractor is required to "Describe the method by which the tidal elevation data are convoluted with the surge data including tidal constants and tidal records." Refer to Benjamin and Cornell (1970) for the definition and use of "convolution integrals" in probability and statistics.

### *Storm Waves and Surge*

The Study Contractor must use the "controlling" wave, defined in Appendix D, in Section D.2.2.6, as 1.6 times the significant wave; the significant wave can be determined using the Shore Protection Manual or Automated Coastal Engineering System (ACES). The waves are assumed to be coincident with the peak surge. There is little other specific (explicit) guidance for this topic in the current FEMA coastal guidelines. In many places, the guidelines refer to the need to choose a factor (deepwater wave height, for example) that somehow corresponds to another process with which it is to be combined (the 1% stillwater level, for example). It is generally not clear from the guidelines how this is to be done, and the matter is left to the judgment of the Study Contractor, along with the injunction that the assumptions be documented. Section D.2.2.6 of the Guidelines, for example, refers to "the meteorology of storms expected to provide approximate realizations of the 1-percent-annual-chance flood" and suggests that such storms would be useful in "assessing wave characteristics likely associated with" that flood. Subsequently, it is suggested that "the 1-percent-annual-chance flood is likely associated with central pressure deficits having exceedance probabilities between 5 and 10 percent," with the implication being that wave height and period estimated from hurricane formulas using pressures in this range would be appropriate (radius to maximum winds and forward speed are not mentioned, although median values might be assumed).

Similarly, there is no guidance regarding the combined probability of separate processes, such as storm surge and rainfall runoff in a tidal river.

### *Pacific Coast*

There are no guidelines for the Pacific Coast.

### 2.1.3 Application of Existing Guidelines to Topic—History and/or Implications for the NFIP

#### *Joint Probability for Hurricane Parameters*

For hurricane flood studies on the East and Gulf Coasts, the original use of the JPM was proposed and developed by NOAA. The approach involves an assumption of independence of storm parameters so that the combined probability of a particular hurricane is the product of the probabilities of each of the governing parameters (i.e., forward speed, storm radius, central pressure depression, and storm position; a dependence on track angle is assumed and accounted for by separation of the storms into directional families). In the early studies by Tetra Tech, this assumption of statistical independence was investigated quite thoroughly by examining cross correlations and factor analysis for a multivariate sample. This aspect of combined probability is considered below as part of Topic 50.

#### *Superposition of Surge and Waves*

For applications that require determination of a wave estimate for superposition (through setup, runup, and overtopping) on a 1% stillwater surge level, two approaches have commonly been used. One approach has been to estimate a 1% deepwater wave condition from WIS data or other similar wave data. The second common approach has been to adopt a design-like storm, such as a storm with the 5–10% pressure deficit, and use this for computations based on hurricane wave formulas.

For the combination of overland wave propagation and surge (WHAFIS), the greatly simplifying assumption is made that depth-limited breaking waves occur at the shoreline during 100-year surge conditions (with an appropriate period), so that there is no need to attach a return period to wave height. The initial wave represents waves of all heights above the minimum necessary to produce breaking conditions.

#### *Combination of Surge and Riverine Flood Profiles*

In past studies, the combined probability of riverine runoff and coastal surge in tidal areas has been treated inconsistently by Study Contractors, including federal agencies. The correct treatment, if independence of the runoff and surge episodes at the mouth of a tidal river is assumed, is to simply add the rates of occurrence of specified flood elevations from each source, at several locations along the affected river reach. There is a great deal of inconsistency among existing studies in this common instance of combined probability, with many studies simply mapping the greater of the two levels (so that the level at the intersection of the two 100-year profiles would actually correspond to the 50-year level). As discussed below, however, the case of non-independence should also be considered. This issue is also discussed in Topic 19 of the Hazard Zone Mapping Focused Study.

### Superposition of Tides and Tsunamis

The combination of tides and tsunamis is not specifically addressed in the Guidelines but has been considered in past FEMA studies, and so is included in this section. In the case where the total water level is the sum of two independent processes that combine in a linear manner, the probability of the expected sum is found by convolution. That is, if the probability density of the tide level is denoted by $p_a(Z)$ and the probability density of the tsunami water level is $p_t(Z)$, then the probability density of the sum of the two is given by:

$$p(Z) = \int_{-\infty}^{\infty} p_a(T)\, p_t(Z-T)\, dT \qquad (1)$$

The process is easily extended to the sum of three or more independent variables that add together. In the early Tetra Tech report (1982), the convolution theory was expanded to include cases where one component had a shorter duration than the other. In other words, it would include cases where, for example, the peak storm tide or tsunami could occur at tide levels other than the maximum.

In the limit it is known that, for a process that has a Gaussian probability density function and is narrow banded in frequencies, the envelope will have a Rayleigh distribution. The Tetra Tech report (1982) showed that, if the storm tide or tsunami lasts for the duration of a half tide cycle (i.e., including a high tide event), the resulting level would be the sum of the two and would tend to a Rayleigh probability distribution. In practice, this may be questionable because, at the extremes, the tidal water levels have an asymptotic limit, whereas the Gaussian and Rayleigh functions are unbounded.

As an example, the "modified Rayleigh" distribution can be written in the form:

$$P = \exp\left(-\left(\frac{H - H_O}{H_C - H_O}\right)^{\gamma}\right) \qquad (2)$$

where $H$ is the measured height, $H_O$ is the minimum that is reported, $H_C$ is a scaling factor, and $\gamma$ is 2.0. (Often, the denominator is replaced with a single "scaling factor" and is referred to as the three-parameter Weibull distribution function.) Figure 1 shows an example from the predicted tides at San Diego. The 1-percent-annual-event would have a probability level of about 1/70,600 or $1.4\text{x}10^{-5}$ (assuming 706 high tides per year). The Rayleigh distribution function clearly overestimates the maximum tide elevation at this probability level. An alternative fit is shown that has $\gamma$ = 5.5 and the values of $H_O$ and $H_C$ have been adjusted.

### Other Approximations

In the early FEMA studies for the Southern Atlantic and Gulf coasts, the combined probability of storm surge and tide was approximated assuming that half of the peak water levels of the storm surge occurred at high tide and half at low tide. This approximation yielded a shift of about half the maximum tide level. With the typical tidal ranges in the Southern Atlantic and Gulf coasts, this approximation yields distributions that are close to those arrived at by the more exact methods.

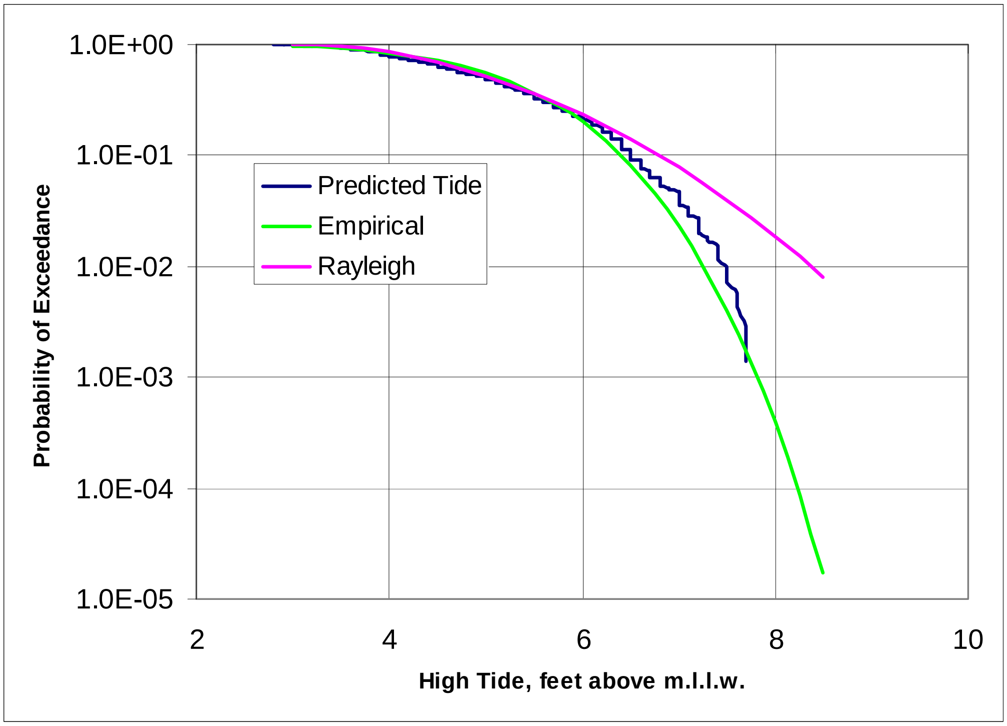



**Figure 1. Probability distribution of high tide elevation at San Diego Bay.**

In a recent study by Philip Williams and Associates (PWA, 2002), the potential for combined tides and waves was estimated by taking:

- the 1% water elevation with an average wave height,
- a 1% wave height with an average high tide (averaged from a set of water levels occurring at the time of high wind waves), and
- a third approximation as an intermediate case, calculated based on marginal joint probability.

The results (wave runup and overtopping) calculated for these three approximations were compared and the worst one was selected. This approach, discussed in more detail below, is very similar to what is recommended in this Focused Study, based on an extensive research effort by the Hydraulic Research Station at Wallingford, U.K.

### 2.1.4 Alternatives for Improvement

There are five major types of combinations of extreme water levels that result from the simultaneous occurrence of more than one event:

1. Two independent, simultaneous contributors that can be added in a linear manner (or nearly so)—In this case, the convolution process applies; the method can be extended to multiple independent contributors that contribute to the sum.

2. Two independent contributors that have major differences in their durations—In such cases, the modified convolution process developed by Tetra Tech may be appropriate, with the proviso that the integration limits must be confined to realistic bounds.

3. Two independently occurring contributors that interact with each other in a non-linear fashion—An example is wind setup in shallow water areas with large tidal ranges because, among other factors, the water response to wind stress is inversely proportional to the total water depth.

4. Two correlated or partially correlated processes—If the processes are completely correlated, these reduce to single events; if they are partially correlated, there are no simplified methods.

5. Three or more processes that are partially correlated—For example, tide level, wave height, wave period, and wave direction; wave overtopping is an example of a combination of processes of this type.

Astronomic tide occurs daily and everywhere, around the world. Therefore, methods must be developed to account for tidal effects in combination with everything else including storm surge, tsunamis, and all manner of wave effects, including wave crest elevation, runup, setup, and overtopping. In addition, the statistical combination of surge and riverine flooding must be accounted for.

#### *Sample Combination Methods*

The guideline methods to be developed must consider each significant combination of two or more factors chosen together. For illustration, consider two such combinations: surge plus tide and waves plus high water.

**Surge Plus Tide**

As mentioned earlier, one approximate method of combining surge plus tide that has been used in past studies. The method is based on the assumptions that it is equally likely for peak surge to occur at either high or low tide, and that the duration of the peak does not last long. For example, let S be a particular surge elevation computed from mid-tide, and let A be the tide amplitude around its mid-level. Then one simply reallocates the estimated rate (frequency) of occurrence of surge elevation S, assigning half of the total rate to elevation S+A and half to elevation S-A.

This reallocation of the probability-mass of the surge S (computed at mid-tide) to both higher and lower levels has the net effect of shifting the frequency curve slightly toward higher elevations. A more accurate statistical determination in the same spirit can be made using the convolution method. These statistical procedures, however, are not appropriate when the surge and tide interact physically, thus affecting each other's behavior.

An improvement over the linear approximation of surge plus tide assumes that the surge can occur at high tide, mid-tide ebb, low tide, and mid-tide flood. For high tide, the surge is simulated with tide; for the other three phases, it is assumed that the surge without tide can be added to the tide linearly. This approach helps identify maximum-interaction nonlinear effects. The assumption here is that combined-effect interactions are at their maximum level at high tide and are less important at mid-tide and low tide. This surge plus tide simulation procedure is generally used only for severe events for which the surge is significantly greater than high tide.

To account more fully for the interactions, more detailed hydrodynamic calculations are necessary. Two approaches will be described: the method adopted in FEMA's two-dimensional (2-D) storm surge model, and the approach used by the State of Florida Department of Environmental Protection (DEP).

**FEMA Method**

In the FEMA (1988) methodology, a large number of storms are simulated using the numerical surge model, and the computed water levels around the study area are recorded. These calculations are made with respect to the mean water level and do not account for tide. The large number of simulations is determined by taking all possible combinations of five parameters defining a storm: pressure depression, radius to maximum winds, storm forward speed, track angle, and track position. Tide could be included among these parameters and appropriately incorporated in the simulations through the boundary conditions. For example, both tide amplitude and tide phase could be taken as additional parameters, increasing the parameter set from five to seven types. If just a small number of values were chosen for each new tide parameter, say three values of amplitude and six values of phase, then the simulation costs would increase by a factor of 18. This was not an acceptable alternative when the surge methodology was developed, owing to the extremely high cost of computer time, although it might be considered acceptable today.

Instead, the FEMA methodology adopted a method by which simulations made around mean sea level (MSL), with no tide, are adjusted to approximate the levels that would be achieved with various tides. The first step is to perform a detailed simulation of a small set of storms, covering a range of peak surge elevations, with tide hydrodynamically added on the grid boundary; it is assumed that, in the offshore region, the tide will add linearly to the surge. For each storm, approximately 20–30 tide combinations are simulated, for a range of tide amplitudes and phases. These *combined* surge and tide simulations account for the interactions and provide the basis for the subsequent adjustment of the no-tide calculations.

The second step is to simulate each tide without surge, and then to linearly add the separately computed tide onto the tideless surge hydrographs computed for the small storm set. These *added* hydrographs will differ from the cases simulated with surge and tide *combined*. A simple regression expression is derived at each grid point, expressing the combined peaks as functions of the added peaks. Finally, these corrective expressions are applied to the very large data set computed without tide, to estimate what the surge would have been in each case for all of the selected tide conditions. Although this is a laborious procedure involving use of several intermediate utility programs, it is practical and far less costly than full 2-D simulations including tide.

**Florida DEP method**

The Florida DEP (see, for example, Dean et al., 1992) uses a different technique that is very simple in concept but relies partly on the use of a one-dimensional (1-D) surge model instead of a 2-D model.  The DEP procedure, which is described in more detail under Topics 54 and 55 of the Stillwater Focused Study Report, begins with simulations of selected storms using a detailed 2-D model over the entire study area. This is followed by simulations of the same storms over several transects using a simpler 1-D model (Freeman et al., 1957).  Of course, the 1-D model does not reproduce the 2-D results exactly, but—as with the FEMA approach relating added and combined tides described above—it is possible to perform both 2-D and 1-D simulations of a small set of storms and, from these, derive regression expressions relating the 2-D results and the 1-D results at all points along the transects. These expressions can then be used to adjust all subsequent 1-D calculations, thereby approximating 2-D calculations.

Using the very efficient 1-D model, the DEP procedure is to simulate a large number of storms with tide boundary conditions imposed at the seaward limits of the transects.  The tide condition chosen is an actual tide history selected at random from the hurricane season.  For example, if a storm is to be simulated for a total of three days, the procedure is to pick a starting time at random from within the hurricane season and use the following three days of tide predictions as the water elevation at the seaward boundary of the transect.

This procedure is repeated many hundreds or thousands of times, each time selecting a new storm (by Monte Carlo selection from the storm parameter cumulative distribution functions) and a new tide history segment for the 1-D boundary condition. In this way, the full range of possible tide conditions is automatically accounted for in a realistic, natural way because the physical interaction between surge and tide is implicit in the calculations.

**Waves plus High Water**

A second major problem is the choice of waves to be associated with high water.  The combination of waves and surge has been mentioned above with respect to overland propagation, for which the assumption of limit-height breaking at the shore eliminated all difficulty. However, in general the user is faced with the difficult problem of selecting a combination of wave and high water that will reasonably represent the1% total event. Three approaches are mentioned here.

**1) Southern California Flood Insurance Study**

The first approach, used by Tetra Tech in its 1982 flood insurance work for Southern California (Tetra Tech, 1982), was simply to consider all significant storms or wave and high water sources, and to follow their effects from source to shore, computing shoreline processes such as setup and runup. A limited number of extreme deepwater conditions were used.

The stillwater processes that were considered included astronomic tide and surge from local tropical cyclones. The wave sources included intense winter storms in the north Pacific; local storms; and tropical cyclones, both local and remote (off Baja California). Extratropical wave data were taken from Fleet Numerical Weather Central (FNWC) data for the period 1946–1974, summarized at three points off Southern California. These three points defined two connected offshore line segments along which wave heights, directions, and periods were specified by interpolation.

Wave rays were initiated along these lines, at very fine spacing, and were carried toward shore using a refraction algorithm developed specifically for the project to permit efficient handling of such large data sets. Wave setup and runup were determined for these waves, once they reached the coast, using practical engineering methods. The treatment of tropical cyclone wave generation and propagation was similarly straightforward. Locally generated storm surge was investigated by a numerical simulation of the 1939 storm, which was unusual for reaching as far as the Los Angeles area. It was found that the wind-driven component of high water from this storm was small compared to the inverse barometer contribution; consequently, the total surge component was simply approximated by the barometric component for each storm considered in the study.

Once the results from each factor, assumed to be independent, were determined, combinations with tide were determined by convolution calculations, and an extremal analysis was performed based on fitting multiple versions of the Gumbel and Weibull distributions. Correlations (joint probability) between high water levels and large swells (as related by El Niño conditions, for example) were not investigated.

**2) Sandy Point Study**

A second example is the more recent Sandy Point study referenced above (PWA, 2002). This study considered the joint probability of estuarine high stillwater levels and local wind waves. Stillwater levels were analyzed using tide gage data collected nearby, applying extreme-value analysis on the highest recorded tides and the largest residuals (residual equals the measured water level minus the predicted astronomic tide for the same time). The probability of high wind speed was used as a surrogate for the high seas generated by local winds (because the site was in sheltered waters, swell was not considered, but the approach is also applicable to swell). An attempt was made to define joint probability directly using a coincident time series of winds and water level data covering a period of 29 years. However, it was found that this period was not sufficiently long to define the low-probability events of interest. No attempt was made to fit the data to a bivariate extreme-value distribution and then estimate low-probability conditions by extrapolation, although this may be a viable alternative approach (see general discussion below). Consequently, an Averaging Method and a Marginal Probability Method were used to estimate the flood events, as described below. Three estimates of the 100-year probability event were made (Table 1), each of which consisted of a high stillwater level and high wind speed.

These events are shown graphically in Figure 2. The selected events are reasonably close to “rule of thumb” guidance used in other studies, as depicted in the figure (labeled “Standard Practice” in the legend).

**Table 1. Wind Speeds and Total Stillwater Level Conditions with Return Periods**

| Event | Label | Wind Speed | | | Stillwater Level | |
|---|---|---|---|---|---|---|
| | | km/h, 2-min avg | mph, 1-sec avg | RT (year) | ft NGVD | RT (year) |
| 100-year WL, average simultaneous wind | A | 69 | 57 | 6 | 8.14 | 100 |
| Intermediate case (marginal probability) | B | 110 | 90 | 37 | 6.75 | 2 |
| 100-year wind, average simultaneous WL | C | 139 | 114 | 100 | 5.87 | < 1 |

NGVD = National Geodetic Vertical Datum, RT = return time, WL = water level

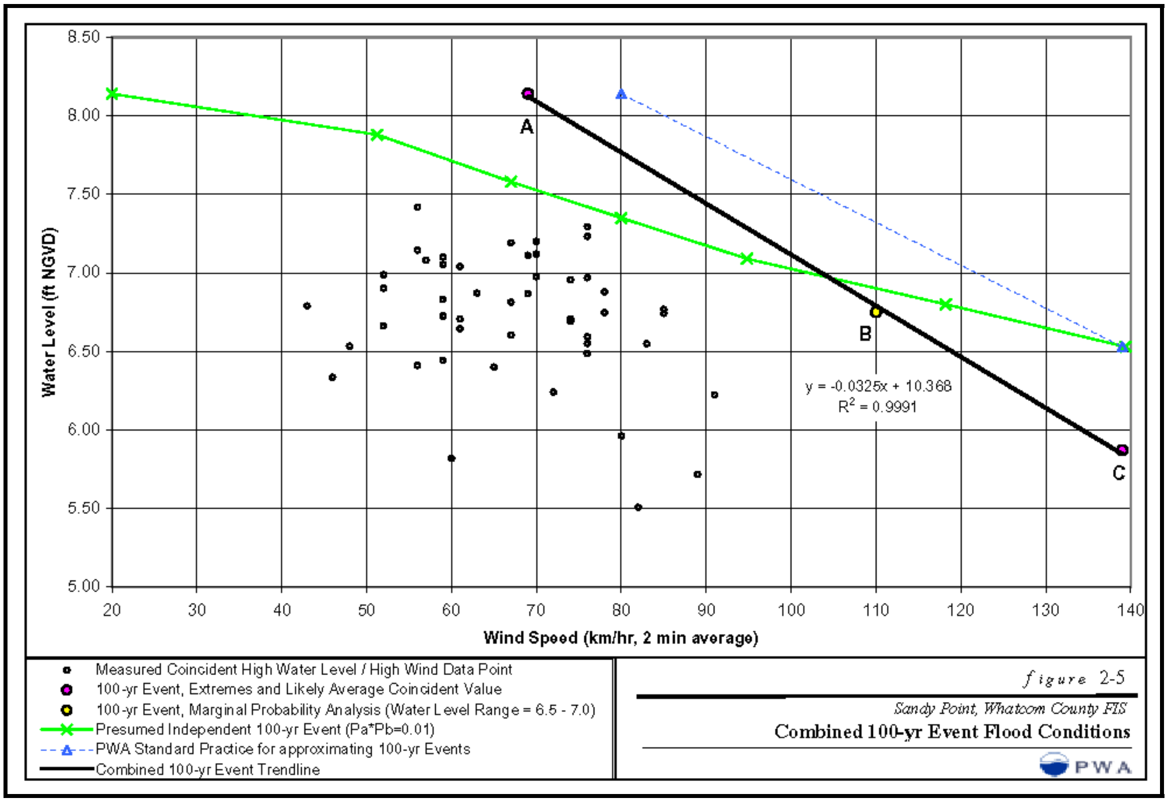



**Figure 2. Combined 100-year event flood conditions from the Sandy Point Study.**

***2a) Averaging Method***

The averaging method used standard extreme-value analysis to determine the 100-year event for one parameter. The other parameter was selected based on the average of values observed to occur at the same time as extreme values of the first parameter. Attempts to correlate wind speed with extreme high-water levels indicated no linear correlation with magnitude, but did indicate a narrow range of values from which an average wind speed could be selected as being likely to recur during a 100-year water level. Similarly, an average high-water level residual was selected as being likely to recur during a 100-year wind event. This resulted in two estimates of the 100-year event: a 100-year wind speed and a high water level of less than 1-year return period, and a 100-year water level and a wind speed with a 6-year return period.

***2b) Marginal Probability Method***

Besides the two estimates resulting from the averaging method, a third estimate used marginal probabilities. This estimate was arrived at by analyzing all wind speeds occurring coincidentally with water levels that fell within a certain range. A water level range from 6.5 to 7.0 ft NGVD was chosen because 13 measurements of extreme coincident wind and water level were within this range, allowing some confidence in the results. The median of this range corresponded to a two-year-return-period tide level (6.8 ft NGVD), based on the residual analysis (see below). An extremal analysis of wind speeds occurring coincidentally with water levels in this range was performed to determine the probability of wind speeds for a two-year-return-period tide. The rules of marginal probability defined the total probability as the quotient of the conditional probability and the probability for the condition to occur. In this case, the probability of the water level being within the 6.5–7.0 ft NGVD range was one time per two years (or P1 = 0.5). Therefore, the 50-year-return-period wind speed, which occurs one time per 50 years (P2 = 0.02), was determined so that the total probability would be equivalent to a 100-year event (P1 * P2 = 0.5 × 0.02 = 0.01). The selected wind speed had a 37-year return period based on the single-parameter return-period analysis conducted on the wind speed.

***2c) Residual Water Level Analysis—Event Selection Method***

Residuals were calculated by subtracting the predicted astronomic tide from the observed (recorded) water level at the tide gage for the 29-year period of record. Extreme-value analysis was applied to the residuals, allowing the residual values for different return periods to be estimated. An extreme-value analysis was also applied to the high tide data directly, providing an estimate of the high-water levels for different return periods. Subtracting the residuals from the extreme tide values for the same return periods provided an estimate of the astronomic tide likely to occur during the extreme event.

The above method was characterized as the Event Selection Method, which could involve various approaches. The implicit assumption was that the probability of coastal flooding caused by high wave runup or surge and overland wave propagation would be the same as the joint probability of occurrence of the environmental forcing parameters, namely water levels, winds, and waves. Comparison with observations of flooding during the Sandy Point study indicated that this approach may have underestimated the probability of flooding and that other approaches estimating the extreme value of the flood event directly, such as runup and overtopping, might have provided better estimates.

**3) Wallingford JOIN-SEA Method**

A series of directly pertinent reports have been prepared by the Hydraulic Research Station (HRS) at Wallingford, U.K., and the University of Lancaster, U.K. In these reports the joint occurrences of astronomical tide, storm tide, and waves were assessed to determine the risks, at different levels, of overtopping of seawalls; the quantity of overtopping; and other potential structural responses.

The method proceeded with the following steps:

1. Preparation of input data, consisting of many independent records (or hindcasts) of wave heights, wave periods, and water levels.
2. Fitting of statistical distributions separately to the wave heights, water levels, and wave steepnesses.

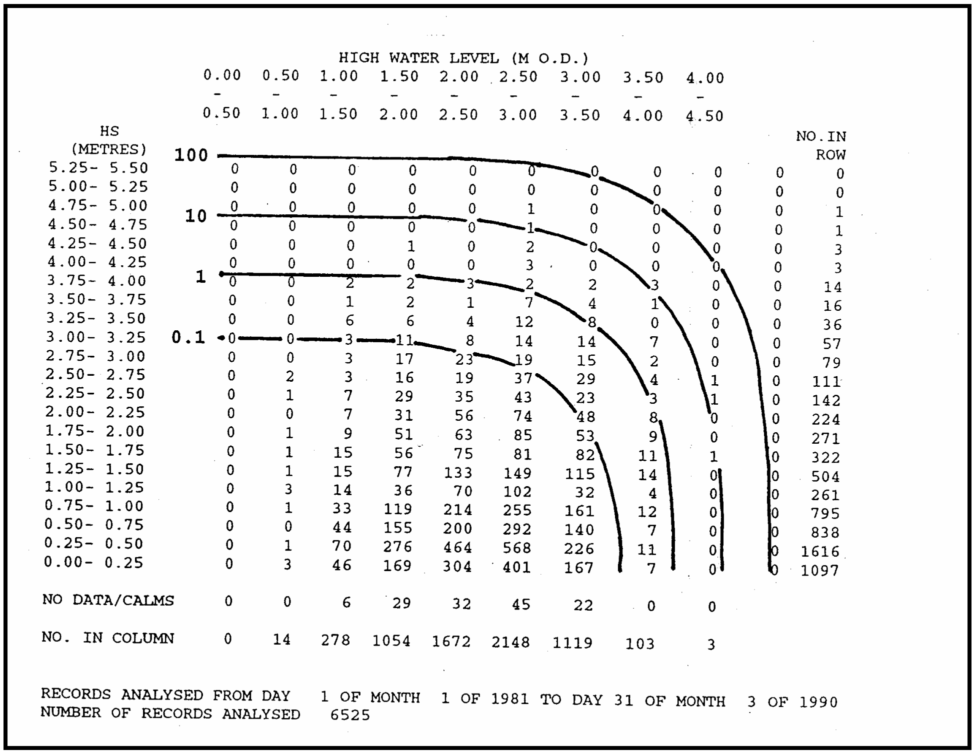
3. Fitting of the dependence between wave heights and water levels, and between wave heights and steepnesses.
4. Statistical simulation of a large sample of data on wave height, wave period, and water level using the fitted distributions and Monte Carlo simulations.
5. Extremal analysis of the range of response variables based on the simulated data.

The methods in the reports were supported by a set of FORTRAN programs that were used to fit the statistical databases and to derive "objective" estimates of the desired extremes. The principal reports included:

1. *Validation of Joint Probability Methods for Large Waves and High Water Levels*, by P. Hawkes and R. Hague, Report SR 347, November 1994.
2. *The Joint Probability of Waves and Water Levels: JOIN-SEA*, H.R. Wallingford Report SR 537, November 1998 with minor amendments, May 2000.
3. *The joint probability of waves and water levels in coastal engineering design*, by P.J. Hawkes, B.P. Gouldby, J.A. Twain, and M. W. Owen, in Journal of Hydraulic Research, Vol. 40, April 2002.

The reports include examples in which the individual contributing processes (wave height, wave period, wave direction, tide, and storm surge anomalies) were both correlated and uncorrelated. The analysis started with wave conditions and water levels on a common time database. Scatter plots of wave heights versus wave periods, wave heights versus storm surge, wave heights versus directions, and so forth, were made to identify the degree of independence of the contributors. If and when these relationships were identified, the appropriate computation method was chosen to make the simulations of long-term records. The reports indicate that a three-year database is sufficient, but this will probably not be the case for any of the U.S. coastal regions. Atlantic and Gulf Coast hurricanes will not be properly represented in a three-year period, and the Pacific Coast, particularly the southern part, will have longer term variability owing to El Niño effects. The 30-year database for waves and swells that is available from Oceanweather (see the Focused Study Report for Storm Wave Characteristics) would provide a more useful source, and long-term tide gauge records from NOS would provide water level data (see discussion of Topics 54 and 55 in the Stillwater Focused Study Report).

Figures 3 and 4 show a sample result for wave height and surge based on recorded data and a summary of the final estimates of extremes. The special cases of "independent" and "dependent" are compared.




| HS (METRES) | HIGH WATER LEVEL (M O.D.) 0.00 - 0.50 | 0.50 - 1.00 | 1.00 - 1.50 | 1.50 - 2.00 | 2.00 - 2.50 | 2.50 - 3.00 | 3.00 - 3.50 | 3.50 - 4.00 | 4.00 - 4.50 | | NO. IN ROW |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 5.25- 5.50 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 |
| 5.00- 5.25 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 |
| 4.75- 5.00 | 0 | 0 | 0 | 0 | 0 | 1 | 0 | 0 | 0 | 0 | 1 |
| 4.50- 4.75 | 0 | 0 | 0 | 0 | 0 | 1 | 0 | 0 | 0 | 0 | 1 |
| 4.25- 4.50 | 0 | 0 | 0 | 1 | 0 | 2 | 0 | 0 | 0 | 0 | 3 |
| 4.00- 4.25 | 0 | 0 | 0 | 0 | 0 | 3 | 0 | 0 | 0 | 0 | 3 |
| 3.75- 4.00 | 0 | 0 | 2 | 2 | 3 | 2 | 2 | 3 | 0 | 0 | 14 |
| 3.50- 3.75 | 0 | 0 | 1 | 2 | 1 | 7 | 4 | 1 | 0 | 0 | 16 |
| 3.25- 3.50 | 0 | 0 | 6 | 6 | 4 | 12 | 8 | 0 | 0 | 0 | 36 |
| 3.00- 3.25 | 0 | 0 | 3 | 11 | 8 | 14 | 14 | 7 | 0 | 0 | 57 |
| 2.75- 3.00 | 0 | 0 | 3 | 17 | 23 | 19 | 15 | 2 | 0 | 0 | 79 |
| 2.50- 2.75 | 0 | 2 | 3 | 16 | 19 | 37 | 29 | 4 | 1 | 0 | 111 |
| 2.25- 2.50 | 0 | 1 | 7 | 29 | 35 | 43 | 23 | 3 | 1 | 0 | 142 |
| 2.00- 2.25 | 0 | 0 | 7 | 31 | 56 | 74 | 48 | 8 | 0 | 0 | 224 |
| 1.75- 2.00 | 0 | 1 | 9 | 51 | 63 | 85 | 53 | 9 | 0 | 0 | 271 |
| 1.50- 1.75 | 0 | 1 | 15 | 56 | 75 | 81 | 82 | 11 | 1 | 0 | 322 |
| 1.25- 1.50 | 0 | 1 | 15 | 77 | 133 | 149 | 115 | 14 | 0 | 0 | 504 |
| 1.00- 1.25 | 0 | 3 | 14 | 36 | 70 | 102 | 32 | 4 | 0 | 0 | 261 |
| 0.75- 1.00 | 0 | 1 | 33 | 119 | 214 | 255 | 161 | 12 | 0 | 0 | 795 |
| 0.50- 0.75 | 0 | 0 | 44 | 155 | 200 | 292 | 140 | 7 | 0 | 0 | 838 |
| 0.25- 0.50 | 0 | 1 | 70 | 276 | 464 | 568 | 226 | 11 | 0 | 0 | 1616 |
| 0.00- 0.25 | 0 | 3 | 46 | 169 | 304 | 401 | 167 | 7 | 0 | 0 | 1097 |
| NO DATA/CALMS | 0 | 0 | 6 | 29 | 32 | 45 | 22 | 0 | 0 | | |
| NO. IN COLUMN | 0 | 14 | 278 | 1054 | 1672 | 2148 | 1119 | 103 | 3 | | |

RECORDS ANALYSED FROM DAY 1 OF MONTH 1 OF 1981 TO DAY 31 OF MONTH 3 OF 1990
NUMBER OF RECORDS ANALYSED 6525

**Figure 3. Example of joint occurrences from recorded data (after HRS Report SR-347).**

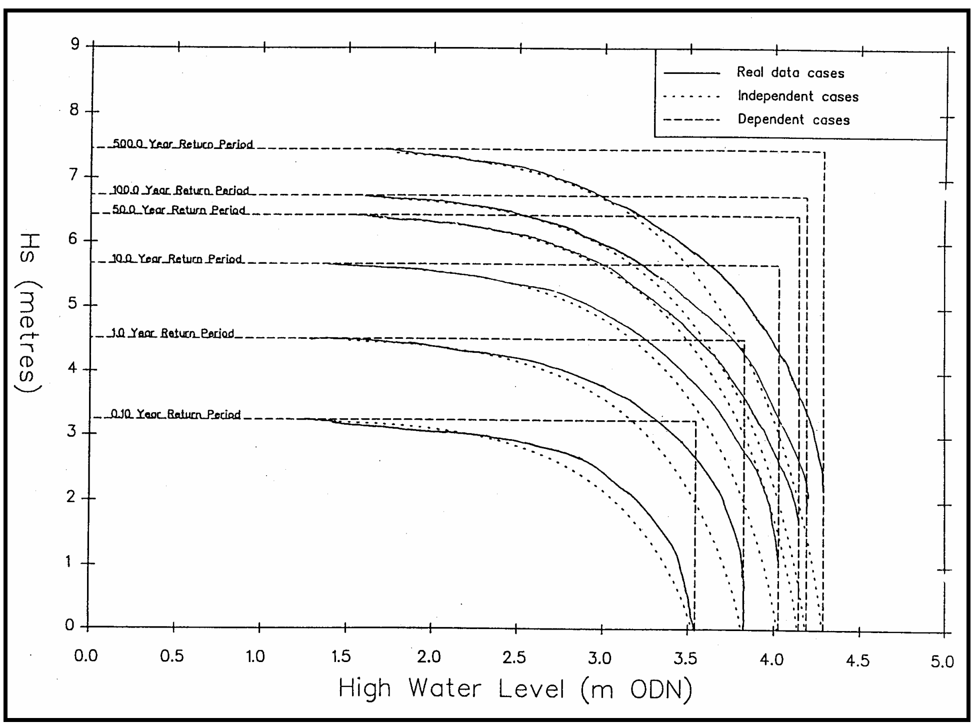



**Figure 4. Example of Final Estimated Extremes (after HRS Report SR-347).**

It is noted that the “worst” 100-year event may not be the same for all responses, as illustrated in Figure 5. Differing water level and wave height combinations may be the most critical determinants for different responses. For example, the 1% erosion will tend to depend strongly on the duration of high water levels in the event that it occurs for slow-moving storms that cause high water to persist for long periods. Thus, the 1% storm surge levels and the 1% erosion may be caused by different hurricane events. For example, Hurricane Andrew (1992) crossed the east Florida coast rapidly and caused little erosion of the nourished Miami Beach.

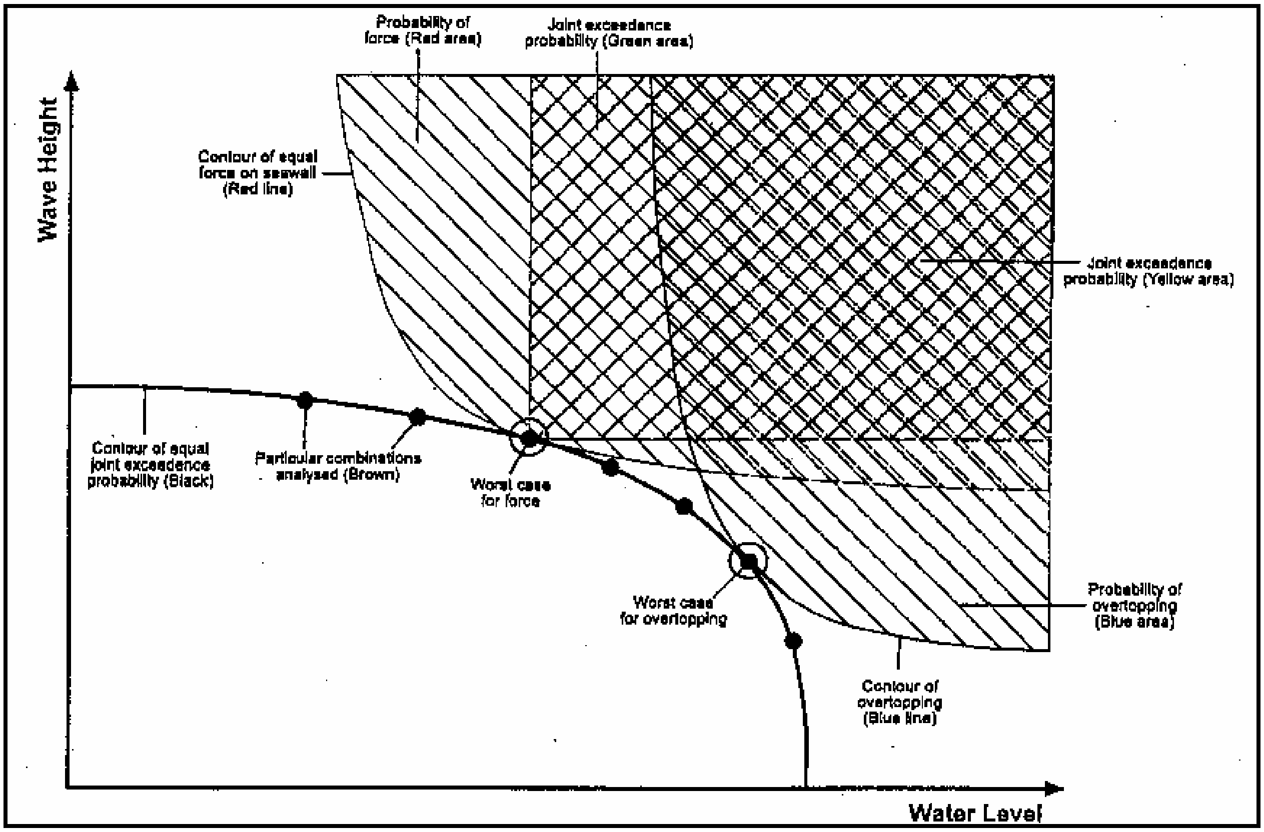



**Figure 5. Illustration of differing combinations of waves and water levels governing different design cases.**

Figure 5 presents results for a two-parameter process (probability of force on a seawall and probability of overtopping). In principle, the processes could be extended to include separate inputs for waves, swells, river flows in estuaries, wave heights, wave periods, and wave directions. However, presentation of multiple combinations of results for each input would become multidimensional and very complex.

## 2.2 COMBINATION OF SURGE AND RIVERINE FLOODS

The common case of combined surge and riverine flood near the mouth of a tidal river was considered in the early FEMA methodology but is missing from the latest Guidelines. As noted above, the recommended approach has been to assume a mismatch between the times of surge and runoff peaks, so that the two floods are effectively separate events (even if sometimes correlated by being derived from the same storm). With this assumption, one simply adds the rates of occurrence of a given flood level, $Z$, to obtain the total rate:

$$R_{TOTAL}(Z) = R_{RIV}(Z) + R_{SURGE}(Z) \tag{3}$$

However, the assumption of independence may not always be appropriate, and the question then is whether practical methods can be found to account for interdependence. There are two major difficulties that must be overcome to accomplish this: First, one would require knowledge of rainfall characteristics of the hurricanes and tropical storms that contribute to the 100-year surge, as well as how those rains are incorporated in the rainfall data upon which the riverine flood profile is based. Second, the riverine (HEC-RAS) modeling would have to be repeated many times to account for physical interactions that would occur over the entire range of possible surge-runoff combinations. One might also look at historical data for simultaneous surge level and stream discharge and, from those data, develop a mean relationship between surge and directly related runoff. To this must still be added the truly separate rainfall events, determined from hurricane-free rainfall data.

This appears to be a daunting task and might require a major investment with little significant return for the flood insurance program. Certainly, it could not be classified as *Critical*, but would instead become *Important*, requiring a much longer period of effort. In other words, the simple addition of rates, while approximate, may remain the most suitable approach. This issue is also discussed under Topic 19 in the Hazard Zone Mapping Focused Study Report.

### 2.3 EST POST-PROCESSED APPROACH

A related Empirical Simulation Technique (EST) approach, briefly described in the following paragraphs, uses an input database of total surge that results from simulating (generally) all recorded events at a specific location. A full discussion of this approach is found in Scheffner et al. (1999). These surge-only values are combined in a linear manner with a finite combination of tides, computed historic wave distributions, computed historic setups, computed historic runup, and so forth, to generate a database of total surge, that is, surge plus tide, waves, setup, and so forth. The EST uses this input data to generate $n$ repetitions of $T$ years of simulated storm activity that includes those processes; a study might involve, say, $n$ =100 repetitions of a $T$ =200-year sequence of storm activity. From the output database of life-cycle simulations, frequency-of-occurrence relationships are computed. An empirical estimate of the cumulative probability distribution function (PDF) $F_X\left(x_{(r)}\right)$, denoted by $\hat{F}_X\left(x_{(r)}\right)$, and is given by the plotting position formula

$$\hat{F}_X\left(x_{(r)}\right) = \frac{r}{(n+1)} \tag{4}$$

for $\{x_{(r)}, r = 1,2,3,\ldots,n\}$. This form of the estimate satisfies Gumbel's requirements, allowing future values of *x* to be less than the smallest observation $x_{(1)}$ with a cumulative PDF of $1/(n+1)$, and to be larger than the largest value $x_{(n)}$ with cumulative PDF of $n/(n+1)$. In the example approach, the 100-year total surge elevation can be determined for each of the 100 simulations of 200 years of simulated storm activity. Mean value and standard deviation analyses can then be used to determine any return-year elevation estimate with an estimate of error based on (for example) the standard deviation. An example of 100 stage-frequency plots and the computed average determined in this way are shown in Figure 6 (see, for example, Borgman and Scheffner, 1991; Scheffner and Borgman, 1992). In a sense, this method is a numerical simulation substitute for convolution methods.

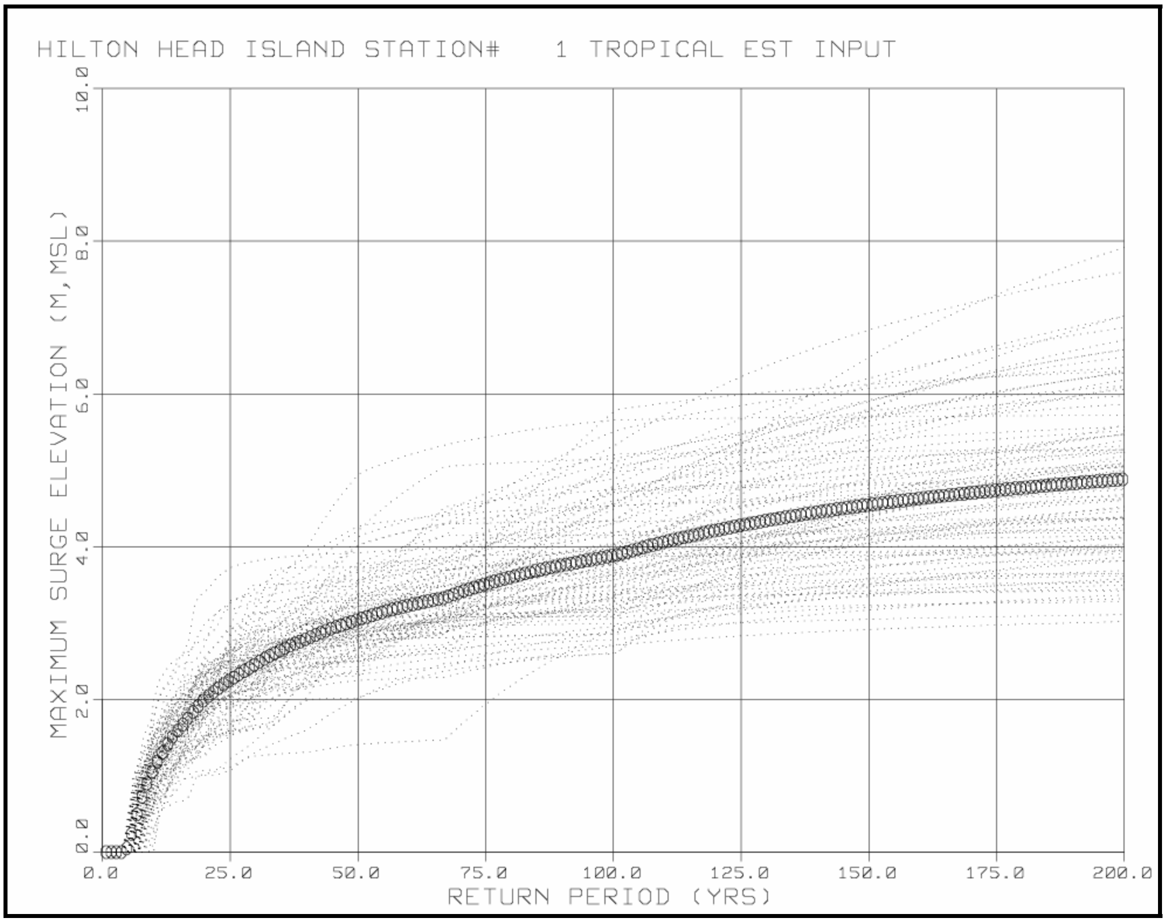



**Figure 6. Distributions derived from 100 simulations of 200-year periods.**

## 2.4 RECOMMENDATIONS

It is recommended that each important variety of flood combination—waves plus high water, tsunami plus tide, and so forth—be included as topics in the new Guidelines, along with a suggested methodology and illustrative examples derived from test studies and/or hypothetical cases. Performing a small set of test studies in typical environments will permit detailed evaluation and comparison of alternatives, as well as development of suitable approximations and practical advice for flood insurance applications.

For example, an approach such as that presented in the HRS (U.K.) reports, which shares features with the PWA (2002) approach, appears to be the most comprehensive and suitable for addressing combinations of water levels and wave conditions, subject to the proviso that a period of record longer than 3 years would probably be required for the initial statistical summaries. The technique can be adapted to consider joint occurrences of both dependent and independent contributors to flood levels. A test study using this approach is recommended at a site (or sites) on the Pacific Coast. More than 10 years of water level data can be obtained from a NOS tide gage, with a corresponding record from a NOAA wave buoy. Within the available time constraint for critical studies, however, an effort separating swell heights and periods, wave heights and periods, and tides might not be feasible.

A test study using data from the recent Sandy Point flood insurance study (FIS) is recommended. The test study would include the HRS Monte Carlo, Tetra Tech, and time series approaches. The results would be compared to the EST results used in the Sandy Point FIS for a limited range of parameters selected for use with the other approaches. Applying the test to Sandy Point allows use of a 29-year data set that has already been analyzed. Additionally, Sandy Point is a simple, two-variable case—water level and locally generated wind waves—with actual flood data available for verification purposes. The output will be 100-year wave runup elevations calculated with each method for selected shore profiles and common input data. It would also be advantageous to apply these methodologies to an Open Coast situation, which would include several variables (e.g., water level variables; heights, periods, and directions of swell and storm seas).

The combination of tsunami and tide levels can probably be handled as a straightforward application of the convolution theorem because these are independent events and the resulting water levels are likely to approximate the simple sum of the two processes. Where the environment might indicate important physical interactions between the two processes, there may be no good alternative to hydrodynamic modeling of numerous joint probability cases.

On the Atlantic and Gulf Coasts in hurricane-dominated areas, the system presently in use is acceptable, with the possible exception of the Atlantic Coast north of Long Island. The latter region could be addressed using the HRS Monte Carlo process.

## 2.5 PRELIMINARY TIME ESTIMATE FOR GUIDELINE IMPROVEMENT PREPARATION

Table 3 in section 6.0 Summary, summarizes the preliminary estimates of time required for Critical Topic 51. These time estimates do not include responding to comments and suggestions associated with the review of the Guideline improvements.

# 3 AVAILABLE TOPICS

None identified.

# 4 IMPORTANT TOPICS

## 4.1 TOPIC 50: STATISTICAL METHODS FOR SURGE MODELING (FOR BOTH ATLANTIC/GULF COASTS AND PACIFIC COAST)

### 4.1.1 Description of the Topic and Suggested Improvement

The basic approach to estimating storm surge frequency, as implemented in FEMA coastal surge model studies, assumes two sorts of knowledge for a given study region. First, the approach assumes that if the characteristics of a particular storm are specified, then modeling tools exist to determine the flood elevations that would occur everywhere within the study area as a consequence of that storm. The storm characteristics might include direction of forward motion, location of the shoreline crossing point, speed of travel, and measures of storm size and intensity. The modeling tool is the storm surge hydrodynamic model, implemented for the local bathymetry, topography, and terrain.

The second assumption is a method for attaching a frequency to the simulated storm, which is then also attached to the computed flood levels for that one storm simulation. This requires knowledge of the storm history for the area, from which the frequency information can be derived. By simulating numerous storms in this manner, one effectively simulates a long period of record at the site, from which flood statistics can be derived. Topic 50 is concerned with the methods by which this process can be achieved.

### 4.1.2 Description of Potential Alternatives

***Joint Probability and Monte Carlo Methods***

The primary method used in past FEMA coastal surge model studies has been the so-called Joint Probability Method (JPM), pioneered for coastal surge applications by NOAA (e.g., Myers, 1970). In the JPM method, a hurricane has usually been defined by five parameters. Track angle, track position, and forward speed are the three kinematic parameters; storm radius to maximum winds and central pressure depression are the two dynamic parameters.

By defining a sample window around the study area, one identifies all recorded storms that have passed within the site vicinity, and from those storms one establishes empirical probability distribution functions for each of the five storm parameters. Each of those distributions can then be discretized into a small number of representative values and probabilities, say, on the order of five each. Taking one value from each set defines a storm; all possible combinations represent all possible storms. In the event that even only five values were selected for each of the five discrete approximations, all possible storms would be represented by a set of more than 3,000 combinations (in practice, the actual simulation set is usually on the order of a few hundred storms).

The frequency represented by one particular storm is calculated as the rate of storm occurrence (events per year obtained from the count of storms caught during a known number of years within the sample window of known size) multiplied by the product of the probabilities assigned to the five parameters (if independence is assumed). This storm rate is attached to each surge elevation computed throughout the basin for that one particular storm. As each storm in the simulation set is run, a histogram is developed at each location in the grid, with rate being accumulated into bins defined by small flood-elevation ranges. For example, if a particular storm with rate R produces a flood elevation of S at point P, then bin 10S (if elevations are resolved to the nearest tenth of a foot) in histogram H(P) is incremented by R. At the conclusion of the entire set of runs, the histogram at any point constitutes a discrete density function. By summing such histograms from the top down, one obtains a cumulative distribution from which the elevation corresponding to an exceedance rate of 0.01 can be read.

An objection to this procedure is the independence assumption, which permits simple multiplication of individual parameter probabilities. This objection is only partly justified, however, because usual practice has been to divide the storm sample into three families: storms that approach land from the sea; those that exit from land to the sea; and those that travel more or less parallel to the local mean coast. A dependence between track angle and the other parameters is clear (storms exiting land, for example, may be less intense than those that make landfall), and this separation into subfamilies accounts for that dependence.

Another lack of independence frequently cited as a point of special concern is the possible dependence between central pressure and radius to maximum winds. The intense Labor Day storm of 1935, for example, had both a very small radius (6 nm) and a very large central pressure depression (3.6 inches Hg). It is sometimes asserted that a storm of this intensity cannot exist with a significantly larger radius, owing to energy limitations. However, this lack of independence, if it exists as a practical matter, has not been well demonstrated using standard statistical tests, and energy calculations show that the radius could be made much larger without exceeding energies frequently encountered in other storms.

A related approach is the Monte Carlo Method, in which continuous distributions are used instead of discretized distributions. Individual storms are constructed by choosing a value randomly from each of the parameter distributions. In general, the questions of independence raised concerning the JPM apply equally to the Monte Carlo Method, although it should be noted that if a correlation between parameters can be specified, then it can also be accounted for in both Monte Carlo and JPM applications. The Monte Carlo Method has an advantage over JPM; because the distributions are not separated, the set of possible storms is not limited to a finite set, as with JPM. The Monte Carlo Method has the potential disadvantage that a greater number of simulations may be necessary to ensure that the tails of the distributions are adequately sampled.

Both the JPM and Monte Carlo approaches require distribution functions for each of the storm parameters and an estimate of the corresponding storm density (number of storms per nautical mile per year at the study site). Two data sources in particular have been relied on in past FEMA work: first, publications of the NWS (currently, NWS 38); and second, storm data files from NOAA (currently, NOAA Hurricane Research Division, data file *HURDAT*). HURDAT is a digital file of storm data for all identified tropical storms in the North Atlantic (now including storms since as early as 1851). In addition to storm tracks (position at six-hour intervals), HURDAT also contains wind and pressure information (although central pressure data are scattered for storms before the 1960s), but no information regarding storm radius. From its storm track information, HURDAT provides a complete data source for three of the five storm parameters that are needed for JPM and Monte Carlo studies: forward speed, track direction, and track position.

NWS 38 (National Weather Service, 1987) was commissioned by FEMA as a comprehensive source of the data needed in a hurricane surge flood study. NWS presents an atlas of the required data in graphical format for all locations along the Atlantic and Gulf Coasts (although northeasters are not included). Track information is based on HURDAT data, whereas pressure and radius data are based on a reevaluation of the available data from NOAA sources. Figure 7 shows the coastline coordinate system used in NWS 38.

Figures are provided in NWS 38 showing the necessary storm parameters as functions of the coastal coordinate. For example, Figures 8 and 9 show the storm frequency for entering and exiting storms.  Figure 10 shows the manner in which a cumulative distribution is displayed (this example is for central pressure on the East Coast).

HURDAT is relatively current, at present including storms through the 2002 season. In fact, the data contained in HURDAT have been updated throughout during the past few years as part of a major reanalysis.

NWS 38, however, is almost 20 years old, which is significant in that a large proportion of the high-quality pressure and radius data that are now available may postdate the study. Consideration should be given, therefore, to recommending more up-to-date data sources to replace or supplement NWS 38.

There is a second difficulty with NWS 38 that is not widely recognized: Data are developed and presented with respect to the shoreline-based coordinate system instead of natural geographic coordinates. The reason for this is historical. The pioneering numerical surge model developed by NOAA, the SPLASH model (Jelesnianski, 1972) originally assumed a straight shoreline with a fixed offshore bottom slope. Three sorts of storms were allowed: entering perpendicular to the straight shoreline, exiting similarly, or running parallel to the shoreline. In developing storm data for the early coastal flood studies performed with the SPLASH model, statistics were naturally specified in the same manner. Storm data categorized in this way was published in NWS 15, the predecessor of NWS 38.

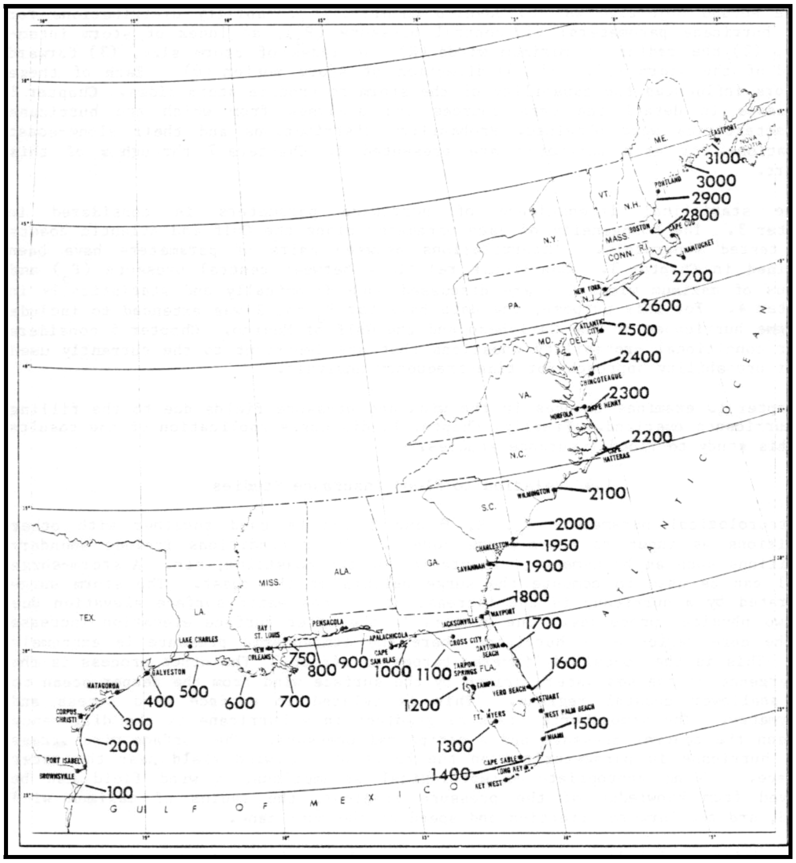



**Figure 7. Coastline coordinate system from NWS 38.**

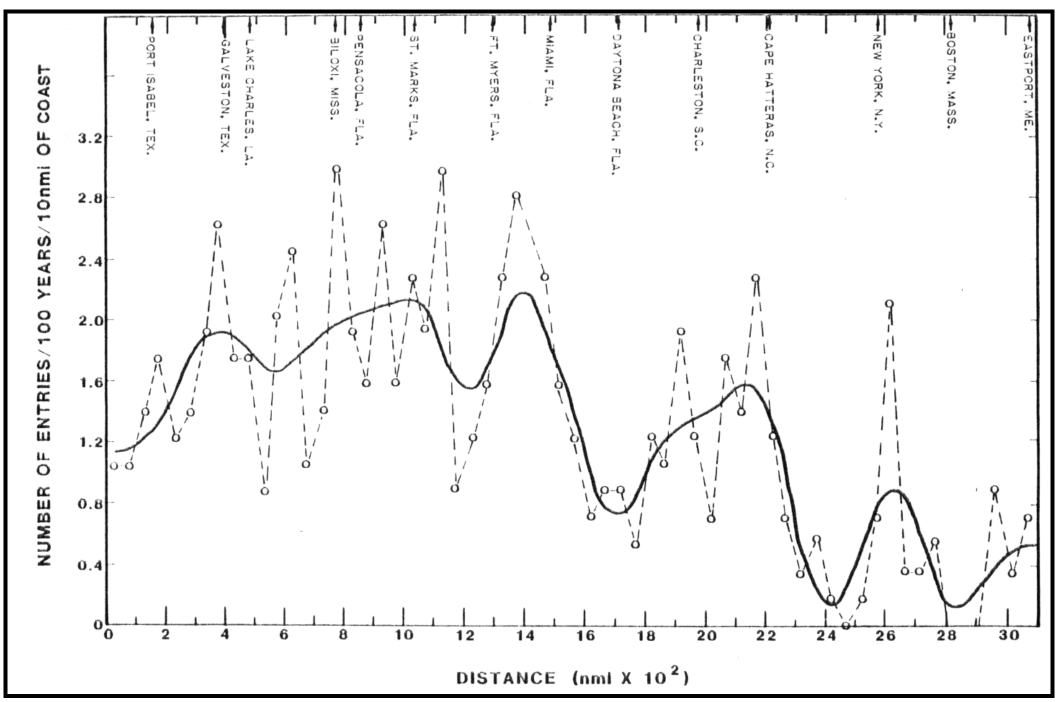



**Figure 8. Storm frequency for entering storms from NWS 38.**

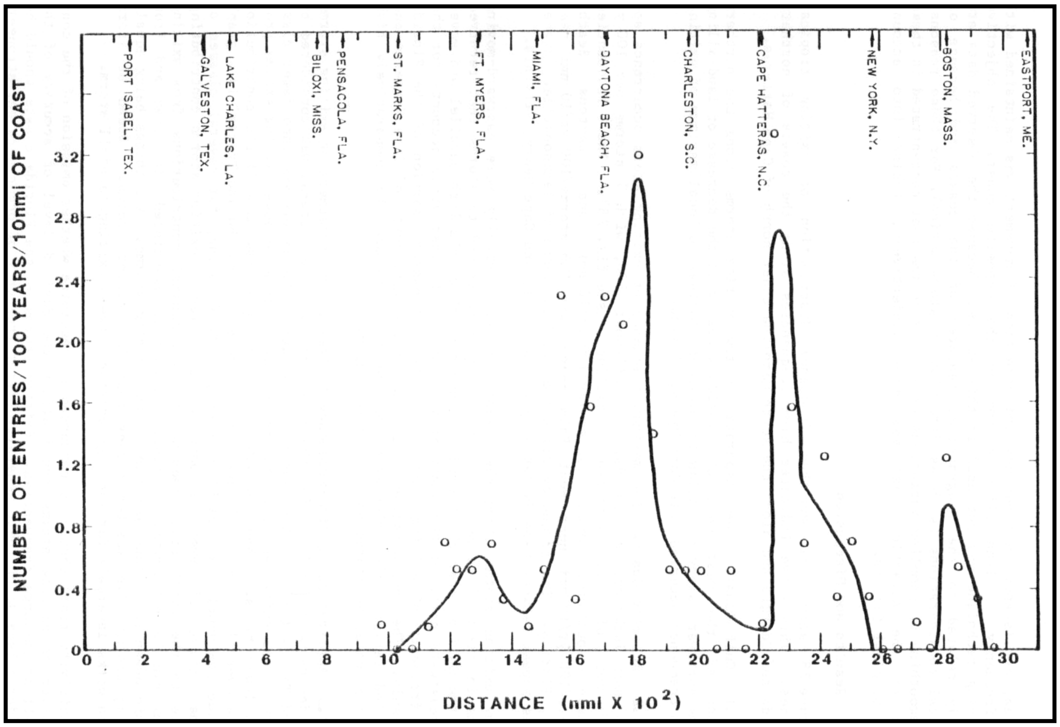



**Figure 9. Storm frequency for exiting storms from NWS 38.**

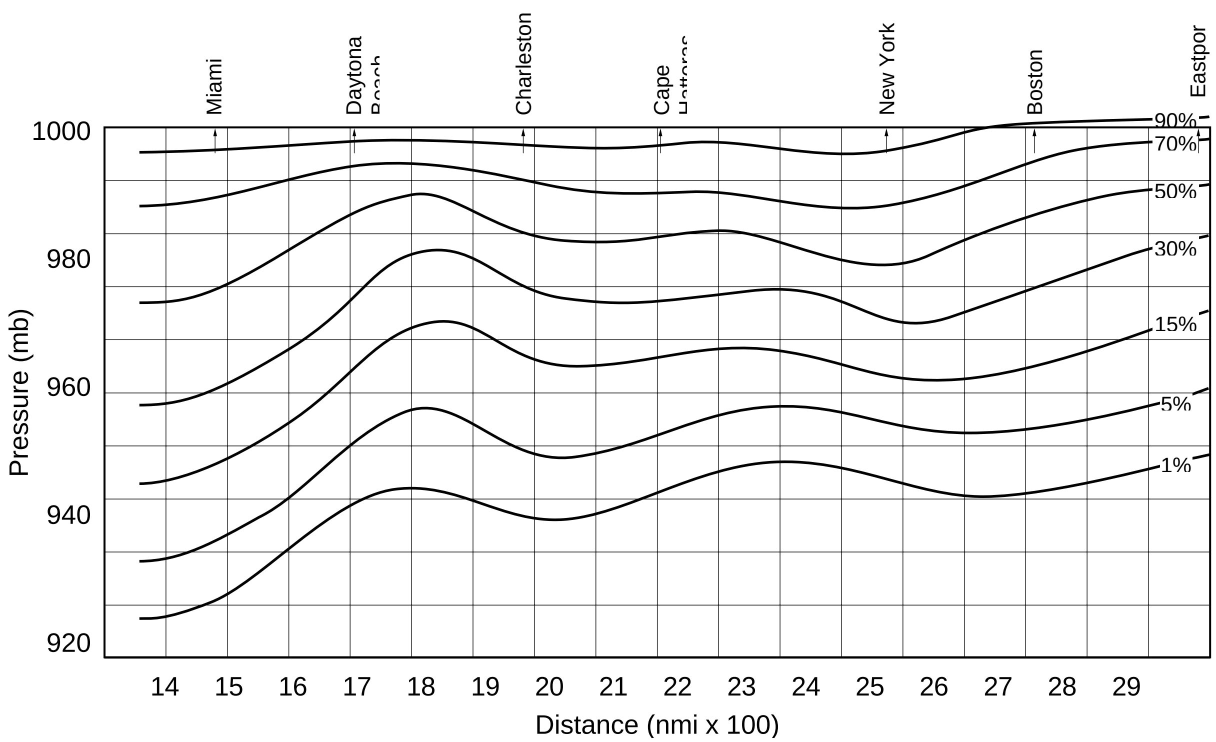



**Figure 10. Cumulative distribution of central pressure on the East Coast from NWS 38.**

The difficulty is that, by dividing storms into three relative path categories and then smoothing data alongshore as shown in the figures above, one inevitably combines the three storm families in an unknown way in the vicinity of every change in coastline orientation. For example, consider points below and above the right-angle coastal bend at Cape Hatteras. Entering storms below the cape belong primarily to the alongshore family above the cape, and vice versa. It is unclear what the practical implication of this is, and has been, for FEMA coastal studies. It should be noted that the objective smoothing procedure used in NWS 38 involves points as far distant as 250 nautical miles on each side of a given point. It should also be noted that a smoothing operation has the one-way effect of reducing all peaks and troughs; fluctuations in direction-family storm counts that are solely caused by bends in the coastline should be retained in the data (because they do not represent undesirable sample variation, which could, legitimately, be smoothed) and should not be reduced by smoothing. Current storm surge models, such as the FEMA surge model, ADCIRC; NOAA's SLOSH; DHI's Mike models, have no coastal orientation restrictions. The most natural way to provide the necessary storm data for these models is simply in terms of natural geographic coordinates.

Consequently, as part of the proposed effort, the appropriate storm data sources for coastal studies should be reviewed and recommendations made. The methods for developing the kinematic parameters (track and forward speed) may be relatively simple, using HURDAT as a source. The appropriate radius and pressure data may be more difficult to specify, requiring the participation of appropriate NOAA specialists. It would be extremely useful if the best available estimates of both R and $\Delta P$, as functions of track position, could be added to HURDAT or summarized in a similar format. Existing data on these parameters are scattered through many sources, and some are unpublished; bringing these data together in HURDAT form would be of substantial value for coastal flood insurance studies.

### *Empirical Simulation Technique*

A newer technique, the Empirical Simulation Technique (EST), has been developed for the U.S. Army Corps of Engineers and has recently been approved for flood insurance applications by FEMA. EST is a statistical technique that simulates long-period sequences of cyclic but non-deterministic multi-parameter systems such as storm events and their corresponding impacts (Scheffner et al., 1999; Scheffner and Borgman, 1992). The approach is based on bootstrap resampling-with-replacement, random-neighbor walk, and subsequent smoothing techniques in which a random sampling of a finite-length historical event database is used to generate a larger, long-period database. The only assumption is that future events will be statistically similar in magnitude and frequency to past events.

The EST begins with an analysis of historical storm events that have affected the study area. Characteristics of these events can be extracted from the HURDAT database and other sources. The selected events are then broken down to define the following components: relevant input parameters that are used to define the dynamics of the storms (the components of the so-called input vectors); factors that may contribute to the total response of the storm (i.e., surge), such as tidal phase, waves, and setup; and response vectors, which define storm-generated effects, such as total surge. Input vectors are simply sets of selected parameters that define the total storm; response vectors are sets of values that summarize the effects. Basic response vectors are determined by simulating historical storms with a suitable hydrodynamic model (ADCIRC has been adopted for surge). These input and response vectors are then used as the basis for the estimations of long-term surge history.

The recent South Carolina Storm Surge Study (Scheffner and Carson, 2001) is a typical example of a surge study, involving the following general sequence of steps:

- First, input vectors were developed for the base historical storms, including as components flood/ebb/slack tidal phase, spring-neap phase of the lunar month, minimum distance from the eye of the storm to the station location of interest, central pressure deficit, maximum wind speed, and forward speed of the eye of the hurricane.

- Next, corresponding response vectors were determined by simulating each historical event with ADCIRC. Each of 24 historic surges was combined with tide at four phases to generate a 96-event input database for the EST.

- The EST then generates multiple life cycles of surge-plus-tide activity. A total of 100 repetitions of 200 years was used for the South Carolina study. The large number of generated events is consistent with the local history (chosen by random sampling of the input vector space with random near-neighbor walk) in both frequency and magnitude.

- The long-period simulation was then post-processed (rank ordering and frequency analysis) to establish surge frequency relationships.

In this way, the EST uses observed and/or computed parameters associated with site-specific historical events and does not rely on assumed parameter independence, but rather uses the joint probability relationships inherent in the local data. Consequently, probabilities are site specific, do not depend on fixed parametric relationships, and do not assume parameter independence; the EST is distribution-free and nonparametric.

However, it is noted that owing to the extremely sporadic nature of hurricanes, the recorded experience at a site may not always adequately represent the range of events actually possible at that site; this is why it frequently happens that a new coastal flood is reported to be of unprecedented magnitude.

### 4.1.3 Recommendations

To better gauge the strengths and weaknesses of the three approaches discussed here for determination of storm surge frequency estimation—JPM, Monte Carlo, and EST—it is recommended that all three methods be implemented in a test study using a common hydrodynamic surge model. The particular model and coastal location used for this study may not be a critical matter. However, it is noted that studies using the ADCIRC/EST combination have been performed for the following sites, from which one might be selected:

- Coast of Delaware
- American Samoa
- Brunswick, SC
- Ponce and Guaynilla, Puerto Rico
- Long Island, Raritan Bay (unpublished)
- New Orleans/Morganza Flood Plain (unpublished)
- Galveston, TX
- Hilton Head, SC
- Guam

The modeling information for studies performed with the FEMA surge model (model grids and storm parameters) has largely been lost or discarded since completion of those studies in the 1980s.

It is important also that NOS water-level time series (or similar data) be available as a benchmark to assist in the interpretation of the statistical results for the three methods. By using a single surge model, differences are isolated to the statistical procedure formulated into the model and the quality of the storm data.

To address such issues as unaccounted-for parameter interdependence and sensitivity to sample error from a finite sample window, it would be desirable to perform numerical experiments using storm parameter distributions specified *a priori* (but mimicking the observed data at the test site) and including specified parameter correlations. From these *a priori* distributions, representing known “true” conditions, one could draw, say, a set of 100-year samples and perform the statistical studies using each of the alternative approaches.

This test effort should use both NWS 38 data and alternative data newly developed from HURDAT and other sources. This would permit an assessment of the suitability of NWS 38 for future use by FEMA and would provide some insight into the impact of storm data coastal smoothing in existing studies. To achieve this, the test site should be selected from a coastal region included in NWS 38 (eliminating three of the ADCIRC/EST sites listed above) that is near a significant coastal bend.

As a longer term effort, a data compendium similar in spirit to NWS 38 might be developed or recommended, or very specific procedures might be devised that would permit Study Contractors to determine parameters in an objective, reliable, and reproducible way.

#### 4.1.4 Preliminary Time Estimate for Guideline Improvement Preparation

Table 3 summarizes the preliminary estimates of time required for Critical Topic 50. These time estimates do not include responding to comments and suggestions associated with the review of the Guideline improvements.

# 5 ADDITIONAL OBSERVATIONS

Storm surge simulations rely critically upon knowledge of a storm's wind field and the surface stress created by the wind. These factors are appropriately considered here under the umbrella of Storm Meteorology, although they might equally well be addressed under the category of Stillwater. The following additional observations are made, and corresponding additional tasks are identified in the time and cost estimate above. These tasks will be applicable to all settings, including the Pacific Coast.

## 5.1 WIND FIELDS

Work performed at NOAA's Hurricane Research Division (HRD) has included the reconstruction of several hurricane wind fields using all available data. HRD has identified systematic differences between those reconstructions and the hypothetical descriptions given by planetary boundary layer models currently used in storm surge models. Systematic differences are important because they may lead to a systematic bias in surge predictions and would compromise calibrations that assume accurate storm knowledge. A task to review the available wind models and to suggest a model for flood insurance applications is recommended. This review should cover not only tropical storms, but also northeasters and Pacific storms, from the standpoint of storm surge modeling.

## 5.2 WIND STRESS

Recent dropsonde observations made by HRD indicate that wind stress on the ocean surface may decrease at high wind speeds. This can occur if extreme winds blow the crests off waves, creating a smoother surface that offers less traction to the wind. Current representations of the wind stress in storm surge models do not include such an effect. Consequently, it is recommended that an additional task be undertaken to review the many wind stress formulations available, and to suggest an appropriate treatment for flood insurance studies.

# 6 SUMMARY

The Storm Meteorology Focused Study Report addresses two broad topics: the specification of the 100-year (or 1%) event (Topic 51) and methods to determine storm surge flood frequency (Topic 50). The specification of the 1% flood is categorized as *Critical* for all regions, whereas surge frequency methodology is categorized as *Important* for the Atlantic and Gulf Coasts.

The specification of the 1% flood requires consideration of combinations of processes that may be independent or correlated, and that may combine in a linear or nonlinear manner. Important examples are the combinations of surge and tsunami with astronomic tide, the combination of surge with riverine rainfall flood profiles, and the combination of waves and high water.

The primary effort will be to perform test studies using the well-documented HRS joint probability approach to the problem of waves and high water, and, from these, to derive general guidance for flood insurance studies. Other methods and simplifications will also be considered, although the HRS procedures, which have been developed over many years, appear to be comprehensive and appropriate. Simpler tasks will include preparation of guidelines for the other identified combinations.

The recommendations for surge frequency determination include a comparison of the JPM and EST methods and the preparation of guidelines for the use of each. Appropriate data sets will be recommended, including not only sets for hurricanes and tropical storms, but also for northeasters. Consideration will be given to the applicability of NWS 38 for continued FEMA use.

Two additional tasks beyond the initial scope are suggested, dealing with the representation of wind fields and wind stresses in storm surge models.

Table 2 summarizes the Topics and recommendations of the Storm Meteorology Focused Study report.

**Table 2. Summary of Findings and Recommendations for Storm Meteorology**

| Topic Number | Topic | Coastal Area | Priority Class | Availability Adequacy | Recommended Approach | Related Topics |
|---|---|---|---|---|---|---|
| 50 | Modeling Procedures | AC | I | PRODAT | Identify and summarize data sources for storm parameters, and compare storm surge statistical methods (EST, JPM, Monte Carlo approaches may all be valuable); prepare guidelines describing the use of each alternative; revisit treatment of storm wind fields and wind stress formulation | 53-55 |
| | | GC | I | PRODAT | | |
| | | PC | -- | -- | | |
| | | SW | -- | -- | | |
| 51 | Combined Probabilities, Determination of the 1% Flood | AC | C | MAJ | For each major process combination, prepare Guidelines with recommended methodology and illustrative examples. For wave-plus-high-water perform (2 open/sheltered) case studies for Pacific sites to: (1) Implement Wallingford approach, (2) use NOS tide gage data, (3) use NOAA wave buoy data. Develop practical Guidelines from study findings, with examples | All |
| | | GC | C | MAJ | | |
| | | PC | C | MAJ | | |
| | | SW | C | MAJ | | |

Key:
Coastal Area
AC = Atlantic Coast; GC = Gulf Coast; PC = Pacific Coast; SW = Sheltered Waters
Priority Class
C = critical; A = available; I = important; H = helpful
Availability/Adequacy
"Critical" Items: MIN = needed revisions are relatively minor; MAJ = needed revisions are major
"Available" Items: Y = availability confirmed; N = data or methods are not readily available
"Important" Items: PRO = procedures or methods must be developed; DAT = new data are required; PRODAT = both new procedures and data are required

| Table 3. Preliminary Time Estimate for Guideline Improvement Preparation | | |
|---|---|---|
| Topic Number | Item | Time (Person months) |
| 50 | **Statistical Methods for Surge Modeling (for both Atlantic/Gulf Coasts and Pacific Coast)** | |
| | Identify and analyze historical long-term gage data | 2 |
| | Identify and analyze storm parameter data sources | 1 |
| | Establish procedures for methodology comparisons | 3 |
| | Apply and compare methodologies (JPM, EST, Monte Carlo) using a common hydrodynamic model and storm data set | 4 |
| | Analyze results; summarize and prepare new guidelines with examples of application drawn from test studies, and including recommended data sources | 2 |
| | Additional Topic: Review best available data regarding wind fields and compare with fields used in storm surge models; recommend the most appropriate models for FIS use (tropical storms, northeasters, and Pacific storms) | 2 |
| | Additional Topic: Review best available data for wind stress and compare with formulations used in storm surge models; recommend the most appropriate formulation for FIS use | 2 |
| | Total | 16 |
| 51 | **Combined Probability (for all geographic regions)** | |
| | Develop guidelines for the combination of tsunami and tide, including a worked hypothetical example | 1 |
| | Develop guidelines for the combination of surge and tide, including examples drawn from past studies (with consideration of FEMA surge studies, ADCIRC/EST, and the FL-DEP Monte Carlo method) | 1 |
| | Prepare recommendations for the combination of surge and a riverine runoff profile | 1 |
| | Plan test studies (2) for Pacific Coast wave and high water combination; obtain necessary data | 2 |
| | Sandy Point, Pacific Coast Sheltered Waters test study | 2 |
| | Perform and evaluate Pacific Coast test studies | 6 |
| | Prepare guidelines based on findings, including illustrative examples | 2 |
| | TOTAL | 15 |

# 7 REFERENCES

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# Stillwater

## FEMA Coastal Flood Hazard Analysis and Mapping Guidelines Focused Study Report

February 2005


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## Acronyms

| | |
|---|---|
| 1-D | one-dimensional |
| 2-D | two-dimensional |
| ADCIRC | Advanced Circulation Model for Coastal Ocean Hydrodynamics |
| BFEs | Base Flood Elevations |
| CO-OPS | Center for Operational Oceanographic Products and Services |
| FEMA | Federal Emergency Management Agency |
| FL-DEP | Center for Operational Oceanographic Products and Services |
| *G&S* | *Guidelines and Specifications* |
| GROW | Global Re-analysis of Ocean Waves |
| LIDAR | Airborne Light Detection and Ranging |
| NFIP | National Flood Insurance Program |
| NOAA | National Oceanic and Atmospheric Administration |
| PWA | Philip Williams & Associates |
| USACE | U.S. Army Corps of Engineers |
| USGS | U.S. Geological Survey |
| WES | Waterways Experiment Station |

# 1 INTRODUCTION

## 1.1 CATEGORY AND TOPICS

This report describes a proposed approach for the development of new FEMA Guidelines for the determination of stillwater levels in coastal areas. *Stillwater* means the flood level not including the effects of waves (wave amplitude and wave setup; setup is addressed in a separate Focused Study Group report) or tsunamis, but including storm surge and astronomic tide. The particular topics addressed in this report were determined during Workshop 1 of the project and are identified below.

**Stillwater Topics and Priorities**

| Topic Number | Topic | Topic Description | Priority | | |
|---|---|---|---|---|---|
| | | | Atlantic / Gulf Coast | Pacific Coast | Non-open Coast |
| 52 | Non-Stationary Processes | Provide guidance on non-stationary processes (for example, relative sea level change) when establishing current conditions | A | A | A |
| 53 | Reliable Surge Data | Identify reliable existing data to compare to existing FEMA flood studies to test performance of surge models | C | -- | -- |
| 54 & 55 | Surge vs. Wave Height | Develop database for surge versus wave height; develop interim Pacific Coast model for surge (possibly ADCIRC); Review the reliability of Pacific tide data to see if surge is imbedded in the data sets for the purpose of developing surge factors for regions where there are little or no tide data; provide guidance | -- | C | C |
| Key: C = critical; A = available; I = important; H = helpful | | | | | |

Topic 52 is judged to be relatively straightforward, amounting to identification of available information on such non-stationary factors as sea level rise and land subsidence that might affect a coastal study.

Topics 53–55 are construed to address development of general guidelines for storm surge evaluation on both the Atlantic/Gulf (Topic 53) and Pacific (Topics 54 and 55) Coasts, including Pacific bays and estuaries (sheltered water areas). Furthermore, the necessary storm surge guidance is considered to be of two types: 1) guidance regarding storm surge hydrodynamic modeling, which will apply to both the Atlantic/Gulf and Pacific Coast insofar as general tools and principles are involved (addressing both Topics 53 and 54), and 2) guidance regarding other methods to estimate storm surge on the Pacific Coast and in Pacific bays, such as analysis of tide gage records (addressing both Topics 54 and 55). Note that additional guidance is provided in an accompanying Focused Study prepared by the TWG on "Sheltered Waters," which addresses many of the coastal flood issues found in non-open coastal areas.

### 1.2 STILLWATER FOCUSED STUDY GROUP

The Stillwater Focused Study Group is made up of Robert Battalio, Ian Collins, Robert Dean, Darryl Hatheway, Norm Scheffner, and David Divoky who served as Team Leader.

# 2 CRITICAL TOPICS

## 2.1 TOPIC 53: ATLANTIC/GULF STORM SURGE

### 2.1.1 Description of the Topic and Suggested Improvement

This topic includes not only the identification of data sets and methods for verifying and testing surge models, but also development of general guidelines regarding storm surge modeling. The general modeling guidelines developed under this topic will apply equally to modeling on the Pacific Coast (Topics 54 and 55).

### 2.1.2 Description of Procedures in the Existing Guidelines

Existing guidelines found in Appendix D are relatively brief, consisting primarily of checklists of requirements for data submission and documentation during a study. The material concerned with general surge modeling is contained in Section D.1.2.4, "Hydrodynamic Storm Surge Model," which, in full, is as follows:

- Report the unique model characteristics used for the study, including a discussion of the specific grid system and sub-grid systems employed, the grid used for bottom topography and shoreline, small-scale features such as harbors and barrier islands, and the location and conditions applied for the open boundaries to the grid.

- Describe and document the adjustment to land features to account for erosion.

- Describe and document the method used to determine average ground elevations and water depths within the cells of the grid system. This discussion is to be augmented by diagrams that show the grid systems as computer listings of the grid data used in the actual model calculations.

- Describe the method used to relate windspeed and surface drag coefficient.

- Discuss the Manning's "n" values used in the calculation of bottom and overland friction and provide values in tabular form. This information will include a discussion of any sensitivity tests used to estimate these values in nearshore water. Nearshore bottom and overland friction is an important part of the overall analysis and, therefore, shall be described with care and sufficient detail.

- Provide a graphical depiction of the model cells and grid system as an overlay to the bathymetric charts and topographic maps covering the study area, annotated with the individual cell inputs for the grid system.

- Discuss the method by which barriers, inlets, and rivers have been treated.

- Explain the procedures used to determine inland flooding, including parameterization of local features and selection of the friction factors used for the terrain.

Additional storm surge guidance is contained in Section D1.2.5, "Storm Surge Model Calibration and Verification," which consists of two paragraphs commenting on verification procedures and required backup documentation; Section D1.4.1, "[Intermediate Data Submission] Before Storm Surge Model Calibration Runs," consisting of a list of eight items to be submitted for review before proceeding with model runs; and Section D1.4.2, "Before Operational Storm Surge Runs," consisting of a checklist of seven items to be submitted for review before performing the main set of statistical simulation runs. Additional general material is provided in Section D2.2, "Data Requirements."

These guidelines are generally based on the use of the FEMA storm surge model, although brief mention is made of the Stone and Webster (1978) northeaster model and the possible determination of stillwater elevations using statistical analysis of available tide gage records, provided those records include 20 or more years of data. Section D.2.2 also states that synthetic computer models for storm surge assessments shall be used where tide gage data is limited and complex shorelines are present which cause appreciable variation in flood elevations for a community.

### 2.1.3 Alternatives for Improvement

#### *Storm Surge Modeling Guidelines*

A numerical storm surge model simulates the effects of a hurricane, tropical storm, northeaster, or other storm type passing over a given study area. Two basic types of data must be provided to the model. First, the model implementation must include an accurate description of the physical characteristics of the study area, including:

- Offshore bathymetry and onshore topography;

- Roughness characteristics of the ocean bed and landcover that may affect the flow of water;

- The nature of barriers and structures that may impede or divert the overland flow of the flood;

- The extent of elements (especially tall vegetation) that may partially shield the water surface from wind stress.

Second, the model must include a realistic representation of the storm being simulated; in particular, the time- and space-varying wind and pressure fields of the storm must be reflected in the model through use of an appropriate storm submodel. Note that sheltered waters may pose special requirements for both basin and storm description, to account for the sheltering effects of terrain, complex flow resistance through developed areas, and changes in storm properties associated with the on-land weakening known as *filling*. Further details regarding Sheltered Waters are provided herein in a separate Focused Study Report on Sheltered Waters.

In addition to these factors describing the basin and the forcing disturbance, the model must solve a set of equations capable of capturing the essential features of the process, including the effects of wind, pressure, friction, overland flow (wetting and drying of land areas), and tidal forcing and tidal potential terms. This also requires the selection of a large number of empirical factors and functional expressions to describe, for example, bottom friction and wind stress.

Figure 1 (adapted from an unpublished diagram by Professor Robert Reid (Texas A&M) illustrates the primary aspects of surge modeling, including the determination of the types of waves that produce wave setup.

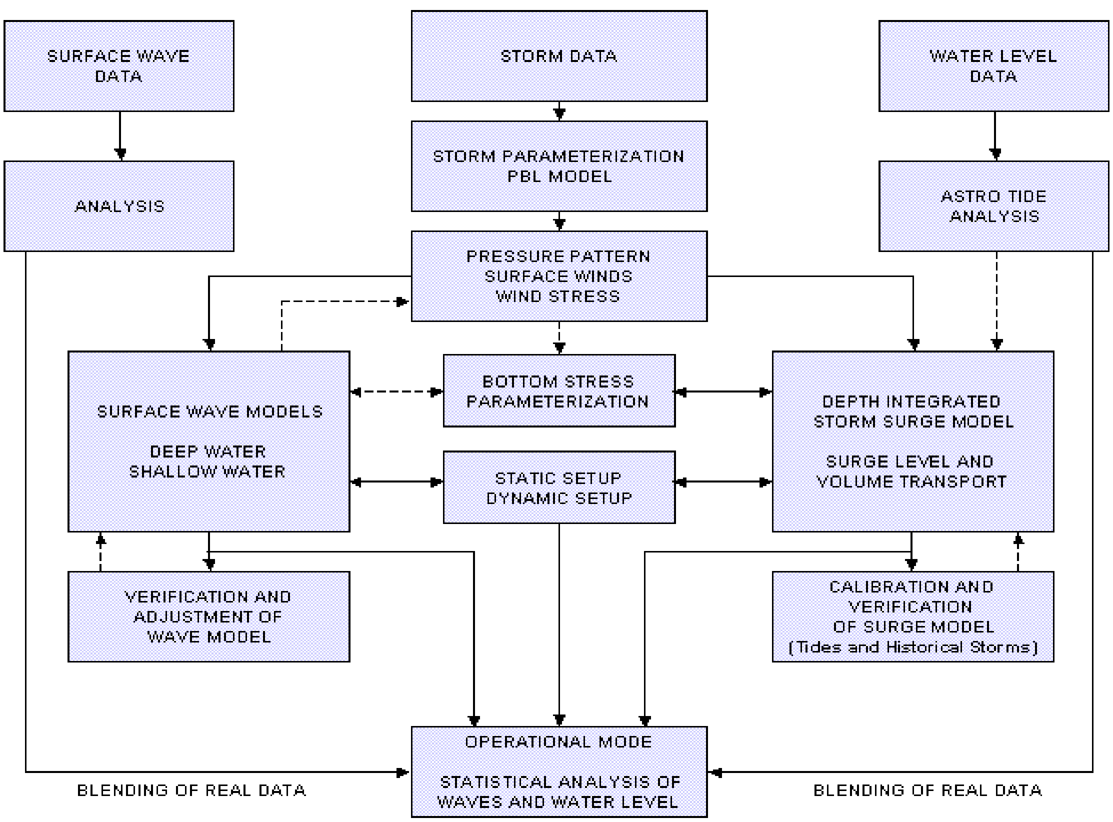


**Figure 1. Illustration of the primary aspects of surge modeling.**

The static component of setup is, strictly speaking, a stillwater component generated by radiation stress rather than wind stress. The development and application of methods that unify the setup and surge computation remain tasks for the future, however, and are discussed in the separate Focused Study Group report on Wave Setup.

It is proposed that new guidelines should include general guidance regarding these factors. Several candidate storm surge models are in current use or development that might be accepted by FEMA for future storm surge studies. It would not be the intention of the proposed work to evaluate specific models or to attempt to describe the details of use of any of those models at this time, because model documentation and user's manuals are not available at the present time.

Instead, the proposed new guidelines would involve the development of more general, high-level guidance incorporating explanatory discussions of modeling factors that should be understood and considered by a study contractor or a FEMA project officer. Among these factors (in italics) are the following:

- The *governing equations* of the model, typically the nonlinear long wave equations accounting for conservation of mass and momentum, with surface wind and barometric pressure terms representing the influence of the storm

- The *numerical scheme* used by the model, whether finite differences computed on a grid of rectangular cells (commonly of fixed size) or in curvilinear coordinates, or finite elements represented by triangular or quadrilateral cells (of varying sizes); the numerical scheme may also be explicit or implicit, affecting time step constraints

- The *flooding/drying* treatment of cells as the flood advances onto land and then recedes

- The *storm representation*, such as a planetary boundary layer model (for a hurricane) or a simpler empirical/parametric description, including both wind and pressure; the storm representation will be quite different for hurricanes, northeasters, and Pacific storms, although the modeling principles remain the same in each case; on-land filling will be significant for sheltered waters

- The *wind stress coefficient*, which relates the wind speed at the surface to the stress felt by the fluid

- The *sheltering treatment*, adjusting the effective wind stress to account for partial reduction by tall vegetation, terrain, and structures (especially significant for sheltered waters)

- The offshore *bottom friction* treatment over the relatively smooth ocean or bay bottom, which retards the flow

- The onshore flow resistance treatment accounting for *bottom friction* and *resistance* offered by tall vegetation and structures (critical for sheltered waters)
- The *source and quality of bathymetric data*, defining the varying depths at the site
- The *source and quality of topographic data*, such as traditional quad sheets or newer LIDAR data
- The manner in which normal *storm erosion* alters the topography used in the model
- The manner in which *catastrophic erosion* might affect the modeling assumptions, in the event of loss of a major barrier to inland flooding
- The *representation* of the bathymetry and topography in the model grid system, which depends on the numerical scheme
- The *faithfulness of the grid* to the irregular bathymetry and terrain, including conformance to boundary shapes and inclusion of small sub-grid barriers
- The *resolution of the grid*, whether fixed or varying through the study area
- The *boundary conditions*, which impose approximate rules along the edges of the model area, both offshore and onshore, permitting termination of the calculations at the expense of accuracy
- The *treatment of astronomic tide*, which might be handled as part of the simulation through the boundary conditions or treated as an added effect separate from the surge simulations; if the computational domain is large, tidal potential terms need to be accounted for in a simulation
- The *types and limits of calibration* that might be done, including small-amplitude astronomic tide reproduction, for which calibration data are reliable
- The role of *verification hindcasts* to confirm the apparent reasonableness of the final model when compared with historical surge records
- The role of *wave setup* (a separate topic in this guideline development project)
- The general manner in which *surge statistics* are generated from multiple surge simulations (the subject of Topic 50 of the separate Storm Meteorology effort)

These guidelines will be developed through review of the storm surge literature and consultation with developers and users of major storm surge models. Although hurricanes are usually the focus of this discussion, northeasters are also to be included in the guidelines. Numerical hydrodynamic modeling, and the use of tide gage analysis, will be addressed as envisioned below in Topics 54 and 55 for the Pacific Coast.

No new methodology development is proposed, with one possible exception. The problem of catastrophic erosion of a coastal dune should be considered as a special modeling problem. Consider an embayment and an inland region protected from surge by a high dune ridge. In previous FEMA surge modeling, such dunes may have been overtopped in the course of a simulation, but were treated as being simply submerged. However, as happened near Hatteras during Hurricane Isabel, overtopping can lead to washout of a considerable portion of the dune, creating a new inlet and permitting a sudden large increase in flood penetration not envisioned in the model. For Pamlico Sound, this may or may not have been significant for the overall determination of surge, since the sound is so large that the additional flow occurring during the few hours of high storm tide may not have appreciably affected sound-side water levels. However, a similar circumstance could make a significant difference in a region with a high barrier protecting low, developed areas. (It is noted that Scheffner, in a study for Fire Island to Montauk Bay, included erosion and breaching of the barrier island as part of a surge simulation and found a significant effect in Great South Bay.) This is not a deterministic process, although it is a frequent event during very large storms. It is proposed that its importance to storm surge modeling and stillwater determination be assessed and that, if it is found to be significant, then suggestions for future study beyond the present scope should be developed.

The style of the proposed guidelines will be consistent with the general approach of the existing guidelines, although more descriptive than prescriptive. Topic discussions may be illustrated by examples drawn from past surge studies performed with both finite difference and finite element models (perhaps both the FEMA surge model and the newer ADCIRC model).

The existing guidelines described above are primarily concerned with documentation and interim review of the storm surge modeling effort. That material, added to the guidelines in 2002–2003, was a significant improvement over the original 1995 draft, which was essentially mute on surge modeling.  The proposed guidelines would preserve and refine the 2002–2003 documentation and review sections of the most recent existing guidelines.

### *Extremal Analysis of Tide Gage Data*

Although the discussion above assumes only two-dimensional (2-D) hydrodynamic modeling would be used to determine storm surge levels, the direct use of tide data is another approach that must be considered. As will be discussed below for surge estimates on the Pacific Coast, it is possible to extract stillwater data from tide gage records by subtracting the known astronomical component. The residual data represents the contribution of all other low-frequency (i.e., stillwater) processes, including wave setup, although it should be noted that owing to large spatial variability, the setup captured at the gage may not be representative of setup in even relatively nearby areas.

With a sufficient period of record (the existing guidelines mention 20 years), an extremal analysis of the residual record after tide removal can be used to estimate the 100-year stillwater level at the gage site. Consideration of this approach will be included in the work outlined below for Topics 54 and 55. As discussed there, limiting factors include the quality and duration of the available data and the possibility of significant spatial variation with increasing distance from the gage site. The new work will include reconsideration of the required period of record as it affects confidence levels. The general approach to this task is not unlike extremal analysis in other hydrologic applications, including the problems of selecting an appropriate idealized probability distribution function, such as an extreme value distribution, and a method (e.g., moments, maximum likelihood) of determining the parameters of that distribution based on the local data sample (which could be the annual series of peak events). Many approaches are possible, with a great variety of choices of specific procedures. The proposed work will evaluate these alternatives and specify recommended procedures.

### *Evaluating the Accuracy of Storm Surge Estimates*

A perceived need in the present coastal flood study program is a way to determine whether or not an existing study gives a reliable 100-year estimate, or whether a restudy that uses newer assumptions or tools is warranted. This is a difficult question, especially on the Atlantic and Gulf coasts where hurricanes are the dominant flood contributors, because hurricanes are extremely sporadic and variable, and because mapped flood levels cannot be identified with any particular storm. Many agencies have different purposes and numerical modeling approaches for evaluating hypothetical storm effects which may also confuse this issue. For example the National Oceanic and Atmospheric Administration (NOAA) prepares hurricane evacuation maps, which depict the inundation of particular hypothetical storms defined by storm track and a Saffir/Simpson rating. The purpose of these maps are different than those needed for FIS studies.

In performing these surge studies, there is little opportunity for so-called model calibration. Beyond minimal calibration of ordinary small-amplitude conditions based on the simulated behavior of astronomic tide, for example, storm surge models are relatively closed-box affairs, assumed to be pre-wired with all the essential physics of the flood processes. In any case, the basic requirements for calibration are rarely well satisfied. To calibrate, one needs accurate knowledge of both the forcing disturbance (the storm) and the basin response (the resulting high water); neither of these are abundant for hurricane surge, although data are available from long-term National Ocean Survey (NOS) stations, publications such as *Characteristics of the Hurricane Storm Surge* (Harris, 1963), and in a variety of reports from the U.S. Army Corps of Engineers (USACE). Storm details are not known with any great accuracy because storms can fluctuate rapidly in size and intensity, and may appear chaotic when compared with the idealized representations used in models. Similarly, the basin response is seldom known with accuracy at more than a very small number of points inside surviving structures and at tide gages; highwater marks obtained in open areas may be contaminated with an undetermined amount of runup and setup. Gages commonly fail during the most significant events; for example, the gage at Duck Pier, North Carolina, failed just as the surge from last year's Hurricane Isabel began to rise.

In view of these twin deficiencies, robust calibration of a storm surge model is not a common option in a FEMA study. To calibrate a model against typical storm and high water data (for example, by adjusting the wind stress coefficient) would be to build a systematic error into the model that cancels the unknown random errors in the storm description and flood observations. This systematic error would then be imposed on all subsequent simulations made during the development of the surge statistics. In lieu of calibration, modelers perform model validation tests by hindcasting historical storms to ensure that the model produces results that are in qualitative, if not quantitative, agreement with observations. With the luxury of several storms, the modeler might simply hope to be high in some cases and low in others. Still, without a real calibration, it is reasonable to question whether the basic hydrodynamic model might contain a systematic bias, either high or low, affecting all simulations that contribute to the 100-year determination.

After a study has been completed and mapped, new storms will eventually occur at the site and will inevitably be compared with the study. If a storm produces elevations less than those mapped, the conclusion might be reached that it simply was not a 100-year storm because weaker storms occur all the time and so are not surprising. Of course, the entire past history at the site can also be compared with the mapped levels. If the record contains no severe events, then the temptation might be to assume that the study was biased to the high side. Conversely, if a new storm creates levels above those mapped, then it is very likely that the accuracy of the study will be questioned. Worse, if two or more such strong storms occur within a few years after the study, or if the record at the site contains several such events, then it may seem natural to conclude that the study was biased to the low side, is understating the hazard, and should be redone.

This reasoning is not decisive, however, and (when clarified) suggests a way to test the accuracy of the existing 100-year coastal flood levels, and perhaps to help perform a *global* calibration, where a local calibration had been impossible. The key observation is that random events do not occur more or less uniformly over their domain, but instead must exhibit predictable irregularities of occurrence. In the case of floods observed at a large number of sites, some sites must be found that have gone for extremely long periods without experiencing a severe event, whereas other areas must have experienced multiple severe events. There must be “good luck” and “bad luck” communities. If the mapping were to be fine-tuned so that experience and mapping were highly consistent throughout, then the mapping would be flawed.

This suggests the possibility of a statistical test of the reliability of the existing 100-year values, which might proceed along the following conceptual lines. Imagine that the coastline were divided into a series of zones, each large enough so that floods within them could be considered statistically independent—i.e., large enough that a particular storm tends to affect only one such zone, yet small enough that occurrence of a 100-year event affects the majority of the zone. Considering floods of 100-year magnitude, the zone size might be on the order of the radius of maximum winds typical of an area—perhaps just a few tens of miles. This would suggest on the order of 100 zones covering the entire area of the Gulf and Atlantic Coasts.

Next, imagine that, for all zones, there are N years of historical flood data (high water marks). In any given zone, there is a certain probability of having experienced no event exceeding the 100-year level during those N years, another probability of having experienced one such event, or two, or three, and so forth. From these considerations, one can estimate how many of the conceptual coastal zones should have experienced 0, 1, 2, ... floods exceeding the 100-year level in the N years of record. These expected numbers can then be compared with observation. If it were found that the count of observed exceedances was significantly greater than expected, then one would suspect that the mapping systematically understates the flood hazard. Conversely, if the count of exceedances was substantially less than expected, the mapping might be suspected to overstate the hazard.

Had all studies been performed in a systematic way using exactly the same surge modeling techniques, one could imagine performing a global calibration of the model to raise or lower the general levels of the mapping, in order to achieve a reasonable fit between the observed and expected rates of extreme occurrences. In reality, the existing flood studies were not all performed in a systematic way, even when the same surge model was used—different Study Contractors undoubtedly made differing assumptions that would affect the homogeneity of the data used in this conceptual approach. However, a statistical review (such as that recommended above) might help reveal such anomalous local studies, which would be identified as zones of inconsistency with adjacent zones.

This section discusses how an approach might be developed. There are difficulties with the zone idea (presented as a conceptual aid), especially in the definition of such zones (large enough to ensure independence, yet small enough to respond as a unit to the 100-year flood). Consider, for example, a strong alongshore storm that could affect a long stretch of coast, and so violate the independence assumption. For the present, we propose only to investigate (in consultation with a statistician such as Professor Borgman [University of Wyoming]) whether such an approach could prove fruitful and, if so, to outline specific methods for future work. A substantial portion of the effort required in this task would be the identification of suitable data sources. The immediate effort described above remains in the *critical* category; if successful, the follow-on effort would be categorized as *important*, requiring a longer performance period than is presently available.

### *Regional Modeling*

In early FEMA storm surge studies, it was common to perform a separate study for each county. One major reason for this was limited computer capacity, which severely restricted the grid sizes that could be accommodated in even the largest machines at the time. For example, even the vaunted CDC 7600 supercomputer had only 64K words of small-core memory and 512K words of large-core memory, with comparably limited disk storage capacity, and a 36 MHz clock speed (1% of the speed and capacity typical of desktop personal computers today). Use of the CDC 7600 typically cost on the order of $1 per second. Because each study area was restricted in size, many separate studies were required; because computing costs were high, the original coastal flood studies were extremely expensive (typically involving computer charges of about $100,000 per county).

With tremendous recent advances in computational power, as measured by both speed and capacity, many of those early modeling constraints have been eliminated, and direct machine charges are now negligible (although proprietary modeling software may be a substantial cost). This suggests that it might be preferable to plan future surge modeling efforts on a regional, rather than a community, basis.

It is proposed, therefore, to provide general guidance on factors that should be considered in scoping a regional modeling effort. In particular, surge modeling is significantly challenged with the problem of boundary conditions. Performing a regional study encompassing many counties would not only reduce costs, but also enhance modeling accuracy by greatly reducing the number of problematic boundaries. Furthermore, through judicious placement of the regional study boundaries, difficult open-water boundaries may be traded for more tractable land boundaries. Recent work of a regional nature includes studies of the coast of South Carolina and Texas from Sabine to San Luis Pass (Scheffner et al., 2001, and in prep.)

#### 2.1.4 Recommendations

It is recommended that four distinct tasks be undertaken in response to Topic 53. The first is a general review of storm surge modeling requirements from the perspective of FEMA and coastal flood insurance studies, leading to the development of a set of broad guidelines for conducting storm surge studies. This will require an assessment of many factors that go into conducting a surge study, ranging from the inherent abilities and limitations of numerical surge models to practical considerations of model selection and implementation in particular cases. The guidelines should include illustrative materials drawn from past studies and an annotated bibliography as a resource for more detailed study. It is beyond the scope and intent of the proposed work to evaluate the merits of particular models; that effort will remain separate as part of FEMA’s accepted models review process, although the material developed in this study will help to provide a framework for that determination.

The second recommendation is for an outline of procedures to extract stillwater data from tide gage records. This overlaps with Topics 54 and 55 for the Pacific Coast, including Non-Open Coast regions, and is discussed in the following section.

Third, the Focus Study Group recommends an effort to develop a global method to assess the accuracy of FEMA’s coastal storm surge studies. The random and sporadic nature of local surge history makes it difficult to determine whether coastal maps are appropriate. Recent catastrophic events may be given more weight than they deserve, since it is to be expected that several events exceeding local determinations must occur at some locations over an interval, while a lack of extreme events should characterize other areas. By considering the global history over the entire length of the U.S. coastline, it may be possible to determine whether the established coastal elevations are exceeded more or less frequently than expected for the assumed case of accurate maps and random local experience.

The fourth recommendation is for development of guidance regarding study planning—in particular, how studies might be grouped regionally to minimize costs while at the same time improving accuracy. Whereas existing FEMA studies were typically performed on a county-by-county basis, the enormous advances in modeling technology over the past 20 years now permit much greater flexibility in model design. Multi-county and statewide (or larger) efforts are entirely feasible, and may also result in improved accuracy of results.

### 2.1.5 Related "Available" and "Important" Topics

Table 3 at the end of this report presents estimates of times required to accomplish the tasks in this topic.

## 2.2 TOPICS 54 AND 55: PACIFIC STORM SURGE (INCLUDING NON-OPEN COAST)

### 2.2.1 Description of the Topic and Suggested Improvement

Storm surge is of smaller magnitude on the Pacific Coast than on the Atlantic and Gulf coasts and so may not commonly require a detailed numerical model to obtain reasonable estimates. Instead, it may be possible to derive estimates of storm surge from tide gage records or simplified computations. When a 2-D hydrodynamic modeling effort is required, the proposed guidelines discussed above for Topic 53 will be appropriate, provided that the selected surge model has the capability to represent the wind and pressure fields appropriate to the Pacific Coast.

### 2.2.2 Description of Procedures in the Existing Guidelines

No specific guidelines have been identified for Pacific Coast storm surge, although the Atlantic and Gulf Coast guidelines discussed above are generally applicable. That is, basic numerical modeling considerations will be the same, although site-specific differences including especially the wind and pressure model must be accounted for.

### 2.2.3 Alternatives for Improvement

***Tide Gage Analysis***

Instead of the storm surge modeling discussed above in Topic 53, an alternate approach is to derive the 100-year stillwater estimate from an analysis of historical data. For this purpose, a wealth of tide gage data are available for coastal stations on both the Pacific and Atlantic/Gulf Coasts. The NOAA CO-OPS data archive (http://co-ops.nos.noaa.gov/data_res.html), for example, includes 117 coastal gages with 25 or more years of data. These data, by their nature, include all stillwater components but do not include the higher frequency wave effects, which are not appropriate to use in a stillwater determination.  The stillwater components captured in the gage data include storm surge (i.e., wind setup and pressure effects), wave setup, tsunamis, astronomic tide, and possibly a freshwater contribution from stream discharges. Most gages are located in protected. Sheltered Waters areas in bays and harbors and areas on the open coast without gage data will be discussed later. Owing to the spatial variability of wave setup, it is noted that although the local setup is captured in gage data, it may not be representative of other, relatively nearby areas.

The portion of the record attributable to astronomic tide is considered to be reasonably well known for each gage site by previous determination of the local tidal constituents. This fact makes it possible to compute the expected tide contribution at any time and then to subtract it from the record, leaving as the difference the sum of all other stillwater contributors. In this approach, wave setup is automatically included with the storm surge component, unlike present surge modeling practice, in which surge and setup are computed separately and appropriately added. In fact, all long-period processes, including tsunamis, are automatically included.

After subtracting the predicted tide from the gage records, an extremal analysis can be performed on the residual data to estimate the local 100-year level. The quality of this estimate will depend on both the reliability of the data and the duration of the record. Examples of the available NOAA CO-OPS data for two storms are shown in Figures 2 and 3.

Figure 2 shows data recorded during a January 1988 storm in Southern California, which is thought to approximate the 100-year (or greater) event; despite the severity of the storm, the storm tide component is seen to be quite small. Figure 3 shows data recorded at San Francisco during a 1998 storm. In this case, the water level was elevated above the expected tide by about two feet at the Presidio tide gage. Part of the anomaly (residual) was attributable to the El Niño climatic condition, which was strong in winter 1997–98. Water levels in the vicinity were elevated an average of one foot for the entire winter. It is interesting to note that Sausalito is within 5 miles of the Presidio tide gage but experienced noticeably higher stillwater levels. The additional elevation was probably caused by local wind setup induced by strong southeasterly winds, and by rainfall runoff entering San Francisco Bay from upstream drainage basins, including the Sacramento and San Joaquin Rivers (Philip Williams and Associates Ltd. [PWA], 2002). This demonstrates that local variability may be substantial in large sheltered-water embayments, so that direct use of gage data may be limited to the near vicinity of the gage. It is noted, however, that in large sheltered waters where gage data is not comprehensive, and where the simplified one-dimensional (1-D) storm surge model discussed below is not appropriate or adequate, the full capability of one of FEMA’s approved 2-D surge hydrodynamic models can be used to determine surge behavior and statistics.

The proposed task is threefold: to identify candidate sources of appropriate tide data, to examine a sample set to determine the extent to which the candidate sources can be used for flood insurance studies, and to estimate the reliability of the derived 100-year flood elevations. As discussed above, the methods of data analysis are similar to the analysis of other stochastic hydrologic data, including selection of an appropriate probability distribution function, determination of distribution parameters from the site sample, and so forth. Reliability considerations will include not only sample error associated with the duration of the record, but also the potential significance of variability near the site. This is particularly important in sheltered waters where tidal hydrology can vary substantially with location. Recent FEMA flood studies in Puget Sound/Strait of Georgia provide examples of tide gage data analysis approaches (PWA, 2002). Previous baywide studies have also addressed the distribution of high waters using tide gage data (U.S. Army Corps of Engineers [USACE], 1984). The importance of variability is not limited to sheltered waters, however. For example, the contribution from wave setup can vary rapidly from place to place, even along the open coast. The suggested effort will also provide case study examples for inclusion in the proposed guidelines.

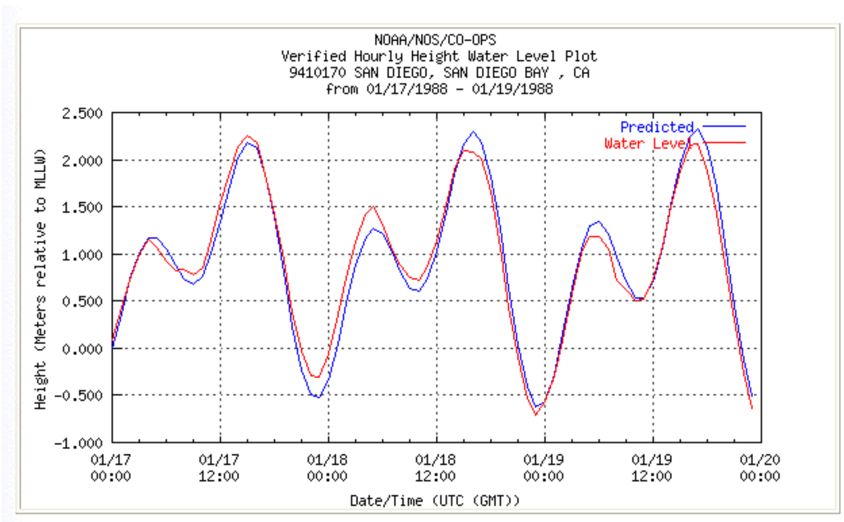


**Figure 2. Sample comparison of predicted and recorded tides during a severe storm at San Diego.**

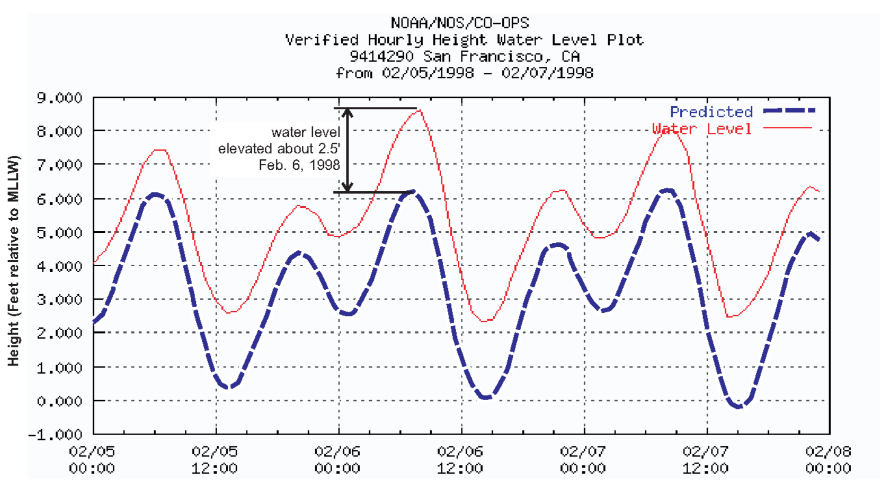


**Figure 3. Sample comparison of predicted and recorded tides during a severe storm at San Francisco.**

### Simplified Surge Modeling

Where adequate records are not available, such as on the Open Coast in areas without gage sites, more traditional efforts such as numerical simulation of surge, wave hindcasts, tsunamis, and combined probability studies may be necessary. However, because the surge component is expected to be relatively small, it may also be possible in many cases to derive estimates of sufficient accuracy from simplified computations. This might be done, for example, following the approach used by the Florida Department of Environmental Protection (FL-DEP) for determining coastal construction control lines.

The FL-DEP applies a storm surge calculation approach that uses both 1-D and 2-D storm surge models (see, for example, Dean et al., 1992). A primary benefit of this approach is the fact that a very large number of simulations (including an appropriate representation of astronomical tide) can be made at minimal cost, from which the 100-year surge levels can be derived. The 2-D model is applied for verification of historical storms and for calibration of the one-dimensional model. Once calibrated, the 1-D model is used for the numerous production runs.

A flow chart of the procedure, taken from a FL-DEP study, is presented in Figure 4. Any valid 2-D model, such as the FEMA Surge Model or ADCIRC, could be used, although the FL-DEP uses a variable-grid explicit-implicit model that allows for overland flooding. The 2-D model is first applied for comparison with historical storm data (although the chart specifically mentions hurricanes and factors specific to the source study, the procedure would be modified to use Pacific storms for West Coast applications). Generally, no adjustments are made to the 2-D model, which is used at this stage primarily for validation and/or to estimate the degree to which it agrees with the historical data.

Following the verification stage, the 2-D and 1-D models are run for a common set of storms with ranges of storm parameters bracketing those anticipated to produce the 100-year surge. For various classes of storms, correlations are developed between the 2-D and 1-D generated maximum surges in the linear form:

$$(\eta_{\max})_{2-D} = m(\eta_{\max})_{1-D} + b \tag{1}$$

An example result is shown in Figure 5 for landfalling hurricanes on a particular transect (profile) in Palm Beach County, Florida.

It should be noted that the average difference between the 1-D and 2-D simulations in this example is only 7%, and that this is the level of difference found in the FL-DEP study for Palm Beach County, Florida.  For the hurricane surge conditions in Florida, a difference of this magnitude approaches 1 foot and so is significant. However, the situation is quite different for the Pacific Coast.

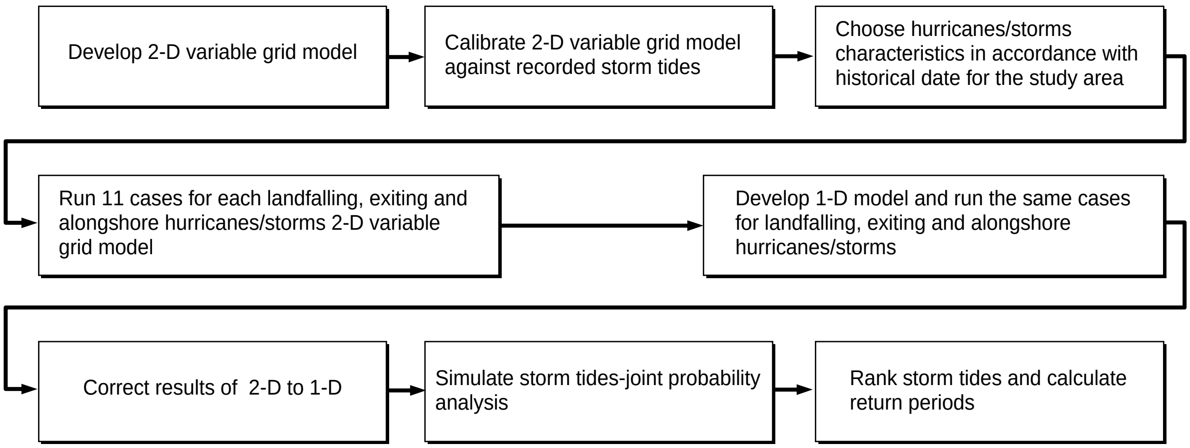


**Figure 4. Flow Chart of Florida Department of Environmental Protection**

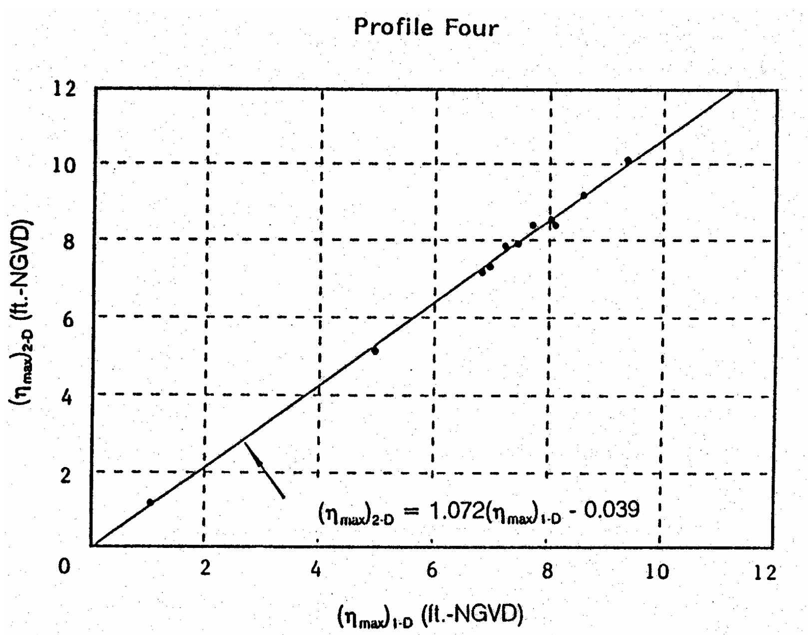


**Figure 5. Example of correlation between one-dimensional and two-dimensional numerical surge models.**

If the expected 100-year surge at a Pacific Coast site is on the order of 2 feet, then a 7% difference would amount to less than two inches. In other words, the added effort and cost of 2-D simulations might not generally be needed on the Pacific Coast, unless the 1-D estimate was calculated to be more than, for example, 7 feet, corresponding to a 0.5-foot error. Tests would need to be done for a few typical Pacific Coast conditions (bathymetry and wind fields) to verify the degree of 1-D model accuracy and to provide guidance about when additional 2-D simulations would be needed.

An attractive feature of the FL-DEP approach is that using such an efficient and economical 1-D surge model makes it possible to handle the combination of surge and tide in an extremely natural way. The procedure, discussed under Topic 51 of the Storm Meteorology Focused Study Report, is to randomly choose a different tide history (drawn from the peak storm surge season) to be used as the seaward boundary condition for each 1-D simulation. That is, to determine the 100-year surge, one simulates a large number of storms with different combinations of characteristics drawn from the local storm population. For each of these storms, a starting time is chosen at random from the appropriate storm season. Then the nearshore tide variation is determined, starting at that time and continuing for the duration of the surge simulation. By taking this time-varying random tide segment as the boundary condition, the influence of that tide is accounted for. By repeating this for many hundreds or thousands of storm simulations (fast and inexpensive with a 1-D model), all likely tide amplitudes and phases are reflected properly in the results.

### 2.2.4 Recommendations

The Focused Study Group's recommendations consist of two major tasks. The first is to establish procedures for extracting the required surge data from tide gage records and prepare corresponding guidelines for Study Contractors. Recent flood studies in Puget Sound/Strait of Georgia (Region X) can be used as examples of analysis methods (PWA, 2002). This does not require the development of any fundamentally new methodology. However, it will be useful to clearly lay out the procedures for Study Contractors and it will be necessary to identify data sources and perform test studies to verify the suggested procedures and assess limitations of the approach. Discussions of limitations will include statistical limits inherent in the varying lengths of available data records. Separate discussions and guidance should be developed regarding the physical limitations and temporal and spatial variation often found within large bays and sheltered waters. The guidelines to be developed should include illustrative examples drawn from the test studies.

The second major task will be to develop procedures for surge estimation in areas for which an adequate tide gage record does not exist, including most Open Coast areas. Procedures for defining the modeling domain and selecting an appropriate model will be presented. When warranted, the detailed numerical modeling methods used for hurricane studies on the Atlantic/Gulf Coasts would also serve for the Pacific Coast, as long as the adopted numerical models are able to properly simulate Pacific Coast wind and pressure fields. However, because surge is much smaller on the Pacific Coast than on the Atlantic/Gulf Coasts, simplified methods may suffice. In particular, the use of a 1-D surge model may be adequate for most cases, minimizing the costs of model implementation and simulation. An assessment of storm meteorology and data sources would be necessary to determine the best manner for specifying winds and pressures and their associated frequencies. Test studies should be performed at selected sites to verify the feasibility of the recommended approach. New guidelines summarizing the procedures would be developed, including illustrative examples.

### 2.2.5 Related "Available" and "Important" Topics

Table 2 at the end of this report presents estimates of times required to accomplish the tasks for these topics.

# 3 AVAILABLE TOPICS

## 3.1 TOPIC 52: STILLWATER NON-STATIONARY PROCESSES

### 3.1.1 Description of the Topic and Suggested Improvement

The task identified under Topic 52 is a straightforward effort to provide guidance alerting a Study Contractor to the possible importance of non-stationary (or non-steady) processes in a study.  The guidance might include, relative sea level rise, tectonic uplifting, land subsidence, or a combination of these processes (effective elevation change).  These might need to be accounted for in the interpretation of historical data, whereas ongoing subsidence would need to be considered for its immediate impact on a new study and discussed with the FEMA project officer.  The effort suggested here is primarily one of providing guidance alerting the user to these possibilities and advising on the availability of suitable data. In addition to relative sea level changes, changes in winds and waves and other climatic features should be addressed. These aspects have been summarized in several books and papers by Komar, including the individual processes of sea level rise, uplift, and subsidence and the effects of combining these, including data and statistics for areas on the Pacific Coast (Komar, 1998, 1988, and 1997).

### 3.1.2 Confirm "Availability"

Both sea level rise and land elevation changes (uplift and subsidence) contribute to relative sea level changes; a great deal of data and data summaries exist for both of these processes. For example, the Philadelphia District of the U.S. Army Corps of Engineers maintains a web page (www.nap.usace.army.mil/cenap-en/slr_links.htm) with links to numerous government data sources for sea level change, including the NOAA CO-OPS *Sea Levels Online* site. NOAA has determined the rate of mean sea level rise/fall for 117 long term water level stations and, from these, has determined trends, seasonal cycles, and interannual variations caused by fluctuations in ocean conditions, including El Niño effects. Figure 6 indicates the distribution of those study sites and the approximate magnitudes of the long-term trends that have been determined.

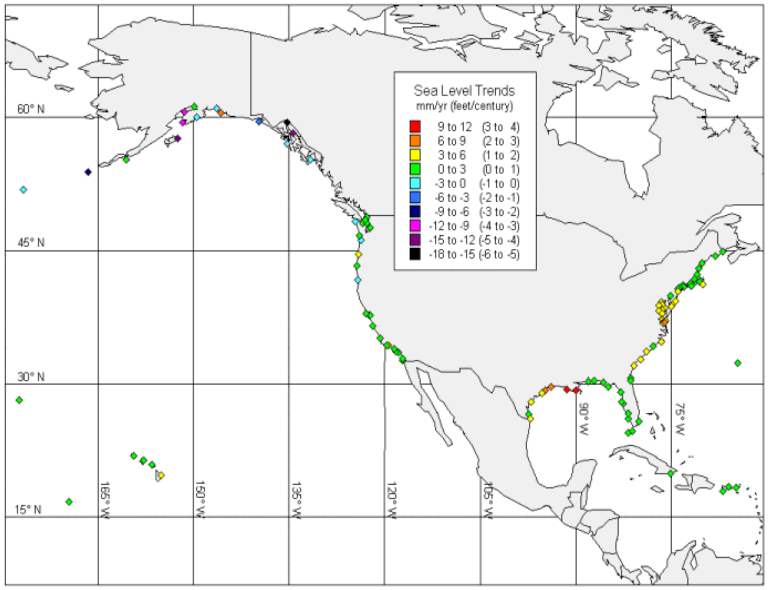


**Figure 6. Observed sea level trends along U.S. coastlines.**

The estimated trends in many regions along the Pacific Coast are seen to be small and may have little importance for flood insurance studies; however, as noted below, it may still be valuable to document the changes and indicate their significance as part of a flood insurance study.

Land subsidence may be more significant than area-wide sea level change for many study sites. The U.S. Geological Survey (USGS) and other sources have documented land subsidence throughout the United States, although subsidence is frequently a very local result of groundwater extraction or oil and gas extraction. Along the Pacific Coast, however, significant tectonic uplifting occurs as a result of regional geologic processes and active plate tectonics. Consequently, despite the great quantity of large-scale data, it will still be necessary for Study Contractors to explore local data sources to identify local problems and determine whether such effects merit discussion with the responsible FEMA program manager. Such sources of information would include discussions with and information from community officials, resource agencies, and local surveyors. New guidelines should identify the major national and regional data sources and provide general advice regarding ways to locate local data. (For example, see San Francisco Bay Conservation and Development Commission, 1987).

### 3.1.3 Availability—Other Factors

There are indications within the literature that weather patterns are changing, and these could have an impact on the interpretation of flood studies and study data. For example, recently revised historical wind patterns that were undertaken for GROW (Global Re-analysis of Ocean Waves; see, for example, Cox and Swail, 2001) appear to show increasing winds and wave heights in the North Atlantic. However, the real increases in winds may be at least partially the result of the fact that measurement instruments and techniques have changed with time (for example, anemometers on modern ships are at a greater elevation, above the standard 10 meter elevation, than was the case on older vessels). Efforts have been made to account for such effects, but it has not always been possible to determine the actual measurement conditions. Another confounding factor is that the wind measured at most offshore data buoys is at elevation 5 meters rather than the traditional standard of 10 meters.

Additional factors, such as variations in solar (sun spot) activity and El Niño cycles, can also be considered as potentially significant non-stationary factors. However, a database of 20–25 years (the minimum desired to estimate the 100-year event with confidence for FEMA studies) for a process that might be affected should already include the net effects of such phenomena. Study Contractors should be aware of these factors and avoid confusing such cyclic non-stationary influences with other hydrometeorologic processes.

Although standard FEMA practice is to address current conditions only, it could also be appropriate to identify and discuss periodic seasonal changes (such as significant El Niño oceanic conditions) and future changes arising from other significant non-stationary contributions. In a 1991 FEMA report titled *Projected Impact of Sea Level Rise on the National Flood Insurance Program*, for example, the potential impact of rising sea levels was investigated. It was concluded, at that time, that a relative sea level rise of up to 1 foot could be tolerated without major impact, but that a longer term rise of 3 feet would have severe financial consequences. Such background discussion might be appropriately included in the guidelines, even if not deemed essential to performance of a study.

More directly pertinent to a study would be an effort to document the expected magnitudes of non-stationary effects, even though small, and to estimate their projected impact over time; if nothing else, this might allay concerns and questions. If a linear trend were assumed for sea level rise, say, one could easily prepare a table for a given study site showing how the BFEs would change were the trend to continue. With time, the 100-year level would rise in approximately the same way as sea level (as long as the change is small), so that the 100-year level as determined by the study would be a more frequent event at any future date. Were the projected rate of rise to be 2 feet per century, for example, then after ten years (well within the life of a typical flood insurance study) the true BFE would have risen 0.2 foot and the mapped flood would have declined from the 100-year level to, say, the 90-year level. These magnitudes may not be critical in most areas, yet their documentation as part of a study might be useful to both FEMA and the communities.

### 3.1.4 Related “Available” and “Important” Topics

Table 3 at the end of this report presents estimates of times required to accomplish the tasks in this topic.

# 4 IMPORTANT TOPICS

None identified.

# 5 ADDITIONAL OBSERVATIONS

None.

# 6 SUMMARY

The Stillwater Focused Study addressed two broad topics: non-stationary processes such as effective sea level rise, and storm surge issues. Non-stationary processes (Topic 52) are categorized as Available; the primary effort will be to identify data sources, provide a discussion of ways in which non-stationary processes relate to flood insurance studies, and provide guidance to Study Contractors regarding their possible significance in a study and what material should be presented to FEMA for consideration.

The storm surge issues are divided into modeling factors for the Atlantic/Gulf Coasts (Topic 53) and alternate and/or simplified methods for the Pacific Coast (Topics 54 and 55), where surge is of less consequence. The primary effort recommended for the Atlantic/Gulf Coasts is to write detailed guidelines regarding storm surge and storm surge modeling, including discussions and recommendations for the numerous factors that affect a modeling effort. A secondary effort will be to review existing and planned coastal studies to suggest how regional study efforts might prove more economical and more accurate than county-by-county studies, as has been the usual practice. A final recommendation is to investigate ways to assess the accuracy of existing and future coastal studies, including a global statistical review and comparison of mapped BFEs with the historical record.

Table 1 summarizes the Stillwater Focused Study topics and recommendations. Table 2 presents a preliminary estimate of time necessary to complete recommended tasks.

**Table 1. Summary of Findings and Recommendations for Stillwater**

| Topic Number | Topic | Coastal Area | Priority Class | Availability / Adequacy | Recommended Approach | Related Topics |
|---|---|---|---|---|---|---|
| 52 | Non-Stationary Processes | AC | A | Y | Identify and summarize data sources for sea level rise and land subsidence and/or uplift; provide basic guidance regarding significance of non-stationarity in flood insurance applications; include guidance on interpretation of historical data. Suggest documentation of projected map impact | -- |
| | | GC | A | Y | | |
| | | PC | A | Y | | |
| | | SW | A | Y | | |
| 53 | Storm Surge Modeling | AC | C | MAJ | Develop overview guidance for surge modeling; define procedures to assess accuracy of surge estimates ; suggest regional modeling approaches for study economy | 6<br>44-48 |
| | | GC | C | MAJ | | |
| | | PC | -- | -- | | |
| | | SW | -- | -- | | |
| 54 & 55 | Pacific Coast Storm Surge | AC | -- | -- | Identify tide gage data sources; develop procedures for surge extraction from tide gage records for FIS use (including test studies); develop simplified numerical modeling method for areas without data (1-D Pacific Surge Model) | 6<br>44-48 |
| | | GC | | | | |
| | | PC | C | MAJ | | |
| | | SW | C | MAJ | | |

Key:
Coastal Area
AC = Atlantic Coast; GC = Gulf Coast; PC = Pacific Coast; SW = Sheltered Waters
Priority Class
C = critical; A = available; I = important; H = helpful
Availability/Adequacy
"Critical" Items: MIN = needed revisions are relatively minor; MAJ = needed revisions are major
"Available" Items: Y = availability confirmed; N = data or methods are not readily available
"Important" Items: PRO = procedures or methods must be developed; DAT = new data are required; PRODAT = both new procedures and data are required

| Table 2. Preliminary Time Estimate for Guideline Improvement Preparation | | |
|---|---|---|
| Topic Number | Item | Time (Person months) |
| 53 | **Atlantic/Gulf Storm Surge** | |
| | Develop storm surge modeling guidelines | 6 |
| | Develop guidelines for surge extraction from tide gage data | Allocated under Topics 54 and 55 |
| | Develop approach for global assessment of surge accuracy and identify data sources | 4 |
| | Develop guidance for regional modeling | 2 |
| | TOTAL | 12 |
| 54 & 55 | **Pacific Storm Surge (including Non-Open Coast)** | |
| | Identify sources and assess tide gage data for surge extraction | 3 |
| | Perform test/example studies of tide gage surge analysis including assessment of limitations | 4 |
| | Prepare contractor guidelines for tide gage surge evaluation | 3 |
| | Develop simplified surge model for Pacific coast applications, including frequency methods and identification of input data types and sources | 6 |
| | Perform test/example studies using simplified modeling approach | 4 |
| | Prepare contractor guidelines for the simplified Pacific surge modeling approach | 4 |
| | Total | 24 |
| 52 | **Stillwater Non-Stationary Processes** | |
| | Identify and summarize data sources for sea level rise, land subsidence, and other non-stationary processes | 2 |
| | Prepare study contractor guidelines regarding the significance of non-stationary processes, data sources, and documentation requirements | 2 |
| | Total | 4 |

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# Storm Wave Characteristics

## FEMA Coastal Flood Hazard Analysis and Mapping Guidelines Focused Study Report

February 2005


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**Acronyms**

| | |
|---|---|
| 2-D | two-dimensional |
| ACES | Automated Coastal Engineering System |
| CDIP | Coastal Data Information Program |
| CEM | Coastal Engineering Manual |
| CHL | Coastal Hydraulics Laboratory |
| FEMA | Federal Emergency Management Agency |
| FIS | Flood Insurance Study |
| FNMOC | Fleet Numerical Meteorology and Oceanography Center |
| FNWC | Fleet Numerical Weather Center |
| GROW | Global Reanalysis of Ocean Waves |
| IAHR | International Association of Hydraulic Engineering and Research |
| ICCE | International Conference on Coastal Engineering |
| JONSWAP | Joint North Sea Wave Project |
| MII | Meteorology International, Inc. |
| NASA | National Aeronautics and Space Administration |
| NOAA | National Oceanic and Atmospheric Administration |
| NOAA-NCEP | National Oceanic and Atmospheric Administration-National Centers for Environmental Prediction |
| PWA | Philip Williams and Associates |
| SEMs | Spectral Energy Models |
| SOWM | Spectral Ocean Wave Model |
| SPM | Shore Protection Manual |
| USACE | U.S. Army Corps of Engineers |
| WIS | Wave Information Studies |

# 1 INTRODUCTION

This report provides recommendations approaches for improving or preparing the Guidelines and a preliminary time estimate for the four wave-related categories grouped under the Storm Wave Characteristics Focused Study. The four topics and associated need and priority level, which are “C” for Critical and “A” for available, for each geographical area are shown in Table 1.

**Table 1. Storm Wave Characteristics Topics and Priorities**

| Topic Number | Topic | Topic Description | Priority | | |
|---|---|---|---|---|---|
| | | | Atlantic / Gulf Coast | Pacific Coast | Non-Open Coast |
| 1 | Wave Definitions | Definitions of wave types using contemporary terminology: standardize the terms | A | A | |
| 3 | Storm Wave Characteristics | Conversion from Shore Protection Manual to Coastal Engineering Manual | A | A | |
| 4 | Swell: Open Coast | Swell exposure: Use hind cast databases, select based on evaluation | A *(C)* | C | |
| 5 | Local Seas: Non-Open Coast (Sheltered Waters) and Open Coast | Local seas: Nearshore representation of wind waves rather than offshore hindcast | A *(C)* | C | Atlantic *(A)*<br>Pacific *(C)* |

Key: C = critical; A = available; I = important; H = helpful
(Recommend priority italicized if focused study recommended a change in priority class)

It was clear in the scoping phase of this study that Topic 3 included issues on wave generation, but also on wave setup and wave runup. Wave generation related topics developed under Topic 3 were included under Topic 5 in the Local Seas: Non-Open Coast (Sheltered Waters) and Open Coast. Topic 3 was also considered by the Focused Study Leaders for wave setup and wave runup. Topic 3 was considered under other items, and was not pursued independently. The priority level for Topic 5: Local Seas, was assigned after Workshop 1, in consultation with Focus Study Team Members and Leaders. While an available priority was determined for the Atlantic and Gulf Coasts, the priority may be critical in some circumstances. If so, it is expected that this Focused Study report and the upcoming Pacific Coast Guidelines can be used.

In addition to the categories described above, the group also contributed to the definition of the 1-percent-annual-chance event for coastal flood hazard mapping. The term extreme is used in this Focused Study to indicate an event with a low probability of occurrence. No specific value for the probability is associated with this terminology, other than it has a low probability.

The Topics were re-organized after Workshop 1. The revised grouping, which is used in the remainder of this report, is shown below. This grouping is organized to address regional differences and to address similar topics together. These results are summarized in Table 2.

**Table 2. Revised Wave Characteristics Topics and Priorities (Post Workshop 1)**

| Topic Number | Topic | Topic Description | Priority | | |
|---|---|---|---|---|---|
| | | | Atlantic / Gulf Coast | Pacific Coast | Non-Open Coast |
| 1 | Wave Definitions | Definitions of wave types using contemporary terminology: standardize the terms | A | A | -- |
| 3 | Storm Wave Characteristics | Conversion from Shore Protection Manual to Coastal Engineering Manual | A | A | -- |
| 4 & 5 | Sea and Swell | Sea and Swell for the Pacific Coast | | C | -- |
| 4 & 5 | Offshore Wave | Offshore Wave Data for the Atlantic and Gulf Coasts | C | | -- |
| 5 | | Nearshore Representation of Southern California Bight | | C | -- |
| 5 | Local Sea (Sheltered Water) | Wave Generation in Sheltered Water | -- | -- | Pacific<br>C<br>Atlantic<br>A |

Key: C = critical A = available I = important H = helpful NE = not essential

The report is organized according to the Guidance document developed by Northwest Hydraulic Consultants on January 29, 2004, and discusses Critical Topics first and available topics next.

## 1.1 STORM WAVE CHARACTERISTICS FOCUSED STUDY GROUP

The Focused Study Group members were Ian Collins, Dick Seymour, Bob Battalio, Darryl Hatheway, Jeff Gangai, Carmela Chandrasekera, Ron Noble, and Shyamal Chowdhury. Shyamal Chowdhury was the Leader of this Study Group. The group had two phone conference meetings on January 13, 2004, and January 26, 2004, when the group exchanged ideas, discussed directions, shared available information and procedures. The Team Leader was responsible for writing the scope, assembling the team, providing direction and coordination and final drafting of the report. Ron Noble was the internal reviewer and responsible for quality control of this report. Team members shared research and report writing tasks as shown below.

**Team Member Responsibilities**

| Person Responsible | Study Topic |
|---|---|
| Darryl Hatheway and Ron Noble | Topic 1: Wave Definitions |
| Jeff Gangai | Topic 3: Conversion from Shore Protection Manual (SPM) to Coastal Engineering Manual (CEM) |
| Ian Collins | Topics 4 and 5: Swell and Sea for All Coasts |
| Carmela Chandrasekera and Bob Battalio | Topic 5: Local Sea for All Non-Open Coasts |
| Dick Seymour | Topic  5: Local Sea for Southern California |

# 2 CRITICAL TOPICS

## 2.1 TOPICS 4 AND 5: SWELL AND SEA – PACIFIC COAST

### 2.1.1 Description of Topic and Suggested Improvement

Coastal flooding generally occurs with a combination of high water levels accompanied by large waves.  The purpose of this task is to identify and document the sources of wave and swell data that would provide the most useful input for wave transformation models.  The wave transformation models would be applied to route the waves to the inshore areas where knowledge of the waves is required to predict wave setup and runup, and overland propagation.

Since the preparation of previous guidelines for the determination of potential coastal flooding, several additional long duration data sources have become available.  These have incorporated improved developments in the modeling of winds, wind-wave generation, and swell propagation. Significant improvements in accuracy have been demonstrated by comparisons with offshore buoy recordings and satellite scatterometer data.

The two principal developments have been:

- Improvements in models of wind fields using worldwide meteorological stations and ships.  This has led to improved models of the planetary boundary layer to re-analyze historical, measured, barometric pressure data from ships and coastal meteorological stations.  The resulting “improved” winds have been compared with the measurements of winds at many offshore buoys.

- Improvements in numerical modeling of wave generation and propagation.  Continued research into the physics of energy transfer from wind to waves and subsequent wave propagation have led to significant improvements in the accuracy of wave forecasting and hindcasting.

These developments are now available and have been incorporated into extensive databases of waves and swells.

### 2.1.2 Description of Procedures in the Existing Guidelines

For the Pacific Coast the existing Guidelines for "Wave Elevation Determination and V Zone Mapping" contain the instruction:

> "No FEMA guidance documents have been published for the Pacific Ocean coastal flood studies. Guidance is to be developed based on existing methodologies recommended by FEMA and coastal states for coastal analyses in the Pacific Ocean. Mapping Partners that are undertaking a flood hazard analysis of a Pacific Coast site should consult with FEMA RPO for that area."

However, the Guidelines do refer to the U.S. Army Corps of Engineers (USACE) Wave Information Studies (WIS) and the availability of offshore and near shore measurements from buoys has been recognized and used by study contractors.

### 2.1.3 Applications of Existing Guidelines for Pacific Coast

On the Pacific Coast the waves determined from the Fleet Numerical Weather Central, as documented in a report by Meteorology International, Inc. (MII) were used for the Southern California area (by Tetra Tech, Inc.) and the WIS stations for Northern California by OTT Water Engineers, Inc. and for Oregon (Coos Bay County) by CH2M Hill.

The principal source of offshore wave data at the time of the earliest studies was the Fleet Numerical Weather Central (FNWC as summarized by MII, 1977) model for the Pacific Coast. The FNWC wave model, as covered at the time of the development of the guidelines (Tetra Tech, 1982) did not include the effect of hurricane generated swell off the West coast of Mexico and the swell from major storms in the southern hemisphere. The latter wave sources may govern in a few locations due to exposure to the more southerly wave directions.

Currently there are no Guidelines and Specifications for swell data. The FEMA Pacific Coast studies (TetraTech, Ott Water Engineers, CH2M Hill and Michael Baker) have used the WIS data and the MII (FNWC) hindcasts and NOAA data buoys. Other contemporary coastal studies have used the Coastal Data Information Program (CDIP) data buoys (Recordings) and WAVEWATCH III wave hindcasting model (described herein).

### 2.1.4 Alternatives for Improvement

#### *Overview*

Potential sources of wave and swell databases are identified. The general forms of the databases are summarized. These are generally available in a suitable format for input into wave modification models that compute the changes in waves as the shorelines are approached. In turn, such models are essential to predict the wave conditions in the surf zone that would ultimately be used to predict water levels and flooding.

Significant improvements in the analysis of historical meteorology have been developed in recent years. Windfields have been much reanalyzed to yield significant improvement and have been used with so-called third-generation wave hindcast models to yield improvements in wave predictions over long periods (20 years or more). These models have been calibrated and verified by comparison with measured data at offshore buoys. Further improvements are expected.

### *Definitions*

Seas (or Storm Seas) are normally considered to be the result of local storm activity and are being directly influenced by local winds.

Swell is normally considered to be waves that are arriving at a location that is remote from the generation area. Typically, swells have longer periods than waves, but not always so.

Swells and seas may occur together (as is usually the case on the Pacific Coast). When this is so, their energies should be added, corresponding to vector addition (square root of sum of squares) but directions and periods will generally be different.

### *Data Sources*

There have been further developments in wave and swell prediction models since the earlier FNWC data as reported in the MII documents. In 1985 FNWC published the results of a more comprehensive wave climate for many oceans of the world as Spectral Ocean Wave Model (SOWM). This methodology has been improved by several organizations such as:

- CHL Field Research Facility (http://frf.usace.army.mil)
- CHL Operations and Analysis Group (http://sandbar.wes.army.mil)
- National Data Buoy Center (http://seaboard.ndbc.noaa.gov)
- Coastal Data Information Program (http://cdip.ucsd.edu)
- National Oceanographic Data Center (http://www.nodc.noaa.gov)
- Fleet Numerical Meteorology and Oceanography Center (http://www.fnoc.navy.mil/PUBLIC, https://www.fnmoc.navy.mil/PUBLIC/)
- Naval Oceanographic Office (http://www.navo.navy.mil)
- OceanWeather, Inc. (http://www.oceanweather.com)

The listed data sources include measurements from offshore buoys and extensive hindcast data. The measurements are generally somewhat sporadic as the installation and maintenance of offshore wave measuring devices is expensive.

### *Specific Comments of Listed Sources*

**CHL Field Research Facility (Coastal Hydraulics Laboratory)**

This database of hindcasts is known as WIS (Wave Information Studies). They provide a 20-year hindcast database for 134 selected stations between Cape Flattery, Washington, and Point Conception, California. (WIS Report 17, "Pacific Coast Hindcast Phase III, North Wave Information" by Jensen, Hubertz and Payne, 1989) and 47 selected stations between Point Conception and the Mexican border (WIS Report 20, "Southern California Hindcast Wave Information" by Jensen, Hubertz, Thompson, Reinhard, Borup, Brandon, Payne, Brooks and McAneny, 1992). Figure 1 illustrates the coverage of part of Northern California Coast and Figure 2 shows the Southern California stations. The stations are relatively close to shore.

The WIS data reports for the Pacific Coast are reportedly under major revision. Existing reports (2003) should be used with care as they do not include the contributions from swells from the Southern Hemisphere or from tropical storms. Published WIS results have also been found to be less accurate. Tillotson and Komar (1997) found that "[s]ignificant wave heights derived from the WIS hindcasts are 30 to 60 percent higher than measured by the deep-water buoys and microseismometer."

