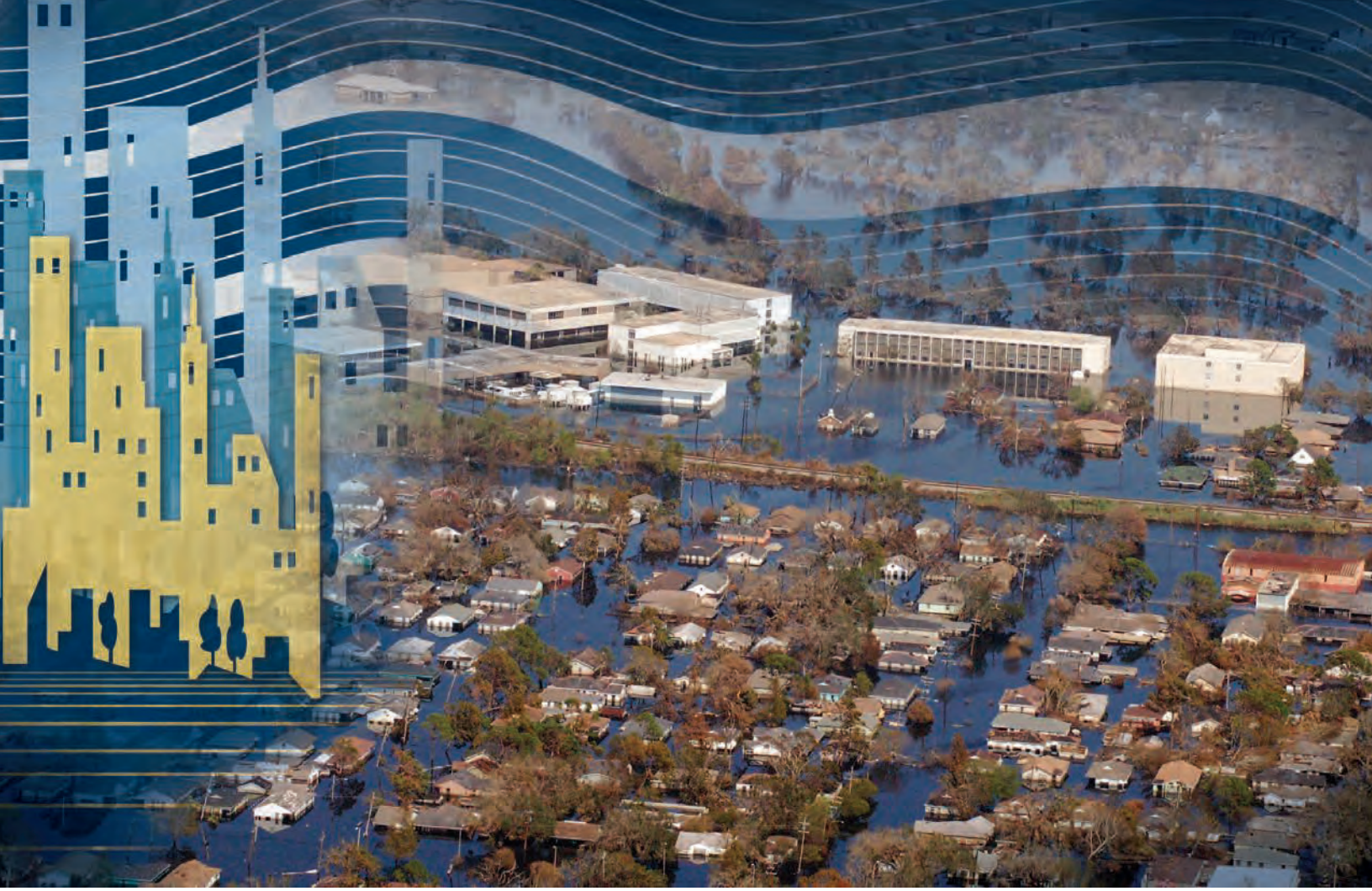
DOCKETED

Docket Number:	15-AFC-01
Project Title:	Puente Power Project
TN #:	213677
Document Title:	California Coastal Commission 30413(d) Report - File Docs Part 2
Description:	Selected file documents cited in CCC's 30413(d) Report on P3
Filer:	Joseph Street
Organization:	California Coastal Commission
Submitter Role:	Public Agency
Submission Date:	9/15/2016 3:43:21 PM
Docketed Date:	9/15/2016



Risk Management Series

Design Guide

for Improving Critical Facility Safety from Flooding and High Winds

FEMA 543 / January 2007

FEMA

FEMA 543 / *January 2007*

RISK MANAGEMENT SERIES

Design Guide *for* Improving Critical Facility Safety from Flooding and High Winds

PROVIDING PROTECTION TO PEOPLE AND BUILDINGS

FEMA

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BACKGROUND

On August 29, 2005, Hurricane Katrina caused extensive damage to the coast along the Gulf of Mexico, resulting in an unprecedented relief, recovery, and reconstruction effort. This reconstruction presents a unique opportunity to rebuild the communities and public infrastructure using the latest hazard mitigation techniques proven to be more protective of lives and property.

Critical facilities comprise all public and private facilities deemed by a community to be essential for the delivery of vital services, protection of special populations, and the provision of other services of importance for that community. This manual concentrates on a smaller group of facilities that are crucial for protecting the health and safety of the population: health care, educational, and emergency response facilities.

The *Design Guide for Improving Critical Facility Safety from Flooding and High Winds* (FEMA 543) was developed with the support of the Federal Emergency Management Agency (FEMA) Region IV in the aftermath of Hurricane Katrina. This manual recommends incorporating hazard mitigation measures into all stages and at all levels of critical facility planning and design, for both new construction and the reconstruction and rehabilitation of existing facilities. It provides building professionals and decisionmakers with information and guidelines for implementing a variety of mitigation measures to reduce the vulnerability to damage and disruption of operations during severe flooding and high-wind events. The underlying theme of this manual is that by building more robust critical facilities that will remain oper-

ational during and after a major disaster, people's lives and the community's vitality can be better preserved and protected.

This manual is a part of FEMA's Risk Management Series, which provides guidelines for mitigating against multiple hazards. The series emphasizes mitigation best practices for specific building uses, such as schools, hospitals, higher education buildings, multi-family dwellings, commercial buildings, and light industrial facilities.

OBJECTIVES

The poor performance of many critical facilities in the affected areas was not unique to Hurricane Katrina. It was observed in numerous hurricanes dating back more than three decades. Several reasons may explain this kind of performance. In many cases the damaged facilities were quite old and were constructed well before the introduction of modern codes and standards. Some of the older facilities were damaged because building components had deteriorated as a result of inadequate maintenance. Many facilities occupy unsuitable buildings that were never intended for this type of use. Some newer facilities suffered damage as a result of deficiencies in design and construction or the application of inappropriate design criteria and standards.

The primary objective of this manual is to assist the building design community and local officials and decisionmakers in adopting and implementing sound mitigation measures that will decrease the vulnerability of critical facilities to major disasters.

The goals of this manual are to:

- Present and recommend the use of building design features and building materials and methods that can improve the performance of critical facilities in hazard-prone areas during and after flooding and high-wind events.

- Introduce and provide guidelines for implementing flooding and high-wind mitigation best practices into the process of design, construction, and operation and maintenance of critical facilities.

SCOPE

To aid in the reconstruction of the Gulf Coast in the wake of Hurricane Katrina, this manual presents an overview of the principal planning and design considerations for improving the performance of critical facilities during, and in the aftermath of, flooding and high-wind events. It provides design guidance and practical recommendations for protecting critical facilities and their occupants against these natural hazards. The focus is on the design for new construction, but this manual also addresses rehabilitation of existing critical facilities. It presents incremental approaches that can be implemented over time to decrease the vulnerability of buildings, but emphasizes the importance of incorporating the requirements for mitigation against flooding and high winds into the planning and design of critical facilities from the very beginning of the process.

The material and recommendations contained in this manual are applicable to many facilities, but address primarily the following critical facilities:

- Schools
- Health Care Facilities
- Fire Stations
- Police Stations
- Emergency Operation Centers (EOCs)

The information presented in this manual provides a comprehensive survey of the methods and processes necessary to protect critical facilities from natural hazards, but is necessarily limited. It is not expected that the reader will be able to use this information directly to develop plans and specifications. It is intended as an introduction to a broader understanding of the fundamental approaches to risk mitigation planning and design. This will help building officials and professionals move on to the implementation phase that involves consultants, procurement personnel, and project administrators, with a better grasp of the task in front of them—improving the safety and welfare of their communities.

TARGET AUDIENCE

This manual describes various mitigation measures that have been successful in the past and could be implemented quickly, especially in areas recovering from a disaster. The intended audience comprises the people who own, operate, design, build, and maintain critical facilities in hazard-prone areas. This includes planning, building design, and construction professionals working for private organizations, State and local government officials working in the building sector, and relevant technical and management personnel involved with the operation of critical facilities.

TRAINING

In tandem with the publication of this manual, FEMA has developed a companion training course targeted to building design professionals and facility managers interested in improving the functionality of critical facilities in natural disasters. For transfer of its content, FEMA will promote a series of workshops directed at these professionals and others involved in planning, design, construction, rehabilitation, and management of critical facilities in areas exposed to flooding and high winds.

The training course emphasizes the best practices in mitigating against flooding and winds hazards. It is organized around a series of exercises, starting with vulnerability analysis, and progressing through assessment of risks, assessment of building systems performance (especially the effects of physical damage on the facility's functionality), and the selection of appropriate mitigation measures to be incorporated into the design of critical facilities.

The expansive scope of reconstruction activities in the aftermath of Hurricane Katrina requires that this course be initially offered and delivered only to affected communities in the Gulf States. However, the relevance and usefulness of this course extends far beyond the hurricane-prone regions, to areas exposed to other wind hazards, as well as areas subject to all types of coastal and riverine flooding.

ORGANIZATION AND CONTENTS

This manual is divided into four main chapters.

Chapter 1 presents an overview of the principal design considerations to help owners, managers, and designers when determining the location, building characteristics, and hazard resistance of critical facilities. It outlines the basic principles and design tools for improving the safety of critical facilities exposed to flooding and high winds. It also provides guidelines for facility managers and building professionals on how to coordinate the process of planning and design of critical facilities.

Chapter 2 discusses the nature of flood forces and their effects on buildings. It outlines the procedures for risk assessment, and describes current mitigation methods for reducing the effects of flooding. It underlines the need to avoid high-risk areas for the construction of new critical facilities, and encourages the application of mitigation measures when critical facilities must remain in high-risk areas. This chapter provides extensive review of the requirements of the National Flood Insurance Program (NFIP), model building codes and other standards, as well as new FEMA policy updates resulting from the experiences of Hurricanes Katrina and Rita.

Chapter 3 discusses the effects of wind forces on the structural and nonstructural building components of critical facilities. By reviewing numerous examples of wind-induced damage to these facilities, it points out the best practices pertaining to new construction and rehabilitation of existing facilities. It concentrates on building components most critical for protecting the uninterrupted operation of critical facilities and provides detailed guidelines for improving their design and construction in hurricane- and tornado-prone areas.

Chapter 4 discusses the performance of hospital, schools, and emergency response facilities (i.e., EOCs, and police and fire rescue stations) during Hurricane Katrina. This chapter emphasizes the lessons learned about the adverse effects on the functionality of critical facilities arising from damage to buildings and contents, especially the ways that various types of physical damage disrupted their operations.

Appendix A contains a list of acronyms and Appendix B contains a glossary of terms that appear in this manual. Appendix C contains an overview of the FEMA grant programs available for funding construction or rehabilitation of critical facilities.

ACKNOWLEDGMENTS

The preparation of this document was sponsored by FEMA Region IV. It has been developed based on the work of several recent publications (as listed below), new insights from the Katrina Mitigation Assessment Teams in the field, and other sources of information on Hurricane Katrina. The principal and contributing authors of the following documents are gratefully acknowledged: FEMA 424, *Design Guide for Improving School Safety in Earthquakes, Floods, and High Winds*; FEMA 489, *Mitigation Assessment Team Report for Hurricane Ivan in Alabama and Florida*; FEMA Region IV, *Hurricane Mitigation: A Handbook for Public Facilities*; FEMA Region X, *Earthquake Hazard Mitigation Handbook for Public Facilities*; Structural Engineers Association of Washington (SEAW) *Commentary on Wind Provisions* (Volumes 1 and 2); and other reference documents presented in Chapter 1.

For this manual, the following individuals are acknowledged:

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The authors and editors gratefully acknowledge the invaluable assistance and advice provided by the following reviewers:

Christopher Arnold, Building Systems Development Inc.; Ronald B. Bailey, Bailey Engineering Corp.; Bob Baker, BBJ Environmental Solutions, Inc.; Ron A. Cook, University of Florida; Jim Fields; Doug Fitts, St. Louis County Public Works; Lois

Forster, FEMA; David Godschalk, University of North Carolina; Neil Grigg, Colorado State University; John Ingargiola, FEMA; Philip Keillor, American Society of Flood Plain Managers; Leon Kloostra; Richard Kuchnicki, International Code Council; Lawrence Larson, ASFPM; Sheila McKinley, Christopher B. Burke Engineering; Nancy McNabb, NFPA; Elliott Mittler; Ann Patton, Ann Patton Company; Dan Powell, FEMA; Dennis Quan, FEMA; Jim Rossberg, ASCE; Michael P. Sheerin, TLC Engineering for Architecture; Emil Simiu, NIST; Robert Slynn, University of Florida; Alan Springett, FEMA; Thomas Sputo, University of Florida; Jim Tauby; Anton TenWolde; Scott Tezak, URS; David Underwood; Wallace A. Wilson.

This manual will be revised periodically and FEMA welcomes comments and suggestions for improvements in future editions. Please send your comments and suggestions via e-mail to: RiskManagementSeriesPubs@dhs.gov

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1.1 INTRODUCTION

This chapter introduces the role of hazard mitigation in the planning, design, and construction of critical facilities. It describes the way building design determines how well a critical facility is protected against natural hazard risks, specifically the risks associated with flooding and high winds. Critical facilities, and the functions they perform, are the most significant components of the system that protects the health, safety, and well-being of communities at risk.

The devastating effects of recent hurricanes, especially Hurricane Katrina, underscored the vulnerability of coastal areas of the United States, the fastest growing regions of the country. The population pressure and the aggressive coastal development in areas subject to hurricanes and coastal storms created the conditions that require careful consideration of the effects of natural hazards on the sustainability of this development. One of the most important determinants of the sustainability of coastal communities is the reliability of their physical and social infrastructure. The communities that cannot rely on their own critical infrastructure are extremely vulnerable to disasters. This is why the design of critical facilities to improve their resistance to damage, and their ability to function without interruption during and in the aftermath of hazard events, deserves special attention.

To ensure safe and uninterrupted operation of critical facilities, which is vital in the post-disaster period, facility owners must incorporate a comprehensive approach to identify hazards and

avoid them when feasible. In cases when exposure to hazards is unavoidable, it is recommended that they build new facilities, or rehabilitate the existing ones to resist the forces and conditions associated with these hazards.

1.1.1 CRITICAL FACILITIES

In general usage, the term "critical facilities" is used to describe all manmade structures or other improvements that, because of their function, size, service area, or uniqueness, have the potential to cause serious bodily harm, extensive property damage, or disruption of vital socioeconomic activities if they are destroyed, damaged, or if their functionality is impaired.

Critical facilities commonly include all public and private facilities that a community considers essential for the delivery of vital services and for the protection of the community. They usually include emergency response facilities (fire stations, police stations, rescue squads, and emergency operation centers [EOCs]), custodial facilities (jails and other detention centers, long-term care facilities, hospitals, and other health care facilities), schools, emergency shelters, utilities (water supply, wastewater treatment facilities, and power), communications facilities, and any other assets determined by the community to be of critical importance for the protection of the health and safety of the population. The adverse effects of damaged critical facilities can extend far beyond direct physical damage. Disruption of health care, fire, and police services can impair search and rescue, emergency medical care, and even access to damaged areas.

The number and nature of critical facilities in a community can differ greatly from one jurisdiction to another, and usually comprise both public and private facilities. In this sense, each community needs to determine the relative importance of the publicly and privately owned facilities that deliver vital services, provide important functions, and protect special populations.

Minimum requirements for the design of new critical facilities and for improvements to existing facilities are found in the model building codes and the design and construction standards. ASCE 7, *Minimum Design Loads for Buildings and Other Structures,* is the

best known standard. Published by the American Society of Civil Engineers (ASCE), it classifies buildings and other structures into four categories based on occupancy. Most critical facilities fall into Category III or Category IV, described below:

Category I includes buildings and other structures whose failure would represent a low hazard to human life, such as agricultural buildings and storage facilities.

Category II includes all buildings not specifically included in other categories.

Category III includes buildings and other structures that represent a substantial hazard to human life in the event of failure. They include buildings with higher concentrations of occupants (i.e., where more than 300 people congregate in one area). These are typically educational facilities with capacities greater than 250 for elementary and secondary facilities, 500 for colleges and adult education facilities, or 150 for daycare facilities.

Category IV includes essential facilities such as hospitals, fire and police stations, rescue and other emergency service facilities, power stations, water supply facilities, aviation facilities, and other buildings critical for the national and civil defense.

This manual concentrates on a number of critical or, as they are sometimes called, essential facilities, that deal with health and safety in emergencies, and include health care facilities, police and fire stations, EOCs, and schools. These facilities are chosen because of their vitally important role in protecting the health and safety of the community. Although limited in scope to several specific types of facilities, the information and recommendations in this manual are valuable and applicable to other types of critical facilities located in areas prone to flooding and exposed to high winds.

1.1.2 HURRICANE KATRINA

Although not the strongest storm to hit the coast of the United States, Hurricane Katrina caused the greatest disaster in the nation's history. The hurricane made its first landfall on August 25, 2005, on the southeast coast of Florida as a Category 1 hurricane.

It then crossed Florida into the Gulf of Mexico, where it gained strength to a Category 5 hurricane. Before making its second landfall near Buras in southeast Louisiana, Katrina weakened to a Category 3 hurricane. Moving across southeast Louisiana, Katrina continued northward, pushing storm surge into coastal areas of Alabama, Mississippi, and Louisiana. After crossing over Lake Borgne, it finally made a third landfall as a Category 3 hurricane near Pearlington, Mississippi, at the Louisiana/Mississippi border (see Figures 1-1 and 1-2). The hurricane caused extensive devastation along the gulf coast, with southeast Louisiana and the coast of Mississippi bearing the brunt of the catastrophic damage.

Wind damage was widespread and severe in many areas; however, the greatest damage was caused by Hurricane Katrina's storm surge flooding. Although the storm weakened from a powerful Category 5 to a Category 3 hurricane just before making landfall in Louisiana and Mississippi, the storm surge appears to have maintained a level associated with a Category 5 hurricane. The surge built by the stronger winds over open water could not dissipate as quickly as the wind speeds decreased, and the shallow depth of the off-shore shelf and the shape of the shoreline contributed to the high surge elevations. The Mississippi coastline experienced the highest storm surge on record. The storm surge also contributed to failures of a number of levees, notably the levee system that protects the City of New Orleans from Lake Borgne and Lake Pontchartrain. An estimated 80 percent of the city subsequently flooded.

The disaster was further compounded by the poor performance of critical facilities during and after the storm. Critical facilities typically did not perform any better than ordinary commercial buildings, but the extent of the damage to these facilities and the subsequent disruption of their operations caused much greater hardship. Facilities such as hurricane evacuation shelters, police and fire stations, hospitals, and EOCs were severely damaged and many were completely destroyed. Some facilities experienced a loss of function when critical support equipment, such as vehicles and communication equipment, were damaged or destroyed. While most of the damage to critical facilities was caused by the storm surge, wind damage also was widespread and substantial. In several instances, critical facilities were destroyed completely or damaged so severely that all the occupants had to be evacuated

Figure 1-1: Hurricane Katrina's path through Louisiana and Mississippi
(BASED ON HURRICANE STORM TRACK DATA FROM THE NATIONAL HURRICANE CENTER)

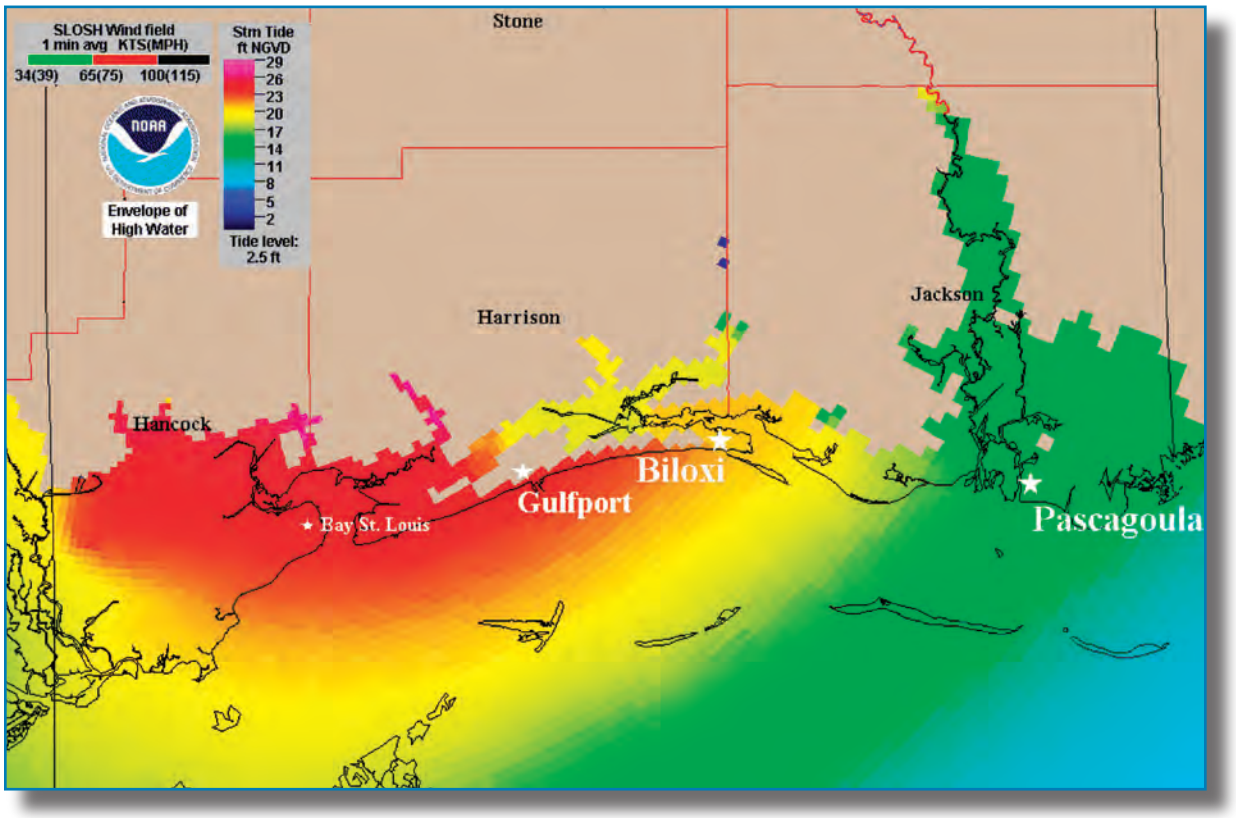

Figure 1-2: Mississippi coast SLOSH NOAA data

SOURCE: NATIONAL OCEANIC AND ATMOSPHERIC ADMINISTRATION (NOAA)

after the hurricane had moved inland. The loss of so many critical facilities placed a severe strain on the emergency operations and recovery efforts.

The estimated death toll of Hurricane Katrina exceeded 1,800. More than 85 percent of casualties were recorded in Louisiana and about 13 percent of victims lost their lives in Mississippi. Other deaths attributed both directly and indirectly to Katrina were reported in Florida, Alabama, Georgia, Kentucky, and Ohio. Hurricane Katrina ranks as the third deadliest hurricane in the United States, surpassed only by the Texas Hurricane at Galveston in 1900, where at least 6,000 and possibly as many as 10,000 lives were lost, and the Florida Hurricane at Lake Okeechobee in 1928, which claimed 2,500 lives. Estimated total economic losses from Katrina are in excess of $150 billion, and insured losses are $40 billion, making Katrina the most expensive natural disaster in the nation's history.

1.2 HAZARD MITIGATION

1.2.1 HAZARD MITIGATION FOR CRITICAL FACILITIES

Mitigation can reduce the enormous cost of disasters to property owners, communities, and the government. Since the late 1980s, hazard mitigation has become well known in many parts of the country for initiatives in land use planning, adoption of building codes, elevation of homes, floodplain buyouts, and retrofitting buildings to resist damage in flooding, high winds, or seismic events. Incorporating mitigation measures in the planning and design of buildings is recommended because these measures reduce injuries and damage resulting from building failures during hazard events. Incorporating mitigation measures in the design of critical facilities, however, is crucial for minimizing the disruption of their operations and protecting the uninterrupted provision of critical services.

"Mitigation" is defined as any sustained action taken to reduce or eliminate long-term risk to life and property from hazard events. The goal is to save lives and reduce property damage in ways that are cost-effective and environmentally sound.

The first Federal program to support State and local mitigation programs was established by the Stafford Act in 1988. Growing support and recognition of the need to improve disaster resistance led to passage of the Disaster Mitigation Act of 2000, which amended the Stafford Act. This statute reinforces the importance of comprehensive, multi-hazard mitigation planning, and emphasizes planning for disasters before they occur. As part of the

Since 1977, Federal agencies have been charged by Executive Order 11988 to provide leadership "to reduce the risk of flood loss, to minimize the impact of floods on human safety, health and welfare, and to restore and preserve the natural and beneficial values served by floodplains in carrying out their responsibilities for (1) acquiring, managing, and disposing of Federal lands and facilities; (2) providing federally undertaken, financed, or assisted construction and improvements; and (3) conducting Federal activities and programs affecting land use, including but not limited to water and related land resources planning, regulating, and licensing activities."

planning process, States and communities are encouraged to identify existing critical facilities and to evaluate their vulnerability to natural hazards. To qualify for certain Federal mitigation grant programs, projects to rehabilitate critical facilities must be consistent with State and local mitigation plans. Appendix C provides an overview of conditions and requirements for obtaining funding assistance from major mitigation funding programs administered by the Federal Emergency Management Agency (FEMA).

There is no single procedure mandated for the planning, site selection, and design of critical facilities, because none can be assumed to be universally applicable. The decision to build a critical facility depends on many factors and requires a rigorous and comprehensive analysis of all the conditions that may affect the operation of a facility. This manual primarily addresses the design of new facilities and measures to improve the disaster resistance of existing facilities exposed to flooding and high winds, based on the assumption that all other alternatives to minimize or avoid such risks have been thoroughly evaluated and rejected as infeasible or impractical. It is outside the scope of this manual to try to depict in detail this evaluation process in its full range and complexity. Communities, as well as the owners and operators of critical facilities, must evaluate all alternatives, assess all risks, and consider all short-term and long-term effects of proposed projects, whenever construction or rehabilitation of these facilities is considered. Careful analysis of alternatives and the potential adverse effects of exposing critical facilities to natural hazards is also intended to help identify the most appropriate hazard-resistant measures when avoidance is not practical.

1.2.2 SITE SELECTION

Site selection is a particularly significant step when planning new critical facilities or when planning substantial improvements to existing facilities in hazard-prone areas. The earliest steps in the planning process should be to identify hazards and assess the

risks for the facility at the proposed site. In addition, alternative solutions should be considered in order to avoid site-specific hazards like floods. After decisions about the building location have been made, hazard mitigation involves acquiring a full understanding of the prevalent hazards and considering all appropriate hazard-resistance measures to ensure the uninterrupted operation of critical facilities.

All work on critical facilities must meet the minimum requirements of building codes and related regulations. However, the importance of uninterrupted operation of critical facilities frequently makes it necessary to go beyond the code requirements to provide acceptable levels of protection for the facility's functionality during, and in the immediate aftermath of, a hazard event.

Typically, the selection of a site for a critical facility is based on specific functions of a facility and the characteristics of its service area. In cases where critical facilities may be exposed to flooding and wind hazards, it is recommended that the final site decision be made only after all alternative sites have been evaluated for hazard exposure and the resulting effects of the hazard exposure on the design, construction, and operation of a facility.

Considering that critical facilities should avoid hazard-prone areas, site selection may sometimes be a difficult and prolonged process. This is especially true in situations when the facility service requirements cannot be easily reconciled with requirements to minimize the exposure to hazards. Sometimes a facility, like a fire station for example, cannot fulfill its rapid response function if it is located outside the hazard zone, far from the area the facility is intended to serve. Additionally, site selection is not always controlled by the community. Many local jurisdictions report that the high cost and the scarcity of available land can severely limit the consideration of alternative locations. The consequences of accepting a flood-prone site include not only the potential physical damage, but also the loss of services provided by the critical facility. This loss of service can adversely affect the community as a whole, both in the immediate post-event period and during its long-term recovery. Section 2.5.1 contains a discussion and a number of questions that can help guide determinations about whether the risks associated with building a critical facility in a floodplain are acceptable.

If the site selection process determines that no other practical and feasible alternatives are available and that a facility must be located in a hazard-prone area, the highest level of protection should be a design priority.

1.2.3 FACILITY DESIGN

The nature of services provided by critical facilities requires that designers and decisionmakers define a design objective of achieving building performance levels beyond the minimum requirements prescribed by the building code. While compliance with the building code may satisfy the requirements to protect the facility's occupants, it may be insufficient to ensure the continued operation of the facility. When designing or rehabilitating a critical facility located in an area subject to high-wind or flooding risks, this manual recommends a set of guidelines intended to minimize the interruption in operation of critical facilities, both during and in the aftermath of hazard events.

- Conduct an in-house assessment of the facility needs, with the assistance of decisionmakers and consultants. Public committees may contribute advice and guidance throughout the programming and design process. For large programs, committees may acquire specialists at different stages as necessary.

- Determine the size and scope of the proposed program. In a smaller area, an architect may be employed to assist the decisionmakers with this task, possibly later becoming the design architect.

- Assess the needs of the facility to determine the availability of suitable sites (and lease/purchase as necessary).

- Develop occupant specifications, seeking advice from facility managers and both in-house and consulting professionals.

- Assess financial needs.

- Identify financial resources, including alternative sources of funding (e.g., Federal and State programs, local taxes, bond issues, and utility fees).

- Ensure funding (e.g., bond issue, establishment of utility districts, etc.).

- Appoint a building program management staff (appointed officials or a committee).

- Determine the design and construction process (i.e., conventional design and bid, design/build, or construction management).

- Select and hire architects and other special design consultants or design/build team members. The timing of this phase varies depending on the number of variables.

- Develop building programs, including building size, room size, equipment, and environmental requirements. This may be done in-house, or architects and independent program consultants may assist.

- Appoint a local representative to the staff and a public stakeholders committee for the design phase.

- Develop designs with cost estimates. Hold public meetings, with the architects in attendance, and encourage public input into the design. Implement local area progress reviews.

- Complete the design and solicit a local review of the contract documents.

- Submit construction documents to the local jurisdiction and any permitting agencies for review and approval.

- Submit documents to the building department.

- Select the contractor (if bidding is used), or finalize design/build or construction management contracts.

- Undertake critical facility construction.

- Administer the construction contract.

- Monitor the construction progress and conduct inspections, as required.

- Complete contracted tasks.

- Conduct inspections and provide proof of the architect's acceptance.

- Inspect the critical facility and obtain concurrence/acceptance by the owner.

- Commission the facility and occupy it.

The sequence of the above steps may vary, depending on the complexity of the program; some steps may be implemented simultaneously. Figure 1-3 shows a flow chart of this typical process. Also shown (in the five boxes to the right) are specific activities related to designing for multiple hazards and how these activities fit into the construction process.

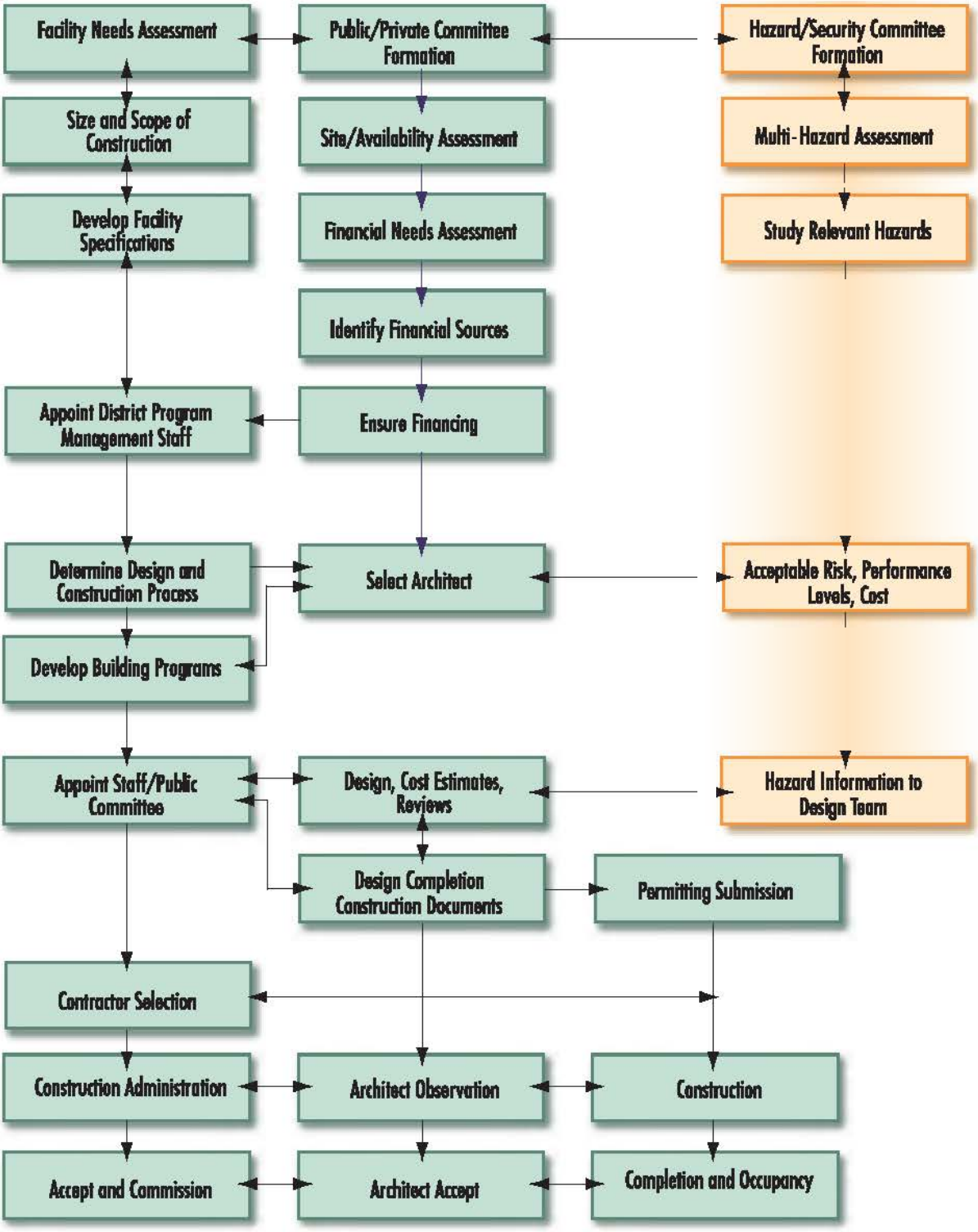

Figure 1-3: Process flow chart for decisionmakers

1.3 PERFORMANCE-BASED DESIGN

1.3.1 BACKGROUND

The model building codes define the minimum design requirements to ensure occupants' safety in critical facilities. Recent natural disasters have forced recognition that damage can occur even when buildings are compliant with the building code. The fact that a large number of critical facilities in communities affected by Hurricane Katrina were shut down (frequently as a result of minor building or equipment damage) suggests that satisfying the minimum code criteria may not be sufficient to ensure continued availability of critical services. Communities depend on the uninterrupted operation of critical facilities, especially during and immediately following natural disasters. In order to meet that need, critical facilities should be designed and constructed according to criteria that result in continued and uninterrupted provision of critical services.

Building performance indicates how well a structure supports the defined needs of its users. The term "performance," as it relates to critical facilities exposed to natural

Performance-based codes define acceptable or tolerable levels of risk for a variety of health, safety, and public welfare issues. Currently available are the *International Code Council Performance Code for Buildings and Facilities* by the International Code Council (ICC, 2006), *101 Life Safety Code* (NFPA, 2006a), and the *NFPA 5000 Building Construction and Safety Code* (NFPA, 2006b) by the National Fire Protection Association (NFPA). The ICC performance code addresses all types of building issues, while the provisions of the *101 Life Safety Code,* "Performance-Based Option," address only issues related to "life safety systems." The *NFPA 5000 Building Construction and Safety Code* sets forth both performance and prescriptive options that apply to all traditional building code issues.

hazards, usually refers to a building's condition after a disaster, i.e., it signifies a level of damage or a load. Acceptable performance indicates acceptable levels of damage or a building condition ,that allows uninterrupted facility operation. Consequently, performance-based design for critical facilities is the process or methodology used by design professionals to create buildings that protect a facility's functionality and the continued availability of services. This approach represents a major change in perception that gives performance-based design considerations a greater importance in the decisionmaking process for design and construction of critical facilities.

The performance-based design approach is not proposed as an immediate substitute for design to traditional codes. Rather, it is seen as an opportunity for enhancing and tailoring the design to match the objectives of the community.

FEMA recently funded the development of next-generation, performance-based seismic design guidelines for new and existing buildings. This process includes detailed modeling; simulation of building response to extreme loading; and estimation of potential casualties, loss of occupancy, and economic losses. The process allows the design of a building to be adjusted to balance the level of acceptable risks and the cost of achieving the required level of building performance. Currently the process focuses on seismic hazards, but it is general enough to be used with other hazards, as soon as the development of performance-based design criteria for wind and other extreme loads advances to the point that they can be incorporated into standardized models.

1.3.2 PRESCRIPTIVE VS. PERFORMANCE-BASED DESIGN

Design and construction in the United States is generally regulated by building codes and standards. Building codes typically seek to ensure the health, safety, and well-being of people in buildings. Toward this purpose, the building codes and standards set minimum design and construction requirements to address structural strength, adequate means of egress for facilities, sanitary equipment, light and ventilation, and fire safety. Building regulations may also promote other objectives, such as energy efficiency, serviceability, quality or value, and accessibility for persons with disabilities. These prescriptive standards are easy to understand and follow, and easy to monitor. This is their great strength.

Historically, building codes were based on a prescriptive approach that limited the available solutions for compliance, which did not encourage creativity and innovation. Prescriptive or specifica-

tion-based design emphasized the "input," or the materials and methods required. In contrast, the focus of performance-based design is the "output," or the expectations and requirements of the users of a building.

Performance-based design requirements define goals and objectives to be achieved and describe methods that can be used to demonstrate whether buildings meet these goals and objectives. This approach provides a systematic method for assessing the performance capabilities of a building, system, or component, which can then be used to verify the equivalent performance of alternatives, deliver standard performance at a reduced cost, or confirm the higher performance needed for critical facilities.

1.3.3 THE PROCESS OF PERFORMANCE-BASED DESIGN OF CRITICAL FACILITIES

The performance-based design process explicitly evaluates how building systems are likely to perform under a variety of conditions associated with potential hazard events. The process takes into consideration the uncertainties inherent in quantifying potential risks and assessing the actual responses of building systems and the potential effects of the performance of these systems on the functionality of critical facilities. Identifying the performance capability of a facility is an integral part of the design process and guides the many design decisions that must be made. Figure 1-4 presents the key steps in this iterative performance-based design process.

Performance-based design starts with selecting design criteria articulated through one or more performance objectives. Each performance objective is a statement of the acceptable risk of incurring different levels of damage and the consequential losses that occur as a result of this damage. Losses can be associated with structural or nonstructural damage, and can be expressed in the form of casualties, direct economic costs, and loss of service costs. Loss of service costs may be the most important loss component to consider for critical facilities. Acceptable risks are typically expressed as acceptable losses for specific levels of hazard intensity and frequency. They take into consideration all the potential hazards that could affect the building and the probability of their

occurrence during a specified time period. The overall analysis must consider not only the intensity and frequency of occurrence of hazard events, but also the effectiveness and reliability of the building systems to survive the event without significant interruption in the operation of a facility.

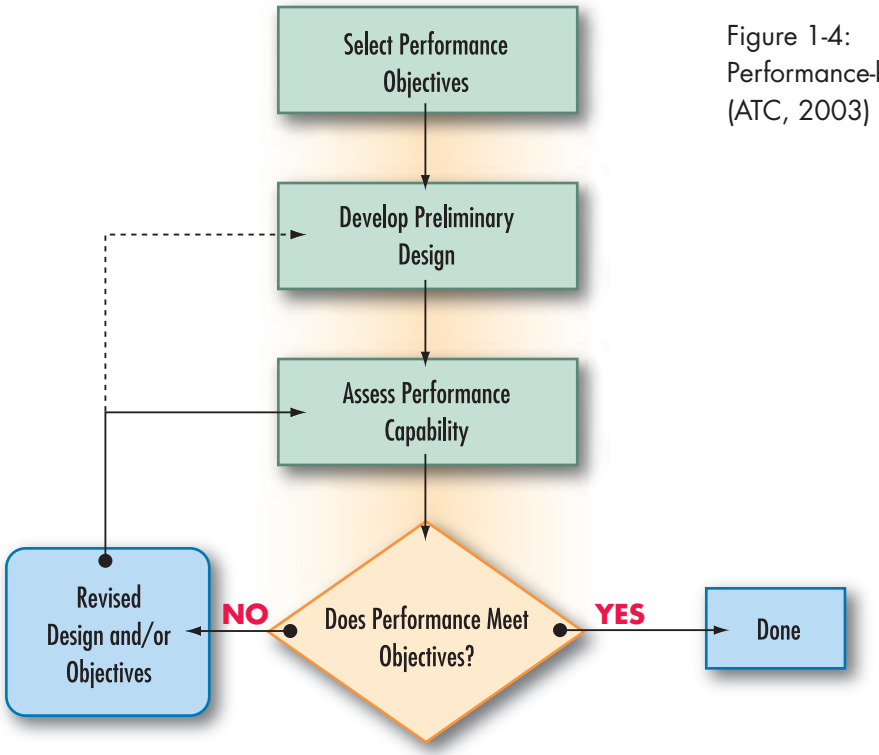

Figure 1-4:
Performance-based design flow diagram (ATC, 2003)

1.3.4 ACCEPTABLE RISK AND PERFORMANCE LEVELS

Performance-based design requires a quantitative measure of risk. It also establishes the basis for evaluating acceptable losses and selecting appropriate designs. While specific performance objectives can vary for each project, the notion of acceptable performance generally follows a trend corresponding to:

- Little or no damage for small, frequently occurring events
- Moderate damage for medium-sized, less frequent events
- Significant damage for very large, very rare events

Performance objectives should be higher and the corresponding acceptable levels of damage lower for critical facilities and other important buildings than for non-critical facilities. This trend is illustrated in Figure 1-5, taken from the ICC Performance Code for Buildings and Facilities (ICC, 2006). This document defines acceptable performance for facilities in one of four performance groups (I, II, III, and IV), using four damage levels (mild, moderate, high, and severe), and given four hazard levels (small, medium, large, and very large). The relative return periods (length of time between occurrences) commonly associated with the hazard levels for each type of hazard event (seismic, flood, and wind) are indicated in Figure 1-6.

Since losses can be associated with structural damage, nonstructural damage, or both, performance objectives must be expressed in terms of the potential performance of both structural and nonstructural systems. The *ICC Performance Code for Buildings and Facilities* has formalized the following four design performance levels, each of which addresses structural damage, nonstructural systems, occupant hazards, overall extent of damage, and release of hazardous materials. These definitions are general to all hazards and are related to tolerable limits of impact to the building, its contents, and its occupants.

Mild Impact: At the mild impact level, the building has no structural damage and is safe to occupy. The nonstructural systems needed for normal building or facility use and emergency operations are fully operational. The number of injured occupants is minimal, and the nature of the injuries minor. The overall extent of the damage is minimal. Minimal amounts of hazardous materials may be released into the environment.

Moderate Impact: At the moderate impact level, structural damage is repairable and some delay in re-occupancy can be expected. The nonstructural systems needed for normal building or facility use and emergency operations are fully operational, although some cleanup and repair may be needed. Injuries to occupants may be locally significant, but generally moderate in numbers and in nature. There is a low likelihood of a single life loss and very low likelihood of multiple life loss. The extent of the damage can be locally significant,

but is moderate overall. Some hazardous materials may be released into the environment, but the risk to the community is minimal.

MAGNITUDE OF DESIGN EVENT (Increasing Magnitude of Event)	INCREASING LEVEL OF PERFORMANCE (Performance Groups)			
	Performance Group I	Performance Group II	Performance Group III	Performance Group IV
Very Large (Very rare)	Severe	Severe	High	Moderate
Large (Rare)	Severe	High	Moderate	Mild
Medium (Less Frequent)	High	Moderate	Mild	Mild
Small (Frequent)	Moderate	Mild	Mild	Mild

Figure 1-5: Maximum level of damage to be tolerated (Table 303.3, ICC, 2006b)
Note: Performance Group I: Buildings that represent a low hazard to human life in the event of failure. Performance Group II: All buildings except those in Groups I, III, and IV. Performance Group III: Buildings with a substantial hazard to human life in the event of failure. Group IV: Buildings designed as essential facilities, including emergency operations centers and designated disaster shelters.

MAGNITUDE OF DESIGN EVENT	DESIGN EVENT		
	Seismic	Flood	Wind
Very Large (Very rare)	2,475 Years	Determined on Site-Specific Basis	125 Years
Large (Rare)	475 Years (Not to Exceed Two-Thirds of the Intensity of Very Large	Determined on Site-Specific Basis	100 Years
Medium (Less Frequent)	72 Years	500 years	75 Years
Small (Frequent)	25 Years	100 Years	50 Years

Figure 1-6: Relative magnitude and return period for seismic, flood, and wind events (ICC, 2006b)

High Impact: At the high impact level, there is significant damage to structural elements, but no falling debris. Significant delays in reoccupancy can be expected. The nonstructural systems needed for normal building use are significantly damaged and inoperable. Emergency systems may be damaged, but remain operational. Injuries to occupants may be locally significant with a high risk to life, but are generally moderate in numbers and nature. There is a moderate likelihood of a single life loss, with a low probability of multiple life loss. The extent of damage can be generally significant and at some locations total. Hazardous materials are released into the environment, with localized relocation required in the immediate vicinity.

Severe Impact: At the severe impact level, there is substantial structural damage. Repair may not be technically possible. The building is not safe for re-occupancy due to the potential for collapse. The nonstructural systems for normal use and emergency systems may be nonfunctional. Injuries to occupants may be high in number and significant in nature. Significant risk to life may exist. There is a high likelihood of single life loss and a moderate likelihood of multiple life loss. Overall damage is substantial. Significant amounts of hazardous materials may be released into the environment, with relocation needed beyond the immediate vicinity.

Once the preliminary design has been developed, a series of simulations (analyses of building response to loading) are performed to estimate the probable performance of the building under various design scenario events. Using fragility relationships (vulnerability functions defining the relationship between load and damage) developed through testing or calculation, building responses are equated to damage states expressed as levels of performance. If the simulated performance meets or exceeds the performance objectives, the design is completed. If not, the design must be revised in an iterative process until the performance objectives are met. In some cases it will not be possible to meet the stated objective at a reasonable cost, in which case the team of decisionmakers may elect to relax some of the original performance objectives.

Continued and uninterrupted operation is the most important performance requirement of any critical facility, regardless of the level of structural and nonstructural building damage. In other

words, the acceptable performance of a critical facility is achieved as long as the structural and nonstructural damage to the building does not disrupt or impair the continued operation of that facility. In recent hurricanes, however, undamaged structures were frequently rendered inoperable as a result of nonstructural damage resulting in unacceptable performance (FEMA, 2006).

In terms of affecting the ability of a facility to function, the failure of nonstructural systems (roofing; exterior envelope; heating, ventillating, and air conditioning [HVAC]; emergency systems) can be as significant as the failure of structural components. Performance-based design provides a framework for considering the potential hazards that can affect a facility or site, and for explicitly evaluating the performance capability of the facility and its components.

Consideration must also be given to the likely possibility that at least a portion of the distribution systems for critical infrastructure services (e.g., electrical power, communications, potable water, and sanitary sewer) could be interrupted. The impact of such an interruption in service should be assessed for the facility, along with an estimate of the time it would take until service could be restored or supplemented. For protecting the continued operation of critical facilities, the most reliable approach is to provide alternative onsite sources for critical infrastructure needs in the form of: (1) emergency power generation capabilities; (2) local wireless communications; (3) potable water supplies; and (4) temporary onsite storage for sanitary waste.

While the practice of performance-based design is currently more advanced in the field of seismic design than the fields of flood and high-wind design, the theory of performance-based design is completely transferable to all hazards. The practice of performance-based design will prompt designers and owners of buildings in flood- or high-wind-prone regions to begin thinking in terms of a few basic objectives:

- Can the real probabilities and frequencies of high-wind and flood events during the useful life of the building be defined with an acceptable degree of accuracy?

- Can the extent and kinds of damage that can be tolerated be defined?

- Are there ways in which an acceptable level of performance can be achieved?

- Are there alternative levels of performance that can be achieved, and how much do they cost over the lifetime/ownership of the building compared to the benefits of reduced damage and improved performance?

- How do these levels compare to the performance levels of designs using the minimum requirements of the applicable building code?

1.3.5 PERFORMANCE-BASED FLOOD DESIGN

The performance levels and objectives for flood hazards, first outlined in FEMA 424 (2004), have been expanded and generalized for performance-based flood design of critical facilities as follows:

Level 1 (Operational): The facility sustains no structural or nonstructural damage, emergency operations are fully functional, and the building can be immediately operational. The site is not affected by erosion, but may have minor debris and sediment deposits.

Level 2 (Moderate Impact): The facility is affected by flooding above the lowest floor, but damage is minimal due to low depths and short duration of flooding. Cleanup, drying, and minor repairs are required, especially of surface materials and affected equipment, but the building can be back in service in a short period of time.

Level 3 (High Impact): The facility may sustain structural or nonstructural damage that requires repair or partial reconstruction, but the threat to life is minimal and occupant injuries should be few and minor. Water damage to the interior of the facility requires cleanup, drying, and repairs, and can prohibit occupancy of all or a portion of the facility for several weeks to several months.

Level 4 (Severe Impact): The facility is severely damaged and likely requires demolition or extensive structural repair. Threats to occupants are substantial, and warning plans should prompt evacuation

prior to the onset of this level of flooding. Level 4 is applicable to facilities affected by all types of flooding, including those that result from failure of dams, levees, or floodwalls.

Planning and design to achieve an appropriate level of flood protection for critical facilities should include avoidance of flood hazard areas and adding a factor of safety (freeboard) to the anticipated flood elevation. Performance evaluation of a facility affected by flooding needs to include consideration of the building response to the following load conditions (fragility functions must be developed to relate calculated response to actual damage states):

- Lateral hydrostatic forces
- Vertical (buoyant) hydrostatic forces
- Hydrodynamic forces
- Surge forces
- Impact forces of flood-borne debris
- Breaking wave forces
- Localized scour

1.3.6 PERFORMANCE-BASED HIGH-WIND DESIGN

The performance objectives for wind hazards, outlined in FEMA 424, have been expanded and generalized for performance-based flood design of critical facilities as follows:

Level 1 (Operational): The facility is essentially undamaged and can be immediately operational.

Level 2 (Moderate Impact): The facility is damaged and needs some repairs, but can remain occupied and be functional after minor repairs to nonstructural components are complete.

Level 3 (High Impact): The facility may be structurally damaged but the threat to life is minimal and occupant injuries should be few and minor. However, damage to nonstructural components (e.g., roofing, building envelope, exterior-mounted equipment) is great, and the cost to repair the damage is significant. If rain accompanies the windstorm, or if rain occurs prior to execution of emergency repairs, water damage to the interior of the facility can prohibit occupancy of all or a portion of the facility for several weeks to several months.

Level 4 (Severe Impact): The facility is severely damaged and will probably need to be demolished. Significant collapse may have occurred, and there is a great likelihood of occupant casualties unless the facility has a specially designed occupant shelter. Level 4 is applicable to facilities struck by strong or violent hurricanes or tornadoes. For other types of windstorms, Level 4 should not be reached.

The challenge with respect to performance-based high-wind design is assessing the wind resistance of the building envelope and exterior-mounted equipment, and the corresponding damage susceptibility. This is challenging because of several factors:

- Analytical tools (i.e., calculations) are currently not available for many envelope systems and components, and there is a lack of realistic long-term wind resistance data.

- Because of the complexity of their wind load response, many envelope systems and components require laboratory testing, rather than analytical evaluation, in order to determine their load-carrying capacity.

- It is likely that finite element analysis will eventually augment or replace laboratory testing, but substantial research is necessary before finite element analysis becomes available for the broad range of existing building envelope systems.

- Before performance-based design for high winds can become a reality, a solid research base on the response of buildings and components to the effects of high winds must be established.

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2.1 GENERAL DESIGN CONSIDERATIONS

This chapter introduces the physical nature and mechanics of floods and explains how flood probabilities are determined and how flood hazard areas are identified. It describes the types of flood damage that can result when critical facilities are located in flood hazard areas or are affected by flooding. A series of requirements and best practices are introduced that facility owners, planners, and designers should consider for reducing the risks from flooding to new critical facilities and to existing facilities already located in areas prone to flooding.

This chapter demonstrates why avoidance of flood hazard areas is the most effective way to minimize the life-safety risk to the occupants and general public who rely on these facilities, as well as to minimize the potential for damage to buildings and other elements of critical facilities. When an existing facility is exposed to flooding, or if a new facility is proposed for a flood hazard area, steps need to be taken to minimize the risks. A well-planned, designed, constructed, and maintained critical facility should be able to withstand damage and remain functional after a flooding event, even one of low probability.

2.1.1 NATURE AND CHARACTERISTICS OF FLOODING

Flooding is the most common natural hazard in the United States, affecting more than 20,000 local jurisdictions and representing more than 70 percent of Presidential disaster declarations. Several

evaluations have estimated that 7 to 10 percent of the Nation's land area is subject to flooding. Some communities have very little flood risk; others lie entirely within a floodplain.

Flooding is a natural process that may occur in a variety of forms: long-duration flooding along rivers that drain large watersheds; flash floods that send a devastating wall of water down a mountain canyon; and coastal flooding that accompanies high tides and onshore winds, hurricanes, and nor'easters. When the natural process does not affect human activity, flooding is not a problem. In fact, many species of plants and animals that live adjacent to bodies of water are adapted to a regimen of periodic flooding.

Flooding is only considered a problem when human development is located in flood-prone areas. Such development exposes people to potentially life threatening situations and makes property vulnerable to serious damage or destruction. It also can disrupt the natural surface flow, redirecting water onto lands not normally subject to flooding.

Flooding along waterways normally occurs as a result of excessive rainfall or snowmelt that creates water flows exceeding the capacity of channels. Flooding along shorelines is usually a result of coastal storms that generate storm surges or waves above normal tidal fluctuations. Factors that can affect the frequency and severity of flooding and the resulting damage include:

- Channel obstructions caused by fallen trees, accumulated debris, and ice jams
- Channel obstructions caused by road and rail crossings where the bridge or culvert openings are insufficient to convey floodwaters
- Erosion of shorelines and stream banks, often with episodic collapse of large areas of land
- Deposition of sediment that settles out of floodwaters or is carried inland by wave action
- Increased upland development of impervious surfaces and manmade drainage improvements that increase runoff volumes

- Land subsidence, which increases flood depths

- Failure of dams (resulting from seismic activity, lack of maintenance, flows that exceed the design, or destructive acts), which may suddenly and unexpectedly release large volumes of water

- Failure of levees (associated with flows that exceed the design, weakening by seismic activity, lack of maintenance, or destructive acts), which may result in sudden flooding of areas behind levees

Each type of flooding has characteristics that represent important aspects of the hazard. These characteristics should be considered in the selection of critical facility sites, the design of new facilities, and the expansion or rehabilitation of existing flood-prone facilities.

Riverine flooding results from the accumulation of runoff from rainfall or snowmelt, such that the volume of flow exceeds the capacity of waterway channels and spreads out over the adjacent land. Riverine flooding flows downstream under the force of gravity. Its depth, duration, and velocity are functions of many factors, including watershed size and slope, degree of upstream development, soil types and nature of vegetation, topography, and characteristics of storms (or depth of snowpack and rate of melting). Figure 2-1 illustrates a cross-section of the generic riverine floodplain.

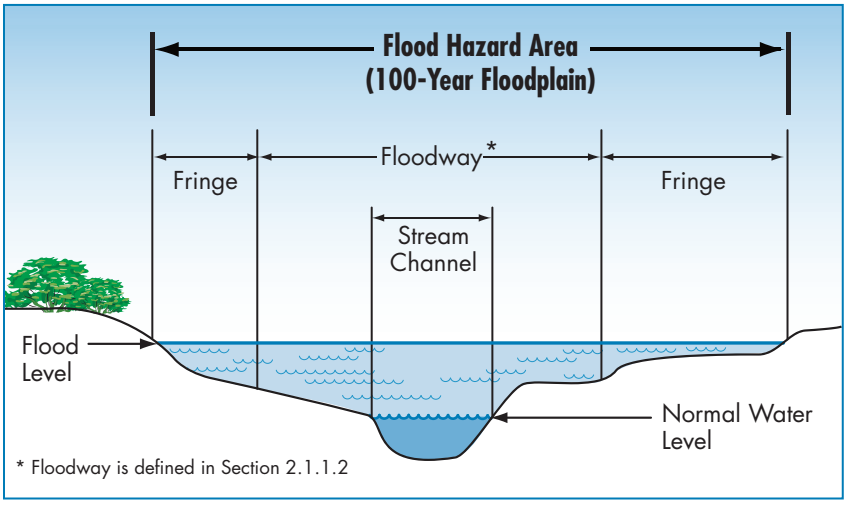


Figure 2-1:
The riverine floodplain

Coastal flooding is experienced along the Atlantic, Gulf, and Pacific coasts, and many larger lakes, including the Great Lakes. Coastal flooding is influenced by storm surges associated with tropical cyclonic weather systems (hurricanes, tropical storms, tropical depressions, typhoons), extratropical systems (nor'easters), and tsunamis (surge induced by seismic activity). Coastal flooding can also be characterized by wind-driven waves, which also affect reaches along the Great Lakes shorelines; winds blowing across the broad expanses of water generate waves that can rival those experienced along ocean shorelines. Some Great Lakes shorelines experience coastal erosion, in part because the erosion is associated with fluctuations in water levels. Figure 2-2 is a schematic of the generic coastal floodplain.

A number of factors associated with riverine and coastal flooding are important in the selection of sites for critical facilities, in site design, and in the architectural and engineering design of critical facilities.

Depth: The most obvious characteristic of any flood is the depth of the water. Depending on many factors, such as the shape of a river

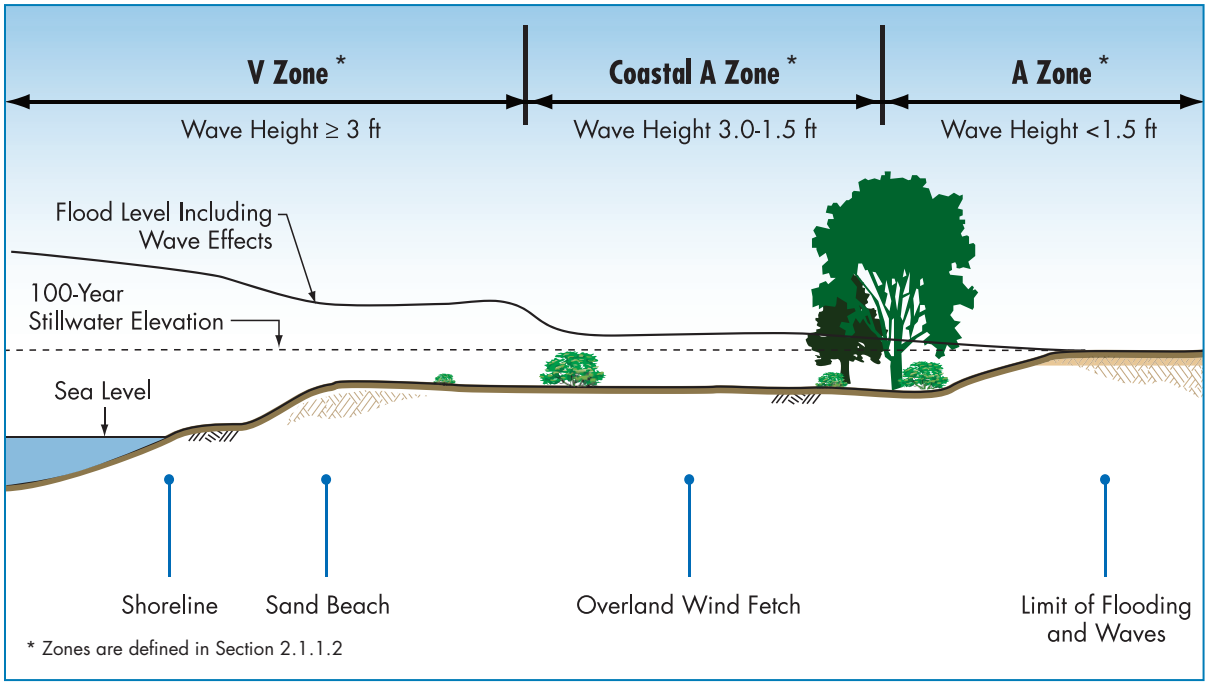


Figure 2-2: The floodplain along an open coast

valley or the presence of obstructing bridges, riverine flooding may rise just a few feet or tens of feet above normal levels. The depth of coastal flooding is influenced by such factors as the tidal cycle, the duration of the storm, the elevation of the land, and the presence of waves. Depth is a critical factor in building design, because the hydrostatic forces on a vertical surface (such as a foundation wall) are directly related to depth, and because costs associated with protecting buildings from flooding increase with depth. Under certain conditions, hurricanes can produce storm surge flooding that is 20 to 30 feet above mean sea level or, in extreme cases such as reported during Hurricane Katrina, as much as 35 feet above mean sea level.

Duration: Duration is the measure of how long the water remains above normal levels. The duration of riverine flooding is primarily a function of watershed size and the longitudinal slope of the valley (which influences how fast water drains away). Small watersheds are more likely to be "flashy," which refers to the rapidity with which floodwaters rise and fall. Areas adjacent to large rivers may be flooded for weeks or months. Most coastal flooding is influenced by the normal tidal cycle, as well as how fast coastal storms move through the region. Areas subject to coastal flooding can experience long duration flooding where drainage is poor or slow as a result of topography or the presence of flood control structures. For example, there may be depressions in the land that would hold water, or water may be trapped behind a floodwall or levee with inadequate drainage. More commonly, coastal flooding is of shorter duration, on the order of 12 to 24 hours, especially if storms move rapidly. Flooding along large lakes, including those behind dams, can be of very long duration because the large volume of water takes longer to drain. For building design, duration is important because it affects access, building usability, and saturation and stability of soils and building materials. Information about flood duration is sometimes available as part of a flood study, or could be developed by a qualified engineer.

Local drainage problems create ponding and local flooding that often is not directly associated with a body of water such as a creek or river. Although such flooding is relatively shallow and not characterized by high velocity flows, considerable damage may result. Areas with poor drainage frequently experience repetitive damage. Some local drainage problems are exacerbated by old or undersized drainage system infrastructure. Flooding caused by drainage problems typically occurs as sheetflow or along waterways with small drainage areas. This type of flooding is often not mapped or regulated.

Velocity: The velocity of floodwaters ranges from extremely high (associated with flash floods or storm surge) to very low or nearly stagnant (in backwater areas and expansive floodplains). Velocity is important in site planning because of the potential for erosion. In structural design, velocity is a factor in determining the hydrodynamic loads and impact loads. Even shallow, high-velocity water can threaten the lives of pedestrians and motorists. Accurate estimates of velocities are difficult to make, although velocity information may be found in some floodplain studies.

Wave action: Waves contribute to erosion and scour (see Figure 2-2), and also contribute significantly to design loads on buildings. The magnitude of wave forces can be 10 to more than 100 times greater than wind and other design loads, and thus may control many design parameters. Waves must be accounted for in site planning along coastal shorelines, in flood hazard areas that are inland of open coasts, and other areas where waves occur, including areas with sufficient fetch that winds can generate waves (such as lakes and expansive riverine floodplains). Waves on top of storm surges may be as much as 50 percent higher than the depth of the surge.

Impacts from debris and ice: Floating debris and ice contribute to the loads that must be accounted for in structure design. The methods and models used to predict and delineate flood hazard areas do not specifically incorporate debris loads. Thus, there are few sources to determine the potential effects of debris impact, other than past observations and judgment.

Erosion and scour: Erosion is the lowering of the ground surface as a result of a flood event, or the gradual recession of a shoreline as a result of long-term coastal processes. Scour refers to a localized lowering of the ground surface due to the interaction of currents and/or waves with structural elements, such as pilings. Soil characteristics influence an area's susceptibility to scour. Erosion and scour may affect the stability of foundations and filled areas, and may cause extensive site damage.

2.1.1.1 Probability of Occurrence or Frequency

The probability of occurrence, or frequency, is a statement of the likelihood that an event of a certain magnitude will occur in a given period of time. For many decades, floodplain management has been based on the flood that has a 1 percent chance of occurring in any given year, commonly called the "100-year flood." For certain critical actions and decisions, such as planning or constructing a critical facility, the basis of risk decisions should be the flood that has a 0.2 percent probability of occurring in any given year, commonly called the "500-year flood."

The assigned frequency of a flood (e.g., 100-year) is independent of the number of years between actual occurrences. Hurricane Camille hit the Mississippi coast in 1969 with storm surge flooding that far exceeded previous events, and Hurricane Katrina affected much the same area. Although just 36 years apart, both storms produced flood levels significantly higher than the 100-year flood.

The term "100-year flood" as an expression of probability or frequency is often misunderstood because it conveys the impression that a flood of that magnitude will occur only once every 100 years. Actually, the 1-percent-annual-chance flood has one chance in 100 of occurring in any given year. The fact that a 1-percent-annual-chance flood is experienced at a specific location does not alter the probability that a comparable flood could occur at the same location in the next year, or even multiple times in a single year. As the length of the period increases, so does the probability that a flood of a specific magnitude or greater will occur. For example, during a 30-year period (the usual lending period for a home mortgage), the probability that a 100-year flood will occur is 26 percent. And during a 70-year period (the potential useful life of many buildings), the probability increases to 50 percent. Similarly, the 500-year flood has a 0.2-percent probability of being equaled or exceeded in any given year, and during a 70-year period the probability of occurrence is 18 percent.

Regardless of the flood selected for design purposes (the "design flood"), the designer must determine specific characteristics associated with that flood. Determining a flood with a specific probability of occurrence is done in a multi-step process that typically involves using computer models that are in the public domain. If a sufficiently long record of flood information exists, the design flood may be determined by applying statistical tools to the data. Alternatively, water resource engi-

Flood frequency analyses are performed using historical records, and the results are influenced by the length of the record. Such analyses do not account for recent changes to the land (upland development or subsidence) or future changes (additional development, greater subsidence, or climatic variations).

neers sometimes apply computer models to simulate different rainfall events over watersheds, to predict how much water will run off and accumulate in channels. Other computer models are used to characterize the flow of water down the watershed and predict how high the floodwaters will rise.

For coastal areas, both historical storms and simulated storm surge models can be used to predict the probability that floodwaters will rise to a certain level and be accompanied by waves of certain heights. Many coastal storms will produce storm surge flooding that, depending on local topography, may extend inland significantly farther than anticipated for the 1-percent-annual-chance flood. Statistically, such extreme storm surges occur less frequently than the 1-percent or 0.2-percent-annual-chance floods, but their consequences can be catastrophic.

Planners and designers should research the relationship between the flood levels for different frequency events and extreme events, especially in hurricane-prone communities. The difference in flood levels may be extreme in some situations, depending on local conditions and the source of flooding. In other areas the lower probability flood depths might not be much higher than the 1-percent-annual-chance flood.

The National Flood Insurance Program (NFIP) is a Federal program that encourages communities to regulate flood hazard areas and, in return, offers property owners insurance protection against losses from flooding (see Sections 2.1.3.1 and 2.1.3.2). The NFIP uses the 1-percent-annual-chance flood as the basis for flood hazard maps, for setting insurance rates, and for application of regulations in order to minimize future flood damage. The 1-percent-annual-chance flood is also used as the standard for examination of older buildings to determine the measures to apply in order to reduce future damage.

Satisfying the minimum requirements of the NFIP does not provide adequate protection for critical facilities that need to be functional even after low probability events. Nearly every

year, a very low probability flood occurs somewhere in the United States, often with catastrophic consequences. Therefore, for planning and design of critical facilities, use of a lower probability flood (at least the 500-year) is strongly recommended. As noted in Section 2.1.3.3, the 500-year level of protection is required if Federal funds are involved in constructing facilities that are vital for emergency response and rapid recovery, including hospitals, EOCs, emergency shelters, and other buildings that support vital services. This reinforces the importance of protecting both the functionality and financial investment in a critical facility with stricter standards than those applied to other buildings.

The Saffir-Simpson Hurricane Scale categorizes hurricanes based on sustained wind speeds (see Section 3.1.1). Storm surge is not always correlated with the category because other factors influence surge elevations, notably forward speed of the storm, tide cycle, offshore bathymetry, and land topography.

2.1.1.2 Hazard Identification and Flood Data

Flood hazard maps identify areas of the landscape that are subject to flooding, usually flooding by the 1-percent-annual-chance flood. Maps prepared by the NFIP are the minimum basis of State and local floodplain regulatory programs. Some States and communities have prepared maps of a floodplain based on the assumption that the upper watershed area is fully developed according to existing zoning. Some communities base their regulations on a flood of record or a historically significant flood that exceeds the base flood shown on the NFIP maps.

The flood hazard maps used by the appropriate regulatory authority should be consulted during planning and site selection, site design, and architectural and engineering design (whether for the design of new buildings or rehabilitation of existing buildings). Regardless of the flood hazard data required for regulatory purposes, additional research should be conducted on past major floods and other factors that could lead to more severe flooding.

The NFIP produces Flood Insurance Rate Maps (FIRMs) for more than 20,000 communities nationwide. FIRMs are prepared for each local jurisdiction that has been determined to have some degree of flood risk. The current effective maps are typically available for

It is important to note that the number of revised and updated FIRMs is increasing rapidly. During the last few years FEMA, in partnership with many States and communities, has been implementing an initiative to modernize and update all maps that are determined to be out of date. The modernization process may involve an examination of flood experience in the period since the original flood studies were prepared, use of more detailed topography and base maps, re-computation of flood discharges and flood heights, and re-delineation of flood hazard area boundaries.

viewing in community planning or permit offices.[1] It is important to use the most recent flood hazard map when determining site-specific flood hazard characteristics. Although many FIRMs are more than 15 years old, often one or more panels or portions of a map panel have been revised and republished. Communities must adopt revised maps to continue participating in the NFIP.

Some FIRMs do not show the 0.2-percent-annual-chance flood hazard area (500-year floodplain), and many FIRMs do not provide detailed information about predicted flood elevations along every body of water, especially smaller streams and tributaries. Determining the 500-year flood is especially difficult when records of past flood events are limited. When existing data are insufficient, additional statistical methods and engineering analyses are necessary to determine the flood-prone areas and the appropriate characteristics of flooding required for site layout and building design.

If a proposed facility site or existing facility is affected by flooding, a site-specific topographic survey is critical to delineate the land that is below the flood elevation used for planning purposes. If detailed flood elevation information is not available, a floodplain study may be required to identify the important flood characteristics and data required for sound design. Having flood hazard areas delineated on a map conveys a degree of precision that may be misleading. Flood maps have a number of limitations that should be taken into consideration, especially during site selection and design of critical facilities. Some of the well-known limitations are:

- Flood hazard areas are approximations based on probabilities; the flood elevations shown and the areas delineated should not be taken as absolutes, in part because they are based on numerical approximations of the real world.

1. Flood maps may also be viewed at FEMA's Map Store at http://www.fema.gov. For a fee, copies may be ordered online or by calling (800) 358-9616. The Flood Insurance Study (FIS) and engineering analyses used to determine the flood hazard area may be ordered through the FEMA Web site.

- FIRMs and Flood Insurance Studies (FISs) are prepared to meet the requirements of the NFIP. For the most part, floodplains along smaller streams and drainage areas (less than 1 square mile) are not shown.

- Especially for older maps, the topography used to delineate the flood boundary may have had contour intervals of 5, 10, or even 20 feet, which significantly affects the precision with which the boundary is determined. The actual elevation of the ground relative to the flood elevation is critical, as opposed to whether an area is shown as being in or out of the mapped flood hazard area.

In communities along the Gulf and Atlantic coasts, facility owners, planners, and designers should check with emergency management offices for maps that estimate storm surge flooding from hurricanes. Local planning or engineering offices may have post-disaster advisory flood maps and documentation of past storm surge events. The FIRMs and regulatory design flood elevations (DFEs) do not reflect low probability/high magnitude flooding that may result from a hurricane making landfall at a specific location.

- Maps are based on the data available at the time they were prepared, and therefore do not account for subsequent upland development (new development that increases rainfall-runoff tends to increase flooding).

- The scale of the maps may impede precise determinations (many older maps are 1 inch = 2,000 feet).

- Flooding characteristics may have been altered by development, sometimes by upland development that has increased runoff, and other times by local modifications that have altered the shape of the land surface of the floodplain (such as fills or levees).

- Local conditions are not reflected, especially conditions that change regularly, such as stream bank erosion and shoreline erosion.

- Areas exposed to very low probability flooding are not shown, such as flooding

Designers and property owners in coastal regions should be aware that current FIRMs may not fully account for natural and manmade changes to beaches, wetlands, and other coastal environments (e.g., the erosion of protective dunes during the base flood). Since the original FIRMs were published in the early 1980s, FEMA has made significant improvements in the models and methods used to identify coastal flood hazards. Before any action is considered, the Flood Insurance Study report should be checked to verify that all pertinent hazards have been addressed. A coastal engineer or similar professional should be consulted if there are any questions concerning the coastal flood data.

from extreme hurricane storm surges, extreme riverine flooding, dam failures, or overtopping or failure of levees.

The flood hazard maps prepared by the NFIP show different flood zones to delineate different floodplain characteristics (see Figures 2-3 and 2-4). The flood zones shown on the NFIP maps, and some other designations, are as described below.

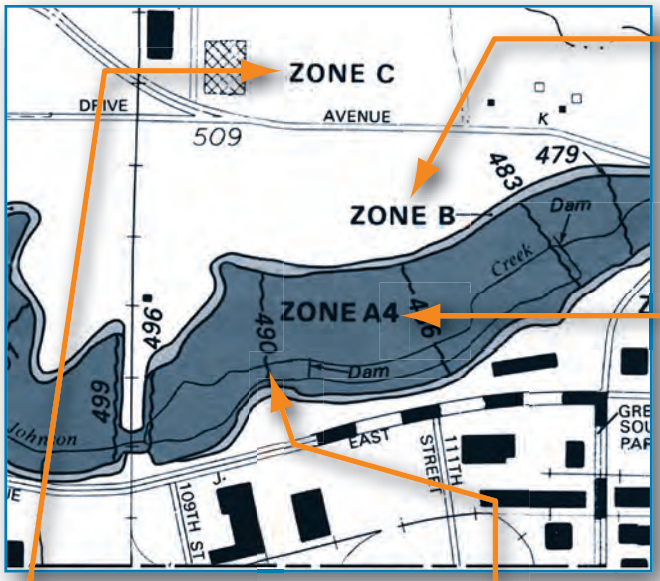
A Zones: (also called "unnumbered A Zones" or "approximate A Zones"). This designation is used for flood hazard areas where engineering analyses have not been performed to develop detailed flood elevations. Base flood elevations (BFEs) are not provided. Additional engineering analyses and site-specific assessments usually are required to determine the design flood elevation.

"Base flood elevation" is the elevation above a datum to which floodwaters are predicted to rise during the 1-percent-annual-chance flood (also called the "base flood" or the 100-year flood).

AE Zones or A1-A30 Zones: (also called "numbered A Zones"). These designations are used for flood hazard areas where engineering analyses have produced detailed flood elevations and boundaries for the base flood (1-percent-annual-chance flood). BFEs are provided. For riverine waterways with these zones, FISs include longitudinal profiles showing water surface elevations for different frequency flood events.

Floodways: The floodway includes the waterway channel and adjacent land areas that must be reserved in order to convey the discharge of the base flood without cumulatively increasing the water surface elevation above a designated height. Floodways are designated for most waterways that have AE Zones or numbered A Zones. FISs include data on floodway widths and mean floodway velocities.

AO and AH Zones: These zones include areas of shallow flooding and are generally shown where the flood depth averages from 1 to 3 feet, where a clearly defined channel does not exist, where the path of flooding is unpredictable, and where velocity flow may be evident. These zones are characterized by ponding or sheetflow. BFEs may be provided for AH Zones; flood depths may be specified in AO Zones.



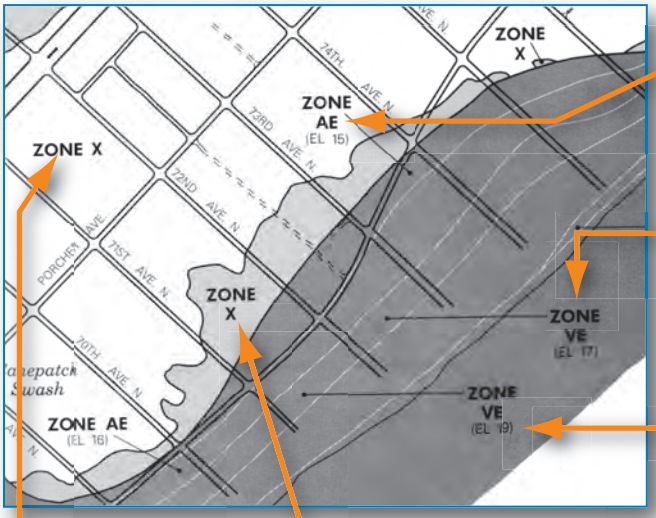
Zone B (or shaded Zone X) is subject to flooding by the 500-year flood (0.2 percent annual chance), and is a moderate risk area.

Zone A, A Zones A1-A30, and **Zone AE** are subject to flooding by the base of the 1percent annual chance (100-year) flood, and are considered high-risk areas.

Zone C (or Zone X) is all other areas, considered low-risk.

Base Flood Elevation (BFE), is the predicted water surface elevation of the base flood at specific locations (in feet above the datum).

Figure 2-3: Riverine flood hazard zones



Zone A, A Zones A1-A30, and **Zone AE** are subject to flooding by the base or 100-year flood (1percent annual chance), and waves less than 3 feet.

Zone V, V Zones V1-V30, and **Zone VE** are where waves are expected to be 3 feet or more.

Base Flood Elevation (BFE) is the predicted water surface elevation (in feet above datum).

Zone X is all other areas.

Shaded Zone X (or Zone B) is subject to flooding by the 500-year flood (0.2 percent annual chance).

Figure 2-4: Coastal flood hazard areas

Shaded X (or B) Zones: This zone shows areas of the 500-year flood (0.2-percent-annual-chance flood), or areas protected by flood control levees. This zone is not shown on many NFIP maps; its absence does not imply that flooding of this frequency will not occur.

Unshaded X (or C) Zones: These zones are all land areas not mapped as flood hazard areas that are outside of the floodplain and designated for the purposes of regulating development pursuant to the NFIP. These zones may still be subject to small stream flooding and flooding from local drainage problems.

V Zones (V, VE, and V1-V30): Also known as coastal high-hazard areas or special flood hazard areas subject to high-velocity wave action. V Zones are relatively narrow areas along open coastlines and some large lake shores that are subject to high-velocity wave action from storms or seismic sources. V Zones extend from offshore to the inland limit of a primary frontal dune, or to an inland limit where the height of breaking waves drops below 3 feet.

Coastal A Zone: This zone, which is not delineated on NFIP maps, is where the potential of breaking wave heights is between 1.5 feet and 3 feet during base flood conditions. Coastal A Zones are landward of the mapped V Zone, or landward of open coasts that do not have a V Zone because breaking waves are predicted to be less than 3 feet high. In these areas, the principal sources of flooding are tides, storm surges, seiches, or tsunamis, not riverine flooding.

Coastal A Zone: The current editions of the model building codes refer to ASCE 7 and ASCE 24, which are two design standards that include requirements for Coastal A Zones that account for the increased risk from the additional wave height (see Section 2.3.2).

Flood hazards and characteristics of flooding must be identified to evaluate appropriately the impact of site development, to calculate flood loads, to design floodproofing measures, and to identify and prioritize retrofit measures for existing critical facilities. Table 2-3 in Section 2.5 outlines a series of questions to facilitate this objective.

Many characteristics of flooding are not shown on the FIRMs but may be found in the FIS or the study or report prepared by the entity that produced the flood hazard map. Hurricane storm surge inundation maps based on the National Hurricane Center models

have been prepared by the U.S. Army Corps of Engineers for most reaches of the Atlantic and Gulf coasts. The maps combine the results of many scenarios to show the maximum potential surge inundation associated with different categories of hurricanes. State and local emergency management offices use the maps for evacuation planning.

Hurricanes can produce storm surge flooding and waves that rise much higher than the BFE shown on the FIRMs.

2.1.1.3 Design Flood Elevation

The DFE establishes the minimum level of flood protection that must be provided. The DFE, as used in the model building codes, is defined as either the BFE determined by the NFIP and shown on FIRMs, or the elevation of a design flood designated by the community, whichever is higher. The DFE will always be at least as high as the BFE. Communities may use a design flood that is higher than the base flood for a number of reasons. For example, a design flood may be used to account for future upland development, to recognize a historic flood, or to incorporate a factor of safety, known as freeboard.

"Freeboard" is a factor of safety usually expressed in feet above a flood level. Freeboard compensates for the many unknown factors that could contribute to flood heights, such as wave action, constricting bridge openings, and the hydrological effect of urbanization of the watershed. A freeboard from 1 to 3 feet is often applied to critical facilities.

Facility owners, planners, and designers should check with the appropriate regulatory authority to determine the minimum flood elevation to be used in site planning and design. Although the NFIP minimum is the BFE, State or local regulations commonly cite the 0.2-percent-annual-chance flood (500-year flood) as the design requirement for critical facilities, or the regulations may call for added freeboard above the minimum flood elevation. Even if there is no specific requirement to use the 0.2-percent-annual-chance flood for siting and design purposes, it is strongly recommended that decisionmakers take into consideration the flood conditions associated with this lower probability event or from other floods of record.

2.1.1.4 Advisory Base Flood Elevation

The flood maps and flood hazard data described in Section 2.1.1.2 are the minimum information required to be used for regulatory purposes. The updating of FIRMs is a continuous process and it relies heavily on examination of storm event data and physical changes to the landscape. If significant flood events have occurred since the effective date of the FIRM, these events may change the statistical analyses, which would then prompt an update of the flood maps and produce revised elevations for the 1-percent-annual-chance flood. Critical facility owners, planners, and designers should contact community officials to determine whether there have been any significant flood events or other changes that may affect flood hazards since the effective date of the FIRM. The best available information should be used at all times.

FEMA works closely with communities to develop new flood hazard data or revise existing data during the standard flood study process. Updating flood hazard data includes the analysis of historical data. If a major flood event significantly alters the physical environment or if it is determined to be statistically significant, FEMA may decide to release Advisory BFEs (ABFEs) and Flood Recovery Maps (see Figure 2-5).

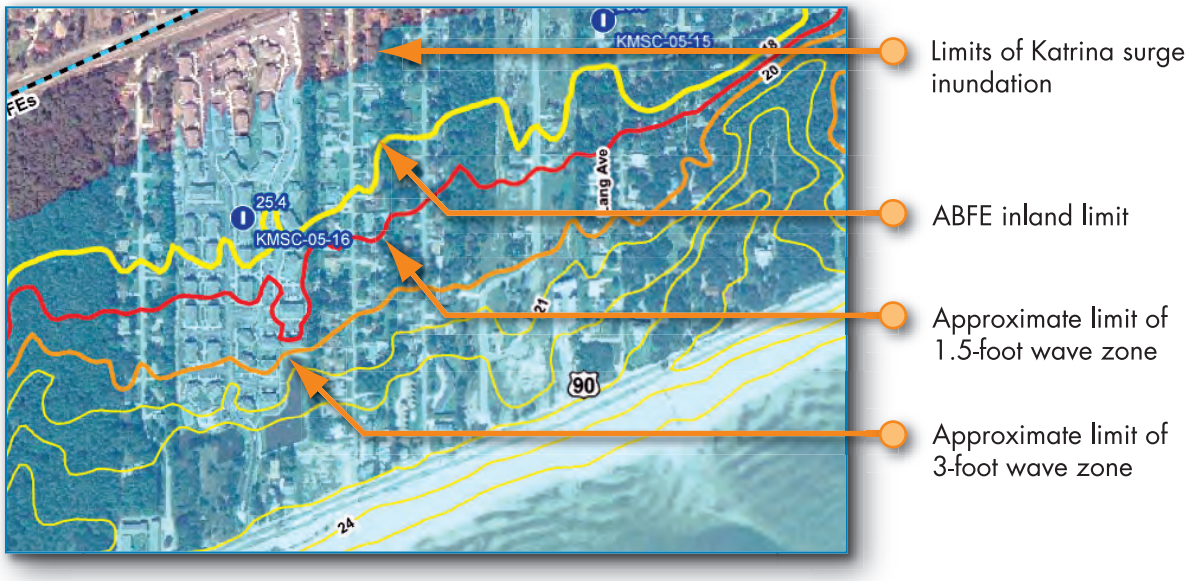


Figure 2-5: Example of a flood recovery map showing ABFEs and other flood hazard information.

ABFEs represent the best estimate of the expected 1-percent-annual-chance flood elevations. They are provided as interim flood hazard information until more detailed flood hazard data become available. Flood Recovery Maps depict the ABFEs, and in general, reflect additional information such as inundation limits and surveyed high water marks (but not the 500-year flood hazard area). For coastal areas, the Flood Recovery Maps may also show the inland debris line and the limit of the 1.5-foot wave (Coastal A Zone).

After Hurricane Katrina, FEMA expedited development of Flood Recovery Maps and ABFEs for the Mississippi coast—the new maps were delivered just 3 months after the storm.

When ABFEs and Flood Recovery Maps are produced and released, FEMA strongly encourages States and communities, as well as private property owners and critical facility owners, to use the information to make decisions about reconstruction until more definitive data become available. FEMA issues guidance to help the users apply the updated flood elevation information at specific locations.

After Flood Recovery Maps are released, FEMA begins the formal process of updating the FIRMs. The community and property owners are notified through public notices and meetings when the preliminary revised maps are available and a formal comment period is opened. The final maps are prepared after consideration of comments. Communities are required to adopt and use the revised FIRMs in order to continue their participation in the NFIP.

2.1.2 FLOOD LOADS

Floodwaters can impose a variety of loads on buildings and building elements. This section provides a brief overview of flood loads and factors that are important for calculating flood loads, including:

- Hydrostatic loads, including buoyancy, which increase as the depth of water increases
- Hydrodynamic loads, which result from moving water

- Breaking wave loads, which are most likely to occur in coastal areas
- Impact loads resulting from floating debris striking a building or building element
- Long-term erosion and localized scour, which can increase the effects and magnitudes of other loads

2.1.2.1 Design Flood Depth

Water depth associated with the design flood is computed by determining the DFE (see Section 2.1.1.3 or 2.1.1.4) and subtracting the elevation of the ground at the building site. Since these elevation data usually are obtained from different sources, it is important to determine whether they are based on the same datum. If not, standard corrections must be applied.

Flood depth is the most important factor required to compute flood loads, because almost every other flood load calculation depends directly or indirectly on this factor. In riverine areas, the flood depth rarely accounts for waves. In coastal areas, the total flood depth is composed of a "stillwater" depth, plus the expected height of waves (see Figure 2-6).

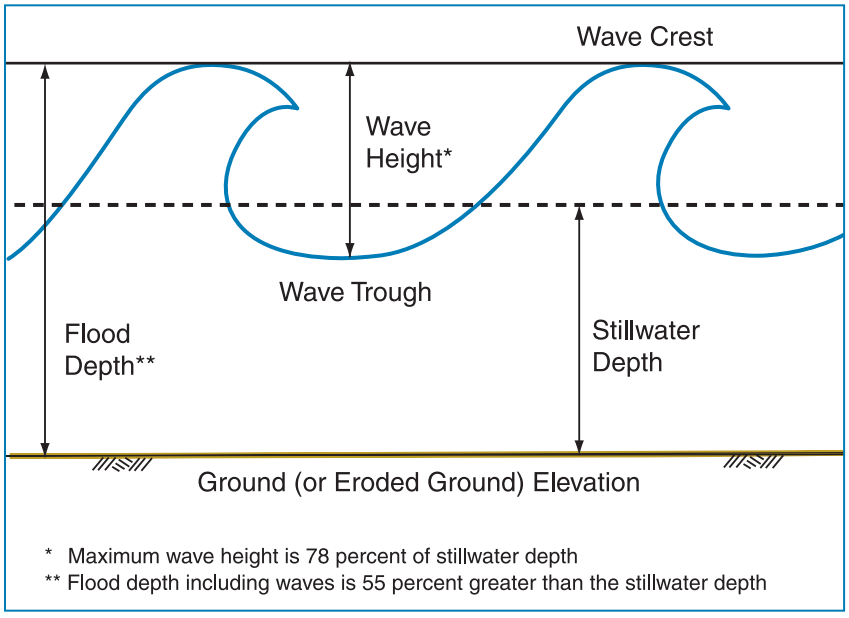


Figure 2-6: Definition sketch—wave height and stillwater depth

The following characteristics that may add to the flood depth should be taken into consideration.

Small waves: In Coastal A Zones (see Section 2.3.2), the DFE shown on FEMA's maps does not include the wave height. Coastal A Zones are characterized by 1.5- to 3-foot high waves. The flood depth should be increased by 3 feet for sites close to the V Zone boundary or the shoreline. For sites farther inland, where the flood depth is at least 3 feet, it should be increased by 1.5 feet. Interpolation may be used to determine the amount that should be added to the flood depth to account for waves in the Coastal A Zone.

> Waves and storm-induced erosion are most common in coastal areas. However, wide rivers and lakes may experience wind-driven waves and erodible soils are found throughout the United States. For more information about waves and erosion, refer to FEMA 55, *Coastal Construction Manual*.

Erosion and scour: Flood depths in areas with erodible soils should consider the effects of erosion where floodwaters lower the ground surface or cause local scour around foundation elements. In these areas, the flood depth determined using the design flood elevation should be increased to account for changes in conditions during a flood event. Not only does lowering the ground surface effectively result in deeper water against the foundation, it may also remove supporting soil from the foundation, which must be accounted for in the foundation design.

2.1.2.2 Design Flood Velocity—Riverine

There are few sources of information that are readily available for estimating design flood velocities at specific locations along riverine bodies of water. If a riverine source has been studied using detailed hydraulic methods, some information may be available in summary form in published studies. Studies prepared for the NFIP (see Section 2.1.1.2) contain tables of data for waterways for which floodways were delineated. For specified cross-sections along the waterway, the Floodway Data Table includes a mean velocity expressed in feet per second. This value is the average of all velocities across the floodway. Generally, velocities in the flood fringe (landward of the floodway) will be lower than in the floodway.

For waterways without detailed studies, methods that are commonly used in civil engineering for estimating open channel flow velocities can be applied.

2.1.2.3 Design Flood Velocity—Coastal

Estimating design flood velocities in coastal flood hazard areas is subject to considerable uncertainty, and there is little reliable historical information or measurements from actual coastal flood events. In this context, velocity does not refer to the motion associated with breaking waves, but the speed of the mass movement of floodwater over an area.

The direction and velocity of floodwaters can vary significantly throughout a coastal flood event. Floodwaters can approach a site from one direction as a storm approaches, then shift to another direction (or through several directions) as the storm moves through the area. Floodwaters can inundate some low-lying coastal sites from both the front (e.g., ocean) and the back (e.g., bay, sound, or river). In a similar manner, at any given site, flow velocities can vary from close to zero to very high. For these reasons, when determining flood loads for building design, velocities should be estimated conservatively and it should be assumed that floodwaters can approach from the most critical direction and that flow velocities can be high.

Despite the uncertainties, there are methods to approximate coastal flood velocities. One common method is based on the stillwater depth (flood depth without waves). Designers should consider the topography, the distance from the source of flooding, and the proximity to other buildings and obstructions before selecting the flood velocity for design. Those factors can direct and confine floodwaters, with a resulting acceleration of velocities. This increase in velocities is described as the "expected upper bound." The "expected lower bound" velocities are experienced in areas where those factors are not expected to influence the direction and velocity of floodwaters.

Upper bound velocities caused by Hurricane Katrina along the Mississippi coast, where storm surge depths neared 35 feet deep, have been estimated at nearly 30 feet per second (20 miles per hour).

Figure 2-7 shows the general relationship between velocity and stillwater depth. For design purposes, actual flood velocities are assumed to lie between the upper and lower bounds. Conservative designs will take into account the upper bound velocities.

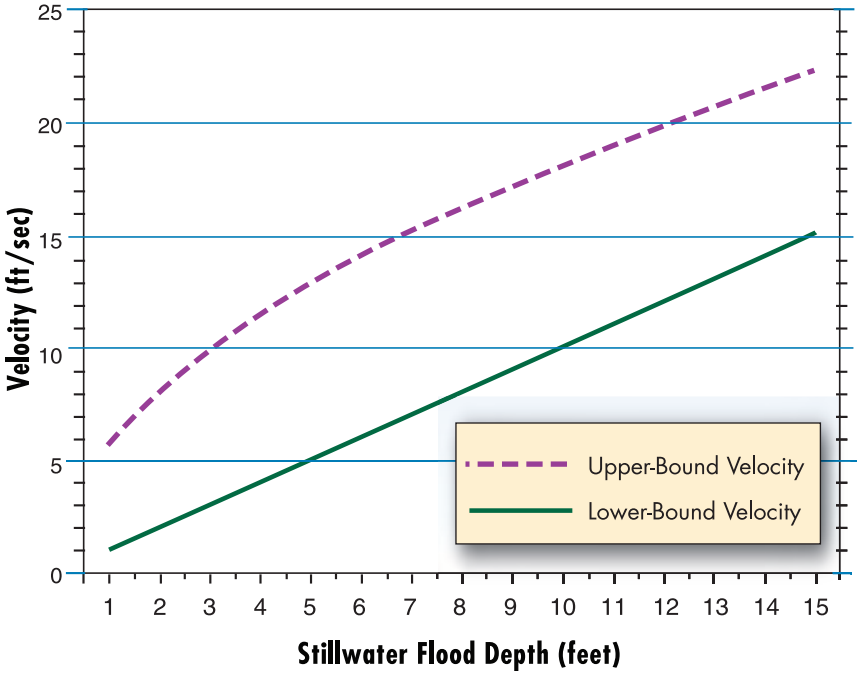


Figure 2-7:
Velocity as a function of stillwater flood depth

2.1.2.4 Hydrostatic Loads

Hydrostatic loads occur when water comes into contact with a building or building component, both above and below the ground level. They act as lateral pressure or vertical pressure (buoyancy). Hydrostatic loads on inclined or irregular surfaces may be resolved into lateral and vertical loads based on the surface geometry and the distribution of hydrostatic pressure.

Lateral hydrostatic loads are a direct function of water depth (see Figure 2-8). These loads can cause serious deflection or displacement of buildings or building components if there is a substantial difference in water levels on opposite sides of the component (or inside and outside of the building). Hydrostatic loads are balanced on foundation elements of elevated buildings, such as piers

and columns, because the element is surrounded by water. If not oriented parallel to the flow of water, shearwalls may experience hydrostatic loads due to a difference of water depth on either side of the wall. To reduce excessive pressure from standing water, floodplain management requirements in A Zones call for openings in walls that enclose areas below the flood elevation (see description of continuous perimeter wall foundation in Section 2.3.1.2).

Buoyant forces resulting from the displacement of water are also of concern, especially for dry floodproofed buildings and aboveground and underground tanks. Buoyancy force is resisted by the dead load of the building or the weight of the tank. When determining buoyancy force, the weight of occupants or other live loads (such as the contents of a tank) should not be considered. If the building or tank does not weigh enough "empty," then additional stabilizing measures need to be taken to avoid flotation. This becomes a significant consideration for designs intended to dry floodproof a building. Buoyancy force is slightly larger in saltwater, because saltwater weighs slightly more than fresh water.

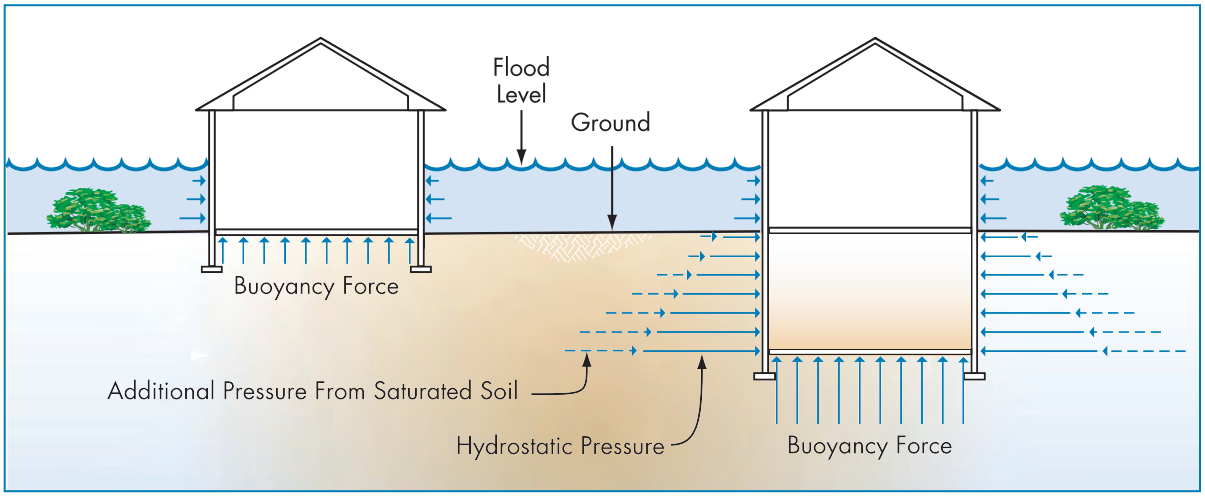


Figure 2-8: Hydrostatic loads on buildings

2.1.2.5 Hydrodynamic Loads

Water flowing around a building or a structural element that extends below the flood level imposes hydrodynamic loads. The loads, which are a function of flow velocity and structure geometry, include frontal impact on the upstream face, drag along the sides, and suction on the downstream side (see Figure 2-9). Ways to determine or estimate flood velocities are described in Section 2.1.2.2 and Section 2.1.2.3.

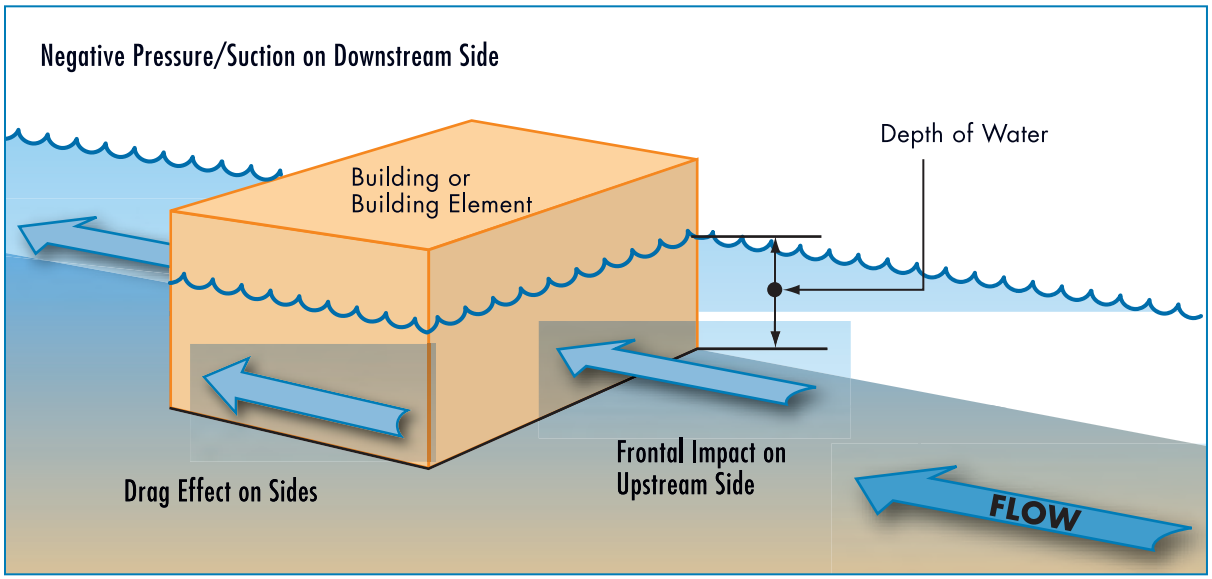


Figure 2-9: Hydrodynamic loads on a building or building element

The most common computation methods for hydrodynamic loads are outlined in the design standard *Minimum Design Loads for Buildings and Other Structures*, produced by the American Society of Civil Engineers' Structural Engineers Institute (ASCE/SEI, 2005). Those methods assume that the flood velocity is constant (i.e., steady state flow) and that the dynamic load imposed by floodwaters moving at less than 10 feet per second can be converted to an equivalent hydrostatic load. This conversion is accomplished by adding an equivalent surcharge depth to the depth of water on the upstream side. The equivalent surcharge depth is a function of the velocity. Loads imposed by floodwaters with ve-

locities greater than 10 feet per second cannot be converted to equivalent hydrostatic loads. Instead, they must be determined according to the principles of fluid mechanics or hydraulic models.

Hydrodynamic loads become important when flow reaches moderate velocities of 5 feet per second. The components of hydrodynamic loads are laterally imposed, caused by the impact of the mass of water against the building, and drag forces along the wetted surfaces. Drag coefficients for common building elements, such as columns and piers, can be found in a number of sources. ASCE 7 recommends values for a variety of conditions.

Another component of hydrodynamic loads is wave loads. As described in ASCE 7, "design and construction of buildings and other structures subject to wave loads shall account for the following loads: waves breaking on any portion of the building or structure; uplift forces caused by shoaling waves beneath a building or structure, or portion thereof; wave runup striking any portion of the building or structure; wave-induced drag and inertia forces; and wave-induced scour at the base of a building or structure, or its foundation."