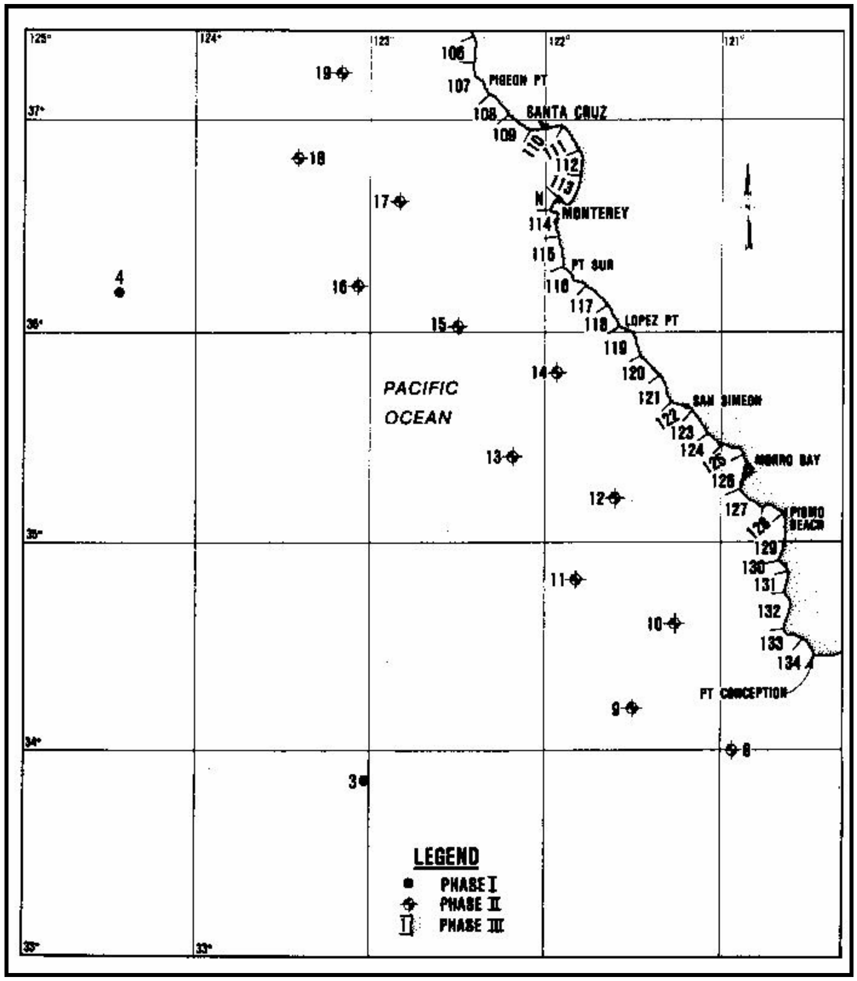


**Figure 1. Illustration of WIS hindcast area for northern part of the Pacific Coast.**

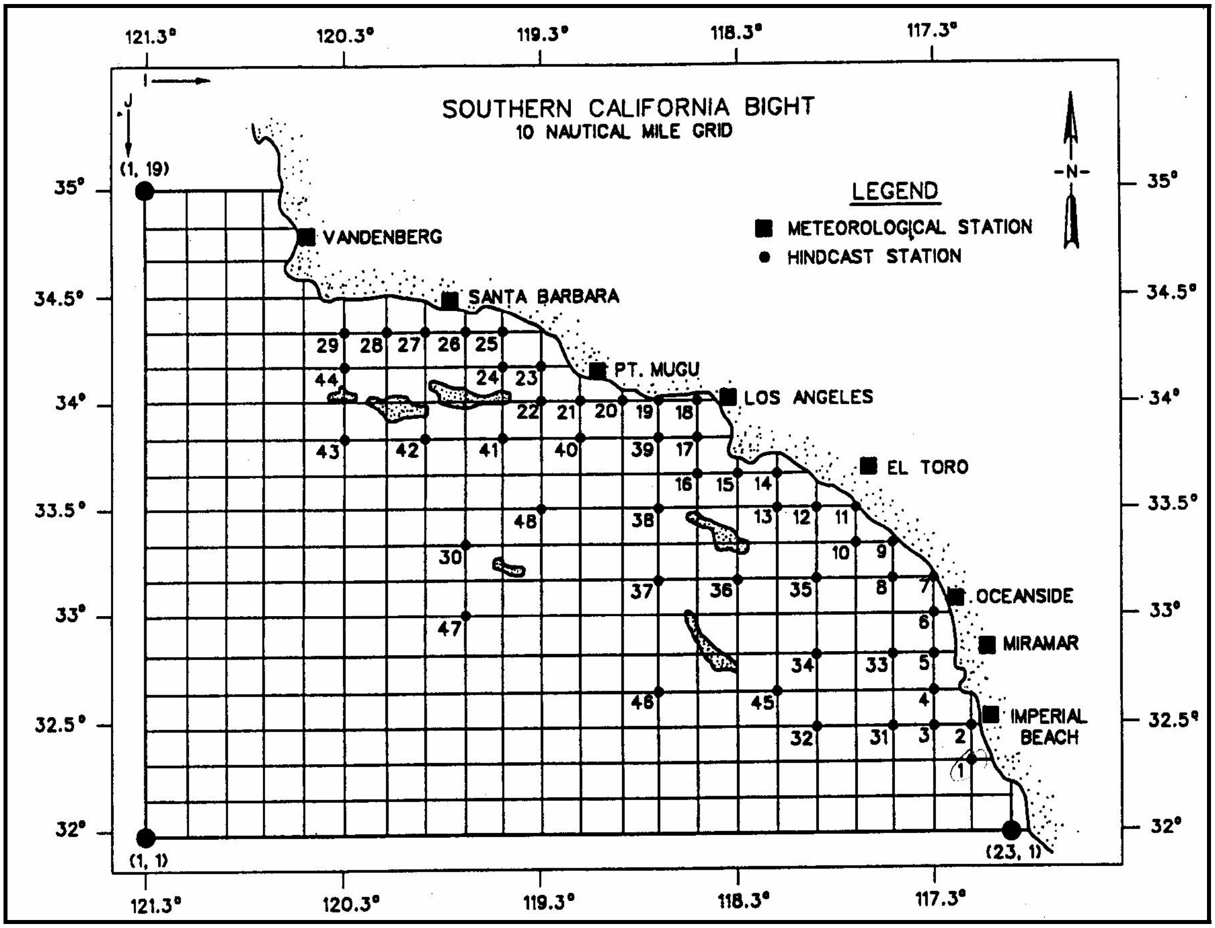


**Figure 2. WIS stations in the Southern California Bight.**

**National Data Buoy Center**

The National Data Buoy Center is a branch of NOAA. They have been installing and maintaining offshore meteorological and oceanographic buoys since the late 1960s. Many of these buoys have been in place for a sufficiently long period (typically, 20 years of data, and preferably longer is required to estimate the 0.01 probability extreme event with confidence) that reasonably accurate wave height statistics can be derived. Many other buoy locations are available for limited periods. Such buoys cannot be used for direct statistical prediction of extremes but are still very useful to check wave hindcast models during the overlapping times.

Figure 3 shows an example of the locations of the MetOcean buoys in the Southern California area and Figure 4 shows locations in the North Pacific. Not all of the buoys that are shown on the maps are always present and often the ones shown are removed for maintenance and may be replaced in a slightly different location. Data inventories (dates of installation and recording) are also included on the website. Most wave data are in the form of one-dimensional spectra with summaries of wave height and periods (spectral peak and average). Very few have wave directional information. The wind and wave data from the buoys have been used extensively to check calibration and validity of wave hindcast models.

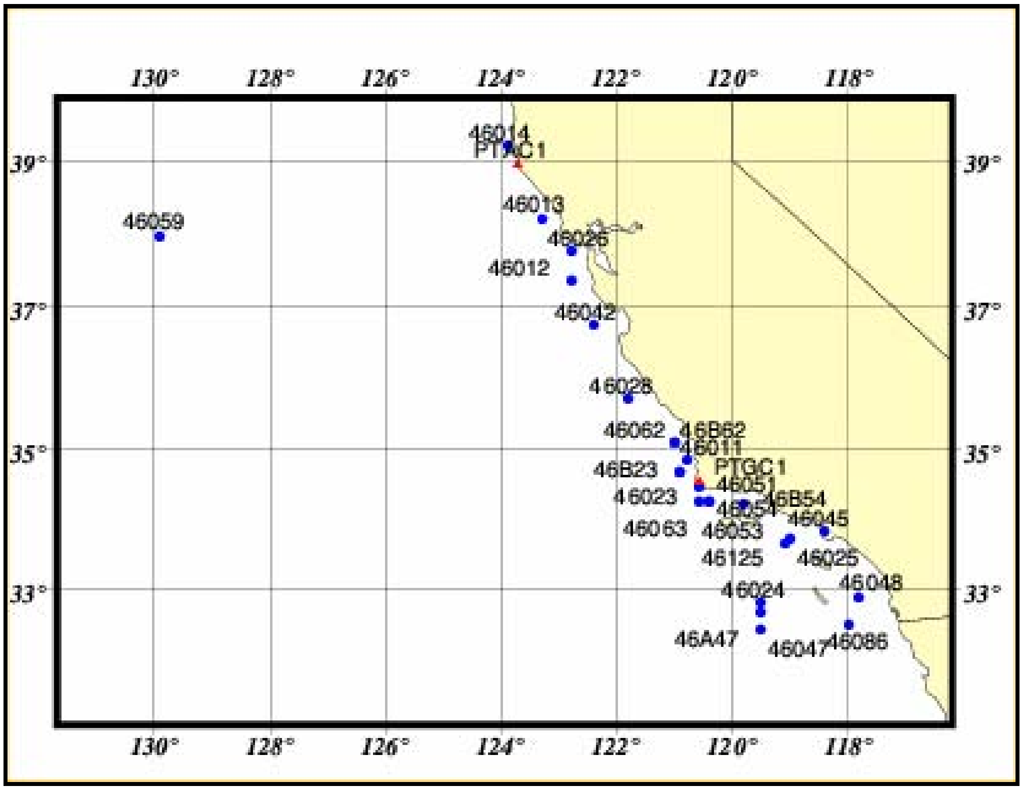


**Figure 3. NDBC buoy locations (southern California).**

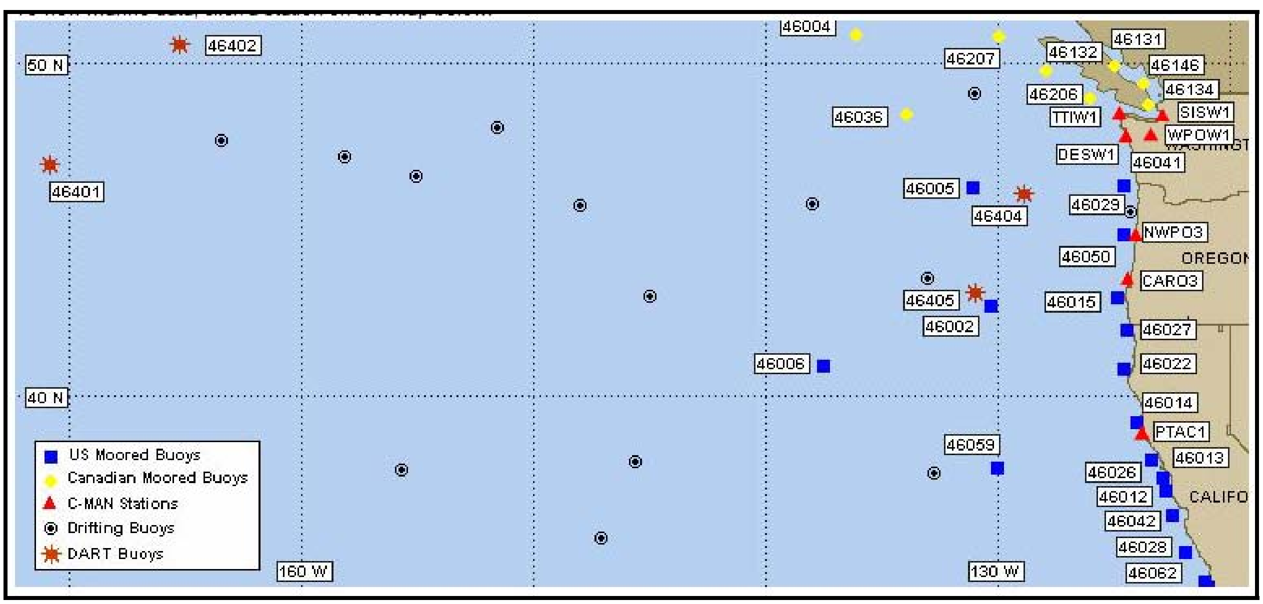


**Figure 4. NODC buoy locations in the north Pacific.**

**Coastal Data Information Program (Mostly in California)**

The CDIP consists of a number of nearshore buoys that record directional wave spectra. They are installed and maintained by Scripps Institution of Oceanography under the sponsorship of USACE and the State of California. The program has been expanded recently to include some installations on the Atlantic Coast. Some earlier data included waves measured by pressure sensor arrays.

Figure 5 summarizes the locations of many of the buoys. The buoys are generally located in water depths of 100 to 550 meters. There are a few buoys in shallower water. The duration of available records is generally too short for reliable estimates of conditions that would be characteristic of the 1-percent extreme value but are useful to calibrate and verify wave modification modeling. Previous deployments included bottom-mounted pressure arrays in shallow water. Data from these instruments includes the estimates of wave directions. However, pressure sensors have been discontinued in all but one site at Scripps Institute of Oceanography Pier.

The CDIP program includes a wave forecasting and shallow water swell height modeling capability that provides wave information near the California Coast. These shallow water conditions are covered more extensively in the Wave Transformation Focused Study.

| | | | | | | |
|---|---|---|---|---|---|---|
| 138 | BEGG ROCK BUOY | 33 22.800 | 119 39.800 | USACE | CDIP | d-01/30/1991 |
| 052 | SAN CLEMENTE ARRAY | 33 24.900 | 117 37.800 | USACE | CDIP | d-05/05/1998 |
| 096 | DANA POINT BUOY | 33 27.506 | 117 46.003 | USACE/CDBW | CDIP | o |
| 092 | SAN PEDRO BUOY | 33 37.091 | 118 19.008 | USACE/CDBW | CDIP | o |
| 072 | HUNTINGTON BEACH ARRAY | 33 37.900 | 117 58.700 | USACE | CDIP | d-09/18/2001 |
| 027 | SUNSET BEACH ARRAY | 33 42.300 | 118 04.200 | USACE | CDIP | d-05/24/1990 |
| 028 | SANTA MONICA BAY BUOY | 33 51.230 | 118 37.920 | USACE/CDBW | CDIP | o |
| 104 | HERMOSA NEARSHORE BUOY | 33 51.791 | 118 25.280 | USACE | CDIP | o |
| 080 | SANTA CRUZ CANYON BUOY | 33 55.000 | 119 44.000 | USACE | CDIP | d-06/01/1989 |
| 103 | TOPANGA NEARSHORE BUOY | 34 01.383 | 118 34.698 | CDBW | CDIP | n |
| 087 | SANTA ROSA ISLAND BUOY | 34 02.300 | 120 05.500 | USACE/CDBW | CDIP | d-12/18/1995 |
| 089 | SANTA CRUZ ISLAND EAST BUOY | 34 03.500 | 119 35.000 | USACE | CDIP/CDBW | d-11/30/1995 |
| 088 | SANTA CRUZ ISLAND WEST BUOY | 34 04.200 | 119 50.000 | USACE/CDBW | CDIP | d-12/18/1995 |
| 141 | PORT HUENEME BUOY | 34 05.200 | 119 10.000 | USACE | CDIP | d-04/17/1991 |
| 038 | POINT MUGU BUOY | 34 05.400 | 119 06.800 | USACE | CDIP | d-09/30/1985 |
| 005 | CHANNEL ISLANDS | 34 10.000 | 119 14.200 | USACE | CDIP | d-09/19/1983 |
| 081 | VENTURA BUOY | 34 10.800 | 119 28.600 | USACE/CDBW | CDIP | d-03/08/1995 |

**Figure 5. Summary of CDIP buoy locations and dates of installation.**

**National Oceanographic Data Center**

This agency and website include similar data to the National Data Buoy Center but covers the entire world, not just U.S. waters.

**Fleet Numerical Meteorology and Oceanography Center**

Fleet Numerical Meteorology and Oceanography Center (FNMOC) prepares weather and wave forecasting for all oceans of the world. An example of the Pacific Ocean data for wave height by direction is given as Figure 6. The basic model is known as WAVEWATCH III. Figure 6 shows a particular presentation of wave height and direction. Additional products include wave period and direction, swell heights by direction, and several other forms. The emphasis of the available data appears to be forecasting. They have a historical database that only goes back to July 1997. This would be too short to use for estimation of extreme waves. However, given that the model is readily available and can be downloaded from the WAVEWATCH site the hindcasting model could be extended by a user as long as the analyzed wind fields for earlier years are prepared or available.

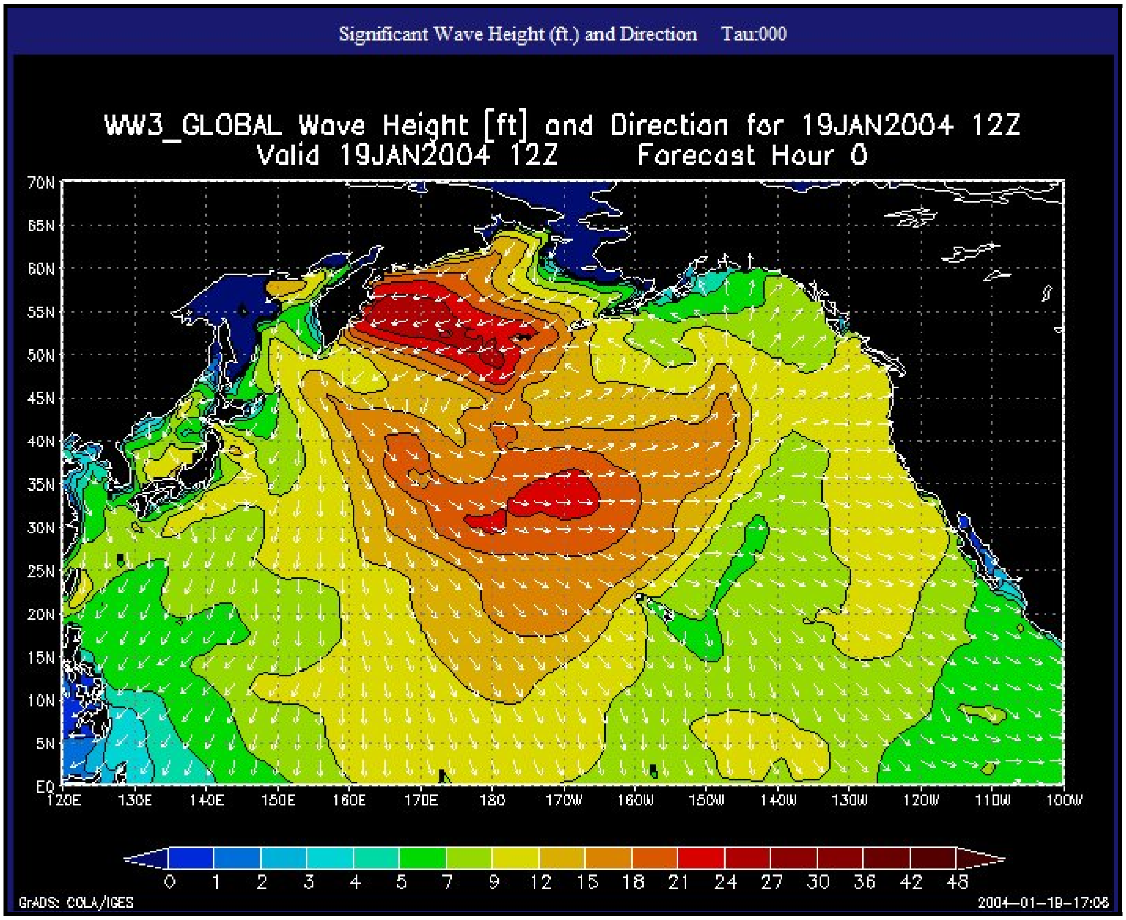


**Figure 6. Example of wave forecast from WAVEWATCH III.**

WAVEWATCH III (Tolman 1997, 1999a) is a third-generation wave model developed at National Oceanic and Atmospheric Administration-National Centers for Environmental Prediction (NOAA-NCEP) in the spirit of the WAM model (WAMDI Group, 1988; Komen et a1., 1994). It is a further development of the model WAVEWATCH I, as developed at Delft University of Technology (Tolman 1939, 1991) and WAVEWATCH II, developed at NASA, Goddard Space Flight Center (e.g., Tolman 1992). It nevertheless differs from its predecessors on all important points: the governing equations, the models structure, numerical methods, and physical parameterizations.

WAVEWATCH III solves the spectral action density balance equation for wavenumber-direction spectra. The implicit assumption of these equations is that the medium (depth and current) as well as the wave field vary on time and space scales that are much larger than the corresponding scales of a single wave. Furthermore, the physics included in the model do not cover conditions where the waves are severely depth influenced. This implies that the model can generally by applied on spatial scales (grid increments) larger than 1 to 10 km, and outside the surf zone.

The following physical features are extracted from WAVEWATCH III homepage

http://polar.wwb.noaa.gov/waves/wavewatch/wavewatch.html :

- The governing equations include refraction and straining of the wave field due to temporal and spatial variations of the mean water depth and the mean current (tides, surges etc.), and wave growth and decay due to the actions of wind, nonlinear resonant interactions, dissipation (‘whitecapping’) and bottom friction.

- Wave propagation is considered to be linear. Relevant nonlinear effects such as resonant interactions are therefore included in the source terms (physics).

- The model includes two source term options, the first based on cycles 1 through 3 of the WAM model (WAMDI Group, 1988), the second based on Tolman and Chalikov (1996), which is used by FNMOC. The source term parameterizations are selected at the compile level.

- The model includes dynamically updated ice coverage.

Many other products are available, including separate displays of waves, swell and wave periods. The software is available for free download. However, the model requires input in the form of a specified windfield. This would require some effort on the part of a Study Contractor. Although, the WAVEWATCH model would be acceptable, the extra processing of wind data that would be required probably makes it more expensive to apply. For the above reasons the model is not recommended at this time for use in Flood Studies, although it may be acceptable to use if properly applied. The model does not calculate wind-related surge.

**Naval Oceanographic Office**

This agency generally provides summaries of other oceanographic data, including temperature profiles and currents as well as waves. There are extensive data archives but wave information is generally cross referenced to FNMOC and WAVEWATCH III.

**OceanWeather, Inc.**

OceanWeather, Inc. is a private company that has specialized in wave hindcasting since its inception in 1977. The particular model that would be most useful for FEMA studies is GROW (Global Re-analysis of Ocean Waves). Figure 7 presents examples showing the locations for which wave data are available. The grids are at 0.625 degrees longitude by 1 degree latitude and cover the entire Pacific and Atlantic Oceans.

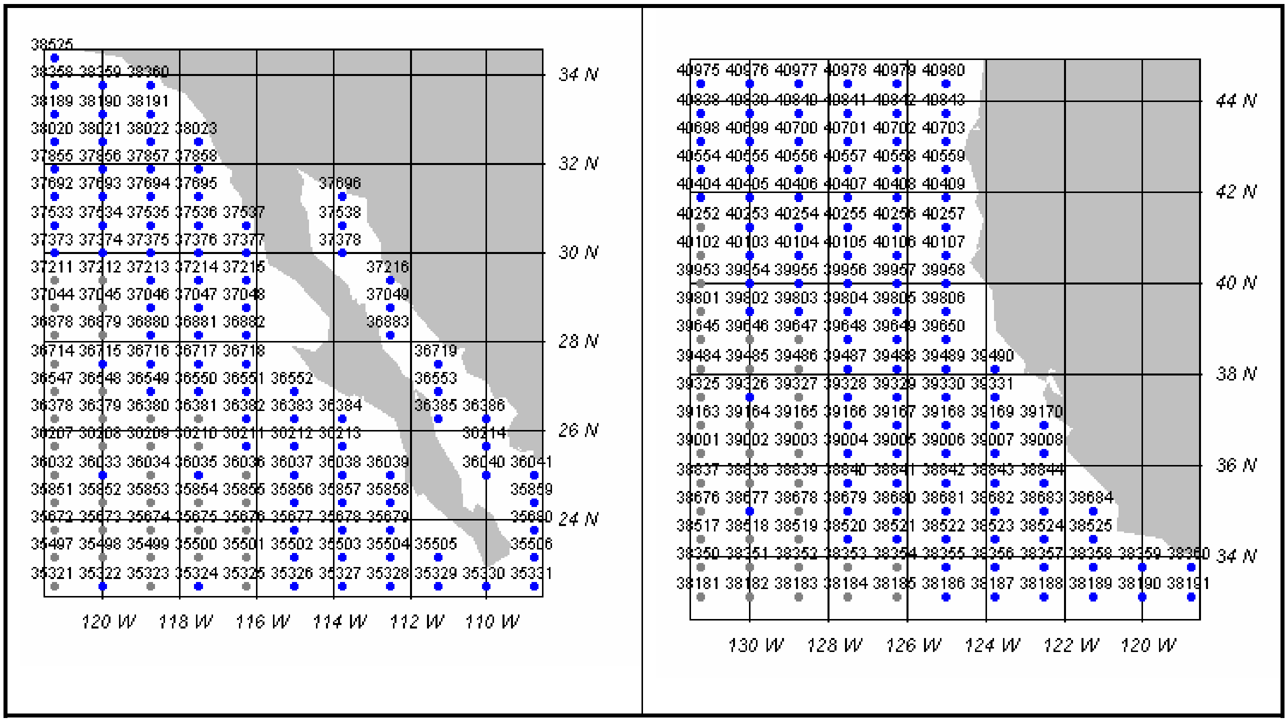


**Figure 7. Examples of available locations for GROW hindcasts.**

GROW couples Oceanweather's global wave model, planetary boundary layer model, and its vast experience in developing marine surface wind fields to produce a global wave hindcast.

The result is a long-term analysis of the global wave climate that can be applied to offshore structure design, tow-analysis, operability, and other applications where wind and wave data are required. Typical data types include:

- Time series of wind and wave parameters (including sea/swell partitions) in ASCII or OSMOSIS format
- Return period extremes for wind speed, wave height (significant, maximum and crest) and wave period
- Operability statistics expressed as frequency-of-occurrence tables and persistence/duration statistics
- Directional wave spectra

The wave hindcast data are generated following an extensive re-analysis of global windfields. Several technical publications have documented this and compared hindcast wind, waves and swell to measurements by NOAA data buoys and satellite scatterometer data.

The available database includes directional wave spectra every 3 hours over a period of 30 years. The swell directional spectra are on the same time base, but are provided as a separate database. In order to manage this database OceanWeather also sells a software suite known as OSMOSIS. OSMOSIS is an engineering analysis tool for displaying and calculating a variety of metocean hindcast statistics. GROW products are available in OSMOSIS format and are purchased separately from the database.

OSMOSIS permits several Display and Export features:

- DataSelect area of interest by clicking on map or entering location
- Select time period of interest
- Display time series as tables or graphs of all or some variables and dates
- Display tables of normals and extremes computed by Oceanweather
- All tables and graphs can be printed or saved to disc
- Export multiple time series to disc at once by selecting points from a map

Statistical analyses include:

- Frequency of Occurrence tables on any two variables
- Persistence/Duration tables on any variable
- Objective identification of storm peaks based on any variable
- Interactive modification of storm peak selection
- Extremal analysis with Gumbel, Borgman and Weibull distributions
- Scatter plots of time series or storm peaks

### 2.1.5 Recommendations

Offshore waves become the drivers for nearshore waves that in turn induce wave setup and runup. The “best” sources of offshore waves and swell need to be identified. An assessment of their accuracy and general quality is needed.

For the Pacific Coast, the GROW data is recommended but updated WIS data is under development and is expected to include input from GROW. Consequently this could become the database of choice for the Pacific Coast. The WIS database that is currently available for the Pacific Coast does not include Southern Hemisphere swell or swell from tropical storms. Wave recordings from the CDIP buoys could be used to verify the validity of wave and swell modification modeling between the offshore and the nearshore.

GROW is available as an off-the-shelf product and is presented in the form of directional spectra for both waves and swells for every 3 hours for 30 years or more. This is believed to be the most useful and comprehensive data source. WAVEWATCH III is heavily oriented for use as a forecasting tool but the source code is available and has been used to develop deep water wave statistics for coastal studies (Noble Consultants for USACE, 2003). Two drawbacks to using WAVEWATCH III would be the need to derive, process and set up the required 20–30 years of windfields or limit the database because the data is only archived back to July 1997.

#### 2.1.6 Preliminary Time Estimates for Guideline Improvement

The remaining tasks that can be completed within the time being allocated for the revised Guidelines and Specifications would be:

1. Review the technical publications on GROW and perform a critical analysis to confirm the claimed lack of bias. (40 hours)

2. Examine the detailed reports from GROW and describe the necessary steps to prepare the input data for wave transformations as the waves propagate to shore. (80 hours plus cost to obtain a data set for a selected Pacific Coast station)

3. Recommend a methodology to apply the shallow water wave transformation models to a suitable matrix of GROW directional spectra to ensure complete coverage of the deep water wave properties envelope. (40 hours)

4. Review the available databases for offshore and near shore wave buoys to see whether they can be used as input to shoaling water wave models. (Leave to Study Contractor)

5. Keep in touch with the progress on the revisions to WIS for the Pacific Coast to see whether this database can be used for wave inputs to local wave modification models. (up to 40 hours, as needed)

Table 4 at the end of this document summarizes the estimated hours for these portions of Topics 4 and 5.

### 2.2 TOPICS 4 AND 5: OFFSHORE WAVE DATA FOR ATLANTIC AND GULF COASTS

#### 2.2.1 Description of Topic and Suggested Improvement

This topic was actually listed as “Available” during the December planning meeting. This is true as long as the methods for wave determination that are given the SPM are considered to be adequate. The procedure takes a “standard” synthetic hurricane and uses the Bretschneider method, which gives wave heights and periods in terms of the hurricane’s central pressure deficit, radius to maximum winds, and forward speed. Such an approximation assumes coincidence of the waves with the peak of the storm surge and assumes that the waves are approaching normal to the shoreline. The method may be adequate since the “controlling” wave height (1.6 times the significant wave height) will often, but not always, be the limit breaking wave at the original shoreline. Wave heights are needed for overland wave propagation, wave runup, and wave setup computations.

However, there may be cases where the Bretschneider hurricane wave approximation is not valid. In such cases, a more complete knowledge of the directional spectrum of waves and swell implies that this becomes a "critical" topic. In such a case, the recommended alternative would be to use the available WIS database or follow the procedures starting with GROW and running an acceptable shallow water wave modification process. The approach would be similar to that described above for the Pacific Coast.

To use a wave height other than the "controlling" wave, an "equivalent" deep water wave height will be needed. This is the *H*o' that is used on many nomographs of wave properties. *H*o' is the equivalent deep water wave height that can be derived from the local wave height after being "de-shoaled" and "de-refracted." In other words, it is what the deepwater wave height would have been if it had not been modified by shoaling and refraction. It allows the use of a local wave (from WIS) or measurement. The effect of energy losses from bottom friction, percolation, and fluid mud bottoms becomes irrelevant. In some cases, if the local wave height has to be derived by wave transformation, the effects of such energy losses have to be included before the derivation of Ho'.

### 2.2.2 Description of Procedures in the Existing Guidelines

It must be expected that there will be waves present and propagating toward the shore when the 1-percent water level occurs. The present guidelines (Appendix D of the Guidelines and Specifications [*G&S*]) apply primarily to the Atlantic and Gulf Coasts and are summarized in the following.

Three specific approaches are suggested in the existing Guidelines and Specifications:

- Wave data from wave measurements at offshore buoys
- Wave data from hindcasts or numerical modeling based on historical effects
- Wave data from specific calculations based on assumed storm meteorology

It was recommended that two or all three methods be applied where feasible to ensure the most accurate assessment of wave conditions. The *G&S* then include the following:

> "Wave measurements for many sites over various intervals have been reported primarily by the USACE and by the National Data Buoy Center. Available data includes records from nearshore gages in relatively shallow water (Thompson, 1977) and from sites further offshore in moderate water depths (Gilhousen et al., 1990). The potential sources of storm wave data also include other Federal agencies and some State or university programs."

"The USACE is the primary source for long-term wave hindcasts along open coasts. That information is conveniently summarized as extreme wave conditions expected to recur at various intervals for Atlantic hurricanes in "Hurricane Hindcast Methodology and Wave Statistics for Atlantic and Gulf hurricanes from 1956-1975" (Abel et al., 1989) and for extratropical storms in "Hindcast Wave Information for the U.S. Atlantic Coast" (Hubertz, Brooks, Brandon, & Tracy, 1993) and "Southern California Hindcast Wave Information" (Jensen et al., 1992), as examples. In some vicinities, other wave hindcasts may be available from the design activities for major coastal engineering projects."

"Either measurements or hindcast results pertain to some specific (average) water depth. However, the Mapping Partner may need to convert such wave information into an equivalent condition at some other water depth for appropriate treatment of flood effects. The Mapping Partner shall consult the following publications for guidance regarding transformation of storm waves between offshore and nearshore regions, where processes to be considered include wave refraction, shoaling, and dissipation: "The USACE Shore Protection Manual" (USACE, 1984), "Random Seas and Design of Maritime Structures" (Goda, 1985), and "Automated Coastal Engineering System, Version 1.07" (Leenknecht, Szuwalski, & Sherlock, 1992)."

"The Mapping Partner may also consider determining local storm wave conditions by developing a specific estimate for storm meteorology taken to correspond to the 1-percent-annual-chance flood. That can be done with relative ease for deep-water waves associated with a hurricane of specified meteorology, using the estimation technique provided in the USACE Shore Protection Manual (USACE, 1984). For extratropical storms, the ACES program in Automated Coastal Engineering System, Version 1.07 (Leenknecht, Szuwalski, & Sherlock, 1992) executes a modern method of wave estimation for specified water depth, incorporating some basic guidance from the Shore Protection Manual (USACE, 1984) and Random Seas and Design of Maritime Structures (Goda, 1985). The Mapping Partner may prepare an outline of important considerations to assist in developing a site-specific wave estimate."

"The resulting wave field is commonly summarized by the significant wave height and wave period; namely, average height of the highest one-third of waves and the corresponding time for a wave of that height to pass a point. Another useful measure is wave steepness, the ratio of wave height to wavelength: in deep water, the wavelength is 0.16 times the gravitational acceleration, times the wave period squared, that is, ($gT^2/2\pi$). On larger water bodies and in relatively deep water, typical wave steepness is approximately 0.03 for extreme extratropical storms and 0.04 for major hurricanes. The Mapping Partner may use these values for wave steepness to determine the wave period if only the wave height is known and the wave height if only the wave period is known."

### 2.2.3 Applications of Existing Guidelines for Atlantic and Gulf Coasts (Waves and Swell)

Study contractors on the Atlantic (South) and Gulf Coasts have generally assumed that waves would be present whenever high-water levels occur at the coast because high water is associated with hurricane activity. The general practice has been to use the SPM procedure for "model" hurricanes. Appropriate values of central pressure deficit and size are assumed and deep water significant wave heights and periods computed. Some studies used the local 5 to 10% central pressure depression, and local median values for other parameters such as radius and forward speed (Personal communication, David Divoky). These waves would then be used to determine local setup and runup that would be present at the time of high water.

The existing FEMA guidelines use direct hurricane wind-wave generation models for the major part of the Atlantic Coast and the Gulf of Mexico Coast because the extreme water levels along these coasts are usually controlled by hurricane events where the simultaneous arrival of the highest water levels is accompanied by waves that will be controlled by depth limited breaking. A reasonable approximation of the offshore wave heights is probably adequate. In other words, the waves are limited by breaking criteria. The relatively wide continental shelf also tends to limit the wave conditions along these coasts because higher offshore waves are reduced by non-linear friction effects more than lower waves. Consequently, large differences in offshore wave heights translate into smaller differences near shore. However, the wave setup at the shoreline is sensitive to deep water wave conditions.

For the Northern part of the Atlantic Coast the governing extreme storm may be a Northeaster, although hurricanes from the south should not be neglected.

Currently there are no Guidelines and Specifications for swell data.

### 2.2.4 Alternatives for Improvement

***Overview***

Similar databases that have been discussed in the Focused Study report on waves and swell for the Pacific Coast exist for the Atlantic and Gulf Coasts. These include WIS (USACE) for local water depths, WAVEWATCH (U.S. Navy) and GROW (commercial) for deep water.

***Definitions***

The definitions for sea and swell are the same as presented in section 2.1.5 above. However, for the Atlantic and Gulf Coasts it is expected that at the times of extreme water levels there will be waves related to hurricane condition. Swells have generally been ignored, but swell heights and directions are available in the GROW databases.

### *Data Sources*

Potential data sources for waves and swell can be found at the same locations that were listed in the Pacific Coast section.  These include:

- CHL Field Research Facility (http://frf.usace.army.mil)
- CHL Operations and Analysis Group (http://sandbar.wes.army.mil)
- National Data Buoy Center (http://seaboard.ndbc.noaa.gov)
- National Oceanographic Data Center (http://www.nodc.noaa.gov)
- Fleet Numerical Meteorology and Oceanography Center (http://www.fnoc.navy.mil/PUBLIC, https://www.fnmoc.navy.mil/PUBLIC/)
- Naval Oceanographic Office (http://www.navo.navy.mil)
- OceanWeather, Inc. (http://www.oceanweather.com)

The listed data sources include measurements from offshore buoys and extensive hindcast data. The measurements are generally somewhat sporadic as the installation and maintenance of offshore wave measuring devices is expensive.

### *Specific Comments of Listed Sources*

#### CHL Field Research Facility

WIS provides a 25-year hindcast database for selected points that are relatively close to shore. An example of the station locations is presented in Figure 8.

The WIS data for the Atlantic and Gulf Coasts have recently been updated and are available from the website in several forms.  Examples are given in Figures 9 and 10.

#### National Data Buoy Center

National Data Buoy Center, as described in the previous section, has systems of offshore meteorological and oceanographic buoys in the Atlantic and Gulf Coast regions. Figure 11 shows a part of the coverage on the Atlantic Coast.  Not all buoys that are shown on the maps are always present and often the ones shown are removed for maintenance and may be replaced in a slightly different location.

The locations of the buoys in other areas are readily determined at the website.  Data inventories (dates of installation and recording) are also given on the website.  Most wave data is in the form of one-dimensional spectra with summaries of wave height and periods (spectral peak and average).  Very few have wave directional information.  The wind and wave data from the buoys have been used extensively to check calibration and validity of wave hindcast models.

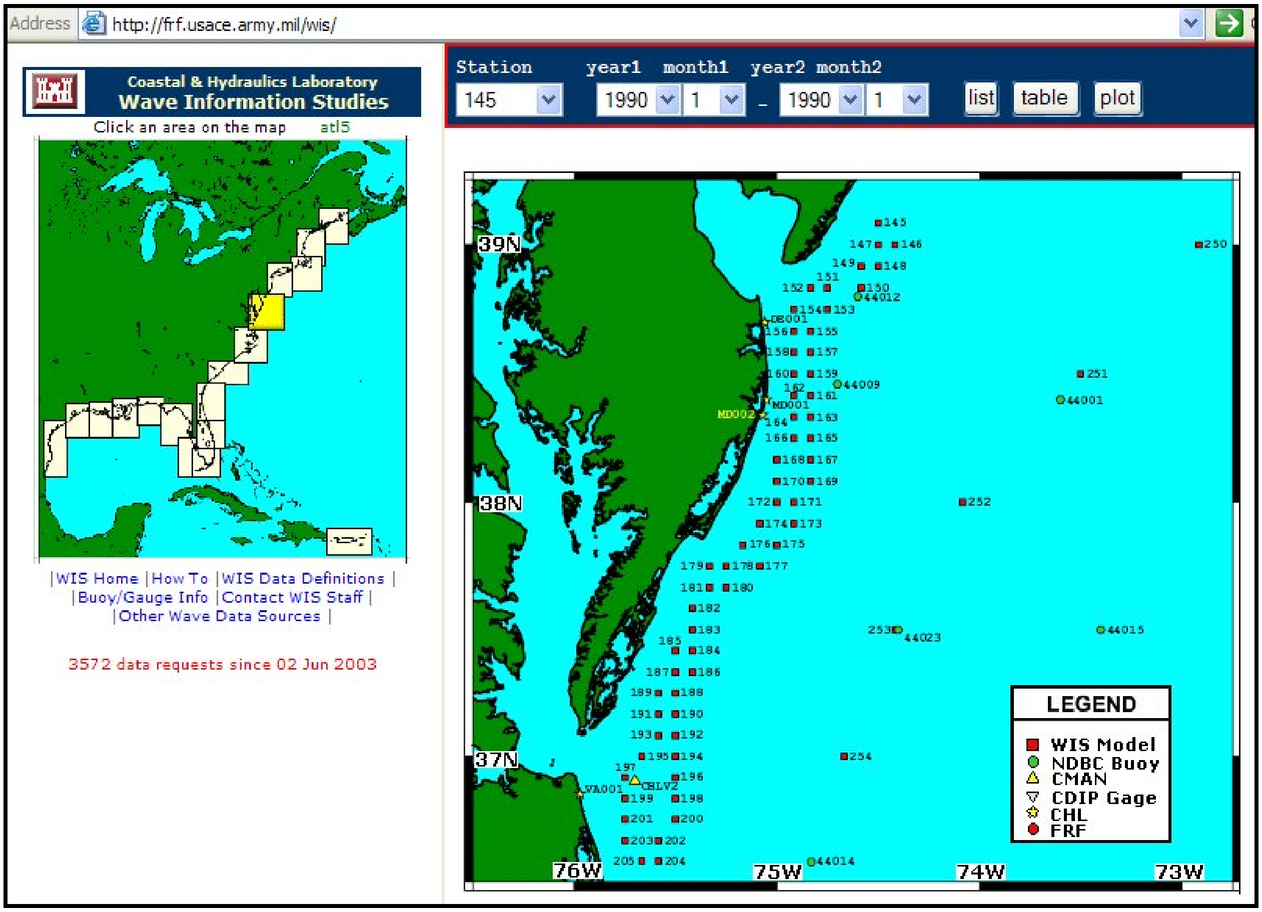


**Figure 8. Example of WIS locations.**

ip = nuthin

| ID | YEAR | MM | DD | HH | LONG | LAT | DPTH | Hmo | DTp | Atp | tmean | wdvmn | wv | wsp | wdir |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 145 | 1990 | 1 | 1 | 1 | -74.50020 | 39.08309 | 17.0 | 0.34 | 3.33 | 3.33 | 3.33 | 180.0 | 180.0 | 10.5 | 180. |
| 145 | 1990 | 1 | 1 | 2 | -74.50020 | 39.08309 | 17.0 | 0.55 | 3.33 | 3.33 | 3.42 | 184.2 | 184.2 | 10.3 | 184. |
| 145 | 1990 | 1 | 1 | 3 | -74.50020 | 39.08309 | 17.0 | 0.79 | 3.33 | 3.33 | 3.52 | 184.1 | 184.2 | 10.2 | 185. |
| 145 | 1990 | 1 | 1 | 4 | -74.50020 | 39.08309 | 17.0 | 0.93 | 4.00 | 3.90 | 3.86 | 184.0 | 182.7 | 10.0 | 189. |
| 145 | 1990 | 1 | 1 | 5 | -74.50020 | 39.08309 | 17.0 | 1.08 | 4.00 | 4.03 | 4.10 | 185.6 | 186.6 | 9.9 | 190. |
| 145 | 1990 | 1 | 1 | 6 | -74.50020 | 39.08309 | 17.0 | 1.14 | 4.00 | 4.50 | 4.40 | 184.9 | 179.9 | 9.6 | 190. |
| 145 | 1990 | 1 | 1 | 7 | -74.50020 | 39.08309 | 17.0 | 1.30 | 5.00 | 4.68 | 4.68 | 187.3 | 181.4 | 9.6 | 208. |
| 145 | 1990 | 1 | 1 | 8 | -74.50020 | 39.08309 | 17.0 | 1.50 | 5.00 | 4.74 | 4.88 | 196.2 | 192.7 | 9.5 | 228. |
| 145 | 1990 | 1 | 1 | 9 | -74.50020 | 39.08309 | 17.0 | 1.59 | 5.00 | 5.17 | 5.03 | 212.5 | 223.1 | 9.4 | 243. |
| 145 | 1990 | 1 | 1 | 10 | -74.50020 | 39.08309 | 17.0 | 1.65 | 5.00 | 5.37 | 5.10 | 227.3 | 190.9 | 9.5 | 268. |
| 145 | 1990 | 1 | 1 | 11 | -74.50020 | 39.08309 | 17.0 | 1.42 | 5.00 | 5.46 | 5.00 | 221.3 | 181.8 | 9.6 | 279. |
| 145 | 1990 | 1 | 1 | 12 | -74.50020 | 39.08309 | 17.0 | 1.26 | 5.56 | 5.59 | 4.96 | 217.3 | 179.7 | 9.8 | 294. |
| 145 | 1990 | 1 | 1 | 13 | -74.50020 | 39.08309 | 17.0 | 1.22 | 4.00 | 5.74 | 4.95 | 219.5 | 181.3 | 10.0 | 295. |
| 145 | 1990 | 1 | 1 | 14 | -74.50020 | 39.08309 | 17.0 | 1.20 | 4.00 | 6.06 | 5.01 | 221.5 | 163.8 | 10.4 | 295. |
| 145 | 1990 | 1 | 1 | 15 | -74.50020 | 39.08309 | 17.0 | 1.21 | 4.00 | 6.20 | 5.07 | 223.5 | 165.8 | 10.7 | 290. |
| 145 | 1990 | 1 | 1 | 16 | -74.50020 | 39.08309 | 17.0 | 1.25 | 4.00 | 3.96 | 5.04 | 235.4 | 278.9 | 11.0 | 290. |
| 145 | 1990 | 1 | 1 | 17 | -74.50020 | 39.08309 | 17.0 | 1.29 | 4.00 | 3.99 | 5.03 | 242.2 | 281.7 | 11.4 | 290. |
| 145 | 1990 | 1 | 1 | 18 | -74.50020 | 39.08309 | 17.0 | 1.34 | 4.00 | 4.00 | 5.07 | 245.8 | 283.3 | 11.7 | 290. |
| 145 | 1990 | 1 | 1 | 19 | -74.50020 | 39.08309 | 17.0 | 1.37 | 4.00 | 4.00 | 5.24 | 243.7 | 286.9 | 11.7 | 290. |
| 145 | 1990 | 1 | 1 | 20 | -74.50020 | 39.08309 | 17.0 | 1.41 | 4.00 | 3.99 | 5.44 | 238.1 | 287.4 | 11.7 | 290. |

**Figure 9. Example of WIS time series.**

1990-1990 ATL WIS STATION: 145 LAT: 39.08 N, LON:-74.50 W, DEPTH: 17 M
PERCENT OCCURRENCES OF WAVE HEIGHT BY MONTH

| Hmo (m) | JAN | FEB | MAR | APR | MAY | JUN | JUL | AUG | SEP | OCT | NOV | DEC | CASES | PCT |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 0.00 - 0.49 | 1.87 | 1.32 | 2.34 | 1.62 | 2.28 | 1.91 | 3.61 | 2.43 | 0.79 | 0.35 | 1.86 | 1.03 | 1876 | 21.4 |
| 0.50 - 0.99 | 3.88 | 2.89 | 3.23 | 3.71 | 3.65 | 5.47 | 3.44 | 3.98 | 3.25 | 3.52 | 3.69 | 2.67 | 3800 | 43.4 |
| 1.00 - 1.49 | 1.96 | 1.91 | 1.30 | 2.31 | 2.25 | 0.67 | 1.21 | 1.74 | 3.48 | 2.91 | 2.00 | 2.02 | 2081 | 23.8 |
| 1.50 - 1.99 | 0.67 | 1.15 | 1.05 | 0.39 | 0.10 | 0.17 | 0.24 | 0.34 | 0.69 | 0.80 | 0.33 | 1.48 | 650 | 7.4 |
| 2.00 - 2.49 | 0.09 | 0.33 | 0.26 | 0.07 | 0.18 | . | . | . | 0.01 | 0.32 | 0.23 | 1.04 | 222 | 2.5 |
| 2.50 - 2.99 | . | 0.07 | 0.19 | 0.13 | 0.02 | . | . | . | . | 0.30 | 0.11 | 0.23 | 92 | 1.1 |
| 3.00 - 3.49 | . | . | 0.11 | . | . | . | . | . | . | 0.19 | . | 0.02 | 29 | 0.3 |
| 3.50 - 3.99 | . | . | . | . | . | . | . | . | . | 0.03 | . | . | 3 | 0.0 |
| 4.00 - 4.49 | . | . | . | . | . | . | . | . | . | 0.07 | . | . | 6 | 0.1 |
| 4.50 - 4.99 | . | . | . | . | . | . | . | . | . | . | . | . | 0 | 0.0 |
| 5.00 - GREATER | . | . | . | . | . | . | . | . | . | . | . | . | 0 | 0.0 |
| TOTAL CASES | 743 | 672 | 744 | 720 | 744 | 720 | 744 | 744 | 720 | 744 | 720 | 744 | 8759 | |

1990-1990 ATL WIS STATION: 145 LAT: 39.08 N, LON:-74.50 W, DEPTH: 17 M
PERCENT OCCURRENCES OF PEAK PERIOD BY MONTH

| Tp(sec) | JAN | FEB | MAR | APR | MAY | JUN | JUL | AUG | SEP | OCT | NOV | DEC | CASES | PCT |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 3.0 - 3.9 | 1.85 | 1.12 | 1.00 | 0.67 | 1.03 | 0.69 | 1.05 | 0.76 | 1.31 | 1.18 | 2.99 | 1.04 | 1287 | 14.7 |
| 4.0 - 4.9 | 1.64 | 1.44 | 1.39 | 0.91 | 1.52 | 0.92 | 2.25 | 0.66 | 0.91 | 1.70 | 2.29 | 1.00 | 1459 | 16.7 |
| 5.0 - 5.9 | 0.86 | 0.86 | 1.19 | 0.83 | 1.18 | 1.14 | 2.47 | 0.42 | 0.96 | 0.50 | 0.11 | 0.47 | 962 | 11.0 |
| 6.0 - 6.9 | 0.90 | 0.74 | 0.57 | 0.48 | 1.10 | 1.98 | 1.42 | 1.29 | 0.69 | 0.51 | 0.69 | 1.07 | 1001 | 11.4 |
| 7.0 - 7.9 | 0.98 | 0.63 | 0.89 | 0.72 | 2.16 | 2.52 | 0.69 | 2.64 | 1.48 | 1.02 | 0.33 | 0.42 | 1268 | 14.5 |
| 8.0 - 8.9 | 0.59 | 0.95 | 0.53 | 1.46 | 0.45 | 0.39 | . | 1.08 | 0.42 | 1.55 | 0.13 | 0.83 | 734 | 8.4 |
| 9.0 - 9.9 | 0.86 | 0.91 | 1.02 | 2.02 | 0.54 | 0.29 | . | 0.27 | 0.05 | 0.65 | 0.35 | 1.22 | 716 | 8.2 |
| 10.0 - 10.9 | 0.56 | 0.63 | 1.14 | 0.71 | 0.37 | 0.27 | . | 0.41 | 0.32 | 0.37 | 0.43 | 1.08 | 551 | 6.3 |
| 11.0 - 13.9 | 0.24 | 0.40 | 0.66 | 0.41 | 0.17 | 0.02 | 0.63 | 0.80 | 1.94 | 0.94 | 0.89 | 1.22 | 729 | 8.3 |
| 14.0 - LONGER | . | . | 0.10 | . | . | . | . | 0.15 | 0.14 | 0.08 | . | 0.13 | 52 | 0.6 |
| TOTAL CASES | 743 | 672 | 744 | 720 | 744 | 720 | 744 | 744 | 720 | 744 | 720 | 744 | 8759 | |

**Figure 10. Example of WIS statistical summaries.**

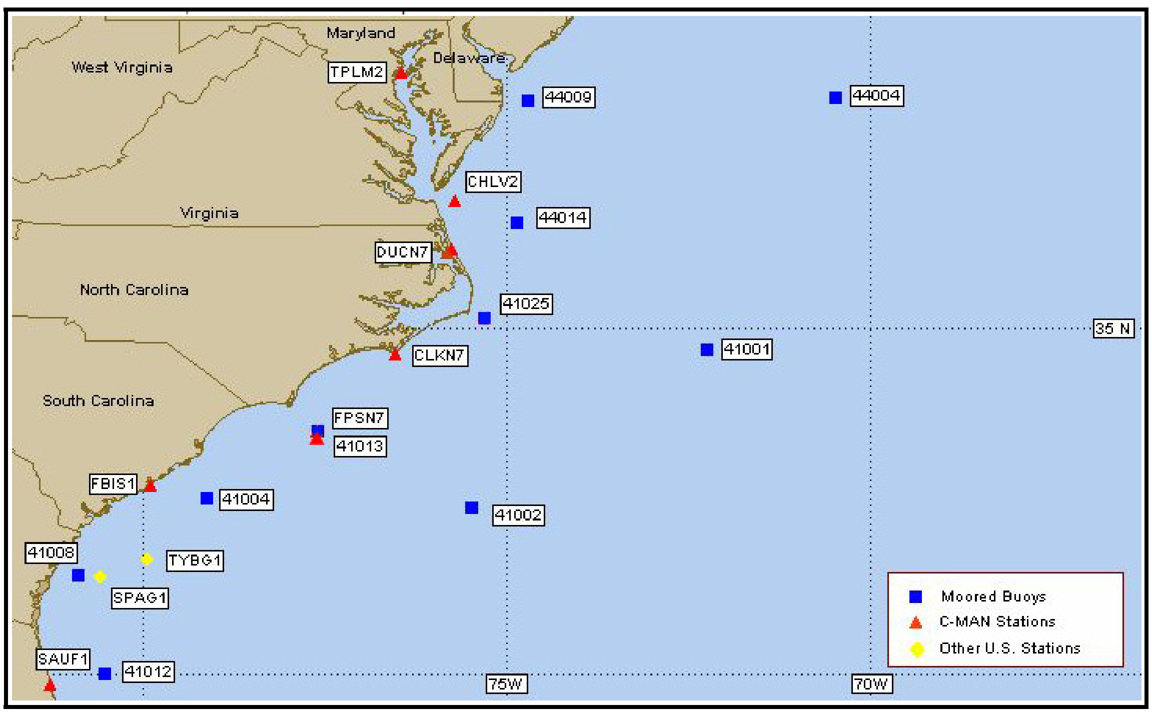


**Figure 11. NDBC buoy stations (East Coast, partial).**

**National Oceanographic Data Center**
This agency and website include similar data to the National Data Buoy Center, but covers the entire world, not just U.S. waters.

**Fleet Numerical Meteorology and Oceanography Center**
FNMOC prepares weather and wave forecasting for all oceans of the world. For the Atlantic Ocean, an example of the data for wave height by direction is given as Figure 12, and Figure 13 presents a sample illustration for swell height versus direction. The basic model is known as WAVEWATCH III. Figures 12 and 13 show a particular presentation of wave height and direction. Additional products include wave period and direction, swell heights by direction and several other forms. The emphasis of the available data appears to be forecasting. The data are available in tabular formats going back to July 1997.

**Naval Oceanographic Office**
This agency generally provides summaries of other oceanographic data, including temperature profiles and currents as well as waves. There are extensive data archives, but wave information is generally cross referenced to FNMOC and WAVEWATCH III.

**OceanWeather, Inc.**
Similar to the Pacific Ocean data that were discussed in an earlier section, 30 plus years of hindcast data for deep water that is based on carefully revised wind field analyses has been prepared for the Atlantic Ocean and Gulf of Mexico. Figure 14 presents examples showing the locations for which wave data are available. The grids are at 0.625 degrees longitude by 1 degree latitude.

### 2.2.5 Recommendations

The presently used procedure as outlined in the existing *G&S* should be retained. Checking the selected storm condition with general wave statistics from WIS should be included. A third check would be to use GROW with a suitable shallow water wave transformation model.

The Technical Working Group and a representative of the USACE (Dr. Don Resio) opined during Workshop 2 that the WIS database had been adequately updated over the years in terms of windfield modeling and is sufficient for wave data needed in the Flood Insurance Studies. Hence, the recommendation was to continue using the WIS database for the Atlantic and Gulf. The Working Group recommended the following items regarding the use of this database:

- Investigate the appropriateness of using either the 100-year significant wave height or the 20-year maximum wave height while modeling WHAFIS;
- Clarify use of equivalent deep water condition**;** and
- Clarify extrapolation to 1-percent-per-year risk level.

### 2.2.6 Preliminary Time Estimate and Cost for Guideline Improvement

The estimated time required for development of guidelines based on the use of WIS database is approximately 60 hours.

Table 4 at the end of this document summarizes the estimated hours for these portions of Topics 4 and 5.

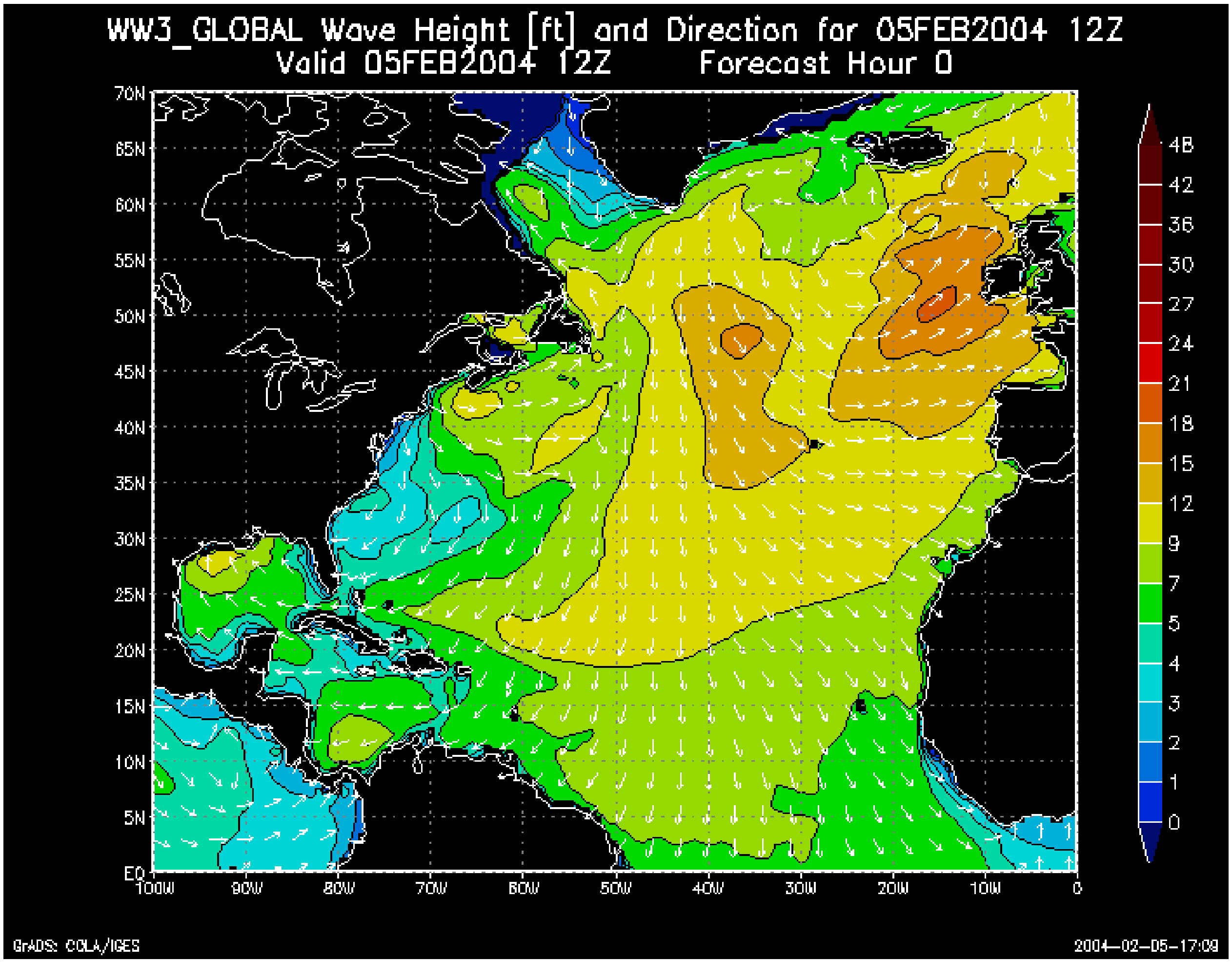


**Figure 12. Example of wave forecast from WAVEWATCH III.**

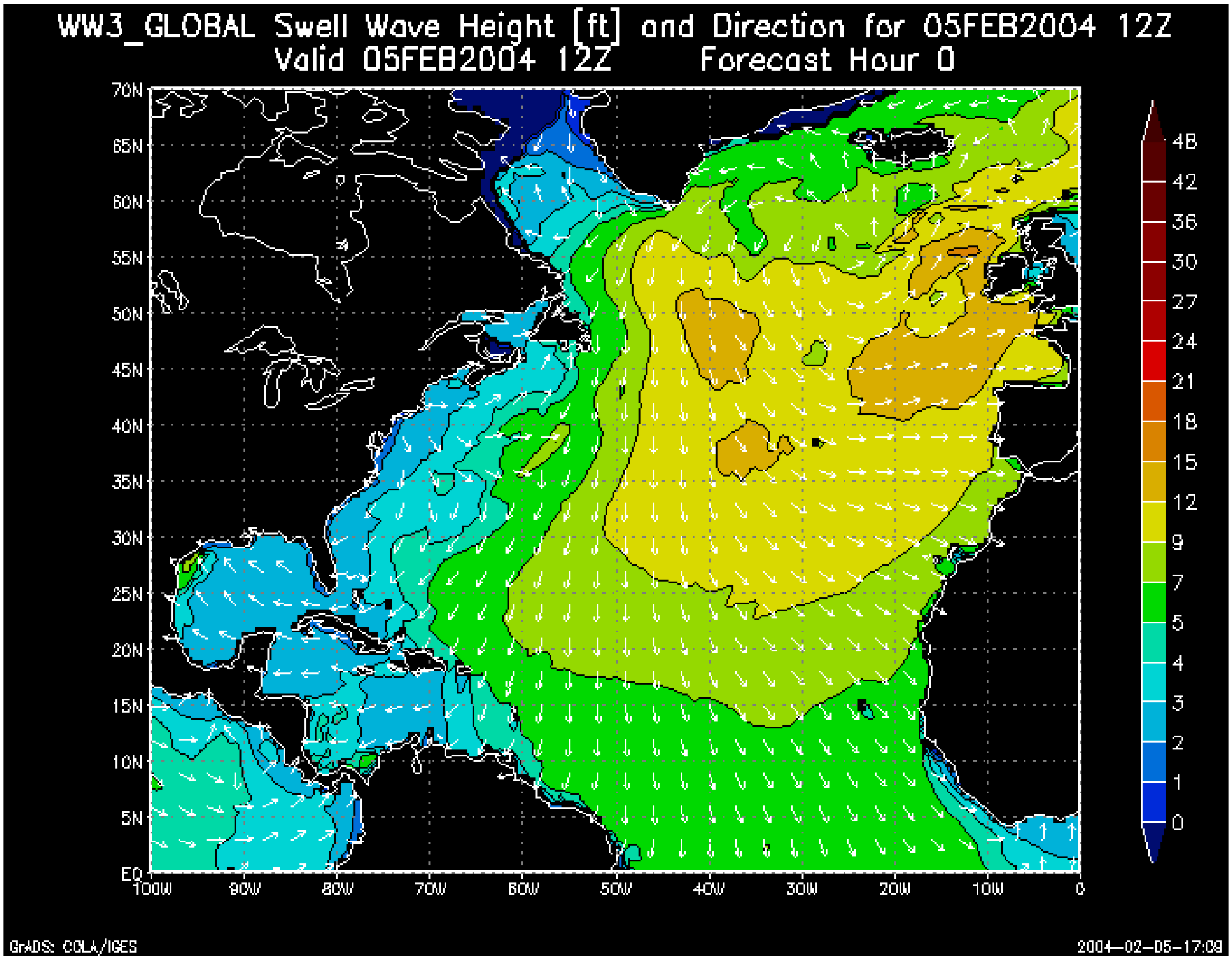


**Figure 13. Example of swell forecast from WAVEWATCH III.**

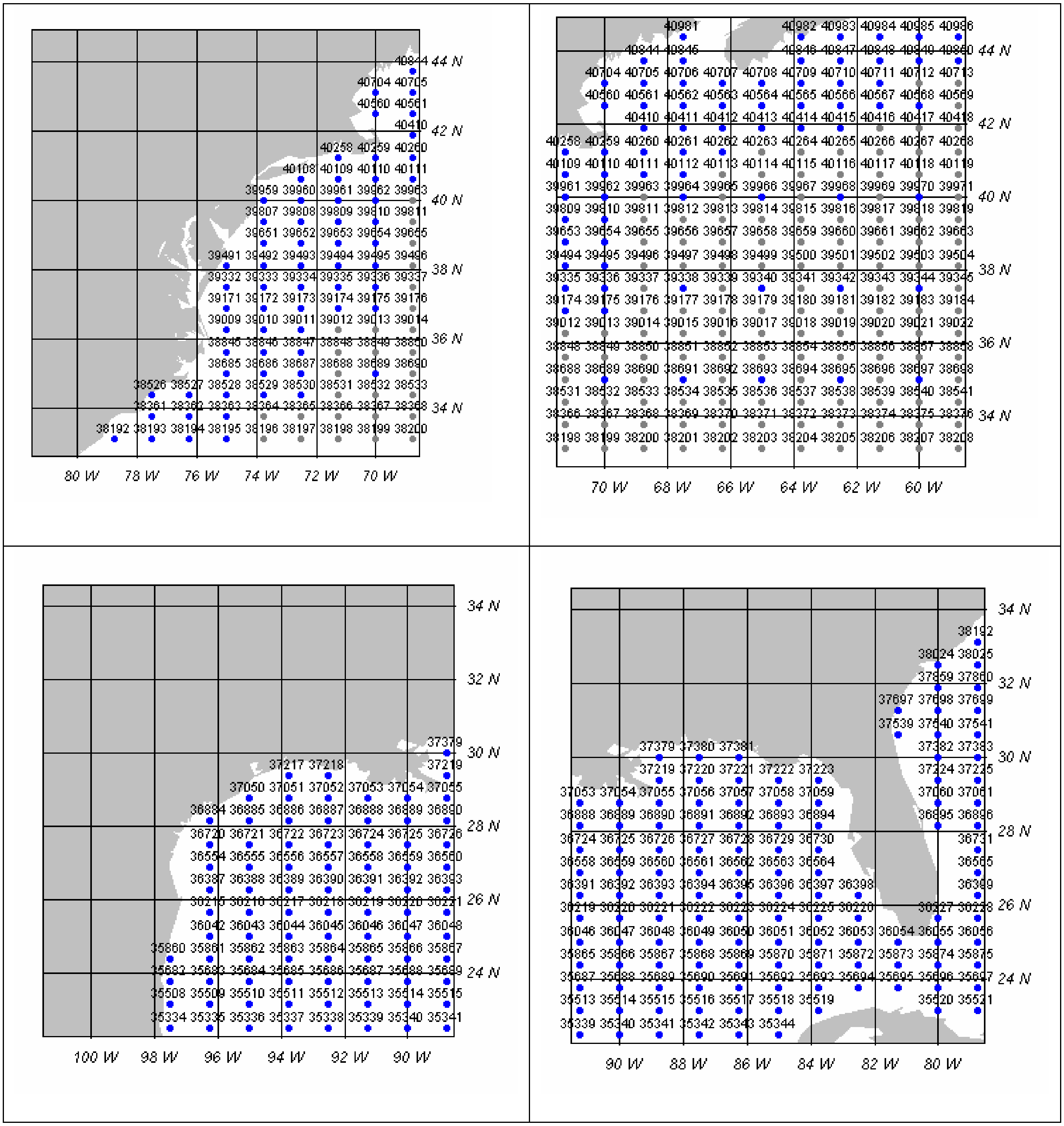


**Figure 14. Examples of available locations for GROW hindcasts.**

## 2.3 TOPIC 5: USE NEARSHORE REPRESENTATION OF WIND WAVES RATHER THAN OFFSHORE WAVE HINDCAST- SPECIFIC TO SOUTHERN CALIFORNIA BIGHT

### 2.3.1 Description of the Topic and Suggested Improvement

In the Southern California Bight (Point Conception to the Mexico border) the shelf is extremely broad and complicated by many islands and shoals. Deep water directional spectra are typically measured or hindcast at the edge of the shelf and wave transformation models that ignore wave generation or dissipation are used to predict nearshore wave conditions. The higher frequency portion of the spectrum (typically periods less than 9 seconds) can be affected by wind conditions encountered during the transit across the shelf. This process is difficult to model because of the lack of wind data and a very complicated wind field. An approach is needed to resolve the impact of local winds on high frequency portion of the spectrum for the Southern California Bight.

### 2.3.2 Description of the Procedure in the Existing Guidelines

There are no existing Guidelines on this topic. However, CDIP assumes that there is no wind-induced change in the spectrum in the Southern California Bight.

### 2.3.3 Applications of Existing Guidelines to Topic

This issue was not resolved in past Flood Insurance Studies.

### 2.3.4 Alternatives to Improvement

There are three alternatives for resolution of this issue. Alternatives are: (a) assume no wind-induced change in the spectrum, (b) attempt to model wind-induced changes, or (c) treat changes to the wind wave portion of the spectrum as an independent variable and use joint probability analysis techniques. Alternative (a) is presently used in the CDIP model. Alternative (b) requires the development and validation of a wind model of much higher spatial resolution than is presently available and could not be accomplished at present. Because the generation area for extreme swell events is typically very distant from the Bight, the local winds cannot be inferred from measured or hindcast wave data at the shelf edge. Alternative (c) considers that winds over the shelf are independent of the height of the extreme waves.

### 2.3.5 Recommendations

Substantial nearshore data exist to validate the magnitude of changes to the high frequency part of the spectrum during large events. A study of these data should be undertaken and the errors evaluated to determine if they are significant. This may require a subregional approach (i.e., wind effects in the Santa Barbara Channel may differ significantly from those off San Diego County). If the potential error is small, then alternative (a) in 2.3.4 should be used to establish the standard database of nearshore waves in Southern California. Note that this would result in a uniform approach being taken for the entire West Coast wave database because the broad shelf problem does not exist elsewhere on this coast. If the error is too large to be ignored, then a separate database of measured variations in the wind wave spectra should be undertaken. This will allow for the correction to be treated as an independent variable additive to the modeled nearshore spectrum.

### 2.3.6 Preliminary Time Estimate

The task could require from 120 to 140 hours, depending on whether alternative (a) or (c) in 2.3.4 is taken. Table 2 at the end of this document summarizes the estimated hours for this portion of Topic 5.

## 2.4 TOPIC 5: WAVE GENERATION IN SHELTERED WATERS – PACIFIC COAST

### 2.4.1 Description of the Topic and Suggested Improvement

Local wind conditions typically control wave heights in sheltered waters (non-open coast), such as Chesapeake Bay, San Francisco Bay, and Puget Sound. Storm seas in sheltered waters are typically limited by the size and shape of the water body, called “fetch-limited” seas. The procedures for estimating seas in this situation are referenced in the *G&S* for the Gulf and Atlantic Coasts, and the Great Lakes. The references refer to the USACE Shore Protection Manual (1984) and the USACE Automated Coastal Engineering System (1996) (ACES). No *G&S* are available for the Pacific Coast. The suggested improvements entail:

- Enhancing the *G&S* to include better guidance for calculating seas in sheltered waters;
- Updating the *G&S* to be consistent with the recent USACE Coastal Engineering Manual;
- Including improved methodologies used in the recent Region X flood studies; and
- Including contemporary methodologies, specifically third-generation wave generation models now widely available and in use.

### 2.4.2 Description of Procedures in the Existing Guidelines

There are no *G&S* procedures for the Pacific Coast. In this case, guidance can be derived from the *G&S* for other geographical areas. The same guidance is provided in the *G&S* for the other regions: Section D.2.2.7 Storm Wave Characteristics (page D-24 through D-26) for the Gulf and Atlantic Coasts, and Section D.3.2.6 Offshore Wave Characteristics (pages D-117 through D-121) for the Great Lakes. The guidance refers to the USACE SPM (1984) and ACES (1996) procedures for wind wave generation. The more involved analysis procedure is recommended where wind wave generation fetches are restricted by the complex geometry of water bodies such as sheltered waters. The method entails calculating a “restricted fetch” as the weighted average of a fan of fetches arrayed around the primary wind direction selected. This is described in the following section, Procedures for Restricted Fetches. This is one of several restricted fetch methods. This methodology is well documented in the USACE SPM and ACES listed above, including very specific guidance on the selection of wind parameters, adjustments to wind parameters for site conditions, and application of wind wave generation equations for both deep and shallow water (relative to generated wave length).

### 2.4.3 Application of Existing Guidelines to Topic-History and/or Implications for the NFIP

The existing *G&S* listed above are serviceable, but are based on older technology. A recent study in Region X (Sandy Point, Whatcom County, Washington – located in the Strait of Georgia) adopted an enhanced version of the restricted fetch method, called the "composite fetch" method (PWA, 2002). The USACE have updated their coastal analysis guidance with the Coastal Engineering Manual (USACE CEM, 2003), which supercedes the Shore Protection Manual (USACE SPM, 1984). Specifically, the wind wave generation equations for shallow water have been updated. Also, as noted in the CEM, more advanced and convenient computer-assisted analysis methods by the USACE and others are readily available and being used by many persons. These models are not presently approved for use on FEMA FISs.

### 2.4.4 Alternatives for Improvement

***Overview of Wave Generation in Sheltered Water***

Waves in sheltered water are characterized by locally generated waves (wind-waves) rather than swells (waves that have traveled some distance away from where they were generated).

Currently approved FEMA methods for wave generation are the SPM and ACES for restricted fetch wind growth and MIKE OSW model for deep and intermediate depth applications.

A discussion on wave hind-casting procedures is available in the CEM, (2003). There are two general types of prediction methods:

- **Empirical prediction methods:** These are based on the principle that universal laws govern interrelationships among dimensionless wave parameters. Relations between wave generating parameters and wave conditions have been established using wave observations during the 1940s and 1950s, and updated with more recent studies. The SPM and ACES methods traditionally used in FEMA studies are Empirical Prediction Methods.

- **Spectral Energy Models:** These are based on an energy balance equation that accounts for wave propagation processes and processes that add or remove energy from a particular frequency and direction component, at a fixed point at a given time. Spectral Energy Models have developed into first-generation, second-generation and third-generation models with successive improvements in wave prediction. The third-generation models are widely used today in deep-ocean, shelf-sea wave models such as WAM (WAMDI Group, 1988). In the present context other models that can be applied to shallower water are considered, such as SWAN, STWAVE and MIKE21 OSW).

Improved methods ranging from enhancements to the SPM (empirical prediction) methods to more advanced computer-aided analysis approaches are available. The more advanced computer-based Spectral Energy Models or wave action model are considered superior, but application procedures need to be developed for coastal flood studies.

The alternatives for improvement include:

- Updating the *G&S* to be consistent with the recent USACE Coastal Engineering Manual;
- Enhancing the *G&S* to include better guidance for calculating seas in sheltered waters;
  - Including improved methodologies used in the recent Region X flood studies; and
  - Including contemporary methodologies, specifically third-generation wave generation models now widely available and in use.

#### *Technical Background*

**Existing Procedures – Empirical Prediction Models:** Procedures for estimating storm seas in sheltered waters have traditionally followed the USACE Shore Protection Manual (SPM, 1984), classified as Empirical Prediction Models herein.

**SPM Procedures**

The SPM procedures are defined in Volume 1, Chapter 3, Section IV, Estimation of Surface Winds for Wave Prediction; Section V, Simplified Methods for Estimating Wave Conditions; and Section VI, Wave Forecasting for Shallow Water. The procedures are detailed in "cookbook" fashion, with enough technical background to allow appropriate enhancements. The heart of the procedures is the Sverdrup-Munk-Bretschneider (SMB) set of equations that relate wind speed, duration, and fetch to wind wave height and period. Modified equations are provided for shallow water (relative to wave length).

A key component of the SPM method is an iterative procedure to identify the fetch limited (maximum) seas. A wind speed is typically selected based on extremal analysis. Wind fields are assumed to include a distribution of speeds and durations, and each wind speed averaged over a particular duration (SPM, page 3-26). The wind field can therefore be considered as an array of wind speed – duration pairs, with faster speeds associated with shorter durations. This is depicted graphically in Figures 15 and 16 (SPM, Figures 3-12 and 3-13 of Pages 3-28 and 29). To calculate fetch limited seas, the fastest wind speed with long enough duration must be selected. Typically, this is accomplished by starting with a high wind speed, calculating the fetch-limited wave height, and checking that the duration-limited wave height is not smaller. If it is, then a slower wind with longer duration is tried. This iteration is repeated until the maximum fetch limited condition is established.

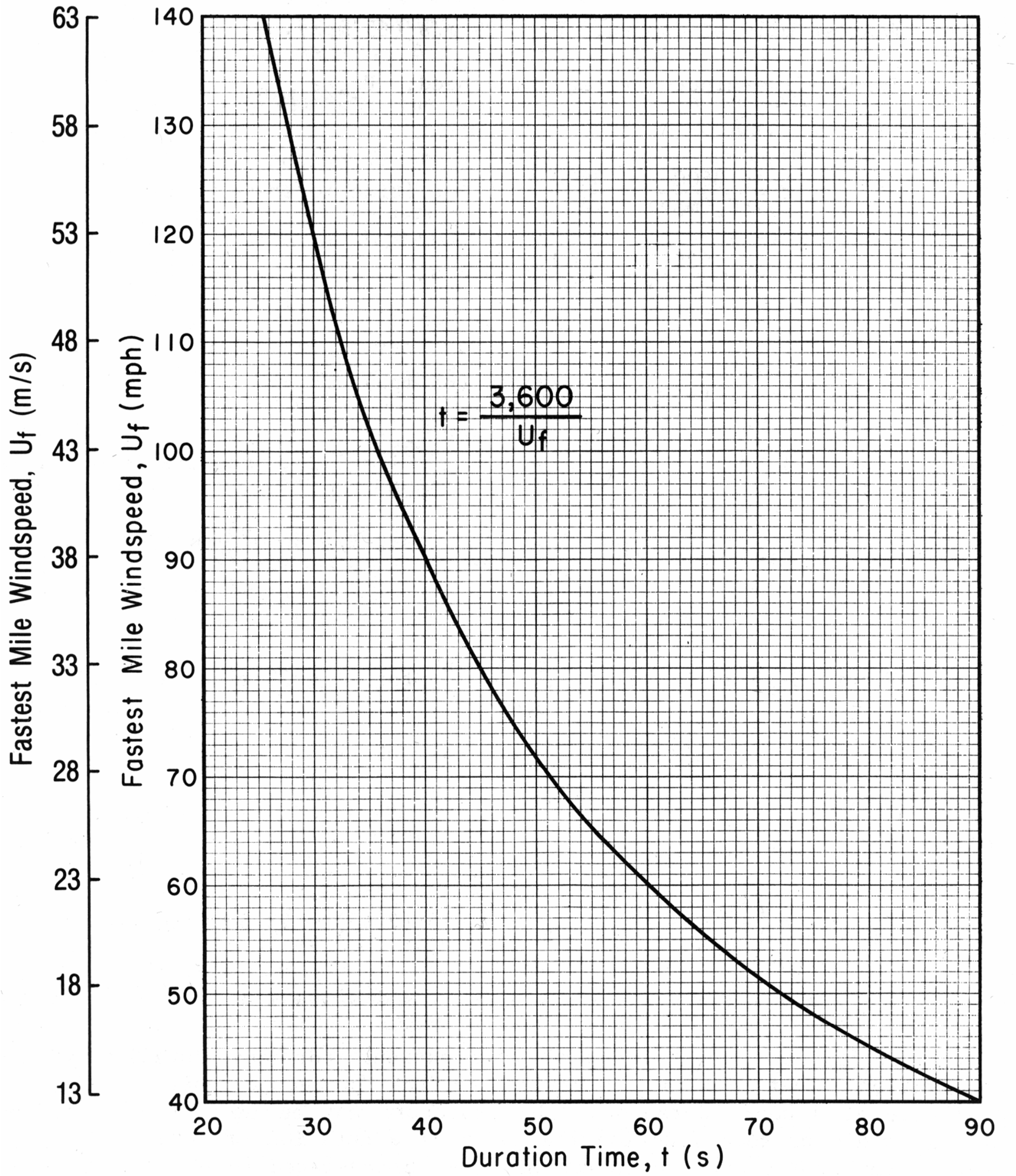


Figure 3-12. Duration of the fastest mile windspeed as a function of windspeed.

**Figure 15. Fastest mile windspeed vs. duration.**

(Source: Shore Protection Manual, 1984)

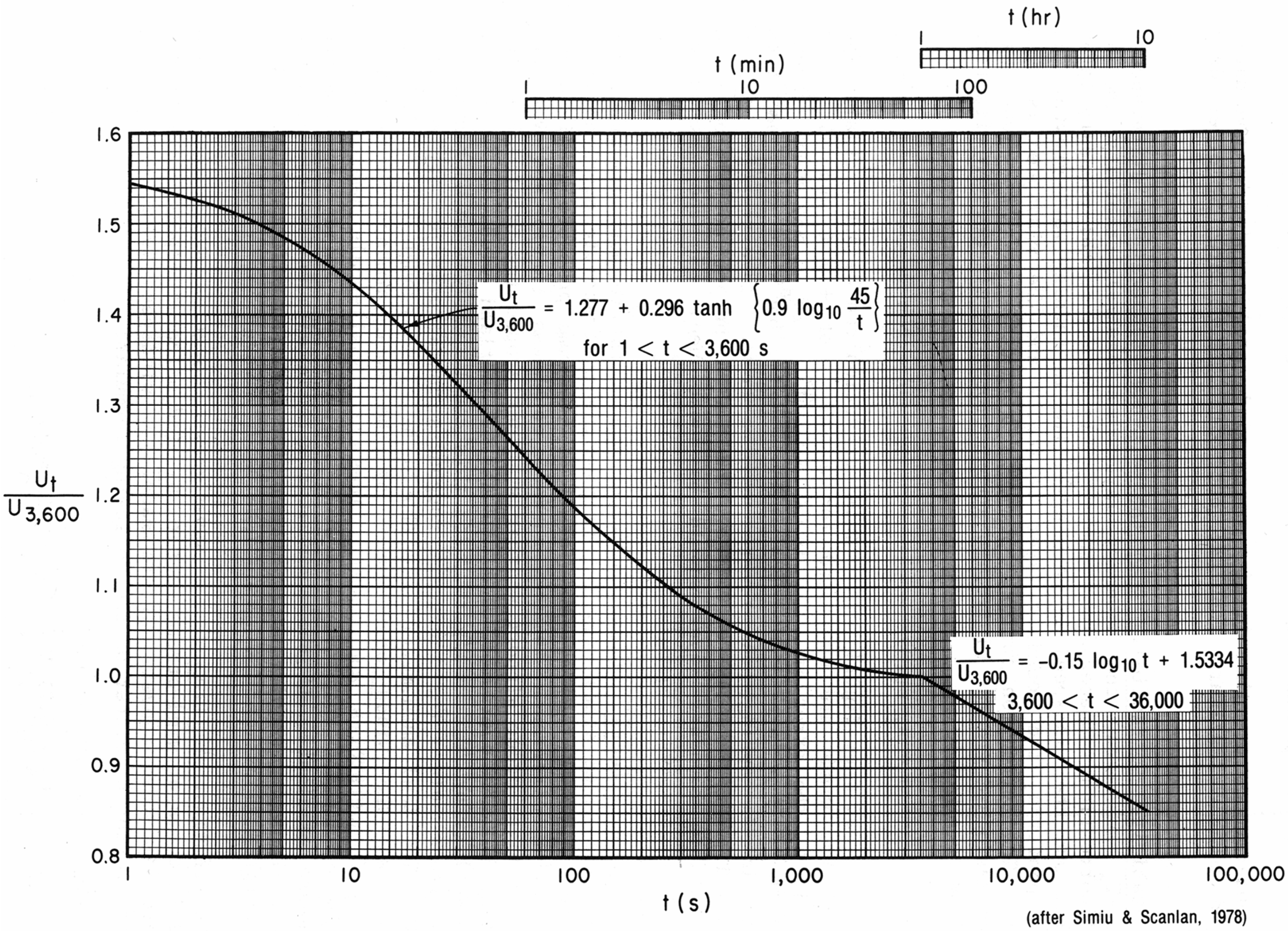


Figure 3-13. Ratio of windspeed of any duration, $U_t$ , to the 1-hour windspeed, $U_{3600}$ .

**Figure 16. Windspeed ratio to 1-hour windspeed vs. duration.**

(Source: Shore Protection Manual, 1984)

**Procedures for Restricted Fetches**

Special procedures are often applied for water bodies (embayments) with irregular planforms not easily represented by a single fetch length. This is typically called a "restricted fetch" condition. There are several ways of addressing a restricted fetch condition. The SPM notes that one procedure for addressing restricted fetch conditions, called the "narrow fetch" or "effective fetch" method, is no longer considered appropriate. This older method shortened the fetch based on considering the fetch width. This was based on the observation that wind waves were smaller in restricted fetch areas than open water areas. However, detailed field data indicated that the directional spread of wind waves was most narrow at the spectral peak, and therefore a simple shortening of the fetch could underpredict height and period. In an irregular embayment with the main axis of the open water in line with the primary wind direction, a straight-line fetch provided better results than the "effective fetch" method (SPM, page 3-51). However, the USACE does allow for restricted fetch analysis in cases where a straight line fetch may underpredict wave height and or period, such as when there are multiple but divergent open fetch areas, or the primary wind direction is not aligned with the axis of a longer open water area. These methods are called "restricted fetch" methods (Figure 17 from *G&S*, Figure D-37, page D-121).

**ACES Method**

The ACES method extends the standard SPM methods to account for restricted fetches. This method is referenced in the *G&S*, and was developed by the USACE. It is called the **ACES Method**, based on the name of the suite of computer programs within which the method is provided (Automated Coastal Engineering System [ACES] Version 1.07, USACE, 1992).

One wind direction and several radial fetch directions (up to +/-90 degrees) are considered. First, the minimum wind duration for a wave field to become fetch limited is evaluated. Then, the character of wave growth is determined (duration limited or fetch limited) and depending on the character, appropriate equations are used to estimate the wave conditions. Winds are not restricted to one direction during storm events and the winds from more than one direction can affect the wave growth.

The wave direction is found by maximizing an expression (product of a weighted fetch length and the weighted cosine of the angle between the fetch and the wind direction), which is assumed to then yield the maximum the wave period. The spectrum-based wave height (Hmo) corresponding to the above condition is calculated. The method does not explicitly consider energy transfers from the adjacent fetches in this approach. However, the method is based on the consideration of these processes. To provide a foundation for consideration of other restricted fetch methods, the physical processes are outlined below.

*Guidelines and Specifications for Flood Hazard Mapping Partners*

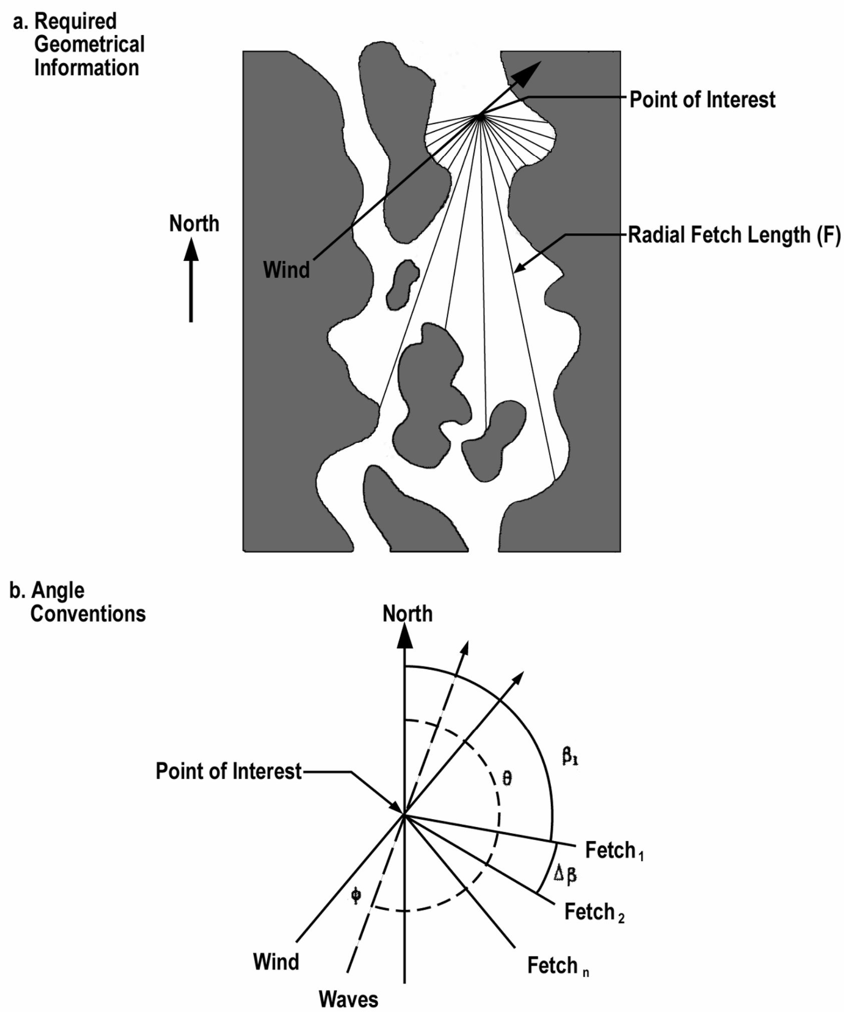


**Figure D-37. Outline of Geometry for Wave Development on Restricted Fetches, from ACES Direction of Wave Development (θ) is Defined by Maximizing the Product $[(F_\phi)^{0.25} \cdot (\cos\phi)^{0.44}\}$.**

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**Figure 17. Illustration of restricted fetch method.**

(Source: Appendix D, Guidelines and Specifications for Flood Hazard Mapping Partners, FEMA, 2002)

### Directional Wave Spectra in the Wave Generation Area

Within the wind wave generation area, seas have a broad directional distribution (Goda, 1985, Section 2.3.2; Seymour, 1977). The directional distribution is often conceptualized by a broad curve with a maximum energy (height) at the peak wave direction, decreasing with angular spread from the peak direction as shown in Figure 18 (Goda, 1985, Figure 2.12, page 30). A curve proportional to the cosine of direction squared, or higher power, is typically used to approximate the direction distribution. Near the frequency peak, a higher power is used to represent a narrower directional distribution typically found in the wave field. This concept of directional distribution of wave power in a wind wave field is used to account for restricted fetch conditions. The ACES method described above uses a weighted average of a fan of fetches to develop a single “effective restricted fetch” to use in the wave generation equations: The weighted average is based on the empirical directional distribution with selected power terms. The composite fetch method described below also uses this concept, but in a different manner.

### Composite Fetch Method

The composite fetch method applies the SMB equations of the SPM method to an array of fetches, and then combines the resulting wave conditions for each fetch using a weighting function (Seymour, 1977; USACE, 1989). The method described by Seymour (1977) uses a cosine squared directional distribution and the Joint North Sea Wave Project (JONSWAP) frequency spectrum. The methodology was found to give good results when compared to field data in San Diego Bay, California and English Bay, Vancouver, Canada. The method described by USACE (1989) is a computer program called NARFET, and also uses the cosine based directional distribution. This formulation is based on data collected in sheltered waters including Puget Sound, Washington, and inland lakes. The primary advantage of the composite fetch method is that it allows a reasonable wave estimate for very irregular embayments, where large fetch areas exist in the primary wind direction.

The Composite Fetch Method was recently applied in an FIS at Sandy Point, Washington, which is in Puget Sound–Strait of Georgia sheltered waters (PWA, 2002). Figure 19 shows the site and the fan of fetches used in the analysis. Wave hind-casting for Sandy Point followed the methods outlined in the USACE Shore Protection Manual (1984) and the spectral contribution method using the JONSWAP spectrum (Seymour, 1977).

Figure 20 was the calculated spectrum for waves arriving from the northwest direction. Note that the spectrum was bimodal, with two peaks corresponding to 8 and 11 second period. The lower frequency peak resulted from the long, deep fetch up the Strait of Georgia (300 degrees on Figure 19), which was the primary wind direction used to develop this spectrum. The other frequency peak resulted from the remaining shorter fetches. While the frequency spectra were not used for subsequent analysis, a range of wave periods were employed, consistent with the two peaks.

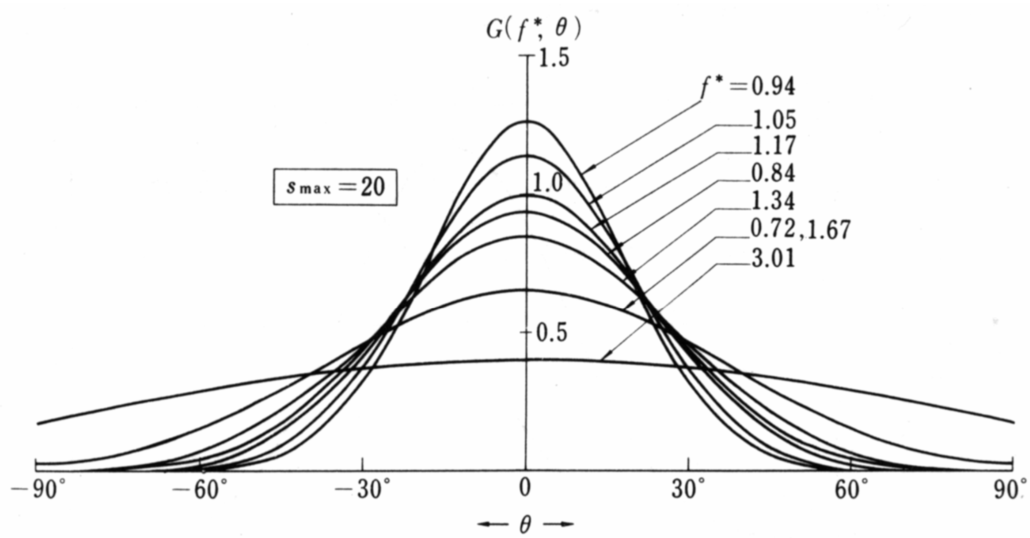


**Fig. 2.12 Example of Mitsuyasu-type spreading function for $s_{max}$=20.**

**Figure 18. An example of a spreading function.**

**(Source: Figure 2.12, Random Sea and Design of Maritime Structues, Y. Goda, 1985)**

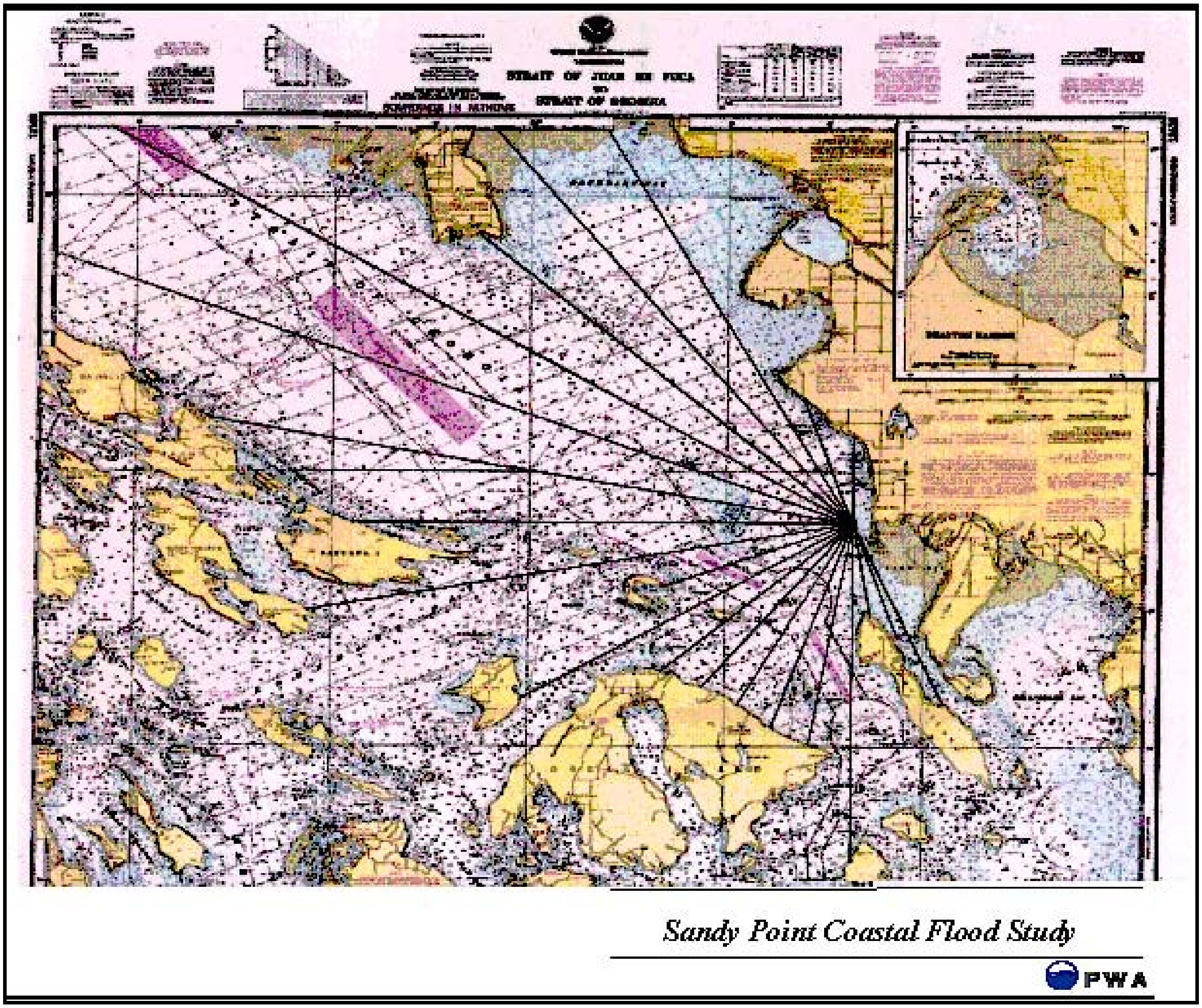


**Figure 19. Composite fetch method application at Sandy Point, WA.**

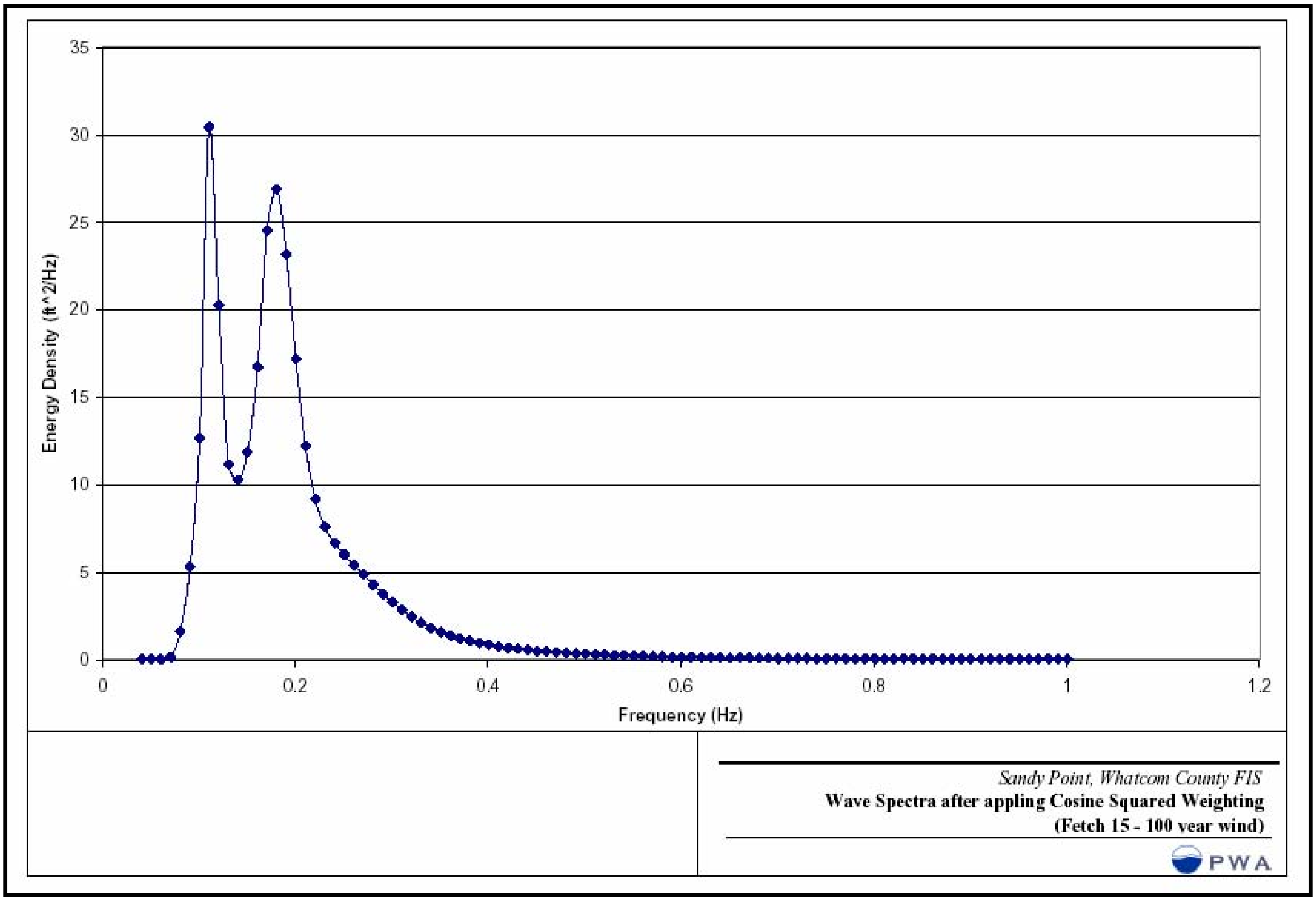


**Figure 20. Bimodal wave spectra.**

**<u>Changes in CEM (2003) compared to SPM (1984)</u>**

CEM suggests that where possible numerical models (e.g., Third Generation, Spectral Energy Models (SEMs)) should be used instead of the parametric models (Empirical Prediction Models). However, for shorter fetch lengths and simple situations where project costs would be minimal, CEM suggests the use of ACES program version of the parametric models (Called ACES Method herein). CEM also provides the Empirical Prediction Models similar to the SPM. Wind speeds in the equations are represented as friction velocities in the CEM, as opposed to wind stress factors in the SPM (1984). The CEM methods are described in Demirbilek et al. (1993). CEM and SPM methods are slightly different but results are expected to be comparable (Resio D. personal communication, 2004). Nomographs are also provided in the CEM, which states that these can be obtained using ACES more expediently.

The CEM recommends the use the deepwater wave growth formulae for all depths, including shallow water with the constraint that no wave period can grow past a limiting value for a given depth (Vincent 1985). This is a significant deviation from the SPM, which included different equations for shallow waters. This revisions result from studies by Bouws et al. (1985) and others. Interestingly, these studies indicate that the wave growth in shallow water is not dependent on the type of bottom sediment, but rather on the depth. A memorandum comparing the SPM and CEM methods have been prepared by Dewberry and Davis, LLC (2004) identifies the changes to wind-wave generation methods. The effect on results (calculated wave heights and periods) in FEMA flood studies should be evaluated before adopting the CEM changes.

An evaluation of the CEM method vs. the SPM method in shallow sheltered water areas would involve a comparison of the wave heights using both methods. An existing flood study (e.g., Sandy Point) can be used for the comparison because wind wave generation results based on the SPM method are already available. Testing can be accomplished in Phase 2 of this project.

### *New Procedures – Spectral Energy Models*

The spectral energy models are two-dimensional, computer-assisted numerical routines that use wave growth and decay (dissipation) terms to represent energy sources and sinks in the wave action balance or energy equations. These are also called third-generation wave models. The computer model packages listed below are capable of generation and transformation of waves. There may be several other similar third-generation wave models that are compatible and mentioning a few of the models as examples below does not endorse these codes to be superior to the others. An added benefit of using the third-generation models is that output can include a wave spectra useful as input into other spectral wave models that need the detailed spectra.

**SWAN**

SWAN is a numerical wave model used to obtain realistic estimates of wave parameters in coastal areas, lakes, and estuaries from given wind, bottom, and current conditions (SWAN user manual). The model represents the following generation and dissipation processes:

- Generation by wind
- Dissipation by white capping
- Dissipation by depth induced breaking
- Dissipation by bottom friction
- Wave-wave interactions
- Obstacles

The model is free and is widely used today but is not pre-approved by FEMA for flood studies. Recent investigation of wave growth and decay in the SWAN model shows good comparisons with measured data for limited fetch conditions in wind wave frequency ranges (Rogers et. al., 2002; Boil et. al., 1999). See Figure 21 (Fig.7 extracted from Rogers et. al., 2002). The model was applied to Lake Michigan and the Mississippi Bight, and “tuning” of the model is discussed. It is important to compare these two-dimensional models with the other approved models and measured data to evaluate the merits or de-merits of the models.

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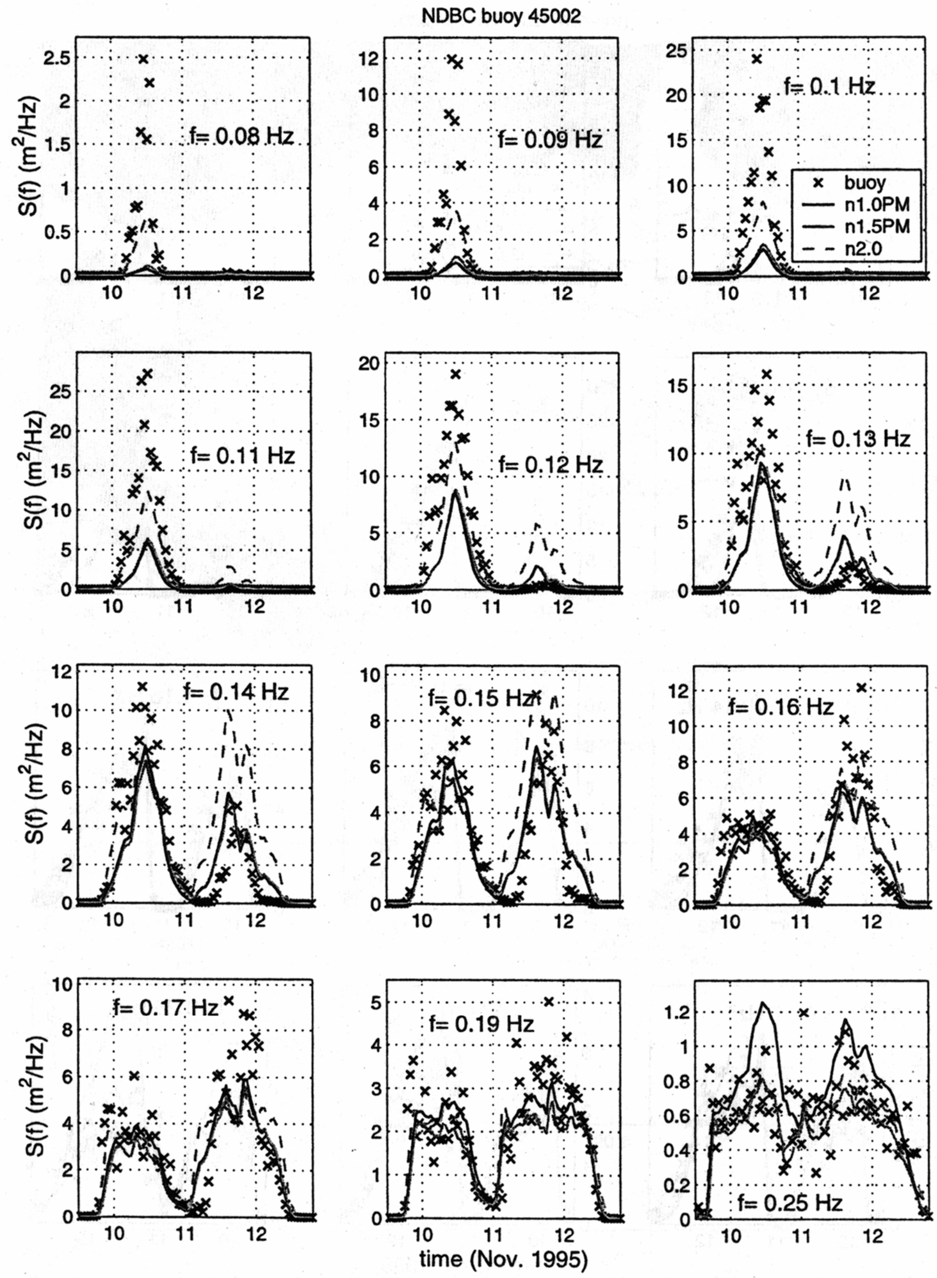


FIG. 7. Lake Michigan time series of frequency spectra at two NDBC buoy locations. The SWAN models are compared with measurements. The n1.0PM,NDS model is not included in this comparison, because it is practically identical to the n1.0PM model in this simulation: (a) NDBC buoy 45002 and (b) NDBC buoy 45007.

**Figure 21. A sample comparison of SWAN Model results with measured data.**

(Source: Journal of Physical Oceanography, Rogers, et al., 2002)

**STWAVE**

STWAVE is a steady-state wave transformation model that can include wind input and model wave growth. This model is widely used in USACE studies and has been used in small enclosed basins for wave generations and validated with the benchmarking system through a joint effort with Delft University of Technology, Delft, The Netherlands. Results are documented in voluminous comparisons on the Office of Naval Research (ONR) testbed project (testbed is discussed in the International Conference on Coastal Engineering [ICCE] 2002 proceedings, Smith, 2000, 2004).

Bottom friction is not implemented since there is little data for validation (Smith, personal communications, 2004). Unless propagation takes place over long distances in intermediate to shallow water, bottom friction may not be significant and STWAVE could still be used. However, Dally (personal communications) has measured surprisingly large damping over hard bottom (reefs), and to a lesser degree, sandy bottoms. In a very shallow basin bottom friction is potentially more important (say for propagation onto broad tidal flats) and STWAVE should be used with caution (see the Wave Transformations Focus Study Report).

**MIKE 21 OSW**

(following excerpts from http://www.dhisoftware.com/mike21/News/MIKE_21_OSW.htm):

> "MIKE 21 OSW is a fully spectral wind-wave model, which describes the propagation, growth and decay of short-period and short-crested waves in offshore areas. It includes wind generation, shoaling, refraction, wave breaking, bottom friction and wave-wave interaction. The output from the model consists of wave parameters including the significant wave height, peak wave period, average wave period, peak wave direction and mean wave direction."
>
> "Application of MIKE21 OSW in coastal areas (February 2001)"
>
> "Until recently Chi's fully spectral wind-wave model has mainly been used for large offshore areas and regional scale applications. New development and improvements have made the model also applicable in coastal and shallow water environment for various forcing conditions, see e.g. Johnson and Cooed-Hansen (J. Phys. Oceanogr., 30, pp. 1743-1756, 2000)."
>
> "In a recent paper wind-wave and air-sea interaction parameters were studied in two fetch-restricted coastal areas using the improved third-generation module in MIKE21 OSW. In the paper model results are compared with field data collected in water depth of 5 m (Femer Belt Model) and 7-10 m (Øresund Model)."
>
> "Recently, DHI has developed MIKE SW (not pre-approved by FEMA), which contains all the features of the MIKE OSW model but has a more flexible grid, making it more appropriate for deepwater to shallow water applications. MIKE NSW (approved by

FEMA) can also be used for wind wave generation and shallow water application, but this model is not a direct extension of MIKE OSW."

***Theory***

In the third-generation wave models (e.g., SWAN, STWAVE and MIKE OSW), the evolution of the wave spectrum is described by the spectral action balance equation. Wave growth and dissipation are accounted for by the source/sink terms, due to wind input, steepness and depth induced breaking and bottom friction. The equation solves the wave propagation in space and/or time and includes terms that represent frequency shifting and refraction due to variations in depth and currents. While STWAVE is stationary, SWAN and MIKE 21 OSW can be stationary or non-stationary (time dependent). The equations are solved on a forward marching technique over a finite difference grid.

***Application***

The models can be applied from deep to shallow water and for areas approximately in the range of 25–40 km (Although it can be applied to larger regions, the numerical scheme works better for mid-sized to smaller regions). The input data generally required to run the models are bathymetry, boundary conditions, wave spectra at the boundary (if any), wind speed and direction (one speed and direction for the stationary case). The output would be wave parameters (wave height, period and direction) and spectra at user selected grid points. Optionally other input (current, surge etc.) and output (wave setup etc.) are available depending on the model type. *G&S* could also include methods of converting data into usable input formats and also converting output into input needed in the other models for wave runup and setup.

The above third-generation models are widely used for wave generation in restricted fetches and sheltered water for design purposes but applications in FEMA Flood studies are not seen in the literature. Most of the applications in the literature are for validation and verification of the models using experimental measured data, or for tuning of model parameters. Some of the relevant applications of the SWAN model in the literature are at Lake Michigan and Mississippi Bight, (Rogers et al., 2002), partially enclosed basin between isles of Raasay and Isle of Skye, Lake George, Australia (Booij et al., 1999; Ris et al, 1999) and at Dutch Lake Ijssel, (Bottema et. al. 2003).

STWAVE has been applied in small enclosed basins for wave generations and validated with the benchmarking system through a joint effort with Delft. Results are documented in voluminous comparisons on the ONR testbed project (testbed is discussed in the ICCE 2002 proceedings, Smith, 2000, 2004)

Even with all the above testing, it is not clear how the results from SEMs differ from the parametric models traditionally used in FEMA flood studies, and in particular with extreme winds and waves.

#### *Guidance with Wind Input*

The above models are well documented in the respective user manuals. Wind speed and direction are important input parameters in the wave generation process and their usage in the models can vary from a simple uniform stationary wind field to a time and space varying wind field (in speed and direction) in the non-stationary modules. In the case of a flood study, the extreme event wind speed is parameterized as a single wind speed. Selection and conversion of wind data to model input needs guidance. Adequate guidance was not found in the literature, and therefore needs to be researched.

The wind input into non-stationary models is in the form of a time series. The other alternative is to run the model in stationary mode with a constant wind speed and direction. The assumption that waves have reached a steady state is implicit with this approach. This assumption is valid if the storm system lasts until the waves reach the maximum wave height for a given wind speed (fetch limited). Guidance is needed on using stationary vs. non-stationary modules.

#### *Uncertainty – Need to Evaluate Further*

Comparisons of simplified methods with the third-generation 2-D models are scarcely known although the third-generation model validations with wave measurements are ubiquitous in the literature. CEM (2003) recommends the third-generation models in design and planning situations and in most circumstances instead of the parametric models. Therefore a comparison of parametric methods (ACES, SPM, etc.) and the third-generation 2-D models is necessary as a baseline to continue using parametric methods and also for introducing 2-D models as an alternate method of wind wave generation for FEMA FIS. As a test case, the results from parametric methods and 2-D models can be compared with the measured data from an extreme event. The test cases also would help in defining wind input parameters for the 2-D models. An existing flood study site or an alternate site can be selected for testing. An existing flood study site would allow use of prior calculations and results.

### 2.4.5 Recommendations

Recommended improvements are:

- Write *G&S* for sheltered waters as part of the new *G&S* for the Pacific Coast geographic area, and include as an update to the existing *G&S* for the Gulf and Atlantic Coasts (could also be used for the Great Lakes geographic area, but this is not included in the present study);

- Update the existing language to be consistent with the USACE CEM. Specifically, evaluate the guidance in the CEM for revisions and clarify applications in FEMA studies. A focused study to compare results using CEM procedures to results using SPM procedures is recommended. An available FIS site or an alternative location can be selected for testing. Use of an available FIS site could simplify the study, if prior results and calculations are available, although the scope and purpose of the comparison should be clearly stated. The Sandy Point FIS is recommended because PWA recently completed this work and is familiar with the data and results;

- Describe a range of procedures that could be employed, as appropriate:
  - Existing Parametric Models Guidance, for Restricted Fetches, updated for CEM;
  - Enhanced Parametric Models, using the Composite Fetch Method recently employed in West Coast Sheltered Waters FISs;
  - Contemporary computer-assisted Spectral Energy Models (SEMs).
- A focused study to compare results from the SEMs and traditional Parametric Models, using restricted fetch methods. Application procedures for the SEMs would be clarified, specifically wind field definition.

### 2.4.6 Preliminary Time Estimates for Guideline Preparation

The Recommendations can be applied in about 400 to 500 person-hours, and in about 3 months elapsed time. Another 100 hours is recommended to allow participation of a technical review/steering committee, to be comprised of management and technical leaders presently working on the *G&S* review. Additional elapsed time to complete work may be needed to accomplish appropriate review and oversight: This indicates a 4-month timeframe is most appropriate. This estimate is based on use of the Sandy Point FIS data, which included all input data and results of the Parametric Model using Enhanced Composite Fetch Methods. Approximately, another 100 to 200 person-hours would be needed for additional analysis, if an alternate site is selected for testing. This estimate is for the analysis and report only. Review time for technical and institutional quality control is not included. These estimates are summarized in Table 2 at the end of this report.

### 2.4.7 Related Available and Important Topics if Any

Wave Transformations Focused Study, Study Topic 8: Swell and seas originating in the open ocean can penetrate coastal inlets, and may control coastal flood risk near the mouths of sheltered waters.

Wave Transformations Focused Study, Study Topic 9: Bottom friction factor used for very shallow waters may affect wind wave generation.

Storm Surge and wind setup may affect depths to the extent that wind wave generation is affected.

# 3 AVAILABLE TOPIC

## 3.1 WAVE DEFINITION- ATLANTIC/GULF AND PACIFIC (TOPIC 1)

### 3.1.1 Description of the Topic and Suggested Improvement

Matrix summary of need for Topic 1: Definitions of wave types using contemporary terminology and standardize the terminology.

The scope of this effort required that the focus report include definitions of wave types (swell, sea, storm, tsunami, etc.) and representative wave parameters such as significant wave height, controlling wave height for use in the Coastal Guidelines. The definitions are intended to provide descriptions of the storm wave characteristics in both the time domain and the spectral frequency domain. The research and review for this task required review of definitions presented in existing published materials, such as USACE Coastal and Hydraulics Lab (Coastal Engineering Manual), NOAA, and other national and international literature sources.

The reason this was considered a topic for further exploration is based on the Workshop 1 assessment that FEMA should have a glossary of wave terminology with definitions. The glossary would provide terminology related to commonly applied FEMA storm and wave characteristics and include other terms and notations that may be unfamiliar to those using or reviewing FEMA coastal flood study methodologies and techniques or coastal engineering in general.

The addition to the *G&S* of a direct link to a common resource for terminology would be useful for Study Contractors. To enhance Flood Mapping Partners ability to correctly use and understand the terminology of the coastal environment and physical processes that affect hazard assessment, Appendix D should require a specific section dedicated to providing the best available definition of this unique terminology.

The following was proposed for consideration and inclusion in Appendix D:

- Recommend the adoption of commonly used wave and hazard related terms encountered in the coastal environment (offshore and onshore). The following primary resources for inclusion in this task of the Storm Wave Characteristics Focus Study are:

    - Incorporate and refine the specific "Glossary of Coastal Terminology" from the CEM. It is comprehensive and ties in with past practices of FEMA reliance on the USACE as a Federal partner for assistance on coastal technical matters.

    - Incorporate entirely, the five listings of notations and parameters in the January 1986 publication from the International Association for Hydraulic Research titled, “List of Sea State Parameters.” These include:

(1) basic notations,
(2) general parameters and functions,
(3) standard parameters and functions,
(4) directional parameters and functions, and
(5) supplementary parameters and functions.

- A more significant and important task for this Focused Study group would be to provide specific guidance on how these terms relate to each other and should be applied relative to the following:

  - FEMA guidance for coastal flood studies,
  - Physical processes that are directly associated with FEMA coastal hazard assessments and flood mapping, and
  - Required coastal hazard study methodologies, techniques and models.

#### 3.1.2 Confirm Availability

Both the CEM and the IAHR lists are available for immediate use. Wherever possible in development of the guidance as a digital document, a link to these resources would be important in each section of the guidance.

#### 3.1.3 Preliminary Time Estimates for Guideline Improvement Preparation

Table 2 at the end of this document summarizes the preliminary Time Estimates for the Wave definition topic.

### 3.2 WAVE GENERATION IN SHELTERED WATER–ATLANTIC/GULF COASTS

#### 3.2.1 Description of the Topic and Suggested Improvement

The current practice is to apply the parametric models using the straight line fetch method (USACE SPM, 1984), restricted fetch method, or ACES program to generate the wave conditions at the site of interest. The wind-speed inputs into these methods are 60 mph for Northeaster-dominated areas (Northern Atlantic), and 80 mph for hurricane-dominated areas. The appropriateness of these wind conditions should be analyzed based on more recent information, and new guidelines should be provided for wind input selection. Also, the *G&S* should be clarified as to whether CEM and or SPM methods are to be employed.

The *G&S* for the Great Lakes and Gulf and Atlantic geographic areas are slightly out of date but functional. A suggested improvement is to update these based on the new version of the Pacific Coast *G&S*.

### 3.2.2 Confirm Availability

The current wind speeds adopted in FEMA FIS were suggested by the National Academy of Sciences (NAS 1977). These can be evaluated against more recent results from extremal analyses that are based on measured extreme wind speeds (see for e.g., National Hurricane Center web site, http://www.nhc.noaa.gov/HAW2/pdf/cat1.pdf). This and other available literature can be used to update guidance on wind speeds to be used in the event that wind data are not available for a particular FIS site. In simple terms, the currently used wind speeds could be increased to represent a higher category hurricane (e.g., Category 3 instead of category 1, etc.) that represents a 100-year return period wind speed.

The USACE CEM is readily available and in use. Required adjustments to update from the SPM to the CEM for the restricted fetch method are minimal. It is presumed that the guidance in the USACE CEM is sound, but implications to results for FEMA applications should be evaluated prior to use.

### 3.2.3. Preliminary Time Estimates for Guideline Preparation

To develop guidelines for the Atlantic and Gulf Coasts, based on Pacific Coast *G&S* and additional research, about 60 hours will be required.

Table 2 at the end of this document summarizes the preliminary Time Estimate for this topic.

# 4 ADDITIONAL OBSERVATIONS

The special case for hurricane-induced storm seas in sheltered waters has not been addressed, but may be important. There may be recent experiences, for example, Chesapeake Bay in 2003, from which observations and data can be used to evaluate the range of methods available.

The selection of waves for the open coast and sheltered water will be dependent on the methods chosen for analysis.  Two methods are under consideration: the Events Selection Method and Response-Based Method. The first method is a deterministic method that selects a single large forcing event, while the second method is a statistical method that performs frequency analysis on the response events as the result of many large waves. In Phase 2, these concepts will be further developed.

# 5 SUMMARY

The Storm Wave Characteristics Focused Study group was charged with developing recommendations on wave definitions; conversion from SPM to CEM on shallow water waves; and available sea and swell databases for Atlantic /Gulf and Pacific Coasts; and local seas for Sheltered Water. The swell and wave information from offshore is necessary for wave transformation from deepwater to nearshore and definition of wave conditions for the 1%-annual-chance-flood-event.

## 5.1 CRITICAL TOPICS

This study lists and critically looks at several sources of wave and swell data and recommends the following:

- For the Pacific Coast, GROW data is recommended, but updated WIS data is under development and is expected to include input from GROW. After this work is completed WIS may be the database of choice for the Pacific Coast.
- For the Atlantic and Gulf Coasts, the WIS database is sufficient.

For the Pacific, further studies are necessary to critically examine the lack of bias in the databases, formulate a methodology to prepare input data for wave transformation, and develop a suitable matrix of GROW directional spectra to ensure complete coverage of the deep water wave properties envelope. About 200 hours will be required for the Pacific Coast to complete these tasks over 3 months duration.

For the Atlantic/Gulf Coasts, the following guidelines on the use of WIS databases are needed:

- extrapolation to 100 years;
- appropriateness of using either the 100-year significant wave height or 20-year maximum; and
- clarification on extrapolation to 100 years.

The measured directional spectra from CDIP buoys contain the contribution from local wind. The modeled nearshore swell estimates for the Southern California Bight do not contain the contribution from local wind. A study of the available nearshore buoy records will be made to assess whether inclusion of the local wind will make a significant change in the high frequency part of the spectrum (typically periods less than 9 seconds). If there are significant changes, then a separate database will be proposed for measured variations in the wind wave spectrum. The task will take approximately 120 hours.

Improvements to the *G&S* are recommended for Storm Wave Characteristics in Sheltered Waters for the Pacific Coast. Traditional methods are available and have been successfully applied in recent FISs. These traditional methods are based on SPM guidance, and need to be reconciled with revised guidance in the CEM. In addition, the traditional methods rely on parametric models while more sophisticated spectral analysis models are now available and are being used in the industry. Hence, the updates to the *G&S* should address whether the spectral analysis models are approved for FEMA FISs, and how they should be applied. Further analysis is necessary to better understand how the results of the revised and new methodologies would compare with results from the traditional methods. It is recommended that analysis be conducted prior to revising the *G&S*. The proposed analysis will generally consist of applying the revised and new methodologies to the same data set, reviewing the results, and noting key steps and factors affecting the results. The proposed analysis is estimated to take up to 600 person-hours over a 3-month duration. An additional 100 person-hours and 1-month duration is estimated for technical oversight and review. These estimates presume that the study will be applied to data already available, probably from a recently completed FIS (the Sandy Point FIS is proposed), and additional time and costs are expected if the analysis is applied to a new site. The recommendations for all critical topic is summarized in Table 1.

### 5.2 AVAILABLE TOPICS

Several sources of wave definitions have been identified, including CEM and IAHR, to assist in the creation of a comprehensive set of definitions for all coasts of the continental U.S. in the time and frequency domain. Two separate sets of standardized definitions, and a specific listing and definition of common notations will be created for Atlantic/Gulf and Pacific coasts. About 240 person-hours will be required for this effort.

It is suggested that the wave generation issues in the sheltered waters for the Atlantic and Gulf Coasts can be improved based on the Pacific Coast *G&S* and additional research on wind conditions based on measured wind speeds. This effort will take about 60 person-hours. The recommendations for all available topics is summarized in Table 1.

## 6 RECOMMENDATIONS

Table 1 is a summary of recommendations for Storm Wave Characteristics Critical Topics and Available Topics. Note that the focused study combined Topics 4 and 5, incorporated a portion of Topic 3 into Topics 4 and 5. Other elements of Topic 3 (e.g., wave runup and wave setup) were considered in other focused studies.

**Table 3 Summary of Findings and Recommendations Storm Wave Characteristics**

| Topic Number | Topic | Coastal Area | Priority Class | Availability / Adequacy | Recommended Approach | Related Topics |
|---|---|---|---|---|---|---|
| 4 and 5 | Sea and Swell | AC | C | MIN | WIS database is recommended for use. Clarify extrapolation to 100-year; investigate appropriateness of using either 100-year significant wave height or 20-year maximum. Clarify use of equivalent deepwater wave - definition (Topic 1) | 8, 9, 51 |
| | | GC | C | MIN | | |

**Table 3 Summary of Findings and Recommendations Storm Wave Characteristics**

| Topic Number | Topic | Coastal Area | Priority Class | Availability / Adequacy | Recommended Approach | Related Topics |
|---|---|---|---|---|---|---|
| | | PC | C | MAJ | 1. GROW database is recommended for use in near term for swell and sea. Confirm lack of bias in GROW database. WIS can be used after completion of current revision. CDIP data can be used for model verification.<br>2. Develop *G&S* for preparation of input data for wave modification models based on GROW directional spectra.<br>3. Conduct a study of the available nearshore data for Southern California Bight to assess whether inclusion of the local wind will make a significant change in the high frequency part of the spectrum | |
| | | SW | C | MAJ | Add guidance on use of Coastal Engineering Manual (CEM); conduct a focused study to confirm that Shore Protection Manual (SPM) results are similar (validation for previous studies). Conduct a focused study and describe procedures for: (1) existing parametric model guidance; (2) enhanced parametric models; (3) spectral energy models | 6, 8, 9, 51 |
| 1 | Wave Definitions | AC | A | Y | The recommended approach includes: (1) adopt the CEM “Glossary of Coastal Terminology” and International Association of Hydraulic Engineering and Research “List of Sea State Parameters” (for notations); and (2) clarify the correlation of these terms to the actual guidance and various methodologies to ensure consistency | 4, 5, 50, 51 |
| | | GC | A | Y | | |
| | | PC | A | Y | | |
| | | SW | A | Y | | |
| 5 | Local Sea - Guidelines for Local Sea | SW (Atl) | A | Y | The recommended approach is to update *G&S* based on Pacific Sheltered Water G &S. | 6, 51 |

| Table 3 Summary of Findings and Recommendations Storm Wave Characteristics | | | | | | |
|---|---|---|---|---|---|---|
| Topic Number | Topic | Coastal Area | Priority Class | Availability / Adequacy | Recommended Approach | Related Topics |

Key:
Coastal Area
AC = Atlantic Coast; GC = Gulf Coast; PC = Pacific Coast; SW = Sheltered Waters
Priority Class
C = critical; A = available; I = important; H = helpful
(Recommend priority italicized if focused study recommended a change in priority class)
Availability/Adequacy
"Critical" Items: MIN = needed revisions are relatively minor; MAJ = needed revisions are major
"Available" Items: Y = availability confirmed; N = data or methods are not readily available
"Important" Items: PRO = procedures or methods must be developed; DAT = new data are required;
PRODAT = both new procedures and data are required

| Table 4 Preliminary Time Estimate for Guideline Improvement Preparation | | |
|---|---|---|
| Topic Number | Item | Time (Hours) |
| **Swell and Sea- Pacific Coast** | | |
| 4 & 5 | Review GROW Publication | 40 |
| | Develop and define techniques for input format for wave modification models | 80 |
| | Prepare description of interface process | 40 |
| | Coordinate with WIS Pacific Coast Revisions | 40 |
| | TOTAL | 200 |
| **Offshore Wave Data-Atlantic/Gulf** | | |
| 4 & 5 | Investigate 100-year significant wave height or 20-year max. | 60 |
| | Clarify use of equivalent deep water condition | 40 |
| | Clarify extrapolation to 100-year | 20 |
| | TOTAL | 120 |
| **Wind waves in Southern California Bight** | | |
| 5 | Evaluate error in nearshore wave data with respect to local sea | 90 |
| | Recommend an approach | 30 |
| | TOTAL | 120 |
| **Wave Generation in Sheltered Waters-Pacific** | | |
| 5 | Write *G&S* for sheltered water and include as an update to the existing *G&S* for Gulf and Atlantic Coasts. Describe a range of procedures that could be employed. | 100 |
| | Compare CEM and SPM procedures using a case study (an existing FIS site) and clarify application of CEM in FEMA studies | 100 |
| | A focused study to compare SEMs and traditional parametric models using restricted fetch methods. Application procedure for SEMs including wind field definition | 300 |
| | Allow participation of a technical review | 100 |
| | TOTAL | 600 |
| **Guideline Preparation-Pacific Coast** | | |

| Table 4 Preliminary Time Estimate for Guideline Improvement Preparation | | |
|---|---|---|
| Topic Number | Item | Time (Hours) |
| 1 | Using the compiled glossary of terms and notations (from CHL and IAHR sources), correlate each of key terms with the coastal methodologies and application. | 80 |
| | Prepare for application within Appendix D | 80 |
| | Prepare for application for Pacific Coast Guidelines | 80 |
| | TOTAL | 240 |
| **Wave Generation in Sheltered Water-Atlantic/Gulf** | | |
| 5 | Develop Guidelines based on Pacific Coast | 60 |
| | TOTAL | 60 |

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# Wave Transformation

## FEMA Coastal Flood Hazard Analysis and Mapping Guidelines Focused Study Report

February 2005


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**Acronyms**

| | |
|---|---|
| ACES | Automated Coastal Engineering System |
| CDIP | Coastal Wave Data Information |
| CEM | Coastal Engineering Manual |
| FHF | Flood Hazard Factor |
| GROW | Global Re-analysis of Ocean Waves |
| NGCD | National Geological Data Center |
| USACE | U.S. Army Corps of Engineers |
| WES | Waterways Experiment Station |
| WIS | Wave Information System |

# 1 INTRODUCTION

This document describes an approach to develop guidelines for addressing Wave Transformation, as part of new Guidelines and Specifications (*G&S*). Four study topics are addressed as listed below.

## 1.1 CATEGORY AND TOPICS

This paper addresses Wave Transformations, which is a focus study area comprising four Study Topics:

| Wave Transformation Topics and Priorities | | | | | |
|---|---|---|---|---|---|
| Topic Number | Topic | Topic Description | Priority | | |
| | | | Atlantic / Gulf Coast | Pacific Coast | Non-Open Coast |
| 7 | CDIP California | California Regional Wave Transformation Models | -- | C | -- |
| 8 | Overall Wave Transformation | Wave Transformations With and Without Regional Models | H | C | C |
| 9 | Dissipation | Wave Energy Dissipation over Shallow, Flat Bottoms | C | H *(C)* | C |
| 10 | WHAFIS | Overland Wave Propagation; Candidate Improvements to WHAFIS | I *(C)* | I *(C)* | H |
| Key: C = critical; A = available; I = important; H = helpful<br>(Recommend priority italicized if focused study recommended a change in priority class) | | | | | |

Study Topic 8 also encompasses Topics 7, 9, and 10. Therefore, topic 8 is discussed before topic 7 in this report. Revisions were made based on information exchanged in Workshop 2, held in Sacramento, February 23-26, 2004. Version 2.0 was provided before the workshop. This is Version 5.0.

## 1.2 WAVE TRANSFORMATION FOCUSED STUDY GROUP

The Wave Transformation Focused Study group members are Bob Battalio, the leader of the study group, Carmela Chandrasekera, Richard Seymour, Bill O'Reilly, Darryl Hatheway, Terry Hull, Rajesh Srinivas, and David Divoky.

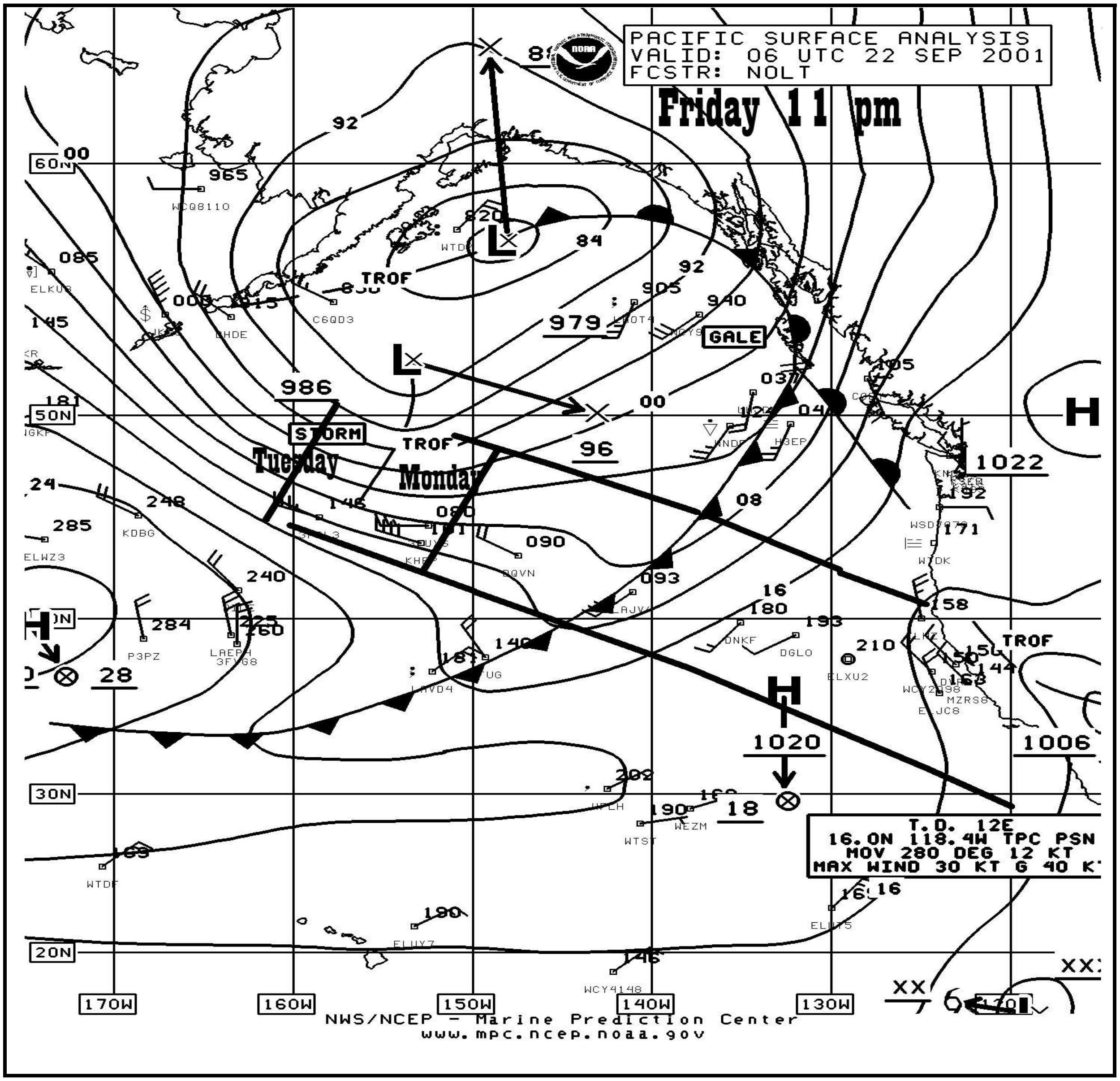
## 1.3 GENERAL DESCRIPTION OF WAVE TRANSFORMATION PROCESSES AND PERTINENCE TO COASTAL FLOOD STUDIES

Wave Transformation refers to changes in wave characteristics during propagation, generally propagating from deep through shallow water. The primary processes affecting wave transformations are refraction, diffraction, shoaling, dissipation, and nonlinear effects. Wave refraction is a key process affecting the distribution of wave energy and power, and hence the potential for coastal flooding along a shoreline. Wave refraction results from a change in local wave propagation speed due primarily to local depth changes. Wave refraction can result in convergence or divergence of the wave energy producing changes in wave height as well as wave direction in the nearshore. Diffraction of water waves is a phenomenon in which energy is transferred laterally along the wave crest. As waves slow down in shallow water, wave-length reduces and wave height increases. The increase in wave height is referred to as wave shoaling. As waves move into shoaling water they eventually become unstable and break. Wave breaking is the prominent method of wave energy dissipation. Waves also lose energy due to bottom friction and viscous damping when they propagate over shallow and intermediate waters, and through inundated marshes.

Wave transformations are important processes to consider in coastal flood studies, especially where long period swell is prevalent, and where coastal morphology focuses wave action. Wave transformations are addressed as an intermediate step between forcing processes (wave generation) and response processes (wave runup and overtopping). An example of forcing processes is provided in Figure 1, which shows the surface atmospheric pressure and wind fields estimated for a north Pacific storm.  The heavy black lines and text identify the zone expected to generate swell incident to the California shore approximately 3 days later. Figure 2 characterizes swell exposure for the central California Coast from close range and distant storms, in terms of swell travel path and time. The West Coast of the United States is exposed to large swell from distant storms, resulting in very long wave lengths that are especially sensitive to wave transformation processes such as depth-induced refraction. An example of response processes is provided in Figure 3, which shows coastal flooding in Pacifica, California, caused by wave runup and overtopping.

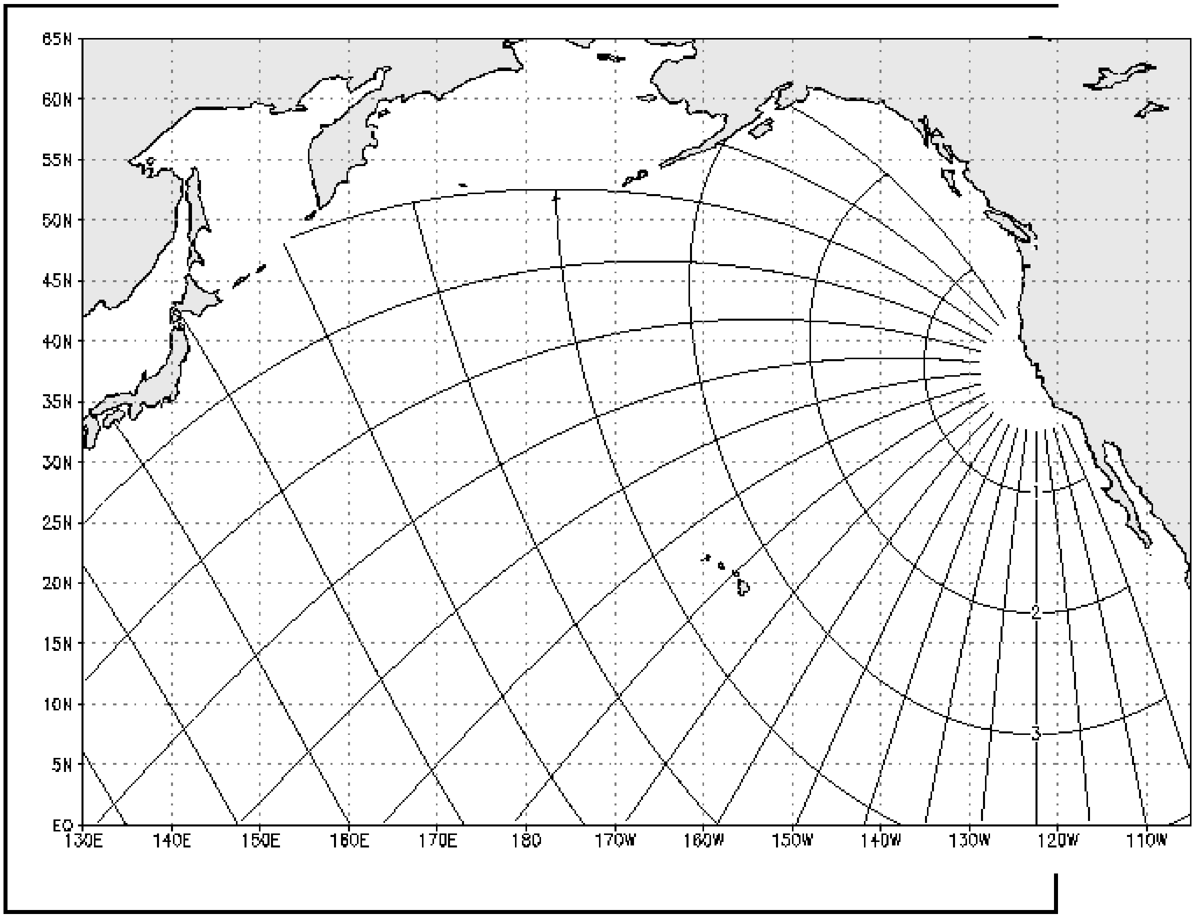
Regional wave transformation modeling is ongoing in California, most notably via the Coastal Wave Data Information Program (CDIP). Figure 4 shows a public-domain output of wave refraction modeling by CDIP for the San Francisco, California area. The graphic shows that wave transformations can greatly increase swell exposure for some areas and decrease it in others. This point will be amplified by looking at swell observations for three locations identified in Figure 4:

1. Ocean Beach, San Francisco;
2. Bolinas Bay and Lagoon Inlet, Marin County;
3. Crissy Field Shore and Lagoon Inlet, San Francisco (inside San Francisco Bay).




Map source, NOAA internet site: Forecasting Marks, Approximate Analysis by Bob Battalio.

**Figure 1. Offshore wave generation by a North Pacific Storm, with Forecast Swell arrival on the West Coast of the U.S. about three days later.**




Source: Public Internet Site.