Wave forces striking buildings and building elements can be 10 to 100 or more times higher than wind forces and other forces. Forces of this magnitude can be substantial, even when acting over the relatively small surface area of the supporting structure of elevated buildings. Post-storm damage inspections show that breaking wave loads overwhelm virtually all wood-frame and unreinforced masonry walls below the wave crest elevation. Only engineered and massive structural elements are capable of withstanding breaking wave loads. The magnitude of wave forces is the rationale behind the floodplain management requirement for the bottom of the lowest horizontal structural member to be at or above the design flood elevation in environments where waves are predicted to be 3 feet or higher (V Zones). Because waves as low as 1.5 feet can impose considerable loads, there is a growing awareness of the value of accounting for waves in areas that are referred to as "Coastal A Zones."

Computation of wave loads depends on the determination of wave height. Equations for wave height are based on the assumption that waves are depth-limited (on the order of 75 to 80 percent of

stillwater depth) and that waves propagating into shallow water break when the wave height reaches a certain proportion of the underlying stillwater depth. These assumptions are used by FEMA to determine coastal high hazard areas (V Zones) where breaking waves are predicted to be 3 feet or higher. At any given site, wave heights may be moderated by other factors. Designers should refer to ASCE 7 for detailed discussion and computation procedures.

Breaking wave loads on vertical walls or supporting structural members reach a maximum when the direction of wave approach is perpendicular to the wall. The duration of individual loads is brief, with peak pressures probably occurring within 0.1 to 0.3 seconds after the wave breaks. It is common to assume that the direction of approach will be perpendicular to the shoreline, in which case the orientation of the wall to the shoreline will influence the magnitude of the load placed on the wall. ASCE 7 provides a method for reducing breaking wave loads on vertical walls for waves that approach a building from a direction other than straight on. Structures should be designed for repetitive impact loads that occur during a storm. Some storms may last for just a few hours, as hurricanes move through the area, or for several days, as during some winter coastal storms (nor'easters) that affect the Mid-Atlantic and northeastern States.

2.1.2.6 Debris Impact Loads

Debris impact loads are imposed on a building or building elements by objects carried by moving water. Objects commonly carried by floodwaters include trees, dislodged tanks, and remnants of manmade structures such as docks and buildings (see Figure 2-10). Extreme impact loads result from less common sources, such as shipping containers, boats, and barges. The magnitude of these loads is very difficult to predict, yet some reasonable allowance should be made during the design process.

Impact loads are influenced by the location of the building in the potential debris stream. The potential for debris impacts is significant if a building is located immediately adjacent to, or downstream from, other buildings, among closely-spaced buildings, or downstream from large floatable objects. While these conditions may be observable in coastal areas, it is more diffi-

cult to estimate the potential for debris in riverine flood hazard areas. Any riverine waterway, whether a large river or smaller urban stream, can carry large quantities of debris, especially uprooted trees.

Figure 2-10:
The South Cameron Memorial Hospital, Cameron, LA, was damaged by debris carried by Hurricane Katrina's storm surge (2005).

SOURCE: LSU AGCENTER

The basic equation for estimating the magnitude of impact loads depends on several variables that must be selected by the designer. These variables include several coefficients, building or building element stiffness, debris weight, debris velocity, and duration of impact. The latter three variables, described in more detail in ASCE 7, are briefly described below.

Debris weight: Debris weight is one of the more difficult variables to estimate. Unless otherwise indicated by field conditions, ASCE 7 recommends using an average object weight of 1,000 pounds. This weight corresponds to a 30-foot long log only 1 foot in diameter, small in comparison to large trees that may be uprooted during a flood. In coastal areas, expected debris weights depend on the nature of the debris. In the Pacific Northwest, large trees and logs are common, with weights in excess of 4,000 pounds. In areas where piers and pilings are likely to become debris, 1,000 pounds is reasonable. In areas where most debris is likely to result from building damage (failed decks, steps, failed walls, propane tanks), the average debris weight may be less than 500 pounds.

Debris velocity: The velocity of the debris depends on the nature of the debris and the velocity of floodwaters. For the impact load computation, the velocity of the water-borne object is assumed to be the same as the flood velocity. Although this assumption is reasonable for smaller objects, it is conservative for large objects.

Debris impact duration: Duration of impact is the elapsed time during which the impact load acts on the building or building element. The duration of impact is influenced primarily by the natural frequency[2] of the building or element, which is a function of the building's stiffness. Stiffness is determined by the properties of the material, the number of supporting members (columns or piles), the height of the building above the ground, and the height at which the element is struck. Despite all the variables that may influence duration of impact, early assumptions suggested a 1-second duration. A review of results from several laboratory tests that measured impacts yielded much briefer periods, and ASCE 7 currently recommends a duration of 0.03 second.

2.1.2.7 Erosion and Localized Scour

Erosion generally refers to a lowering of the ground surface as a result of a flood event. Erosion may occur in riverine and coastal flood hazard areas. In coastal areas, erosion may affect the general ground surface and may cause a short-term or long-term recession of the shoreline. Erosion should be considered during load calculations, because it increases the local flood depth, which in turn influences load calculations. In areas subject to gradual erosion of the ground surface, additional foundation embedment depth can mitigate the effects. However, where waterways are prone to changing channels and where shoreline erosion is significant, engineered solutions are unlikely to be effective. Avoidance of sites in areas subject to active erosion is the safest and most cost-effective course of action.

Localized scour results from turbulence at the ground level around foundation elements. Scour occurs in both riverine and coastal flood hazard areas, especially in areas with erodible soils.

2. The frequency at which an object will vibrate freely when set in motion.

Determining potential scour is critical in the design of foundations to ensure that failure during and after flooding does not occur as a result of the loss in either bearing capacity or anchoring resistance around the posts, piles, piers, columns, footings, or walls (see Figure 2-11). Scour determinations require knowledge of the flood depth, flow conditions, soil characteristics, and foundation type.

At some locations, soil at or below the ground surface can be resistant to localized scour, and calculated scour depths based on unconsolidated surface soils below will be excessive. In instances where the designer believes the underlying soil at a site will be scour-resistant, the assistance of a geotechnical engineer or geologist should be sought.

Figure 2-11:
Local scour undermined this shallow foundation (also note that the building was not anchored to the foundation).

2.1.3 FLOODPLAIN MANAGEMENT REQUIREMENTS AND BUILDING CODES

The NFIP is the basis for the minimum requirements included in model building codes and standards for design and construction methods to resist flood damage. The original authorizing legislation for the NFIP is the National Flood Insurance Act of 1968 (42 U.S.C. 4001 et seq.). In that act, Congress expressly found that "a program of flood insurance can promote the public interest by encouraging sound land use by minimizing exposure of property to flood losses…"

The most convincing evidence of the effectiveness of the NFIP minimum requirements is found in flood insurance claim payment statistics. Buildings that pre-date the NFIP requirements are, by and large, not constructed to resist flood damage. Buildings that post-date the NFIP (i.e., those that were constructed after a community joined the program and began applying the minimum requirements) are designed to resist flood damage. The NFIP reports that aggregate loss data indicate that buildings that meet the minimum requirements experience 70 percent less damage than buildings that pre-date the NFIP. There is ample evidence that buildings designed to exceed the minimum requirements are even less likely to sustain damage.

2.1.3.1 Overview of the NFIP

The NFIP is based on the premise that the Federal government will make flood insurance available in communities that agree to recognize and incorporate flood hazards in land use and development decisions. In some States and communities this is achieved by guiding development to areas with a lower risk. When decisions result in development within flood hazard areas, application of the criteria set forth in Federal regulation 44 CFR §60.3 are intended to minimize exposure and flood-related damage. State and local governments are responsible for applying the provisions of the NFIP through the regulatory permitting processes. At the Federal level, the NFIP is managed by FEMA and has three main elements:

- Hazard identification and mapping, under which engineering studies are conducted and flood maps are prepared in partnership with States and communities. These maps delineate areas that are predicted to be subject to flooding under certain conditions.

- Floodplain management criteria for development, which establish the minimum requirements to be applied to development within mapped flood hazard areas. The intent is to recognize hazards in the entire land development process.

- Flood insurance, which provides some financial protection for property owners to cover flood-related damage to buildings and contents.

"Substantial damage" is damage of any origin sustained by a structure whereby the cost of restoring the structure to its condition before the damage would equal or exceed 50 percent of the market value of the structure before the damage occurred.

"Substantial improvement" is any repair, reconstruction, rehabilitation, addition, or improvement of a building, the cost of which exceeds 50 percent of the market value of the building before the improvement or repair is started (certain historic structures may be excluded).

Federal flood insurance is intended to shift some of the costs of flood disasters away from the taxpayer by providing property owners an alternative to disaster assistance and disaster loans. Disaster assistance provides limited funding for repair and cleanup, and is available only after the President signs a major disaster declaration for the area. NFIP flood insurance claims are paid any time damage from a qualifying flood event[3] occurs, regardless of whether a major disaster is declared. Community officials should be aware that public buildings may be subject to a mandated reduction in disaster assistance payments if the building is in a mapped flood hazard area and is not covered by flood insurance.

Another important objective of the NFIP is to break the cycle of flood damage. Many buildings have been flooded, repaired or rebuilt, and flooded again. Before the NFIP, in some parts of the country this cycle occurred every couple of years, with reconstruction taking place in the same flood-prone areas, using the same construction techniques that did not adequately resist flood damage. NFIP provisions guide development to lower risk areas by requiring compliance with performance measures to minimize exposure of new buildings and buildings that undergo major renovation or expansion (called "substantial improvement" or repair of "substantial damage"). This achieves the long-term objective of building disaster-resistant communities.

3. For the purpose of adjusting claims for flood damage, the NFIP defines a flood as "a general and temporary condition of partial or complete inundation of two or more acres of normally dry land area or of two or more properties (at least one of which is the policyholder's property) from: overflow of inland or tidal waters; unusual and rapid accumulation or runoff of surface waters from any source; mudflow; or collapse or subsidence of land along the shore of a lake or similar body of water as a result of erosion or undermining caused by waves or currents of water exceeding anticipated cyclical levels that result in a flood as defined above."

2.1.3.2 Summary of the NFIP Minimum Requirements

The performance requirements of the NFIP are set forth in Federal regulation 44 CFR Part 60. The requirements apply to all development, which the NFIP broadly defines to include buildings and structures, site work, roads and bridges, and other activities. Buildings must be designed and constructed to resist flood damage, which is primarily achieved through elevation (or floodproofing). Additional specific requirements apply to existing development, especially existing buildings. Existing buildings that are proposed for substantial improvement, including restoration following substantial damage, are subject to the regulations.

Although the NFIP regulations primarily focus on how to build structures, one of the long-term objectives of the program is to guide development to less hazardous locations. Preparing flood hazard maps and making the information available to the public is fundamental in satisfying that objective. With that information, people can make informed decisions about where to build, how to use site design to minimize exposure to flooding, and how to design buildings that will resist flood damage.

The NFIP's broad performance requirements for site work in flood hazard areas are as follows:

- Building sites shall be reasonably safe from flooding.
- Adequate site drainage shall be provided to reduce exposure to flooding.
- New and replacement sanitary sewage systems shall be designed to minimize or eliminate infiltration of floodwaters into the systems and discharges from the systems into floodwaters.
- Development in floodways shall be prohibited, unless engineering analyses show that there will be no increases in flood levels.

The NFIP's broad performance requirements for new buildings proposed for flood hazard areas (and substantial improvement of existing flood-prone buildings) are as follows:

- Buildings shall be designed and adequately anchored to prevent flotation, collapse, or lateral movement resulting from hydrodynamic and hydrostatic loads, including the effects of buoyancy.
- Building materials used below the design flood elevation shall be resistant to flood damage.
- Buildings shall be constructed to minimize flood damage (primarily by elevating to or above the base flood level, or by specially designed and certified floodproofing measures).
- Buildings shall be constructed with electrical, heating, ventilation, plumbing, and air conditioning equipment and other service facilities designed to prevent water from entering or accumulating within the components.

Owners, planners and designers should determine if there are any applicable State-specific requirements for floodplain development. Some States require that local jurisdictions apply standards that exceed the minimum requirements of the NFIP. In particular, some States require that critical facilities be located outside of the floodplain (including the 500-year floodplain) or they are to be designed and constructed to resist conditions associated with the 500-year flood. Some States have regulations that impose other higher standards, while some States have direct permitting authority over certain types of construction or certain types of applicants.

States often use governors' executive orders to influence State-constructed and State-funded critical facilities, requiring location outside of the 500-year floodplain where feasible, or protection to the 500-year flood level if avoiding the floodplain is not practical. In 2004, a review of State and local floodplain management programs determined that Alabama, Illinois, Michigan, New York, North Carolina, Ohio, and Virginia have requirements for critical facilities (ASFPM, 2004).

As participants in the NFIP, States are required to ensure that development that is not subject to local regulations, such as State construction, satisfies the same performance requirements. If critical facilities are exempt from local permits, this may be ac-

complished through a State permit, a governor's executive order, or other mechanisms that apply to entities not subject to local authorities.

2.1.3.3 Executive Order 11988 and Critical Facilities

When Federal funding is provided for the planning, design, and construction of new critical facilities, or for the repair of existing critical facilities located within the 500-year floodplain, the funding agency is required to address additional considerations. Executive Order 11988, Floodplain Management, requires Federal agencies to apply a decisionmaking process to avoid, to the extent possible, the long- and short-term adverse impacts associated with the occupancy and modification of floodplains, and to avoid the direct or indirect support of floodplain development whenever there is a practicable alternative. If there is no practicable alternative, the Federal agency must minimize any adverse impacts to life, property, and the natural and beneficial functions of floodplains.

The executive order establishes the base flood elevation as the minimum standard for all Federal agencies. Implementation guidance specifically addresses "critical actions," which are described as those actions for which even a slight chance of flooding would be too great. The construction or repair of critical facilities, such as fire stations, hospitals and clinics, EOCs, the storage of hazardous wastes, and the storage of critical records, are examples of critical actions.

After determining that a site is in a mapped flood hazard area, and after giving public notice, the Federal funding agency is required to identify and evaluate practicable alternatives to locating a critical facility in a 500-year floodplain. If the Federal agency has determined that the only practicable alternative is to proceed, then the impacts of the proposed action must be identified. If the identified impacts are harmful to people, property, and the natural and beneficial functions of the floodplain, the Federal

FEMA's eight-step decisionmaking process for complying with Executive Order 11988 must be applied before Federal disaster assistance is used to repair, rehabilitate, or reconstruct damaged existing critical facilities in the 500-year floodplain.

agency is required to minimize the adverse effects on the floodplain and the funded activity.

Having identified the impacts of the proposed action and the methods to minimize these impacts, the Federal agency is required to re-evaluate the proposed action. The re-evaluation must consider whether the action is still feasible, whether the action can be modified to relocate the facility or eliminate or reduce identified impacts, or if a "no action" alternative should be chosen. If the finding results in a determination that there is no practicable alternative to locating a critical facility in the floodplain, or otherwise affecting the floodplain, then a statement of findings and a public explanation must be provided.

2.1.3.4 Scope of Model Building Codes and Standards

The *International Building Code* (IBC, 2003) and the *Building Construction and Safety Code*™ (NFPA 5000, 2003) were the first model codes to include comprehensive provisions that address flood hazards. Both codes are consistent with the minimum provisions of the NFIP that pertain to the design and construction of buildings. The NFIP requirements that pertain to site development, floodways, coastal setback lines, erosion-prone areas, and other environmental constraints are found in other local ordinances. The codes require designers to identify and design for anticipated environmental loads and load combinations, including wind, seismic, snow, and flood loads, as well as the soil conditions.

The IBC and NFPA 5000 incorporate by reference a number of standards that are developed through a rigorous consensus process. The best known is *Minimum Design Loads for Buildings and Other Structures* (ASCE 7-05). The model building codes require that applicable loads be accounted for in the design. The 1998 edition of ASCE 7 was the first version of the standard to include flood loads explicitly, including hydrostatic loads, hydrodynamic loads (velocity and waves), and debris impact loads.

The IBC and NFPA 5000 also incorporate by reference a standard that was first published by ASCE in 1998 and revised in 2005, *Flood Resistant Design and Construction* (ASCE/SEI 24-05).

Developed through a consensus process, ASCE 24 addresses specific topics pertinent to designing buildings in flood hazard areas, including floodways, coastal high hazard areas, and other high-risk flood hazard areas such as alluvial fans, flash flood areas, mudslide areas, erosion-prone areas, and high floodwater velocity areas.

ASCE 7-05 outlines methods to determine design loads and load combinations in flood hazard areas, including hydrostatic loads, hydrodynamic loads, wave loads, and debris impact loads.

ASCE 24-05 addresses design requirements for structures in coastal high-hazard areas (V Zones).

Section 1.2 describes the four categories used by ASCE 7 to classify structures based on occupancy; different requirements apply based on a structure's category. The same categories are used in ASCE 24 and different flood-resistant requirements apply to the different categories. Table 2-1 summarizes the elevation requirements of ASCE 24 that exceed the NFIP minimum requirements for the critical facilities addressed by this manual (Category III or Category IV structures).

Table 2-1: ASCE/SEI 24-05 provisions related to the elevation of critical facilities

		Category III	Category IV
Elevation of Lowest Floor or Bottom of Lowest Horizontal Structural	A Zone: elevation of lowest floor	BFE +1 ft or DFE, whichever is higher	BFE +2 ft or DFE, whichever is higher
	V Zone and Coastal A Zone: where the lowest horizontal structural member is parallel to direction of wave approach	BFE +1 ft or DFE, whichever is higher	BFE +1 ft or DFE, whichever is higher
	V Zone and Coastal A Zone: where the lowest horizontal structural member is perpendicular to direction of wave approach	BFE +2 ft or DFE, whichever is higher	BFE +2 ft or DFE, whichever is higher
Elevation Below which Flood-Damage-Resistant Materials Shall	A Zone	BFE +1 ft or DFE, whichever is higher	BFE +2 ft or DFE, whichever is higher
	V Zone and Coastal A Zone: where the lowest horizontal structural member is parallel to direction of wave approach	BFE +2 ft or DFE, whichever is higher	BFE +2 ft or DFE, whichever is higher
	V Zone and Coastal A Zone: where the lowest horizontal structural member is perpendicular to direction of wave approach	BFE +3 ft or DFE, whichever is higher	BFE +3 ft or DFE, whichever is higher
Minimum Elevation of Utilities and Equipment	A Zone	BFE +1 ft or DFE, whichever is higher	BFE +2 ft or DFE, whichever is higher
	V Zone and Coastal A Zone: where the lowest horizontal structural member is parallel to direction of wave approach	BFE +2 ft or DFE, whichever is higher	BFE +2 ft or DFE, whichever is higher
	V Zone and Coastal A Zone: where the lowest horizontal structural member is perpendicular to direction of wave approach	BFE +3 ft or DFE, whichever is higher	BFE +3 ft or DFE, whichever is higher
Dry Floodproofing	A Zone: elevation to which dry floodproofing extends	BFE +1 ft or DFE, whichever is higher	BFE +2 ft or DFE, whichever is higher
	V Zone and Coastal A Zone: dry floodproofing not allowed	Not applicable	Not applicable

2.2 CRITICAL FACILITIES EXPOSED TO FLOODING

2.2.1 EVALUATING RISK AND AVOIDING FLOOD HAZARDS

Flood hazards are very site-specific. When a flood hazard map is prepared, lines drawn on the map appear to precisely define the hazard area. Land that is on one side of the line is "in" the mapped flood hazard area, while the other side of the line is "out." Although the delineation may be an approximation, having hazard areas shown on a map facilitates avoiding such areas to the maximum extent practical. Where it is unavoidable, facility owners should carefully evaluate all of the benefits and all of the costs in order to determine long-term acceptable risks, and to develop appropriate plans for design and construction of new facilities.

Even in communities with expansive floodplains, it should be possible to avoid locating new critical facilities in floodways and coastal areas subject to significant waves (V Zones).

Section 2.2.2 describes the damage sustained by existing buildings exposed to flood hazards, including site damage, structural and nonstructural building damage, destruction or impairment of service equipment, and loss of contents. These types of damage, along with loss of function and community service, are avoided if critical facilities are located away from flood hazard areas. Damage is reduced when critical facilities that must be located in flood hazard areas are built to exceed the minimum requirements.

2.2.1.1 Benefits/Costs: Determining Acceptable Risk

Extreme hurricane storm surge flooding may be a very low-probability event, but the flood water depths and waves may be much more severe than the conditions of the base flood shown on the FIRMs. The potential impacts on a critical facility must be carefully considered in order to make an informed decision regarding acceptable risk and potential damage. If possible, it is always best to avoid locating critical facilities in areas subject to extreme storm surge flooding.

Many decisions that are made with respect to critical facilities are, in part, based on a determination of acceptable risk. Risk includes the potential losses associated with a hazard. Ideally, risk is defined in terms of expected probability and frequency of the hazard occurring, the people and property exposed, and the potential consequences. Choosing a site that is affected by flooding is a decision to accept some degree of risk. Although the flood-prone land may have a lower initial cost, the incremental costs of construction, plus the likely increased costs of maintenance, repair, and replacement, may be significant. Another cost of locating a critical facility in a flood-prone area is related to access problems if roads and driveways are impassable. Although the building may be elevated and protected, if access is restricted periodically, then the use of the facility is affected.

The building owner and the design team can influence the degree of risk (e.g., the frequency with which flooding may affect the site). They control it through the selection of the site design and the building design measures. Fundamentally, this process is a balancing of the benefits of an acceptable level of disaster resistance with the costs of achieving that degree of protection. With respect to mitigation of future hazard events:

- Benefits are characterized and measured as future damages avoided if the mitigation measures (including avoiding flood hazard areas) are implemented.

- Costs are the costs associated with implementing measures to eliminate or reduce exposure to hazards.

Section 2.2.2 describes damage and losses that are incurred by buildings exposed to flooding. Direct damage includes damage to physical property, including the site, the building, building materials, utilities, and building contents. Indirect damage that is not

listed includes health hazards, loss of functionality, emergency response, evacuation, and expenses associated with occupying another building during repairs.

Benefits other than avoided physical damage are difficult to measure. They are associated with future damage that does not occur because of the mitigation activity, cleanup that is not required because of the mitigation activity, and service that is not interrupted because flooding does not affect normal operation of the facility. In addition, benefits accrue over long periods of time, thus making it more difficult to make a direct comparison of the benefits with the up-front costs of mitigation. Mitigation costs can be more readily expressed in terms of the higher costs of a flood-free site, or the initial capital costs of work designed to resist flood damage. Thus, without a full accounting of both benefits and costs, decisionmakers may not be able to make fully informed decisions. Some questions that should be answered include:

Sometimes developers are required to set aside land to meet adequate facilities requirements, or land may be donated to support community or non-profit facilities, such as fire stations. If the donated land is affected by flood hazards, it may be difficult to avoid floodplain impacts entirely. Careful consideration should be made whether the benefits of accepting the land outweigh the costs and risks associated with mitigating the flood risk.

- If the site is flood-prone and the building is out of the flood hazard area or is elevated on fill, what are the average annual cleanup costs associated with removal of sand, mud, and debris deposited by floods of varying frequencies?

- If the facility building is elevated by means other than fill, will periodic inundation of the exposed foundation elements cause higher average annual maintenance costs?

- If the facility is protected with floodproofing measures, what are the costs of annual inspections, periodic maintenance and replacement of materials, and staff training and periodic drills?

- If the critical facility meets only the minimum elevation requirements, what are the average annual damage and cleanup costs over the anticipated useful life of the building, including the occurrence of floods that exceed the design flood elevation?

- How do long-term costs associated with periodic inundation compare to up-front costs of selecting a different site or building to a higher level of protection?

- If the facility is located in a hurricane-prone community, how should the facility design account for low probability, but high impact, storm surge flooding?

- If access to the facility is periodically restricted due to flooding, especially long-duration flooding, what are the cost effects? How often should an alternate location be provided to continue normal operations?

2.2.1.2 Identifying Flood Hazards at Critical Facilities Sites

As part of site selection and to guide locating a new critical facility and other improvements on a site, facility owners, planners, and designers should investigate site-specific flood hazard characteristics. Similarly, when examining existing critical facilities and when planning improvements or rehabilitation work, an important step is to determine the site characteristics and flood hazards. The best available information should be examined, including flood hazard maps, records of historical flooding, storm surge maps, and advice from local experts and others who can evaluate flood risks. Table 2-3 in Section 2.5 outlines questions that should be answered prior to initiating site layout and design work.

2.2.1.3 Critical Facilities as Emergency Shelters

Emergency managers regularly identify facilities (especially schools) to serve as short-term and long-term shelters. Schools are attractive sites for shelters because they have kitchen facilities designed to serve many people, restroom facilities likely to be adequate for many people, and plenty of space for cots in gymnasiums, cafeterias, and wide corridors.

New schools that will function as emergency shelters warrant a higher degree of protection than other schools and should be appropriately designed as critical facilities (see Section 1.3 for an

overview on performance-based design). If located in, or adjacent to, flood hazard areas, it is appropriate to provide protection for the building and utility systems to at least the 0.2-percent-annual-chance (500-year) flood level or, at a minimum, 2 to 3 feet above the DFE. Additional guidance on hazard-resistant shelters is found in FEMA 361, *Design and Construction Guidance for Community Shelters.*

Additional measures that may be appropriate for consideration when flood-prone critical facilities are used as shelters include the following:

- Wastewater service must be functional during conditions of the flooding.
- Emergency power service must be provided.
- Dry ground access is important, in the event flooding exceeds design levels.

2.2.2 VULNERABILITY: WHAT FLOODING CAN DO TO EXISTING CRITICAL FACILITIES

Existing flood-prone facilities are susceptible to damage, the nature and severity of which is a function of site-specific flood characteristics. As described below, damage may include: site damage, structural and nonstructural building damage, destruction or impairment of utility service equipment and loss of contents.

Regardless of the nature and severity of damage, flooded facilities typically are not functional while cleanup and repairs are undertaken. The length of closure, and thus the impact on the ability of the facility to become operational, depends on the severity of the damage and lingering health hazards. Sometimes repairs are put on hold pending a decision on whether a facility should be rebuilt at the flood-prone site. When damage is substantial, rehabilitation or reconstruction is allowed only if compliance with flood-resistant design requirements is achieved (see Section 2.1.3.2).

2.2.2.1 Site Damage

The degree of site damage associated with flooding is a function of several variables related to the characteristics of the flood, as well as the site itself.

Erosion and scour: All parts of a site subject to flooding by fast moving water could experience erosion, and local scour could occur around any permanent obstructions to flow. Graded areas, filled areas, and cut or fill slopes are especially susceptible. Stream and channel bank erosion, and erosion of coastal shorelines, are natural phenomena that may, over time, threaten site improvements and buildings.

Debris and sediment removal: Even when buildings are not subject to water damage, floods can produce large quantities of debris and sediment that can damage a site and be expensive to remove.

Landscaping: Grass, trees, and plants suffer after floods, especially long-duration flooding that prevents oxygen uptake, and coastal flooding that stresses plants that are not salt-tolerant. Fast-moving floodwaters and waves also can uproot plants and trees.

Fences: Some types of fences that are relatively solid can significantly restrict the free flow of floodwaters and trap floating debris. Fences can be damaged by flowing water, and can be knocked down under pressure of flowing water or if the buildup of debris results in significant loads (see Figure 2-12).

Accessory structures: Accessory structures can sustain both structural and nonstructural damage. In some locations, such structures can be designed and built using techniques that minimize damage potential, without requiring elevation above the DFE.

Access roads: Access roads that extend across flood-prone areas may be damaged by erosion, washout of drainage culverts, failure of fill and bedding materials, and loss of surface (see Figure 2-13). Road damage could prevent uninterrupted access to a facility and thus impair its functionality.

Figure 2-12:
Katrina's storm surge flooding knocked down this fence adjacent to a fire station (2005).

Figure 2-13:
Flooding caused the failure of this road bed.
SOURCE: U.S. ARMY CORPS OF ENGINEERS

Parking lots and parking garages: Paved parking lots may be damaged by failure of bedding materials and loss of driving surface. Vehicles left in parking lots and parking garages could also be damaged. Most large parking garages are engineered structures that can be designed to allow for the flow of water.

Stormwater management facilities and site drainage: Site improvements such as swales and stormwater basins may be eroded, filled with sediments, or clogged by debris.

2.2.2.2 Structural Damage

Structural damage includes all damage to the load-bearing portions of a building. Structural damage can be caused by each of the characteristics of flooding described in Section 2.1.1.

Damage to other components of buildings is described below, including saturation of materials (Section 2.2.2.3), utility service equipment (Section 2.2.2.4), and contents (Section 2.2.2.5).

Depth: The hydrostatic load or pressure against a wall or foundation is directly related to the depth of water (refer to Figure 2-9). Standard stud and siding, or unreinforced brick veneer walls, may collapse under hydrostatic loads associated with relatively shallow water. Reinforced masonry walls perform better than unreinforced masonry walls (see Figure 2-14), although an engineering analysis is required to determine performance. Walls and floors of below-grade areas (basements) are particularly susceptible to damage by hydrostatic pressure. When soils are saturated, pressures against below-grade walls are a function of the total depth of water, including the depth below-grade and the weight of the saturated soils.

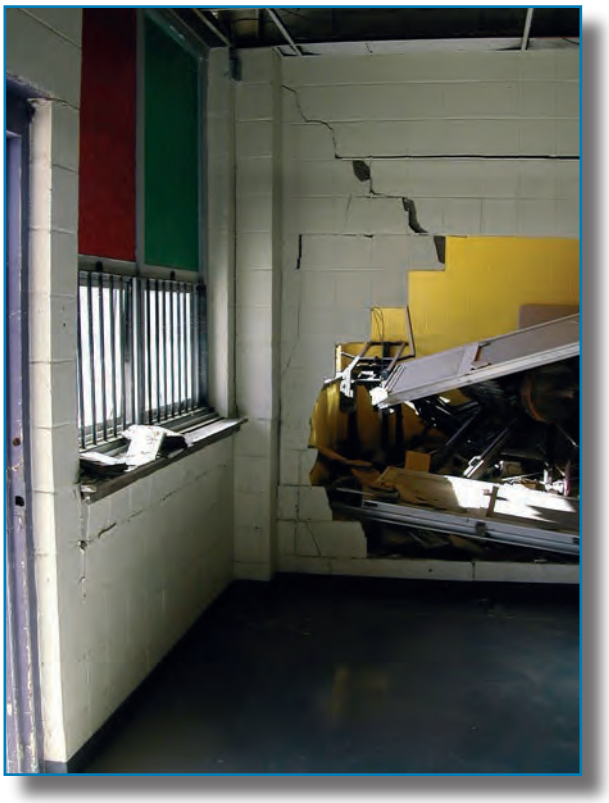


Figure 2-14:
Interior unreinforced masonry walls of the Port Sulphur High School in Louisiana were damaged by hydrostatic loads associated with Hurricane Katrina's storm surge (2005).

Buoyancy and uplift: If below-grade areas are essentially watertight, buoyancy or uplift forces can float a building out of the ground or rupture concrete floors (see Figure 2-15). Buildings that are not adequately anchored can be floated or pushed off foundations. Although rare for large and heavy critical facility buildings, this is a concern for outbuildings and portable (temporary) units.

Figure 2-15: Concrete floor ruptured by hydrostatic pressure (buoyancy). Hurricane Katrina (2005)

Duration: Long duration saturation can cause dimensional changes and contribute to deterioration of wood members. By itself, saturation is unlikely to result in significant structural damage to masonry construction. Saturation of soils, a consequence of long duration flooding, increases pressure on below-grade foundation walls.

Velocity, wave action, and debris impacts: Each of these components of dynamic loads can result in structural damage if buildings are not designed to resist overturning, repetitive pounding by waves, or short-duration impact loads generated by floating debris.

Erosion and scour: Structural damage is associated with foundation failure when erosion or scour results in partial or complete removal of supporting soil (see Figure 2-16). Erosion of slopes, especially unprotected slopes, can lead to slope failures and loss of foundation supporting soil.

Figure 2-16:
Scour around the foundation of this building contributed to significant damage.

2.2.2.3 Nonstructural Damage

Many flood-prone buildings are exposed to floodwaters that are not fast moving, or that may be relatively shallow and not result in structural damage. Simple inundation and saturation of the building and finish materials can result in significant and costly nonstructural damage, including long-term health complications associated with mold. Floodwaters often are contaminated with chemicals, petroleum products, or sewage. Under such circumstances, recovery generally involves removal of nonstructural materials and finishes because cleanup and decontamination is expensive and time-consuming. Damage to contents is discussed in Section 2.2.2.5.

Nonstructural damage can vary as a function of the duration of water exposure. Some materials are not recoverable even after very brief inundation, while others remain serviceable if in contact with water for only a few hours. Use of water-resistant materials will help to minimize nonstructural damage caused by saturation and reduce the costs of cleanup and restoration to service (see *Flood-Resistant Materials Requirements*, FIA-TB-2).

Wall finishes: Painted concrete and concrete masonry walls usually resist water damage, provided the type of paint can be readily cleaned, such as high strength epoxy paints. Tiled walls may be

acceptable, depending on the type of adhesive and foundation (gypsum board substrate and wood-framed walls with tile typically do not remain stable).

Flooring: Many critical facilities have durable floors that resist water damage. Ground floors often are slab-on-grade and finished with tile or sheet goods. Flooring adhesives in use since the early 1990s likely are latex-based and tend to break down when saturated. Most carpeting, even the indoor-outdoor kind, is difficult to clean. Wood floors are particularly susceptible to saturation damage, although short duration inundation may not cause permanent de-formation of some wood floors. However, because of low tolerance for surface variations, gymnasium floors in schools are particu-larly sensitive and tend to warp after flooding of any duration (see Figure 2-17).

Figure 2-17:
This parquet wood gymnasium floor was damaged by dimensional changes due to saturation. Hurricane Katrina (2005)

Wall and wood components: When soaked for long periods of time, some building components change composition or shape. Most types of wood will swell when wetted and, if dried too quickly, will crack, split or warp. Plywood can delaminate and wood door and

window frames may swell and become unstable. Gypsum wallboard, wood composition panels, other wall materials, and wood cabinetry not intended for wet locations can fall apart (see Figure 2-18). The longer these materials are wet, the more moisture, sediment, and pollutants they absorb. Some wall materials, such as the paper facing on gypsum wallboard, "wick" standing water, resulting in damage above the actual high-water line (see Figure 2-19).

Figure 2-18:
Damaged walls and cabinets

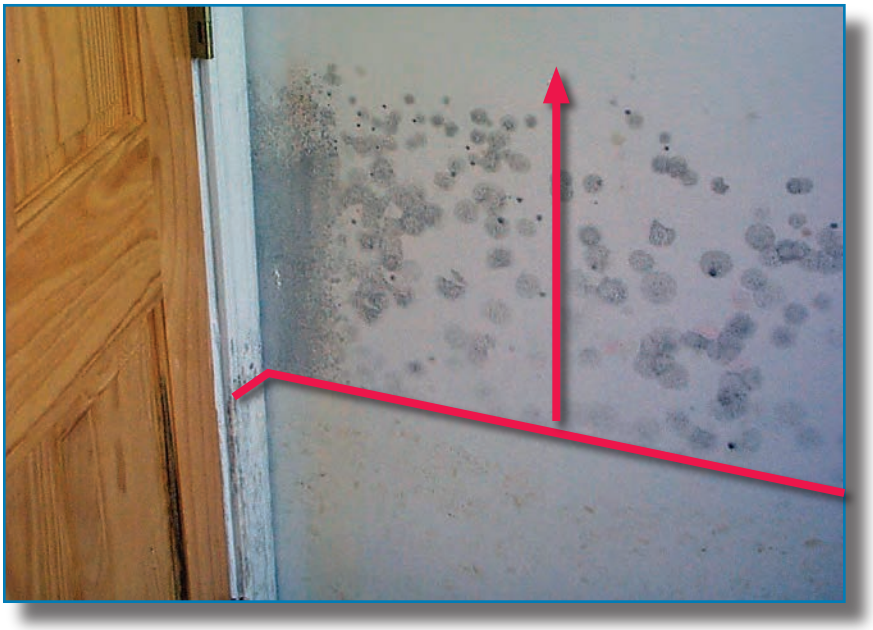


Figure 2-19:
Water damage and mold growth extend above the water line

SOURCE: OAK RIDGE NATIONAL LABORATORY

Metal components: Metal structural components are unlikely to be permanently damaged by short-term inundation. However, hollow metal partitions are particularly susceptible when they come into contact with water because they cannot be thoroughly dried and cleaned. Depending on the degree of corrosion protection on the metal, repetitive flooding by saline coastal waters may contribute to long-term corrosion.

Metal connectors and fasteners: Depending on the composition of the metal, repetitive flooding, especially by saline coastal waters, may contribute to long-term corrosion. Connectors and fasteners are integral to the structural stability of buildings; therefore, failure caused by accelerated corrosion would jeopardize the building.

2.2.2.4 Utility System Damage

Utility system service equipment that is exposed to flooding is vulnerable to damage. Damage may result in a total loss, or may require substantial cleaning and restoration efforts. The degree of damage varies somewhat as a function of the characteristics of flooding. Certain types of equipment and installation measures will help minimize damage and reduce the costs of cleanup and restoration to service.

Displacement of equipment and appliances: Installation below the flood level exposes equipment and appliances to various flood forces, including drag resulting from flowing water and buoyancy. Gas-fired appliances are particularly dangerous: flotation can separate appliances from gas sources, resulting in fires and explosive situations. Displaced equipment may dislodge lines from fuel oil tanks, contributing to the threat of fires and causing water pollution and environmental damage.

Elevators: If located in areas subject to flooding, elevator component equipment and controls will be damaged, and communication between floors will be impaired.

Corrosion: Corrosion related to inundation of equipment and appliances may not be apparent immediately, but can increase maintenance demand and shorten the useful life of some equipment and appliances.

Electrical systems and components: Electrical systems and components, and electrical controls of heating, ventilation, and air conditioning systems, are subject to damage simply by getting wet, even for short durations. Unless specifically designed for wet locations, switches and other electrical components can short out due to deposits of sediment, or otherwise not function even when allowed to dry before operation. Wiring and components that have been submerged may be functional, although generally it is more cost-effective to discard flooded outlets, switches, and other less expensive components than to attempt thorough cleaning.

Communications infrastructure: Critical communications infrastructure, such as control panels and wiring for warning systems, 911 systems, and regular telephone and wireless networks, are most susceptible to failure during emergencies if located in below-grade basements.

Specialized piping: Unprotected piping for medical gas supply systems may be damaged and threaten care that depends on uninterrupted supply of oxygen and other gasses for the treatment of patients.

Ductwork damage: Ductwork is subject to two flood-related problems. Flood forces can displace ductwork, and saturated insulation can overload support straps, causing failure.

Mold and dust: Furnaces, air handlers, and ductwork that have been submerged must be thoroughly cleaned and sanitized. Otherwise, damp conditions contribute to the growth of mold and accumulated sediment can be circulated throughout the critical facility, causing respiratory problems. Fiberglass batt or cellulose insulation that has been submerged cannot be sanitized and must be replaced. In sensitive environments, ductwork should be replaced rather than cleaned.

Gas-fired systems: Water-borne sediment can impair safe functioning of jets and controls in gas-fired furnaces and water heaters, necessitating professional cleaning and inspection prior to restoration of service. Control equipment (valves, electrical switches, relays, temperature sensors, circuit breakers, and fuses) that have been submerged may pose an explosion and fire hazard and should be replaced.

Emergency power generators: Generators that are installed at-grade are susceptible to inundation and will be out of service after a flood (see Figure 2-20).

Tanks (underground): Underground storage tanks are subjected to significant buoyant forces and can be displaced, especially when long-duration flooding occurs. Computations of stability should be based on the assumption that the tank is empty in order to maximize safety. Tank inlets, fill openings, and vents should be above the DFE, or designed to prevent the inflow of floodwaters or outflow of tank contents during flood conditions.

Tanks (aboveground): Aboveground storage tanks are subject to buoyant forces and displacement caused by moving water. Standard strapping of propane tanks may be inadequate for the anticipated loads. Tank inlets, fill openings, and vents should be above the DFE, or designed to prevent the inflow of floodwaters or outflow of tank contents during flood conditions.

Public Utility Service: Damage to public utility service (potable water supply and wastewater collection) can affect operations and may cause damage to critical facilities:

Figure 2-20:
Although it was anchored and not displaced by floodwaters, this generator was out of service after being submerged.

- Potable water supply systems may become contaminated if public water distribution lines or treatment facilities are damaged, or if wellheads are submerged.

- During heavy rains, sewers back up from infiltration and inflow of stormwater into the sewer lines and manholes, cross connections between storm and sanitary sewers, and flooded wastewater treatment plants. Sewer backup into a critical facility poses a major health hazard. Even when the water has receded, exposed building components, finish materials, and contents are contaminated, and usually must be removed because adequate cleaning is difficult, if not impossible.

2.2.2.5 Contents Damage

Critical facilities may contain high-value contents that can be damaged and become unrecoverable when subjected to flooding. For the purpose of this discussion, the term "contents" includes items such as furniture, appliances, computers, laboratory equipment and materials, records, and specialized machinery. The following types of contents are often considered a total loss.

Furniture: In long-duration flooding, porous woods become saturated and swollen, and joints may separate. Furniture with coverings or pads generally cannot be restored. Metal furniture is difficult to thoroughly dry and clean, is subject to corrosion, and typically is discarded. Some wood furniture may be recoverable after brief inundation.

Computers: Flood-damaged computers and peripheral equipment cannot be restored after inundation, although special recovery procedures may be able to recover information on hard drives.

Communications equipment: Even though some communications equipment may be able to be restored with appropriate cleaning, the loss of functionality would seriously impair the ability of the facility to provide critical services immediately after a flood. Equipment with printed circuit boards generally cannot be restored.

Office records and police files: When facilities are located in flood-prone space, valuable records may be lost. Although expensive, some recovery of computerized and paper records may be possible with special procedures (see Figure 2-21).

Health care equipment and laboratory materials: Most medical and health care equipment cannot be cleaned and restored to safe functioning, and would need to be replaced. Depending on the nature of laboratory materials and chemicals, complete disposal or special cleanup procedures may be required.

Kitchen goods and equipment: Floodwaters can dislodge appliances that can float and damage other equipment (see Figure 2-22). Stainless steel equipment generally has cleanable surfaces that can be disinfected and restored to service. Because of contamination, all food stuffs must be discarded.

Vehicles associated with critical facilities: If left in flood-prone areas, fire engines, police cars, ambulances, and other vehicles require replacement or cleaning to be serviceable and may not be functional and available for service immediately after a flood.

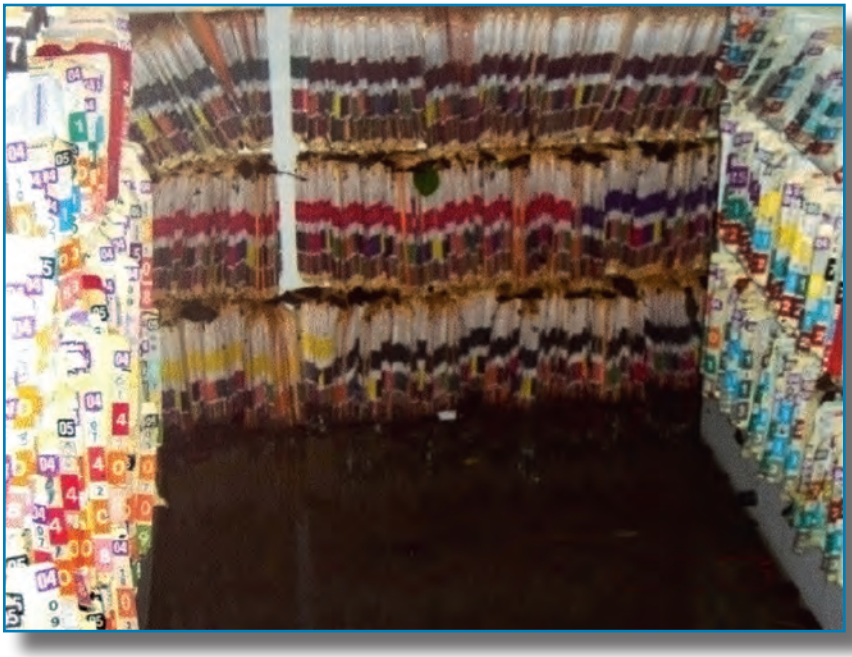


Figure 2-21:
Medical records saturated by floodwaters

SOURCE: HANCOCK MEDICAL CENTER

Figure 2-22:
Kitchen appliances and equipment from Port Sulphur High School were displaced and damaged by Hurricane Katrina floodwaters (2005).

2.3 REQUIREMENTS AND BEST PRACTICES IN SPECIAL HAZARD AREAS

2.3.1 RISK REDUCTION IN "A ZONES"

Flood hazard areas designated as A Zones on FIRMs are areas where significant wave action is not expected (see Section 2.1.1.2). A Zones are found along riverine bodies of water (rivers, streams, creeks, etc.), landward of V Zones, and on some open coastlines that do not have mapped V Zones. When constructing a critical facility on a site affected by an A Zone flood hazard, site design is influenced by several constraints, such as the presence of flood hazard areas, wetlands, poor soils, steep slopes, sensitive habitats, mature tree stands, and the environmental requirements set by the various regulatory authorities and the agency that approves development plans.

Four aspects of the design of flood-resistant buildings and sites are described in this section: site modifications, elevation considerations, flood-resistant materials, and floodproofing considerations. Section 2.3.5 addresses related facilities, including access roads, utility installations, water and wastewater systems, storage tanks, and accessory structures.

2.3.1.1 Site Modifications

When sites being considered for critical facilities are affected by flood hazards, planners and designers may want to evaluate the feasibility of certain site modifications in order to provide an in-

Site modifications are not appropriate in floodways along riverine waterways, where obstructions to flows can increase flood elevations. Engineering analyses are required to determine the impact of such modifications.

creased level of protection to buildings. The evaluations involve engineering analyses to determine whether the desired level of protection is cost-effective, and whether the proposed site modifications alter the floodplain in ways that could increase flooding. The effectiveness of typical site modifications and their ramifications must be examined for each specific site.

Earthen fill: Fill can be placed in the flood hazard area for the purpose of elevating a site above the design flood elevation. If the fill is placed and compacted so as to be stable during the rise and fall of floodwaters, and if the fill is protected from erosion, then modifying a site with fill to elevate a facility is preferred over other methods of elevation. Not only will buildings be less exposed to flood forces, but, under some circumstances (such as long duration floods), critical facilities may be able to continue to function. Whether nonstructural fill is placed solely to modify the site, or structural fill is placed for the purpose of elevating buildings, placement of fill can change flooding characteristics, including increased flooding on other properties. Engineering analyses can be conducted to determine whether eliminating floodplain storage by filling will change the direction of the flow of water, create higher flow velocities, or increase the water surface elevation in other parts of the floodplain. Fill is a less effective elevation method in flood hazard areas exposed to wave action, such as the banks of wide rivers, back bays, or Coastal A Zones, because wave action may erode the fill and adequate armoring or other protection methods can be expensive.

Excavation: Excavation alone rarely results in significantly altering the floodplain on a given parcel of land. Excavation that modifies a site is more commonly used in conjunction with fill in order to offset or compensate for the adverse impacts of fill.

Earthen levee: A levee is a specially designed barrier that modifies the floodplain by keeping the water away from certain areas (see Figure 2-23). Levees are significant structures that require detailed, site-specific geotechnical investigations; engineering analyses to identify whether flooding will be made worse on other properties; structural and site design to suit existing constraints; design of in-

terior drainage (on the land side); and long-term commitment for maintenance, inspection, and repairs. It is important to remember that areas behind levees are protected only up to a certain design flood level—once overtopped or breeched, most levees fail and catastrophic flooding results. Levees that protect critical facilities usually are designed for at least the 0.2-percent-annual-chance flood (500-year) and have freeboard to increase the factor of safety.

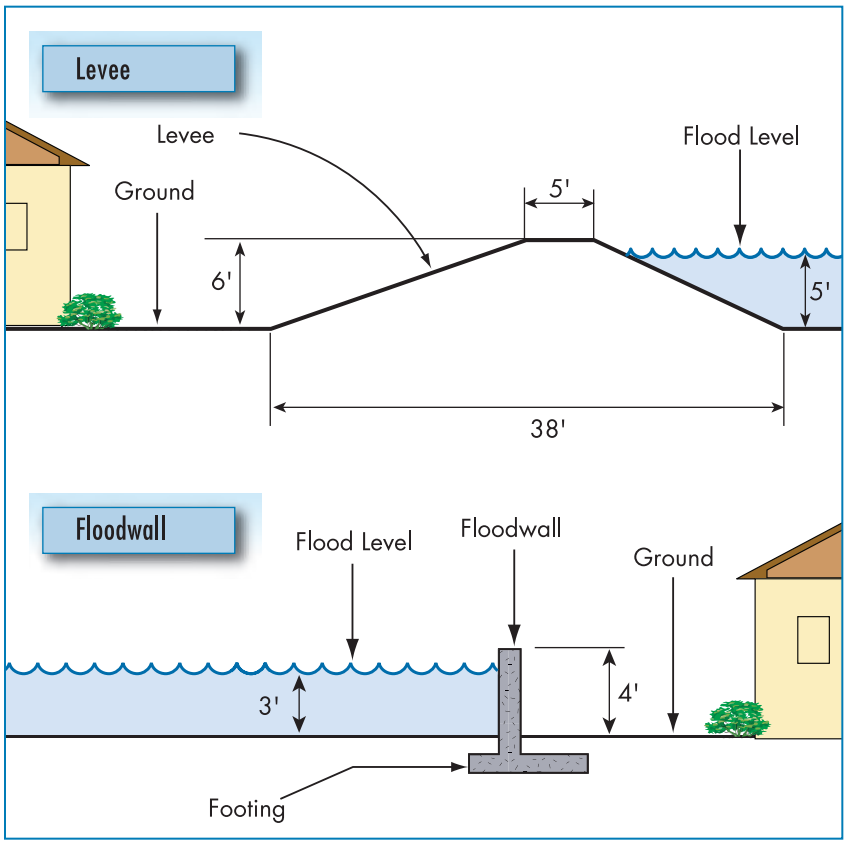


Figure 2-23: Schematic of typical earthen levee and permanent floodwall

Floodwall: Floodwalls are similar to levees in that they provide protection to certain areas (see Figure 2-23). Failure or overtopping of a floodwall can result in catastrophic flooding. A floodwall is a significant structure that is designed to hold back water of a certain depth based on the design flood for the site. Generally, due to design factors, floodwalls are most effective in areas with relatively shallow flooding and minimal wave action. As with levees, designs must accommodate interior drainage on the land side, and maintenance and operations are critical for adequate performance.

Floodwalls that protect essential and critical facilities usually are designed for the 0.2-percent-annual-chance flood (500-year) and have freeboard to increase the factor of safety.

2.3.1.2 Elevation Considerations

"Lowest floor" is the floor of the lowest enclosed area (including the basement). An unfinished or flood-resistant enclosure, usable solely for parking of vehicles, building access, or storage in an area other than a basement, is not the lowest floor, provided the enclosure is built in compliance with applicable requirements.

The selection of the appropriate method of elevating a critical facility in an A Zone flood hazard area depends on many factors, including cost, level of safety and property protection determined as acceptable risk, nature of the flood hazard area, and others. Methods of elevation are described below. The minimum elevation requirement is that the lowest floor (including the basement) must be at or above the DFE (plus freeboard, if desired or required). Table 2-1 in Section 2.1.3.4 summarizes the elevation requirements in ASCE 24. Given the importance of critical facilities, elevation of the lowest floor to or above the 0.2 percent-annual-chance flood (500-year) elevation is crucial.

ASCE 7 outlines methods to determine design loads and load combinations in flood hazard areas, including hydrostatic loads, hydrodynamic loads, wave loads, and debris impact loads. ASCE 24 addresses design requirements for structures in flood hazard areas.

For elevation methods other than fill, the area under elevated buildings in A Zones may be used only for limited purposes: parking, building access, and limited storage (crawlspaces are treated as enclosures, see below). Owners and designers are cautioned that enclosures below the design flood elevation are exposed to flooding and the contents will be damaged or destroyed by floodwaters. The walls surrounding an enclosure must have flood openings that are intended to equalize interior and exterior water levels during rising and falling flood conditions, to prevent differential hydrostatic pressures that could lead to structural damage. The enclosed area must not contain utilities and equipment (including ductwork) below the required elevation.

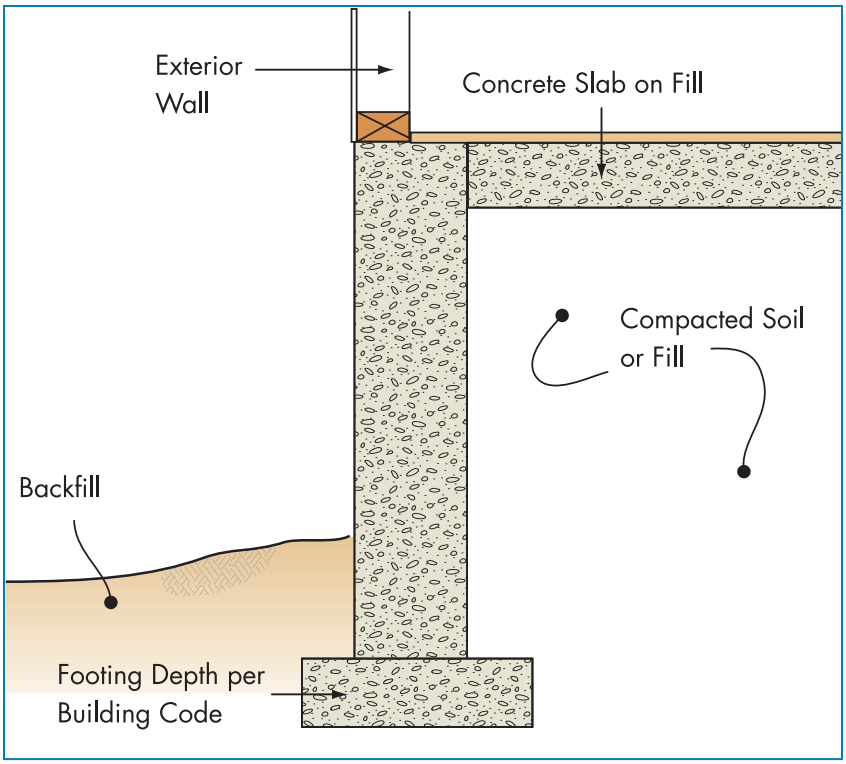
Slab-on-grade foundation on structural fill: This is considered to be the safest method to elevate a building in many flood hazard areas, except those where waves and high velocity flows may cause erosion. Structural fill can be placed so that, when water rises up to the DFE, it will not touch the building (see Figure 2-24) and building access is maintained. The fill must be designed to minimize adverse impacts, such as increasing flood elevations on adjacent properties, increasing erosive velocities, and causing local drainage problems. To ensure stability, especially as floodwaters recede and the soils drain, fill must be designed for the anticipated water depths and duration. A geotechnical engineer or soil scientist may need to examine underlying soils to determine if the bearing capacity is sufficient to carry the added weight of fill, or if consolidation over time may occur. In addition, the effects of long-term compaction of the fill should be considered, and may prompt additional elevation as a factor of safety. The horizontal extent of fill from the foundation should be designed to facilitate access by emergency and fire vehicles, with a minimum 25-foot width recommended. Designers are cautioned to avoid excavating a basement into fill without added structural protection (and certification that the design meets requirements for dry floodproofing), due to the potential for significant hydrostatic loads and uplift on basement floors.

Communities may require a registered design professional to certify that buildings elevated on fill are reasonably safe from flooding. FEMA NFIP Technical Bulletin 10 (2001) discusses criteria for this certification.

Figure 2-24: Municipal building elevated on fill

Stem wall foundations: Stem wall foundations have a continuous perimeter grade beam, or perimeter foundation wall, that is backfilled with compacted earth to the underside of the concrete floor slab (see Figure 2-25). Because this foundation type is backfilled and has no crawlspace, hydrostatic pressures are minimized. Stem wall foundations are designed to come in contact with floodwaters on the exterior. They are more stable than perimeter wall foundations with crawlspaces, but could experience structural damage if undermined by local scour and erosion. Designs must account for anticipated debris and ice impacts, and incorporate methods and materials to minimize impact damage.

Figure 2-25: Typical stem wall foundation



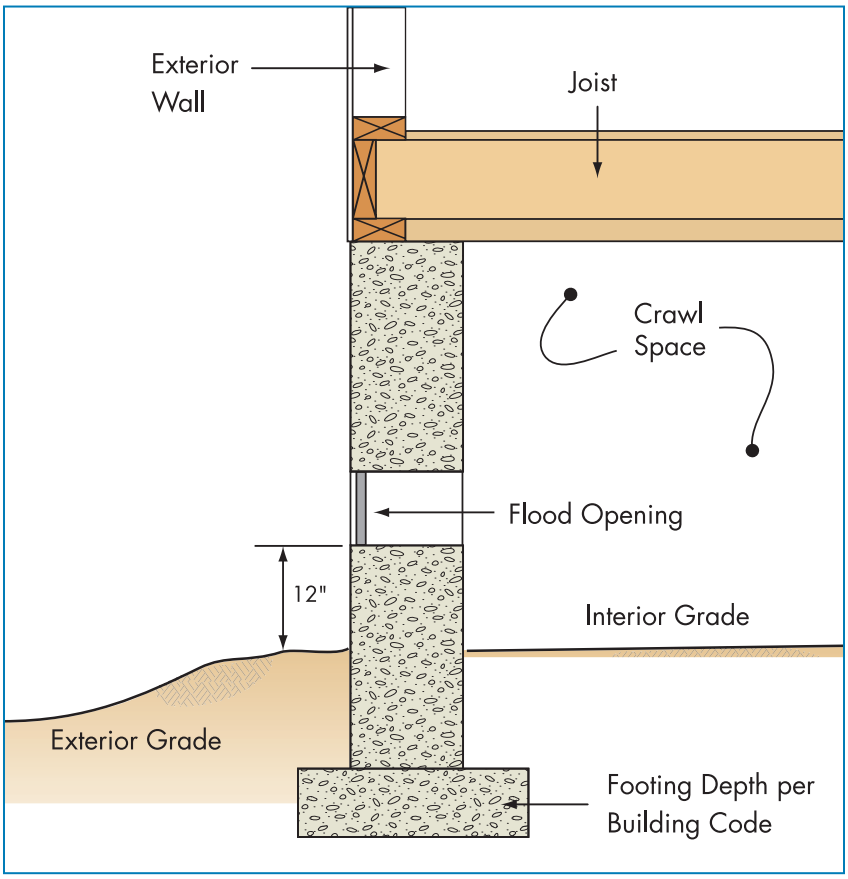
Columns or shear wall foundations (open foundations): Open foundations consist of vertical load bearing members (columns, piers, pilings, and shear walls) without solid walls connecting the vertical members. Open foundations minimize changes to the floodplain and local drainage patterns, and the area under the building can be used for parking or other uses (see Figure 2-26). The design of the vertical members must also account for hydrodynamic loads

and debris and ice impact loads. Flood loads on shear walls are reduced if they are oriented parallel to the anticipated direction of flow. Erodible soils may be present and local scour may occur; both must be accounted for in designs by extending the load-bearing members and foundation elements well below the expected scour depth. Depending on the total height of the elevated facility, the design may need to take into consideration the increased exposure to wind and uplift, particularly where breaking waves are expected.

Figure 2-26: School elevated on columns

Continuous perimeter walls (enclosed foundations with crawlspace): Unlike stem wall foundations, continuous perimeter walls enclose an open area or crawlspace (see Figure 2-27). The perimeter walls must have flood openings that are intended to equalize interior and exterior water levels automatically during changing flood conditions to prevent differential hydrostatic pressures that could lead to structural damage. Flood openings may be engineered and certified for the required performance level, or must meet prescriptive requirements (notably, the opening must provide at least 1 square inch of net open area for each square foot of area enclosed). Perimeter wall design must also account for hydrodynamic loads, and debris and ice impact loads. Enclosed crawlspaces must not contain utilities or equipment (including ductwork) below the required elevation. Designers must provide adequate underfloor ventilation and subsurface drainage to minimize moisture problems after flooding.

Figure 2-27:
Typical crawlspace with flood openings



Pier supports for manufactured and portable units: Manufactured buildings and portable units must be elevated above the DFE (plus freeboard, if required). Pier supports must account for hydrodynamic loads and debris and ice impact loads and units must be anchored to resist wind loads. Although written specifically for manufactured housing units, FEMA 85, *Manufactured Home Installation in Flood Hazard Areas*, has useful information that is applicable to portable units.

2.3.1.3 Flood-Resistant Materials

All structural materials, nonstructural (finish) materials, and connectors that are used below certain elevations (see Table 2-2) should be flood-resistant. Flood-resistant materials have sufficient strength, rigidity, and durability to adequately resist flood loads and damage due to saturation. They are building materials that

are capable of withstanding direct and prolonged contact with floodwaters without sustaining any damage that requires more than cosmetic repair. As defined in ASCE 24, the term "prolonged contact" means partial or total inundation by floodwaters for 72 hours for non-coastal areas (fresh water) or 12 hours for coastal areas.

FEMA NFIP Technical Bulletin FIA-TB-2, *Flood-Resistant Materials Requirements,* provides some additional information. Many types of materials and application products are classified by degrees of resistance to flood damage.

In general, materials that are exposed to floodwaters are to be capable of resisting damage, deterioration, corrosion, or decay. Typical construction materials range from highly resistant to not at all resistant to water damage. FEMA NFIP Technical Bulletin FIA-TB-2 contains tables with building materials, classified based on flood resistance (Table 2-2).

In areas away from the coast, exposed structural steel should be primed, coated, plated, or otherwise protected against corrosion. Secondary components such as angles, bars, straps, and anchoring devices, as well as other metal components (plates, connectors, screws, bolts, nails angles, bars, straps, and the like) should be stainless steel or hot-dipped galvanized after fabrication.

Table 2-2: Classes of Flood-Resistant Materials

NFIP	Class	Class Description
Acceptable	5	Highly resistant to floodwater damage. Materials in this class are permitted for partially enclosed or outside uses with essentially unmitigated flood exposure.
	4	Resistant to floodwater damage. Materials in this class may be exposed to and/or submerged in floodwaters in interior spaces and do not require special waterproofing protection.
Unacceptable	3	Resistant to clean water damage. Materials in this class may be submerged in clean water during periods of intentional flooding.
	2	Not resistant to water damage. Materials in this class require essentially dry spaces that may be subject to water vapor and slight seepage.
	1	Not resistant to water damage. Materials in this class require dry conditions.

SOURCE: FROM U.S. ARMY CORPS OF ENGINEERS, *FLOODPROOFING REGULATIONS* (1995).

Concrete and masonry that are designed and constructed in compliance with applicable standards are generally considered to be flood-resistant. However, masonry facings are undesirable finishes unless extra anchoring is added to prevent separation (see Figure 2-28). Wood and timber members exposed to flood waters should be naturally decay resistant species or pressure treated with appropriate preservatives.

Structural steel and other metal components exposed to corrosion should be stainless steel or hot-dipped galvanized after fabrication.

Figure 2-28:
Brick facing separated from the masonry wall at Port Sulpher High School, LA.

2.3.1.4 Dry Floodproofing Considerations

Dry floodproofing involves a combination of design and special features that are intended to prevent the entry of water into a building and its utilities while also resisting flood forces. It involves structural reinforcement so that exterior walls are sufficiently robust to withstand the loads described in Section 2.1.2 (hydrostatic

pressure, hydrodynamic loads, wave loads, and debris impact loads). Exterior walls must also be designed to prevent infiltration and seepage of water, whether through the wall itself or through any openings, including where utility lines penetrate the envelope. Floodproofed buildings constructed on permeable soils require additional design attention, because they are susceptible to hydrostatic pressure from below.

Communities that participate in the NFIP will require that a registered professional engineer or architect develop or review the structural design, specifications, and plans, and certify that the dry floodproofing design and methods of construction to be used are in accordance with accepted standards of practice. The standards of practice require that the building, together with attendant utility and sanitary facilities, be designed so that it is watertight, with walls substantially impermeable to the passage of water and with structural components having the capability of resisting hydrostatic and hydrodynamic loads and effects of buoyancy associated with the design flood event.

According to the NFIP regulations, nonresidential buildings and nonresidential portions of mixed-use buildings may be dry floodproofed. Areas used for living and sleeping purposes in health care facilities and dormitory rooms at fire stations may not be dry floodproofed. Although floodproofing of the nonresidential spaces is allowed, careful consideration must be given to the possible risk to occupants and additional physical damage.

All flood protection measures are designed for certain flood conditions. Therefore, there is some probability that the design will be exceeded (i.e., water will rise higher than accounted for in the design). When this happens to a dry floodproofed building, the consequences can be catastrophic. As a general rule, dry floodproofing is a poor choice for new critical facilities when avoidance of the floodplain or elevation methods to raise the building above the flood level can be applied. Floodproofing may be acceptable for retrofitting existing buildings under certain circumstances (see Section 2.4.4).

A number of dry floodproofing limitations and requirements are specified in ASCE 24:

- Dry floodproofing is limited to areas where flood velocities at the site are less than or equal to 5 feet per second.

- If human intervention is proposed, such as measures to protect doors and windows, the flood warning time shall be a minimum of 12 hours unless the community operates a flood

warning system and implements a notification procedure that provides sufficient time to undertake the measures requiring intervention.

- At least one door satisfying building code requirements for an exit door or primary means of escape must be provided above the level of protection.

- An emergency plan, approved by the community and posted in at least two conspicuous locations, is required in floodproofed buildings; the plan is to specify the location of panels and hardware, methods of installation, conditions that activate deployment, a schedule for routine maintenance of any aspect that may deteriorate over time, and periodic practices and drills.

Windows and doors that are below the flood level used for dry floodproofing design present significant potential failure points. They must be specially designed units (see Figure 2-29) or be fitted with gasketed, mountable panels that are designed for the anticipated flood conditions and loads. Generally speaking, it is difficult to protect window and door openings from water more than a few feet deep. The framing and connections must be specifically designed for these protective measures, or water pressure may cause window and door frames to separate from the building.

The documents *Flood Resistant Design and Construction* (ASCE 24-05), *Flood Proofing: How to Evaluate Your Options* (USACE, 1993), *Flood Proofing Regulations* (USACE, 1995), *Floodproofing Non-Residential Structures* (FEMA 102, 1986), *Non-Residential Floodproofing – Requirements and Certification* (FIA-TB-3 [FEMA NFIP, 1993]), *Flood Proofing Systems & Techniques* (USACE, 1984) provide additional information about floodproofing.

Dry floodproofing is required to extend to 1 or 2 feet above the BFE (see Table 2-1). For the purpose of obtaining NFIP flood insurance, the floodproofing must extend at least 1 foot above the BFE, or the premiums will be very high. Therefore, a higher level of protection is recommended.

Floodproofing techniques are considered to be permanent measures if they are always in place and do not require any specific human intervening action to be effective. Use of contingent floodproofing measures that require installation or activation, such as window shields or inflatable barriers,

may significantly reduce the certainty that floodproofing will be effective. Rigorous adherence to a periodic maintenance plan is critical to ensure proper functioning. The facility must have a formal, written plan, and people responsible for implementing the measures must be informed and trained. These measures also depend on the timeliness and credibility of the warning. In addition, floodproofing devices often rely on flexible seals that require periodic maintenance and that, over time, may deteriorate and become ineffective. Therefore, a maintenance plan must be developed and a rigorous annual inspection and training must be conducted.

Dry floodproofed critical facilities must never be considered safe for occupancy during periods of high water; floodproofing measures are intended only to reduce physical damage.

Figure 2-29: Permanent watertight doors for deep water

SOURCE: PRESRAY CORPORATION

Safety of occupants is a significant concern with dry floodproofed buildings. Regardless of the degree of protection provided, dry floodproofed buildings should not be occupied during flood events, because failure or overtopping of the floodproofing measures is likely to cause catastrophic structural damage. When human intervention is required, the people responsible for implementing those measures remain at risk while at the building, even if a credible warning system is in place, because of the many uncertainties associated with predicting the onset of flood conditions.

2.3.2 RISK REDUCTION IN "V ZONES"

Flood hazard areas designated as "V Zones" on FIRMs are relatively narrow areas along open coasts and lake shores where the base flood conditions are expected to produce 3-foot or higher waves. V Zones, sometimes called coastal high hazard areas or special flood hazard areas subject to high-velocity wave action, are found on the Pacific, Gulf, and Atlantic coasts, and around the Great Lakes.

Every effort should be made to locate critical facilities outside of V Zones, because the destructive nature of waves makes it difficult to design a building to be fully functional during and after a flood event. This is particularly true in coastal areas subject to hurricane surge flooding (see Section 2.3.4). However, when a decision is made to build a critical facility in a V Zone or Coastal A Zone, the characteristics of the site and the nature of the flood hazards must be examined prior to making important design decisions.

Beach front areas with sand dunes pose special problems. Man-made alterations of sand dunes are not allowed unless analyses indicate that such modifications will not increase potential flood damage. The site modifications described in Section 2.3.1.1 that may be used in some A Zones to reduce flood hazards generally are not feasible in V Zones because of wave forces and potential erosion and scour. In particular, structural fill is not allowed as a means to raise a building site above the flood level.

The NFIP and ASCE 24 do not allow use of dry floodproofing measures to protect nonresidential structures in V Zones.

2.3.2.1 Elevation Considerations

The selection of the appropriate method of elevating a critical facility in a V Zone flood hazard area depends on many factors, including cost, desired level of safety and property protection, and the nature of the flood hazard area. The NFIP regulations and the building codes require the elevation of the bottom of the lowest horizontal structural member of the lowest floor (including basement) to be at or above the DFE (plus freeboard, where required). Given the importance of critical facilities, elevation to or above the 0.2-percent-annual-chance flood (500-year) elevation is appropriate and strongly recommended.

> Communities that participate in the NFIP will require that a registered professional engineer or architect develop or review the structural design, specifications, and plans, and certify that the design and methods of construction to be used are in accordance with accepted standards of practice. The standards of practice require that the foundation and structure attached thereto is anchored to resist flotation, collapse, and lateral movement due to the effects of wind and water loads acting simultaneously on all building components. Water loading values shall be those associated with the base flood conditions, and wind loading values shall be those required by applicable State or local building codes and standards.

Buildings in V Zones must be elevated using open foundations, which consist of vertical load bearing members (columns, piers, pilings, and shear walls) without solid walls connecting the vertical members. The design of the vertical members must also account for hydrodynamic loads and debris impact loads. Flood loads on shear walls are reduced if the walls are oriented parallel to the anticipated direction of flow. Since erodible soils may be present and local scour may occur, both conditions must be accounted for in designs of load-bearing members and foundations.

The area under elevated buildings in V Zones may be used only for limited purposes: parking, building access, and limited storage. Owners and designers are cautioned that enclosures below the DFE are exposed to flooding. Areas under elevated buildings may be open or enclosed by lattice walls or screening. However, if areas are enclosed by solid walls, the walls must be specifically designed to break away under certain flood loads to allow the free passage of floodwaters under the building. Breakaway walls are non-load bearing walls, i.e., they do not provide structural support for the building. They must be designed and constructed to collapse under the impact of floodwaters in such a way that the supporting foundation system and the structure are not affected.

2.3.2.2 Flood-Resistant Materials

Section 2.3.1.3 addresses the general requirement that all structural materials, nonstructural (finish) materials, and connectors that are used below certain elevations are to be flood-resistant materials. In coastal areas, airborne salt aerosols and inundation with saline water increase the potential for corrosion of some metals. Structural steel and other metal components that are exposed to corrosive environments should be stainless steel or hot-dipped galvanized after fabrication.

2.3.3 RISK REDUCTION IN "COASTAL A ZONES"

Coastal A Zones are areas of the mapped floodplain where breaking waves that are between 1.5 to 3 feet high are expected under base flood conditions. Coastal A Zones are part of the area shown as the A Zone on a FIRM, landward of the mapped V Zone or landward of open coasts that do not have a V Zone. FIRMs do not distinguish between Coastal A Zones and A Zones. Designers should determine whether Coastal A Zone conditions are likely to occur at a critical facility site because of the anticipated wave action and loads. This determination is based on an examination of the site and its surroundings, the actual surveyed ground elevations, and the predicted stillwater elevations found in the Flood Insurance Study.

Coastal A Zones are present where two conditions exist: where the expected floodwater depth is sufficient to support waves 1.5 to 3 feet high, and where such waves can actually occur (see Figure 2-30). The first condition occurs where stillwater depths (vertical distance between the stillwater elevation and the ground) are more than 2 feet deep. The second condition occurs where there are few obstructions between the shoreline and the site. The stillwater depth requirement is necessary, but is not sufficient by itself to warrant designation as a Coastal A Zone, because obstructions in the area may block wind and dampen waves. Obstructions that may dampen waves include buildings, locally high ground, and dense tree stands.

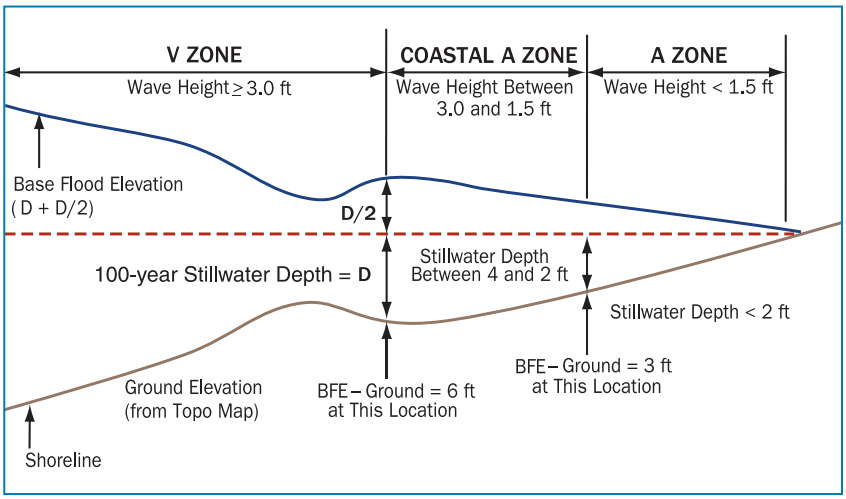


Figure 2-30:
Flood hazard zones in coastal areas

Field observations and laboratory research have determined that flooding with breaking waves between 1.5 and 3 feet high produces more damage than flooding of similar depths without waves. Therefore, ASCE 24 specifically requires application of the NFIP's V Zone design requirements in Coastal A Zones. Section 2.3.2.1 addresses elevation requirements and foundation types, and Section 2.3.2.2 addresses flood-resistant materials, used in V Zones and Coastal A Zones. The designers are advised to pay special attention to two additional considerations:

Although the NFIP regulations and the model building codes allow dry floodproofing of nonresidential buildings in flood hazard areas where waves are predicted to be between 1.5 and 3 feet during the base flood (called Coastal A Zones), designers are cautioned to fully consider the additional forces associated with wave impacts, which may make dry floodproofing a less feasible alternative.

- Debris loads may be significant in Coastal A Zones landward of V Zones where damaged buildings, piers, and boardwalks can produce battering debris. Foundations designed to account for debris loads will minimize damage.

- Especially in high wind regions, designers must pay special attention to the entire roof-to-foundation load path when designing and specifying connections. If designed to meet V Zone requirements, designs for buildings in Coastal A Zones will account for simultaneous wind and flood forces. Corrosion-resistant connections are especially important for the long-term integrity of the structure.

2.3.4 RISK REDUCTION IN HURRICANE STORM SURGE AREAS

Coastal communities along the Atlantic and Gulf coasts are subject to storm surge flooding generated by hurricanes and tropical storms. Depending on a number of variables, storm surge flood depths may significantly exceed the BFE. In addition, waves are likely to be larger than predicted for the base flood, and will occur in areas where significant wave action during the base flood is not expected. Application of the minimum requirements related to elevation of the lowest floor and foundation design does not result in flood resistance for such extreme conditions. The following special considerations will provide a greater degree of protection for critical facilities located in areas subject to storm surges.

Higher foundations: Foundations should be designed to elevate the building so that the lowest horizontal structural members are higher than the minimum required elevation. Additional elevation not only reduces damage that results from lower probability events, but the cost of Federal flood insurance is usually lower. However, accessibility may be affected and there will be some additional construction costs that must be balanced against avoided future damage and a higher likelihood that a facility can be more rapidly restored to full function.

Scour and erosion: Storm surge flooding and waves can cause scour and erosion, even at locations that are some distance from the shoreline. Foundation designs for critical facilities in coastal communities should account for some erosion and local scour of supporting soil during low probability surge events.

Water-borne debris: Storm surge flooding can produce large quantities of floating debris, even at locations that are some distance from the shoreline. Debris damages nonstructural building components and, in some cases, prolonged battering can lead to structural failure. Foundation designs for critical facilities in coastal communities should account for debris loads. This is especially important where damage to other buildings in the area may generate additional debris, thereby increasing the loads.

Continuous load path: Especially in high wind regions, designers should pay special attention to the entire roof-to-foundation load

path when designing and specifying connections. Connections must be capable of withstanding simultaneous wind and flood forces. Poorly connected buildings may fail or float off of foundations when floodwaters and waves are higher than the design flood elevation. Corrosion-resistant connections are critical for the long-term integrity of the structure, and should be inspected and maintained periodically.

Emergency equipment: Equipment that is required for emergency functioning during or immediately after a storm surge event, such as emergency generators and fuel tanks, is best installed well above the design flood elevation.

Occupancy of surge-prone areas: Designers and owners should plan to use the lowest elevated floor for non-critical uses that, even if exposed to flooding more severe than the design flood, will not impair critical functioning during post-flood recovery.

2.3.5 RISK REDUCTION FOR RELATED FACILITIES

Critical facilities do not exist as purely independent buildings. They usually are accompanied by a variety of related facilities, such as utility installations both inside and outside of buildings, gas and electric services, water and wastewater services, aboveground or underground storage tanks, accessory structures and outbuildings, and access roads and parking lots.

2.3.5.1 Access Roads

Access roads to critical facilities should be designed to provide safe access at all times, to minimize impacts on flood hazard areas, to minimize damage to the road itself, and to minimize exposing vehicles to dangerous situations. Depending on the site and specific flood characteristics, balancing those elements can be difficult. Designers should take the following into consideration.

Safety factors: Although a critical facility's access road may not be required to carry regular traffic like other surface streets, a flood-prone road always presents a degree of risk to public safety. To

minimize those risks, some State or local regulatory authorities require that access roads be designed so that the driving surface is no more than 1 to 2 feet below the DFE. To maximize evacuation safety, two separate accesses to different feeder roads are recommended. In some circumstances, especially long-duration flooding where a critical facility is built on fill, dry access may allow continued operations.

Floodplain impacts: Engineering analyses may be required to document the effects on flood elevations and flow patterns if large volumes of fill are required to elevate a road to minimize or eliminate flooding above the driving surface.

Drainage structure and road surface design: The placement of multiple drainage culverts, even if not needed for local drainage, can facilitate the passage of floodwaters and minimize the potential for a road embankment to act as a dam. Alternatively, an access road can be designed with a low section over which high water can flow without causing damage. Embankments should be designed to remain stable during high water and as waters recede. They should be sloped and protected to resist erosion and scour. Similarly, the surface and shoulders of roads that are intended to flood should be designed to resist erosion. The increased resistance to erosion may be accomplished by increasing the thickness of the road base.

2.3.5.2 Utility Installations

Utilities associated with new critical facilities in flood hazard areas must be protected either by elevation or special design and installation measures. Utilities subject to this provision include all systems, equipment, and fixtures, including mechanical, electrical, plumbing, heating, ventilating, and air conditioning. Potable water systems (wellheads and distribution lines) and wastewater collection lines are addressed in Section 2.3.5.3.

Utility systems and equipment are best protected when elevated above the DFE (plus freeboard, if required). In some cases, equipment can be located inside protective floodproofed enclosures, although it must be recognized that flooding that exceeds the design level of such an enclosure will adversely affect the equipment (see Figure 2-31). Designers should pay partic-

ular attention to underfloor utilities and ductwork to ensure that they are properly elevated. Plumbing conduits, water supply lines, gas lines and electric cables that must extend below the DFE should be located, anchored, and protected to resist the effects of flooding. Equipment that is outside of elevated building also must be elevated:

For more information on utility installations, see *Protecting Building Utilities from Flood Damage: Principles and Practices for the Design and Construction of Resistant Building Utility Systems* (FEMA 348, 1999).

- In A Zones, equipment may be affixed to raised support structures or mounted on platforms that are attached to or cantilevered from the primary structure.

- In V Zones and Coastal A Zones, equipment may be affixed to raised support structures designed for the flood conditions (waves, debris impact, erosion, and scour) or mounted on platforms that are attached to or cantilevered from the primary structure. If an enclosure is constructed under the elevated building, the designer must take care that utilities and attendant equipment are not mounted on or pass through walls that are intended to break away.

Figure 2-31:
Equipment room with watertight door

SOURCE: PRESRAY CORPORATION

Although it is difficult to achieve, the model building codes and NFIP regulations provide an alternative that allows utility systems and equipment to be located below the DFE. This alternative requires that such systems and equipment be designed, constructed, and installed to prevent floodwaters from entering or accumulating within the components during flood events.

2.3.5.3 Potable Water and Wastewater Systems

New installations of potable water systems and wastewater collection systems are required to resist flood damage, including damage associated with infiltration of floodwaters and discharge of effluent. Health concerns arise when water supply systems are exposed to floodwaters. Contamination from flooded sewage systems poses additional health and environmental risks. Onsite water supply wellheads should be located on land elevated from the surrounding landscape to allow contaminated surface water and runoff to drain away. Well casings should extend above the design flood elevation, and casings should be sealed with a tight-fitting, floodproof, and vermin-proof well cap. The space between the well casing and the side of the well must be sealed to minimize infiltration and contamination by surface waters.

Sewer collection lines should be located and designed to avoid infiltration and backup due to rising floodwaters. Devices designed to prevent backup are available and are recommended to provide an added measure of protection.

Onsite sewage systems usually are not used as the primary sewage disposal system for new critical facilities. However, it would be prudent for owners, planners, and designers to consider a backup onsite sewage system if a facility's functionality will be impaired if the public system is affected by flooding. Designers are advised that local or State health departments may impose constraints that limit or prevent locating septic fields in floodplain soils or within a mapped flood hazard area. If allowed, septic fields should be located on the highest available ground to minimize inundation and impairment by floodwaters. An alternative to a septic field is installation of a holding tank that is sized to contain wastewater for a period of time, perhaps a few days if the municipal system is expected to be out of service.

2.3.5.4 Storage Tank Installations

Aboveground and underground storage tanks located in flood hazard areas must be designed to resist flotation, collapse, and lateral movement. ASCE 24 specifies that aboveground tanks are to be elevated or constructed, installed and anchored to resist at least 1.5 times the potential buoyant and other flood forces under design flood conditions, assuming the tanks are empty. Similarly, underground tanks are to be anchored to resist at least 1.5 times the potential buoyant forces under design flood conditions, assuming the tanks are empty. In all cases, designers are cautioned to address hydrodynamic loads and debris impact loads that may affect tanks that are exposed to floodwaters. Vents and fill openings or cleanouts should be elevated above the DFE or designed to prevent the inflow of floodwaters or outflow of the contents of tanks.

2.3.5.5 Accessory Structures

Depending on the type of accessory structures, full compliance with floodplain management regulations is appropriate and may be required. For example, buildings or portable classrooms that serve educational purposes (e.g., offices, classrooms), even if detached from the primary school building, are not considered to be accessory in nature and must be elevated and protected to the same standards as other buildings.

Some minor accessory structures need not fully comply, but may be "wet floodproofed" using techniques that allow them to flood while minimizing damage. Examples include small storage sheds, garages, and restrooms. Accessory structures must be anchored to resist flotation, collapse, and lateral movement. Flood-resistant materials must be used and utilities must be elevated above the DFE (plus freeboard, if required). Openings in walls must be provided to allow the free inflow and outflow of floodwaters to minimize the hydrostatic loads that can cause structural damage. Because wet floodproofed accessory buildings are designed to flood, critical facility staff must be aware that contents will be damaged.

2.4 RISK REDUCTION FOR EXISTING CRITICAL FACILITIES

2.4.1 INTRODUCTION

Section 2.2.2 describes the type of damage that can be sustained by critical facilities that already are located in flood hazard areas. The vulnerability of these facilities can be reduced if they can be made more resistant to flood damage. Decisionmakers may take such action when flood hazards are identified and there is a desire to undertake risk reduction measures proactively. Interest may be prompted by a flood or by the requirement to address flood resistance as part of proposed substantial improvement or an addition. Some questions and guidance intended to help identify building characteristics of importance when considering risk reduction measures for existing facilities are included in the checklist in Section 2.5.

Work on existing buildings and sites is subject to codes and regulations, and the appropriate regulatory authority with jurisdiction should be consulted. With respect to reducing flood risks, work generally falls into the categories described in the following subsections.

Owners and operators of public and not-for-profit critical facilities should be aware of the importance of flood insurance coverage for facilities that are located in the flood hazard areas shown on NFIP maps. If not insured for flood peril, the amount of flood insurance that should have been in place will be deducted from any Federal disaster assistance payment that would otherwise have been made available. A particular facility may have to absorb up to $1 million in unreimbursed flood damage per building, because the NFIP offers $500,000 in building coverage and $500,000 in contents coverage for nonresidential buildings (coverage limits as of early 2006).

2.4.2 SITE MODIFICATIONS

A plan to modify the site of an existing facility that is subject to flooding requires careful examination by an experienced professional engineer. Determining the suitability of a specific measure requires a complex evaluation of many factors, including the nature of flooding and the nature of the site. The first part of Table 2-3 in Section 2.5 identifies elements that influence the choice of mitigation measures applicable to existing sites. Some flood characteristics may make it infeasible to apply site modification measures to existing facilities (e.g., depths greater than 3 to 4 feet, very high velocities, insufficient warning because of flash flooding or rapid rate of rise, and very long duration).

A common problem with all site modifications is the matter of site access. Depending on the topography of the site, construction of barriers to floodwaters may require special access points. Access points may be protected with manually installed stop-logs or designed gates that drop in, slide, or float into place. Whether activated by automatic systems or manually operated, access protection requires sufficient warning time.

Other significant constraining factors include poor soils and insufficient land area which can make site modifications either infeasible or very costly. A critical facility may be among several buildings and properties that can be protected, increasing the benefits. For any type of barrier, rainfall that collects on the land side must be accounted for in the design, whether through adequately sized stormwater storage basins constructed on land set aside for this purpose, or by providing large-capacity pumps to move collected water to the water side of the barrier.

Each of these site modification measures described below has limitations, including the fact that floods larger than the design flood will exceed the level of protection.

Regrading the site (berm): Where a facility is exposed to relatively shallow flooding and sufficient land area is available, regrading the site or constructing an earthen berm may provide adequate protection.

Earthen levee: Earthen levees are engineered structures that are designed to keep water away from land area and buildings. Hydraulic analyses and geotechnical investigations are required to determine their feasibility and effectiveness. For existing sites, constraints include the availability of land (levees have a large "footprint" and require large land areas), cost (including availability of suitable fill material and long-term maintenance), and difficulties with site access. Levees are rarely used to protect a single site, although they may offer a reasonable solution for a group of buildings. Locating levees and floodwalls within a designated floodway is generally not allowed. Rapid onset flooding makes it impractical to design a flood levee with access points that require installation of a closure system. Earthen levees may also be subject to high velocity flows that cause erosion and affect their stability.

Permanent floodwall: Floodwalls are freestanding, permanent engineered structures that are designed to prevent encroachment of floodwaters. Typically, a floodwall is located some distance from a building, so that structural modification of the existing building is not required. Floodwalls may protect only the low side of a site (in which case they must "tie" into high ground) or completely surround a site (which may affect access because special closure structures are required and must be installed before the onset of flooding, see Figure 2-32).

Figure 2-32:
A masonry floodwall with multiple engineered openings protected the Oak Grove Lutheran School in Fargo, ND from flooding in 2001.

SOURCE: FLOOD CENTRAL AMERICA, LLC

Mobilized floodwall: This category of flood protection measures includes fully engineered flood protection structures that have permanent features (foundation and vertical supports) and features that require human intervention when a flood is predicted (horizontal components called planks or stop-logs). Mobilized floodwalls have been used to protect entire sites, or to tie into permanent floodwalls or high ground. Because of the manpower and time required for proper placement, these measures are better suited to conditions that allow long warning times.

2.4.3 ADDITIONS

All model building codes generally treat additions as new construction and require additions to critical facilities in flood hazard areas to be elevated or dry floodproofed to minimize exposure to flooding. However, full compliance with the code and NFIP requirements is only required if an addition is a substantial improvement (i.e., the cost of the addition plus all other work equals or exceeds 50 percent of the market value of the building). Designers are cautioned that existing buildings as well may be required to be brought into compliance with the flood-resistant provisions of the code or local ordinances if the addition is structurally connected to the existing building.

For more information on additions and substantial improvements, see *Answers to Questions About Substantially Damaged Buildings* (FEMA 213, 1991).

Section 2.3.1.2 outlines elevation options that are applicable to additions in A Zones (see Section 2.3.2.1 for elevation considerations applicable to additions in V Zones). Elevation of an addition on fill may not be feasible unless structural fill can be placed adjacent to the existing building. Utility service equipment for additions must meet the requirements for new installations (see Section 2.3.5.2).

With respect to code compliance and designing additions to resist flood damage, one of the more significant issues that may come up is ease of access. If the lowest floor of the existing facility is below the DFE, steps, ramps, or elevators will be required for the transition to the new addition. Some jurisdictions may wish to allow variances to the requirement for elevation, because alternative means of access are available, such as ramps and elevators.

Under the regulations of the NFIP and FEMA guidance, it is not considered appropriate to grant such a variance.

2.4.4 REPAIRS, RENOVATIONS, AND UPGRADES

Every critical facility that is considered for upgrades and renovations, or that is being repaired after substantial damage from any cause, must be examined for structural integrity and stability to determine compatibility with structural modifications that may be required to achieve acceptable performance. When an existing facility is located in a flood hazard area, that examination should include consideration of measures to resist flood damage and reduce risks.

The model building codes and the regulations of the NFIP require that work constituting "substantial improvement" of an existing building be in compliance with the flood-resistant provisions of the code. Non-substantial improvements should take into account measures to reduce future flood damage, such as those described in Section 2.4.8, and wet floodproofing measures that allow water to enter the building to avoid structural damage, as well as emergency measures (see Section 2.4.9).

Additional information on rehabilitation of existing buildings is provided in *Flood Proofing: How to Evaluate Your Options* (USACE, 1993), *Floodproofing Non-Residential Structures* (FEMA 102), *Floodproofing—Requirements and Certification* (FIA-TB-3), and *Engineering Principles and Practices for Retrofitting Flood-prone Buildings* (FEMA 259, 1995). Although written primarily for homes, this last reference contains very detailed checklists and worksheets that can be modified. They also provide some guidance for evaluating the costs and benefits of various measures.

Compliance with flood-resistant provisions means the existing building must be elevated or dry floodproofed. Both options can be difficult for existing critical facilities, given the typical use, size, and complexity of some of these buildings. Retrofit dry floodproofing (described in Section 2.4.5) is generally limited to water depths of 3 feet or less, unless the structural capacity of the buildings have been assessed by a qualified design professional and found to be capable of resisting the anticipated loads.

Elevating an existing building presents an entirely different set of challenges and also requires detailed structural engineering analyses. It involves the same equipment and

methods used to move other types of buildings; expert building movers have successfully moved large, heavy, and complex buildings, sometimes by segmenting them. A critical facility that is elevated in-place must meet the same performance standards set for new construction.

2.4.5 RETROFIT DRY FLOODPROOFING

"Dry floodproofing" refers to measures and methods to render a building envelope substantially impermeable to floodwater.

Modification of an existing building may be required or desired in order to address exposure to design flood conditions. Modifications that may be considered include construction of a reinforced supplementary wall, measures to counter buoyancy (especially if there is below-grade space), installation of special watertight door and window barriers (see Figure 2-33), and providing watertight seals around the points of entry of utility lines. The details of structural investigations and structural design of such protection measures are beyond the scope of this manual.

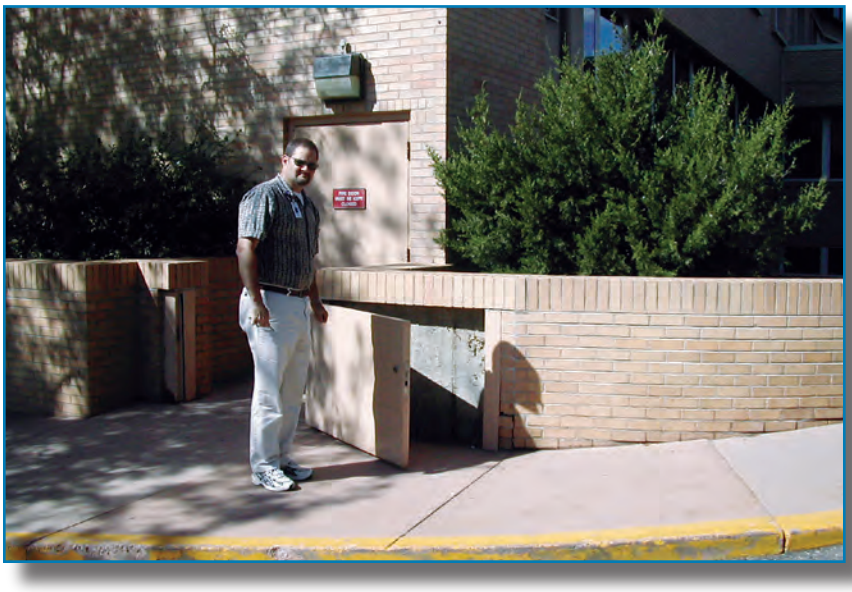


Figure 2-33: Boulder Community Hospital, Boulder, CO installed this permanently mounted floodgate in a low floodwall; the floodgate swings to the left to close off the door that leads to the mechanical equipment room.

Because of the tremendous flood loads that may be exerted on a building not originally designed for such conditions, detailed structural engineering evaluations are required to determine

whether an existing building can be dry floodproofed. The following elements must be examined:

- The strength of the structural system
- Whether non-load bearing walls can resist anticipated loads; secondary walls can be constructed immediately adjacent to existing walls, with a waterproof membrane, to provide adequate strength
- The effects of buoyancy on the walls and floors of below-grade areas
- Effective means to install watertight doors and windows, or mountable panels
- Protection where utilities enter the building
- Methods to address seepage, especially where long-duration flooding is anticipated
- Whether there is sufficient time for human intervention measures, given the availability of official warnings of predicted flood conditions

Application of waterproofing products or membranes directly to exterior walls may minimize infiltration of water, although there are concerns with durability and limitations on use (this measure is most effective for shallow, short-duration flooding). Retrofit measures that require human intervention are considered emergency measures and are discussed in Section 2.4.9.

2.4.6 UTILITY INSTALLATIONS

Some aspects of an existing flood-prone critical facility's utility systems may be modified to reduce damage. The effectiveness of such measures depends not only on the nature of the flooding, but the type of utility and the degree of exposure. Table 2-3 in Section 2.5 lists some questions that will help facility planners and designers to examine risk reduction measures.

Even if a facility is unlikely to sustain extensive structural damage from flooding, high costs and delayed reoccupancy may result from flood-damaged utility systems. The risk reduction design measures described below can be applied, whether undertaken as part of large-scale retrofits of existing buildings, or as separate projects.

Additional guidance on improving the flood resistance of utility installations in existing buildings is found in FEMA 348, *Protecting Building Utilities From Flood Damage: Principles and Practices for the Design and Construction of Flood Resistant Building Utility Systems.*

Relocate from below-grade areas: The most vulnerable utility installations are those located below grade, and the most effective protection measure is to relocate them to properly elevated floors or platforms that are at least 2 feet above the DFE. The complexity of rerouting pipes, conduits, ductwork, electrical service, lines, and connections will depend on site-specific factors.

Elevate components: Whether located inside or outside of the building, some components of utility systems can be elevated-in-place on platforms, including electric transformers, communications switch boxes, water heaters, air conditioning compressors, furnaces, boilers, and heat pumps (see Figure 2-34).

Figure 2-34:
Elevated utility box

Anchor tanks and raise openings: Existing tanks can be elevated or anchored, as described in Section 2.3.5.4. If anchored below the DFE, tank inlets, vents, fill pipes, and openings should be elevated above the DFE, or fitted with covers designed to prevent the inflow of floodwaters or outflow of the contents of the tanks.

Protect components: If utility components cannot be elevated, it may be feasible to construct watertight enclosures, or enclosures with watertight seals that require human intervention to install when flooding is predicted.

Elevate control equipment: Control panels, gas meters, and electrical panels can be elevated, even if the equipment they service cannot be protected.

Separate electrical controls: Where areas within an existing facility are flood-prone, separation of control panels and electrical feeders will facilitate shutdown before floodwaters arrive, and help protect workers during cleanup.

Protect against electrical surges: Current fluctuations and service interruptions are common in areas affected by flooding. Equipment and sensitive electrical components can be protected by installing surge protection and uninterruptible power supplies.

Connections for portable generators: Pre-wired portable generator connections allow for quick, failure-free connection and disconnection of the generators when needed for continued functionality.

2.4.7 POTABLE WATER AND WASTEWATER SYSTEMS

All plumbing fixtures that are connected to the potable water system may become weak points in the system if they allow floodwaters to contaminate the system. Relocating the uses that require plumbing to elevated floors and removing the fixtures that are below the DFE provides protection. Wellheads can be sealed with watertight casings or protected within sealed enclosures.

Wastewater system components become sources of contamination during floods. Rising floodwaters may force untreated sewage to backup through toilets. Specially designed back-flow devices can be installed, or restrooms below the DFE can be provided with overhead piping that may require specially designed pumps to operate properly. Septic tanks can be sealed and anchored.

2.4.8 OTHER DAMAGE REDUCTION MEASURES

A number of steps can be taken to make existing facilities in flood hazard areas more resistant to flood damage, which also facilitates rapid recovery, cleanup, and reoccupancy. Whether these measures are applicable to a specific facility depends, in part, on the characteristics of the flood hazard and the characteristics of the building itself. Facility planners and designers should consider the following:

- Rehabilitate and retrofit the building envelope with openings specifically designed to allow floodwaters to flow in and out to minimize hydrostatic pressure on walls (called wet floodproofing, see Figure 2-35). Although it allows water to enter the building, this measure minimizes the likelihood of major structural damage. Walls that enclose interior spaces would also be retrofitted with openings.

Figure 2-35:
The enclosed entry and storage area to the right of the fire truck bays were retrofitted with flood openings.

SOURCE: SMART VENT, LLC

- Replace interior walls that have cavities with flood-resistant construction or removable panels to facilitate cleanup and drying.

- Abandon the use of below-grade areas (basements) by filling them in to prevent structural damage.

- Permanently relocate high-value or sensitive functions that are often found on the ground floor of critical facilities (e.g., offices, records, libraries, and computer laboratories) to higher floors or elevated additions.

- Install backflow devices in sewer lines.

- Pre-plan actions to move high-value contents from the lower floors to higher floors when a flood warning is issued.

- Replace wall, flooring, and finish materials with flood-resistant materials.

- Use epoxy or other impervious paints on concrete and other permeable surfaces to minimize contamination.

- Install separate electric circuits and ground fault interrupter circuit breakers in areas that will flood. Emergency measures should be provided so that electrical service can be shut down to avoid electrocution hazards.

- Relocate chemicals to storage areas not subject to flooding.

2.4.9 EMERGENCY MEASURES

Emergency response to flooding is outside the scope of this manual. However, it is appropriate to examine feasible emergency measures that may provide some protection. The following discussion pertains only to emergency measures that have been used to reduce flood damage to older buildings that are already located in flood hazard areas. These measures do not achieve compliance with building and life safety codes, they may not provide protection to occupants, and they can experience a high frequency of failure depending on human factors related to deployment.

Emergency barriers are measures of "last resort," and should be used only when a credible flood warning with adequate lead-time is available and dependable. These measures have varying degrees of success, depending on the available manpower, skill required, long-term maintenance of materials and equipment, suitability for site-specific flood conditions, and having enough advanced warning. Complete evacuation of protected buildings is required, as these measures should not be considered adequate protection for occupants. Furthermore, emergency barriers are not acceptable in lieu of designed flood resistant protection for new buildings.

Sandbag walls: Unless emergency placement is planned well in advance or under the direction of trained personnel, most sandbag barriers are not constructed in accordance with proper practices, leading to leakage and failures. Because of the intensive work effort and length of time required for protection from even relatively shallow water, sandbag walls are not a reliable protection measure. To be effective, sandbags and sand should be stockpiled and checked regularly to ensure that sandbags have not deteriorated. Sandbags have some other drawbacks, including high disposal costs and their tendency to absorb pollutants from contaminated floodwaters, which necessitates disposal as hazardous waste.

Water-filled barriers: A number of vendors make water-filled barriers that can be assembled with relative ease, depending on the source of water for filling. The barriers must be specifically sized for the site. Training and annual drills are important so that personnel know how to place and deploy the barriers. Proper storage, including cleaning after deployment, is necessary to protect the materials over long periods of time.

Panels for doors: For shallow and short-duration flooding, panels of sturdy material can be made to fit doorways to minimize the entry of floodwaters. Effectiveness is increased significantly if a flexible gasket or sealant is provided, and the mounting hardware is designed to apply even pressure. Personnel must know where the materials are stored and be trained in their deployment. A number of vendors make special doors for permanent installation and drop-in panels or barriers that are designed to be watertight (see Figure 2-36).

Figure 2-36:
Example of an aluminum flood barrier used for flooding less than 3 feet deep.

SOURCE: SAVANNAH TRIMS

2.5 CHECKLIST FOR BUILDING VULNERABILITY OF FLOOD-PRONE CRITICAL FACILITIES

The Checklist for Building Vulnerability of Flood-Prone Critical Facilities (Table 2-3) is a tool that can be used to help assess site-specific flood hazards and building vulnerability. The checklist is useful during site selection, preliminary design of a new building, or when considering rehabilitation of an existing facility. In addition to examining building design issues that affect vulnerability, the checklist also helps users to examine the functionality of the critical and emergency systems upon which most critical facilities depend. The checklist is organized into separate sections, so that each section can be assigned to a subject expert for greater accuracy of the examination. The results should be integrated into a master vulnerability assessment to guide the design process and the choice of appropriate mitigation measures.

Table 2-3: Checklist for Building Vulnerability of Flood-Prone Critical Facilities

Vulnerability Sections	Guidance	Observations
Site Conditions		
Is the site located near a body of water (with or without a mapped flood hazard area)? Is the site in a flood hazard area shown on the community's map (FIRM or other adopted map)? If so, what is the flood zone?	All bodies of water are subject to flooding, but not all have been designated as a floodplain on FIRMs. Flood hazard maps usually are available for review in local planning and permit offices. Electronic versions of the FIRMs may be available online at www.fema.gov. Paper maps may be ordered by calling (800) 358-9616.	