**Figure 2. North Pacific swell paths incident to central California. Rings are one-day travel distances for moderate period swell, and radial lines are swell travel paths.**

*(Photos by Bob Battalio).*

Source: CDIP Internet Site

**Figure 3. Wave setup, runup, overtopping and coastal structure damage, Pacifica, CA.**

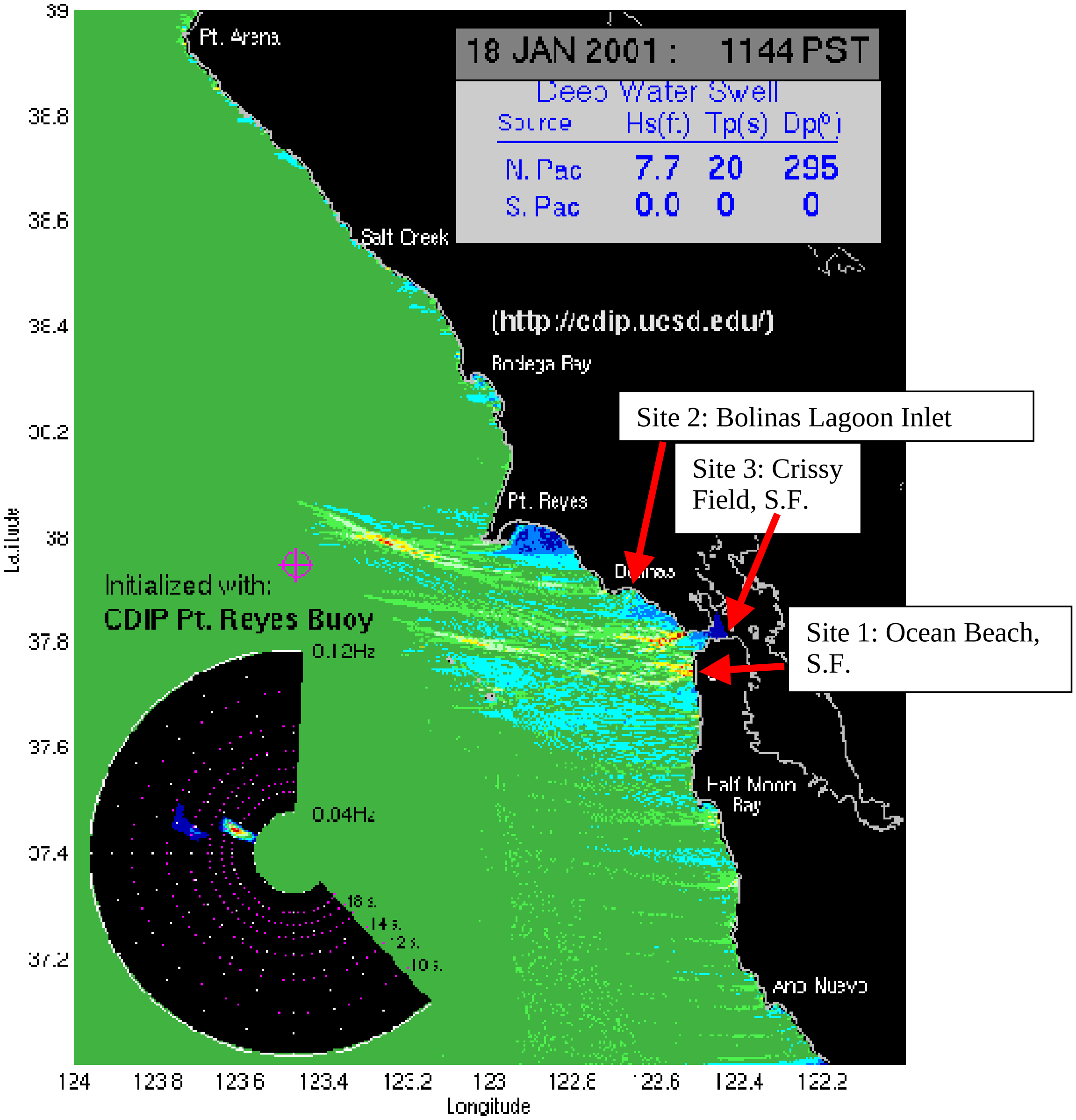



**Figure 4. Wave transformation modeling by the Coastal Data Information Program at Scripps Institution of Oceanography.**

The CDIP regional wave modeling predicts large waves at Site 1, smaller waves at Site 2 and much smaller waves at Site 3.

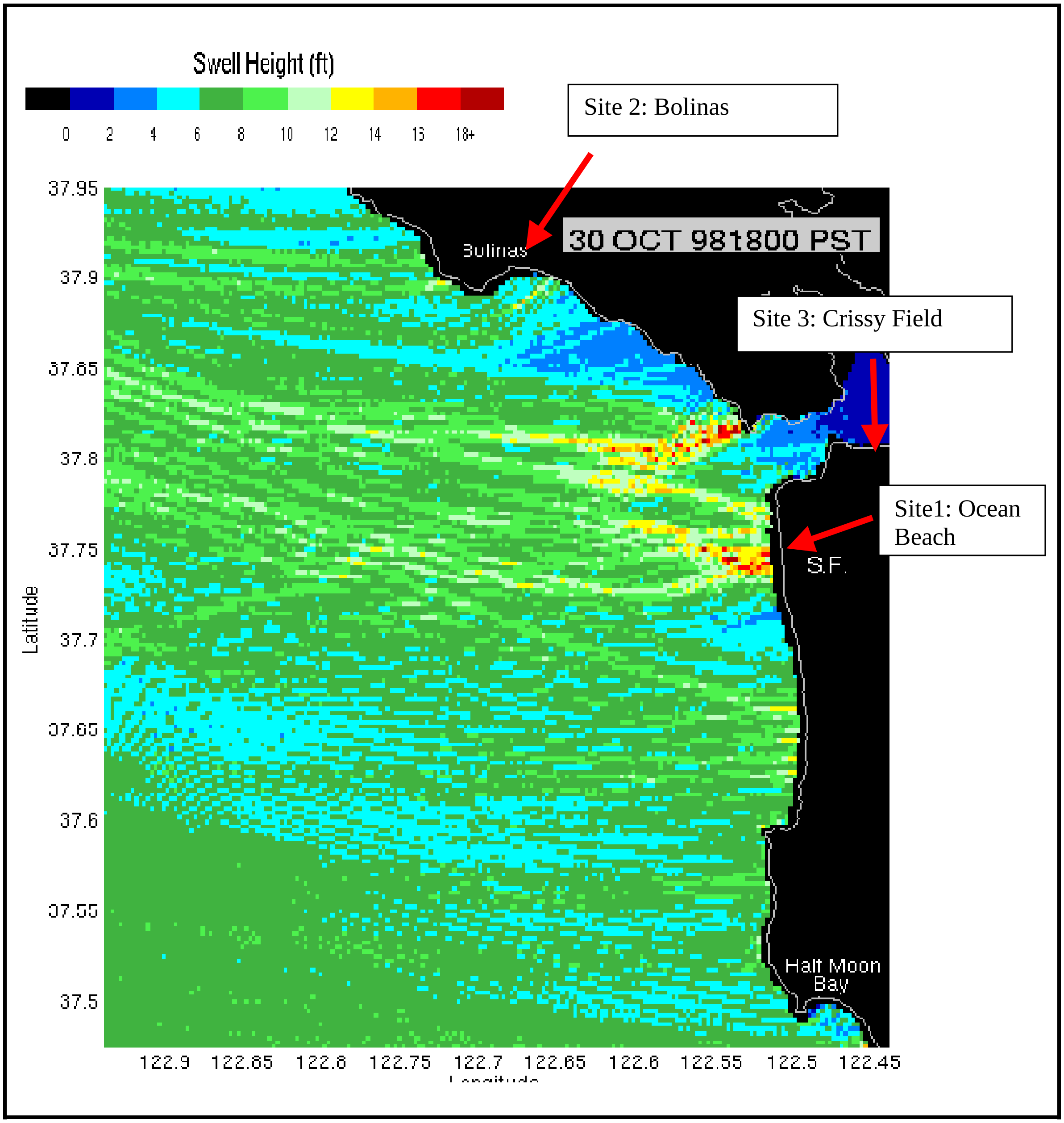
Figure 5 is another CDIP product showing a close-up view of wave refraction effects around the San Francisco Golden Gate area. An aerial photograph of Site 1 (Ocean Beach) shows wave crossing patterns consistent with the CDIP modeling (Figure 6). Figures 7 and 8 are photographs of breaking waves at Site 1 with heights on the order of 20 to 40 feet. Note that these waves are long-period swell with little relation to local weather.

Wave exposure at Site 2, Bolinas Bay, is characterized in Figure 9. This figure shows wave heights measured offshore and used as input to CDIP refraction modeling, wave heights measured nearshore, and wave height output from the CDIP modeling for the nearshore location. A comparison between the modeled and measured wave heights shows good agreement and confirms that wave transformations greatly reduce incident wave heights for this section of the coast.

Wave exposure at Site 3, Crissy Field, is characterized in Figures 10 and 11. Crissy Field is located in the San Francisco Bay, and swell has propagated through the Golden Gate. Figure 10 shows a time series of wave heights and periods measured offshore in the Pacific Ocean and near the Crissy Field shore. While the wave heights are much lower at Site 3 than in the open ocean, maximum heights from swell can approach the height of other locally generated wind waves potentially governing coastal flooding potential. Figure 11 is a photograph of a swell breaking at Crissy Field.

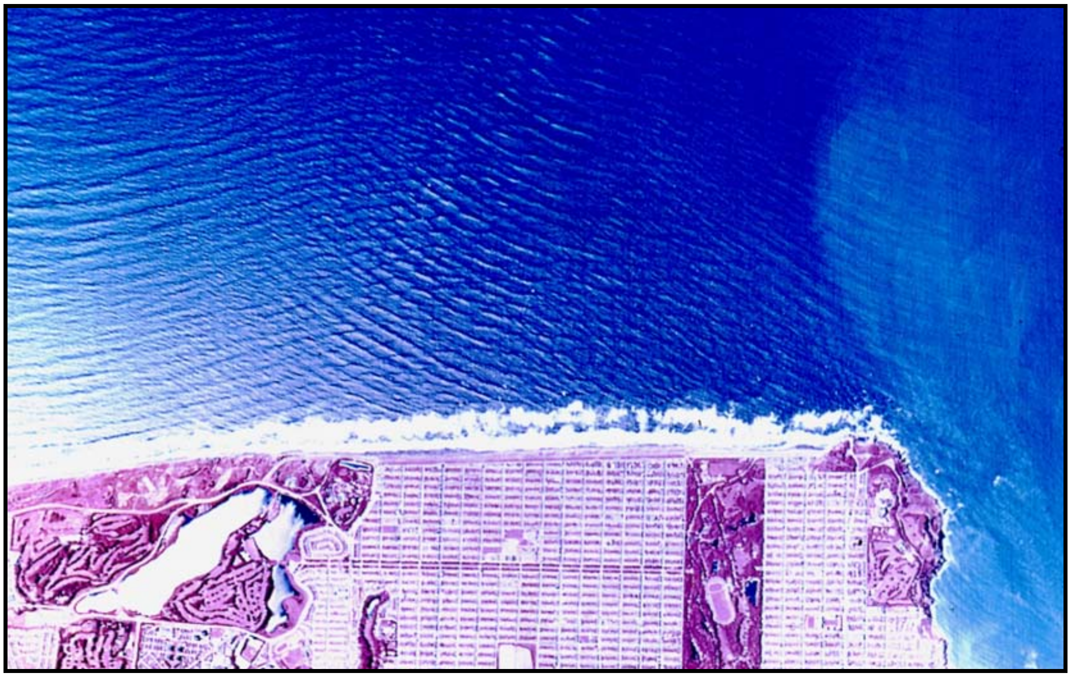
Wave transformations can also be important in sheltered water areas such as Puget Sound, as determined in a recent flood study at Sandy Point, Whatcom County (Figure 12). Sandy Point is exposed to wind waves generated within the greater Puget Sound, with particular exposure to a long open fetch in the Straight of Georgia (Figures 13 and 14). 100-year wind wave heights over 16 feet were calculated, with peak periods up to 11 seconds (Figure 15). The bathymetry offshore of Sandy Point includes a shallow area called Alden Bank (Figure 16), which was found to focus wave energy at Sandy Point (Figure 17). The wave focusing results in increased flood potential for a part of the Sandy Point community, as verified by observations during a moderate flooding event (Figure 18).

Wave Transformations discussed in this report include all changes to wave conditions during propagation from offshore waters to nearshore waters pertinent to coastal flood studies. Wave Transformation analyses are typically applied after offshore wave conditions are defined, with results used as input for nearshore runup analysis or overland wave propagation, both used for flood risk mapping.




Source: CDIP Internet Site.

**Figure 5. Wave transformation close-up at (1) San Francisco, (2) Bolinas and (3) Half Moon Bay, CA.**



Source: U.S. Army Corps of Engineers, South Pacifica Division.

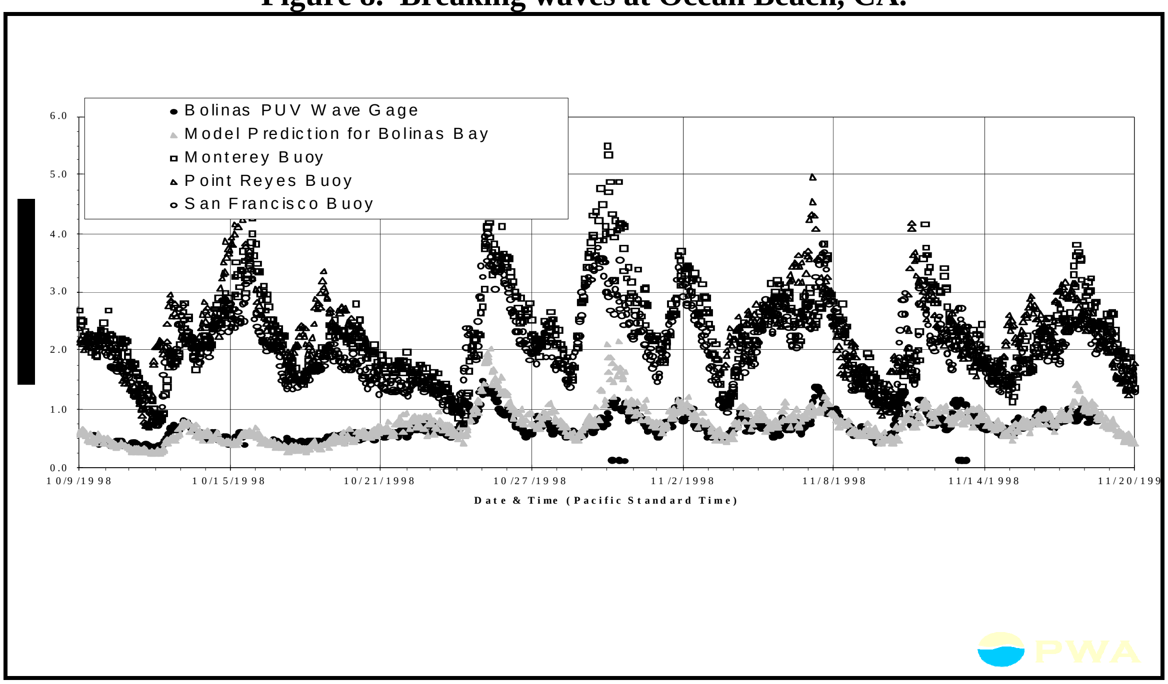
**Figure 6. Wave Refraction Resulting in Large Breaking Waves at Ocean Beach, CA (Site 1 in Figures 4 and 5).**

Photograph: Tim Britton

**Figure 7. Breaking waves at Ocean Beach, CA.**

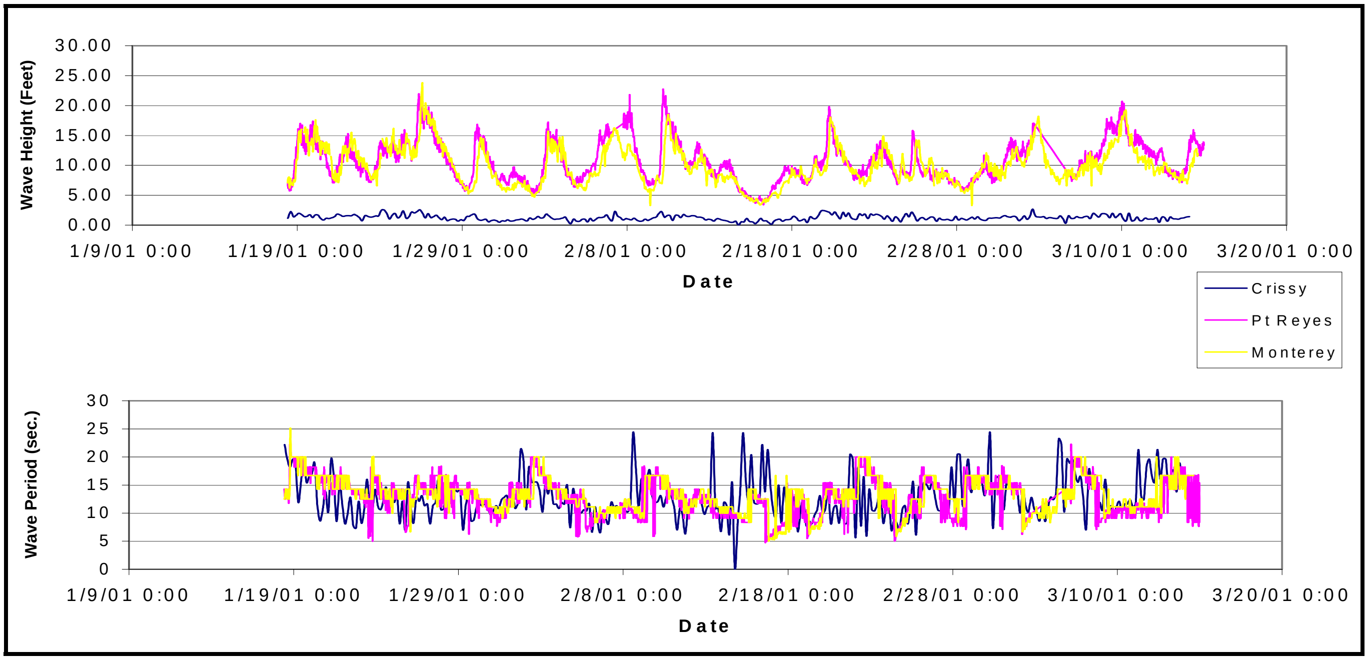
Photograph: Tim Britton. Note surfer paddling up wave, for scale.

**Figure 8. Breaking waves at Ocean Beach, CA.**




Offshore Data from CDIP (Point Reyes Buoy) and NABCO (Monterey Bay Buoy). Nearshore (Bolinas PUV Wave Gauge) Wave Data from PWA. Model Predictions from CDIP Refraction Analysis. Bolinas Bay is Site 2 in Figures 4 and 5 (PWA, 1999).

**Figure 9. Wave height comparison at Bolinas, California vs. offshore and nearshore wave measurements.**




Offshore wave data from CDIP (Point Reyes Buoy) and NDBC (Monterey Bay Bouy). Crissy Field data from PWA. Crissy Filed is Site 3 in Figures 4 and 5.

**Figure 10. Wave height and period comparison at Crissy Field vs. offshore buoy measurements.**

Photograph by Bob Battalio.

**Figure 11. Reduced swell wave heights at Crissy Field, east of Golden Gate Bridge, San Francisco, CA (Site 3).**

**Figure 12. Example in sheltered waters, Sandy Point, Whatcom County, WA.**

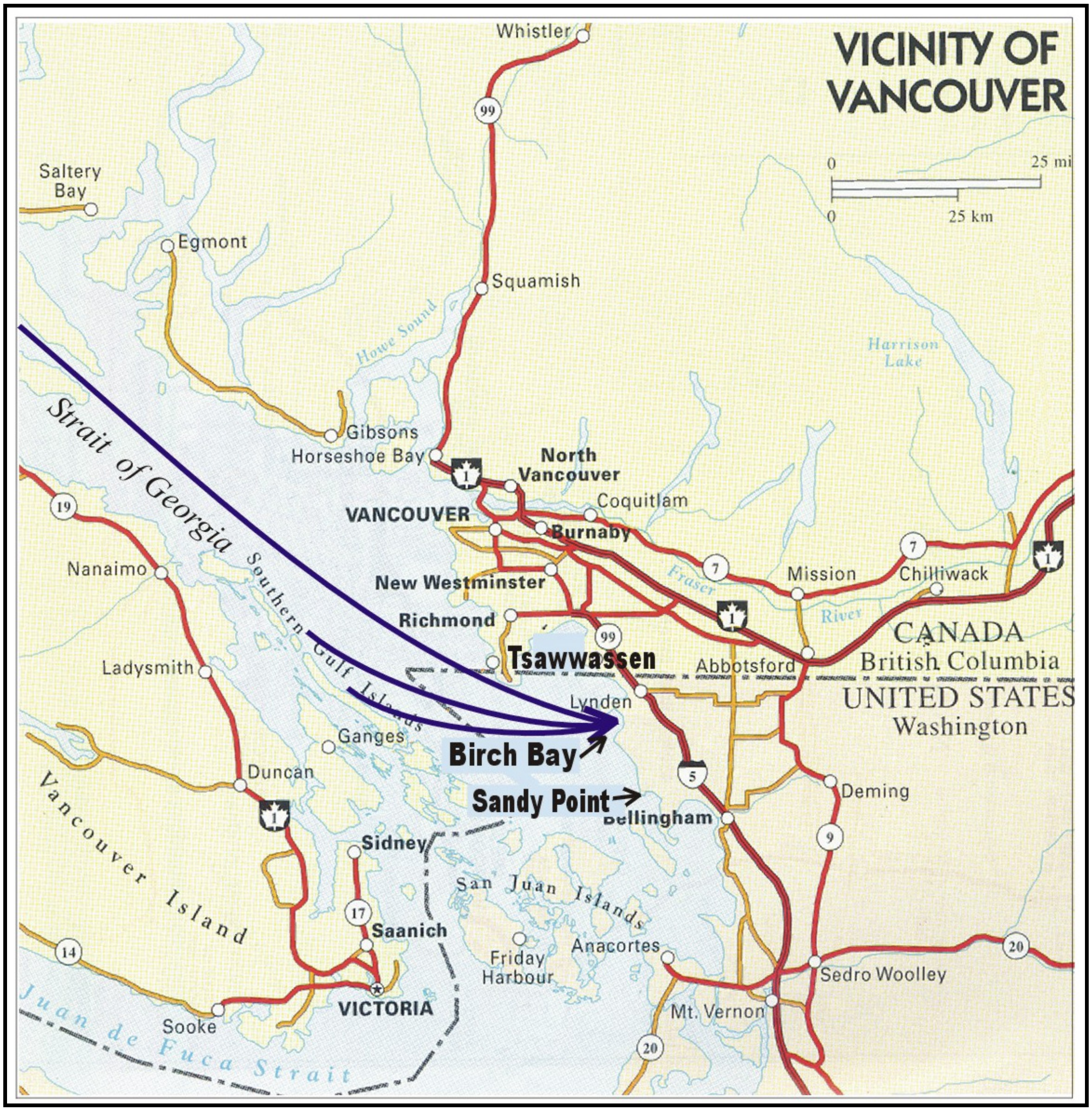
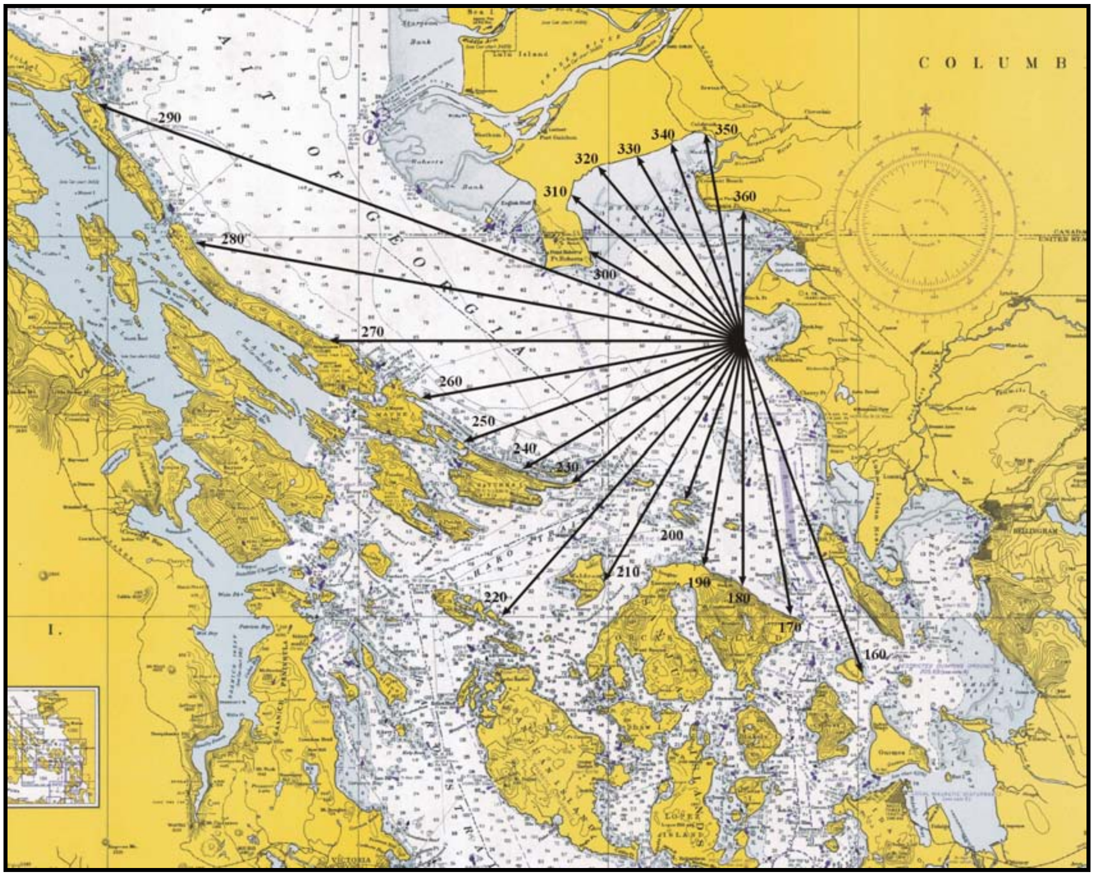



**Figure 13. Local wind sea forcing in Strait of Georgia, WA.**




Example Shown for Birch Bay is Similar to Fetches for Sandy Point, Whatcom County, Washington.

**Figure 14. Composite Fetch Hindcast Method application.**

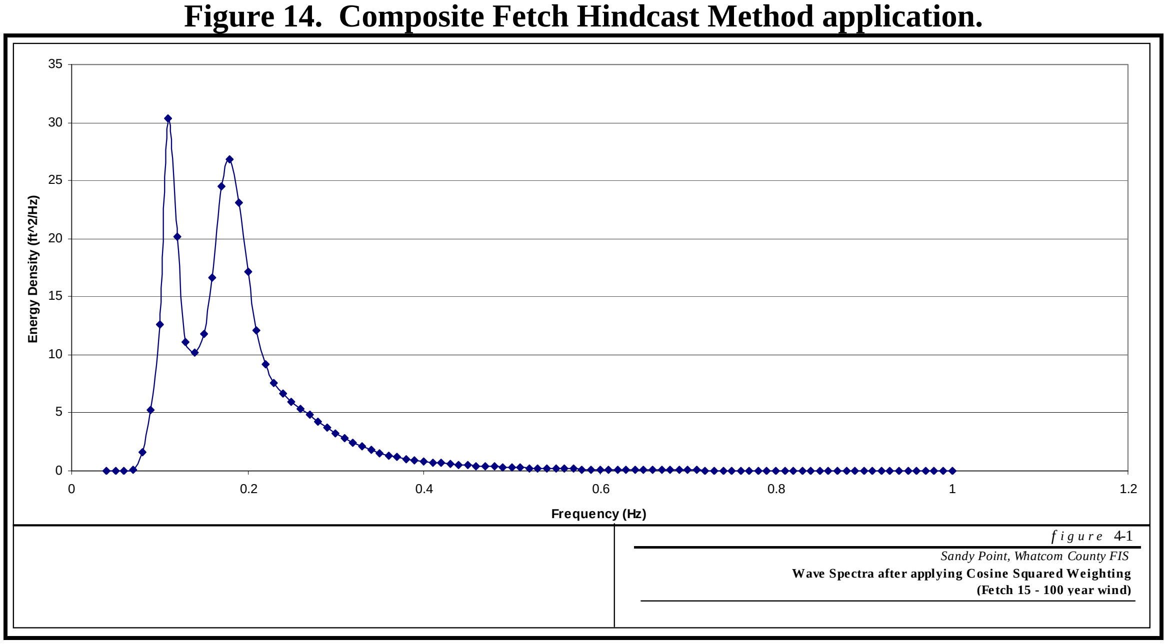
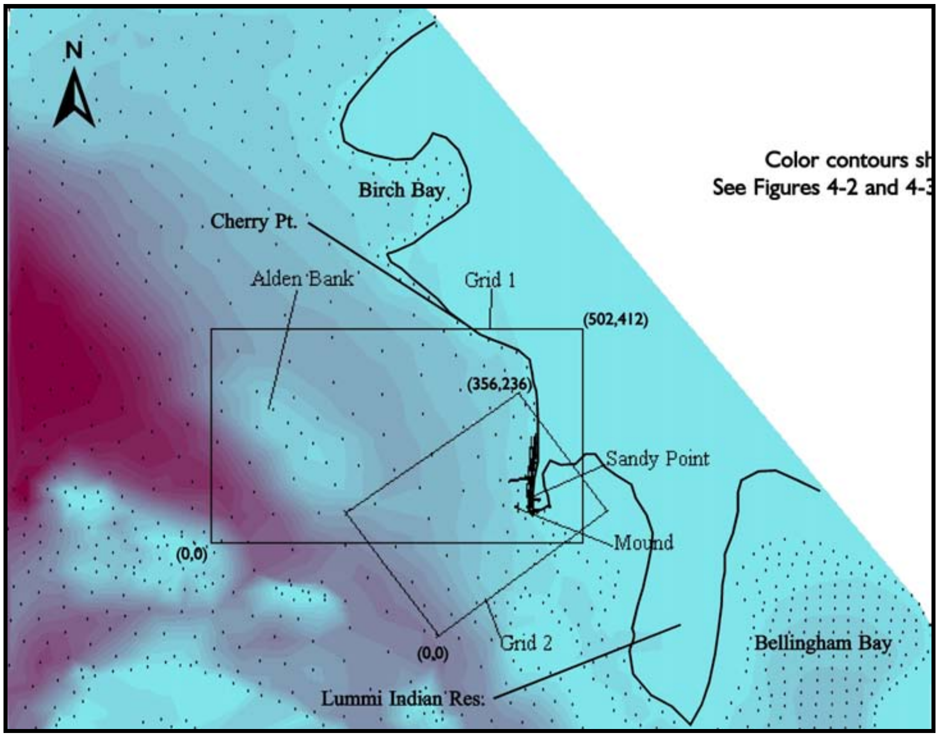
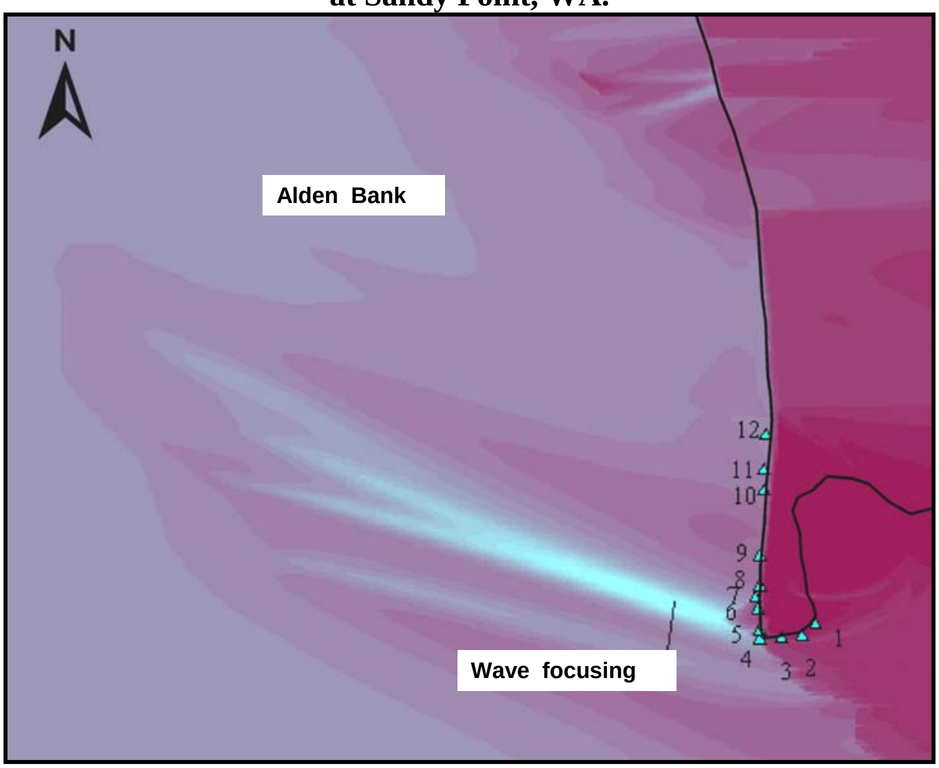



**Figure 15. Calculated wind wave energy spectrum for 1% event.**




Note Shallow Reef 'Alden Bank'.

**Figure 16. Bathymetry grids for wave transformation modeling at Sandy Point, WA.**




Focusing Caused by Shallow Area (See Figure 16, Alden bank).

**Figure 17. Wave focusing due to wave transformation affects flood risk.**

Flood Risk Shown to be Affected by Wave Refraction.

**Figure 18. Coastal flooding event 12/15/2000 at Sandy Point, WA.**

# 2 CRITICAL TOPICS

## 2.1 TOPIC 8: WAVE TRANSFORMATIONS WITH AND WITHOUT REGIONAL MODELS

### 2.1.1 Description of the Topic and Suggested Improvement

Wave Transformations refer to changes in wave characteristics during propagation. The primary processes are refraction, diffraction, shoaling, dissipation, and nonlinear effects. For practical reasons, Wave Transformations are often considered in the regime bracketed by wave generation (typically in “deep water”) and depth-induced breaking (typically “near shore”). See the Storm Wave Characteristics Focused Study for guidance on developing offshore wave conditions for input to wave transformations. See the following topics for guidance on related nearshore processes that use the output from Wave Transformation: Wave Setup; Wave Runup and Overtopping, and Overland Propagation. FEMA *G&S* address coastal flooding by wave action via wave runup (RUNUP 2.0 software) and or overland propagation (WHAFIS software), both of which require wave conditions at the beginning of the surf zone. However, wave transformations through the surf zone are important to wave setup and wave dissipation processes and hence shallow water wave breaking processes are included in Wave Transformations and in Wave Setup. Wave reflection and current-induced refraction are typically ignored, and guidance can be found elsewhere (USACE SPM, 1984; USACE CEM, 2003).

Refraction, diffraction, shoaling, and dissipation are strongly dependent on the wave length, with longer waves (higher wave periods) being affected the most (wave height is important, and dissipation due to propagation through vegetation can be greater for shorter, steeper waves). Irregular and steep bathymetry also increase wave transformations. Wave transformations are important for Pacific Coast flood studies owing to the longer waves, and generally steeper and less regular bathymetry. Wave transformations on the Pacific Coast are graphically depicted by near-real-time wave models applied under CDIP see for example http://cdip.ucsd.edu/models/socal_now.shtml. In Southern California, near shore wave heights can vary by a factor of 5 over a few miles of shoreline. Wave energy can be significantly dissipated (wave heights attenuated) during propagation over extensive shallow areas and intertidal marsh due to friction effects, viscous damping, and flow obstruction. These processes are particularly important in the Gulf Coast where sand and mud flats and marsh may extend for miles. Similar conditions can be found in some estuaries (Sheltered Waters) such as San Francisco Bay (West Coast) and Chesapeake Bay (East Coast).

Presently, the *G&S* do not include a description of wave transformations, and no *G&S* are written for the Pacific Coast (FEMA, 2003). Yet, prior Pacific Coast studies have addressed wave transformations in some detail (Tetra Tech, 1982; PWA, 2002a, b). Hence it is recommended that the Pacific Coast *G&S* be written to include Wave Transformations. Other regions could use the information in the Pacific Coast *G&S* as appropriate.

### 2.1.2 Description of Procedures in the Existing Guidelines

There are no *G&S* procedures for the Pacific Coast. In this case, guidance can be derived from the *G&S* for other geographical areas. Wave Transformations are addressed in Appendix D of the FEMA *G&S* in terms of overland travel (Sections D.2.6 - 2.6.4) and application of the WHAFIS model. This treatment is one-dimensional (defined by a profile), and limited to shallow water breaking and dissipation processes. Dissipation due to propagation over shallow areas and marsh plants is included. However, wave refraction, diffraction and shoaling are not addressed, except in passing references such as on page D-70: “Where land shelter or wave refraction may result in reduced incident waves, it is appropriate to specify an initial significant wave height for the transect.” The emphasis of the *G&S* on depth-limited, shallow water propagation and dissipation is logical given the bias toward the Atlantic and Gulf Coasts.

### 2.1.3 Application of Existing Guidelines to Topic–History and/or Implications for NFIP

The existing *G&S* are not adequate for Pacific Coast Flood Insurance Studies, and depending on site characteristics, are often not adequate for other regions, including sheltered waters. However, some wave transformation methods have been used in the following case studies.

#### *Case Studies on the West Coast*

Following are selected flood insurance studies on the Pacific Coast that address wave transformation at different levels of complexity.

**Sandy Point, Whatcom County, Washington (PWA, 2002a)**

Sandy Point is located close to the southern end of the Strait of Georgia, in the Pacific Northwest (Figure 12). The morphology consists of a 2-mile-long southward prograded sand and gravel spit. Swell wave existence at Sandy Point was ruled out because of its sheltered location. Governing wave conditions are locally generated seas and the highest waves are caused by winds blowing along the Strait of Georgia (Figures 13 and 14). The longest fetch to the northwest dominates the deepwater wave characteristics, and effects of varying fetch lengths in different directions were visible in wave spectra (Figure 15).

Deepwater waves were transformed to near breaking waves using RCPWAVE, a two-dimensional numerical model for wave refraction, diffraction, and shoaling. The main bathymetric features include a large offshore shoal, the Alden Bank (Figures 16 and 19). The grids generated for wave transformation are shown in Figure 16. The wave transformation results revealed wave energy focusing by the shoal, which accounted for the extreme flood hazards close to the tip of Sandy Point (Figure 17). Although wave focusing is real, the degree of variation of wave heights from focusing to de-focusing areas was overestimated. This is attributed to the monochromatic (non-spectral, single period) calculation method used by RCPWAVE and extreme refraction. Therefore, a parameterized directional spectrum weighting function (Goda, 1985) was used to average the distribution of wave energy in shallow water, for waves of all applicable directions. The highest averaged breaking wave heights were selected for wave runup calculations. Wave setup due to the highest average breaking wave was calculated. The stillwater level (SWL) was increased appropriately inside the surf zone when calculating smaller waves breaking close to the shore. Simplified methods from the SPM (1984) were used for wave breaking and setup calculations. The results were quantitatively verified by comparison with flood limits resulting from a large event that occurred during the study period.

**Birch Bay, Whatcom County, Washington (PWA, 2002b, ongoing)**

Birch Bay is located within the unincorporated limits of Whatcom County, Washington. Principal coastal flood problems occur at Birch Bay when strong northwest or southwest winds occur during periods of low barometric pressure, resulting in high storm surge conditions. The morphology at Birch Bay is different than Sandy Point because of the bay bathymetry and the extended mudflats.

The wave analysis for Birch Bay consisted of three steps:

- a windwave-hindcast for three large wind and water level cases,
- transformation of the deepwater waves to breaking, and,
- selection of wave conditions to be used for each shoreline reach.

The focus of the wave analysis was to select an appropriate range of wave conditions for each section of shore as input to the runup and overtopping analyses, including the effect of wave setup by the largest waves. The important wave characteristics were the wave periods (spectral average and range) and the wave heights, (the largest average breaking waves). The RCPWAVE computer program was used to transform deepwater waves to shallow water, and directional smoothing procedures of Goda (1985) were applied to the near breaking wave heights. The selected highest average waves for each reach was used for wave setup calculations and for wave runup and overtopping calculations. Wave dissipation over the shallow mud and sand flats was ignored. The approach used for wave transformations was similar to those used for Sandy Point.

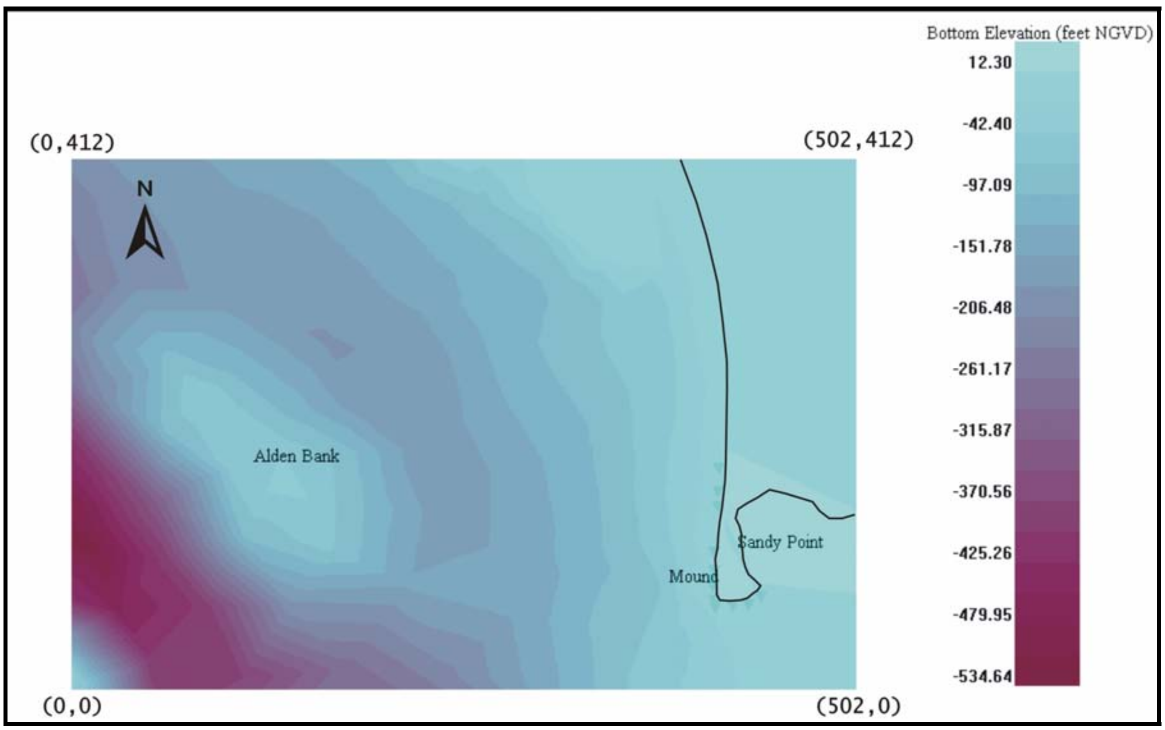



**Figure 19. Bathymetry - Sandy Point, WA.**

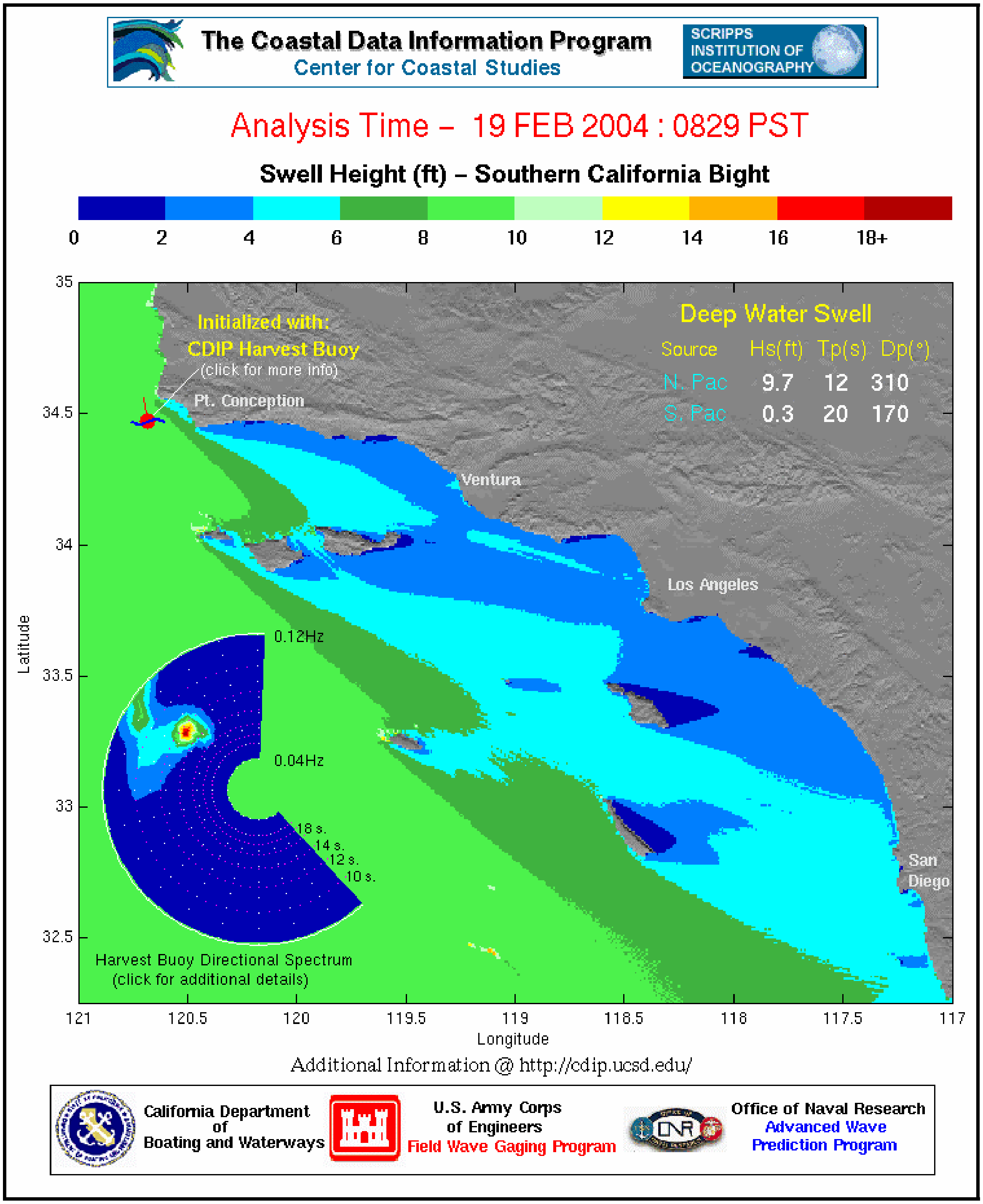



**Figure 20. Example of CDIP wave predictions for Southern California.**

**Bandon, Oregon (CH2M Hill, 1995)**

The City of Bandon is located at the mouth of Coquille River in the southwestern Oregon, in Coos County on the Pacific Ocean. The Flood Study was performed based on corrections to SWLs at a long-term tide station and return period wave runups based on the U.S. Army Corps of Engineers (USACE) Phase III Wave Information Studied (WIS).

Phase III WIS data (at 33 ft depth) were used in the wave analysis work. CH2M Hill compared the WIS monthly mean wave heights to the waves recorded by a Corps pressure gauge and found that WIS waves were slightly higher than the gauge records. The report mentions that wave refraction and shoaling were investigated to the extent necessary to verify that results from simplified methods were reasonable. Namely, to confirm that the selected large high waves could approach the study site, and confirm the limiting assumptions used in WIS data to transform Phase II waves (deepwater) to Phase III waves (at 33 ft depth), of uniform bottom slope and parallel, straight depth contours. Applicability of WIS Phase III results were verified by calculating nearshore wave heights using SPM (1984), Plate C-6 and ACES software. Plate C-6 shows change in wave direction and height due to refraction on slopes with straight, parallel depth contours including wave shoaling. This is an example of the simplified method usage for wave transformation.

**Northern California Coastal Flood Studies (Ott Water Engineers, 1984)**

Several sites along the northern California Coastline were included in the study. Offshore wave data were obtained from the U.S. Navy Weather Prediction Model. Storm waves (local) were calculated from the Sverdrup, Munk, and Bretschneider (SMB) method and tsunami levels were obtained from the U.S. Army Engineer Waterways Experiment Station in Vicksburg, Mississippi. The Wave Track model was used to obtain nearshore wave conditions due to shoaling and refraction. This model outputs wave height at breaking, and direction in the shallow water.

**San Francisco Bay and Puget Sound (Baker, 1989)**

Baker (1989) proposed an effective-fetch method for wave analysis and the SPM (1984) methods for wave refraction, shoaling and determining breaking wave locations. The breaking wave heights are input into the WHAFIS program to determine changing wave heights, as the waves progress further landward along representative shore profiles.

**Puget Sound (Coulton, 1988)**

A finite difference program called WAVES2 was used to compute refraction and shoaling of incoming deepwater, fetch limited waves as they approach the study sites. Input data include nearshore bathymetry, deepwater wave height and period, and the direction and starting location of the wave train. Wave ray location and directions are established on a two-dimensional depth grid and the output includes, refraction and shoaling coefficients, shallow water wave height and length, and the water depth to wave height ratio. Graphical output enables the refinement of the starting location of the wave rays enabling intercept of the study site transects.

**Pacific Northwest (Dorratoague, et al.1977)**
Deepwater wave conditions at the continental shelf were obtained by the SMB method, using deepwater wave charts, with wind speed, fetch, and duration as input. These waves were tracked to the shore using a wave shoaling and refraction computer program that was not specified.

**Southern California Coastal Flood Studies (Tetra Tech, 1982)**
Tetra Tech completed coastal flood studies for southern California in the 1980s. Their approach to wave transformations was documented in a report titled *Methodology for Computing Coastal Flood Statistics in Southern California*. A linear wave refraction routine was applied to transform waves from deep to shallow water for winter swell and hurricane wave sources. Approximately 183 model runs were required. Wave setup was calculated using the change in wave radiation stress using a spectral wave model. The spectral wave model uses simplified assumptions, such as linear super-position of spectral components (with consideration of maximum high-frequency energy), parameterized directional spectral shapes and wave breaking. However, the results indicate benefits relative to non-spectral approaches. Wave runup was calculated using a similar approach, with Hunt's method (similar to RUNUP 2.0) as the runup calculator. The methodology employed in this study is the most detailed and robust for open Pacific Coast conditions.

### 2.1.4 Alternatives for Improvement

#### *Introduction*

Guidelines and Specifications for Wave Transformations need to be written, as part of the *G&S* for the Pacific Coast. *G&S* for other regions could be left as is, with the presumption that guidance on wave transformations could be derived for the *G&S* for the Pacific Coast.

Key areas that need to be included and expanded in the *G&S* are identified below in "Guidelines for Wave Transformation."

#### *Guidelines for Wave Transformation*

There are many methods that can be employed to successfully simulate nearshore wave characteristics in an FIS. The Study Contractor faces the important task of selecting the appropriate methods for the study. The *G&S* need to address the selection of methods based on the physical parameters that are encountered in the wave transformation process. Guidance can be provided based on the following criteria:

- *Region and Site Geomorphology:* A starting point is to select methods based on the site conditions at the regional level (e.g., exposure, island sheltering, etc.) and at the site level (mild sandy slopes, or steep cobbles, etc.) ;
- *Contour regularity / irregularity:* The irregularity of farshore and nearshore bathymetry has a major affect on the degree of wave refraction and diffraction that will occur, and hence the level of analysis necessary to achieve reasonable accuracy;

- *Seabed steepness:* Bottom slope affects shoaling rates, refraction and diffraction, and dissipation;
- *Wave parameters:* Wave period (length), steepness, height and possibly spectral shape affect wave transformations;
- Information needed for subsequent analyses, such as setup and runup, may affect the methods used.
- Evaluation of analysis results: Identify results that would indicate a more detailed methodology is appropriate, such as wave ray crossings. Identify methods for validating the results from model applications.

**Description of Wave Transformations**

The *G&S* will include a description of wave transformations and pertinent factors as background for subsequent analysis. Appendix 1, Section A-1.1 provides a "feel" for the content and level of detail proposed

**Relationship with Other Analyses Steps**

In a flood study, the final task is to determine the flood elevations and landward extent by evaluating storm surge elevations, wave runup, and overtopping during a 100-year return-period flood event. Wave transformation accounts for the changes in wave characteristics between offshore and nearshore. The nearshore waves are important as input into the runup and overtopping calculations and also to estimate the increase in stillwater elevation due to wave setup. *G&S* need to be written to identify methods that will provide adequate information for subsequent analyses. The text in Appendix1, Section A-1.1.1, is an example of the proposed coverage and content.

The following topics would be addressed in the *G&S* to identify linkages. Cross references would be provided.

- Storm Wave Characteristics
- Wave Setup
- Wave Runup and Overtopping
- Overland Propagation (WHAFIS)
- Tsunami

**Processes**

The *G&S* should provide a description of the following processes addressed within Wave Transformations. The section A-1.1.2 in Appendix 1 provides an example of what may be written, with additional polishing, graphics, and references.

- Wave Refraction and Diffraction

- Wave Shoaling and Breaking
- Wave Energy Dissipation, Non-Breaking
- Wave Propagation Over Inundated Land Areas
- Wave Generation
- Wave Reflection
- Nonlinear Effects

**Regional and Geomorphic Considerations**

The *G&S* should include information to help determine the type and level of analyses to use. The *G&S* should categorize the coastal areas in terms of regional and local site conditions, and link these characterizations to appropriate methods.

**Regional Models**

Regional wave transformation models have been developed for most of California under the CDIP, jointly funded by USACE and the State of California Department of Boating and Waterways. These models address wave refraction using a spectral back-refraction model and have been calibrated and verified using wave data collected with directional wave gauges. The models and the resulting data represent a significant potential resource for future coastal flood insurance studies. For a given site, wave height transformation coefficients can be used to transfer selected deepwater wave conditions to the nearshore. Alternatively, where available, nearshore hind cast time series can be analyzed directly. Also, where available, radiation stresses can be obtained for wave setup calculations.

As discussed in Section 2.2, regional wave modeling using the CDIP approach is recommended for the California Coast, including the proposed development of a nearshore wave climate based on transformed wave hindcast data (see also the Storm Wave Characteristics Focus Study). While expansion of the CDIP is recommended to satisfy FEMA's needs for coastal flood studies in California, interim procedures are needed both for use of CDIP data and other regional models that may become available, to address site-specific wave transformation studies, and for other locations.

Wave transformation coefficients have been developed by the CDIP for much of the California Coast. The data are generally more developed for Southern California and progressively less developed for Central and Northern California and other West Coast regions. In Southern California, very detailed and well-verified data exist. Guidance is required for the use of these data, including how to address wave growth due to winds within the domain of wave transformation modeling. For Central and Northern California, substantial data are also available but have not been verified to the same extent and require additional guidance for use.

For the Pacific Northwest (Oregon and Washington), some data are available now and more may be developed as part of the CDIP over time, and hence appropriate guidance will be needed. In these areas, the 40-year commercially available GROW data set could serve as the deep water input and simplified wave transformation models may be appropriate.

The *G&S* will probably recommend the use of regional modeling where appropriate and use of output from regional modeling where available. It is recommended that the CDIP regional modeling products be used for California, to the extent appropriate, with a reference to a "user manual" or other document by FEMA and or CDIP. It is important to note that regional modeling is not an absolute need. That is, a coastal flood study could include wave transformation analyses only as required for a given community. This may be the case along a sparsely populated coast where only limited detailed coverage is needed in the foreseeable future, or where results are needed before regional modeling can be accomplished.

### *Input and Output Parameters*

The *G&S* should provide details on required input and possible output for different analysis methods. The text given in Appendix 1, section A-1.2 would be augmented as the other portions of the Pacific *G&S* are developed. References and graphics would be added to clarify concepts. Input and output parameters appropriate for a given coastal flood study can be selected by considering the following topics.

#### Geographic / Geomorphic

Input data requirements should be identified along with guidance on spatial domain and boundaries, based on regional and geomorphic characteristics. Graphics and quantified criteria will be developed based on available guidance. The text given under section A-1.2.2 in Appendix A is the beginning.

#### Wave Characteristics

Descriptions of wave characterization appropriate for the different types and levels of wave transformation analyses will be provided in the Wave Transformations *G&S* to be written. The text provided in Appendix 1, section A-1.2.3, outlines the range of characterizations to be described. Text will be augmented based on available literature with references and graphics. As described in the Storm Wave Characteristics Focused Study report, a deepwater wave climate should be available for input to the wave transformations. The objective is to allow a nearshore shallow water wave climate to be developed, including directional spectra. Common representations of waves and concerns are:

- Mono chromatic - basic characteristics such as significant height, peak period, and central direction.
- Frequency Spectra – wave height is a function of wave frequency. Guidance on shallow water spectra is needed.
- Directional Spectra – both wave height and direction are a function of wave frequency. Guidance on deepwater and especially shallow water spectra is needed.

- Groups and Infra-gravity Waves – this subject requires further research.
- Breaker Parameters – guidance other than a constant breaker height to breaker depth ratio of 0.78 is necessary Adequate information exists in the literature to write this guidance, and should be used to develop recommended methods.

**Radiation Stress**

The *G&S* will build upon published methods for regular and irregular wave setup calculation. One methodology that could be employed is described in Tetra Tech (1982). Coordination is required with Wave Setup Focused Study.

### *Wave Refraction and Diffraction Methods*

**Method Selection**

A range of techniques is available for transformation of waves from deep to shallow water. The type of bathymetry is a key parameter in selecting the appropriate method. Simple techniques can be applied in the case of simple bathymetry (straight and parallel bottom contours) to account for wave shoaling and refraction. For random, directional waves it is necessary to transform all component waves in the spectrum and use superposition to obtain wave conditions in finite water depths. Model selection is subject to the key parameters of input/output terms, bathymetric features, and wave characteristics. Guidance on methods selection will be provided in the *G&S*.

**Simplified Methods**

The simple techniques can be applied in the case of simple bathymetry (straight and parallel bottom contours).

**Refraction by Snell's Law**

The path traced by the wave orthogonal as a wave crest propagates onshore is called a wave ray. Simple wave propagation problems can readily be visualized by construction of wave rays manually or by graphical techniques. In the case of straight and parallel contours, and for monochromatic waves the Snell's law ($\sin \theta / C$ = constant) can be applied to draw the path of the wave ray.

In addition, the wave height variation can be estimated by considering two closely spaced wave rays. Assuming no transfer of energy takes place across the wave ray boundary, wave height at any location along the wave ray is given in terms of the offshore wave height, shoaling, and refraction coefficients. These coefficients can be calculated in terms of the water depth and the orthogonal distance between wave rays at the interested location. The CEM provides solution nomograms (Figure II-3-6) which are also automated in the ACES program.

**Linear Refraction**

If the bathymetry has variations along the shore, then the simple Snell's law approach cannot be used, rather a 2-D approach must be used. One common method is wave ray tracing. The ray approach for wave refraction has had problems caused by wave ray crossing, at which point wave height becomes infinite. These problems are caused by the fact that each ray is traced independently of the other rays and there is no refraction or breaking. Some numerical methods overcome this problem by artificial smoothing techniques. Results need to be checked for signs of wave ray crossings (caustics) and in that event a simple refraction-diffraction model is more appropriate.

**Graphical Diffraction**

Graphical Diffraction methods are available in SPM (1984); Goda (1985); and CEM (2003). Methods include monochromatic and simplified spectral approaches. These methods can be applied relatively easily and are reliable for most cases. A description of application of Goda's methods using the s factor (directional spread) will be included.

**Refraction / Diffraction Models**

The following text provides a summary of contemporary wave refraction / diffraction analysis methods. Some are approved for use by FEMA and some are not. As part of the *G&S*, it is recommended that those not approved be applied to a test case to identify the differences in results, and that further literature review be accomplished to gauge the accuracy of the models. Based on the results, recommendations for approval and guidance on application will be developed and included in the *G&S*.

When waves propagate into water depth that is less than about one-half of the wave length, the direction of wave propagation gradually changes. These changes can cause energy concentrations or spreading depending on the bathymetry. Sometimes when diffraction is not considered in the wave transformation method, wave heights can increase to unrealistic elevations. In reality, wave heights are limited by breaking either because of depth or steepness constraints. Diffraction effects (the spread of energy along the wave crest) can also, reduce locally high wave heights and reduce the tendency for local wave breaking. For more complex bathymetry with shoals, islands or other major geological features, both refraction and diffraction need to be modeled.

A series of programs are available that deal with diffraction, in addition to modeling wave refraction and shoaling. A brief discussion of these models is available in CEM, 2003. The CEM lists the computer programs RCPWAVE (Ebersole, 1985; Ebersole, Cialone, and Prater, 1986), REFDIF1 (Kirby and Dalrymple, 1991) for monochromatic wave refraction, as available and in use by USACE but cautions the users to apply these models within the limits of their use.

FEMA pre-approved RCPWAVE is a steady-state linear wave model based on the mild-slope equation and includes wave breaking. The program is limited to open coast areas without structures or islands etc. A comparison of wave refraction and diffraction models was performed by Maa et al., (2000). Wave transformations were estimated across the elliptic shoal and compared with experiments carried out by Berkhoff et al., (1982).RCPWAVE performed poorly in simulating the wave height distribution and wave direction. Therefore this model may be inadequate in modeling areas with sand shoals and other complex bathymetry.

The CDIP has applied a linear, spectral back-refraction model along the California Coast. Detailed application of this model has included verification using directional wave data collected at deep and shallow water wave gauges. Very good results have been obtained.

REFDIF1 is a steady-state model based on the parabolic approximation solution to the mild-slope equation. Although this model is not pre-approved by FEMA, it is known to provide more accurate wave heights than from the RCPWAVE model in certain bathymetric situations (Maa et al., 2000). However, if the study domain has complicated geography and/or bathymetry, or if there is a strong wave diffraction and /or reflection, elliptic mild slope models are appropriate.

MIKE 21 EMS is based on the numerical solution of the Elliptic Mild-Slope equation formulated by Berkhoff (1972) and is capable of reproducing the combined effects of shoaling, refraction, diffraction, and back-scattering. Energy dissipation from wave breaking and bed friction, is included along with partial reflection and transmission through pier structures and breakwaters. MIKE 21 EMS can be used to study wave dynamics in smaller coastal areas and in harbors. The Module is particularly useful for the detection of harbor resonance and seiching due to, for instance, long-period swell.

The extended mild-slope models may be more appropriate for steep and rapidly varying bathymetry. These models are computationally expensive and therefore only applicable to smaller areas.

## Spectral Refraction Models

### STWAVE

Developed by the USACE Waterways Experiment Station (WES), STWAVE is a steady state, spectral wave transformation model, based on the wave action balance equation. A wave action approach can handle a current correctly, where as an energy spectrum approach cannot. STWAVE is able to simulate wave refraction and shoaling induced by changes in bathymetry and by wave interactions with currents. The model includes wave breaking based on water depth and wave steepness. Other features of STWAVE include wind induced wave growth, and influences of wave white capping on the distribution and dissipation of energy in the wave spectrum.

STWAVE is most applicable to wave transformation problems where the following assumptions can be made:

- Mild bottom slope and negligible wave reflection.

- Spatially homogeneous offshore wave conditions with steady state wave, current, and wind conditions.

- Linear wave refraction and shoaling with negligible effects from bottom friction

Wave energy dissipation due to bottom friction and viscous damping effects may occur as waves propagate over shallow areas. Dissipation is not included in the standard version of STWAVE. However, a version that addresses dissipation is being used by the USACE (personal communication, Resio, at Workshop 2) and other versions have been developed and used by others with good results.

**SWAN**

The numerical wave transformation model SWAN was developed at the Delft University of Technology, Delft, Netherlands. SWAN and STWAVE have many similarities. Like STWAVE, the formulation of SWAN is based on the spectral wave action balance equation. This model currently has many well-developed features, which provide the user with many execution options These features range from purely convenient options that allow several different formats for input and output data, to options that allow control of fundamental physical processes in the model, for example wave generation, dissipation, and interaction. Linear wave refraction and shoaling are included in the model. Some differences from STWAVE are:

- Input wave conditions can be varied spatially along open boundary, and wind, water level elevation, and current inputs can be varied spatially over the entire computational domain.

- Simulations may be steady state or dynamic. SWAN has the ability to compute a time varying solution, rather that just a series of steady state solutions.

- Users of SWAN must consider the following model assumptions in a specific application:

- SWAN does not model wave diffraction or reflection, and therefore is most useful in applications where accuracy of the computed wave field is not required in the immediate vicinity of obstacles.

- Mild bottom slope with negligible wave reflection

**REF/DIF S**

REF/DIF S was developed at the Center for Applied Coastal Research, at the University of Delaware. This spectral wave transformation model is a further development of the REF/DIF 1 model, which solves for monochromatic waves only. REF/DIF uses the parabolic form of the mild-slope equation, and the complex amplitude of each separate wave component. Because the mild-slope form of the governing equation is used, the model includes the effects of wave diffraction, unlike STWAVE and SWAN.

**Alternatives to Spectral Models**

If the wave-wave interactions can be ignored, a simplified method of "energy transfer functions" can be used to construct the nearshore wave energy spectrum at a specified location for any given off-shore spectrum. The procedure involves calculating the response matrix using a linear refraction-diffraction model with a unit incident wave height (or amplitude) for the range of wave frequencies and directions. The transfer functions need to be calculated only once since the refraction-diffraction model is linear. The response to any desired incident directional spectrum is then constructed by appropriately weighting each discrete component. This method has been used by O'Reilly and Guza (1991, 1993) for wave predictions in an analytical circular shoal configuration and at Southern California locations. They used the spectral refraction model of LeMehaute and Wang, (1982) and a spectral refraction-diffraction model (linear version of the higher order PEM derived by Kirby 1986a, Kirby 1986b, and Kirby and Dalrymple, 1986). The CDIP has applied a linear, spectral back-refraction model along the California Coast. Detailed application of this model has included verification using directional wave data collected at deep and shallow water wave gauges. Very good results have been obtained. See the write-up for Topic 7: California Regional Wave Transformation Models for a more complete description.

**Use of Directional Spreading Functions**

Wave directional spectra are not available as output from many of the above discussed models. In such a situation, if wave directional spectra are required as input to another model or for smoothing out the artificial wave energy focusing effects, an approximate method would be to use directional spreading functions. This is a semi-parametric approach for generating directional wave spectra. Goda (1985) discusses a couple of functions including the Mitsuyasu type (Mitsuyasu et al., 1975).

### *Wave Shoaling and Breaking*

**Method Selection**

Adequate information exists in the literature to complete the following *G&S* for Wave Shoaling and Breaking. Method selection is primarily based on wave characteristics and morphology.

**Linear Shoaling**

Waves slow down upon entering shoaling water and consequently wave height increases and sometimes decreases depending on group/phase velocity relations. The change in wave height due only to the change in wave group velocity is referred to as shoaling. Linear shoaling assumes the waves are of small amplitude and therefore the linear wave theory can be used to derive the shoaling coefficient (Ks = H/Ho) by equating the offshore wave power to the wave power at any nearshore location (before breaking). When other processes such as wave refraction, diffraction, and dissipation are involved in the transformation process, equivalent deepwater wave height is used instead of the deepwater wave height in the shoaling equation (Ks = H/Ho')

**Non-Linear Shoaling**

As waves approach very shallow water, several wave lengths seaward of breaking, shoaling becomes highly non-linear and the linear shoaling coefficient may significantly under predict the wave height, especially for long waves in shallow water. Non-linear shoaling coefficients are available in several publications, which relate shoaling coefficients to parameters of wave steepness, relative depth and beach slopes (Goda, 1985, SPM and others).

**Breaking Indices**

In shallow water, breaking is limited by water depth and the point of breaking is influenced by wave steepness and beach slope. Simple wave breaking indices for regular and irregular waves are discussed in the CEM (2003), Part II-4.

A breaking wave model (series of equations) that operates on a site-specific nearshore profile (one-dimensional) is needed to calculate wave setup, as described in the Focused Study report for Setup (see in particular Topics 44 through 48, Wave Setup). The breaking wave model should be adequate to calculate wave radiation stress through the surf zone for irregular wave conditions. The wave radiation stress is used to calculate wave setup. Guidance is also needed for the dynamic component of wave setup, using available information. The breaking wave model shall be applicable for the Gulf, Atlantic and Pacific Coasts, including sheltered waters, but is critical for the Pacific Coast.

**Spectral Transformations**

Vincent and Briggs (1989) showed by their lab experiments that wave transformation over a shoal is sensitive to the shape of the incident wave directional spectrum and differ significantly from a single unidirectional wave. Therefore, the approach of defining a single wave height to represent the offshore spectrum and using this wave height in the unidirectional wave transformation models does not prove to be satisfactory when shoals and complex bathymetries exist.

Transformation of incident wave frequency-directional spectra can be achieved by combining multiple model runs, each for a single frequency and direction (Izumiya and Horikawa, 1987; Panchang et al., 1990). These spectral models do not explicitly predict the directional spectrum, but have been used to estimate the directionally integrated energy to determine the wave height.

### *Wave Energy Dissipation, Non-Breaking*

**Method Selection**

Method selection will be based on bed and wave conditions and or region and other site conditions. To the extent practicable, coefficients will be provided for the described methods based on published data. Where data are not adequate to calculate wave dissipation, calibration will be recommended.

**Friction**

Friction related energy dissipation occurs mainly in shallow water (Tubman and Suhayda, 1976). The friction effect varies with the type of bottom material and also as a function of wave parameters, relative depth, propagation distance etc. Guidelines for selection of criteria are in Section 2.3

**Viscous Bottom**

Unlike friction related dissipation, which usually occurs in shallow water, soft (flexible) bottom dissipation can also cause significant wave attenuation in intermediate water depths.

Suhayda (1984) documents the use of a numerical model to develop wave crest elevation attenuation coefficients by simulating the effects of wave generation by wind, shoaling, and dissipation due to breaking, bottom friction, and soft muds during extreme hurricanes. The author models wave height/energy to change exponentially with distance along the wave travel direction. To compute its effects on wave dissipation, he models the soft muddy bottom as a visco-elastic medium, in accordance with the MacPherson (1980) model. The results summarize wave height to water depth ratios in the range of 0.42 to 0.78 for the 21 transects, that he used in this study. Guidelines for selection of criteria are addressed in Section 2.3.

**Marsh Vegetation**

*G&S* Appendix D (2002) considers marsh vegetation (pg. D-72 to D-80) under description of the WHAFIS 3.0 model. Eight parameters are used to describe the dissipation properties. This procedure was specifically developed for the Gulf and Atlantic Coasts. Applicability of these guidelines for the Pacific Coast wetland areas need to be explored. Also see the section "Method for Wave Attenuation in Pacific Marsh Conditions" under Topic 9.

#### *Wave Propagation over Inundated Land Areas*

This condition is commonly observed in the Atlantic and Gulf Coasts, and WHAFIS 3.0 approved by FEMA is applied in the present FIS. Although not common, overland wave propagation can be significant in marshes surrounding bays (e.g., San Francisco Bay). The changes to wind characterization may be necessary to use the WHAFIS model for the Pacific conditions.

Continuation of the two-dimensional wave transformation models into the inundated regions may be the next step of improvement. However, application of two-dimensional models may be constrained by data availability. The *G&S* will address use of WHAFIS for Pacific Coast FIS. Extensive *G&S* exist for application of WHAFIS to the Gulf and Atlantic Coast FIS, with additional guidance in Section 4.1.

### 2.1.5 Recommendations

Recommended improvements are:

1. Write *G&S* for Wave Transformations as a section within the *G&S* for the Pacific Coast;

2. Include several focused studies to demonstrate the Wave Transformations *G&S*;

3. Use available publications to identify a range of methods from simplified to more detailed so that study managers and contractors have a range of "tools" to select from, to provide defensible and cost beneficial studies;

4. Develop criteria for level of analysis required based on region, site geomorphology, wave characteristics, available input data and regional models, and required output data. These criteria will guide the procedures used for refraction, diffraction, shoaling, and dissipation. Include development of guidelines for spatial coverage and wave parameters, and address use of regional models such as CDIP;

5. Research available literature to adequately define wave groups, infragravity waves, shallow water spectra, and radiation stress formulations for input into wave setup and runup calculations;

6. Evaluate adequacy of linear wave transformation models and needs to supplement these models. Place emphasis on representation of infragravity waves;

7. Evaluate wave transformation models using available case studies or a selected data set, in order to compare results. Review available literature and guidance on the range of applicability of contemporary computer models. Recommend models for inclusion on the FEMA pre-approved coastal model list, and provide guidance on their application to the FEMA FIS.

8. Incorporate applicable sections of existing *G&S* for other geographical areas that cover the overland propagation and wave energy dissipation topics.

#### 2.1.6 Preliminary Time Estimates for Preliminary Guideline Preparation

Table 2 summarizes the preliminary estimates of time required for Critical Topic 8. These time estimates do not include responding to comments and suggestions associated with the review of the Guideline improvements.

#### 2.1.7 Related Available and Important Topics

Wave Characteristics Focus Study Topic 4: Swell and seas originating in the open ocean can penetrate coastal inlets, and may control coastal flood risk near the mouths of sheltered waters.

Wave Transformations Focus Study Topic 9: Bottom friction factor used for very shallow waters may affect wind wave generation.

Wave Transformations Focus Study Topic 10: WHAFIS is included in Wave Transformations.

Storm Surge and wind setup may affect depths to the extent that wind wave generation is affected.

### 2.2 TOPIC 7: CALIFORNIA REGIONAL TRANSFORMATION MODELS

CDIP regional modeling can now provide transformation coefficients for most locations in the Southern California bight and some locations in Central California.

### 2.2.1 Description of the Topic and Suggested Improvement

The CDIP at Scripps Institution of Oceanography maintains a database of linear, spectral refraction-diffraction transformation coefficients for shallow coastal areas from the U.S.-Mexico border to Point Arena north of San Francisco (O'Reilly and Guza, 1991). The database nominally extends into depths as shallow as 10m with alongshore spacing of approximately 200m. The wave model coefficients are for waves with periods longer than 8 seconds (frequencies less than or equal to 0.12 Hz) and are primarily used to produce the swell wave height maps on the CDIP website. Figure 20 is an example of the CDIP product for the southern California Coast.

The spectral refraction-diffraction model uses a parabolic approximation to the mild slope equation and is computationally well suited for making wave predictions across large regions like the Southern California Bight. However, waves refracting around islands and over submarine canyons can propagate at high angles to the x-axis of the model bathymetry grid. This violates the small angle approximation in the underlying parabolic equations, resulting in numerical noise that makes it difficult to extract directional wave information from the model output. In addition, anomalously large transformation coefficients can occur near extreme bathymetry owing to the high wave angle propagation errors. Because of the model limitations, only nearshore frequency spectra between 0.04–0.12Hz (no direction information) can be estimated from input deep water frequency-directional spectrum, and care must be taken to ensure that numerical errors have not corrupted any of the coefficients if they are going to be used for FEMA Coastal FIS. As a result, CDIP does not widely distribute specific data from this transformation coefficient database without careful QC by the CDIP staff.

Because of the numerical limitations of the spectral refraction-diffraction model for nearshore coastal engineering and scientific studies, CDIP is now implementing a simpler spectral refraction modeling method to derive regional, alongcoast wave predictions just seaward of the surfzone. This technique has recently been applied to the Los Angeles County coastline, as part of the region's USACE Storm and Tidal Waves Study, with good results. See also the discussion in the Introduction to this report and Figure 9 for an application in northern California at Bolinas Bay (PWA, 1999).

### 2.2.2 Description of Procedures in the Existing Guidelines

There are no *G&S* procedures for the Pacific Coast and regional model use is not covered in FEMA existing *G&S*.

### 2.2.3 Application of Existing Guidelines to Topic–History and/or Implications for NFIP

There are no *G&S* procedures for the Pacific Coast and regional model use is not covered in FEMA existing *G&S*.

### 2.2.4 Alternatives for Improvement

#### *Basic Methodology*

A linear, spectral refraction model will be used to transform deep water hindcasts of extreme storm wave spectra to nearshore wave spectra at locations just outside the surf zone along the entire U.S. West Coast. The resulting nearshore database will be validated against wave measurements on a regional basis, and made available to FEMA contractors as an approved source of incident wave information for coastal hazard modeling and mapping.

#### *The Spectral Refraction Model*

The transformation of deep ocean directional wave spectra to the nearshore will be performed using a spectral wave refraction model (Longuet-Higgins, 1957; LeMehaute and Wang, 1982; O'Reilly and Guza, 1991). The model accounts for island blocking, wave refraction, and wave shoaling. Spectral refraction back-refracts wave rays from the site of interest to unsheltered deep water over the entire range of possible wave frequencies and wave directions. The retained starting and ending ray angles are then used to map a deep water directional spectrum to a sheltered or shallow water spectrum at the back-refraction site. The resulting solutions are more realistic than those obtained using an assumption of unidirectional, monochromatic deep ocean waves.

The spectral model is linear; therefore the model calculation needs to only be performed once for a particular location and sea level elevation to obtain linear transformation coefficients between the offshore and nearshore wave spectra. As a result, the creation of a nearshore wave spectra database for the entire U.S. is feasible, and future improvements to all or parts of the database should be straightforward. The spectral refraction model has undergone extensive validation in Southern California (O'Reilly and Guza, 1993a, O'Reilly et al., 1993b) and is well suited for the U.S. West Coast, where the continental shelf is relatively steep and narrow and bottom dissipation effects are small. Recent field validation of the spectral refraction model in the vicinity of a submarine canyon (Ray, 2003) demonstrated that diffraction effects are small over even extreme natural bathymetries, and spectral refraction is an adequate methodology in these situations.

#### *Model Application and Validation on the U.S. West Coast*

From a wave modeling perspective, the U.S. West Coast can be divided into two distinct regions:

1. Southern California, from the U.S.-Mexico border to Point Conception.

2. The open coast from Point Conception north to the U.S.-Canada border.

The Southern California region is partially sheltered from deep ocean waves by islands, resulting in a local wind generated sea wave climate that must be considered separately from incident swell waves in some areas. In addition, the coastal wave climate at the east end of the Santa

Barbara Channel, approximately between the cities of Ventura and Santa Barbara, is significantly affected by the reflection of northwest swell off the coastal cliffs on the north side of Santa Cruz Island (O'Reilly et al., 1999). The spectral refraction model has been tested extensively in Southern California and a large database of wave measurements exist to assist in the development and validation of a FEMA extreme wave database for this region.

North of Point Conception, owing to a lack of islands, the transformation of deep water waves to the coast is more straightforward. However, far fewer directional wave measurements have been made outside Southern California, particularly north of San Francisco, so the model has undergone little validation along most of the U.S. West Coast. Based on CDIP's experience with the model in Southern California, and an application of the model to a site at the entrance of Bolinas Lagoon near San Francisco by Philip Williams and Associates, it is anticipated that the spectral refraction model will perform well north of Point Conception. Nevertheless, new directional wave measurements specifically for model validation in Oregon and Washington are needed.

### *Wave Model Information Needs*

The primary boundary condition information needed to develop the nearshore wave model database is bathymetry and hindcasts of extreme deep water wave spectra.

#### Bathymetry

Adequate bathymetric data is believed to exist for the West Coast. CDIP currently maintains a bathymetric wave model grid for the California Coast. In addition, the National Geological Data Center (NGDC) has recently released high resolution bathymetric grids for the West Coast, and maintains a database of digital bathymetric survey data for this region.

Combining the various data sets into an optimal wave model bathymetry grid for Oregon and Washington will be required, but is not foreseen as a significant hurdle in the development of the nearshore wave database.

#### Deep Water Spectra

Several deep water hindcast databases currently exist or are being developed in the public and private sector. The USACE has been revising its Wave Information Study (WIS) database for the Pacific Coast, and several private companies (e.g., Oceanweather) have developed similar databases. FEMA will need to acquire an extreme deep water spectra database, nominally with 1 to 2 degree latitude spacing up the West Coast, for use as an offshore boundary for the wave model. Extreme event hindcasts, and resulting nearshore model predictions for approximately 1980 to the present, will be validated against deep water wave data collected by CDIP and NOAA where possible. As mentioned previously, significantly more wave data are available in California (and Southern California in particular) for this purpose.

### Nearshore Database Limitations

Linear spectral refraction modeling of wave spectra makes several important assumptions and has known limitations.

- The model is a "propagation only" model, and assumes that additional wind input to the spectrum is small from the deep water boundary to the nearshore site. In Southern California, this means incident swell from the open Pacific and seas generated within the islands by local winds must be treated separately. North of Pt. Conception, this assumption needs further validation, particularly in Oregon and Washington where the continental shelf is widest and additional wind input to the spectrum across the shelf may be large enough to measure.

- The model assumes currents are weak and bottom dissipation effects are small. Validation of the model in Southern California has confirmed that these assumptions are reasonable. However, the model may not be appropriate for a nearshore site directly seaward of an inlet where tidal currents are strong.

- The model assumes wave diffraction effects are weak. This assumption has also been confirmed through inter-model comparisons in Southern California. An exception would be very close (1 wavelength) to a coastal structure like a jetty or groin. However, it is anticipated that the nearshore model prediction sites will be seaward of any coastal structures in order to remain outside the surf zone when modeling large wave events.

- The model assumes the bathymetry seaward of the nearshore model sites does not change. CDIP's experience in Southern California, comparing old bathymetric surveys to recent ones, suggests this is a reasonable assumption. In addition, spectral wave model results are generally not sensitive to small changes in the model bathymetry, or tidal elevation, in water depths greater than 10m. Nevertheless, the nearshore database may require periodic updating in some coastal areas if local knowledge suggests that significant bathymetric changes occur seaward of the nearshore model site (e.g., near river mouths or large inlets). An example of this would be the San Francisco Bar, which has not been surveyed since the 1950s (Battalio and Trivedi, 1996).

- Nonlinear effects are not included. Specifically, the effects that generate infragravity (IG) waves as a result of the interaction of two linear spectral components are not taken into consideration seaward of the inshore location of the wave transformation. Although this interaction is known to be strongest in shallower water, it is also known to be strong on the Pacific Coast and there may be substantial contributions to the growth of the IG portion of the spectrum seaward of the landward point of linear wave transformation.

### Required Tasks for Wave Database Development and Validation (Long-term)

- Task 1: Literature and Data Search.

- Gather all the literature (reviewed and gray) on the application of the spectral refraction model on the West Coast.

- Locate all the sources of digital bathymetric data for the West Coast.

- Locate all potential sources of deep water wave hindcast spectra for the West Coast.

- Task 2: Model and Field Measurement Validation Planning (Based on what we learn from Task 1).

  - Decide on one or more hindcast data sets to acquire/use.

  - Decide on what bathymetric data sets to use to make the official FEMA West Coast bathymetric grid.

  - Decide where to make additional wave measurements. The goal is for FEMA to have a comprehensive set of studies/references to support the use of the final database.

- Task 3: Model Development and Field Measurements.

  - Deploy additional wave buoys to begin acquiring optimal validation data.

  - Assemble U.S. West Coast wave model bathymetry grid.

  - Make initial model runs at various locations on the West Coast with simulated high energy wave spectra. Use these to decide how to select locations of nearshore sites (water depths, and alongshore spacing) and where to apply special regional modeling needs (local seas inside islands, island wave reflection).

- Task 4: Field Validation of Deep Water Hindcasts and Nearshore Predictions.

  - Use all the existing data, both historical and newly acquired, to validate the deep water hindcast and nearshore model accuracy during large wave events. Modify modeling methodology in some areas if necessary. Document findings with appropriate reports and/or peer reviewed papers.

- Task 5: Evaluate need to include nonlinear effects in some manner.

- Task 6: Create FEMA Nearshore Wave Spectra Database.

  - Generate the database using field validated hindcast data and wave transformation code.

  - Prepare a simple instruction manual on the use of the database.

- Install the data and the manual on a secure Internet site.

***Required Tasks for Interim and Short Term***

See Short-Term (Phase 2) Recommendations, in the following section.

### 2.2.5 Recommendations

***Long-Term Recommendations***

1. A substantial amount of nearshore data exists to validate the magnitude of changes to the high frequency part of the spectrum during extreme events. A study of these data should be undertaken and the errors evaluated to determine if they are significant. This may require a subregional approach (i.e., wind effects in the Santa Barbara Channel may differ significantly from those off San Diego County.) If the potential error is small, then Approach (a) should be used in establishing the standard database of nearshore waves in Southern California. Approach (a) is to assume no wind-induced change in the spectrum. Note that this would result in a uniform approach being taken for the entire West Coast wave database because the broad shelf problem does not exist elsewhere on this coast. If the error is too large to be ignored, then a separate database of measured variations in the wind wave spectra should be undertaken. This will allow for the correction to be treated as an independent variable additive to the modeled nearshore spectrum.

2. Adopt regional wave modeling for the Southern California Coast.

3. Expand CDIP for the California Coast of the US:

    a. Use regional models to develop near shore directional spectral wave climate,

    b. Acquire and process bathymetry,

    c. Acquire hindcast offshore wave database,

    d. Verify hindcast by comparison with recent (after 1980 buoy deployment) buoy data, and

    e. Verify nearshore wave spectra with wave measurements.

4. Evaluate any limitations due to the linearity of the transformation models.

5. Consider expanding regional wave modeling for Washington and Oregon Coasts using CDIP or other programs (e.g., WIS) at the appropriate time and depending on the need, recognizing that regional wave models are more logical in densely populated areas. Individual studies may be performed in sparsely located communities (see Topic 8).

### *Short-Term (Phase 2) Recommendations*

1. Develop Interim Guidance for: (This work is proposed as critical for the Study Topic 8 Wave Transformations, but is included here for completeness.)

    a. Southern California: Develop *G&S* for use of CDIP information for this region where the CDIP program is the most mature and wind wave growth may be important within the modeling domain.

    b. Central California: Develop *G&S* for use of CDIP data for this region where the CDIP program is less mature;

    c. Northern California: Develop *G&S* for use of CDIP data for this region where the CDIP program is the least mature.

2. Use existing CDIP bathymetry grids for the California Coast.

3. Use an alongshore spacing of 400m on the 20m depth contour for the entire coastline.

4. Create 2 sets of transformation coefficients in Southern California. One set for swell (waves modeled from outside the islands to the 20m contour) and a second set for seas (waves modeled from the mainland shelf break, inside the islands, to the 20m contour).

5. In each of 3 regions (Southern California, Central California, Northern California) demonstrate the models capability for predicting nearshore wave conditions during large winter storms using existing buoy data (very limited data available for Northern California).

6. Create a database on the CDIP server that is accessible to FEMA contractors. Provide a user's manual, and simple Fortran and MATLAB code, to assist contractors in using the model coefficients with their hindcast wave spectra.

**Limitations of the short-term plan:**

- The short-term modeling effort will not address known underprediction of wave heights between Santa Barbara and Ventura owing to reflection of NW swells from Santa Cruz Island.

- Recent bathymetric survey data for some areas of California will not be included in the fast-track product.

- The 400m spacing of alongshore points may be somewhat coarse in areas with extreme nearshore bathymetry (e.g., around submarine canyon heads).

- It is assumed that FEMA will provide CDIP with at least minimal funding to maintain the database after the 6-month contract period.

### 2.2.6 Preliminary Time Estimates for Preliminary Guideline Preparation

Table 2 summarizes the preliminary estimates of time required for Critical Topic 8. These time estimates do not include responding to comments and suggestions associated with the review of the Guideline improvements.

### 2.2.7 Related Available and Important Topics

Not Applicable.

## 2.3 TOPIC 9: PROPAGATION OVER DISSIPATIVE BOTTOMS

### 2.3.1 Description of the Topic and Suggested Improvement

The sea floor starts to influence the heights and directions of waves when they enter regions with water depths less than half a wavelength. Common mechanisms for such change include refraction, reflection, shoaling, breaking, diffraction, and bottom dissipation.

This section addresses the effects of bottom dissipation on wave transformation. The nature of the bottom (roughness, porosity, rigidity, etc.) and its interaction with surface waves causes wave damping and changes in wave kinematics. Appreciable wave height attenuation may occur if the wave propagation distance is long or if the bottom is not very rigid.

Ignoring bottom dissipation mechanisms can lead to overestimated nearshore wave heights, particularly when the transformation distances are great or when the bottom contains soft muds. In turn, the overestimated wave heights may lead to overestimates of flood hazard risk for shorefront development.

*G&S* needs to address this topic, because wave energy dissipation is a significant part of wave transformation in the Gulf of Mexico and for beaches with similar or other local geomorphologic conditions in the Atlantic and Pacific regions.

### 2.3.2 Description of Procedures in the Existing Guidelines

Presently there is little or no guidance on wave dissipation mechanisms for wave transformation analysis in FEMA guidelines. For overland wave propagation, WHAFIS model includes wave dissipation from marsh vegetation (*G&S*, Appendix D, 2002). However, wave dissipation from muddy bottoms has not been included in WHAFIS.

### 2.3.3 Application of Existing Guidelines to Topic–History and/or Implications for NFIP

Significant wave dissipation and damping can occur before waves travel overland during extreme wave events. The guidance in the current *G&S* are inadequate, given different site characteristics encountered in FIS.

### 2.3.4 Alternatives for Improvement

#### *Overview*

Typical wave propagation analysis involves transforming hindcast or buoy data in deep water to the nearshore through numerical models that simulate generation, shoaling, refraction, diffraction, and breaking; bottom dissipation effects are not routinely considered.

A literature review on the above topic demonstrates that bottom dissipation mechanisms can lead to significant wave height attenuation in the nearshore. Consideration of such dissipation mechanisms can help increase the accuracy of predicting nearshore wave heights. In the *G&S* development, guidance shall be provided for when and where dissipation can become significant to consider in FIS. The guidance can be based on bottom type, propagation distance, relative depth (depth/wave length or depth/wave height), wave steepness, wave height and length, and shall identify what methods to use for each bottom type.

#### *Technical Background*

**Existing Procedures**

Bottom dissipation mechanisms can be mathematically expressed as a negative forcing term in the conservation of wave energy equation as follows.

$$\frac{\partial E}{\partial t} + \nabla_h \bullet \left( E\,\overrightarrow{C_G} \right) = -\varepsilon \qquad (1)$$

where $E$ is the wave energy, $C_G$ the wave group velocity, $\varepsilon$ the energy dissipation rate per unit area, and t time. $\nabla_h$ is the horizontal gradient operator. For steady state, longshore uniform conditions, Equation (1) reduces to

$$\frac{dEC_G}{dx} = -\varepsilon \qquad (2)$$

where $x$ is the direction of wave propagation. Dissipation can occur at the surface, at the bottom, and due to wave breaking. One may consider ε as the sum of energy dissipations due to wave breaking and bottom effects. Dissipation due to bottom effects dominates seaward of the break point; dissipation due to breaking dominates landward of the break point. The following sections describe commonly accepted relations for dissipation due to rough, porous, and mud bottoms and vegetated marshes.

**Rough Bottom**

Dean and Dalrymple (1991) express energy dissipation due to bottom friction as

$$\varepsilon_F = \frac{\rho f}{48\pi} \left( \frac{H\sigma}{\sinh kh} \right)^3 \qquad (3)$$

where $\rho$ is the density of water, $f$ the friction factor, $H$ the wave height, $\sigma$ the angular wave frequency, $k$ the wave number, and $h$ the water depth. The friction factor is a function of the Reynolds number of the flow at the bottom and relative bed roughness (a ratio of the excursion of the water particles at the bottom to the bottom roughness). Typical friction factor values lie in the range $10^{-3}$ to 100. An alternate form of Equation 3 used by some researchers defines a modified friction factor $c_f$ equal to $f/8$.

**Porous Bottom**

Dean and Dalrymple (1991) express energy dissipation due to bottom percolation as

$$\varepsilon_P = \frac{\rho g^2 H^2 Kk}{8\upsilon \cosh^2 kh} \tag{4}$$

where $g$ is the acceleration due to gravity, $K$ is the permeability constant, $v$ the kinematic viscosity of water. Typical values for $K$ for sand are in the range of $10^{-9}$ to $10^{-12}$ m$^2$.

**Viscous Bottom**

Dean and Dalrymple (1991) express energy dissipation due to a viscous bottom as

$$\varepsilon_V = \frac{\rho_2 \sqrt{\sigma \upsilon_2} H^2}{16\sigma^2} e^{2kh} \left(\sigma^2 - gk\right)^2 \tag{5}$$

where $\rho_2$ and $\upsilon_2$ are the density and kinematic viscosity of the mud layer, $H$ the wave height of the surface wave. The angular wave frequency is

$$\sigma^2 = \frac{gk(\frac{\rho_2}{\rho} - 1)\tanh kh}{\frac{\rho_2}{\rho} + \tanh kh} \quad \text{and} \quad \sigma^2 = gk \tag{6}$$

in which $\rho$ is the density of the water.

**Other Formulations**

Other formulations for non-rigid beds are also possible. Lee (1995) provides a general summary of dissipation described by different models for non-rigid beds, Lee (1995) also suggests a wave attenuation function of the form

$$H = H_0 e^{-k_i x} \tag{7}$$

to model the effects of soft mud on wave propagation in uniform water depth. $H_0$ is the incident wave height and $k_i$ is the wave attenuation coefficient for soft muds. Lee recommends the range $10^{-4} \le k_i \le 0.05$.

Substituting Equation 7 into Equation 2, assuming a horizontal bottom for a wave traveling a distance X from location 1 to location 2, yields

$$\varepsilon_V = C_G E_1 (1 - e^{-2k_i X}) / X \tag{8}$$

where ε is the energy dissipation experienced by the wave as it travels from location 1 to 2 and $E_1$ is the wave energy at location 1. Note that Suhayda (1984) adopts a similar approach in his numerical model to simulate wave energy dissipation due to both soft muds and bottom dissipation—in fact, he uses the form of Equation 7 to model the effects of both bottom friction and soft muds on wave height evolution.

**Vegetated Marsh**

WHAFIS simulates the effects of energy dissipation by flexible and rigid vegetation on wave height. When necessary, this WHAFIS methodology, developed for overland energy dissipation by marsh plants, can also be adopted for computing the effects of such vegetation seaward of the shoreline. In practice, this suggests that the WHAFIS computations should begin at the seaward edge of the marsh vegetation rather than at the generally-adopted mean sea level shoreline start point.

**Method for Wave Attenuation in Vegetated Marsh Conditions**

Many investigators have suggested that vegetation damps wave energy. Knutson et al. (1982) performed a field experiment to quantify this phenomena in Smooth cordgrass (*Spartina alterniflora* also called Atlantic cordgrass) marshes. Dean (1978) suggests that marshes will affect waves in much the same manner as an array of vertical cylinders. Knutson et al. (1982) modified this equation to calculate wave damping by marsh plants and calibrated the coefficients for smooth cordgrass. The equation relates $H_1$, the incident wave height seaward of a stand of marsh grass to $H_2$, wave height landward of the stand of marsh grass as follows.

$$H_2 = \frac{H_1}{1 + AH_1 w} \tag{9}$$

where $w$ = width of the stand of grass from seaward to landward through which waves propagate and

$$A = \frac{C_p C_d D}{3\pi\, S^2 d} \tag{10}$$

in which, $C_p$ =plant drag coefficient, $C_d$ = typical drag coefficient approximately 1.0, $S$ = stem spacing, and $d$ =water depth. This was derived for constant depths. For smooth cordgrass marshes, Knutson et. al found that $C_p$ =5.

It should be noted that marsh vegetation and morphology differ from region to region and with salinity levels. Pacific cordgrass (*Spartina Foliosa*) is less substantial than Atlantic cordgrass (Figure 21). Application of these equations can be considered for the Pacific Coast marshes with test cases to calibrate the coefficients. The above equations have been used to evaluate a minimum distance of vegetation required to damp wave energy in large and normal wind wave and tide conditions in the marshes in San Francisco Bay. The wave-damping model assumes that the transmitted waves actually encounter the vertical plants, therefore is less valid for larger water depths.

Hansen (2002) measured the effectiveness of tules (cat tails) in dissipating incoming boat wake energy in the Sacramento-San Joaquin Delta. Prior to commencing his research, a literature review was done and he found that studies on wave energy dissipation along the West Coast are sparse. Knutsen et al. (1981) conducted a study on erosion control by inter-tidal vegetation. His work included California marshes, but provided few results relevant to the present topic.

**Method for Wave Attenuation in Pacific Mudflats**

Waves are damped when traveling across mudflats, because of the sediment rich water column and movement of the bed, friction, and wave breaking. The attenuation of ferry wakes propagating over mudflats in San Francisco Bay has been analyzed by applying the Ippen-Keulegan (Ippen and Kulin, 1955; Keulegan, 1948) equations (PWA, 1995). The best-fit dynamic viscosity value was found using wave measurements. This reference can be used to provide guidance on wave dissipation over mudflats in San Francisco Bay. Figure 22 (PWA, 1995) shows the attenuation of wave heights with mudflat distance. Waves over mudflats have a wave height to depth ratio that is much lower than the standard depth limited breaking criteria; therefore, wave breaker location for soft mud beds can be different from that of a sandy bottom for the same gradients.

## Selected Literature Review and Recent Studies

Recent wave measurements and modeling work by Surfbreak Engineering Sciences (unpublished work) suggests that bottom friction can be an important dissipation mechanism for waves traveling over both sandy bottoms and hard bottoms. In fact, wave dissipation because of bottom friction over a hard bottom may be an order of magnitude higher than that over a sandy bottom.

Previous work by Suhayda (1984), Forristal and Reece (1985), Sheremet and Stone (2003a, 2003b), and ongoing work by Taylor Engineering suggests that dissipation by soft muddy bottoms can cause substantial attenuation in the offshore delta regions of the Mississippi and Atchafalaya Rivers.

**Sheremet and Stone (2003a, 2003b)**

The authors present concurrent measurements of wave height, wave period, wind speed, and wind direction at two sites 100 km apart in the Gulf of Mexico offshore Louisiana. Bottom sediments at one site (CSI 3) are cohesive and at the other sandy (CSI 5); both sites are located in about 5 m water depths and exposed to similar atmospheric and fetch conditions.

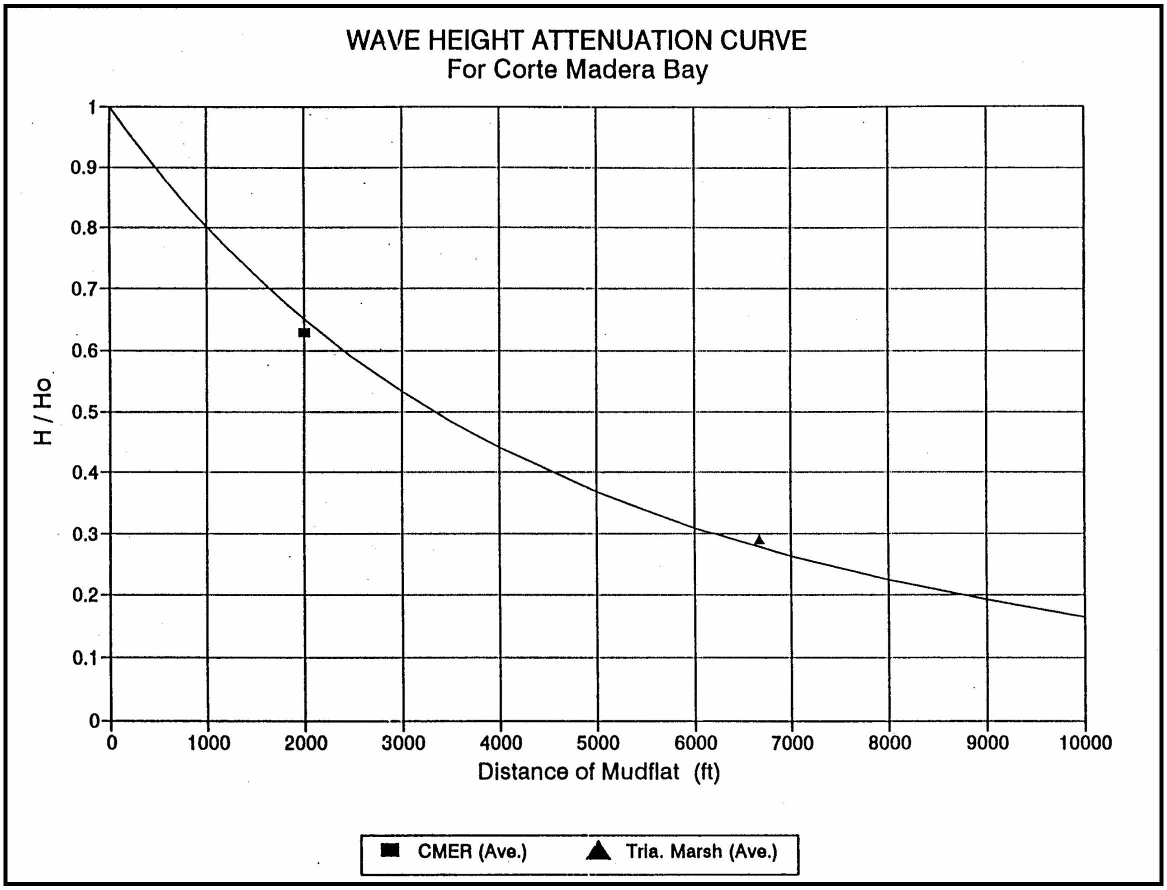
The analyses show that wave heights at CSI 3 were strongly damped relative to those at CSI 5; the damping was especially high for larger wave heights. They also tracked the evolution of wave energy with the passage of a storm whose wind speeds monotonically increased to a maximum and then decreased rapidly. Swell energy increased monotonically to a maximum and then decreased monotonically at both stations; however, the energy level at CSI 3 was much lower than that at CSI 5. Sea energy increased non-uniformly to a maximum at both locations and then attenuated rapidly; though this phenomena seemed to appear at both stations, the attenuation rate was higher for CSI 3. Thus, at the muddy location, seas rapidly dissipated after the wind forcing ceased.

Native Pacific cordgrass meadow at Blackie's Pasture, Marin County.

A tall stand of Atlantic smooth cordgrass hybrids invading a native pacific cordgrass meadow near Tiburon, Marin County.

Source: CSCC, 2003

**Figure 21. Cordgrass Species.**




Source: PWA (1995)

**Figure 22. Wave height attenuation curve for Corte Madera Bay. CA.**

Sheremet and Stone also applied the wave propagation model SWAN at the two measurement stations. SWAN does not model the effects of viscous bed dissipation. Predicted waves at CSI 5 were close to those measured; while the predicted wave heights at CSI 3 were too high compared to those measured. The authors hypothesize that the lack of an appropriate mud-related dissipation mechanism in SWAN was the cause of the inaccurate predictions.

A significant finding of their studies is that mud-induced wave dissipation extends across both low and high frequencies. They point out that bottom friction-type bottom dissipation mechanisms should only affect low frequency waves with little effect on high frequency waves. They suggest that short wave damping is related to cohesive sediment resuspension and fluid mud layer formation.

**Forristal and Reece (1985)**
The authors measured directional spectra of waves propagating in the Gulf of Mexico offshore Louisiana from deep (310 m) to relatively shallow ( 6 m ) water depths. In particular, the authors report on measurements and analyses of data for seven storms, including Hurricane Frederic

(1979) and Hurricane Allen (1980) and five winter storms, between 1979 and 1981. Maximum deepwater significant wave heights ranged from 1.88 to 8.59 m.

The primary purpose of the field data collection was to measure the attenuation of waves, caused by energy dissipation due to a soft bottom (a result of deposits of the Mississippi River), as they propagated from the offshore to the shallow water station. The authors shoaled and refracted the waves and accounted for wave travel time to transfer the deepwater spectra to the shallow water location. They then compared the transformed and measured spectra at the shallow water location.

Spectral analysis for Hurricane Frederic showed strong attenuation of energy across frequencies less than 0.2 Hz. For low frequencies, wave attenuation increased with wave height and was almost independent of frequency. During a modest winter storm in December 1979, wave energy attenuation was apparent for frequencies less than the peak spectral frequency; minimal change was observed for frequencies greater than the peak. In contrast, during the strongest measured winter storm in November 1980, spectral energy between 0.07 and 0.20 Hz was strongly attenuated.

Data from three storms, Frederic, Allen, and Winter 1980, show that wave height attenuation was a strong function of the deepwater energy and a weak function of frequency for low frequencies.

Bottom motion data showed that the bottom was 180 degrees out of phase with the surface wave.

In summary, bottom dissipation did not appear to be important for small waves; such a mechanism became increasingly important for larger waves. The attenuation rate was a strong function of deepwater wave height and a weak function of wave frequency.

**Tubman and Suhayda (1976)**

The authors measured wave characteristics and bottom oscillations in East Bay offshore Louisiana, an area covered by muds. The actual work done on the mudline was found to correspond to the dissipation on the surface wave. The authors computed wave energy dissipation to be one order of magnitude larger than that computed by general expressions for energy dissipation by bottom friction or percolation. Unlike friction-related dissipation, which occurs mostly in shallow water, soft (flexible) bottom dissipation cause significant wave attenuation in intermediate water depths.

**Suhayda (1984)**

The report documents the use of a numerical model to develop wave crest elevation attenuation coefficients by simulating the effects of wave generation by wind; shoaling; and dissipation due to breaking, bottom friction, and soft muds during extreme hurricanes. The author models wave height/energy to change exponentially with distance along the wave travel direction. To compute its effects on wave dissipation, he models the soft muddy bottom as a visco-elastic medium, in accordance with the MacPherson (1980) model.

The results summarize representative wave height to water depth ratios for 21 transects spanning St. Bernard to Vermilion Parishes along the Louisiana Coast. The ratios range from a low of 0.42 at Plaquemines Parish to a high of 0.78 at Lafourche Parish. The report also presents nomographs, developed from the numerical model, to relate the wave height to water depth ratio to mud shear strength, bottom friction coefficient, fetch length, and wind speed.

**Surfbreak Engineering Sciences (SES) Unpublished, Ongoing Work**

In 2001, SES collected two months of wave data at a nearshore location in water depths of 10 m offshore Brevard County, Florida (Figure 23). This open water site has a large offshore fetch and sandy bottom. Using the wave propagation model STWAVE, an offshore wave hindcast in 80 m was transformed to the wave measurement site; model predictions were compared to measured data. To accurately simulate the measured time series, SES had to modify STWAVE to include dissipation due to friction caused by a sandy bottom. In 2002-2003, SES collected wave data off the seaward and landward edges of low relief hardbottom offshore Indian River County, Florida (Figure 24). Water depths at the measurement sites, spaced about 700 m apart, were 10 m and 4 m respectively. These open water sites have large offshore fetches. SES used STWAVE to transform the offshore wave data to the nearshore site. To accurately mimic the lower wave heights recorded landward of the hardbottom, SES had to include dissipation due to friction by a hardbottom in STWAVE. The appropriate friction factor for hardbottom was one order of magnitude larger than that for sand.

In summary, the standard STWAVE model tended to overpredict wave heights in the nearshore for beaches with both predominantly sandy and predominantly hard bottoms when bottom friction, was not included. By modifying STWAVE to include a dissipation mechanism related to bottom friction, SES was able to substantially improve the accuracy of the model predictions.

**Taylor Engineering Unpublished, Ongoing Work**

Taylor Engineering is applying the STWAVE wave propagation model for an ongoing project investigating the feasibility of restoring Acadiana Bays, located offshore southwestern Louisiana. Preliminary results suggest that the standard STWAVE model routinely overpredicts wave heights for modeled cases in both Terrebonne and Acadiana Bays. Modification of the STWAVE code to include bottom dissipation mechanisms, either due to friction or soft muds, increased the accuracy of the model predictions in 3.3 m water depth for storm waves which occurred in 1981 (Figure 25). Exclusion of bottom dissipation clearly overpredicts nearshore wave heights. Ongoing wave measurements will be compared to STWAVE predictions to further refine model dissipation parameters and investigate their dependence on common non-dimensional coastal parameters (e.g., wave steepness).

**Wave Dissipation Availability in Wave Transformational models**

REFDIF (Kirby and Dalrymple, 1994), a monochromatic wave propagation model, accounts for dissipation because of laminar surface and bottom boundary layers, turbulent bottom boundary layers, porous bottoms, and wave breaking. It does not simulate the effects of soft mud bottoms.

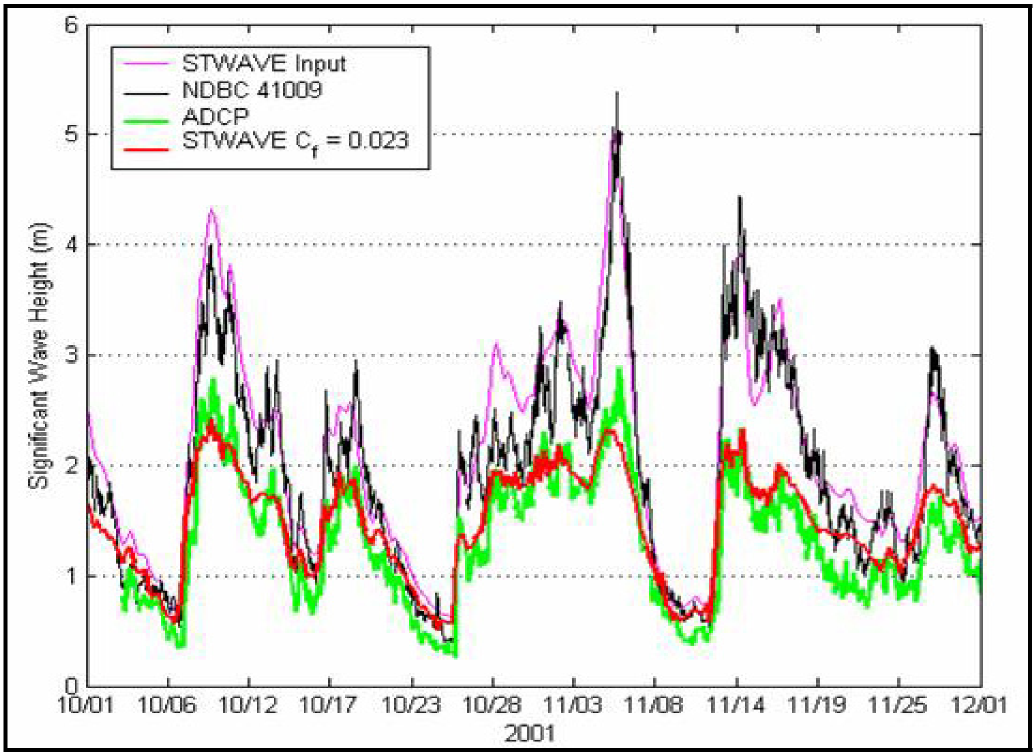
STWAVE (Smith et al., 2001), a spectral wave propagation model, accounts for dissipation due to energy transfer to high frequencies and white capping; but does not account for bottom dissipation mechanisms. Surfbreak Engineering Sciences and Taylor Engineering have recently modified STWAVE to include the effects of bottom friction and soft mud bottoms, respectively.

#### 2.3.5 Recommendations

A Study Contractor may cautiously employ numerical wave transformation models to obtain nearshore storm wave heights. Blindfolded applications of bottom dissipation mechanisms in wave propagation models is inadvisable given the large ranges in the possible values of the attenuation coefficients. Consequently, a study contractor should calibrate the numerical models so that predictions accurately mimic measured wave data. The contractor should select the appropriate bottom dissipation mechanisms and calibrate the relevant dissipation coefficients for site-specific conditions.

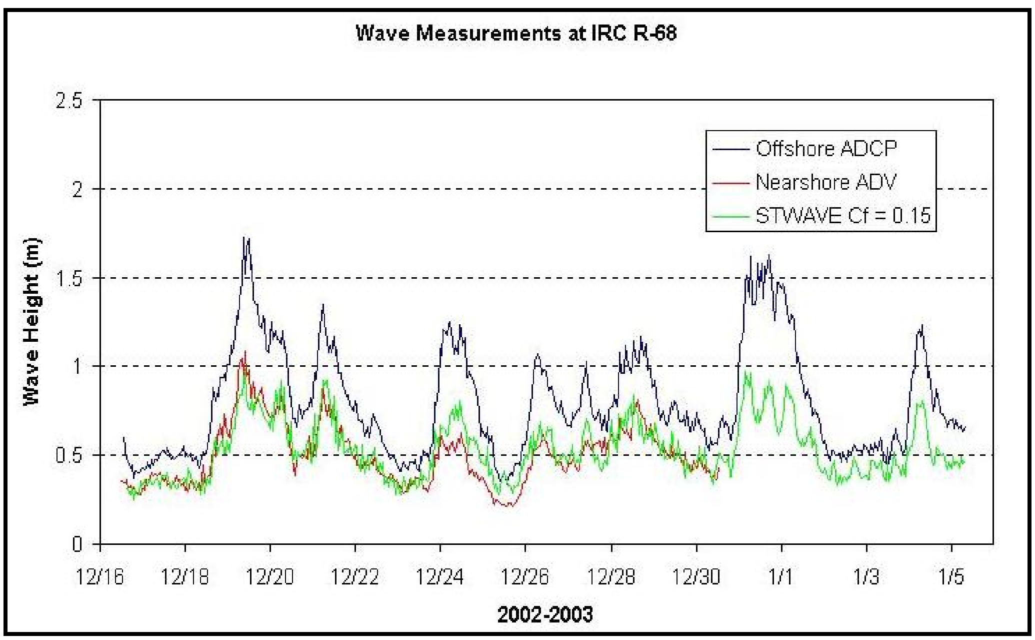
To provide guidance, typical ranges for dissipation coefficients corresponding to a variety of bottom conditions could be included in the Guidelines and Specifications. The data collected by Surfzone Engineering Sciences may help determine representative dissipation coefficients for sandy beaches and for beaches fronted by large expanses of hardbottom. Suhayda's (1984) work and ongoing work by Taylor Engineering may provide further information on wave dissipation by soft beds. The Guidelines and Specifications could also include regional-scale data about nearshore bottom conditions whether it is rough, soft, porous, or marshy  between, say, the 100-foot contour and the shoreline. The Study Contractor can then include the appropriate bottom dissipation mechanism(s) in the wave propagation model when transforming offshore wave hindcast/buoy data to nearshore conditions.

Until verified guidance is available, the Study Contractor should locate reliable offshore and nearshore wave height data and calibrate a suitable wave transformation model. The contractor should also have knowledge of the bottom conditions throughout the study area. The calibrated model should only be applied to areas with bottom conditions similar to those characterizing the wave gauge area. The contractor should then employ the calibrated wave transformation model where appropriate to obtain the design wave heights (starting conditions for WHAFIS) required for flood insurance studies. Sensitivity studies are advisable to apprise the contractor of the effects of poorly known coefficients.



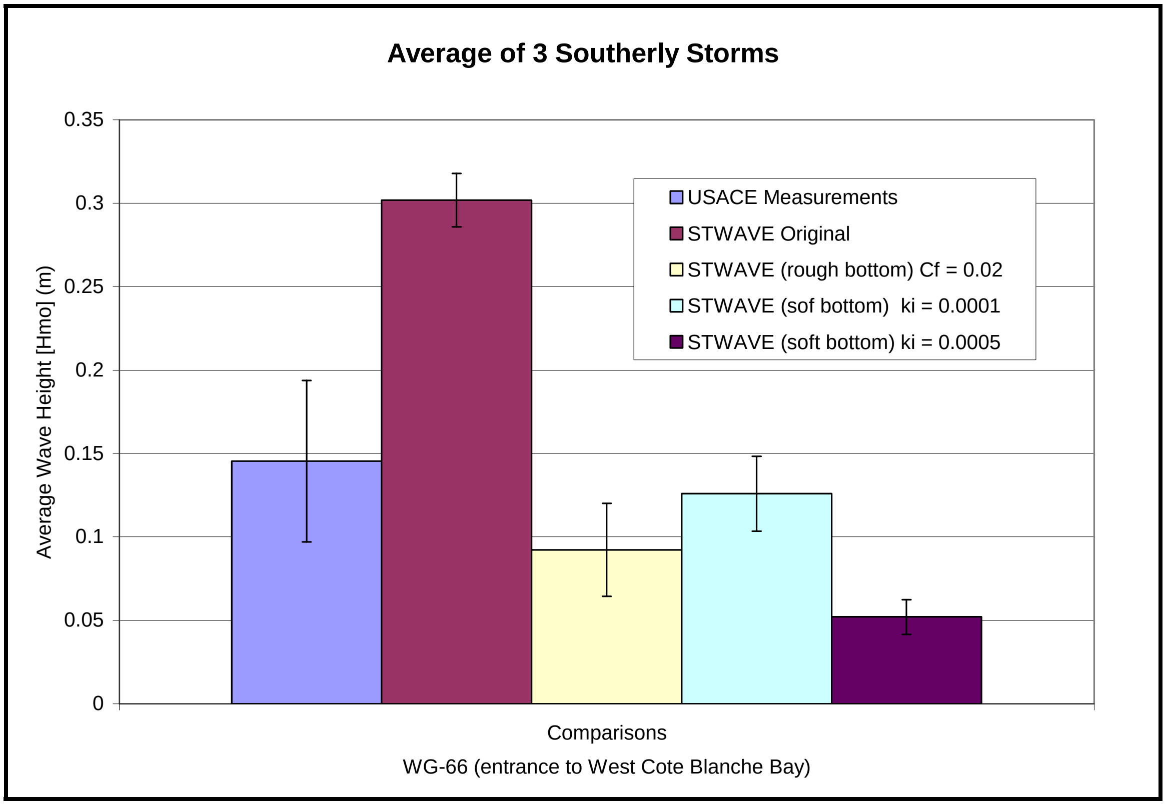

(courtesy of Surbreak Engineering Sciences, Inc.)

**Figure 23. Comparison of bottom friction-included STWAVE wave height predictions with measurements for sandy bottom.**




(courtesy of Surbreak Engineering Sciences, Inc.)

**Figure 24. Comparison of bottom friction-included STWAVE wave height predictions with measurements for hard bottom.**




(Taylor Engineering, ongoing work)

**Figure 25. Average wave heights measurements, in 3.3 m water depth, for three storms compared to predictions with different formulations of STWAVE.**

If model calibration is not possible, the contractor should, with one exception, ignore bottom dissipation effects in wave transformation analyses. Suhayda's (1984) methods and results appear reasonable and may be used to develop nearshore waves in Louisiana. Improvement could be gained by updating his results.

Recommended improvements are:

1. Write *G&S* to include a section on wave energy dissipation over shallow and flat bottoms.

2. Develop typical ranges for dissipation coefficients for a variety of bed and wave conditions to include in the *G&S*

3. Categorize bed and wave conditions for the U.S. coastlines.

4. Revise *G&S* to adopt Suhayda (1984) method.

5. Develop better guidance for West Coast conditions, namely for shallow surge over mudflats and West Coast marsh vegetation.

#### 2.3.6 Preliminary Time Estimates for Guideline Preparation

Table 2 summarizes the preliminary estimates of time required for Critical Topic 8. These time estimates do not include responding to comments and suggestions associated with the review of the Guideline improvements.

#### 2.3.7 Description and Suggested Improvement

Issues related to the WHAFIS program were categorized as Important for the Atlantic and Gulf coasts and as Helpful for the Pacific Coast. However, in considering possible improvements to WHAFIS, several topics have been identified that could be considered Critical instead of Important or Helpful, since they could be accomplished within a 6-month period, and because they would be valuable for near-term use in studies and map revisions on both the Atlantic/Gulf and Pacific Coasts.

Of particular importance are changes to the program that would make it more suitable for Pacific Coast applications. These include:

- The capability to specify wind speeds that would be more appropriate to the Pacific than those used by default for Atlantic/Gulf hurricanes
- Addition of new Pacific Coast marsh grass types
- Incorporation of dissipation over a muddy bottom

These and other Important/Helpful but potentially Critical topics are included in the WHAFIS discussion in Section 4.

# 3 AVAILABLE TOPICS

There are no available topics for Topic 10, Overland Wave Propagation; Candidate Improvements to WHAFIS.

# 4 IMPORTANT TOPICS

## 4.1 CONTINUATION OF TOPIC 10: WAVE PROPAGATION – WHAFIS IMPROVEMENTS

#### 4.1.1 Description of the Topic and Suggested Improvement

In the current coastal guidelines, FEMA's WHAFIS program is prescribed for modeling the overland propagation of waves. WHAFIS, now version 3, is an implementation and expansion of the wave propagation methods suggested by the NAS in *Methodology for Calculating Wave Action Effects Associated with Storm Surges* (1977).

WHAFIS is applied along *wave transects* which consist of one-dimensional descriptions of the terrain over reaches oriented approximately perpendicular to the shoreline. Each transect is divided into multiple segments defined by: station along the transect measured from the initial seaward point of the transect; ground elevation at each station; and nature of the segment just prior to each station.

Transect segments may be of several sorts, each either dissipative or regenerative for waves. The regenerative segments are identified as either *over water fetch* or *inland fetch*, where the two character identifiers are used to identify each line of data in an input file (in earlier parlance, they are *card types*). Over water fetch and inland fetch segments differ only in the wind speed which is used to compute the wave regeneration rate. The most important dissipative segments are regions of energy loss through *vegetation* and through rows of buildings . Areas in which the terrain rises above the local surge are identified with *above surge* stations, while natural and man-made narrow *dune*-like barriers are sometimes identified as DU stations.

Energy loss through vegetation is modeled as the loss through an equivalent stand of rigid vertical cylinders, defined by fixed values of diameter, height, average spacing, and drag coefficient throughout a segment. Energy propagation through rows of buildings is determined by the fractional open aperture between the buildings along a row, and the number of rows within the transect segment. The fraction of incident energy passing through a row is assumed to be equal to the average fractional open aperture between adjacent buildings; between rows, energy is assumed to be laterally redistributed before encountering the subsequent row. A rather detailed treatment of marsh grass was added to the initial WHAFIS implementation. It is denoted by the VH segment designation, and is the only segment type which considers the effects of both damping and regeneration.

Transects are spaced alongshore at intervals as necessary to reasonably represent the variation of conditions encountered by waves. The spacing may range from more than a mile on very uniform coasts, to on the order of a thousand feet in developed areas with rapid variation in landcover. A spacing of ½ mile is typical in many studies.

Conditions at the first transect point (usually at zero elevation on the beach) are defined by an *initial elevation* card which provides starting information such as the 100-year surge level (including setup), and the wave period and initial height. Alternatively, a fetch length can be specified, and the program will determine default values for height and period. A key assumption of the model is that wave heights are always limited to the breaking wave height (taken to be 78% of depth) at the local depth (surge elevation minus land elevation). This means, for example, that the wave height computed at the beach will generally be 78% of the surge height, irrespective of the starting wave height specified (neglecting wave setup).

The WHAFIS program follows the propagation of the wave along the transect, determining the variation of height and period. The surge elevation may be revised at any segment, in order to represent the surge variation over a barrier and across a protected bay. WHAFIS produces an output table dividing the total transect into zones (VE and AE) and base flood elevations, according to FEMA's coastal mapping rules.

### 4.1.2 Description of Potential Alternatives

A number of WHAFIS improvements have been identified for consideration. Although this topic is categorized as Important for the Atlantic/Gulf Coasts, and as Helpful for the Pacific and non-open waters, it may be that the WHAFIS program will continue to be used for some time, both in restudies and in coastal LOMRs, and that improvements categorized as Critical might be considered and completed within a relatively short time (less than 6-months). The following items have been identified, arranged in order of increasing level of effort.

***Minor Effort (Could be re-categorized as Critical)***

- Eliminate use of the 10-year stillwater elevation throughout WHAFIS. The 10-year level was needed in earlier mapping which included determination of the so-called Flood Hazard Factor (FHF). This is no longer necessary, and introduces confusion to the user.
- Eliminate the FHF information from Part 7 of the WHAFIS output.
- Revise WHAFIS to insert the run date and time into the output file, as an aid to documentation.
- Revise the code to locate and delete scratch files left over from a prior failed run; WHAFIS fails again if, upon loading, it encounters prior scratch files.
- Revise the code (eliminating SUBROUTINE READIN) so that the marsh grass data file is bound into the exe file. Even if WHAFIS is on the path, it does not look in its own directory for mg.dat, only in the current directory.
- Revise the code to allow comment lines, CM cards, which would simply be ignored during execution. This would be helpful in making the input files self-documenting.
- Revise the code to permit free-format input data files (as an option).
- Although it purports to do so, WHAFIS does not always reinitialize the wave period properly if the water depth (and wave height) goes exactly to zero (but if the terrain rises epsilon above the surge, then the period is reset to zero for subsequent regeneration). The regeneration rate is strongly affected by the period, sometimes leading to large excursions of zone boundaries between two adjacent transects, one reset and one not reset.

***Moderate Effort (Could be re-categorized as Critical)***

- Presently, the computation of starting wave period, if not specified on the IE line, is based on shallow water wave growth resulting in wave periods of less than 7 seconds.

This should be modified to also allow computation of a reasonable default suitable for open coast conditions.

- Restrict the use of Above Surge cards, making them unnecessary or seldom necessary. There may still be good reasons to use Above Surge cards in some cases (perhaps to help the interpretation of surge height changes), but the program should often be able to determine where the terrain rises above the surge, and where it falls back down.

- Improve the so-called "interpolation" that WHAFIS does between surge changes (SUBROUTINE SCANE). This is closely tied to the AS issue, above.

- Improve the default internal subdivision of long reaches, to better reflect the variation across the reach. If a reach, VE say, is very long, the ending wave height is OK, but the variation may not be well represented in mid-reach. Dividing the reach into two shorter reaches may be necessary – WHAFIS should take care of this better than it does.

- Presently, the WHAFIS treatment of regeneration over over-water fetch and inland fetch reaches is governed by the assumption of fixed wind speeds of 80 and 60 mph, respectively, deemed appropriate for hurricane conditions on the Atlantic/Gulf Coasts. It is proposed that the program be revised to use different default values for the Pacific, and also to permit the user to specify arbitrary wind speeds.

- Modify the program to include additional marsh grass varieties encountered on the Pacific Coast. This would actually be achieved through additions to the MG.DAT file which, as suggested above, should be bound into WHAFIS.

- Modify the program to include a new dissipative category, MB say, representing wave damping over a muddy bottom. This feature should account not only for damping, but also for the continuing influence of wind of user specified speed (with an appropriate default). The procedures would most likely follow the suggestions of Topic 9 included in another section of this Focused Study report.

- Modify the code to include output of the zone breaks and zone data in shapefile format, or in dbf format which can be imported by ArcView, for example, as a point theme.

- Modify the code to include creation of an optional HP/GL output file of the terrain and wave crest profiles.

### *Clarify Existing Guidelines and Incorporate New Guidance for the Items Above*

- The input requirements for all data fields need to be clarified, including such factors as proper units.

- Provide better guidance for the Dune card (DU).

- Provide guidance for choosing between inland fetch and over water fetch cards.
- Provide Pacific Coast guidance for optional windspeeds to be used with inland fetch and over water fetch cards (if implemented).
- Provide guidance for representation of elevated structures. This could include, for example, simulation as vegetation (rigid cylinders) if dissipation is appropriate; as fetch, if the effects of wind should be dominant; or as buildings with open fraction of 1.0, representing a zone with neither damping nor growth.
- Provide additional guidance regarding the selection of vegetation parameters, perhaps including photos showing various types of typical coastal vegetation along with recommended values of parameters. Include new guidance for Pacific Coast vegetation type, including Pacific marsh grasses (if implemented).
- Provide new guidance for dissipation over muddy bottoms (if implemented).

***Significant Effort (categorized as Important)***

WHAFIS divides the transects into reaches that are either dissipative or regenerative, not both (except marsh grass). It is frequently problematic whether one should account for whatever dissipation might exist, or whether to account for wind effects. A typical example is a reach of scattered vegetation, perhaps overtopped by the surge. In some cases, such a reach could be subdivided into a succession of alternating fetches and VE regions, but this would be laborious and seldom desirable. Similarly, the first few rows of elevated structures may offer little wave resistance, while (near the coast) wind effects may still be large. In this case, the problem is not only how to account for the damping (by representing the structural supports by a VE card, for example), but also how to account for the wind (as could be done by using an OF or IF card instead). A significant improvement to the accuracy of wave estimates would be made if WHAFIS were revised to consider the combined effects of damping and wind action over each segment, accounting for vegetation height and for wind sheltering by non-submerged elements.

### 4.1.3 Recommendations

Recommended improvements are:

- Change code for more user-friendly program (Minor Effort)
- Significant code changes for improvement in accuracy and graphics (Moderate Effort)
- Clarify Existing Guidelines and Incorporate New Guidance for the Items Above
- Improve WHAFIS to include combined effects of damping and wind action over each segment (Significant Effort)

- Evaluate applicable 1-percent wind condition for each region (e.g., Pacific, Atlantic, Gulf of Mexico) and change WHAFIS to incorporate a series of wind conditions to choose from.

- Revisions to WHAFIS should include associated revisions to the software CHAMP, which is a "shell" program used to operate WHAFIS in the Microsoft Windows environment.

### 4.1.4 Preliminary Time Estimate for Guideline Preparation

Table 2 summarizes the preliminary estimates of time required for Critical Topic 8. These time estimates do not include responding to comments and suggestions associated with the review of the Guideline improvements.

## 5 ADDITIONAL OBSERVATIONS

None.

## 6 SUMMARY

The Wave Transformation Focused Study is comprised of four topics: Topic 7 (California Regional Wave transformation Models), Topic 8 (Wave Transformations with and without Regional Models), Topic 9 (Wave Energy Dissipation over shallow flat bottoms), and Topic 10 (Overland Wave Propagation; Candidate improvements to WHAFIS). The recommendations are shown in Table 1.

Wave Transformations will be a key component of the *G&S* for the Pacific Coast Region. These *G&S* will be of use to other regions as well, including sheltered waters, owing to the very limited treatment in the existing *G&S*. Wave Transformations are processes that are conceptually intermediate between the flood forcing functions (see Focused Study for Storm Wave Characteristics) and flood response functions (See Focused Studies for Wave Setup and Wave Runup and Overtopping). Wave transformations are generally crucial for Pacific Coast Flood Studies, as demonstrated by prior efforts in Region IX and X, because of different geographic and oceanographic conditions than found in the Gulf and Atlantic Coast Regions. For example, the Pacific Coast is exposed to very long wave length swell that "feels bottom" much farther offshore, resulting in varying nearshore conditions that can only be quantified via wave transformation analysis.

It is recommended that the *G&S* identify a range of tools that can be applied based primarily on consideration of the site characteristics. Other factors should be identified, such as whether regional analysis of wave transformations has already been accomplished. Recommended actions will clarify the use of regional model products, the U.S. Army Coastal Engineering Manuel

(CEM), and readily available, contemporary computer programs of wave transformation processes. Also recommended is improved guidance to determine when wave dissipation effects are important to coastal flood studies, and how these effects can be quantified. Finally, a range of improvements to WHAFIS software used to quantify overland wave propagation in flood studies is identified and prioritized for implementation.

**Table 1. Summary of Findings and Recommendations for Wave Transformation**

| Topic Number | Topic | Coastal Area | Priority Class | Availability / Adequacy | Recommended Approach | Related Topics |
|---|---|---|---|---|---|---|
| 7 | CDIP California | AC | -- | -- | Use existing CDIP bathymetry grids for California Coast; Create 2 sets of transformation coefficients in Southern California; Demonstrate the model skill for predicting nearshore wave conditions during large winter storms using existing buoy data (for southern Central and northern California. Coast); Create database, Provide user's manual, and Fortran and MATLAB codes to assist contractors in using the model coefficients. | 8 |
| | | GC | -- | -- | | |
| | | PC | C | MAJ | | |
| | | SW | -- | -- | | |
| 8 | Overall Wave Transformation | AC | H | MIN | Refer to PC *G&S* for potential use of regional models | 6, 7, 9, 10, 11, 44, 45, 47, 48, 49, 54, 55 |
| | | GC | H | MIN | | |
| | | PC | C | MAJ | Write *G&S* for Wave Transformations. Tasks:<br>1. Conduct several focused studies to inform the Wave Transformations *G&S*.<br>2. Use available publications to identify a range of methods.<br>3. Develop criteria for level of analysis.<br>4. Include development of guidelines for spatial coverage and wave parameters, and include use of regional models such as CDIP.<br>5. Research available literature to adequately define wave groups, infragravity waves, shallow water spectra, etc. for input into wave setup and runup calculations.<br>6. Evaluate wave transformation models using a selected data set.<br>7. Review available literature and guidance on the range of applicability of contemporary computer models, recommend models for inclusion on the FEMA pre-approved coastal model list, and provide guidance on their application to FEMA FISs.<br>8. Incorporate applicable sections of existing *G&S* for other geographical areas that cover the overland propagation and wave energy dissipation topics. (Topics 9 &10) | |
| | | SW | C | **MIN** | Include in PC *G&S*; reference for AC and GC | |

**Table 1. Summary of Findings and Recommendations for Wave Transformation**

| Topic Number | Topic | Coastal Area | Priority Class | Availability / Adequacy | Recommended Approach | Related Topics |
|---|---|---|---|---|---|---|
| 9 | Dissipation | AC | C | MAJ | Write *G&S* to include a section on wave energy dissipation over shallow and flat bottoms based on available information. Develop typical ranges for dissipation coefficients for a variety of bed and wave conditions to include in the *G&S* from available information. Revise *G&S* to adopt Suhayda (1984) method. Provide guidance on calibration if available data not adequate to select coefficients. Conduct studies to develop typical ranges for dissipation coefficients for variety of bed and wave conditions to include in the *G&S*. Categorize bed and wave conditions for U.S. coastlines. Revise *G&S* to provide dissipation coefficients on a geographic basis to the extent appropriate | 8,10 |
| | | GC | C | MAJ | | |
| | | PC | C | MAJ | Evaluate wave dissipation over marsh and mudflats in the Pacific using available information provide interim guidance for calculating wave dissipation. Conduct field data collection to characterize wave dissipation over marsh and mudflats in the Pacific; provide guidance for calculating wave dissipation. | |
| | | SW | C | MIN | Include in PC *G&S*; reference for AC and GC | |
| 10 | WHAFIS propagation; evaluate new methods to better represent vegetation effects, treatment of elevated pile supported buildings | AC | I (C) | PRO | Clarify where WHAFIS, 1-D, and 2-D models are most appropriate Update WHAFIS and tie back to CHAMP. Minor Effort – code changes for more user friendly program. Moderate Effort – more intense code changes for improvement in accuracy and graphics, add wind direction. Update *G&S* accordingly. Significant Effort – improve WHAFIS to include combined effects of damping and wind action over each segment. Include realistic wave breaking model for setup and other processes after developed. | 8, 9 |
| | | GC | I (C) | PRO | | |
| | | PC | I (C) | PRODAT | | |
| | | SW | H | PRO | | |

Key:
Coastal Area
AC = Atlantic Coast; GC = Gulf Coast; PC = Pacific Coast; SW = Sheltered Waters
Priority Class
C = critical; A = available; I = important; H = helpful
(Recommend priority italicized if focused study recommended a change in priority class)
Availability/Adequacy
"Critical" Items: MIN = needed revisions are relatively minor; MAJ = needed revisions are major
"Available" Items: Y = availability confirmed; N = data or methods are not readily available
"Important" Items: PRO = procedures or methods must be developed; DAT = new data are required; PRODAT = both new procedures and data are required

**Table 2. Time Estimates for Wave Transformation Topics**

| Topic Number | Topic | Time (person months) |
|---|---|---|
| 7 | **Evaluate regional models for California** | |
| | Task 1: Literature and Data Search | 6 |
| | Task 2: Model and Field Measurement Validation Planning | 5 |
| | Task 3: Model Development and Field Measurements (includes 4 buoys. Measurement time period includes 2 winters) | 19 |
| | Task 4: Field Validation of Deep Water Hindcasts and Nearshore Predictions. (Can Critical Topics include field measurements?) | 13 |
| | Task 5: Evaluate need to include nonlinear effects in some manner. | 2 |
| | Task 6: Create FEMA Nearshore Wave Spectra Database | 7 |
| | Note: Short-term recommendations can be achieved within 6-months, and will be tailored to available budgets to the extent practicable. | |
| | TOTAL | 52 |
| 8 | **Assess need for regional models** | |
| | Write *G&S* for wave transformations as a section within the *G&S* for the Pacific Coast | 1.7 |
| | Use available publications and thorough literature survey to identify range of methods to select from | 0.5 |
| | Develop general criteria for level of analysis required | 0.6 |
| | Research available literature to adequately define wave groups, infragravity waves, shallow water spectra, communicate with the other analysis groups which need output from the wave transformation study, and develop approaches for *G&S* | 1.2 |
| | Evaluate adequacy of linear wave transformation models and needs to supplement these models. Place emphasis on representation of infragravity waves; | 1.2 |
| | Evaluate wave transformation and, review available literature and guidance of the range of applicability of contemporary computer models and recommend models for inclusion on the FEMA pre-approved coastal model list | 3.5 |
| | Incorporate applicable sections of existing *G&S* for other geographical areas that cover overland propagation and wave energy dissipation topics | 0.5 |
| | TOTAL | 9.2 |
| 9 | **Wave Energy Dissipation over shallow, flat bottoms** | |
| | Develop typical ranges for dissipation coefficients corresponding to a variety of bed and wave conditions. Revise Guidelines to reflect recommended dissipation relationships and coefficients. | 6 |
| | Categorize bed and wave conditions for U.S. coastlines. Revised Guidelines to provide dissipation coefficients on a geographic basis. | 6 |
| | Revise Guidelines to adopt Suhayda's (1984) results for Louisiana. Provide guidance on use by study contractors. | 0.3 |
| | TOTAL | 12.3 |
| 10 | **Candidate Improvements to WHAFIS** | |
| | Perform Minor Effort tasks | 1 |
| | Perform Moderate Effort tasks | 4 |
| | Clarify guidelines and incorporate items above | 3 |
| | TOTAL | 8 |

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# APPENDIX 1

# WAVE TRANSFORMATIONS DISCUSSION

# APPENDIX 1
# WAVE TRANSFORMATIONS DISCUSSION

## A-1 INTRODUCTION

The following text provides additional, detailed discussion supporting Topic 8, Wave Transformations. The organization was selected to be consistent with anticipated future Guidelines and Specifications, for the Pacific Coast Region.

### A-1.1 DESCRIPTION OF WAVE TRANSFORMATIONS

Wave Transformations refers to changes in wave characteristics during propagation. The primary processes are refraction, diffraction, shoaling and dissipation. Wave Transformations are considered in the regime bracketed by wave generation (typically in "deep water") and depth-induced breaking (typically "near shore").

Wave reflection and current-induced refraction are typically ignored. Refraction, diffraction, shoaling and dissipation are strongly dependent on the wavelength, with longer waves (higher wave periods) being affected the most (wave height is important, and dissipation due to propagation through vegetation can be greater for shorter, steeper waves). Irregular and steep bathymetry, also increase wave transformations.

Wave transformations are more likely to be important for West Coast flood studies owing to the longer waves, and generally steeper and less regular bathymetry. In southern California, nearshore wave heights can vary by a factor of 5 over a few miles of shoreline.

#### A-1.1.1 Relationship with Other Analyses Steps

##### *A-1.1.1.1 Storm Wave Characteristics*

Storm wave characteristics will be established, and used as input to wave transformations. Hence, *G&S* for Wave Transformations need to identify input requirements.  Simple situations will allow the use of simple input, such as wave height and period. More complex situations may require directional spectra.

##### *A-1.1.1.2 Wave Setup*

Wave setup is the super-elevation of the mean water level caused by wave action (USACE CEM, 2003). As waves break, the still water level lowers at the breaking point and then increases between the break point and the shoreline. The simplified methods available for calculation of wave setup require the knowledge of the wave parameters at breaking (steepness) and other geometric parameters like the foreshore slope past breaking point. Output from wave transformation models, serve as input into wave setup models. Some wave transformation models calculate the radiation-stress within the program, which is used in wave setup calculations.

#### *A-1.1.1.3 Wave Runup and Overtopping*

Runup is the maximum elevation of wave uprush above still-water level (USACE CEM, 2003). The current *G&S* supports programs RUNUP 2.0 and ACES for runup calculations and ACES for overtopping calculations. The maximum breaker height, height to depth ratio, and equivalent deepwater heights are needed as input into these models. More accurate assessments of runup and overtopping may require additional information such as wave height distributions and wave group characteristics.

#### *A-1.1.1.4 Overland Propagation (WHAFIS)*

WHAFIS currently uses the depth limited breaking wave height to provide the upper bound of the wave crest profile. Improved estimates of height to depth ratios may be useful.

#### *A-1.1.1.5 Tsunami*

Tsunamis are extremely long waves generated by disturbances associated primarily with earthquakes occurring below or near the ocean floor. Underwater volcanic eruptions and landslides can also generate tsunamis. In the deep ocean, their length from wave crest to wave crest may be a hundred miles or more but with a wave height of only a few feet or less.

Tsunami Predictions for parts of the West Coast are available for FEMA FIS in a report prepared by the US Army Engineer Waterways Experiment Station (1975) and more recent studies. This document does not consider Tsunami propagation as part of the guidelines.

### A-1.1.2 Processes

#### *A-1.1.2.1 Wave Refraction and Diffraction*

When parts of the wave are in different water depths, the part of the wave in shallow water moves slower than the rest of the wave in deeper water and causes the wave to bend, changing the spatial distribution of wave energy and height. This phenomenon is known as wave refraction.

Wave refraction can result in convergence or divergence of the wave energy, producing changes in wave height as well as wave direction in the nearshore. The output from refraction analysis provides input into wave runup and overtopping processes.

Wave diffraction is a process of wave propagation that can be important as refraction and shoaling (USACE CEM 2003). Diffraction of water waves is a phenomenon in which energy is transferred laterally along the wave crest (USACE SPM, 1984). Diffraction is mostly noticeable when waves encounter surface-piercing obstacles, such as a breakwater or an island. The waves after passing the barrier, would penetrate into the geometric shadow zone turning toward the lee side of the barrier with reduced heights along the wave crests.

#### A-1.1.2.2 Wave Shoaling and Breaking

The speeds at which waves travel depend on the water depth. In deep water, the wave propagation speed does not change. When the water depth reaches about one half of the wave length, the waves begin to slow down. As they slow, their lengths diminish and their heights increase due to a reduction in group velocity, increasing the wave steepness. The increase in wave height due to slowing down of the waves is referred to as wave shoaling.

In shallow water, wave crests become sharper and wave troughs flatter, approaching a Solitary wave form. As a solitary wave moves into shoaling water it eventually becomes unstable and breaks. A solitary wave breaks when the water particle velocity at the wave crest becomes equal to the wave speed. The ratio of breaking wave height to water depth is commonly used to define the breaking point, mathematically. Guidance is available in CEM(2003). Wave breaking is the prominent method of wave energy dissipation.

#### A-1.1.2.3 Wave Energy Dissipation, Non-Breaking

Waves lose energy due to bottom friction and viscous damping when they propagate over shallow and intermediate waters, and through inundated marshes.  While it is “conservative” to ignore wave energy dissipation, in some cases dissipation must be considered to achieve realistic results.

#### A-1.1.2.4 Wave Propagation Over Inundated Land Areas

If the land areas are inundated due to storm surge, the non-broken waves or the regenerated waves will continue to propagate over these areas, with wave growth due to wind energy input and decay due to frictional effects. This condition is important in the US East Coast, and is modeled with FEMA’s WHAFIS 3.0 program.

#### A-1.1.2.5 Wave Generation

While waves from distant storms are propagating from deep to shallow water, the local winds can impart energy and generate new waves which are called wind waves or local seas. These waves have shorter wave periods than the swells arriving from distant origin.

#### A-1.1.2.6 Wave Reflection

Wave energy could reflect off steep shorelines and barriers causing changes in wave height and direction. In the case of a vertical, hard structure, the fraction of wave energy reflected can be very large. Wave reflections from the channel islands (Santa Cruz) have been observed in the measured data near Santa Barbara in southern California. The transformational model used did not account for swell wave reflection and therefore the wave heights were under predicted.

#### *A-1.1.1.7 Nonlinear Effects*

As waves propagate, their nonlinear character results in interaction between the various spectral components. This nonlinear interaction is most significant in shallow water, for large wave heights and long periods and is evident along the Pacific Coast in terms of infragravity waves also called surf beat and harbor oscillations.

### A-1.1.3 Regional and Geomorphic Considerations

Different regions of the coast have different wave exposures and other characteristics that help frame the range of analysis methods typically appropriate. The major regions and their characteristics pertinent to wave transformations will be identified. A table would be developed to include a list of regions (Sheltered waters: San Diego Bay, San Francisco Bay, Puget Sound, etc.) and characterization of distinguishing coastal morphologies (offshore islands, canyons, wind sandy beaches, etc.).

The simplest geomorphic form consists of straight and parallel bottom contours leading to straight coastlines without offshore or nearshore features affecting the wave propagation. Although some coastlines can be simplified in the above category, most existing shorelines are far from the ideal straight and parallel contours. Curved coastlines carry their shape well into the deeper water contours.

Offshore shoals and reefs of different geometry and scale, and water depth at the shoal can affect the wave propagation. Wave energy focusing behind the shoal and defocusing in other areas is possible in the case of non-breaking waves, and the same shoal can be a source of wave dissipation for larger breaking waves.

Special cases include submarine canyons, river deltas, estuary ebb shoals and inlets, islands, submerged reefs and rock shelves, and distinct holes and channels.

## A-1.2 INPUT AND OUTPUT PARAMETERS

In wave transformation modeling, offshore wave parameters are needed as input to generate the nearshore wave parameters. Output from wave transformation models are necessary as input in to other models for evaluation of wave setup, runup and overtopping etc.