Table 2-3: Checklist for Building Vulnerability of Flood-Prone Critical Facilities (continued)

Vulnerability Sections	Guidance	Observations
Site Conditions (continued)		
(continued) Is the site affected by a regulatory floodway ? Is the site located in a storm surge inundation zone (or tsunami inundation area)?	(continued) Development in floodways, where floodwaters typically are fast and deep, must be supported by engineering analyses. In coastal communities, even sites at some distance inland from the shoreline may be exposed to extreme storm surge flooding. Storm surge maps may be available at State or local emergency management offices.	
What is the DFE (or does an analysis have to be done to determine the DFE)? What is the minimum protection level required by regulatory authorities? Does the FIS or other study have information about the 500-year flood hazard area? Has FEMA issued post-disaster advisory flood elevations and maps? What are the expected depths of flooding at the site (determined using flood elevations and ground elevations)?	Reference the FIS for flood profiles and data tables. Site-specific analyses should be performed by qualified engineers. Check with regulatory authorities to determine the required level of protection. If a major flood event has affected the community, FEMA may have issued new flood hazard information, especially if areas not shown on the FIRMs have been affected. Sometimes these maps are adopted and replace the FIRMs; sometimes the new data are advisory only.	
Has the site been affected by past flood events? What is the flood of record?	Records of actual flooding augment studies that predict flooding, especially if historic events resulted in deeper or more widespread flooding. Information may be available from local planning, emergency management, and public works agencies, or State agencies, the U.S. Army Corps of Engineers, or the Natural Resources Conservation Service. The flood of record is often a lower probability event (with higher flood elevations) than the 100-year flood.	

Table 2-3: Checklist for Building Vulnerability of Flood-Prone Critical Facilities (continued)

Vulnerability Sections	Guidance	Observations
Site Conditions (continued)		
What is the expected velocity of floodwaters on the site? Are waves expected to affect the site?	Velocity is a factor in computing loads associated with hydrodynamic forces, including drag on building surfaces. Approximations of velocity may be interpolated from data in the FIS Floodway Data Table if the waterway was studied using detailed methods, application of approximation methods based on continuity, local observations and sources, or site-specific studies. Waves can exert considerable dynamic forces on buildings and contribute to erosion and scour. Wind-driven waves occur in areas subject to coastal flooding and where unobstructed winds affect wide floodplains (large lakes and major rivers). Standing waves may occur in riverine floodplains where high velocities are present.	
Is there information on how quickly floodwaters may affect the site? What is the expected duration of flooding?	Warning time is a key factor in the safe and orderly evacuation of critical facilities. Certain protective measures may require adequate warning so that actions can be taken by skilled personnel. Duration has bearing on the stability of earthen fills, access to a site and emergency response, and durability of materials that come into contact with water. Records of actual flooding are the best indicator of duration as most floodplain analyses do not examine duration.	
Is there a history of flood-related debris problems or erosion on the site?	Site design should account for deposition of debris and sediment, as well as the potential for erosion-related movement of the shoreline or waterway. Buildings exposed to debris impact or undermining by scour and erosion should be designed to account for these conditions.	
Is the site within an area predicted to flood if a levee or floodwall fails or is overtopped? Is the site in an area predicted to be inundated if an upstream dam were to fail?	Flood protection works may be distant from sites and not readily observable. Although a low probability event, failure or overtopping can cause unexpected and catastrophic damage because the protected lands are not regulated as flood hazard areas.	

Table 2-3: Checklist for Building Vulnerability of Flood-Prone Critical Facilities (continued)

Vulnerability Sections	Guidance	Observations
Site Conditions (continued)		
	(continued) The effects of an upstream dam failure are not shown on the FIRMs or most flood hazard maps prepared locally. Although dam failure generally is considered an unlikely event, the potential threat should be evaluated due to the catastrophic consequences. (Note: owners of certain dams should have emergency action plans geared toward notification and evacuation of vulnerable populations and critical facilities.)	
Does the surrounding topography contribute to the flooding at the site? Is there a history of local surface drainage problems due to inadequate site drainage?	If areas with poor local drainage and frequent flooding cannot be avoided, filling, regrading, and installation of storm drainage facilities may be required.	
Given the nature of anticipated flooding and soils, is scour around and under the foundation likely?	Scour-prone sites should be avoided, in part due to likely long-term maintenance requirements. Flooding that is high velocity or accompanied by waves is more likely to cause scour, especially on fills, or where local soils are unconsolidated and subject to erosion.	
Has water from other sources entered the building (i.e., high groundwater, water main breaks, sewer backup, etc.)? Is there a history of water intrusion through floor slabs or well-floor connections? Are there underground utility systems or areaways that can contribute to basement flooding? Are there stormwater sewer manholes upslope of window areas or openings that allow local drainage to enter the basement/lower floor areas?	These questions pertain to existing facilities that may be impaired by water from sources other than the primary source of flooding. The entire building envelope, including below-grade areas, should be examined to identify potential water damage.	

Table 2-3: Checklist for Building Vulnerability of Flood-Prone Critical Facilities (continued)

Vulnerability Sections	Guidance	Observations
Site Conditions (continued)		
Is at least one access road to the site/building passable during flood events? Are at-grade parking lots located in flood-prone areas? Are below-grade parking areas susceptible to flooding?	Access is increasingly important as the duration of flooding increases. For the safety of occupants, most critical facilities should not be occupied during flood events. Areas where vehicles could be affected should have signage to warn users of the risk. Emergency response plans should include notification of car owners.	
Are any portions of the building below the Design Flood Elevation? Has the building been damaged in previous floods?	For existing buildings, it is important to determine which portions are vulnerable in order to evaluate floodproofing options. If flood depths are expected to exceed 2 or 3 feet, dry floodproofing may not be feasible. Alternatives include modifying the use of flood-prone areas.	
Are any building spaces below-grade (basements)?	Below-grade spaces and their contents are most vulnerable to flooding and local drainage problems. Rapid pump out of below-grade spaces can unbalance forces if the surrounding soil is saturated, leading to structural failure. If below-grade spaces are intended to be dry floodproofed, the design must account for buoyant forces.	
Are any critical building functions occupying space that is below the elevation of the 500-year flood or the Design Flood Elevation? Can critical functions be relocated to upper levels that are above predicted flood elevations? If critical functions cannot be relocated, is floodproofing feasible? If critical functions must continue during a flood event, have power, supplies, and access issues been addressed?	New critical facilities built in flood hazard areas should not have any functions occupying flood-prone spaces (other than parking, building access and limited storage). Existing facilities in floodplains should be examined carefully to identify the best options for protecting functionality and the structure itself.	

Table 2-3: Checklist for Building Vulnerability of Flood-Prone Critical Facilities (continued)

Vulnerability Sections	Guidance	Observations
Site Conditions (continued)		
Have critical contents (files, computers, servers, equipment, research, and data) been located on levels of the facility above the flood elevations? Are critical records maintained off-site?	For existing facilities that are already located in flood hazard areas, the nature of the facility may require continued use of flood-prone space. However, the potential for flooding should be recognized and steps taken to minimize loss of expensive equipment and irreplaceable data. If critical contents cannot be permanently located on higher floors, a flood response plan should take into account the time and attention needed to move such contents safely.	
Building Envelope		
Are there existing floodproofing measures in place below the expected flood elevation? What is the nature of these measures and what condition are they in? Is there an annual inspection and maintenance plan? Is there an "action plan" to implement floodproofing measures when flooding is predicted? Do the building operators/occupants know what to do when a flood warning is issued?	Floodproofing measures are only as good as the design and their condition, especially if many years have passed since initial installation. Floodproofing measures that require human intervention are entirely dependent on the adequacy of advance warning, and the availability and ability of personnel to properly install the measures.	
For existing buildings, what types of openings penetrate the building envelope below the 500-year flood elevation or the DFE (doors, windows, cracks, vent openings, plumbing fixtures, floor drains, etc.)?	For dry floodproofing to be effective, every opening must be identified and measures taken to permanently seal or to prepare special barriers to resist infiltration. Sewage backflow can enter through unprotected plumbing fixtures.	
Are flood-resistant materials used for structural and nonstructural components and finishes below the 500-year elevation or the DFE?	Flood-resistant materials are capable of withstanding direct and prolonged contact with floodwaters without sustaining damage that requires more than cosmetic repair. Contact is considered to be prolonged if it is 72 hours or longer in freshwater flooding areas, or 12 hours or longer in areas subject to coastal flooding.	

Table 2-3: Checklist for Building Vulnerability of Flood-Prone Critical Facilities (continued)

Vulnerability Sections	Guidance	Observations
Utility Systems		
Is the potable water supply for the facility protected from flooding? If served by a well, is the wellhead protected?	Operators of critical facilities that depend on fresh water for continued functionality should learn about the vulnerability of the local water supply system and the system's plans for recovery of service in the event of a flood.	
Is the wastewater service for the building protected from flooding? Are any manholes below the DFE? Is infiltration of floodwaters into sewer lines a problem? If the site is served by an onsite system that is located in a flood-prone area, have backflow valves been installed?	Most waste lines exit buildings at the lowest elevation. Even buildings that are outside of the floodplain can be affected by sewage backups during floods.	
Are there any aboveground or underground tanks on the site in flood hazard areas? Are they installed and anchored to resist flotation during the design flood? Are tank openings and vents elevated above the 500-year elevation or the DFE, or otherwise protected to prevent entry of floodwater or exit of product during a flood event?	Dislodged tanks become floating debris that pose special hazards during recovery. Lost product causes environmental damage. Functionality may be impaired if tanks for heating fuel, propane, or fuel for emergency generators are lost or damaged.	
Mechanical Systems		
Are air handlers, HVAC systems, ductwork, and other mechanical equipment and systems located above the 500-year elevation or the DFE? Are the vents and inlets located above flood level, or sealed to prevent entry of floodwater?	In existing buildings, utility equipment that is critical for functionality should be relocated to higher floors or into elevated additions.	

Table 2-3: Checklist for Building Vulnerability of Flood-Prone Critical Facilities (continued)

Vulnerability Sections	Guidance	Observations
Plumbing and Gas Systems		
Are plumbing fixtures and gas-fired equipment (meters, pilot light devices/burners, etc.) located above the 500-year elevation or the DFE?	In existing buildings, utility equipment that is critical for functionality should be relocated to higher floors or into elevated additions.	
Is plumbing and gas piping that extends below flood levels installed to minimize damage?	Piping that is exposed could be impacted by debris.	
Electrical Systems		
Are electrical systems, including backup power generators, panels, and primary service equipment, located above the 500-year elevation or the DFE? Are pieces of electrical stand-by equipment and generators equipped with circuits to turn off power? Are the switches and wiring required for safety (minimal lighting, door openers) located below the flood level designed for use in damp locations?	In existing buildings, utility equipment that is critical for functionality should be relocated to higher floors or into elevated additions.	
Fire Alarm Systems		
Is the fire alarm system located above the 500-year elevation or the DFE?	In existing buildings, utility equipment that is critical for functionality should be relocated to higher floors or into elevated additions.	
Communications and IT Systems		
Are the communication/IT systems located above the 500-year elevation or the DFE?		

Table 2-3: Checklist for Building Vulnerability of Flood-Prone Critical Facilities (continued)

Vulnerability Sections	Guidance	Observations
Structural		
What is the construction type and the foundation type and what is the load bearing capacity? Has the foundation been designed to resist hydrostatic and hydrodynamic flood loads? If the building has below-grade areas, are the lower floor slabs subject to cracking and uplift?	If siting in a floodplain is unavoidable, new facilities are to be designed to account for all loads and load combinations, including flood loads. Building spaces below the design flood level can be dry floodproofed, although it must be recognized that higher flood levels will overtop the protection measures and may result in severe damage. Dry floodproofing creates large unbalanced forces that can jeopardize walls and foundations that are not designed to resist the hydrostatic and hydrodynamic loads.	
If the building is elevated on a crawlspace an open foundation, are there any enclosed areas?	New buildings may have enclosures below the flood elevation provided the use of the enclosures is limited (crawlspace, parking, building access, limited storage). In addition, the enclosures must have flood openings to automatically allow for inflow and outflow of floodwaters to minimize differential hydrostatic pressure. Existing buildings that are elevated and have enclosures below the flood elevation can be retrofit with flood openings.	
For an existing building with high value uses below the flood elevation, is the building suitable for elevation-in-place or can it be relocated to higher ground?	Elevating a building provides better protection than dry floodproofing. Depending on the type and soundness of the foundation, even large buildings can be elevated on a new foundation or moved to a site outside of the floodplain.	

2.6 REFERENCES AND SOURCES OF ADDITIONAL INFORMATION

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American Society of Civil Engineers, Structural Engineering Institute, *Minimum Design Loads for Buildings and Other Structures*, ASCE/SEI 7-05, Reston, VA, 2005.

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- *User's Guide to Technical Bulletins*, FIA-TB-0, April 1993.
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U.S. Army Corps of Engineers, *Flood Proofing Programs, Techniques and References,* Washington, DC, 1996.

U.S. Army Corps of Engineers, *Flood Proofing Performance—Successes & Failures*, Washington, DC, 1998.

Organizations and Agencies:

FEMA: FEMA's regional offices (www.fema.gov) can be contacted for advice and guidance on NFIP mapping and regulations.

NFIP State Coordinating offices help local governments to meet their floodplain management obligations, and may provide technical advice to others; the offices are listed by the Association of State Floodplain Managers, Inc. (www.floods.org/stcoor.htm).

State departments of education or agencies that coordinate State funding and guidelines for critical facilities may have State-specific requirements.

U.S. Army Corps of Engineers: District offices offer Flood Plain Management Services (www.usace.army.mil/inet/functions/cw/).

FEMA publications may be obtained at no cost by calling (800) 480-2520, faxing a request to (301) 497-6378, or downloaded from the library/publications section online at http://www.fema.gov.

3.1 GENERAL DESIGN CONSIDERATIONS

Wind with sufficient speed to cause damage to weak critical facilities can occur anywhere in the United States and its territories. Even a well-designed, constructed, and maintained critical facility may be damaged in a wind event much stronger than one the building was designed for. However, except for tornado damage, this scenario is a rare occurrence. Rather, most damage occurs because various building elements have limited wind resistance due to inadequate design, poor installation, or material deterioration. Although the magnitude and frequency of strong windstorms vary by locale, all critical facilities should be designed, constructed, and maintained to minimize wind damage (other than that associated with tornadoes—see Section 3.5).

This chapter discusses structural, building envelope, and nonstructural building systems, and illustrates various types of wind-induced damage that affect them. Numerous examples of best practices pertaining to new and existing critical facilities are presented as recommended design guidelines. Incorporating those practices applicable to specific projects will result in greater wind resistance reliability and will, therefore, decrease expenditures for repair of wind-damaged facilities, provide enhanced protection for occupants, and avoid disruption of critical services.

1. The U.S. territories include American Samoa, Guam, Northern Mariana Islands, Puerto Rico, and the U.S. Virgin Islands. ASCE provides basic wind speed criteria for all but Northern Mariana Islands.

3.1.1 NATURE OF HIGH WINDS

A variety of windstorm types occur in different areas of the United States. The characteristics of the types of storms that can affect the site should be considered by the design team. The primary storm types are listed below.

Straight-line wind: This type of wind generally blows in a straight line and is the most common. Straight-line wind speeds range from very low to very high. High winds associated with intense low pressure can last for approximately a day at a given location. Straight-line winds occur throughout the United States and its territories.

Down-slope wind: Wind blowing down the slope of mountains is referred to as down-slope wind. Down-slope winds with very high speeds frequently occur in Alaska and Colorado. In the continental United States, mountainous areas are referred to as "special wind regions" (see Figure 3-1). Neither ASCE 7 nor model building codes provide specific wind speeds in special wind regions. ASCE 7 does provide guidance on how to determine design wind speeds in these regions. If the local building department has not established the basic speed, use of regional climatic data and consultation with a wind engineer or meteorologist is advised.

ASCE 7, *Minimum Design Loads for Buildings and Other Structures*, provides guidance for determining wind loads on buildings. The IBC and NFPA 5000 refer to ASCE 7 for wind load determination.

Thunderstorm: This type of storm can form rapidly and produce high wind speeds. Approximately 10,000 severe thunderstorms occur in the United States each year, typically in the spring and summer. They are most common in the Southeast and Midwest. Besides producing high winds, they often create heavy rain and sometimes spawn tornadoes and hail storms. Thunderstorms commonly move through an area quite rapidly, causing high winds for only a few minutes at a given location. However, thunderstorms can also stall and become virtually stationary.

Downburst: Also known as a microburst, this is a powerful downdraft associated with a thunderstorm. When the downdraft reaches the ground, it spreads out horizontally, and may form one or more horizontal vortex rings around the downdraft. The out-

flow is typically 6,000 to 12,000 feet across, and the vortex ring may rise 2,000 feet above the ground. The lifecycle of a downburst is usually 15 to 20 minutes. Observations suggest that approximately 5 percent of all thunderstorms produce a downburst, which can result in significant damage in a localized area.

Northeaster (nor'easter): A northeaster is a cyclonic storm occurring off the east coast of North America. These winter weather events are notorious for producing heavy snow, rain, and high waves and wind. A nor'easter gets its name from the continuously strong northeasterly winds blowing in from the ocean ahead of the storm and over the coastal areas. These storms may last for several days.

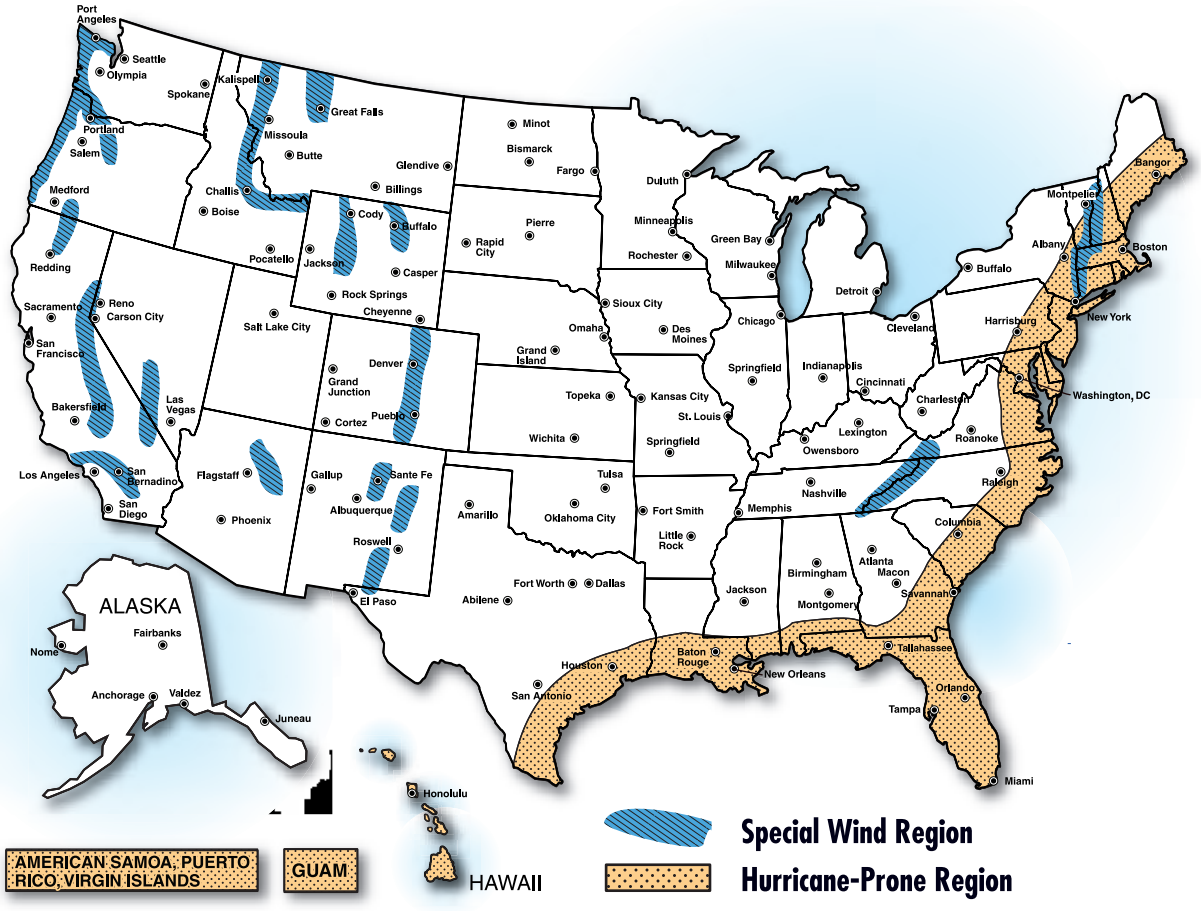


Figure 3-1: Hurricane-prone regions and special wind regions

SOURCE: ADAPTED FROM ASCE 7-05

Hurricane: This is a system of spiraling winds converging with increasing speed toward the storm's center (the eye of the hurricane). Hurricanes form over warm ocean waters. The diameter of the storm varies and can be between 50 and 600 miles. A hurricane's forward movement (translational speed) can vary between approximately 5 miles per hour (mph) to more than 25 mph. Besides being capable of delivering extremely strong winds for several hours and moderately strong winds for a day or more, many hurricanes also bring very heavy rainfall. Hurricanes also occasionally spawn tornadoes. The Saffir-Simpson Hurricane Scale (see Table 3-1) categorizes the intensity of hurricanes. The five-step scale ranges from Category 1 (the weakest) to Category 5 (the strongest). Hurricane-prone regions are defined in Section 3.1.3.

The Saffir-Simpson Hurricane Scale is based on measurements of sustained wind speeds in hurricanes; these measurements are taken over open water. The wind speeds described in the Saffir-Simpson Hurricane Scale are used to prepare storm response actions and are not intended to be used for building design. Design wind loads on buildings should be determined using the basic wind speeds given in ASCE 7.

Of all the storm types, hurricanes have the greatest potential for devastating a large geographical area and, hence, affect the greatest number of people. The terms "hurricane," "cyclone," and "typhoon" describe the same type of storm. The term used depends on the region of the world where the storm occurs. See Figure 3-1 for hurricane-prone regions.

Table 3-1: Saffir-Simpson Hurricane Scale

Strength	Sustained Wind Speed (mph)*	Gust Wind Speed (mph)**	Pressure (millibar)
Category 1	74-95	89-116	>980
Category 2	96-110	117-134	965-979
Category 3	111-130	135-159	945-964
Category 4	131-155	160-189	920-944
Category 5	>155	>189	<920

* 1-minute sustained over open water
** 3-second gust over open water

Tornado: This is a violently rotating column of air extending from the base of a thunderstorm to the ground. The Fujita scale categorizes tornado severity based on observed damage. The six-step scale ranges from F0 (light damage) to F5 (incredible damage). Weak tornadoes (F0 and F1) are most common, but strong tornadoes (F2 and F3) frequently occur. Violent tornadoes (F4 and F5) are rare. Tornado path widths are typically less than 1,000 feet; however, widths of approximately 2.5 miles have been reported. Wind speed rapidly decreases with increased distance from the center of a tornado. A critical facility on the periphery of a strong or violent tornado could be subjected to moderate to high wind speeds, depending upon the distance from the core of the tornado. However, even though the wind speed at a given facility might not be great, a facility on the periphery could still be impacted by many large pieces of wind-borne debris. Tornadoes are responsible for the greatest number of wind-related deaths each year in the United States. Figure 3-2 shows the frequency of tornado occurrence for a period between 1950 and 1998, and Figure 3-3 shows the design wind speeds recommended by FEMA for designing community shelters.

The majority of the tornadoes spawned during hurricanes are classified as F2 or weaker. However, a few F3 and at least two F4 tornadoes have been reported.

Beginning in February 2007, the National Weather Service will use the Enhanced Fujita Scale (EF-Scale) to categorize tornado severity. This new scale has six steps, ranging from EF0 to EF5. The new scale was developed by Texas Tech University's Wind Science and Engineering Center. See www.spc.noaa.gov/efscale/ for further information on the EF-Scale.

3.1.2 PROBABILITY OF OCCURRENCE

Via the importance factor,[2] ASCE 7 requires Category III and IV buildings to be designed for higher wind loads than Category I and II buildings (see Section 1.1.1). Hence, critical facilities designed in accordance with ASCE 7 have greater resistance to stronger, rarer storms. When designing a critical facility, design professionals should consider the following types of winds.

Routine winds: In many locations, winds with low to moderate speeds occur daily. Damage is not expected to occur during these events.

2. The importance factor accounts for the degree of hazard to human life and damage to property. Importance factors are given in ASCE 7.

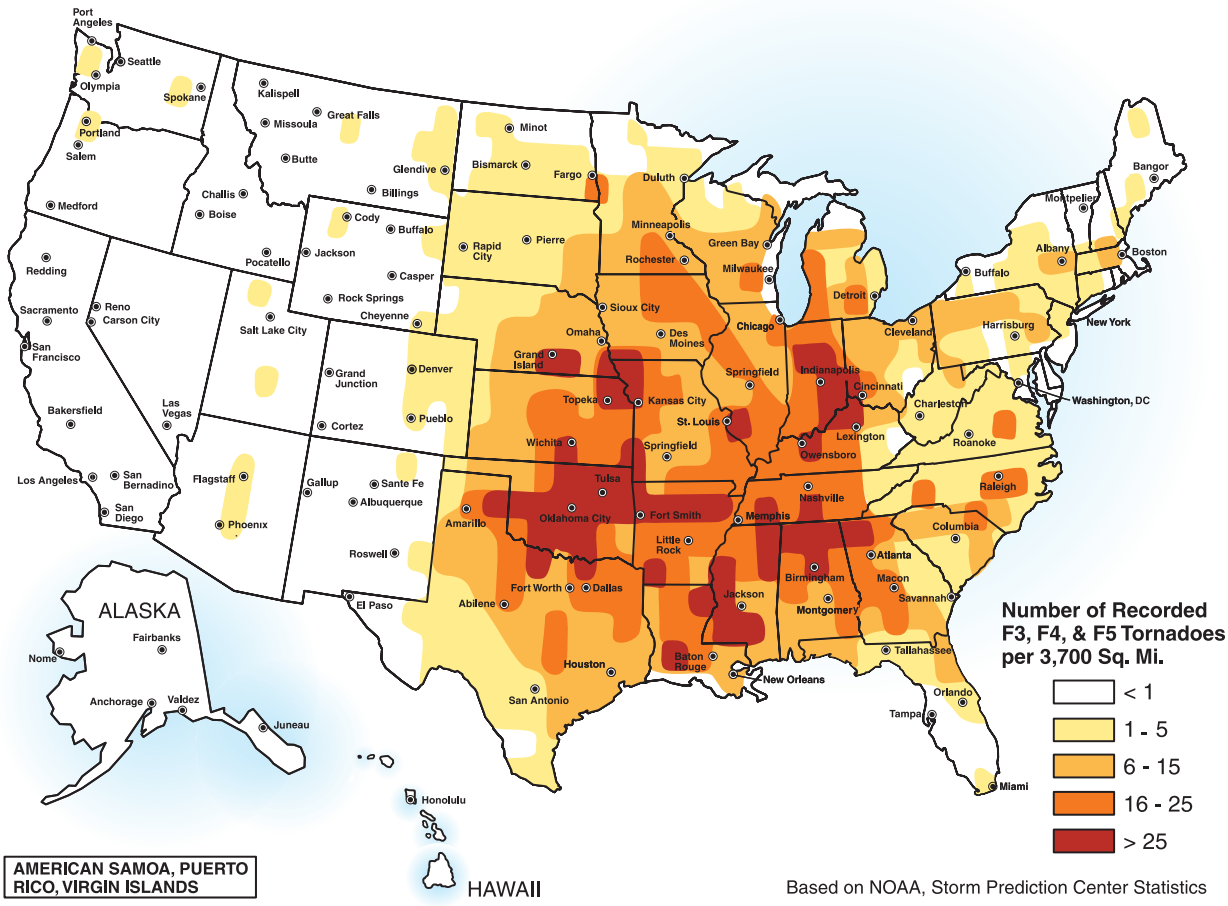


Figure 3-2: Frequency of recorded F3, F4, and F5 tornadoes (1950-1998)

Stronger winds: At a given site, stronger winds (i.e., winds with a speed in the range of 70 to 80 mph peak gust, measured at 33 feet in Exposure C — refer to Section 3.1.3) may occur from several times a year to only once a year or even less frequently. This is the threshold at which damage normally begins to occur to building elements that have limited wind resistance due to problems associated with inadequate design, insufficient strength, poor installation, or material deterioration.

Design level winds: Critical facilities exposed to design level events and events that are somewhat in excess of design level should experience little, if any damage. Actual storm history, however, has shown that design level storms frequently cause extensive building envelope damage. Structural damage also occurs, but less

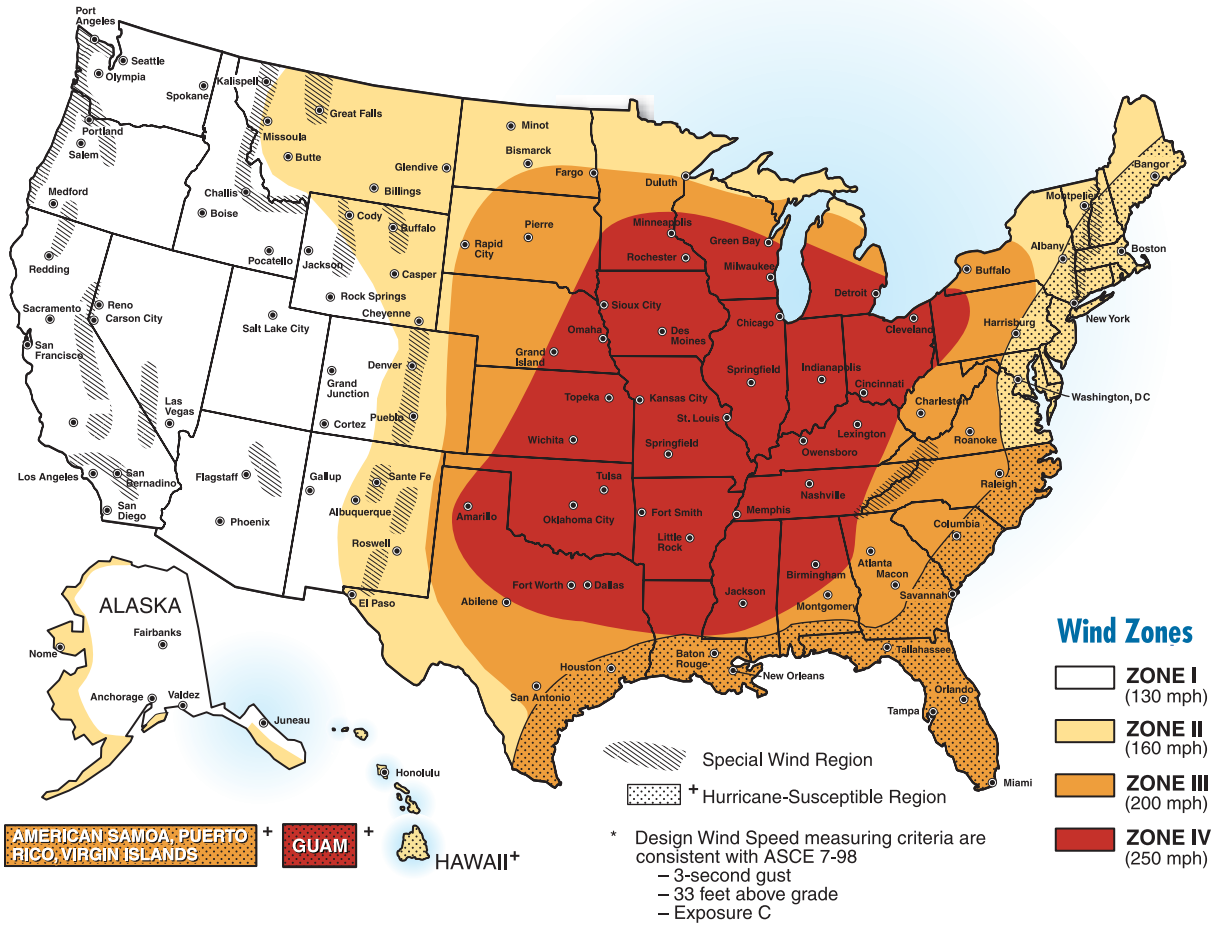


Figure 3-3: Design wind speeds for community shelters

frequently. Damage incurred in design level events is typically associated with inadequate design, poor installation, or material deterioration. The exceptions are wind-driven water infiltration and wind-borne debris (missiles) damage. Water infiltration is discussed in Sections 3.3.3.1, 3.3.3.2, and 3.3.3.3.

Tornadoes: Although more than 1,200 tornadoes typically occur each year in the United States, the probability of a tornado occurring at any given location is quite small. The probability of occurrence is a function of location. As shown in Figure 3-2, only a few areas of the country frequently experience tornadoes, and tornadoes are very rare in the west. The Oklahoma City area is the most active location, with 112 recorded tornadoes between 1890 and 2003 (www.spc.noaa.gov/faq/tornado/#History).

Missile damage is very common during hurricanes and tornadoes. Missiles can puncture roof coverings, many types of exterior walls, and glazing. The IBC does not address missile-induced damage except for glazing in wind-borne debris regions. (Wind-borne debris regions are limited to portions of hurricane-prone regions.) In hurricane-prone regions, significant missile-induced building damage should be expected, even during design level hurricane events, unless special enhancements are incorporated into the building's design (discussed in Section 3.4).

Well designed, constructed, and maintained critical facilities should experience little if any damage from weak tornadoes, except for window breakage. However, weak tornadoes often cause building envelope damage because many critical facilities have wind resistance deficiencies. Most critical facilities experience significant damage if they are in the path of a strong or violent tornado because they typically are not designed for this type of storm.

3.1.3 WIND/BUILDING INTERACTIONS

When wind interacts with a building, both positive and negative (i.e., suction) pressures occur simultaneously (see Figure 3-4). Critical facilities must have sufficient strength to resist the applied loads from these pressures to prevent wind-induced building failure. Loads exerted on the building envelope are transferred to the structural system, where in turn they must be transferred through the foundation into the ground. The magnitude of the pressures is a function of the following primary factors.

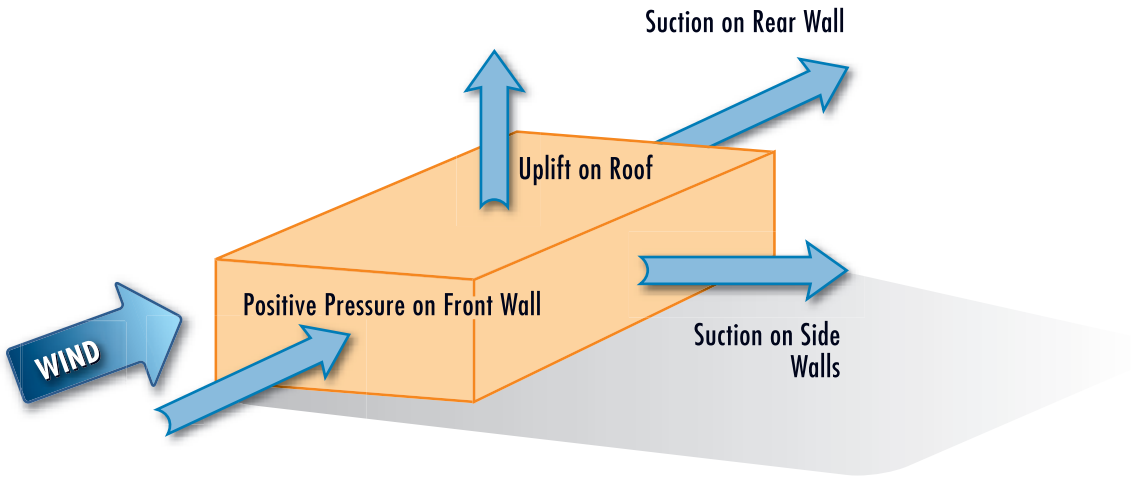


Figure 3-4:
Schematic of wind-induced pressures on a building

Exposure: The characteristics of the terrain (i.e., ground roughness and surface irregularities in the vicinity of a building) influence the wind loading. ASCE 7 defines three exposure categories, Exposures B, C, and D. Exposure B is the roughest terrain category and Exposure D is the smoothest. Exposure B includes urban, suburban, and wooded areas. Exposure C includes flat open terrain with scattered obstructions and areas adjacent to water surfaces in hurricane-prone regions (which are defined below under "basic wind speed"). Exposure D includes areas adjacent to water surfaces outside hurricane-prone regions, mud flats, salt flats, and unbroken ice. Because of the wave conditions generated by hurricanes, areas adjacent to water surfaces in hurricane-prone regions are considered to be Exposure C rather than the smoother Exposure D. The smoother the terrain, the greater the wind pressure; therefore, critical buildings located in Exposure C would receive higher wind loads than those located in Exposure B, even at the same basic wind speed.

For additional exposure information, see the *Commentary* of ASCE 7, which includes several aerial photographs that illustrate the different terrain conditions associated with Exposures B, C, and D.

Basic wind speed: ASCE 7 specifies the basic wind speed for determining design wind loads. The basic wind speed is measured at 33 feet above grade in Exposure C (flat open terrain). If the building is located in Exposure B or D, rather than C, an adjustment for the actual exposure is made in the ASCE 7 calculation procedure.

Since the 1995 edition of ASCE 7, the basic wind speed measurement has been a 3-second peak gust speed. Prior to that time, the basic wind speed was a fastest-mile speed (i.e., the speed averaged over the time required for a mile-long column of air to pass a fixed point).[3] Most of the United States has a basic wind speed (peak gust) of 90 mph, but much higher speeds occur in Alaska and in hurricane-prone regions. The highest speed, 170 mph, occurs in Guam.

Hurricane-prone regions include Atlantic and Gulf coastal areas (where the basic wind speed is greater than 90 mph), Hawaii, and the U.S. territories in the Caribbean and South Pacific (see Figure 3-1).

3. Peak gust speeds are about 15 to 20 mph higher than fastest-mile speeds (e.g., a 90-mph peak basic wind speed is equivalent to a 76-mph fastest-mile wind speed). IBC Chapter 16 provides a table of equivalent basic wind speeds.

The MWFRS is an assemblage of structural elements assigned to provide support and stability for the overall structure. The system generally receives wind loading from more than one surface. The C&C are elements of the building envelope that do not qualify as part of the main wind-force resisting system.

In the formula for determining wind pressures, the basic wind speed is squared. Therefore, as the wind speed increases, the pressures are exponentially increased, as illustrated in Figure 3-5. This figure also illustrates the relative difference in pressures exerted on the main wind-force resisting system (MWFRS) and the components and cladding (C&C) elements.

Topography: Abrupt changes in topography, such as isolated hills, ridges, and escarpments, cause wind to speed up. Therefore, a building located near a ridge would receive higher wind pressures than a building located on relatively flat land. ASCE 7 provides a procedure to account for topographic influences.

Building height: Wind speed increases with height above the ground. Taller buildings are exposed to higher wind speeds and greater wind pressures. ASCE 7 provides a procedure to account for building height.

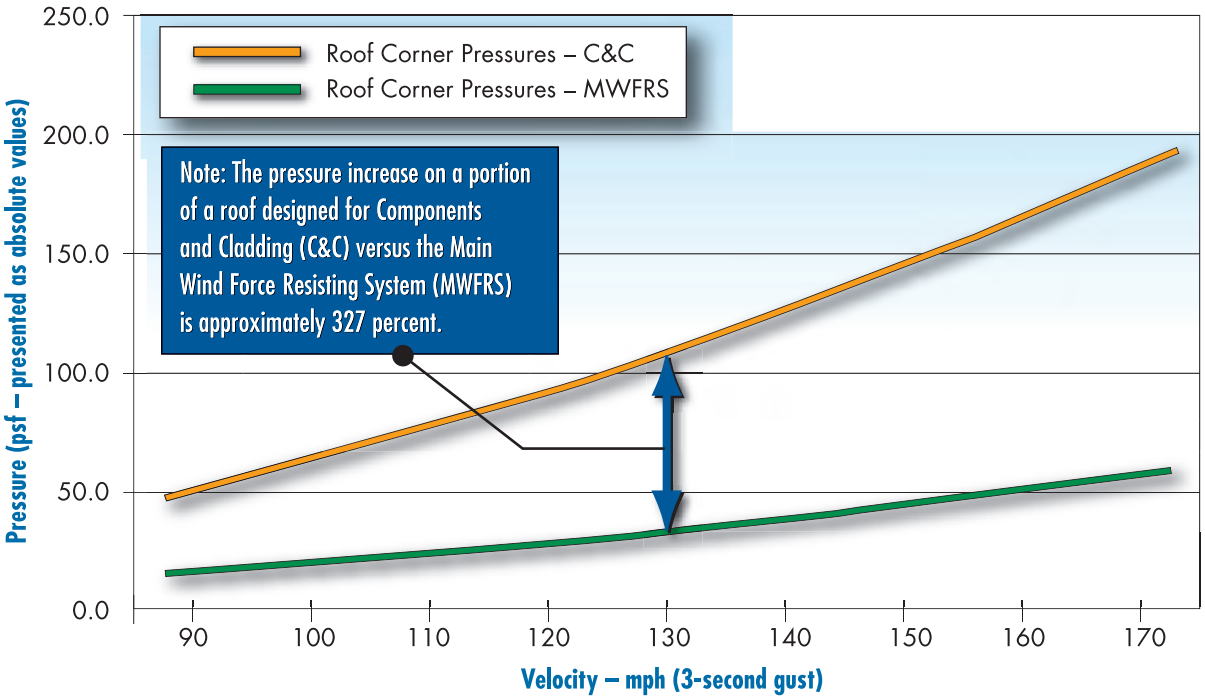


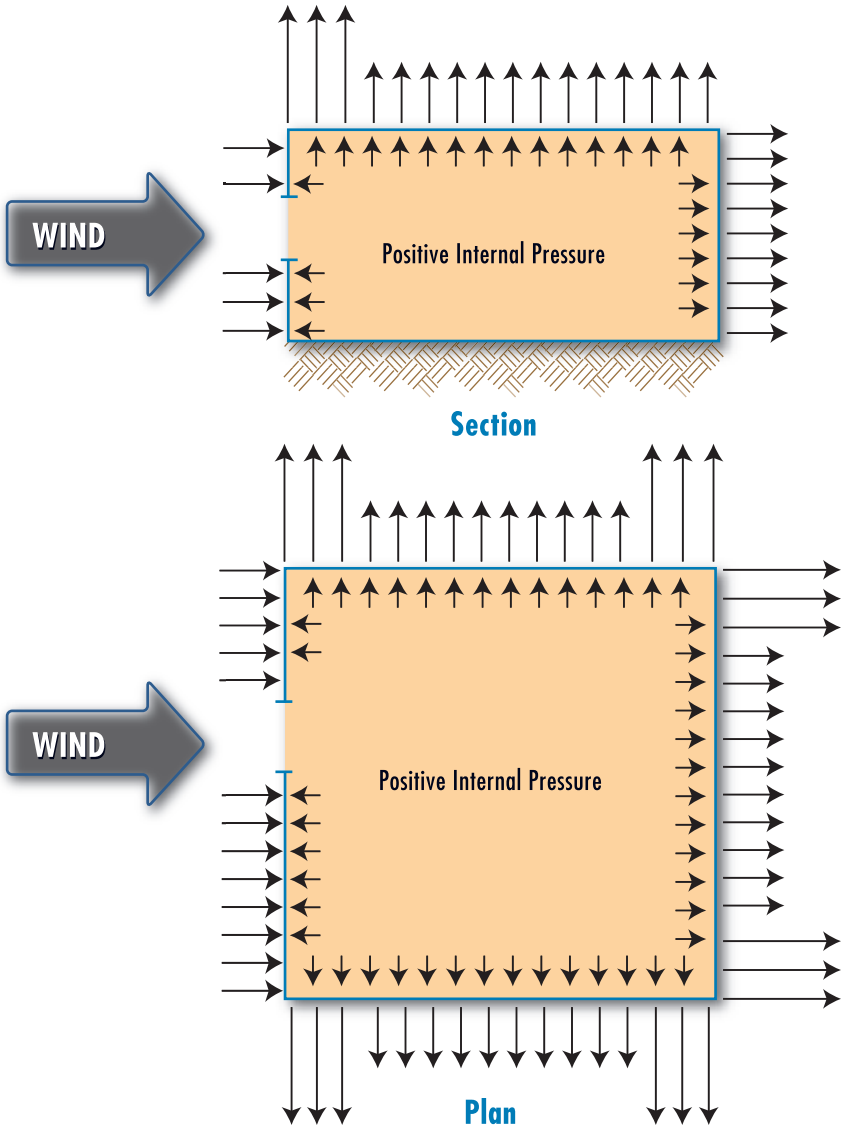
Figure 3-5: Wind pressure as a function of wind speed

Internal pressure (building pressurization/depressurization): Openings through the building envelope, in combination with wind interacting with a building, can cause either an increase in the pressure within the building (i.e., positive internal pressure), or it can cause a decrease in the pressure (i.e., negative internal pressure). Building envelope openings occur around doors and window frames, and by air infiltration through walls that are not absolutely airtight. A door or window left open, or glazing that is broken during a storm, can greatly influence the magnitude of the internal pressure.

Wind striking an exterior wall exerts a positive pressure on the wall, which forces air through openings and into the interior of the building (this is analogous to blowing up a balloon). At the same time that the windward wall is receiving positive pressure, the side and rear walls are experiencing negative (suction) pressure from winds going around the building. As this occurs, air within the building is pulled out at openings in these walls. As a result, if the porosity of the windward wall is greater than the combined porosity of the side and rear walls, the interior of the building is pressurized. But if the porosity of the windward wall is lower than the combined porosity of the side and rear walls, the interior of the building is depressurized (this is analogous to letting air out of a balloon).

When a building is pressurized, the internal pressure pushes up on the roof. This push from below the roof is combined with suction on the roof above, resulting in an increased upward wind pressure on the roof. The internal pressure also pushes on the side and rear walls. This outward push is combined with the suction on the exterior side of these walls (see Figure 3-6). When a building becomes fully pressurized (e.g., due to window breakage or soffit failure), the loads applied to the exterior walls and roof are significantly increased. The rapid build-up of internal pressure can also blow down interior partitions and blow suspended ceiling boards out of their support grid. The breaching of a small window can be sufficient to cause full pressurization of the facility's interior.

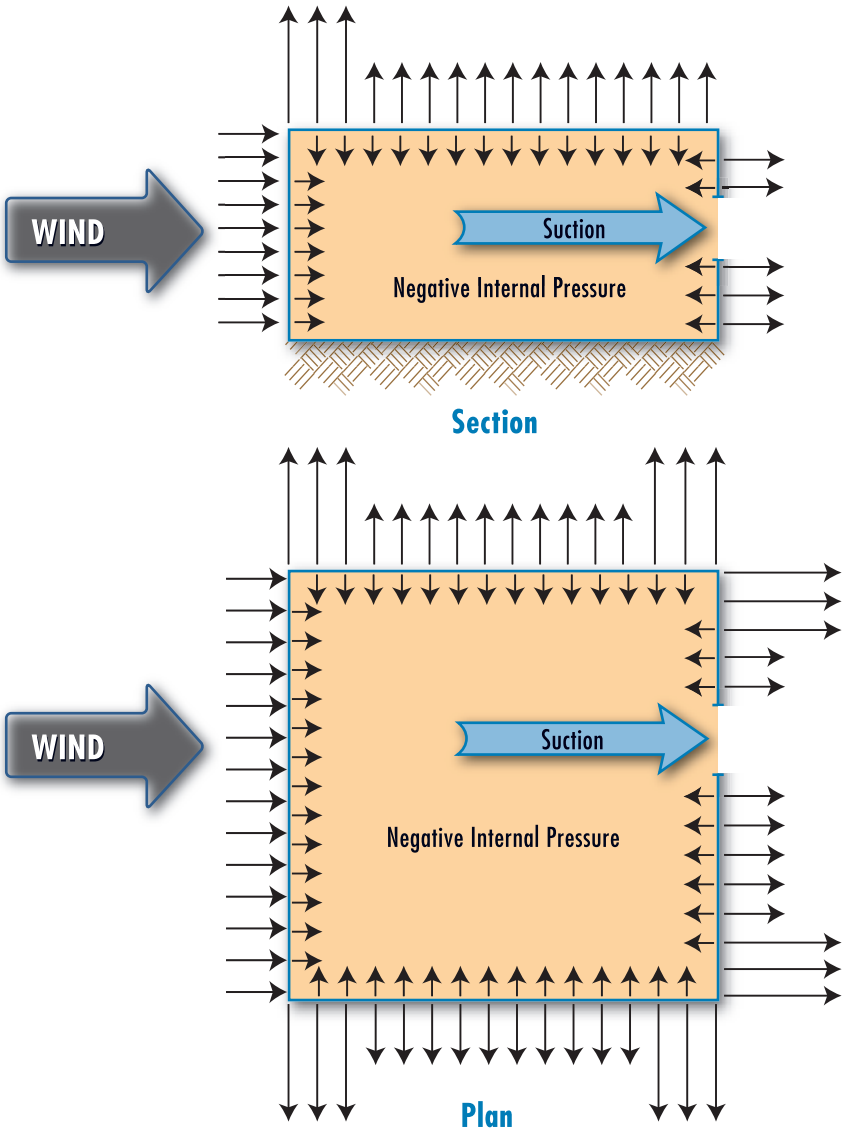
Figure 3-6:
Schematic of internal pressure condition when the dominant opening is in the windward wall



NOTE: Arrows indicate direction and magnitude of applied force.

When a building is depressurized, the internal pressure pulls the roof down, which reduces the amount of uplift exerted on the roof. The decreased internal pressure also pulls inward on the windward wall, which increases the wind load on that wall (see Figure 3-7).

The ASCE 7 wind pressure design procedure accounts for the influence of internal pressure on the wall and roof loads, and it provides positive and negative internal pressure coefficients for



NOTE: Arrows indicate direction and magnitude of applied force.

Figure 3-7:
Schematic of internal pressure condition when the dominant opening is in the leeward wall

use in load calculations. Buildings that are designed to accommodate full pressurization are referred to as partially enclosed buildings. Buildings that are only intended to experience limited internal pressurization are referred to as enclosed buildings. Buildings that do not experience internal pressurization are referred to as open buildings (such as covered walkways and most parking garages).

Building shape: The highest uplift pressures occur at roof corners because of building aerodynamics (i.e., the interaction between the wind and the building). The roof perimeter has a somewhat lower load compared to the corners, and the field of the roof has still lower loads. Exterior walls typically have lower loads than the roof. The ends (edges) of walls have higher suction loads than the portion of wall between the ends. However, when the wall is loaded with positive pressure, the entire wall is uniformly loaded. Figure 3-8 illustrates these aerodynamic influences. The negative values shown in Figure 3-8 indicate suction pressure acting upward from the roof surface and outward from the wall surface. Positive values indicate positive pressure acting inward on the wall surface.

Aerodynamic influences are accounted for by using external pressure coefficients in load calculations. The value of the coefficient is a function of the location on the building (e.g., roof corner or field of roof) and building shape as discussed below. Positive coefficients represent a positive (inward-acting) pressure, and negative coefficients represent negative (outward-acting [suction]) pressure. External pressure coefficients for MWFRS and C&C are listed in ASCE 7.

Building shape affects the value of pressure coefficients and, therefore, the loads applied to the various building surfaces. For example, the uplift loads on a low-slope roof are larger than the loads on a gable or hip roof. The steeper the slope, the lower the uplift load. Pressure coefficients for monoslope (shed) roofs, sawtooth roofs, and domes are all different from those for low-slope and gable/hip roofs.

Building irregularities, such as re-entrant corners, bay window projections, a stair tower projecting out from the main wall, dormers, and chimneys can cause localized turbulence. Turbulence causes wind speed-up, which increases the wind loads in the vicinity of the building irregularity, as shown in Figures 3-9 and 3-10. Figure 3-9 shows the aggregate ballast on a hospital's single-ply membrane roof blown away at the re-entrant corner and in the vicinity of the corners of the wall projections at the window bays. The irregular wall surface created turbulence, which led to wind speed-up and loss of aggregate in the turbulent flow areas.