### A-1.2.1 Input and Output Parameter Selection

Input parameter selection is based on the requirements of the wave transformation technique that will be adopted subjected to constraints in availability of data (or type of data). In the least, wave transformation models require information on bathymetry, offshore wave height(s), period(s) and direction(s) or wave spectral information. Additionally, two-dimensional numerical models would need, lateral boundary conditions, wind, current or water level input and other coefficients for friction etc.

Output parameters are basically the wave parameters (wave height or breaking wave height, period, and direction in the nearshore or nearshore spectra and other available information such radiation stress for calculation of wave setup. The input requirements of other wave runup and overtopping models control the output selection.

#### A-1.2.2 Geographic / Geomorphic

Wave transformation processes are mostly dominated by the bathymetry in the intermediate and shallow water. Regularity or irregularity of bathymetry is a key factor determining the appropriate method of transformation.

Some experience or guidance is necessary for study contractors to select between different methods of wave transformation depending on the bathymetry of the area in consideration. Also, questions arise about the geographic limits and resolution of the bathymetric information necessary in setting up refraction-diffraction models. The former is governed by the wave period information, and the latter by the type of model used. In addition, geological and coastal bathymetric features can largely influence the wave transformation process.

#### A-1.2.3 Wave Characteristics

Characteristics of waves can be defined in many different ways. Basically waves can be defined by a height, period and direction or in terms of energy distribution in each wave period and direction (a directional or two-dimensional spectrum).

##### *A-1.2.3.1 Simplified*

In the analysis of ocean waves, the irregular seas are represented by parameters of varying complexity. The simplest form of these is a sinusoidal wave defined by a wave height and a wave period. The complex irregular sea is random, and therefore can be considered as a superposition of waves of several wave heights and periods. However, a single wave height and period is some times selected to represent the random sea. The most commonly used simplified parameters are shown in Table D-2 (pg. D-25), Appendix D, *G&S* (2002).

##### *A-1.2.3.2 Frequency Spectra*

A more realistic approach of defining the random sea is the energy distribution at different frequencies (reciprocal of wave periods)  A graph of wave energy vs. frequency is commonly known as a wave spectrum and usually has a single peak or multiple peaks of energy. Statistical methods are available to convert the energy spectrum to various wave height representations (Hmo, Hrms, Hs etc., SPM (1984)). Energy spectra without the wave direction information (directional distribution) are called frequency (or one dimensional) spectra S(f).

Two parameters are commonly used to describe the spectra. These are spectral width $\nu$ and spectral bandwidth $\varepsilon$ and are used to determine the narrowness of the spectra (USACE CEM, 2003). For a narrow-band spectrum, both of these parameters are close to zero. Since $\varepsilon$ tends to amplify noise in the high frequencies of spectra, an alternative parameters called the spectral peakedness parameter Qp was introduced by Goda (1974). The two parameters are not directly related, but in general, a small ε implies that Qp is large. Approximate relations for most common wave parameters by statistical analysis are given in CEM, Part II, Chapter 1, pg 87.

Wave spectra are measured by buoys or by other wave gages. These are archived commonly as bulk parameters. In instances where wave spectra are needed as input for wave transformation modeling, these parameters need to be converted back to spectral form. For this purpose, theoretical wave spectra (parametric spectrum models) developed by validating with measured data may be used. Two commonly used parametric model spectra are the single parameter Pierson-Moskowitz, PM spectrum (Pierson and Moskowitz; 1964), and the five- parameter JONSWAP spectrum (Hasselemann et. al.; 1973, 1976). In the JONSWAP model, three of the parameters are usually held constant. Other parametric spectral models which are essentially derivatives of the above two spectra are given in CEM (2003).

Wave energy spectra change from deep to shallow water due to the effects of depth and the interaction between the spectral components. It can be a simple reduction of wave energy in each of the wave frequency bands without change in the spectral shape, or a different spectral shape due to energy being transferred to different frequencies. Typically, high frequency portions of the spectrum decrease more rapidly owing to limitation on wave steepness, and a depth-dependent maximum can be applied for frequencies above the peak frequency. Bouws et. al. (1984) proposed a variation to the JONSWAP energy spectrum called the TMA (Texel, Marsen, and Arsole) spectrum to represent the wave spectra in finite-depth water. Also, energy can move into the lower frequency bands in the surf zone, frequently called infra gravity waves and often associated with wave groups and wave crossings.

#### *A-1.2.3.3 Directional Spectra*

Directional (or two-dimensional) spectra S(f, θ) show the distribution of wave energy as a function of frequency and direction. These are essential input for advanced wave propagation models, and are increasingly available through recent wave measurement programs or global scale model (WAM, WaveWatch III, etc.) outputs.

If wave directional spectra are not available but are required as input to another model or for smoothing out the artificial wave energy focusing effects, an approximate method can be applied using directional spreading functions. This is a semi-parametric approach for generating directional wave spectra. Goda (1985) discusses a couple of functions including the Mitsuyasu type (Mitsuyasu et al., 1975). Directional distributions typically become more focused with propagation into shallow water owing to refraction and wave / wave interactions.

#### *A-1.2.3.4 Groups and Infra-gravity Waves*

Long wave motions on the order of 30 sec to several minutes contain a considerable portion of the surf zone energy. These motions are termed infragravity waves (USACE CEM, 2003).

Bounded long waves are one type of infragravity waves related to wave groups. The mean water level is lowered at the higher waves of the group while it is raised at the lower waves of the group and that forms the oscillation of water level at group wave period. The oscillation is not free, but is bounded to the group and travels at the group speed (USACE CEM, 2003).

Although energy spectra at incident wave frequencies are usually saturated due to breaking, at infragravity frequencies, the energy density can increase linearly with increasing wave heights offshore. Therefore, infragravity energy becomes considerable part of energy during storm conditions.

#### *A-1.2.3.5 Breaker Parameters*

The ratio of breaking wave height (Hb) to the equivalent deepwater wave height (Ho')is frequently called the breaker height index. Breaking wave height and the breaking depth (db) are functions of the bottom slope and wave steepness (ratio of wave height to wave length). SPM (1984) provides curves to obtain breaker index given the wave steepness and beach slope. Goda (1985) provides guidance on calculating depth-dependant wave height distributions.

### A-1.2.4 Radiation Stress

Radiation stresses arise because of the excess momentum flux due to the presence of waves. Radiation stresses can be used to calculate Wave Setup.

## A-1.3 WAVE REFRACTION AND DIFFRACTION METHODS

### A-1.3.1 Method Selection

A range of techniques is available for transformation of waves from deep to shallow water. The type of bathymetry is a key parameter in selecting the appropriate method. The simple techniques can be applied in the case of simple bathymetry (straight and parallel) to account for wave shoaling and refraction (for offshore waves arriving at an angle to the shoreline). However, the randomness and directionality of waves may need to be addressed in complex sea states, using an offshore spectrum to transform all component waves in the spectrum and use superposition to obtain wave conditions in finite water depths, in order to model storms and extreme events. Model selection is subject to the key parameters of input/output, bathymetric features, and wave characteristics. Guidance on methods selection will be provided in the *G&S*.

### A-1.3.2 Simplified Methods

The simple techniques can be applied in the case of simple bathymetry (of straight and parallel contours). These would be hand calculations or simple graphical methods.

#### *A-1.3.2.1 Refraction by Snell's Law*

The path traced by the wave orthogonal as wave crest propagates onshore is called a wave ray. Simple wave propagation problems can readily be visualized by construction of wave rays manually or by graphical techniques. In the case of straight and parallel contours, and for monochromatic waves the Snell's law (sin θ /C = constant) can be applied to draw the path of the wave ray.

In addition, the wave height variation can be estimated by considering two closely spaced wave rays. Assuming no transfer of energy takes place across the wave ray boundary, wave height at any location along the wave ray is given in terms of the offshore wave height, shoaling and refraction coefficients. These coefficients can be calculated in terms of the water depth and the orthogonal distance between wave rays at the location of interest. The Coastal Engineering Manual (USACE CEM, 2003) provides solution nomograms (Figure II-3-6) which are also automated in the Automated Coastal Engineering System (ACES) program.

#### *1.3.2.2 Linear Refraction*

If the bathymetry has variations along the shore then the simple Snell's law approach cannot be made but a 2-D- wave ray approach has to be used. The ray approach for wave refraction has had problems caused by wave ray crossing, at which point wave height becomes infinite. These problems are caused by the fact that each ray is traced independently of the other rays. Some numerical methods overcome this problem by artificial smoothing techniques. Results need to be checked for signs of wave ray crossings (Caustics) and in that event a simple refraction-diffraction model is more appropriate.

#### *A-1.3.2.3 Graphical Diffraction*

Graphical Diffraction methods are available in SPM (1984); Goda (1985); and CEM(2003). Methods include monochromatic and simplified spectral approaches.  These methods can be applied relatively easily and are reliable for most cases. A description of application of Goda's methods using the s factor (directional spread) will be included.

### A-1.3.3 Refraction / Diffraction Models

The following text provides a summary of contemporary wave refraction / diffraction analysis methods. Some are approved for use by FEMA and some are not. As part of the *G&S*, it is recommended that those not approved be applied to a test case to identify the differences in results, and that further literature review be accomplished to gauge the accuracy of the models. Alternatively, existing literature describing case studies can be used as the sole basis of evaluation. Based on the results, recommendations for approval and guidance on application will be developed and included in the *G&S*.

When waves enter into a region of shallower water (water depth is less than about one-half of the wave length), the direction of wave propagation gradually changes. These changes can cause energy concentrations or spreading depending on the bathymetry. Sometimes when diffraction is not considered in the wave transformation method, wave heights can increase to unrealistic elevations. In reality wave heights are limited by breaking either due to depth or steepness constraints. Diffraction effects (spread of energy along the wave crest) can however, reduce the wave heights and keep it below the breaking height. For more complex bathymetry with shoals, islands or other major geological features, both refraction and diffraction need to be calculated or modeled.

A series of programs are available that deal with diffraction, in addition to modeling wave refraction and shoaling. A brief discussion of these models is available in the CEM, 2003. The CEM lists the computer programs RCPWAVE (Ebersole, 1985; Ebersole, Cialone and Prater, 1986), REFDIF1 (Kirby and Dalrymple, 1991) for monochromatic wave refraction, as available and in use by the Corps of Engineers but cautions the users to apply these models within the limits of their use.

FEMA pre-approved RCPWAVE is a steady-state linear wave model based on the mild-slope equation and includes wave breaking. The program is limited to open coast areas without structures or islands etc. A comparison of wave refraction and diffraction models was performed by Maa et al., (2000) based on the performance of wave transformation across the elliptic shoal experiment carried out by Berkhoff et al (1982), and RCPWAVE performed poorly in simulating the wave height distribution and wave direction. Therefore this model may be inadequate in modeling areas with sand shoals and other complex bathymetry.

The California Data Information Program (CDIP) has applied a linear, spectral back-refraction model along the California Coast. Detailed application of this model has included verification using directional wave data collected at deep and shallow water wave gauges. Very good results have been obtained. See the write-up for Topic 7: California Regional Wave Transformation Models for a more complete description.

REFDIF1 is a steady-state model based on the parabolic approximation solution to the mild-slope equation. Although this model is not pre-approved by FEMA, it is known to provide more accurate wave heights than from the RCPWAVE model in certain bathymetric situations (Maa et. al. 2000). However, if the study domain has complicated geography and/or bathymetry, or if there is a strong wave diffraction and /or reflection, elliptic mild slope models are appropriate.

MIKE 21 EMS is based on the numerical solution of the Elliptic Mild-Slope equation formulated by Berkhoff (1972) and is capable of reproducing the combined effects of shoaling, refraction, diffraction and back-scattering. Energy dissipation, due to wave breaking and bed friction, is included as well as partial reflection and transmission through, for instance pier structures and breakwaters. MIKE 21 EMS can be used to study wave dynamics in smaller coastal areas and in harbors. The Module is particularly useful for the detection of harbor resonance and seiching due to, for instance, long-period swell.

The extended mild-slope models may be more appropriate for steep and rapidly varying bathymetry. These models are computationally expensive and therefore only applicable to smaller areas.

### A-1.3.4 Spectral Refraction Models

#### *A-1.3.4.1 STWAVE*

Developed by the U.S. Army Corps of Engineers (USACE) Waterways Experiment Station (WES), STWAVE is a steady state, spectral wave transformation model, based on the wave action balance equation. A wave action approach can handle a current correctly where as an energy spectrum approach cannot. STWAVE is able to simulate wave refraction and shoaling induced by changes in bathymetry and by wave interactions with currents. The model includes wave breaking based on water depth and wave steepness. Other features of STWAVE include wind induced wave growth, and influences of wave white capping on the distribution and dissipation of energy in the wave spectrum.

STWAVE is most applicable to wave transformation problems where the following assumptions can be made:

- Mild bottom slope and negligible wave reflection.
- Spatially homogeneous offshore wave conditions with steady state wave, current, and wind conditions.
- Linear wave refraction and shoaling with negligible effects from bottom friction

Wave energy dissipation due to propagation over shallow areas (bottom friction and viscous damping effects are not included in the standard version of STWAVE; however, a version that addresses dissipation is being used by the Corps of Engineers (personal communication, Resio at Workshop 2) and other versions have been developed and used by others (see write-up for Topic 9: Wave Energy Dissipation over shallow,  flat bottoms) with good results.

#### *A-1.3.4.2 SWAN*

The numerical wave transformation model SWAN was developed at Delft University of Technology, Delft, The Netherlands. SWAN and STWAVE have many similarities. Like STWAVE, the formulation of SWAN is based on the spectral wave action balance equation. This model currently has many well developed features, which give the user many options on how each model run is executed. These features range from purely convenient options that allow several different formats for input and output data, to options that allow control of fundamental physical processes in the model, like wave generation, dissipation, and interaction. Linear wave refraction and shoaling are included in the model. Some differences from STWAVE are:

- Input wave conditions can be varied spatially along open boundary, and wind, water level elevation, and current inputs can be varied spatially over the entire computational domain.

- Simulations may be steady state or dynamic. SWAN has the ability to compute a time varying solution, rather that just a series of steady state solutions.

Users of SWAN must consider the following model assumptions in a specific application:

- SWAN does not model wave diffraction or reflection, and is therefore is most useful in applications where accuracy of the computed wave field is not required in the immediate vicinity of obstacles.

- Mild bottom slope with negligible wave reflection

#### *A-1.3.4.3 REF/DIF S*

REF/DIF S was developed at the Center for Applied Coastal Research, at the University of Delaware. This spectral wave transformation model is a further development of the REF/DIF 1 model, which solves for monochromatic waves only. REF/DIF uses the parabolic form of the mild-slope equation, and the complex amplitude of each separate wave component. Because the mild-slope form of the governing equation is used, the model includes the effects of wave diffraction, unlike STWAVE and SWAN.

#### *A-1.3.4.4 Alternatives to Spectral Models*

If the wave-wave interactions can be ignored, the simple method of “energy transfer functions” can be used to construct the nearshore wave energy spectrum at a specified location, for any given off-shore spectrum. The procedure involves calculating the response matrix using a linear refraction-diffraction model with a unit incident wave height (or amplitude) for the range of wave frequencies and directions. The transfer functions need to be calculated only once since the refraction-diffraction model is linear. The response to any desired incident directional spectrum is then constructed by appropriately weighting each discrete component. This method has been used by O’Reilly and Guza (1991, 1993) for wave predictions in an analytical circular shoal configuration and at Southern California locations. They use the spectral refraction model of LeMehaute and Wang, (1982) and a spectral refraction-diffraction model (linear version of the higher order PEM derived by Kirby 1986a, Kirby 1986c, and Kirby and Dalrymple, 1986 ). The California Data Information Program (CDIP) has applied a linear, spectral back-refraction model along the California Coast. Detailed application of this model has included verification using directional wave data collected at deep and shallow water wave gauges. Very good results have been obtained. See the write-up for Topic 7: California Regional Wave Transformation Models for a more complete description.

#### A-1.3.4.5 Use of Directional Spreading Functions

Wave directional spectra are not available as output from many of the above discussed models. In such a situation, if wave directional spectra are required as input to another model or for smoothing out the artificial wave energy focusing effects, an approximate method would be to use directional spreading functions. This is a semi-parametric approach for generating directional wave spectra. Goda (1985) discusses a couple of functions including the Mitsuyasu type (Mitsuyasu et al., 1975).

## A-1.4 WAVE SHOALING AND BREAKING

### A-1.4.1 Method Selection

Adequate information exists in the literature to complete the following *G&S* for Wave Shoaling and Breaking. Method selection is mainly based on wave characteristics and morphology.

### A-1.4.2 Linear Shoaling

Waves slow down in entering shoaling water and consequently wave height increases and sometimes decreases depending on group/phase velocity relations. The change in wave height due only to the change in wave group velocity is referred to as shoaling. Linear shoaling assumes the waves are of small amplitude and therefore the linear wave theory can be used to derive the shoaling coefficient (Ks = H/Ho) by equating the offshore wave power to the wave power at any nearshore location (before breaking). When other processes such as wave refraction, diffraction, and dissipation are involved in the transformation process, equivalent deepwater wave height is used instead of the deepwater wave height, in the shoaling equation (Ks = H/Ho’)

### A-1.4.3 Non-Linear Shoaling

As waves approach very shallow water, several wave lengths seaward of breaking, shoaling becomes highly non-linear and the linear shoaling coefficient may significantly under predict the wave height, especially for long waves in shallow water. Non-linear shoaling coefficients are available in several publications, which relate shoaling coefficients to parameters of wave steepness, relative depth and beach slopes (Goda, 1985, SPM and others).

### A-1.4.4 Breaking Indices

In shallow water breaking is limited by water depth and the point of breaking is influenced by wave steepness and beach slope. Simple wave breaking indices for regular and irregular waves are discussed in the CEM(2003), Part II-4.

A breaking wave model (series of equations) that operates on a site-specific nearshore profile (one-dimensional) is needed to calculate wave setup, as described in the Focused Study report for Setup (see in particular Topic 51 Interim Approach, and also Topics 44 through 48).  The breaking wave model should be adequate to calculate wave radiation stress through the surf zone for irregular wave conditions. The wave radiation stress is used to calculate wave setup. Guidance is also needed for the dynamic component of wave setup, using available information. The breaking wave model shall be applicable for the Gulf, East and Pacific Coasts, including sheltered waters, but is critical for the Pacific Coast.

#### A-1.4.5 Spectral Transformations

Vincent and Briggs (1989) showed by their lab experiments that wave transformation over a shoal is sensitive to the shape of the incident wave directional spectrum and differ significantly from a single unidirectional wave. Therefore, the approach of defining a single wave height to represent the offshore spectrum and using this wave height in the unidirectional wave transformation models does not prove to be satisfactory when shoals and complex bathymetries exist.

Transformation of incident wave frequency-directional spectra can be achieved by combining multiple model runs, each for a single frequency and direction (Izumiya and Horikawa, 1997 and Panchang et. al., 1990). These spectral models do not explicitly predict the directional spectrum but have been used to estimate the directionally integrated energy to determine the wave height.

### A-1.5 WAVE ENERGY DISSIPATION, NON-BREAKING

#### A-1.5.1 Method Selection

Method selection will be based on bed and wave conditions and or region and other site conditions, as described in the write-up for Study Topic 9: Wave Dissipation over Shallow, Flat Bottoms. To the extent practicable, coefficients will be provided for the described methods based on published data. Where data are not adequate to calculate wave dissipation, calibration will be recommended.

#### A-1.5.2 Friction

Friction related energy dissipation occurs mainly in shallow water (Tubman and Suhayda, 1976). The friction effect varies with the type of bottom material and also as a function of wave parameters, relative depth, propagation distance etc. Guidelines for selection of criteria will be addressed under study topic 9.

#### A-1.5.3 Viscous Bottom

Unlike friction related dissipation, which occurs mostly in shallow water, soft (flexible) bottom dissipation cause significant wave attenuation in intermediate water depths.

Suhayda (1984), documents the use of a numerical model to develop wave crest elevation attenuation coefficients by simulating the effects of wave generation by wind; shoaling; and dissipation due to breaking, bottom friction, and soft muds during extreme hurricanes. The author models wave height/energy to change exponentially with distance along the wave travel direction. To compute its effects on wave dissipation, he models the soft muddy bottom as a visco-elastic medium, in accordance with MacPherson's (1980) model. The results summarize wave height to water depth ratios in the range of 0.42 to 0.78 for the 21 transects, that he used in this study.

#### A-1.5.4 Marsh Vegetation

*G&S* Appendix D (2002) considers marsh vegetation (pg. D-72 to D-80) under description of the WHAFIS 3.0 model. Eight parameters are used to describe the dissipation properties. This procedure was specifically developed for the Gulf and Atlantic Coasts. Applicability of these guidelines for the Pacific Coast wetland areas, need to be explored. Also see the section "Method for wave attenuation in Pacific marsh conditions" under Topic 9.

### A-1.6 WAVE PROPAGATION OVER INUNDATED LAND AREAS

This condition is commonly observed in the US East Coast, and WHAFIS 3.0 approved by FEMA is applied in the present FIS. Although not common, overland wave propagation can be significant in marshes surrounding bays ( e.g., San Francisco Bay). The changes to wind characterization may be necessary to use the WHAFIS model for the Pacific conditions.

Continuation of the two-dimensional wave transformation models into the inundated regions may be the next step of improvement. However, application of two-dimensional models may be constrained by data availability. The *G&S* will address use of WHAFIS for Pacific Coast FIS, based on the write-up for Study Topic 10: Overland Wave Propagation, Candidate Improvements to WHAFIS. Extensive *G&S* exist for application of WHAFIS to the Gulf and Atlantic Coast FIS, with additional guidance in the Topic 10 write-up.

# Wave Setup

## FEMA Coastal Flood Hazard Analysis and Mapping Guidelines Focused Study Report

February 2005


Focused Study Leader

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**<u>Tables</u>**

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## Acronyms

ANWM advanced numerical wave models

EBP Equilibrium Beach Profile

ERDC Engineer Research and Development Center

CEM Coastal Engineering Manual

GROW Global Reanalysis of Ocean Waves

NOPP National Oceanographic Partnership Program

SPM Shore Protection Manual

SWL stillwater level

# 1 INTRODUCTION

This report provides recommendations for a program leading to improvement of the current FEMA Guidelines related to Wave Setup. Six Wave Setup topics were developed at the December 2003 Workshop. Three of these topics were labeled "Critical" and applied to all three geographic areas, two were designated "Important" and also applied to all three geographic areas, and one was designated "Available" and was later transferred to another group. Therefore, the five topics addressed by the Wave Setup Group are as follows:

**Wave Setup Topics and Priorities**

| Topic Number | Topic | Topic Description | Priority | | |
|---|---|---|---|---|---|
| | | | Atlantic / Gulf Coast | Pacific Coast | Non-Open Coast |
| 44 & 45 | Define, Document, Compile Data | Better define and document, summarize what to consider and how to approach; data requirements. Compile example data/sets to perform tests | C | C | C |
| 46 | Interim Method | Develop "Interim Method". (Look at CEM as a fall back, or Univ. of HI SPM procedure) | C | C | C |
| 47 | Develop Ideal Method - Coupled | Develop "Ideal Method" coupled with storm surge and waves to develop setup | I | I | I |
| 48 | Dynamic Wave Setup | Develop procedure for dynamic wave setup | I | I | I |
| Key: C = critical; A = available; I = important; H = helpful | | | | | |

## 1.1 WAVE SETUP FOCUSED STUDY GROUP AND APPROACH

The Wave Setup Group is made up of Ian Collins, David Divoky, Darryl Hatheway, Norman Scheffner and Bob Dean who served as Team Leader for this effort.

To provide structure to our efforts and to avoid unnecessary duplication, the following approach was adopted—the Team Leader developed background material, reviewed available information, and developed draft writeups for the approaches. The draft write up was then distributed to the Team Members who contributed information of which they were uniquely aware, critiqued and contributed to the draft writeups and accomplished specific components of the overall effort leading to this report.

## 1.2 CURRENT FEMA GUIDANCE ON WAVE SETUP

The current FEMA guidance for Mapping Partners to calculate wave setup relies on the 1984 Shore Protection Manual (SPM) that focuses on the average (or static) wave setup. The guidance recognizes the effect of beach slope and deep water wave steepness ($H_{os}/L_{os}$) based on the deep water significant wave height ($H_{os}$) and associated length ($L_{os}$). Figure 1 presents current FEMA guidance (page D-66 f Guidelines and Specifications). As seen from this figure, wave setup increases with steeper beach slopes and smaller wave steepness, $H_{os}/L_{os}$. The guidance also briefly discusses wave setup in the presence of a reef or offshore berm, but offers no specific guidance on these settings. Figure 1 shows predicted wave setup values of 7% to 8% of the deep

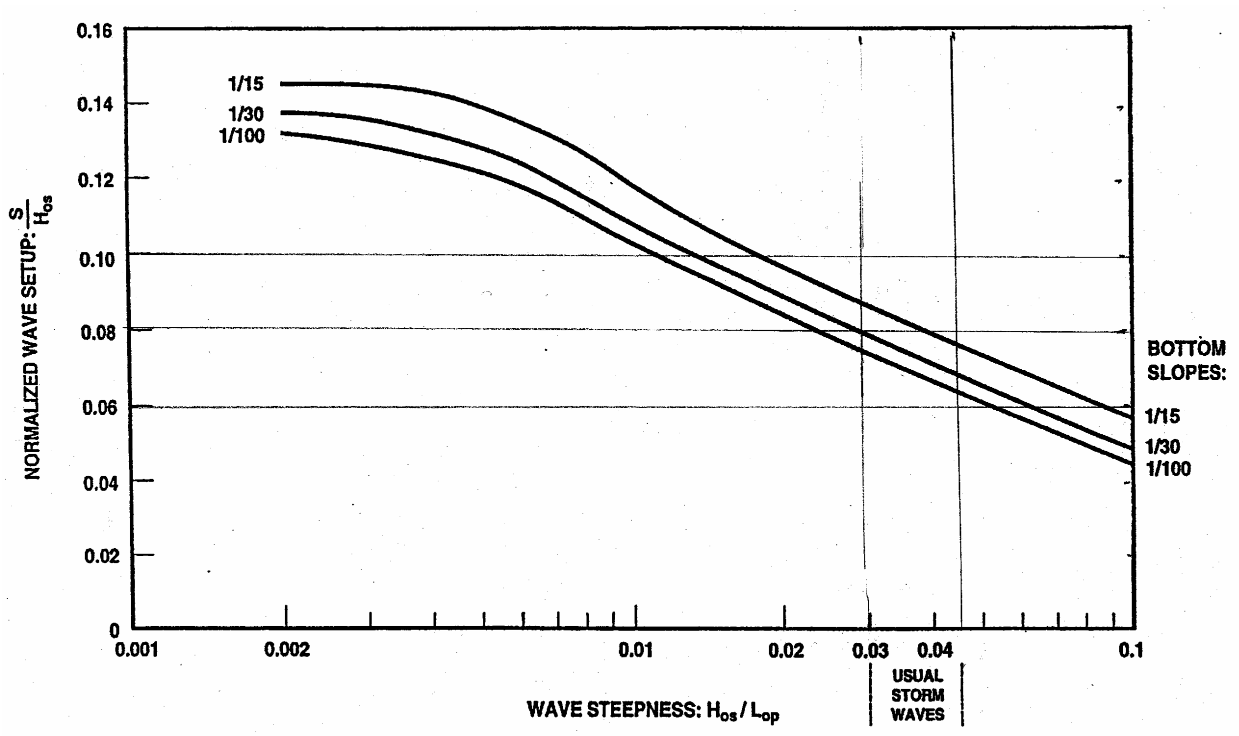



**Figure 1. Current FEMA guidance on wave setup based on 1984 Shore Protection Manual.**

water wave height for deep water wave steepness values of 0.03 to 0.04–typical for storm seas. Wave setup values of up to 10% are predicted for waves of lower steepness, which could govern for areas exposed to large, long period swell, such as the Pacific Coast. The recommended beach slope is the average from an offshore distance corresponding to a depth of $2H_{os}$ to the shoreline. The current guidelines do not contain any mention of dynamic wave setup, i.e., the fluctuating component of wave setup caused by groups of waves.

## 1.3 APPLICATIONS OF EXISTING GUIDELINES TO WAVE SETUP TOPICS

Wave setup can be a significant component of the total 100-year surge elevation on all coasts. The narrow Pacific continental shelf results in the combination of wave setup and astronomical tide being the two largest components of the 100-year surge. On the Atlantic and Gulf shorelines, wave setup can range up to 50% of the total 100-year surge in areas with narrow continental shelves.

As noted, current guidance is based on the 1984 U.S. Army Corps of Engineers (USACE) Shore Protection Manual (SPM 1984) for irregular waves on an open coast and for planar beach profiles (uniform slopes) and does not address many settings related to FEMA's responsibilities. The recent USACE Coastal Engineering Manual (CEM), which replaces the SPM, provides guidance for both regular and irregular waves. The CEM results for irregular waves are presented in graphical form and do not extend to the shoreline; however, if these results are extrapolated to the shoreline for comparison with the current guidance (SPM), the CEM wave setup values are consistently higher than the SPM values. Two common beach slopes are presented in SPM and CEM: for the 1:30 slopes, the CEM values are approximately 1.6 times (60% higher than) the SPM values and for the 1:100 slope, the CEM values are approximately a factor of 1.4 times (40% higher than) the SPM values.

Of the coastal counties where FIS studies have been conducted, approximately 40% have included wave setup in the 100-year FIS elevations. Those counties that have included wave setup in the 100-year elevations are predominantly those that were conducted in recent years and/or those that have been restudied after elevations were judged to be too low, in some cases based on high water marks or other data following major storms. For those counties where setup has been included, the methodologies employed have not been entirely consistent, but have relied predominantly on guidance provided by the USACE through various editions of the SPM. In addition to establishing a consistent procedure to be applied at the coast, the issue of wave setup variation over inland flooded areas is of concern and is not addressed in the SPM guidance. Updates of the FIRM's to include wave setup (i.e., increase flood levels) have led to expensive and counterproductive appeals. Two examples of such appeals have been in Pinellas County and Collier County, Florida, where much of the concern was focused on the incorporation of wave setup. Thus it is considered essential to establish a consistent methodology for all calculations of wave setup with as much adherence to the physics of the system as possible.

# 2 CRITICAL TOPICS

As noted, the December 2003 Workshop identified three "Critical Topics" on wave setup: 1) "Better define and document; summarize what to consider and how to approach; data requirements (Topic 44)"; 2) "Compile example/data sets to perform tests (Topic 45)"; and 3) "Develop interim method (look at CEM as a fall back, or University of Hawaii SPM procedure) (Topic 46)." "Critical Topics" are those that could be accomplished within six months. All three of the critical Wave Setup Topics apply to the three geographic areas defined: 1) Atlantic/Gulf Coasts, 2) Pacific Coast, and 3) Sheltered Waters.

## 2.1 TOPIC 44: BETTER DEFINE AND DOCUMENT; SUMMARIZE WHAT TO CONSIDER AND HOW TO APPROACH; DATA REQUIREMENTS

### 2.1.1 Definitions

Wave setup is the increase in mean water level above the stillwater level (defined as including the effects of all other forcing except wave setup) due to momentum transfer to the water column by waves that are breaking or otherwise dissipating their energy, see Figure 2. Wave setup is the

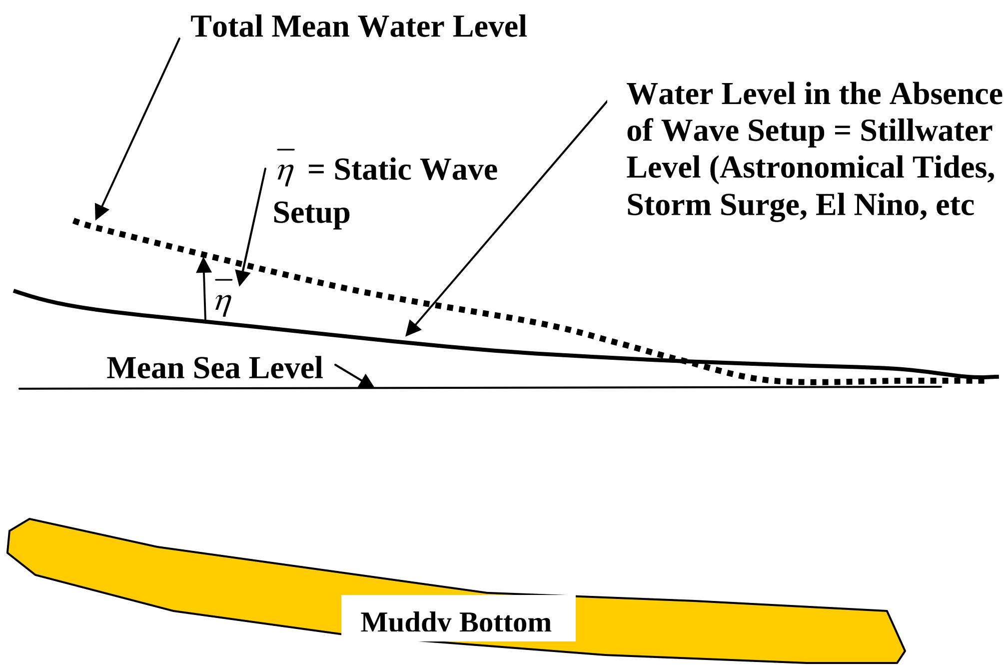



**Figure 2. Definition Sketch for Wave Setup**

increase in water level with periods ranging from several to tens of periods of the dominant incident wind wave period. A typical wind wave period is in the range of 8 to 15 seconds.

Wave setup is a component in wave runup in the same manner as the wind and barometric components of the storm surge are components in wave runup. In those portions of the nearshore zone where water is always present, the definition of wave setup is simpler than in the runup zone that is alternately wet and dry. In locations where water is always present, wave setup is the deviation of the mean water level from the stillwater level (SWL). The SWL is defined as the water level in the absence of waves but with all other processes present.

Wave setup includes a static component and a dynamic component with the dynamic component varying much more slowly than the dominant wave period. Figure 3 is a sketch illustrating these components.

A challenge in this and the wave runup issues will be to ensure that the effect of wave setup is not “double counted”, i.e., not included twice because the wave setup is included to some degree in wave runup measurements. A useful and practical working definition distinguishing wave setup from wave runup elevations is: "Wave setup contributes to high water marks inside reasonably small buildings; however, wave runup does not." A second challenge is the development of an acceptable method to predict the inland excursion of the steady and dynamic wave setup components.

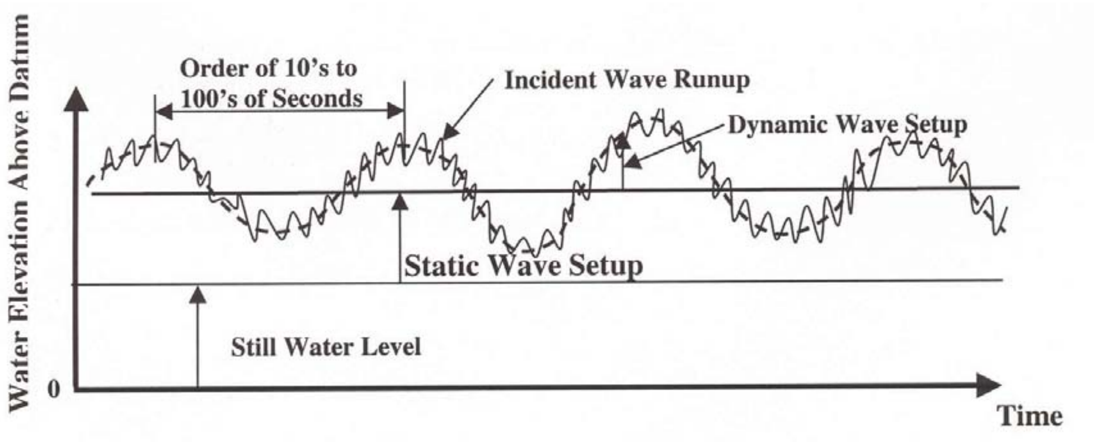



**Figure 3. Definitions of static and dynamic wave setup components.**

### 2.1.2 Physiographic Settings

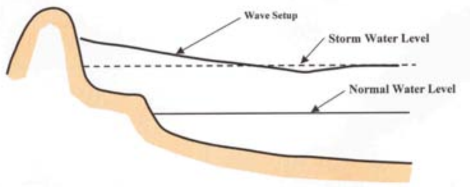
Wave setup can occur in a variety of physiographic settings that are relevant to FEMA's flooding responsibilities. Eight such settings have been identified and are shown in Figure 4. The mechanics of wave setup in some of these settings may be similar or identical; however, the range of possible settings is included here for completeness.

### 2.1.3 Considerations and Approaches

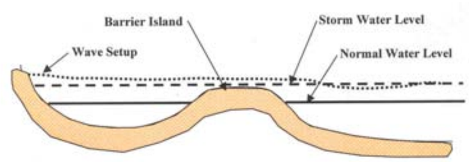
As the NFIP Program matures, it is clear that the programs and procedures employed will to be more complete and represent the physics more effectively. This is also the case for wave setup. The systems of interest are three dimensional and complex and it is believed that the next generation of models and procedures will be able to consider the physical system and forcing more completely and realistically. If this is correct, the problem of predicting realistic values of wave setup will be on a much more solid footing and should minimize future appeals based on considerations of out-of-date methodology. It is anticipated that the next generation of models will still require some empiricism and ad hoc approaches; however, artificialities will be reduced considerably relative to present methodology.

The physics of the static wave setup component are reasonably well understood and governed by the following equation

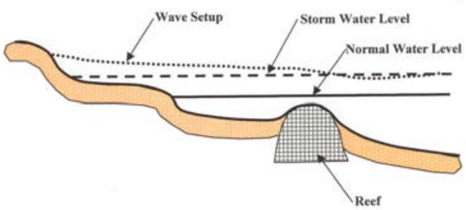
$$\frac{\partial \bar{\eta}}{\partial x} = \frac{1}{\rho g (h+\bar{\eta})}\left(-\frac{\partial S_{xx}}{\partial x} + \tau_b\right) \qquad (1)$$



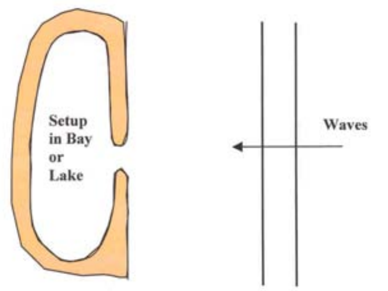

**Setting 1. Long Straight Beach.**



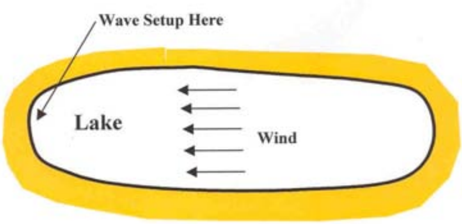

**Setting 2. Flooded Barrier Island.**



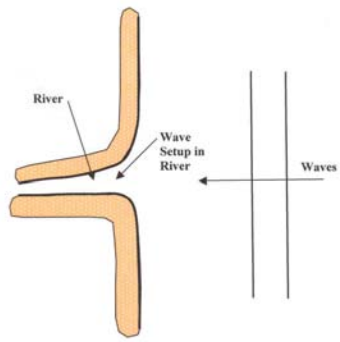

**Setting 3. Fringing Reef.**



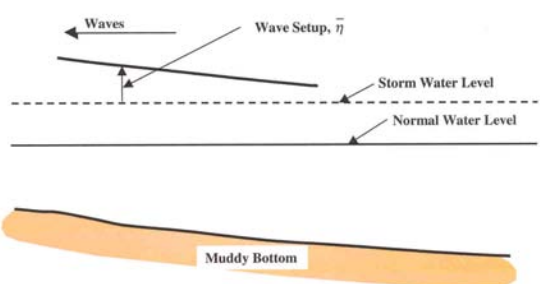

**Setting 4. Wave Setup in Lake or Bay.**



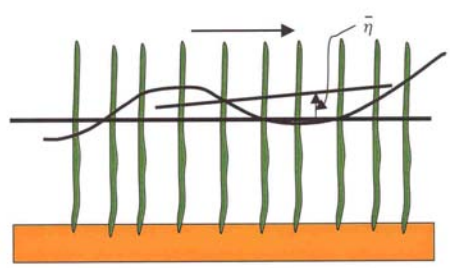

**Setting 5. Wave Setup in Enclosed Water Body.**




**Setting 6. Wave Setup in River.**




**Setting 7. Wave Setup Over Muddy Bottom.**




**Setting 8. Wave Setup Through Vegetation.**

**Figure 4. Eight wave setup settings relevant to FEMA's responsibilities.**

in which $\overline{\eta}$ is the steady state component of the wave setup, *x* is the shoreward directed axis, $\rho$ is the mass density of water, *g* is gravity, *h* is the stillwater depth, $S_{xx}$ is the flux of momentum in the direction of wave propagation, and $\tau_b$ is the bottom friction. The $S_{xx}$ term is defined as

$$S_{xx} = \overline{\int_{-h}^{\eta} (p + \rho u^2) dz} \tag{2}$$

where *p* is wave related pressure, *u* is the horizontal component of the wave related water particle velocity, $\eta$ is the instantaneous water surface elevation relative to the stillwater level, *z* is the vertical coordinate directed upward with its origin at the stillwater level, and the overbar indicates averaging over the wind wave period. The quantity $S_{xx}$ can be calculated readily for linear waves; however, as will be demonstrated, nonlinearities must be taken into consideration and can result in significantly smaller values of $S_{xx}$ than those based on linear wave theory for the same wave height. In the very nearshore (surf zone), the wave propagation direction will be nearly shore normal. But there may be regions where the wave direction and the normal to the bathymetry are not in line. In this case, the momentum stress tensor must be corrected for the relative angle.

The term $(h + \overline{\eta})$ in the denominator of Eq. (1) is relevant as it indicates that a rational wave setup model will require an appropriate wave breaking model and use of valid nearshore bathymetry rather than the assumption that the waves are depth limited. In summary, referring to Eq. (1), momentum transfer ( $-\frac{\partial S_{xx}}{\partial x}$ ) in deeper water will cause less tilt of the water surface and since wave breaking (which governs $\frac{\partial S_{xx}}{\partial x}$ ) depends on the bathymetry, both wave breaking modeling and valid bathymetry will be required. Furthermore, the fact that the waves do not have infinite crest lengths implies that the momentum fluxes are not unidirectional. Also, spatial variations can result from multiple wave trains incident simultaneously from different directions.

### 2.1.4 Data Requirements

As noted above, improvements to this topic will derive primarily due to approaches that are more comprehensive and more inclusive of the relevant physics. At present, a fairly large number of laboratory experiments on wave setup have been conducted and several field experiments have been carried out for the express purpose of investigating wave setup. However, considerable questions remain in interpreting some of the results, especially the field data in which similar approaches have yielded substantially different quantitative results. It is noted here that establishment of the offshore (still) water level is quite difficult in most field experiments which may account for some of the differences since the wave setup is relative to the stillwater level. There are several cases in which wave setup has been identified in the field in what may be called “experiments of opportunity”, i.e., the setup appeared in either tide gage readings or high water marks. These are of direct interest to FEMA as they are usually associated with severe storm events.

It will be necessary to summarize and interpret the data (a partial such effort is included in the wave setup supporting documentation developed as part of this effort) and to locate and analyze other related data. Undoubtedly additional relevant data are available that have not been identified during this relatively brief effort, especially internationally. It is believed that an effort directed to glean wave setup information from existing tide gage and high water mark information would be fruitful. Also, a more thorough analysis of the existing experimental results (laboratory and field) may provide further quantified understanding of these results and clarify significant relationships, for example wave setup in the runup region.

Finally, it is possible that, after completion of the efforts above, additional laboratory and/or field efforts will be warranted. If this is the case, the details of these recommended efforts will be established.

Table 1 at the end of this report contains a summary of the key findings and recommendations for Topic 44. Table 2 at the end of this report presents estimates of times required to accomplish the various tasks in this topic.

## 2.2 TOPIC 45: COMPILE EXAMPLE/DATA SETS TO PERFORM TESTS

### 2.2.1 Compilation of Example/Data Sets

The compilation of data sets has been discussed in Critical Topic 1 under 2.1.4, Data Requirements, and will be addressed here only briefly. It appears that a sufficiently large unexplored data base on wave setup exists and could assist in shaping the next generation of wave setup models. Additionally, the capability of the new generation of wave models in addressing the dynamic wave setup component should be useful.

Table 1 at the end of this report contains a summary of the key findings and recommendations for Topic 45. Table 2 at the end of this report presents estimates of times required to accomplish the various tasks in this topic.

## 2.3 TOPIC 46: DEVELOP INTERIM METHOD FOR CALCULATING WAVE SETUP

### 2.3.1 General

The current FEMA guidelines for calculating wave setup have been discussed earlier in this document. This guidance is based on planar beaches (i.e., uniform slopes) and does not recognize the nonlinear effects that can be significant to the quantification of $S_{xx}$ at breaking. Additionally, current guidance does not address the dynamic wave setup component that is relevant to beach erosion and other processes, especially on the Pacific Coast. The Coastal Engineering Manual (CEM) treatment of wave setup has been reviewed and compared with the current guidance and it is recommended that the current guidance be retained until an alternate interim method is developed.

It is recommended that an interim methodology account for the following: 1) Steady and dynamic wave setup components, 2) Irregular waves (implicit in (1) above), 3) Characterization of nearshore bathymetry, 4) A valid wave breaking model, 5) Nonlinearities in $S_{xx}$, and 6) Wave damping seaward of the breaking zone where appropriate. Our assessment is that the required information is available to accomplish these objectives within the time frame of six months for the most common physiographic settings of concern (Figure 4). It is anticipated that the interim methodology will be applicable to two-dimensional situations and will apply reasonably well to Settings 1, 2, 3, 5, and 8. Because of the different causes of flooding and wave setup on the Pacific Coast and Sheltered Waters (P&SW), and the Gulf and Atlantic (G&A) Coasts, the interim methods will likely be different and are presented separately in the following sections. The common elements of the two interim methods occur landward of the breaking locations. Thus, the following sections present likely procedures for the Pacific Coast and Sheltered Waters and Gulf and Atlantic Coasts separately followed by a discussion of the common elements.

### 2.3.2 Possible Interim Methodologies

#### *Seaward of Breaking Region*

**Possible Interim Method for the Pacific Coast and Sheltered Waters**

The deliberations of FEMA Workshop 2 (February 2004) established that the wave input to the Pacific Coast flooding studies will likely be the Global Reanalysis of Ocean Waves (GROW) data available from Oceanweather, Inc. These data are available commercially and represent the results of reanalysis of wind fields and wave prediction and are available at a spacing of approximately 40 nautical miles along the Pacific Coast. The information contained in these data sets is assumed to include directional wave spectra. In application to the computation of coastal flooding, these spectra and the astronomical tides are expected to serve as the primary input to the calculations.

For wave setup and wave runup, the GROW wave characteristics may be transformed to the breaking zone accounting for refraction, shoaling, and energy dissipation caused by bottom friction. This will be accomplished by the Wave Transformation Study (Topics 7–10) efforts and will not be discussed further here. As noted previously, within the breaking zone, a wave breaking model will be used to establish the wave height characteristics and to provide the basis for integration of the wave setup equation. The procedures within the surf zone are common to all coastlines and will be discussed separately.

**Wave Prediction and A Possible Interim Method for the Gulf and Atlantic Coasts**

It is unlikely that an interim methodology will include a combination of a storm surge model and a wave calculation capability. However, all storm surge models include a wind field model. It is envisioned that the available winds could provide reasonable estimates of waves. The method would require testing to ensure its reasonableness for wave setup purposes. A potential method is described below and outlined in the flow chart in Figure 5.

As noted, all storm surge models include a wind model for forcing; however, none of which we are aware include direct wave calculations, although efforts are underway to accomplish this objective. Since wave setup requires waves as input, a parameterization of a hurricane wave field originally developed by Bretschneider (1972) can be applied. This relationship is illustrated in Figure 6.

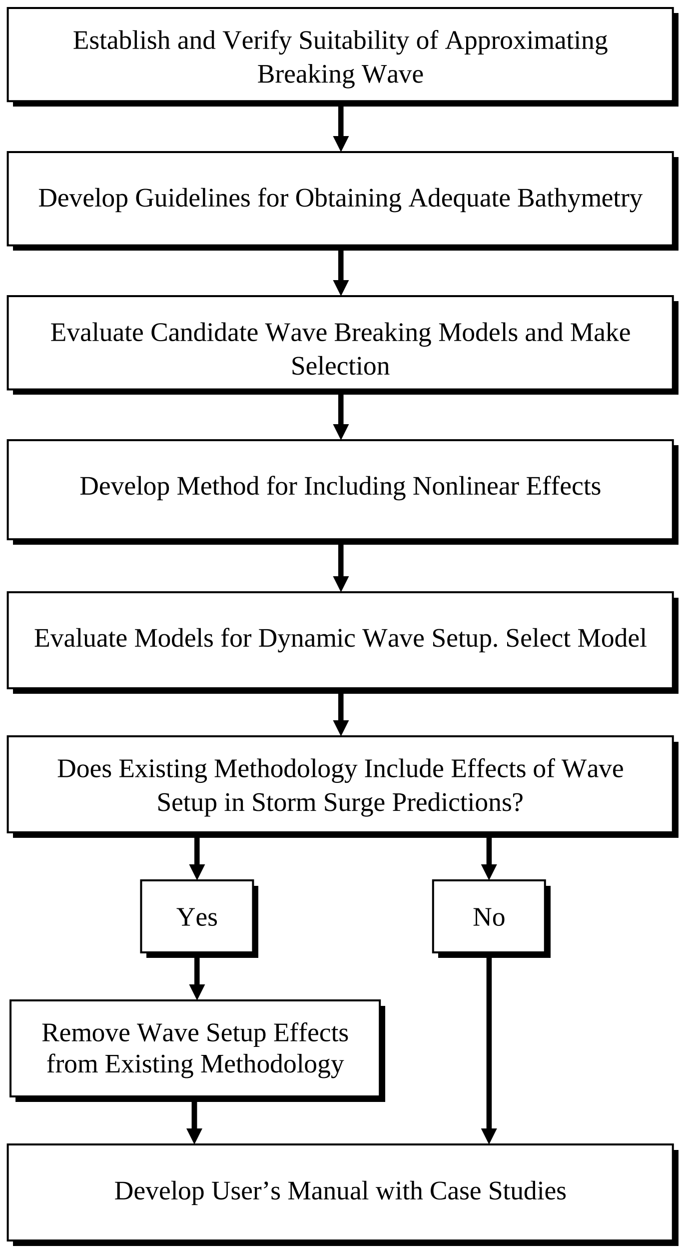



**Figure 5. Flow chart for development of interim wave setup methodology.**

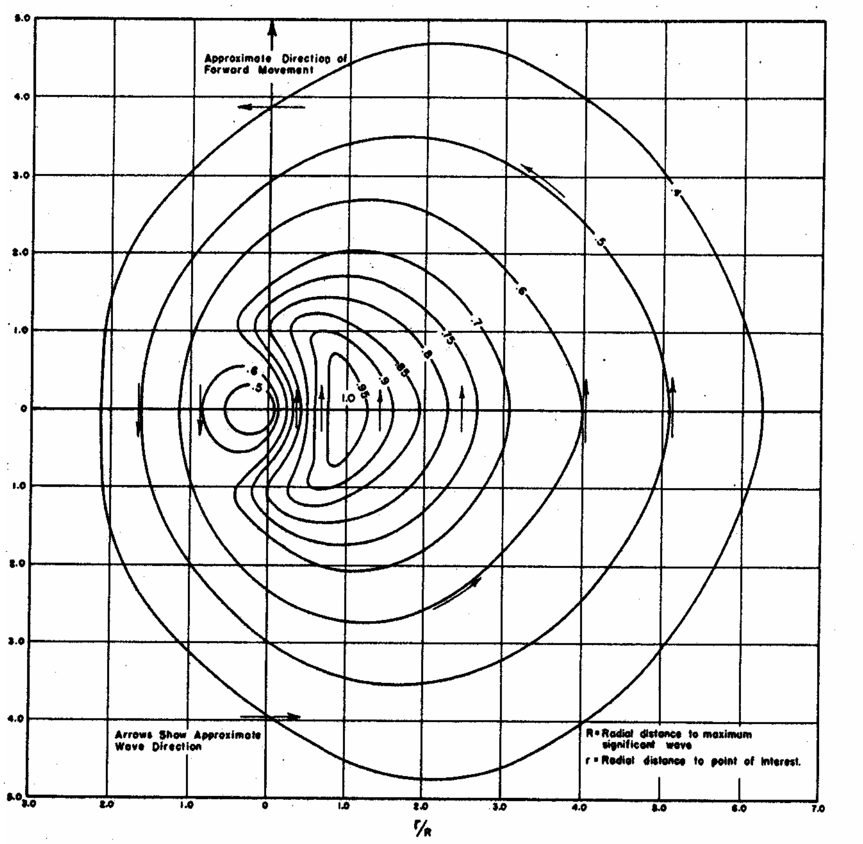



**Figure 6. Non-dimensional wave height field (from Bretschneider, 1972)**

The significant wave height and associated period at the location of maximum winds are described by

$$H_o = 16.5 e^{\frac{R\Delta p}{100}} \left[ 1 + \frac{0.208 \alpha V_F}{\sqrt{U_R}} \right] \tag{3}$$

and

$$T_s = 8.6 e^{\frac{R\Delta p}{200}} \left[ 1 + \frac{0.104 \alpha V_F}{\sqrt{U_R}} \right] \tag{4}$$

where the units of $H_o$ and $T_s$ are feet and seconds, respectively; and $R$ is the radius to maximum winds in nautical miles, $\Delta p$ is the central pressure deficit in inches of mercury, $V_F$ is the forward translational speed of the hurricane in knots, $U_R$ is the maximum sustained wind speed in knots, calculated at 30 feet above the mean sea surface at radius $R$, where

$$U_R = 0.865 U_{\max} + 0.5 V_F \tag{5}$$

and $U_{max}$ is the maximum gradient wind speed in knots at 30 feet above the water surface. Finally, the parameter α is a coefficient that depends on the forward speed of the hurricane. For slowly moving hurricanes, the suggested value of α is 1.0. In Figure 5, the horizontal and vertical axes are non-dimensionalized by the radius to maximum winds, *R*.

Thus, with Equations (3) and (4), for the maximum significant wave height and the results in Figure 5, it would be possible to calculate the wave height at any location of interest and this may be a useful approach. Alternately, the wave height at any location of interest, $H_*$, can be approximated by the following:

$$H_* = H_{\max} \frac{U_*^2}{U_{\max}^2} \qquad (6)$$

where $U_*$ is the wind speed at the location of interest and $U_{\max}$ is the maximum wind speed in the hurricane wind field. The square relationship in Eq. (6) is consistent with the physics governing wave generation by wind, basically the Froude relationship.

Eq. (6) is applicable for deep water conditions. A method needs to be developed and incorporated to account for refraction, shoaling and wave damping that would occur across broad continental shelves. It is recommended that damping be based on a reasonable friction factor and the geometric characteristics of the shelf profile. It is likely that a set of curves and/or empirical equations could be developed to represent several characteristic shelf widths, etc.

**Sheltered Waters**

For purposes here, it is considered that the Storm Wave Characteristics efforts (Topics 1–5) and Wave Transformation efforts (Topics 7–10) will provide a basis for developing wave spectra outside the breaking zone for sheltered waters.

### *Interim Methodology Common to the Pacific Coast and Sheltered Waters and Gulf and Atlantic Coasts Landward of Breaking*

Two interim methods will be described. Method 1 is more of a parameterized method based on as much proven engineering methodology as is available. Method 2 would apply advanced numerical Boussinesq wave models that have found applicability in the surf and swash (runup) regions. Because of the present uncertainty regarding the applicability of these more physics-based models to FEMA issues, the first phase of the interim method effort would be an evaluation of these models to establish whether or not they are capable of providing suitable estimates of static and dynamic wave setup for applications of interest here.

**Method 1: Based on Proven Engineering Methodology**

**Static Wave Setup Component**

The components of the interim methodology that are common to all coastlines commence at a nearshore reference depth outside the breaking zone. As noted previously, the nearshore wave information will be a product of the Wave Transformation effort. At this stage, it appears that the directional wave spectra provided at the nearshore reference depth will be a result of linear superposition. Since infragravity (nonlinear) waves can be significant to wave setup, runup, and beach erosion, especially on the Pacific Coast, the possibility of adding these infragravity components to the linear spectrum will be explored. Following this, a realistic wave transformation model that accounts for the particular characteristics of the nearshore profile will be applied to represent the wave characteristics as the waves propagate toward shore and through the surf zone. The $S_{xx}$ term and other momentum flux terms will be calculated and the wave setup equation (Eq. (1)) integrated to determine $\bar{\eta}$ according to the particular setting. It is likely that a "WHAFIS-like" computer program will be developed to carry out calculations from the seaward location of nearshore data wave storage (again, directional spectra, a product of the Wave Transformation effort) to wave setup and runup.

**Dynamic Wave Setup Component**

Two rather direct procedures have been established to account for the dynamic wave setup. The method of Lo (1981) is to augment the static setup, $\bar{\eta}$, associated with the significant wave height by 50% (with possibly a reduction factor to account for two-dimensional effects).

A second approach to the dynamic wave setup would be to utilize the expression of Goda (1985)

$$\eta_{rms} = \frac{0.01 H_o}{\sqrt{\frac{H_o}{L_o}\left(1 + \frac{h}{H_o}\right)}} \tag{7}$$

where h is the water depth at any location in the surf zone. The methods of Lo and Goda have been compared for one case and have been shown to yield reasonably similar results. Thus, either (or both) of these two approaches would appear to be appropriate for an interim methodology.

A third possible approach (discussed in more detail in Method 2 below) to predicting dynamic wave setup would be to utilize one of the more physics-based wave models (such as a Boussinesq model) that can represent both the static and dynamic components of wave setup and runup. Through exercising the model for a range of conditions, it could be possible to develop guidelines for the dynamic (and/or static) component of wave setup. This approach could facilitate exploration of the effect of wave "groupiness" on wave setup. Informal observations support that setup is dependent on the time series of breaking waves, including the grouping of larger waves. Therefore, very groupy wave trains may have relatively low static setups but large dynamic setups. Model runs using measured wave time series with different groupiness levels may yield results that could be used to develop a simplified procedure for Pacific Coast, large swell conditions.

**Method 2: Based on Advanced Numerical Models**

Advanced numerical models have been developed over the last several decades and have found applicability in the surf and runup zones (Madsen, et al., 1997a, 1997b; Sorensen, et al., 1998; Kennedy, et al., 2000). With specification of a directional spectrum seaward of the breaking zone as input, these models can calculate both the static and dynamic setup and runup; however, to the best of our knowledge, these models have not been applied or evaluated for purposes of addressing issues within the purview of FEMA's responsibilities. Therefore, the first phase of the interim methodology will be the evaluation of the applicability of these advanced models to provide suitable predictions of static and dynamic wave setup. This will be based on comparisons of predictions with measurements. If this method is successful, a separate wave breaking wave model would not be required.

### Beach Profile Representation

Regardless of which of the two methodologies is selected for development of an interim methodology, beach profiles will be required. Under a flooding scenario, the profiles of interest will include those contours that are normally above water. which, for purposes here will be assumed to be reasonably well known. As noted, most of the wave setup results for which beach profiles are taken into account are for the case of uniform beach slopes. However, beach profiles in nature tend to be concave upwards and may include bar features. In some areas of application, reasonably good information describing beach profiles will be available whereas in others there may be only limited data. In the absence of any quality beach profile data, it is suggested that some nearshore profiles be surveyed and correlated with Equilibrium Beach Profile (EBP) theory (e.g., Dean and Dalrymple, 2000) to determine whether EBP theory is adequate for wave setup calculations.

EBP theory considers the beach profile to be described by

$$h(y) = Ay^{2/3} \tag{8}$$

in which $h$ is the stillwater depth under normal conditions (say, relative to NGVD) at a seaward distance, $y$, from the normal shoreline and $A$ is a dimensional parameter (units of length$^{1/3}$ termed a "Profile Scale Parameter") which depends on sediment size. The profile predicted by Eq. (8) is concave upwards and is monotonic. The value of $A$ for most Florida profiles is on the order of 0.1 m$^{1/3}$ (0.15 ft$^{1/3}$), a value that corresponds to a mean sediment size of approximately 0.2 mm.

To summarize, there are several approaches by which beach profile information can be developed for a particular application.

### Wave Breaking Model

As noted, improved models will be required to provide a realistic basis for wave breaking which governs the transfer of wave related momentum to the water column. Candidate wave breaking models include those by Goda, 1985; Guza and Thornton, 1981; Battjes and Jannsen, 1978;

Svendsen, 1984; and Dally, Dean and Dalrymple, 1985. An advantage of the latter model (termed the $D^3$ model) is that the same quantity (wave energy) that governs the wave momentum flux is modeled directly. Also, this model predicts, in accordance with observations, that initially breaking waves propagating over a horizontal bottom will approach an equilibrium wave height after which they will become stable (non-breaking). This feature has advantages for profiles in which a longshore bar and landward bar system is present.

In summary of this issue, the manner in which waves break and thus momentum transferred is important to obtaining the correct wave setup. Several models are available which predict much more realistic wave breaking than the commonly applied model in which the wave height is assumed proportional to the local total water depth.

### *Nonlinear Effects on $S_{xx}$ at Breaking*

Breaking waves tend to be quite nonlinear at breaking with peaked crest regions and broad flat troughs. Associated with this nonlinear feature is a momentum flux ($S_{xx}$) which is considerably smaller than that predicted by linear breaking waves. Figure 7 presents, for periodic waves, the ratio of nonlinear to linear $S_{xx}$ at breaking versus relative water depth, $h/L_o$.

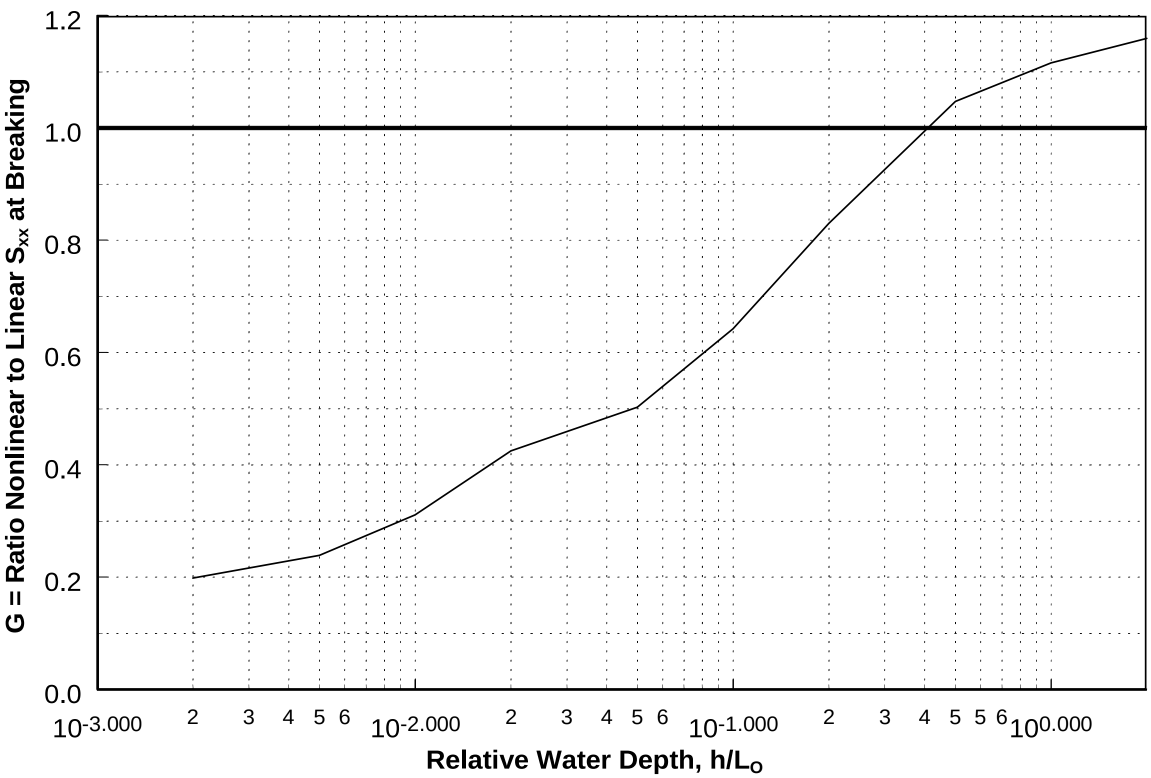



**Figure 7. Relationship of nonlinear to linear $S_{xx}$ at breaking versus relative water depth, $h/l_o$. based on stream function wave theory.**

These results are based on stream function wave theory; however, other valid nonlinear theories exist. Clearly the magnitude of this effect justifies accounting for these nonlinearities in the quantification of $S_{xx}$.

In summary of the nonlinear $S_{xx}$ issue, the effects of nonlinearities warrant their inclusion in a design methodology and several wave theories exist which can provide realistic results for the modification of $S_{xx}$ at breaking.

Table 1 at the end of this report contains a summary of the key findings and recommendations for Topic 46. Table 2 at the end of this report presents estimates of times required to accomplish the various tasks in this topic.

# 3 AVAILABLE TOPICS

As noted in the Introduction, initially the Wave Setup topics included one “Available Topic”– “Topic 49: Review WRUP, available equation based program for wave run-up”. This topic was reassigned to the Runup and Overtopping Focused Study.

# 4 IMPORTANT TOPICS

There were two “Important” Topics in the wave setup category: (1) Topic 47: Develop Ideal Method Coupled With Storm Surge and Waves to Develop Setup, and (2) Topic 48: Develop Procedures for Dynamic Wave Setup. Each of these is discussed below.

## 4.1 TOPIC 47: DEVELOP IDEAL METHOD COUPLED WITH STORM SURGE AND WAVES TO DEVELOP SETUP

### 4.1.1 General

The so-called “Ideal Method” will be one in which the wave setup calculations are integrated into the storm surge model, requiring that the storm surge model also include the capability to compute or access wind fields and calculate the spatial and temporal distributions of waves. This so-called integrated model would include “wetting and drying” capabilities available in many advanced models and would have the capability to calculate realistic values of bottom friction coefficients. The model will also represent three dimensional features such as inlets, flows over barrier islands, and the gradients of the storm surge field due to the limited lateral dimension of the hurricane.

Some of these features are now represented in available storm surge models. The previously discussed nonlinear effects on $S_{xx}$ could be represented by a subroutine that runs a nonlinear model such as a Boussinesq (see earlier references) or other model to evaluate $S_{xx}$ (and potentially the other momentum flux terms) at breaking for the particular wave conditions. A practical difficulty in the direct application of the momentum flux contributions in long wave models is that the nearshore grid would need to be extremely small in order to resolve the breaker zone because the setup is a function of the gradients of radiation stresses, which could require grid resolution on the order of 10 m. An alternate approach would be to have look up tables based on the stream function or other nonlinear wave theory providing information similar to that presented in Figure 7. In this approach, wave setup could be computed external to the hydrodynamic model and either added linearly to the stillwater elevation or ideally included as a stress gradient in the hydrodynamic forcing. The first option would not require detailed nearshore resolution; however, the second option probably would.

Several groups are now actively pursuing the addition of a wave setup capability to the long wave model ADCIRC. These groups include the U. S. Army Engineer Research and Development Center (ERDC), (formerly the Waterways Experiment Station) and the National Oceanographic Partnership Program (NOPP). If one or both of these efforts are successful, it is possible that little additional work will be required for a portion of this topic. However, realistically, the models established for other than FEMA's applications probably will require further development for FEMA's specific purposes. Reasons include the need to retain as much of the governing physics as possible in the models and to ensure that the models are robust and can be applied over a wide range of physical settings by non-model specialists while still providing reasonably correct results. Thus, it is probably both realistic and prudent to consider the requirement of a considerable amount of development and testing over a wide range of conditions relevant to FEMA's responsibilities. The latter would naturally lead to the development of a User's Manual that would include results and guidance for a wide range of coastal settings (Figure 4).

Table 1 at the end of this report contains a summary of the key findings and recommendations for Topic 47. Table 2 at the end of this report presents estimates of times required to accomplish the various tasks in this topic.

## 4.2 TOPIC 48: DEVELOP PROCEDURES FOR DYNAMIC WAVE SETUP

### 4.2.1 General

The dynamic component of wave setup is a result of groups of waves that cause a variable setup/setdown in the offshore region and the further wave setup generation in the surf zone. Wave groups are more prominent for narrow energy spectra in the frequency domain with a narrow directional spread. According to some of the analytical and numerical models that have been developed to investigate wave setup oscillations induced in the surf zone, it appears possible that a type of resonance may occur further enhancing the dynamic wave setup. The so-called "sneaker waves" may be the result of two energetic spectral components propagating in almost precisely the same wave direction. A slight difference in wave direction causes a significant "detuning" away from resonance to the propagating forced wave.

In view of the above, a rational approach to the calculation of the dynamic component of wave setup would require a detailed description of the incident wave spectrum, including the directional and nonlinear wave characteristics. Recognizing the uncertain paths available for this topic and questions regarding the most appropriate pathway, a two-stage effort is proposed: 1) The first stage would be exploratory and would establish whether a rational approach or one or more ad hoc approaches is most suitable. The decision of whether or not a rational approach is feasible will depend on the prognosis for the required models being available within the next few years, and 2) A second phase to pursue the approach identified in the first phase. Each of these phases is discussed below and the overall effort is depicted in the flow chart below (Figure 8).

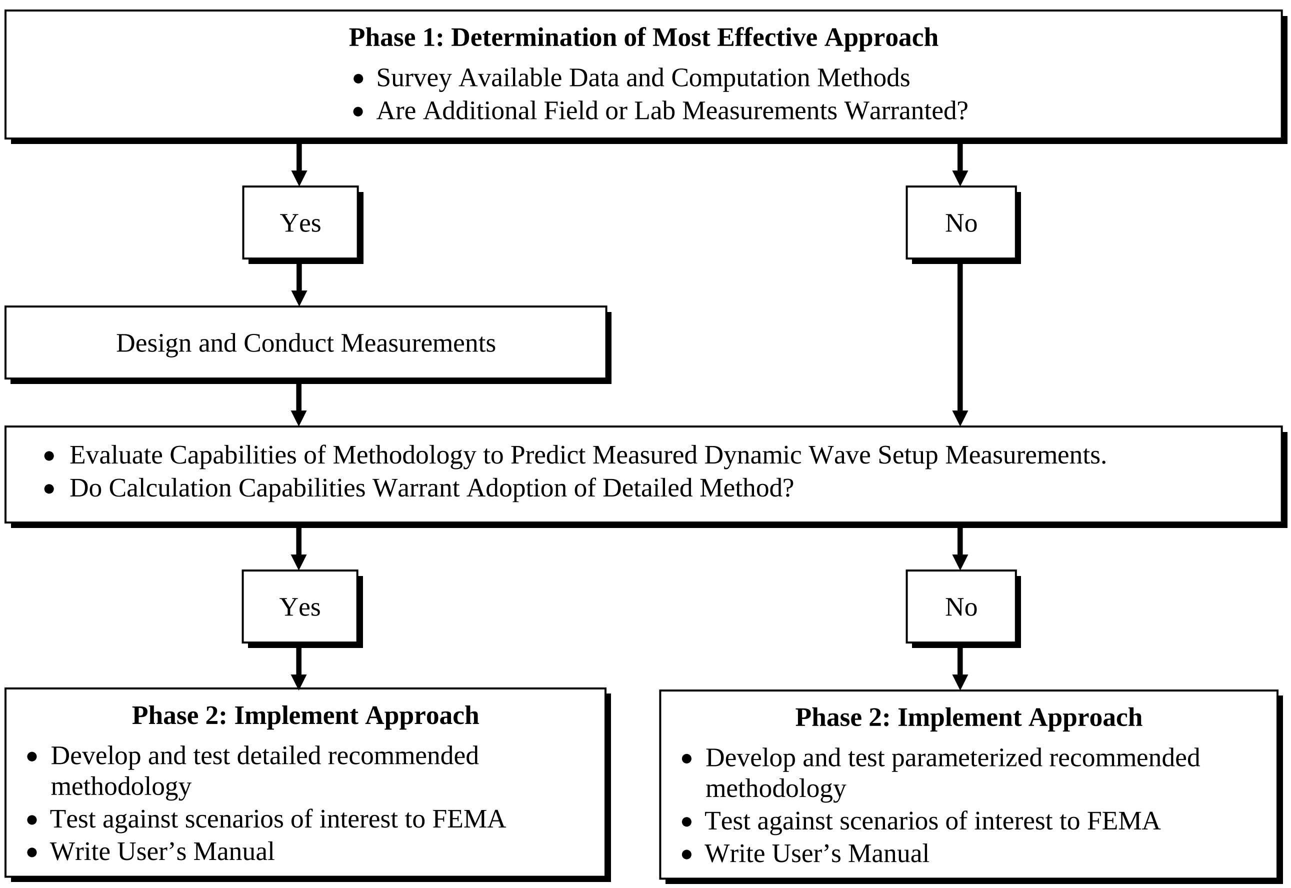



**Figure 8. Flow Chart: Process of determining methodology for dynamic wave setup.**

#### 4.2.2 Phase 1. Determination of Most Effective Approach for Representing Dynamic Wave Setup

This phase of the task would be exploratory and would include establishment of existing documentation and development and comparison with calculation procedures. Each of these is discussed below.

The literature and available data relating to dynamic wave setup would be reviewed to identify data for further examination and methodologies for calculating wave setup. Argus camera systems (Holman and others) have been installed in a number of locations in the world and contain dynamic wave setup and dynamic wave runup information. Also, the field studies by Guza and Thornton (1981) that were carried out with a runup wire parallel to and a few centimeters above the beach surface contain dynamic setup information if the raw data are still available. Selection of data for further examination should include a preference for field situations in which quality offshore wave data are available. Efforts should be made to locate international sources of quality dynamic wave setup data. For example, Goda (1975) has published guidance for calculating dynamic wave setup and may have valuable data sets. The studies of Nielsen and colleagues, while not conducted for the purpose of measuring dynamic wave setup (the dynamic component was purposely averaged from the data), contain a lower limit of dynamic setup that may be useful for checking. Several authors (Schaffer and Svendsen, 1988; Schaffer and Jonsson, 1990; Symonds, Huntley and Bowen, 1982; and others) have presented methodologies for calculating long period waves in the surf zone resulting from wave groupiness. Additional laboratory and/or field experiments designed to address FEMA's responsibilities may be warranted and recommended. Additionally, as discussed earlier, the detailed Boussinesq wave models (see earlier references) that have been developed during the last few decades may be suitable for predicting wave setup and wave runup.

The second phase of the effort is to assess the available data and the capabilities of the existing computational methodologies to be evaluated by comparing predictions with available data and to decide on a procedure for proceeding toward an adopted methodology. The review here identified only two existing readily applied approaches for predicting dynamic wave setup (Goda, 1985; Lo, 1982). Advantages of developing a methodology based on detailed representations of the forcing spectrum will be based on the availability and/or prognosis of the development of such information.

#### 4.2.3 Develop Selected Approach for Application

At this stage of the effort, it is considered that a decision will have been made to adopt either a detailed methodology or a parameterized approach for calculating dynamic wave setup. Subsequent efforts will include development and testing the recommended methodology against scenarios of interest to FEMA's flooding responsibilities and the writing of a User's Manual.

Table 1 at the end of this report contains a summary of the key findings and recommendations for Topic 48. Table 2 at the end of this report presents estimates of times required to accomplish the various tasks in this topic.

# 5 ADDITIONAL OBSERVATIONS

Although the underlying physics of wave setup is well understood, current guidance relating to the calculation of wave setup for the wide range of settings within FEMA's area of responsibility (Figure 3) is lacking. With the emphasis on the nearshore region over the last three decades or so, the capability to improve current guidance is substantial.

Two general methods are considered, either of which would represent a significant advancement: 1) Use of available and proven engineering procedures, and 2) Use of advanced numerical wave models (ANWM), in particular the Boussinesq models. The first method is definitely possible and can be packaged to be applied by a Study Contractor (SC). A question exists as to whether the advanced wave models can be applied by a SC over a broad scale of settings and wave and nearshore geometries. Further, some of these ANWMs are proprietary, they are computationally intensive, advancing rapidly and undoubtedly their capabilities will be greater in a decade than at present. Finally, even if a decision is made to progress with an ANWM which would be run by a SC as a "black box", it would be desirable that the SC have a less computationally intensive procedure as a general check. On the other side, the potential (present?) capabilities of the ANWM are very attractive, being able to predict both wave setup and wave runup without concern if wave setup is included twice in wave runup.

Regardless of the method adopted, a significant effort will be completed in a search for high-quality wave setup data with an emphasis on field data. It is expected that some of the more valuable data will be based on carefully documented high water marks during extreme events which are conditions of special concern to FEMA.

# 6 SUMMARY

## 6.1 CATEGORY SUMMARY

The Wave Setup Focused Study Group on was tasked with identifying programs that would lead to state-of-the-art improved capabilities of Study Contractors to better accomplish FEMA's responsibilities in establishing hazard zones. These tasks were organized in six topics with one topic later transferred to the Wave Runup and Overtopping Focused Study Group. Of the five remaining topics, three were listed as "critical" and two were "important". All five were considered of concern to the Atlantic and Gulf coats, Pacific and Sheltered coasts. The alternatives above were discussed at Workshop 2 in Sacramento in February 2004, and recommendations developed based on the consensus of the Technical Working Group.

## 6.2 SUMMARY TABLES

**Table 1. Summary of Findings and Recommendations for Wave Setup**

| Topic Number | Topic | Coastal Area | Priority Class | Availability/ Adequacy | Recommended Approach | Related Topics |
|---|---|---|---|---|---|---|
| 44 & 45 | Define, Document, Compile Data | AC | C | MAJ | The recommended approach for this topic is the same for all geographic regions: Conduct a thorough examination of all available relevant literature with an emphasis on quality field data sets. These would include experiments conducted especially to investigate wave setup and especially "experiments of opportunity" in major storms including high water marks. Organize data by "settings" identified in this report. | 11 |
| | | GC | C | MAJ | | |
| | | PC | C | MAJ | | |
| | | SW | C | MAJ | | |
| 46 | Interim Method | AC | C | MAJ | Several possibilities exist. The "Interim Method" should include consideration of the following: (1) static and dynamic setup, (2) irregular waves (implicit in (1) above). (3) characterization of nearshore bathymetry, (4) a valid wave breaking model, (5) nonlinearities in $S_{xx}$, and (6) wave damping where appropriate. An attempt should be made to ensure that the interim method address as many of the settings identified as possible | 1, 6, 9 |
| | | GC | C | MAJ | | |
| | | PC | C | MAJ | | |
| | | SW | C | MAJ | | |
| 47 | Develop Ideal Method - Coupled | AC | I | PRODAT | The recommended approach for this topic is the same for all geographic regions. The ideal method would be one in which the storm surge model also incorporates a wave generation model. The wave generation model would predict directional spectra so that the characteristics of the dynamic setup could be calculated directly. It is recommended that this topic be approached as a two phase effort with the first phase evaluating approaches and the second phase pursuing the approach identified. | 9, 10, and many beyond those identified in Table 1 |
| | | GC | I | PRODAT | | |
| | | PC | I | PRODAT | | |
| | | SW | I | PRODAT | | |
| 48 | Dynamic Wave Setup | AC | I | PRODAT | This topic could be incorporated into Topic 47, but a more realistic approach is to parallel Topic 47 with a first phase to evaluate existing methodologies that could | 9, 10, and many beyond |
| | | GC | I | PRODAT | | |
| | | PC | I | PRODAT | | |

| Table 1. Summary of Findings and Recommendations for Wave Setup | | | | | | |
|---|---|---|---|---|---|---|
| Topic Number | Topic | Coastal Area | Priority Class | Availability/ Adequacy | Recommended Approach | Related Topics |
| | | SW | I | PRODAT | be applied. The results of the first phase would guide the second phase, which would implement the optimal approach identified. It is anticipated that the actual procedures developed would be somewhere between a full physics-based approach which would proceed from a directional spectrum, and the approaches available from Lo and Goda which are either based on somewhat simple calculations or empirical. A probable approach would be one in which the dynamic wave setup is based on parameterized spectra determined as a function of wind fields and continental shelf width of interest. | those identified in Table 1 |

Key:
Coastal Area
AC = Atlantic Coast; GC = Gulf Coast; PC = Pacific Coast; SW = Sheltered Waters
Priority Class
C = critical; A = available; I = important; H = helpful
Availability/Adequacy
"Critical" Items: MIN = needed revisions are relatively minor; MAJ = needed revisions are major
"Available" Items: Y = availability confirmed; N = data or methods are not readily available
"Important" Items: PRO = procedures or methods must be developed; DAT = new data are required;
PRODAT = both new procedures and data are required

| Table 2. Time Estimates for Wave Setup Topics | | |
|---|---|---|
| Topic Number | Topic | Time (person months) |
| 44 | **Better Define and Document; Summarize What to Consider and How to Approach; Data Requirements** | |
| | Improve Definitions in Guidelines | 1 |
| | Develop Approach Strategy | 2 |
| | Write Report | 2 |
| | Incorporate Feedback, Finalize | 2 |
| | TOTAL | 7 |
| 45 | **Compile Example/Data Sets to Perform Tests** | |
| | Compile Data Sets From US Literature | 2 |
| | Visit US Investigators to Obtain Data Sets as Necessary | 3 |
| | Visit International Investigators to Obtain Data Sets as Necessary | 3 |
| | Compile Data Sets Into Useful Data Base | 3 |
| | TOTAL | 11 |

| Table 2. Time Estimates for Wave Setup Topics | | |
|---|---|---|
| Topic Number | Topic | Time (person months) |
| 46 | **Develop Interim Method (look at CEM as a fall back, or University of Hawaii SPM Procedure)** | |
| | 1. Select Engineering Based or Boussinesq Model Method | 4 |
| | 2. Develop Recommendations for Nearshore Profiles | 2 |
| | 3. Evaluate and Make Recommendations for Wave Breaking Model (Not Required if Boussinesq Model Selected) | 3 |
| | 4. Develop Recommendations for Representing Nonlinear Wave Effects on $S_{xx}$ at Breaking Model (Not Required if Boussinesq Model Selected) | 2 |
| | 5. Evaluate Candidate Methods for Dynamic Wave Setup and Develop Recommendation Model (Not Required if Boussinesq Model Selected) | 2 |
| | 6. Test Model Over a Wide Range of Settings Consistent With FEMA's Responsibilities | 2 |
| | 7. Evaluate Whether Existing Methods Include Wave Setup Effects Implicitly and if so, Account for These | 2 |
| | 8. Develop Report (User's Manual) Describing Recommended Interim Methodology | 2 |
| | TOTAL | 10–19 |
| 47 | **Develop Ideal Method Coupled With Storm Surge and Waves to Develop Setup** | |
| | Evaluate Various Available Models, Select Model for Further Development | 4 |
| | Further Develop Model for FEMA Applications | 12 |
| | Incorporate Nonlinear Effects on $S_{xx}$ (Reduced effort if Boussinesq Model Selected) | 2 |
| | Ensure That Recommended Methodology Does Not Include Wave Setup Effects Implicitly Model (Reduced effort if Boussinesq Model Selected) | 3 |
| | Test Model Over a Wide Range of Settings Consistent With FEMA's Responsibilities | 4 |
| | Develop Report (User's Manual) Describing Recommended Model | 4 |
| | TOTAL | 24–29 |
| 48 | **Develop Procedures for Dynamic Wave Setup** | |
| | Evaluate Various Available Models, Select Model for Further Development | 4 |
| | Further Develop Model for FEMA Applications | 8 |
| | Exercise Model for Scenarios and Settings of FEMA Interest | 4 |
| | Test Model Over a Wide Range of Settings Consistent With FEMA's Responsibilities | 4 |
| | Develop Report (User's Manual) Describing Recommended Model | 4 |
| | TOTAL | 24 |

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# Wave Runup and Overtopping

## FEMA Coastal Flood Hazard Analysis and Mapping Guidelines Focused Study Report

February 2005


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## Acronyms

| | |
|---|---|
| ACES | Automated Coastal Engineering System |
| ANEMONE | Advanced Non-Linear Engineering Suite of Models for the Nearshore Environment |
| BFEs | Base Flood Elevations |
| CCSTWS | Coast of California Storm and Tidal Wave Study |
| CDIP | Coastal Data Information Program |
| CEDAS | Coastal Engineering Design and Analysis |
| CEM | Coastal Engineering Manual |
| CHAMP | Coastal Hazard Analysis Modeling Program |
| FEMA | Federal Emergency Management Agency |
| FIRM | Flood Insurance Rate Map |
| FIS | Flood Insurance Study |
| GIS | Geographic Information Systems |
| LIDAR | LIght Detection And Ranging |
| LOMR | Letter of Map Revision |
| NFIP | National Flood Insurance Program, |
| NGVD | National Geodetic Vertical Datum |
| PWA | Phillip Williams & Associates |
| SPM | Shore Protection Manual |

| | |
|---|---|
| TAW | Technical Advisory Committee for Water Retaining Structures |
| USACE | U.S. Army Corps of Engineers |
| WHAFIS | Wave Height Analysis for Flood Insurance Studies |

# 1 INTRODUCTION

Water levels along coastal shorelines vary through time, depending upon tides and incident wave conditions. These water levels can be thought of as being composed of two components: 1) a static (or assumed static or slowly varying) mean water level associated with astronomical tides, storm surges, and wave setup; and 2) a fluctuation about that mean (swash) associated with surf beat and the motion of individual waves at the shoreline.

As used in this report*, *wave runup* refers to the height above the stillwater elevation (tide and surge) reached by the swash (see Figure 1). Runup is a very complex phenomenon, that is known to depend on the local water level (including surf beat or infragravity wave effects), the incident wave conditions (height, period, steepness, direction), and the nature of the beach or structure being run up (e.g., slope, reflectivity, height, permeability, roughness).

Runup guidance is largely empirical, and typically is based either on field measurements on beaches or on laboratory measurements on structures. Most guidance relates runup to the surf similarity parameter $\xi$ (ratio of the barrier slope to the square root of the wave steepness) as a means of reducing the number of variables and generalizing the applicability of specific measurements or tests.

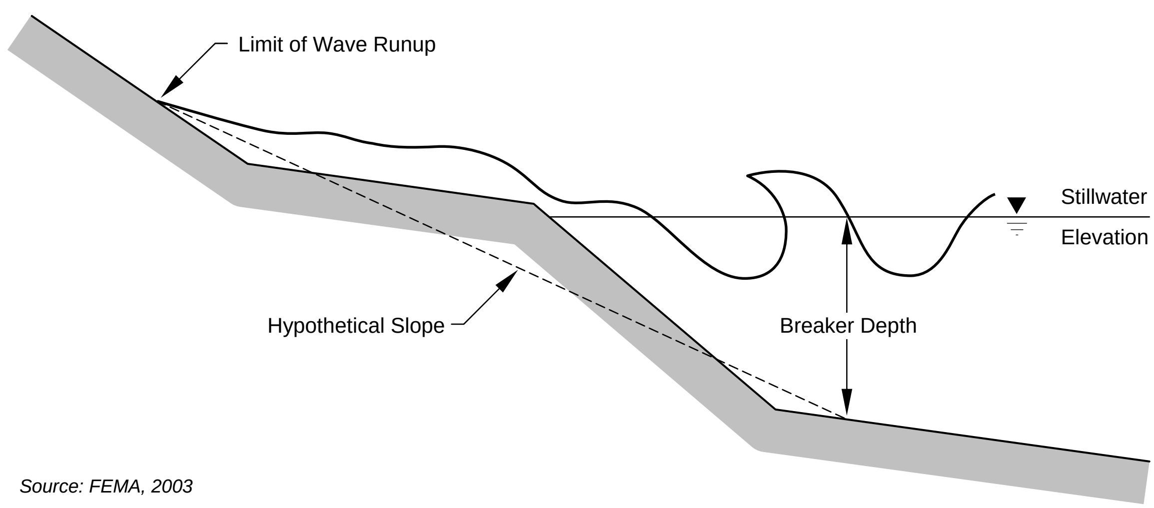



**Figure 1. Wave runup sketch.**

* Using this definition, which is consistent with current Federal Emergency Management Agency (FEMA) guidance, wave runup includes wave setup. An alternate definition for wave runup would exclude the wave setup component such that the runup is equal to the height above the stillwater elevation plus setup reached by the swash. The definition selected for use should be determined in conjunction with work carried out by the Wave Transformation and Wave Setup Study Groups.

As used in this report, *wave overtopping* refers to the volumetric rate at which runup flows over the top or crest of a slope, be it a beach, dune, or structure.

This report provides recommendations for:

- development of wave runup and overtopping guidance for Study Contractors completing Flood Insurance Studies (FIS) or restudies along the Pacific shorelines of California, Oregon, and Washington;
- development of wave runup and overtopping guidance for use by Study Contractors along sheltered (i.e., non-open coast) shorelines throughout the continental United States; and
- review of existing wave runup and overtopping guidance for use along the shorelines of the Atlantic Ocean and Gulf of Mexico.

Note that any recommendations or work on runup and overtopping must be integrated with recommendations and work on other topics, e.g., stillwater, wave setup, wave transformation, coastal structures, event based erosion, hazard zones, and tsunamis.

## 1.1 CATEGORIES AND TOPICS

Five wave runup/overtopping topics were identified at Workshop 1, and are identified below. The topic with the highest priority was Topic 12 (use of mean vs. higher values for runup and overtopping), followed by Topics 11 (review methods and models), 49 (WRUP™), 13 (overtopping volumes), and 14 (wavecast debris). Note that some of the workshop-assigned priorities and topic details were revised during the focused study.

**Wave Runup and Overtopping Topics and Priorities**

| Topic Number | Topic | Topic Description | Priority | | |
|---|---|---|---|---|---|
| | | | Atlantic / Gulf Coast | Pacific Coast | Non-Open Coast |
| 11 | Methods and Models | Review runup programs and methods; provide explicit guidance on where each should be applied | H *(I)* | A *(C)* | A *(C)* |
| 12 | Mean v. Higher Value | Review appropriateness of using mean vs. higher values for runup and overtopping | H *(C)* | C | C |
| 13 | Overtopping Volumes | Develop improved guidance for determining and mapping overtopping volumes | -- *(A)* | A | A |
| 14 | Wavecast Debris | Review available methods and develop guidance for wavecast debris | H | I | I |
| 49 | WRUP | Review WRUP™ (available wave runup program) | A | A | A |

Key: C = critical; A = available; I = important; H = helpful
(Recommend priority italicized if focused study recommended a change in priority class)

## 1.2 WAVE RUNUP AND OVERTOPPING FOCUSED STUDY GROUP

This report was prepared using information and comments submitted by Ida Brøker (Danish Hydraulics Institute), Kevin Coulton (HDR), Jeff Gangai (Dewberry & Davis), Darryl Hatheway (Baker), Chris Jones (focused study leader), Jeremy Lowe (Phillip Williams & Associates), Ron Noble (Noble Engineering Consultants, Inc.), and Rajesh Srinivas (Taylor Engineering).

## 1.3 CURRENT FEMA GUIDANCE FOR WAVE RUNUP AND OVERTOPPING

### 1.3.1 Introduction

FEMA's existing guidance for runup and overtopping is limited to the coasts of the Atlantic Ocean, Gulf of Mexico, and Great Lakes*, as summarized in Appendix D of the *Guidelines and Specifications for Flood Hazard Mapping Partners* (FEMA, 2003). Although it is not stated explicitly, the inference is that existing Atlantic/Gulf and Great Lakes guidance will be appropriate for associated sheltered shorelines, given the proper selection of base flood water levels and wave conditions. There is no runup and overtopping guidance for the Pacific Coast in Appendix D.

Figures D-1 (page D-18) and D-35 (page D-113) of the *Guidelines and Specifications* (*G&S*) illustrate the overall procedures to be used for Atlantic/Gulf and Great Lakes flood insurance studies. In both cases, runup analyses must be preceded by the definition of a shore profile (transect). This shore profile must evaluate the durability (during the base flood) of any coastal structures present, and assess base flood erosion along any erodible shorelines. Runup estimates must be made along transects that have been adjusted for event-based erosion (not long-term erosion) and for any expected failures of coastal structures. Although it is not mentioned in the *G&S*, Study Contractors should check for possible breaches and failures between transects before interpolating runup and overtopping results to adjacent beaches.

FEMA calls for runup (and therefore, overtopping) analyses only in certain instances, as shown in Appendix D, Tables D-1 (Atlantic/Gulf) and D-14 (Great Lakes). These tables are summarized in Table 1 below.

FEMA presumes that runup on low-profile beaches—without a sizable landward barrier (e.g., dune, bluff, cliff, or structure)—will not be significant, and therefore need not be analyzed or calculated. This presumption is reasonable on low-profile shorelines where storm surges flood upland areas and wave heights tend to control base flood elevations (BFEs). This presumption, however, is probably invalid for the Pacific Coast, where storm surge heights tend to be small, swell periods can be large, infragravity motions can be substantial, and wave runup on beaches and structures tends to control BFEs.

* Note that FEMA's Great Lakes runup methods are based on the USACE Detroit District procedures (USACE, 1989).

**Table 1. Shore Types where Runup Estimates are Required for Flood Insurance Studies (Atlantic/Gulf Coasts and Great Lakes)**

| Shore Type | Runup Analysis |
|---|---|
| Rocky bluff | yes |
| Sandy/sediment bluff or bank, little beach | yes |
| Sandy beach, small dune | no |
| Sandy beach, large dune | yes |
| Open wetlands | no |
| Shore protection structure | yes |

Source: FEMA, 2003

### 1.3.2 Wave Runup

Runup guidance for the Atlantic Ocean and Gulf of Mexico is contained on pages D-42 through D-60 of FEMA (2003). FEMA calls for the use of its RUNUP 2.0 model, except for vertical- or near-vertical-faced coastal structures; on such structures, FEMA (2003) calls for use of procedures contained in the *Shore Protection Manual* (USACE, 1984). Although it is not stated in the *G&S*, FEMA also permits use of the Automated Coastal Engineering System (ACES) (USACE, 1992) for runup and overtopping calculations against vertical and sloping structures. (Note that ACES v. 1.07 is on the FEMA list of accepted models of coastal wave effects, which can be found at <http://www.fema.gov/fhm/en_coast.shtm>). It should also be noted that ACES uses more up-to-date methods than those contained in the *Shore Protection Manual* or those used in RUNUP 2.0

RUNUP 2.0 is a 1990 update and revision to FEMA's first runup model (RUNUP 1.0), which was originally developed for use in New England flood insurance studies in 1981. RUNUP 2.0 is discussed in Hallermeier, et al. (1990) and documented in Dewberry & Davis (1991).

RUNUP 2.0 is based largely on the reanalysis by Stoa (1978) of small-scale laboratory runup tests (regular waves on smooth, impermeable, uniform slopes); on the composite slope procedure developed by Saville (1958); and on roughness coefficients taken from the *Shore Protection Manual* (USACE, 1984). However, RUNUP 2.0 results were compared against field and large-scale laboratory runup measurements (using irregular waves), and Hallermeier et al. (1990) determined that the model predictions were in agreement with the measurements. Although not stated explicitly in the *G&S*, input wave conditions for RUNUP 2.0 will likely be irregular waves (specified as the equivalent deepwater mean wave height and period).

RUNUP 2.0 calculates wave runup along shore-perpendicular transects. It uses the 1% (100-year) stillwater elevation (tide plus surge, not including wave setup) and the equivalent deepwater *mean* wave conditions (height and period) as model inputs. It then estimates the *mean* wave runup height, which is added to the 1% stillwater elevation to determine the *mean* wave runup elevation. FEMA (2003) recommends using ranges of input wave heights and periods as inputs (+/- 5% or whatever percentage suits the level of uncertainty) in cases where it is difficult to specify the 1% flood conditions. The *G&S* call for averaging the RUNUP 2.0 output values for the nine input combinations of water level, wave height, and wave period.

One key difference between RUNUP 2.0 and RUNUP 1.0 is the fact that the latter predicted wave runup using unspecified combinations of offshore wave heights and periods (i.e., neither mean [50%], nor significant [33%], nor controlling [1%]) that were expected to occur during northeasters (or hurricanes). It was assumed by RUNUP 1.0 that the results (when added to the 1% stillwater elevation) represented the *maximum* runup elevation (Stone & Webster, 1981), while RUNUP 2.0 computes the *mean* runup elevation. Thus, there is a significant disparity between the results of flood insurance studies in communities based on RUNUP 1.0 and 2.0 models (Hatheway, pers. comm., 2003). This can be seen in New England, where many flood studies were based on the RUNUP 1.0 model.

Finally, unlike the case of wave height analyses using WHAFIS, FEMA (2003) states that wave setup is not to be added to the 100-year stillwater elevation before wave runup analyses, because RUNUP 2.0 assumes that wave setup is already included in the calculated wave runup. This assumption may be reasonable if the measurements and model tests used to develop the procedures contained in RUNUP 2.0 included wave setup effects (these data should be reviewed). However, the validity of this assumption should be reexamined for the Pacific Coast subject to infragravity waves, and as FEMA's wave setup calculation methods evolve.

### 1.3.3 Wave Overtopping

Overtopping guidance for the Atlantic Ocean and the Gulf of Mexico is contained on pages D-61 through D-69 of FEMA (2003), and is based largely on the work of Owen (1980) and Goda (1985).

FEMA (2003) does not call for overtopping calculations in all instances. Instead it first calls for a comparison of the freeboard, F (the vertical distance between the base flood stillwater elevation and the crest elevation), and the mean runup height, $\overline{R}$. If F > 2$\overline{R}$, then the guidance assumes that overtopping can be neglected. If F ≤ 2$\overline{R}$, then the mean overtopping rate $\overline{Q}$ for a nonvertical slope is calculated according to:

$$\overline{Q} = Q^*(gH_S^3)^{0.5} \quad (1)$$

$$Q^* = 8 \bullet 10^{-5}\ \exp[3.1\ (rR^* - F/H_S)] \quad (2)$$

$$R^* = [1.5m/(H_s/L_{op})^{0.5}] \quad (3)$$

where:

$Q^*$ = dimensionless overtopping,

$R^*$ = estimated extreme runup normalized by Hs (note: the *G&S* do not define "extreme" runup),

$r$ = the roughness coefficient,

$F$ = freeboard,

$H_s$ = incident significant wave height at toe of overtopped barrier,

$g$ = gravitational constant,

$m$ = the cotangent of the slope angle of the overtopped barrier, and

$L_{op}$ = deepwater wavelength.

FEMA (2003) also includes guidance (Figure D-19) that can be used to estimate the dimensionless overtopping on smooth slopes (see Figure 2), from which $\overline{Q}$ can be calculated (adjustments for roughness can be made according to the text).

Overtopping of a vertical wall is calculated using the methods of Goda (1985) and summarized in *G&S* Figure D-20 (page D-68).

Table 2 (Table D-7 on page D-69, repeated below) relates flood hazard zones landward of an overtopped structure/feature to the mean overtopping rate.

**Table 2. Interpretation of Mean Wave Overtopping Rates**

| $\overline{Q}$ Order of Magnitude | Flood Hazard Zone Behind Barrier |
|---|---|
| <0.9991 cfs/ft | Zone X |
| 0.0001-0.01 cfs/ft | Zone AO (1 ft depth) |
| 0.01-0.1 cfs/ft | Zone AO (2ft depth) |
| 0.1-10. cfs/ft | Zone AO (3ft depth) |
| >1.0 cfs/ft* | 30-ft width** of Zone VE (elevation 3 ft above barrier crest), landward Zone AO (3 ft depth) |

*With estimated $\overline{Q}$ much greater than 1 cfs/ft, removal of barrier from transect representation may be appropriate

**Appropriate inland extent of velocity hazards should take into account structure width, incident wave period or wavelength, and other factors.

Source: FEMA, 2003

Note that one hazard zone associated with overtopping and rapid sheet flow—the VO zone—has been designated in the National Flood Insurance Program (NFIP) regulations, but is not contained in Table 2 and has not been implemented. The Hazard Zone Focused Study may recommend use of the VO zone; if so, procedures governing its use should be coordinated with the Runup/Overtopping Study Group.

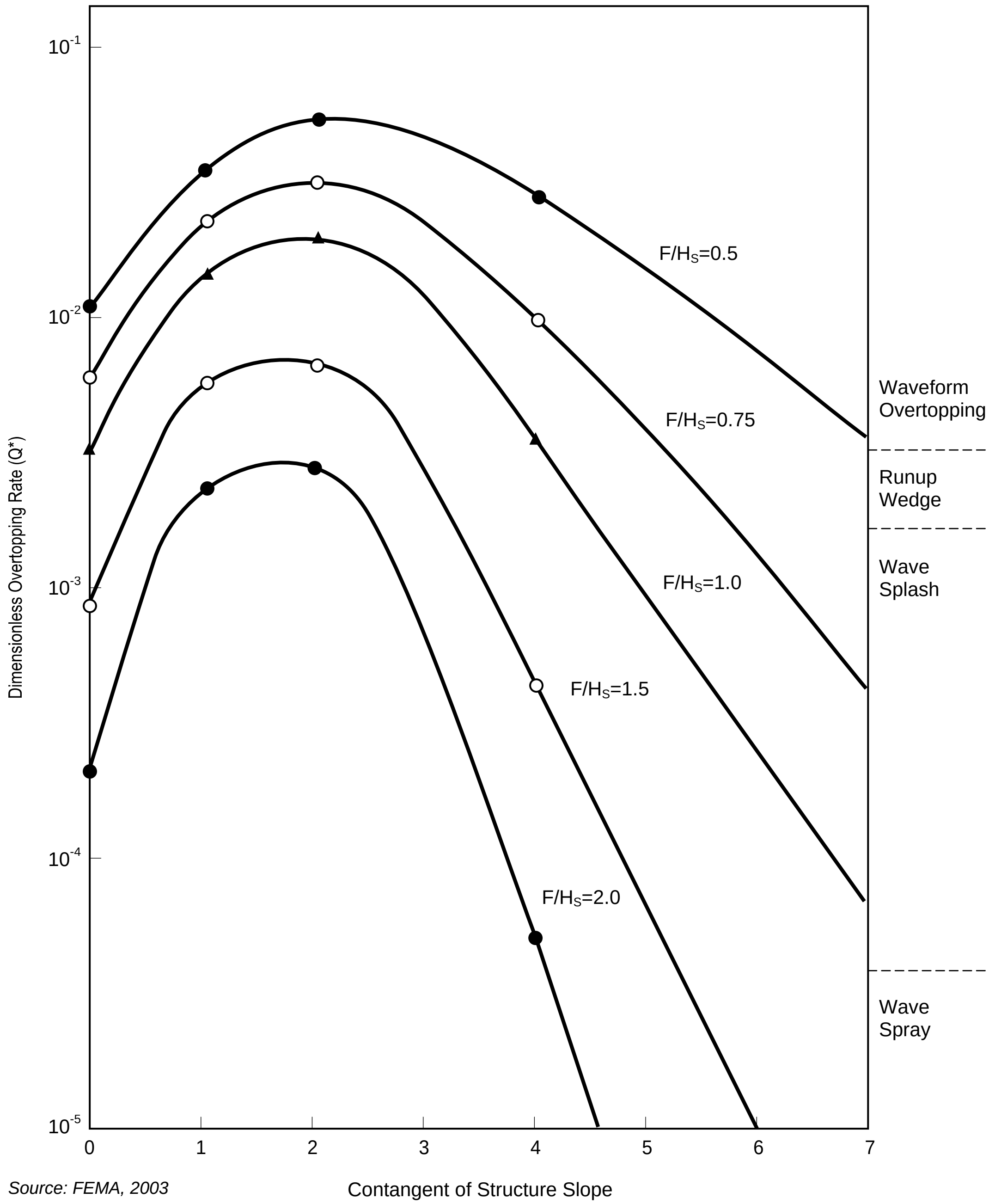



**Figure 2. Overtopping of smooth, sloping structures.**

FEMA (2003) provides simplified guidance for mapping flood hazard zones on overtopped dunes/barriers without calculating overtopping values (see Figure 3), and provides some guidance for runup onto low bluffs and plateaus, based largely on the work of Cox and Machemehl (1986)—see Figure 4. These procedures should be reviewed based on recent experience and other more recent methods.

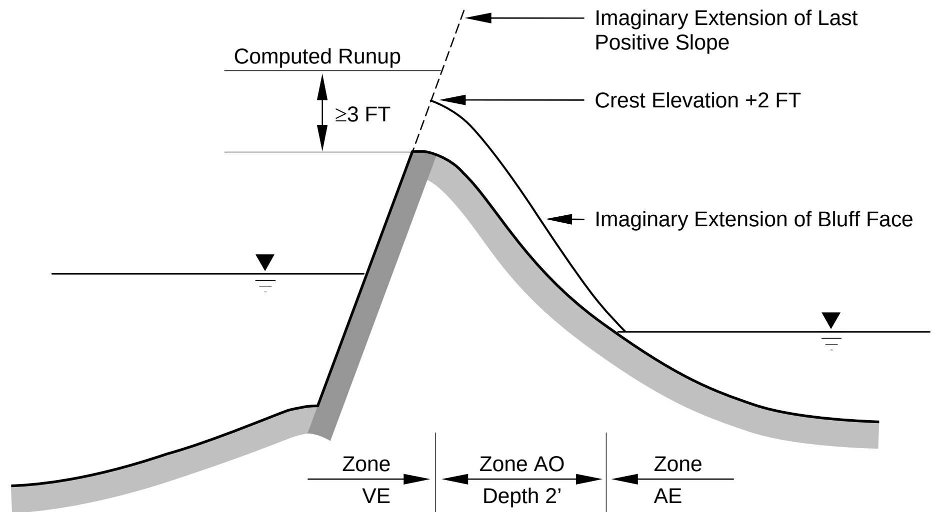



**Figure 3. Simplified mapping of overtopped dune where runup exceeds crest by 3 feet or more.**

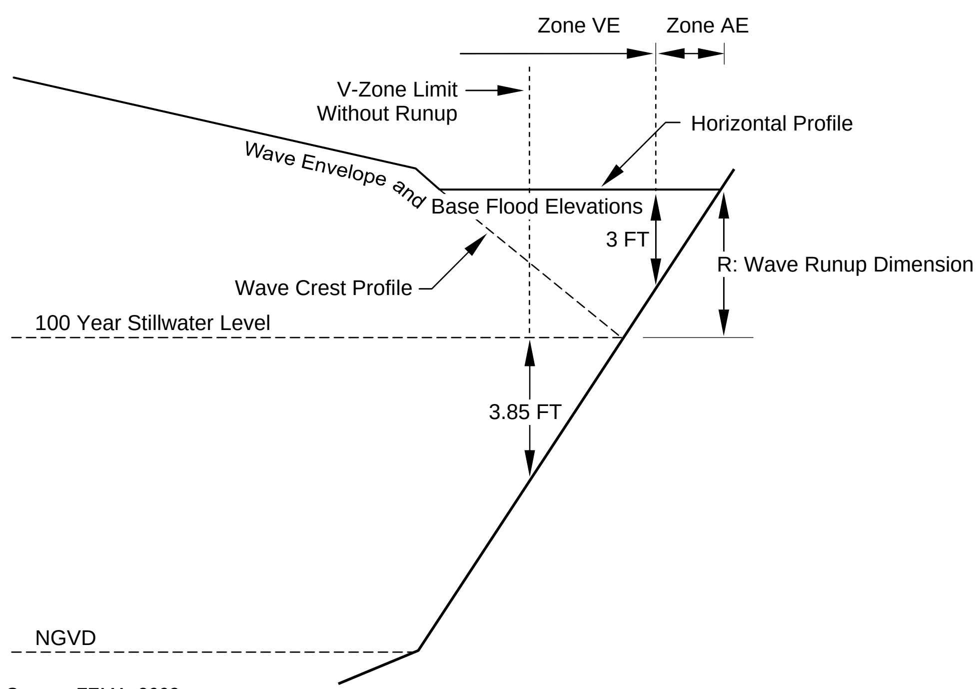



**Figure 4. Wave envelope and base flood elevations resulting from combination of wave heights and wave runup.**

# 2 CRITICAL TOPICS

## 2.1 TOPIC 12: REVIEW APPROPRIATENESS OF USING MEAN VS. HIGHER VALUES FOR RUNUP AND OVERTOPPING

### 2.1.1 Description of the Topic and Suggested Improvement

This topic can be summarized by asking three questions:

- Is calculating the mean runup elevation consistent with other FEMA guidance and procedures?
- Does mapping to the mean runup elevation provide adequate protection for building's which are in compliance with NFIP requirements?
- Does mapping to the mean overtopping rate provide adequate protection for NFIP-compliant buildings?

The conclusion of the Focused Study Group is that the answer to the first two questions is no, and the study group recommended that consideration be given to calculating and mapping to a higher runup level (the exact level is yet to be determined).

The answer to the third question is closely tied to how the overtopping rate is used to identify hazard zones. Use of the mean overtopping rate may be acceptable for calculation purposes, but the hazard zone delineations based on the mean overtopping rate may need to be revised.

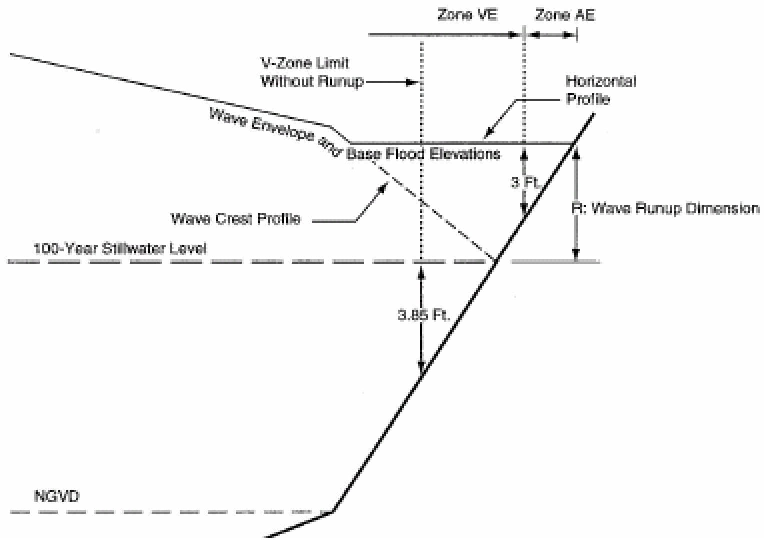
### 2.1.2 Description of Procedures in the Existing Guidelines

Current FEMA guidance calls for calculating (and mapping based on) the mean runup elevation and the mean overtopping rate.

Although there may be some exceptions, the average of the RUNUP 2.0 computed mean runup elevations is used to establish the BFE and flood hazard zones on the slope/structure subject to runup. The crest elevation and mean overtopping rate are used to establish the BFE and flood hazard zone landward of the overtopped structure/feature.

In areas not dominated by storm surge and wave heights, or by primary frontal dune considerations (see Hazard Zone Topics 17 and 39), FEMA differentiates between V zones and A zones based on the wave runup depth and the overtopping rate, as follows:

Areas on slopes subject to runup, where the ground is lower than 3.0 feet below the mean runup elevation (i.e., where the runup "depth" is greater than or equal to 3.0 feet), are classified as V zones. Where runup "depths" are less than 3.0 feet the areas are classified as A zones. Note the similarity to V zones based on wave heights (V zones have runup depth $\geq$ 3.0 feet or breaking wave heights $\geq$ 3.0 feet). See Figure 5.




Source: FEMA, 2003

**Figure 5. Wave envelope and base flood elevations resulting from combination of wave heights and wave runup.**

Landward areas subject to mean overtopping rates ≥ 1.0 cubic foot per second (cfs)/foot are mapped as V zones (see Table 1 above); otherwise, they are mapped as AO zones.

#### 2.1.3 Application of Existing Guidelines to Topic—History and/or Implications for the NFIP

There are three key implications associated with application of the existing guidance. These implications are described below.

***Consistency with Other FEMA Procedures****

FEMA typically—but with an important exception—maps hazards associated with the 100-year event at the mean (50%) level. Review of the *G&S* shows that the mean runup elevation, mean overtopping rate, and median erosion value are all used in mapping the 1% flood elevations in coastal areas. However, for Atlantic/Gulf of Mexico situations, FEMA uses its WHAFIS model to establish BFEs using the "controlling" (1%) wave height, not the mean wave height. The controlling wave height is equivalent to approximately 1.6 times the significant wave height (or approximately 2.6 times the mean wave height) in deepwater, but all

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* Another inconsistency can be found with the incident wave conditions used as model inputs for RUNUP 2.0 versus WHAFIS. Although the inconsistency may be correct technically, it can be confusing to those using the RUNUP 2.0 and WHAFIS models: RUNUP 2.0 requires input of the equivalent deepwater mean wave height and period (approximated as 0.65 times the equivalent deepwater significant wave height, and 0.85 times the peak wave period); WHAFIS requires input of the significant wave height and peak wave period at the start of the analysis transect (which WHAFIS converts to the controlling [1%] wave height, assumed to be 1.6 times the significant wave height).

reduce to the depth-limited wave height (0.78 times stillwater depth) in shallow water. The WHAFIS model calculates wave crest elevations based on the controlling wave height. This procedure can be traced to the National Academy of Sciences (1977).

Dewberry & Davis (1991) acknowledges this discrepancy (between mapping the controlling wave height and the mean runup height), but calls for use of the mean runup value because there are "limitations in assuming a Rayleigh probability distribution for runup elevations." In other words, use of the mean runup value avoids having to estimate what a maximum runup elevation might be, when there is uncertainty associated with the actual runup distribution. Uncertainty arguments aside, there can sometimes be an inconsistency between mapping wave heights to a 1% level and mapping wave runup to a 50% level. The significance of this inconsistency increases as the runup velocity increases, and will be most apparent for mapping tsunami runup. The inconsistency may also be important in Pacific regions where infragravity motions can be substantial.

### *Adequacy of Base Flood Elevations and Hazard Zones Identified using Mean Values*

This issue should be viewed in light of the principal purposes of the NFIP—to map flood and flood-related hazards, and to establish minimum development regulations (principally those related to the design and construction of buildings) using those maps.

If one examines the history of NFIP coastal mapping, the original coastal BFE was simply the stillwater level, and wave effects were ignored. Insurance premiums for areas subject to wave heights were surcharged, and building standards for V zones were more restrictive than those in A zones, but BFEs ignored the presence of waves. The National Academy of Sciences recognized the problem, as did those who inspected new homes in coastal Alabama, built to the stillwater elevation but destroyed by Hurricane Frederic in 1979. It was after Hurricane Frederic that the NFIP produced *Wave Height Supplement* reports and modified BFEs to reflect the 1% wave crest elevation.

Ignoring runup elevations above the 50% level means that buildings elevated to the mean runup elevation may be reached many times (and likely damaged) by wave runup during a coastal storm event. Although the impact of wave runup of a certain depth is generally less than that contained in a breaking wave of similar height (and, therefore, building damage may be less), the omission seems similar in nature (if not in magnitude) to the early omission of wave heights by the NFIP. This argument is supported by a recent flood insurance study on the Pacific Coast at Sandy Point, in Whatcom County, Washington. This study determined that use of the mean runup calculation procedure could under-predict damage to upland structures caused by flooding and associated wavecast debris. The determination was based on observed flooding and damage during a 5% (20-year) flood event (Phillip Williams & Associates, 2002).

The design of coastal structures is not the main focus of the NFIP (although coastal structure design is considered in mapping flood hazards). However, the present project can be informed by guidance on the design of coastal structures. The durability and crest elevation of a coastal structure are usually dictated by the importance of the area being protected, and by the frequency and rate of overtopping deemed acceptable. Structural designs are typically based on wave heights greater than $H_{50\%}$, and crest elevations are usually set to prevent overtopping at runup elevations higher than the mean value. These practices indicate that protection at a level higher than 50% is common. Regarding overtopping, mean overtopping rates are generally used for coastal structure design purposes. This practice may underestimate flooding in some cases, however. For example, if the structure has a high crest elevation but is attacked by several large, unbroken waves over a short period of time, the mean overtopping rate may be low, but the overtopping associated with those few large waves may cause significant flooding behind the structure.

#### *RUNUP 1.0 vs. RUNUP 2.0*

In 1991, FEMA adopted RUNUP 2.0 and discontinued use of RUNUP 1.0. RUNUP 1.0 calculated maximum runup elevations for a variety of combinations of input wave heights and periods assumed to be representative of conditions for a northeaster (or hurricane), not mean runup elevations. No systematic comparison of the results has been made for communities where Flood Insurance Rate Maps (FIRMs) are based on RUNUP 1.0. However, such a comparison might reveal substantially lower BFEs would result from use of RUNUP 2.0 mean runup elevations. Granted, some of the differences would be the result of other revisions made between versions 1.0 and 2.0, but the difference attributable to mapping a mean vs. maximum runup level could be significant. Further comparisons should be made for the northeastern Atlantic Coast to better define the difference between the results of runup models 1.0 and 2.0.

### 2.1.4 Alternatives for Improvement

#### *Wave Runup*

Several alternative runup values are considered for flood hazard mapping purposes:

- Maintain present FEMA use of $\overline{R}$,
- $R_{33\%}$ (significant runup, $R_s$),
- $R_{10\%}$,
- $R_{2\%}$, and
- $R_{max}$ (maximum runup).

The selected value should account for the duration, frequency, and magnitude of runup elevations that may potentially damage upland structures. Use of FEMA's present $\overline{R}$ guidance seems to violate this criterion. However, the selected value need not be so conservative that it precludes all contact between runup and upland structures during the base flood event (use of the $R_{max}$ value clearly violates this criterion), nor must it prevent contact by runup that has a low frequency of occurrence and/or a low likelihood of causing structural damage to upland structures (use of the $R_{2\%}$ value may violate this criterion).

Thus, use of a runup value in the range of $R_{33\%}$ to $R_{10\%}$ seems reasonable. Once a runup value is adopted, the next step is to define the $R_{x\%}$ height and elevation based on an existing runup calculation procedure that calculates $R_{x\%}$ directly (or uses a runup distribution relating $R_{x\%}$ to $\bar{R}$), or based on a more rigorous analysis (e.g., Monte Carlo). As a first approximation, and for the purposes of the present analysis, the $R_{33\%}$ and $R_{10\%}$ values would correspond to approximately $1.5\bar{R}$ and $2.0\bar{R}$, respectively. Incorporation of conversion factors such as these would allow the continued use of the RUNUP 2.0 model and methods in their present form, with only a scaling of the output runup height—an easy adjustment.

### *Wave Overtopping*

As was the case with runup, several alternative overtopping values could be considered:

- Maintain present FEMA use of mean overtopping rate $\bar{Q}$,
- Q33% (significant overtopping rate, Qs),
- Q10%,
- Q2%, and
- $Q_{max}$ (maximum overtopping rate).

However, overtopping calculations are subject to much more uncertainty than runup calculations, and selection of a specific $Q_{x\%}$ may be problematic. Kobayashi (1999) points out that while mathematical and numerical runup models may replicate measured runup values with errors of about 20%, predicted overtopping rates are often in error by a factor of 2 or more. Some overtopping predictions may be even less accurate, given the fact that subtle changes in wave conditions, water levels, barrier geometry and characteristics, or wave breaking can have a very large effect on overtopping rates. Unlike the case of wave runup, there appears to be no compelling reason to adopt an overtopping value different from $\bar{Q}$. It is recommended that FEMA continue to use the $\bar{Q}$ calculation, but reevaluate flood hazard zone designations based on mean overtopping rates (see Table 1 above and Section 3.2).

### 2.1.5 Recommendations

Recommendations for Topic 12 are as follows (see Table 5 at the conclusion of this report):

1. Revise the guidance to call for runup analyses in the sandy beach, small dune shore type (because runup will control BFEs on many low-profile beaches along the Pacific and sheltered shorelines).

2. Evaluate use of the mean runup $\overline{R}$ with a value; if $\overline{R}$ fails to capture historical evidence of damaging runup, then consider an alternate value for mapping purposes (probably in the range of $R_{33\%}$ to $R_{10\%}$, or as indicated by historical data).

3. Develop an interim procedure for adjusting the results of RUNUP 2.0 (for FIS or Letter of Map Revision [LOMR] evaluations).

4. Conduct a similar analysis specific to the tsunami runup value appropriate for flood hazard mapping.

5. Retain use of the mean overtopping rate $\overline{Q}$ for overtopping calculation purposes, but consider revising overtopping values that distinguish among flood hazard zones.

### 2.1.6 Preliminary Time and Cost Estimate for Guideline Improvement Preparation

Table 6 at the end of this report presents estimates of times required to accomplish the tasks in this topic.

### 2.1.7 Related Available and Important Topics

Available and Important Topics related to Topic 12 are listed in Table 5, at the conclusion of this report.

## 2.2 TOPIC 11: REVIEW RUNUP METHODS AND PROGRAMS; PROVIDE EXPLICIT GUIDANCE ON WHERE EACH SHOULD BE APPLIED

Overtopping considerations have been removed from Topic 11 and grouped with those in Topic 13; although overtopping depends upon runup, it can be treated differently for NFIP flood hazard mapping purposes.

### 2.2.1 Description of the Topic and Suggested Improvement

Current FEMA runup guidance has been developed on an ad-hoc basis over the years. The guidance may or may not represent the procedure(s) most appropriate for a contemporary FIS. It may or may not be transferable to the Pacific Coast.

In fact, experience suggests that this guidance may not be directly transferable without some revision or modification. The Pacific Coast, unlike the open-coast Atlantic and Gulf of Mexico, does not lend itself to a simple characterization of the 1% flood event. Much of the Pacific Coast is composed of dissipative beaches, and the relative contributions of storm surge, wave setup, and wave runup can differ substantially from those along the coasts of the Atlantic Ocean and Gulf of Mexico. Pacific wave spectra may differ substantially from those used to develop the FEMA runup methods used along the coasts of the Atlantic Ocean and Gulf of Mexico.

This is not to say that wave runup has not been computed for the Pacific Coast. It has been computed using a variety of available methods: the FEMA RUNUP 2.0 model, ACES, *Shore Protection Manual* SPM (1984) methods, tsunami runup models, and other methods, some of which are based on local experience.

The issue is not whether runup methods are available; the issue is which of the available methods are best suited to FISs and yield the best results for the Pacific Coast. Therefore, the Focused Study Group has chosen to revise the Topic 11 priorities assigned at Workshop 1 from “Available” to “Critical” for the Pacific, and from “Helpful” to “Available” for the Atlantic and Gulf Coasts.

Clearly, the identification of appropriate runup guidance is most needed for Pacific FISs, and that issue is given the highest priority. Existing guidance for the Atlantic and Gulf can be used without major modification (notwithstanding the mean runup issue discussed in Topic 12), but the New England Coast especially will benefit from the development of guidance for the Pacific Coast.

The Focused Study Group for Topic 11 sought to facilitate the development of sound, practical runup guidance for the Pacific Coast, and to evaluate similar guidance for the coasts of the Atlantic Ocean and Gulf of Mexico. With this in mind, the study group’s primary recommendation is to develop test scenarios and perform side-by-side comparisons of existing runup methods and models. The testing should include evaluation of the sensitivity of the various runup methods and models to various parameters (e.g., profile shape and roughness, incident wave characteristics, infragravity motions). Infragravity motions must be included in any Pacific Coast testing; infragravity waves are more common on the Pacific Coast than on the Atlantic and Gulf Coasts, and such waves can amplify runup and overtopping considerably.

A similar approach may be useful for evaluating Pacific Coast event-based erosion or wave setup and wave transformation. As many categories as possible should be evaluated using common test conditions.

### 2.2.2 Description of Procedures in the Existing Guidelines

See Sections 1.2 and 2.1.2.

### 2.2.3 Application of Existing Guidelines to Topic—History and/or Implications for the NFIP

See Section 2.1.3.

### 2.2.4 Alternatives for Improvement

At least a dozen methods and models can be used to predict wave runup, not counting site-specific field measurements and laboratory modeling (both of which are unlikely during an FIS). Relevant issues and parameters associated with these methods and models are as follows:

- Each method or model is based on certain assumptions and empirical data, and each is valid over a range of morphologic, hydraulic, and sometimes geographic conditions.
- Some use deepwater wave conditions as input; others use local (i.e., transformed) wave conditions at the toe of the barrier.
- Some methods or models are applicable to beaches and others to coastal structures.
- Some are applicable to transect-type analyses while others are appropriate to grid- or element-based analyses.
- Each requires tradeoffs among simplicity, accuracy, data requirements, ease of use, and economy.

***Wave Runup***

The runup methods and models considered are described below.

**RUNUP 2.0**

This model was described in Section 1.2.1.

**Shore Protection Manual**

The Shore Protection Manual (SPM) (USACE, 1984) contains several graphs that relate the runup of normally incident regular (monochromatic) waves on impermeable slopes to deepwater wave steepness, barrier slope, and deepwater wave height. Refraction, diffraction, and bottom friction are not considered. Graphs are provided for smooth slopes, quarrystone and stepped revetments, and vertical and curved-face seawalls. These graphs are based on small-scale laboratory work; guidance is provided for adjustment of calculated runup for scale effects and roughness. Any effects of wave setup are included in the computed runup values. Saville's (1958) composite slope procedure is included.

The SPM gives limited guidance for estimating runup resulting from irregular waves. According to Dewberry & Davis (1991), the 1984 SPM did not make use of Stoa's (1978) reanalysis of wave runup data.

**WRUP$^{TM}$**

WRUP$^{TM}$ was developed by Noble Software, Inc., for the runup of regular waves (Noble, 1984). A menu-driven program designed to facilitate the calculation of wave runup based on SPM methods, WRUP$^{TM}$ uses equations, curves, and methodology presented in the 1984 edition of the SPM.

The program can be applied to composite slopes (up to eight variable slopes per profile) including revetted slopes, vertical slopes, and three defined complex structures. It can calculate runup that exceeds the top of a vertical wall or other steep slope by adding a fictitious flat slope directly behind the top of vertical or steep slopes. Wave input can be at deepwater, intermediate water, or depth-limited breaking waves. WRUP$^{TM}$ has been applied to the Coast of California Storm and Tidal Waves Study (CCSTWS) in Orange County for the U.S. Army Corps of Engineers (USACE). The advantage of using WRUP$^{TM}$ is that it is faster and more convenient than interpolating from graphs in the SPM. A flow chart for WRUP$^{TM}$ is shown in Figure 6.

**Parabolic Profile Representation**

Taylor et al. (1980) developed an alternate to the composite-slope approach by describing the beach profile between the seaward edge of the dune and the wave breakpoint by an equilibrium profile, a parabolic function of the form:

$$x = a\ y^{\upsilon} \qquad (4)$$

The formulation does not include longshore bars. It uses small-scale laboratory data of Saville (1956, 1958), Savage (1958) and Hunt (1959) to relate runup to the deepwater wave height and period.

Limited comparisons with the profiles produced by the composite-slope method for Volusia County, Florida, show generally poor agreement, with the parabolic method producing generally lower runup. This was thought to have occurred partly because the parabolic approach smoothed the bar and resulted in seaward shifting of the wave breakpoint, which reduced the mean slope relative to the composite-slope method. It was not possible at the time of the study to determine which approach more accurately predicted runup.

**ACES v. 1.07**

The most widely used version of ACES is the freely distributed ACES v. 1.07 (USACE, 1992). Later versions are available only as part of the CEDAS (Coastal Engineering Design and Analysis System) software sold by Veritech.

ACES v. 1.07 has three wave runup programs: *Irregular Wave Runup on Beaches*, *Irregular Wave Runup on Riprap*, and *Wave Runup and Overtopping on Impermeable Structures*. Wave setup contributions are included in each of the runup calculations.

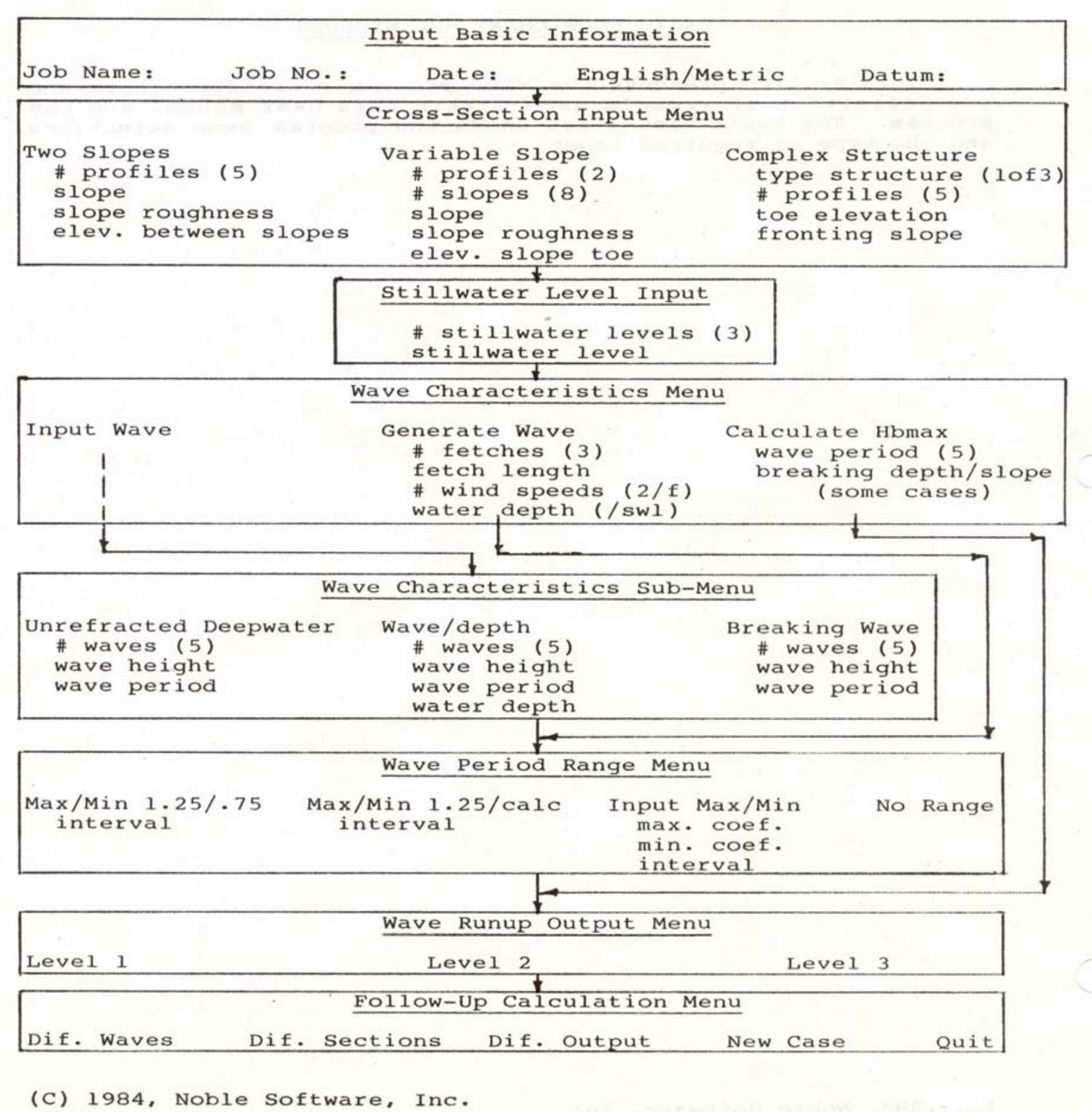



**Figure 6. Flow chart for WRUP™.**

The *Irregular Wave Runup on Beaches* module calculates several values of runup ($R_{max}$, $R_{2\%}$, $R_{10\%}$, $R_{33\%}$, and $\overline{R}$) based on laboratory experiments of runup on smooth impermeable slopes. The calculations are made given the deepwater significant wave height, peak wave period, and foreshore slope (which yield the surf similarity parameter, $\xi = \tan\theta / (H_o/L_o)^{1/2}$), and using the general relationship

$$\frac{R_{x\%}}{H_0} = a\xi^{\,b} \qquad (5)$$

where $a$ and $b$ are constants that depend on the statistic ($x$%) desired, from Mase (1989).

The *Irregular Wave Runup on Riprap* calculation is part of the *Rubble-mound Revetment Design* module. The method calculates the expected maximum runup elevation and provides a conservative estimate of the maximum runup elevation, based on small-scale laboratory tests of Ahrens and Heimbaugh (1988). The calculations are made given the deepwater significant wave height, peak wave period, and foreshore slope (which yield the surf similarity parameter), and using the general relationship

$$\frac{R_{max}}{H_0} = a\,\xi/(1+b\,\xi) \qquad (6)$$

where $a$ and $b$ are constants given by Aherns and Heimbaugh (1989).

The *Wave Runup and Overtopping on Impermeable Structures* module calculates the runup elevation associated with incident uniform waves at the structure toe (described by $H_i = H_s$) acting on smooth or rough structures. Other inputs are the peak wave period, nearshore slope, structure slope, and roughness coefficients. The pertinent relationships are

$$\frac{R}{H_i} = c\,\xi/(1+d\,\xi) \qquad \text{for rough slopes} \qquad (7)$$

$$\frac{R}{H_i} = C \qquad \text{for smooth slopes} \qquad (8)$$

where $c$ and $d$ are armor unit coefficients given by Ahrens and McCartney (1975), and coefficient $C$ varies with the surf similarity parameter $\xi$, based on the work of Ahrens and Titus (1985).

The ACES runup modules represent improved guidance over that contained in the SPM. ACES guidance may be preferable to RUNUP 2.0 in some instances. The *Irregular Wave Runup on Beaches* calculation is maintained in the Coastal Engineering Manual (CEM). The *Irregular*

*Wave Runup on Riprap* calculation is reported to be advantageous because it works well for both shallow water and deep water at the toe of the revetment.

**Coastal Engineering Manual**

A replacement for the *Shore Protection Manual*, the CEM (2003) (Section II-4-4) contains guidance for calculation of regular and irregular wave runup on beaches (Smith, 2003). Wave setup contributions are included in the runup results. Runup by regular breaking waves on smooth impermeable slopes is based on small-scale model tests and is a function of the deepwater wave conditions (expressed using the surf similarity parameter). Such runup is calculated using relationships developed by Hunt (1959), and rewritten in nondimensional form by Battjes (1974):

$$\frac{R}{H_0} = \xi_0 \qquad \text{for} \qquad 0.1 \le \xi_0 \le 2.3 \ \ \text{with} \quad \xi_0 = \tan\beta\left(\frac{H_0}{L_0}\right)^{-\frac{1}{2}} \tag{9}$$

Walton et al. (1989) revised the formulation to determine the upper limit of runup by nonbreaking regular waves:

$$\frac{R}{H_0} = (2\pi)^{\frac{1}{2}}\left(\frac{\pi}{2\beta}\right)^{\frac{1}{4}} \tag{10}$$

where $\beta$ = slope (in radians).

The guidance for runup from irregular breaking waves on smooth impermeable slopes is similar to the guidance contained in ACES 1.07 (see above). The CEM (2003) (Section VI-5-2) contains guidance for calculation of irregular wave runup on structures (Burcharth and Hughes, 2003). The guidance is based largely on the small- and large-scale laboratory tests summarized in van der Meer and Stam (1992), and van der Meer and Janssen (1995). It uses a Battjes-type formulation

$$\frac{R_{x\%}}{H_s} = (A\xi + C)\gamma_r\gamma_b\gamma_h\gamma_\beta \tag{11}$$

where $A$ and $C$ are coefficients related to the surf similarity parameter and runup probability for the reference case (smooth, straight impermeable slope, normally incident long-crested waves with wave heights given by a Rayleigh distribution); and where the coefficients $\gamma_r$ , $\gamma_b$ , $\gamma_h$ , $\gamma_\beta$ adjust for surface roughness, influence of a berm, shallow water, and angle of wave incidence ($\gamma$ = 1.0 for reference case).

The CEM provides several graphs and formulas for $R_{2\%}$ and $R_S$ as a function of the significant wave height at the toe of the structure, not as a function of the deepwater wave height. Also, note that $R_{2\%}$ refers to the runup level exceeded by 2% of the incoming waves, not by 2% of the runup levels, etc.

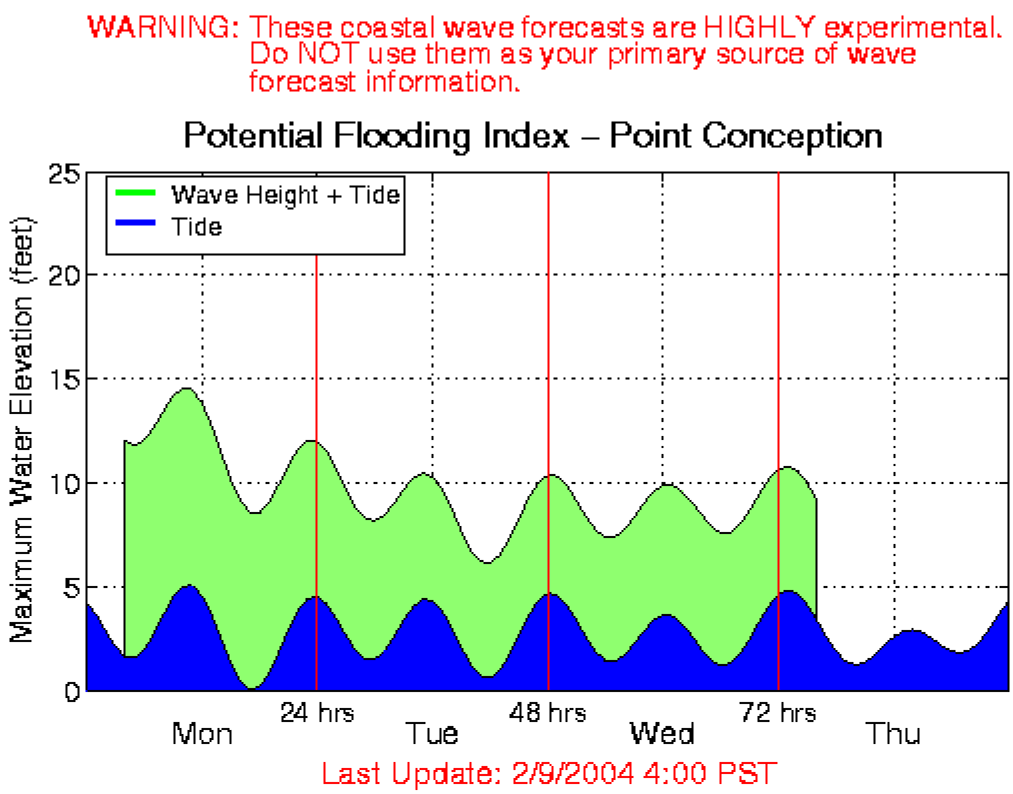
The CEM provides no methods for calculating irregular wave runup against vertical walls, although the method of Walton et al. (1989) mentioned above in the *Regular Wave Runup on Beaches* section could be used.

**Wave Momentum Flux Parameter**

Hughes (2003a, 2003b) developed and used a wave momentum flux parameter to improve on the predictive accuracy of the CEM's irregular wave runup guidance for smooth-sloped, impermeable structures. Like the CEM, this revised method calculates the $R_{2\%}$ value using inputs of local wave height and period, structure slope, and depth at structure toe.

**Coastal Data Information Program (Potential-Flooding Index for Southern California)**

The Coastal Data Information Program (CDIP) is an experimental tool used to forecast the maximum runup elevation based on predicted (astronomical) tide elevations and the predicted significant wave height outside the surf zone (Seymour, 2003). The experimental CDIP tool is illustrated in Figure 7.




Source: CDIP 2004

**Figure 7. Coastal Data Information Program, potential flood index tool.**

The CDIP is not a wave runup model per se; therefore, use of the CDIP Potential Flood Index Tool as a proxy for runup elevations should be considered an interim approach until runup analyses are completed. Actual forecasts can be found under *Wave Forecast Models* (see "Coast Waves + Tide, southern California") at <http://cdip.ucsd.edu/elnino_htmls/homepage.shtml>. The Potential Flood Index Tool assumes that the combined setup plus runup at the shoreline is equal to the significant wave height beyond the surf zone. (The latter can be forecast using wave buoy data and numerical models.)

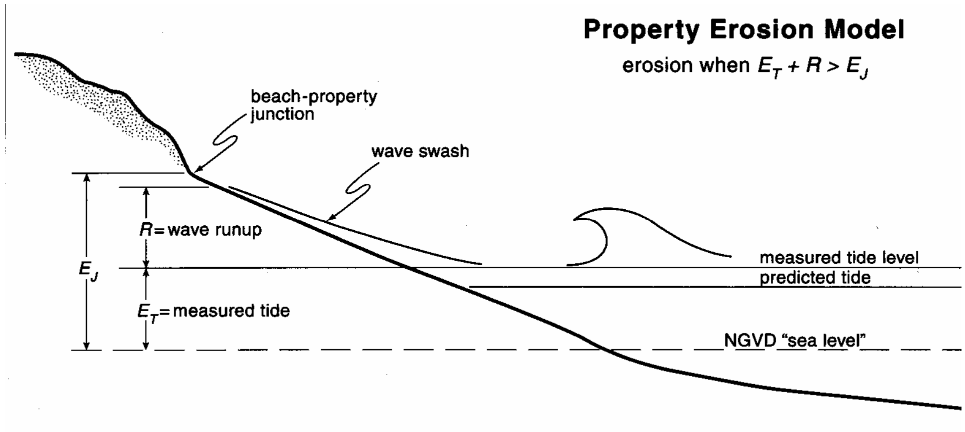
**Oregon Property Erosion Model**

Ruggiero et al. (2001) summarize development of a model to evaluate the susceptibility of coastal property to wave-induced erosion. The model is predicated on the observation that foredune erosion occurs when the runup elevation (actual tide elevation plus runup height) exceeds the elevation of the beach-foredune junction (see Figure 8). Wave setup is embedded in the runup.

The study points to the importance of both runup elevation and duration (hours/year) of high runup elevations. It found a good correlation between the number of hours per year that the predicted $R_{2\%}$ elevation would exceed the beach-foredune elevation, and observed erosion characteristics. Using field data from Oregon and North Carolina (USACE Field Research Facility, Duck, North Carolina), the predicted $R_{2\%}$ (2% exceedance elevation, measured in meters above National Geodetic Vertical Datum [NGVD]) was defined using beach slope, and deepwater significant wave height and wavelength as:

$$R_{2\%} = 0.27\,(S\,H_{os}\,L_o)^{1/2} \qquad \text{(metric units)} \qquad (12)$$

Where the shore was subject to less than 1 hour of attack per year ("attack" is defined as when $R_{2\%}$ exceeds the beach-foreshore junction), the shore tended to be stable or accretional. Where the shore was subject to more than 10 hours of attack per year, the shore was erosional. Higher durations were associated with greater erosion.




Source: Ruggerio et al., 2001

**Figure 8. Oregon property erosion model.**

**Technical Advisory Committee for Water Retaining Structures**

The TAW (2002) report updates the earlier guidance of van der Meer (upon which much of the CEM runup guidance is based). This report is available at <http://www.tawinfo.nl/engels/downloads/TRRunupOvertopping.pdf>. It includes the results of recent model tests, and considers cases with very shallow foreshores and with vertical walls atop slopes. The report also replaces use of the peak wave period at the structure toe with the spectral wave period, and increases estimates of maximum wave runup.

**Boussinesq Wave Models**

This type of model solves the so-called Boussinesq type equations in the time domain. It resolves the waves in detail, and is suited for simulation of propagation and interaction of nonlinear directional waves. It is capable of reproducing the combined effects of most wave phenomena of interest in ports, harbors, and coastal engineering: shoaling and refraction, diffraction, bottom dissipation, partial reflection and transmission, nonlinear wave-wave interactions, and wave breaking for directional, irregular waves.

DHI's suite of models, MIKE 21, includes two Boussinesq modules, 2DH and 1DH. The "2DH" module calculates wave disturbance in ports and harbors; the 1DH module calculates wave transformation across an arbitrary profile from offshore up to the shoreline for the study of surf zone and swash zone dynamics (see Figure 9). The 1DH module solves the equations along a transect, and can therefore represent the dynamics for unidirectional, irregular waves.

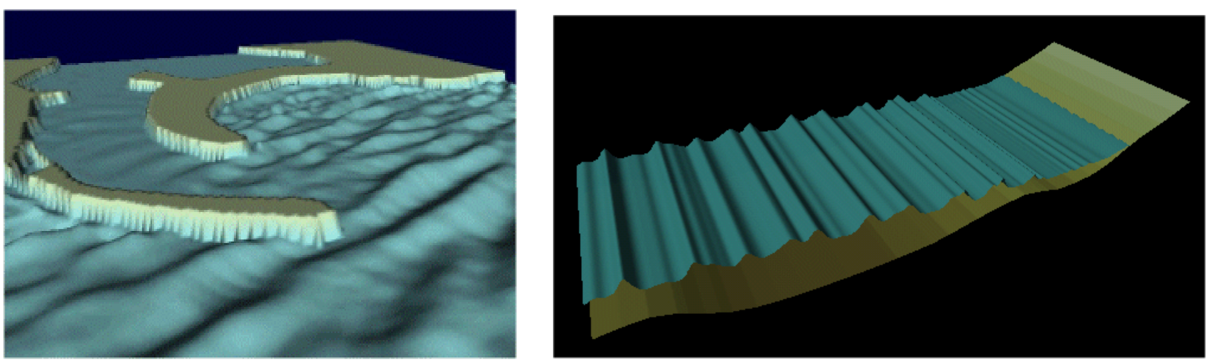
The 1-D BW model is a relevant tool for the study of runup, and its strength is its computational speed. The 1-D BW can simulate the combination of setup and runup, and phenomena such as wave groups and surf beat can be included (provided that the driving forces are included in the boundary conditions). The results can be analyzed into frequency of exceedance runup levels.

**Detailed 3-D Hydrodynamic Model, Navier-Stokes Solvers**

DHI's Navier-Stokes solver, NS3, is a numerical model that solves the full three-dimensional Navier-Stokes equations including modeling of the free surface. The model is designed especially for modeling of refined flow problems, such as eddies around structures, details of run-up on structures, etc.

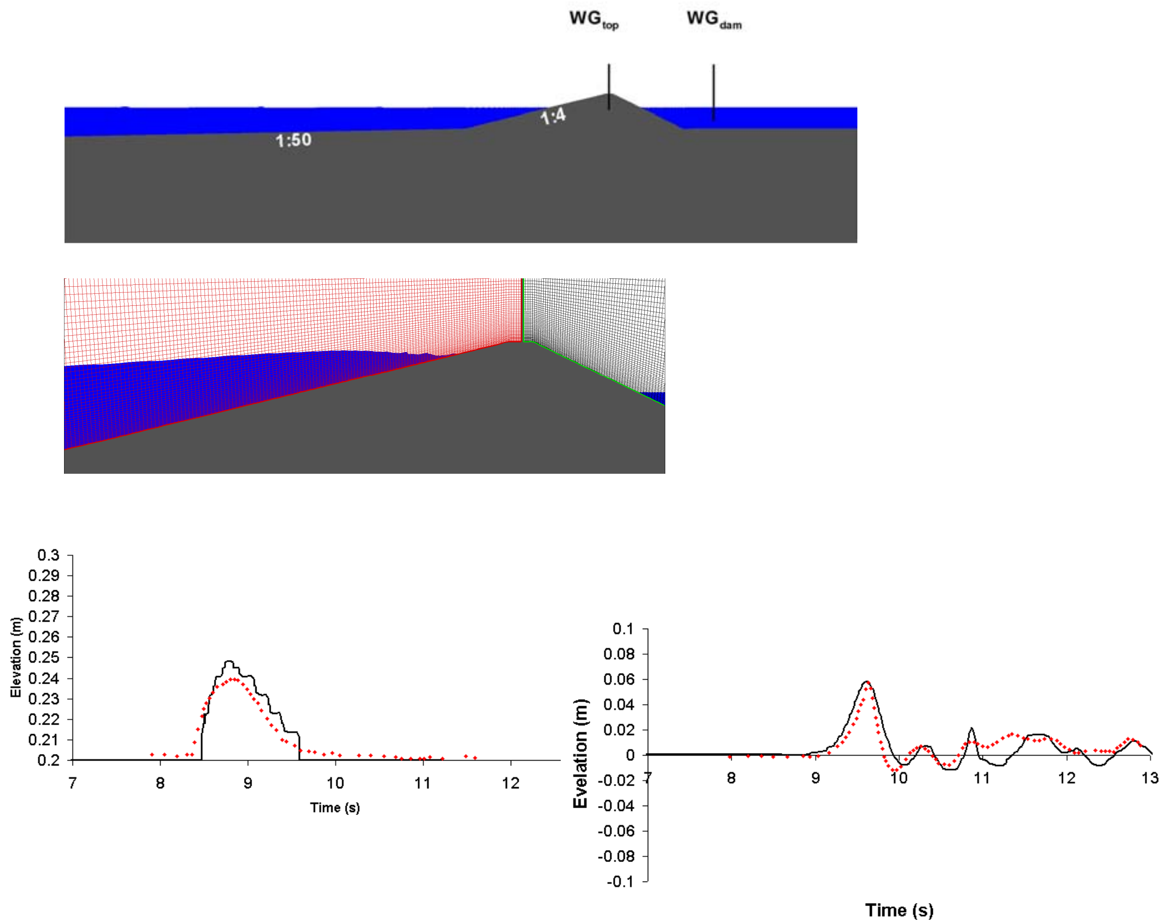
The model can be run in full 3-D or can be used as a "slice model" representing, for instance, a coastal transect. Figure 10 shows an example where NS3 has been used along a transect analysis to calculate runup and overtopping of a solitary wave on a dike. The example shows a comparison between modeled and measured water levels on the crest and behind the dike.

NS3 can be used as a numerical tool that replaces physical model tests in a flume. Output from the model is a time series of water levels, velocity fields, overtopping rates, and pressure fields. This model is also a useful tool for the calculation of forces on structures, e.g., wave forces on a wave screen. The Navier-Stokes solver is more accurate in the prediction of wave overtopping than the Boussinesq models, which are strong tools for wave runup calculations.



Source: Danish Hydraulics Institute

**Figure 9. Illustration of the 2-D BW Model (wave penetration into a harbor) and the 1D BW Model (wave transformation across a beach profile).**




*Notes*:
Upper panel, layout of experiment; middle panel, close-up of computational grid near the crest of the dike; lower left, comparison of measured water level at the crest (dots) and modeled level (line); lower right, measured water level behind the structure (dots) and modeled (line).

Source: Danish Hydraulics Institute

**Figure 10. Runup and overtopping calculated by DHI NS3.**

The numerical model is complex and computationally demanding. NS3 is presently not released as a commercial software product and runs presently without Graphical User Interfaces. However, conceptual model setups can be prepared so experienced modelers can adjust the boundary conditions and the geometry and can run specific simulations without detailed knowledge of the coding.

### *Deterministic vs. Statistical Approaches*

Two general methods for computing 1% annual chance flood elevations were discussed in Workshop 2: the Event Selection Method and the Response Method.

- The Event Selection Method is deterministic; it uses one or more user-identified combinations (each defined as a 1% flood event) of water level and wave conditions, and computes the resulting flood elevation for each combination. The user then selects a flood elevation for mapping purposes.

- The Response Method is based on a statistical approach, where input parameter values are selected (randomly) from defined parameter distributions, and are then used to compute a flood elevation (response). The process is repeated many times, a response distribution is developed, and the 1% response is determined.

Given the difficulties (particularly on the Pacific Coast and on sheltered shorelines) in defining the 1% flood event, including all relevant parameters—water level, transformed wave conditions, wave setup, erosion, and runup—it may be useful to consider a statistical type analysis for determining the $R_{x\%}$ elevation used for flood hazard mapping. A statistical (response) approach can account for the random combination of storm wave conditions, tide elevations, and other parameters, and can determine a statistical distribution of wave runup frequency and wave runup elevations.

The statistical approach requires distributions and constraints for input parameters to be defined. It allows determination of the wave-tide combination(s) responsible for the $R_{x\%}$ elevation. The statistical approach is not limited to a single runup calculation procedure (it can be employed with many different procedures), but can provide statistical meaning to the results from the runup calculation procedure employed. A flow chart for one statistical approach is shown in Figure 11.

***Using Models vs. Using Simple Procedures***

The main advantage of numerical runup (and overtopping) models over simple procedures (empirical formulas) is that with models, arbitrary profile shapes can be studied in combination with widely varying water level and wave parameters. The utility of simple formulas is restricted by the empirical data and conditions that led to their development, and extrapolation to other geometries and conditions may be questionable.

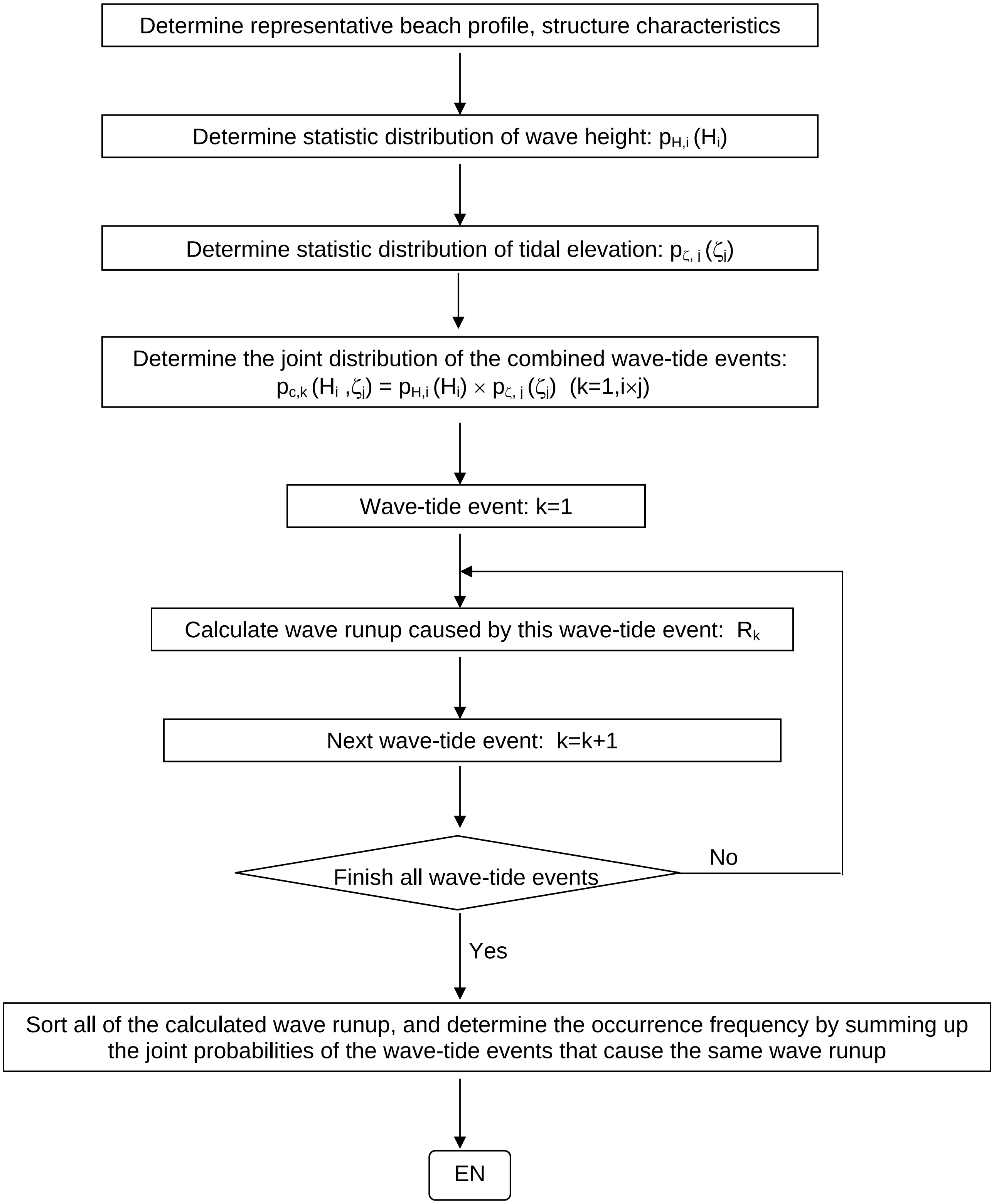



**Figure 11. Flow chart for statistical approach.**

However, if a shoreline/structure profile under investigation has geometric characteristics and hydraulic conditions similar to those that form the basis for a simple procedure, use of the simple procedure will be acceptable, and will probably be more cost effective for FEMA. Numerical models may be better suited to complex shoreline shapes, geometries, and situations (and less restricted by ranges of conditions over which they are applicable), but they also require more data, preparation, expertise and expense to yield acceptable results.

Numerical models, computing capabilities, and data acquisition/manipulation techniques (including Light Detection and Ranging [LIDAR] and Geographic Information Systems [GIS]) have advanced significantly over the past two decades. During that time, however, FEMA's basic approach to identifying coastal flood hazards has remained unchanged. (Improvements have been made to various FEMA methods, but the basic transect analysis process has remained intact.) Model development has been driven, in large part, by the need for improved coastal structure design capabilities, and for shoreline management purposes. Flood hazard mapping can benefit from these advancements.

Ultimately, FEMA's methods will be overtaken and replaced by numerical models. This is likely to occur first for large study areas where coastal storm surges (including wave transformation, wave setup, and other wave effects) must be recomputed, and last for situations where previously computed storm surges and related parameters are judged adequate for FEMA use. This evolution should also occur first where critical infrastructure and development exist, and where the uncertainty associated with use of the simple formulas may not be acceptable. Note that FIS and FIRM appeals may hasten this evolution, through the use of more advanced models by appellant representatives.

In the interim, runup (and overtopping) calculations can be carried out by a variety of methods (which may include numerical models), but carefully chosen and applied simple procedures should be adequate for most coastal FISs and restudies.

The Runup/Overtopping Study Group recommends that the procedures and models described above be evaluated carefully, with an eye toward improving the accuracy of flood hazard maps using simple procedures (where possible), and eventually migrating to numerical models for most flood hazard mapping tasks.

### *Wave Runup, Wave Setup, and Wave Transformation*

Wave runup is typically estimated using the stillwater elevation (without wave setup) as an input, and runup estimates generally include the combined effects of swash and wave setup. This has been the tendency because the majority of field and laboratory runup measurements to date—upon which most estimation procedures are based—have made no attempt to separate out the exact effects of wave setup. Relying on wave inputs is likewise a function of the evolution of empirical runup methods; some rely on deepwater wave conditions while others rely on the local waves at the structure toe.

As models advance, the capacity to resolve water level constituents, wave transformation, and complex hydraulic interactions will increase. It is important to take advantage of these capabilities where they serve flood mapping needs, but the need should drive the technique (not the other way around).

Irrespective of the exact path, as FEMA's coastal flood hazard mapping methods change, the treatment of wave setup and wave runup (and other components, e.g., stillwater elevations, event-based erosion, overland wave propagation) must be consistent. Thus, the Runup/Overtopping Study Group sees the need for close coordination with other Focused Study Groups, particularly the Wave Setup and Wave Transformation groups.

### 2.2.5 Recommendations

Recommendations for Topic 11 are as follows (see Table 5, at the conclusion of this report):

Investigate use of Oregon-type and/or CDIP-type methods as interim methods for all of California, Oregon, and Washington. While not probability-based at present, it is reasonable to expect that probabilities could be assigned and a base flood runup elevation could be estimated using these methods. Bear in mind the previously mentioned caution, that the CDIP does not resolve the surf zone and compute wave runup—its Potential Flood Index Tool is an experimental proxy for runup.

Develop test scenarios for side-by-side comparisons of existing runup methods and models (give priority to the Pacific Coast, followed by New England, then the south Atlantic and Gulf of Mexico). This will require selecting representative beach profiles and structure geometries—including low-profile, sandy-beach, small-dune barriers not presently modeled for runup (see Table 1)—then locating existing data sets that can be used as a basis for comparing the accuracy and sensitivity of results. These data sets may also serve as historical data of potential use in future FISs. (Coordinate development of test scenarios with other study groups.)

Perform the side-by-side comparisons. Eliminate methods or models that do not provide acceptable results or that cannot be used efficiently. (Remember that these will have to be used for FISs with time, budget, and expertise constraints.) Identify which methods and models are appropriate for use in various geographic areas and morphologic/hydraulic conditions. Consider appropriate ranges of input parameters to address event definition uncertainty.

Coordinate work with the Wave Setup and Wave Transformation Study Groups. Inputs to wave runup methods/models must be available and consistent with the results of wave setup and transformation tasks.

### 2.2.6 Preliminary Time and Cost Estimate for Guideline Improvement Preparation

Table 6 at the end of this report presents estimates of times required to accomplish the tasks in this topic.

#### 2.2.7 Related Available and Important Topics

Available and Important Topics related to Topic 11 are listed in Table 6 at the conclusion of this report.

# 3 AVAILABLE TOPICS

## 3.1 TOPIC 49: REVIEW WRUP™ (AVAILABLE WAVE RUNUP PROGRAM)

#### 3.1.1 Description of the Topic and Suggested Improvement

See Section 2.2.1.

#### 3.1.2 Description of Procedures in the Existing Guidelines

See “Wave Runup” in Section 2.2.4.

#### 3.1.3 Application of Existing Guidelines to Topic—History and/or Implications for the NFIP

FEMA *G&S* are predicated on SPM calculations for many items, including wave runup on vertical walls. WRUP™ is a program built around SPM methods, and therefore it should satisfy current flood hazard calculation requirements. However, the model has not been accepted by FEMA per se, and its widespread use would not be permitted. (The developer is free to use the model and submit its results for specific projects; this is one issue that has not been clarified by FEMA.) Formal acceptance and widespread use of WRUP™ should be predicated upon: 1) the continued use of SPM methods by FEMA, and 2) a detailed technical review of WRUP™ for consistency with the SPM.

#### 3.1.4 Alternatives for Improvement

See “Wave Runup” and “Deterministic vs. Statistical Approaches” in Section 2.2.4.

#### 3.1.5 Recommendations

The recommendation for Topic 49 is to include the evaluation of WRUP™ in the Topic 11 evaluation of runup methods and models.

#### 3.1.6 Preliminary Time and Cost Estimate for Guideline Improvement Preparation

Table 6 at the end of this report presents estimates of times required to accomplish the tasks in this topic.

## 3.2 TOPIC 13: DEVELOP IMPROVED GUIDANCE FOR DETERMINING AND MAPPING OVERTOPPING VOLUMES

### 3.2.1 Description of the Topic and Suggested Improvement

Current FEMA overtopping guidance has been developed on an ad-hoc basis over the years. The guidance may or may not represent the procedure(s) most appropriate for contemporary FISs.

There are a variety of overtopping methods and procedures that should be evaluated as part of this topic. The focus of the work should be on the following steps:

Review available overtopping methods and models, and determine appropriate procedure(s) for calculating the mean overtopping discharge, including those over low-profile beaches and barriers, dune remnants, revetments, and vertical walls.

Evaluate FEMA’s current guidance, which limits the runup elevation to 3 feet above a barrier’s crest elevation

Evaluate procedures for calculating overtopping onto low bluffs with gently sloping, flat, or adverse slopes. Evaluate methods for determining ponding landward of overtopped barriers

Review the current literature on “acceptable” overtopping, and work with the Hazard Zone Study Group to evaluate the overtopping rates FEMA (2003) uses to identify flood hazard zones landward of an overtopped barrier.

### 3.2.2 Description of Procedures in the Existing Guidelines

See Section 1.2.3.

### 3.2.3 Application of Existing Guidelines to Topic—History and/or Implications for the NFIP

See “Wave Overtopping” in Section 2.1.4.

### 3.2.4 Alternatives for Improvement

***Calculating Wave Overtopping***

The overtopping methods and models to be considered are described below.

<u>**FEMA Guidelines and Specifications Method**</u>
See Section 1.2.3.

<u>**Shore Protection Manual**</u>
For regular waves, an empirical expression is used based on a reanalysis of laboratory data reported by Saville (1955) and by Saville and Caldwell (1953):

$$q = \sqrt{gQ_0^* H_0^3}\,\exp\left[-\frac{0.217}{\alpha}\tanh^{-1}\left(\frac{h-d_s}{R}\right)\right] \quad \text{(Equation 7-10 in the SPM)} \qquad (13)$$

where $\alpha$ and $Q^*_0$ are empirical coefficients given in SPM Figures 7-24 to 7-32, based on experiments for various wave conditions, structure slopes and structure types. Weggel (1976) provided guidance on determining approximate values of $\alpha$ and $Q^*_0$ when better estimates are not available. Inputs are deepwater wave height, runup, height of structure, depth of water at the structure, and various coefficients. A procedure is included in the SPM to estimate the increase in overtopping rate with wind speed (Equation 7-12).

Ahrens (1977) extended the formula for regular waves by applying a method for determining runup for irregular waves. This procedure was included in the SPM as an interim procedure.

$$q_{p\%} = \sqrt{gQ_0^* H_{0,s}^3}\,\exp\left[-\frac{0.217}{\alpha}\tanh^{-1}\left(\frac{h-d_s}{R_s}\right)\frac{R_s}{R_{p\%}}\right] \quad \text{(Equation 7-14 in the SPM)} \qquad (14)$$

**ACES v. 1.07**

Wave overtopping is provided in ACES for both monochromatic waves and irregular waves. For monochromatic wave overtopping, ACES uses the SPM method developed by Weggel (1976). For irregular wave overtopping, ACES uses a method based on Ahrens (1977) and Douglass (1986), which uses Weggel’s monochromatic formula, but uses the significant deepwater wave height. The method computes and sums overtopping contributions of the individual members of the runup distribution.

**Cox and Machemehl (low bluff)**

See Section 1.2.2.

**Coastal Engineering Manual**

The CEM presents a variety of wave overtopping formulas from many different sources (see Table 3). Each source presents wave overtopping for a different structure configuration or scenario and is based mostly on empirical formulas from laboratory testing. Two types of overtopping formulations dominate the literature:

$$Q = a\, e^{-(bR)} \qquad (15)$$

$$Q = a\, R^{-b} \qquad (16)$$

where $Q$ is a dimensionless average overtopping rate per meter, $R$ is a dimensionless freeboard, and $a$ and $b$ are coefficients related to structure geometry.

**Table 3. Summary of CEM Overtopping Guidance**

**EM 1110-2-1100 (Part VI)**
**Proposed Publishing Date: 30 Apr 03**

**Table VI-5-7**
**Models for Average Overtopping Discharge Formulae**

| Authors | Structures | Overtopping model | Dimensionless discharge $Q$ | Dimensionless freeboard $R$ |
|---|---|---|---|---|
| Owen (1980,1982) | Impermeable smooth, rough, straight and bermed slopes | $Q = a\ exp(-b\,R)$ | $\frac{q}{g\,H_s\,T_{om}}$ | $\frac{R_c}{H_s}\left(\frac{s_{om}}{2\,\pi}\right)^{0.5}\frac{1}{\gamma}$ |
| Bradbury and Allsop (1988) | Rock armored impermeable slopes with crown walls | $Q = a\ R^{-b}$ | $\frac{q}{g\,H_s\,T_{om}}$ | $\left(\frac{R_c}{H_s}\right)^2\left(\frac{s_{om}}{2\,\pi}\right)^{0.5}$ |
| Aminti and Franco (1988) | Rock, cube, and Tetrapod double layer armor on rather impermeable slopes with crown walls, (single sea state) | $Q = a\ R^{-b}$ | $\frac{q}{g\,H_s\,T_{om}}$ | $\left(\frac{R_c}{H_s}\right)^2\left(\frac{s_{om}}{2\,\pi}\right)^{0.5}$ |
| Ahrens and Heimbaugh (1988b) | 7 different seawall/revetment designs | $Q = a\,exp(-b\,R)$ | $\frac{q}{\sqrt{g\,H_s^3}}$ | $\frac{R_c}{(H_s^2\,L_{op})^{1/3}}$ |
| Pedersen and Burcharth (1992) | Rock armored rather impermeable slopes with crown walls | $Q = a\,R$ | $\frac{q\,T_{om}}{L_{om}^2}$ | $\frac{H_s}{R_c}$ |
| van der Meer and Janssen (1995) | Impermeable smooth, rough straight and bermed slopes | $Q = a\,exp(-b\,R)$ | $\frac{q}{\sqrt{g\,H_s^3}}\sqrt{\frac{s_{op}}{\tan\alpha}}$ for $\xi_{op} < 2$ | $\frac{R_c}{H_s}\frac{\sqrt{s_{op}}}{\tan\alpha}\frac{1}{\gamma}$ for $\xi_{op} < 2$ |
| | | | $\frac{q}{\sqrt{g\,H_s^3}}$ for $\xi_{op} > 2$ | $\frac{R_c}{H_s}\frac{1}{\gamma}$ for $\xi_{op} > 2$ |
| Franco, de Gerloni, and van der Meer (1994) | Vertical wall breakwater with and without perforated front | $Q = a\,exp(-b\,R)$ | $\frac{q}{\sqrt{g\,H_s^3}}$ | $\frac{R_c}{H_s}\frac{1}{\gamma}$ |
| Pedersen (1996) | Rock armored permeable slopes with crown walls | $Q = R$ | $\frac{q\,T_{om}}{L_{om}^2}$ | $3.2 \cdot 10^{-5}\,\frac{H_s^5\,\tan\alpha}{R_c^3\,A_c \cdot B}$ |



Source: USACE, 2003

The method by Owen (1980), adopted by FEMA (2003), is still presented in the CEM for runup on impermeable, smooth and rough bermed slopes. The work of Goda (1985), also referenced by FEMA (2003), is mentioned in the CEM. The CEM provides a method to estimate the overtopping volume of an individual wave. (The average overtopping rate provides no information on the overtopping of single waves, yet most overtopping damage occurs with single large waves.)

**Wallingford (W178 Method)**

The HR Wallingford Ltd. (1999) report summarizes the current United Kingdom methodology for determining wave overtopping for a variety of structures. The report is available at <http://www.environment-agency.gov.uk/commondata/105385/w178.pdf>.

Design curves are based on small-scale (1:40, 1:20) laboratory tests performed on a variety of seawall configurations, beach slopes, and wave angles. Prototype measurements of overtopping have been made to validate the laboratory tests, but the results are seen as conservative, when compared with the Delft (TAW) guidance. Guidance was developed with pseudo-random waves described by a JONSWAP spectrum (the spectrum does not include a swell component). Therefore, its application is most applicable to unimodal, narrow banded seas (i.e., storm seas with a single spectral peak).

The guidance is summarized in Table 4. The required inputs are structure geometry and characteristics, significant wave height and mean wave period at the toe of the structure, height of the crest of the wall above the stillwater level, angle of wave attack, etc. (Note: The input stillwater level does not include wave setup.)

The procedures allow calculation of the mean overtopping discharge, as well as the maximum individual wave overtopping discharge (using a method similar to CEM).

A discussion of tolerable discharges (for seawalls, pedestrians, vehicles, buildings) is also presented; this appears to have been adopted by the CEM.

**Technical Advisory Committee for Water Retaining Structures)**

TAW (2002) provides revised procedures for calculating overtopping discharge for breaking and nonbreaking waves. This guidance supersedes the older guidance (which is included in the CEM). Higher-than-average overtopping discharge levels are recommended for structure design (see Figure 12). Procedures for computing overtopping volumes per wave are provided.

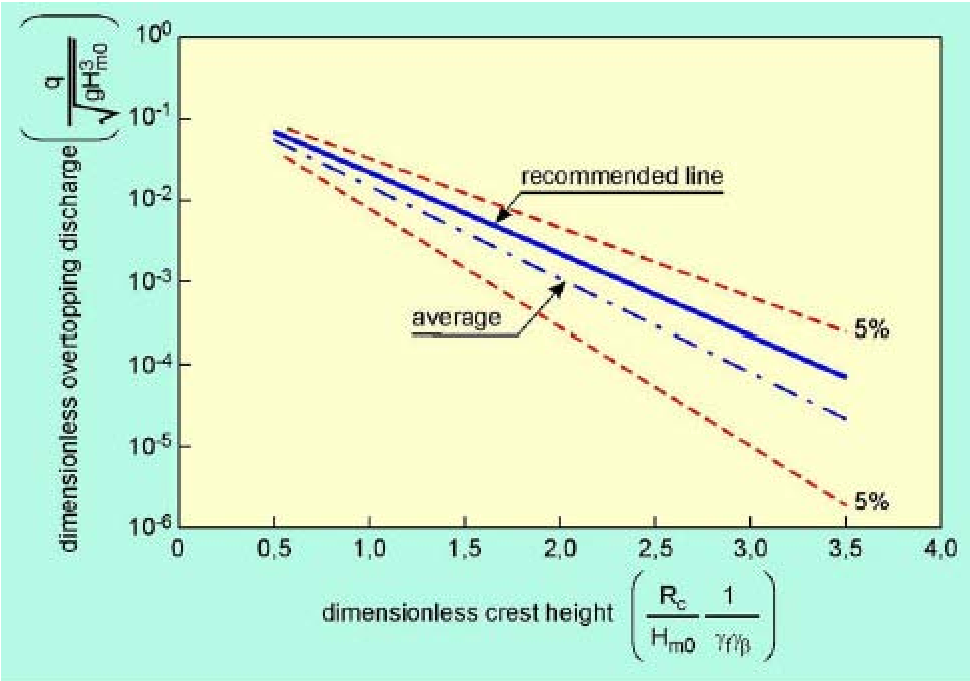
**Numerical Models**

See "Wave Runup" in Section 2.2.4. Other runup/overtopping models exist or are under development, such as the OTT-1d and OTT-2d models, which are part of HR Wallingford Ltd.'s ANEMONE (Advanced Non-linear Engineering suite of Models for the Nearshore Environment). More information has been requested.

**Table 4. Methods to Determine Wave-Overtopping Discharges for Various Structures**

| | Mean overtopping discharges Derived from small-scale laboratory experiments | | | Maximum individual wave discharge $V_{max}=a(\ln(N_{ow}))^{1/b}$ | Other |
|---|---|---|---|---|---|
| | Normal wave attack | Angled wave attack | Return walls | Number of overtopping waves | |
| Smooth impermeable simple and bermed slopes | Owen (1980): $Q^* = A \exp(-BR^*)$ where $Q^*$ is dimensionless overtopping rate and $R^*$ is dimensionless freeboard. *A* and *B* are empirically derived coefficients. | Banyard and Herbert (1995): $O_r= f(\beta)$ where $O_r$ is the ratio of overtopping at a given wave attack angle, β, compared to that under normal wave attack. | Owen and Steele (1991) is used to determine a discharge factor, $D_f$, which is the ratio of overtopping for a return wall that without a return wall. Dependent mainly upon the height of the wall and the incident overtopping rate. Banyard and Herbert (1995) provide a method for calculating $D_f$ with angled wave attack. | Owen (1982): $N_{ow}/N_w=\exp(-C(R^*/r)^2)$ where *C* is an empirical coefficient dependent upon slope. Determined by the number of waves with calculated runup greater than the crest height. For slopes between 1:1 and 1:4. Advice is given for angled wave attack. | Advice is given in HR Wallingford Ltd. (1999) for estimating rates for composite slopes and multiple berms |
| Rough and armored slopes | Owen (1980): $Q^* = A \exp(-BR^*/r)$ where *r* is a roughness coefficient based upon the relative runup performance of the different surfaces (e.g., smooth concrete, single layer armor unit, one layer of rock with impermeable core, two layers of rock). | Advice for angled wave attack is to use the method of Banyard and Herbert (1995) as for smooth slopes. | Bradbury and Allsop (1988), reanalyzed in HR Wallingford Ltd. (1999), used to determine $D_f$. Banyard and Herbert (1995) provide a method for calculating $D_f$ with angled wave attack. | Owen (1982): $N_{ow}/N_w=\exp(-CR^{*2})$ for slopes between 1:1 and 1:2. Advice is given for angled wave attack. | Permeable crest berms are accounted for with a reduction factor, $C_r$, based on the crest width. |
| Plain vertical walls | Allsop et al. (1995): Functions provided for calculating overtopping for both impacting and reflecting waves. | Franco (1996) gives $O_r$ function for reflecting waves only. | | HR Wallingford Ltd. (1999) gives functions to determine $N_{ow}$ for impacting and reflecting waves. Advice is given for angled wave attack | |
| Composite vertical walls (sitting on a mound) | Allsop et al. (1995): Overtopping is dependent upon whether the mound is large or small compared to the depth of water. | Advice for angled wave attack is to use the method of Franco (1996) as for plain vertical walls. | | | |

Source: HR Wallingford Ltd., 1999




Source: TAW, 2002

**Figure 12. Maximum wave overtopping by nonbreaking waves.**

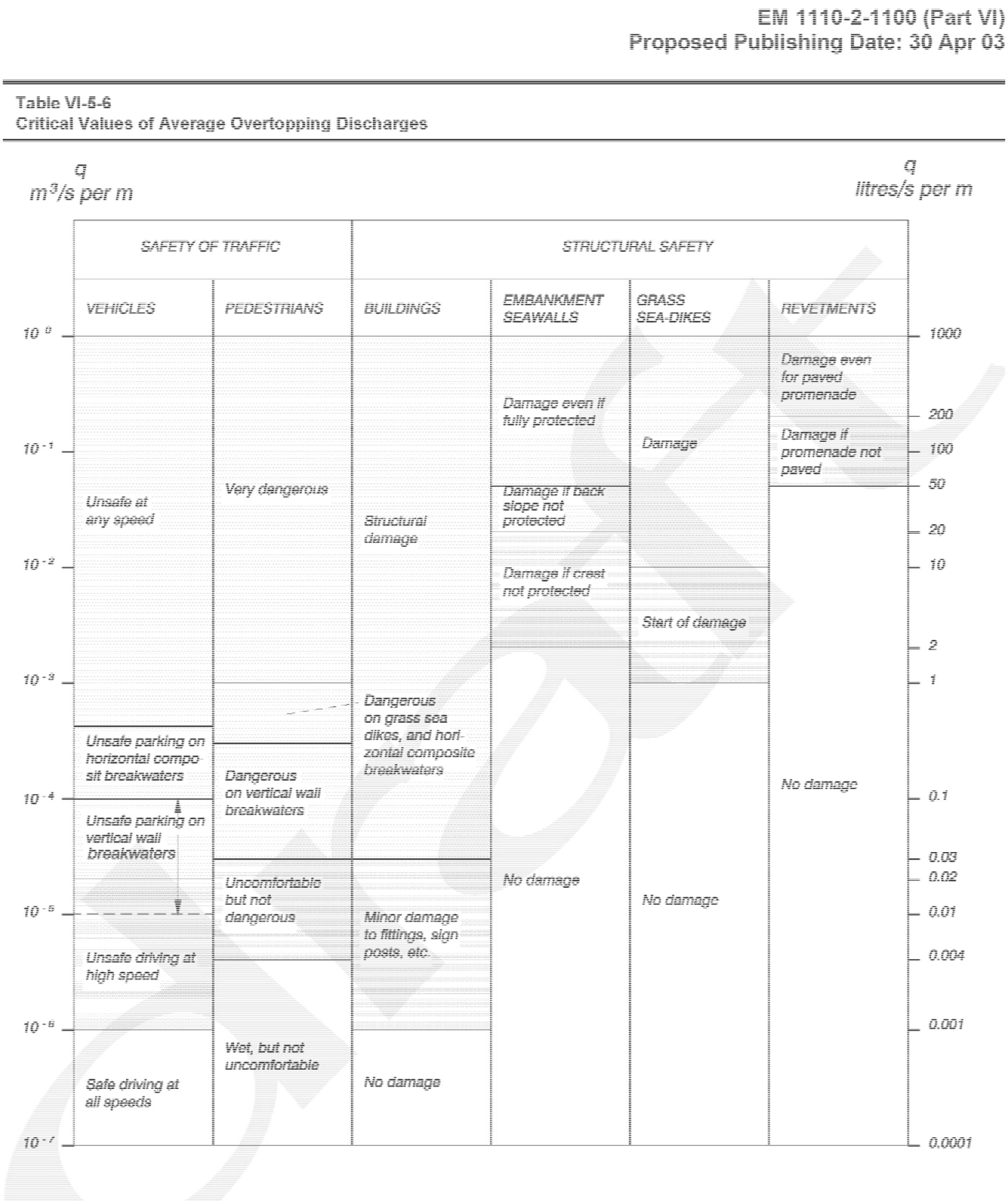
### *"Acceptable" Overtopping*

FEMA (2003) maps flood hazard zones landward of an overtopped barrier using the mean overtopping rate—the higher the rate, the higher the flood elevation/depth and the more hazardous the zone designation (see Table 2). The source of the overtopping rates separating the zones and depths is unknown.

Several authors and studies have attempted to define "tolerable" or "critical" rates of overtopping, which will vary with the object being affected by the overtopping, the distance from the overtopped barrier, etc. The CEM has assembled much of this information into a single figure, which is reproduced here as Figure 13. A more recent study (Geeraerts et al., 2003) provides field measurements of overtopping velocities and overtopping forces (on vertical walls, window glass, people [using dummies], and pipelines). These data should be reviewed to evaluate whether FEMA's overtopping rates are appropriate. (The building/wall/glass data should be especially pertinent for NFIP mapping purposes.) This work should be coordinated with the Hazard Zone Study Group.

### *Tsunami Overtopping*

FEMA (2003) does not contain any guidance for estimating overtopping of coastal structures by tsunamis. A cursory review of the literature located a USACE document, *Tsunami Engineering* (Camfield, 1980), which contains two empirical methods for estimating tsunami overtopping of




Source: USACE, 2003

**Figure 13. Critical values of average overtopping values.**

seawalls, the Kaplan (1955) method and the Wiegel (1970) method. These empirical methods are described below.

**Kaplan (1955) Method**

Under this method,

$$V = 21.65\left(Kh_s - h_w\right)^3 \big/ K^2 h_s \qquad (17)$$

where: $V$ = volume of overtopping the wall in cubic meters per meter ($m^3/m$) or cubic feet per foot ($ft^3/ft$);

$h_s$ = wave height at the shoreline in meters or feet;

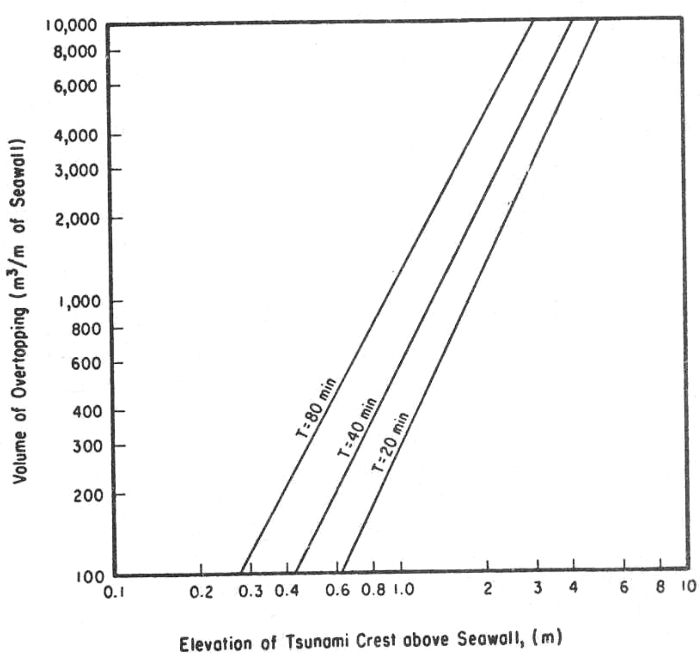
$h_w$ = height of wall in meters or feet; and

$K$ = $R/h_s$ where $R$ is the wall height required to prevent overtopping.

**Wiegel (1970) method**

Wiegel gives a relationship for estimating tsunami overtopping volumes that includes tsunami period and time dependence. The results of this relationship are summarized in Figure 14.

A more thorough literature search and coordination with the Tsunami Study Group should be undertaken for this topic.




Source: Wiegel, 1970

**Figure 14. Tsunami overtopping volume at a seawall.**

#### 3.2.5 Recommendations

Recommendations for Topic 13 are as follows (see Table 5, at the conclusion of this report):

1. Review available overtopping methods and models, and determine appropriate procedure(s) for calculating the mean overtopping discharge, including those over low-profile natural barriers, dune remnants, revetments, and vertical walls.

2. Evaluate procedures for calculating overtopping onto low bluffs with gently sloping, flat, or adverse slopes. Evaluate methods for determining ponding landward of overtopped barriers.

3. Review the current literature on "acceptable" overtopping, and work with the Hazard Zone Study Group to evaluate the overtopping rates that FEMA (2003) uses to identify flood hazard zones landward of an overtopped barrier.

4. Evaluate FEMA's current guidance, which limits the runup elevation to 3 feet above a barrier's crest elevation.

5. Coordinate work with the Tsunami Study Group.

#### 3.2.6 Preliminary Time and Cost Estimate for Guideline Improvement Preparation

Table 6 at the end of this report presents estimates of times required to accomplish the tasks in this topic.

#### 3.2.7 Related Available and Important Topics

Available and Important Topics related to Topic 13 are listed in Table 5, at the conclusion of this report.

# 4 IMPORTANT TOPICS

## 4.1 TOPIC 14: REVIEW AVAILABLE METHODS AND DEVELOP GUIDANCE FOR WAVECAST DEBRIS

### 4.1.1 Description of the Topic and Suggested Improvement

The existing *G&S* do not provide any guidance for estimating the hazards caused by wavecast debris, e.g., waterborne logs and wave-sprayed stone. Some guidance on estimating debris characteristics and its effects (on both upland structures and shore protection structures) may exist in the literature, however, and this should be reviewed. For example:

- Knowles and Terich (1977) described the hazards associated with logs and debris at Sandy Point, Whatcom County, WA (see Figure 15).

Source: Knowles and Terich, 1977

**Figure 15. March 1975 storm, drift logs driven into coastal houses at Sandy Point, Washington.**

- Edens (pers. comm., 1978) acknowledged the relative importance of floodborne debris in a memorandum that outlined a coastal flood study methodology for Puget Sound. The memorandum stated, "There was a general agreement…that damage due to water-borne logs and other forms of debris is the greatest danger to the destruction of property associated with the breaking wave of the magnitude that is experienced in Puget Sound."

- Kriebel, Buss, and Rogers (2000) reviewed the literature on impact loads caused by floodborne debris, including riverine debris, hurricane debris, tsunamis, and West Coast log debris. The report was background for a study on floodborne debris impacts, which helped plan the laboratory study of Haehnel and Daly (2002), and informed floodborne debris impact load calculations in *ASCE 7-02* (ASCE, 2002).

- Allan and Komar (2002) documented the inland penetration of small stone from a revetment at Cape Lookout State Park (see Figure 16).

Anticipated revisions to the *G&S* will include more discussion and guidance on defining hazards to insured property from wavecast debris, and will provide Mapping Partners with more information on how drift logs can contribute to the failure of coastal structures and shoreline erosion. Work on this topic will be coordinated with the Sheltered Water, Hazard Zone, Coastal Structures, Event Based Erosion, and Tsunami Study Groups.

Haehnel and Daly (2002) used a laboratory flume with logs (ranging in size from 380 pounds to 730 pounds) and traveling at speeds up to 4 feet per second to measure debris impact loads, and to develop a method for estimating floodborne debris impact loads.

Source: Allan and Komar, 2002

**Figure 16. Inland Penetration of small revetment stone during 1998-1999 winter, Cape Lookout State Park.**

#### 4.1.2 Description of Procedures in the Existing Guidelines

Current coastal flood study guidance from FEMA (2003) indicates that the landward extent of the VE Zone is established at a point where the runup depth drops below 3 feet (see Figure 5). The VE zone may be extended inland by 30 feet if overtopping rates exceed 1.0 cfs/foot (see Table 2).

Some accounts of flooding at flood insurance study communities along Puget Sound indicate that flooding, overtopping, and/or ponding can extend more than 30 feet inland at many locations, even during storms much less severe than the base flood (e.g., Phillip Williams & Associates, 2002). Thus, the current guidance may not capture all of those coastal areas subject to high hazards during the base flood.

#### 4.1.3 Application of Existing Guidelines to Topic—History and/or Implications for the NFIP

See Section 2.1.3.

#### 4.1.4 Alternatives for Improvement

Given the lack of guidance for determining hazards from wavecast debris, FIS contractors have had to develop methods to address these hazards during past flood insurance studies in FEMA Region X. Among these studies have been a 1989 sheltered water flood study in the harbor of Port Angeles, Washington, and the Sandy Point and Birch Bay studies in Whatcom County, Washington, in 2001 and 2002. More details on these studies are provided in Section 2.g. of the Sheltered Water Focused Study report.

The resulting methods represent simple efforts that were developed, applied, and approved by FEMA within existing flood study budget and schedule limitations at the time. These methods should be reviewed, refined, and considered for adoption as guidelines for defining flood hazards from wavecast debris.

#### 4.1.5 Recommendations

Recommendations for Topic 14 are as follows (see Table 5, at the conclusion of this report):

1. Review the current literature and quantify the significance of coastal flood damages from drift logs and wave-sprayed stone.

2. Review past flood insurance studies that have resulted in methods for defining flood hazards from wavecast debris, and refine these methods for possible incorporation into the *G&S*.

3. Incorporate results into flood zone mapping. Do not attempt to map debris specifically; map the water that carries the debris. Coordinate work with other Focused Study Groups as appropriate.

#### 4.1.6 Preliminary Time and Cost Estimate for Guideline Improvement Preparation

Table 6 at the end of this report presents estimates of times required to accomplish the tasks in this topic.

#### 4.1.7 Related Available and Important Topics

Available and Important Topics related to Topic 14 are listed in Table 5, at the conclusion of this report.

## 5 ADDITIONAL OBSERVATIONS

### 5.1 SEPARATING WAVE SETUP FROM WAVE RUNUP

FEMA (2003) methods currently add wave setup to the 1% water level for wave height (WHAFIS) calculations, but do not do so for wave runup calculations (also note that FEMA's event-based erosion calculations use the stillwater elevation without setup). This inconsistency results from the underlying data and methods used by FEMA to develop its wave height and wave runup procedures. In effect, FEMA has determined that its computed wave runup already includes a wave setup component.

In Phase 2 of the current project, Pacific Coast methods will be developed and wave setup calculations will be reconsidered. The issue of how wave setup is treated relative to wave runup, wave heights, and event-based erosion must be resolved in a consistent and sound manner during Phase 2.

## 5.2 IMPLICATIONS OF USING THE RESPONSE METHOD

The Event Selection Method is relatively easy (and appropriate) to employ along the Atlantic and Gulf of Mexico—there is a high correlation between storm surge and wave conditions, and combining the 1% stillwater elevation with the 1% wave conditions is appropriate in most situations.

In general, use of this simple procedure is not valid along the Pacific Coast (and along many sheltered shorelines on all coasts) where water levels and wave conditions are not highly correlated. In these cases, either the Mapping Partner must identify other water level–wave condition combinations (which can be difficult and subject to error), or resort to a statistical analysis of response. The Response Method may be preferable for FISs.

However, use of the response method to determine the 1% flood elevation (or 1% profile geometry) will likely introduce extreme complexity into the flood map revision process. Coastal map revision requests are usually submitted to and processed by FEMA based on a defined event and improved (or altered) topography. Methods should be sought to avoid requiring all map revision requestors to also use the Response Method. One approach might be to back-calculate a 1% event (or events) based on the results of the Response Method, and allow revisions to be based on the event(s). Obviously, the details need to be worked out and this procedure needs to be tested during Phase 2.

## 5.3 USE OF 2-D MODELS

Procedures currently approved by FEMA for use in coastal FISs include both simple 1-D approaches and more complex 2-D models. At present, the only approved wave runup procedures are 1-D procedures (e.g., RUNUP 2.0, ACES, CHAMP, GLWRM). 2-D models have been approved for storm surge calculations (e.g., RMA2, MIKE 21, FLOW2D) and for wave height modeling (e.g., RCPWAVE, MIKE 21 offshore and nearshore wave models), although use of the 1-D WHAFIS methodology is dominant for overland wave height calculations.

FEMA's Approved Models Committee has and will continue to evaluate other 2-D models for use by Mapping Partners. Undoubtedly, more and more 2-D models will be approved for FISs, including models that calculate wave runup and overland wave heights. The migration away from the transect approach will continue. Phase 2 of the current study should consider how 2-D models, especially those on the approved models list (<http://www.fema.gov/mit/tsd/en_coast.shtm>), can be incorporated into Pacific flood studies.

# 6 SUMMARY

Focused Study findings and recommendations for runup and overtopping are summarized in Table 5 below.

**Table 5. Summary of Findings and Recommendations for Runup and Overtopping**

| Topic Number | Topic | Coastal Area | Priority Class | Availability/ Adequacy | Recommended Approach | Related Topics |
|---|---|---|---|---|---|---|
| 12 | Runup and Overtopping | AC | H *(C)* | MIN | 1. Revise guidance to call for runup analyses for sandy beach, small dune shore type.<br>2. Review runup distributions for beaches and structures during El Niño, coastal storm, and hurricane conditions; review runup damages; evaluate use of $R_{50\%}$ and select alternate $R_{x\%}$ value (probably between $R_{33\%}$ and $R_{10\%}$ ) if $R_{50\%}$ understates the hazard.<br>3. Tsunami runup should be treated by runup procedures developed specifically for tsunami events (rely on Tsunami Study Group).<br>4. Investigate feasibility of interim procedure for modifying the results of RUNUP 2.0. | 11<br>16 |
| | | GC | H *(C)* | MIN | | |
| | | PC | C | MIN | | |
| | | SW | C | MIN | | |
| 11 | Runup and Overtopping | AC | H *(A)* | Y | 1. Evaluate expansion of “Oregon-type” and “CDIP-type” methods as interim Pacific runup method<br>2. Develop test scenarios for side-by-side comparisons of existing runup methods, models (give priority to Pacific and New England scenarios)<br>3. Perform comparisons and sensitivity tests, eliminate methods, models; identify appropriate runup methods, models by location, morphology and hydraulic conditions | 4, 5<br>7, 8<br>12<br>16<br>44-48<br>49 |
| | | GC | H *(A)* | Y | | |
| | | PC | A *(C)* | MAJ | | |
| | | SW | A | Y | | |
| 49 | Runup and Overtopping | AC | A | Y | Evaluate with other runup methods and models in Topic 11 work. | 11 |
| | | GC | A | Y | | |
| | | PC | A | Y | | |
| | | SW | A | Y | | |

| Table 5. Summary of Findings and Recommendations for Runup and Overtopping | | | | | | |
|---|---|---|---|---|---|---|
| Topic Number | Topic | Coastal Area | Priority Class | Availability/ Adequacy | Recommended Approach | Related Topics |
| 13 | Runup and Overtopping | AC | NE *(A)* | Y | 1. Evaluate existing methods and models for calculating mean overtopping rates.<br>2. Determine appropriate procedures for calculating overtopping at structures, remnant dunes, low-profile beaches and barriers.<br>3. Evaluate procedures for calculating overtopping at low bluffs.<br>4. Review literature for data on "acceptable" overtopping rates, revise landward flood hazard zones.<br>5. Review FEMA practice to limit runup elevations to 3 feet above barrier crests. | 11<br>12<br>14 |
| | | GC | NE *(A)* | Y | | |
| | | PC | A | Y | | |
| | | SW | A | Y | | |
| 14 | Runup and Overtopping | AC | H | PRODAT | 1. Review the literature and quantify the significance of coastal flood damages from drift logs and wave-sprayed stone.<br>2. Review past flood insurance studies that have resulted in methods for defining flood hazards from wavecast debris, and refine methods where appropriate. Incorporate results into flood hazard zone mapping, but do not attempt to specifically map debris (map the water that carries debris, not debris itself). | 6<br>13<br>18<br>20<br>22 |
| | | GC | H | PRODAT | | |
| | | PC | I | PRODAT | | |
| | | SW | I | PRODAT | | |

Key:
Coastal Area
AC = Atlantic Coast; GC = Gulf Coast; PC = Pacific Coast; SW = Sheltered Waters
Priority Class
C = critical; A = available; I = important; H = helpful
(Recommend priority italicized if focused study recommended a change in priority class)
Availability/Adequacy
"Critical" Items: MIN = needed revisions are relatively minor; MAJ = needed revisions are major
"Available" Items: Y = availability confirmed; N = data or methods are not readily available
"Important" Items: PRO = procedures or methods must be developed; DAT = new data are required;
PRODAT = both new procedures and data are required

| Table 6. Time Estimates for Runup and Overtopping Topics | | |
|---|---|---|
| Topic Number | Topic | Time (person months) |
| 12 | **Review Appropriateness of Using Mean vs. Higher Values for Runup and Overtopping** | |
| | Make final recommendation regarding appropriate $R_{X\%}$ value for use in wave runup calculations; coordinate with Tsunami Study Group (topic 16) | 1 |
| | Develop interim procedure for modifying the results of RUNUP 2.0 outputs (until RUNUP 2.0 is modified or replaced) | 0.5 |
| | TOTAL | 1.5 |
| 11 / 49 | **Review Runup Methods and Programs; Provide Explicit Guidance on Where Each Should Be Applied / Review WRUPTM (Available Wave Runup Program)** | |
| | Evaluate Oregon and CDIP methods for use as interim runup methods | 1 |
| | Develop test scenarios for side-by-side comparisons of existing runup methods, models (give priority to Pacific and New England scenarios); include sandy beach small dune scenario | 1 |
| | Perform comparisons, eliminate methods, models; identify appropriate runup methods, models by location, morphology, hydraulics; consider input condition uncertainties | 2 |
| | Coordinate work with Wave Setup and Wave Transformation groups – make sure required wave runup inputs are available and methods are consistent | 1 |
| | TOTAL | 5 |
| 13 | **Develop Improved Guidance for Determining and Mapping Overtopping Volumes** | |
| | Review available overtopping methods and models, and determine appropriate procedure(s) for calculating the mean overtopping discharge | 0.7 |
| | Evaluate FEMA's current guidance which limits the runup elevation to 3 feet above a barrier's crest elevation | 0.1 |
| | Evaluate procedures for calculating overtopping onto low bluffs with gently sloping, flat or adverse slopes. Evaluate methods for determining ponding landward of overtopped barriers | 1 |
| | Review the current literature on "acceptable" overtopping, and coordinate with the Hazard Zone Study Group | 0.2 |
| | TOTAL | 2 |
| 14 | **Review Available Methods and Develop Guidance for Wavecast Debris** | |
| | Review the current literature and quantify the significance of coastal flood damages from drift logs and wave-sprayed stone | 0.75 |
| | Review past flood insurance studies that have resulted in methods for defining flood hazards from wave-cast debris, and refine these methods for possible incorporation into the *G&S* | 0.75 |
| | TOTAL | 1.5 |

# 7 REFERENCES

## 7.1 PRINTED REFERENCES

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# Event-Based Erosion

## FEMA Coastal Flood Hazard Analysis and Mapping Guidelines Focused Study Report

February 2005


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**Appendix**



**Acronyms**

| | |
|---|---|
| BFEs | Base Flood Elevations |
| CCCL | State of Florida’s Coastal Construction Control Line |
| CCM | Connecticut Conference of Municipalities |
| CEDAS | Coastal Engineering Design and Analysis System |
| CERC | Coastal Engineering Research Center |
| CFR | Code of Federal Regulations |
| CHL | Coastal and Hydraulics Laboratory |
| D&D | Dewberry & Davis |
| DHI | Danish Hydraulic Institute |
| EBE | Event-Based Erosion |
| EBEACH | time-dependant, two-dimensional beach and dune erosion model |
| ERDC | Engineer Research and Development Center |
| FEMA | Federal Emergency Management Agency |
| FIRM | Flood Insurance Rate Map |
| FIS | Flood Insurance Study |
| HAZUS | Flood Loss Model |
| LIDAR | Airborne Light Detection and Ranging |
| NAS | National Academy of Sciences |
| NFIP | National Flood Insurance Program |
| NOAA | National Oceanic and Atmospheric Administration |

| | |
|---|---|
| PNW | Pacific Northwest |
| REFDIF | wave model |
| SBEACH | Storm-induced Beach Change numerical model |
| SF | square feet |
| SHORECIRC | Quasi-3D nearshore circulation model |
| SWEL | Stillwater Flood Elevation |
| TWG | Technical Working Group |
| USACE | U.S. Army Corps of Engineers |
| USGS | U.S. Geological Survey |

# 1 INTRODUCTION

This report provides recommendations for a program leading to improvement of the current Federal Emergency Management Agency (FEMA) Guidelines related to Event-Based Erosion (EBE) and develops preliminary time estimates to accomplish these improvements. Fourteen technical categories related to EBE were developed by the Technical Working Group (TWG) during the December 2003 Workshop. Some of the EBE category needs and priorities were revised during Workshop 2 in February 2004. Four of these topics are prioritized as "Critical" with two of the Topics (Topic Nos. 33 & 35) applied to all three geographic areas (Atlantic, Gulf, and Pacific Coasts), five were designated "Important", four were designated "Available," and two were "Helpful." Topic 39, Primary Fontal Dune, was moved to topics covered under Hazard Zones. All thirteen remaining Topics addressed by the EBE Team are listed below (*Items in parentheses were revised at WS-2, 02-26-02*) and discussed in this report. Erosion during tsunamis and erosion due to winds (aeolian erosion) are topics not considered in the Focused Studies.

EBE Topic priorities were categorized by The TWG in light of the project schedule, which allowed approximately six months for development of new guidelines for the Pacific Coast. Based on this practical consideration, topics were characterized as follows:

- *Critical* – topics that were considered important to improve coastal flood hazard analysis and mapping for the NFIP, that required significant effort to analyze or develop, but could be developed or resolved in six months or less.
- *Important* – topics that were considered important to improve coastal flood hazard analysis and mapping for the NFIP, that required significant effort to analyze or develop, and are likely to require more than six months to be developed or resolved.
- *Available* – topics that could be improved with relatively available data or procedures in less than six months.
- *Helpful* – topics that would be helpful to the NFIP, but were considered less significant or lower priority.

**Event-Based Erosion Topics and Priorities**
(Items in parentheses were revised at WS-2, 02-26-02)

| Topic Number | Category | Topic Description | Atlantic / Gulf Coast | Pacific Coast | Non-Open Coast |
|---|---|---|---|---|---|
| 30 | Geometric Techniques - PC | Review empirical geometric techniques and pre- and post-event data for CA, OR, WA; review OR setback methods, develop geometric techniques for pacific shorelines, including sea cliff, bluff, dunes beaches | -- | C | -- |

**Event-Based Erosion Topics and Priorities**
(Items in parentheses were revised at WS-2, 02-26-02)

| Topic Number | Category | Topic Description | Atlantic / Gulf Coast | Pacific Coast | Non-Open Coast |
|---|---|---|---|---|---|
| 31 | Bluff Erosion - AC/GC/(PC) | Add/revise *G&S* language regarding bluff erosion in Atlantic/Gulf areas – better descriptions and discussions needed | A | (A) | (A) |
| 32 | Geometric Method for Bluffs - AC/GC/(PC) | Develop geometric method for bluff erosion in Atlantic/Gulf areas | I (A) | (A) | (A) |
| 33 | Cobble/ Shingle Effects | Add *G&S* descriptions/discussion regarding effect of cobble/shingle materials (including sediment mixtures/layers) on geometric erosion techniques | C | C | C |
| 34 | Cobble/ Shingle -Geometric Method | Develop improved geometric methods which consider cobble/shingle effects | I | I | I |
| 35 | Erosion – Sheltered Waters | Add *G&S* descriptions/discussions regarding erosion assessments in sheltered areas | (C) | (C) | C |
| 36 | Geometric Method – Sheltered Waters | Review data and develop geometric methods for determining eroded profiles in sheltered areas | (I) | (I) | I |
| 37 | Review 540 SF Criterion | Expand database from which 540 was determined; review use of median value | I | -- | -- |
| 38 | Process-Based Approach | Develop assessment procedures that consider temporal and longshore effects/variability | I | I | I |
| 39 | PFD | Develop better definition of landward limit of PFD (used for V zone limit);gather and evaluate MA CZM and other approaches NOTE: Topic 39 moved to Hazard Zones | C *(H)* 39 moved to Hazard Zones | I *(H)* 39 moved to Hazard Zones | I *(H)* 39 moved to Hazard Zones |
| 40 | Vertical Erosion Depths | Maintain data and make available for use in building performance and insurance tasks | H Nominal Needs | H Nominal Needs | H Nominal Needs |
| 41 | Long-Term Erosion | Revise *G&S* D.5 language and put warning on the FIRM to state that "present methods may understate/overstate future flood hazards; reference CCM and other reports; discuss implications of study data selection" (e.g., older data may have better resolution, but be out of date as a result of erosion, sea level change, effects of subsidence, etc.) | A | A | A |
| 42 & 43 | Nourished Beaches | Ensure clarity in *G&S* that references FEMA policy statement regarding treatment of nourished beaches | A | A | -- |

Key: C = critical; A = available; I = important; H = helpful
(Recommend priority italicized if focused study recommended a change in priority class)

## 1.1 THE EBE TEAM AND APPROACH

The EBE study team consists of Kevin Coulton, Bob Dean, Darryl Hatheway, Maria Honeycutt, Jeff Johnson, Chris Jones, Paul Komar, Chia-Chi Lu, Ron Noble, Trey Ruthven, and Dick Seymour. Robert MacArthur served as Team Leader for this effort and Bob Dean provided significant guidance and review.

In order to provide structure to our efforts and to avoid unnecessary duplication of effort, the following approach was employed. The Team Leader assigned lead technical and writing responsibilities for specific topics to the following individuals: Paul Komar, Trey Ruthven, and Robert MacArthur (Topics 30-34), Ron Noble and Chia-Chi Lu, (Topics 35, 36, 38, and 41), Chris Jones (37, 40, 41, 42, and 43). All EBE Team Members contributed significant information of which they were uniquely aware, critiqued and contributed to the draft write-ups, and accomplished specific components of the overall effort leading to this report.

## 1.2 PRESENT FEMA GUIDANCE ON EVENT-BASED EROSION RELATED TO ALL PRIORITY CATEGORIES

Prior to 1986, specific FEMA guidance and objective procedures were not available for treating the effects of erosion in coastal flood hazard assessments. Studies by Hallermeier and Rhodes (1988) and Dewberry & Davis (1989) developed and discussed the method recommended in present FEMA "*Guidelines and Specifications for Flood Hazard Mapping Partners: Appendix D – Guidance for Coastal Flooding Analyses and Mapping*" (April 2003), hereinafter referred to as *Appendix D*. Present geometric erosion assessment methods in *Appendix D* rely on empirical results from an assessment of 38 notable dune erosion cases documented primarily along the Atlantic and Gulf Coasts of the U.S. Present methods apply only to coastal sandy dunes and erodible bluffs according to the FEMA criteria associated with the definition of a primary frontal dune and only apply to coasts along the Atlantic, Gulf, or Great Lakes. In order to enact and adopt the procedures recommended by Dewberry & Davis, FEMA published new rules and definitions in the May 6, 1988 *Federal Register*, pages 16269-16273 (that became effective on October 1, 1988), which included the following revised definitions in 44 CFR sec. 59.1 and 65.11 of the National Flood Insurance Program (NFIP) regulations:

> "*Primary frontal dune* means a continuous or nearly continuous mound or ridge of sand with relatively steep seaward and landward slopes immediately landward and adjacent to the beach and subject to erosion and overtopping from high tides and waves during major coastal storms. The inland limit of the primary frontal dune occurs at the point where there is a distinct change from a relatively steep slope to a relatively mild slope." (From 44 CFR sec 59.1)
>
> *Evaluation criteria.* Primary frontal dunes will not be considered as effective barriers to the base flood storm surges and associated wave action where the cross-sectional area of the primary frontal dune, as measured perpendicular to the shoreline and above the 100-

year stillwater flood elevation and seaward of the dune crest, is equal to, or less than, 540 square feet [from 44 CFR sec 65.11 (b)].

The adopted procedure established a relationship of dune erosion area (and volume as a function of beach length) to storm intensity as measured by flood recurrence interval. For the 1-percent-annual-chance storm, *Appendix D* determined that, “to prevent dune breaching or removal, an average cross-sectional area of 540 square feet is required above the SWEL and seaward of the dune crest.”  This standard for dune cross section continues to occupy a central role in erosion assessment procedures (also known as the 540 SF criterion). Material characteristics and storm duration are empirically included in this simple geometric relationship; however, application of this criterion may be limited to the coastal region for which it was developed.

Previous research by the Corps of Engineers Coastal Engineering Research Center (CERC, 1987) determined that quantitative (process-based) numerical models had not been developed to the point necessary for reliable application in FEMA-type assessments and mapping projects. Therefore, it was recommended by CERC that only empirically based models (for storm-induced or event-based erosion) produced reasonable results with a minimum of effort and input data. Further, it was recommended that this approach be used even though it has certain limitations, and that dune overwash processes are poorly documented and unquantified.  FEMA performed additional investigations on erosion models and procedures before adopting the 540 SF criterion in 1988, but decided to employ these very simplified procedures for erosion assessments based upon empirical data from historical storm-induced erosion events.  These procedures were considered capable of reasonable depiction of documented effects of extreme storms (resulting from either Atlantic and Gulf hurricanes, or extratropical storms such as northeasters) and were judged appropriate for treating dune erosion in Flood Insurance Studies (FISs) for coastal communities along the Atlantic and Gulf Coasts.  As presented above, FEMA included a new section in 44 CFR sec. 65.11 of the NFIP regulations, identifying an (average) cross sectional area of 540 square feet as the basic criterion to be used in evaluation whether a primary frontal dune will serve as an effective barrier during a 1-percent-annual-chance (100-year) flood event. Figure D-4 from the *G&S* provides a flowchart summarizing FEMA’s present approach for assessing the effects of erosion during a Coastal Flood Insurance Study.

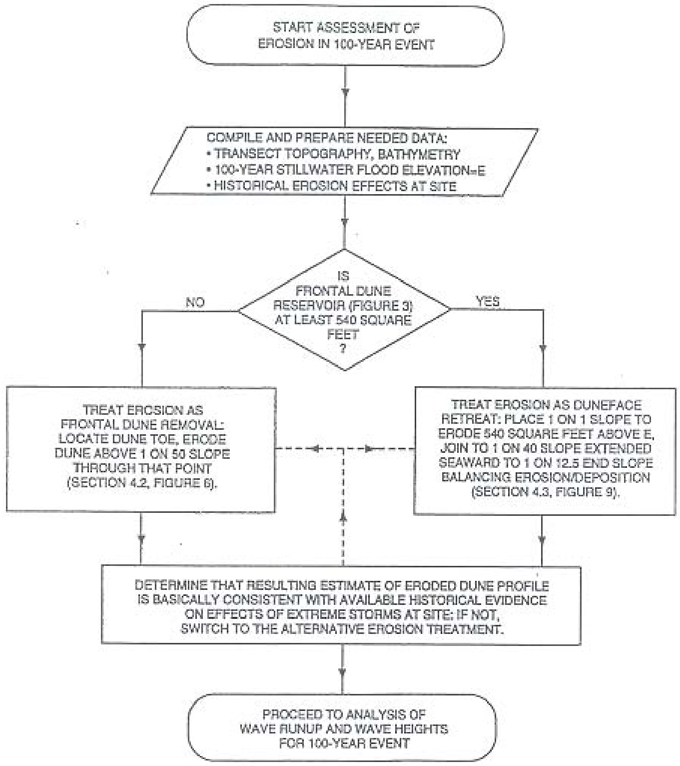



Figure D-4. Flowchart of Erosion Assessment for a Coastal Flood Insurance Study

The following Figures 1 and 2, summarize the "frontal dune reservoir" and "dune removal and dune retreat geometries" according to present FEMA criteria.

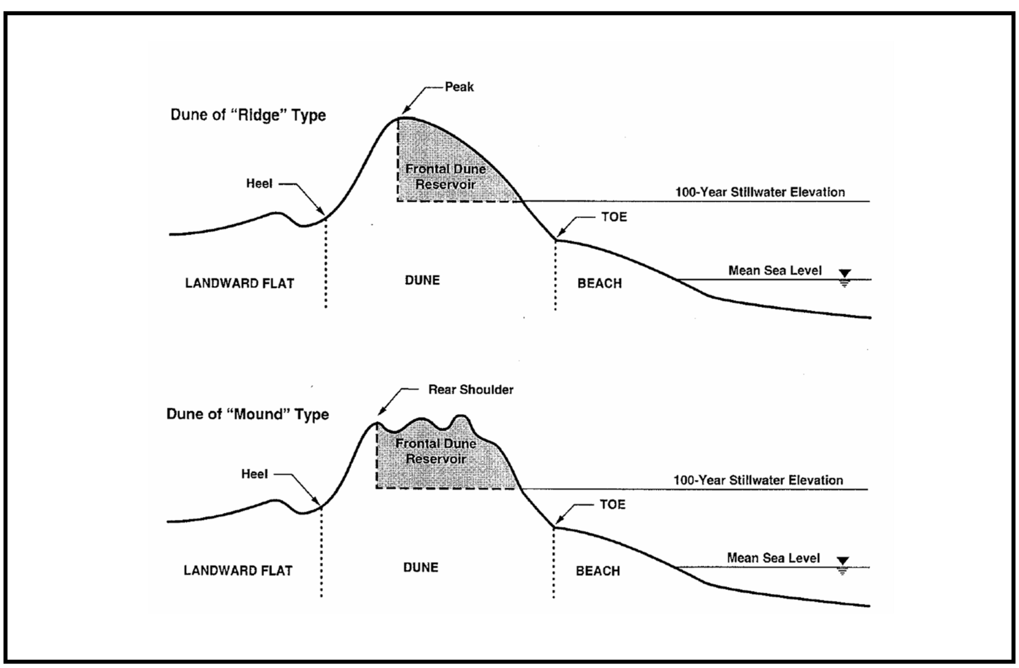



**Figure 1. Definition sketch of frontal dune reservoir (from FEMA, 2003).**

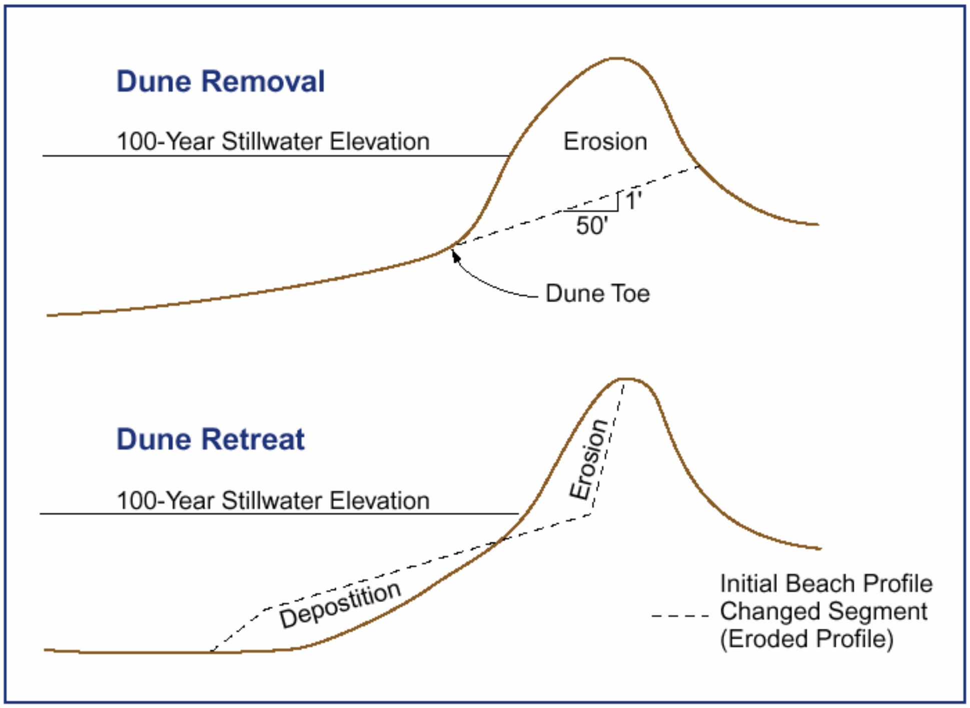
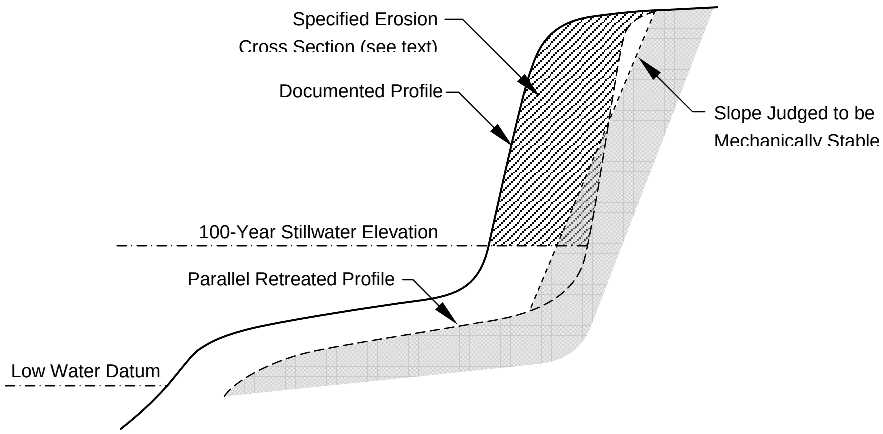
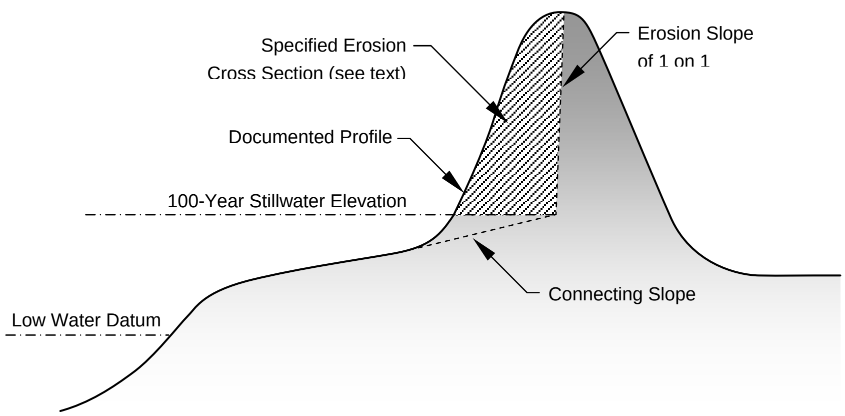



**Figure 2. Current FEMA treatment of dune retreat and dune removal.**

*Appendix D* also provides similar recommended procedures for assessing the effects of event-based erosion in the Great Lakes regions (see Section D-3). In the case of the Great Lakes, the average cross sectional area of the dune reservoir (above the 100-year stillwater level) required to prevent dune breaching and removal is 270 square feet for Superior, Michigan, Huron and Erie, and 190 square feet for Lake Ontario. These criterion were developed from observed data in each of the geographic areas. These same values are used to estimate bluff retreat in those locations, see Figures D-38 and D-39 from Appendix D, shown below. Although presented here, the Great Lakes geometric erosion assessment may be a lower value than expected for Pacific erosion events or other Sheltered Water locations. However, these values developed for the Great Lakes show how different material types and wave climates affect the limiting dimensions of the frontal dune reservoir for retreat and removal in different regions and will not be considered further in this study.




**Basic Erosion Considerations for Coastal Bluff Provides Shaded Shore Profile for Great Lakes Base Flood. (from Figure D-38, Appendix D.)**




**Basic Erosion Considerations for Coastal Sand Dune Provides Shaded Shore Profile for Great Lakes Base Flood. (from Figure D-39, Appendix D)**

Sheltered areas and coastlines with mixed grain-size, cobble-shingle or muddy bottom beaches and dunes are not explicitly covered by *Appendix D*. It is also clearly stated in Section D.4 that: "No FEMA guidance documents have been published for Pacific Ocean coastal flood studies. Guidance is to be developed based on existing methodologies recommended by FEMA and coastal states for coastal analyses in the Pacific Ocean. Mapping Partners that are undertaking a flood hazard analysis of a Pacific Coast site should consult with the FEMA Regional Project Officer for that area." Phase 2 of the present FEMA mapping project will develop such guidance for the Pacific.

Therefore, present *G&S* do not provide specific guidance for assessing event-based erosion in coastal areas of the Pacific, Sheltered Waters on either coast, or non-sandy beach and dune coastal areas, and provide only simplified empirical-based geometric relationships for the Atlantic and Gulf. Therefore, the following sections of this report discuss specific topics deemed important for consideration by the TWG during the December 2003 and February 2004 Workshops in order to improve the present guidelines for the Atlantic and Gulf and to develop recommendations for the development of new guidelines for the Pacific Coast and all Sheltered Water coastal areas of the continental U.S.

### 1.2.1 Factors in Beach, Dune and Bluff Erosion

To set the stage for discussions that follow on the various topics, it is useful to consider key characteristics of the erosion processes. The main erosion related factors are: 1) The forcing elements that include the time histories of the wave characteristics, currents and water levels, and runup, and 2) The response elements that include the physiographic setting and the beach and dune/bluff characteristics. The elevated water level places the profile out of equilibrium and the waves provide the energy and the offshore extent of sand redistribution to result in a reestablishment of equilibrium. If the forcing elements occur over a relatively short time period, there may not be enough time for the erosion processes to reestablish equilibrium. This is especially the case if the bluff is composed of durable material in which the processes proceed on more of a geological time scale than a storm event time scale. Some researchers prefer to relate periodic changes in beach and dune profiles to the exceedance of an erosion threshold within the beach setting with a new resultant state dependant on the forces imposed on it. This concept recognizes the importance of antecedent beach conditions when a storm event occurs and that erosion thresholds will vary between events of different duration, intensity and location.

Because the physics are the same for erosion on all types of shorelines, it is desirable in further developments and recommendations related to FEMA applications, to attempt to develop and recommend procedures that embody the same fundamental structure, and are applicable to different physiographic regions. In brief, this requires that the following considerations be included: 1) Physiographic setting, 2) Sediment characteristics across the active profile, 3) Time histories of wave and storm tide characteristics, and 4) Local or regional oceanic (El Niño) or topographic (recent tectonic adjustments) characteristics that may affect the study area. Within this common framework, it will be necessary to make assumptions and approximations in which, depending upon local conditions, some factors can be neglected; however, the fundamental structure of the erosion process will be consistent for all applications.

# 2 CRITICAL TOPICS

As noted, outcomes from the December 2003 Workshop identified four "Critical Topics" for Event-based erosion: 1) Topic 30 for the Pacific, 2) Topic 33 for all coastal regions, 3) Topic 35 for non-open coasts, 4) and Topic 39 for the Atlantic coast. (NOTE: Topic 39 is now covered in the Hazard Zones Focused Study.) Workshop 2 (February 23-26, 2004) adjusted the priorities and needs to those now listed in Table 1, with Topic 35 critical for *all three* geographical areas, and Topic 39 now being covered by the Hazard Zones Technical Working Group.

## 2.1 CRITICAL TOPIC 30: GEOMETRIC EROSION ASSESSMENT FOR THE PACIFIC

### 2.1.1 Description of Topic and Suggested Improvement

Dunes backing beaches along some of the U.S. coasts can reach sufficient elevations that they provide a barrier to the flooding of backshore areas. However, these dunes can be subject to significant erosion during extreme storms, potentially leading to their failure as a barrier. FEMA procedures divided EBE effects into two basic categories, retreat and removal (failure), as was shown in Figure 2.

The primary factor controlling the basic type of dune erosion is the pre-storm cross section lying above the 1-percent-annual-chance SWEL (frontal dune reservoir). This is recognized in the FEMA methodology as applied to the Atlantic and Gulf coasts, which first assesses the vulnerability of the dunes to erosion failure (using the 540 $ft^3$/ft dune-volume criterion). If the median dune volume above the 100-year SWEL is greater than 540 $ft^3$/ft of dune length, then the dune does not fail during the event but retreats with the dune remnant remaining as a surge and wave barrier. If there is less than this available volume per foot of dune length it is assumed that the dune will be breached and will fail, and will be washed away, resulting in a new "eroded beach profile" for use in calculating: 1) wave propagation landward, 2) surf transformation at the shoreline, and 3) wave runup at the coast. According to *Appendix D* guidance on dune retreat and dune removal, "different treatments for erosion are required for these two conditions because no available model of dune erosion suffices for the entire range of coastal [settings] situations."

Similar problems with dune erosion processes exist along the Pacific Coast, although dunes are a less common feature in this region. No FEMA methodology has been established for the Pacific coastal environment where shoreline characteristics are more complex and where the cumulative effects of multiple storms must be considered rather than the single extreme storms typically found along the Atlantic and Gulf. Methodologies have been developed for application to West Coast conditions, but have been directed primarily toward the establishment of erosion hazard setback lines rather than focusing on short-term EBE impacts.

Therefore, new improved methods for assessing coastal erosion hazards according to FEMA standards and guidelines for conducting such assessments are required for the Pacific region. It is also agreed that improved methods are needed for the Pacific, Atlantic, and Gulf, especially where beaches, dunes, and bluffs are comprised of sediment materials other than uniform sand.

### 2.1.2 Description of Procedures in the Existing Guidelines

The methodology employed by FEMA to assess the potential extent of primary frontal dune erosion during a major storm on the U.S. Atlantic and Gulf Coasts is based on analyses developed in the 1989 report by Dewberry & Davis (D&D), with a published summary by Hallermeier and Rhodes (1988). There are two components to their analyses supporting the FEMA methodology, first the establishment of the average 540 $ft^3/ft$ frontal dune volume as that required to survive the estimated 100-year stillwater flood elevation (measured tide) produced by either a hurricane or extratropical storm (northeaster), and second to establish an erosion profile needed for subsequent wave height and wave runup analyses. An additional analysis deals with the case where the dune is breached and failure of the dune as an effective barrier to storm surge and wave propagation occurs, leading to backshore flooding and wave effects. However, this application of the dune removal geometric erosion assessment technique has received far less evaluation and testing on the West Coast because there are very few Pacific Coast study areas with significant dune formations protecting highly developed coastal areas. Additional discussions of "geometric erosion assessment techniques" found in the Existing Guidelines are presented in Section 1.2, "Present FEMA Guidance on EBE Related to all Priority Categories, Critical, Important and Available," above.

### 2.1.3 Application of Existing Guidelines to Topic – History and/or Implications for NFIP

As previously stated, there is no specific *Appendix D* guidance presently available for the Pacific, and the existing empirical database of pre- and post-storm erosion events used to develop the 540 SF Criterion are specific to the Atlantic and Gulf coasts and are not intended to be applied in the Pacific. Therefore, new improved and specific methods for assessing coastal erosion hazards, according to FEMA standards and guidelines for conducting such assessments, are required for the Pacific region.

#### *Application of Geometric Models on the U.S. Pacific Coast and Their Use for Establishing Setback Distances (Erosion Hazard Zones)*

Although there is no established FEMA methodology for dune-erosion assessments on the U.S. Pacific Coast, methodologies have been developed for the evaluation of coastal setback distances on the coast of the Pacific Northwest (Oregon and Washington), herein referred to as PNW methods. With appropriate modifications, these methodologies could be adapted to the FEMA applications. The California Coast represents a more complex problem, exacerbated by significantly greater coastal development, with wide variations in exposure to storms and a variety of geological settings and material characteristics. California has no adopted program with formal coastal setback distance methodologies that can guide FEMA efforts.

There are two stages in the PNW method, the first to determine the "design" erosion event, the second yielding a projected dune erosion or susceptibility assessment for potential sea cliff erosion. Considerable research has been undertaken documenting the processes responsible for the erosion of beaches and backshore properties in the Pacific Northwest (Komar, 1997; Komar, et al., 1999 and 2002). This coast experiences high wave energies generated by intense extratropical storms that cross the North Pacific, with landfall generally occurring on the coast from Northern California to British Columbia. The most recent wave climate assessment has yielded approximately a 16-meter deep-water significant wave height for the 100-year storm, and with winter storms frequently producing 10-meter significant wave heights. Of interest and concern, research has shown that the wave heights along the West Coast have been increasing during the 25- to 30-year records provided by buoy data, with studies of the intensities of the storm systems indicating that the increase likely extends back to at least 1950 (Allan and Komar, 2000, 2001; Graham and Diaz, 2001). Based on such studies of West Coast wave conditions and their climate controls, a fairly firm basis exists for a determining wave conditions for establishing the design erosion event.

While an extreme extratropical storm can occur during any winter, the overall greatest erosion impacts on the West Coast have occurred during major El Niños like those in 1982–83 and 1997–98. It is well documented that on average the storm-wave heights are greatest during El Niños, this increase being most significant on the coast of Central and Southern California because of the more southerly tracks of El Niño storms (Seymour, 1996). Also important are the elevated tides during an El Niño, produced by reversals in the average wind stress across the Pacific, the thermal expansion of the warmer water, and the geostrophic effects of stronger northward flowing currents. Monthly mean water levels are elevated by about 0.3 meter in Southern California to 0.5 meter on the coast of the Pacific Northwest, and are maintained by those amounts throughout the entire El Niño winter.

From this, the assessment of the design erosion event for application in the Pacific Northwest is represented by the occurrence of an extreme storm during a major El Niño winter. The methodology for this assessment was developed by Ruggiero et al. (2001), a procedure that in essence involves the summation of the processes that determine the total water level at the shore—the sum of the predicted tide, the effects of the several processes that elevate measured tides above predicted levels during El Niño, and the addition of the surge and swash runup produced by a storm. Ultimately of importance is the total water level achieved during the storm in comparison with the elevation of the dunes or bluffs.

In the assessments of setbacks for the long-term protection of homes on the Pacific Northwest coast, Komar included the local relative sea level and its potential rise during the next 50 to 100 years, and also an increase in the storm surge and swash runup levels that could result from a continued increase in storm intensities and generated wave heights at the rates experienced during the past 25 years. Having defined the design erosion event, the next step in the analysis to establish recommended setback distances is the application of a geometric dune-erosion model that has been adapted to conform with the conditions found on the Pacific Northwest coast.

Those conditions include: (1) most beaches in the Pacific Northwest are "dissipative" as defined in the classification of Wright and Short (1983), that is, they are low in slope with a wide surf zone, being effective in dissipating the energy of the waves so the beach profile does not experience marked changes in sand levels during storms and through the seasons; (2) during major storms the surf zone is hundreds of meters wide and the waves and currents rapidly disperse the sand eroded from the dunes; and (3) the beach face within the swash zone at the base of the dunes has a nearly uniform slope (typically about 1:25), which is maintained and extended landward as the dunes are eroded. These observed conditions made it possible to formulate a simple geometric model (Komar et al. 1999). Like geometric erosion models adopted by the Dutch (Vellinga, 1982, 1983, 1986), it is accepted that the cut back of the dunes will originate at the level reached by the water, but rather than focusing on the storm surge which is only a minor factor on the Pacific Northwest coast, the total water level as analyzed by the Ruggiero et al. (2001) model governs, with the level reached by the intense wave-swash runup being a particularly important factor. Unlike the Dutch model, the Komar et al. model is not concerned with the conservation of sand because the sand released by the dune erosion is rapidly dispersed, rather than raising the elevation of the beach immediately in front of the dune. Quite the opposite, the geometric model includes a factor that accounts for the local lowering of the beach in that embayments eroded by rip currents into the beach face have been observed to be important to the zones of maximum dune erosion, and therefore could be included in the analysis as a lowered beach elevation.

### 2.1.4 Alternatives for Improvement

In that the level of a Pacific Northwest beach within the inner surf zone undergoes little change during the erosion event, the Komar et al. (2002) geometric procedure simply extends that slope landward, cutting away the dunes up to the total water-level elevation established by the design storm event. Accordingly, the derivation yields the simple formulation:

$$DE_{\max} = \frac{(WL - E_J) + \Delta BL}{S} \tag{1}$$

Where $DE_{\max}$ is the horizontal distance of dune erosion, $WL$ is the total water level achieved by the design event relative to the elevation of the toe of the dunes prior to the erosion, $E_J$ is the elevation of the beach-dune junction and $\Delta BL$ is beach level change or vertical shift in the profile that might be produced by a rip-current embayment or other process. $DE_{\max}$ represents the “maximum dune erosion” and forms the horizontal leg of a right triangle, while the other parameters combine to determine its vertical leg, so they are related by $S = \tan\beta$ the slope of the beach within the swash zone fronting the dunes. Figures 3 and 4 provide schematic sketches of these variables.

This model yields the maximum potential dune retreat for the total water level *WL*, in that it does not account for the duration over which the water may only reach the design erosion level and the erosional response will lag behind the causative processes. Attempts to assess this lag through application of process-based models for beach profile and dune erosion, specifically SBEACH (Larson and Kraus, 1989), EBEACH (Kriebel and Dean, 1985) and COSMOS (Nairn and Southgate, 1993) were not successful. It was found that these models are inadequate in applications on the Pacific Northwest coast due to their having been calibrated to much lower energy beaches (or in laboratory wave tanks), and in particular because processes important to the erosion of West Coast beaches are not included (e.g., long-wave infragravity surf motions, important on dissipative beaches). Thus, it is possible that if the hydrodynamic variables (infragravity processes) were better defined, process models could be applied. These models not only predicted less dune retreat during a storm than the geometric model, they also under predicted the actual extent of dune erosion that has been experienced during major storms. The USACE Waterways Experiment Station is presently evaluating SBEACH and other process-based models to see if they can be modified for reliable applications on the West Coast. Further detailed discussion of SBEACH and EBEACH are provided in Section 3.

The use of the Komar geometric model to assess the potential extent of dune erosion and to establish setbacks has been supported by tests under extreme storm conditions experienced on the Pacific Northwest Coast in recent years. The winters of 1997–98 and 1998–99 caused unusually extreme erosion and thus provided the opportunity to test these methodologies developed to assess the potential extent of foredune erosion. Before and after beach and dune profiles were obtained at a number of sites, documenting the resulting extent of the cumulative erosion. Confirmation of the calculated total water levels, *WL*, resulting from the combined processes, was provided by general agreement with the surveyed elevations at the seaward toe of the eroded duneface. This also represented partial confirmation of the geometric dune-erosion model in that a basic assumption in its derivation is that the total water level controls the elevation at which the dunes are cut back. However, as expected, it was found that the surveyed horizontal retreat of the dunes was less than the calculated $DE_{max}$. On the other hand, under the "one-two punch" of those successive winters, with the last storm in the series having been the largest and yielding the highest total water levels at most coastal sites, the resulting surveyed cumulative dune retreat increased to the extent that it nearly reached the calculated $DE_{max}$. Thus, although one storm may not have sufficient duration to produce dune erosion to the extent calculated with the geometric model, a series of storms could, justifying the use of the evaluated $DE_{max}$ in coastal management to establish setback distances. This emphasizes the need to incorporate the effect of storm duration in the models as was done by Ruggerio, et al. (2001) by calculating the number of hours per year that the 2% runup exceeded the dune toe elevation. Ruggerio et al.'s estimate of the 2% runup elevation includes the vertical component of runup as well as setup and the swash runup elevation. Komar et al. (2002) have shown good results and agreement with measured beach profiles for applications of the relationships shown in Equation 1 and Figures 3 and 4 at sites along the Oregon Coast for a wide range of beach slopes.

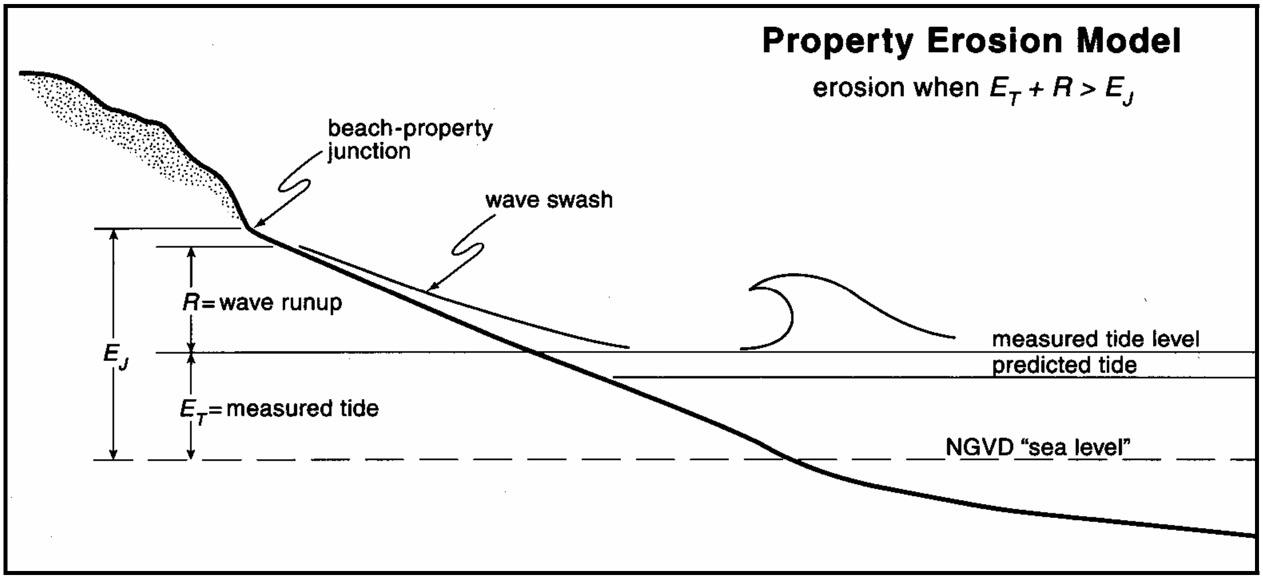



**Figure 3. Pacific Northwest erosion model (Komar et al., 2002).**

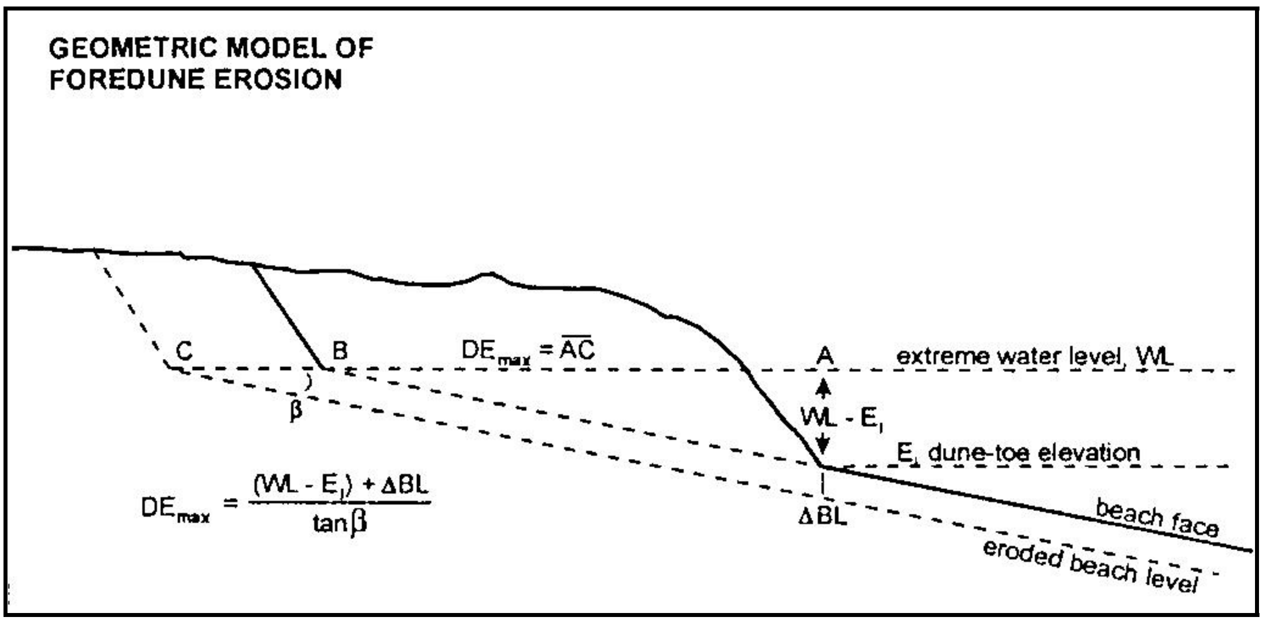



**Figure 4. Pacific Northwest geometric dune erosion model (Komar et al., 2002).**

This methodology has been applied to establish setback distances along significant stretches of the Oregon Coast, with the geometry of the dunes and fronting beach determined from LIDAR surveys, together with spot checks from ground surveys. Other methods by Judge et al. (2003) may have applicability to sandy Pacific Coast beaches and dunes. Their approach includes use of a new dune vulnerability indicator that shows promise for improving predictions of dune failure during hurricanes. At this time, this focused study cannot present a discussion on studies to test Judge et al.'s methods for the Pacific. Such tests are recommended.

Two essential items are needed for effective estimates of beach and dune erosion. It is essential to have a reliable definition of the most likely 100-year storm event and SWEL, and it is important to understand the primary geomorphic and geologic characteristics of the study site in order to address the dominant erosion processes responsible for changes in beach, dune or bluff geometry during the 100-year event. Seasonal winter beach profile data should be gathered and examined for each study site to determine whether there is an average, or most-likely, profile that represents winter beach conditions for the site.

Note that dune backed beaches represent only a small fraction of the California coastline and that many of these are further backed by bluffs or cliffs of differing erodibility and may be undeveloped. As a result, coastal engineering efforts to develop predictive tools for erosion effects in California have concentrated on losses to low slope beaches and cliff or bluff damage.

In relating the geometric model by Komar and colleagues to the framework discussed previously, it appears that a main difference is that there is no explicit dependency on duration or specific material properties. It is possible that some of the principles elucidated in process-based models combined with characteristics of the Komar geometric model could yield improved model transient predictions and a model consistent with the desired consistent framework.

### 2.1.5 Conclusions for Topic 30 (Geometric Erosion Assessment for the Pacific)

Following are key points and conclusions related to the evaluation of geometric techniques for assessing erosion effects on dune-backed beaches along the Pacific Coast and brief descriptions of possible alternatives for improving these methods:

- Existing and new guidelines need to clearly state that EBE is “storm induced erosion.”

- There is no specific *Appendix D* guidance presently available for the Pacific. The existing empirical database of pre- and post-storm erosion events used to develop the 540 SF criterion are specific to the Atlantic and Gulf coasts and are not intended to be applied in the Pacific. Therefore, new improved and specific methods for assessing coastal erosion hazards, according to FEMA standards and guidelines for conducting such assessments, are required for the Pacific region.

- Studies by Kuhn and Shepard (1983) have shown that bluffs in Southern California tend to retreat most during “wet years”. Therefore, rates and extent of cliff and bluff erosion may also be affected by material characteristics and geotechnical stability processes as well as coastal erosion processes.

- Geometric models employed to assess the dune erosion produced by extreme storms are useful for simple determinations of the maximum potential dune retreat and sand volume loss. While the use of empirical data sets for development and validation of geometric erosion assessment procedures for the Pacific region (like those in the Atlantic and Gulf) may be a viable alternative, there may only be limited pre- and post-storm beach profile data available on which to base the procedure. Further research and inquiries are necessary with state resource agencies, universities, the USGS, and NOAA to determine whether such historical storm-induced erosion data sets are readily available.

- The most extensively developed geometric model for dune erosion is that of the Dutch as presented by Vellinga (1982, 1983, 1986), which yields a calculated dune volume loss and position of the fronting beach for a 5-hour storm tide elevation, with guidelines for the additional erosion that occurs for each hour beyond that 5-hour duration. (This type of short duration event may not be appropriate for Pacific storms.)

- Existing FEMA methodology is based on a modified form of the Dutch model (Vellinga, 1986), with a two-segment profile approximation to the Dutch concave profile, employed to analyze the wave runup and potential for dune overwash on the erosion adjusted profile. The FEMA methodology uses a geometric erosion assessment procedure to adjust the post-erosion profile for varying stillwater elevations and dune configuration, but does not utilize a geometric model to evaluate the volume of sand eroded from the dune by the storm, having opted instead to fix that volume at the average volume of 540 $ft^3/ft$ for the estimated 100-year event for each erosion assessment application (dune removal or dune retreat).

- FEMA analyses comparing dune-erosion volumes to storm recurrence flood levels is very sensitive to an accurate determination of the stillwater elevation and return period of the storm, which in itself can have a significant degree of uncertainty. The return periods for the median erosion values for the Atlantic and Gulf Coast data set are based on a comparison of the measured tide gauge data or observed high-water marks from the storm with the published FIS return period elevations. Measured tide gauge data for each storm are considered the best available information for storm recurrence interval determination. Development of similar procedures for the Pacific Coast require the location and development of similar data sets.

- The geometric model by Komar and others (2002) that has been applied on the U.S. Pacific Coast to evaluate dune erosion during recent El Niño related storms and high-water levels should be tested and refined as a possible method for evaluating the extent of sandy beach and dune retreat.

- Methods developed by Judge et al. (2003) may have applicability to sandy Pacific Coast beaches and dunes, and merit further investigation. However, it is noted that this model does not include the duration of storm characteristics nor the erodibility of the sediments.

- It is essential to understand the primary geomorphic and geologic characteristics of the Pacific Coast study site in order to address the dominant erosion processes responsible for changes in beach, dune or bluff geometry during the 100-year event. Therefore, one of the first steps during an erosion assessment is to clearly define the project setting, the underlying erosion processes, and the erodibility of the sediments.

- In areas where sea cliffs or bluffs are present, but not composed of sand, geometric models may not be appropriate. A second approach consistent with Pacific Northwest methods discussed earlier is to define the erodibility time scales differently for loose sand and other materials.

- Research for this focus study found no reliable geometric models applicable to mixed grain sizes and/or cobble and gravel based beaches and dunes. Simplified methods for evaluating single-event erosion hazards in coastal regions comprised of coarse grained materials may not be readily available for the Pacific Coast or the North Atlantic. This is discussed further in Topics 33 and 34, herein.

### 2.1.6 Recommendations for Topic 30 (Geometric Erosion Assessment for the Pacific)

Table 1, at the conclusion of this report, summarizes the key findings and recommendations for this topic, and Table 2 provides an estimate of the amount of time required to accomplish tasks recommended for this topic. Following are recommendations for Topic 30 (Geomtric Erosion Assessment for the Pacific).

1. In the short term: A review should be undertaken for the Pacific Coast, based on available LIDAR, photogrametric, or physical surveys of beach and dune erosion produced by major storms, including periods of El Niño conditions, to see if available data sets can successfully document dune volume losses and beach profile changes for a variety of beach types and settings in California, Oregon and Washington. Limited data sets are available from the NOAA Coastal Services Center from LIDAR investigations conducted before and after the 1997–98 El Niño event. The goal is to develop a geometric model capable of: 1) predicting the extent of dune retreat during a 100-year storm scenario, 2) determining whether the dune persists or fails as a flooding barrier, and 3) determine the ultimate beach and dune profile during the 100-year event upon which runup and overtopping can be computed. It may be determined that the issue of the magnitude of the “100-year erosion” may be less important than concurrent or sequential EBE (duration of erosion) assumptions for whatever return frequency storm event is being assessed for the FEMA NFIP.

2. New EBE assessment methods are needed and should be applicable to different physiographic regions and must consider the following: 1) physiographic setting, 2) sediment characteristics across the active profile, 3) time histories of wave and storm tide characteristics, and 4) whether local or regional oceanic (El Niño) or topographic (recent tectoic adjustments) characteristics affect the study area and the magnitude of runup.

3. Study Contractors should examine available state and federal coastal resources mapping and documentation to determine the geomorphic, geologic and erosional setting for each

Pacific Coast project site. A determination of the erosion assessment procedure to be utilized should be based upon the history of significant erosion at the site and whether there are data and evidence of a consistent seasonal winter beach profile for the study region. The seasonal winter beach profile, and perhaps a "Most Likely Winter Beach Profile" would represent the typical beach profile configuration expected for the storm events and upon which the procedure would be applied.

4. The geometric model by Komar and others (2002) that has been applied on the U.S. Pacific Coast to evaluate dune erosion during recent El Niño related storms and high-water levels should be tested and refined as a possible method for evaluating the extent of sandy beach and dune retreat. Study Contractors may use the "Most Likely Winter Beach Profile" as an interim approach for estimating the eroded beach profile shown in Figure 4.

5. A longer term program (possibly a multi-agency cooperative program) could include expansion of the present USGS/NOAA coastal survey program for the Pacific Coast. Results from this program will help determine the "Most Likely Winter Beach Profile" to use for Pacific Coastal areas prior to the 100-year event.

6. The post-storm profiles obtained in the long-term field studies could be used to develop and test new geometric models (or process-based models) for sandy beach and dune systems along the Pacific Coast.

7. The performance and reliability of geometric versus numerical modeling procedures should be tested for sand beaches and dunes on the Pacific Coast and verified with available data sets.

8. Methods for assessing other types of non-sandy beach settings, such as cobble and gravel beaches, should be developed and based as much as possible on the underlying physical processes controlling those coastal settings.

9. Establish the definition of the most likely 100-year storm event and SWEL for any location along the Pacific coastline. A program to measure and determine the magnitude and approximate recurrence frequency of Pacific storms is necessary. It is essential to define the most likely 100-year storm event and SWEL for use in FEMA coastal hazard assessments.

## 2.2 CRITICAL TOPIC 33: (SHINGLE/COBBLE EROSION ASSESSMENT)

### 2.2.1 Description of Topic and Suggested Improvement

Present guidance in *Appendix D* focuses primarily on simplified methods for estimating single storm event erosion for sand-dominated beaches and dunes. The *G&S* do not provide methods for estimating erosion in coastal systems comprised of mixed grain sizes, gravel, cobbles or shingle. Note that shingles are not a standard American Geophysical Union size class descriptor and refer to very coarse beach gravel consisting of flat cobble and flattish pebbles found on higher parts of the beach. The TWG recommends developing and adding new guidelines with the capabilities to address erosion in these types of coastal areas found along the Atlantic, Gulf, Pacific, and in Sheltered Water areas.

### 2.2.2 Description of Procedures in the Existing Guidelines

The *G&S* do not provide methods for estimating erosion in coastal systems comprised of mixed grain sizes, gravel, cobbles, or shingles. Shingle/cobble beaches do not have a similar response to the storm induced erosion on a sand beach. This may preclude the use of a simplified "540 SF-type" method. It is likely that different methods are required, in part because there is a greater degree of variability found in mixed- and coarse-grain beaches.

### 2.2.3 Application of Existing Guidelines to Topic – History and/or Implications for NFIP

The key issues with cobble and shingle beaches and dunes are defining their degree of similarity to sandy beach areas during significant storms, and whether the present "540 SF" approximation is appropriate for application in these areas. If cobble/shingle areas are unique enough to require their own technical approach, existing historical beach profile data and literature may provide sufficient information for the derivation of an assessment method. This problem has significant implications along the North Atlantic, portions of the Pacific and in some Sheltered Water areas. Therefore, this topic is considered "Critical" for all three regions.

Historically, cobble beaches, also commonly referred to as gravel or shingle beaches, have not received as much scientific and engineering focus as sandy beaches in the United States. However, there is a rich literature in the United Kingdom because of the considerable extent of beaches of this type. Cobble beaches tend to be stable over a wide range of wave conditions and thus tend not to be as erosive as sandy beaches. Therefore, the level of research focused on cobble beach design is relatively limited. Previous studies and design involving cobble beaches have utilized existing formulas and concepts to describe the dynamics of sand beaches to explain and predict cobble beach behavior with varied results. In recent years, more studies have been undertaken to understand cobble beaches because of their stable nature. Cobble beaches are being explored as viable alternatives to hard coastal engineering structures for beach stabilization.

There are a few main physical differences between sandy beaches and cobble beaches. First, cobble beaches have much steeper foreshore slopes (~1:10) than sandy beaches (~1:40 to 1:100). Cobble beaches are also usually marked by steep berms that correspond to the maximum height reached by the swash runup. Cobble beaches tend to contain a wide range of materials, varying from sand to cobbles. This results in beach profiles with a steep foreshore slope, which is naturally armored with coarsest material in the littoral system. Along the lower portion of the beach profile, sand and finer materials commonly form a very shallow or flatly sloped low tide terrace. Figure 5, illustrates the difference in profile shape for cobble, sand, and mixed beaches. Since gravel and cobbles also are less susceptible to motion in a given wave environment, these beaches are more stable under wave and current attack. However, cobble beaches still remain very dynamic, with constant readjustment to variations in wave climate and tidal conditions. Another feature is the high hydraulic conductivity of the stone. This increases the potential for infiltration during swash and is probably responsible for the formation of the berm at the maximum swash runup (Van Wellen, 2000).

Nourishment of cobble beaches along the coasts of England and Wales and the North Atlantic shores of the U.S. has led to the development of procedures for assessing the dynamics of cobble beaches and dunes during significant storm events. Similarly, along the Pacific and New Zealand coastlines, researchers and engineers have designed “cobble berms” or “dynamic revetments” to reduce severe erosion of back beach areas subjected to high water levels and wave action (Komar et al., 2003; Powell, 1988; Powell, 1990; Ahrens, 1990). Research has determined that there is a great variety of so-called “mixed grain size” beaches. Depending on the relative proportions of sand versus coarse particles (gravel/shingle and cobbles) the patterns of grain sorting and beach morphology vary depending on the tide range and local wave energies. It is well established that a sand beach responds by the cross-shore movement of sand from the berm to offshore bars, its average slope decreasing in the process so that is more dissipative of the wave energy. While many field studies have found a similar pattern for gravel and cobble beaches, Bluck (1967) for example found a net landward movement of coarse particles and beach accretion during storms, so both the crest elevation and slope of the beach increased. Pacific Coast researchers found the same response in the study of the cobble berm constructed at Cape Lookout State Park, and for the natural cobble beaches in the Pacific Northwest. Everts et al. (2002) also found similar patterns for cobble beaches on the Southern California Coast. At both West Coast sites the cobble beach was fronted by a sand beach, and when impacted by storms, the sand beaches decreased in slope to become more dissipative, while the cobble beaches on their landward sides increased in slope and become more reflective. There has not been sufficient study to understand this response of cobble beaches, or to discern why it is different from one site to another. Researchers suspect it is related to the content of sand within the otherwise coarse-grained deposit, which affects the permeability of the beach and hence the balance between the swash and backwash, and the competence of the landward-flowing swash to transport cobbles up the beach face. However, further research and field validation is needed.

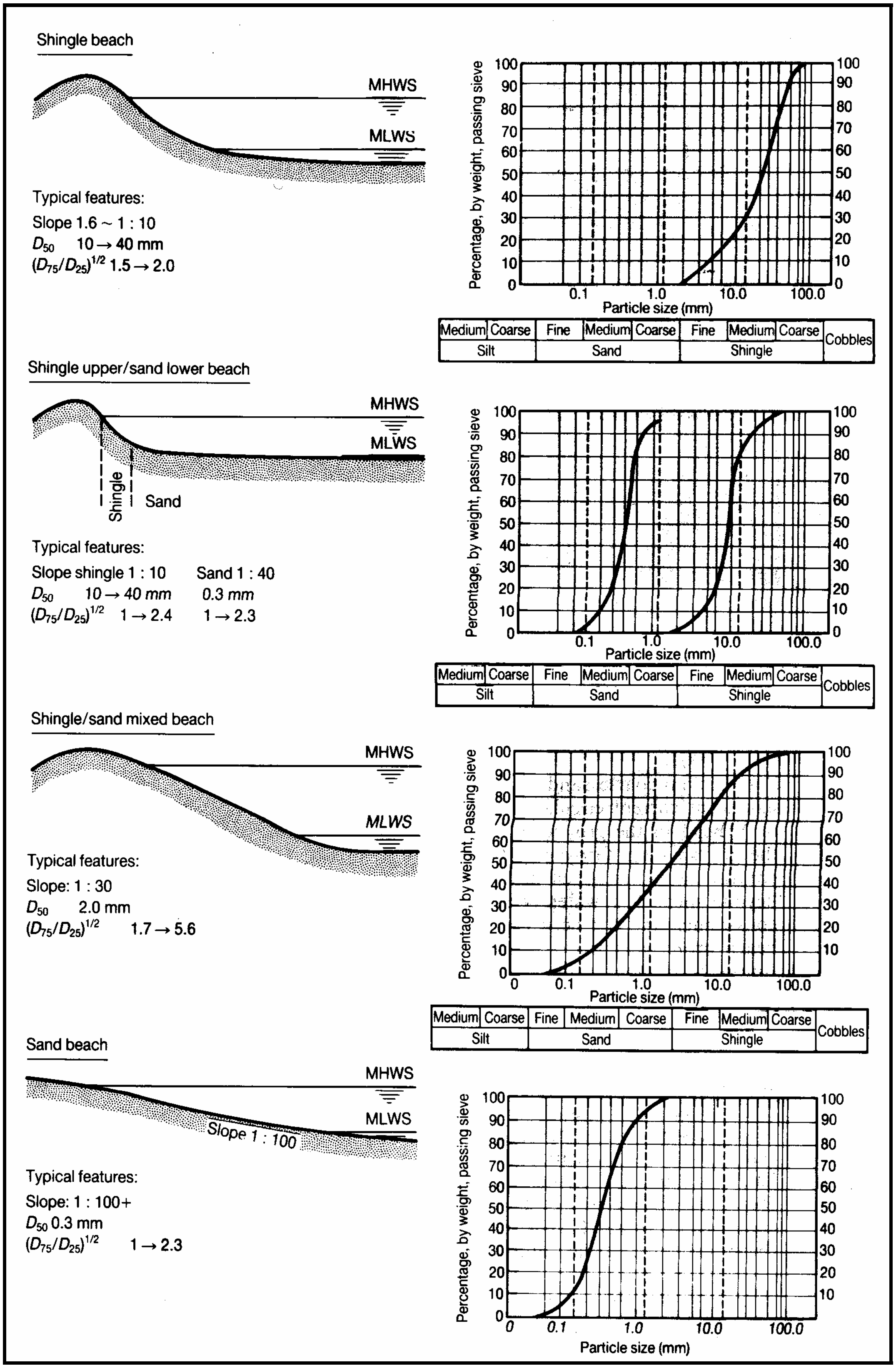



**Figure 5. Variation in beach profiles and gradations (CIRIA, 1991).**

### 2.2.4 Alternatives for Improvement

In summary, present guidance in *Appendix D* does not provide methods for estimating erosion in coastal systems comprised of mixed grain sizes, gravel, cobbles, or shingles. Shingle/cobble beaches do not have a similar response to storm induced erosion as a sand beach that would allow the use of a simplified "540 SF-type" method. It is apparent that different methods (more process-based) are required and that there is a greater degree of variability found in mixed- and coarse-grain beaches. Further research is required to better describe erosion processes in gravel and cobble beach settings.

### 2.2.5 Recommendations

Following are recommendations for this Topic. Table 1, at the conclusion of this report, summarizes the key findings and recommendations for this topic, and Table 2 provides an estimate of the amount of time needed to accomplish tasks recommended for this topic.

1. Prepare new sections in the existing *Appendix D* guidance to describe differences between sand dominated beaches and mixed- and coarse-grained beaches. Provide photos and profile information.

2. Gather existing literature on natural cobble, shingle, and coarse-grained beaches to summarize the existing state of knowledge and provide references until specific guidelines can be developed and adopted.

3. Review literature on the design of and construction of dynamic revetments and cobble berms to provide guidance on their stability and long term development.

4. Examine other possible guidance and available beach and dune data sets for possible clarifications to the 540 SF Criterion for sand-dominated beaches versus mixed- and coarse-grained beaches. Attempt to develop "equivalents" between sand and coarse grained beaches. Attempt to develop methodology that will allow computation of erosion within framework described.

5. Discuss the limitations of applying geometric models to cobble/shingle beach and dune areas.

6. Examine the applicability of existing equilibrium beach profile concepts and relationships to represent the response of cobble and mixed grain beaches to storms, for example, Dean (1991).

7. Prepare case studies of actual coarse grained beaches demonstrating application of the recommended methodology.

8. Prepare new guidelines for the Pacific Coast describing the physical processes associated with mixed- and coarse-grained beaches.

## 2.3 CRITICAL TOPIC 35: GUIDANCE FOR EROSION ASSESSMENTS IN SHELTERED AREAS

### 2.3.1 Description of Topic and Suggested Improvement

EBE in major sheltered areas such as San Francisco Bay, Puget Sound, and Chesapeake Bay, is dependent on fetch-limited wave characteristics, inshore water levels that consist of the stillwater level and wave-induced setup, and beach morphology defined by sediment type, inshore slope, etc. Sheltered water areas tend to have a wide variety of shoreline sediment/material types and beach/shoreline profiles due to their local geomorphology, local geology, and watershed characteristics. Watershed size, hydrology, geology, land use, and resulting sediment production and delivery to the coastal zone affect the beach characteristics and processes found within sheltered water areas.

For example, although much of the San Francisco Bay shoreline is composed of silty sediment (bay mud), marshes, and steep coarse cobble and revetted areas that are more resistant to the EBE induced by wind-driven waves, some existing sandy beach areas are still prone to erosion, particularly in shoreline segments that are semi-exposed to ocean swells (e.g., Crissy Field). Past field observations indicate that horizontal bank erosion without vertical scouring is most likely to occur in shoreline segments that consist of bay mud only. Unlike the open-coast EBE where recovery processes do occur depending on the subsequent wave climate, no recovery of bank erosion is to be expected after the sheltered bank is eroded away.

In Puget Sound, the shorelines in sheltered areas may be characterized as consisting of narrow to non-existent sandy to cobble beaches backed by high, wave cut coastal cliffs. The sandy beach has only a thin lens of sand topping the cobble or the natural bedrock planform. The rocky and steep shorelines mostly resist EBE, and the event-based vertical scouring for sandy pocket beaches would be limited to the upper thin sandy lens, as fetch-limited wind-driven waves are probably not capable of removing the underlying cobble material. However, most of the depleted thin sandy shorelines do however recover afterwards. As cliff erosion occurred over time, the eroded material contributed to the formation of low-tide terraces fronting the cliffs. These wide terraces now provide a shallow water zone where wave energy is dissipated. Thus, the majority of the shorelines within Puget Sound have experienced relatively stable conditions in the recent past.

Historical beach and dune erosion events have been documented along inland bays and sheltered waters in the Atlantic/Gulf Coast regions. It is believed that the physical processes of the event-based dune erosion are similar to those occurring along the open coast. Although the original guidance on the 540 SF Criterion for EBE was primarily based upon historical field investigations along open coast beaches in the Atlantic and Gulf regions, no *G&S* of erosion assessment in sheltered areas for any coastal regions, including the Atlantic/Gulf and Pacific

Coasts, are presented in *Appendix D*. Based upon historical field observations of the EBE pattern between these regions that demonstrate a strong dependence of EBE on individual beach morphology, suggestions can be made to establish the guidance to the EBE in sheltered waters as presented in the recommendation section.

### 2.3.2 Description of Procedures in the Existing Guidelines

Guidelines and procedures for assessing erosion in sheltered areas for any coastal regions, including the Atlantic/Gulf and Pacific coasts, are not presently available in *Appendix D* (FEMA, 2003).

### 2.3.3 Application of Existing Guidelines to Topic

In sheltered water areas along the Pacific Coast, large sand dune systems are not typical and the NFIP regulations and existing Guidelines that provide methods for delineating Base Flood Elevations (BFEs) with Primary Frontal Dunes, typically exclude the lower energy EBE (horizontal erosion and vertical scouring) in sheltered waters that are induced by wind-driven waves. In some occasions, even the effects of these smaller wind-driven waves are not incorporated into hazard zone delineations in these coastal flood studies and only the 1-percent-annual-chance stillwater elevation is used to define the sheltered water BFE.

In the Atlantic/Gulf region, the 540 SF Criterion has been applied for inland bays where beach and dune erosion has been documented and known to be a historical EBE associated with the base coastal flood event (Hatheway, 2004). The application usually results in minor but necessary adjustments to the beach profiles prior to the wave height analyses in these applications (e.g., recent Mobile Bay coastal analyses in Baldwin County). Although the 540 SF Criterion commonly used for the Atlantic/Gulf open coast has been applied to the sheltered waters in the same region, this appears to be a very conservative approach and could result in unrealistically large flood level assumptions. For example, extreme water levels can extend well inland of the open coast as seen in the extensive flooding of the Severn River at Annapolis during Hurricane Isabel in 2003. However, the local wave field which is implicit in the 540 SF Criterion cannot exist in such a width-limited and length-limited fetch. Therefore, application of the typical 540 Criterion for this scenario is not recommended. Dune erosion rates will necessarily be greatly reduced or non-existent in that scenario. In all likelihood, a much smaller geometric prism will provide equivalent protection in these environments. Additional field verification is necessary to confirm the applicability of this geometric criterion to the sheltered water zones. However, given the scarcity of extensive natural dunes in most sheltered waters, relevant field data will be difficult to obtain. Reductions in the recommended eroded cross-sectional area, based upon adjustments to the probable local wave conditions, may provide the only practical solution to this problem.

The existing Guidelines focus on the erosion of open coast sand dune systems and do not provide guidance for addressing EBE of sheltered water beaches and backshore low bluffs and coastal cliffs.

### 2.3.4 Alternatives for Improvement

The alternatives for improvement to the *G&S* regarding erosion assessments in sheltered areas are:

1. Classify the specific characteristics of EBE in sheltered waters based on the location of the flood study site with respect to the geographic setting, local shore forms, and past field observations for different types of beach sediment such as bay mud, cobble, and coarse to fine sand.

2. Differentiate guidance, if guidelines are required, based on observed historical event-erosion patterns that are applicable to each setting and geomophic category.

3. For the Atlantic/Gulf region where some applications have been made using the established 540 SF Criterion, existing historical data and publications related to the application of the 540 SF Criterion should be reviewed to determine inland bay and sheltered water response to coastal storms so that the existing or revised 540 SF Criterion can be readily added to the erosion assessment in sheltered areas for the Atlantic/Gulf region.

4. A survey of coastline types in major West Coast sheltered water areas should be made to determine the extent of regions in which the 540 SF type geometric criterion, might be applicable and an assessment made of the need for development of revised geometric criteria for this region (as presented previously in Topic 30).

5. As defined in the NFIP regulations, "flood-related erosion means the collapse or subsidence of land along the shore of a lake or other body of water as a result of undermining caused by waves or currents of water exceeding anticipated cyclical levels or suddenly caused by an unusually high water level in a natural body of water, accompanied by a severe storm, or by an unanticipated force of nature, such as a flash flood or an abnormal tidal surge, or by some similarly unusual and unforeseeable event which results in flooding." Since FEMA is to provide the data upon which floodplain management regulations for flood-related erosion-prone areas shall be based (44 CFR sec. 60.5), guidance should be provided to Mapping Partners on how to obtain, review and reasonably utilize these data.

6. Explore the possibility of developing a rational basis for predicting erosion in sheltered waters which is consistent with the general framework discussed previously. Such a framework should account for the time histories of water level and wave forcing, and the durability of the eroded material.

### 2.3.5 Recommendations

Following are recommendations for this Topic. Table 1, at the conclusion of this report, summarizes the key findings and recommendations for this topic, and Table 2 provides an estimate of the amount of time needed to accomplish tasks recommended for this topic.

1. Prepare a new *G&S* description of EBE for sheltered waters in accordance with typical beach morphology and sheltered-water wave characteristics.

2. Provide the interim *G&S* for EBE in sheltered waters, based primarily on historical field observations during various storm events.

3. Attempt to develop rational guidance based on a model consistent with the general framework discussed previously.

4. Develop case studies for testing new guidance in previously studied sheltered area settings.

5. Future research: Incorporate the EBE models that may ultimately be developed from Topic 36 to establish the final *G&S* that can be applied to all identified major sheltered waters for the Atlantic/Gulf and Pacific coasts (i.e., San Francisco Bay, Puget Sound, and Chesapeake Bay) and other small sheltered waters including those located in Southern California if the EBE conditions are justified.

## 2.4 CRITICAL TOPIC 39: PRIMARY FONTAL DUNE

This topic was determined to be more appropriately associated with Hazard Zones, and so it was moved to that section and will be included in the TWG for Hazard Zones.

This completes the discussion of Critical Topics.

# 3 IMPORTANT TOPICS

## 3.1 TOPIC 34: DEVELOP IMPROVED GEOMETRIC METHODS WHICH CONSIDER COBBLE/SHINGLE EFFECTS

### 3.1.1 Description of Topic 34 and Suggested Improvement

Present guidance in *Appendix D* focus primarily on simplified methods for estimating single storm event erosion for sand-dominated beaches and dunes. The Guidelines do not provide methods for estimating erosion in coastal systems comprised of mixed grain sizes, gravel, cobbles, or shingles. The TWG recognizes the need for addressing beach profile changes that occur during base flood events and how those changes may affect runup and flooding along coasts comprised of mixed grain sizes, gravel, cobble, and boulders. Given the need to assess these types of coastal settings, one key issue with FEMA is whether the present 540 SF Criterion used for sand-dominated beaches can be used or modified for shingle/cobble beaches and dunes. Therefore, this is considered to be an "Important Topic." The TWG recommends developing new guidelines with the capabilities to address erosion in these types of coastal areas found along the Atlantic, Gulf, and Pacific Coasts and in some Sheltered areas.

### 3.1.2 Description of Procedures in the Existing Guidelines

The present Guidelines do not provide methods for estimating erosion in coastal systems comprised of mixed grain sizes, gravel, cobbles, or shingles. Coastal engineering research has focused primarily on preventing or controlling erosion along shingle/cobble beaches rather than predicting how such beaches may erode during rare storm events. Shingle/cobble beaches do not display similar responses to storm induced erosion as do sand-dominated beaches; therefore, application of the present simplified "540 SF-type" method should be avoided. It is apparent that different methods are required and that there is a greater degree of variability found in mixed- and coarse-grain beaches.

### 3.1.3 Alternatives for Improvement

There has been sporadic interest in mixed grain, gravel, cobble, or shingle beaches over the years by engineers and scientists. The result is a scattered body of literature and knowledge that has never been organized and combined into a coherent base of knowledge on the dynamics, characteristics, and variability of the cobble, shingle, and mixed grain systems. The first step in developing a quantitative guidance for assessing the dynamics of these systems is to conduct extensive research of the available literature on natural gravel, cobble, and mixed sand and gravel beaches to summarize the knowledge that has been developed and to examine the quantitative methodologies that have been used and proposed.

It is not clear whether the morphological differences between systems will allow direct application of knowledge and typical system responses during storm events from one site to another. For example, some of the local gravel, cobble, boulder beaches found in California, Washington, and Oregon contain substantial quantities of natural, rounded large cobble and boulders, whereas in Europe and Japan the common constituent is flat shingle. Therefore, where possible, available data should be compared to see how the various systems differ. It may be found that the systems comprised of similar material characteristics (grain size, shape, and density) respond similarly regardless of the variations in morphology and wave climate. Making this determination may allow currently developed methodologies to be applied and developed for a wide range of different systems and locations. However, until those relationships are understood, caution is required when attempting to use data developed in regions with significantly different wave climates and geomorphic characteristics and beach material characteristics.

The writers are unaware of reliable numerical models that are capable of simulating dynamic beach morphology. Available models are very simplified (Powell, 1990), but may eventually be refined as more is learned of these types of beach processes (see Figure 6). However, studies examining dynamic revetments and berm breakwaters should be reviewed. The physics governing how dynamic revetments and berm breakwaters respond during storms differ somewhat from cobble and shingle beaches because of increased grain size and reduced grain size composition (dynamic revetments and berm breakwaters generally do not contain fine material to allow for wave absorption). These types of structures rely on profile development and response to dissipate wave energy. This is very similar to what naturally occurs on natural cobble, shingle, and coarse-grained beaches and may closely correspond to processes important to FEMA. Certainly, qualitative information can be extracted from previous studies. Van der Meer (1992) has done extensive model tests on the stability of different cobble slopes and how they relate to hydraulic and structural parameters of berm breakwaters. Those relationships were used to develop the computational model called BREAKWAT for assessing and designing cobble berm, breakwater. It is possible that the basis for this model could be further developed to predict the profile evolution of cobble and shingle beaches. Sayao (2004)has done extensive 2- and 3-dimensional flume tests on profile development and stability in berm breakwaters and dynamic revetments which could also be incorporated. The Dutch have used similar methods to protect dikes in the Netherlands. Dynamic revetments are beginning to be more commonly used in Massachusetts in place of more traditional seawalls and revetments.

Development of simple geometric (empirical) models is possible, but it will require careful evaluation of regional and perhaps site-specific data. Case studies of historical and current profile data along with site-specific information would provide examples of the shoreline types encountered and summarize the differences in beach characteristics and wave conditions found along natural cobble, shingle, and coarse-grained beaches. Combining this information with the approaches and methodologies already in use could provide the necessary guidance for evaluation of natural cobble, shingle, and coarse-grained beaches. Also, available equilibrium beach profile concepts and relationships may provide useful information (Dean, 1991).

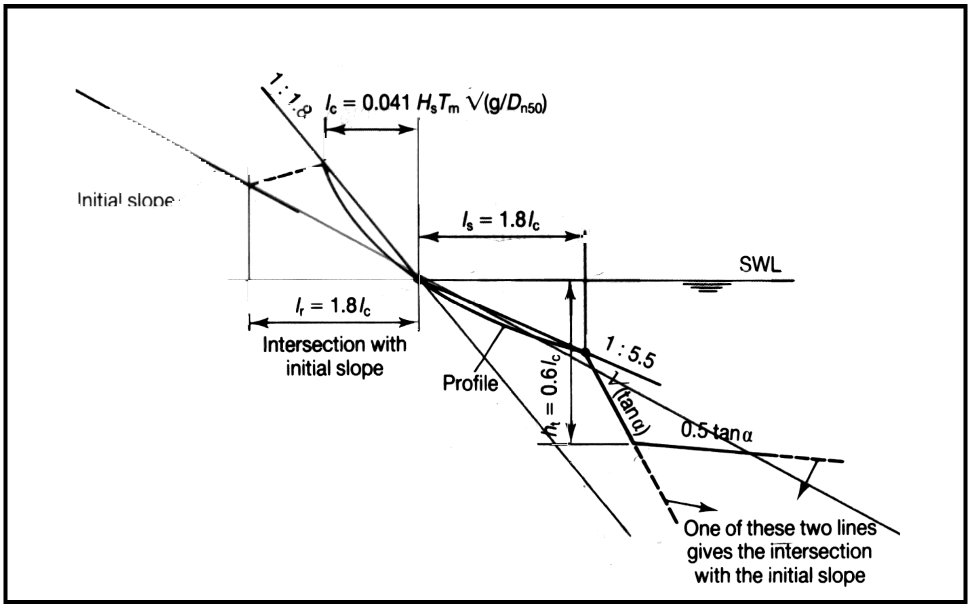



**Figure 6. Schematized pre- and post- storm profiles of rock and gravel beaches (CIRIA, 1991).**

### 3.1.4 Recommendations

Following are recommendations for this Topic. Table 1, at the conclusion of this report, summarizes the key findings and recommendations for this topic, and Table 2 provides an estimate of the amount of time required to accomplish tasks recommended for this topic.

1. Gather, compile, and summarize existing literature on natural cobble, shingle, and coarse-grained beaches to summarize the existing state of the knowledge and provide references Mapping Partners can use until specific guidelines are developed and adopted.

2. Review literature on the design of and construction of dynamic revetments and cobble berms to provide guidance on their stability and long-term development (morphologic changes during varying wave conditions).

3. Review and assess historical applications of the existing geometric model (540 SF Criterion) to the Atlantic/Gulf for natural gravel, cobble, and mixed sand and gravel shorelines to determine its validity for these types of beach conditions.

4. Perform a demonstration test of 540 SF Criterion on a natural gravel, cobble, and mixed sand and gravel beach to assess its reliability (or not). Document results in terms of a case study; recommend discussion paragraph for *G&S*.

5. Gather and summarize documentation of historical erosion and beach profile surveys during extreme storm events, particularly for Northeasters on the Atlantic and El Niño years such as 1982–1983 and 1997–1998 for the Pacific Coast. Develop interim eroded

gravel, cobble, and mixed sand and gravel beach profiles for the Atlantic and Pacific Coast regions separately, based primarily on the historical data.

6. Examine the applicability of existing equilibrium beach profile concepts and relationships to represent the response of cobble and mixed-grain beaches to storms.

7. Determine whether generic process-based models can be developed in a relatively short period of time for application to both the Atlantic and Pacific coasts.

8. A process-based model would be consistent with the desirable framework discussed earlier. If a model is recommended that is not process based, ensure that the model incorporates elements consistent with the framework.

## 3.2 TOPIC 36: GUIDANCE FOR EROSION ASSESSMENTS IN SHELTERED AREAS

### 3.2.1 Description of Topic and Suggested Improvement

As described in Topic 35, the physical processes of the EBE in sheltered waters are similar to those along the open coast. Beach morphology for major sheltered waters can be categorized as those described in Topic 30, except that silty sediment instead of sandy material is more common in many Pacific Coast regions (e.g., San Francisco Bay). In San Francisco Bay, past field observations during storm events indicate that horizontal bank erosion without vertical scouring is most likely to occur in the shoreline segments that consist of bay mud only. Eroded beaches within sheltered water areas may not recover in the same manner as seasonal beach profiles do along the open coast because the post-storm wave characteristics are significantly different in sheltered waters. In Puget Sound, the event-based vertical scouring for sandy pocket beaches is likely to be limited to the upper thin sandy lens, as described in Topic 35. Since no *G&S* regarding EBE are available for sheltered waters, new guidance is needed. Potential alternatives and suggestions are presented in Section 3.2.4.

### 3.2.2 Description of Procedures in the Existing Guidelines

As discussed in Topic 35, no guidance is provided in the present *G&S* for assessing erosion in sheltered areas for any coastal region along the Atlantic, Gulf, or Pacific.

### 3.2.3 Application of Existing Guidelines to Topic

See Section 2.1.3

### 3.2.4 Alternatives for Improvement

Alternatives for improving the *G&S* regarding erosion assessments in sheltered areas include:

- Characterize beach, back beach, bluff, and cliff morphology, historic stability, and dominant material properties typically found in Sheltered Waters and discuss the differences with those properties found along open coasts.

- Determine whether available process-based erosion models for the open coast are applicable to the sheltered water areas.

- Consider/recommend possible guidance clarifications or modifications to the 540 SF Criterion for sheltered waters. Review existing historical data and literature for the Pacific to determine inland bay and sheltered water responses to coastal storms, and their consistency with the Atlantic and Gulf coastal areas. Test the applicability of the 540 SF Criterion for sheltered with reliable beach profile data.

- Evaluate the process-based models (e.g., EBEACH) that are presented in Topic 38 to determine if they would be suitable for estimating storm induced erosion along inland bays and sheltered waters for Atlantic/Gulf and Pacific coastal areas.

### 3.2.5 Recommendations

Following are recommendations for this Topic. Table 1, at the conclusion of this report, summarizes the key findings and recommendations for this topic, and Table 2 provides an estimate of the amount of time required to accomplish tasks recommended for this topic.

1. Review and assess the historical applications of the existing 540 SF Criterion to sheltered shorelines. Summarize the results regarding its applicability to the sheltered water regions.

2. Develop interim eroded profiles for the Pacific Coast region, based primarily on documented histories of erosion and beach profile surveys during extreme storm events, particularly in El Niño years such as 1982–1983 and 1997–1998 to provide interim *G&S* suitable to the Pacific Coast.

3. Conduct case studies to test and illustrate the recommended approach using actual data sets.

4. Explore the possibility of developing a rational basis for predicting erosion in sheltered waters which is consistent with the general framework discussed previously. Such a framework should account for the time histories of water level and wave forcing, and the durability of the eroded material.

5. Test process-based models that are to be developed under Topic Number 38 to determine if they are suitable for implementation in sheltered waters in all regions.

## 3.3 TOPIC 37: REVIEW ATLANTIC-GULF COAST 540 SF CRITERION

### 3.3.1 Description of Topic and Suggested Improvement

Section D.2.4 of *Appendix D* directs the Study Contractor to perform an erosion assessment of open coast shorelines bordering the Atlantic Ocean and Gulf of Mexico, that is, to determine any erosion likely to occur during the base flood event, and to adjust the existing profile to reflect the anticipated eroded profile shape prior to use of the wave height and wave runup models.

As previously stated in earlier sections, the present default erosion assessment procedure determines the cross-sectional area of a sand dune above the 100-year stillwater elevation (without wave setup) and seaward of the dune peak* (see Figure 7), then compares that cross-section against the critical value required to prevent dune loss (removal) during the base flood event – 540 SF. If this "frontal dune reservoir" is less than 540 SF, the dune is presumed to be destroyed (removed) by the base flood event. If the primary frontal dune reservoir is at least 540 SF in size, then the dune is presumed to sustain retreat, but survive the storm (see Figure 8). In other words, the 540 SF Criterion for the frontal dune reservoir is a trigger for dune removal (less than 540 SF) and retreat (greater than 540 SF).

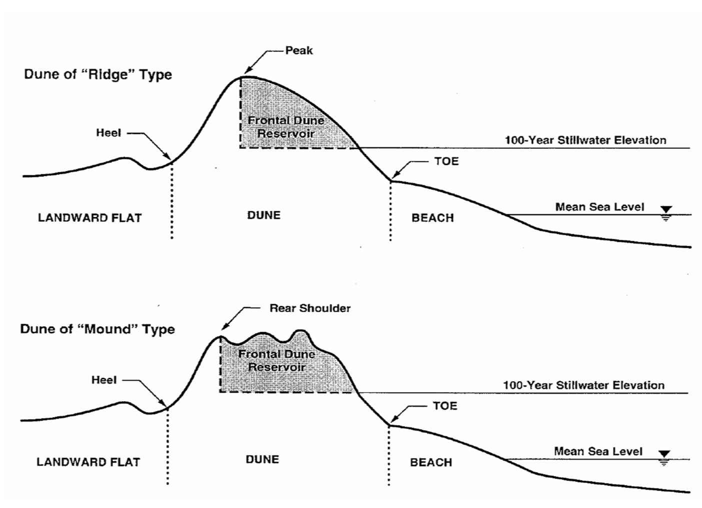



**Figure 7. Frontal dune reservoir.**
**(from Appendix D, FEMA, 2003)**

* Section D.2.4.1 of FEMA (2003) states that the dune erosion treatment is also appropriate in cases with sandy bluffs or headlands extending above the 1-percent-annual-chance stillwater elevation.

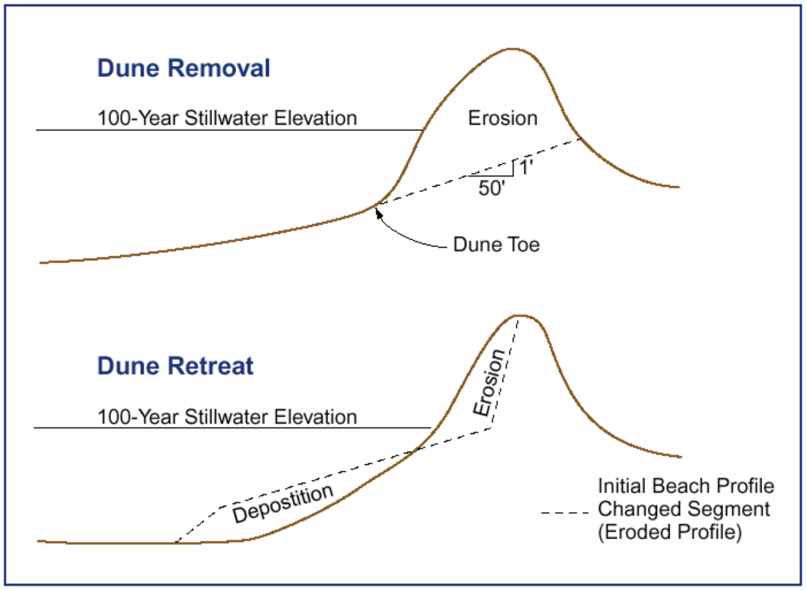



**Figure 8. Dune removal and dune retreat geometries.**

The critical value used by FEMA – 540 SF – was determined to be the *median* erosion area above the stillwater elevation during the 100-year event. The value was determined by a review of pre- and post-storm profile data for 38 erosion events on the Atlantic, Gulf or Mexico and Dutch Coasts [Hallermeier and Rhodes, 1986; Dewberry & Davis, 1989].

The 540 SF Criterion became effective following a change to the NFIP regulations (see Appendix A at the end of this discussion Topic 37, for a copy of the final rule published in the Federal Register [Vol. 53, No. 88, pages 16,269 to 16,275]).

### 3.3.2 Review of 540 SF Criterion

The 540 SF Criterion is reviewed and discussed below in terms of two central questions:

1. What is the best estimate for an erosion area-frequency relationship (and is 540 SF the correct value for median erosion above the 100-year stillwater elevation)?

2. Is use of the median erosion area appropriate for dune removal-retreat determinations?

Regarding the first question, Hallermeier and Rhodes (1986) reviewed storm erosion data for 38 storms between 1894 and 1985. Dewberry & Davis (1989) added eight storm erosion events to the databases and repeated the analysis. Both analyses yielded the erosion-frequency relationship

$$E = 85.6\ T^{0.4} \qquad (2)$$

where:

E = erosion area above storm stillwater elevation ($ft^2$, or $ft^3/ft$)

T = storm return interval (years)

The 540 SF value corresponds to the 100-year stillwater recurrence interval.

Many storm-related beach profile data have been collected since the relationship was developed, and much of that data have been very detailed (much more detailed than the original 38-storm database). It is recommended that the erosion-frequency relationship be revisited by adding more data to the 38-storm database, and by a second evaluation of the 38 storms. It is noted that Judge et al. (2003) have documented dune erosion at 90 transects on Topsail Beach, North Carolina, as a result of Hurricane Fran (1996).

Regarding the second question, FEMA Typically analyzes coastal flood hazards by considering the 100-year stillwater level in conjunction with other flood parameters at the mean (50%) level. Review of *Appendix D* shows the median erosion value, mean runup elevation, and mean overtopping rate are all used in mapping the 1% flood elevations in coastal areas. However, for Atlantic/Gulf of Mexico open coast situations where uplands are submerged by storm surge, FEMA establishes BFEs using the "controlling" (1%) wave height, not the mean wave height. Use of the 1% wave height was recommended by the National Academy of Sciences (1977). The NAS committee obviously believed mapping and regulating to a lower wave height was inappropriate, given the consequences of breaking waves striking buildings in coastal areas (severe building damage or destruction).

The current practice of using the median value to trigger dune removal will, by definition, underestimate dune erosion 50% of the time. This is not a concern where variability about the median value is small or where the consequences of underestimation are minor. However, the reports upon which the 540 SF criterion is based (Hallermeier and Rhodes, 1986; Dewberry & Davis, 1989) documented significant variability about the median value. Other studies (e.g., Chiu 1977, USACE 1984, Savage and Birkemeier 1987, and Birkemeier et al., 1988 ) also found wide variability in above-stillwater level erosion from one location to another —generally, the maximum eroded area was found to range from 1.5 to 6.6 times the median value. The State of Florida's Coastal Construction Control Line (CCCL) erosion model uses a factor of 2.5 to adjust the average erosion upward to a value more consistent with post-storm observations of maximum vs. average erosion.

### 3.3.3 Recommendations

Thus, it is recommended that the review of the erosion-frequency relationship consider – if preliminary assessments suggest – that a larger cross-section (than the median erosion value) be considered as a trigger between dune removal and dune retreat.

Note that the above recommendation is not inconsistent with FEMA guidance to Study Contractors in Sec. D.2.4.4, which recognizes the variability of dune erosion during a given storm, and which cautions that use of a single value to characterize dune erosion may be inaccurate. *Appendix D* recommends historical data be used, wherever possible, to guide erosion assessments for the 100-year flood event.

An other consideration is the use of the present 540 SF value may or may not be the best characterization of the median erosion value during a 100-year event, but a 540 SF frontal dune reservoir represents a large dune, and few dunes exceed this value. The net result of using 540 SF is that most dunes are removed during the erosion assessment. Moreover, frontal dune reservoir determinations are not the source of flood insurance study appeals or challenges.

However, use of the 540 SF median value does not account for the effects of multiple storms on large dunes (or bluffs, if the method is applied there). In addition to capturing more of the erosion affected areas during a 100-yer event, use of a value higher than the median value may extend the "shelf life" of Flood Insurance Rate Maps by compensating for multiple storms or erosion over a period of time.

***Determine erosion area-frequency relationship (is 540 SF the median?)***

Following are specific recommendation for re-evaluating the area-frequency relationship:

- Update 38-storm database to include other Atlantic and Gulf of Mexico open coast storm (profile and water level) data
- Re-evaluate existing 38-storm and updated data set, including use of updated flood elevation-frequency data and wave setup information in published FISs.
- Consider effects of storm duration in the analysis of 38 original storms and more recent storm erosion data
- Develop an updated erosion-frequency relationship, determine median and other values
- Evaluate data from the 38 original storms and the more recent storm erosion data to determine whether FEMA eroded geometries for retreat and removal profiles are appropriate

***Review use of the median value as the trigger for dune retreat***

Following are recommendations for further evaluation of the uses of the median value as the trigger for dune retreat. Table 1, at the conclusion of this report, summarizes the key findings and recommendations for this topic, and Table 2 provides an estimate of the amount of time needed to accomplish tasks recommended for this topic.

- Review erosion variability for the 38 original storms and more recent storm erosion data

- Contingent on the results of the erosion area-frequency and variability analyses, determine whether the median value trigger should be maintained or revised. If a revised trigger is indicated, determine the appropriate value(s)

## 3.4 TOPIC 38: PHYSICS- OR PROCESS- BASED EROSION ASSESSMENT

### 3.4.1 Description of the Topic and Suggested Improvement

The severity of a storm-induced erosion event for a subject sandy beach can be characterized by vertical scouring and horizontal erosion. The vertical scouring establishes the likely lowest beach elevation in front of a coastal protective device or in the frontshore area during a storm event. The potentially highest wave runup associated with the storm-induced waves and a high tidal water level during the wave attack period can then be estimated. The horizontal erosion is used to determine a safe setback of coastal dwellings as well as ground clearance required to prevent wave-runup flooding. Typical beach conditions applicable to the coastal regions including the Atlantic/Gulf, Great Lakes, and Pacific regions are:

a) Sandy beach backed by protective dune formation

b) Sandy beach with shore protective device (i.e., revetment or seawall)

c) Sandy beach without either shore protective device or dune formation

d) Wave-cut coastal bluff fronted by narrow sandy beach

e) Cobble or shingle beach with or without the presence of sea cliff

***Sandy Beach Backed by Protective Dune Formation***

This type of beach morphology exists mostly in the Atlantic/Gulf region. Coastal sand dunes usually extend above the designated Stillwater Flood Elevation (i.e., one-percent occurrence), but such barriers used for storm flood protection may not be permanent, as the protective dunes will be eroded away during a severe storm event and may require decades to rebuild under the action of wind. Storm-induced erosion that removes and modifies the geometric formation of the barrier dunes allows impinging waves to propagate further inland and results in overwash flood in the coastal low-lying areas. The 540 SF Criterion (i.e., geometric dune-erosion model) has been extensively applied to the Atlantic/Gulf Coast to determine the required Stillwater Flood Elevation (SWEL). A detailed review of this criterion is being presented under Topic 37 of this focus study. In addition, process-based erosion models (e.g., EBEACH) can also be used in some settings to simulate the erosion scenario based on the physical process of storm wave attack combined with the induced high water level.