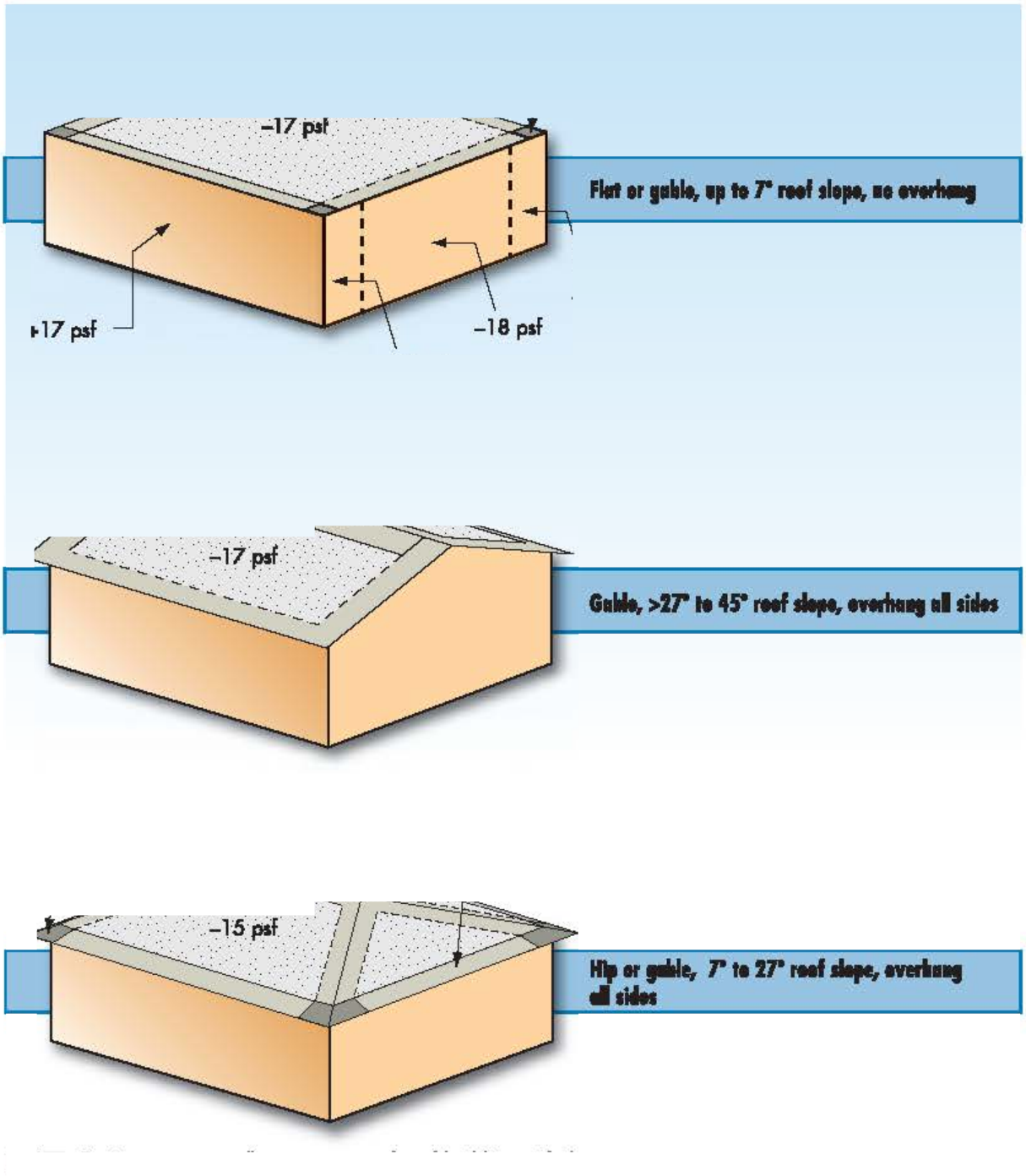


Figure 3-8: Relative roof uplift pressures as a function of roof geometry, roof slope, and location on roof, and relative positive and negative wall pressures as a function of location along the wall

Figure 3-9: Aggregate blow-off associated with building irregularities. Hurricane Hugo (South Carolina, 1989)

Information pertaining to load calculations is presented in Section 3.3.1.2. For further general information on the nature of wind and wind-building interactions, see *Buildings at Risk: Wind Design Basics for Practicing Architects*, American Institute of Architects, 1997.

Figure 3-10 shows a stair tower at a hospital that caused turbulence resulting in wind speed-up. The speed-up increased the suction pressure on the base flashing along the parapet behind the stair tower. The built-up roof's base flashing was pulled out from underneath the coping because its attachment was insufficient to resist the suction pressure. The base flashing failure propagated and caused a large area of the roof membrane to lift and peel. Some of the wall covering on the stair tower was also blown away. Had the stair tower not existed, the built-up roof would likely not have been damaged. To avoid damage in the vicinity of building irregularities, attention needs to be given to the attachment of building elements located in turbulent flow areas.

To avoid the roof membrane damage shown in Figure 3-10, it would be prudent to use corner uplift loads in lieu of perimeter uplift loads in the vicinity of the stair tower, as illustrated in Figure 3-11. Wind load increases due to building irregularities can be identified by wind tunnel studies; however, wind tunnel studies are rarely performed for critical facilities. Therefore, identification of wind load increases due to building irregu-

larities will normally be based on the designer's professional judgment. Usually load increases will only need to be applied to the building envelope, and not to the MWFRS.

Figure 3-10:
The irregularity created by the stair tower (covered with a metal roof) caused turbulence resulting in wind speed-up and roof damage. Hurricane Andrew (Florida, 1992)

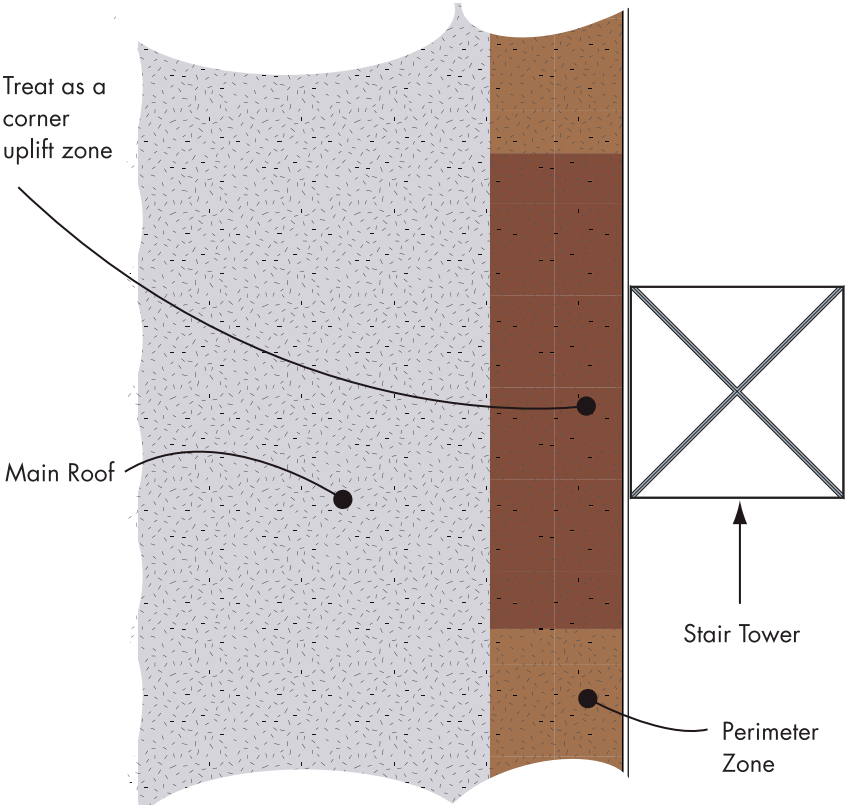


Figure 3-11:
Plan view of a portion of the building in Figure 3-10 showing the use of a corner uplift zone in lieu of a perimeter uplift zone on the low-slope roof in the vicinity of the stair tower

3.1.4 BUILDING CODES

The IBC is the most extensively used model code. However, in some jurisdictions NFPA 5000 may be used. In other jurisdictions, one of the earlier model building codes, or a specially written State or local building code, may be used. The specific scope and/or effectiveness and limitations of these other building codes will be somewhat different than those of the IBC. It is incumbent upon the design professionals to be aware of the specific code (including the edition of the code and local amendments) that has been adopted by the authority having jurisdiction over the location of the critical facility.

3.1.4.1 Scope of Building Codes

With respect to wind performance, the scope of the model building codes has greatly expanded since the mid-1980s. Some of the most significant improvements are discussed below.

Recognition of increased uplift loads at the roof perimeter and corners: Prior to the 1982 edition of the Standard Building Code (SBC), Uniform Building Code (UBC), and the 1987 edition of the National Building Code (NBC), these model codes did not account for the increased uplift at the roof perimeter and corners. Therefore, critical facilities designed in accordance with earlier editions of these codes are very susceptible to blow-off of the roof deck and/or roof covering.

Adoption of ASCE 7 for design wind loads: Although the SBC, UBC, and NBC permitted use of ASCE 7, the 2000 edition of the IBC was the first model code to require ASCE 7 for determining wind design loads. ASCE 7 has been more reflective of the current state of the knowledge than the earlier model codes, and use of this procedure typically has resulted in higher design loads.

Roof coverings: Several performance and prescriptive requirements pertaining to wind resistance of roof coverings have been incorporated into the model codes. The majority of these additional provisions were added after Hurricanes Hugo (1989) and Andrew (1992). Poor performance of roof coverings was wide-

spread in both of those storms. Prior to the 1991 edition of the SBC and UBC, and the 1990 edition of the NBC, these model codes were essentially silent on roof covering wind loads and test methods for determining uplift resistance. Code improvements continued to be made through the 2006 edition of the IBC, which added a provision that prohibits aggregate roof surfaces in hurricane-prone regions.

Glazing protection: The 2000 edition of the IBC was the first model code to address wind-borne debris requirements for glazing in buildings located in hurricane-prone regions (via reference to the 1998 edition of ASCE 7). The 1995 edition of ASCE 7 was the first edition to address wind-borne debris requirements.

ASCE 7 requires impact-resistant glazing in wind-borne debris regions within hurricane-prone regions. Impact-resistant glazing can either be laminated glass, polycarbonate, or shutters tested in accordance with standards specified in ASCE 7. The wind-borne debris load criteria were developed to minimize property damage and to improve building performance. The criteria were not developed for occupant protection. Where occupant protection is a specific criterion, the more conservative wind-borne debris criteria given in FEMA 361, *Design and Construction Guidance for Community Shelters* is recommended.

Parapets and rooftop equipment: The 2003 edition of the IBC was the first model code to address wind loads on parapets and rooftop equipment (via reference to the 2002 edition of ASCE 7, which was the first edition of ASCE 7 to address these elements).

3.1.4.2 Effectiveness and Limitations of Building Codes

A key element of an effective building code is for a community to have an effective building department. Building safety depends on more than the codes and the standards they reference. Building safety results when trained professionals have the resources and ongoing support they need to stay on top of the latest advancements in building safety. An effective building safety system provides uniform code interpretations, product evaluations, and professional development and certification for inspectors and plan reviewers. Local building departments play an important role in helping to ensure buildings are designed and constructed in accordance with the applicable building codes. Meaningful plan review and inspection by the building department are particularly important for critical facilities.

General limitations to building codes include the following:

- Because codes are adopted and enforced on the local or State level, the authority having jurisdiction has the power to eliminate or modify wind-related provisions of a model code, or write its own code instead. In places where important wind-related provisions of the current model code are not adopted and enforced, critical facilities are more susceptible to wind damage. Additionally, a significant time lag often exists between the time a model code is updated and the time it is implemented by the authority having jurisdiction. Buildings designed to the minimum requirements of an outdated code are, therefore, not taking advantage of the current state of the knowledge. These buildings are prone to poorer wind performance compared to buildings designed according to the current model code.

- Adopting the current model code alone does not ensure good wind performance. The code is a minimum tool that should be used by knowledgeable design professionals in conjunction with their training, skills, professional judgment, and the best practices presented in this manual. To achieve good wind performance, in addition to good design, the construction work must be effectively executed, and the building must be adequately maintained and repaired.

- Critical facilities need to perform at a higher level than required by codes and standards. See Section 1.3 on performance based design.

IBC 2006: The 2006 edition of the IBC is believed to be a relatively effective code, provided that it is properly followed and enforced. Some limitations of the 2006 IBC have, however, been identified:

- With respect to hurricanes, the IBC provisions pertaining to building envelopes and rooftop equipment do not adequately address the special needs of critical facilities. For example: (1) they do not account for water infiltration due to puncture of the roof membrane by missiles; (2) they do not adequately address the vulnerabilities of brittle roof coverings (such as tile) to missile-induced damage

and subsequent progressive failure; (3) they do not adequately address occupant protection with respect to missiles; (4) they do not adequately address protection of equipment in elevator penthouses; (5) they do not account for interruption of water service or prolonged interruption of electrical power; and (6) the current requirements for shelters are limited. All of these elements are of extreme importance for critical facilities, which need to remain operational before, during, and after a disaster. Guidance to overcome these shortcomings is given in Section 3.4.

- The 2006 IBC does not account for tornadoes; therefore, except for weak tornadoes, it is ineffective for this type of storm.[4] Guidance to overcome this shortcoming is given in Section 3.5.

The International Code Council (ICC) is developing a consensus standard, ICC/NSSA Standard for the Design and Construction of Storm Shelters, to provide basic requirements for the design and construction of emergency shelters in areas affected by hurricanes and tornadoes. If it is adopted by a community when it becomes available in early 2007, it will provide design and construction standards for buildings intended to resist the impact of high winds and wind-borne debris. This stand-alone standard will be linked to the IBC and IRC. It is the intent of the ICC that the shelter standard be incorporated by reference into the IBC and IRC in 2009. FEMA should be consulted prior to designing or constructing shelters to the ICC/NSSA standard with FEMA funds to ensure all program requirements are met, as some components may need to be designed to stricter criteria than those included in the consensus standard.

The 2000, 2003, and 2006 IBC rely on several referenced standards and test methods developed or updated in the 1990s. Prior to adoption, most of these standards and test methods had not been validated by actual building performance during design level wind events. The hurricanes of 2004 and 2005 provided an opportunity to evaluate the actual performance of buildings designed and constructed to the minimum provisions of the IBC. Building performance evaluations conducted by FEMA revealed the need for further enhancements:

- A limitation of the 2006 IBC pertains to some of the test methods used to assess wind and wind-driven rain resistance of building envelope components. However, before this code limitation can be overcome, research needs to be conducted and new test methods need to be developed.

4. Except for glass breakage, code-compliant buildings should not experience significant damage during weak tornadoes.

- Except to the extent covered by reference to ASCE 7, the 2006 IBC does not address the need for continuity, redundancy, or energy-dissipating capability (ductility) to limit the effects of local collapse, and to prevent or minimize progressive collapse after the loss of one or two primary structural members, such as a column. Chapter 1 of ASCE 7 addresses general structural integrity, and the Chapter 1 Commentary provides some guidance on this issue.

3.2 CRITICAL FACILITIES EXPOSED TO HIGH WINDS

3.2.1 VULNERABILITY: WHAT HIGH WINDS CAN DO TO CRITICAL FACILITIES

This section provides an overview of the common types of wind damage and their ramifications.

3.2.1.1 Types of Building Damage

When damaged by wind, critical facilities typically experience a variety of building component damage. The most common types of damage are discussed below in descending order of frequency.

Roof: Roof covering damage (including rooftop mechanical, electrical, and communications equipment) is the most common type of wind damage. The school, illustrated in Figure 3-12, was being used as a hurricane shelter at the time a portion of the roof membrane blew off. In addition to the membrane damage, several pieces of rooftop equipment were damaged, and virtually all of the loose aggregate blew off and broke many windows in nearby houses. The cast-in-place concrete deck kept most of the water from entering the building.

Glazing: Exterior glazing damage is very common during hurricanes and tornadoes, but is less common during other storms. The glass shown in Figure 3-13 was broken by the aggregate

from a built-up roof. The inner panes had several impact craters. In several of the adjacent windows, both the outer and inner panes were broken. The aggregate flew more than 245 feet (the estimated wind speed was 104 mph, measured at 33 feet in Exposure C).

Figure 3-12:
A portion of the built-up membrane lifted and peeled after the metal edge flashing lifted. Hurricane Andrew (Florida, 1992)

Figure 3-13:
The outer window panes were broken by aggregate from a built-up roof. Hurricane Hugo (South Carolina, 1989)

Wall coverings, soffits, and large doors: Exterior wall covering, soffit, and large door damage is common during hurricanes and tornadoes, but is less common during other storms. At the building shown in Figure 3-14, metal wall covering was attached to plywood over metal studs. The CMU wall behind the studs did not appear to be damaged. The building was located on the periphery of a violent tornado.

Wall collapse: Collapse of non-load-bearing exterior walls is common during hurricanes and tornadoes, but is less common during other storms (see Figure 3-15).

Figure 3-14:
Collapsed metal stud wall—the wall was blown completely away in another part of the building. (Oklahoma, 1999)

Figure 3-15:
The unreinforced CMU wall collapsed at a school during Hurricane Marilyn. (U.S. Virgin Islands, 1995)

Structural system: Structural damage (e.g., roof deck blow-off, blow-off or collapse of the roof structure, collapse of exterior bearing walls, or collapse of the entire building or major portions thereof) is the principal type of damage that occurs during strong and violent tornadoes (see Figure 3-16).

Figure 3-16:
The school wing was destroyed by a violent tornado. (Oklahoma, 1999)

3.2.1.2 Ramifications of Damage

The ramifications of building component damage on critical facilities are described below.

Property damage: Property damage requires repairing/replacing the damaged components (or replacing the entire facility), and may require repairing/replacing interior building components, furniture, and other equipment, and mold remediation. Even when damage to the building envelope is limited, such as blow-off of a portion of the roof covering or broken glazing, substantial water damage frequently occurs because heavy rains often accompany strong winds (particularly in the case of thunderstorms, tropical storms, hurricanes, and tornadoes). At the newly constructed gymnasium shown in Figure 3-17, the structural metal

Modest wind speeds can drive rain into exterior walls. Unless adequate provisions are taken to account for water infiltration (see Sections 3.3.3.1 to 3.3.3.3), damaging corrosion, dry rot, and mold can occur within walls.

roof panels were applied over metal purlins. The panels with 3-inch-high trapezoidal ribs at 24 inches on center detached from their concealed clips. A massive quantity of water entered the building and buckled the wood floor.

Figure 3-17:
A massive quantity of water entered the building after the roof blew off. Typhoon Paka (Guam, 1997)

Wind-borne debris such as roof aggregate, gutters, rooftop equipment, and siding blown from buildings can damage vehicles and other buildings in the vicinity. Debris can travel well over 300 feet in high-wind events.

Portable classrooms on school campuses are often particularly vulnerable to significant damage because they are seldom designed to the same wind pressures as permanent buildings. Portable classrooms are frequently blown over during high-wind events, because the anchoring techniques typically used are inadequate to secure the units to the ground. Wind-borne debris from portable classrooms, or an entire portable classroom, may strike the permanent school building and cause serious damage.

Ancillary buildings (such as storage buildings) adjacent to critical facilities are also vulnerable to damage. Although loss of these buildings may not be detrimental to the operation of the critical facility, debris from ancillary buildings may strike and damage the critical facility. The damaged building shown in

Figure 3-18 contained the hospital's supplies and maintenance shop. With the loss of this building, tents had to be set up to provide supply storage. Almost all of the tools and stock materials for repairs were lost.

Figure 3-18:
A hurricane-damaged, pre-engineered storage building adjacent to a hospital. Hurricane Charley (Florida, 2004)

Although people are not usually outside during hurricanes, it is not uncommon for people to seek shelter or assistance in critical facilities during a storm. Missiles, such as roof aggregate or tile shedding from a critical facility, could injure or kill late arrivals before they have a chance to enter the building.

Injury or death: Although infrequent, critical facility occupants or people outside the facility have been injured and killed when struck by collapsed building components (such as exterior masonry walls or the roof structure) or wind-borne debris. The greatest risk of injury or death is during strong hurricanes and strong/violent tornadoes.

Interrupted use: Depending on the magnitude of wind and water damage, it can take days, months, or more than a year to repair the damage or replace a facility. In addition to the costs associated with repairing/replacing the damage, other social and financial costs can be even more significant. The repercussions related to interrupted use of the critical facility can include the loss of emergency and first-responder services, lack of medical care, and the costs to rent temporary facilities. These additional costs can be quite substantial.

3.2.2 EVALUATING CRITICAL FACILITIES FOR RISK FROM HIGH WINDS

This section describes the process of hazard risk assessment. Although no formal methodology for risk assessment has been adopted, prior experience provides a sufficient knowledge base upon which a set of guidelines can be structured into a recommended procedure for risk assessment of critical facilities. The procedures presented below establish guidelines for evaluating the risk to new and existing buildings from wind storms other than tornadoes. These evaluations will allow development of a vulnerability assessment that can be used along with the site's wind regime to assess the risk to critical facilities.

In the case of tornadoes, neither the IBC nor ASCE 7 requires buildings (including critical facilities) to be designed to resist tornado forces, nor are occupant shelters required in buildings located in tornado-prone regions. Constructing tornado-resistant critical facilities is extremely expensive because of the extremely high pressures and missile impact loads that tornadoes can generate. Therefore, when consideration is voluntarily given to tornado design, the emphasis is typically on occupant protection, which is achieved by "hardening" portions of a critical facility for use as safe havens. FEMA 361 includes a comprehensive risk assessment procedure that designers can use to assist building owners in determining whether a tornado shelter should be included as part of a new critical facility. See Section 3.5 for the design of tornado shelters and other recommendations pertaining to critical facilities in tornado-prone regions.

In this manual, the term "tornado-prone regions" refers to those areas of the United States where the number of recorded F3, F4, and F5 tornadoes per 3,700 square miles is 6 or greater per year (see Figure 3-2). However, an owner of a critical facility may decide to use other frequency values (e.g., 1 or greater, 16 or greater, or greater than 25) in defining whether the building is in a tornado-prone area. In this manual, tornado shelters are recommended for all critical facilities in tornado-prone regions.

Where the frequency value is 1 or greater, and the facility does not have a tornado shelter, the best available refuge areas should be identified, as discussed in Section 3.5.

3.2.2.1 New Buildings

When designing new critical facilities, a two-step procedure is recommended for evaluating the risk from wind storms (other than tornadoes).

Step 1: Determine the basic wind speed from ASCE 7. As the basic wind speed increases beyond 90 mph, the risk of damage increases. Design, construction, and maintenance enhancements are recommended to compensate for the increased risk of damage (see Section 3.3).

Step 2: For critical facilities in hurricane-prone regions, refer to the design, construction, and maintenance enhancements recommended in Section 3.4.

For particularly important critical facilities (such as hospitals) in remote areas outside of hurricane-prone regions, it is recommended that robust design measures be considered to minimize the potential for disruption resulting from wind damage. Because of their remote location, disruption of such facilities could severely affect the occupants or community. Some of the recommendations in Section 3.4 may therefore be prudent.

3.2.2.2 Existing Buildings

The resistance of existing buildings is a function of their original design and construction, various additions or modifications, and the condition of building components (which may have weakened due to deterioration or fatigue). For existing buildings, a two-step procedure is also recommended.

Step 1: Calculate the wind loads on the building using the current edition of ASCE 7, and compare these loads with the loads for which the building was originally designed. The original design loads may be noted on the contract drawings. If not, determine what building code or standard was used to develop the original design loads, and calculate the loads using that code or standard. If the original design loads are significantly lower than current loads, upgrading the load resistance of the building envelope and/or structure should be considered. An alternative to comparing current loads with original design loads is to evaluate the resistance of the existing facility as a function of the current loads to determine what elements are highly overstressed.

Step 2: Perform a field investigation to evaluate the primary building envelope elements, rooftop equipment, and structural system elements, to determine if the facility was generally constructed as indicated on the original contract drawings. As part of the investigation, the primary elements should be checked for deterioration. Load path continuity should also be checked.

If the results of either step indicate the need for remedial work, see Section 3.6.

3.3 CRITICAL FACILITY DESIGN CONSIDERATIONS

3.3.1 GENERAL DESIGN CONSIDERATIONS

The performance of critical facilities in past wind storms indicates that the most frequent and the most significant factor in the disruption of the operations of these facilities has been the failure of nonstructural building components. While acknowledging the importance of the structural systems, Chapter 3 emphasizes the building envelope components and the nonstructural systems. According to the National Institute of Building Science (NIBS), the building envelope includes the below-grade basement walls and foundation and floor slab (although these are generally considered part of the building's structural system). The envelope includes everything that separates the interior of a building from the outdoor environment, including the connection of all the nonstructural elements to the building structure. The nonstructural systems include all mechanical, electrical, electronic, communications, and lightning protection systems. Historically, damage to roof coverings and rooftop equipment has been the leading cause of building performance problems during wind storms. Special consideration should be given to the problem of water infiltration through failed building envelope components, which can cause severe disruptions in the functioning of critical facilities.

The key to enhanced wind performance is paying sufficient attention to all phases of the construction process (including site selection, design, and construction) and to post-occupancy maintenance and repair.

3.3.1.1 Site

When selecting land for a critical facility, sites located in Exposure D (see Section 3.1.3 for exposure definitions) should be avoided if possible. Selecting a site in Exposure C or preferably in Exposure B would decrease the wind loads. Also, where possible, avoid selecting sites located on an escarpment or the upper half of a hill, where the abrupt change in the topography would result in increased wind loads.[6]

Trees with trunks larger than 6 inches in diameter, poles (e.g., light fixture poles, flag poles, and power poles), or towers (e.g., electrical transmission and large communication towers) should not be placed near the building. Falling trees, poles, and towers can severely damage a critical facility and injure the occupants (see Figure 3-19). Large trees can crash through pre-engineered metal buildings (which often house fire stations) and wood frame construction (which is commonly used for nursing homes). Falling trees can also rupture roof membranes and break windows.

Figure 3-19:
Had this tree fallen in the opposite direction, it would have landed on the school. Hurricane Ivan (Florida, 2004)

6. When selecting a site on an escarpment or the upper half of a hill is necessary, the ASCE 7 design procedure accounts for wind speed-up associated with this abrupt change in topography.

In the past, design professionals seldom performed load calculations on the building envelope (i.e., roof and wall coverings, doors, windows, and skylights) and rooftop equipment. These building components are the ones that have failed the most during past wind events. In large part they failed because of the lack of proper load determination and inappropriate design of these elements. It is imperative that design professionals determine the loads for the building envelope and rooftop equipment, and design them to accommodate such loads.

Providing at least two means of site egress is prudent for all critical facilities, but is particularly important for facilities in hurricane-prone regions. If one route becomes blocked by trees or other debris, or by floodwaters, the other access route may still be available.

3.3.1.2 Building Design

Good wind performance depends on good design (including details and specifications), materials, installation, maintenance, and repair. A significant shortcoming in any of these five elements could jeopardize the performance of a critical facility against wind. Design, however, is the key element to achieving good performance of a building against wind damage. Design inadequacies frequently cannot be compensated for with other elements. Good design, however, can compensate for other inadequacies to some extent. The following steps should be included in the design process for critical facilities.

Step 1: Calculate Loads

Calculate loads on the MWFRS, the building envelope, and rooftop equipment in accordance with ASCE 7 or the local building code, whichever procedure results in the highest loads. In calculating wind loads, design professionals should consider the following items.

Uplift loads on roof assemblies can also be determined from FM Global (FMG) Data Sheets. If the critical facility is FMG insured, and the FMG-derived loads are higher than those derived from ASCE 7 or the building code, the FMG loads should govern. However, if the ASCE 7 or code-derived loads are higher than those from FMG, the ASCE 7 or code-derived loads should govern (whichever procedure results in the highest loads).

Importance factor: The effect of using a 1.15 importance factor versus 1 is that the design loads for the MWFRS and C&C are increased by 15 percent. The importance factor for most critical facilities is required to be 1.15. However, ASCE 7 permits a factor of 1 for some critical facilities. For example, schools with an occupant load of less than 250 people are permitted to be designed with an importance factor of only 1 (provided they are not used as a shelter—if used as a shelter,

schools are required to be designed with an importance factor of 1.15, regardless of occupant load). Other facilities where occupant load controls the importance factor include certain health care facilities, such as nursing homes with 50 or more residents, for which a factor of 1.15 is required. For nursing homes with less than 50 residents, a factor of 1 can be used. Some critical facilities are not specifically addressed in ASCE 7. For example, various buildings on a hospital campus, such as medical office buildings that are integrally connected to the hospital and various types of non-emergency treatment facilities (such as storage, cancer treatment, physical therapy, and dialysis) are not specifically required by ASCE 7 to be designed with a 1.15 factor. This manual recommends a value of 1.15 for all critical facilities.

Wind directionality factor: The ASCE 7 wind load calculation procedure incorporates a wind directionality factor (k_d). The directionality factor accounts for the reduced probability of maximum winds coming from any given direction. By applying the prescribed value of 0.85, the loads are reduced by 15 percent. Because hurricane winds can come from any direction, and because of the historically poor performance of building envelopes and rooftop equipment, this manual recommends a more conservative approach for critical facilities in hurricane-prone regions. A directionality factor of 1.0 is recommended for the building envelope and rooftop equipment (a load increase over what is required by ASCE 7). For the MWFRS, a directionality factor of 0.85 is recommended (hence, no change for MWFRS).

Step 2: Determine Load Resistance

When using allowable stress design, after loads have been determined, it is necessary to select a reasonable safety factor in order to determine the minimum required load resistance. For building envelope systems, a minimum safety factor of 2 is recommended. For anchoring exterior-mounted mechanical, electrical, and communications equipment (such as satellite dishes), a minimum safety factor of 3 is recommended. When using strength design, load combinations and load factors specified in ASCE 7 are used.

ASCE 7 provides criteria for combining wind loads with other types of loads (such as dead and flood loads) using allowable stress design.

When using allowable stress design, a safety factor is applied to account for reasonable variations in material strengths, construction workmanship, and conditions when the actual wind speed somewhat exceeds the design wind speed. For design purposes, the ultimate resistance an assembly achieves in testing is reduced by the safety factor. For example, if a roof assembly resisted an uplift pressure of 100 pounds per square foot (psf), after applying a safety factor of 2, the assembly would be suitable where the design load was 50 psf or less. Conversely, if the design load is known, multiplying it by the safety factor equals the minimum required test pressure (e.g., 50 psf design load multiplied by a safety factor of 2 equals a minimum required test pressure of 100 psf).

For structural members and cladding elements where strength design can be used, load resistance can be determined by calculations. For other elements where allowable stress design is used (such as most types of roof coverings), load resistance is primarily obtained from system testing.

The load resistance criteria need to be provided in contract documents. For structural elements, the designer of record typically accounts for load resistance by indicating the material, size, spacing, and connection of the elements. For nonstructural elements, such as roof coverings or windows, the load and safety factor can be specified. In this case, the specifications should require the contractor's submittals to demonstrate that the system will meet the load resistance criteria. This performance specification approach is necessary if, at the time of the design, it is unknown who will manufacture the system.

Regardless of which approach is used, it is important that the designer of record ensure that it can be demonstrated, via calculations or tests, that the structure, building envelope, and nonstructural systems (exterior-mounted mechanical, electrical, and communications equipment) have sufficient strength to resist design wind loads.

Connections are a key aspect of load path continuity between various structural and nonstructural building elements. In a window, for example, the glass must be strong enough to resist the wind pressure and must be adequately anchored to the window frame, the frame adequately anchored to the wall, the wall to the foundation, and the foundation to the ground. As loads increase, greater load capacity must be developed in the connections.

Step 3: Detailed Design

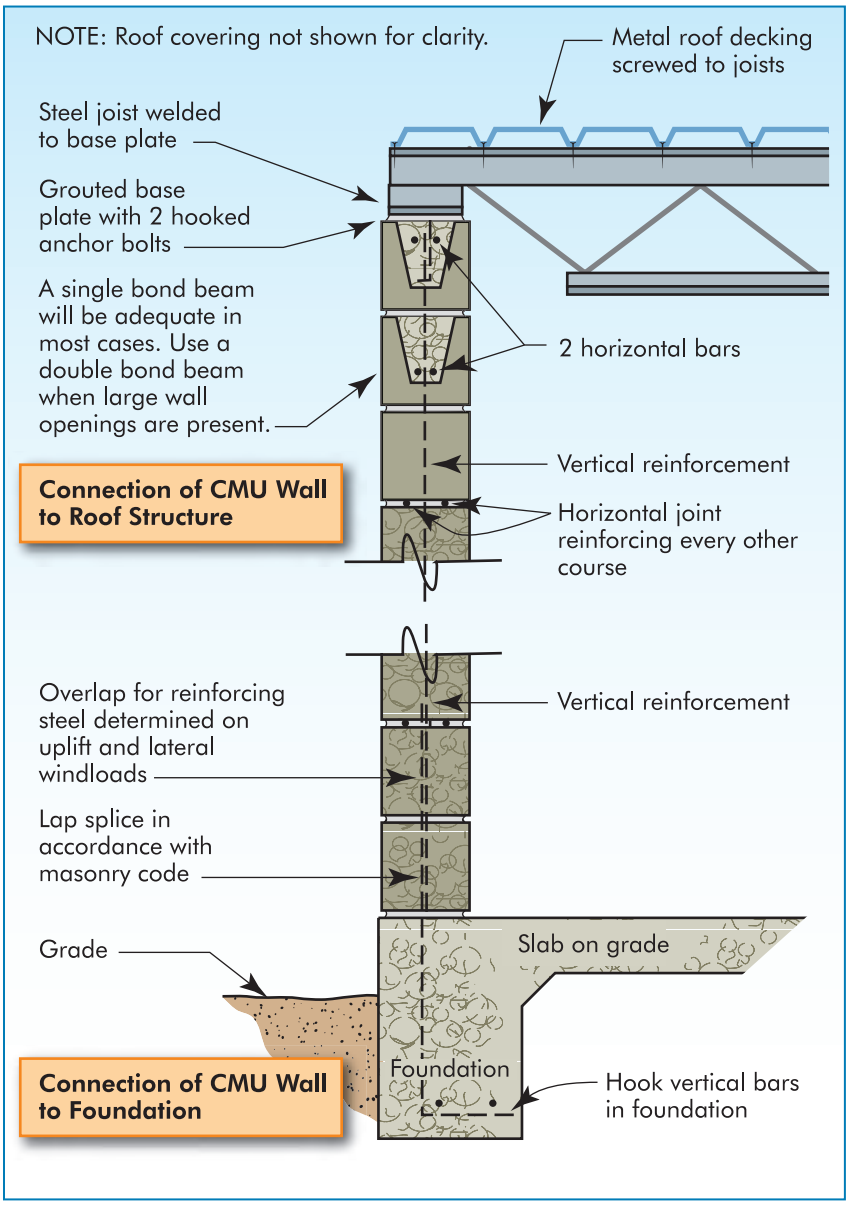
It is vital to design, detail, and specify the structural system, building envelope, and exterior-mounted mechanical, electrical, and communications equipment to meet the factored design loads (based on appropriate analytical or test methods). It is also vital to respond to the risk assessment criteria discussed in Section 3.2.2, as appropriate.

As part of the detailed design effort, load path continuity should be clearly indicated in the contract documents via illustration of connection details. Load paths need to accommodate design uplift, racking, and overturning loads. Load path continuity obviously applies to MWFRS elements, but it also applies to building envelope elements. Figure 3-20 shows a load path discontinuity between a piece of HVAC equipment and its equipment stand. The equipment on this new Federal courthouse blew away because it was resting on vibration isolators that provided lateral resistance, but no uplift resistance (also see Figure 3-75).

Figure 3-20: Temporary coverings placed over two large openings in the roof that were left after the ductwork blew away. Hurricane Katrina (Mississippi, 2005)

Figure 3-21 illustrates the load path concept. Members are sized to accommodate the design loads. Connections are designed to transfer uplift loads applied to the roof, and the positive and negative loads applied to the exterior bearing walls, down to the foundation and into the ground. The roof covering (and wall covering, if there is one) is also part of the load path. To avoid blow-off, the nonstructural elements must also be adequately attached to the structure.

Figure 3-21:
Illustration of load path continuity



As part of the detailed design process, special consideration should be given to the durability of materials and water infiltration.

Durability: Because some locales have very aggressive atmospheric corrosion (such as areas near oceans), special attention needs to be given to the specification of adequate protection for ferrous metals, or to specify alternative metals such as stainless

steel. FEMA Technical Bulletin, *Corrosion Protection for Metal Connectors in Coastal Areas* (FIA-TB-8, 1996) contains information on corrosion protection. Attention also needs to be given to dry rot avoidance, for example, by specifying preservative-treated wood or developing details that avoid excessive moisture accumulation. Appendix J of the *Coastal Construction Manual,* (FEMA 55, 2000) presents information on wood durability.

Durable materials are particularly important for components that are inaccessible and cannot be inspected regularly (such as fasteners used to attach roof insulation). Special attention also needs to be given to details. For example, details that do not allow water to stand at connections or sills are preferred. Without special attention to material selection and details, the demands on maintenance and repair will be increased, along with the likelihood of failure of components during high winds.

Water infiltration (rain): Although prevention of building collapse and major building damage is the primary goal of wind-resistant design, consideration should also be given to minimizing water damage and subsequent development of mold from the penetration of wind-driven rain. To the extent possible, non-load-bearing walls and door and window frames should be designed in accordance with rain-screen principles. With this approach, it is assumed that some water will penetrate past the face of the building envelope. The water is intercepted in an air-pressure equalized cavity that provides drainage from the cavity to the outer surface of the building. See Sections 3.3.3.2 and 3.3.3.3, and Figure 3-40 for further discussion and an example.

Coastal environments are conducive to metal corrosion, especially in buildings within 3,000 feet of the ocean. Most jurisdictions require metal building hardware to be hot-dipped galvanized or stainless steel. Some local codes require protective coatings that are thicker than typical "off-the-shelf" products. For example, a G90 zinc coating (0.75 mil on each face) may be required. Other recommendations include the following:

- Use hot-dipped galvanized or stainless steel hardware. Reinforcing steel should be fully protected from corrosion by the surrounding material (masonry, mortar, grout, or concrete). Use galvanized or epoxy-coated reinforcing steel in situations where the potential for corrosion is high.
- Avoid joining dissimilar metals, especially those with high galvanic potential.
- Avoid using certain wood preservatives in direct contact with galvanized metal. Verify that wood treatment is suitable for use with galvanized metal, or use stainless steel.
- Metal-plate-connected trusses should not be exposed to the elements. Truss joints near vent openings are more susceptible to corrosion and may require increased corrosion protection.

Note: *Although more resistant than other metals, stainless steel is still subject to corrosion.*

Further information on the rain-screen principle can be found in the National Institute of Building Sciences' *Building Envelope Design Guide* (www.wbdg.org/design/envelope.php).

In conjunction with the rain-screen principle, it is desirable to avoid using sealant as the first or only line of defense against water infiltration. When sealant joints are exposed, obtaining long-lasting watertight performance is difficult because of the complexities of sealant joint design and installation (see Figure 3-40, which shows the sealant protected by a removable stop).

Step 4: Peer Review

If the design team's wind expertise and experience is limited, wind design input and/or peer review should be sought from a qualified individual. The design input or peer review could be arranged for the entire building, or for specific components such as the roof or glazing systems, that are critical and beyond the design team's expertise.

Regardless of the design team's expertise and experience, peer review should be considered when a critical facility:

- Is located in an area where the basic wind speed is greater than 90 mph (peak gust)—particularly if the facility is a hospital, or will be used as an EOC or hurricane shelter.
- Will incorporate a tornado shelter.

3.3.1.3 Construction Contract Administration

After a suitable design is complete, the design team should endeavor to ensure that the design intent is achieved during construction. The key elements of construction contract administration are submittal reviews and field observations, as discussed below.

Submittal reviews: The specifications need to stipulate the submittal requirements. This includes specifying what systems require submittals (e.g., windows) and test data (where appropriate). Each submittal should demonstrate the development of a load path through the system and into its supporting element. For example,

a window submittal should show that the glazing has sufficient strength, its attachment to the frame is adequate, and the attachment of the frame to the wall is adequate.

During submittal review, it is important for the designer of record to be diligent in ensuring that all required documents are submitted and that they include the necessary information. The submittal information needs to be thoroughly checked to ensure its validity. For example, if an approved method used to demonstrate compliance with the design load has been altered or incorrectly applied, the test data should be rejected, unless the contractor can demonstrate the test method was suitable. Similarly, if a new test method has been developed by a manufacturer or the contractor, the contractor should demonstrate its suitability.

Field observations: It is recommended that the design team analyze the design to determine which elements are critical to ensuring high-wind performance. The analysis should include the structural system and exterior-mounted electrical equipment, but it should focus on the building envelope and exterior-mounted mechanical and communications equipment. After determining the list of critical elements to be observed, observation frequency and the need for special inspections by an inspection firm should be determined. Observation frequency and the need for special inspections will depend on the results of the risk assessment described in Section 3.2.2, complexity of the facility, and the competency of the general contractor, subcontractors, and suppliers.

See Section 3.4 for additional information pertaining to critical facilities located in hurricane-prone regions.

3.3.1.4 Post-Occupancy Inspections, Periodic Maintenance, Repair, and Replacement

The design team should advise the building owner of the importance of periodic inspections, maintenance, and timely repair. It is important for the building owner to understand that a facility's wind resistance will degrade over time due to exposure to weather unless it is regularly maintained and repaired. The goal should

be to repair or replace items before they fail in a storm. This approach is less expensive than waiting for failure and then repairing the failed components and consequential damage.

The building envelope and exterior-mounted equipment should be inspected once a year by persons knowledgeable of the systems/materials they are inspecting. Items that require maintenance, repair, or replacement should be documented and scheduled for work. For example, the deterioration of glazing is often overlooked. After several years of exposure, scratches and chips can become extensive enough to weaken the glazing. Also, if an engineered film was surface-applied to glazing for wind-borne debris protection, the film should be periodically inspected and replaced before it is no longer effective.

A special inspection is recommended following unusually high winds. The purpose of the inspection is to assess whether the strong storm caused damage that needs to be repaired to maintain building strength and integrity. In addition to inspecting for obvious signs of damage, the inspector should determine if cracks or other openings have developed that may allow water infiltration, which could lead to corrosion or dry rot of concealed components.

See Section 3.4 for additional information pertaining to buildings located in hurricane-prone regions.

3.3.2 STRUCTURAL SYSTEMS

Based on post-storm damage evaluations, with the exception of strong and violent tornado events, the structural systems (i.e., MWFRS and structural components such as roof decking) of critical facilities have typically performed quite well during design wind events. There have, however, been notable exceptions; in these cases, the most common problem has been blow-off of the roof deck, but instances of collapse have also been documented (see Figure 3-22). The structural problems have primarily been caused by lack of an adequate load path, with connection failure being a common occurrence. Problems have also been caused by reduced structural capacity due to termites, workmanship errors (commonly associated with steel decks attached by puddle welds),

and limited uplift resistance of deck connections in roof perimeters and corners (due to lack of code-required enhancement in older editions of the model codes).

Figure 3-22:
Collapse of a large portion of a new pre-engineered metal building used as a shelter for approximately 1,400 people. Hurricane Charley (Florida, 2004)

With the exception of strong and violent tornado events, structural systems designed and constructed in accordance with the IBC should typically offer adequate wind resistance, provided attention was given to load path continuity and to the durability of building materials (with respect to corrosion and termites). However, the greatest reliability is offered by cast-in-place concrete. There are no known reports of any cast-in-place concrete buildings experiencing a significant structural problem during wind events, including the strongest hurricanes (Category 5) and tornadoes (F5).

The following design parameters are recommended for structural systems (see Section 3.4.2 for critical facilities located in hurricane-prone regions):

- If a pre-engineered metal building is being contemplated, special steps should be taken to ensure the structure has more redundancy than is typically the case with pre-engineered buildings.[7] Steps should be taken to ensure the structure is not vulnerable to progressive collapse in the event a primary bent (steel moment frame) is compromised or bracing components fail.

- Exterior load-bearing walls of masonry or precast concrete should be designed to have sufficient strength to resist external and internal loading when analyzed as C&C. CMU walls should have vertical and horizontal reinforcing and grout to resist wind loads. The connections of precast concrete wall panels should be designed to have sufficient strength to resist wind loads.

- For roof decks, concrete, steel, plywood, or oriented strand board (OSB) is recommended.

- For steel roof decks, it is recommended that a screw attachment be specified, rather than puddle welds or powder-driven pins. Screws are more reliable and much less susceptible to workmanship problems. Figure 3-23 shows decking that was attached with puddle welds. At most of the welds, there was only superficial bonding of the metal deck to the joist, as illustrated by this example. Only a small portion of the deck near the center of the weld area (as delineated by the circle) was well fused to the joist. Figures 3-24 and 3-25 show problems with acoustical decking attached with powder-driven pins. The pin shown on the left of Figure 3-25 is properly seated. However, the pin at the right did not penetrate far enough into the steel joist below.

- For attaching wood sheathed roof decks, screws, ring-shank, or screw-shank nails are recommended in the corner regions of the roof. Where the basic wind speed is greater than 90 mph, these types of fasteners are also recommended for the perimeter regions of the roof.

7. The structural system of pre-engineered metal buildings is composed of rigid steel frames, secondary members (including roof purlins and wall girts made of Z- or C-shaped members) and bracing.

Figure 3-23:
View looking down at the top of a steel joist after the metal decking blew away. Only a small portion of the deck was well fused to the joist (circled area). Tornado (Oklahoma, 1999)

Figure 3-24:
Looking down at a sidelap of a deck attached with powder-driven pins. The washer at the top pin blew through the deck.

Figure 3-25:
View looking along a sidelap of a deck attached with powder-driven pins. The right pin does not provide adequate uplift and shear resistance.

- For precast concrete decks it is recommended that the deck connections be designed to resist the design uplift loads because the deck dead load itself is often insufficient to resist the uplift. The deck in Figure 3-26 had bolts to provide uplift resistance; however, anchor plates and nuts had not been installed. Without the anchor plates, the dead load of the deck was insufficient to resist the wind uplift load.

Figure 3-26:
Portions of this waffled precast concrete roof deck were blown off. Typhoon Paka (Guam, 1997)

- For precast Tee decks, it is recommended that the reinforcing be designed to accommodate the uplift loads in addition to the gravity loads. Otherwise, large uplift forces can cause member failure due to the Tee's own pre-stress forces after the uplift load exceeds the dead load of the Tee. This type of failure occurred at one of the roof panels shown in Figure 3-27, where a panel lifted because of the combined effects of wind uplift and pre-tension. Also, because the connections between the roof and wall panels provided very little uplift load resistance, several other roof and wall panels collapsed.

Figure 3-27:
Twin-Tee roof panel lifted as a result of the combined effects of wind uplift and pre-tension. Tornado (Missouri, May 2003)

- For buildings that have mechanically attached single-ply or modified bitumen membranes, designers should refer to the decking recommendations presented in the *Wind Design Guide for Mechanically Attached Flexible Membrane Roofs*, B1049 (National Research Council of Canada, 2005).

- If an FMG-rated roof assembly is specified, the roof deck also needs to comply with the FMG criteria.

- Walkway and entrance canopies are often damaged during high winds (see Figure 3-28). Wind-borne debris from damaged canopies can damage nearby buildings and injure people, hence these elements should also receive design and construction attention.

Figure 3-28:
The destroyed walkway canopy in front of a school became wind-borne debris. Hurricane Ivan (Florida, 2004)

ASCE 7-05 provides pressure coefficients for open canopies of various slopes (referred to as "free roofs" in ASCE 7). The free roof figures for MWFRS in ASCE 7-05 (Figures 6-18A to 6-18D) include two load cases, Case A and Case B. While there is no discussion describing the two load cases, they pertain to fluctuating loads and are intended to represent upper and lower limits of instantaneous wind pressures. Loads for both cases must be calculated to determine the critical loads. Figures 6-18A to 6-18C are for a wind direction normal to the ridge. For wind direction parallel to the ridge, use Figure 6-18D in ASCE 7-05.

3.3.3 BUILDING ENVELOPE

The following section highlights the design considerations for building envelope components that have historically sustained the greatest and most frequent damage in high winds.

3.3.3.1 Exterior Doors

For further general information on doors, see "Fenestration Systems" in the National Institute of Building Sciences' *Building Envelope Design Guide* (www.wbdg.org/design/envelope.php)

This section addresses primary and secondary egress doors, sectional (garage) doors, and rolling doors. Although blow-off of personnel doors is uncommon, it can cause serious problems (see Figure 3-29). Blown-off doors allow entrance of rain and tumbling doors can damage buildings and cause injuries.

Blown off sectional and rolling doors are quite common. These failures are typically caused by the use of door and track assemblies that have insufficient wind resistance, or by inadequate attachment of the tracks or nailers to the wall. At the relatively new fire station shown in Figure 3-30, two of the windward doors were pushed out of their tracks. At the third door, the track was pushed out from the nailer. With the collapse of these doors, the apparatus bay was fully pressurized. Because the connections between the trusses and the beam were too weak to accommodate the uplift load, the entire roof structure over the apparatus bay blew off.

Particular attention should be given to the design and installation of fire station apparatus bay doors which have been blown-in or blown-out frequently (see Figure 3-30). If doors blow inward, they can damage fire engines and ambulances and impair emergency response.

See Section 3.4.3.1 for critical facilities located in hurricane-prone regions.

Loads and Resistance

The IBC requires that the door assembly (i.e., door, hardware, frame, and frame attachment to the wall) be of sufficient strength to resist the positive and negative design wind pressure. Design professionals should require that doors comply with wind load testing in accordance with ASTM E 1233. Design profes-

sionals should also specify the attachment of the door frame to the wall (e.g., type, size, spacing, and edge distance of frame fasteners). For sectional and rolling doors attached to wood nailers, design professionals should also specify the attachment of the nailer to the wall.

For design guidance on attachment of door frames, see Technical Data Sheet #161, *Connecting Garage Door Jambs to Building Framing*, published by the Door & Access Systems Manufacturers Association, 2003 (available at www.dasma.com).

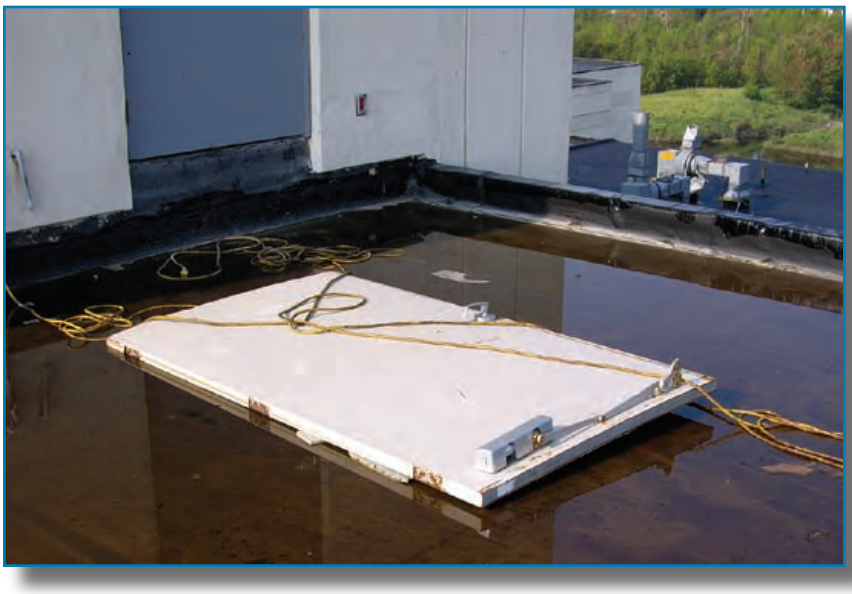


Figure 3-29:
Door on a hospital penthouse blown off its hinges during Hurricane Katrina (Mississippi, 2005)

Figure 3-30:
The roof structure over the apparatus bay blew off following the failure of sectional doors. Hurricane Charley (Florida, 2004)

Water Infiltration

Heavy rain that accompanies high winds (e.g., thunderstorms, tropical storms, and hurricanes) can cause significant wind-driven water infiltration problems. The magnitude of the problem increases with the wind speed. Leakage can occur between the door and its frame, the frame and the wall, and between the threshold and the door. When wind speeds approach 120 mph, some leakage should be anticipated because of the very high wind pressures and numerous opportunities for leakage path development.

Where corrosion is problematic, anodized aluminum or galvanized doors and frames, and stainless steel frame anchors and hardware are recommended.

The following recommendations should be considered to minimize infiltration around exterior doors.

Vestibule: Adding a vestibule allows both the inner and outer doors to be equipped with weatherstripping. The vestibule can be designed with water-resistant finishes (e.g., concrete or tile) and the floor can be equipped with a drain. In addition, installing exterior threshold trench drains can be helpful (openings must be small enough to avoid trapping high-heeled shoes). Note that trench drains do not eliminate the problem, since water can still penetrate at door edges.

Door swing: Out-swinging doors have weatherstripping on the interior side of the door, where it is less susceptible to degradation, which is an advantage when compared to in-swinging doors. Some interlocking weatherstripping assemblies are available for out-swinging doors.

For primary swinging entry/exit doors, exit door hardware is recommended to minimize the possibility of the doors being pulled open by wind suction. Exit hardware with top and bottom rods is more secure than exit hardware that latches at the jamb.

The successful integration of the door frame and the wall is a special challenge when designing doors. See Section 3.3.3.2 for discussion of this juncture.

ASTM E 2112 provides information pertaining to the installation of doors, including the use of sill pan flashings with end dams and rear legs (see Figure 3-31). It is recommended that designers use ASTM E 2112 as a design resource.

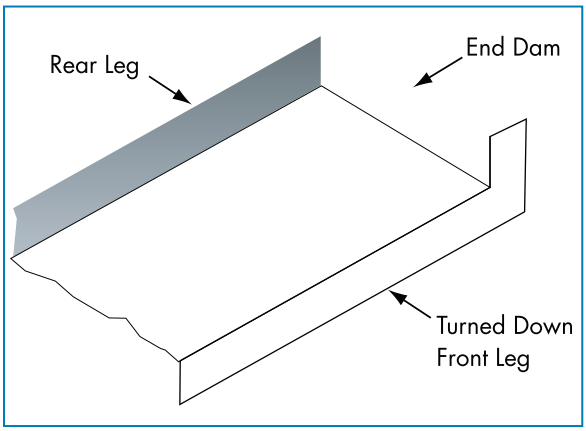


Figure 3-31:
Door sill pan flashing with end dams, rear leg, and turned-down front leg

Weatherstripping

A variety of pre-manufactured weatherstripping components is available, including drips, door shoes and bottoms, thresholds, and jamb/head weatherstripping.

Drips: These are intended to shed water away from the opening between the frame and the door head, and the opening between the door bottom and the threshold (see Figures 3-32 and 3-33). Alternatively, a door sweep can be specified (see Figure 3-33). For high-traffic doors, periodic replacement of the neoprene components will be necessary.

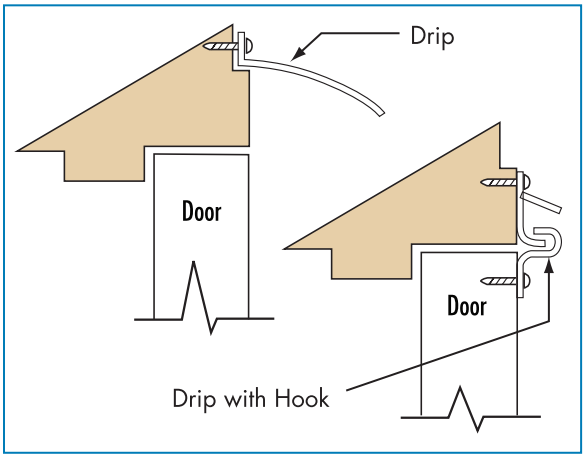


Figure 3-32:
Drip at door head and drip with hook at head

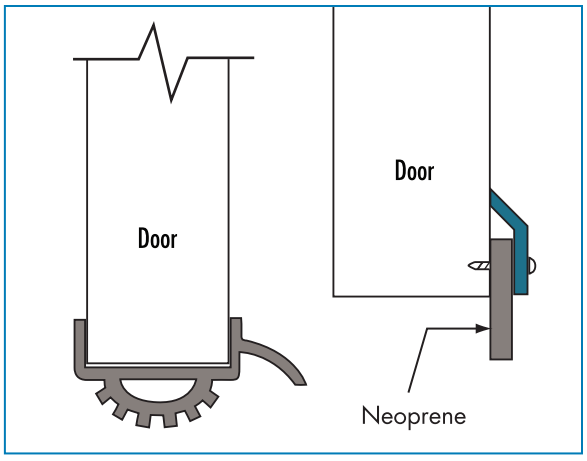


Figure 3-33:
Door shoe with drip and vinyl seal (left). Neoprene door bottom sweep (right)

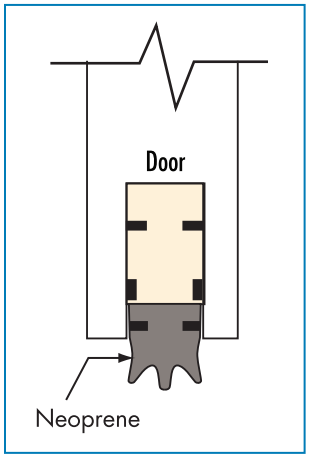


Figure 3-34:
Automatic door bottom

Door shoes and bottoms: These are intended to minimize the gap between the door and the threshold. Figure 3-33 illustrates a door shoe that incorporates a drip. Figure 3-34 illustrates an automatic door bottom. Door bottoms can be surface-mounted or mortised. For high-traffic doors, periodic replacement of the neoprene components will be necessary.

Thresholds: These are available to suit a variety of conditions. Thresholds with high (e.g., 1-inch) vertical offsets offer enhanced resistance to wind-driven water infiltration. However, the offset is limited where the thresholds are required to comply with the Americans with Disabilities Act (ADA), or at high-traffic doors. At other doors, high offsets are preferred.

Thresholds can be interlocked with the door (see Figure 3-35), or thresholds can have a stop and seal (see Figure 3-36). In some instances, the threshold is set directly on the floor. Where this is appropriate, setting the threshold in butyl sealant is recommended to avoid water infiltration between the threshold and the floor. In other instances, the threshold is set on a pan flashing, as previously discussed in this section. If the threshold has weep holes, specify that the weep holes not be obstructed (see Figure 3-35).

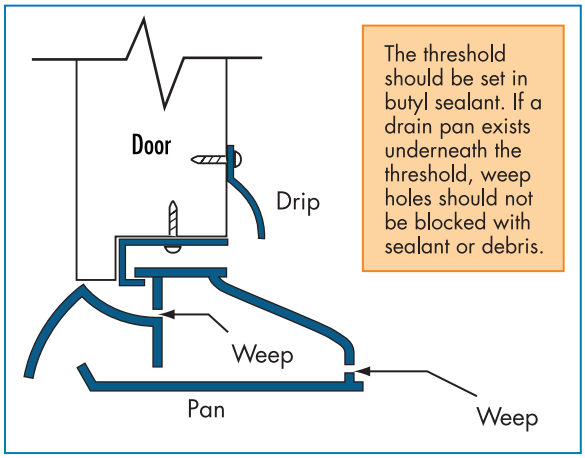


Figure 3-35:
Interlocking threshold with drain pan

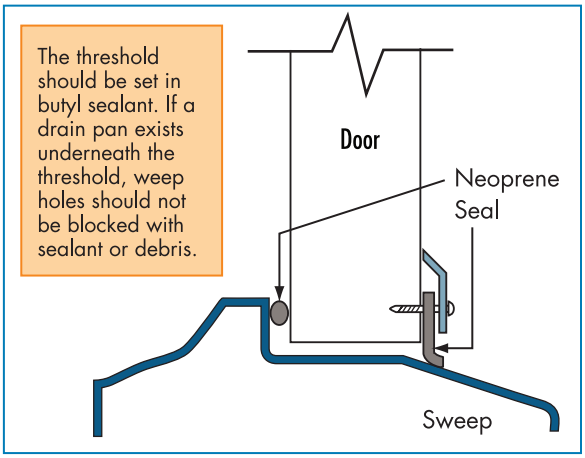


Figure 3-36:
Threshold with stop and seal

Adjustable jamb/head weatherstripping: This type of weatherstripping is recommended because the wide sponge neoprene offers good contact with the door (see Figure 3-37). The adjustment feature also helps to ensure good contact, provided the proper adjustment is maintained.

Meeting stile: At the meeting stile of pairs of doors, an overlapping astragal weatherstripping offers greater protection than weatherstripping that does not overlap.

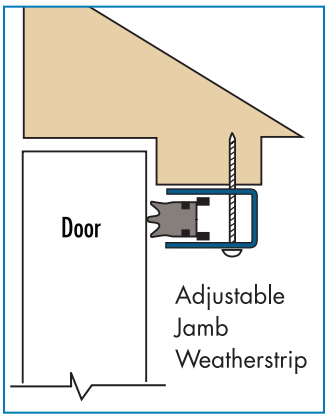


Figure 3-37: Adjustable jamb/head weatherstripping

3.3.3.2 Windows and Skylights

This section addresses general design considerations for exterior windows and skylights in critical facilities. For additional information on windows and skylights in critical facilities located in hurricane-prone regions, see Section 3.4.3.2, and for those in tornado-prone regions, see Section 3.5.

Loads and Resistance

The IBC requires that windows, curtain walls, and skylight assemblies (i.e., the glazing, frame, and frame attachment to the wall or roof) have sufficient strength to resist the positive and negative design wind pressure (see Figure 3-38). Design professionals should specify that these assemblies comply with wind load testing in accordance with ASTM E 1233. It is important to specify an adequate load path and to check its continuity during submittal review.

For further general information on windows, see the National Institute of Building Sciences' *Building Envelope Design Guide* (www.wbdg.org/design/envelope.php).

Water Infiltration

Heavy rain accompanied by high winds can cause wind-driven water infiltration problems. The magnitude of the problem increases with the wind speed. Leakage can occur at the glazing/frame interface, the frame itself, or between the frame and wall. When the basic wind speed is greater than 120 mph, because of the very high design wind pressures and numerous opportunities for leakage path development, some leakage should be anticipated when the design wind speed conditions are approached.

Figure 3-38:
Two complete windows, including frames, blew out as a result of an inadequate number of fasteners. Typhoon Paka (Guam, 1997)

The successful integration of windows and curtain walls into exterior walls is a challenge in protecting against water infiltration. To the extent possible when detailing the interface between the wall and the window or curtain wall units, designers should rely on sealants as the secondary line of defense against water infiltration, rather than making the sealant the primary protection. If a sealant joint is the first line of defense, a second line of defense should be designed to intercept and drain water that drives past the sealant joint.

Where corrosion is a problem, use of anodized aluminum or stainless steel frames and stainless steel frame anchors and screws are recommended.

When designing joints between walls and windows and curtain wall units, consider the shape of the sealant joint (i.e., a square joint is typically preferred) and the type of sealant to be specified. The sealant joint should be designed to enable the sealant to bond on only two opposing surfaces (i.e., a backer rod or bond-breaker tape should be specified). Butyl is recommended as a sealant for concealed joints, and polyurethane for exposed joints. During installation, cleanliness of the sealant substrate is important (particularly if polyurethane or silicone sealants are specified), as is the tooling of the sealant. ASTM

The maximum test pressure used in the current ASTM test standard for evaluating resistance of window units to wind-driven rain is well below design wind pressures. Therefore, units that demonstrate adequate wind-driven rain resistance during testing may experience leakage during actual wind events.

E 2112 provides guidance on the design of sealant joints, as well as other information pertaining to the installation of windows, including the use of sill pan flashings with end dams and rear legs (see Figure 3-39). Windows that do not have nailing flanges should typically be installed over a pan flashing. It is recommended that designers use ASTM E 2112 as a design resource.

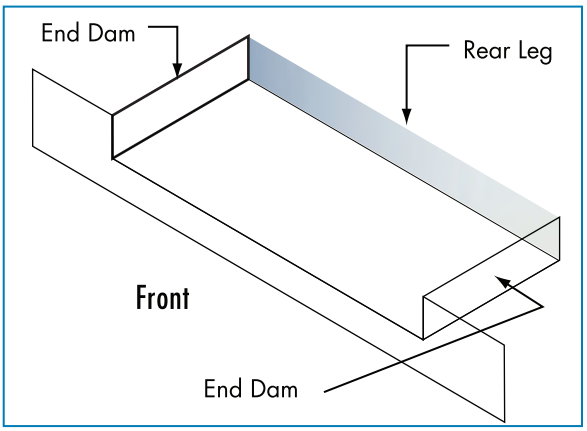


Figure 3-39:
View of a typical window sill pan flashing with end dams and rear legs

SOURCE: ASTM E 2112

Sealant joints can be protected with a removable stop, as illustrated in Figure 3-40. The stop protects the sealant from direct exposure to the weather and reduces the possibility of wind-driven rain penetration.

Where water infiltration protection is particularly demanding and important, it is recommended that onsite water infiltration testing in accordance with ASTM E 1105 be specified.

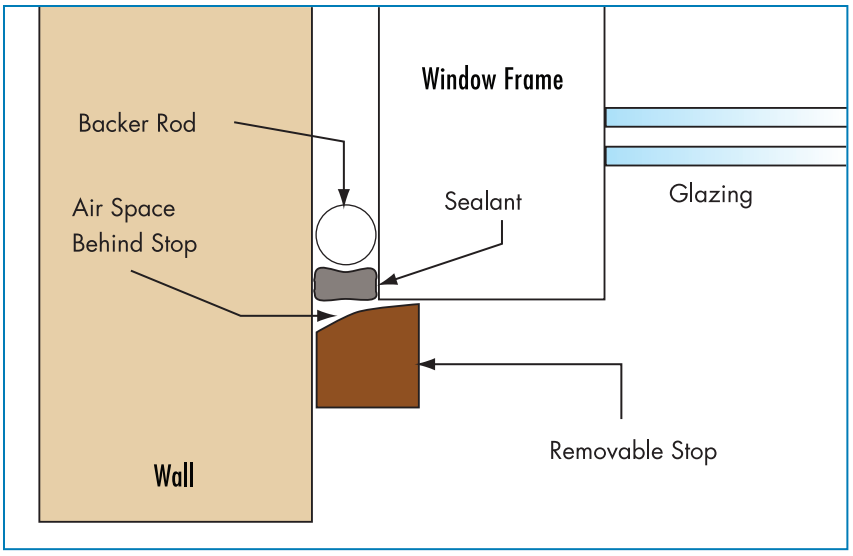


Figure 3-40:
Protecting sealant with a stop retards weathering and reduces the exposure to wind-driven rain.

3.3.3.3 Non-Load-Bearing Walls, Wall Coverings, and Soffits

This section addresses exterior non-load-bearing walls, exterior wall coverings, and soffits, as well as the underside of elevated floors, and provides guidance for interior non-load-bearing masonry walls. See Section 3.4.3.3 for additional information pertaining to critical facilities located in hurricane-prone regions, and Section 3.5 for additional information pertaining to critical facilities located in tornado-prone regions.

For further general information on non-load-bearing walls and wall coverings, see the National Institute of Building Sciences' *Building Envelope Design Guide* (www.wbdg.org/design/envelope.php).

Loads and Resistance

The IBC requires that soffits, exterior non-load-bearing walls, and wall coverings have sufficient strength to resist the positive and negative design wind pressures.

To ensure the continuity of elevator service, elevator penthouse walls must possess adequate wind and water resistance. If the walls blow away or water leaks through the wall system, the elevator controls and/or motors can be destroyed. Loss of elevators may critically affect facility operations because the restoration can take weeks even with expedited work (see Figure 3-41).

Figure 3-41:
The wall covering blew off the penthouse at this hospital complex allowing rainwater to destroy the elevator controls. Hurricane Ivan (Florida, 2004)

Soffits: Depending on the wind direction, soffits can experience either positive or negative pressure. Besides the cost of repairing the damaged soffits, wind-borne soffit debris can cause property damage and injuries (see Figure 3-42). Failed soffits may also provide a convenient path for wind-driven rain to enter the building. Storm-damage research has shown that water blown into attic spaces after the loss of soffits caused significant damage and the collapse of ceilings. At the relatively new fire station shown in Figure 3-43, essentially all of the perforated aluminum soffit was blown away. Wind-driven water entered the attic and saturated the batt insulation, which caused the ceiling boards to collapse. After the storm, the fire station was evacuated solely because of this damage. Even in instances where soffits remain in place, water can penetrate through soffit vents and cause damage. At this time, there are no known specific test standards or design guidelines to help design wind- and water-resistant soffits and soffit vents.

Where corrosion is a problem, stainless steel fasteners are recommended for wall and soffit systems. For other components (e.g., furring, blocking, struts, and hangers), nonferrous components (such as wood), stainless steel, or steel with a minimum of G-90 hot-dipped galvanized coating are recommended. Additionally, access panels are recommended so components within soffit cavities can be periodically inspected for corrosion or dry rot.

Figure 3-42:
This suspended metal soffit was not designed for upward-acting wind pressure. Typhoon Paka (Guam, 1997)

Figure 3-43:
This fire station was abandoned after Hurricane Charley because of soffit failure. (Florida, 2004)

Exterior non-load-bearing masonry walls: Particular care should be given to the design and construction of exterior non-load-bearing masonry walls. Although these walls are not intended to carry gravity loads, they should be designed to resist the external and internal loading for components and cladding in order to avoid collapse. When these types of walls collapse, they represent a severe risk to life because of their great weight (see Figure 3-15).

Interior non-load-bearing masonry walls: Special consideration should also be given to interior non-load-bearing masonry walls. Although these walls are not required by building codes to be designed to resist wind loads, if the exterior glazing is broken, or the exterior doors are blown away, the interior walls could be subjected to significant load as the building rapidly becomes fully pressurized. To avoid casualties, it is recommended that interior non-load-bearing masonry walls adjacent to occupied areas be designed to accommodate loads exerted by a design wind event, using the partially enclosed pressure coefficient (see Figure 3-44). By doing so, wall collapse may be prevented if the building envelope is breached. This recommendation is applicable to critical facilities located in areas with a basic wind speed greater than 120

mph, those used for hurricane shelters, and to critical facilities in tornado-prone regions that do not have shelter space designed in accordance with FEMA 361.

Figure 3-44:
The red arrows show the original location of a CMU wall that nearly collapsed following a rolling door failure. Hurricane Charley (Florida, 2004)

Wall Coverings

There are a variety of exterior wall coverings. Brick veneer, exterior insulation finish systems (EIFS), stucco, metal wall panels, and aluminum and vinyl siding have often exhibited poor wind performance. Veneers (such as ceramic tile and stucco) over concrete, stone veneer, and cement-fiber panels and siding have also blown off. Wood siding and panels rarely blow off. Although tilt-up precast walls have failed during wind storms, precast wall panels attached to steel or concrete framed buildings typically offer excellent wind performance.

Brick veneer:[8] Brick veneer is frequently blown off walls during high winds. When brick veneer fails, wind-driven water can enter and damage buildings, and building occupants can be vulnerable to injury from wind-borne debris (particularly if the walls are

8. The brick veneer discussion is from *Attachment of Brick Veneer in High-Wind Regions - Hurricane Katrina Recovery Advisory* (FEMA, December 2005).

sheathed with plastic foam insulation or wood fiberboard in lieu of wood panels). Pedestrians in the vicinity of damaged walls can also be vulnerable to injury from falling bricks (see Figure 3-45). Common failure modes include tie (anchor) fastener pull-out (see Figure 3-46), failure of masons to embed ties into the mortar (Figure 3-47), poor bonding between ties and mortar, a mortar of poor quality, and tie corrosion.

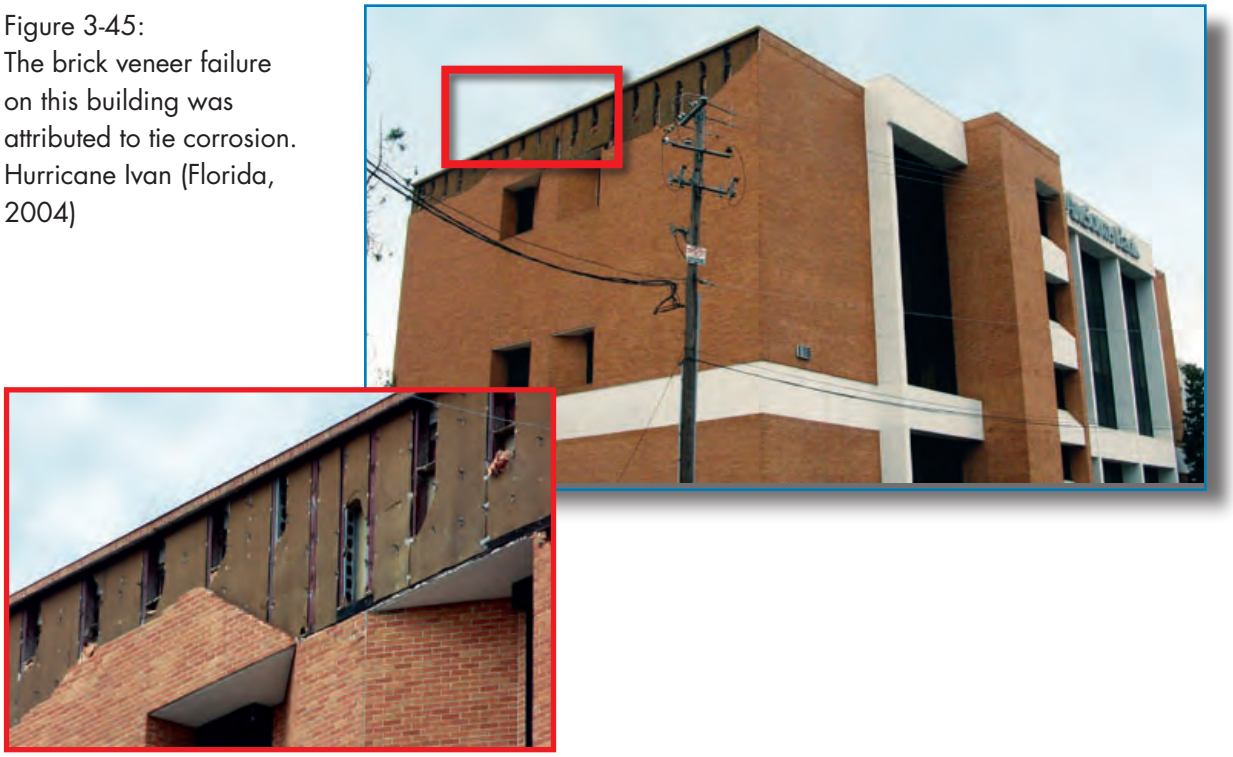


Figure 3-45:
The brick veneer failure on this building was attributed to tie corrosion. Hurricane Ivan (Florida, 2004)

Figure 3-46:
This tie remained embedded in the mortar joint while the smooth-shank nail pulled from the stud.

Figure 3-47:
These four ties were never embedded into the mortar joint.

Ties are often installed before brick laying begins. When this is done, ties are often improperly placed above or below the mortar joints. When misaligned, the ties must be angled up or down to be embedded into the mortar joints (Figure 3-48). Misalignment not only reduces the embedment depth, but also reduces the effectiveness of the ties, because wind forces do not act in parallel direction to the ties.

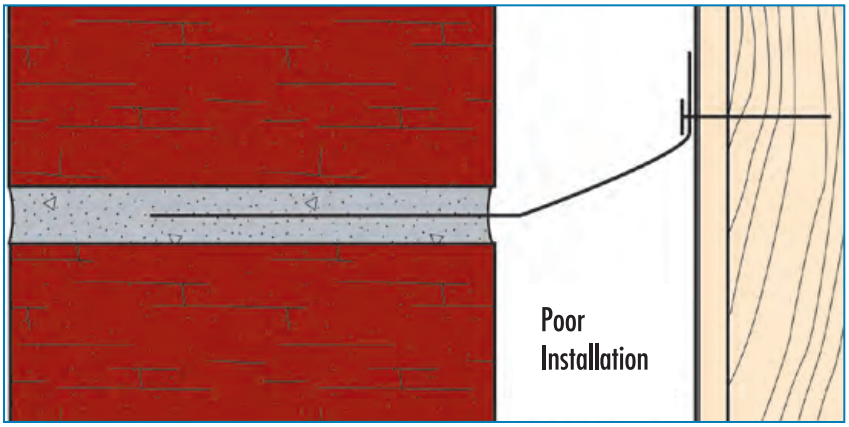


Figure 3-48:
Misalignm ent of the tie reduces the embedment and promotes veneer failure.

Corrugated ties typically used in residential and nursing home veneer construction provide little resistance to compressive loads. The use of compression struts would likely be beneficial, but off-

the-shelf devices do not currently exist. Two-piece adjustable ties (Figure 3-49) provide significantly greater compressive strength than corrugated ties.

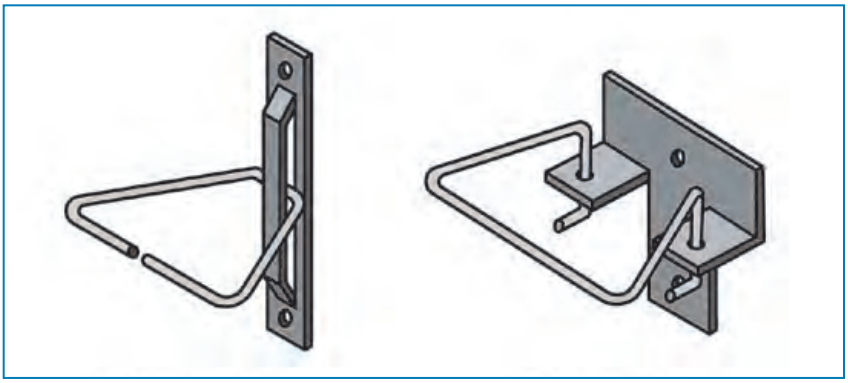


Figure 3-49: Examples of two-piece adjustable ties

The following Brick Industry Association (BIA) technical notes provide guidance on brick veneer: Technical Notes 28: *Anchored Brick Ve- neer, Wood Frame Construction* (2002); Technical Notes 28B: *Brick Veneer/Steel Stud Walls* (2005); and Technical Notes 44B: *Wall Ties* (2003) (available online at www.bia.org). These technical notes provide attachment recommendations; however, they are not specific for high-wind regions. To enhance wind performance of brick veneer, the following are recommended:

- Calculate wind loads and determine tie spacing in accordance with the latest edition of the *Building Code Requirements for Masonry Structures,* ACI 530/ASCE 5/TMS 402 (ACI 530, 1996)). A stud spacing of 16 inches on center is recommended so that ties can be anchored at this spacing.

- Ring-shank nails are recommended in lieu of smooth-shank nails for wood studs. A minimum embedment of 2 inches is suggested.

- For use with wood studs, two-piece adjustable ties are recommended. However, where corrugated steel ties are used, they should be 22-gauge minimum, ⅞-inch wide by 6-inch long, and comply with ASTM A 1008, with a zinc coating complying with ASTM A 153 Class B2. For ties used with steel studs, see BIA Technical Notes 28B, *Brick Veneer/Steel Stud Walls.* Stainless steel ties should be used for both wood and steel studs in areas within 3,000 feet of the coast.

- Install ties as the brick is laid so that the ties are properly aligned with the mortar joints.

- Locate ties within 8 inches of door and window openings, and within 12 inches of the top of veneer sections.

- Although corrugated ties are not recommended, if used, bend the ties at a 90-degree angle at the nail head to minimize tie flexing when the ties are loaded in tension or compression (Figure 3-50).

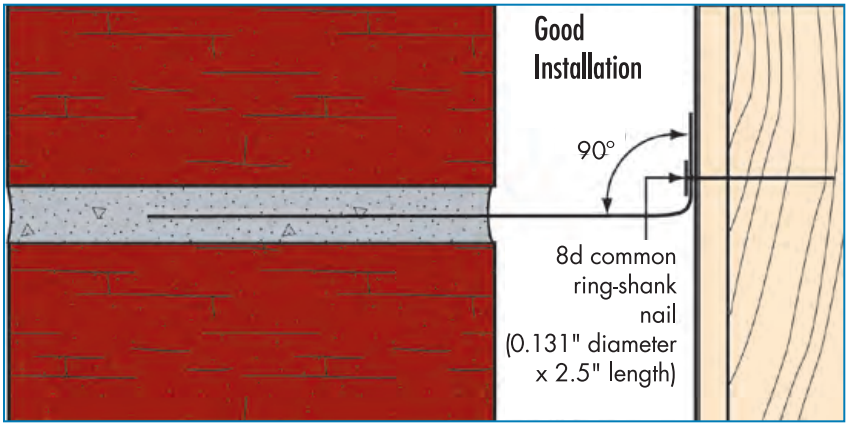


Figure 3-50:
Bend ties at nail heads

- Embed ties in joints so that the mortar completely encapsulates the ties. Embed a minimum of 1½ inches into the bed joint, with a minimum mortar cover of ⅝-inch to the outside face of the wall (see Figure 3-51).

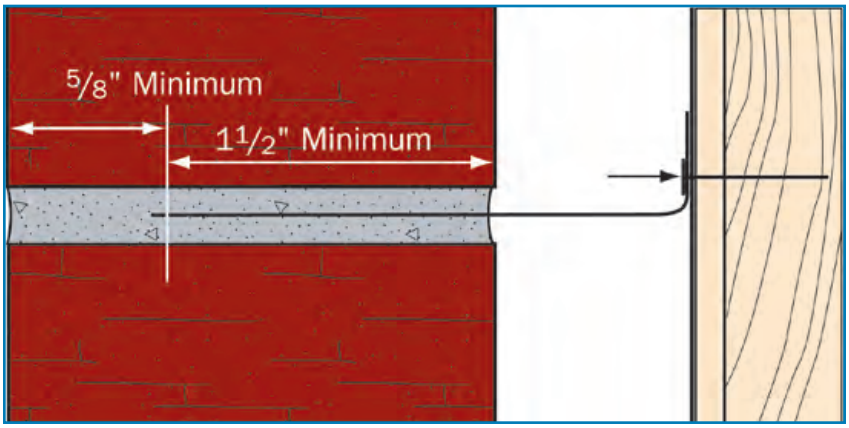


Figure 3-51:
Tie embedment

To avoid water leaking into the building, it is important that weep holes be adequately spaced and not be blocked during brick installation, and that through-wall flashings be properly designed and installed. At the hospital shown in Figure 3-52, water leaked into the building along the base of many of the brick veneer walls. When high winds accompany heavy rain, a substantial amount of water can be blown into the wall cavity.

Figure 3-52:
Water leaked inside along the base of the brick veneer walls. Hurricane Katrina (Louisiana, 2005)

EIFS: Figure 3-53 shows typical EIFS assemblies. Figures 3-41, 3-54, and 4-6 show EIFS blow-off. In these cases, the molded expanded polystyrene (MEPS) was attached to gypsum board, which in turn was attached to metal studs. The gypsum board detached from the studs, which is a common EIFS failure. When the gypsum board on the exterior side of the studs is blown away, it is common for gypsum board on the interior side to also be blown off. The opening allows the building to become fully pressurized and allows the entrance of wind-driven rain. Other common types of failure include wall framing failure, separation of the MEPS from its substrate, and separation of the synthetic stucco from the MEPS.

At the hospital shown in Figure 3-55, the EIFS was applied over a concrete wall. The MEPS debonded from the concrete. In general, a concrete substrate prevents wind and water from entering a building, but if the EIFS debonds from the concrete, EIFS debris can break unprotected glazing.

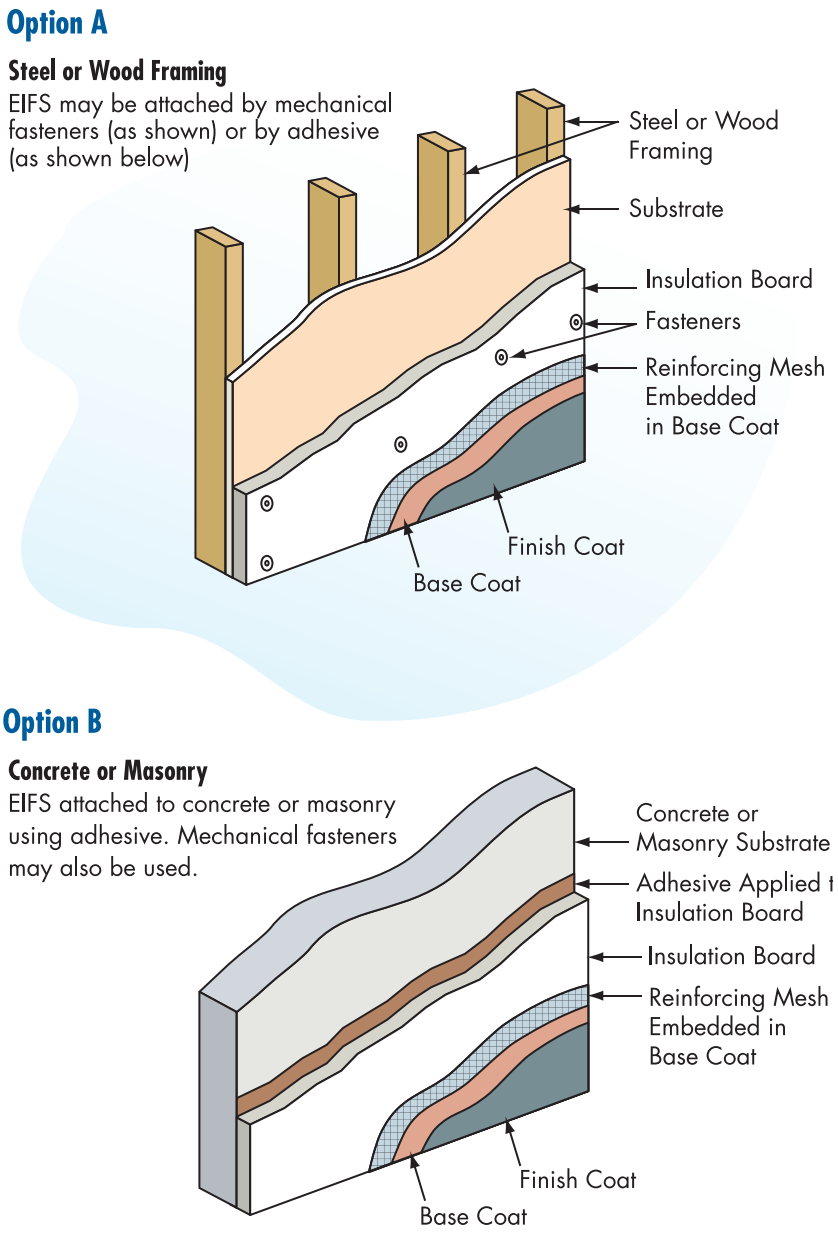
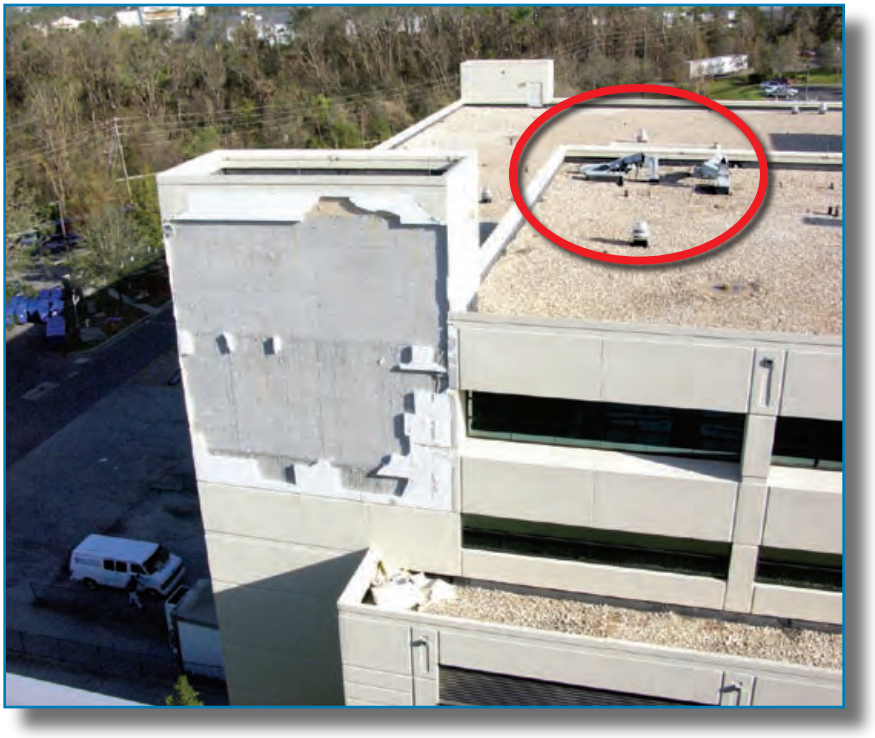


Figure 3-53:
Typical EIFS assemblies

Figure 3-54:
EIFS blow-off at building corner. In places, metal fascia was also blown in. Tornado (Oklahoma, 1999)

Figure 3-55:
EIFS blown off a cast-in-place concrete wall. Note the damaged rooftop ductwork. Hurricane Ivan (Florida, 2004)



Wind-borne EIFS debris can be devastating to unprotected glazing. At the hospital shown in Figure 3-56, the hospital's original concrete wall panels had been furred with metal hat channels and covered with EIFS. In a large corner area, the EIFS and gypsum board substrate blew off the hat channels and broke a large number of windows in the multi-story connecting walkway

between the hospital and the medical office building (MOB). The EIFS debris also broke a large number of windows in the MOB (see Figure 3-56). Glass shards from the MOB punctured the roof membrane over the dialysis unit. The costly damage resulted in loss of several rooms in the MOB and hampered functioning of the hospital complex.

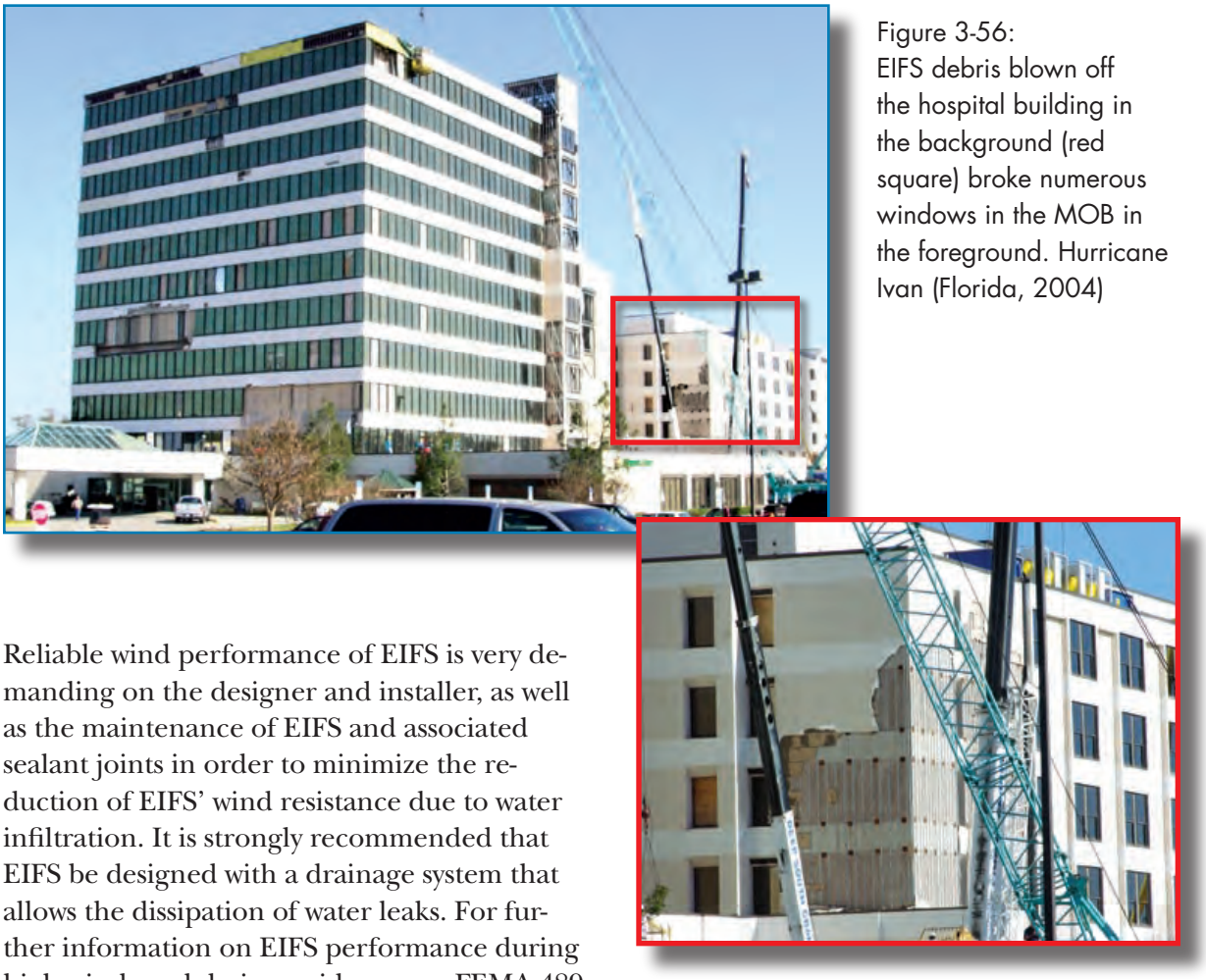


Figure 3-56:
EIFS debris blown off the hospital building in the background (red square) broke numerous windows in the MOB in the foreground. Hurricane Ivan (Florida, 2004)

Reliable wind performance of EIFS is very demanding on the designer and installer, as well as the maintenance of EIFS and associated sealant joints in order to minimize the reduction of EIFS' wind resistance due to water infiltration. It is strongly recommended that EIFS be designed with a drainage system that allows the dissipation of water leaks. For further information on EIFS performance during high winds and design guidance see FEMA 489 and 549.

Another issue associated with EIFS is the potential for judgment errors. EIFS applied over studs is sometimes mistaken for a concrete wall, which may lead people to seek shelter behind it. However, instead of being protected by several inches of concrete, only two layers of gypsum board (i.e., one layer on each side of the studs)

and a layer of MEPS separate the occupants from the impact of wind-borne debris that can easily penetrate such a wall and cause injury.

Stucco: Wind performance of traditional stucco walls is similar to the performance of EIFS, as shown in Figure 3-57. In several areas the metal stud system failed; in other areas the gypsum sheathing blew off the studs; and in other areas, the metal lath blew off the gypsum sheathing. The failure shown in Figure 3-57 illustrates the importance of designing and constructing wall framing (including attachment of stud tracks to the building and attachment of the studs to the tracks) to resist the design wind loads.

Figure 3-57:
The stucco wall failure was caused by inadequate attachment between the stud tracks and the building's structure. Hurricane Ivan (Florida, 2004)

Metal wall panels: Wind performance of metal wall panels is highly variable. Performance depends on the strength of the specified panel (which is a function of material and thickness, panel profile, panel width, and whether the panel is a composite) and the adequacy of the attachment (which can either be by concealed clips or exposed fasteners). Excessive spacing between clips/fasteners is the most common problem. Clip/fastener spacing should be specified, along with the specific type and size of fastener. Figures 3-14 and 3-58 illustrate metal wall panel problems. At the school shown in Figure 3-58 (which was being used as a shelter), the

metal panels were attached with concealed fasteners. The panels unlatched at the standing seams. In addition to generating wind-borne debris, loss of panels allowed wind-driven rain to enter the building.

Figure 3-58:
The loss of metal wall panels allowed a substantial amount of wind-driven rain to penetrate this building. Hurricane Ivan (Florida, 2004)

To minimize water infiltration at metal wall panel joints, it is recommended that sealant tape be specified at sidelaps when the basic wind speed is in excess of 90 mph. However, endlaps should be left unsealed so that moisture behind the panels can be wicked away. Endlaps should be a minimum of 3 inches (4 inches where the basic wind speed is greater than 120 mph) to avoid wind-driven rain infiltration. At the base of the wall, a 3-inch (4-inch) flashing should also be detailed, or the panels should be detailed to overlap with the slab or other components by a minimum of 3 inches (4 inches).

Vinyl siding: Vinyl siding blow-off is typically caused by nails spaced too far apart and/or the use of vinyl siding that has inadequate wind resistance. Vinyl siding is available with enhanced wind resistance features, such as an enhanced nailing hem, greater interlocking area, and greater thickness.

The Vinyl Siding Institute (VSI) sponsors a Certified Installer Program that recognizes individuals with at least 1 year of experience who can demonstrate proper vinyl siding application. If vinyl siding is specified, design professionals should consider specifying that the siding contractor be a VSI-certified installer. For further information on this program, see www.vinylsiding.org.

Secondary line of protection: Almost all wall coverings permit the passage of some water past the exterior surface of the covering, particularly when the rain is wind-driven. For this reason, most wall coverings should be considered water-shedding, rather than waterproofing coverings. To avoid moisture-related problems, it is recommended that a secondary line of protection with a moisture barrier (such as housewrap or asphalt-saturated felt) and flashings around door and window openings be provided. Designers should specify that horizontal laps of the moisture barrier be installed so that water is allowed to drain from the wall (i.e., the top sheet should lap over the bottom sheet so that water running down the sheets remains on their outer surface). The bottom of the moisture barrier needs to be designed to allow drainage. Had the metal wall panels shown in Figure 3-58 been applied over a moisture barrier and sheathing, the amount of water entering the building would have likely been eliminated or greatly reduced.

In areas that experience frequent wind-driven rain, incorporating a rain screen design, by installing vertical furring strips between the moisture barrier and siding materials, will facilitate drainage of water from the space between the moisture barrier and backside of the siding. In areas that frequently experience strong winds, enhanced flashing is recommended. Enhancements include use of flashings that have extra-long flanges, and the use of sealant and tapes. Flashing design should recognize that wind-driven water could be pushed up vertically. The height to which water can be pushed increases with wind speed. Water can also migrate vertically and horizontally by capillary action between layers of materials (e.g., between a flashing flange and housewrap). Use of a rain screen design, in conjunction with enhanced flashing design, is recommended in areas that frequently experience wind-driven rain or strong winds. It is recommended that designers attempt to determine what type of flashing details have successfully been used in the area where the facility will be constructed.

Underside of Elevated Floors

If sheathing is applied to the underside of joists or trusses elevated on piles (e.g., to protect insulation installed between the joists/trusses), its attachment should be specified in order to

avoid blow-off. Stainless steel or hot-dip galvanized nails or screws are recommended. Since ASCE 7 does not provide guidance for load determination, professional judgment in specifying attachment is needed.

3.3.3.4 Roof Systems

Because roof covering damage has historically been the most frequent and the costliest type of wind damage, special attention needs to be given to roof system design. See Section 3.4.3.4 for additional information pertaining to critical facilities located in hurricane-prone regions, and Section 3.5 for critical facilities located in tornado-prone regions.

For further general information on roof systems, see the National Institute of Building Sciences' *Building Envelope Design Guide* (www.wbdg.org/design/envelope.php).

Code Requirements

The IBC requires the load resistance of the roof assembly to be evaluated by one of the test methods listed in IBC's Chapter 15. Design professionals are cautioned that designs that deviate from the tested assembly (either with material substitutions or change in thickness or arrangement) may adversely affect the wind performance of the assembly. The IBC does not specify a minimum safety factor. However, for the roof system, a safety factor of 2 is recommended. To apply the safety factor, divide the test load by 2 to determine the allowable design load. Conversely, multiply the design load by 2 to determine the minimum required test resistance.

For structural metal panel systems, the IBC requires test methods UL 580 or ASTM E 1592. It is recommended that design professionals specify use of E 1592, because it gives a better representation of the system's uplift performance capability.

The roof of the elevator penthouse must possess adequate wind and water resistance to ensure continuity of elevator service. It is recommended that a secondary roof membrane, as discussed in Section 3.3.3.3, be specified over the elevator penthouse roof deck.

Load Resistance

Specifying the load resistance is commonly done by specifying a Factory Mutual Research (FMR) rating, such as FM 1-75. The first number (1) indicates that the roof assembly passed the FMR tests for a Class 1 fire rating. The second number (75) indicates the uplift resistance in pounds per square foot (psf) that the assembly achieved during testing. With a safety factor of two this assembly would be suitable for a maximum design uplift load of 37.5 psf.

The highest uplift load occurs at the roof corners because of building aerodynamics as discussed in Section 3.1.3. The perimeter has a somewhat lower load, while the field of the roof has the lowest load. FMG Property Loss Prevention Data Sheets are formatted so that a roof assembly can be selected for the field of the roof. For the perimeter and corner areas, FMG Data Sheet 1-29 provides three options: 1) use the FMG Approval Guide listing if it includes a perimeter and corner fastening method; 2) use a roof system with the appropriate FMG Approval rating in the field, perimeter, and corner, in accordance with Table 1 in FMG Data Sheet 1-29; or 3) use prescriptive recommendations given in FMG Data Sheet 1-29.

FM Global (FMG) is the name of the Factory Mutual Insurance Company and its affiliates. One of FMG's affiliates, Factory Mutual Research (FMR) provides testing services, produces documents that can be used by designers and contractors, and develops test standards for construction products and systems. FMR evaluates roofing materials and systems for resistance to fire, wind, hail, water, foot traffic and corrosion. Roof assemblies and components are evaluated to establish acceptable levels of performance. Some documents and activities are under the auspices of FMG and others are under FMR.

When perimeter and corner uplift resistance values are based on a prescriptive method rather than testing, the field assembly is adjusted to meet the higher loads in the perimeter and corners by increasing the number of fasteners or decreasing the spacing of adhesive ribbons by a required amount. However, this assumes that the failure is the result of the fastener pulling out from the deck, or that the failure is in the vicinity of the fastener plate, which may not be the case. Also, the increased number of fasteners required by FMG may not be sufficient to comply with the perimeter and corner loads derived from the building code. Therefore, if FMG resistance data are specified, it is prudent for the design professional to specify the resistance for each zone of the roof separately. Using the example cited above, if the field of the roof is specified as 1-75, the perimeter would be specified as 1-130 and the corner would be specified as 1-190.

If the roof system is fully adhered, it is not possible to increase the uplift resistance in the perimeter and corners. Therefore, for fully adhered systems, the uplift resistance requirement should be based on the corner load rather than the field load.

Roof System Performance

Storm-damage research has shown that sprayed polyurethane foam (SPF) and liquid-applied roof systems are very reliable high-wind performers. If the substrate to which the SPF or liquid-applied membrane is applied does not lift, it is highly unlikely that these systems will blow off. Both systems are also more resistant to leakage after missile impact damage than most other systems. Built-up roofs (BURs) and modified bitumen systems have also demonstrated good wind performance provided the edge flashing/coping does not fail (which happens frequently). The exception is aggregate surfacing, which is prone to blow-off (see Figures 3-12 and 3-13). Modified bitumen applied to a concrete deck has demonstrated excellent resistance to progressive peeling after blow-off of the metal edge flashing. Metal panel performance is highly variable. Some systems are very wind-resistant, while others are quite vulnerable.