#### *Sandy beach with shore protective device (i.e. revetment or seawall)*

A shore protective device is constructed when the fronting sandy beach cannot provide an adequate buffer against storm wave attack, even if the subject sandy beach is in relatively stable conditions, except for experiencing seasonal variation in beach width. There is an ongoing debate among coastal engineers and coastal geologists as to the long-term effects of a shore protective device on the fronting sandy beach. However, there is little argument as to the additionally induced short-term impacts. The degree of the short-term impacts depends on the type of shore protective device. Under conditions of large storm surges, a shore protective device can be subject to wave impacts before any substantial erosion of the beach can occur. Waves reflecting from the shore protective device, particularly during the storm-attacking period, can result in increased scour at base of the structure. Additionally, seawalls and revetments have been documented to place additional erosional stress on the adjacent shorelines during storms. The short-term scouring potential on the beach fronting a shore protective device is critical to the estimate of wave runups and potential overtopping that would subsequently determine the Stillwater Flood Elevation (SWEL). On low storm surge conditions, such as found along much of the West Coast beaches, appropriately sited seawalls at the back of the beach are not subjected to significant wave reflection until after the beach has been eroded, typically to bed rock or cobble – usually by a series of storm events. An applicable process-based model should be able to account for the effects due to the presence of a shore protective device.

#### *Sandy beach without either shore protective device, a bluff or dune formation*

A sandy beach without a shore protective device and not backed by a bluff or a coastal dune generally implies a relatively wide backbeach berm that provides an adequate buffer against storm wave attack. In Southern California, beach profiles in the low-lying coastal area typically consist of an inshore zone, a foreshore with beach fronting slope, and a backshore berm without a dune formation. Under this type of beach morphology it is necessary to characterize a storm-induced erosion event for the subject sandy beach into two primary parameters; vertical scouring and horizontal erosion, as addressed in the previous section. However, much of the Southern California beach consists of a thin layer of sand overlying a wave-cut rock terrace, such that there is a well-defined limit to both the vertical scour and the horizontal erosion, regardless of the storm intensity.

#### *Wave-cut coastal bluff fronted by narrow sandy beach*

The bluff base is exposed to wave attack after the narrow sandy beach acting as a buffer is stripped away, particularly during the winter months. Bluff toe erosion occurs mostly during severe storm events when waves impinge upon the coastal cliff and induce mechanical abrasion at the base, forcing impact on small joints and fissures in consolidated earth and rock units, and hydraulic action on the bluff face. When bluff toe erosion extends to a threshold depth, the upper bluff loses its support at the base and subsequently collapses. Strictly speaking, this failure mechanism is not EBE in that no single storm event is responsible and the failure could occur under benign conditions. However, if the collapse itself is considered the event, it can be assigned a probability of occurrence. Kuhn and Shepard (1983) have documented the significant contribution of heavy rainfall years to episodic bluff failure along the Southern California coast.

***Cobble or shingle beach with or without the presence of sea cliff***

This type of beach morphology is commonly observed in Oregon and the Atlantic Northeast region. The shoreline segments with a cobble berm backed by sea cliff are also observed in Southern California. The resistance capability of a cobble berm/shingle beach against short-term wave-induced erosion is still not well understood. Field applications of constructing a cobble berm that acts as a shore protective device against storm wave attack have been initiated in Oregon and Southern California. A more detailed discussion of its erosion processes during a storm event can be found in Topics 33 and 34.

### 3.4.2 Description of Procedures in the Existing Guidelines

Presently, there are no guidelines and procedures for applying process-based erosion methods for any coastal regions, including the Atlantic/Gulf and Pacific coasts, are presently available (FEMA, 2003). Only an empirical geometric model (i.e., 540 SF Criterion) with detailed guidelines and procedures is provided for the applications of erosion assessment in the Atlantic/Gulf Coast and Great Lakes regions. Topic Numbers 30 through 33 provide a thorough discussion of this erosion assessment method.

### 3.4.3 Application of Existing Guidelines to Topic

Several process-based erosion models are available, particularly the SBEACH model that was developed by the USACE. Such models have been applied with limited success along the Atlantic and Gulf Coasts. Presently available process-based models have not been fully tested for wide-spread application and are therefore not recommended by present guidance in *Appendix D*. Such models are discussed further in the following section.

### 3.4.4 Alternatives for Improvement

Researchers have developed several process-based models, which are applicable to beach conditions of Categories a, b and c, which are briefly described above. These models that may improve the predicting capability of erosion assessments can be classified into two groups; simple (or “closed loop”) and comprehensive (or “open loop”) models. Closed loop models signify that the profile is constrained to converge to a specified (equilibrium) profile for constant wave and water level conditions whereas there is no such constraint for open loop models. The open and closed loop terminology was introduced by Dean (1995) in a review of cross-shore sediment transport models. Brief discussions of several of the models are provided in the following sections. In addition, a statistical model that can be used to predict the episodic occurrence of coastal bluff failure for the beach condition described in Category d, “Wave-cut coastal bluff fronted by narrow sandy beach,” is also presented. It is noted that most profile evolution models can account for additions or removals of sand from the profile; however, most applications have not included this capability. The applicability of geometric and numerical models for the cobble/single beach is addressed in Topic Numbers 33 and 34.

### *1) Simplistic (Closed Loop) Process-Based Models for Storm-Induced Beach Erosion*

**SBEACH**

The Storm-induced BEAch CHange (SBEACH) numerical model was developed by the U.S. Army Corps of Engineers as an engineering tool for simulating beach profile evolution in response to storms. Detailed information on model development and application is provided in a series of technical and instruction reports (Larson and Kraus 1989; Larson, Kraus, and Byrnes 1990; Rosati et al. 1993; Wise, Smith, and Larson 1996; Sommerfeld, Kraus and Larson 1996; Larson and Kraus 1998).

SBEACH is an empirically based numerical model for simulating two-dimensional cross-shore beach change. The model was initially formulated using data from prototype-scale laboratory experiments and has been further developed and verified with laboratory and field data primarily from beaches on the Atlantic Coast. SBEACH calculates meso-scale beach profile change with emphasis on beach and dune erosion as well as bar formation and movement. The model is intended for predicting the short-term profile response to storms (i.e. single- or multiple-storm events)

As noted, a fundamental assumption of SBEACH and other closed loop models is that the profile change is produced solely by cross-shore processes, resulting in a redistribution of sediment across the profile with no net gain or loss of material. Longshore processes are considered to be uniform and neglected in calculating profile change. This assumption limits the model to be valid only for short-term storm-induced profile response on open coasts away from tidal inlets and coastal structures. However, if the details of volume change are available, this can be taken into consideration by this and other closed loop models.

In SBEACH the beach profile change is calculated from application of the mass conservation equation and a cross-shore sediment transport equation. The mass conservation equation relates the temporal change of the beach profile to the cross-shore gradient of the net cross-shore sediment transport. The net sediment transport rate relationships are developed based on physical considerations and analysis of large wave tank data. The sediment transport computations are separated into four zones: swash zone, broken wave zone, breaker transition zone, and pre-breaking zone. A transport formula similar to that used by Kriebel and Dean (1985) in the development of EBEACH is applied for the surf zone, and transport relationships in the other zones are empirical and based directly on the data from the wave tank experiments. In applications, sand is exchanged between the four zones of the profile, and the volume of total sediment is conserved to maintain a balance within the evolving profile.

SBEACH requires data typically available in engineering studies to calculate beach profile response. For project applications, primary input to SBEACH includes time-histories of storm wave height and period (direction is optional) and water level; beach profile survey data; and median sediment grain size. Sampling intervals of input wave and water level time-histories usually range from 1 to 4 hours. Input required for model configuration includes parameters such as grid size, time-step, and calibration coefficients (default values are available). Typical values of model grid size and time-step are 3 meters and 5 minutes, respectively. SBEACH can be operated as a module in the commercial software package such as the Coastal Engineering Design and Analysis System (CEDAS) with a user-friendly interface.

The model enhancements after initial development of SBEACH include a random wave model and refined sediment transport relationships to improve calculation of beach response under random waves, an algorithm to simulate beach and dune erosion produced by overwash, seawall representation, and simulation of profile change over non-erodible bottoms. The wave model is now relatively sophisticated and computes wave shoaling, refraction, breaking, breaking wave-re-formation, wave- and wind-induced setup/setdown, and runup. Areas of future model development include representation of variable sediment grain size across the profile, and improved calculation of sediment transport in the offshore zone to describe movement of dredged material placed in submerged mounds.

Because of the empirical foundation of SBEACH and natural variability that occurs along the beach during storms, the model should be tested or calibrated using data from specific beach profiles surveyed before and after storms on the project coast. The model prediction should be carefully evaluated based on coastal engineering experience and knowledge, and observation of the project coast. If reliable calibration data are not available, SBEACH should be used with caution and validation is recommended.

The SBEACH model has been calibrated with data from prototype-scale wave basin, field research facility, and field studies. It has been successfully applied to numerous field case studies on the East and Gulf Coasts, and to a degree in the Great Lakes, environments that most closely fit the conditions for which it was developed and calibrated. However, several less-successful experiences using SBEACH on the coast of California (USACE-LAD, 1994) and Oregon (Komar, 2004b) seem to indicate that SBEACH may under-predict the erosion during storms on the West Coast, where the beach morphology and storm characteristics differ from its development. Recently, the USACE has officially recognized the inadequacy of SBEACH to predict erosion on West Coast beaches and has funded a research program to determine the causes and to suggest ways to overcome the deficiencies. One likely cause of the problem is that SBEACH contains a switch to turn on the erosion prediction methodology which is based upon calculating the fit of the profile to the Dean $Ay^{2/3}$ model. Another possibility is the lack of infragravity swash predicted by SBEACH, but which is central to erosion of Pacific Coast beaches. The importance to the USACE of a viable SBEACH-type tool for the West Coast would seem to indicate that the model will eventually be improved or replaced. However, as of this writing, no schedule is set for completing this.

**EBEACH**

The EBEACH model, a closed-loop-type model, was developed by Kriebel and Dean (1985) for predicting time-dependent, two-dimensional beach and dune erosion during severe storms due to elevated water levels and waves. Detailed information on model development and application can be found in a series of publications (Kriebel, 1982, 1984a, 1984b; Kriebel and Dean, 1985; Kriebel, 1986, 1990).

While conceptually similar to the geometric dune erosion models (as e.g., Edelman, 1968; Vellinga, 1982), the EBEACH model represents a distinct improvement in that it evaluates the dissipation of the wave energy within the nearshore and calculates the cross-shore sediment transport based on that wave dissipation. Therefore, while the geometric models predict the maximum potential dune erosion that might occur during a storm, EBEACH and SBEACH provide an evaluation of the actual cross-shore profile adjustment of natural beaches to storm conditions and account for the time varying wave heights and water levels in a natural manner.

As SBEACH does, a fundamental assumption of EBEACH is that profile change is produced solely by cross-shore processes. Like the geometric models and SBEACH, EBEACH assumes the existence of an equilibrium beach profile that is governed by the median grain size or fall velocity of the beach sediment. In EBEACH and SBEACH, the local cross-shore sand transport rate in the surf zone is linked to the difference between the local wave energy dissipation per unit volume and equilibrium wave energy dissipation per unit volume corresponding to the equilibrium beach profile. In EBEACH, a general equilibrium beach profile found by Bruun (1954) and further developed by Dean (1977) was used in the outer surf zone, while the profile of the inner surf zone is taken to have a uniform slope, the angle depending on the sediment grain size.

The model employs an equation of sediment mass conservation to relate the time-dependent profile evolution to the cross-shore gradient of the cross-shore sand transport rate, and a dynamic equation governing the cross-shore sand transport due to the disequilibrium of wave energy dissipation levels. This methodology was essentially used in the development of SBEACH.

The recent enhancements to EBEACH include the addition of the swash runup of the waves at the shore, calculated with the Hunt formula, and a more accurate depiction of the dune profile variations.

EBEACH has also been calibrated to the large-scale laboratory wave-tank experiments and field data on the East and Gulf Coasts. EBEACH can be operated as a module in a commercial software package such as the Automated Coastal Engineering System (ACES). Komar, et al (1999) has tested both SBEACH and EBEACH and found that they tend to under predict erosion on the Oregon Coast. This may be due, in part, to the infragravity wave setup and runup that are present on the Pacific Coast during severe events, but not included in the inputs to these models.

**SBEACH versus EBEACH**

SBEACH is conceptually similar to EBEACH in many respects. Although, they both assume the beach profile evolution during a storm is solely caused by the cross-shore gradient of the cross-shore sand transport and thus use the same equation to link beach evolution to sand transport rate, if sand addition or removal were specified along with the cross-shore locations of the addition and removal, these models could take this effect into account. The semi-empirical formulas for the cross-shore sand transport rate in the surf zone are both based on the similar concept in that the transport rate is linked to the difference between the wave energy dissipation and the equilibrium energy dissipation. Both models have been calibrated to laboratory experiments and field data on the East and Gulf Coasts, but with less effort and success for the West Coast.

While SBEACH is conceptually similar to EBEACH in many respects, the capability of SBEACH appears to be more comprehensive. SBEACH accounts for the formation of break-point bars, has a relatively detailed consideration of sand transport rate, has a more appropriate wave model, and is capable of being applied to cases with more complex bottom features such as non-erodible hard bottoms. In addition, SBEACH is designed to be run by technicians having only modest training and thus has been well documented by accompanying manuals.

Both SBEACH and EBEACH can be potentially used as the simple process-processed models for the short-term beach and dune evolution during storms. Both models have been calibrated and successfully applied to the East and Gulf Coasts. As discussed previously, significant efforts to reformulate and validate these models are necessary in order to apply them to the West Coast.

### *2) Comprehensive (Open Loop) Process-Based Models for Storm-Induced Beach Erosion*

The major advantage of the simplistic models, such as SBEACH and EBEACH, lies in their theoretical simplicity and computational efficiency. However, many aspects of these models are empirical rather than based directly on the nearshore processes. The fundamental assumption of the beach profile evolution solely caused by cross-shore sand transport and the empirical formulations of cross-shore sediment transport rate result in these models being used with limited application and less accuracy. A more accurate and detailed analyses of beach evolution demands a more sophisticated model that is less empirical, but based more on the nearshore processes and a "state-of-the-art" assessment of the sediment transport processes. Such models have been developed during the last two decades, but may not be fully tested, documented and ready for application in coastal FISs.

**European Models**

Several sophisticated models for nearshore processes have been individually developed by European research institutes such as the Danish Hydraulic Institute (DHI), Delft Hydraulics, and the University of Liverpool. These models are fundamentally similar but differ in detail as to how they simulate nearshore hydrodynamics and sediment transport, and differ in their computational procedures. Hedegaard et al. (1992) has presented a thorough review of European cross-shore sediment transport models available at the time of her review.

These models variously incorporate simulations of wave transformations, wave-induced mean water level variation (setup or setdown), wave induced undertow, and alongshore currents, the transport of the suspended and bedload sediments as well as the beach evolution. The wave and circulation modules incorporated in these models predict wave transformation and wave induced circulation in the nearshore region, and provide the flow particle velocity consisting of the wave induced current component and wave orbital velocity component as inputs to the sediment transport modules. Some models include both suspended load and bedload while others only include the suspended sediment load. The bedload transport rate is calculated using formulas that directly link the transport rate to the flow velocity or bottom friction. The suspended load transport rate is obtained by solving the sediment diffusion equation and is dependent on flow conditions such as flow velocity, bottom friction and turbulent diffusion as well as the sediment characteristics. By using the sediment mass conservation equation the temporal evolution of the beach profile is related to the spatial variation of the total sediment transport rate in both the cross-shore and alongshore directions.

These sophisticated models provide a more comprehensive depiction of coastal processes and the mechanism of nearshore sediment transport and beach evolution, and thus are superior in their physics. These models are also capable of providing more comprehensive and detailed information about nearshore processes and beach response. These models continue to be improved and have been tested against extensive laboratory experiments and a few field cases. Application requires significantly more data and effort than SBEACH- or EBEACH-type models. However, the results provide far more information on beach adjustment during and after storm events. The sophistication of these models is offset, to some degree, by the possibility of them providing unrealistic results and tendencies for instability.

**A Nearshore Processes Model developed by University of Delaware and U.S. Army Engineer Research and Development Center**

Another effort in the modeling of nearshore processes (Qin, 2003; Svendsen, 2003) has recently been performed at the University of Delaware in a joint research effort with the U.S. Army Engineer Research and Development Center's (ERDC) Coastal and Hydraulics Laboratory (CHL). The primary developer of this model, Wenkai Qin, is currently employed with Noble Consultants, Inc. in California.

The capability of this model is similar to the European models in that the complex nearshore processes including wave transformation, wave-induced circulation, sediment transport and beach evolution can be comprehensively simulated. However, other important improvements have also been incorporated in the model.

The wave module in this model can be selected from REFDIF (Kirby and Dalrymple, 1994), a cnoidal wave-bore model (Svendsen, Qin and Ebersole, 2003), or a kinematic irregular wave (Qin and Svendsen, 2003). The Quasi-3D nearshore circulation model SHORECIRC (Svendsen et al. 2002) is used as the circulation module. In the sediment transport module, the Engelund and Fredsøe (1976) formula, the Bailard and Inman (1981) formula or their modified version can be selected to calculate the bed load transport rate, and the suspended transport can be estimated either by solving the sediment diffusion-convection equation or by using the modified Bailard (1981) equation after including the contribution of the wave breaking process. The model is capable of predicting both alongshore and cross-shore sediment transport rates, the breaker bar formation and migration as well as erosion in the surf zone during a storm.

It is also important to mention that by developing a kinematic irregular wave model, not only the averaged quantities but also the long-wave infragravity motions of the nearshore hydrodynamics and their effect on sediment transport can be accounted for by this model.

### *A Statistical Model for Bluff Failure*

For the beach morphology that is characterized as a hard bottom backed by a coastal bluff, the evolution of this bluff-type shoreline is significantly different from that of a sandy shoreline. Storm waves that directly impinge on the bluff initially induce toe erosion at the base of the bluff, and the accumulation of individual storm-related toe erosion ultimately triggers the bluff face to steepened ultimately collapse. This type of bluff failure is frequently observed along north Atlantic and in many locations in California, Oregon, and Washington.

Previous estimates for coastal bluff retreat have always resorted to a temporally averaged rate over a long period (an average annual rate of retreat) based on long-term records. Though the annualized rate of coastal cliff erosion is a good indicator of the gradual retreat of the bluff top, it does not adequately represent the episodic nature of bluff failure, when several meters of bluff top can instantaneously fail and fall to the beach face below. An annualized retreat rate essentially accounts for the long-term average effect of various episodic failure events combined with the periods of little or no erosion activity. As a result, the annualized retreat rate tends to yield a misleading picture of coastal cliff erosion as well as the resulting damage to bluff-top development and hazards to coastal communities often located on top of coastal bluffs.

During an investigation of the Encinitas/Solana Beach, California, shoreline area, Noble Consultants, Inc. developed a statistical model for the prediction of bluff failure induced by a series of storm attacks (USACE-LAD, 2003). A semi-empirical formulation was developed to quantify the short-term bluff toe erosion rate as a function of the intensity of impinging waves and the rock resistance of the bluff according to Sunamura (1982 and 1983). A Monte Carlo technique was then applied to simulate the random process of storm waves impinging upon the bluff base, inducing toe erosion, and subsequently triggering a bluff failure. The same statistical technique was also used to randomly select the size of upper bluff failure when it occurs. The entire simulations consisted of two Monte Carlo type random sampling procedures based on two formulated statistical distributions: (1) wave height at the bluff base, and (2) bluff failure size on the top. Statistical random populations of wave height at the bluff base were derived from hind cast deepwater waves via the wave propagation process. Bluff-top failure size was randomly selected from a detailed, comprehensive, historical database of bluff failures in the study area.

The results from the Monte Carlo simulations provide a synoptic accounting of bluff failure that closely resembles the natural process of bluff failure in both the short and long term.

This statistical model procedure is in the process of being certified by the U.S. Army Corps of Engineers, CERC as the designated numerical model for storm damage analysis related to coastal bluff failure. A flow chart of this modeling procedure is presented in Figure 9.

### 3.4.5 Recommendations

Since no existing guidelines and procedures are available for process-based modeling approaches flood hazard mapping partners, recommendations are herein presented to provide some preliminary guidelines for assessing EBE for beach conditions of Categories a through d. The procedure of assessing process-based erosion under beach conditions of Categories a, b and c includes two primary steps of 1) choosing an appropriate model for the simulation of the short-term erosion process; and 2) determining the oceanographic parameters (including storm waves and tides) during a storm event that are responsible for the process-based evolution. For Category d, the previously discussed statistical model can be applied. In addition, prior to any final validation of existing process-based erosion models, an interim approach is also recommended to provide a means for estimating the eroded beach profile during a severe storm event.

***Simplistic Models versus Comprehensive Models***

Both the simplistic and comprehensive processes-based models described above can potentially serve as FEMA models for the assessment of process-based erosion of a sandy beach. The simplistic models such as SBEACH and EBEACH are more empirically oriented and involve more assumptions that may limit their application. However, they are theoretically simple and computationally efficient. On the other hand, the more comprehensive models are more physics-based and capable of directly addressing the complex nearshore processes, including the mechanisms of nearshore hydrodynamics, sediment transport, and beach response. They can provide more comprehensive and detailed information of nearshore processes including beach evolution. The disadvantages of more comprehensive models lie in model complexity which may require more detailed data and boundary condition specifications, answers that may vary widely, instabilities, and computational inefficiency (longer model setup and run times).

It is therefore recommended that the selection of a simplistic or comprehensive model should be based on considerations of the specific project objective, beach material properties, and environment specific data requirements and overall budget. If numerous model executions are required for various storm conditions, the simplistic models are recommended to save on computations. On the other hand, if only a few executions are required, or the beach environment is too complex to apply the simplistic models, comprehensive models may be a preferred alternative.

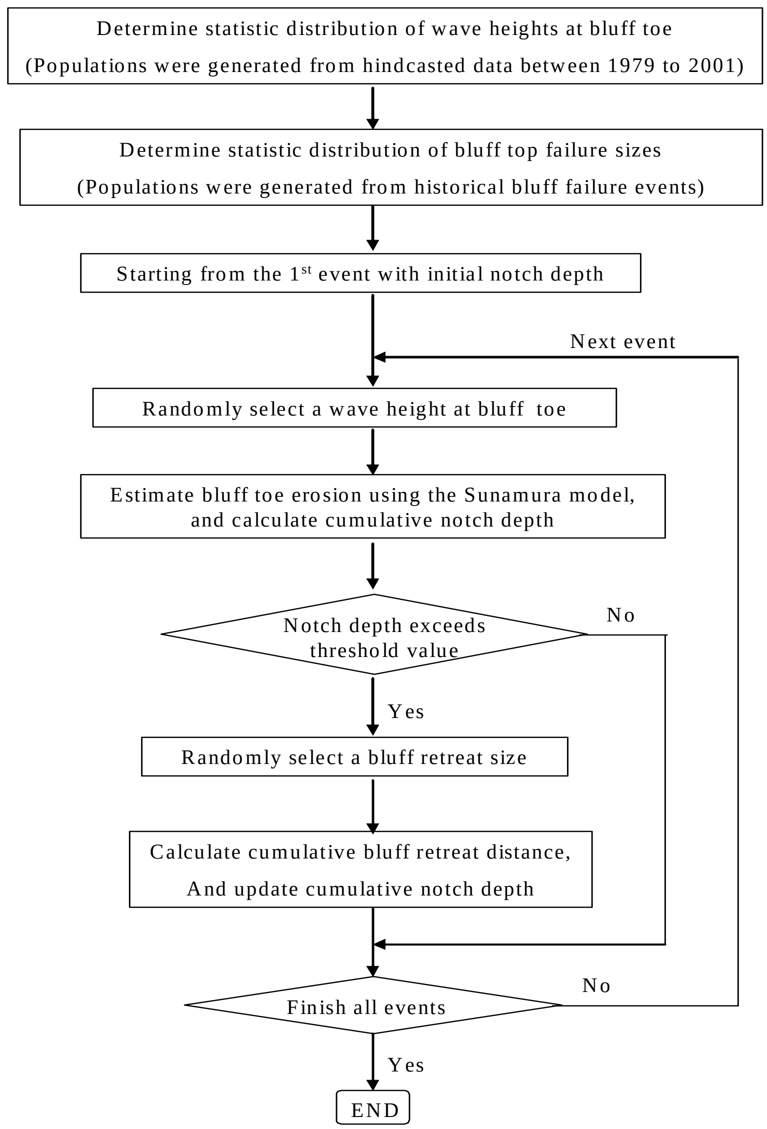



**Figure 9. Flow chart of the statistical model for bluff failure.**

### Random Coincidence Between Storms and Tides

Not only wave conditions, but also water depth, determine the severity of beach erosion during a storm event. The peak storm condition coincident with high or low tides will result in significant differences in the short-term beach erosion. Different combinations of storms and tides may induce the same amount of beach erosion. A 100-year storm may not be necessary to induce beach erosion that is equivalent to a 100-year erosion event, if return storm waves of 100 years arrived at a subject beach during the low tide condition. Therefore, it is essential to include various coincidences (joint occurrences) between the storms and tides in the analyses of the EBE.

It is recommended that a methodology be developed to include the randomness of storm waves, tidal elevations, and coincidence of these two oceanographic parameters. The preliminary concept of this methodology is illustrated in Figure 10. By analyzing all of the calculated results of beach erosion for all possible events, the event-based beach erosion for various return frequencies (as e.g., the one-percent EBE) can be determined.

#### *Interim Approach for Assessing Eroded Beach Profile*

Until process-based models are fully developed and tested, the EBE study group recommends that an interim approach be employed to estimate the eroded beach profile during a severe storm event so that wave runup and overtopping can be computed using the methodology detailed in Topics 11 through 14 (runup and overtopping). In the Atlantic, Gulf, and Great Lakes regions, the existing geometric model (referred to as the 540 SF Criterion) can be used to estimate the eroded beach profile conditions. Until specific methods are developed and accepted by FEMA for the Pacific region, eroded beach profiles can be estimated, using past field observations during historical severe storm events for various types of beach morphology and site conditions.

- For beach profiles that consist of a thin lens of sand overtopping the natural bedrock planform, it can be assumed that all sands will be stripped away during severe storm events. Thus, the profile of the bedrock planform (previously referred to as the “most likely winter beach profiles”) can be used as the beach profile in the calculation of the one-percent wave runup and flood base elevation. Topics 30 and 37, herein discuss this recommended procedure further.
- For sandy beaches that have a thick sand layer, the most eroded beach profiles documented during past storm events should be employed as the storm-eroded beach profiles for wave runup calculations. These most depleted beach profiles probably occurred in the 1983 El Niño year during which a cluster of severe storms sequentially impinged upon the Pacific Coast from California to Washington and resulted in the most wide spread of coastal damages along the West Coast. Historical and recent beach profile surveys that have been regularly conducted by the USACE, NOAA, regional governments, and local agencies such as counties and individual cities.

Table 1, at the conclusion of this report, summarizes the key findings and recommendations for this topic, and Table 2 provides an estimate of the amount of time needed to accomplish tasks recommended for this topic.

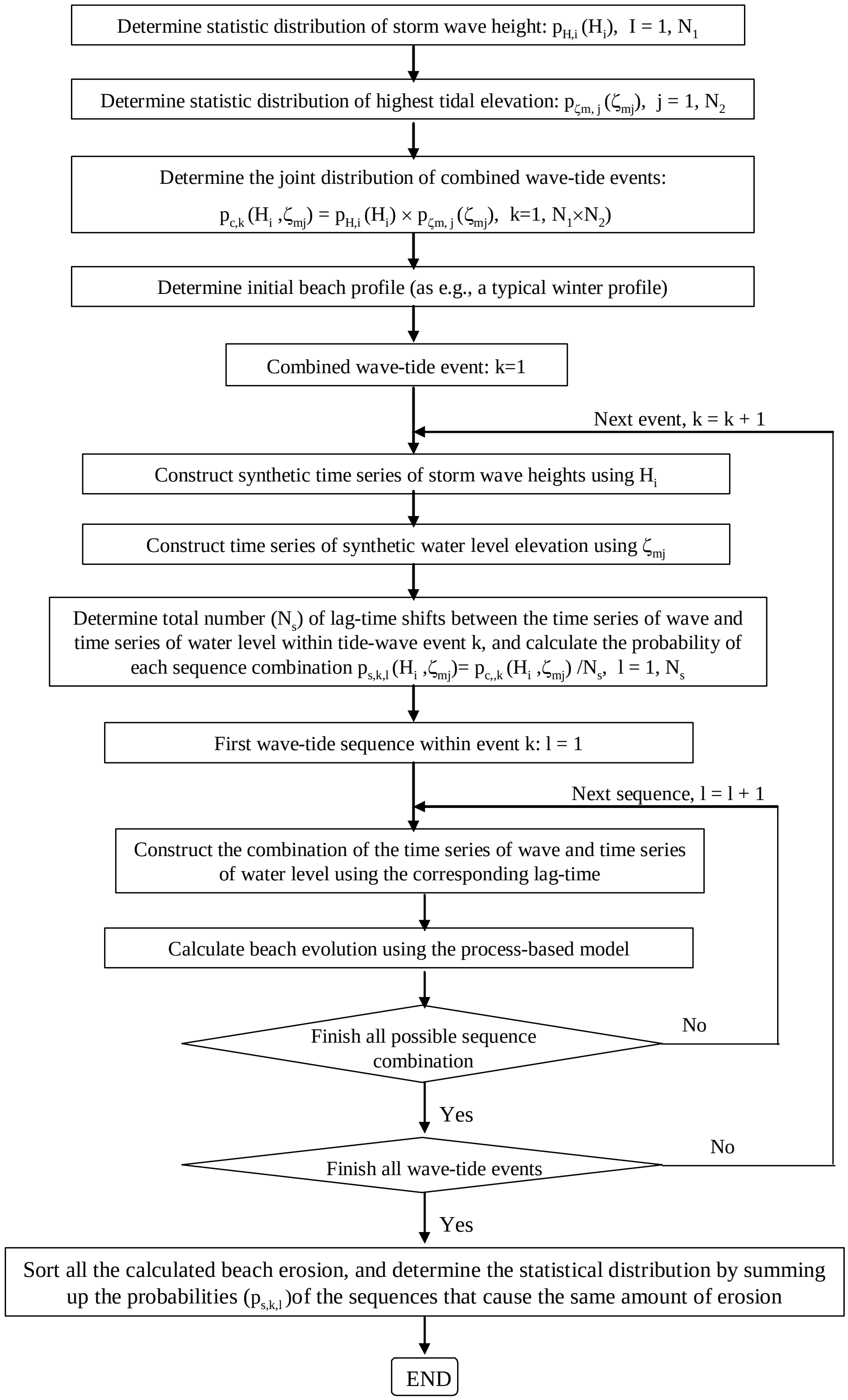



**Figure 10. Flow chart of statistical analysis of event-based erosion.**

# 4 AVAILABLE TOPICS

## 4.1 TOPIC 31: ADD/REVISE *G&S* LANGUAGE REGARDING BLUFF EROSION IN ATLANTIC/GULF AREAS

Topic 31 is categorized as an “Available Topic” for the Atlantic and Gulf areas. Sand-dominated dune erosion is reasonably covered in the present guidance in *Appendix D* by the 540 SF criterion for most Atlantic/Gulf areas with slight modifications to that criterion used in the Great Lakes. Other topics (33 and 34) discuss the needs to develop and provide new guidance for beach, dune, and back beach areas comprised of mixed grain materials, gravel, cobble, and shingles. Topic 31 is directed at better addressing “bluff erosion” in *Appendix D* and discussing whether a simple geometric model similar to the 540 SF criterion is necessary and can be developed for the Pacific Coast.

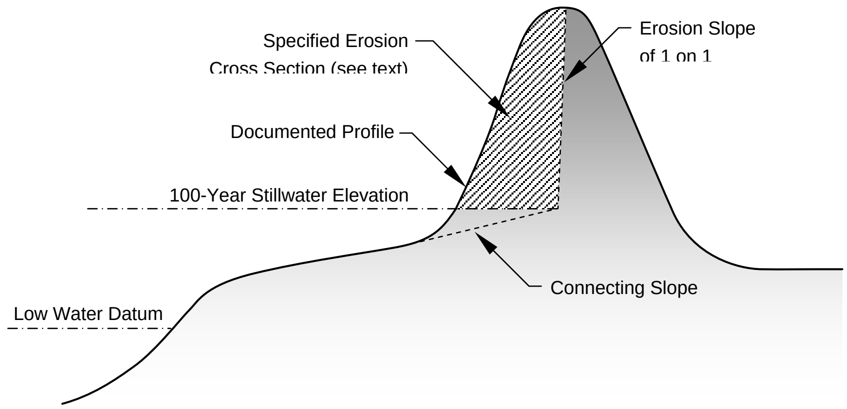
### 4.1.1 Description of Procedures in the Existing Guidelines

Present guidance in *Appendix D* focuses primarily on addressing erosion as sand-dominated dune face retreat or sand dune removal based entirely on the size of the frontal dune reservoir. These guidelines do not specifically address bluff erosion for the Atlantic, Gulf, or Pacific Coasts.

### 4.1.2 Application of Existing Guidelines to Topic

Because bluffs are often comprised of older, more consolidated materials with cohesive mixtures of soil, sand, and gravel materials, they are more erosion resistant than sand dunes. They may erode and retreat landward only periodically during or following rare intense storm events. However, unlike noncohesive sand dunes, bluffs rarely prograde (recover) back toward the ocean on a seasonal basis. Therefore, FEMA assumes that “bluff erosion is more of a long-term process” and not a present concern for FEMA according to present regulations.

The only place in the *G&S* where bluff erosion is mentioned is in Section D.3.4, Erosion Assessment for the Great Lakes areas. In this section of the *G&S* (page D-132), bluff erosion (projection of the retreating bluff face) “is based on a retreated profile assumed parallel to the existing bluff, but with a potential adjustment to the eroded face governed by soil stability consideration for the site.” Figure D-39 from *Appendix D* shows a typical eroding bluff scenario.




**Basic Erosion Considerations for Coastal Sand Dune Provides Shaded Shore Profile for Great Lakes Base Flood. (from Figure D-39 in Appendix D)**

The *G&S* assume that there is unlimited material behind the face of an eroding bluff (or cliff) and that approximately the same beach and back beach-bluff face profile will exist during and after a large storm event. Therefore, all that occurs is that the bluff face retreats landward by an unpredictable amount every so often, but the barrier to landward flooding (the bluff face) remains as it did prior to the event.

#### 4.1.3 Alternatives for Improvement

The present guidance in *Appendix D* would benefit from the addition of more in-depth discussions of the characteristics, settings, and physical processes associated with coastal bluffs and bluff erosion. Many reports and papers are available (Bruun, 1988; Komar, 1997; Komar, Marra, and Allan, 2002; Kriebel and Dean, 1985, Nairn and Southgate, 1993; Roelvink and Broker Hedegaard, 1993; The Heinz Center, 2000; National Research Council, 1990) to provide this type of information. Several states and local agencies have also published coastal erosion mapping and management program documents that are very informative and cover large portions of the Pacific and Atlantic coastlines. A recent report by the U.S. Geological Survey (2004) provides a summary of historical shoreline changes and associated coastal land loss along the Gulf of Mexico. This report represents the first in a series that will eventually include the Atlantic Coast, Pacific Coast, and parts of Hawaii and Alaska.

Inclusion of these types of discussions and more explicit explanations of the physical processes responsible for bluff and cliff retreat will provide valuable information to Mapping Partners.

***Considerations for future development:***

The capabilities and reliability of process-based numerical models is improving each year. Noble Consultants, Inc. (2003) documents successful and practical methods for assessing bluff erosion using statistical procedures and numerical modeling (see detailed discussion in Topic 38, Physics- or Process-Based Erosion Assessments). Refer to "A Statistical Model for Bluff Failure" for a detailed discussion of a statistical model for bluff failure.

#### 4.1.4 Recommendations

1. Review available literature and reporting and select appropriate information for inclusion in the *G&S* to describe the physical and geotechnical processes responsible for bluff (and cliff) erosion and retreat. Include characterization of the durability of the bluff material.

2. Examine reports and documents used to develop the present bluff erosion guidelines for the Great Lakes. Select appropriate information for enhancing the *G&S*.

3. Similar descriptions should be included in the new Pacific *G&S*.

4. Further testing and application of Process-Based numerical/statistical modeling methods is encouraged. These methods are presently being applied in some locations with success. Further development looks promising. FEMA should consider these tools for future inclusion in the NFIP program.

### 4.2 TOPIC 32: DEVELOP IMPROVED GEOMETRIC METHODS FOR BLUFF EROSION IN THE ATLANTIC AND GULF AREAS

#### 4.2.1 Description of Topic 32 and Suggested Improvement

Topic 32 is categorized as an "Available Topic" for the Atlantic and Gulf areas. Sand-dominated dune erosion is reasonably covered in the present guidance in *Appendix D* by the 540 SF criterion for most Atlantic/Gulf areas. This Topic 32 is directed at addressing "bluff erosion" and whether a simple geometric model similar to the 540 SF criterion can be developed for such applications along the Atlantic.

#### 4.2.2 Description of Procedures in the Existing Guidelines

Present guidance in *Appendix D* focuses primarily on addressing erosion as sand-dominated dune face retreat or sand dune removal based entirely on the size of the frontal dune reservoir. The present guidance in *Appendix D* does not specifically address bluff erosion.

#### 4.2.3 Application of Existing Guidelines to Topic

See Section 4.1.3 for a discussion of this topic.

#### 4.2.4 Recommendations and Approach

At the present time FEMA considers periodic bluff (and cliff) erosion as long-term processes that are not considered under present regulations.

1. It is recommended that further descriptions of the physical processes responsible for bluff (and cliff) erosion to the *G&S* be added as described in Topic 31, emphasizing the durability of the bluff material.

2. Development of geometric models may not be necessary at this time to estimate beach and back beach profiles for runup and overtopping calculations.

3. As an interim method, prepare an estimate of the most likely amount of retreat during the 1% event from available information (maps, photos, documentation of the area, and survey data) and develop typical beach and back beach profiles for use in run up/overtopping calculations.

4. If it is determined that the bluffs are retreating rapidly and regularly, then the Mapping Partner should conduct further investigations regarding the rates and causes of the erosion and consult with their FEMA contract manager regarding how that may affect their zoning estimates.

5. Development of more detailed methods is not necessary unless FEMA determines how to change the regulations to include periodic bluff (and cliff) erosion in the NFIP.

6. The writers suggest changing the priority of this topic to "Available" while continuing to investigate opportunities for future implementation of more advanced modeling methods.

### 4.3 TOPIC 41: LONG-TERM EROSION/FUTURE CONDITIONS

#### 4.3.1 Description of Procedures in the Existing Guidelines

The focus study considered the topic of mapping long-term erosion on FIRMs and any necessary changes to the *G&S*. In short, the project team believes mapping long-term erosion is technically feasible, but problematic, given unresolved NFIP policy and implementation issues. This topic has received considerable attention by others (at the federal, state, and local levels), but time and budget constraints prevented this project team from contributing to the topic beyond reiterating its importance.

The project team considered the long-term erosion issues identified at Workshop 1 (expand *G&S* text on the topic; put warning notes on FIRMs, etc.) and concluded that until the many issues related to mapping future conditions on FIRMs are resolved, incorporation of long-term erosion in the *G&S* are premature. However, the project team strongly believes that the topic is important, that the topic should continue to be evaluated, and that better communication regarding erosion risk, impacts and mitigation should be undertaken in the period prior to the mapping of long-term erosion on FIRMs.

#### 4.3.2 Recommendations and Approach

This topic should continue to be evaluated by FEMA. New guidance can be developed once FEMA decides how best to account for long-term erosion processes within FIS's and FIRM's *G&S*.

### 4.4 TOPICS 42 & 43: ADD GUIDANCE REGARDING THE TREATMENT OF NOURISHED BEACHES IN FLOOD HAZARD MAPPING

#### 4.4.1 Description of Procedures in the Existing Guidelines

There is not much dispute that nourished beaches can protect upland development and reduce flood- and erosion-damage. However, there is considerable dispute over certain technical and policy issues, i.e., the longevity of nourishment projects, and whether and how they should be considered for flood hazard mapping purposes. One of the more thorough treatments of these topics is contained in the National Academy of Sciences report, *Beach Nourishment and Protection* (NRC, 1995). Dean (2002) presents methodology for predicting longevity of beach nourishment projects.

At present, the *G&S* provide no specific guidance to Mapping Partners and Study Contractors relative to beach nourishment. FEMA policy on the matter is best summarized by Davison, et al. (1996), written in response to the National Academy of Sciences report. In essence, FEMA policy has been to ignore the presence of nourishment projects in the establishment of flood hazard zones/BFEs and in the setting of coastal building standards. This procedure is similar to that used to remove "uncertified" coastal structures (structures not capable of withstanding the base flood event and/or structures without acceptable maintenance plans) from transects before erosion and wave analyses are performed.

What is not clear, however, is how a Mapping Partner or Study Contractor would actually "remove" a nourishment project before conducting erosion and wave analyses for flood hazard mapping. The result has been that some flood insurance studies have become effective using city- or county-wide topographic mapping updates (that include the nourished area) obtained through surveys following nourishment, while at the same time other communities have been discouraged from seeking revisions to FIRMs following beach nourishment.

#### 4.4.2 Recommendations and Approach

Table 1, at the conclusion of this report, summarizes the key findings and recommendations for this topic, and Table 2 provides an estimate of the amount of time needed to accomplish tasks recommended for this topic.

- The project team considered the beach nourishment issues identified at Workshop 1, and concluded that the *G&S* should be revised to direct mapping partners/study contractors to use the following procedure:

- Notify FEMA when a study area contains a shoreline that has been nourished in the past.

- Research the nourishment project(s) and conduct preliminary analysis to determine whether the nourishment is likely to have an impact on hazard zone designations or BFEs over the long term.

- If the presence of nourishment is likely to affect hazard zone designations or BFEs over the long term, contact FEMA to discuss a possible exception to existing FEMA beach nourishment policy.

- The project team also recommends that the *G&S* be revised to include a listing of the types of information that may be required to assess special cases where exceptions to FEMA's beach nourishment policy may be granted.

#### 4.4.3 Topics 42 & 43: Availability

Information to address Topics 42 & 43 is available and easily incorporated into existing guidance.

# 5 HELPFUL TOPICS

## 5.1 TOPIC 40: CALCULATE VERTICAL EROSION DEPTHS

### 5.1.1 Description of Topic and Suggested Improvement

Topic 40, Calculate Vertical Erosion Depths, is the only topic categorized as "Helpful" during Workshop 1 in December 2003. Most economic flood damage models use "depth-damage" functions to calculate flood damages. Depth-damage functions relate the percentage of building damage to the depth of flooding (from the top of the wave crest or the stillwater surface to the ground). Functions vary by flood hazard zone and building type.

There is a trend in flood loss modeling to include other flood-related hazards. For example, HAZUS considers flood depth and vertical erosion depth (the vertical distance between the original ground elevation and the [event-based] eroded ground elevation). These analyses require erosion depth-damage functions, which relate the percentage of building damage to the vertical erosion depth. Erosion depth-damage functions vary with foundation type.

This topic is merely a placeholder for future use–as flood hazard methods and models are coded, we should build in the capability to calculate and store vertical erosion depths (along transects or grids). These vertical erosion depths can then be used by economic models to estimate building damages due to erosion.

### 5.2 AVAILABILITY

No specific information is required to address Topic 40 at this time. Future development and refinement of erosion depth-damage functions will be required, but these tasks are not included in the time and cost estimate below.

## 6 SUMMARY

Present *G&S* do not provide specific guidance for assessing EBE in coastal areas of the Pacific, Sheltered Waters on either coast, or non-sandy beach and coastal dune areas, and provide only simplified empirical-based geometric relationships (the 540 SF Criterion) for the Atlantic and Gulf. Therefore, new or improved methods are needed for the Pacific, Atlantic and Gulf, especially where beaches, dunes and bluffs are comprised of sediment materials other than uniform sand.

The EBE Study Team was tasked to: 1) develop improved language, descriptions and discussions related to coastal erosion assessments for consideration in revised and/or new FEMA *G&S*, 2) to review empirical geometric techniques and process-based methods for estimating beach and back beach profiles resulting from a 1-percent-annual-chance storm event in various settings along the Atlantic Coast, Gulf Coast and Pacific Coast, 3) review the present 540 SF Criterion for assessing EBE, 4) review and discuss methods for assessing EBE along cobble/shingle beaches, 5) recommend improved geometric model procedures for the Atlantic, Gulf and Pacific coastal regions, 6) prepare descriptions and discussions regarding erosion assessments in sheltered areas, 7) discuss steps to take and list the types of information that Study Contractors should provide to FEMA in cases where beach nourishment may be considered in determining hazard zones and BFEs, and 8) recommend approaches for improving or preparing guidelines in each topic area.

Following are brief summaries of the findings and recommendations for the key topics associated with EBE. The following tabular summaries are grouped into *Critical*, *Important,* and *Available* categories of topics as were defined by the TWG during Workshops 1 and 2. Table 2 provides an estimate of the amount of time required to accomplish tasks that are recommended for each topic.

**Table 1. Summary of Findings and Recommendations for Event Based Erosion**

| Topic Number | Topic | Coastal Area | Priority Class | Availability/ Adequacy | Recommended Approach | Related Topics |
|---|---|---|---|---|---|---|
| 30 | Geometric Techniques | AC | -- | -- | 1. Select and evaluate existing geometric methods and models for application along Pacific Coast. Methods should include effects of storm duration and sediment erodibility. Document results.<br>2. Develop guidance for determination of a Most Likely Winter Beach Profile (Pacific) including areas of beach nourishment for Pacific coastal areas prior to the occurrence of the 100-year event. These profiles will be developed from historical beach profiles and recent LIDAR mapping of the Pacific coastline.<br>3. Evaluate geometric versus numerical modeling procedures for sand beaches and dunes on Pacific Coast and test with available data sets. Document results.<br>4. Recommend that FEMA to expand/support the present USGS/NOAA coastal survey program for the Pacific coast;<br>***Future, Long-Term Program Considerations:***<br>1. Expand/support the present USGS/NOAA coastal survey program for the Pacific coast; update likely winter profiles for various geomorphic settings; determine whether joint probability methods related to initial beach profiles, duration and material erodibility are necessary.<br>2. Develop and test new geometric models (or process-based models) for sandy beach and dune systems along the Pacific using data from the long-term program above.<br>3. Develop methods for assessing other types of non-sandy beach settings, such as cobble and gravel beaches based on the underlying physical processes controlling those coastal settings (See Topics 33 and 34)<br>4. Develop long-term data sets for model testing and validation. | 31, 32, 35, 36, 37 |
| | | GC | -- | -- | | |
| | | PC | C | MAJ | | |
| | | SW | | | | |

**Table 1. Summary of Findings and Recommendations for Event Based Erosion**

| Topic Number | Topic | Coastal Area | Priority Class | Availability/ Adequacy | Recommended Approach | Related Topics |
|---|---|---|---|---|---|---|
| 33 | Cobble/ Shingle Effects | AC | C | MAJ | 1. Prepare new section of Guidelines to describe differences between sand dominated beaches and gravel/cobble/shingle beaches found along the north Atlantic, Gulf, Pacific and in sheltered areas. Provide photos and profile information.<br>2. Gather existing literature on cobble, shingle and coarse-grained beaches to summarize the existing state of knowledge until specific guidelines can be developed and adopted.<br>3. Review literature on the design of and construction of dynamic revetments and cobble berms to provide guidance on their stability and long term development.<br>4. Examine other possible guidance and available beach and dune data sets for possible clarifications to the 540 SF Criterion for sand-dominated beaches versus gravel/cobble/shingle beaches.<br>5. Discuss the limitations of applying geometric models to cobble/shingle beach and dune areas.<br>Future Considerations:<br>6. Examine the applicability of existing equilibrium beach profile concepts and relationships to represent the response of cobble and mixed grain beaches to storms.<br>7. Prepare case studies using actual coarse grain beaches demonstrating application of the recommended methodology.<br>8. Prepare new guidelines for the Pacific Coast describing the physical processes associated with gravel/cobble/shingle beaches. | 30-32, 34, 37 |
| | | GC | C | MAJ | | |
| | | PC | C | MAJ | | |
| | | SW | C | MAJ | | |
| 35 | Erosion - Sheltered Waters | AC | (C) | Y | 1. Provide definitions and discussion for Guidelines for sheltered water types of beach morphology, materials, & wave characteristics.<br>2. Provide interim *G&S* based primarily on historical beach profiles & field observations. | 5, 6 36, 41 |
| | | GC | (C) | Y | | |
| | | PC | (C) | Y | | |
| | | SW | C | Y | | |

| Table 1. Summary of Findings and Recommendations for Event Based Erosion | | | | | | |
|---|---|---|---|---|---|---|
| Topic Number | Topic | Coastal Area | Priority Class | Availability/ Adequacy | Recommended Approach | Related Topics |
| 34 | Cobble/ Shingle -Geometric Method | AC | I | PRODAT | 1. Review literature on natural cobble, shingle and coarse-grained beaches. Provide key results to Mapping Partners for interim consideration.<br>2. Review literature regarding design and project response of "dynamic revetments and cobble berms." Summarize useful guidance and methodologies for application to cobble and single beaches.<br>3. Perform assessment and test of 540 SF criterion for cobble and single beaches. Document results as Case Studies.<br>4. Summarize pertinent national and international literature on gravel, shingle, cobble beach assessment methods.<br>5. Examine the applicability of existing equilibrium beach profile concepts and relationships to represent the response of cobble and mixed grain beaches to storms.<br>6. Determine whether process-based models can be developed in a relatively short period of time for application to both the Atlantic and Pacific coasts.<br>7. Provide interim *G&S* based primarily on historical beach profiles and documented case studies (AC and PC will be presented separately).<br>8. Recommend how to incorporate new procedures into *G&S*. | 12, 21, 33, 35, 38, 42 |
| | | GC | I | PRODAT | | |
| | | PC | I | PRODAT | | |
| | | SW | I | PRODAT | | |
| 36 | Geometric Method – Sheltered Waters | AC | I | Y | 1. Provide interim *G&S* for the AC & GC based primarily on historical applications of the 540 SF criterion on AC/GC.<br>2. Provide interim *G&S* for the PC based primarily on historical field observations developed on PC.<br>3. Perform pilot studies; refine procedures and describe methods for *G&S*.<br>4. Test models and incorporate event-based models where feasible into final *G&S* Sheltered Waters.<br>5. Provide guidance on appropriate models for erosion in sheltered waters. | 5, 6, 35, 38 |
| | | GC | I | Y | | |
| | | PC | I | Y | | |
| | | SW | I | Y | | |

**Table 1. Summary of Findings and Recommendations for Event Based Erosion**

| Topic Number | Topic | Coastal Area | Priority Class | Availability/ Adequacy | Recommended Approach | Related Topics |
|---|---|---|---|---|---|---|
| 37 | Review 540 SF Criterion | AC | I | DAT | 1. Expand database beyond 38 storm events for AC and GC using more recent data.<br>2. Re-evaluate existing data points,<br>3. Consider storm duration in analyses,<br>4. Evaluate geometry of retreat and removal profiles.<br>5. Consider variability of erosion about median at each data point.<br>6. Contingent on 1–5, determine whether median erosion trigger should be maintained or revised. | 32, 34, 36 |
| | | GC | I | DAT | | |
| | | PC | -- | -- | | |
| | | SW | -- | -- | | |
| 38 | Process-Based Approach | AC | I | Y | 1. Further develop & test process based models using field data and compare results with geometric models.<br>2. Develop method to include randomness of return storm waves & tides & coincidence in Item 1.<br>3. Provide *G&S* for erosion assessment to coastal bluff fronted by a narrow beach<br>4. As an interim method continue to use the 540 SF Criterion for AC & GL, and most likely winter beach profile or best documented winter profile for the PC | 30-32, 35, 36 |
| | | GC | I | Y | | |
| | | PC | I | Y | | |
| | | SW | I | Y | | |
| 31 | Bluff Erosion | AC | A | Y | ***Interim Task;***<br>1. Review available literature and reporting and select appropriate information for inclusion in the *G&S* to describe the physical and geotechnical processes responsible for bluff (and cliff) erosion and retreat. Try to characterize the durability of the bluff material.<br>2. Provide appropriate definitions and process descriptions in the Pacific *G&S*.<br>***Future considerations:***<br>1. Provide interim *G&S* based primarily on historical beach profiles and documented case studies.<br>2. Provide interim *G&S* based primarily on historical field observations.<br>3. Incorporate event-based models to establish final *G&S*.<br>4. FEMA should consider process-based numerical/statistical modeling methods for future inclusion in the NFIP program. In the mean time completed case studies should be documented and provided to FEMA for review. | 30, 32, 35, 36-38, 41 |
| | | GC | (A) | Y | | |
| | | PC | (A) | Y | | |
| | | SW | (A) | Y | | |

**Table 1. Summary of Findings and Recommendations for Event Based Erosion**

| Topic Number | Topic | Coastal Area | Priority Class | Availability/ Adequacy | Recommended Approach | Related Topics |
|---|---|---|---|---|---|---|
| 32 | Geometric Method for Bluffs | AC | I (A) | Y | ***Interim recommendation:***<br>1. Review and summarize existing bluff erosion assessment procedures and selected literature.<br>2. Consider development of geometric procedure for bluff erosion and cliff retreat.<br>***Future Tasks to consider;***<br>1. Develop geometric procedure for bluff erosion and cliff retreat.<br>2. Add further descriptions of the physical processes responsible for bluff (and cliff) erosion to the *G&S* as described in Topic 31.<br>3. Recommend how to incorporate new procedures into future *G&S*. | 12, 21, 33, 35, 38, 42 |
| | | GC | I (A) | Y | | |
| | | PC | (A) | Y | | |
| | | SW | (A) | Y | | |
| 41 | Long-Term Erosion | AC | A | Y | 1. Topic considered important to NFIP, but FEMA action on previous work pending, therefore, guidance best developed outside of current project.<br>2. Provide better risk communication to public - outside of *G&S*. | 30-32, 35, 36 |
| | | GC | A | Y | | |
| | | PC | A | Y | | |
| | | SW | A | Y | | |
| 42 & 43 | Beach Nourishment | AC | A | Y | Prepare guidance to:<br>1. Notify FEMA that study area includes beach nourishment area;<br>2. Conduct research and preliminary analysis to determine whether beach nourishment is likely to have an effect on hazard zone designations and/or BFEs;<br>3. Provide list of types of information that may be required to assess special cases where beach nourishment may be considered in determining hazard zones and BFEs (as an exception to existing FEMA policy). | 39, 41 |
| | | GC | A | Y | | |
| | | PC | A | Y | | |
| | | SW | A | Y | | |
| 40 | Vertical Erosion Depths Erosion depths | AC | H | Y | Document depths of erosion following storm events and maintain data for depths of erosion and damages to buildings in order to better determine “depth-damage” relationships. As methods and models are coded, calculate and store vertical erosion depths along transects and grids. . These vertical erosion depth data can then be used in economics models to estimate building damages due to EBE. | 30-36 |
| | | GC | H | Y | | |
| | | PC | H | Y | | |
| | | SW | H | Y | | |
| | | | | | | |

| Table 2. Preliminary Time Estimate for Guideline Improvement Preparation | | |
|---|---|---|
| Topic Number | Item | Time (Person months) |
| 30 | **Review empirical geometric techniques and pre- and post-event data for CA, OR, WA; review OR setback methods, develop geometric techniques for pacific shorelines, including sea cliff, bluff, dunes beaches** | |
| | 1. Select and evaluate existing geometric methods and models for application along Pacific Coast. Methods should include effects of storm duration and sediment erodibility. Document results. | 4 |
| | 2. Develop guidance for determination of a Most Likely Winter Beach Profile (Pacific) including areas of beach nourishment for Pacific coastal areas prior to the occurrence of the 100-year event. These profiles will be developed from historical beach profiles and recent LIDAR mapping of the Pacific coastline. | 3 |
| | 3. Evaluate geometric versus numerical modeling procedures for sand beaches and dunes on PC and test with available data sets. Document results. | 3 |
| | ***Long-Term Program:*** 4. Expand/support the present USGS/NOAA coastal survey program for the Pacific coast; update likely winter profiles for various geomorphic settings; update likely winter profiles for various geomorphic settings. | Future Programs |
| | Total | 10 |
| 33 | **Add *G&S* descriptions/discussion regarding effect of cobble/shingle materials (including sediment mixtures/layers) on geometric erosion techniques** | |
| | Prepare new section of Guidelines to describe differences between sand dominated beaches and gravel/cobble/shingle beaches found along the north Atlantic, Gulf, Pacific and in Sheltered areas. Provide photos and profile information | 1 |
| | Gather existing literature on cobble, shingle and coarse-grained beaches to summarize the existing state of knowledge until specific guidelines can be developed and adopted | 1 |
| | Review literature on the design of and construction of dynamic revetments and cobble berms to provide guidance on their stability and long term development | 0.5 |
| | Examine other possible guidance and available beach and dune data sets for possible clarifications to the 540 SF Criterion for sand-dominated beaches versus gravel/cobble/shingle beaches | 1 |
| | Discuss the limitations of applying geometric models to cobble/shingle beach and dune areas | 0.5 |
| | Prepare New Guidelines for the Pacific coast describing the physical processes associated with gravel/cobble/shingle beaches | 1 |
| | Future Research: Examine the applicability of existing equilibrium beach profile concepts and relationships to represent the response of cobble and mixed grain beaches to storms. | N/A |
| | Future Research: Prepare Case Studies using actual cobble and coarse grain beaches demonstrating application of the recommended methodology. | Future Research |
| | TOTAL | 5 |
| 35 | **Add *G&S* descriptions/discussions regarding erosion assessments in sheltered areas** | |
| | 1. Provide definitions and discussion for Guidelines for sheltered water types of beach morphology, materials, & wave characteristics. | 1 |
| | 2. Provide interim *G&S* based primarily on historical beach profiles & field observations. | 2 |

| Table 2. Preliminary Time Estimate for Guideline Improvement Preparation | | |
|---|---|---|
| Topic Number | Item | Time (Person months) |
| | 3. Attempt to develop rational guidance based on a model consistent with the general framework discussed previously | Future Research |
| | 4. Develop Case Studies based on actual settings. | Future Research |
| | TOTAL | 3 |
| 34 | **Develop methods that consider cobble/shingle effects** | |
| | Gather, compile and summarize existing literature on natural cobble, shingle, and coarse-grained beaches to summarize the existing state of knowledge and provide references Mapping Partners can use until specific guidelines can be developed and adopted. | 2.5 |
| | Develop geometric procedure for estimating eroded profiles for cobble/shingle beaches | 3 |
| | Review literature on the design of and construction of dynamic revetments and cobble berms to provide guidance on their stability and long-term development (changes) | Future Research |
| | Review and assess the historical applications of the existing geometric model (SF540 Criterion) to the Atlantic/Gulf for natural gravel, cobble and mixed sand and gravel shorelines to determine its validity for these types of beach conditions. | Future Research |
| | Perform a demonstration test of 540 Criterion on a natural gravel, cobble and mixed sand and gravel beach. | Future Research |
| | Examine the applicability of existing equilibrium beach profile concepts and relationships to represent the response of cobble and mixed grain beaches to storms. | Future Research |
| | Determine whether generic process-based models can be developed in a relatively short period of time for application to both the Atlantic and Pacific coasts. | Future Research |
| | TOTAL | 5.5 |
| 36 | **Review data and develop geometric methods for determining eroded profiles in sheltered areas** | |
| | Review and assess the historical applications of the existing geometric model (540 SF Criterion) to the Atlantic/Gulf sheltered shorelines to determine the reliability of its applicability to the sheltered water regions. | 1 |
| | Develop interim eroded profiles for the Pacific Coast region, based primarily on historical erosion and beach profile surveys during extreme storm events, particularly in El Niño years such as 1982–1983 and 1997–1998 to provide interim *G&S* suitable to the Pacific Coast | 2 |
| | Test process-based models that are to be developed under Topic Number 38 to determine if they are suitable for the implementation in sheltered waters in all regions. | 3 |
| | Explore the possibility of developing a rational basis for predicting erosion in sheltered waters which is consistent with the general framework discussed previously. Such a framework should account for the time histories of water level and wave forcing, and the durability of the eroded material. | 3 |
| | Conduct Case Studies illustrating application of recommended approach using actual situations. | 3 |
| | TOTAL | 12 |
| 37 | **Expand database from which 540 was determined; review use of median value** | |
| | 37a. Determine erosion area-frequency relationship (is 540 SF the median?) | 4 |

| Table 2. Preliminary Time Estimate for Guideline Improvement Preparation | | |
|---|---|---|
| Topic Number | Item | Time (Person months) |
| | 37b. Review use of the median value as the trigger for dune retreat | 2 |
| | TOTAL | 6 |
| 38 | **Develop assessment procedures that consider temporal and longshore effects/variability** | |
| | Select simplistic or comprehensive process-based models based on site conditions & perform further model development & testing. | 4 |
| | Develop methodology to include random-ness of return storm waves, tidal elevations & coincidence of these two oceanographic parameters. | 4 |
| | Provide *G&S* for erosion assessment to coastal bluffs fronted by a narrow beach | 2 |
| | Develop and use interim "Most Likely Winter Beach Profile" approach until process based models are acceptable. | 1 |
| | TOTAL | 11 |
| 31 | **Add/revise *G&S* language regarding bluff erosion in Atlantic/Gulf areas – better descriptions and discussions needed** | |
| | Review available national and international literature and reporting and select appropriate information for inclusion in the *G&S* to describe the physical and geotechnical processes responsible for bluff (and cliff) erosion and retreat. Provide descriptions and examples. Include characterization of the durability of the bluff material. | 1.5 |
| | Future consideration: Examine reports and documents used to develop the present bluff erosion guidelines for the Great Lakes. Select appropriate information for enhancing the *G&S*. | 1 |
| | Future consideration: Improve descriptions of the physical processes affecting bluff (and cliff) erosion in Atlantic and Gulf areas. | 1 |
| | Future consideration: FEMA should consider Process-Based numerical/statistical modeling methods for future inclusion in the NFIP program. In the mean time completed case studies should be documented and provided to FEMA for review. | -- |
| | TOTAL | 3.5 |
| 32 (Assumed SAME as 31) | **Develop geometric method for bluff erosion in Atlantic/Gulf areas** | |
| | Review available national and international literature and reporting and select appropriate information for inclusion in the *G&S* to describe the physical and geotechnical processes responsible for bluff (and cliff) erosion and retreat. | 1.5 |
| | Examine reports and documents used to develop the present bluff erosion guidelines for the Great Lakes. Select appropriate information for enhancing the *G&S*. | 1 |
| | Improve descriptions of the physical processes affecting bluff (and cliff) erosion in Atlantic and Gulf areas. | 1 |
| | TOTAL | 3.5 |
| 42, 43 | **Ensure clarity in *G&S* that references FEMA policy statement regarding treatment of nourished beaches** | |
| | Develop methodology for determining whether a beach nourishment project and procedures in place will provide long-term storm damage reduction benefits | 2 |
| | Provide Clarification in *G&S* to Study Contractor providing procedures to be followed for cases where beach nourishment projects are present | 1 |
| | TOTAL | 3 |

| Table 2. Preliminary Time Estimate for Guideline Improvement Preparation | | |
|---|---|---|
| Topic Number | Item | Time (Person months) |
| 40 | **Maintain data and make available for use in building performance and insurance tasks** | |
| | 40a. placeholder topic | -- |
| | 40b. future development and refinement of erosion depth-damage functions | Not included |
| | TOTAL | -- |

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# APPENDIX A

Final Rule, 540 SF Criterion

Federal Register, May 6, 1988

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| Sections in 42 CFR that contain collections of information | Current OMB control numbers |
|---|---|
| | 0990-0058 and 0938-0247 |
| 433.138 | 0938-0502 |
| 433.139 | 0938-0459 |
| 434.6-434.20, 434.23-434-27, 434.30, 434.32, 434.36, 434.50, 434.53, 434.55 | 0938-0326 |
| 435.910, 435.919, 435.920, 435.940 | 0938-0467 |
| 435.945 | 0938-0467 and 0938-0502 |
| 435.948, 435.952, 435.955 | 0938-0467 |
| 435.960 | 0938-0467 and 0938-0502 |
| 435.965 | 0938-0467 |
| 441.56, 441.58, 441.60, 441.61 | 0938-0354 |
| 441.255-441.259 | 0938-0481 |
| 441.301 | 0938-0449 |
| 441.302 | 0939-0268 and 0938-0449 |
| 441.303 | 0938-0272 and 0938-0449 |
| 442.118 | 0938-0488 |
| 442.307, 442.308, 442.309, 442.311, 442.313, 442.314, 442.318, 442.319, 442.320 | 0938-0370 |
| 442.402, 442.404-442.407, 442.412, 442.413, 442.417, 442.421, 442.423-442.425, 442.427, 442.430, 442.434, 442.441, 442.443, 442.457, 442.460, 442.463, 442.466, 442.468, 442.475, 442.482-442.487, 442.490, 442.492, 442.497, 442.500-442.503, 442.505, 442.506, 442.512 | 0938-0366 |
| 447.30 | 0938-0067 |
| 447.31 | 0938-0287 |
| 447.53 | 0938-0429 |
| 447.253(a) | 0938-0193 |
| 447.255 | 0938-0193 |
| 456.654 | 0938-0445 |
| 466.70, 466.72, 466.74, 466.78, 466.80, 466.94 | 0938-0445 |
| 473.18, 473.34, 473.36, 473.42 | 0938-0443 |
| 474.36, 474.38-474.40 | 0938-0444 |
| 476.104, 476.105, 476.116, 476.134 | 0938-0426 |
| 482.12, 482.22, 482.27, 482.30, 482.41, 482.53, 482.56, 482.57 and 482.60-482.62 | 0938-0328 |
| 488.56, 488.60, 488.64 | 0938-0267 |
| 498.22, 498.40, 498.58, 498.82 | 0938-0508 |

(Catalog of Federal Domestic Assistance Program No. 13.714, Medical Assistance Programs; No. 13.773, Medicare—Hospital Insurance; No. 13.774, Medicare—Supplementary Medical Insurance)

Dated: March 18, 1988.

**William L. Roper,**

*Administrator, Health Care Financing Administration.*

Approved: April 8, 1988.

**Otis R. Bowen,**

*Secretary.*

[FR Doc. 88-10083 Filed 5-5-88; 8:45 am]

BILLING CODE 4120-01-M

**DEPARTMENT OF THE INTERIOR**

**Bureau of Land Management**

**43 CFR Public Land Order 6675**

**[NM-940-08-4220-10; NM NM 66022]**

**Withdrawal of Public Land for Protection of Recreational Values Along the Rio Grande; New Mexico**

**AGENCY:** Bureau of Land Management. Interior.

**ACTION:** Public Land Order.

**SUMMARY:** This order withdraws 264.39 acres of public land from surface entry and mining for a period of 20 years for the Bureau of Land Management to protect, preserve, and maintain existing and future recreational values located along the "Pilar" section of the Rio Grande. The lands have been and remain open to mineral leasing.

**EFFECTIVE DATE:** May 6, 1988.

**FOR FURTHER INFORMATION CONTACT:** Clarence Hougland, BLM, New Mexico State Office, P.O. Box 1449, Santa Fe, New Mexico 87504-1449, 505-988-6554.

By virtue of the authority vested in the Secretary of the Interior by section 204 of the Federal Land Policy and Management Act of 1976, 90 Stat. 2751; 43 U.S.C. 1714, it is ordered as follows:

1. Subject to valid existing rights, the following described public lands are hereby withdrawn from settlement, sale, location, or entry under the general land laws, including the United States mining laws (30 U.S.C., Ch. 2), but not from leasing under the mineral leasing laws, to protect two sites important for recreational use on the Rio Grande:

New Mexico Principal Meridian

County Line Site

T. 23 N., R. 10 E.,
Sec. 14, lot 4;
Sec. 15, lot 4.

Fishing Hole Site

T. 24 N., R. 11 E.,
Sec. 32, lots 5, 6, 7, 8, SW¼SE¼.

The areas desc... aggregate 264.39 acres in Taos and Rio Arriba Counties.

2. The withdrawal made by this order does not alter the applicability of those public land laws governing the use of the lands under lease, license, or permit or governing the disposal of their mineral or vegetative resources other than under the mining laws.

3. This withdrawal will expire 20 years from the effective date of this order unless, as a result of a review conducted before the expiration date pursuant to section 204(f) of the Federal Land Policy and Management Act of 1976, 43 U.S.C. 1714(f), the Secretary determines that the withdrawal sh... extended.

May 3, 1988.

James W. Ziglar,

*Assistant Secretary of the Interior.*

[FR Doc. 88-10087 Filed 5-5-88; 8:45 am]

BILLING CODE 4310-FB-M

**FEDERAL EMERGENCY MANAGEMENT AGENCY**

**44 CFR Parts 59, 60, 61, 62, 65, 70, and 72**

**National Flood Insurance Program; Flood Plain Management Standards**

**AGENCY:** Federal Insurance Administration (FIA), Federal Emergency Management Agency (FEMA).

**ACTION:** Final rule.

**SUMMARY:** This final rule revises the National Flood Insurance Program (NFIP) regulations dealing with: flood plain management standards; criteria for the identification of coastal high hazard areas, more commonly referred to as V-zones, and delineated as Zone V, VO, V1-30 or VE on NFIP maps; requirements for maintenance of altered watercourses; criteria under which communities may permit flood plain and floodway developments which could increase base flood elevations; procedures for map correction; reimbursement procedures for the review of proposed projects to determine if they would qualify for NFIP map revisions upon their completion; and changes in the Standard Flood Insurance Policy (SFIP) terms and provisions.

**EFFECTIVE DATE:** October 1, 1988.

**FOR FURTHER INFORMATION CONTACT:** Charles M. Plaxico, Federal Emergency Management Agency, Federal Insurance Administration, 500 C Street SW., Washington, DC 20472; telephone number (202) 646-3422.

**SUPPLEMENTARY INFORMATION:** On November 3, 1987, FEMA published for comment in the Federal Register (Vol. 52, page 42117) a proposed rule containing revisions to the NFIP which were the result of a continuing reappraisal of the NFIP to achieve greater administrative and fiscal effectiveness in the operation of the program and to encourage sound flood plain management so that reductions in loss to life and property and in disaster expenditures can be realized. This reappraisal included the risk assessment (i.e., mapping of flood hazard areas) component of the NFIP, the loss

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reduction (i.e., flood plain management) component and the claims, coverage, rating and sale of insurance component of the NFIP.

In the process of developing this final rule 32 comments were received, logged and analyzed based on the 7 subject areas discussed in the proposed rule supplementary information. The tally of comments included 1 individual, 17 representatives of private companies, 3 associations (including one on behalf of three associations), 4 local governments and 7 State governments. Many of the comments generally concurred with the proposed rule while specifically addressing one or more of its provisions. The comment contents ranged from strong support for, to strong opposition to, one or more of the proposed changes.

The analysis of the comments resulted in language clarification, a minor change to the numbering of provisions and, due to editorial oversight in the proposed rule, inclusion of changes to §§ 60.3(d) and 60.3(e) to incorporate appropriate cross references for consistency. Also, the Standard Flood Insurance Policy duplicate policy provision in the proposed rule was revised to make it more flexible. The comments received are addressed under the subject headings below.

**Community Ordinances**

Two commentators expressed a general concern that the final rules will require corresponding changes in community ordinances. In fact, the final rules are primarily procedural and will not require ordinance revisions at the local level. However, the new provisions at §§ 60.3(c)(13) and 60.3(d)(4), requiring a community to apply to FEMA for a conditional Flood Insurance Rate Map (FIRM) revision prior to permitting development or encroachments within the special flood hazard areas or within the regulatory floodway that would result in an increase in base flood elevations exceeding NFIP's standards, are voluntary and need only be adopted by a community if it wishes to permit such development. FEMA believes these situations should affect relatively few communities.

As for the final rule amending § 60.3(d)(3) which specifies use of hydraulic and hydrologic analyses in connection with a community's review of proposed development in floodways, this requirement is a clarification of the meaning implicit in the current regulations and, therefore, local ordinances do not need to be changed to reflect this clarification.

**Coastal High Hazard Area and Erosion Considerations for Sand Dunes**

Six commentators addressed this proposed rule and all expressed general support for including primary frontal dunes in V-zones and considering dune erosion during the base flood event in order to better reflect coastal areas actually at risk. However, clarification of the intent of the final rule is necessary to resolve apparent misunderstandings of the proposed rule as expressed by several of the commentators.

The final rule definition of coastal high hazard area includes all primary frontal dunes. Therefore, the boundary line of the V-zone, at a minimum, becomes the landward "toe" of the dune. Figure 1 clarifies the evaluation criterion for considering such dunes as effective barriers to base flood storm surges and associated wave action and depicts the inland limit of the dune. This figure also illustrates another provision of the rule that the cross-sectional area of the dune, as measured from the ocean side of the dune crest and above the 100-year stillwater flood level (SWFL), must be at least 540 square feet in order for the dune to be considered effective in attenuating wave action. Under the rule all dunes with a cross-sectional area less than 540 square feet will be treated as completely eroded during the 100-year storm in the preparation of FIRMs. Such a dune will not be credited with offering any protection against flooding or wave action to areas on the landward side of that dune. For dunes with a cross-sectional area smaller than 540 square feet, the methods used to determine areas subject to wave action will be employed as if the dune did not exist. Conversely, any dune with a cross-sectional area greater than 540 square feet may not be totally destroyed during the 100-year event and the dune will be credited with offering some level of protection to areas on its landward side. In this case, the method used to determine areas subject to wave action will be employed in conjunction with a dune erosion model which predicts how much of the dune will be eroded and how much protection it provides.

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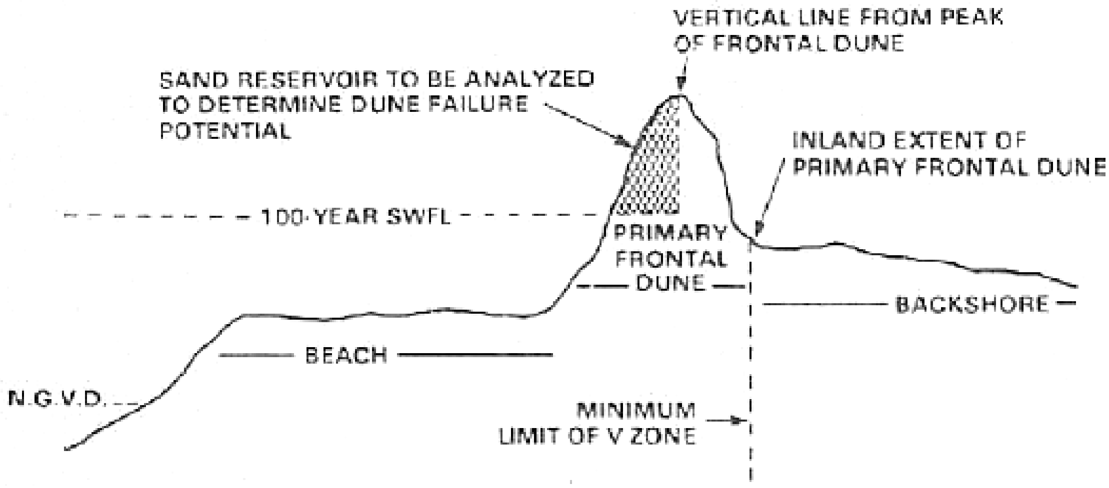
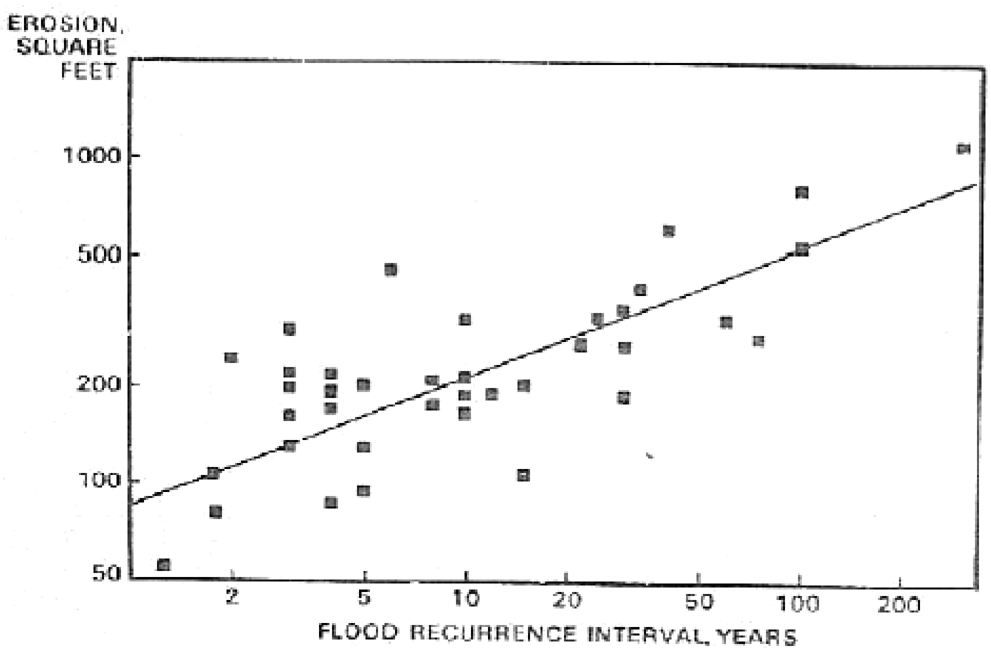



FACTORS TO BE CONSIDERED IN DETERMINING
DUNE FAILURE POTENTIAL AND V ZONE MAPPING

FIGURE 1




MEDIAN CROSS-SECTIONAL EROSION ABOVE FLOOD ELEVATION VERSUS FLOOD RECURRENCE INTERVAL BASED ON 38 CASES OF DUNE RETREAT DURING VARIOUS COASTAL STORMS.

FIGURE 2

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1. One State government and one association commented that by choosing the 50 percentile (median) cross-sectional area of 540 square feet as a criterion, FEMA has, by definition, underestimated dune erosion in half of all base (100-year) flood events (see figure 2). The commentators suggested using the 90–95 percentile in figure 2, which represents a cross-sectional area of 1,000 square feet or more.

The risk analysis methodology decisions incorporated in the NFIP have traditionally been based on selection of a median value, i.e., an estimate that has an equal chance of being too high or too low. Such has been the case in estimation of 100-year flood discharges and base flood elevations. It would be inconsistent to select a criterion for evaluating dune integrity which would overestimate dune erosion more than 50% of the time. Further, a value other than the median value would be inequitable in the determination of actuarial insurance rates.

In addition, dunes that meet the cross-sectional criteria of 540 square feet will not necessarily be considered as providing total protection from the 100-year flood. A post-flood eroded profile of the dune will be considered in computing wave runup on the dune face and overtopping on the dune crest when appropriate.

It is important to note that a criterion of 540 square feet for the cross-sectional area above the storm surge elevation and seaward of the dune crest represents a dune of significant size. FEMA believes use of this area is appropriate because it is consistent with other median values utilized in the NFIP and is based on an empirical relationship between the quantity of sand that would be removed from a frontal dune and the recurrence interval of the local storm tide.

2. Several commentators expressed concern that site-specific conditions should be analyzed in determining potential dune erosion rather than adopting a criterion for general dune integrity. FEMA has assessed the state-of-the-art in erosion modeling approaches and determined that they are not always effective in differentiating between local conditions. Moreover, site specific analyses will be considered in each flood risk analysis performed. Measurements of dunes will be taken locally to determine the volume of sand in the dune and whether the dune meets the 540 square feet criterion. Where the dune does not meet the 540 square feet criterion, site specific conditions will be considered in estimating a post-storm dune profile for wave runup and overtopping computations. Where the dune does meet the criterion, the protection afforded by the post-storm configuration of the dune will be factored into the wave runup calculations.

3. A State commentator suggested that FEMA develop provisions which take into account long-term erosional retreat rates of oceanfront shorelines in its mapping. The issue of using future risk conditions, such as long-term erosion effects, in flood risk analysis and mapping has been previously examined by FEMA. The NFIP legislation has not previously addressed long-term erosion and a consistent data set does not now exist to allow FEMA to uniformly consider long-term erosion in the mapping of flood risks. FEMA has addressed this issue in the past by striving to maintain the accuracy of its maps through periodic map revisions. Additional efforts in erosion risk analysis and management will be forthcoming as FEMA implements the provisions of section 544 of the Housing and Community Development Act of 1987 pertaining to erosion.

4. Several associations and a State agency commented that FEMA should incorporate restrictive building requirements in areas subject to wave action. Language to amend § 60.3(e)(7) was suggested. It was also suggested that FEMA establish regional coastal construction requirements for structural foundations and that FEMA incorporate its Coastal Construction Manual into the final rule. Revision of flood plain management regulations for coastal areas is not being considered by FEMA during FY 1988 rulemaking. Presently, the NFIP regulations for construction in V-zones at § 60.3(e)(4) and § 60.3(a)(5)(ii) do require more rigorous construction standards such as piling or column foundations and anchoring for wind and water forces designed to exceed 100-year values for wind and water loading. With the final rule, these restrictive standards will become more comprehensive in geographic extent by the inclusion of all primary frontal dunes in the V-zone, and the extension of V-zones into areas behind dunes which do not have substantial cross sections.

Further, § 60.3(e)(7) prohibits any man-made alteration of sand dunes in V-zones which would increase potential flood damage. Most development that currently alters frontal dunes occurs on dunes mapped as outside the V-zone. The final rule incorporates all primary frontal dunes into the V-zone and thereby makes § 60.3(e)(7) much more encompassing and effective in protecting the integrity of frontal dune systems. It is envisioned that, as new V-zone delineations are established under the rule, construction activity (e.g., excavation and grading) which would jeopardize the integrity of primary frontal dunes will be prohibited by communities participating in the NFIP.

With regard to coastal foundation construction, while § 60.3(e)(4) does not specifically state that foundation design should incorporate an increase in foundation loadings due to erosion of supporting soil during a base flood event, this requirement is implicit in subparagraph (ii) of this section.

The current regulation is in agreement with the Coastal Construction Manual and, thus, the latter does not need to be incorporated into the rule. However, communities and interested parties are encouraged to utilize the manual for additional design considerations. Further, FEMA recognizes and encourages those local and State governments that wish to adopt more restrictive requirements than FEMA's standards to do so.

5. Another State suggested that FEMA include a definition in the rule for "alterations of sand dunes." FEMA believes the term, "man-made alteration of sand dunes" as used in § 60.3(e)(7), is self-explanatory; therefore, a definition is not necessary.

6. A joint comment submitted by three associations suggested that bluffs, secondary dunes, wetlands and other coastal barriers be included in V-zones. This suggestion is outside the realm of the proposed rule. Further, the definition of coastal high hazard area (V-zone) remains related to areas impacted by significant wave action. Primary frontal dunes are being included in V-zones because they are features that absorb the brunt of the wave action. Areas such as coastal wetlands, secondary dunes, and bluffs are not consistently affected by wave action during flood events and, thus, should not be included in V-zones by definition. Where such features can be shown to be subject to significant wave action in major storms through the engineering analyses performed in a flood risk study, they will be included in V-zones.

7. A State agency suggested that FEMA immediately provide revised FIRM's reflecting the new criterion for erosion consideration for sand dunes to any coastal community requesting such a revision, rather than waiting to incorporate the changes in new flood risk assessment studies or restudies. It is FEMA's intention to eventually revise all FIRM's where the V-zones may be presently underestimated. These revisions must be processed through the

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normal map revision procedures rather than by special restudy effort because of current budgetary constraints. Priorities for performing map updates through the restudies and revisions procedures for communities impacted by the new erosion criteria will be based on cost/benefit considerations, as is presently the case with determining priorities for other types of restudies. A community's need for revised V-zone mapping will be a factor in determining its priority. Where costs can be reduced through local cost-sharing, or other approaches, adjustments in priority will be made.

**Requirements for Maintenance of Altered Watercourses**

Seven commentators addressed this proposed rule including three State governments, two local governments and two associations. Four commentators generally supported the rule but proposed modifications or substitute language; one commentator did not directly address the issue; and two commentators objected to the proposed rule, citing problems with access to private property, cost and legal liability associated with mandatory maintenance.

1. Two local governments and two associations pointed out that not all altered or relocated watercourses will require maintenance of their flood carrying capacity by virtue of their design. They asserted that the design criteria for a watercourse may include factors that account for regrowth of vegetation, sediment deposition, etc., thus obviating the need for maintenance. FEMA agrees that such situations should be addressed in the final rule and a new paragraph (a)(13) has been added to § 65.6.

The new paragraph provides that in lieu of the requirement to submit documentation that the provisions of § 60.3(b)(7) will be met prior to FEMA's revising the NFIP map to reflect the flood hazard mitigation effects of the altered or relocated watercourse, a community may submit certification by a registered professional engineer that the project has been designed to retain its flood carrying capacity without periodic maintenance.

2. A local government and an association suggested that altered or relocated watercourses on open space areas such as golf courses where no existing development will be impacted should be exempted from the maintenance requirement. FEMA's concern in this rule deals only with the maintenance of modifications of watercourses for which flood control benefits have been reflected or are proposed to be reflected on flood maps. This is very rarely, if ever, the case in undeveloped areas. Further, if a map revision is not sought on the basis of watercourse alteration, then FEMA agrees that the data submission requirement regarding watercourse maintenance does not apply and FEMA would not request a community to make such a submission.

3. Comments from a local government and an association suggested that the proposed amendments to § 65.6 be deleted or revised due to problems of access to private property, cost, legal liability and environmental impacts. It is important to note that FEMA is not creating a new requirement for maintenance of altered or relocated watercourses by the final rule. Such a requirement has existed for many years under § 60.3(b)(7). Instead, the final rule merely establishes a procedure whereby FEMA can verify that maintenance, where appropriate, will be carried out for new watercourse alterations for which map revisions are being sought. The final rule enables FEMA to obtain documentation as to the nature of the maintenance activities to be performed, the frequency with which they will be performed, and the title of the local official who will be responsible for assuring that the maintenance activities are accomplished. If it is prohibitive for a community to maintain its altered or relocated watercourses (for whatever reason), FEMA will not credit the flood control benefit of those projects on its maps. It is up to each community to evaluate the benefit of lower BFE's and reduced Special Flood Hazard Areas versus the costs of maintaining its watercourse modifications originally intended for these purposes.

4. A local government, a State agency and two associations addressed the environmental considerations connected with maintenance of altered watercourses. Some commented that implementation policies for maintenance of altered watercourses should be developed in concert with other Federal agencies and that such policies should specify criteria which is environmentally sensitive. While FEMA agrees that there is a need for better coordination among appropriate Federal agencies in regard to environmental issues and the Federal permitting process in connection with the maintenance of altered watercourses, the final rule is not the appropriate mechanism by which to address these issues. Further, it is unlikely that situations will arise where the alteration of watercourses for flood control purposes would be permitted, but maintenance of such alterations would not. Should such situations occur, then FEMA will not revise the map to reflect the mitigating effects of the altered watercourse and, therefore, no maintenance responsibilities will be imposed.

5. A State government and an association suggested that FEMA should require communities to report periodically regarding their maintenance activities and the continued effectiveness of their altered watercourses. FEMA agrees with the concept of monitoring community compliance with this requirement and is considering modification of the Community Assistance Visit Program to include elements relative to the maintenance of altered watercourses. However, establishing a formal reporting requirement would create an unnecessary paperwork burden on NFIP communities and an unnecessary administrative burden on the Agency. Therefore, reporting requirements will not be included in the final rule.

6. The same two commentators suggested that in rapidly urbanizing areas, future development of the watershed upstream from the altered watercourse should be considered in estimating design flows and flood plain limits. They asserted that with any flood control project, there is a potential for the benefits to be partially or completely negated by upstream development which increases runoff and the 100-year flood discharge. FEMA has taken these comments under advisement. A study is presently underway to examine the feasibility of considering future development in flood risk determinations. However, certain administrative issues become immediately apparent. First is the issue of equitable charging of actuarial premium rates for flood insurance coverage when rates are based on future rather than current risk conditions. Secondly, the estimation of future development and its effects on hydrologic conditions adds an additional level of uncertainty on the regulatory data established. This data remains subject to individual rights to appeal, and therefore, must be scientifically, technically and legally defensible. While FEMA is aware of the impact of possible future urbanization on the flood risk, these significant administrative concerns may prove too complex to implement effectively. Nevertheless, communities are encouraged to address the issue of the impacts of upstream future development on the potential benefits of a flood control project and adopt more restrictive local standards as necessary.

# Coastal Structures

## FEMA Coastal Flood Hazard Analysis and Mapping Guidelines Focused Study Report

February 2005


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**Acronyms**

| | |
|---|---|
| BFEs | Base Flood Elevations |
| CEM | Coastal Engineering Manual |
| CFR | Code of Federal Regulations |
| FEMA | Federal Emergency Management Agency |
| FIRM | Flood Insurance Rate Map |
| FIS | Flood Insurance Study |
| NGDC | National Geophysical Data Center |
| PWA | Philip Williams & Associates |
| SFHA | Special Flood Hazard Area |
| SPM | Shore Protection Manual |
| USACE | U.S. Army Corps of Engineers |

# 1 INTRODUCTION

Existing FEMA guidance for treatment of coastal structures refers to seawalls, bulkheads, revetments and coastal levee-type structures, i.e., those that are intended to retain fill and offer protection against flooding and waves, and that are constructed along or parallel to the shoreline. Groins, jetties and detached breakwaters are not mentioned specifically, but should also be considered for flood hazard mapping purposes.

A coastal structure can modify flood levels, wave effects and topography, both landward of, seaward of and adjacent to the structure, and must be considered during the mapping of coastal flood hazards. Two scenarios are commonly encountered:

1. *Existing* coastal structures are analyzed during a Flood Insurance Study, and their effects (if any) must be reflected by the resulting FIRM. This process is described in Appendix D to the *G&S* (FEMA, 2003).

2. *Existing, new or proposed* coastal structures often serve as the basis for revisions to FIRMs, and their stability and effects must be evaluated. The map revision instructions and form MT-2 (FEMA 2002) address this scenario.

## 1.1 CATEGORY AND TOPICS

Seven coastal structures topics were identified at Workshop 1 and are identified below. There were no “Critical” topics identified. Five topics were designated “Available” and two were identified as “Helpful.” Each of these will be considered in this paper.

## 1.2 COASTAL STRUCTURES FOCUSED STUDY GROUP

The Coastal Structures Study Group is made up of Bob Batallio, Ida Brøker, Kevin Coulton, Jeff Gangai, Darryl Hatheway, Jeremy Lowe, Ron Noble, and Chris Jones, who served at Team Leader.

## 1.3 CURRENT FEMA GUIDANCE FOR COASTAL STRUCTURES

FEMA’s existing guidance for coastal structures is limited to the Atlantic, Gulf of Mexico and Great Lakes Coasts, as summarized in the *G&S for Flood Hazard Mapping Partners* (FEMA, 2003). Sections D.2.2.8 and D.2.3 address the Atlantic and Gulf of Mexico; nearly identical sections D.3.2.7 and D.3.3 address the Great Lakes. No coastal structure guidance specific to sheltered shorelines or the Pacific Coast exists in Appendix D, although it is reasonable to expect that existing guidance for other coasts will apply.

| Coastal Structures Topics and Priorities | | | | | |
|---|---|---|---|---|---|
| Topic Number | Topic | Topic Description | Priority | | |
| | | | Atlantic / Gulf Coast | Pacific Coast | Non-Open Coast |
| 25 | Flood Protection Structures | Review *G&S* language – (SC not required to evaluate all structures using 89-15); add new procedure for flood hazard modeling in the presence of coastal structures | A | A | A |
| 21 | Failed Structures | Clarify guidance that when a structure is determined to fail under base flood conditions, the structure is removed, fill/topo remains and is subject to erosion, wave analyses | A | A | A |
| 23 | Buried Structures | Add *G&S* language that buried structures are to be evaluated | A | A | A |
| 27 | Coastal Levees v. Structures | Review *G&S* and regs regarding treatment of coastal levees and structures; identify conflicts; clarify *G&S* that evaluations of all "structures" to be per 89-15 | A | A | A |
| 24 | Structures - Tsunamis | Review 89-15 and other literature for tsunami failure information/guidance | -- | A | I |
| 22 | Failed Structure Configuration | Investigate configuration of failed structures | H | H | H |
| 26 | Adjacent Properties | Review data on (and add to *G&S*) effects of structures on flood hazards on adjacent properties, flooding/waves behind structures via adjacent properties | H | H | H |
| Key: C = critical; A = available; I = important; H = helpful | | | | | |

**Excerpts and major elements of the existing coastal structure guidance are summarized below:**

- "The crucial first consideration in evaluating a coastal structure is whether it was properly designed and has been maintained to provide protection during the 1-percent-annual-chance flood. If it can be expected to survive the 1-percent-annual-chance flood, the structure should figure in all ensuing analyses of wave effects (erosion, runup, and wave height). Otherwise, it should be considered destroyed before the 1-percent-annual-chance flood and removed from subsequent transect representations." (Section D.3.3, paragraph 1).

- Specific criteria for evaluating coastal structures are contained in a memorandum (FEMA 1990), reproduced in Appendix CS-1. The criteria are based in large part on a study performed by the USACE for FEMA (Walton, et al., 1989; also referred to as "TR-89-15"), and cover such topics as:

  - Design parameters (water levels and wave heights; breaking wave forces),
  - Freeboard (above 1% stillwater level, and relative to the runup elevation),

- Toe protection,
- Backfill protection,
- Structural and geotechnical stability (sliding, overturning, settlement, soil slip, ice and impact forces, etc.),
- Materials (strength and durability, including stone size, filter characteristics, expected lifetime, etc.),
- Adverse impacts,
- Maintenance plan, and
- Engineering certification.

Similar criteria are contained in the *Coastal Structures Form* (MT-2, Form 5, reproduced in Appendix CS-2) used to evaluate coastal structures as the basis for FIRM revisions.

In performing analyses for a Flood Insurance Study (FIS) FEMA (2003) directs the mapping contractor (partner) to obtain documentation for each coastal structure possibly providing protection from 1-percent-annual-chance flood. That documentation is to include the following:

- Type and basic layout of structure;
- Dominant site particulars, (e.g., local water depth, structure crest elevation, ice climate);
- Construction materials and present integrity;
- Historical record for structure, including construction date, maintenance plan, responsible party, repairs after storm episodes; and
- Clear indications of effectiveness or ineffectiveness.

Unfortunately, few FIS projects have sufficient funds to support a detailed evaluation of coastal structures, and the *G&S* call for development of "much of this information through office activity, including a careful review of aerial photographs. In some cases of major coastal structures, site inspection would be advisable to confirm preliminary judgments." (Section D.2.2.8, last paragraph).

Cost considerations aside, the *G&S* also recognize that information about existing coastal structures may not be available or sufficient to complete a detailed evaluation. In such cases, the mapping contractor (partner) "shall make an engineering judgment about its likely stability based on a visual inspection of physical conditions and any historical evidence of storm damage and maintenance." (Section D.2.3, second paragraph).

# 2 CRITICAL TOPICS

There were no "Critical" topics identified in Workshop 1.

# 3 AVAILABLE TOPICS

## 3.1 TOPIC 25: ADD GUIDELINES AND SPECIFICATIONS TEXT THAT STATES STUDY CONTRACTORS ARE NOT REQUIRED TO EVALUATE ALL COASTAL STRUCTURES USING THE CRITERIA IN FEMA (1990) AND WALTON, ET AL. (1989); ADD A RECOMMENDED PROCEDURE FOR MAPPING FLOOD HAZARDS AT TRANSECTS WITH COASTAL STRUCTURES

### 3.1.1 Description of Topic and Suggested Improvement

Sections D.2.3 and D. 3.3 of the existing guidance make reference to the FEMA criteria for the evaluation of coastal structures (FEMA, 1990; Walton, et al., 1989), and imply these criteria should be applied by study contractors, unless available information is not sufficient to perform detailed evaluations. The *G&S* should be revised to state clearly that detailed evaluations of all structures are not required of study contractors.

Instead, the following structure evaluation procedure is recommended for inclusion in the *G&S*:

1. The Study Contractor should determine whether available information clearly indicates a coastal structure will fail or survive a base flood event, then perform the subsequent erosion and wave analyses on the indicated (intact or failed structure) profile. In the case of revetment type structures that tend to fail progressively, study contractors should be allowed the discretion to allow for partial – rather than complete – failure (see Topics 21a and 22). It should be clearly communicated to communities and property owners that Study Contractor structure performance determinations are for mapping purposes only, are not intended to substitute for detailed structural evaluations, and should not serve as a basis for Study Contractor liability in the event of structure failure.

2. If available information does not clearly point to survival or failure of a coastal structure, the Study Contractor may either: a) conduct a detailed evaluation using TR-89-15 procedures, or b) perform the erosion and wave analyses for both the intact and failed structure cases, and map the flood hazards associated with the more hazardous case. If option 2.b) is selected, the Study Contractor should clearly document the results of both cases (structure intact, structure failed) and specify which case is used for mapping purposes. Also, see section 5.1.1, Topic 22.

***Implications of not Performing Detailed Coastal Structure Evaluations During the FIS***

Flood study contracts typically do not have sufficient budget to carry out detailed evaluations of coastal structures, and study contractors commonly assume the structures will fail as a default condition (since they have not performed detailed evaluations). There are two important implications of this assumption:

- Failed coastal structures may or may not yield the highest BFEs and greatest flood hazards. See Topic 22 for additional discussion.
- Property owners frequently request (and receive) revisions to FIRMs after retaining engineers who perform detailed evaluations and certify that coastal structures will withstand the 1% flood event. As a result, the revised FIRMs may display highly irregular flood hazard zone boundaries and BFE lines, and may be constantly changing as additional detailed evaluations are performed. See Topic 27 for additional discussion.

### 3.1.2 Availability

Information to address Topic 25 is available and easily incorporated into existing guidance.

## 3.2 TOPIC 21: CLARIFY GUIDANCE REGARDING TREATMENT OF BACKFILL/TOPOGRAPHY WHEN A STRUCTURE IS DETERMINED TO FAIL UNDER BASE FLOOD CONDITIONS, AND IS REMOVED FROM THE TRANSECT

### 3.2.1 Description of Topic and Suggested Improvement

Existing guidance calls for the removal of a coastal structure (from analysis transects) when it has been determined that the structure will not withstand the 1% event (see Section D.2.3, first paragraph; Section D.3.3, first paragraph).

However, no details are provided as to how such a removal should be accomplished for those types of structures contemplated by the *G&S* (seawalls, bulkheads, revetments, levees), and no details are provided regarding other types of coastal structures whose failure during a base flood event could affect coastal flood hazards (e.g., groins, jetties, detached breakwaters).

Dealing with the former issue will be straightforward, but dealing with the latter will not. Guidance on how to predict the failure of groins and jetties – which usually fail by loss of profile (through settlement or displacement) and/or by becoming detached at their landward ends – is not readily available. Likewise, guidance on how to predict the failure of detached breakwaters (usually through loss of profile) is not readily available.

The recommended approach can be divided into two components:

- <u>Topic 21a</u>. For *seawalls, bulkheads, revetments and coastal levees*: remove the failed structure (or estimate a partial collapse of revetment structures, where appropriate) and alter the remaining soil to achieve its likely slope immediately after structure failure (note that this is not necessarily the same as the long-term stable slope in the case of bluffs and cliffs). This slope will then be subjected to an event-based erosion analysis and wave height and runup analyses.

Topic 21b. For *groins, jetties and detached breakwaters*: evaluate the overall condition and performance of the structures over time; determine whether the structures (or similar structures nearby) have been damaged or detached during prior major storms; document the structural damage and any resulting shoreline recession attributable to the structural damage; use this information to predict the likely shoreline configuration (in plan view) if the structures fail during the base flood. The altered shoreline will then be subjected to an event-based erosion analysis and wave height and runup analyses. Note that in the case of groins and jetties, it is unlikely that their failure will require "removal" from analysis transects (removal of a detached breakwater from a transect is more likely to occur). The effects of the structures on the shoreline configuration, however, will be removed.

### 3.2.2 Availability

Information to address Topics 21a is available and easily incorporated into existing guidance.

Existing guidance can be modified to mention Topic 21b, but detailed guidance is not readily available. Developing detailed guidance could require site-specific studies using analytical or numerical methods.

Therefore, it is recommended that guidance be expanded to discuss removal of seawalls, bulkheads, revetments, coastal levees and that guidance allow for partial failure of revetments, where appropriate. Mention in guidance removal of the effects of groins, jetties, detached breakwaters on the shoreline. Develop specific guidance on how to remove the effects of groins, jetties, and detached breakwaters on the shoreline.

## 3.3 TOPIC 23: ADD GUIDELINES AND SPECIFICATIONS TEXT THAT BURIED STRUCTURES ARE TO BE EVALUATED

### 3.3.1 Description of Topic and Suggested Improvement

Existing guidance is vague regarding those coastal structures that should be evaluated for their durability during the 1% flood event. The guidance is clear that exposed structures must be evaluated, but does not mention coastal structures that are known to exist, but are buried. The recommended approach is simple:

Modify the *G&S* text to state that study contractors should:

1. Inquire as to whether buried coastal structures exist within their study area,

2. Mention the apparent presence or absence of buried coastal structures in the study documentation,

3. Apply evaluation techniques to buried coastal structures that are similar to those applied to exposed coastal structures.

4. Add examples to the *G&S*.

### 3.3.2 Availability

Information to address Topic 23 is available and easily incorporated into existing guidance.

## 3.4 TOPIC 27: REVIEW GUIDELINES AND SPECIFICATIONS AND NFIP REGULATIONS REGARDING TREATMENT OF COASTAL LEVEES AND STRUCTURES; IDENTIFY CONFLICTS; REVIEW AND UPDATE TR-89-15 STRUCTURE EVALUATION CRITERIA; CONSIDER REQUIRING ALL COASTAL STRUCTURES (EXISTING AND NEW) TO MEET THE SAME EVALUATION CRITERIA

### 3.4.1 Description of Topic and Suggested Improvement

There are potential inconsistencies in the treatment of coastal levees and other coastal flood protection structures, and in the evaluation of coastal structures. The issues are as follows:

- Topic 27a – incomplete explanation of the differences between coastal levees and other coastal structures, and how the designation affects their treatment in flood hazard mapping;
- Topic 27b – the evaluation criteria in Walton et al. (1989) should be reviewed in light of the methods contained in the *Coastal Engineering Manual* (USACE, 2002); and
- Topic 27c – existing coastal flood protection (non-levee) structures can be incorporated into a coastal flood study based on engineering judgment, without meeting the same engineering and certification requirements for new or proposed structures; consider requiring all structures to meet the same criteria; maintenance plan criteria for private structures are problematic.

***Topic 27a: Coastal Levees vs. Other Coastal Structures***

There are two general classes of coastal structures that can provide some degree of protection against coastal flooding: coastal levees and other coastal structures.

> *Coastal levees* are structures that are designed to provide low-lying coastal areas with total protection during the 1% flood. In other words, the coastal levee must be substantial enough to prevent any flooding or wave overtopping landward of the levee crest. NFIP regulations (44CFR part 65.10; reproduced in Appendix CS-3) spell out the requirements

a coastal levee must meet to be credited as providing complete protection from flooding, including a freeboard requirement specific to coastal levees – the crest elevation of the levee must be elevated at least two feet above the 1% stillwater elevation, and above the elevation of the 1% wave height or the maximum wave runup elevation (whichever is greater).

*Other coastal structures (seawalls, bulkheads, revetments)* can be recognized on flood hazard maps as long as they remain intact during the 1% flood, even if they are overtopped. They can provide limited protection against flooding and waves, yet still be considered for flood hazard mapping purposes. These types of structures are often used by property owners to reduce flood hazards and to revise flood hazard zones on the FIRM (i.e., to change V zones to A zones or X zones).

The *G&S* do not explain the differences between coastal levees and other coastal structures, do not discuss the different design and certification requirements, and do not discuss how the designation affects their treatment for flood hazard mapping purposes.

On a related matter, one source of much discussion has been the maintenance plan criteria in FEMA (1990) and 44CFR65.10. The maintenance plan requirements in the regulations only apply to coastal levees, but in FEMA (1990), the same criteria apply to all coastal structures. This has been problematic since the criteria only allow certification of levees/structures where a maintenance plan has been adopted by and maintenance activities are the responsibility of a federal, state or community agency. Private structures will not be able to meet this requirement. As a practical matter, however, government agencies can require private owners to maintain their coastal structures. This effectively satisfies the intent of the maintenance plan requirement.

### *Topic 27b: Update to Coastal Structure Evaluation Criteria*

FEMA coastal structure evaluation criteria (adopted in 1990) are based on a USACE report (Walton, et al., 1989). The report also forms the basis for the evaluation criteria contained in the *G&S*, in 44CFR 65.10, and in the flood map revision form for coastal structures (*MT-2, Form 5*).

The criteria should be reviewed in light of more recent guidance and methods contained in the USACE's *Coastal Engineering Manual* (CEM).

### *Topic 27c: Consistency of Coastal Structure Design/Certification in Flood Studies and Map Revisions, Including Maintenance Plan Criteria*

Existing non-levee coastal flood protection structures can be incorporated into a coastal flood insurance study or restudy, without meeting all the specific requirements that new structures are expected to meet to justify a map revision.

The study contractor documentation specified in Section D.2.2.8 can serve as the basis for the evaluation of existing coastal structures. The documentation includes:

- Type and basic layout of structure;
- Dominant site particulars, (e.g., local water depth, structure crest elevation, ice climate);
- Construction materials and present integrity;
- Historical record for structure, including construction date, maintenance plan, responsible party, repairs after storm episodes; and
- Clear indications of effectiveness/ineffectiveness.

Given the fact that the *G&S* allow the Study Contractor to develop much of this documentation through an office review of available data, *engineering judgment* using the above factors can determine whether an existing coastal flood protection structure is incorporated into the coastal hazard assessment, and whether it influences BFEs and flood hazard zones.

In contrast, a new coastal flood protection structure is required to be certified with all supporting calculations and technical documentation specified in FEMA (1990) and Walton et al. (1989), including the maintenance plan requirement.

It would appear – for consistency purposes – that a similar level of engineering and certification should be required of both existing and new/proposed structures. It is recommended that consistent engineering and certification requirements be used for existing and new/proposed structures, with an exception for the maintenance plan criteria for private structures (which are not adopted by government agencies; such agencies will not be responsible for maintenance). Maintenance for private structures should be the responsibility of private owners and enforced through deed restrictions instituted at the time of the FIS or map revision.

Note that these recommendations will not only require a revision to the existing guidance in the *G&S,* they will require a significant increase in the level of effort (and cost) required for flood insurance studies, and will require a revision to FEMA's (1990) adopted criteria for privately owned coastal structures. Making such changes is more than a technical issue, and will require FEMA policy change.

### 3.4.2 Availability

Information to address Topic 27a is available and incorporated into existing guidance; however, inconsistencies will have to be resolved by FEMA.

Information on Topic 27b is available in the CEM and changes to evaluation criteria can be proposed based on this information.

Information related to Topic 27c is available; however, changes to require consistent engineering and certification requirements will necessitate FEMA policy changes and could have significant time and cost consequences.

## 3.5 TOPIC 24: REVIEW WALTON, ET AL. (1989) AND OTHER LITERATURE FOR DAMAGE TO COASTAL STRUCTURES DURING TSUNAMIS

### 3.5.1 Description of Topic and Suggested Improvement

Sections D.2.3 and D. 3.3 of the existing guidance do not reference evaluation criteria that may be appropriate for coastal structures in tsunami-prone areas. While the existing guidance may be pertinent for non-bore type tsunamis, it will probably not be adequate for bore-type tsunamis.

A review of the literature should be undertaken to document tsunami damage to coastal structures. Camfield (1980) summarizes the state-of-the-art as of two decades ago, and should be included in the review.

More recent reports and information sources should also be reviewed. For example:

- National Tsunami Hazard Mitigation Program: http://www.pmel.noaa.gov/tsunami-hazard/index.htm;
- Tsunami data at the National Geophysical Data Center (NGDC): http://www.ngdc.noaa.gov/seg/hazard/tsu.html;
- NOAA Tsunami Research Program: http://www.pmel.noaa.gov/tsunami/;
- International Journal of the Tsunami Society, *Science of Tsunami Hazards* (available at http://epubs.lanl.gov/tsunami/ (see the article by J.F. Landers, L.S. Whiteside and P.A. Lockridge, *Two decades of Global Tsunamis – 1982-2002*, in Vol. 21, No. 1, 2003);
- The Tsunami Research Group at the University of Southern California is dedicated to the investigation of tsunamis and some information may be found from their works: http://www.usc.edu/dept/tsunamis/;
- Mitigation of local tsunami effects project: http://engr.smu.edu/waves/index.html;
- Professor Philip L-F Liu at Cornell University is devoted to studies of the causes and effects of tsunami, and some information may be found in his publications: http://www.cee.cornell.edu/index.cfm; and
- The O.H. Hinsdale Wave Research Laboratory at Oregon State University is designated by the National Science Foundation as a site for tsunami research. This tsunami model basin is presently the largest one in the world for analyzing the impacts of tsunami waves: http://wave.oregonstate.edu/.

The *G&S* should be revised to incorporate revised coastal structure evaluation criteria for areas subject to bore-type tsunamis.

### 3.5.2 Availability

Information to address Topic 24 is available. This effort should be coordinated with the Tsunami Study Group.

# 4 IMPORTANT TOPICS

There were no "Important" topics identified in Workshop 1.

# 5 ADDITIONAL OBSERVATIONS - HELPFUL TOPICS

## 5.1 TOPIC 22: INVESTIGATE CONFIGURATIONS OF FAILED COASTAL STRUCTURES

### 5.1.1 Description of Topic and Suggested Improvement

The discussion in Section 3.1 summarizes the current *G&S* treatment of failed coastal structures, namely, they are to be removed from the analysis transects. However, in the case of seawalls, revetments and similar structures, outright removal may not result in the highest BFEs and flood conditions. Moreover, in the case of revetments, partial failure rather than complete failure (and removal) may be a more appropriate scenario for analysis due to the creation of higher runup condition or greater depths of ponding.

A proposed procedure for handling this situation was developed during the Whatcom County, WA, FIS (PWA, 2002). A modified PWA procedure is recommended for incorporation into the *G&S* as follows:

- In the absence of structure certification, conduct coastal flood analysis for intact and failed conditions, and use the worst case for flood mapping; note that maintaining the results of both analyses may be useful in the event that map revisions are requested in the future based on intact structures;

- Apply simple geometric approaches to estimate the failed condition for vertical or near-vertical rigid structures:

  - Estimate toe scour based on the Shore Protection Manual (SPM) or similar approximations (scour to the water depth at the structure toe, based on the largest unbroken wave anticipated at the toe);

  - Extend the toe erosion offshore a distance related to the incident wave length;

  - Presume the rigid structure breaks apart, into a rough, porous failed slope at 1.5:1. The slope is selected with the understanding that runup typically reaches a maximum at about this slope, which is also consistent with the potential angle of repose of rough angular material; and

- Note that assuming a failed slope of 1.5:1 may lead to undermining of buildings situated very close to the coastal structure. This scenario should be investigated during Phase 2 to determine the appropriate mapping course of action.

- In the case of revetments, consider whether complete or partial failure is more likely during the base flood, and model the selected failed condition. If the failure condition is uncertain, modeling of total and partial revetment failure can be carried out.

In the case of the Sandy Point FIS, application of the above procedure indicated the failed structure condition *typically* did not yield the highest runup elevation, but could result in greater overtopping rates than the intact structure condition.

Parts V-3 (Basco, 2003) and VI-5 (Burcharth and Hughes, 2003) of the CEM (and other documents – see Section 5.2.1) should be reviewed for possible guidance regarding the configurations of failed structures. However, it is proposed that the PWA method be considered an interim method (for seawalls, bulkheads and revetment type structures) and evaluated for future refinement.

Methods for handling failed groins, jetties and breakwaters have not been proposed here, but may be considered for future enhancements of the *G&S* – see Topic 21b.

### 5.1.2 Availability

Information to address Topic 22 is available. This effort should be coordinated with the Runup/Overtopping Study Group.

## 5.2 TOPIC 26: REVIEW DATA ON THE EFFECTS OF COASTAL STRUCTURES ON FLOOD HAZARDS ON ADJACENT PROPERTIES; REVIEW FLOODING/WAVE EFFECTS BEHIND STRUCTURES

### 5.2.1 Description of Topic and Suggested Improvement

One of the coastal structure evaluation considerations included in FEMA (1990), FEMA (2002) and FEMA (2003) is *adverse impacts*. Unfortunately, the level of guidance contained in those documents is *inadequate*:

- FEMA's (1990) memorandum regarding the evaluation of coastal structures states: "All requests for flood map revisions based upon new or enlarged coastal flood control structures shall include an analysis of potential adverse impacts of the structure on flooding and erosion within, and adjacent, to the protected area.";

- FEMA's (2002) flood map revision coastal structures form asks flood map revision requestors, "... will the structure impact flooding and erosion for areas adjacent to the structure? If yes, attach an explanation."; and

- FEMA's (2003) *G&S*, section D.2.3, states, "... a structure might decrease flood hazards in one area while increasing flood and erosion effects at adjacent sites."

### *Impact of Coastal Structures (Seawalls, Revetments) on Adjacent Property*

Impacts can be divided into erosion impacts and hydraulic impacts. Erosion impacts will include the short- or long-term effects of a coastal structure on the topography of adjacent property. Hydraulic impacts will include such things as wave reflection, concentration of flow, etc.

Fortunately, the literature contains numerous papers and studies related to erosion impacts:

- Dean (1987) assessed commonly expressed concerns about seawall impacts. The assessment is summarized in Figure 1.

- Fulton-Bennett and Griggs (1986) document case histories of 32 shore protection structures at sites between San Francisco and Carmel, CA. The report concluded that few of the structures survived the long-term test of time without some damage to the structure or the upland areas. Maintenance costs of the structures were much higher than originally anticipated.

- Griggs, et al. (1994) summarized the results of field monitoring at sites in Monterey Bay, CA. They concluded after seven years of detailed monitoring that there was "an absence of measurable or significant differences" between the seawall backed beach and the natural beach.

- Kraus and Pilkey (1988), and Kraus and McDougal (1996) present detailed literature reviews concerning the effects of seawalls on beaches. Both papers were published in the *Journal of Coastal Research*, the first being in a special issue devoted to the topic (Kraus and Pilkey, 1988).

- McDougal et al. (1987) conducted laboratory and field investigations in Oregon to assess the impacts of shore protection structures on adjacent unprotected properties. The studies found the "excess erosion" on adjacent properties was consistent with the findings of Chiu (1977): the depth of excess erosion was found to be equal to approximately 10% of the seawall length (see Figure 2).

Taken as a whole, these studies indicate the erosion effects of shore protection structures on nearby properties will vary, depending on the local coastal processes and morphology, sediment budget, and structure location/characteristics. However, the effects can be divided into three general categories:

The effects of impoundment (sediment landward of the structure being prevented from eroding and nourishing the beach) and passive erosion (continuation of ongoing shoreline recession, resulting in a narrower beach in front of a structure) are relatively uncontroversial and can be quantified for a site.

EM 1110-2-1100 (Part V)
31 Jul 2003

**Table V-3-3**
**Assessment of Commonly Expressed Concerns Related to Coastal Armoring (Dean 1987)**

| No. | Concern | Assessment | |
|---|---|---|---|
| 1 | Coastal armoring placed in an area of existing erosional stress causes increased erosional stress on the beaches adjacent to the armoring. | True | By preventing the upland from eroding, the beaches adjacent to the armoring share a greater portion of the same total erosional stress. |
| 2 | Coastal armoring placed in an area of existing erosional stress will cause the beaches fronting the armoring to diminish. | True | Coastal armoring is designed to protect the upland, but does not prevent erosion of the beach profile waterward of the armoring. Thus, an eroding beach will continue to erode. If the armoring had not been placed, the width of the beach would have remained approximately the same, but with increasing time, would have been located progressively landward (see 2b). |
| 2a | Beaches on eroding coastlines will diminish in front of fixed dune positions. | True | An eroding beach continues to erode relative to a fixed dune position. The width of the beach must diminish if the shoreline is eroding (Figure 1). |
| 2b | Natural beaches on retreating barriers maintain the same beach width. | True | Relative to a retreating duneline, a shoreline eroding at the same rate results in a stable beach width. |
| 3 | Coastal armoring causes an acceleration of beach erosion seaward of the armoring. | Probably False | No known data or physical arguments support this concern. |
| 4 | An isolated coastal armoring can accelerate downdrift erosion. | True | If an isolated structure is armored on an eroding beach, the structure will eventually protude into the active beach zone and will act to some degree as a groin, interrupting longshore sediment transport and thereby causing downdrift erosion. |
| 5 | Coastal armoring results in a greatly delayed poststorm recovery. | Probably False | No known data or physical arguments support this concern. |
| 6 | Coastal armoring causes the beach profile to steepen dramatically. | Probably False | No known data or physical arguments support this concern. |
| 6a | Coastal armoring destroys foreshore bar and trough features. | Probably False | No known data or physical arguments support this concern. |
| 7 | Coastal armoring placed well-back from a stable beach is detrimental to the beach and serves no useful purpose. | False | In order to have any substantial effects to the beaches, the armoring must be acted upon by the waves and beaches. Moreover, armoring set well-back from the normally active shore zone can provide "insurance" for upland structures against severe storms. |
| 8 | Seawalls increase the longshore sediment transport. | Unknown | No known data exists, physical arguments can support or discredit this concern. Needs research. |
| 9 | Seawalls cause sand transport a far distance offshore. | Probably False | No known data or physical arguments support this concern. |
| 10 | Other | | |

**Figure 1. Review of concerns related to coastal armoring (Dean, 1987, as compiled by USACE, 2003).**

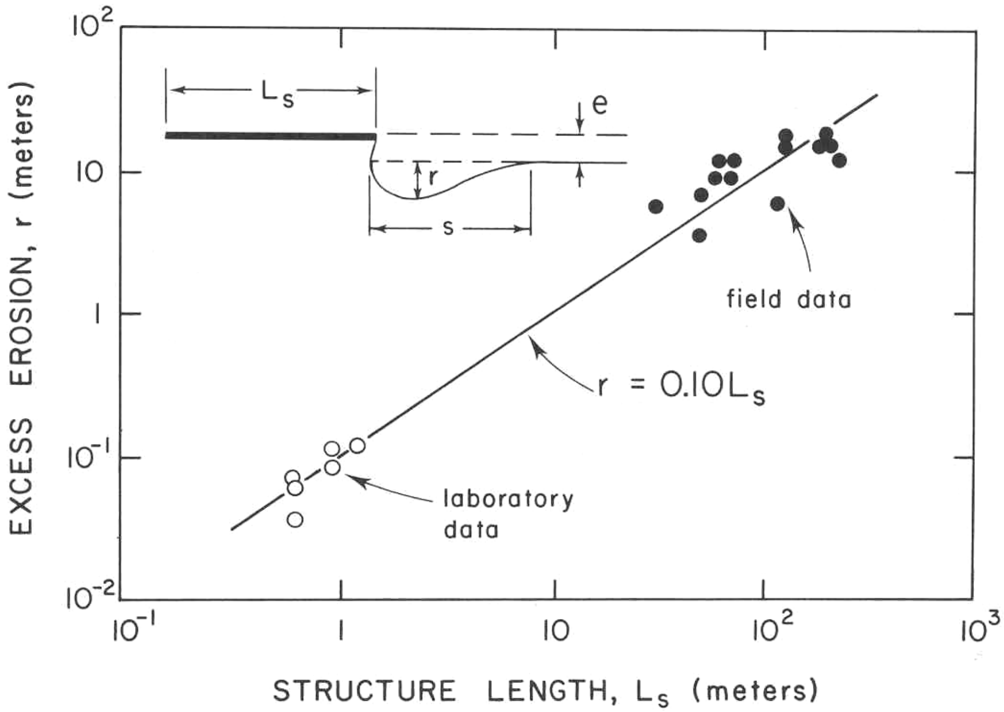



**Figure 2. Excess erosion caused by seawalls (McDougal, et al., 1987).**

The effects of active erosion (postulated erosion and scour due to the presence of the structure) remain the subject of dispute and are more difficult to quantify. The previously mentioned work of Dean (1987), McDougal et al. (1987) and Kraus (1988, 1996) should serve as guidance for evaluating active erosion effects.

Thus, this Focused Study concludes that the approximate or expected erosion effects of coastal structures can be determined for flood hazard mapping purposes. Guidance can be developed for study contractors to use in their evaluations.

Looking forward, the more difficult issue will be how to incorporate this knowledge into FEMA policy regarding treatment of coastal structures:

- If adverse effects of existing coastal structures are documented or of new/proposed structures are predicted, should mitigation be required? If so, in what form?
- Should unmitigated effects be considered in flood hazard mapping (and is this getting into the future conditions area)? Should mitigation efforts be credited in flood hazard mapping (this is similar to the issue surrounding credit for beach nourishment)?
- Should map revisions be permitted based on structures that are predicted or known to cause adverse effects on adjacent properties?

This topic will undoubtedly be the subject of additional debate, and the work described in Table 1 is intended to provide limited technical guidance until the policy issues are resolved.

The wealth of literature devoted to erosion effects of coastal structures does not exist for hydraulic effects. However, the hydraulic effects of many coastal structures can be approximated using the methods of hydraulics, fluid mechanics and wave mechanics, coupled with documents such as the *Coastal Engineering Manual*. There may be some instances where the hydraulic effects of large structures can be better addressed via numerical modeling, but this is expected to be the exception rather than the rule (at least for the near future). For the present, it is recommended that a general discussion of hydraulic effects be included in the *G&S*.

***Flooding and Erosion Behind Coastal Structures (Seawalls, Revetments, etc.)***

A second issue of importance to FEMA is whether the dimensions of a coastal structure are sufficient to prevent flooding and erosion from occurring landward of the structure during the 1% flood event. This issue will be important for both, flood insurance studies and the evaluation of flood map revisions based on coastal structures.

Flooding behind a structure can be caused by overtopping of the shore-parallel section of the structure, or due to overtopping of the shore-perpendicular (return wall) section of the structure.

Erosion behind a structure can be caused by undermining at the structure toe, overtopping, or other structural failures. The erosion can be initiated at or across the shore-parallel or shore-perpendicular sections.

The *G&S* can be expanded to address these hazards, by stating that the TR-89-15-type analyses shall consider both the shore parallel and shore-perpendicular sections of coastal structures.

For the mapping of flood hazard zones landward of structures determined to withstand the 1% flood event, the following procedure is recommended.

<u>Case 1, isolated structure with return walls</u>:

- Evaluate the shore-parallel and shore-perpendicular portions of the structure;
    - if the returns are too short or will not withstand the 1% event, remove the entire structure from the transect prior to further flood analyses (unless the structure is very long compared to the parcel frontage being evaluated), and
    - if the return walls are adequate, determine the mean overtopping rate across the shore-parallel section of the structure.
- Map the resulting BFEs and flood hazard zone boundaries behind and parallel to both the shore-parallel section and any shore-perpendicular sections. This procedure assumes overtopping can occur over any section of the structure. See Figure 3; and

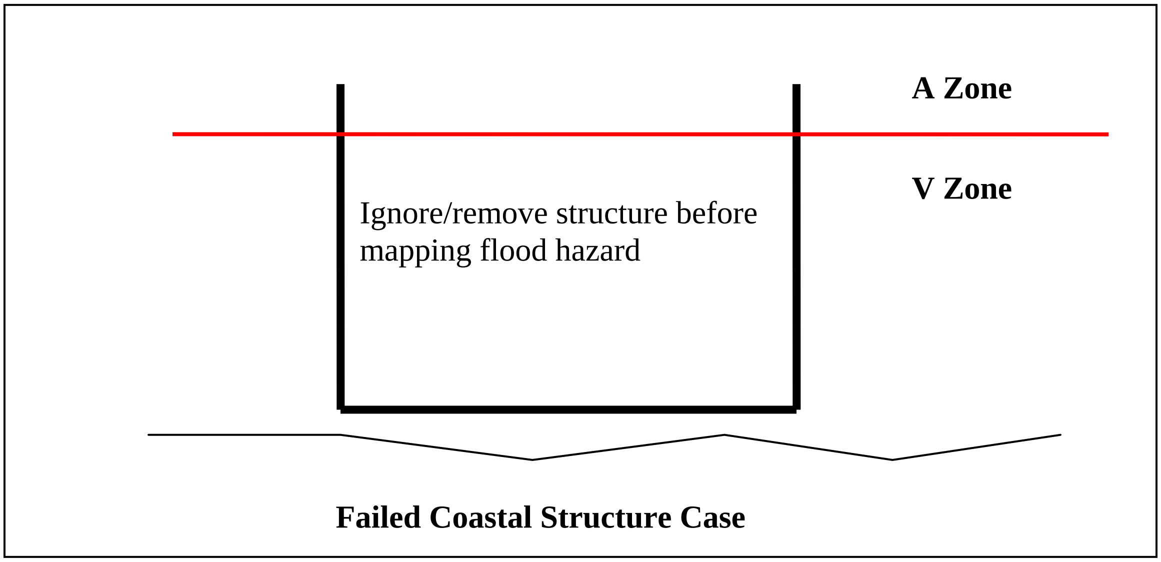
- Calculate the maximum overtopping and determine if any ponding or drainage problems will exist behind the structure; adjust the mapped flood hazard zones and heights/elevations to reflect the ponding or drainage problems.

Case 2, series of structures:

This case will be encountered by Study Contractors, and will likely occur when a one property owner requests a map revision based on a portion of a single structure or one of a series of structures;

- Consider each distinct structure separately – determine whether the land behind the structure is separated from adjacent lands by return walls;
    - if yes, evaluate as in case 1 above, unless the adjacent shore-parallel sections are long and will withstand the 1% flood event (in which case the return wall analysis and mapping are not required); and
    - if no, evaluate the adjacent shore-parallel sections for their stability during the 1% event.
        - if adjacent sections will not withstand the 1% event, the subject coastal structure may be damaged or destroyed as the adjacent structures fail (and may need to be removed prior to any flood analyses); and
        - if adjacent shore-parallel sections will withstand the 1% event, and if they are sufficiently long to preclude flanking behind the subject structure, continue as described below.
- If the analysis goes forward, determine the mean overtopping rate across the shore-parallel section of the structure; and
- Map the resulting BFEs and flood hazard zone boundaries behind and parallel to both the shore-parallel section and any shore-perpendicular sections. This procedure assumes overtopping can occur over any section of the structure. Check for ponding and drainage problems.

Adjust the zones and BFEs along the boundaries with adjacent parcels, as dictated by the stability of adjacent coastal structures.



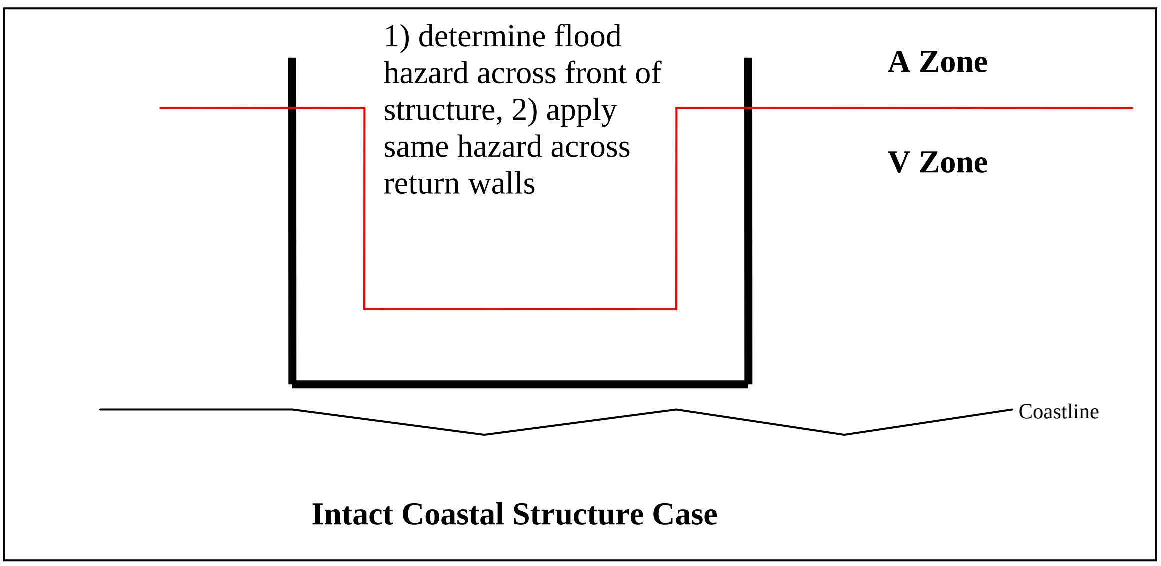




**Figure 3. Sample mapping of flood hazards at failed coastal structure – through physical failure or insufficient return walls – and at intact coastal structure (actual flood hazard zones and BFEs will vary with site/structure conditions).**

Note that the above procedures do not establish a minimum coastal structure length required to gain flood hazard mapping credit (either during an FIS or a map revision). However, as a first approximation, a structure length less than twice the mapped overtopping zone width behind the structure (see *G&S* Table D-7) would probably not provide significant flood hazard reduction for the area landward of the structure. For a more rigorous analysis, the minimum length required will depend upon:

- whether the structure is intended to remove an area from the SFHA or merely to reduce the flood severity/BFE/zone,
- the height of the structure and its associated base flood overtopping rate,
- whether the structure is isolated or part of a longer structure, and
- whether the subject parcel is isolated by return walls that can withstand the base flood event.

Minimum structure lengths might be developed through analyses of selected structures and flood conditions, but this should be considered for future enhancements to the *G&S*.

Recommendations and availability are summarized in Table 1.

### 5.2.2 Availability

Information to address Topics 26a, 26b, and 26d is readily available. Information to address Topic 26e can be gathered and used, but may require greater effort. Addressing Topic 26c requires as much policy development as technical work. Therefore, Workshop 2 deleted Topics 26c and 26e from further consideration during the present project.

# 6 SUMMARY

**Table 1. Summary of Findings and Recommendations for Coastal Structures**

| Topic Number | Topic | Coastal Area | Priority Class | Availability/ Adequacy | Recommended Approach | Related Topics |
|---|---|---|---|---|---|---|
| 25 | Flood Protection Structures | AC | A | Y | Mention in guidance: detailed TR-89-15 evaluation/certification of coastal structures are not required during FIS, but discuss implications (see Topic 22) | 22, 26, 27 |
| | | GC | A | Y | | |
| | | PC | A | Y | | |
| | | SW | A | Y | | |
| 21 | Failed Structures | AC | A | Y | Expand guidance to discuss removal of seawalls, bulkheads, revetments, coastal levees; allow | 13, 22 |
| | | GC | A | Y | | |
| | | PC | A | Y | | |

**Table 1. Summary of Findings and Recommendations for Coastal Structures**

| Topic Number | Topic | Coastal Area | Priority Class | Availability/ Adequacy | Recommended Approach | Related Topics |
|---|---|---|---|---|---|---|
| | | SW | A | Y | for partial failure of revetments, where appropriate. Mention in guidance, removal of the effects of groins, jetties, detached breakwaters on the shoreline. Develop specific guidance on how to remove the effects of groins, jetties, detached breakwaters on the shoreline. | |
| 23 | Buried Structures | AC | A | Y | Mention in guidance: buried structures may exist, should be located and should be considered in analyses. | 22 |
| | | GC | A | Y | | |
| | | PC | A | Y | | |
| | | SW | A | Y | | |
| 27 | Coastal Levees v. Structures | AC | A | Y | Revise Appendix D to differentiate coastal levee requirement from those for other costal flood protection structures; identify conflicts. Review CEM for new or additional guidance on evaluation of coastal structures; Consider requiring all structures (existing and new) to meet the same evaluation criteria. | 11, 25 |
| | | GC | A | Y | | |
| | | PC | A | Y | | |
| | | SW | A | Y | | |
| 24 | Structures - Tsunamis | AC | -- | -- | Review literature and revise guidance for coastal structure evaluation criteria in tsunami-prone areas. | 22 |
| | | GC | -- | -- | | |
| | | PC | I | PRODAT | | |
| | | SW | I | PRODAT | | |
| 22 | Failed Structures | AC | H | Y | Review literature for treatment of failed structures; Revise coastal structure evaluation guidance to reflect PWA Interim method and literature review. | 21, 24 |
| | | GC | H | Y | | |
| | | PC | H | Y | | |
| | | SW | H | Y | | |
| 26 | Adjacent Properties | AC | H | Y | Review literature and develop guidance for evaluating the erosion effects of coastal structures on adjacent properties. Review literature and develop guidance for evaluating the hydraulic effects of coastal structures on adjacent properties. Develop guidance for evaluating flooding and erosion from adjacent properties. | 11, 22 |
| | | GC | H | Y | | |
| | | PC | H | Y | | |
| | | SW | H | Y | | |

**Table 1. Summary of Findings and Recommendations for Coastal Structures**

| Topic Number | Topic | Coastal Area | Priority Class | Availability/ Adequacy | Recommended Approach | Related Topics |
|---|---|---|---|---|---|---|

Key:
Coastal Area
AC = Atlantic Coast; GC = Gulf Coast; PC = Pacific Coast; SW = Sheltered Waters
Priority Class
C = critical; A = available; I = important; H = helpful
(Recommend priority italicized if focused study recommended a change in priority class)
Availability/Adequacy
"Critical" Items: MIN = needed revisions are relatively minor; MAJ = needed revisions are major
"Available" Items: Y = availability confirmed; N = data or methods are not readily available
"Important" Items: PRO = procedures or methods must be developed; DAT = new data are required; PRODAT = both new procedures and data are required

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# COASTAL STRUCTURES FOCUSED STUDY

# APPENDIX A-1

## FEMA 1990 MEMORANDUM:

## CRITERIA FOR EVALUATING COASTAL FLOOD PROTECTION STRUCTURES

Federal Emergency Management Agency
Washington, D.C. 20472

APR 23 1990

MEMORANDUM FOR: FEMA REGIONAL DIRECTORS

FROM: Harold T. Duryee, Administrator
Federal Insurance Administration

SUBJECT: Criteria for Evaluating Coastal Flood Protection Structures for National Flood Insurance Program (NFIP) Purposes

In order to better guide our staff, study contractors, and technical evaluation contractors, in the performance of flood insurance studies and in the review of flood map revision requests based on coastal structures, the Federal Insurance Administration has developed the attached proposed criteria statement. The proposed criteria would establish the conditions, procedures, and standards under which coastal flood protection structures would be credited on NFIP Flood Insurance Rate Maps as providing protection from the base flood.

It is our intention to issue these criteria as rulemaking during FY 1991. Any coments you have should be forwarded to the Office of Risk Assessment by May 25, 1990.

Also, attached is a copy of the Corps' Technical Report CERC-89-15, "Criteria for Evaluating Coastal Flood-Protection Structures" for your reference. CERC-89-15 was used as the basis for this proposed interim procedure.

Attachments

## Criteria for Evaluating Coastal Flood Protection Structures

### Background

Many property owners and communities along the U.S. coast are resorting to the construction of coastal flood control structures to protect existing or new development from potential damage associated with hurricanes and other major coastal storm events. Flooding and erosion caused by natural processes, sea level rise, and/or man-made influences are factors contributing to the decision to construct structures such as seawalls, revetments, bulkheads, and coastal levees/dikes. Although there is continued debate on the overall impact of these coastal structures, their construction and use requires that FEMA evaluate their effectiveness for reducing flood risk and their viability as an alternative to the non-structural flood loss reduction approaches required for community participation in the National Flood Insurance Program (NFIP).

The areas protected by coastal flood protection structures are frequently designated as Coastal High Hazard Areas (V zones) on the Flood Insurance Rate Maps (FIRMs) published by FEMA. FEMA is often requested to revise FIRMs to reflect the protection provided by a coastal structure against the base (100-year) flood. Because of the different types of coastal structures, materials, and construction methods, FEMA must perform a detailed review of these requests to assure that the structure is adequately designed and constructed to provide the stated level of protection, and to withstand the 100-year flooding event.

Part 65 of the NFIP regulations requires that any requester of a FIRM revision based on flood protection structures provide an analysis of the revised flood hazards, demonstrate and certify that the structure is designed and constructed for 100-year flooding conditions, and provide assurance that the structure will be maintained. Revision requests based on coastal structures are currently reviewed on a case-by-case basis using these regulations. A wide variation has been found in the quality of data submitted. Some possible reasons for this variation include the requester's inexperience or unfamiliarity with the different types of structures, the available design guidance, and/or the base (100-year) flood considered by the NFIP. In order to improve the quality of information submitted, and the ability of FEMA to review revision requests based on coastal structures, FEMA has decided to establish minimum design criteria that must be addressed in the request.

FEMA commissioned the U.S. Army Corps of Engineers, Waterways Experiment Station (WES), Coastal Engineering Research Center to identify or develope criteria for evaluating the effectiveness of all types of coastal flood protection structures in preventing or reducing damages and flooding from the 100-year event. This study identified and defined the different coastal structures that provide protection against flooding to property landward of the structure, and documented successful and unsuccessful cases for each structure type. The minimum criteria, considerations, and/or conditions applicable to the 100-year flooding event that are necessary for an evaluation of a coastal structure were also identified. The WES study recommended a procedure using these criteria to evaluate the adequacy, of a coastal flood protection structure to survive the 100-year flooding event, and to provide protection against flooding, wave runup and overtopping, wave forces, and erosion.

The WES Technical Report CERC-89-15 "Criteria for Evaluating Coastal Flood Protection Structures" was used as the basis for these critera. These criteria will also be used to resolve appeal challenges and in the conduct of flood insurance studies, when sufficient design and construction data are available.

Mapping of areas protected by coastal flood protection structures.

(a) General. For purposes of the NFIP, FEMA will only recognize in its flood hazard and risk mapping effort those coastal flood protection structures that meet, and continue to meet, minimum design and maintenance standards that are consistent with the level of protection sought through the comprehensive floodplain management criteria established by 44 CFR Part 60.3. Accordingly, this procedure describes the types of information FEMA needs to recognize, on NFIP maps, that a coastal flood protection structure provides protection from the base flood. This information must be supplied to FEMA by the community or other party seeking recognition of such a coastal flood protection structure at the time a flood risk study or restudy is conducted, when a map revision under the provision of Part 65 of this subchapter is sought based on a coastal flood protection structure, and upon request by the Administrator during the review of previously recognized structures. The FEMA review will be for the sole purpose of establishing appropriate risk zone determinations for NFIP maps and shall not constitute a determination by FEMA as to how a structure will perform in a flood event.

(b) Design Criteria. For coastal flood protection structures to be recognized by FEMA, sufficient evidence must be provided that adequate design, construction, and maintenance have been undertaken to provide reasonable assurance of durable protection from the base flood. The following requirements must be met:

(1) Design Parameters. A coastal flood protection structure must be designed using physical parameters that fully represent the base (100-year) flooding event, including the following:

(i) Design water levels evaluated should range from the mean low water level at the site to the 100-year stillwater surge elevation. The full range of elevations must be examined to determine the critical water level since the most severe conditions may not occur at either extreme.

(ii) Wave heights and periods must be calculated for each water level analyzed. At a minimum, significant wave height and periods should be used for "flexible" structures such as revetments, with larger wave height, up to the one-percent wave height (1.67 times the significant wave height), used for more rigid structures such as seawalls and bulkheads. The U.S. Army Corps of Engineers (COE) Shore Protection Manual (1984 or later edition), provides guidance and procedures for determining appropriate wave heights and periods.

(iii) Breaking wave forces under structure-perpendicular loading must be considered in the design unless it can be demonstrated that the structure will not be subject to breaking waves. The very high, short duration "shock" pressures must be used for low mass structures such as bulkheads, while only the secondary "non-shock" pressures need to be used for massive structures such as gravity seawalls. Analyses of the breaking wave forces using methods such as those identified in the COE report "Criteria for Evaluating Coastal Flood Protection Structures," (WES TR CERC-89-15) must be submitted.

(2) Minimum Freeboard. The minimum freeboard for coastal flood protection structures to be recognized on FEMA flood maps for protection against the storm surge component of the base flood shall be two feet above the 100-year stillwater surge elevation.

(3) Toe Protection. The loss of material and profile lowering seaward of the structure must be included in the design either through the incorporation of adequate toe protection or an evaluation of structural stability with potential scour equal to the maximum wave height on the structure. Engineering analyses such as those recommended in the COE's "Geotechnical Engineering in the Coastal Zone" (WES IR CERC-87-1), or "Design of Coastal Revetments, Seawalls, and Bulkheads" (COE EM 1110-2-1614) must be submitted for the toe protection, or an analysis of scour potential such as found in "Criteria for Evaluating Coastal Flood Protection Structures" (WES TR CERC-89-15) must be submitted.

(4) Backfill Protection. Engineering analyses of wave runup, overtopping, and transmission must be performed using methods provided in the COE report "Criteria for Evaluating Coastal Protection Structures" (WES TR CERC-89-15). Where the structure height is not sufficient to prevent overtopping and/or wave transmission, protection of the backfill must be included in the design. This should address prevention of loss of backfill material by rundown over the structure, by drainage landward, under, and laterally around the ends of the structure; as well as through joints, seams, or drainage openings in the structure.

(5) Structural Stability, Minimum Water Level. Analyses of the ability of the structures to resist the maximum loads associated with the minimum seaward water level, no wave action, saturated soil conditions behind the structure, and maximum toe scour must be submitted.

(i) For coastal dikes and revetments, a geotechnical analyses of potential failure in a landward direction by rotational gravity slip must be submitted.

(ii) For gravity and pile-support seawalls, engineering analyses of seaward sliding, of seaward overturning, and of foundation adequacy using the maximum pressures developed in the sliding and overturning calculations must be submitted.

(iii) For anchored bulkheads, engineering analyses of shear failure, moment failure, and the adequacy of the tiebacks and deadmen to resist the loadings must be submitted.

(6) Structural Stability - Critical Water Level. Analyses of the ability of the structure to resist the maximum loads associated with the critical water level, which may be any water level from the mean low water level to the 100-year stillwater elevation, including hydrostatic and hydrodynamic (wave) loads, saturated soil conditions behind the structure and maximum toe scour, must be submitted.

(i) For coastal dikes and revetments, geotechnical analyses of potential failure in a seaward direction by rotational gravity slip and of foundation failure due to inadequate bearing strength must be submitted.

(ii) For revetments, engineering analyses of the rock, riprap, or armor blocks' stability under wave action; uplift forces on the rock, riprap, or armor blocks; toe stability, and adequacy of the graded rock and geotechnical filters must be submitted.

(iii) For gravity and pile-supported seawalls, engineering analyses of landward sliding, of landward overturning, and of foundation adequacy using the maximum pressures developed in the sliding and overturning calculations must be submitted.

(iv) For anchored bulkheads, engineering analyses of shear and moment failure using "shock" pressures must be submitted.

(7) Material Adequacy. Documentation and/or analyses must be submitted that demonstrate that the materials used for the construction of the structure are adequate and suitable including life expectancy considerations, for the conditions that exist at the site.

(8) Ice and Impact Alignment. Where appropriate, analyses of ice and impact forces must be submitted.

(9) Structure Plan Alignment. A shore protection project should present a continuous structure with redundant return walls at frequent intervals to isolate locations of failure. Isolated structures or structures with a staggered alignment must submit analyses of the additional forces from concentrated, diffracted, and/or reflected wave energy on the different sections and ends.

(10) Other Design Criteria. FEMA will require that flood protection structures, regardless of type described above, be evaluated on the basis of how they may react structurally to applied forces. Therefore, analyses normally required of one structure type may also be required by another type which would react in a similar manner to applied forces. In unique situations, FEMA may require that other design criteria and analyses be submitted to show that the structure provides adequate protection. In such situations, sound engineering practice will be the standard on which FEMA will base its determinations. FEMA will provide the rationale for requiring any additional information.

(c) Adverse Impact Evaluation. All requests for flood map revisions based upon new or enlarged coastal flood control structures shall include an analysis of potential adverse impacts of the structure on flooding and erosion within, and adjacent, to the protected area.

(d) Community and/or State Review. For coastal flood protection structures to be recognized, evidence must be submitted to show that the design, maintenance, and impacts of the structures have been reviewed and approved by the affected communities and by any Federal, state or local agencies that have jurisdiction over flood control and coastal construction activities.

(e) Maintenance Plans and Criteria. For a coastal flood protection structures to be recognized as providing protection from the base flood, the structure must be maintained in accordance with an official adopted maintenance plan, and a copy of this plan must be provided to FEMA by the owner of the structure when recognition is being sought or when the plan for a previously recognized structure is revised in any manner. All maintenance activities must be under the jurisdiction of a Federal or state agency, an agency created by Federal or state law, or an agency of a community participating in the NFIP that must assume ultimate responsibility for maintenance. This plan must document the formal procedure that ensures that the stability and overall integrity of the structure and its associated structures and systems are maintained. At a minimum, maintenance plans shall specify the maintenance activities to be performed, the frequency of their performance, and the person by name or title responsible for their performance.

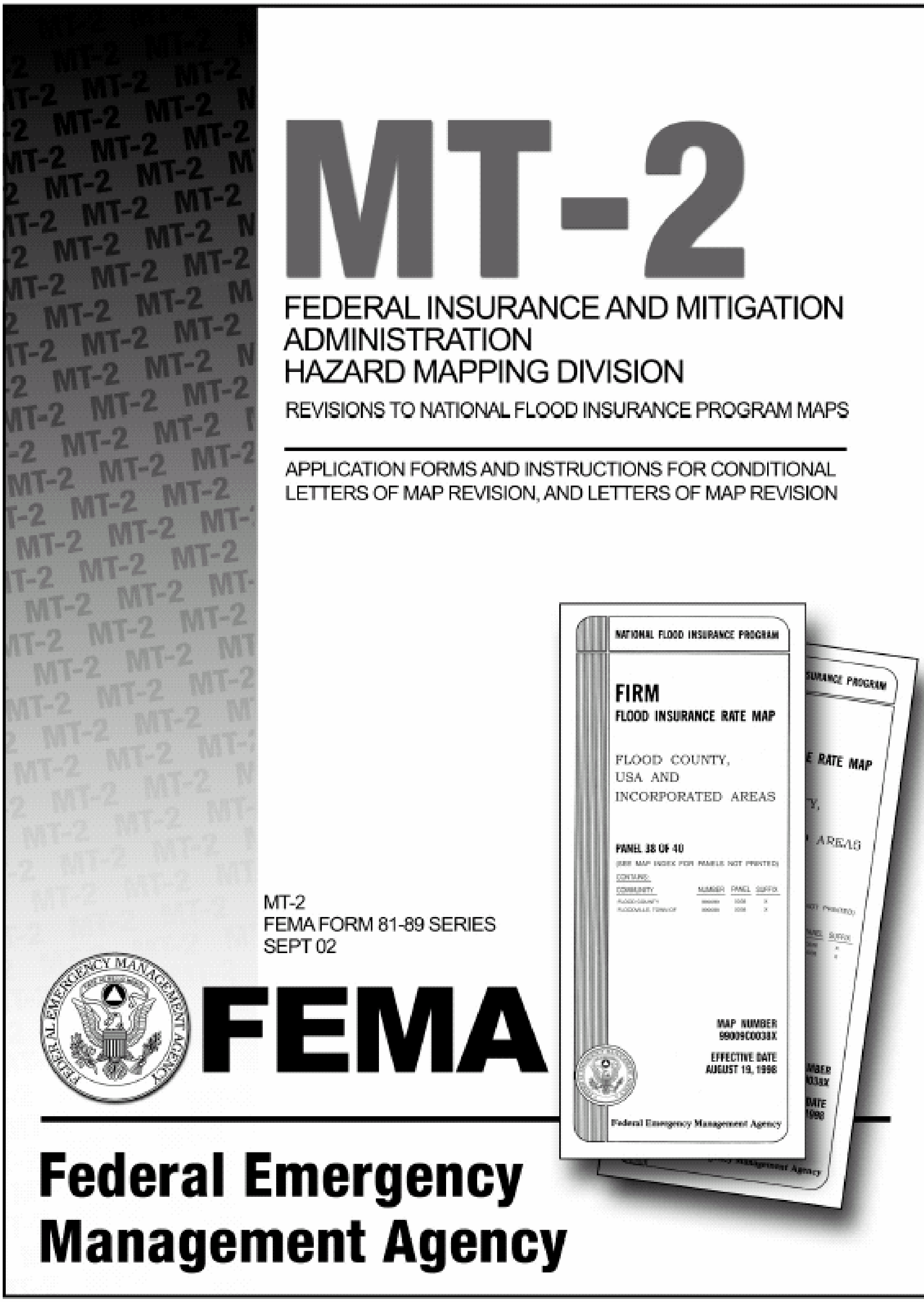
(f) Certification Requirements. Data and analyses submitted to support that a given coastal flood protection structure complies with the structural design requirements set forth in paragraphs (b)(1) through (10) above must be certified by a registered professional engineer. Also, certified as-built plans of the structure must be submitted. Certifications are subject to the definition given at § 65.2 of 44 CFR Part 65. In lieu of these certification requirements, a Federal agency with responsibility for design of coastal flood protection structures may certify that the structure has been adequately designed and constructed to provide protection against the base flood.

# COASTAL STRUCTURES FOCUS STUDY

# APPENDIX A-2

## FEMA MT-2, FORM 5

## COASTAL STRUCTURES ANALYSIS FORM



# MT-2

FEDERAL INSURANCE AND MITIGATION ADMINISTRATION
HAZARD MAPPING DIVISION

REVISIONS TO NATIONAL FLOOD INSURANCE PROGRAM MAPS

APPLICATION FORMS AND INSTRUCTIONS FOR CONDITIONAL LETTERS OF MAP REVISION, AND LETTERS OF MAP REVISION

MT-2
FEMA FORM 81-89 SERIES
SEPT 02



FEMA

Federal Emergency Management Agency

# INSTRUCTIONS FOR COMPLETING THE COASTAL STRUCTURES FORM (FORM 5)

The Coastal Structures Form is to be completed when a revision to coastal flood hazard elevations and/or areas is requested based on coastal structures being credited as providing protection from the base flood. The purpose of the Coastal Structures Form is to ensure that the structure is designed and constructed to provide protection from the base flood without failing or causing an increase in flood hazards to adjacent areas. Refer to the *Consolidated Guidelines and Specifications for Flood Hazard Mapping Partners, Appendix D: Guidance for Coastal Flooding Analyses and Mapping* which can be obtained from the Federal Emergency Management Agency's (FEMA's) Internet site at http://www.fema.gov/mit/tsd/dl_cgs.htm, for the criteria for evaluating flood protection structures.

If the coastal structure is a levee/floodwall, complete the Levee/Floodwall System section of the Riverine Structure Form (Form 3), in addition to this form. When the Coastal Structures Form is submitted, the Coastal Analysis Form (Form 4) should also be submitted.

**Section A: Background**

Information about the type of structure, the location, the material being used, and the age of the structure must be provided. Certified "as built" plans must also be provided. If these plans are not available, an explanation must be given with sketches of the general structure dimensions as described. If the structure design has been certified by a Federal agency to provide flood protection and withstand forces from the 1% annual chance (base) flood, the dates of the project completion and certification of the structure should be provided, and the remainder of the form does not need to be completed.

**Section B: Design Criteria**

Documentation must be provided that ensures a coastal structure is designed and constructed to withstand the wind and wave forces associated with the base flood. The minimum freeboard of the structure must be in compliance with National Flood Insurance Program (NFIP) Regulation 44 CFR Ch. 1, Section 65.10. Additional concerns include the impact to areas directly landward of the structure that may be subjected to overtopping and erosion along with possible failure of the structure due to undermining from the backside and the possible increase in erosion to unprotected properties at the ends of the structure. The evaluation of protection provided by sand dunes must follow the criteria outlined in NFIP Regulation 44 CFR Ch. 1, Section 65.11.

**Section C: Adverse Impact Evaluation**

If the structure is new, proposed, or modified, and will impact flooding and erosion for the areas adjacent to the structure, provide an explanation and documentation to support your conclusions.

**Section D: Community and/or State Review**

Provide documentation of Community and/or State review of the revision.

**Section E: Certification**

The licensed professional engineer and/or land surveyor should have a current license in the State where the affected communities are located. While the individual signing this form is not required to have obtained the supporting data or performed the analyses, he or she must have supervised and reviewed the work.

If the requester is a Federal agency who is responsible for the design and construction of flood control facilities, a letter stating that "the analyses submitted have been performed correctly and in accordance with sound engineering practices" may be submitted in lieu of certification by a registered professional engineer. Regarding the certification of completion of flood control facilities, a letter from the Federal agency certifying its completion and the flood frequency event to which the project protects may be submitted in lieu of this form.

FEDERAL EMERGENCY MANAGEMENT AGENCY

# COASTAL STRUCTURES FORM

*O.M.B. No. 3067-0148*
*Expires September 30, 2005*

## PAPERWORK REDUCTION ACT

Public reporting burden for this form is estimated to average 1 hour per response. The burden estimate includes the time for reviewing instructions, searching existing data sources, gathering and maintaining the needed data, and completing, reviewing, and submitting the form. You are not required to respond to this collection of information unless a valid OMB control number appears in the upper right corner of this form. Send comments regarding the accuracy of the burden estimate and any suggestions for reducing this burden to: Information Collections Management, Federal Emergency Management Agency, 500 C Street, SW, Washington DC 20472, Paperwork Reduction Project (3067-0148). Submission of the form is required to obtain or retain benefits under the National Flood Insurance Program. **Please do not send your completed survey to the above address.**

Flooding Source:
**Note:** Fill out one form for each flooding source studied

## A. BACKGROUND

1. Name of structure (if applicable):

2. Structure location:

3. Type of structure (check one):

☐ Levee/Floodwall* ☐ Anchored Bulkhead ☐ Revetment ☐ Gravity Seawall

☐ Breakwater ☐ Pile supported seawall ☐ Other:

***Note:** If the coastal structure is a levee/floodwall, complete Section E of Form 3 (Riverine Structures Form). The remainder of this form does not need to be completed.

4. Material structure is composed of (check all that apply):

☐ Stone ☐ Earthen fill ☐ Concrete ☐ Steel

☐ Sand ☐ Other

5. The structure is (check one):

☐ New or proposed ☐ Existing ☐ Modification of existing structure

☐ Replacement structure of the same size and design as what was previously at the site

Describe in detail the existing structure and/or modifications being made to the structure and the purpose of the modifications:

If existing, please include date of construction:

6. Copies of certified "as-built" plans ☐ are ☐ are not attached. Attach all design analyses that apply.

If "as-built" plans are not available for submittal, please explain why and attach a sketch with general structure dimensions including: face slope, height, length, depth, and toe elevation referenced to the appropriate datum (e.g. NGVD 1929, NAVD 1988, etc.).

7. Has a Federal agency with responsibility for the design of coastal flood protection structures designed or certified that the structures have been adequately designed and constructed to provide protection against the 1%-annual-chance event?

☐ Yes ☐ No

If Yes, specify the name of the agency and dates of project completion and certification.

**If Yes, then no other sections of this form need to be completed.**

## B. DESIGN CRITERIA

1. Design Parameters

   a. Were physical parameters representing the 1%-annual-chance event or greater used to design the coastal flood protection structure?

   ☐ Yes ☐ No

   b. The number of design water levels that were evaluated (number) range from the mean low water elevation of feet to the 1%-annual-chance stillwater surge elevation of feet. The critical water level is feet. The datum that these elevations are referenced to is (e.g.: NGVD 1929, NAVD 1988, etc.).

   Attach an explanation specifying which water levels and associated wave heights and periods were analyzed.

   c. Were breaking wave forces used to design the structure?

   ☐ Yes ☐ No If No, attach an explanation why they were not used for design.

2. Settlement

   a. What is the expected settlement rate at the site of the structure?

   Please attach a settlement analysis.

3. Freeboard

   a. Does the structure have 1 foot of freeboard above the height of the 1%-annual-chance wave-height elevation or maximum wave runup (whichever is greater)?

   ☐ Yes ☐ No

   b. Does the structure have freeboard of at least 2 feet above the 1% annual chance stillwater surge elevation?

   ☐ Yes ☐ No

4. Toe Protection

   Specify the type of toe protection:

   If no toe protection is provided, provide analysis of scour potential and attach an evaluation of structural stability performed with potential scour at the toe.

5. Backfill Protection

   Will the structure be overtopped during the 1%-annual-chance event? ☐ Yes ☐ No

   If the structure will be overtopped, attach an explanation of what measures are used to prevent the loss of backfill from rundown over the structure, drainage landward, under or laterally around the ends of the structure, or through seams and drainage openings in the structure.

6. Structural Stability - Minimum Water Level

   a. For coastal revetments, was a geotechnical analysis of potential failure in the landward direction by rotational gravity slip performed for maximum loads associated with minimum seaward water level, no wave action, saturated soil conditions behind the structure, and maximum toe scour?

   ☐ Yes ☐ No

   b. For gravity and pile-supported seawalls, were engineering analyses of landward sliding, landward overturning, and of foundation adequacy using maximum pressures developed in the sliding and overturning calculations performed?

   ☐ Yes ☐ No

   c. For anchored bulkheads, were engineering analyses performed for shear failure, moment failure, and adequacy of tiebacks and deadmen to resist loading under low-water conditions?

   ☐ Yes ☐ No

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### B. DESIGN CRITERIA (CONTINUED)

### C. ADVERSE IMPACT EVALUATION

If the structure is new, proposed, or modified, will the structure impact flooding and erosion for areas adjacent to the structure?

☐ Yes ☐ No

If Yes, attach an explanation.

### D. COMMUNITY AND/OR STATE REVIEW

Has the design, maintenance, and impact of the structure been reviewed and approved by the community, and any Federal, State, or local agencies having jurisdiction over flood control and coastal construction activities in the area the structure impacts?

☐ Yes ☐ No

If Yes, attach a list of agencies who have reviewed and approved the project.

If No, attach an explanation why review and approval by the appropriate community or agency has not been obtained.

### E. CERTIFICATION

As a Professional Engineer, I certify that the above structures will withstand all hydraulic and wave forces associated with the 1% annual chance flood without significant structural degradation. All documents submitted in support of this request are correct to the best of my knowledge. I understand that any false statement may be punishable by fine or imprisonment under Title 18 of the United States Code, Section 1001.

Certifier's Name:

License No.: Exp. Date:

Company Name:

Telephone No.: Fax. No.:

Signature: ______________________________ Date:

Seal (optional)

# COASTAL STRUCTURES FOCUS STUDY

# APPENDIX A-3

## FEMA REGULATIONS FOR COASTAL LEVEES:

## CFR PART 44 SECTION 65.10

a reissuance or revision of the flood insurance study or maps and will be deferred until such time as a significant change occurs;

(f) An additional 90 days is required to evaluate the scientific or technical data submitted; or

(g) Additional data are required to support the revision request.

(h) The required payment has not been submitted in accordance with 44 CFR part 72, no review will be conducted and no determination will be issued until payment is received.

[51 FR 30315, Aug. 25, 1986; 61 FR 46331, Aug. 30, 1996, as amended at 62 FR 5736, Feb. 6, 1997]

**§ 65.10 Mapping of areas protected by levee systems.**

(a) *General*. For purposes of the NFIP, FEMA will only recognize in its flood hazard and risk mapping effort those levee systems that meet, and continue to meet, minimum design, operation, and maintenance standards that are consistent with the level of protection sought through the comprehensive flood plain management criteria established by § 60.3 of this subchapter. Accordingly, this section describes the types of information FEMA needs to recognize, on NFIP maps, that a levee system provides protection from the base flood. This information must be supplied to FEMA by the community or other party seeking recognition of such a levee system at the time a flood risk study or restudy is conducted, when a map revision under the provisions of part 65 of this subchapter is sought based on a levee system, and upon request by the Administrator during the review of previously recognized structures. The FEMA review will be for the sole purpose of establishing appropriate risk zone determinations for NFIP maps and shall not constitute a determination by FEMA as to how a structure or system will perform in a flood event.

(b) *Design criteria*. For levees to be recognized by FEMA, evidence that adequate design and operation and maintenance systems are in place to provide reasonable assurance that protection from the base flood exists must be provided. The following requirements must be met:

(1) *Freeboard*. (i) Riverine levees must provide a minimum freeboard of three feet above the water-surface level of the base flood. An additional one foot above the minimum is required within 100 feet in either side of structures (such as bridges) riverward of the levee or wherever the flow is constricted. An additional one-half foot above the minimum at the upstream end of the levee, tapering to not less than the minimum at the downstream end of the levee, is also required.

(ii) Occasionally, exceptions to the minimum riverine freeboard requirement described in paragraph (b)(1)(i) of this section, may be approved. Appropriate engineering analyses demonstrating adequate protection with a lesser freeboard must be submitted to support a request for such an exception. The material presented must evaluate the uncertainty in the estimated base flood elevation profile and include, but not necessarily be limited to an assessment of statistical confidence limits of the 100-year discharge; changes in stage-discharge relationships; and the sources, potential, and magnitude of debris, sediment, and ice accumulation. It must be also shown that the levee will remain structurally stable during the base flood when such additional loading considerations are imposed. Under no circumstances will freeboard of less than two feet be accepted.

(iii) For coastal levees, the freeboard must be established at one foot above the height of the one percent wave or the maximum wave runup (whichever is greater) associated with the 100-year stillwater surge elevation at the site.

(iv) Occasionally, exceptions to the minimum coastal levee freeboard requirement described in paragraph (b)(1)(iii) of this section, may be approved. Appropriate engineering analyses demonstrating adequate protection with a lesser freeboard must be submitted to support a request for such an exception. The material presented must evaluate the uncertainty in the estimated base flood loading conditions. Particular emphasis must be placed on the effects of wave attack and overtopping on the stability of the levee. Under no circumstances, however, will a freeboard of less than two feet above the 100-year stillwater surge elevation be accepted.

(2) *Closures.* All openings must be provided with closure devices that are structural parts of the system during operation and design according to sound engineering practice.

(3) *Embankment protection.* Engineering analyses must be submitted that demonstrate that no appreciable erosion of the levee embankment can be expected during the base flood, as a result of either currents or waves, and that anticipated erosion will not result in failure of the levee embankment or foundation directly or indirectly through reduction of the seepage path and subsequent instability. The factors to be addressed in such analyses include, but are not limited to: Expected flow velocities (especially in constricted areas); expected wind and wave action; ice loading; impact of debris; slope protection techniques; duration of flooding at various stages and velocities; embankment and foundation materials; levee alignment, bends, and transitions; and levee side slopes.

(4) *Embankment and foundation stability.* Engineering analyses that evaluate levee embankment stability must be submitted. The analyses provided shall evaluate expected seepage during loading conditions associated with the base flood and shall demonstrate that seepage into or through the levee foundation and embankment will not jeopardize embankment or foundation stability. An alternative analysis demonstrating that the levee is designed and constructed for stability against loading conditions for Case IV as defined in the U.S. Army Corps of Engineers (COE) manual, "Design and Construction of Levees" (EM 1110–2–1913, Chapter 6, Section II), may be used. The factors that shall be addressed in the analyses include: Depth of flooding, duration of flooding, embankment geometry and length of seepage path at critical locations, embankment and foundation materials, embankment compaction, penetrations, other design factors affecting seepage (such as drainage layers), and other design factors affecting embankment and foundation stability (such as berms).

(5) *Settlement.* Engineering analyses must be submitted that assess the potential and magnitude of future losses of freeboard as a result of levee settlement and demonstrate that freeboard will be maintained within the minimum standards set forth in paragraph (b)(1) of this section. This analysis must address embankment loads, compressibility of embankment soils, compressibility of foundation soils, age of the levee system, and construction compaction methods. In addition, detailed settlement analysis using procedures such as those described in the COE manual, "Soil Mechanics Design—Settlement Analysis" (EM 1100–2–1904) must be submitted.

(6) *Interior drainage.* An analysis must be submitted that identifies the source(s) of such flooding, the extent of the flooded area, and, if the average depth is greater than one foot, the water-surface elevation(s) of the base flood. This analysis must be based on the joint probability of interior and exterior flooding and the capacity of facilities (such as drainage lines and pumps) for evacuating interior floodwaters.

(7) *Other design criteria.* In unique situations, such as those where the levee system has relatively high vulnerability, FEMA may require that other design criteria and analyses be submitted to show that the levees provide adequate protection. In such situations, sound engineering practice will be the standard on which FEMA will base its determinations. FEMA will also provide the rationale for requiring this additional information.

(c) *Operation plans and criteria.* For a levee system to be recognized, the operational criteria must be as described below. All closure devices or mechanical systems for internal drainage, whether manual or automatic, must be operated in accordance with an officially adopted operation manual, a copy of which must be provided to FEMA by the operator when levee or drainage system recognition is being sought or when the manual for a previously recognized system is revised in any manner. All operations must be under the jurisdiction of a Federal or State agency, an agency created by Federal or State law, or an agency of a community participating in the NFIP.

(1) *Closures.* Operation plans for closures must include the following:

(i) Documentation of the flood warning system, under the jurisdiction of Federal, State, or community officials, that will be used to trigger emergency operation activities and demonstration that sufficient flood warning time exists for the completed operation of all closure structures, including necessary sealing, before floodwaters reach the base of the closure.

(ii) A formal plan of operation including specific actions and assignments of responsibility by individual name or title.

(iii) Provisions for periodic operation, at not less than one-year intervals, of the closure structure for testing and training purposes.

(2) *Interior drainage systems.* Interior drainage systems associated with levee systems usually include storage areas, gravity outlets, pumping stations, or a combination thereof. These drainage systems will be recognized by FEMA on NFIP maps for flood protection purposes only if the following minimum criteria are included in the operation plan:

(i) Documentation of the flood warning system, under the jurisdiction of Federal, State, or community officials, that will be used to trigger emergency operation activities and demonstration that sufficient flood warning time exists to permit activation of mechanized portions of the drainage system.

(ii) A formal plan of operation including specific actions and assignments of responsibility by individual name or title.

(iii) Provision for manual backup for the activation of automatic systems.

(iv) Provisions for periodic inspection of interior drainage systems and periodic operation of any mechanized portions for testing and training purposes. No more than one year shall elapse between either the inspections or the operations.

(3) *Other operation plans and criteria.* Other operating plans and criteria may be required by FEMA to ensure that adequate protection is provided in specific situations. In such cases, sound emergency management practice will be the standard upon which FEMA determinations will be based.

(d) *Maintenance plans and criteria.* For levee systems to be recognized as providing protection from the base flood, the maintenance criteria must be as described herein. Levee systems must be maintained in accordance with an officially adopted maintenance plan, and a copy of this plan must be provided to FEMA by the owner of the levee system when recognition is being sought or when the plan for a previously recognized system is revised in any manner. All maintenance activities must be under the jurisdiction of a Federal or State agency, an agency created by Federal or State law, or an agency of a community participating in the NFIP that must assume ultimate responsibility for maintenance. This plan must document the formal procedure that ensures that the stability, height, and overall integrity of the levee and its associated structures and systems are maintained. At a minimum, maintenance plans shall specify the maintenance activities to be performed, the frequency of their performance, and the person by name or title responsible for their performance.

(e) *Certification requirements.* Data submitted to support that a given levee system complies with the structural requirements set forth in paragraphs (b)(1) through (7) of this section must be certified by a registered professional engineer. Also, certified as-built plans of the levee must be submitted. Certifications are subject to the definition given at §65.2 of this subchapter. In lieu of these structural requirements, a Federal agency with responsibility for levee design may certify that the levee has been adequately designed and constructed to provide protection against the base flood.

[51 FR 30316, Aug. 25, 1986]

**§65.11 Evaluation of sand dunes in mapping coastal flood hazard areas.**

(a) *General conditions.* For purposes of the NFIP, FEMA will consider storm-induced dune erosion potential in its determination of coastal flood hazards and risk mapping efforts. The criterion to be used in the evaluation of dune erosion will apply to primary frontal dunes as defined in §59.1, but does not

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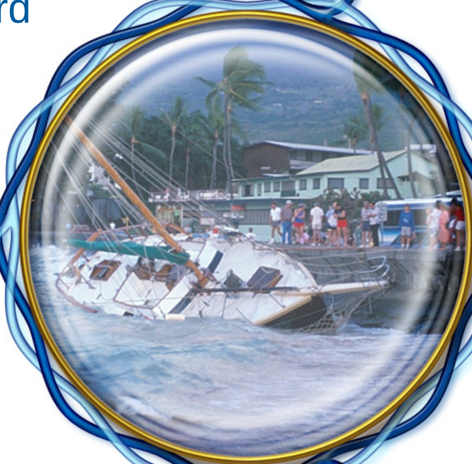
# Tsunami Hazards

## FEMA Coastal Flood Hazard Analysis and Mapping Guidelines Focused Study Report

February 2005



Focused Study Leader

Shyamal Chowdhury, Ph.D., CFM

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Eric Geist
Frank Gonzalez, Ph.D.
Robert MacArthur, Ph.D., P.E.
Costas Synolakis, Ph.D.

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### Tables



### Figures

## Acronyms

| | |
|---|---|
| CCM | Coastal Construction Manual |
| CFR | Code of Federal Regulations |
| ERDC | Engineer Research and Development Center |
| FEMA | Federal Emergency Management Agency |
| FD | finite-difference |
| FE | finite-element |
| FIRM | Flood Insurance Rate Map |
| FSG | Focused Study Group |
| LSW | linearized shallow-water wave |
| MOST | numerical inundation model |
| NFIP | National Flood Insurance Program |
| NSF | National Science Foundation |
| NTHMP | U.S. National Tsunami Hazard Mitigation Program |
| NOAA | National Oceanic and Atmospheric Administration |
| PTHA | Probabilistic Tsunami Hazard Analysis |
| SG | Study Group |
| USGS | U.S. Geological Survey |

# 1 INTRODUCTION

Guidelines do not currently exist for defining tsunami hazard zones for incorporation into FEMA Flood Insurance Rate Maps (FIRMs). This study is therefore focused on recommendations for developing tsunami hazard assessment methodologies for the Pacific Coast, through close coordination and collaboration of the FEMA National Flood Insurance Program (NFIP) with the U.S. National Tsunami Hazard Mitigation Program (NTHMP), a partnership led by the National Oceanic and Atmospheric Administration (NOAA) with participation from FEMA, the U.S. Geological Survey (USGS), NSF, and geotechnical and emergency management agencies of the five Pacific states (Alaska, California, Hawaii, Oregon, and Washington).

This report provides recommendations for preparing guidelines and a preliminary time estimate for four tsunami-related topics grouped under the Tsunami Focused Study. The table below shows the four topics and associated need and priority level.

A critical, short-term (6-month) activity is recommended to develop a methodology to incorporate existing NTHMP products into FIRM by combining Tasks 15 and 16 into a pilot study for a specific locale in California, Oregon, and Washington. The preliminary assessment will also allow for evaluation of the adequacy of existing guidelines, related to wind waves or riverine flows, to account for tsunami hazards. The current FEMA effort to update FIRMs focuses on California, Oregon and Washington, but will consider future efforts in other states, as appropriate. The seriousness of the tsunami threat to the states of Alaska and Hawaii are well-documented, and a rapid expansion of the FIRM update effort to these states is therefore strongly recommended.

**Tsunami Hazard Topics and Priorities**

| Topic Number | Topic | Topic description | Priority Level | | |
|---|---|---|---|---|---|
| | | | Atlantic / Gulf Coast | Pacific Coast | Non-Open Coast |
| 15 | NTHMP | Address use of NTHMP products and approaches in the NFIP | H | C | Pacific C |
| 16 | 100-year Recurrence | Develop methodology for determining the 100-year inundation line | H | C | Pacific C |
| 20 | Structure-Debris Interaction | Tsunami structure–debris interaction to define hazard zones. | H | I | Pacific I |
| 29 | Tsunami-Induced Erosion. | Review methodology for predicting erosion | H | I | Pacific I |
| Key: C = critical; A = available; I = important; H = helpful | | | | | |

The Focused Study Group (FSG) is made up of Frank Gonzalez of NOAA, Eric Geist of the USGS, Shyamal Chowdhury of Northwest Hydraulic Consultants (Team Leader), Costas Synolakis of the University of Southern California, and Robert MacArthur of Northwest Hydraulic Consultants. The FSG had several teleconferences and one meeting and exchanged numerous emails with documents and ideas. This preliminary work discusses the scope and NOAA's current and future plans for preparation of tsunami inundation mapping and possible venues of cooperation between FEMA, NOAA, USGS, and other agencies to address FEMA's needs for guidelines to incorporate tsunami hazard zones on FIRMs, both in the interim and long term.

## 1.1 DESCRIPTION OF THE HAZARD

Prior to discussing the topics from the Workshop a description of the tsunami hazard is included here to provide necessary background for developing guidelines to incorporate tsunami hazard assessments into FEMA maps.

Tsunamis are long waves of small steepness generated by impulsive geophysical events of the seafloor and of the coastline, such as earthquakes and submarine and aerial landslides. Volcanic eruptions and asteroid impacts are less common but more spectacular triggers of tsunamis. The determination of the terminal effects of tsunamis as they strike shorelines and coastal structures is one of the quintessential problems in earthquake engineering and has profound implications for mitigating their effects and saving lives.

Tsunamis are notorious for exporting "death and destruction at distant coastlines," as they sometimes travel across the world's oceans without dissipating sufficient energy to render them harmless. When striking at distances greater than 1,000 miles, tsunamis are referred to as teletsunamis or farfield tsunamis. An example of coastal inundation from teletsunami is shown in Figures 1 and 2.

Tsunamis generated within 200 miles of the coast and striking the adjacent shoreline, are referred to as nearfield tsunamis. Tsunamis originating between 200 to 1,000 miles off the target coastline are known as regional tsunamis. The characterization (farfield, nearfield, or regional) is thus dependent on the target coastline at risk. For example, a Cascadia Subduction Zone tsunami is nearfield in the Pacific Northwest, regional for California, and farfield or teletsunami for Hawaii or Japan.

The recorded tsunami history of the United States is quite short by world standards. In Washington, Oregon, and California, it is at best 180 years long, while a few older events are identifiable from paleotsunami studies. Until 1998, a total of 63 farfield tsunamis had been reported in the Western states, and 47 of them have been recorded instrumentally. Eleven have caused damage and fatalities (Lander et al., 1993) (Table 1). A total of 53 nearfield tsunamis have been reported in the same period, with 17 of them causing damage (Table 2) (Lander et al. 1993; McCarthy et al., 1993; Borrero, 2002).

**Figure 1. Inundation in Crescent City, California, from a tsunami triggered by the 1964 Great Alaskan earthquake.**

In the past 10 years, 12 major tsunamis have struck coastlines around the Pacific Rim, causing more than 3,000 deaths and an estimated $1 billion US (2001) in damage. Fortuitously, these tsunamis have either struck less developed coastlines or developed coastlines at low season with few or no visitors along the coast. Within the contiguous 48 states of the United States, the most significant event was the 1964 Great Alaskan tsunami that killed nine people in Crescent City, CA, and caused more than $30 million US (1984) in damage. Before the 1995 Kobe, Japan and 1999 Izmit, Turkey, earthquakes, it had been estimated that tsunamis cause between 5% and 15% of earthquake damage worldwide. During the past century, tsunamis have killed more people in the United States than earthquakes.

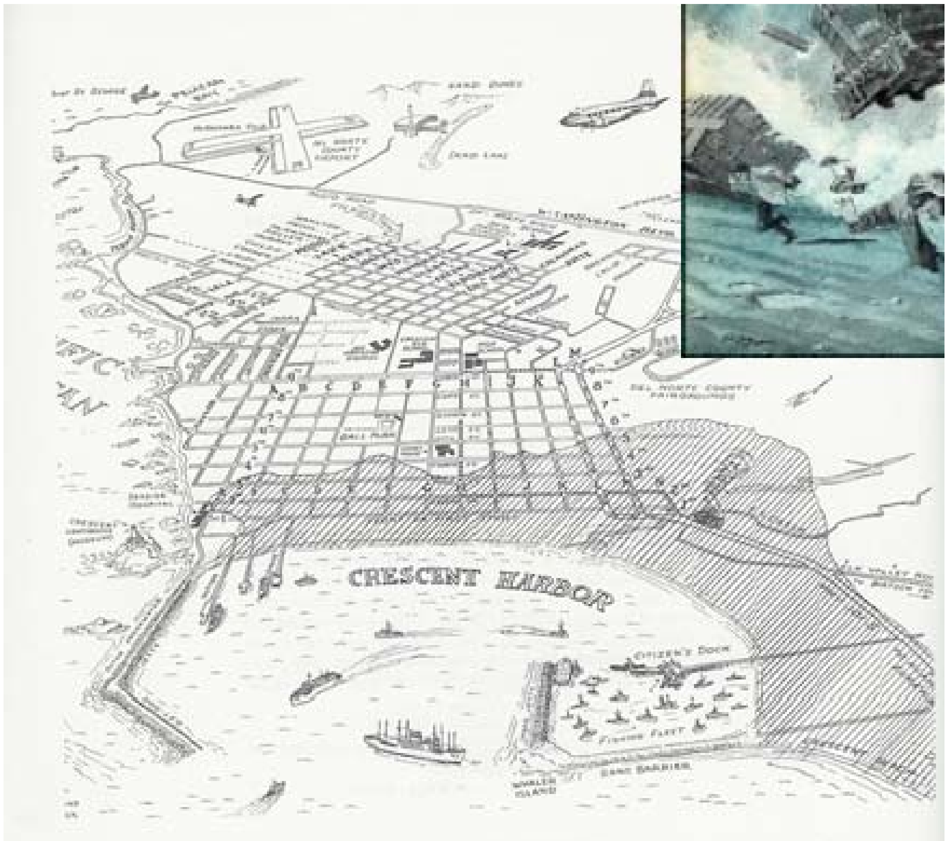


**Figure 2. The inundation line in Crescent City, California from a farfield tsunami triggered by the 1964 Great Alaskan earthquake.**

*The inset (upper right) is an artist's rendering of the nearfield impact around Cook Inlet, Alaska, and is based on eyewitness accounts.*

The term tsunami—also known as seismic sea wave or tidal wave—comes from the Japanese and translates as "harbor wave". Since ancient times, harbors have long been centers of commercial activity, and even a relatively small tsunami entering a harbor can trigger substantial harbor oscillations by bouncing off the harbor's embankments and combining together to form larger waves. Alaska's 1964 infamous Good Friday earthquake triggered large tsunamis that entered harbors throughout the region, including those at Anchorage, Valdez, and Seward, Alaska, and caused catastrophic destruction. The same tsunami manifested itself as a teletsunami in Crescent City, California where it killed nine people and devastated the downtown area, and was even recorded in Long Beach.

| Table 1. Farfield Tsunamis (teletsunamis) impacting Washington, Oregon, and California | | |
|---|---|---|
| Date | Generation Area | Impact in the United States |
| August 13, 1868 | Arica, Peru | San Diego, CA |
| May 10, 1877 | Chile | San Pedro, CA |
| June 15, 1896 | Sanriku, Japan | Santa Cruz, CA |
| January 31, 1906 | Ecuador | San Diego, CA |
| April 1, 1946 | Unimak Island, Alaska | Scott's Cap, Tahola, WA; Seaside, OR; and Noyo Harbor, Half Moon Bay, Santa Cruz, Port Huaneme, Catalina Island, CA |
| November 4, 1954 | Kamchatka, USSR | Brandon, OR; Crescent City, CA |
| March 9, 1957 | Unimak Island, Alaska | San Diego, CA |
| May 22, 1960 | Chile | Seaside, Gold Beach, OR; and Crescent City, Noyo Harbor, Pismo Beach, Morro Bay, Santa Barbara, Los Angeles, and San Diego, CA |
| March 28, 1964 | Prince William Sound, Alaska | $1.75M damage in Washington and Oregon; over $15M damage in Crescent City; $1M in San Francisco, CA 17 people killed |
| November 29, 1975 | Kanapala, Hawaii | Catalina Island, CA |
| Source: Lander et al., 1993 | | |

| Table 2. Nearfield tsunamis impacting Washington, Oregon, and California | |
|---|---|
| Date | Generation Area/Impacted Area |
| December 21, 1812 | Santa Barbara, CA |
| May 31, 1854 | Santa Barbara, CA |
| October 21, 1854 | San Francisco, CA |
| July 10, 1855 | San Juan Capistrano, CA |
| February 15, 1856 | San Francisco, CA |
| May 27, 1862 | San Diego, CA |
| October 8, 1865 | Santa Cruz, CA |
| October 21, 1868 | San Francisco, CA |
| November 22, 1873 | Oregon and Northwest CA |
| November 29, 1891 | Puget Sound, WA. |
| March 2, 1901 | Monterey, CA |
| April 18, 1906 | San Francisco, CA |
| November 4, 1927 | Point Arguello, CA |
| August 30, 1930 | Santa Monica, CA |
| April 13 and 16, 1949 | Tacoma, WA |
| October 19, 1989 | Monterey, CA |
| April 25, 1992 | Cape Mendocino, CA |
| Source: Lander et al., 1993; McCarthy et al., 1993; Borrero, 2002 | |

Such large earthquakes frequently generate strong seismic waves with periods of a few to tens of seconds that propagate outward from the epicenters along the surface of the earth. The ground motions from these seismic surface waves can cause the water in small harbors, marinas, navigation channels and coastal rivers to go into oscillation, sometimes hundreds or even thousands of kilometers from the earthquake epicenters. Called seismic seiches, these oscillations have the potential to cause substantial damage to shoreside facilities, acting either independently or enhancing the destructive power of tsunamis. Local oscillations induced by the seismic waves of the 1964 Alaska earthquake were observed at numerous sites throughout the US. Therefore, seismic seiches should also be included in the long-term plan to map tsunami and other water wave hazards.

Tsunamis also occur in lakes and reservoirs through seismic local ground shaking that triggers resonant oscillations (also known as seiching); by coseismic generation of subaerial or submarine mass movements (landslides, debris flows, slumps); or by aseismic mass movements or passing weather fronts. Lake Geneva (Lac Leman), Switzerland, is the textbook case of meteo-triggered lake oscillations. During the 1755 Lisbon earthquake, significant seiching was triggered in lakes as distant as Loch Lomond in Scotland and persisted for several days, while the 1964 Great Alaskan earthquake reportedly triggered oscillations in numerous reservoirs operated by the U.S. Army Corps of Engineers (Synolakis, 2002). Lander et al. (1993) list eight “reservoir” tsunamis in Franklin D. Roosevelt Lake, Washington, between 1944 and 1993. In Puget Sound, Washington, the inland waterways are exposed to multiple potential tsunami sources, in the form of active seismic fault systems and potential landslide and river delta failures; in Lake Washington and Lake Sammamish, sidescan imagery and seismic profiles have identified subaqueous landslides, submerged forests, coherent block slides, debris flows, sand flows and mixed slumps (Gonzalez et al., 2003). The January 17, 1994 Northridge earthquake triggered waves that sloshed up to 19 feet in the Los Angeles Reservoir, while unconfirmed reports published in the *Los Angeles Times* on January 18, 1994, described 30-foot waves and overtopping of the reservoir. Flooding from overtopping appears to be of growing concern, particularly for smaller reservoirs located in the midst of densely populated cities in many California locales.

## 1.2 TSUNAMIS VERSUS WIND WAVES

It is important to describe briefly the differences between wind waves and tsunamis, because a tsunami with heights comparable to those of swell often has substantially higher impact.

Tsunamis are created by sudden movements or disturbances of the water column by a number of mechanisms, including submarine explosions or impacts of large objects such as landslides from the coastline or asteroids, and/or aerial or submarine mass movements. These events trigger a series of fast-moving, long waves of initial low amplitude that radiate outward similar to the waves radiating when a pebble is dropped into a pond. In contrast, most of the swell waves observed on beaches are generated by wind disturbing the surface of the sea. Tsunamis are generated by disturbing the seafloor, wind waves by disturbing the ocean surface. Another mechanism for triggering tsunamis is shaking of a closed basin such as a reservoir, a lake, or a harbor, also known as sloshing or seiching.

Both wind waves and tsunamis are characterized by a wavelength, the horizontal distance between crests or peaks; a period, the time it takes successive peaks to pass a fixed point; and a height, the vertical distance from the wave trough to its crest. Wind waves tend to have a wavelength from a few inches to about a mile and periods of about 1/2 second to 30 seconds (Prager, 2000). In contrast, tectonic tsunamis near the source typically have a wavelength of hundreds of kilometers and periods of tens of minutes. Wind waves vary in height from tiny ripples on the sea surface to the rare rogue waves imagineered in the movie *The Perfect Storm*. Tsunamis, on the other hand, race across the open ocean as a series of long, low-crested waves, usually less than one meter high. Their steepness is so small that a ship at sea may not feel a tsunami passing beneath the hull.

In general, waves are considered deep-water waves if their wavelength $L$ is relatively small compared to the water depth $d$ through which they travel. Wind waves do not "feel" the seafloor until they are within one to several kilometers from the coastline, depending on the slope of the beach. In the open ocean, where depths average about 2.5 miles, all wind waves with period less than 30 seconds are deep-water waves—a short wavelength relative to depth ($d/L>1$). In contrast, shallow-water waves are those with a long wavelength relative to depth ($d/L<1/20$). The depth and nature of the seafloor strongly influence how shallow-water waves propagate or travel. Because tsunamis have such long wavelengths, even when traveling through very deep water, they are considered shallow-water waves. Thus, although geometric spreading initially reduces the height of tsunami waves, variations in ocean depth can focus or de-focus energy at a distant point. Because the earth is a sphere, simple divergence and convergence of orthogonals can also de-focus and focus tsunami energy over transoceanic distances; in fact, this mechanism will intensify tsunamis at sites antipodal to the source, so that the threat can actually increase at great distances.

In wind-generated waves, the orbital motion of the water particles decreases with depth from the water surface. As energy is transferred through the motion of the water particles, the energy of wind waves traveling through deep water is concentrated near the surface. By contrast, the energy imparted to the water during tsunami formation sets the entire water column in motion. Tsunami orbital velocities do not decrease significantly with depth, and although the wave height at the surface in the open seas is relatively small, the energy contained throughout the entire water column is substantial. Furthermore, the rate at which water waves lose energy is inversely proportional to their wavelength. Hence, tsunamis not only contain a significant amount of energy and move at high speeds (often reaching 450 mph), but they can also travel great distances with little energy loss.

As they move toward the coast, tsunamis pass through varying depths and over complex seafloor topography. Changes in the ocean depth and seafloor cause them to continuously evolve and change shape. A tsunami generated from an earthquake off Peru may look entirely different along the Peruvian coastline compared to when it enters a bay in California and still different, when it strikes a beach in Hawaii. Tsunamis are characterized by much greater length scales than wind waves and this can give rise to differences in nearshore propagation. For example, nearshore tsunamis are almost never forced waves, but it is not uncommon for shallow-water wind waves to be forced and, as a consequence, deviate from the usual free wave propagation characteristics. However, some mechanisms governing nearshore transformation are shared by both tsunamis and wind waves as they approach a coastline; in particular, both undergo refraction and shoaling. Shoaling is the process in which the wave front steepens and the wave height increases. The front of the wave enters shallower water and moves more slowly than the tail of the wave because the depth is smaller, hence the speed slower, at the front. If the wave is sufficiently steep and the continental shelf is wide, the wave eventually breaks, in essence tripping over itself. However, the crest lengths of tsunamis often cause unexpected wave patterns in refraction compared to those of wind waves. Also, because of their long wavelengths, tsunamis dissipate less than wind waves as they evolve up sloping beaches, and frictional effects are less important.

When tsunamis advance up on dry land, they can snap trees, destroy engineered structures, and carry boats far inland. Their impact is often described with the inundation distance and the runup height. The inundation distance refers to the maximum penetration inland of the entire tsunami wave train. Not unexpectedly, Tsunamis can penetrate farther on flat beaches than they do on steep beaches.

Tsunami runup is the maximum elevation the wave climbs up a coastline. One of the highest tsunami runup recorded in the United States occurred during the 1946 Aleutian tsunami at Unimak Island, Alaska. The tsunami destroyed the Scotch Cap lighthouse, killing all four U.S. Coast Guard officers. This nearfield event for Unimak was a teletsunami in Hilo, Hawaii, where it killed more than 150 people and ushered in the modern era of tsunami mitigation studies in the United States. The site of the disaster, along with a picture of the lighthouse as it stood before the event, is shown in Figure 3.

At times, the tsunami runup and inundation distance do not fully describe the tsunami impact. The large sustained currents associated with tsunamis can carry large objects and use them as projectiles to destroy structures behind them. The 1946 tsunami carried large debris, the impact force of which bent several parking meters along the Hilo coastline. During the 1994 Mindoro, Philippines, event, a 6-foot tsunami broke a 6,000-ton power generating barge off its moorings at the delta of the Baryan River, carried it one mile inland, and left it there, so that when the water level receded, there was not sufficient freeboard to tow the barge back to the delta (Imamura et al., 1995). During the 1995 Manzanillo, Mexico, tsunami, another 7-foot wave carried large recreational vehicles 600 feet inland and smashed them against palm trees (Borrero et al., 1995).

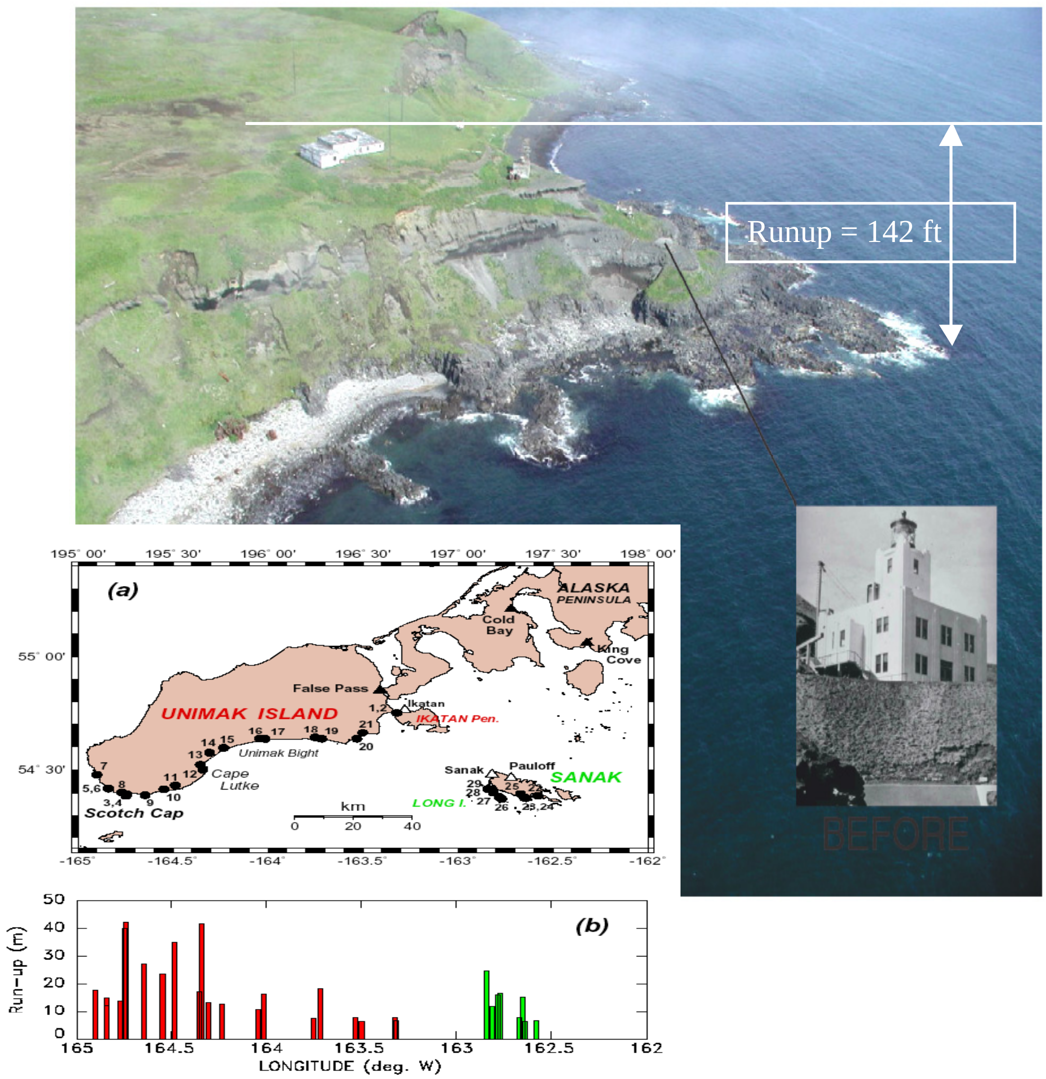


**Figure 3. Aerial view of Scotch Cap, on Unimak Island, Alaska.**

*The lighthouse that was destroyed during the 1946 Aleutian tsunami, killing all four U.S. Coast Guard officers, is shown in the inset at lower right. The inset at lower left shows Unimak and Sanak Islands and the runup distribution along the south side of Unimak Island. Scotch Cap is at the lower left tip of the island. Tsunami runup reached 142 ft. (Photo results from Okal et al. 2003; lighthouse photo from NOAA.)*

Tsunamis can also cause erosion and deposition of sediment or they can rip apart coral reefs that are in their path. Coastal regions that are low lying or located between steep cliffs or bodies of water are particularly vulnerable to tsunami damage. The September 1, 1992, Nicaraguan tsunami deposited a vast sediment blanket over many lowlands along the affected areas. On June 3, 1994, a magnitude 7.2 earthquake triggered a large tsunami that struck the coast of southeast Java and rolled on to hit southwest Bali (Synolakis et al. 1995). About 200 people were killed and 400 injured. Post-tsunami surveys identified watermarks, such as trees with sand-encrusted bark and leaves, indicating that runup reached about 17 feet in west Bali and up to 40 feet in southeast Java. Several beaches were completely washed away, while rivers effectively blocked evacuation routes. The same tsunami was documented along the northwestern Australian coast where a surge of 10 feet of water carried fish and rocks nearly 1,000 feet inland. The 1998 Papua, New Guinea, tsunami deposited a sediment layer that, in some areas, was 3 feet thick (Kawata et al., 1999; Gelfenbaum and Jaffe, 2003).

## 1.3 TECTONIC TSUNAMIS VERSUS LANDSLIDE-GENERATED WAVES

Predicting the initial wave generated by a seafloor motion or mass movement is the first step in modeling tsunami generation and assessing its possible impact. Until the last few months, when a NOAA DART buoy–also known as a tsunameter–recorded a tsunami in the open ocean, there were no measurements of tsunamis near the generation region, and tsunami hazard mitigation relied on untested models. The current state of knowledge differs substantially in reconstructing *ex post facto* tsunamis from tectonic sources and from submarine mass movements. Understanding the limitations of accurately predicting the initial tsunami wave is important in evaluating the accuracy of the predictions and is equally important in proposing recurrence intervals.

Earthquake-induced seafloor deformation was long believed to be the primary cause of most tsunamis, even though numerous major landslides and associated waves were triggered in fiords and lakes of southern Alaska by the 1964 Great Alaska earthquake (Plafker, 1965). It is now suspected that landslides play a much greater role in tsunami generation than was earlier believed. This should not have been all that surprising; Gutenberg suggested in 1939 that “submarine landslides are to be considered at least as one of the chief causes, if not indeed the major cause of tsunamis.” Landslide-generated tsunamis differ from the classic long waves in that they are steeper and disperse (break down) rapidly, particularly in shallow water. Also, mass movements often trigger tsunamis unexpectedly and sometimes aseismically. The 1994 Skagway, Alaska, tsunami was triggered by sediment instabilities at extreme low tides without associated seismic motions.

There are several important differences in the character of tsunamis triggered by mass movements compared to those triggered by earthquakes (tectonic tsunamis) (Prager, 2000). Tectotsunamis tend to have longer wavelengths, longer periods, and a larger source area than those generated by mass movements of earth. Whereas it is clear that the timing of the seafloor deformation is not important to first order in calculating tsunami evolution, it is also clear that the timing of mass movements is more important in the wave evolution; very slow movements will not generate large waves. Nonetheless, this is a parameter that cannot be determined very accurately (Okal and Synolakis, 2003).

When a potential tsunami-triggering earthquake occurs, sufficient information is often available to predict whether or not a massive wave will be created. This is all that can be inferred, however reliably. There are at least four characteristics of a mass movement that determine whether or not a tsunami will form; its length, width, thickness, and the inclination of the slope that fails and triggers the landslide. Controversy remains regarding the ways in which the generated waves are affected by the geomechanical characteristics of the sliding material. This controversy is partially attributable to the lack of knowledge about the effects of the timing of seafloor motion, but is more importantly related to the lack of validated constitutive models. None of these characteristics can yet be accurately predicted; the relevant information on geometric slide characteristics may sometimes be difficult to determine even after the event (Synolakis et al., 2002a) A few empirical and computational methods exist to predict initial waves generated by underwater mass movements (Chiang, et al, 1981).

Compared with the understanding of earthquake induced initial tsunami waves, the understanding of landslide-induced initial waves is marginal. A few empirical and computational methods exist to predict initial waves generated by underwater mass movements. The lack of understanding limits intuition, leading to inadvertent errors. In 1985, a purely arithmetic error in a simple algebraic formula led to underestimating the size of a possible tsunami from the Palos Verdes, California, debris avalanche by a factor of 100 in official U.S. government reports. The error was quoted freely until the arithmetic was redone in 2001. The PV wave, calculated in 1985 at 0.14 m, was found to be 14 m using the identical algebraic formula and the identical landslide parameters. Because no field data are available for verification, the degree of understanding or the lack of understanding embodied in the algebraic formula is unknown. But this interesting anecdote, involving solutions that differ by two orders of magnitude, does demonstrate the lack of intuition mentioned above.

While landslide-induced tsunamis may not be as uncommon as believed before the 1998 Papua, New Guinea, event (Synolakis et al., 2002a), it is now accepted that the most common cause is submarine earthquakes. Note that not all submarine earthquakes generate tsunamis. According to Okal (2002), in the past 31 years, there have been one submarine earthquakes per year of magnitude 8 or higher and about 10 of magnitude 7, yet only 20 of these have reportedly created tsunamis. The pattern and extent of vertical ground deformation from an earthquake uniquely determines whether a tsunami is formed or not. Most seismic faults combine both strike-slip and thrust motions, but primarily only faults that have predominantly vertical displacement and create sufficiently large seafloor deformations appear to trigger a tsunami.

## 1.4 FACTORS IN TSUNAMI MODELING

Generally, the larger the magnitude of an earthquake, the larger the deformed area is, and this deformed area usually contains an area of uplift and subsidence that defines the dipole shape of the wave. The deformation area refers to the horizontal extent of deformation, while slip length is a measure of vertical change. Strong earthquakes not only deform larger areas, but do so by a greater amount of slip, thus producing disproportionally larger tsunamis than smaller events. Tsunami generation is discussed in detail in Geist (1997, 2003) and Geist and Dmowska (1999).

In addition to an earthquake's magnitude, the depth of the earthquake affects tsunami generation. The deeper the hypocenter or focus of an earthquake, the smaller the vertical deformation of the Earth's surface. A deeper hypocenter allows the seismic energy to spread over a larger volume, so that less energy reaches the ground surface. Earthquakes deeper than about 30 km rarely cause sufficient deformation to generate tsunamis.

An earthquake whose epicenter lies inland will only generate a tsunami if it produces sufficient vertical deformation offshore on the seafloor. Therefore, only very strong inland thrust earthquakes, as compared to even moderate offshore earthquakes, are potential tsunami generators (unless they trigger a massive landslide into the sea). For example, the 1994 Northridge earthquake resulted in vertical ground deformations of up to 6 feet but did not produce a tsunami. Had the fault ruptured with the same strength about 40 miles west offshore, it would have probably created a substantial tsunami inside Santa Monica Bay.

Tsunami models use the energy released; the size of the deformed area; the mean displacement at the surface; and the dip, strike, and slip angles, to infer a seafloor displacement pattern. Then, the models assume that water motion occurs instantaneously and, therefore, the initial tsunami wave will have the same shape as the seafloor displacement. Whatever mass of fluid is displaced by the seafloor moving up or down causes an equivalent displacement of the water in the same direction. The instantaneous assumption is based on the fact that tsunamis propagate at speeds up to 700 feet per second (fps), while seismic waves cause rupture at typical speeds of 1 to 2 miles per second.

Once the initial wave conditions are established, tsunami models estimate the evolution of the tsunami from its source to the target coastline, over the underlying seafloor bathymetry. When the simulated wave arrives at the coastline, tsunami models become inundation models and calculate the evolution of the tsunami as it moves inland. Tsunami models are really the synthesis of earthquake, wave evolution, and flood inundation models.

# 2 CRITICAL TOPICS SECTION

## 2.1 TOPIC 15: ADDRESS USE OF NATIONAL TSUNAMI HAZARD MITIGATION PROGRAM PRODUCTS AND APPROACHES IN THE NFIP. (HELPFUL FOR THE ATLANTIC AND GULF COASTS, CRITICAL FOR OPEN AND NON-OPEN PACIFIC COASTLINES.)

The National Tsunami Hazard Mitigation Program (NTHMP) conducts site-specific tsunami inundation modeling efforts for hazard assessment, based on “credible worst-case” tsunami generation scenarios (Bernard et al., 1996). The scenarios are based on identified tsunamigenic sources, typically earthquakes and/or landslides near the threatened site. Source development is key in this approach and involves research into historical and prehistorical events, including geologic fieldwork such as paleotsunami investigations and shallow sediment coring of the seafloor, as well as geophysical investigations such as multibeam bathymetric and seismic reflection surveys. Then numerical computations are undertaken that evolve the wave from its source to the target coastline (either nearshore or farfield), which sometimes involves transoceanic propagation. Model output includes wave height and current speed over the computational domain; from these, various products can be derived, including a line of maximum inundation that occurs over the duration of the simulated event. Tsunami inundation models are tested by simulating historic tsunamis and comparing model results with available tide gauge records. Simulations are also tested using field measurements of tsunami deposit distribution and estimates of tsunami current speed from sediment transport modeling of deposits as described in Section 3. Recommended best practices and quality control procedures for official NTHMP hazard assessment products are discussed by González et al. (2003). The current approach is outlined below, with the state of California as a case study.

### 2.1.1 Description of the Topic and Suggested Improvement

Inundation maps provide emergency managers in coastal communities with the necessary tools to plan for and mitigate tsunami disasters. Inundation maps are not only useful in assessing the population and facilities at risk, but also helpful in planning for emergency response. The preparation of inundation maps involves the assessment of the local geologic hazards, the interpretation of those hazards in terms of tsunami initial conditions, and the calculation of the resulting potential coastal inundation. Inundation maps now exist for most coastal areas of the Pacific states of the U.S., most coastal areas of Japan, and several other vulnerable areas around the world.

In this section, as a case study, the preparation of tsunami inundation maps in California will be presented. Even using these state-of-the-art inundation prediction tools, California presents unique challenges in assessing tsunami hazards: 1) There is an extremely short historic record of tsunamis in the state. Whereas some areas in the Pacific have 1,000-year-long records, in California there are none known before the 19th century. Although 28 more-than-credible tsunami “hits” have been reported, only the impact of the 1964 event has been well documented. 2) The geologic work in the state has been concentrated on identification of the risks associated with onshore faults. There is scant and mostly unpublished information on offshore faults or landslide and slump scars suggestive of past submarine mass failures.  3) Earlier estimates of tsunami hazards had relied almost entirely on farfield sources and had used pre-1980s inundation mapping technology. This had created the impression among policy planners and the general public that the tsunami hazard was small.

The most comprehensive calculation of tsunami hazards for California is the work of Houston and Garcia (1974) and of Houston (1980), both of which focused on the hazard in Southern California from farfield events. McCulloch (1985) also focused on the hazards in the Los Angeles region, primarily from farfield events, but also considered several nearfield events. Satake and Sommerville (1992) analyzed the Lompoc 1927 earthquake and the associated local hazards. In a seminal review, McCarthy et al. (1993) analyzed the historic records of tsunamis in California and predicted qualitatively the hazard over the entire state.  Synolakis et al. (1997) reviewed pre-1997 studies and observed that the earlier runup estimates did not include inundation calculations. When performed with the new generation of inundation models, runup estimates were occasionally up to 100% higher than what the earlier calculations suggested, depending on the nearshore topography.  Borrero et al. (2001, 2002, 2003) studied nearshore tectonic, landslide, and slump sources in East Santa Barbara channel and produced runup estimates ranging from 6 to 40 feet.  For the San Pedro Bay, Borrero et al. (2004) estimated losses ranging from $7 to $40 billion provide estimates for the leading wave heights for landslide-generated waves off Palos Verdes ranging from 30 feet to 120 feet depending on the initiation depth.  The bathymetry off Palos Verdes (shown in Figure 4) has features suggestive even to non-marine geologists of landslide scarps.

The current state of understanding is reviewed in Borrero (2002) and in Synolakis et al. (2002c). They have argued that the 100-year hazard in California is dominated by distant events, similar to the assumption of the earlier FEMA studies.  However, given the recent results of offshore landslide hazards, which have yet to be accurately timed, they argued that the 500-year hazard is dominated by local events, hence revising Houston and Garcia's estimates.

In 1996, the Tsunami Hazard Mitigation Federal/State Working Group prepared a report to the U.S. Congress recommending the preparation of inundation maps for the five Pacific states (Alaska, Hawaii, California, Oregon, and Washington). The report led to mobilization of significant federal resources for tsunami hazards mitigation, and to the establishment of the U.S. National Tsunami Hazard Mitigation Program (NTHMP), which provides resources in all five states for mitigating tsunami hazards. The NTHMP was the focus of a program review during the International Tsunami Symposium held on August 5-7, 2001, in Seattle, Washington (Gonzalez et al, 2001).

As early as 1997, California's Coastal Region Administrator of the Governor's Office of Emergency Services (OES), through a series of workshops and publications, informed local governments and emergency agencies of the plans to address tsunami hazards and presented the NTHMP.  OES solicited input as to the levels of hazards to be represented on the maps because

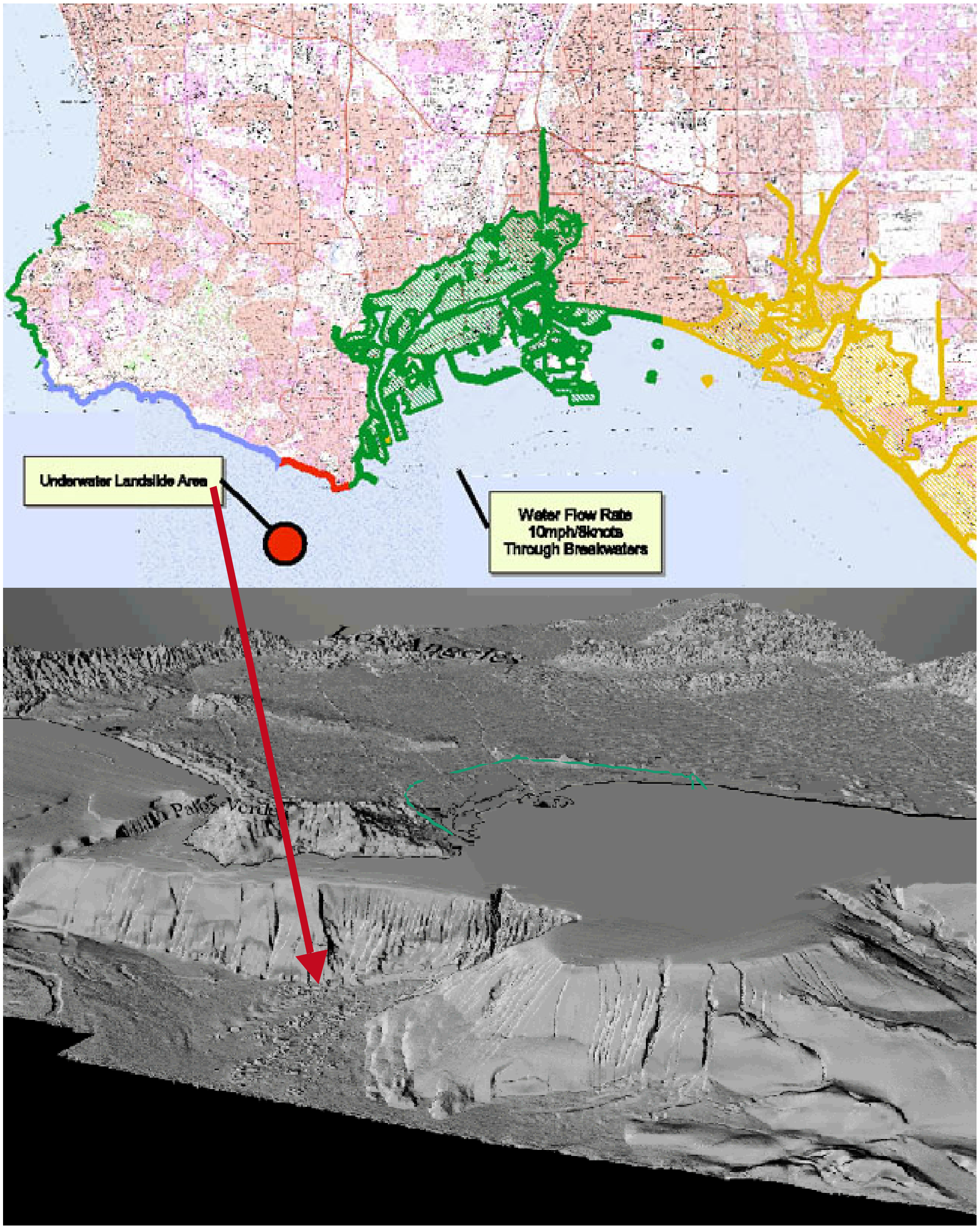


**Figure 4. Images of Santa Monica and San Pedro bays, California.**

*Inundation zone from a tsunami generated from a failure of the Palos Verdes, California debris avalanche in green (i. e. the zone on the left). The "standard" flood zone from a dam break is shown in yellow (inundation zone on the right of the top figure). The lower figure shows the topography off Palos Verdes and identifies the so-called PV debris avalanche. The economic impact ranges from $7 to $40 billion U.S.*

the short length of the historic record did not permit a comprehensive probabilistic hazard assessment. It was decided that the maps would include credible worst-case scenarios, to be identified further in the mapping process. In 1998, as funding became available for the state, OES contracted with the Tsunami Research Program of the University of Southern California for the development of the first generation of inundation maps for the state.

The State of California has the most densely populated coastlines among the five states in the NTHMP. The state had to use the same limited resources as the other four but assess offshore tsunami hazards over a much longer coastline. A comprehensive tsunami hazard evaluation involves both the probabilistic hazard assessment of different farfield and nearfield, onshore and offshore sources and the hydrodynamic computation of the tsunami evolution from the source to the target coastline. Given the level of funding, this was not feasible, and this presented another challenge for California.

Given the limited resources, it was decided to focus on nearshore tsunami hazards, which had never been modeled; even with the acknowledged limitations, the impact from farfield events had at least been attempted by Houston and Garcia (1974). Although return periods for these nearshore sources were not estimated, they are likely longer than the 100-year return period of the Houston and Garcia sources. But current NTHMP hazard assessment policy is to develop "credible worst case" scenarios, rather than events characterized by a specific return period. Thus, if inundation predictions from nearshore events proved smaller than twice the farfield tsunami results of Houston and Garcia, then farfield sources would have to be considered as well. As it turned out, nearfield sources produced inundation predictions more than twice those of Houston and Garcia. Hence, the effort was focused on identifying credible nearfield events, such as submarine mass movements, and then developing the state inundation maps for these nearfield scenarios only.

The state was also faced with the decision of choosing its mapping priorities. By considering the geographic distribution of population centers, the state opted to perform modeling of the Santa Barbara and San Francisco coastlines in year one, of Los Angeles and San Diego in year two, and of Monterey Bay in year three. The next decision was the resolution of the numerical grids to be used in developing the maps. The technology existed for high-resolution maps with grids of sizes as small as 17 feet, but this would result in a relatively small spatial coverage with large computational grids and painful computations. It was decided that the goal would be to produce maps at 400 feet resolution, based on information from Titov and Synolakis (1997, 1998), who had argued that dense grids may improve numerical accuracy but do not improve the realism if the available bathymetric/topographic sets are not of similar resolution. In California, the best available sets varied in resolution between 170 feet and 500 feet. Also, given the uncertainties in locating and understanding source mechanisms for submarine mass movements results with higher resolution would be misleading.

The next question was whether to provide emergency planners with inundation results at different levels of risk. For example, one suggestion was to include low and high risk lines on the inundation maps. Another suggestion was to provide separate lines for nearfield and farfield events. After discussing these issues with emergency preparedness professionals across the state, it was decided that a single line representing a worst-case scenario was preferable, for it simplified the preparedness response of city officials and better informed the general public.

The inundation mapping effort first identified offshore faults and offshore landslide and slump hazards. Difficulties encountered included the lack of detailed high-resolution marine surveys over all target coastlines. Marine surveys have been undertaken by the USGS off Santa Monica Bay and by the Monterey Bay Aquarium Marine Institute (MBARI) off Santa Barbara and Monterey Bay (Greene et al., 2000); high resolution surveys are not available for other parts of the state, if indeed they exist at all. Hence, given that onshore earthquakes can trigger submarine landslides, in regions where marine geology data did not exist, steep submarine soft-sediment slopes were considered as possible sources. Data on offshore faults and slide-prone areas were then used to develop initial tsunami waves as discussed in Borrero et al. (2001), and then the inundation model MOST was used to obtain inundation heights and penetration distances along the target coastline.

Once draft versions of the maps became available, the California OES presented them in regional meetings with emergency preparedness officers and other interested parties such as the State Lands, Seismic Safety, and Coastal commissions. Further input was solicited, and an emergency response manual was produced by OES (2002) with guidelines for mitigation. OES also produced a videotape for school use and distributed numerous copies of other commercial video programs describing tsunami hazards. The development of the state's inundation maps was featured in four Discovery Channel documentaries and in numerous national and local news stories.

### 2.1.2 Description of Procedures in the NTHMP Guidelines

NTHMP tsunami hazard assessment products draw on the collective expertise of NOAA, USGS, and the community of academic scientists and engineers actively involved in tsunami research and hazard mitigation (Bernard et al., 1996; Gonzalez et al., 2003). Thus they represent the best available methodology and information for tsunami hazard assessment. In each state, the historic and prehistoric record is examined to determine whether the worst credible event is likely to occur from a farfield or nearfield tsunami. In summary, the program has identified the following points:

A. Southern and Central California are at risk from local and distant earthquakes and from coseismic or aseismic subaerial and subaqueous slides.

B. Northern California to Northern Washington and Straits of Juan de Fuca are at risk from Cascadia Subduction Zone earthquakes and from coseismic of aseismic subaerial and subaqueous slides.

C. Puget Sound is at risk from local earthquakes (i.e., along the Seattle, Tacoma, and other local fault systems) from coseismic and aseismic subaerial and subaqueous slides and from delta failures.

D. For Alaska, the primary sources of tsunamis are local earthquakes and landslides. In Hawaii, distant earthquakes and local landslides are the primary sources of tsunamis.

To date, 22 inundation modeling efforts have been completed, covering approximately 108 coastal communities in California, Oregon, and Washington with an estimated at-risk population of 1.2 million residents. Work continues on the estimated 40 additional modeling efforts needed to cover the remaining 2.2 million residents at risk.

### 2.1.3 Application of Existing NTHMP Guidelines to Topic–History and/or Implications for the NFIP

#### *Alternatives for Improvement*

Because the NTHMP methodology does not fit a current FEMA template for the assessment of other flooding hazards, we recommend a focused, collaborative FEMA/NTHMP effort to develop such a methodology for incorporation of NTHMP tsunami hazard assessments into the FEMA National Flood Insurance Program–perhaps as a special overlay on existing FIRMs (i.e., development of a separate Tsunami Hazard Zone Delineation) or some other use of the NTHMP products. We have identified the following areas that are critical for this integration.

**A. Probabilistic hazard assessment, 100-year-return period**

The current NTHMP approach does not explicitly address the probability of occurrence of events. In particular, no formal effort is made to develop a "100-year event" or an estimated recurrence period for a particular source. Rather, the focus is on creating a scientifically defensible scenario for generation of a tsunami that poses a potential threat to the community that can be used for emergency management purposes. This critical need is addresses in Task 16 in detail.

**B. Producing inundation maps versus evacuation maps**

Without a probabilistic hazard assessment element in the NTHMP's existing inundation maps, it has been difficult to rank the relative risk from different scenarios. Some states have felt that lines identifying risk zones for nearfield and farfield events would prove cumbersome and confusing for the public. In these cases, it was decided to consider, for every locale in each study region under consideration, the worst credible event based on the available historic earthquake and tsunami information.

The inundation predictions for any given event are highly dependent on bathymetry and topography and vary substantially along the coast. Because the location of the source is seldom accurately known, the source was moved around but remained within the range of uncertainty of its location. Along California's flat coastlines, this relocation of the tsunami sources resulted in relocation of the maximum along the coast. When asked, emergency planners preferred to have a single value for each region identifying the maximum elevation that tsunami waves from the different local offshore sources would attain. This practice would simplify the communication of the risk to the public and would provide information that is easy to remember and implement in regional emergency preparedness. For example, a region could plan for tsunami evacuation areas above a certain minimum elevation across its jurisdiction. Hence, in the development of the maps, sources were relocated along the coast and the highest inundation value among different runs was identified.

Interestingly, in the areas studied, there were no areas that consistently experienced higher runup that adjacent locales. Synolakis et al. (2002c) found that most low-lying coastal areas could experience high runup if the source was relocated in an appropriate direction, within the uncertainties of defining the source. Thus, the inundation maps for California do not represent the inundation from any particular event or characteristic earthquake, but the locus of maximum penetration distances from relocating worst-case scenario events. For the Palos Verdes tsunami, Borrero et al. (2004) estimated direct, indirect, and induced losses ranging from $7 to $40 billion. An interim procedure for incorporating NTHMP maps into FIRMs would be to use the existing 100-year-return maps and designate a separate hazard zone for tsunami risks. The NTHMP boundaries would then be the tsunami hazard zone limits.

Substantial effort would have to be expended to identify the predicted hazard zone limits within existing inundation maps. Because the NTHMP maps are used for evacuation planning and emergency preparedness, most often they reflect local conditions. For example, to effectively implement evacuation plans, if the inundation zone is close to a major highway, the state maps extend to the highway, which is then designed for evacuation. Also, the existing maps often portray an inundation zone that is larger than the zone predicted from the models. For example, if the highest credible tsunami runup within a region is 40 feet, the evacuation maps attempt to follow the 40-foot-elevation contour everywhere. While this is important for evacuation planning, it is not adequate for flood insurance mapping, where detail might be important.

### 2.1.4 Preliminary Time Estimate for Guideline Improvement Preparation

Supplementary support for NOAA, USGS, and FEMA activities will be required to appropriately expand ongoing work and include efforts specific to NFIP needs–conduct a comprehensive review of the relevant literature, examine in-house geologic and geophysical data create the digital database, develop an appropriate methodology, relocate the existing inundation line as appropriate, and produce a report. Interagency discussions will determine the source of new funds. Table 4 at the end of this report presents estimates of times required to accomplish the tasks in this topic.

## 2.2 TOPIC 16: PROBABILISTIC HAZARD ASSESSMENT FOR THE OPEN AND NON-OPEN COASTLINES OF THE PACIFIC STATES

A methodology is recommended for completing a comprehensive probabilistic tsunami hazard assessment for the Pacific Coast, considering both farfield events and nearfield events triggered by seismic sources. For the Pacific Coast, farfield events are those generated a long distance away by seafloor displacement during earthquakes, such as the 1964 Alaska and 1960 Chile earthquakes; and nearfield events are those generated by submarine landslides triggered by earthquakes offshore and onshore, such as the Cascadia Subduction Zone earthquakes that triggered the 1992 Cape Mendocino tsunami along the California coast. Existing FIRMs for the Pacific Coast depict only risk associated with farfield events and the method relies on the definition of a 1% annual-chance-event and a 0.2% annual-chance-event tsunami for the farfield events.

Traditionally, FEMA's policy has been to incorporate tsunami-induced hazard and other storm-related coastal hazard into one coastal high hazard zone, which is defined in 44 Code of Federal Regulations (CFR) Part 59.1 as follows:

> *Coastal high hazard area means an area of special flood hazard extending from offshore to the inland limit of a primary frontal dune along an open coast and any other area subject to high velocity wave action from storms or* ***seismic*** *sources.*

During the course of this study, it became imperative to address not only the statistical aspects of tsunami generation, but the associated geological, numerical modeling, regulatory, and institutional aspects and available resources at NOAA, USGS, and in academic institutions.

### 2.2.1 Description of the Topic and Suggested Improvement

There is no existing guideline for tsunami hazard assessment for the Pacific Coast. However, Houston and Garcia (1974) and Houston (1980) conducted tsunami hazard studies that can be construed as FEMA's methodology on effective studies for the Pacific Coast. In a Type 19 Flood Insurance Study for Southern California, Houston (1980) conducted tsunami prediction studies for the 100-year and 500-year tsunamis based on events in Chile and Alaska. A similar approach was adopted for Type 16 Flood Insurance Studies for the West Coast of the Continental United States and for Monterey and San Francisco bays and Puget Sound.

The runup frequency relationship for tsunamis was combined statistically with combined frequency relationships for swell and wind waves to produce a single runup frequency relationship along a transect. Thus, the resulting coastal hazard zones represented hazards associated with high-velocity wave action from storms or seismic sources. The biggest limitation of this method is that it incorporated only farfield events.

Houston and Garcia (1974) used a combination finite-difference (FD) solution and analytic solution of the linearized shallow-water wave (LSW) equations to calculate tsunami propagation, except in Santa Monica and San Diego bays, where they used a finite-element (FE) solution to resolve possible local resonance effects. They argued that the only reliable data for defining source characteristics at that time were from the 1964 Alaskan and the 1960 Chilean earthquakes. At the time, the tsunamigenic potential of the Cascadia subduction zone had not been recognized (Geist, 1998; Satake et al., 2003). Based on these data, they approximated the initial ground deformation by a hypothetical uplift mass of ellipsoidal shape, about 600 miles long with an aspect ratio of 1:5 and maximum vertical uplift of 25–33 feet. They then divided the Aleutian trench into 12 segments and calculated the wave evolution from each segment, and repeated the procedure for tsunamis from the Peru-Chile trench.

It is important to note that, for their time, the methodology used by Houston and Garcia (1974) was ground breaking, not only in its scope, but also in the combined use of analytical and numerical methods. Houston and Garcia (1974) first solved a linear-form spherical long wave and then propagated the tsunami from the source to the edge of the continental shelf, by using a finite difference model; at the continental shelf, they derived an analytic expression to match the outer and inner wave amplitudes, and then they used that expression to derive a simple amplification factor for a sinusoidal tsunami. Even though they did not match the slope of the water surfaces in the inner and outer continental shelf regions, their results compared extremely well with measurements from tidal gauge records, whenever suitable tidal were available that did not need additional signal processing to filter harbor resonance effects.

The good comparisons with tidal gauge data for the 1964 Alaskan event provided encouragement for the extrapolation of the results for nearfield events, despite Houston and Garcia's (1974) own disclaimers, thereby masking three important aspects of the "inundation" calculations used: 1) Nearfield events are "extremely dynamic in three dimensions," and for this reason the methodology used for farfield events may not be appropriate for nearshore quakes. 2) In the mid-1980s it became apparent that superposition of sinusoids is not as straightforward as had been previously assumed; the reason is that during the reflection process (not accounted for in Houston and Garcia's calculation) a phase lag is introduced that is frequency dependent (Synolakis, 1987; Liu et al., 1991). 3) Comparisons of numerical model predictions with data from the field surveys of the 1992–1996 tsunamis suggested that even small-scale nearshore features can influence inundation to first order, casting doubt on predictions from coarse grid computations because they may miss extreme events.

Houston performed another comprehensive study of tsunami predictions in California in a series of two reports (Houston and Garcia, 1978; Houston, 1980). By this time, numerical solutions of the shallow-water wave equations had begun to be available; they solved, using finite difference algorithms, the nonlinear form of the shallow-water (NSW) wave equations including frictional terms. Still, because of computational power limitations, no runup was evaluated. Instead, computations treated the shoreline as a vertical wall. Houston (1980) noted that the runup elevations (i.e., the elevation of the maximum inland penetration of the tsunami) may not equal shoreline elevations at locations where dunes prevent flooding, or if the land is flat, where inland flooding maybe extensive. As has been shown by Titov (1997) and Titov and Synolakis (1997, 1998), wall-type calculations (as shown in Figure 5) not only underpredict the runup substantially, but may also miss extreme events. Although the degree of underprediction varies with the local topography, it is often a factor of two and sometimes it has been reported as large as five.

Even though these newer computations were a substantial improvement over the 1974 study, it is important again to revisit the computational assumptions used to arrive at these 1978 and 1980 predictions:

1. Only farfield events from Alaska and South America were considered because the potential of the Cascadia Subduction Zone had not been recognized at the time.

2. Faults were modeled simply as an uplift of the seafloor.

3. The entire Pacific Ocean was modeled as a 1,600-foot constant-depth basin with a 2-mile-square grid. In the nearshore region, the bathymetry was also modeled with a 2-mile-grid (i.e., no coastal topography was included).

4. The computational boundary was a vertical wall at the shoreline (i.e., there were no inundation computations).

5. No values were provided for tsunami currents.

To evaluate the possible effects of these limitations, the Focused Study Group looked at results from field measurements and modeling from the 15 tsunamis in the 1992–2003 period. The contemporaneous field surveys that were initiated following every event have provided a high-quality data set with actual inundation measurements. They have thus allowed not only the assessment of earlier assumptions, but also the means to visualize what the impact of even small tsunamis can be. Some of the conclusions from review of the field surveys in the past 15 years are summarized below.

1. Small-scale coastal features have first-order effect on runup, particularly for extreme events. To obtain quantitative agreement with measurements, computational grids of less than 150 feet are needed.

2. Excellent predictions for fault movements now exist. Comparisons with field data (for nearfield events only) suggest that, for first-order accuracy, the details of the three-dimensional fault displacement on the seafloor need to be incorporated in the model (Geist, 2002).

3. Threshold-type computations with vertical boundaries at some reference offshore depth or at the shoreline, as used earlier, may underpredict measurements by factors up to five.

4. Overland flow is a significant hazard, as demonstrated in Wurhing during the 1992 Flores event, or in Sissano, during the 1998 Papua New Guinea tsunami. Residents of narrow

spits of land, such as in Eureka or Coronado Island in San Diego, California, are at higher risk, and extreme care is needed to evaluate inundation areas in such locales.

5. Tsunami currents are important in defining inundation zones. Even small tsunamis can generate large currents.

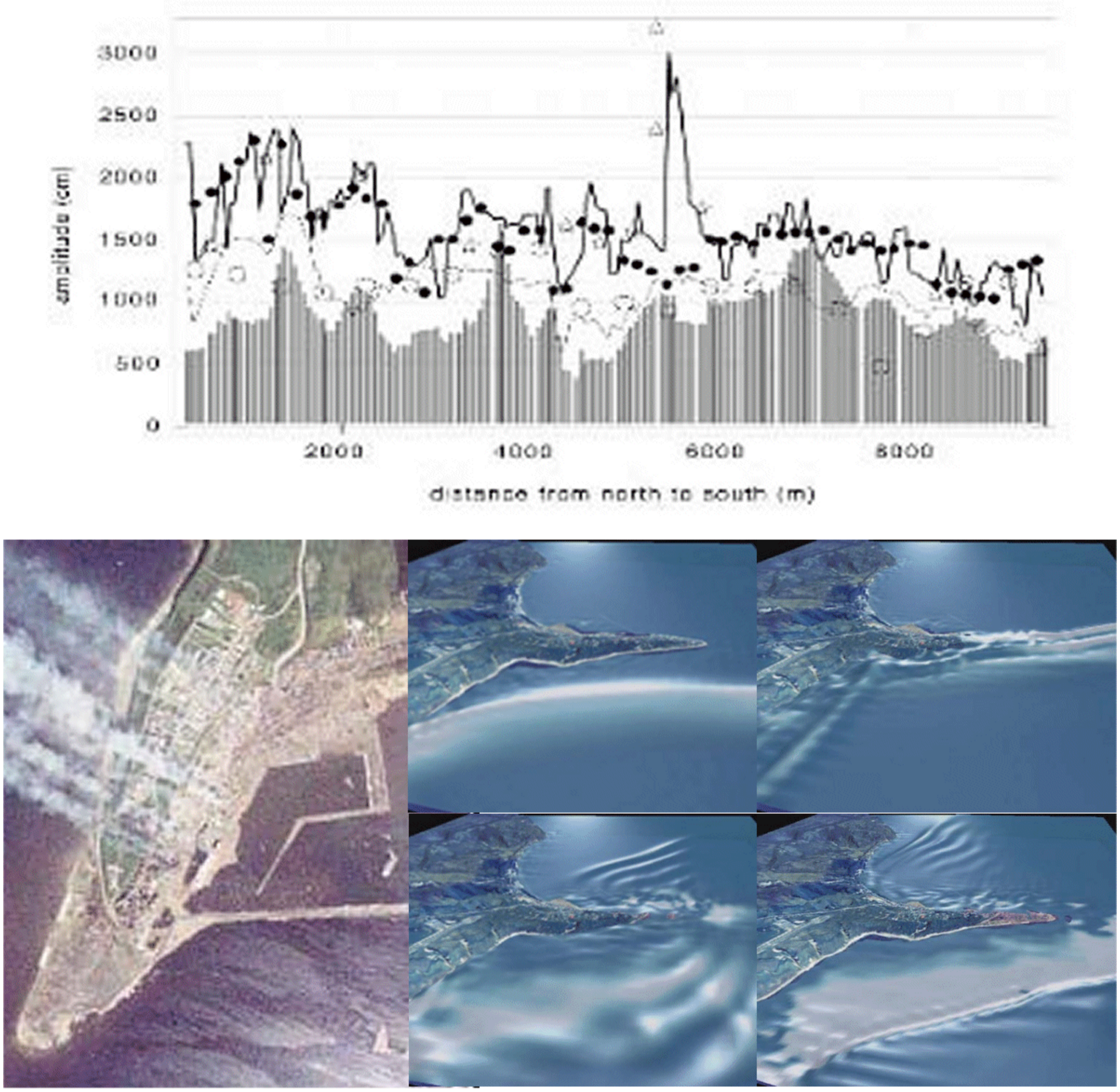


**Figure 5. The lower left panel shows the damage at the town of Aonae at the southern tip of Okushiri Island, Japan, during the 1993 Hokkaido-Nansei-Oki tsunami.**

*The photo was taken the next morning, and fires are still smoldering. Notice the complete devastation of the eastern tip of the peninsula. The lower right panel shows numerical simulations from MOST that accurately display the overland flow that destroyed Aonae. Notice how the wave approaches (upper right), flows over (upper left), and then reflects back (lower right). The upper panel shows a comparison of results from numerical predictions with actual measurements. The solid vertical lines (1) are the predictions of a threshold model such that used by Houston and Garcia (1974) and then by Houston (1978). The empty circles (□) show model predictions at a resolution of 1,600 feet, the solid circles ( ) at 800 feet, and the solid line (–) at a resolution of 1,000 feet or less. The stars indicate the field data. Notice that only at a resolution of 100 feet it is possible to capture the extreme runup, which reached more than 100 feet. The threshold models appear inadequate for modeling extreme runup, which is exactly when accuracy is most needed. Results from Titov and Synolakis (1997, 1998).*

The recent findings stated above are a cause of concern. In view of the large differences with the inundation predictions from nearfield events, it is important to assess what improvements can be made in the existing methodology to determine an inundation line with 100-year return period using the best available technology, in terms of geologic, geophysical, and hydrodynamic computational resources.

### 2.2.2 Alternatives for Improvement

Four preliminary alternatives are articulated, in order of increasing preferences:

A. Do nothing for tsunami hazard assessment and rely on local, state, and NOAA programs to map tsunami risks. For instance, a recent Flood Insurance Study on Sandy Point, Washington, did not include a tsunami hazard assessment.

B. Maintain the same tsunami-related risk assessment in the coastal areas for which Houston developed runup frequency elevations. Update the method used by Houston only for farfield events.

C. As an interim procedure, consider separating tsunami risks from storm-induced risks. Designate a separate hazard zone for tsunami risks. Use NOAA's Tsunami Inundation Map boundaries as hazard zone limits.

D. As a longer term procedure, develop a methodology for comprehensive tsunami hazard assessment for farfield and nearfield events triggered by earthquakes, both offshore and onshore. This includes the analysis of tide gage data, when available, to determine the tsunami anomalies along the Pacific Coast and transfer these results to open coast areas of interest. The tsunami events listed in Tables 1 and 2 will serve as a basis for searching the tide gage data for tsunami anomalies.

#### *Approach A*

Approach A ignores the risk altogether and is not advisable.

#### *Approach B*

Approach B maintains the status quo of hazard identification of effective FIRMs, with tsunami hazard identified, as the new FIRMs under FEMA's Map Modernization Program will also include same type of farfield tsunami hazards as the currently effective FIRM. In this approach, necessary modifications should be applied to the Houston method in terms of event description and numerical models. This approach is better than the first because it incorporates the farfield tsunami hazard; however, it does not incorporate comprehensive tsunami hazard analysis involving both farfield and nearfield events and is based on earlier results that differ substantially with recent finding.

### Approach C

Approach C would involve identifying a tsunami risk hazard zone as the maximum credible tsunami inundation. Although this approach would show a hazard zone, NOAA's current tsunami inundation maps would not serve the actuarial purpose of a FIRM. This approach is analogous to creating a FIRM for maximum possible flood rather than the one-percent-annual chance flood for a riverine hazard. However, if return periods can be assigned to NOAA's tsunami inundation maps, actuarial rates may be worked out separately for the tsunami hazard. In this approach, the NFIP would need to change its regulations to redefine coastal high-hazard zones and add new regulations for tsunami hazard zones.

The argument for designating a tsunami hazard zone merits some investigation because the damage potential of tsunami waves may be much greater than that of storm waves, for which the current guidelines were developed. The rationale for a new tsunami hazard zone is as follows:

1. The definition of V-Zone is based on at least a 3-foot breaking wave during a 100-year storm event. This definition should be reevaluated for tsunami waves because tsunamis are associated with large velocities. Further, empirical relationships often used to associate velocities with wave heights may not be applicable for tsunamis.

2. Current V-zones frequently exclude adjoining areas with virtually indistinguishable hazard characteristics. It is probable that damages from a tsunami would extend beyond a V-Zone limit line because tsunamis are known to induce debris and fire damage in adjacent areas. Statistical combination of tsunami-induced runup with storm-induced runup may mask the actual risk posed by tsunamis because damage may extend beyond mathematically computed elevations and zones.

3. Floodplain management regulations may need to be reevaluated for tsunami-prone coastal communities under the NFIP to reflect appropriate precautions necessary for reducing damages from a tsunami. For new construction and substantial improvement within V-Zones, 44 CFR 60.3 provides certain minimum standards, which may have been developed based on hurricane-induced conditions of the Atlantic and Gulf coasts. Relevant regulations, particularly those under Section 60.3(e), need to be reevaluated for tsunami-prone areas. Some criteria may be developed under the Community Rating System for rewarding "tsunami-ready" communities that undertake precautionary steps beyond the minimum necessary under the NFIP. These evaluations should include conditions from all tsunami-prone states, including Alaska and Hawaii.

4. Because tsunami events are infrequent, the insurance rate structure in a tsunami hazard zone may be less costly than that for the more frequent coastal storm events, which include El Ninos.

5. Designation of a different hazard zone for apparently similar flooding scenarios is not new to the NFIP. In riverine situations, separate zone designations are available for alluvial fans with high velocity debris hazards.

Approach C3 is better than the first two because it includes nearfield events, at least in California, where these events are known to be more hazardous than farfield events. The definition of a new coastal hazard zone for tsunami may be more time consuming because it involves changes to federal regulations. To have the effective regulations ready for use in the Pacific Coast flood studies under the Map Modernization Program, activities leading to tsunami-related floodplain management regulations should start soon. The weakness of adopting NOAA's inundation maps is that the hazards they address are not comprehensive; for instance, they do not include farfield events for California or nearfield events in Alaska.

***Approach D***

Approach D would attempt to incorporate comprehensive tsunami hazard assessment for nearfield and farfield events. The Probabilistic Tsunami Hazard Analysis (PTHA) framework (for example, as proposed by Downes et. al.,2001) can be adopted with some modifications. The framework this Focused Study Group proposes consists of four steps:

1. Identify farfield and nearfield sources: The sources might be submarine landslides and fault sources. State-of-the-art model (Geist, 2002) would be used to predict initial conditions for the numerical computations.

2. Estimate recurrence interval: In this step, recurrence interval of seafloor displacement and landslide volumes would be estimated. This step produces the initial condition for the propagation model and is deemed to be the most formidable step in the process because of the large uncertainties in the underlying rupture process of fault or landslide.

3. Develop runup (or wave height) frequency curves. These can be developed for the shorelines by using numerical models that account for wave transformation from each source (i.e., faults and landslides).

4. Combine runup frequency curves. These can be generated by statistically combining all runup frequency curves developed in Step 3.

In computing recurrence intervals for infrequent events such as tsunamis, one can also consider conditional probabilities and this approach may be the most realistic for tsunami hazard assessment as it represents real risk from infrequent events. For the Pacific Northwest, Priest (2001) argues that "with the known condition that 301 years of strain has already accumulated on the subduction zone, the conditional probability of a recurrence in the next 100 years is much higher than a random event with a 400-600 year recurrence." This approach is a departure from traditional hazard assessment, which is based on purely random events.

Although there are difficulties and uncertainties with recurrence interval estimates, given the lack of sufficient data, groups of scientists in academia, research institutions, and federal agencies are actively working on this problem (Bardet et al., 2003). Most notable are the USGS, the University of Southern California, Northwestern University, and Lawrence Livermore Laboratory. A study can be undertaken comprising national and international tsunami research scientists to assess the state of the art and associated uncertainties of recurrence interval prediction for tsunami sources along the Pacific Coast for California, Oregon, and Washington.

Approach D could take several to many years to perfect, and it is recommended that FEMA remain an active partner with other federal agencies and research institutions and help set the goals and objectives of the PTHA.

### 2.2.3 Recommendations

The Focused Study Group believes that the Approach D is the only defensible approach to tsunami hazard assessment and that a probabilistic hazard assessment can be performed in 2–4 years for all Pacific states. However, before embarking on a large-scale up/down–coast analysis, it is recommended that a focused study be performed within a 6-month period to redo the existing simulations for farfield events done 25 years ago. Tsunami modeling technology has evolved rapidly, particularly in the last ten years, and comparison of new simulations with the 25-year-old results would allow evaluation of the margin of error of the hydrodynamic predictions. Depending on the results, the Focused Study Group will recommend to FEMA a larger followup study to address the needs of all five Pacific states and possibly the Atlantic and Gulf states as well.

The Focused Study Group believes that a good candidate for this interim pilot study is Santa Monica Bay in Southern California. Not only does it have the largest population density on the west coast, but on a Sunday afternoon in the summer months, hundreds of thousands of people are at risk. The property values are some of the highest along the Pacific Coast, and the pilot study results would be extremely useful in local emergency preparedness. However, from the epistemological point of view, any locale at risk in Washington, Oregon, or California would serve the purpose of the pilot study.

It is noted that this pilot study would be useful under either Approach C or D. Even if a probabilistic hazard study is not conducted, the pilot study would determine a better 100-year-return inundation line by improving the farfield estimates of Houston and Garcia (1974, 1978). If, for example, Approach C is used and a different hazard zone defined, the limits of this zone would be calculated using contemporary inundation technology, eliminating most of the significant known limitations of the earlier maps.

Inundation modeling for hazard assessment, based on tsunami source probabilities, will exploit the advanced numerical modeling technology and essential infrastructure developed and maintained in each state by the NTHMP. It will employ NOAA’s model MOST. Organization and planning of this multiyear effort in an efficient and cost-effective way will be a primary objective of the FEMA/NTHMP working group.

NOAA's historical database requires extensive quality control and expansion, including systematic historical research to assess and characterize the nature of tsunami-event sources. Similarly, to identify and characterize additional tsunami events and the nature of their sources, the prehistoric database requires an accelerated effort to acquire and analyze field measurements. Characterization of tsunami events will include estimates of inundation and tsunami currents based on field and laboratory measurements of tsunami deposits and on tsunami sediment transport modeling. This collaborative FEMA/NTHMP effort to assess tsunami hazards will clearly advance the primary goals of each agency–those of FEMA related to insurance rates, those of the NTHMP related to emergency management, and those of both agencies related to mitigation.

Specific goals of this pilot study for a selected locale in Washington, Oregon, or California are the following:

1. Significantly improve estimates of recurrence interval by increasing the coverage and quality of the historic and prehistoric tsunami record. Analyze farfield sources in Alaska and South America and regional sources in Cascadia.

2. Develop probability distributions for tsunamigenic earthquake and landslide sources.

3. Conduct inundation modeling to evaluate the consequences of events from the appropriate sources.

4. Determine the 100-year-recurrence line for communities in one selected locale as a pilot study.

### 2.2.4 Preliminary Time Estimate for Guideline Improvement Preparation

Accelerated research and development focused on joint NFIP/NTHMP goals will require supplementary support for USGS, NOAA, and academic efforts. These efforts will include (a) significant improvement of database coverage and quality, (b) development of probability distributions, (c) tsunami modeling, and (d) FEMA/NTHMP Working Group activities. Interagency discussion and agreement will identify the level and sources of both new and matching in-kind support required for this effort.

# 3 IMPORTANT TOPICS SECTION

## 3.1 TOPICS 20 AND 29: TSUNAMI STRUCTURE–DEBRIS INTERACTION TO DEFINE HAZARD ZONES AND TSUNAMI-INDUCED EROSION.

Debris impact causes the greatest amount of structural damage during tsunami attack, at least for tsunamis with overland flow depths less than 10 feet. As the tsunami evolves in the terminal stages of upwash up a beach, it transports debris such as logs, cars, and the remains of coastal structures, which then become waterborne missiles. An example of waterborne debris damage is shown in Figure 6. An example of tsunami-induced erosion is shown in Figure 7. No guidelines exist for calculating forces from the impact of tsunami-borne debris impact on structures. A comprehensive review of tsunami-debris interaction is recommended, along with a preliminary assessment of the adequacy of guidelines published in FEMA’s Coastal Construction Manual (CCM).

### 3.1.1 Description of the Topic and Suggested Improvement

Tsunamis can generate large onshore currents and move large objects far inland. Historic examples from large tsunamis abound. The most notorious is the report of the U.S. Navy ship Watery, which was moved by the 1868 Arica, Chile tsunami two miles inland and then moved back to shore during the 1877 Arica tsunami so that the ship could sail on. A measure of what even small tsunamis can do is the 1994 Mindoro, Philippines tsunami (Imamura et al., 1995). In an area where the vertical inundation heights did not exceed 10 feet, the generated tsunamis floated a 6,000-ton power-generating barge, broke its mooring lines, and carried it one mile inland down the Baryan river. Impact forces can cause collapse of coastal structures; an excellent visualization of the process can be observed in detail in the videotape Discovery's Tidal Wave (1998). The estimation of impact forces and currents is still an art and far less well understood than hydrodynamic evolution and inundation computations. In what follows, different methods and formulae are described in the literature, although none has been truly validated by comparisons with field data.

Existing analyses extend only so far as suggesting methods for calculating forces on piles and impact forces on seawalls and structures with provisions available for breaking wave loads. No methods exist for calculating debris impact, beyond the suggestions provided in the CCM, which were derived from results from steady flows. The results are discussed in detail in Synolakis (2003).

Tsunami forces on piles are usually calculated in accordance with the classic work of Dean and Harleman (1966), as the sum of a drag and added mass terms developed for periodic waves. To date, no equivalent published analysis exists for transient waves. Research work is under way in a National Science Foundation Collaborative Research study with the University of Washington, Cornell, Southern Methodist University, and the University of Southern California. Preliminary results exist only for solitary waves.

**Figure 6. An example of impact from waterborne debris in Hilo, Hawaii, from the attack of the tsunami triggered by the 1946 Aleutian earthquake, the same tsunami whose impact in Scotch Cap is shown in Figure 3.**

**Figure 7. Rajekwesi, East Java, after the 1994 tsunami.**

*The tsunami eroded a strip about 100 feet wide off the shoreline. The damage seen is from the tsunami; there was no earthquake ground shaking, as this was a slow offshore earthquake. The tsunami penetrated more than 1/2 mile inland. Its height here was estimated (from watermarks inside the only surviving structure) as 12 feet. (Imamura et al., 1995).*

For tsunami forces on seawalls, the existing methodology follows the classic work of Cross (1967). Ramsden and Raichlen (1990) and Ramsden (1993) used Cross's formulation and results from laboratory experiments to calculate the forces of an impinging bore on a vertical seawall. They cautioned against extrapolating their specific results for walls of finite height and horizontal extent.

No published results exist for tsunami loads on rectangular structures. To calculate the hydrodynamic load on a rectangular structure, the CCM recommends using an equivalent "dynamic" flow depth, $d_{dyn}$, calculated from

$$d_{dyn} = (1/2g)\ C_d\ V^2 \qquad (1)$$

where $C_d$ is the drag coefficient, g is the acceleration due to gravity, and V is the velocity. The CCM recommends that $V^2 = 4gd_s$ where $d_s$ is the design flow elevation (DFE).. When the velocity $V < 10$fps, $d_{dyn} = 2\ C_d\ d_s$. While it is unclear how $C_d$ or $d_s$ are to be calculated for a highly transient wave, the velocity of which varies rapidly as it evolves up a shoreline, it is presumed that one would use the maximum flow velocity calculated from the hydrodynamic computational models at the locale of interest. Then the hydrodynamic force is given by

$$F = \rho g\ d_s\ d_{dyn}\ w \tag{2}$$

in which $\rho$ is the water density and w is the structure width. The drag coefficient depends on the relative ratio of width of the structure w to the DFE depth $d_s$ at the front of the structure.

For overland flow velocities $V > 10$fps, the CCM recommends that the force be calculated by

$$F = (1/2)\ \rho\ C_d\ V^2\ A \tag{3}$$

in which A is the surface area of the structure normal to the flow.

The CCM describes a methodology for calculating debris impact forces through the calculation of the impact load $F_p$, given by

$$F_p = W\ V\ /\ gt \tag{4}$$

where W is the weight of the object impacting the structure, V is its velocity, g is the acceleration of gravity, and t is the duration of impact. The formula in essence calculates the impulse force. The CCM recommends that, in the absence of any criteria, one use w=1,000lb with $V=(g\ d_s)^{1/2}$. Assuming that one is careful with the calculation of the DFE, the formula might produce a conservative estimate. One issue that needs clarification is the calculation of the local DFE. As the wave evolves up on dry land, its velocity is not simply related to the square root of the gravitational acceleration times the depth, which is simply the long wave velocity. During tsunami attack, the velocity in the runup zone can be as high as 10 fps, yet the local depth might be smaller than 1 foot.

There are no existing guidelines for erosion due to tsunamis. Scientists from the USGS have performed comprehensive surveys of several historic tsunamis and of most 1992–2002 events. A large amount of data has been accumulated on erosion and deposition during tsunami attack, but the data have yet to be translated into standards and guidelines for engineered structures. While there are many studies of scour around cylindrical piers for steady flows and combinations of steady flows and waves, the Focused Study Group identified only one for tsunamis, by Tonkin et al. (2003). It describes a laboratory experiment with erosion from solitary waves attacking a circular cylinder. The study concluded that the time scale of the tsunami attack is critical in the scouring process. However, given the well-established difficulties with extrapolating sediment scouring experiments from small-scale laboratory measurements to the prototype, it is not possible to draw conclusions from this study

### 3.1.2 Description of Potential Alternatives

There are three preliminary alternatives to mitigating the hazard:

A. Do nothing and rely on the CCM recommendations and on existing guidelines for evaluating coastal structures.

B. Do a preliminary study to determine how relevant and appropriate the CCM recommendations are for tsunamis, in view of recent field observations.

C. Develop standards and guidelines for calculating debris impact loads on structures during tsunami attack.

D. Perform an interim study to determine whether sufficient data exist to develop empirical guidelines for tsunami scour and to suggest a methodology for calculations.

Despite the fact that there are no published studies disputing the CCM recommendations for tsunamis, even casual examination of tsunami damage photos reveals that tsunami impact is a substantial hazard, even for tsunamis of small height. For this reason, Approach A is not recommended.

Approach B involves calculating specific tsunami heights and velocities during tsunami attack on actual coastlines. Numerical models such as MOST calculate velocities in both propagation directions as the tsunami evolves, in addition to the local flow depth. Note that the latter is not known a priori, when the wave advances on dry land. For example, the tip of the advancing wave has very small depth but very high velocity. As the wave reaches its maximum, the flow depths over the inundation area are the largest, but the velocities are smaller than their extreme values. It is possible to obtain numerical data, then compare different methodologies for evaluating the local DFE and velocities, and compare them with the CCM recommendations.

Approach C involves resorting to both numerical modeling and large-scale experiments to determine debris impact forces from tsunamis. Both the Network for Earthquake Engineering Simulation and the Engineer Research and Development Center (ERDC, formerly CERC) basins, have directional wave spectrum generators that have the capability to generate large-scale transient waves. Experiments in these facilities can help validate numerical results on impact forces.

In terms of tsunami scour, it is recommended that an interim study be performed that computes predictions using existing methodology for scour from steady flows using the recent laboratory data and existing field measurements.

The study group believes that the second approach is the most advisable and feasible as an interim measure. If results for force calculations using flow depths and velocities from numerical models differ substantially from the CCM results or are not consistent with existing guidelines for evaluating coastal structures for wind waves, then the Study Group will recommend to FEMA a suggested methodology.

### 3.1.3 Preliminary Time Estimate for Guideline Improvement Preparation

If this preliminary assessment is undertaken in the context of the interim analysis to develop a 100-year-recurrence line for a specific area, as outlined in Section 2.2, numerical modeling results will be readily available to perform a preliminary assessment of the CCM guidelines as they pertain to tsunami debris impact forces. The additional effort involves only comparison of results with the values derived from the methodology in the CCM.

In terms of tsunami-induced scour, it is recommended that the preliminary studies of this Focused Study Group be expanded with the help of the USGS, which has substantial experience in evaluating tsunami erosion and deposition (Jaffe and Gelfenbaum, 2002; Gelfenbaum and Jaffe, 2003). Again, in the context of a pilot study, the USGS could recommend a cohesive approach.

## 4 SUMMARY

This Focused Study had four items to consider that also involve the NTHMP's products and approaches. These are Task 15, to address use of the NTHMP products into FEMA FIRMs; Task 16, to evaluate the possibility of performing probabilistic tsunami hazard analyses to better define the NTHMP hazard zone lines: Task 20, to develop new hazard zones for tsunamis-born debris impact; and Task 29, to assess tsunami induced erosion. Of the four, the first two items have been treated as critical, the third and fourth as important.

The main findings of the study are:

- FEMA's current maps use 25-year-old methodology and only consider farfield tsunami events. They do not reflect hazards from nearfield tsunami events, which may be more dominant in Washington, Oregon, and California. Further, inundation mapping technology did not exist 25 years ago and the potential of the Cascadia Subduction Zone had not been recognized; hence, there are significant known limitations in the earlier projections.

- Current NFIP regulation defines Coastal high-hazard zones as inclusive of both storm wave and tsunami wave hazards. However, since the hazards posed by tsunamis are very different temporally and spatially from storm-related hazards, a case can be made for defining a new hazard zone for tsunamis.

- NTHMP inundation maps use state-specific sources and represent hazard zones of varying recurrence intervals across the five Pacific states. They represent worst credible scenarios, with unspecified return periods. None combine nearfield and farfield events or multiple hazards. Hence, these maps cannot be used in NFIP as they currently exist. Yet, the existing maps would be very useful to the NFIP if return periods could be estimated and included.

- Comprehensive probabilistic tsunami hazard analysis, which includes farfield and nearfield events from both offshore and onshore sources, is the only way for reliable and cross-hazard consistent tsunami risk assessments in NFIP. However, suitable methods have not yet been fully developed for the idiosyncrasies of the Pacific Coast.

This Focused Study Group strongly recommends an integrated, interdisciplinary, and highly focused six-month pilot study to define the national problem of tsunami flooding, forces, and erosion, by carefully examining the limitations of existing NTHMP and NFIP tools, in the context of evaluating the hazards in one specific locale in Washington, Oregon, or California. The proposed study will demonstrate the need and methods for national implementation, which we believe can be accomplished within 2–4 years. Specific tasks for the initial six-month period (to September 30, 2004) are the following six.

1. Significant improvement of recurrence interval estimates by increasing the coverage and quality of the historic and prehistoric tsunami record. Analysis of farfield sources in Alaska and South America and regional sources in Cascadia, and inclusion of nearfield sources. Estimation of credible probabilities for nearfield events.

2. Development of probability distributions for both tsunamigenic earthquake and landslide sources.

3. Inundation modeling to evaluate the consequences of the generated tsunamis for relevant geologic sources for the locale under study.

4. Determination of 100-year recurrence line in one selected locale as a pilot study. If appropriate storm hazard data are available, these will be included because NFIP guidelines allow for combining runup estimates from both storm and seismic sources. Other possible tsunami hazard zone delineations based on both water depth and velocity will be investigated for their potential to improve hazard assessment.

Inundation modeling for hazard assessment, based on tsunami source probabilities, will exploit the advanced numerical modeling technology and essential infrastructure developed and maintained in each state by the NTHMP. It will employ NOAA's model MOST and the state-of-the-art fault models recently developed by USGS to provide estimates of source motions and initial conditions. Efficient and cost-effective organization and planning of this multiyear effort is a primary objective of this FEMA/NTHMP Working Group.

At the conclusion of this pilot study, FEMA will have a methodology that meets its present 100-year-return criteria. FEMA will also have a more realistic cost estimate for properly incorporating tsunamis with other flooding hazards.

This collaborative FEMA/NTHMP effort to assess tsunami hazards will clearly advance the primary goals of each agency–those of FEMA related to insurance rates; those of the NTHMP related to emergency management, and those of both agencies related to mitigation, saving lives, and protecting property.

**Table 3. Summary of Findings and Recommendations for Tsunami Hazards**

| Topic Number | Topic | Coastal Area | Priority Class | Availability / Adequacy | Recommended Approach | Related Topics |
|---|---|---|---|---|---|---|
| 15 | NTHMP | AC | H | -- | The recommended approach includes: (1) develop digital database, and (2) develop a methodology, including recurrence interval estimation, for use of NTHMP products for NFIP for tsunami hazard zone delineation. | 16, 20, 29 |
| | | GC | H | -- | | |
| | | PC | C | MIN | | |
| | | SW | C | MIN | | |
| 16 | 100-year Recurrence Develop method to predict 100-year tsunami events | AC | H | -- | The recommended approach is to perform a comprehensive probabilistic tsunami hazard assessment at a pilot site in California or Oregon or Washington: (1) recurrence interval estimate of forcing functions (2) propagation of tsunamis from Alaska, Chile, Cascadia Subduction Zones; (3) inundation calculations, (4) probability distributions and integration of hazards, (5) Include the analysis of tide gage data, when available, to determine the tsunami anomalies along the Pacific Coast and transfer these results to open coast areas of interest. | 15, 20, 29 |
| | | GC | H | -- | | |
| | | PC | C | MAJ | | |
| | | SW | C | MAJ | | |
| 20 | Structure-Debris Interaction | P | I | PRODAT | Evaluation of Coastal Construction Manual recommendations for impact forces using data for overland flow depths and velocities for the numerical simulations from Topics 15 and 16 for one specific locale | 15, 16 |
| 29 | Erosion | SW | I | PRO | Evaluation and integration of USGS data into empirical relationships for the specific locale under study | |

| Table 4. Preliminary Time Estimate for Guideline Improvement Preparation | | |
|---|---|---|
| Topic Number | Item | Time |
| 15 | Digital database development | 3 months for one NOAA support scientist |
| | Recurrence interval development | 3 months for one USGS scientist |
| | Develop methodology, FEMA, NOAA, USGS and consultants | 3 months for NOAA and USGS scientists, one month for consultants |
| 16 | Recurrence Interval Estimate | 2 months of one NOAA and one USGS scientists, consultants. |
| | Propagation of tsunamis from Alaska, Chile, CSZ | 6 months of one NOAA scientist, consultants and/or USGS |
| | Inundation calculations | 3 months of one NOAA scientist, 3 months of consultants |
| | Probability distributions and integration | 2 months of one NOAA scientist, 2 months of a USGS and 1 month for consultants |
| 20, 29 | Evaluation of CCM recommendations for impact forces using data for overland flow depths and velocities of the numerical simulations from Topics 15 and 16 for one specific locale. | 1 month of combined NOAA and consultants. |
| | Evaluation and integration of USGS data into empirical relationships for the specific locale under study. | 4 months of combined USGS and consultants. |

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# Sheltered Waters

## FEMA Coastal Flood Hazard Analysis and Mapping Guidelines Focused Study Report

February 2005


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**Acronyms**

| | |
|---|---|
| ACES | Automated Coastal Engineering Systems |
| CTP | Coordinating Technical Partners |
| DFIRM | Digital Flood Insurance Rate Map |
| FIS | Flood Insurance Study |
| FIT | Flood Information Tool |
| MAS | Mapping Activity Statement |
| NFIP | National Flood Insurance Program |

# 1 INTRODUCTION

This report provides recommendations for a program leading to improvement of the current FEMA Guidelines related to Sheltered Waters Flood Hazards and develops preliminary time estimates to accomplish these improvements. One sheltered water category was developed at the December 2003 Workshop. This category was labeled “Critical” for the Pacific and Non-Open Coast and “Helpful” for the Atlantic and Gulf Coast.

This was the only focused study category initially established with one related topic; however, the nature of this focused study cuts across many other focused studies, as indicated by the number of ratings in the Non-Open Coast column of the original focused study table. Therefore, this single category has been subdivided into additional topics (needs). The topics either provide detail for recommendations not captured under other focused studies or provide recommendations to refine open coast guidance prepared by other focused study groups, for application to sheltered water studies. The topics were further refined during the February 2004 Workshop. The original topics are shown below.

**Sheltered Waters Topics and Priorities**

| Topic Number | Topic | Topic Description | Priority | | |
|---|---|---|---|---|---|
| | | | Atlantic / Gulf Coast | Pacific Coast | Non-Open Coast |
| 6a | Definitions and Classification | Provide definitions and develop a classification method for sheltered water studies as a framework for an approach | H | C | C |
| 6b | Historical Information | Prepare guidance for reconstructing historic sheltered water flood conditions to validate flood study results | H | C | C |
| 6c | Peer Input | Seek peer input on new sheltered water guidelines needs [Deleted during Workshop 2] | -- | -- | -- |
| 6d | 1% Annual Chance Flood Elevations | Prepare guidance for defining the 1-percent-chance flood event in sheltered water areas | H | C | C |
| 6e | Stillwater Elevations and Tidal Currents | Prepare guidance for estimating stillwater elevations and currents in sheltered water areas | H | C | C |
| 6f | Coastal Structures | Prepare guidance for evaluating coastal flood protection structures in sheltered water [Moved to Topic 21a during Workshop 2] | -- | -- | -- |
| 6g | Hazard Zones | Prepare guidance for identifying flood insurance risk zones in sheltered water [Moved to Topic 17 during Workshop 2] | -- | -- | -- |
| 6h | Interrelationships | Coordinate the preparation of sheltered water guidelines with other Map Modernization objectives and multi-hazard planning initiatives | H | C | C |
| Key: C = critical; A = available; I = important; H = helpful | | | | | |

The preparation of guidance for many of the above topics will be coordinated with other appropriate focused study groups because of the interrelationship of these topics with similar topics found in other focused studies. The purpose of addressing these topics here in the Sheltered Waters Focused Study is to document aspects of the work that may have more relevance to sheltered water flood studies, as opposed to open coast studies.

### 1.1 SHELTERED WATERS FOCUSED STUDY GROUP AND APPROACH

The Sheltered Waters Focused Study group consisted of: Jeff Johnson, David Divoky, Darryl Hatheway, Ron Noble, and Kevin Coulton who served as Team Leader for this effort.

To provide structure to the team efforts and to avoid unnecessary duplication of work, the following approach was used: the Team Leader developed background material, reviewed available information, and developed draft write-ups, which were then distributed to the Team. All Team Members contributed information of which they were uniquely aware, critiqued and contributed to the draft write ups and accomplished specific components of the overall effort leading to this report.

### 1.2 CURRENT FEMA GUIDANCE ON SHELTERED WATER FLOOD HAZARDS

Sheltered waters are water bodies with shorelines that are not subjected to the direct action of undiminished ocean waves. Although similar processes contribute to flooding in sheltered water shorelines as along open coastlines, such as wave setup, runup, and overtopping, there are several aspects of sheltered water flood hazards not addressed in the current FEMA Guidelines. For example, wind-generated waves are highly dependent on the shape and orientation of the surrounding terrain to prevailing wind directions (Figure 1).

Wave generation and transformation in sheltered waters are typically limited by an open water fetch distance, complex bathymetry, and often the presence of structures. See the Storm Wave Characteristics Focus Study Report for more information about seas in sheltered waters.

A sheltering effect typically reduces wave energy and flood potential compared to open coast areas; however, wave runup and overtopping along sheltered water shorelines may present additional hazards from wave-cast debris and backshore flooding because insured buildings are often located closer to the shoreline than along the open coast. Hazard evaluations for these and other characteristics of sheltered waters require special methods not available in the Guidelines.

The existing Guidelines (FEMA, 2003) are generally written to provide guidance for coastal flood studies along the open coasts of the Atlantic Ocean and Gulf of Mexico. Several references to sheltered water areas are made in these Guidelines, but detailed guidance is not provided. The Great Lakes are very large sheltered water bodies and guidelines (Great Lakes *G&S* in *Appendix D*) are provided for flood studies; however, these guidelines are very specific to the storm meteorology and physiography of the Great Lakes and are assumed to not be applicable to Pacific Ocean sheltered water areas.

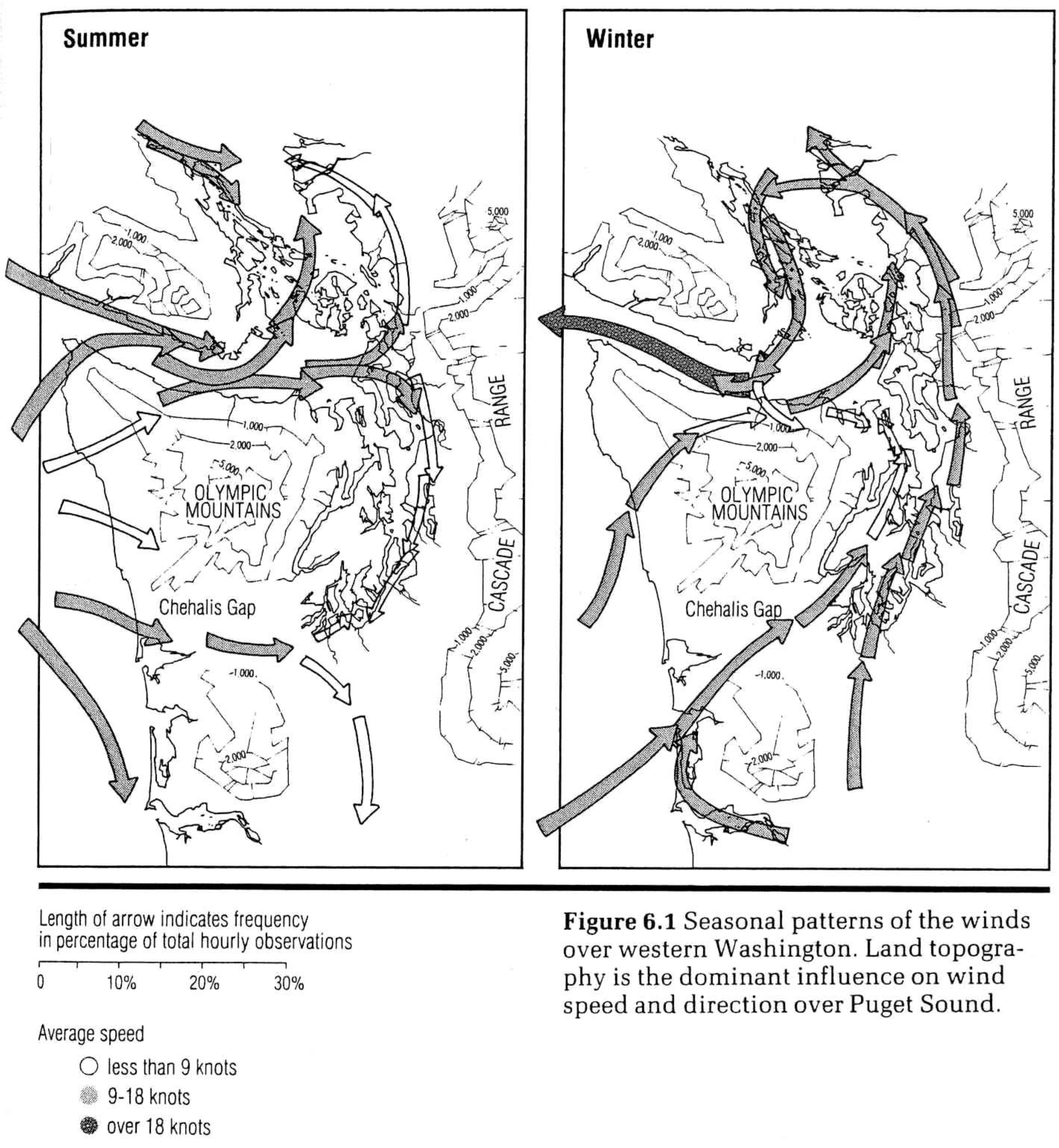


**Figure 1. Seasonal wind patterns in Puget Sound, Washington (Figure from Downing, 1983)**

The following listing provides a brief description of portions of the existing guidelines relevant to the topic of Sheltered Waters flood studies. These items are expanded on in subsequent sections of this report.

- Section D.1.2.4 – "Methods by which barriers, inlets and rivers have been treated" are required in documentation of the hydrodynamic storm surge model. However, no guidance is provided for methods to consider modeling for sheltered waters.

- Section D.2.2.7 – The "analysis of restricted fetches" in "sheltered coastal sites" is addressed in the existing guidelines and the ACES software is referred to; however, more specific guidance can be provided on how to apply this software to fetch-limited conditions.

- Section D.2.5.5 – "Wave runup in special situations" addresses wave runup and overtopping on shoreline barriers where overtopping flows discharge across landward-dipping or level backshore slopes to a "bay, river, or backwater". These situations are prevalent in sheltered water areas. No additional guidance is provided.

The natural processes that result in flooding in sheltered water areas are complex and not adequately addressed in the current guidelines. Sheltered water areas often have unique flood hazards, due to the effects of fluvial drainages and modified tidal and surge hydrology, and relatively strong currents. Wave-cast debris from extreme wave runup and overtopping can be especially problematic, owing to the proximity to sources of such materials in many estuaries. Other unique flood-related characteristics include the complex geometry of the embayments, lack of coincident peak storm surge with peak winds, shallow water and restricted wind fetches for wave growth, and non-sandy shoreline types with special erosion and scour hazards. These unique flood hazards are not adequately addressed in the current guidelines for sheltered water bodies.

New guidelines are needed to inform and guide Mapping Partners in the preparation of coastal flood insurance studies and flood hazard maps in sheltered water areas of the coastal floodplain.

## 1.3 SUGGESTED IMPROVEMENTS

Flood hazards in sheltered waters are caused by physical processes that are also responsible for open coast flooding (e.g., tidal surge, wave runup and overtopping); however, the sheltering effect of high elevation upland terrain on wind wave development and the localized shallow water bathymetry influence on the inland propagation of ocean swell and waves may create flood hazards unique to sheltered water areas.

At a basic level, definitions and descriptions of the physical conditions and physical processes associated with flood hazards in sheltered water areas need to be clearly described to Mapping Partners (and the public) through the guidelines. This information should be documented in the guidelines in a manner that establishes a framework for standardized, repeatable, and defendable flood study methods.

Given the lack of guidance for performing sheltered waters flood studies, Mapping Partners have developed methods to meet the needs of these unique studies over the years. Therefore, a concerted effort should be made to compile, review, and compare past methods that have been employed and approved by FEMA, and may be suitable for documentation in the new Guidelines.

It is proposed that new guidelines for sheltered water areas be developed as a separate section of the Guidelines to accompany the Pacific Ocean, Gulf of Mexico/Atlantic Ocean, and Great Lakes sections. This Sheltered Waters section would consist of methods unique to physical processes influenced by these sheltering effects. Cross-references would be provided to other sections of the guidelines to either instruct Mapping Partners on where and how to import parameters for use in the unique sheltered water methods or to export findings for use in subsequent methods to define final flood hazards.

An interim approach is to include a description of sheltered waters in the Pacific Coast Guidelines, which is to be written in Phase 2 of this project. It is anticipated that this section would also be useful for flood studies in other regions.

# 2 CRITICAL TOPICS

A series of focused study topics were identified to support the original Sheltered Waters issues in Topic 6, which was identified as being a critical need in Workshop 1. Following Workshop 1 eight critical topics were identified, but as of Workshop 2 three topics from this focused study report were either removed entirely or included in another Category and Topic. The remaining five Sheltered Water topics are discussed in the following sections of this report.

## 2.1 CRITICAL TOPIC 6A – PROVIDE DEFINITIONS AND DEVELOP A CLASSIFICATION METHOD FOR SHELTERED WATER STUDIES AS A FRAMEWORK FOR AN APPROACH

### 2.1.1 Description of the Topic 6a

Definitions for coastal and riverine flood studies are provided in the NFIP regulations (44 CFR 59.1). These definitions are useful as they convey basic concepts related to flood insurance studies and they support the Guidelines and Specifications. “Sheltered waters” is not currently defined in the NFIP regulations or the existing Guidelines. One USACE definition of sheltered waters is “shorelines that are not subjected to the direct action of undiminished ocean waves.” There are likely additional definitions that should be reviewed and considered to clarify the terms used in the pending revised guidelines. Examples of sheltered water bodies may also be appropriate to present in the guidelines to demonstrate the range of sheltered water conditions that can be encountered in coastal flood studies.

Pacific Coast examples of important sheltered water areas include Puget Sound (WA), San Francisco Bay (CA), and San Diego Bay (CA). Siletz Bay (OR), Humboldt Bay (CA), Morro Bay (CA), and Newport Bay (CA), San Diego Bay (CA) are examples of smaller embayments which may exhibit similar characteristics of sheltered waters.

Gulf of Mexico examples of sheltered water include Galveston Bay (TX), Mobile Bay (AL), Tampa Bay (FL), and Charlotte Harbor, (FL).

Atlantic Ocean examples of sheltered water include Indian River Lagoon (FL), Albermarle and Pamlico Sounds (NC), Chesapeake Bay (MD-VA), Delaware Bay (DE-PA), Long Island Sound (NY-CT), Narragansett Bay (RI), Buzzards Bay (MA), and Cape Cod Bay (MA).

This focused study proposes the use of a classification system as a way to provide a framework for the sheltered waters guidelines. Given the variety of coastal conditions a Mapping Partner could encounter along the shorelines of bays, river deltas, estuaries, etc. and the variety of coastal processes at work in sheltered water resulting in unique flood hazards, a classification system may assist a Mapping Partner to determine relevant issues and available methods for assessing potential flood hazards. Such an approach would benefit Study Contractors working on the open coast as well.

### 2.1.2 Current FEMA Guidance on the Topic

The following sections from the current FEMA Guidelines (2003) generally address this topic:

- Section D.1 – "General guidance" is primarily oriented to the Atlantic and Gulf Coasts; however, a 1977 technical paper is referenced for Pacific Northwest open coast flood study guidance. No references are provided for sheltered water flood studies in the current Guidelines.

- Section D.1.2.1 – The "geographic setting" of the flood study site is required as part of the engineering report. It is assumed that the general description of the geographic and demographic conditions of the study area are prepared to satisfy this requirement, and then is typically used as text for the FIS narrative report.

- Section D.1.2.4 – "Methods by which barriers, inlets and rivers have been treated" are required in documentation of the hydrodynamic storm surge model.

- Section D.2.1. and Section D.3.1 – "Typical shoreline types" (Table D-1) are provided in guidance for Atlantic and Gulf Coast studies and "basic types of coastal topography" (Table D-14) are provided for Great Lakes studies. No similar specific classification system is provided for Pacific Coast or sheltered water areas.

- Section D.2.2. and Section D.2.2.2 – "Upland regions" and "Topographic data" are discussed in these sections in relation to data requirements for coastal flood hazard analyses. The discussion is primarily focused on shore topography and does not encourage a broader view of regional topographic conditions.
- Section D.2.2.4 – "Land cover data" are discussed in the current guidelines as related to the immediate shoreline and backshore areas. This information is used to establish shoreline reaches with similar cultural features and define the nature of overland wave obstructions caused by vegetation and buildings.

### 2.1.3 Alternatives for Improvement

In lieu of changing the NFIP regulations and adding to the definitions provided in 44 CFR 59.1, a new section in Appendix D of the Guidelines for coastal flood study definitions (sheltered water and open coast) should be provided to augment those in 44 CFR 59.1. It is understood that the Guidelines currently has a "Glossary of Flood Hazard Mapping Terms" following the appendices; however, it may be easier for users of the Guidelines to refer to definitions within the technical appendix itself.

There is an opportunity to expand existing guidance for defining the "geographic setting" of the flood study site. Expanded guidance would be provided to assist the Mapping Partner in undertaking a systematic approach to quickly assess a project setting in order to better understand the regional and site specific characteristics of the study site. With this understanding, the Mapping Partner would then be guided to a series of methods to best define flood hazards given the study site characteristics. This approach would be embodied in a classification system.

Classification has been used in a variety of disciplines to order characteristics of systems into similar categories or relationships. Classification systems have been used since the 1800s to describe rivers and streams. Some examples of these classification systems include Montgomery and Buffington's geomorphological classification of drainage channels in the Pacific coastal ecoregion (Montgomery and Buffington, 1993) and Rosgen's classification of stream channels for restoration design (Rosgen, 1996).

Crafting an approach to the guidelines that helps orient the Mapping Partner to the physical characteristics and processes present at a given flood insurance study site location is an important consideration in planning the content of the new Guidelines. Based on this information the Mapping Partner can work through the guidelines to determine the types and relative importance of coastal hazards to consider (erosion, setup, overtopping, drift logs, etc) and select the appropriate methods to properly define hazard zones. A coastal classification system (perhaps in a flow chart or table format) would be provided in the guidelines for this purpose.

An example of a potential procedural flowchart that could assist Study Contractors to determine site history, conditions and relevant issues related to Sheltered Water areas is shown in Figure 2. The approach shown in the figure would encourage a Mapping Partner to systematically identify and understand the processes and the physiographic setting within, and beyond, a specific study

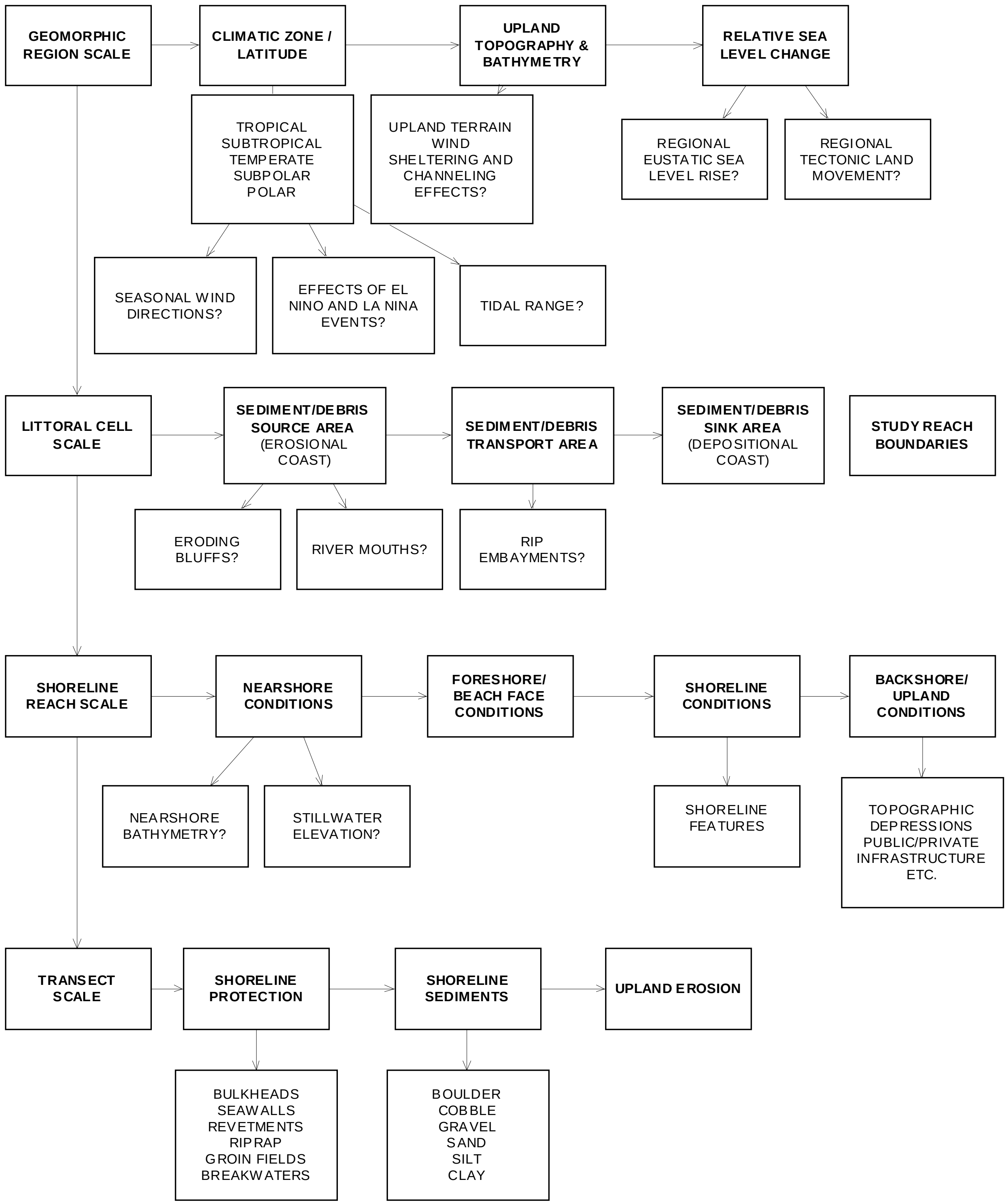


**Figure 2. Conceptual procedural flow chart for a sheltered water classification system.**

site at multiple spatial scales. For example, seasonal wind patterns and the sheltering effect of surrounding terrain would be identified at a regional scale. Sediment and erosion characteristics would then be identified at the littoral cell scale and shoreline and backshore features at the project reach and transect scale. Site classification (determination of site conditions and relevant issues) is typically the first step in conducting a flood hazard assessment and it is an essential part of scoping a study to reduce the chance of missing important study needs specific to a project's location, setting and history.

Additional improvements to existing guidance in the *G&S* are as follows:

- Section D.1 – "General guidance": New references to FEMA-accepted flood insurance studies in sheltered water areas (Port Angeles 1995, Whatcom County 2001) should be included in the references for all coasts. Phase 2 efforts should involve a review of these studies to determine their appropriateness to serve as references for guidance.

- Section D.1.2.4 – "Methods by which barriers, inlets and rivers have been treated": Where these methods involve sheltered water, specialized guidance should be referenced in a separate section to explain how the storm surge model took into account sheltering affects of high terrain, large forested vegetation stands, urban development, deep-cut navigation channels, etc.

- Section D.2.1. and Section D.3.1 – "Typical shoreline types": New guidance should be provided to support the proposed classification system described in the next section.

- Section D.2.2. and Section D.2.2.2 – "Upland regions" and "Topographic data": For sheltered water studies, the wind sheltering and channeling effects of nearby upland terrain on regional storm systems are important, as is the proximity of river mouths and the associated terrain and land use characteristics within river watersheds. New guidelines should provide guidance for evaluating these broader topographic data issues.

- Section D.2.2.4 – "Land cover data": New guidelines should broaden the review of these data to support the proposed geographic setting classification system and allow an evaluation of sources of wave-cast floating debris. In addition, if storm-induced erosion is to be addressed in the new guidelines, then geology and soils data may be best discussed in this section of the guidelines.

### 2.1.4 Recommendations

The recommendations for Topic 6a include:

1. Review previous sheltered water flood studies – Document how studies have been done in the past and compare methods approved by the various FEMA Regions.

2. Define Sheltered Water (non-Open Coasts) for the purposes of FEMA flood insurance studies - Compile a list of sheltered water flood study definitions and clarify terminology used in guidelines.
3. Identify sheltered water physical processes and site characteristics - Identify physical processes that may need to be considered to map the coastal flood hazard; consider processes common to all coastal areas and those unique to the identified landform class. Identify specific site characteristics (in this case sheltered water) that may need to be considered, evaluated and quantified to map the coastal flood hazard within a certain coastal class.
4. Review classification systems established by others and refine/adapt a system for sheltered water areas – Provide a framework for standardizing an approach to coastal flood studies and identifying the relative importance of certain physical characteristics and processes to coastal flood hazards, based on the physical location of a flood study project site.
5. Write Guidelines for Sheltered Water in the Pacific Coast Region with information useful to the Atlantic, Gulf and Great Lakes Regions as appropriate.

Table 1 at the end of this report contains a summary of the key findings and recommendations for Topic 46. Table 2 at the end of this report presents estimates of times required to accomplish the various tasks in this topic.

## 2.2 CRITICAL TOPIC 6B – PREPARE GUIDANCE FOR RECONSTRUCTING HISTORIC FLOOD CONDITIONS TO VALIDATE FLOOD STUDY RESULTS

### 2.2.1 Description of the Topic

The current FEMA Guidelines provide limited guidance on methods that can be used to obtain past flood data, reconstruct historic flood conditions, and use the data to validate new flood study results. This is an extremely important consideration because the complexity of coastal flood events can impart great uncertainty into the estimation of final Base Flood Elevations. The ability to compare theoretical results of the 1-percent-annual-chance flood to observed flood data can greatly reduce the uncertainty involved. This effort can increase the credibility of FEMA studies and reduce the potential for appeals.

Therefore, an objective of this work will be to prepare guidance for using historic flood observations to validate new flood study results.

### 2.2.2 Current FEMA Guidance on the Topic

The following section from the current FEMA Guidelines addresses this topic:

- Section D.2.2.9 – "Historical floods" and "buildings flooded" are discussed and general guidance is provided encouraging acquisition of all available data related to high water marks and tidal flooding from extreme coastal flood events.

### 2.2.3 Alternatives for Improvement

In sheltered water areas, many buildings and other features of the built environment are closer to the shoreline and often more exposed to flooding than along open coast areas. An opportunity exists to expand guidance on historical flood data acquisition to include methods for reconstructing past flood water elevations observed on these landmarks found in sheltered water areas (buildings, mailboxes, street signs, etc.) together with new survey data. Reconstruction implies converting qualitative flood observations (e.g., homeowner account of "flood water up to my deck") to a quantitative elevation using local survey methods employed on recent studies in Puget Sound (Whatcom County, WA - Sandy Point 2001 and Birch Bay 2002) can be reviewed and summarized for use as case study examples in the new guidelines.

Accordingly, expanded field reconnaissance guidance for sheltered water flood studies will be developed. This guidance may include the following:

- Develop sample "flood hazard questionnaire" for Mapping Partners to edit and send out to local government officials and property owners during the reconnaissance phase of a flood study.

- Develop field reconnaissance guidance on how to find and document coastal flood high water marks and wave heights.

- Develop guidance for "historic flood reconstruction" methods that convert qualitative flood observations to more quantitative data for results validation.

### 2.2.4 Recommendations

The following recommendations for Topic 6b include:

1. Review previous sheltered water flood studies and document methods used for validating flood study results – A summary of the review may include a checklist for results validation.

2. Review of previous sheltered water flood studies and compare results of past flood studies to actual damage and flood observations made by community officials and residents.

3. Prepare field reconnaissance guidance for reconstructing historic flood observations.

## 2.3 CRITICAL TOPIC 6C – SEEK PEER INPUT ON NEW SHELTERED WATER GUIDELINES NEEDS

*NOTE: This Topic was deleted during Work Shop 2.*

## 2.4 CRITICAL TOPIC 6D – PREPARE GUIDANCE FOR DEFINING THE 1-PERCENT-ANNUAL-CHANCE FLOOD EVENT IN SHELTERED WATER AREAS

### 2.4.1 Description of the Topic

The current guidance in Appendix D focuses on open coast areas and does not provide any recommended procedures for developing the 1-percent-annual-chance flood event in sheltered water areas. Guidance needs to be provided for the unique conditions found in sheltered waters where high terrain obstructs and channels the winds associated with regional storm systems as they traverse sheltered water bodies. Some sheltered water areas also receive strong influences from seasonal fresh water inflows, often leading to the need for assessing the joint probability of such combined influences of coastal and terrestrial flood conditions.

This topic will be coordinated with Topic 51 addressed by the Storm Meteorology Focused Study Group.

### 2.4.2 Current FEMA Guidance on the Topic

The following sections from the current FEMA Guidelines generally address this Topic:

- Section D.2.2.1 and Section D.2.2.7 – The current guidelines state “only the 1-percent-annual-chance SWEL is required for coastal analyses” and “the basic presumption…is that wave hazards occur coincidentally with the 1-percent-annual-chance flood (“flood” is assumed to mean stillwater).”

- Section D.2.2.6 - The current 1-percent-annual-chance flood event guidance was developed primarily for open-coast settings on the Atlantic and Gulf Coasts; e.g., guidance refers to storm meteorology associated with Northeasters and hurricanes and “rules of thumb” are provided for typical wind speed and surge conditions. As mentioned above, the current Guidelines appear to define the 1-percent-annual-chance “stillwater” as the “flood” event.

- Section D.2.2.7 – The “analysis of restricted fetches” in “sheltered coastal sites” is mentioned in the existing guidelines and the Mapping Partner is referred to ACES user manual for guidance.

- Section D.2.5.2 – Wave height and period estimates “suitable for runup computations at fully exposed coastal sites” are provided in Table D-4.

### 2.4.3 Alternatives for Improvement

Alternatives for improvement may be provided in sheltered water flood studies conducted for FEMA in Region X (CH2M Hill, 1989; PWA, 2002). These studies resulted in different methods that were approved by FEMA to estimate the 1-percent-annual-chance flood event for sheltered water areas of Puget Sound (Figure 3). A comparison of these methods to each other and to observed flood data would be informative to assess the sensitivity and accuracy of the methods. Based on the findings, more definitive guidance could be provided to Mapping Partners for estimating the 1-percent-annual-chance flood event for sheltered water areas.

Where streams are tributary to sheltered water, definition of the 1-percent-chance flood event requires consideration of the joint probability effects of riverine and tidal flooding. The dependence or independence of these two events in a flood frequency analysis is a complex issue, based on the size and speed of the regional storm system and the resulting coincidence or lag between peak occurrences of storm surge and river runoff. Guidance could be provided to Mapping Partners to define and enable mapping of these complex flood hazards.

Additional improvements to existing guidance in the *G&S* are as follows:

- Section D.2.2.1 and Section D.2.2.7: Recent sheltered water studies in Puget Sound recognized the wind sheltering effect of high mountainous terrain on low-pressure storm systems approaching the Pacific Coast and alternate methods were employed to estimate the 1-percent flood event with restricted fetches limiting the wave growth regions throughout Puget Sound. The methods used in these studies involved the estimation of the 1- through 100-year return period tides for use in joint probability statistical analyses. Therefore, new guidelines may actually require Mapping Partners to consider more than just the 1-percent-annual-chance flood (stillwater) elevation for estimation of the 1-percent-annual-chance flood event caused by the combination of wind waves and tide.

- Section D.2.2.6: Definition of the recurrence interval of this stillwater event is deemed important for providing a basis from which to assign recurrence intervals to observed historic extreme stillwater events and for assessing wave conditions likely associated with this stillwater event. These storm systems are addressed because of their potential differences in duration of the surge event. Therefore, new guidelines may need to address the duration of the surge event and not just the peak elevation of the stillwater event alone. The duration, or time history, of the tidal surge event may become important if more detailed guidance is provided on methods for determining the probability of the simultaneous occurrence of two events, as opposed to the related occurrence of the events within a broader time period.

- Section D.2.2.7: New guidelines might expand on the “fetch-limited” wave height calculations made through the use of the currently referenced ACES Windspeed Adjustment and Wave Growth computer routine.

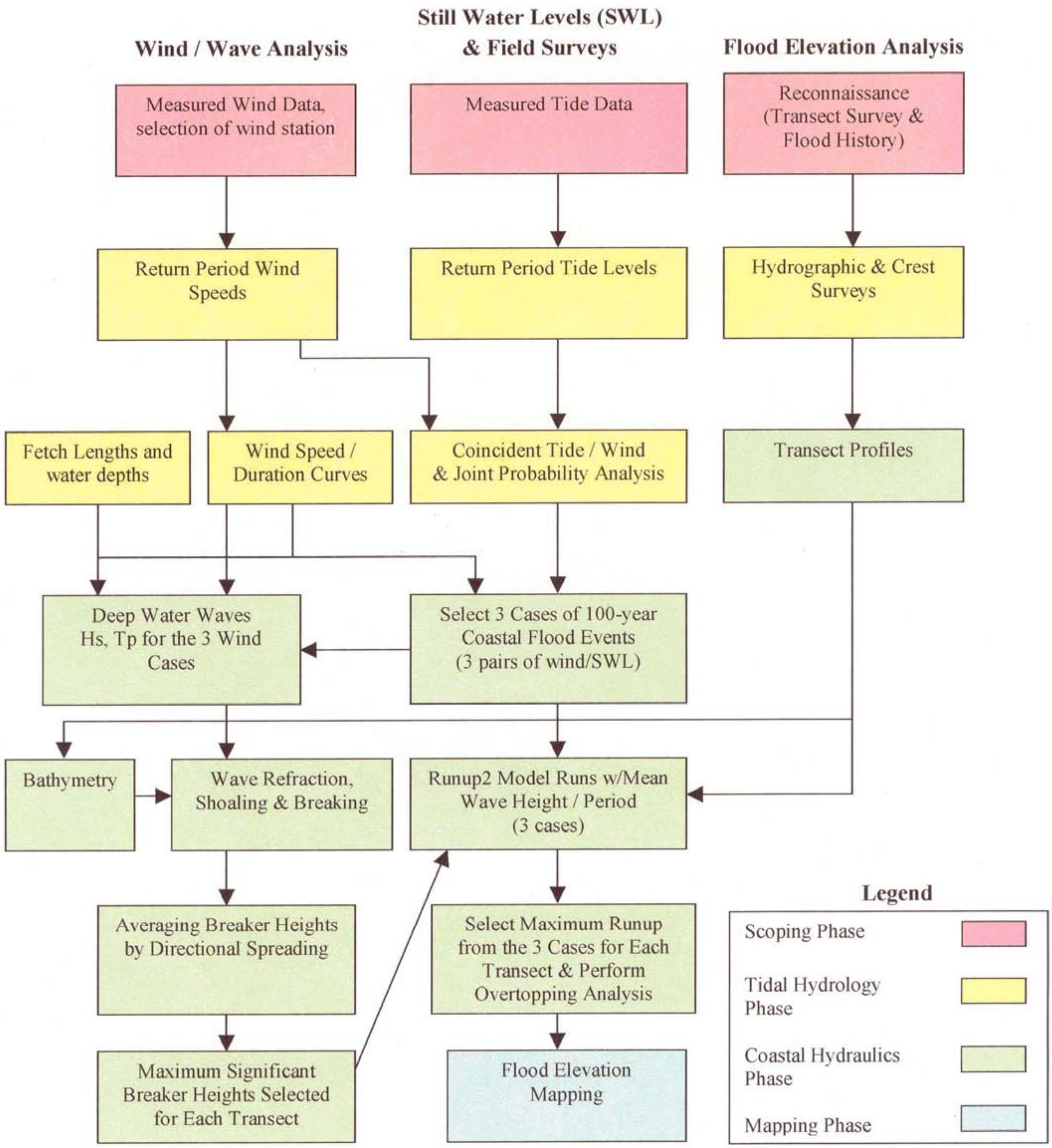


**Figure 3. Representative sheltered water methodology.**

- Section D.2.5.2: A similar table to Table D-4 or nomograph relating wind speed to fetch length, such as the classic Sverdrup-Munk-Bretschnieder diagram (Figure 4), or improved guidance, would be useful in the guidelines to describe wave growth for sheltered water conditions based on fetch length, wind speed and duration.

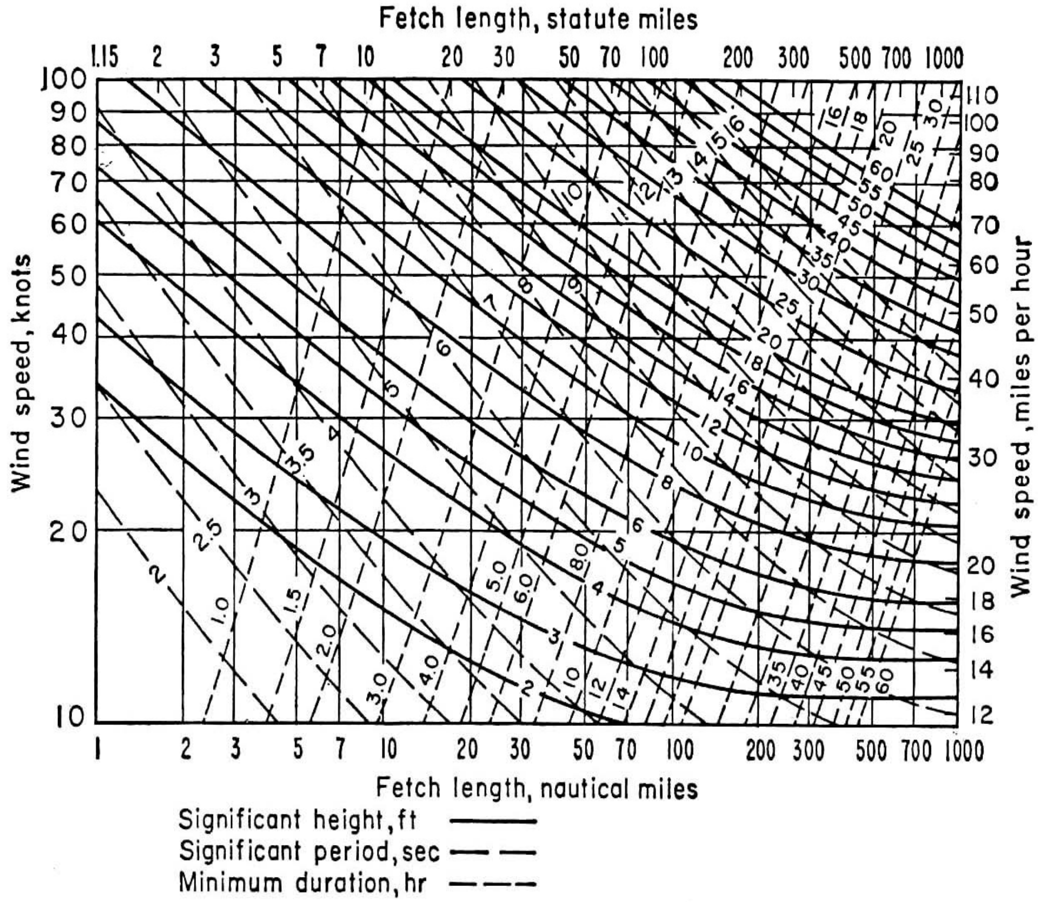


FIG. 10-15. Deep-water wave-forecasting curves as a function of wind speed, fetch length, and wind duration.

**Figure 4. Sverdrup-Munk Bretschnieder diagram for wave forecasting in sheltered waters**

### 2.4.4 Recommendations

The following recommendations for Topic 6d include:

1. Review the methods used in previous FEMA-accepted sheltered water flood insurance studies for possible adoption as methods to reference in the new guidelines.

2. Prepare guidance on independent/dependent joint probability effects on the 1%-chance event considering coastal watersheds and riverine-tidal flooding.

3. Expand discussion of existing guidance on wind data acquisition and analysis and fetch-limited wave forecasting.

## 2.5 CRITICAL TOPIC 6E – PREPARE GUIDANCE FOR ESTIMATING STILLWATER ELEVATIONS AND CURRENTS IN SHELTERED WATER AREAS

### 2.5.1 Description of the Topic

Sheltered water areas encompass a variety of complex shorelines. This complexity in geography and topography can result in unique circumstances where methods derived for open coast assessments may not be directly applicable to sheltered water coastal flood studies. The objective of this work is to prepare guidance interactively with the Stillwater and Wave Transformation Focused Study Groups, refining appropriate open coast guidance (Item 8 and 55) into a reliable methodology for sheltered water areas.

### 2.5.2 Current FEMA Guidance on the Topic

The following sections from the current FEMA Guidelines and NFIP regulations generally address this topic:

- D.2.2.1 – Guidance on the "estimation of stillwater elevations" is provided with recommendations for the use of measured water levels at gauge stations or storm surge modeling. Models are recommended for "complex shorelines" where gage records may not represent true stillwater elevations.

- D.2.6.1 – Various tidal datums (MLW, MTL, MHW) are referred to in Figure D-24 of the existing Guidelines. Tidal datums are also referred to in the NFIP regulations under 44CFR60.3(e)(3) "Mean High Tide" and "normal high water" is referred to in the definition of "reference feature" in 44CFR59.1. There is currently no guidance on how to obtain these data or estimate these elevations.

### 2.5.3 Alternatives for Improvement

Given the concentration of development and insured structures that are typical within sheltered water areas—those adjacent to low gradient shoreline beaches and estuarine wetlands—there is a need to better understand the true natural variation of tide, in addition to the hypothetical 1-percent-annual-chance stillwater elevation. These tidal variations are commonly documented at tidal gauging stations as "tidal datums." Vertical tidal datum references are often used in the more densely populated sheltered water areas to demarcate natural and human boundaries; e.g., legal property boundaries are often establish based on the "Ordinary High Water Mark" along shorelines and rivers. Guidance should be provided 1) on where and how to obtain tidal datum information, 2) how tidal datums can be developed from tide gauge data or derived in conjunction with field investigations, and 3) how tide data and tidal datums should be validated prior to usage.

Recurrence interval stillwater data are typically derived from tidal gauging stations. These stations are normally located within a sheltered water body with open communication to the ocean. For flood study sites located in a nearby ungauged sheltered water body, the direct transfer of tide gauge elevations, or interpolation of data between two gauging stations may not be appropriate. For example, the morphology of an embayment with a narrow tidal inlet may result in “tidal choking”, resulting in stillwater elevations higher than predicted from adjacent gauge data. A sheltered bay influenced by large seasonal fresh water discharges from inland drainage areas are certain to have unique tidal characteristics (datums) during periods of high runoff. Guidance should be provided to Mapping Partners for transferring tide gauges data to ungauged locations. This guidance could be similar to the standard hydrologic methods available for transferring stream gauge data from a gauged to ungauged watershed location.

Tidal, riverine and other nearshore currents may have an effect on wave shoaling and shoreline erosion under flood conditions. The Guidelines should discuss these conditions and provide methods for assessing the significance of these physical processes on flood hazards. This topic should be coordinated with the Wave Transformations Study Group, owing to the effects of currents on wave refraction and shoaling. The key issue is that currents are often stronger in sheltered waters, and hence current effects on waves should not necessarily be neglected.

### 2.5.4 Recommendations

The following recommendations for Topic 6e include:

1. Review scientific literature and resource management practices related to stillwater, tidal currents and tidal datums in sheltered water areas.

2. Prepare guidance for the transfer of tide gauge data to ungauged sheltered water bodies.

3. Prepare guidance for the assessment of tidal and riverine nearshore currents and their significance to flood hazards.

4. Coordinate guideline development with appropriate Focused Study Groups.

## 2.6 CRITICAL TOPIC 6F – PREPARE GUIDANCE FOR EVALUATING COASTAL FLOOD PROTECTION STRUCTURES IN SHELTERED WATERS

*NOTE: This Topic was assigned for coverage under Topic 21a during Work Shop 2.*

## 2.7 CRITICAL TOPIC 6G – PREPARE GUIDANCE FOR IDENTIFYING FLOOD INSURANCE RISK ZONES IN SHELTERED WATERS

*NOTE: This Topic was assigned for coverage under Topic 17 during Work Shop 2.*

## 2.8 CRITICAL TOPIC 6H – COORDINATE THE PREPARATION OF SHELTERED WATER GUIDELINES WITH OTHER MAP MODERNIZATION OBJECTIVES AND MULTI-HAZARD PLANNING INITIATIVES

### 2.8.1 Description of the Topic

The development of new coastal guidelines is one of many Map Modernization objectives identified by FEMA. The sheltered water guidelines should reference other appropriate Map Modernization objectives and multi-hazard planning initiatives. This coordination can broaden the use of the new guidelines to assist Mapping Partners with other hazard mitigation activities. For example, the utility of new guidelines may be enhanced if the methods for riverine-tidal flood assessments in sheltered water areas also provide guidance for incorporating future conditions riverine hydrology into coastal flood hazard maps.

### 2.8.2 Current FEMA Guidance on the Topic

The current FEMA Guidelines reference past coastal guidelines, that have been incorporated into the document, and other documented methodologies that are available as supporting guidance (Section D.1.1). However, the current Guidelines do not reference other FEMA guidelines and initiatives Mapping Partners may consider when they start a flood insurance study. For example, Mapping Partners may want to plan a flood study so that data can be subsequently used for the Coastal Flood Information Tool (FIT) for eventual HAZUS modeling.

### 2.8.3 Alternatives for Improvement

As the draft guidelines for sheltered water flood hazards are being defined, a nominal effort should be made to understand how these guidelines may interrelate to other FEMA initiatives to identify potential benefits that could be relayed to Mapping Partners. The new guidelines may simply include a listing of other FEMA initiatives for the Mapping Partner's reference. Example of related FEMA initiatives may include:

- Mapping future conditions – FEMA has provided guidance for mapping riverine floodplains due to future conditions hydrology. Similar guidance may be appropriate for future hazards in sheltered water areas due to relative sea level change—sea level rise together with local tectonic uplift or subsidence—either separately or in combination with future riverine guidance for flood studies in areas of combined riverine – tidal flooding. The focused study team believes consideration and mapping of long-term changes are technically feasible, but problematic, given unresolved NFIP policy and implementation issues. These topics have received considerable attention by others (at federal, state and local levels), but time and budget constraints prevented this team from contributing to the topic beyond reiterating its importance.

- HAZUS-MH and the Coastal Flood Information Tool (FIT) – Analysis at Levels 2 and 3 using the FIT will likely rely on flood hazards determined from detailed coastal studies performed using the new guidelines.
- Letters of Map Change, Amendment and Revision MT Forms
- Mapping Activity Statement (MAS) Template for Cooperating Technical Partners (CTP) Program
- DFIRM standards

Lastly, Volumes 1 to 3 and the remaining appendices of the FEMA Guidelines should be reviewed to determine if changes to the *G&S* will affect content elsewhere in the document.

### 2.8.4 Recommendations

The following recommendations for Topic 6h include:

1. Identify and assess interrelationships of new guidelines to other Map Modernization objectives and related FEMA initiatives.
2. Review state floodplain management policies related to sheltered water.
3. Review Volumes 1-3 and other appendices of current Guidelines
4. Document and disseminate the findings to the other Focused Study Groups and integrate into the new Guidelines as appropriate.

## 3 AVAILABLE TOPICS

*(Not Applicable)*

## 4 IMPORTANT TOPICS

*(Not Applicable)*

## 5 ADDITIONAL OBSERVATIONS

Seiching may contribute to flood hazards in sheltered water. Seiching involves the movement of long waves that move rhythmically back and forth within an enclosed water body such as a lake or bay. The waves can be caused by a sudden air pressure change from the passage of an intense storm system or a long period wave train entering an embayment from the ocean. The resulting wave period is a function of the size and depth of the water body. Flood hazards from seiching are limited, but could be significant if the wave period of a tsunami entering a sheltered embayment is an even multiple of the natural period of the embayment; in this case the seiching would be amplified and could potentially result in coastal flood hazards. The significance of this physical process should be considered in future Tsunami Studies.

# 6 SUMMARY

The Sheltered Waters Focused Study involves a unique effort because sheltered water aspects are included in many of the other Focused Studies. Accordingly, the preparation of guidance for many of the topics identified in this report will be coordinated with other appropriate focused study groups because of the interrelationship of the sheltered water and open coast topics.

Five sheltered water topics are addressed in this report. The first topic addresses the need to establish a framework for performing coastal studies and proposes the use of a classification system, to guide Mapping Partners through the scoping and execution of a coastal flood study project, and the need to simply define “sheltered waters” for FEMA flood study purposes. The second topic will expand current guidance for documenting and using data on observed flood conditions to provide validation of flood study results. The third topic will result in guidance specific to defining the 1-percent-annual-chance flood event involving dependent and independent joint probability occurrences of riverine and tidal flooding in sheltered water areas. Existing guidance on wind data acquisition and analysis and fetch-limited wave forecasting in sheltered waters will also be expanded. The fourth topic addresses guidance for estimating stillwater elevations in sheltered waters and evaluating the effects of tidal and riverine currents on wave propagation. The last topic addresses the benefits of referencing other appropriate Map Modernization objectives and multi-hazard planning initiatives. This coordination can broaden the use of the new guidelines to assist Mapping Partners with other hazard mitigation activities. These topics and recommendations for preparing associated guidelines are summarized in Table 1.

An additional observation was made to include guidance on defining flood hazards caused by seiching in sheltered water bodies.

| Table 1. Summary of Findings and Recommendations for Sheltered Waters | | | | | | |
|---|---|---|---|---|---|---|
| Topic Number | Topic | Coastal Area | Priority Class | Availability Adequacy | Recommended Approach | Related Topics |
| 6a | Definitions and Classification | AC | H | MAJ | 1. Review previous sheltered water flood studies, compare methods, geomorphic conditions, unique flood hazards.<br>2. Define Sheltered Water (non-Open Coasts) for the purposes of FEMA flood insurance studies.<br>3. Identify and classify Pacific sheltered water physical processes and site characteristics.<br>4. Review classification systems established by others and refine/adapt a system for sheltered water areas.<br>5. Write Guidelines for Sheltered Water in the Pacific Coast Region with information useful to the Atlantic, Gulf and Great Lakes Regions as appropriate. | 1, 5, 9, 10, 11-14, 15-16, 17-19, 20, 21-27, 29, 30, 35-36, 37-43, 44-48, 50-51, 52-55, |
| | | GC | H | MAJ | | |
| | | PC | C | MAJ | | |
| | | SW | C | MAJ | | |
| 6b | Historical Information | AC | H | MIN | 1. Review previous sheltered water flood studies and document methods used for validating flood study results.<br>2. Compare results of past flood studies to actual damage and flood observations.<br>3. Prepare field reconnaissance guidance.<br>4. Write the guidelines. | 9-10, 11-14, 17-19, 21-22, 24, 30-31, 35-36, 53 |
| | | GC | H | MIN | | |
| | | PC | C | MIN | | |
| | | SW | C | MIN | | |
| 6d | 1% Annual Chance Flood Elevations | AC | H | MAJ | 5. Review the methods used in previous FEMA-accepted sheltered water flood insurance studies for possible adoption as methods to reference in the new guidelines.<br>6. Prepare guidance on independent/ dependent joint probability effects on the 1%-chance event considering coastal watersheds and riverine-tidal flooding.<br>7. Expand discussion of existing guidance on fetch-limited wave forecasting.<br>8. Write the guidelines. | 4-5, 8-10, 12, 16, 19, 44-48, 50-51, 52-55 |
| | | GC | H | MAJ | | |
| | | PC | C | MAJ | | |
| | | SW | C | MAJ | | |

**Table 1. Summary of Findings and Recommendations for Sheltered Waters**

| | | | | | | |
|---|---|---|---|---|---|---|
| 6e | Stillwater Elevations and Tidal Currents | AC | H | MAJ | 1. Review scientific literature and resource management practices related to stillwater, currents and tidal datums in sheltered water areas.<br>2. Prepare guidance for the transfer of tide gauge data to ungauged sheltered water bodies.<br>3. Prepare guidance for the estimation of tidal datums in flood insurance studies.<br>4. Prepare guidance for the assessment of tidal and riverine nearshore currents and their significance to flood hazards.<br>5. Coordinate guidelines development with appropriate Focused Study Groups. | 42-48, 52-55 |
| | | GC | H | MAJ | | |
| | | PC | C | MAJ | | |
| | | SW | C | MAJ | | |
| 6h | Inter-relationships | AC | H | MIN | 1. Identify and assess interrelationships of new guidelines to existing FEMA initiatives.<br>2. Review the status of the ongoing Pacific Coast Coastal Barrier Resources System study and review related FEMA guidance (Section 2.2).<br>3. Review state floodplain management policies related to sheltered water.<br>4. Review Volumes 1-3 and other appendices of current Guidelines.<br>5. Document and disseminate the findings to the other Focused Study Groups and integrate into the new Guidelines as appropriate. | All |
| | | GC | H | MIN | | |
| | | PC | C | MIN | | |
| | | SW | C | MIN | | |

Key:
Coastal Area
AC = Atlantic Coast; GC = Gulf Coast; PC = Pacific Coast; SW = Sheltered Waters
Priority Class
C = critical; A = available; I = important; H = helpful
Availability/Adequacy
“Critical” Items: MIN = needed revisions are relatively minor; MAJ = needed revisions are major
“Available” Items: Y = availability confirmed; N = data or methods are not readily available
“Important” Items: PRO = procedures or methods must be developed; DAT = new data are required; PRODAT = both new procedures and data are required

**Table 2. Preliminary Time Estimate for Guideline Preparation for Sheltered Water Flood Hazards**

| Item | Time (Person Months) |
|---|---|
| **Time Estimates for Critical Topic 6a** | |
| Review previous sheltered water flood studies | 1 |
| Define Sheltered Water (non-Open Coasts) for the purposes of FEMA flood insurance studies | 0.5 |
| Identify sheltered water physical processes and site characteristics | 1 |
| Develop a sheltered water classification system | 1 |
| Write the topic guidelines | 0.5 |
| Incorporate Feedback, Finalize | 0.2 |
| TOTALS | 4.2 |
| **Time Estimates for Critical Topic 6b** | |
| Document methods used for validating flood study results | 1 |
| Compare results of past flood studies to actual damage and flood observations | 0.5 |
| Prepare field reconnaissance guidance | 1 |
| TOTALS | 2.5 |
| **Time Estimates for Critical Topic 6d** | |
| Review the methods used in previous FEMA-accepted sheltered water flood insurance studies for possible adoption as methods to reference in the new guidelines | 1 |
| Prepare guidance on independent/dependent joint probability effects on the 1%-chance event considering coastal watersheds and riverine-tidal flooding. | 1 |
| Expand discussion of existing guidance on fetch-limited wave forecasting | 0.5 |
| Write the topic guidelines | 0.5 |
| Incorporate Feedback, Finalize | 0.2 |
| TOTALS | 3.2 |
| **Time Estimates for Critical Topic 6e** | |
| Review scientific literature and resource management practices | 0.5 |
| Prepare guidance for the transfer of tide gauge data to ungauged sheltered water bodies. | 0.5 |
| Prepare guidance for the estimation of tidal datums | 0.5 |
| Prepare guidance for the assessment of tidal and riverine nearshore currents | 0.5 |
| Coordinate guideline development with appropriate Focused Study Groups | 0.3 |
| Write the topic guidelines | 0.5 |
| Incorporate Feedback, Finalize | 0.2 |
| TOTALS | 3 |
| **Time Estimates for Critical Topic 6h** | |
| Identify and assess interrelationships of new guidelines to other Map Modernization objectives and related FEMA initiatives. | 0.5 |
| Review state floodplain management policies related to sheltered water | 0.2 |
| Review Volumes 1-3 and other appendices of current Guidelines | 0.2 |
| Document and disseminate the findings | 0.2 |
| TOTALS | 1.1 |

# 7 REFERENCES

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# Flood Hazard Zones

## FEMA Coastal Flood Hazard Analysis and Mapping Guidelines Focused Study Report

February 2005


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## Acronyms

| | |
|---|---|
| ASCE | American Society of Civil Engineers |
| BFEs | Base Flood Elevations |
| CFR | Code of Federal Regulations |
| CRS | Community Rating System |
| CTP | Cooperating Technical Partner |
| FEMA | Federal Emergency Management Agency |
| FIRM | Flood Insurance Rate Map |
| FIS | Flood Insurance Study |
| *G&S* | *Guidelines and Specifications* |
| MA CZM | Massachusetts Office of Coastal Zone Management |
| NFIP | National Flood Insurance Program |
| nhc | Northwest Hydraulics Consultants |
| PFD | Primary Frontal Dune |
| SC | Study Contractor |
| SFHA | Special Flood Hazard Area |
| WHAFIS | Wave Height Analysis for Flood Insurance Studies |

# 1 INTRODUCTION

The Flood Hazard Zone Focused Study evaluates the procedures in the current FEMA guidelines related to the establishment and mapping of coastal hazard zones. The description of each Flood Hazard Zone topic identified in Workshop 1 is listed below. In addition, each topic and needs description for Flood Hazard Zones are identified by specific coastal regions of application for Atlantic Ocean, Gulf of Mexico, Pacific Ocean, or Non-Open Coast (Sheltered Waters).

**Flood Hazard Zones Topics and Priorities**

| Topic Number | Topic | Topic Description | Atlantic / Gulf Coast | Pacific Coast | Non-Open Coast |
|---|---|---|---|---|---|
| 39 | PFD | Develop better definition of landward limit of PFD (used for VE Zone limit), and gather and evaluate MA CZM and other mapping approaches. | C | I | I |
| 17 | VE Zone Limit | Enhance existing guidelines for defining inland limit of VE Zone including the development of a basis for better guidance for heavily over-topped areas | C | C | C |
| 18 | VE/AE Zone Appropriateness | Investigate the appropriateness of existing VE & AE Zone definitions for coastal areas | I | I | I |
| 19 | Combined Coastal/ Riverine | Flood risk management of combined coastal and riverine flooding hazards | A | A | A |
| Key: C = critical; A = available; I = important; H = helpful | | | | | |

The workshop assigned each topic with a priority rating of "C" for Critical, "I" for Important, and "A" for Available, depending upon its direct impact on the FEMA coastal flood studies program, revision needs in existing *G&S*, and the expected timeline for completion and incorporation of the topic into *Appendix D* and the National Flood Insurance Program (NFIP).

The Flood Hazard Zones topics list initially included only Topics 17, 18, and 19, but was modified to include Topic 39 (previously associated with Event-Based Erosion) during Workshop 2. The topic descriptions are shown above.

## 1.1 FLOOD HAZARD ZONES FOCUSED STUDY GROUP

The Flood Hazard Zones Focused Study Group members are Kevin Coulton, Michael DelCharco, Darryl Hatheway (who served as the Focused Study Leader), and Chris Jones. The group worked collectively to: a) define the scope of the Focused Study and specific assignments, b) resolve any discrepancies or interpretations of the topic and needs descriptions, and c) collect related background information. The group attempted to review and apply available information on each subject to prepare this report. Each member contributed their unique insight on the subject matter, technical reports and publications, and specific report section write-ups. The collaboration of the group within the allotted Focused Study time-span is by no means comprehensive or exhaustive, but represents a considerable effort to evaluate the topics addressed in Workshop 1 (December 2003) and Workshop 2 (February 2004), and to provide recommendations that will benefit FEMA coastal flood hazard mapping guidance.

## 1.2 CURRENT FEMA GUIDANCE FOR FLOOD HAZARD ZONES

As part of the review of FEMA coastal flood insurance study methodologies, the Flood Hazard Zones Focused Study Group reviewed several topics from an overall matrix of specific topics. These topics (39 [formerly with Event-Based Erosion], 17, 18, and 19) addressed needs identified during the FEMA Coastal Guidelines Project Workshop 1 held by Northwest Hydraulics Consultants (nhc) in December 2003. The overall goal of the Flood Hazard Zones Focused Study was to resolve key issues related to hazard zone mapping per the current April 2003 FEMA guidelines document for coastal Flood Insurance Studies titled, "Guidelines and Specifications for Flood Hazard Mapping Partners – *Appendix D*: Guidance for Coastal Flooding Analyses and Mapping" (hereinafter referred to as the *G&S* or *Appendix D*).

Hazard zone mapping is the final product of the detailed analyses of a coastal flood study process undertaken by a Mapping Partner, performed either by the study contractor (SC), map revision requester, or Cooperating Technical Partner (CTP). The results of the coastal flood study are described in the FIS report and delineated onto a Flood Insurance Rate Map (FIRM). The FIRM depiction of the Special Flood Hazard Area (SFHA) for coastal hazards, as determined by detailed studies of storm surge flooding, storm-induced erosion, and wave effects, is generally subdivided into six different zones, including:

- VE Zones, also known as the coastal high hazard areas. They are areas subject to high velocity water including waves; they are defined by the 1% annual chance (base) flood limits (also known as the 100-year flood) and wave effects 3 feet or greater. The hazard zone is mapped with base flood elevations (BFEs) that reflect the combined influence of stillwater flood elevations, primary frontal dunes, and wave effects 3 feet or greater.

- AE Zones, also within the 100-year flood limits, are defined with BFEs that reflect the combined influence of stillwater flood elevations and wave effects less than 3 feet. The AE Zone generally extends from the landward VE zone limit to the limits of the 100-year flood from coastal sources, or until it reaches the confluence with riverine flood sources. The AE Zones also depict the SFHA due to riverine flood sources, but instead of being subdivided into separate zones of differing BFEs with possible wave effects added, they represent the flood profile determined by hydrologic and hydraulic investigations and have no wave effects.

- AO Zones, representing coastal hazard areas that are mapped with flood depths instead of base flood elevations. Depths are mapped from 1 to 3 feet, in whole-foot increments. These SFHAs generally are located in areas of sheet flow and runoff from coastal flooding where a BFE cannot be established. The AO Zone is also used in riverine flood mapping.

- AH Zones, representing coastal hazard areas associated with shallow flow or ponding, with water depths of 1 to 3 feet. These areas are usually not subdivided, and BFEs are mapped.

- X Zone (shaded), representing the coastal (or riverine) floodplain areas between the 100-year flood and 0.2% annual chance (500-year) flood. These areas are located outside the SFHA, but are depicted on the FIRM unless map scale limitations prevent detailed mapping of this area. They were formerly mapped and depicted as Zone B.

- X Zone (unshaded), representing the areas on the FIRM that are located outside the limits of the 500-year flooding. They were formerly mapped and depicted as Zone C.

Before undertaking a detailed evaluation of each topic, the Flood Hazard Zones Focus Study Group believes it is useful to examine the larger context of what hazard zones should (and should not) encompass. Starting with the NFIP definition of the FIRM, we see that the FIRM should identify hazard areas and risk premium zones (44 CFR sec. 59.1 states, *"Flood Insurance Rate Map (FIRM) means an official map of a community, on which the Administrator has delineated both the* ***special hazard areas*** *and the risk premium zones applicable to the community"*). A common interpretation of the FIRM is that it maps only the SFHA (SFHA is defined in 44CFR sec. 59.1 as: *the land in the flood plain within a community subject to a one percent or greater chance of flooding in any given year*) and the risk premium zones. However, the NFIP regulations clearly anticipated the mapping of other hazards on the FIRM as well – *Special Hazard Area* is defined in 44CFR sec. 59.1 as *an area having special flood, mudslide (i.e., mudflow), or flood-related erosion hazards.*

Thus, the Flood Hazard Zones Focus Study Group will view its charge broadly, and advise FEMA on the mapping of flood hazards in coastal areas subject to a 1% or greater chance of occurring in any given year (100-year flood), and on the mapping of other hazards associated with the coastal base flood event. Flood hazards will include standing or slowly moving water, flow velocity, wave height, wave runup, and wave overtopping. Associated hazards will include event-based erosion, overwash and sediment deposition, and flood borne or wave-cast debris.

Because a FIRM is used to regulate construction and to establish flood insurance premium rates, it makes sense to consider a wide range of coastal hazards that can damage buildings in coastal areas. It also makes sense to consider mapping- and policy-related issues (such as Primary Frontal Dune delineation) that affect the delineation of flood and associated hazards in coastal areas.

This Focused Study includes an evaluation of the following issues, but further consideration by FEMA is required prior to implementation because of policy and regulatory implications.

- Changing current coastal mapping/regulation/insurance practices in one or more aspects of the NFIP to substantially reduce damages to NFIP-compliant structures in mapped AE zones.
- Modifying the NFIP's primary frontal dune definition and V Zone delineation to better identify flood risks while maintaining dune protection and coastal construction standards.
- Revising floodplain management requirements for SFHAs included within the primary frontal dunes.
- Reviewing possible regulatory changes to redefine the criteria used to establish VE Zones. In particular, VE Zones may need to consider a variety of conditions (breaking waves, broken waves, runup, velocity, erosion, overwash) in light of recent coastal flood events that caused damage to structures on wall-type and shallow foundations located in the SFHA (Zone AE, AH, and AO) inland of the VE Zone limit.
- Reviewing possible regulatory changes to subdivide the coastal AE Zone into portions where flood conditions resemble those in VE Zones, and portions where flood conditions resemble those in riverine AE Zones.

# 2 CRITICAL TOPICS

Workshop 1 developed matrices listing each of the topics with specific category groupings (e.g., Flood Hazard Zones were number 17, 18, and 19), and priority classes (e.g., C, I, A). The workshop identified the highest priority topics as "Critical." As mentioned above, Topic 39 was originally assigned to Event-Based Erosion and was subsequently moved to the Flood Hazard Zones Focused Study for review and consideration. Flood Hazard Zones topics listed as Critical in one or more regions, include Topics 39 and 17. Flood Hazard Zones Topic 39 was identified as priority "C" for the Atlantic and Gulf region and only "I" for the Pacific Region and Sheltered Waters. However, all discussion for this topic will be included in this Critical topics section. In addition, Topic 17 was identified as priority "C" for all four regions - the Atlantic, Gulf, Pacific and Sheltered Waters.

## 2.1 TOPIC 39: LANDWARD PRIMARY FRONTAL DUNE LIMIT AND DEFINITION

### 2.1.1 Description of the Topic and Suggested Improvement

The December 2003 workshop identified the needs for Topic 39 as follows:

*"Develop better definition of landward limit of Primary Frontal Dune (used for VE Zone limit), and gather and evaluate MA CZM and other mapping approaches."*

The Focused Study group reaffirmed the importance of Topic 39 since the primary frontal dune (PFD) VE Zone (by definition) will dominate over other detailed analyses of the VE Zone limit based on wave height, wave runup, and wave overtopping. The group identified possible improvements to the existing definition within the NFIP, with the objectives of improved clarity for Mapping Partners and improved correlation between the PFD definition and hazard zone mapping (Topic 17).

The February 2004 workshop considered the topic further, and determined that a related issue also needs to be addressed – current mapping procedures result in large BFE differences at the boundary between the PFD-based VE Zone and the next landward Zone AE hazard areas or an abrupt transition into Zone X floodplain areas. These BFE and zone transition differences are a product of mapping procedures, not actual flood hazards, and should be addressed in conjunction with Topic 17.

To prepare the Flood Hazard Zones Focused Study assessment for Topic 39, the following was required:

- Review of NFIP regulations as they apply to primary frontal dune VE Zones;
- Review of existing *Appendix D* guidelines for determination of the landward limit of the primary frontal dune; and
- Assessment of current guidance on the mapping of primary frontal dune VE Zones on the FIRM.

The existing *G&S* for hazard zone mapping show various ways to map the VE Zone coastal high hazard area and BFEs due to inclusion of the primary frontal dune. In many cases, the VE Zone landward limit is defined by the definition of the primary frontal dune, rather than by analysis of water level or wave conditions. The definition of the primary frontal dune is therefore critical to hazard zone mapping on the FIRM. However, the existing definition of the primary frontal dune in NFIP regulations is qualitative, and *Appendix D* contains little direct guidance on estimation of the landward limit. A better methodology for defining the primary frontal dune and its landward limit would improve efficiency and consistency in hazard mapping.

The Focused Study Group also determined that there are other key areas of concern related to VE Zones and primary frontal dunes, which are addressed in Topic 17. The key issue for Topic 39 is the determination of the landward limit of the PFD, based on qualitative methods using the Part 59 definition.

### 2.1.2 Topic 39: Description of Procedures in the Existing Guidelines

*Appendix D* Section D.2 includes a discussion with background on recent coastal guidance updates for storm-induced erosion that also resulted in a change in the official basis for treating flood hazards near coastal sand dunes. FEMA published new rules and definitions in the May 6, 1988 *Federal Register*, pages 16269-16273 (that became effective on October 1, 1988), which included the following revised definitions in 44 CFR sec. 59.1 of the NFIP regulations:

> "*Coastal high hazard area* means an area of special flood hazard extending from offshore to the inland limit of a primary frontal dune along an open coast and any other area subject to high velocity wave action from storms or seismic sources."

> "*Primary frontal dune* means a continuous or nearly continuous mound or ridge of sand with relatively steep seaward and landward slopes immediately landward and adjacent to the beach and subject to erosion and overtopping from high tides and waves during major coastal storms. The inland limit of the primary frontal dune occurs at the point where there is a distinct change from a relatively steep slope to a relatively mild slope."

Figure 1, taken from the May 6, 1988 Federal Register, illustrates the Primary Frontal Dune limit described above.

44 CFR subsection 60.3(e)(7) of the NFIP regulations requires that participating communities prohibit manmade alterations of sand dunes in VE Zones which would increase potential flood damage. Thus, expansion of a VE Zone via the PFD expands the dune areas protected by 60.3(e)(7).

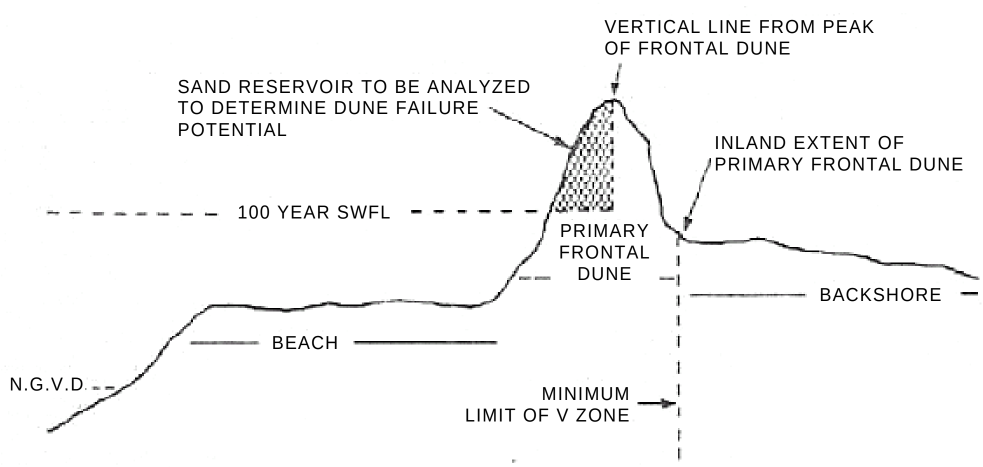


**Figure 1. Primary frontal dune definition sketch (May 6, 1988 Federal Register, p. 16271).**

FEMA also included a new section in 44 CFR Part 65, requiring that the storm-induced erosion cross-sectional area of 540 square feet serve as the basic criterion to be used in evaluating whether a primary frontal dune will act as an effective barrier during the 1-percent-annual-chance flood (see Event-Based Erosion report, Topic 37).

The guidance in *Appendix D* for primary frontal dunes as it pertains to 44 CFR Parts 59, 60,and 65 described above, needs some clarification (which is addressed in some detail in Critical Topic

17 to follow). *Appendix D* Section D.2.7.2 defines the VE Zone as the area where 1) 3-foot or greater breaking wave heights could occur; 2) the eroded ground profile is 3 feet or more below the representative runup elevation; and 3) the entire primary frontal dune, by definition. However, detailed criteria for determining the landward limit of the primary frontal dune are not provided. For hazard zone purposes the definitions in 44 CFR Part 59 are the key issues to be considered by this focused study.

### 2.1.3 Application of Existing Guideline to Topic – History and/or Implications for the NFIP

A general description of hazard risk zones and mapping criteria for VE Zones and other hazard zones is presented in *Appendix D* section D.2.7.2, "Identification of Flood Insurance Risk Zones". Within this section is the first mention of and example for mapping the primary frontal dune, described as follows:

> "Identifying appropriate zones and elevations may require particular care for dunes, given that the entire primary frontal dune is defined as Coastal High Hazard Area. Although the analyses may have determined a dune will not completely erode and wave action should stop at the retreated dune face with only overtopping possibly propagating inland, the Mapping Partner shall designate the entire dune as Zone VE. The Mapping Partner shall assign the BFE at the dune face for the remainder of the dune.
>
> It may seem unusual to use a BFE that is lower than the ground elevation, although this is actually fairly common. Most of the BFEs for areas where the dune was assumed to be eroded are also below existing ground elevations. In these cases, it is the VE Zone designation that is most important to the NFIP, under current regulations, structures in VE Zones must be built on pilings and prohibits alterations to the dune."

The method or practical approach for identifying the landward limit of the primary frontal dune (or heel of the dune) is not discussed, nor is there any clarification provided to assist the Mapping Partner in locating the point for the VE Zone termination based on the "relatively steep to relatively mild slope" criterion from the NFIP regulations (44 CFR sec. 59.1).

However, *Appendix D* does contain several figures that demonstrate inclusion of the primary frontal dune considerations in VE Zone mapping -- see Figure 2 below, from *Appendix D,* and other figures contained within the Appendix (1) located at the end of this report. *Appendix D* does not provide a complete overview of all possible examples for primary frontal dune VE Zone hazard mapping scenarios, but does provide some general examples of hazard assessment results and the mapping of hazard zones and BFEs.

<u>Explanation of Figure 2:</u> Schematic summary for three of the four criteria used to define the landward limit to the Coastal High Hazard Area (i.e., 3-foot wave height, 3-foot runup depth, and primary frontal dune–the fourth criterion, wave overtopping, does not apply to this example).

The VE Zone limit for each of the three applicable criteria is identified, and the VE/AE boundary placed at the one furthest landward, in this example, 3-foot runup depth. Note that in the majority

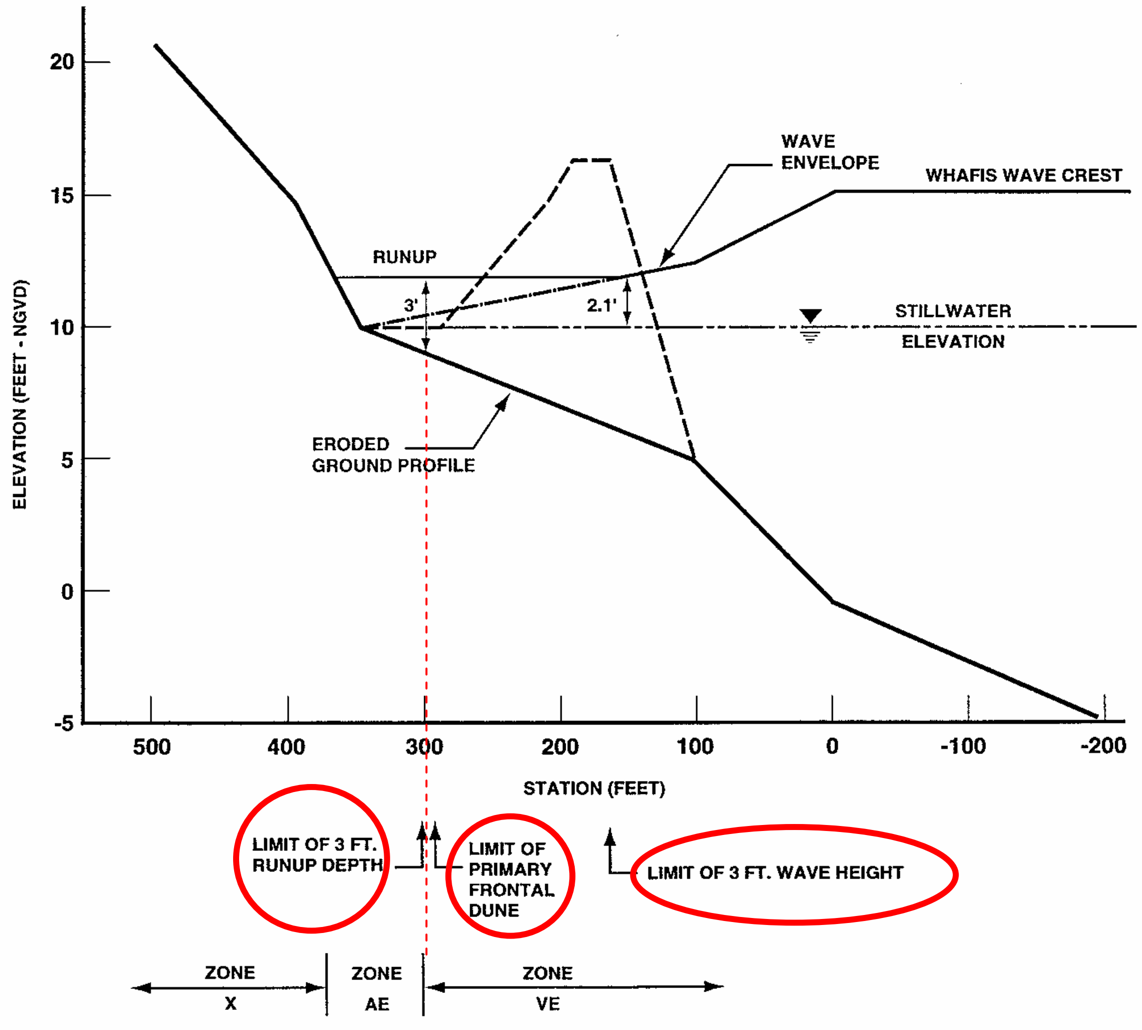


**Figure 2. Possible V-Zone limits at eroded dune.**

of cases with a primary frontal dune, the PFD criterion will be the most landward and will determine the VE limit.

Based on the experience of the Focused Study Group members in preparation and review of numerous coastal analyses and flood insurance studies, the definition of the primary frontal dune has a significant influence on coastal hazard mapping for the NFIP. In many areas, the definition of the primary frontal dune landward limit dominates the VE Zone mapping, yet no quantitative criteria are available for its determination.

In some cases, the PFD VE Zone shown on the FIRM is located landward of the portion of the SFHA subject to active coastal processes during the base flood (erosion, wave height, wave runup, and wave overtopping). Thus it has limited risk to coastal flooding. The landward portion of the PFD-based VE Zone in this scenario does not really represent an area of 1% risk to coastal flooding–it primarily serves as a floodplain management tool to protect dunes from alterations, and to regulate coastal construction practices and building standards. Moreover, the property owner's flood insurance premium rates are set at the maximum level, yet the property does not share a risk equivalent to that of other VE Zones within the SFHA that are mapped based on the effects of erosion and wave action.

***Primary Frontal Dune Definition – Current Practice***

The methodology used by Mapping Partners to define the landward heel of the primary frontal dune is presently very qualitative. The first step in mapping a VE Zone limit using the primary frontal dune criterion for mapping a VE Zone is to determine whether or not dune features meeting the regulatory definition are present in the study area according to the definition in 44 CFR Section 59.1. Note, however, that this definition is not currently provided in *Appendix D*.

If the study area includes primary frontal dunes, the next step requires detailed topographic information and/or beach and dune profiles taken from shoreline to a point landward of the dune feature. These data assist in the assessment of the point where the landward dune face transitions from a "relatively steep slope to relatively mild slope".

In a flood insurance study, the following data are desirable and the following methods typically applied:

- Topographic Mapping Basis
    - Topographic information with 1- to 2-foot contour intervals is needed to accurately determine where the slope change occurs.
    - From the topographic information, the primary frontal dune heel location will be identified and mapped based on the contour interval spacing and transitions (wide contour interval indicates mild slope and narrow contour interval spacing indicates steep slopes).
- Beach and Dune Cross Section (Profile) Basis
    - If beach and dune profiles are used instead of, or in addition to general topography, they should have sufficient vertical and horizontal resolution to capture all major slope transitions (e.g., the dune toe, peak and heel locations).
    - From beach and dune profiles, the point of the landward limit is established and then a smooth interpolation between profiles can be mapped with or without the use of topographic information (depending upon proximity of the profiles to each other and uniform nature of the primary frontal dune ridge).

- In certain situations, man-made impacts to the landward dune face may have altered the terrain (excavations for homes and appurtenant structures) and will indicate a false transition point for the primary frontal dune heel. In those scenarios, the natural dune feature and primary frontal dune heel location may be evaluated based on historic data, unaltered portions of the dune, and/or aerial photography.

It should be noted that FIRM scale limitations, and minor variations in qualitative assessments of primary frontal dune limits using topographic contours, can result in the inadvertent inclusion of the entire first row of homes located directly landward of the dune limits. Verification of mapping limits using aerial imagery and detailed beach and dune profiles can help resolve issues related to inadvertent inclusions in FIRM revisions based solely upon inclusion of the PFD VE Zone.

The methods outlined above are not documented or discussed in any of *Appendix D*. Additional guidance should be included to quantify the definition of the landward limit of the primary frontal dune where it serves as the basis for VE Zone determination. Topic 17 discusses the limitations associated with this use of the primary frontal dune definition for mapping coastal hazards in more detail.

### 2.1.4 Alternatives for Improvement

Based on the above review of the existing hazard zone guidance and problems with identification and mapping of landward limit of the primary frontal dune VE Zones, the following alternatives should be considered for *Appendix D* improvement:

Alternative 1- Revise the PFD definition and consider an improved definition of the slope transition, possibly as a percent of slope change in both the shore perpendicular direction and the shore parallel direction, with limited discretion on the part of Mapping Partners when they evaluate potential primary frontal dune features.

Alternative 2- The MA CZM has an integrated approach that uses high resolution aerial laser topographic data, Geographic Information Systems, and slope transition analyses to quantitatively define the primary frontal dune. The MA CZM proposed methodology provides an example of a technically defensible determination tool for a basic automated delineation of the primary frontal dune feature. There should be further review and consideration of this approach for application in coastal areas outside of MA, but the basic technical approach has merit for inclusion in the update to *Appendix D*.

### 2.1.5 Recommendations

The alternatives discussed above were discussed at Workshop 2 in Sacramento in February 2004, and recommendations developed based on the consensus of the Technical Working Group.

### 2.1.6 Related “Critical”, “Available,” and/or “Important” Topics

Related topics can be found in Focused Study Topics 17 and 18.

## 2.2 TOPIC 17: VE ZONE CRITERIA AND DEFINITIONS

### 2.2.1 Description of the Topic and Suggested Improvement

The workshop identified the needs for Flood Hazard Zones Topic 17 as follows:

*"Enhance existing guidelines for defining inland limit of VE Zone including the development of a basis for better guidance for heavily overtopped areas."*

The Focused Study Group determined that Topic 17 should include all coastal high hazard areas, not just “heavily overtopped areas.” The group evaluated criteria and mapping practices for all VE Zones. The group identified possible improvements to the guidelines and methodologies to best represent the projected hazards and damage potential for this unique SFHA. One goal of the study group is to provide better guidance to Mapping Partners, so that VE Zone inconsistencies are reduced.

The group subdivided Topic 17 into six issues, four associated with the existing definitions of the VE Zone coastal high hazard areas, and two new considerations, listed below as Topics 17 (a) to (f).

***Topic 17 (a) – Primary Frontal Dune VE Zones***

Topic 39 addressed the definition of the primary frontal dune and the delineation of its landward limit. Topic 17 addressed the mapping consequences and BFE discontinuities resulting from mapping VE Zones based on the primary frontal dune. For example, current guidance in *Appendix D*, section D.2.7.2 calls for extending the last computed wave height- or wave runup-based BFE at the dune face landward through the dune feature to the landward limit (heel) of the primary frontal dune. In cases where the landward limit of the primary frontal dune is located within a different flood source and stillwater elevation, or outside the area calculated to have a 1% annual chance of flooding, this can lead to sudden BFE changes of many feet (so-called “waterfalls”) and abrupt transitions in hazard zones. Portions of the areas within the primary frontal dune limits are mapped as VE zones, but may be outside the calculated 1% annual chance limits, and thus do not meet the regulatory definition of the SFHA.

These BFE and zone transition differences are a product of mapping procedures associated with the primary frontal dune definition, and not with analyses of water level or wave conditions. <u>The issue is inconsistency in the delineation of VE Zones using the primary frontal dune definition relative to other SFHAs, and uncertainty of how the VE Zone was established relative to the associated risk.</u> This issue has implications for insurance rating, building standards, and coastal land use.

### *Topic 17 (b) – 3-foot (or Greater) Breaking Wave Height VE Zones*

One NFIP definition of the VE Zone specifies that the hazard area is subject to breaking wave heights of 3 or more feet, which according to NFIP mapping procedures, occurs where stillwater depths equal or exceed 3.85 feet (waves are depth-limited, with the wave height limited to 0.78 times the stillwater depth–see Figure 3 below). A sub-topic for this study is to examine this VE Zone limit and to assess whether the 3-foot breaking wave height provides a reasonable definition of the coastal high hazard area, and whether a different wave height should be adopted as the VE Zone limit. This topic is closely related to an alternate approach that is considered in Topic 18 (Coastal A Zone).

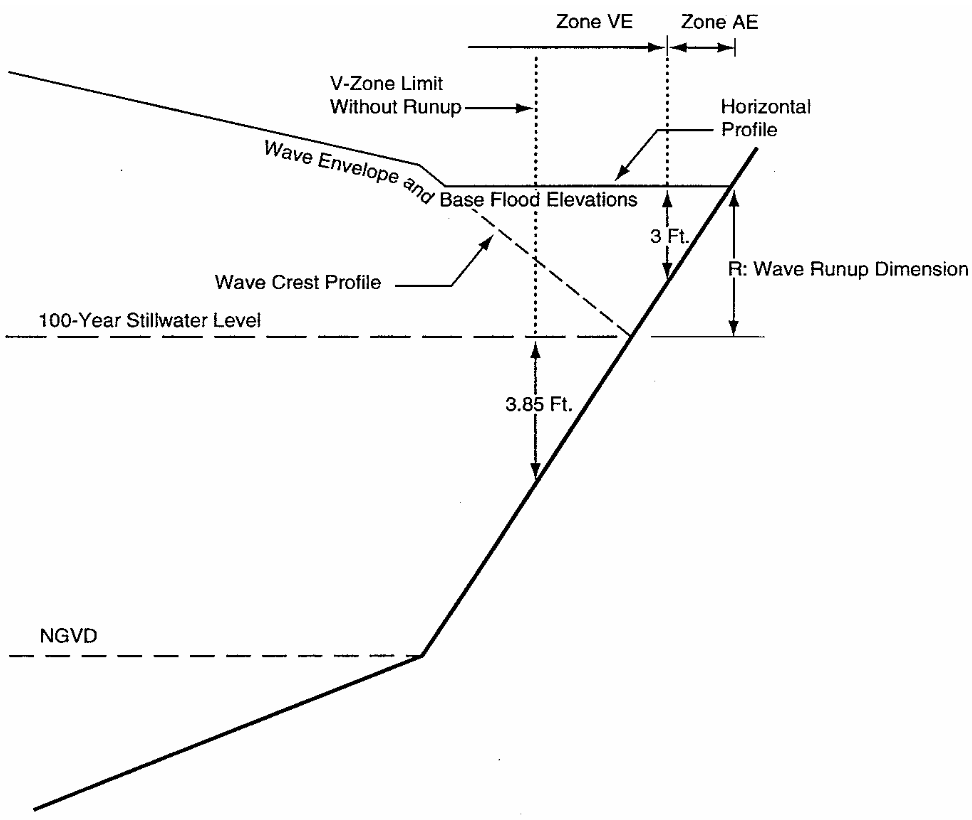


**Figure 3. VE Zone limits based on wave heights and wave runup.**

### *Topic 17 (c) – 3-foot (or Greater) Wave Runup Depth VE Zones*

Another VE Zone definition specifies that the hazard area is subject to wave runup depths of 3 or more feet (see Figure 3 above). A sub-topic for this study is to examine whether the 3-foot runup depth provides a reasonable and accurate limit to the coastal high hazard area.

***Topic 17 (d) – Wave Overtopping (Splash Zone) VE Zones***

The *G&S* also call for the mapping of a VE Zone to account for high levels of wave overtopping, in effect via a splash zone (approximately 30 to 50 feet), that extend landward of a coastal shore protection structure, such as a seawall, bulkhead, or revetment (see Table 4 in the Runup and Overtopping Focus Study). A sub-topic for this study is to review the trigger (mean overtopping rate) and lateral extent of the VE Zone, and to consider use of another coastal high hazard zone defined in the NFIP regulations to represent this type of hazard area – the VO zone.

***Topic 17 (e) –Wave-Cast Debris VE Zones***

The study group identified the need for a new classification of VE Zone or use of the VO Zone criteria for coastal areas that experience damaging wave-cast debris in conjunction with wave action and/or overtopping. This new zone would identify the area as a coastal high hazard and also account for the size, quantity and velocity of the wave-cast debris unique to this zone. Evaluation of this potential coastal high hazard zone criteria and mapping standards for a VE or VO Zone with wave-cast debris is necessary since none exist presently within *Appendix D*. This issue is also related to Topic 17 (d).

***Topic 17 (f) – Structural Load VE Zones***

The study group identified a second potential new VE zone definition that is based on structural loads that act on buildings and structures. There are a variety of existing VE zone definitions that may or may not be consistent in terms of the magnitude and effects of structural loads they induce. A new VE zone definition could be based on the loads themselves, rather than the proxies (e.g., breaking wave height, wave runup depth, mean overtopping rate) used at present. A new VE designation such as this could help guide the evaluation of mapping results discussed in *Appendix D*, section 2.7, “Prior to mapping the flood elevations and zones, the Mapping Partner shall review results from the models and assessments from a common-sense viewpoint and compare them to available historical data.” A structural load-based VE Zone criterion might aid in this review, and help resolve inconsistencies between these and other hazard zones and BFEs.

### 2.2.2 Description of Procedures in the Existing Guidelines

In *Appendix D*, Section D.2.7.2, “Identification of Flood Insurance Risk Zones”, the *G&S* present an overview of the various hazard zone mapping criteria for zones VE, AE, AO, AH, and X, considering the combined effects of storm-induced erosion, wave height, wave runup, wave overtopping, primary frontal dune, and coastal flood protection structures. The general lack of guidance on primary frontal dune definition (Sub-topic 17a) was discussed in Topic 39. Procedures for determining the wave height and runup characteristics (Sub-topics 17b and 17c) are included in the existing guidelines and discussed in separate focused studies on these subjects. Procedures for delineation of splash zones (Sub-topic 17c) are also included in the guidelines. Procedures for delineation of hazards due to wave cast debris (Sub-topic 17d) and structural load-based zones (Sub-topic 17e) are not presently included.

The *G&S* do not presently provide a complete set of examples for wave transect hazard mapping that are applicable in all situations. However, the *G&S* do include several figures that serve as general examples and descriptions of mapping results (these figures have been reproduced in Appendix (1) located at the end of this Focused Study report). It should be noted that all of these transect illustrations relate to Atlantic and Gulf Coast examples.

The examples in the existing guidelines show a WHAFIS wave crest envelope. The existing versions of WHAFIS will not produce results that would govern hazard zone delineation and BFEs on the Pacific Coast. WHAFIS in its present form is not appropriate for use on the Pacific Coast given its wind speed and vegetation subroutines. Based on past experience, the WHAFIS-type analysis for wave heights will not determine significant BFEs and flood hazard zones for most of the Pacific Coast due to low storm surge levels. On the Pacific Coast, extreme wave runup and wave overtopping flooding effects typically control the BFEs, flood depths, and hazard zone mapping criteria.

### 2.2.3 Application of Existing Guideline to Topic – History and/or Implications for the NFIP

As determined by the Focused Study Group, the following are key areas of concern. Resolution of each issue will improve the NFIP regulation and mapping of the VE Zone coastal high hazard areas.

#### *Topic 17 (a) – Primary Frontal Dune VE Zones*

There is no current mechanism in place to allow a Mapping Partner or FIRM reviewer to distinguish the active velocity zone portion of the VE Zone SFHA from the portion of the VE Zone SFHA that is based on the landward extension to the heel of the primary frontal dune Once the VE Zone is mapped, there is no way to distinguish the rationale for and accuracy of the zone and BFE.

Moreover, there are several implications – for the NFIP, participating communities, and owners of coastal property–of the current mapping procedures:

1. The current NFIP definition of "coastal high hazard area" includes the primary frontal dune, but is ambiguous as to where primary frontal dunes are designated. That is, should the NFIP definition ("Coastal high hazard area means an area of special flood hazard extending from offshore to the inland limit of a primary frontal dune along an open coast and any other area subject to high velocity wave action from storms or seismic sources.") be interpreted to mean: (1) primary frontal dunes are designated only along an "open coast" (open coast is not defined by the NFIP), or (2) primary frontal dunes are designated along an open coast and along any "area subject to high velocity wave action

from storms or seismic sources." The difference is critical. The May 6, 1988 Federal Register notice (see Event Based Erosion report, Topic 37, Appendix A) does not clarify this point. The exact meaning of the definition should be determined; especially since the applicability of the PFD designation to sheltered water shorelines is unclear and could be problematic in many Pacific Coast study areas.

2. If not for the primary frontal dune criterion, some portions of the VE zone would typically be mapped as AE or AO Zones, or Zone X (shaded or unshaded). Owners of buildings within PFD-based VE zones that would otherwise be mapped as less hazardous zones are faced with high flood insurance premium rates that do not reflect the actual base flood risk.

3. If not for the primary frontal dune criterion, BFEs along the analysis transects would reflect the 1% flood hazard, including the effects of event based erosion. BFEs based on the primary frontal dune designation can lie several feet higher than they would if mapped as less hazardous zones.

4. A problem associated with Issue (3) is the mapping of "waterfalls". This occurs when the transition from an extended Zone VE BFE to the primary frontal dune limit merges with a much lower BFE associated with Zone AE, Likewise, the sudden transition from a VE Zone to Zone X appears unusual without clarification of the primary frontal dune mapping criteria. In these cases, the landward limit of the primary frontal dune is located at point in the profile and coastal area that is above the stillwater elevation, has no wave effects, and is outside the SFHA entirely.

5. The designation of the primary frontal dune as a VE Zone not only supports hazard-specific building standards and land use requirements, but also protects the dune from man-made impacts or physical alterations. Any changes to the primary frontal dune mapping procedures should attempt to preserve the dune protection and construction standard requirements as intended in FEMA's floodplain management ordinances.

Several questions arise from this discussion above for Topic 17(a). The group feels that the following important questions should be addressed in the subsequent effort. The items presented will be addressed in more detail as part of the recommendations for Topic 17:

- Should primary frontal dunes be identified at dune features along all coasts and wave exposures, or should they be limited to open coast areas, or should they be limited to areas that would otherwise have VE zones designated (e.g., by wave height, wave runup or wave overtopping criteria)?

- Should FEMA identify the basis for VE zone designations (e.g., 3-foot wave height, 3-foot runup depth, primary frontal dune, and wave overtopping zone)?

- Should the portion of the primary frontal dune SFHA be represented by decreasing BFEs to its landward limit? Or should FEMA continue its current procedure of extending the last and most landward-calculated VE Zone BFE (from the wave height or wave runup analyses) to the landward limit of the primary frontal dune (see Figure D-28 in Appendix 1)?

- Should FEMA consider a new VE Zone classification or identification mechanism for the mapped portion of the SFHA for primary frontal dunes? Or should FEMA continue to map the area the same way it maps other velocity hazard zones even though the primary frontal dune is outside any historically known or calculated hazards?

- How should primary frontal dune VE Zones be merged into the adjacent AE Zones or X Zones to avoid BFE waterfalls or sudden transitions?

***Topic 17 (b) – 3-foot (or Greater) Breaking Wave Height VE Zones***

The identification of coastal high hazard areas (areas subject to high velocity waters and wave action) dates to the inception of the NFIP. The 3-foot wave height criterion was used, and continues to be used, as a dividing line between the coastal high hazard area (VE Zone) and other portions of the SFHA.

The Galveston District of the USACE (1975) produced guidelines for identifying coastal high hazard areas, part of which included an assessment of the "critical wave" (a wave possessing sufficient energy to cause major damage on contact with conventional structures). Appendix B of the 1975 USACE study concluded that the critical wave is a 3-foot breaking wave; however, a closer reading of calculations in Appendix B shows the USACE determined breaking waves 2.1 feet high are capable of destroying conventional wood-frame walls and connections. Nevertheless, the 3-foot standard was adopted by the 1975 USACE study, in recognition of the fact that conditions in the field (slope of ground, angle of wave attack, sheltering, etc.) may not be identical to those assumed in the study.

More recent full-scale laboratory tests of breakaway wall sections determined that breaking wave heights as low as 1.5 feet consistently cause failure of traditional stud wall construction. The tests were part of a larger effort to improve breakaway wall design standards (FEMA, 1999).

Post-Hurricane Opal (1995) studies at Pensacola Beach examined building damage caused by storm surge and wave heights in mapped VE and AE Zones, and determined that damages in AE Zones were consistent with damages in VE Zones (EQE, 2000). While the purpose of the study was to test FIA depth-damage functions for VE Zones and AE Zones, it indicates that wave heights less than 3 feet caused significant structural damage to AE Zone-type construction during the 1995 hurricane.

Taken together, the three studies suggest that the 3-foot wave height definition of the VE Zone may underestimate the extent of the coastal high hazard area.

The question that arises is whether FEMA should modify its VE Zone definition to one that uses a breaking wave height less than 3 feet. The Focus Study Group believes this concept should be explored fully. Note that this issue is closely tied to Topic 18, which considers subdividing the AE zone into a seaward "Coastal A Zone" where construction would be regulated like a VE zone, and a more landward AE Zone where current construction standards would be maintained. The expanded VE Zone and the Coastal A Zone should be considered mutually exclusive options – both accomplish the same goal, but in different ways.

### *Topic 17 (c) – 3-foot (or Greater) Wave Runup Depth VE Zones*

Wave runup based flood hazard zones were introduced in the early 1980s in New England. Prior to that point in time, wave heights were the only basis for VE Zone mapping. Present FEMA methods call for mapping a VE Zone seaward of the point where the ground elevation lays 3 or more feet below the mean runup elevation at the shoreline (see Figure 3). The origin of the 3-foot "runup depth" VE Zone criterion is unknown to the Focus Study Group, but it notes that the documentation for FEMA's first runup model states, "a criterion has been adopted which states that a runup value of less than 2 feet is incapable of causing significant damage" (Stone and Webster, 1981).

The 3-foot runup depth criterion should be evaluated under both tsunami runup and non-tsunami runup scenarios to determine if it provides an appropriate limit to the VE Zone.

The Focus Study Group also notes that there may be a consistency issue between the 3-foot wave height and 3-foot runup depth criteria, when the force of each on typical building components (piles, walls) is compared – see Topic 17 (f).

### *Topic 17 (d) – Wave Overtopping (Splash Zone) VE Zones*

There is a need to expand the coastal study guidance on wave overtopping criteria (wave and barrier characteristics, overtopping rate, high hazard zone limit) used to delineate the VE Zone hazard areas beyond overtopped dunes and shore protection structures. The existing guidance calls for VE Zones to be 30 feet wide where the mean overtopping rate exceeds 1.0 cfs/ft (see *G&S* Table D-7 and footnote that reads, "Appropriate inland extent of velocity hazards should take into account structure width, incident wave period or wavelength, and other factors").

Three aspects of this procedure should be evaluated: 1) the threshold overtopping rate used to map Zone VE (this assessment should be part of Topic 13, see the Runup and Overtopping Focused Study report), 2) the inland extent of the VE zone (i.e., provide more specific guidance than that contained in the footnote to Table D-7), which should be evaluated by the Flood Hazard Zones group, and 3) evaluation of the VO Zone designation for use in wave overtopping hazard identification. It is possible that additional factors such as velocity of flow (similar to alluvial fans) may help to expand and better define this hazard area. The VO Zone hazard assessment and identification procedures are not currently included in the guidance of *Appendix D*.

Development of specific hazard assessment criteria for VO Zone could enhance *Appendix D* with respect to these topics, and its evaluation is recommended by the focus study group.

Another approach, derived in a recent flood insurance study in Puget Sound for Whatcom County, Washington, and approved by FEMA, involved the use of an equation that describes the attenuation of an overtopping wave on a flat backshore surface (Cox and Machemehl, 1986). From a flood hazard standpoint, the Cox equation still appears to be a reasonable, simple approach to account for inland decay of wave heights over a distance to estimate flood hazards immediately inland of a shoreline where insured structures may be located.

***Topic 17 (e) –Wave-Cast Debris VE Zones***

FEMA mapping procedures currently define the limits of coastal high hazard areas based on wave action and the primary frontal dune, but fail to consider the wave-cast debris issue. However, flood damages resulting from drift logs (damaging building foundations) and wave-sprayed gravel (breaking windows) has been observed inland of mapped velocity zones. It would be useful to isolate and identify areas subject to damaging wave cast debris, possibly using existing or new hazard zones, special FIRM notes and FIS descriptions of the significance of the hazard. The role of winds in driving wave-spray gravel inland should also be investigated for future use and consideration in coastal hazard assessments. This effort should be tied closely with Topic 17(d) in this focused study, which deals with wave overtopping hazards, and with Topics 13 and 14 in the Wave Runup and Overtopping Focused Study.

***Topic 17 (f) – Structural Load VE Zones***

Preliminary calculations determined that wave runup loads on typical building components, for a 3-foot runup depth (i.e., at the runup-based VE Zone boundary), are of lesser magnitude than the loads due to a 3-foot breaking wave (at the wave height-based VE zone boundary). In fact, the runup loads are similar to loads resulting from breaking wave heights of approximately 1.5 feet. The inconsistency could be resolved by modifying the definition of the VE Zone criteria to use a lesser breaking wave height, or by increasing the wave runup depth used to define the VE zone limit, or by defining a new VE zone limit based on hydrodynamic loads resulting from wave effects. Of the three options, the second is probably contrary to observations of actual flood damages in post-storm assessments. The first was contemplated in Topic 17 (b). The third would be a new method that could resolve inconsistencies in existing coastal high hazard mapping procedures.

### 2.2.4 Alternatives for Improvement

Based on the above review of the existing hazard zone guidance and problems with identification and mapping of VE Zones, the group recommended the following alternatives be considered for *Appendix D* improvement:

- Retain VE zone mapping rationale. Develop ways to display the VE zone mapping rationale (e.g., wave height, runup depth, PFD, overtopping, etc.). Alternative methods include use of a DFIRM layer, archived back-up data, explanation in the FIS report, and designation on the FIRM.

- Revised primary frontal dune mapping procedures. Consider mapping BFEs across the primary frontal dune to minimize discontinuities at the landward limit. Alternatives include use of alternate flood hazard zones in areas where the primary frontal dune dominates. These could include approximate V, D, E, VO, AO, coastal A zone (see Topic 18), or a new PFD zone.

- Revised wave height VE zone criterion. Consider mapping Zone VE using a lesser breaking wave height. Prior studies suggest something on the order of a 1.5- to 2.0-foot wave height might be appropriate. In effect, this alternative would capture some of the area presently mapped as Zone AE, and is closely related to another alternative described in Topic 18.

- Revised wave runup depth VE zone criterion. Consider mapping Zone VE with a wave runup depth other than 3.0 feet. The selection of the appropriate runup depth will depend on the flood source (tsunami, hurricane, other coastal storm) and should reflect flood hazards generally consistent with the wave height VE criterion.

- Revised wave overtopping VE/VO zone criterion. Consideration of new VE Zone definition and/or proposed refinement and utilization of the VO Zone could improve the guidance. The VO Zone is listed in the NFIP regulations, but details regarding its possible mapping and use are not included. The VO zone would be appropriate for a variety of coastal hazard zones subject to high overtopping rates and/or flow velocities, whether on the open coast or on sheltered water shorelines. The VO Zone could be considered for mapping wave runup on the foreshore slopes or sheet flow down the backshore slopes of beaches and dunes (sandy, cobble or other), low coastal ridges, and coastal structures. A simplified procedure outlined in the current *G&S* for mapping wave overtopping AO Zones (which makes a transition from 3 feet or greater depth of wave runup overtopping into a Zone AO, Depth 2 feet, and for less than 3-foot depth of wave runup overtopping a transition into a Zone AO, Depth 1 foot), is shown in Figure 4 (from Appendix D, Figure D-15). This simplified concept can be improved to better mimic the actual physical processes and flood risk, and expanded to consider more energetic flow regimes in VO zones.

- Wave-cast debris hazard delineation. One key problem to be faced is how to evaluate and identify severely overtopped areas with damaging debris loads, separately from the typical VE zone designations (e.g., 3-foot wave height, 3-foot wave runup, and primary frontal dune VE Zones). A methodology is needed and mapping guidance would have to be developed. Data from the Pacific Northwest and New England should be reviewed and potential methods should be formulated and evaluated. Alternatives include a specific zone designation for wave cast debris hazards, or development of methods to consider wave cast debris in delineation of existing or modified zones.

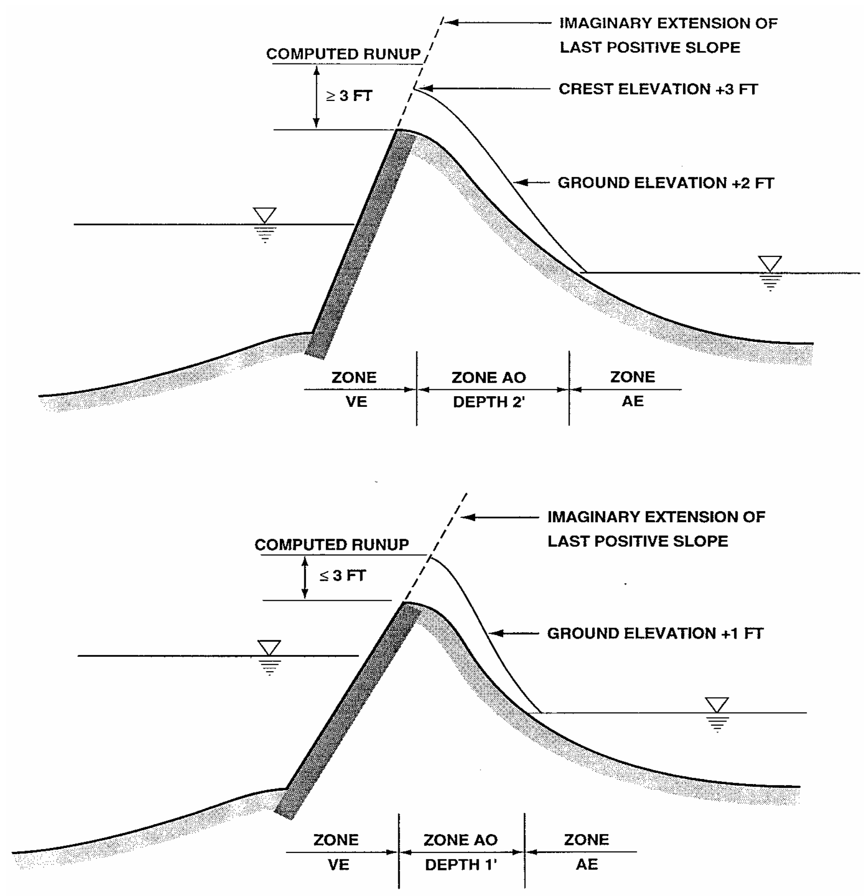


**Figure 4. Simplified runoff procedures, Zone AO (FEMA, 2003).**

- Structural loading VE zone criterion. A possible new VE zone criterion, based on flood loads, should be developed and considered. Such a method should be based on field damage reports and analytical calculations of flood and wave forces acting on typical building components (e.g., piles, columns, walls). The method could be used as a check on other VE zone criteria, and as a way to resolve discrepancies between mapping resulting from those criteria.

### 2.2.5 Recommendations

The alternatives discussed above were discussed at Workshop 2 in Sacramento in February 2004, and recommendations developed based on the consensus of the Technical Working Group. For

Topic 17, the summary of recommendations based on the issues and alternatives presented above include:

1. Investigate and develop guidance to better map the BFE transition between PFD dominated VE Zones and landward SFHA (and Zone X) hazard zones, and test and apply these procedures in a case study;

2. Establish procedures (hazard identification and mapping) to better utilize VO Zones;

3. Establish procedures for identifying and mapping hazard zones for wave overtopping and wave-cast debris hazards; and

4. Establish improved procedures for establishing the landward limit of the PFD (also related to Topic 39 discussions above).

#### 2.2.6 Related "Critical", "Available," and/or "Important" Topics

For Flood Hazard Zones Topic 17, related topics are Topic 39, which discussed the primary frontal dune definition, and Topics 11 to 14 in the Runup and Overtopping Focused Study. Recommendations for the Critical, Important, and Available issues of Topics 11 to 14 that might be applicable to Flood Hazard Zones Topic 17 could be related to new data sets to change hazard definitions or procedures for VE Zone mapping.

# 3 AVAILABLE TOPICS

Topic 19 was determined to be a priority "A" (Available) for the geographical regions of the Atlantic, Gulf, Pacific and Sheltered Waters. Available topics were considered to have information readily available for incorporation into revised guidelines. While an extensive compilation of this information (e.g., data sets, programs, procedures, etc.) is not a part of the focused study, the classification of the topic should be confirmed based on review of the available information and its potential for rapid use in study guidelines.

## 3.1 TOPIC 19: MAPPING FOR COMBINED COASTAL-RIVERINE PROBABILITIES

The description for Topic 19 is as follows:

*"Flood risk management of combined coastal and riverine flood hazards."*

As described, Topic 19 is considered to be an update to implement previous guidance and provide some clarification on its application to coastal flood studies.

### 3.1.1 Description of the Topic and Suggested Improvement

The current *G&S* do not provide guidance for flood level determination or mapping standards for combined coastal-riverine flood hazard areas. Past guidance is available from FEMA coastal storm surge modeling documentation prepared by Tetra Tech (FEMA, 1981) that includes specific guidance for flood level determination, but none is available for special mapping standards for this combined hazard zone. Therefore, the Topic 19 assessment reviews previous FEMA guidance for determining and mapping flood hazard areas when coastal backwater flooding combines with a riverine flood profile – referred to as the combined coastal-riverine hazard area. The combination of coastal and riverine flood hazards has often been overlooked in past coastal and riverine flood studies. These combined coastal-riverine hazard areas may occur far inland from the open coast, and are common in sheltered water areas.

To prepare the Flood Hazard Zones Focused Study assessment for Topic 19, the group performed the following tasks:

- Reviewed previous and any existing guidance for determining and mapping the area between the coastal backwater flood hazard area and riverine profile, referred to as the combined coastal-riverine hazard area.
- Reviewed previous Tetra Tech guidance in the FEMA "Coastal Flooding Handbook" from 1978 and/or FEMA "Coastal Storm Surge Model Users Manual" from 1981 related to this topic for consideration and application in future revision to the *G&S*.
- Reviewed/summarized other published methods to consider joint probability analyses of coastal-riverine flooding effects (combined or independent) to determine range of methods available.
- Identified potential case study coastal FIS examples related to combined coastal-riverine flood hazard study areas.

At this time, the following work is recommended to improve the *G&S*:

- Compile the best available prior guidance from previous FEMA publications (FEMA, 1981) to help clarify and develop new mapping standards for inclusion in *Appendix D*.
- Compile an annotated bibliography of related papers and publications in support of similar or new methods for identifying and mapping flood hazard areas of combined coastal-riverine influence using joint probability techniques.
- Identify the specific sections of *Appendix D* or other sections of the *G&S* that need to be revised or enhanced to include guidance on how to conduct the assessment and mapping of combined coastal and riverine areas.

- Perform a case study and prepare an example using a previously studied coastal area to demonstrate the improvement in guidance on combined coastal-riverine mapping (including standard notes and mapping methods).

During the recent restudy of flood hazards for Pasquotank County, NC, the Mapping Partner required guidance regarding delineation of combined coastal-riverine flooding on the FIRM and in the FIS report. The combined coastal-riverine probability and flood level determination methodology from the FEMA *Coastal Flooding Handbook* (1978) prepared by Tetra Tech were used in the effective Pasquotank County, unincorporated areas FIS report, and have also been used in other areas of North Carolina.

Further investigation may find that little or inconsistent consideration of combined coastal-riverine flooding is occurring in FEMA flood studies in other areas, and that BFEs may be underestimated as a result. For example, if the riverine and coastal flood events are assumed to be independent, the probabilities of the events can be added together to estimate the combined probability of both events. The point at which the 1% annual chance river flood elevation profile passes through the coastal 1% annual chance surge elevation is therefore 0.01 + 0.01 or 0.02, which is the 2% annual chance (50-year) flood event. This is further explained in the following insert.

**Combined Effects: Surge Plus Riverine Runoff**

The following example concerns the determination of the 1% stillwater flood level in a tidal location subject to flooding by both coastal and riverine mechanisms. This is the case in the lower reaches of all tidal rivers.

It is assumed that the extreme levels from coastal and riverine processes are independent, or at least widely separate in time. This assumption is generally true since the storms which produce extreme rainfall and runoff may not be from the same set as the storms which produce the greatest storm surge. Furthermore, if a single storm does produce both large surge and large runoff, the runoff is usually delayed because of overland flow, causing the runoff elevation to peak long after the storm surge. Clearly, there may be particular storms for which these assumptions are not true, but even so they are not expected to be so common as to strongly influence the final statistics.

Given these assumptions, the Study Contractor can determine the appropriate combined flood frequencies by a simple procedure. For a range of elevations covering all elevations of interest at a particular point, one determines the rate of occurrence of that elevation from surge alone, and the rate from riverine runoff alone. The total rate of occurrence of that level is then just the sum of the two contributing rates. This process must be repeated at intervals along the tidal river, from the mouth upstream to a point where the coastal influence in negligible.

This procedure is discussed in more detail in Subsection D.4.8.2. Note that at the coast, the total elevation frequency curve is just that of the surge, since the river runoff cannot raise the ocean level. Conversely, at a distance upstream, the total elevation frequency curve approaches that of the riverine flood. Note, too, that at the intermediate point where the individual 1% profiles of the two floods cross, the crossing elevation equals the 2% (50 year) level, since it is assumed to occur twice in 100 years.

Two problems arise from the lack of guidance for this issue: 1) Inconsistent methodology and application; and 2) Inconsistent presentation of flood elevations in FIRM mapping, flood profile, and floodway data table. The current *G&S* for the issue of combined probabilities does not provide the contractor any background or technical discussion of the problem.

The notation of the combined riverine and coastal flood hazard zone is not clearly defined on the FIRM, flood profile, or the floodway data table. As a result, the FIS report (including coastal stillwater elevation tables and floodway data tables) does not match the data shown on flood profiles and depicted on the FIRM. Of course, this causes some confusion for the communities using the maps as well as subsequent restudies by Mapping Partners.

The recommended approach includes two items that the group determined are in need of clarification: the existing *G&S* needs to be modified to describe the process to combine the probability and flood levels; and the FIRM should be able to distinctly clarify hazard zones and/or flooding reaches affected by the combined probability adjustments.

### 3.1.2 Confirm "Availability"

*Appendix D* in Section 1.2.6 simply states the following regarding Topic 19:

*"Describe and report adjustments to account for the combined probability of coastal and riverine flooding for each area where such an approach was taken."*

The addition of text similar to that provided in previous guidance documents (specifically "Coastal Flooding Storm Surge Model, Part 1, Methodology", February 1981) would greatly improve the existing *G&S*. The referenced report provides a brief narrative presenting the technical problem and appropriate methodology to combine the probabilities and adjust the flood levels accordingly.

Further, this Focused Study recommends new guidelines be developed to clarify the appropriate documentation and description of the combined probability in the FIS text, including floodway data tables and flood profiles. This text should modify section D 1.2.6 of the current *G&S*.

A recent North Carolina mapping update for some combined coastal-riverine areas in Pasquotank County, NC, resulted in the placement of a special note and flood gutter on the FIRM at the lower and upper boundaries of the combined coastal-riverine area that coincided with the same locations on the flood profile. This provided clarification of exactly where this combined coastal-riverine flood hazard area is located along the respective reach of the river. This combined coastal-riverine hazard zone was studied and mapped for many coastal areas in North Carolina. This Focused Study recommends that FEMA consider the mapping approach applied in North Carolina as the basic method for combined coastal-riverine SFHA mapping, and adjust, as necessary, to meet FEMA's nation-wide needs.

The required methodology is confirmed to be generally available and can be incorporated into a new or updated guidelines.

### 3.1.3 Recommendations

The following recommendations were developed for Topic 19:

1. Review the previous guidance from 1981 for adoption into *Appendix D*; and

2. Develop mapping standards to clearly identify areas affected by combined coastal and riverine flood hazards on FIRMS, flood profiles and floodway data tables.

# 4 IMPORTANT TOPICS

As described above, Workshop 1 resulted in the preparation of matrices that listed each of the numbered topics with specific category groupings and specific priority classes. Topic 18 was classified as "Important" ("I"). Topic 18 was determined to be a priority "I" for the geographical regions of the Atlantic, Gulf, Pacific and Sheltered Waters.

## 4.1 TOPIC 18: ADEQUACY OF VE AND AE ZONE DEFINITIONS

The needs description for Flood Hazard Zones Topic 18 is as follows:

*"Investigate the appropriateness of existing VE and AE Zone definitions for coastal areas."*

In essence, Topic 18 asks whether VE and AE Zone mapping methods accurately distinguish and delineate two hazard zones representing different risks and BFEs, but within each of which common building standards can be applied. The Topic 17(b) discussion indicates that mapping VE Zones using the 3.0 ft wave height criterion may fail to capture all of the coastal high hazard area. Topic 18 presents an alternate approach – that of leaving the 3.0 ft VE Zone criterion intact and subdividing the AE Zone into two portions: 1) a more seaward portion of the AE Zone (exposed to direct flood and wave effects from a principle flood source) where hazards are similar in nature (but reduced in magnitude) to the VE Zone, but where VE Zone building standards are deemed to be appropriate, and 2) a more landward portion of the AE Zone where wave effects are negligible and traditional AE Zone building standards are appropriate. The more seaward portion is often referred to as the "Coastal A Zone". The paper included in Appendix 2 provides background information on the proposed Coastal A Zone designation.

### 4.1.1 Description of the Topic and Suggested Improvement

The Coastal A Zone concept investigated in Topic 18 has been or is being employed in several instances. For example:

1. FEMA's revised Coastal Construction Manual (FEMA, 2000) promotes use of VE Zone construction techniques in designated AE Zones subject to waves and erosion. The value of this practice was borne out in Pensacola Beach and Navarre Beach, FL, during Hurricane Opal (1995) when damages to newer pile-supported buildings in AE zones were minimal, while damages to older style construction (on shallow footings with wall-type construction) were more extensive.

2. FEMA's Community Rating System (CRS) awards points toward reduced flood insurance premiums for communities that adopt VE Zone-type construction standards in AE Zones that are subject to coastal flooding and wave effects.

3. The American Society of Civil Engineers (ASCE) national load standard for buildings, ASCE-7, differentiates between load combinations in riverine-type AE Zones and coastal-type AE Zones (the latter uses the same load combinations as in coastal VE Zones);

4. ASCE-24 (standard for flood-resistant construction) and ASCE 7-02 are being updated with a recommendation to apply similar construction practices in Coastal A Zones as in VE Zones – e.g., pile supported buildings instead of slab on grade – (both ASCE-7 and ASCE-24 will, in the next editions, likely support use of the 1.5-foot breaking wave height as the landward limit of the Coastal A Zone), as shown in Figure 5 below. The focus study group finds it preferable to pursue the Coastal A Zone concept as opposed to trying to change the VE Zone wave height criteria to the 1.5-foot breaking wave height. The VE zone delineation has been closely associated with the 3-foot wave height for over 30 years, and changing that designation would be difficult. Moreover, there may be legitimate reasons (i.e., related to flood insurance premiums, floodplain management, and land use policy) to differentiate between VE zones and the Coastal A Zone – those issues must be identified and considered carefully as work on Topic 18 goes forward.

5. The focus study group also acknowledges that adoption of the Coastal A Zone by the NFIP will probably require changes to NFIP regulations, a process which can often be time consuming and where the outcome is uncertain. Nevertheless, the identification of the Coastal A Zone – even if not linked to mandatory NFIP building regulations – is of value as a hazard identification and public outreach tool. It can also provide communities that wish to exceed minimum NFIP requirements with a consistent and defensible approach.

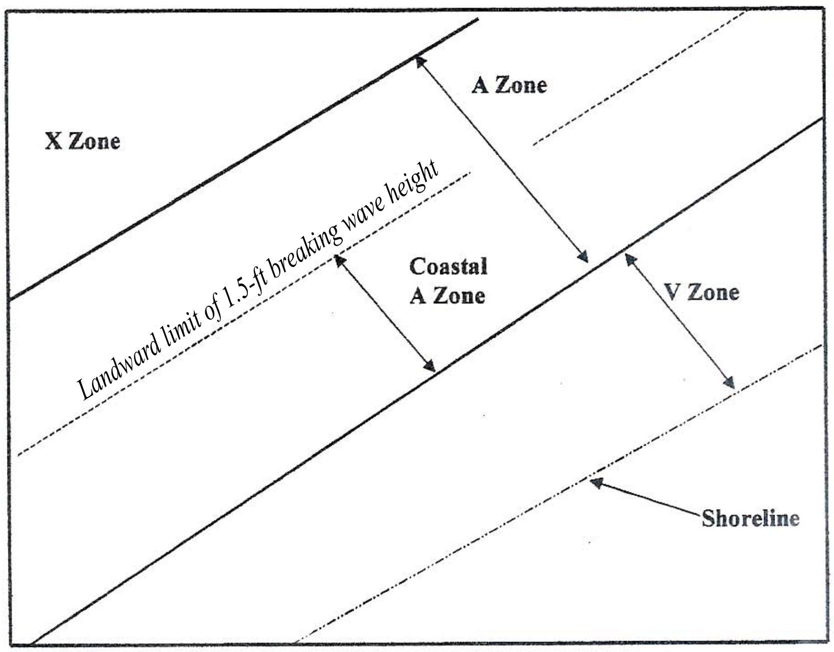


**Figure 5. Proposed coastal A Zone mapping concept being considered for revisions to ASCE 24-98 and ASCE 7-02.**

It is hoped that the inclusion of new guidance in Appendix D for mapping the Coastal A Zone high hazard area (as AE Zone with wave effects in the BFE) can enhance the delineation of the wave-affected floodplains.

### 4.1.2 Description of Potential Alternatives

The adoption of the Coastal A Zone concept is considered a priority, and may address problems associated with existing VE Zone and AE Zone delineations, and reduce storm damage outside of the designated VE Zone.

Some of the work required to support the above goals for Topic 18 would include:

- Identify Coastal A Zone Criteria: The 1.5-foot breaking wave height seems to represent a logical wave height division between the Coastal A Zone and the remainder of Zone AE. Using this as a starting point, develop criteria to address other VE zone hazards (wave runup, wave overtopping, PFD, wave-cast debris, etc.) on a consistent basis. The criteria should be developed so that they can be applied to all coasts (Atlantic, Gulf, Pacific, Great Lakes) and all wave exposures (open coasts and sheltered). This effort will also require an assessment of the NFIP regulations in 44CFR, and *Appendix D* sections, to clarify any regulatory and technical changes that must be made.

- Post-Storm Data Analysis: Perform additional analyses of post-storm hazard and damage data to refine the Coastal A Zone delineation. At present, limited data sets exist (Pensacola Beach, FL, Hurricane Opal, 1995; a soon to be completed study of Topsail Island, NC, Hurricane Fran, 1996). Additional analyses should be performed for shorelines where wave runup and overtopping dominate base flood hazards.

- Mapping Partner and Community Guidance: Prepare new FEMA Policy Memos and Technical Bulletins to help clarify the revised AE Zone (including Coastal A Zone) definitions and their application to the NFIP. These publications should include a bibliography of references and literature that would support expanded or new guidance.

- Case Study: select a community and test Coastal A Zone delineation methods. Include wave height, wave runup, wave overtopping, wave-cast debris and primary frontal dune considerations.

### 4.1.3 Recommendations

The following recommendations were developed for Topic 18:

1. Investigate and develop Coastal A Zone criteria (wave and erosion damage) and procedures for application within the NFIP;

2. Prepare technical bulletins for clarification of proposed revisions to VE Zones, AE Zones, and new VO Zones related to hazard identification and floodplain management;

3. Develop an annotated bibliography of related research and papers to support new guidance; and

4. Apply new concepts in a case study area.

# 5 SUMMARY

This Focused Study addresses two critical topics (39 and 17) dealing with flood hazard zone definitions and means for hazard delineation. Available topic 19 is discussed regarding mapping of combined coastal and riverine hazard zones. Adequacy and needs to define alternative hazard zones are discussed. Recommended approaches for addressing these issues are presented.

**Table 1. Summary of Findings and Recommendations for Flood Hazard Zones**

| Topic Number | Topic | Coastal Area | Priority Class | Availability/ Adequacy | Recommended Approach | Related Topics |
|---|---|---|---|---|---|---|
| 39 | PFD | AC | C | MAJ | (1) Consider an improved and refined definition of the PFD slope transition as revision to NFIP regs, (2) provide further technical guidance in *Appendix D* to clarify the PFD mapping criteria, and (3) consider adoption of new quantitative methodologies for identification and mapping (e.g., MA CZM) | 17, 18 |
| | | GC | C | MAJ | | |
| | | PC | I | PRO | | |
| | | SW | I | PRO | | |
| 17 | VE Zone Limit | AC | C | MAJ | (1) Investigate and develop guidance to better map the BFE transition between PFD and landward hazard zones, and apply in a case study; (2) establish procedures (hazard identification and mapping) to better utilize VO Zones; (3) establish procedures for identifying and mapping wave overtopping and wave-cast debris hazard zones; and (4) establish improved procedures for establishing the landward limit of the PFD (see Topic 39). | 39, 11, 12, 13, & 14 |
| | | GC | C | MAJ | | |
| | | PC | C | MAJ | | |
| | | SW | C | MAJ | | |
| 19 | Combined Coastal/ Riverine | AC | A | Y | (1) Review the previous guidance from 1981 for adoption into *Appendix D*, and (2) develop mapping standards to clearly identify this hazard zone | N/A |
| | | GC | A | Y | | |
| | | PC | A | Y | | |
| | | SW | A | Y | | |
| 18 | VE/AE Zone Appropriateness | AC | I | PRO | (1) Investigate and develop Coastal A Zone criteria (wave and erosion damage) and procedures for application within the NFIP; (2) prepare technical bulletins for clarification of proposed revisions to VE Zones, AE Zones, and new VO Zones related to hazard identification and floodplain management; (3) develop an annotated bibliography of related research and papers to support new guidance; and (4) apply new concepts in a case study area. | 11, 12, 13, & 14 |
| | | GC | I | PRO | | |
| | | PC | I | PRO | | |
| | | SW | I | PRO | | |

Key:
Coastal Area
AC = Atlantic Coast; GC = Gulf Coast; PC = Pacific Coast; SW = Sheltered Waters
Priority Class
C = critical; A = available; I = important; H = helpful
(Recommend priority italicized if focused study recommended a change in priority class)
Availability/Adequacy
"Critical" Items: MIN = needed revisions are relatively minor; MAJ = needed revisions are major
"Available" Items: Y = availability confirmed; N = data or methods are not readily available
"Important" Items: PRO = procedures or methods must be developed; DAT = new data are required; PRODAT = both new procedures and data are required

| Table 2. Time Estimates for Flood Hazard Zones Topics | | |
|---|---|---|
| Topic Number | Topic | Time (person months) |
| 39 | **Landward Primary Frontal Dune Limit and Definition** | |
| | Refine PFD definition in NFIP regs | 0.5 |
| | Review PFS mapping criteria | 1 |
| | Review and consider new PFD identification and mapping methods | 2 |
| | TOTAL | 3.5 |
| 17 | **VE Zone Criteria and Definitions** | |
| | Revise PFD mapping criteria | 0.5 |
| | Review Coastal A Zone mapping criteria | 1 |
| | Define VO Zone mapping & determination criteria | 2 |
| | Review alternative VE Zone wave height | 0.5 |
| | Define wave cast debris VE Zone criteria | 1 |
| | Prepare examples of new criteria | 2 |
| | TOTAL | 7 |
| 19 | **Mapping for Combined Coastal-Riverine Probabilities** | |
| | Convert 1981 and/or new additional guidance for inclusion to *Appendix D* | 1.5 |
| | Prepare FIRM mapping criteria for combined coastal-riverine SFHA | 1 |
| | TOTAL | 2.5 |
| 18 | **Adequacy of VE and AE Zone Definitions** | |
| | Define the Coastal A Zone revisions needed for NFIP regulations & mapping criteria | 2.5 |
| | Review FEMA technical bulletins needs for revised/ new VE, VO, and AE zones. | 0.5 |
| | Case study for revised/new VE, VO, and AE mapping definitions (including PFD) | 1 |
| | Annotated bibliography preparation | 0.5 |
| | TOTAL | 4.5 |
| **Flood Hazard Zones Preliminary Time and Cost Estimate Totals by Topic** | | |
| 39 | Landward Primary Frontal Dune Limit and Definition | 3.5 |
| 17 | VE Zone Criteria and Definitions | 7 |
| 19 | Mapping for Combined Coastal-Riverine Probabilities | 2.5 |
| 18 | Adequacy of VE and AE Zone Definitions | 4.5 |
| | TOTAL | 17.5 |

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# APPENDIX 1

**Coastal Mapping Examples taken from FEMA (2003)**

***Guidelines and Specifications for***

***Flood Hazard Mapping Partners***

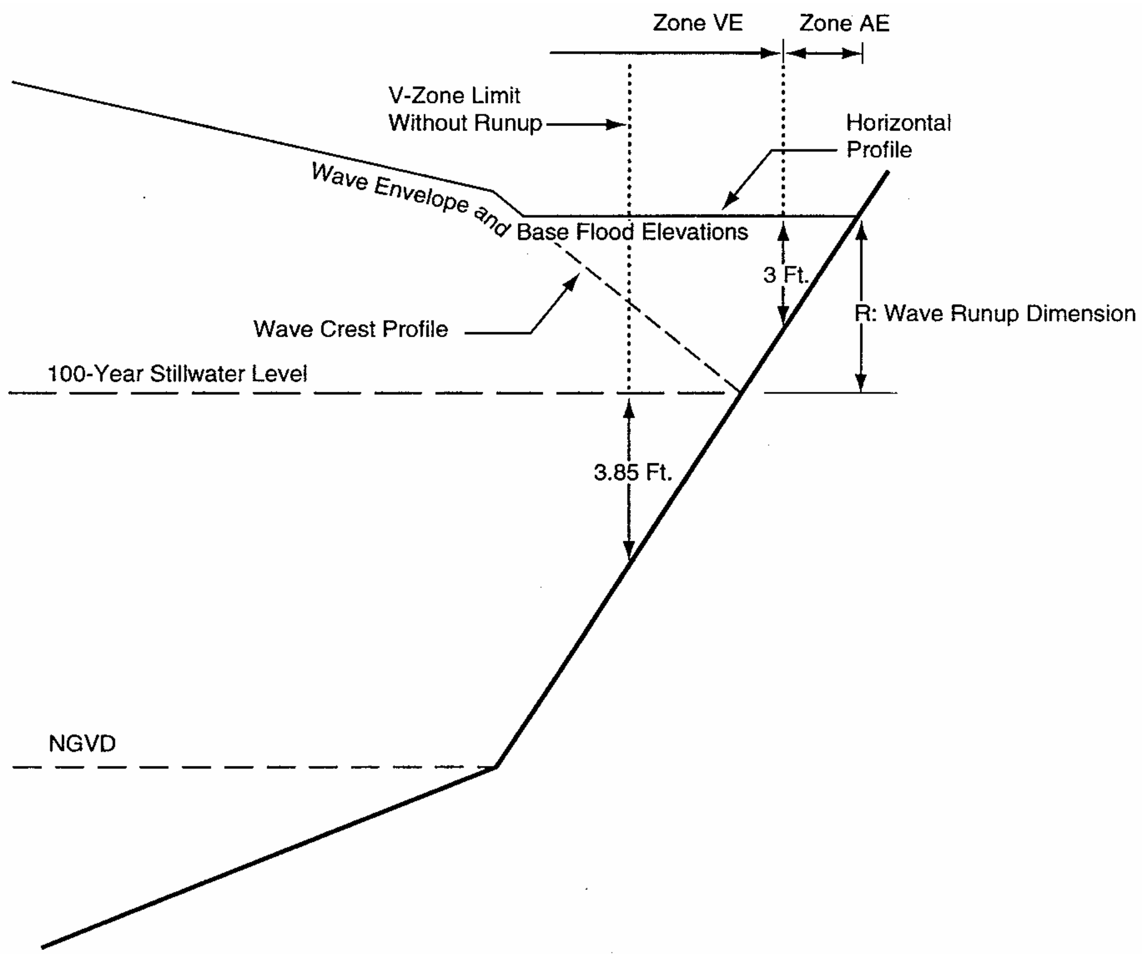


**Figure D-25. Wave Envelope Resulting from Combination of Nearshore Crest Elevations and Shore Runup Elevation**

Explanation of Figure D-25: "Shows that the wave envelope is a combination of representative wave runup elevation with the controlling wave crest profile determined by WHAFIS. The wave crest profile is plotted on the transect from the data in Part 2 of the WHAFIS output. A horizontal line is extended seaward from the wave runup elevation to its intersection with the wave crest profile to obtain the wave envelope, as shown in Figure D-25."

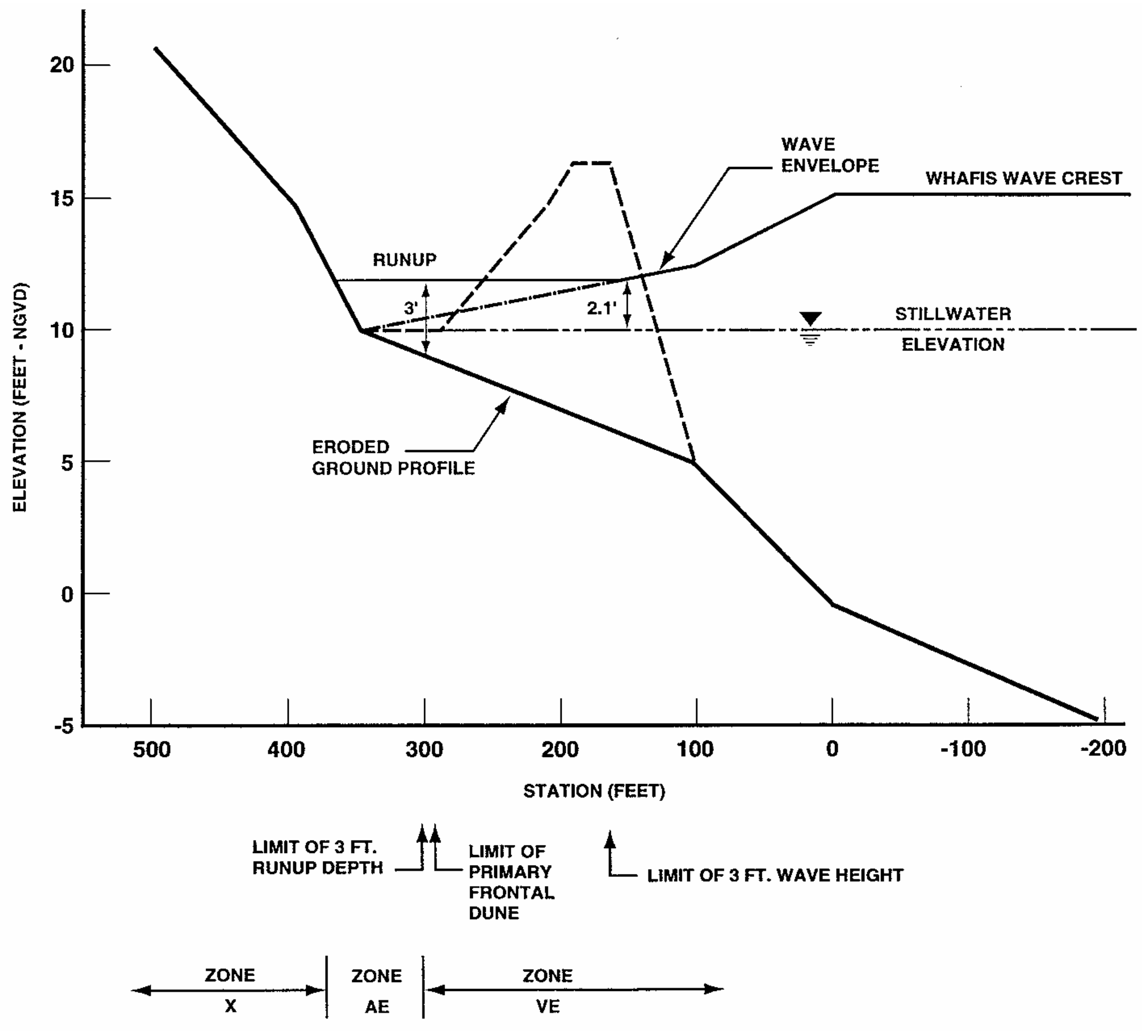


**Figure D-26. Possible V-Zone Limits at Eroded Dune**

Explanation of Figure D-26: "Provides a schematic summary for the three criteria potentially defining the landward limit to the Coastal High Hazard Area. The VE zone limit for each of the three criteria is identified, and the VE/AE boundary placed at the one furthest landward, as shown in Figure D-26."

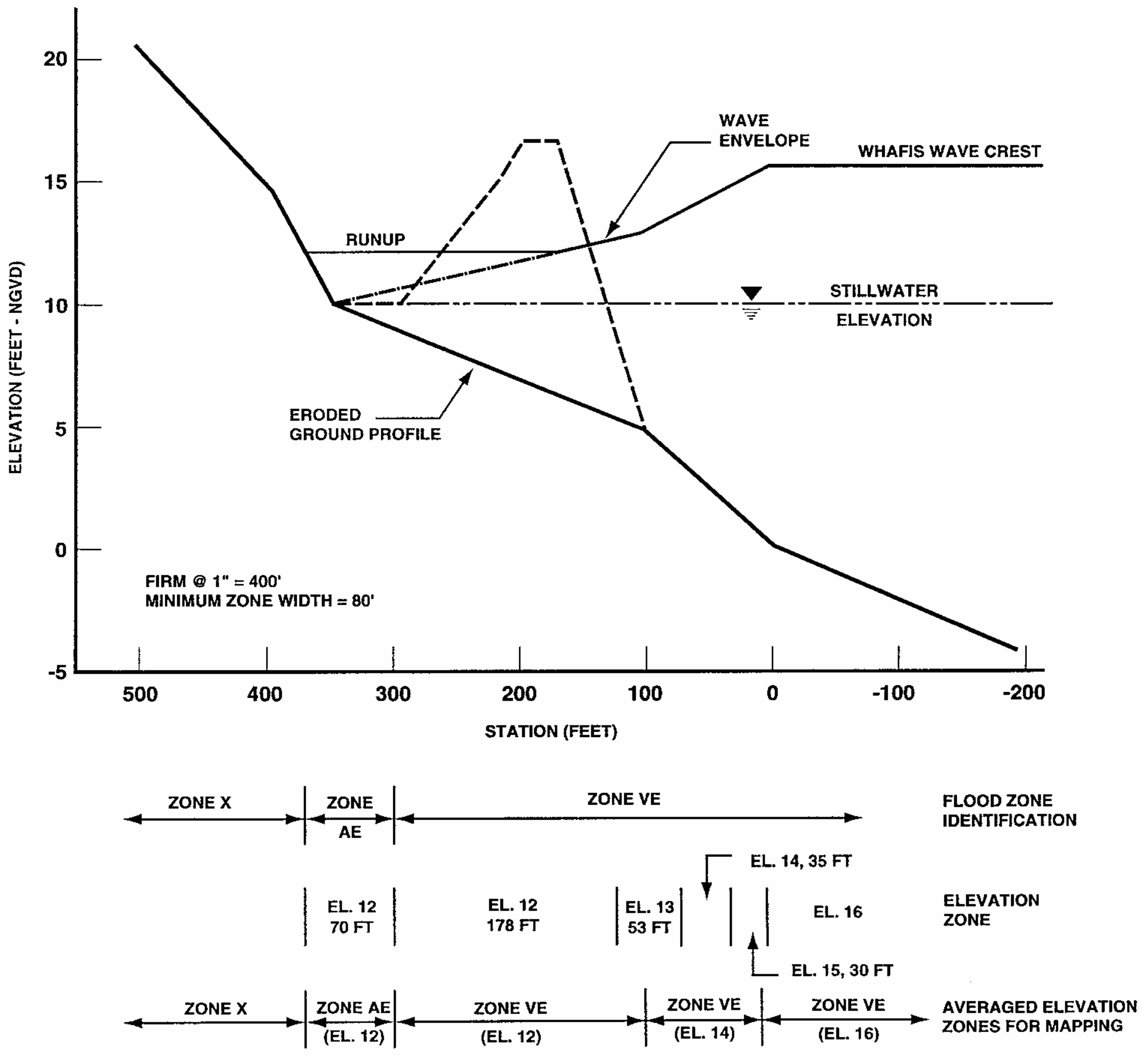


**Figure D-27. Identification of Elevation Zones, Example 1: Dune Removal with Wave Runup Landward**

<u>Explanation of Figure D-27:</u> "Presents an example of dune removal with appreciable runup occurring on the eroded profile. For this transect, the VE Zones with BFEs of 13, 14, and 15 feet are too narrow to be mapped, so they are averaged to a BFE of 14 feet."

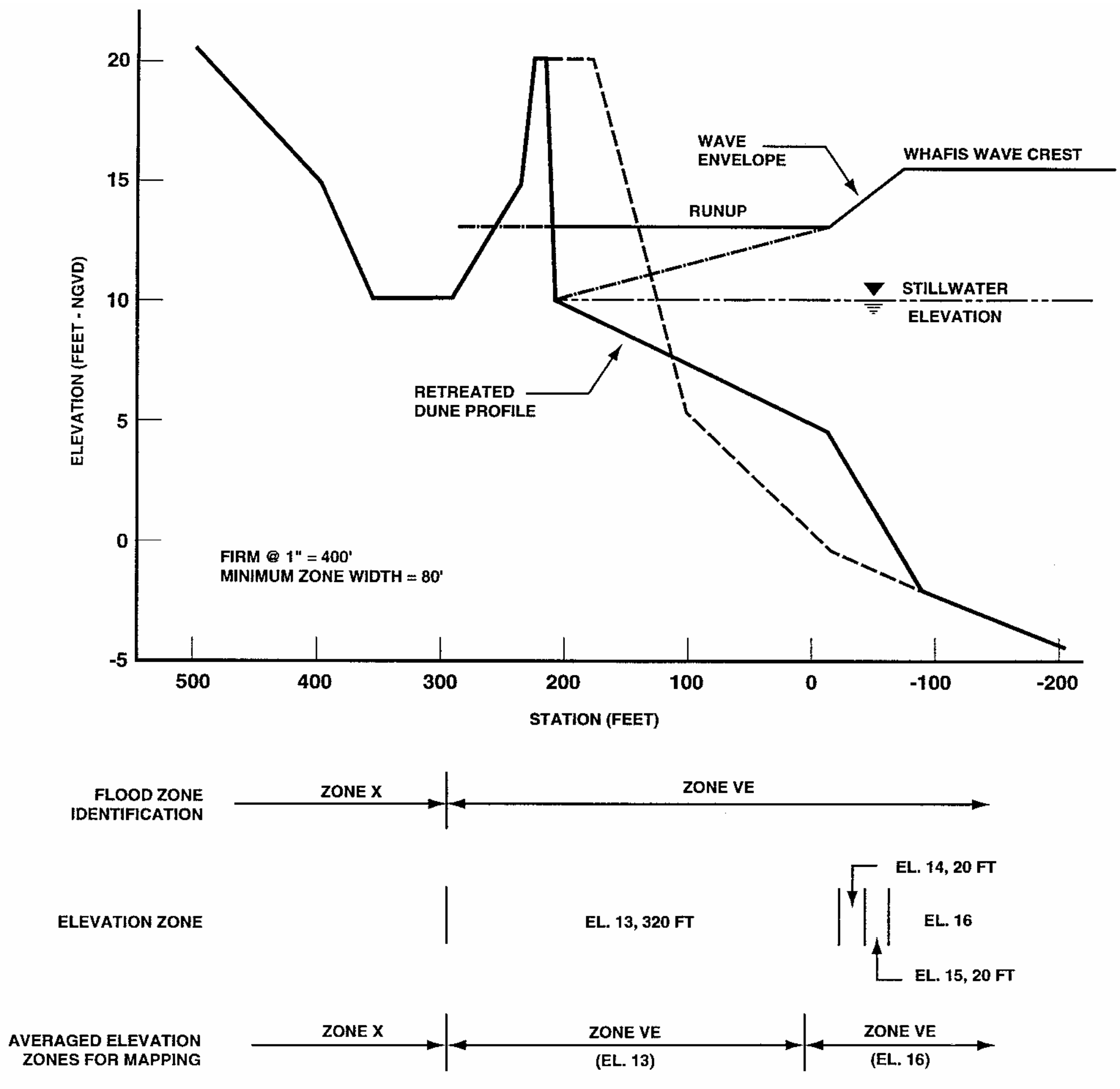


**Figure D-28. Identification of Elevation Zones, Example 2: Duneface Retreat with Relatively High Remnant.**

Explanation of Figure D-28: "Illustrates an example of a relatively high retreated dune face. A mean runup elevation of 13 feet is calculated for the eroded dune face. This elevation is assigned through the dune, all of which is designated as Zone VE. Because the dune remnant extends more than 7 feet above the SWEL, no flooding landward of the dune is indicated by designating the area as Zone X."

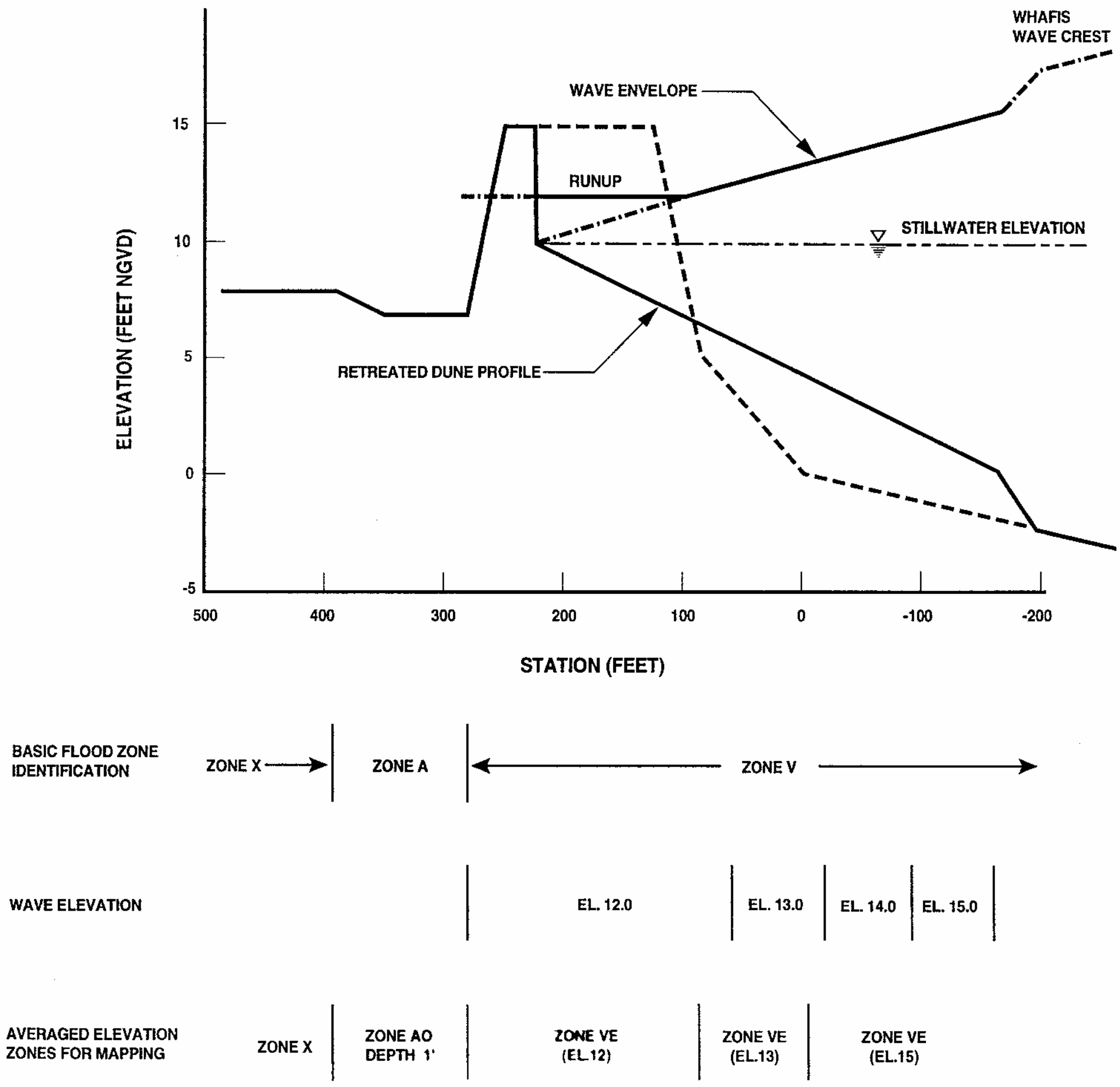


**Figure D-29. Identification of Elevation Zones, Example 3: Low Retreated Dune with Wave Overtopping.**

Explanation of Figure D-29: "Illustrates an example of a retreated dune face with a relatively small remnant having low relief. A mean runup elevation of 12 feet is calculated for the eroded profile, and this flood elevation is assigned through the dune, all of which is designated as Zone VE. The division into separate map zones is similar to the division in Figure D-28."

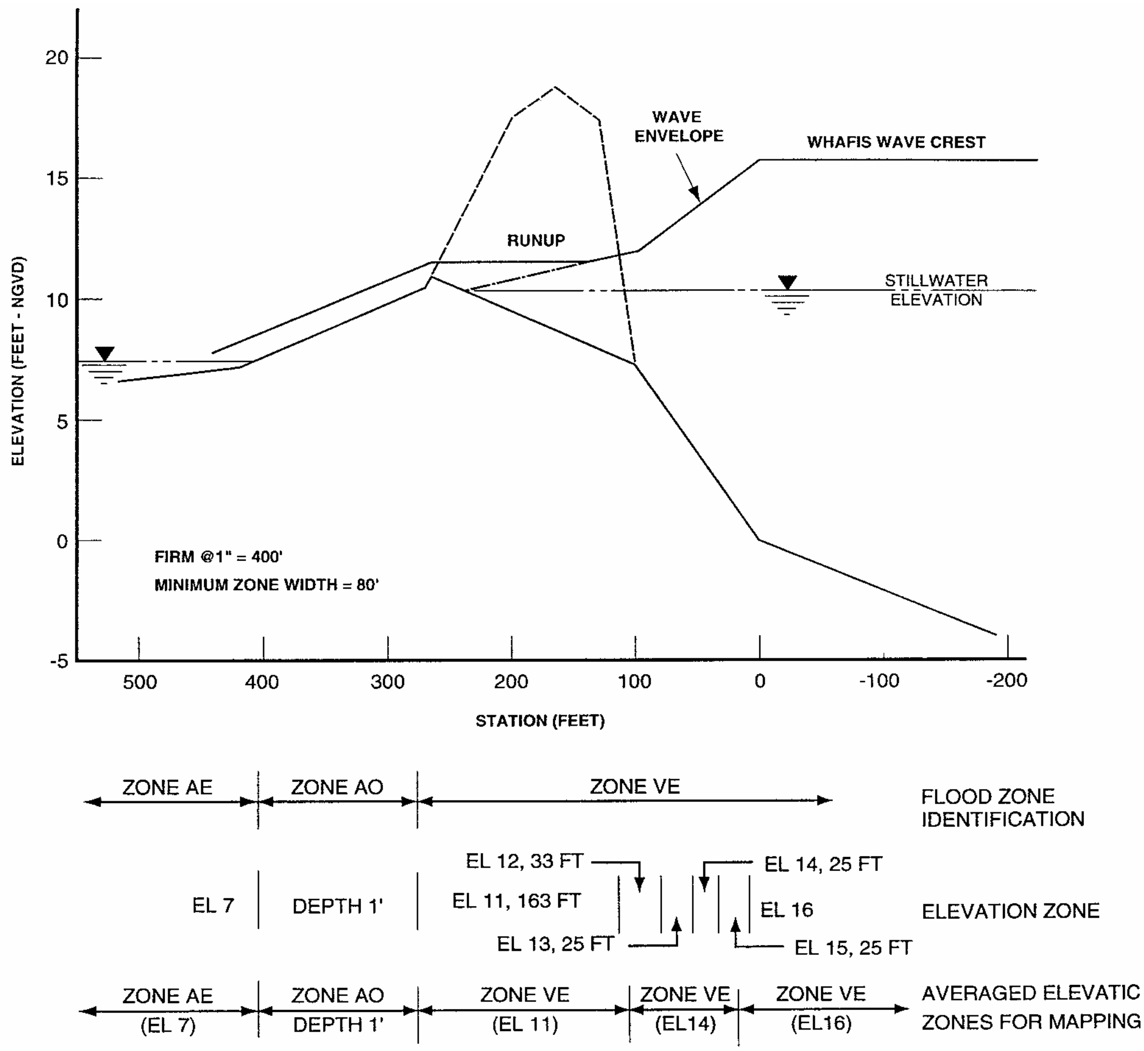


**Figure D-30. Identification of Elevation Zones, Example 4: Dune Removal with Wave Runup and Runoff**

Explanation of Figure D-30: "Illustrates an example of dune removal where there is some runup and overtopping of the remaining stub. As in Figure D-27, the VE zone with a runup elevation of 11 feet is extended to the dune toe and the Zone VE, elevation 16 feet, is located just landward of the shoreline."

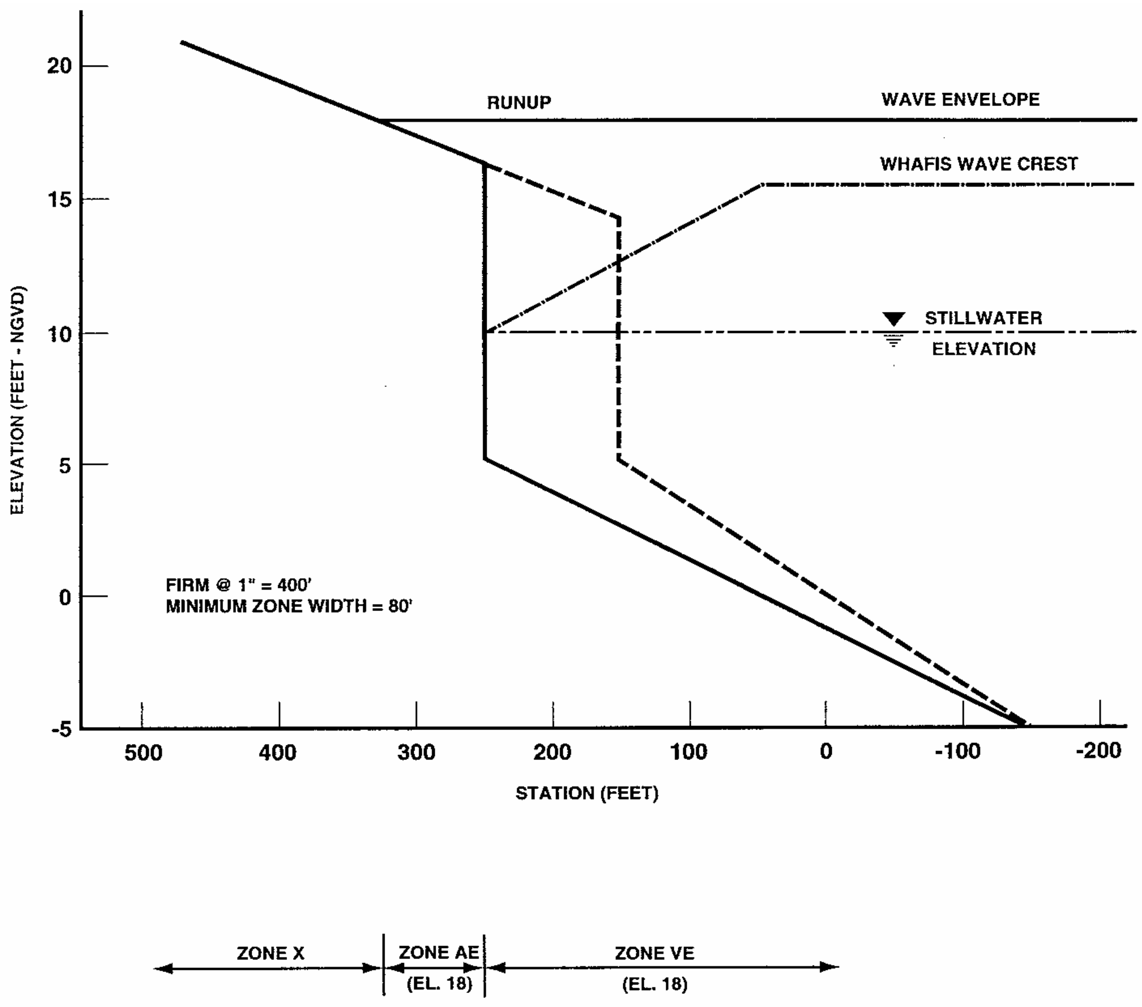


**Figure D-31. Identification of Elevation Zones, Example 5: Eroded Bluff with Wave Runup**

Explanation of Figure D-31: "An eroded bluff is shown in Figure D-31. The angle of the bluff face remains the same while the seaward extension from the toe is a 1 on 40 slope. The computed runup elevation slightly exceeds the bluff crest and is higher than the maximum wave crest elevation. The area is designated Zone VE, elevation 18 feet, until the difference between the runup elevation and the ground is less than 3 feet."

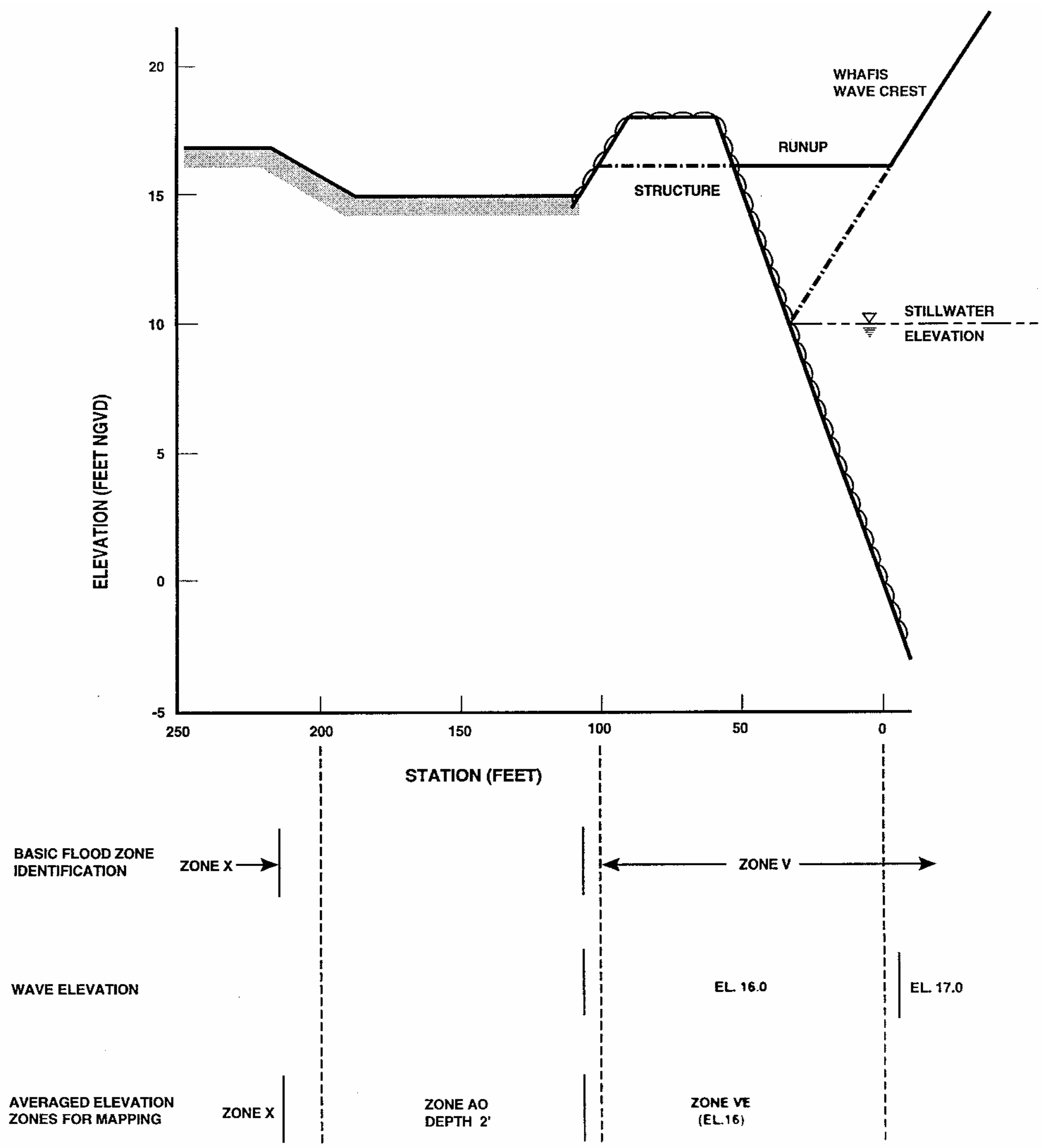


**Figure D-32. Identification of Elevation Zones, Example 6: Coastal Structure with Moderate Wave Overtopping**

<u>Explanation of Figure D-32:</u> "Figure D-32 illustrates an example of moderate structure overtopping expected for waves accompanying the 1-percent-annual-chance flood. The structure crest has sufficient freeboard above the 1-percent-annual-chance SWEL to contain a calculated mean runup of 6 feet, but extreme wave runups are likely to overtop the structure intermittently."

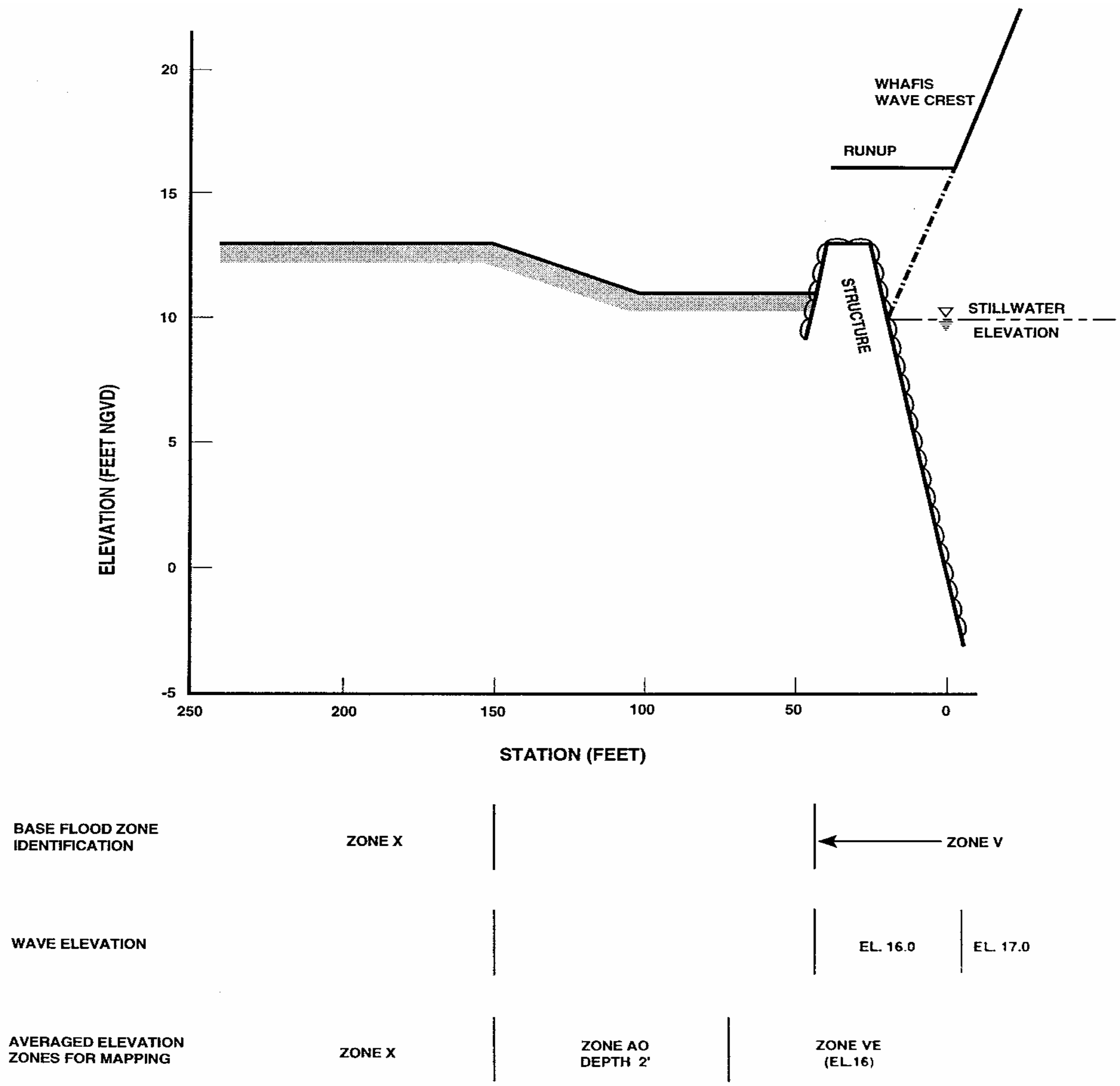


**Figure D-33. Identification of Elevation Zones, Example 7: Coastal Structure with Severe Wave Overtopping**

Explanation of Figure D-33: "Figure D-33 illustrates an example for a structure extending above the 1-percent-annual-chance SWEL but heavily overtopped by wave action. The calculated mean runup elevation is 5 feet above the seaward face, but that is reduced to the maximum excess runup of 3 feet in assigning a flood elevation of 16 feet for the shorefront VE zone. That zone extends through the entire structure and over an additional 30 feet landward, because likely wave impact area reaches beyond the structure during the 1-percent-annual-chance flood."

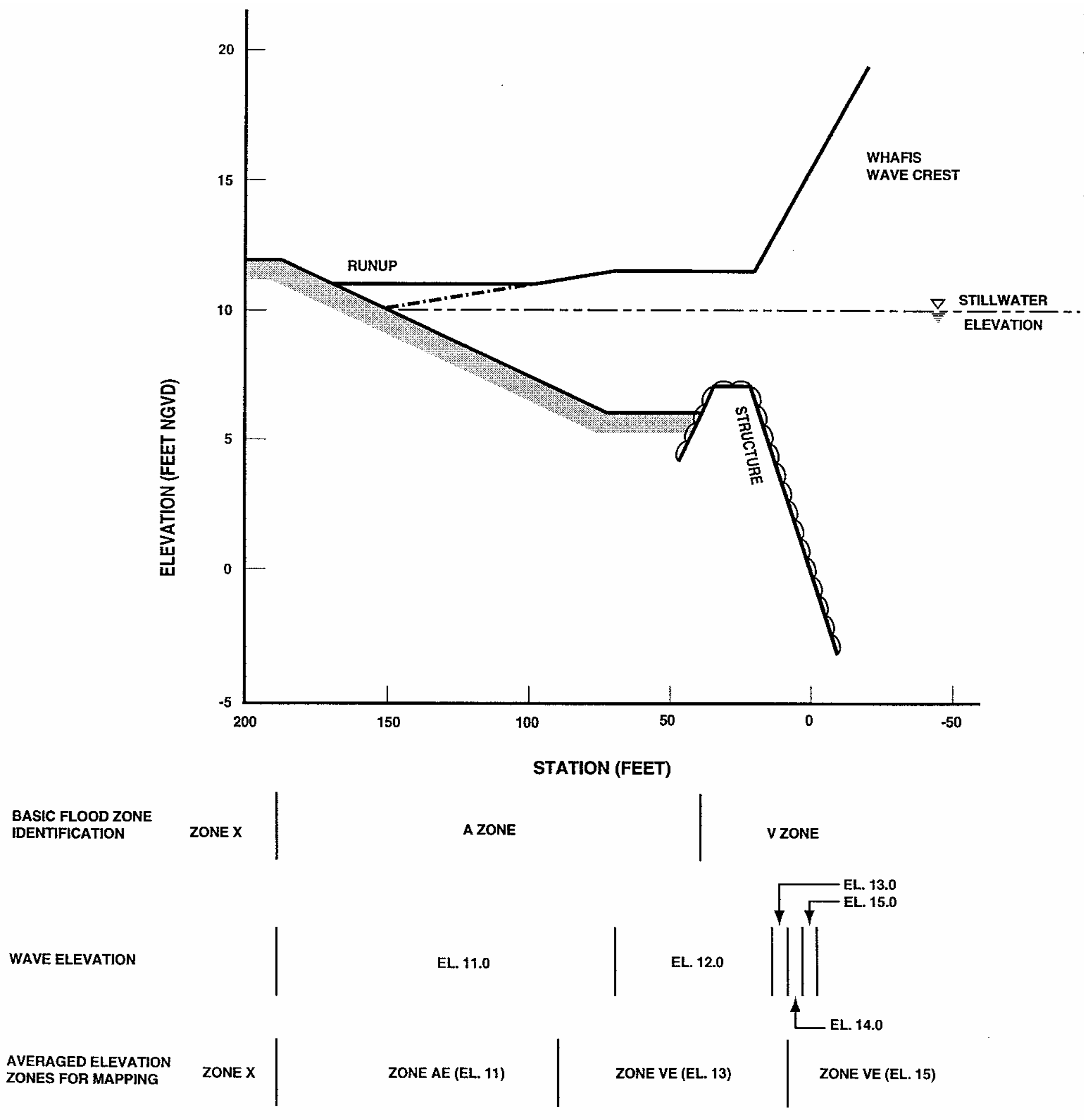


**Figure D-34. Identification of Elevation Zones, Example 8: Coastal Structure with Inundation**

Explanation of Figure D-34: "Figure D-34 illustrates an example of a structure covered by 3 feet of water during the 1-percent-annual-chance flood. Flood depth is not sufficient for waves 3 feet in height to propagate inland of the structure, but the V zone must extend to 30 feet landward of the structure, in view of likely wave impacts through the flood's course."

# APPENDIX 2

## Paper presented at the

## 2001 ASFPM Annual Conference, Charlotte, NC

***Consideration of a New Flood Hazard Zone: the Coastal A Zone***

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## INTRODUCTION

Present NFIP regulations make no distinction between coastal A zones and riverine A zones -- design and construction requirements are the same for both (i.e., elevation on fill, or on solid walls with flood openings, is permissible; reference elevation is the top of the lowest floor, etc.). However, there is a growing body of evidence that design and construction requirements in coastal A zones should be more like those in V zones than those in riverine A zones. This paper will discuss four topics that support this contention: 1) the nature of flood damage, 2) damages observed following coastal flood events, 3) the origin and validity of the V zone (3' breaking wave) criteria, and 4) testing of FIA depth-damage functions following a coastal flood event.

## NATURE OF FLOOD DAMAGE

The types of flood damage experienced by a building are related directly to the flood hazards and forces affecting the building -- different hazards (i.e., stillwater flooding vs. breaking waves) will result in different types of damage to the building. Table 1 provides a crude accounting of the dominant flood hazards present in riverine A zones, coastal A zones and V zones, and shows that coastal A zone flood hazards are similar to those in V zones, not riverine A zones.

**Table 1. Dominant Flood Hazards Present in Different Flood Zones**

| Hazard | Riverine A Zone | Coastal A Zone | V Zone |
|---|---|---|---|
| Elevated water level | X | X | X |
| currents | X | X | X |
| Waves | | X | X |
| Debris | X | X | X |
| Scour & Erosion | | X | X |

## OBSERVED DAMAGES

Post-flood damage inspections in coastal V zones consistently show damage to pre-FIRM buildings supported on fill or solid wall foundations. The same inspections frequently show similar damage to post-FIRM buildings supported on fill or solid wall foundations in coastal A zones. Recent inspections following hurricanes Hugo (South Carolina, 1989), Opal (Florida, 1995) and Fran (North Carolina, 1996) have all documented wave and erosion damage to post-FIRM coastal A zone buildings constructed in compliance with A zone standards (see FEMA's Building Performance Assessment Team reports).

## ORIGIN AND VALIDITY OF THE V ZONE (3-FOOT BREAKING WAVE) CRITERIA

The 3-foot breaking wave height is often used to distinguish V zones from A zones in coastal areas. Where did the 3-foot wave height standard originate? A study by the Galveston District (USACE, 1975) defined the "Critical Wave" as "a wave possessing sufficient energy to cause major damage on contact with conventional structures." Appendix B of the study concluded that the critical wave is a 3-foot breaking wave; however, a closer reading of calculations in Appendix B shows breaking waves 2.1 foot high are capable of destroying conventional wood-frame walls and connections. The 3-foot standard was adopted by the study, in recognition of the fact that conditions in the field (slope of ground, angle of wave attack, sheltering, etc.) may not be identical to those assumed in the study.

Recent full-scale laboratory tests of breakaway wall sections determined that breaking wave heights as low as 1.5- foot high consistently cause failure of traditional stud wall construction. The tests were part of a larger effort to improve breakaway wall design standards (FEMA, 1999).

### TESTING FIA DEPTH-DAMAGE FUNCTIONS

The Federal Insurance Administration has developed damage functions to predict structural and contents damages due to floods. The functions relate flood damage (as a percent of structure/contents value) to flood depth, where depth is measured from a reference elevation -- top of lowest floor in A zones, bottom of lowest horizontal structural member in V zones -- to the top of the water surface (including wave height). Once the A zone and V zone damage functions are shifted to the same reference elevation, a direct comparison between the functions is possible. This comparison (see Table 2) shows there is a great difference in predicted structural damage for a given structure and water depth, depending on the flood zone designation.

**Table 2. Percent Structural Damage vs. Flood Depth Using FIA A Zone and V Zone Damage Functions**

| Flood Depth | A Zone (2-story, no basement) | V Zone (no obstructions) |
|---|---|---|
| -2 ft | 0 % | 10 % |
| 0 ft | 0 % | 15 % |
| 2 ft | 5 % | 35 % |
| 4 ft | 13 % | 58 % |
| 6 ft | 20 % | 66.5 % |

* Flood depth is measured from bottom of lowest horizontal structural member to the top of the flood surface (including wave height). The table assumes the distance between the top of floor and the bottom of lowest horizontal structural member is 2 ft for the A zone building.

Consider the case of the V/A boundary established using the 3-foot breaking wave height (i.e., where the stillwater depth is 3.8 feet and the "depth" between the ground and the wave crest elevation is 5.9 feet). If a pre-FIRM structure was built at this boundary on a slab foundation, with the top of the slab just above the ground, application of the A zone damage function to base flood conditions would predict structural damage at approximately 20% of the structure value. Application of the V zone damage function in the same case would predict approximately 66% structural damage. Consider another comparison, this one for post-FIRM construction built at the same V/A boundary. If the location was classified an A zone and the building was elevated with the top of the lowest floor at the BFE, the A zone damage function predicts 5% structural damage under base flood conditions. The same structure would suffer an estimated 35% damage with a V zone designation (bottom of lowest horizontal structural member 2 feet below BFE).

Given the importance of the zone designation and corresponding damage function in predicting coastal flood damages, testing was carried out as part of the development of the coastal flood module for HAZUS (EQE International, 2000). Predicted structural damage was compared against Hurricane Opal flood claims data for 81 residential structures (63 A zone, 18 V zone) at Pensacola Beach, FL. Predicted damages were based on ground elevations, building characteristics (value, number of stories, lowest floor elevation), estimated flood elevations during Opal, and FIA depth-damage functions. The results of the analysis are summarized in Table 3. The general conclusion is that, in this case, application of the V zone damage function to coastal A zone buildings provided a better aggregate estimate of structural damage due to Opal. Use of the A zone damage function underestimated structural damage to coastal A zone structures.

**Table 3. Comparison of Predicted and Actual Structural Damage due to Hurricane Opal, Pensacola Beach, FL (81 residential structures)**

| Building Type and Damage Fn | Aggregate Value ($1,000) | Predicted Damage ($1,000) | Actual Damage ($1,000) | Predicted / Actual Ratio |
|---|---|---|---|---|
| 1-story, A zone | $1,565 | $490 | $958 | 0.51 |
| 1-story, A zone | $1,360 | $261 | $710 | 0.37 |
| V zone | $1,180 | $731 | $615 | 1.19 |
| V Zone function applied to all | $2,925 | $1,770 | $1,668 | 1.06 |

Recognizing the uncertainty associated with some of the parameters used to estimate structural damage, sensitivity tests were carried out. In order for predicted damages to equal actual damages, the following adjustments would be required: increasing the A zone flood depth over 20 feet, and reducing the V zone flood depth 1.7 feet; multiplying A zone building valuations by a factor of 2.2, and multiplying V zone building valuations by a factor of 0.84. The A zone flood depth adjustment is unreasonable. The required building value adjustments would make A zone buildings more expensive than V zone buildings, contrary to actual experience. In summary, the difference between predicted and actual structure damage cannot be due to errors in flood depths or building values, but must be due to the damage function applied.

Application of the FIA damage function to buildings demolished after the storm (and presumed to have sustained > 50% actual damage) yielded similar results. The A zone damage function failed to predict > 50% damage to any of 22 demolished A zone buildings. Application of the V zone damage function to those same 22 buildings predicted > 50% damage in 18 out of 22 cases.

## FINAL COMMENTS

This paper has discussed the issue of distinguishing between coastal A zones and riverine A zones. The authors believe sufficient justification exists for doing so, and this concept is finding its way into the literature [e.g., the national load standard (ASCE, 1998) and the revised Coastal Construction Manual (FEMA, 2000)]. However, on a broad scale, how should this concept be implemented? Is it best to implement mapping changes (redefine V zone delineation) or management changes (leave FIRMs as they are and apply V zone standards to coastal A zones)? Those questions have yet to be answered. More work on the subject and more discussion are required.

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