Of the single-ply attachment methods, the paver-ballasted and fully adhered methods are the least problematic. Systems with aggregate ballast are prone to blow-off, unless care is taken in specifying the size of aggregate and the parapet height (see Figure 3-9). The performance of protected membrane roofs (PMRs) with a factory-applied cementitious coating over insulation boards is highly variable. When these boards are installed over a loose-laid membrane, it is critical that an air retarder be incorporated to prevent the membrane from ballooning and disengaging the boards. ANSI/SPRI RP-4 (which is referenced in the IBC) provides wind guidance for ballasted systems using aggregate, pavers, and cementitious-coated boards.

The National Research Council of Canada, Institute for Research in Construction's Wind Design Guide for Mechanically Attached Flexible Membrane Roofs (B1049, 2005) provides recommendations related to mechanically attached single-ply and modified bituminous systems. B1049 is a comprehensive wind design guide

that includes discussion on air retarders. Air retarders can be effective in reducing membrane flutter, in addition to being beneficial for use in ballasted single-ply systems. When a mechanically attached system is specified, careful coordination with the structural engineer in selecting deck type and thickness is important.

If a steel deck is selected, it is critical to specify that the membrane fasteners be attached in rows perpendicular to the steel flanges to avoid overstressing the attachment of the deck to the deck support structure. At the school shown in Figure 3-59, the fastener rows of the mechanically attached single-ply membrane ran parallel to the top flange of the steel deck. The deck fasteners were overstressed and a portion of the deck blew off and the membrane progressively tore. At another building, shown in Figure 3-60, the membrane fastener rows also ran parallel to the top flange of the steel deck. When membrane fasteners run parallel to the flange, the flange with membrane fasteners essentially carries all the uplift load because of the deck's inability to transfer any significant load to adjacent flanges. Hence, at the joists shown in Figure 3-60, the deck fasteners on either side of the flange with the membrane fasteners are the only connections to the joist that are carrying substantial uplift load.

Figure 3-59:
The orientation of the membrane fastener rows led to blow-off of the steel deck. Hurricane Marilyn (U.S. Virgin Islands, 1995)

Figure 3-60:
View of the underside of a steel deck showing the mechanically attached single-ply membrane fastener rows running parallel to, instead of across, the top flange of the deck.

For metal panel roof systems, the following are recommended:

- When clip or panel fasteners are attached to nailers, detail the connection of the nailer to the nailer support (including the detail of where nailers are spliced over a support).
- When clip or panel fasteners are loaded in withdrawal (tension), screws are recommended in lieu of nails.
- For concealed clips over a solid substrate, it is recommended that chalk lines be specified so that the clips are correctly spaced.
- When the basic wind speed is 110 mph or greater, it is recommended that two clips be used along the eaves, ridges, and hips.
- For copper panel roofs in areas with a basic wind speed greater than 90 mph, it is recommended that Type 316 stainless steel clips and stainless steel screws be used in lieu of copper clips.
- Close spacing of fasteners is recommended at hip and ridge flashings (e.g., spacing in the range of 3 to 6 inches on center, commensurate with the design wind loads.)

Edge Flashings and Copings

Roof membrane blow-off is almost always a result of lifting and peeling of the metal edge flashing or coping, which serves to clamp down the membrane at the roof edge. Therefore, it is important for the design professional to carefully consider the design of metal edge flashings, copings, and the nailers to which they are attached. The metal edge flashing on the modified bitumen membrane roof shown in Figure 3-61 was installed underneath the membrane, rather than on top of it, and then stripped in. In this location, the edge flashing was unable to clamp the membrane down. At one area, the membrane was not sealed to the flashing. An ink pen was inserted into the opening prior to photographing to demonstrate how wind could catch the opening and lift and peel the membrane.

Figure 3-61:
The ink pen shows an opening that the wind can catch, and cause lifting and peeling of the membrane.

ANSI/SPRI ES-1, *Wind Design Standard for Edge Systems Used in Low Slope Roofing Systems* (2003) provides general design guidance including a methodology for determining the outward-acting load on the vertical flange of the flashing/coping (ASCE 7 does not provide this guidance). ANSI/SPRI ES-1 is referenced in the IBC. ANSI/SPRI ES-1 also includes test methods for assessing flashing/coping resistance. This manual recommends a minimum safety factor of 3 for edge flashings, copings, and nailers for critical facil-

ities. For FMG-insured facilities, FMR-approved flashing should be used and FM Data Sheet 1-49 should also be consulted.

The traditional edge flashing/coping attachment method relies on concealed cleats that can deform under wind load and lead to disengagement of the flashing/coping (see Figure 3-62) and subsequent lifting and peeling of the roof membrane (as shown in Figure 3-12). When a vertical flange disengages and lifts up, the edge flashing and membrane are very susceptible to failure. Normally, when a flange lifts such as shown in Figure 3-62, the failure continues to propagate and the metal edge flashing and roof membrane blows off.

Storm-damage research has revealed that, in lieu of cleat attachment, the use of exposed fasteners to attach the vertical flanges of copings and edge flashings has been found to be a very effective and reliable attachment method. The coping shown in Figure 3-63 was attached with ¼-inch diameter stainless steel concrete spikes at 12 inches on center. When the fastener is placed in wood, #12 stainless steel screws with stainless steel washers are recommended. The fasteners should be more closely spaced in the corner areas (the spacing will depend upon the design wind loads). ANSI/SPRI ES-1 provides guidance on fastener spacing and thickness of the coping and edge flashing.

Figure 3-62:
The metal edge flashing disengaged from the continuous cleat and the vertical flange lifted. Hurricane Hugo (South Carolina, 1989)

Figure 3-63:
Both vertical faces of the coping were attached with exposed fasteners instead of concealed cleats. Typhoon Paka (Guam, 1997)

Gutters

Storm-damage research has shown that gutters are seldom constructed to resist wind loads (see Figure 3-64). When a gutter lifts, it typically causes the edge flashing that laps into the gutter to lift as well. Frequently, this results in a progressive lifting and peeling of the roof membrane. The membrane blow-off shown in Figure 3-65 was initiated by gutter uplift. The gutter was similar to that shown in Figure 3-64. The building, housing the county Sheriff's office, suffered water leakage that shut down the county 911 call center, destroyed the crime lab equipment, and caused significant interior water damage.

Figure 3-64:
This gutter, supported by a type of bracket that provides no significant uplift resistance, failed when wind lifted it together with the metal edge flashing that lapped into the gutter. Hurricane Francis (Florida, 2004)

Figure 3-65:
The original modified bitumen membrane was blown away after the gutter lifted in the area shown by the red arrow (the black membrane is a temporary roof). Hurricane Francis (Florida, 2004)

Special design attention needs to be given to attaching gutters to prevent uplift, particularly for those in excess of 6 inches in width. Currently, there are no standards pertaining to gutter wind resistance. It is recommended that the designer calculate the uplift load on gutters using the overhang coefficient from ASCE 7. There are two approaches to resist gutter uplift.

- Gravity-support brackets can be designed to resist uplift loads. In these cases, in addition to being attached at its top, the bracket should also be attached at its low end to the wall. The gutter also needs to be designed so it is attached securely to the bracket in a way that will effectively transfer the gutter uplift load to the bracket. Bracket spacing will depend on the gravity and uplift load, the bracket's strength, and the strength of connections between the gutter/bracket and the bracket/wall. With this option, the bracket's top will typically be attached to a wood nailer, and that fastener will be designed to carry the gravity load. The bracket's lower connection will resist the rotational force induced by gutter uplift. Because brackets are usually spaced close together to carry the gravity load, developing adequate connection strength at the lower fastener is generally not difficult.

- The other option is to use gravity-support brackets only to resist gravity loads, and use separate sheet-metal straps at 45-degree angles to the wall to resist uplift loads. Strap spacing

will depend on the gutter uplift load and strength of the connections between the gutter/strap and the strap/wall. Note that FMG Data Sheet 1-49 recommends placing straps 10 feet apart. However, at that spacing with wide gutters, fastener loads induced by uplift are quite high. When straps are spaced at 10 feet, it can be difficult to achieve sufficiently strong uplift connections.

When designing a bracket's lower connection to a wall or a strap's connection to a wall, designers should determine appropriate screw pull-out values. With this option, a minimum of two screws at each end of a strap is recommended. At a wall, screws should be placed side by side, rather than vertically aligned, so the strap load is carried equally by the two fasteners. When fasteners are vertically aligned, most of the load is carried by the top fastener.

Since the uplift load in the corners is much higher than the load between the corners, enhanced attachment is needed in corner areas regardless of the option chosen. ASCE 7 provides guidance about determining a corner area's length.

Parapet Base Flashings

Information on loads for parapet base flashings was first introduced in the 2002 edition of ASCE 7. The loads on base flashings are greater than the loads on the roof covering if the parapet's exterior side is air-permeable. When base flashing is fully adhered, it has sufficient wind resistance in most cases. However, when base flashing is mechanically fastened, typical fastening patterns may be inadequate, depending on design wind conditions (see Figure 3-66). Therefore, it is imperative that the base flashing loads be calculated, and attachments be designed to accommodate these loads. It is also important for designers to specify the attachment spacing in parapet corner regions to differentiate them from the regions between corners.

Figure 3-66:
If mechanically attached base flashings have an insufficient number of fasteners, the base flashing can be blown away. Hurricane Andrew (Florida, 1992)

Steep-Slope Roof Coverings

For a discussion of wind performance of asphalt shingle and tile roof coverings, see FEMA 488 (2005), 489 (2005), and 549 (2006). For recommendations pertaining to asphalt shingles and tiles, see Fact Sheets 19, 20, and 21 in FEMA 499 (2005).

3.3.4 NONSTRUCTURAL SYSTEMS AND EQUIPMENT

Nonstructural systems and equipment include all components that are not part of the structural system or building envelope. Exterior-mounted mechanical equipment (e.g., exhaust fans, HVAC units, relief air hoods, rooftop ductwork, and boiler stacks), electrical equipment (e.g., light fixtures and lightning protection systems), and communications equipment (e.g., antennae and satellite dishes) are often damaged during high winds. Damaged equipment can impair the operation of the facility, the equipment can detach and become wind-borne missiles, and water can enter the facility where equipment was displaced (see Figures 3-20, 3-67, 3-68, 3-72, 3-76, and 3-78). The most common problems typically relate to inadequate equipment anchorage, inadequate strength of the equipment itself, and corrosion.

Figure 3-67:
Toppled rooftop mechanical equipment. Hurricane Andrew (Florida, 1992)

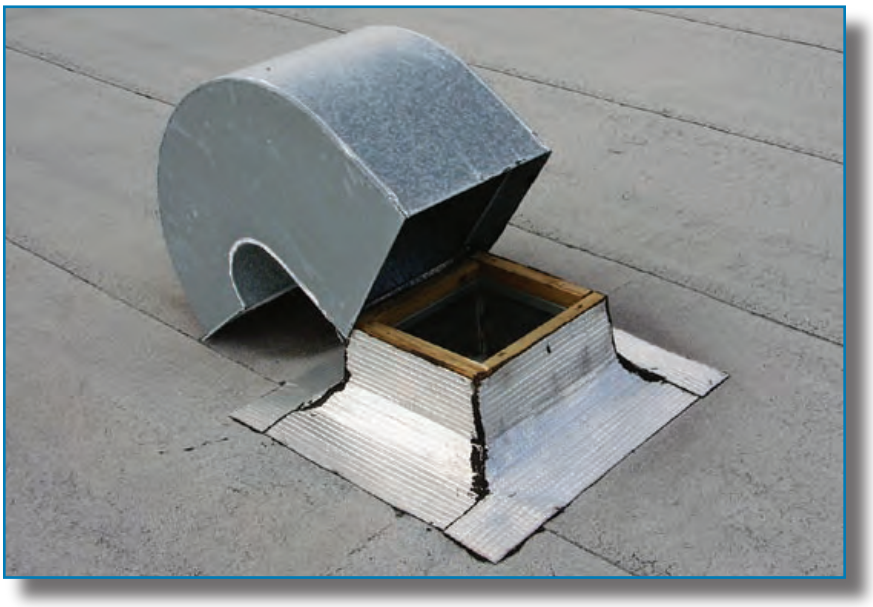


Figure 3-68:
This gooseneck was attached with only two small screws. A substantial amount of water was able to enter the building during Hurricane Francis. (Florida, 2004)

See Section 3.4.4 for additional information pertaining to critical facilities located in hurricane-prone regions.

3.3.4.1 Exterior-Mounted Mechanical Equipment

This section discusses loads and attachment methods, as well as the problems of corrosion and water infiltration.

Loads and Attachment Methods[9]

Information on loads on rooftop equipment was first introduced in the 2002 edition of ASCE 7. For guidance on load calculations, see "Calculating Wind Loads and Anchorage Requirements for Rooftop Equipment" (ASHRAE, 2006). A minimum safety factor of 3 is recommended for critical facilities. Loads and resistance should also be calculated for heavy pieces of equipment since the dead load of the equipment is often inadequate to resist the design wind load. The 30' x 10' x 8' 18,000-pound HVAC unit shown in Figure 3-69 was attached to its curb with 16 straps (one screw per strap). Although the wind speeds were estimated to be only 85 to 95 miles per hour (peak gust), the HVAC unit blew off the medical office building.

To anchor fans, small HVAC units, and relief air hoods, the minimum attachment schedule provided in Table 3-2 is recommended. The attachment of the curb to the roof deck also needs to be designed and constructed to resist the design loads.

Mechanical penetrations through the elevator penthouse roof and walls must possess adequate wind and water resistance to ensure continuity of elevator service (see Section 3.3.3.3). In addition to paying special attention to equipment attachment, air intakes and exhausts should be designed and constructed to prevent wind-driven water from entering the penthouse.

Figure 3-69: Although this 18,000-pound HVAC unit was attached to its curb with 16 straps, it blew off during Hurricane Ivan. (Florida, 2004)

9. Discussion is based on: *Attachment of Rooftop Equipment in High-wind Regions—Hurricane Katrina Recovery Advisory* (May 2006, revised July 2006)

Table 3-2: Number of #12 Screws for Base Case Attachment of Rooftop Equipment

Case No	Curb Size and Equipment Type	Equipment Attachment	Fastener Factor for Each Side of Curb or Flange
1	12″ x 12″ Curb with Gooseneck Relief Air Hood	Hood Screwed to Curb	1.6
2	12″ x 12″ Gooseneck Relief Air Hood with Flange	Flange Screwed to 22 Gauge Steel Roof Deck	2.8
3	12″ x 12″ Gooseneck Relief Air Hood with Flange	Flange Screwed to 15/32″ OSB Roof Deck	2.9
4	24″ x 24″ Curb with Gooseneck Relief Air Hood	Hood Screwed to Curb	4.6
5	24″ x 24″ Gooseneck Relief Air Hood with Flange	Flange Screwed to 22 Gauge Steel Roof Deck	8.1
6	24″ x 24″ Gooseneck Relief Air Hood with Flange	Flange Screwed to 15/32″ OSB Roof Deck	8.2
7	24″ x 24″ Curb with Exhaust Fan	Fan Screwed to Curb	2.5
8	36″ x 36″ Curb with Exhaust Fan	Fan Screwed to Curb	3.3
9	5′-9″ x 3′- 8″ Curb with 2′- 8″ high HVAC Unit	HVAC Unit Screwed to Curb	4.5*
10	5′-9 ″x 3′- 8″ Curb with 2′- 8″ high Relief Air Hood	Hood Screwed to Curb	35.6*

Notes to Table 3-2:

1. The loads are based on ASCE 7-05. The resistance includes equipment weight.
2. The Base Case for the tabulated numbers of #12 screws (or ¼ pan-head screws for flange-attachment) is a 90-mph basic wind speed, 1.15 importance factor, 30′ building height, Exposure C, using a safety factor of 3.
3. For other basic wind speeds, multiply the tabulated number of #12 screws by $\left(\frac{V_D^2}{90^2}\right)$ to determine the required number of #12 screws (or ¼ pan-head screws) required for the desired basic wind speed, V_D (mph).
4. For other roof heights up to 200′, multiply the tabulated number of #12 screws by (1.00 + 0.003 [h - 30]) to determine the required number of #12 screws or ¼ pan-head screws for buildings between 30′ and 200′.

   Example A: 24″ x 24″ exhaust fan screwed to curb (table row 7), Base Case conditions (see Note 1): 2.5 screws per side; therefore, round up and specify 3 screws per side.

   Example B: 24″ x 24″ exhaust fan screwed to curb (table row 7), Base Case conditions, except 120 mph: $120^2 \times 1 \div 90^2$ = 1.78 x 2.5 screws per side = 4.44 screws per side; therefore, round down and specify 4 screws per side.

   Example C: 24″ x 24″ exhaust fan screwed to curb (table row 7), Base Case conditions, except 150′ roof height: 1.00 + 0.003 (150′ - 30′) = 1.00 + 0.36 = 1.36 x 2.5 screws per side = 3.4 screws per side; therefore, round down and specify 3 screws per side.

* This factor only applies to the long sides. At the short sides, use the fastener spacing used at the long sides.

Fan cowling attachment: Fans are frequently blown off their curbs because they are poorly attached. When fans are well attached, the cowlings frequently blow off (see Figure 3-70). Blown off cowlings can tear roof membranes and break glazing. Unless the fan manufacturer specifically engineered the cowling attachment to resist the design wind load, cable tie-downs (see Figure 3-71) are recommended to avoid cowling blow-off. For fan cowlings less than 4 feet in diameter, 1/8-inch diameter stainless steel cables are recommended. For larger cowlings, use 3/16-inch diameter cables. When the basic wind speed is 120 mph or less, specify two cables. Where the basic wind speed is greater than 120 mph, specify four cables. To minimize leakage potential at the anchor point, it is recommended that the cables be adequately anchored to the equipment curb (rather than anchored to the roof deck). The attachment of the curb itself also needs to be designed and specified.

To avoid corrosion-induced failure (see Figure 3-78), it is recommended that exterior-mounted mechanical, electrical, and communications equipment be made of nonferrous metals, stainless steel, or steel with minimum G-90 hot-dip galvanized coating for the equipment body, stands, anchors, and fasteners. When equipment with enhanced corrosion protection is not available, the designer should advise the building owner that periodic equipment maintenance and inspection is particularly important to avoid advanced corrosion and subsequent equipment damage during a windstorm.

Figure 3-70: Cowlings blew off two of the fans on a police building that housed the county's EOC. Hurricane Ivan (Florida, 2004)

Figure 3-71:
Cables were attached to prevent the cowling from blowing off. Typhoon Paka (Guam, 1997)

Ductwork: To avoid wind and wind-borne debris damage to rooftop ductwork, it is recommended that ductwork not be installed on the roof (see Figure 3-72). If ductwork is installed on the roof, it is recommended that the ducts' gauge and the method of attachment be able to resist the design wind loads.

Figure 3-72:
Two large openings remained (circled area and inset to the left) after the ductwork on this roof blew away. Hurricane Katrina (Mississippi, 2005)

Condenser attachment: In lieu of placing rooftop-mounted condensers on wood sleepers resting on the roof (see Figure 3-73), it is recommended that condensers be anchored to equipment stands. The attachment of the stand to the roof deck also needs to be designed to resist the design loads. In addition to anchoring the base of the condenser to the stand, two metal straps with two side-by-side #12 screws or bolts with proper end and edge distances at each strap end are recommended when the basic wind speed is greater than 90 mph (see Figure 3-74).

Three publications pertaining to seismic restraint of equipment provide general information on fasteners and edge distances:

- *Installing Seismic Restraints for Mechanical Equipment* (FEMA 412, 2002)
- *Installing Seismic Restraints for Electrical Equipment* (FEMA 413, 2004)
- *Installing Seismic Restraints for Duct and Pipe* (FEMA 414, 2004)

Figure 3-73: Sleeper-mounted condensers displaced by high winds. Hurricane Katrina (Mississippi, 2005)

Vibration isolators: If vibration isolators are used to mount equipment, only those able to resist design uplift loads should be specified and installed, or an alternative means to accommodate uplift resistance should be provided (see Figure 3-75).

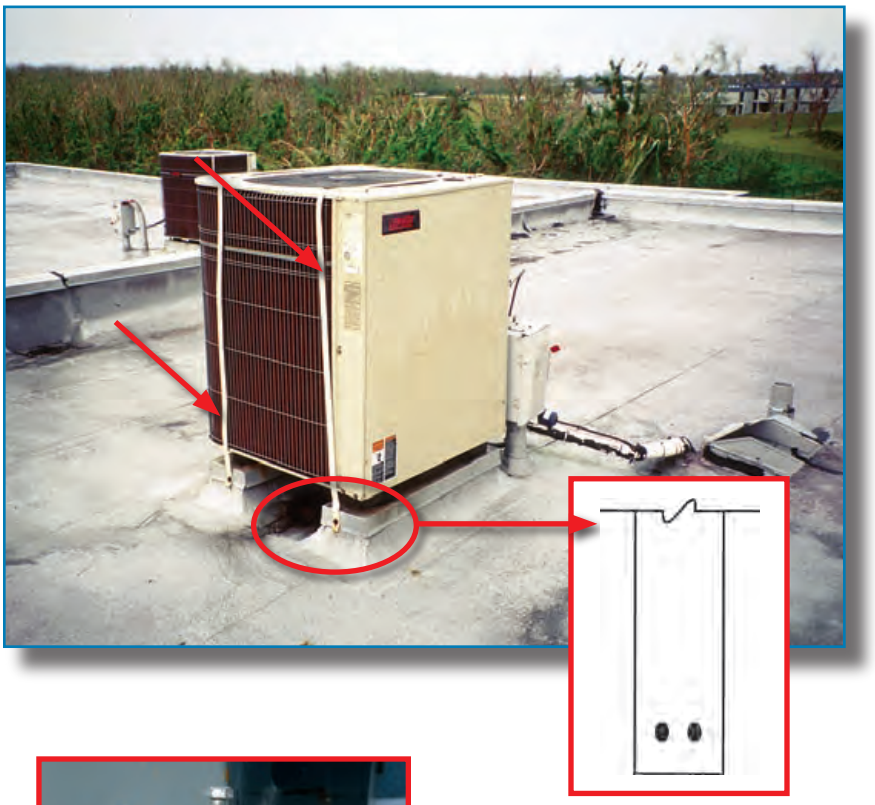


Figure 3-74:
This condenser had supplemental attachment straps (see red arrows). Typhoon Paka (Guam, 1997)

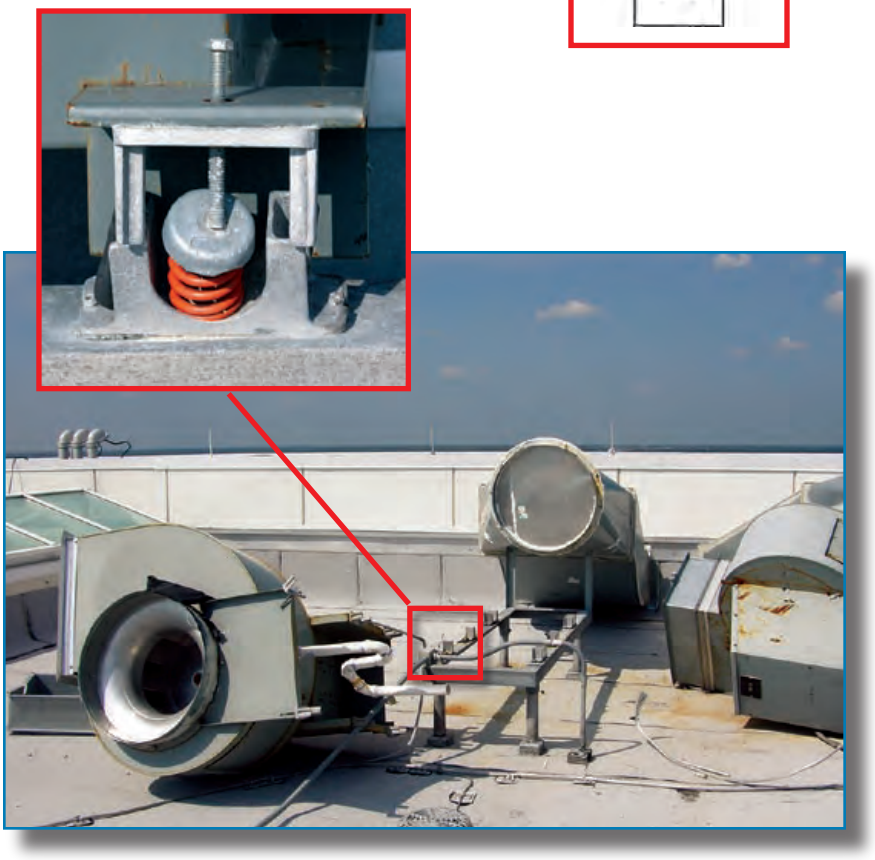


Figure 3-75:
Failure of vibration isolators that provided lateral resistance but no uplift resistance caused equipment damage. A damaged vibration isolator is shown in the inset. Hurricane Katrina (Mississippi, 2005)

Boiler and exhaust stack attachment: To avoid wind damage to boiler and exhaust stacks, wind loads on stacks should be calculated and guy-wires should be designed and constructed to resist the loads. Toppled stacks, as shown at the hospital in Figure 3-76, can allow water to enter the building at the stack penetration, damage the roof membrane, and become wind-borne debris. The designer should advise the building owner that guy-wires should be inspected annually to ensure they are taut.

Figure 3-76:
Three of the five stacks that did not have guy-wires were blown down. Hurricane Marilyn (U.S. Virgin Islands, 1995)

Access panel attachment: Equipment access panels frequently blow off. To minimize this, job-site modifications, such as attaching hasps and locking devices like carabiners, are recommended. The modification details need to be customized. Detailed design may be needed after the equipment has been delivered to the job site. Modification details should be approved by the equipment manufacturer.

Equipment screens: Screens around rooftop equipment are frequently blown away (see Figure 3-77). Screens should be designed to resist the wind load derived from ASCE 7. Since the effect of screens on equipment wind loads is unknown, the equipment attachment behind the screens should be designed to resist the design load.

Figure 3-77:
Equipment screen panels, such as these blown away at a hospital, can break glazing, puncture roof membranes, and cause injury. Hurricane Ivan (Florida, 2004)

Water Infiltration

During high winds, wind-driven rain can be driven through air intakes and exhausts unless special measures are taken. Louvers should be designed and constructed to prevent leakage between the louver and wall. The louver itself should be designed to avoid water being driven past the louver. However, it is difficult to prevent infiltration during very high winds. Designing sumps with drains that will intercept water driving past louvers or air intakes should be considered. ASHRAE 62.1 (2004) provides some information on rain and snow intrusion. The *Standard 62.1 User's Manual* provides additional information, including examples and illustrations of various designs.

3.3.4.2 Exterior-Mounted Electrical and Communications Equipment

Damage to exterior-mounted electrical equipment is infrequent, mostly because of its small size (e.g., disconnect switches). Exceptions include communication towers, surveillance cameras, electrical service masts, satellite dishes, and lightning protection systems. The damage is typically caused by inadequate mounting as a result of failure to perform wind load calculations and anchorage design. Damage is also sometimes caused by corrosion (see Figure 3-78 and the text box on corrosion in Section 3.3.4.1).

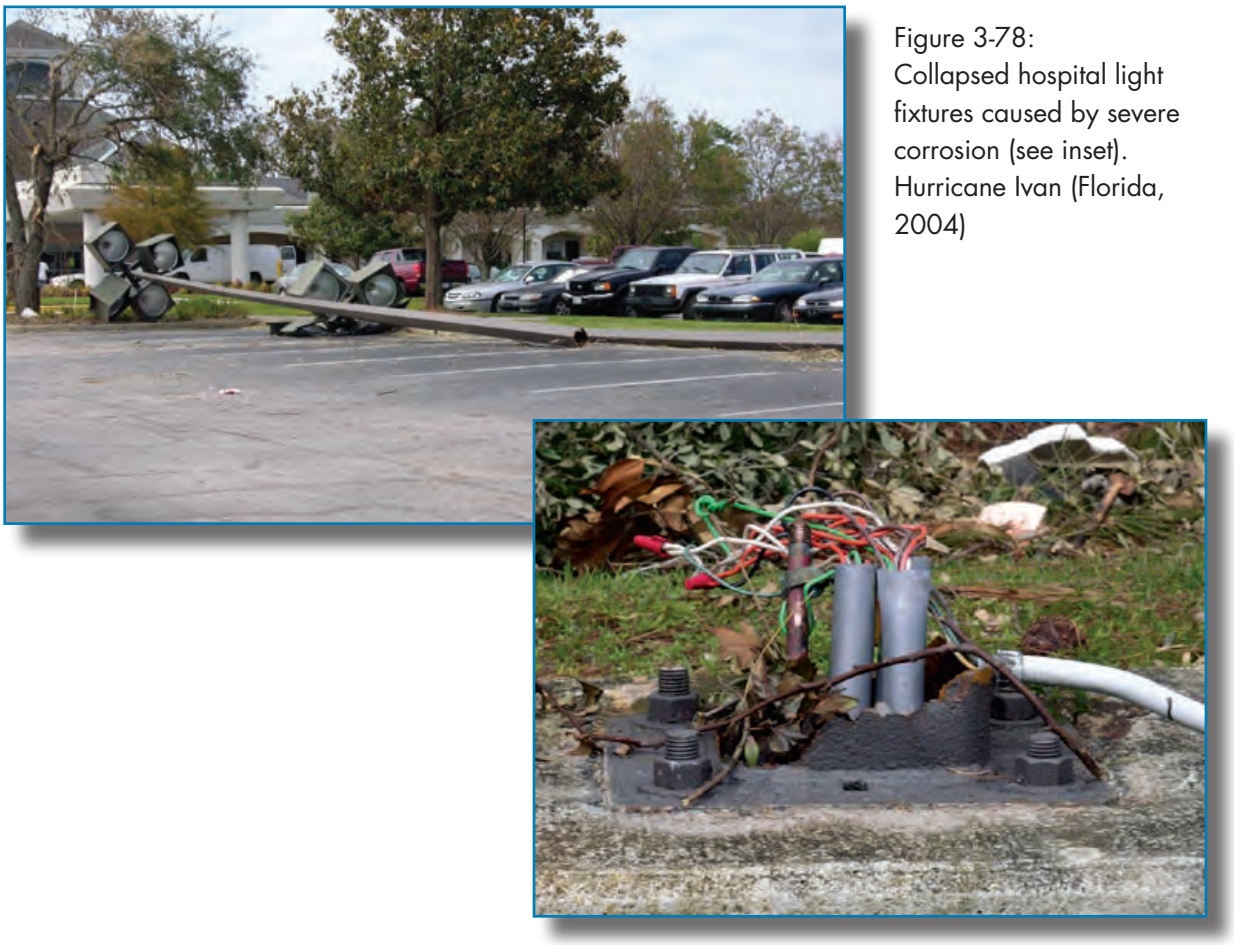


Figure 3-78:
Collapsed hospital light fixtures caused by severe corrosion (see inset). Hurricane Ivan (Florida, 2004)

Communication towers and poles: ANSI/C2 provides guidance for determining wind loads on power distribution and transmission poles and towers. AASHTO LTS-4-M (amended by LTS-4-12, 2001 and 2003, respectively) provides guidance for determining wind loads on light fixture poles (standards).

Both ASCE 7 and ANSI/TIA-222-G contain wind load provisions for communication towers (structures). The IBC allows the use of either approach. The ASCE wind load provisions are generally consistent with those contained in ANSI/TIA-222-G. ASCE 7, however, contains provisions for dynamically sensitive towers that are not present in the ANSI/TIA standard. ANSI/TIA classifies towers according to their use (Class I, Class II, and Class III). This manual recommends that towers (including antennae) that are mounted on, located near, or serve critical facilities be designed as Class III structures.

Collapse of both large and small communication towers at emergency operation centers, fire and police stations, and hospitals is quite common during high-wind events (see Figures 3-79 and 3-80). These failures often result in complete loss of communication capabilities. In addition to the disruption of communications, collapsed towers can puncture roof membranes and allow water leakage into the facilities, unless the roof system incorporated a secondary membrane (as discussed in Section 3.4.3.4). At the tower shown in Figure 3-79 the anchor bolts were pulled out of the deck, which resulted in a progressive peeling of the fully adhered single-ply roof membrane. Tower collapse can also injure or kill people.

See Section 3.3.1.1 regarding site considerations for light fixture poles, power poles, and electrical and communications towers.

Figure 3-79:
The collapse of the antenna tower caused progressive peeling of the roof membrane. Also note that the exhaust fan blew off the curb, but the high parapet kept it from blowing off the roof. Hurricane Andrew (Florida, 1992)

Figure 3-80:
The antenna tower at this fire station buckled. Hurricane Katrina (Mississippi, 2005)

Electrical service masts: Service mast failure is typically caused by collapse of overhead power lines, which can be avoided by using underground service. Where overhead service is provided, it is recommended that the service mast not penetrate the roof. Otherwise, a downed service line could pull on the mast and rupture the roof membrane.

Satellite dishes: For the satellite dish shown in Figure 3-81, the dish mast was anchored to a large metal pan that rested on the roof membrane. CMU was placed on the pan to provide overturning resistance. This anchorage method should only be used where calculations demonstrate that it provides sufficient resistance. In this case the wind approached the satellite dish in such a way that it experienced very little wind pressure. In hurricane-prone regions, use of this anchorage method is not recommended (see Figure 3-82).

Lightning protection systems: For attachment of lightning protection systems on buildings higher than 100 feet above grade, and for buildings located where the basic wind speed is in excess of 90 mph, see Section 3.4.4.3.

Figure 3-81:
Common anchoring method for satellite dish. Hurricane Ivan (Florida, 2004)

Figure 3-82:
A satellite dish anchored similarly to that shown in Figure 3-81 was blown off of this five-story building. Hurricane Charley (Florida, 2004)

The recommendations given in Section 3.3 are summarized in Table 3-3.

Table 3-3: Risk Reduction Design Methods

Site and General Design	See Section 3.3.1
Exposure	Locate in Exposure B if possible. Avoid escarpments and upper half of hills.
Presence of trees or poles	Locate to avoid blow-down on facility.
Site access	Minimum of two roads.
General design issues	See recommendations in Section 3.3.1.2.
Wind loads on MWFRS, building envelope and rooftop equipment	Use ASCE 7 or local building code, whichever procedure results in highest loads.
Load resistance	Determine via calculations and/or text data. Give load resistance criteria in contract documents, and clearly indicate load path continuity.
Durability	Give special attention to material selection and detailing to avoid problems of corrosion, wood decay, and termite attack.
Rain penetration	Detail to minimize wind-driven rain penetration into the building.
Structural Systems (MWFRS)	**See Section 3.3.2**
Pre-engineered metal buildings	Take special steps to ensure structure is not vulnerable to progressive collapse.
Exterior load-bearing walls	Design as MWFRS and C&C. Reinforce CMU. Sufficiently connect precast concrete panels.
Roof decks	Concrete, steel, or wood sheathing is recommended. Attach steel decks with screws. Use special fasteners for wood sheathing. Anchor precast concrete to resist wind loads. If FMG-rated assembly, deck must comply with FMG criteria.
Walkways and canopies	Use pressure coefficients from ASCE 7.
Exterior Doors	**See Section 3.3.3.1**
Door, frame, and frame fasteners	Must be able to resist positive and negative design load, verified by ASTM E 1233 testing. Specify type, size, and spacing of frame fasteners.
Water infiltration	Consider vestibules, door swing, and weatherstripping. Refer to ASTM E 2112 (2001) for design guidance.

Table 3-3: Risk Reduction Design Methods (continued)

Windows and Skylights	See Section 3.3.3.2
Glazing, frame, and frame fasteners	Must be able to resist positive and negative design load, verified by ASTM E 1233 (2000) testing. Specify type, size, and spacing of frame fasteners.
Water infiltration	Carefully design junctures between walls and windows/ curtain walls. Avoid relying on sealant as the first or only line of defense. Refer to ASTM E 2112 for design guidance. Where infiltration protection is demanding, conduct onsite water infiltration testing per ASTM E 1105 (2000).
Non-Load-Bearing Walls, Wall Coverings, and Soffits	**See Section 3.3.3.3**
Exterior non-load-bearing walls, wall coverings, soffits, and elevated floors	See recommendations in Section 3.3.3.3
Load resistance	Must be able to resist positive and negative design load. Design as C&C.
Elevator penthouses	Design to prevent water infiltration at walls, roof, and mechanical penetrations.
Soffits	Design to resist wind and wind-driven water infiltration.
Interior non-load-bearing masonry walls	Design for wind load per Section 3.3.3.3.
Brick veneer	See recommendations in Section 3.3.3.3.
Secondary protection	Provide moisture barrier underneath wall coverings that are water-shedding.
Roof Systems	**See Section 3.3.3.4**
Testing	Avoid designs that deviate from a tested assembly. If deviation is evident, perform rational analysis. For structural metal panel systems, test per ASTM E 1592 (2000).
Load resistance for field, perimeter, and corner areas	Specify requirements. See recommendations in Section 3.3.3.4.
Edge flashings and copings	Follow ANSI/SPRI ES-1 (2003). Use a safety factor of three.
Gutters	Calculate loads and design attachment to resist uplift.
System selection	Select systems that offer high reliability, commensurate with the wind-regime at facility location.

Table 3-3: Risk Reduction Design Methods (continued)

Roof Systems (continued)	See Section 3.3.3.4
Mechanically attached modified bitumen and single-ply membrane systems	Refer to *Wind Design Guide for Mechanically Attached Flexible Membrane Roofs,* B1049 (NRCC, 2005).
Metal panel systems	See recommendations in Section 3.3.3.4.
Parapet base flashing	Calculate loads and resistance. This is particularly important if base flashing is mechanically attached.
Asphalt shingles and tile coverings	See Fact Sheets 19, 20, or 21 in FEMA 499.
Exterior-mounted Mechanical, Electrical, and Communications Equipment	**See Section 3.3.4**
Load resistance	Specify anchorage of all rooftop and wall-mounted equipment. Use a safety factor of three for equipment anchorage. See recommendations in Section 3.3.4.1.
Equipment strength	Specify cable tie-downs for fan cowlings. Specify hasps and locking devises for equipment access panels. See recommendations in Section 3.3.4.1.
Rooftop satellite dishes	Design the attachment to resist the design wind loads.
Antennae towers	See recommendations in Section 3.3.4.2.
After Completing Contract Documents	**See Section 3.3.1**
Peer review	Consider peer review of contract documents. See Section 3.3.1.2.
Submittals	Ensure required documents are submitted, including all necessary information. Verify that each submittal demonstrates the load path through the system and into its supporting element. See Section 3.3.1.3.
Field observations	Analyze design to determine which elements are critical to ensuring high-wind performance. Determine observation frequency of critical elements. See Section 3.3.1.3.
Post-occupancy inspections, maintenance, and repair	Advise the building owner of the importance of periodic inspections, special inspections after unusually high winds, maintenance, and timely repair. See Section 3.3.1.4.

3.4 BEST PRACTICES IN HURRICANE-PRONE REGIONS

This section presents the general design and construction practice recommendations for critical facilities located in hurricane-prone regions. These recommendations are additional to the ones presented in Section 3.3 and in many cases supersede those recommendations. Critical facilities located in hurricane-prone regions require special design and construction attention because of the unique characteristics of this type of windstorm. Hurricanes can bring very high winds that last for many hours, which can lead to material fatigue failures. The variability of wind direction increases the probability that the wind will approach the building at the most critical angle. Hurricanes also generate a large amount of wind-borne debris, which can damage various building components and cause injury and death.

Although all critical facilities in hurricane-prone regions require special attention, three types of facilities are particularly important because of their function or occupancy: EOCs, healthcare facilities, and shelters. EOCs serve as centralized management hubs for emergency operations. The loss of an EOC can severely affect the overall response and recovery in the area. Healthcare facilities normally have vulnerable occupants (patients) at the time of a hurricane, and afterwards, many injured people seek medical care. Significant damage to a facility can put patients at risk and jeopardize delivery of care to those seeking treatment. Shelters often have a large number of occupants. The collapse of a shelter building or entrance of wind-borne debris into a shelter has the potential to injure or kill many people. See

Chapter 4 for information on the performance of some EOCs, healthcare facilities, shelters, and other types of critical facilities that were affected by Hurricane Katrina.

In order to ensure continuity of service during and after hurricanes, the design, construction, and maintenance of the following critical facilities should be very robust to provide sufficient resiliency to withstand the effects of hurricanes.

EOCs: Communications are important for most types of critical facilities, but for EOCs they are vital. To inhibit disruption of operations, water infiltration that could damage electrical equipment must be prevented, antenna towers need to be strong enough to resist the wind, and the emergency and standby power system needs to remain operational.

Healthcare facilities: Full or partial evacuation of a hospital prior to, during, or after a hurricane, is time consuming, expensive, and for some patients, potentially life-threatening. Water infiltration that could damage electrical equipment or medical supplies, or inhibit the use of critical areas (such as operating rooms and nursing floors) needs to be prevented. The emergency and standby power systems need to remain operational and be adequately sized to power all needed circuits, including the HVAC system. Provisions are needed for water and sewer service in the event of loss of municipal services, and antenna towers need to be strong enough to resist the wind.

Nursing homes are often no more hurricane-resistant than residential buildings. Evacuating these facilities (particularly skilled nursing homes and facilities caring for patients with Alzheimer's disease) can be difficult. Except for antenna towers, the issues identified for hospitals are applicable to nursing homes.

Shelters: During and after hurricanes, these facilities are often occupied by more than 1,000 people. The primary purpose of shelters is to protect occupants from injury or death as a result of building collapse or entrance of wind-borne debris. However, beyond meeting this basic requirement, providing a degree of occupant comfort during a stressful time is important. The building's design and construction should avoid significant water infiltration and provide at least a minimum level of lighting and mechanical ventilation using emergency generators. Shelters should also have provisions for sewage service (such as portable toilets) in the event of loss of municipal water or sewer service.

FEMA recommends that shelters be designed in accordance with FEMA 361, *Design and Construction Guidance for Community Shelters (2000).*

3.4.1 SITE AND GENERAL DESIGN CONSIDERATIONS

Via ASCE 7, the 2006 edition of the IBC has only one special wind-related provision pertaining to Category III and IV buildings in hurricane-prone regions. It pertains to glazing protection within wind-borne debris regions (as defined in ASCE 7). This single additional requirement does not provide adequate protection for occupants of a facility during a hurricane, nor does it ensure a critical facility will remain functional during and after a hurricane. A critical facility may comply with IBC but still remain vulnerable to water and missile penetration through the roof or walls. To provide occupant protection, the exterior walls and the roof must be designed and constructed to resist wind-borne debris as discussed in Sections 3.4.2 and 3.4.3.

The following recommendations are made regarding siting:

- Locate poles, towers, and trees with trunks larger than 6 inches in diameter away from primary site access roads so that they do not block access to, or hit, the facility if toppled.

- Determine if existing buildings within 1,500 feet of the new facility have aggregate surfaced roofs. If roofs with aggregate surfacing are present, it is recommended that the aggregate be removed to prevent it from impacting the new facility. Aggregate removal may necessitate reroofing or other remedial work in order to maintain the roof's fire or wind resistance.

- In cases where multiple buildings, such as hospitals or school campuses, are occupied during a storm, it is recommended that enclosed walkways be designed to connect the buildings. The enclosed walkways (above- or below-grade) are particularly important for protecting people moving between buildings during a hurricane (e.g., to retrieve equipment or supplies) or for situations when it is necessary to evacuate occupants from one building to another during a hurricane.

Figure 3-83: Open walkways do not provide protection from wind-borne debris. Hurricane Katrina (Mississippi, 2005)

3.4.2 STRUCTURAL SYSTEMS

Because of the exceptionally good wind performance and wind-borne debris resistance that reinforced cast-in-place concrete structures offer, a reinforced concrete roof deck and reinforced concrete or reinforced and fully grouted CMU exterior walls are recommended as follows:

Roof deck: A minimum 4-inch thick cast-in-place reinforced concrete deck is the preferred deck. Other recommended decks are minimum 4-inch thick structural concrete topping over steel decking, and precast concrete with an additional minimum 4-inch structural concrete topping.

If these recommendations are not followed for critical facilities located in areas where the basic wind speed is 100 mph or greater, it is recommended that the roof assembly be able to resist complete penetration of the deck by the "D" missile specified in ASTM E 1996 (2005, see text box in Section 3.4.3.1).

If precast concrete is used for the roof or wall structure, the connections should be carefully designed, detailed, and constructed.

Exterior load-bearing walls: A minimum 6-inch thick cast-in-place concrete wall reinforced with #4 rebars at 12 inches on center each way is the preferred wall. Other recommended walls are a minimum 8-inch thick fully grouted CMU reinforced with #4 rebars in each cell, and precast concrete that is a minimum 6 inches thick and reinforced equivalent to the recommendations for cast-in-place walls.

3.4.3 BUILDING ENVELOPE

The design considerations for building envelope components of critical facilities in hurricane-prone regions include a number of additional recommendations. The principal concern that must be addressed is the additional risk from wind-borne debris and water leakage, as discussed below.

3.4.3.1 Exterior Doors

Although the ASCE-7 wind-borne debris provisions only apply to glazing within a portion of hurricane-prone regions, it is recommended that all critical facilities located where the basic wind speed is 100 mph or greater comply with the following recommendations:

- To minimize the potential for missiles penetrating exterior doors and striking people inside the facility, it is recommended that doors (with and without glazing) be designed to resist the "E" missile load specified in ASTM E 1996. The doors should be tested in accordance with ASTM E 1886 (2005). The test assembly should include the door, door frame, and hardware.

- It is recommended that the doors on shelters meet the wind pressure and missile resistance criteria found in FEMA 361. Information on door assemblies that meet these criteria is included in FEMA 361.

ASTM E 1996 specifies five missile categories, A through E. The missiles are of various weights and fired at various velocities during testing. Building type (critical or non-critical) and basic wind speed determine the missiles required for testing. Of the five missiles, the E missile has the greatest momentum. Missile E is required for critical facilities located where the basic wind speed is greater than or equal to 130 mph. Missile D is permitted where the basic wind speed is less than130 mph. FEMA 361 also specifies a missile for shelters. The shelter missile has much greater momentum than the D and E missiles, as shown below:

Missile	Missile Weight	Impact Speed	Momentum
ASTM E 1996—D	9 pound 2x4 lumber	50 feet per second (34 mph)	14 lb_f -s*
ASTM E 1996—E	9 pound 2x4 lumber	80 feet per second (55 mph)	22 lb_f -s*
FEMA 361 (Shelter Missile)	15 pound 2x4 lumber	147 feet per second (100 mph)	68 lb_f -s*

*lb_f -s = pounds force per second

3.4.3.2 Windows and Skylights

Exterior glazing that is not impact-resistant (such as laminated glass or polycarbonate) or protected by shutters is extremely susceptible to breaking if struck by wind-borne debris. Even small, low-momentum missiles can easily break glazing that is not protected (see Figures 3-84 and 3-85). At the hospital shown in Figure 3-84, approximately 400 windows were broken. Most of the breakage was caused by wind-blown aggregate from the hospital's aggregate ballasted single-ply membrane roofs, and aggregate from built-up roofs. With broken windows, a substantial amount of water can be blown into a building, and the internal air pressure can be greatly increased (as discussed in Section 3.1.3) which may damage the interior partitions and ceilings.

Figure 3-84:
Plywood panels (black continuous bands) installed after the glass spandrel panels were broken by roof aggregate. Hurricane Katrina (Mississippi, 2005)

Figure 3-85:
A small piece of asphalt shingle (red arrow) broke the window at this nursing home. Hurricane Katrina (Mississippi, 2005)

In order to minimize interior damage, the IBC, through ASCE 7, prescribes that exterior glazing in wind-borne debris regions be impact-resistant, or be protected with an impact-resistant covering (shutters). For Category III and IV buildings in areas with a basic wind speed of 130 mph or greater, the glazing is required to resist a larger momentum test missile than would Category II buildings and Category III and IV buildings in areas with wind speeds of less than 130 mph.

ASCE 7 refers to ASTM E 1996 for missile loads and to ASTM E 1886 for the test method to be used to demonstrate compliance with the E 1996 load criteria. In addition to testing impact resistance, the window unit is subjected to pressure cycling after test missile impact to evaluate whether the window can still resist wind loads. If wind-borne debris glazing protection is provided by shutters, the glazing is still required by ASCE 7 to meet the positive and negative design air pressures.

Although the ASCE 7 wind-borne debris provisions only apply to glazing within a portion of hurricane-prone regions, it is recommended that all critical facilities located where the basic wind speed is 100 mph or greater comply with the following recommendations:

- To minimize the potential for missiles penetrating exterior glazing and injuring people, it is recommended that exterior glazing up to 60 feet above grade be designed to resist the test Missile E load specified in ASTM E 1996 (see text box in Section 3.4.3.1). In addition, if roofs with aggregate surfacing are present within 1,500 feet of the facility, glazing above 60 feet should be designed to resist the test Missile A load specified in ASTM E 1996. The height of the protected glazing should extend a minimum of 30 feet above the aggregate surfaced roof per ASCE 7.

  Because large missiles are generally flying at lower elevations, glazing that is more than 60 feet above grade and meets the test Missile A load should be sufficient. However, if the facility is within a few hundred feet of another building that may create debris such as EIFS, tiles, or rooftop equipment, it is recommended that the test Missile E load be specified instead of the Missile A for the upper-level glazing.

- For those facilities where glazing resistant to bomb blasts is desired, the windows and glazed doors can be designed to accommodate wind pressure, missile loads, and blast pressure. However, the window and door units need to be tested for missile loads and cyclic air pressure, as well as for blast. A unit that meets blast criteria will not necessarily meet the E 1996 and E 1886 criteria, and vice versa.

For further information on designing glazing to resist blast, see the "Blast Safety" resource pages of the National Institute of Building Sciences' *Building Envelope Design Guide* (www.wbdg.org/design/enve-lope.php).

With the advent of building codes requiring glazing protection in wind-borne debris regions, a variety of shutter designs have entered the market. Shutters typically have a lower initial cost than laminated glass. However, unless the shutter is permanently anchored to the building (e.g., an accordion shutter), storage space will be needed. Also, when a hurricane is forecast, costs will be incurred each time shutters are installed and removed. The cost and difficulty of shutter deployment and demobilization on upper-level glazing may be avoided by using motorized shutters, although laminated glass may be a more economical solution. For further information on shutters, see Section 3.6.2.2.

3.4.3.3 Non-Load-Bearing Walls, Wall Coverings, and Soffits

For buildings not constructed using concrete roof decks and concrete or CMU walls (as recommended), shelters can be constructed within buildings for occupant protection. FEMA 320—*Taking Shelter From the Storm: Building a Safe Room Inside Your Home (2004)* describes how restrooms and storage rooms can be designed for sheltering inside new and existing buildings.

It is recommended that wood-framed and pre-engineered metal buildings in areas with a basic wind speed of 100 mph or greater, that will be occupied during a hurricane, have a designated storage room(s), office(s), or small conference room(s) designed in accordance with FEMA 320 to protect the occupants. Although FEMA 320 is intended for residential construction, the guidance is suitable for small shelters inside critical facilities such as fire and police stations. For large shelters, FEMA 361 criteria are recommended.

In order to achieve enhanced missile resistance of non-load-bearing exterior walls, the wall types discussed in Section 3.4.2 (i.e., reinforced concrete, or reinforced and fully grouted CMU) are recommended.

To minimize long-term problems with exterior wall coverings and soffits, it is recommended that they be avoided to the maximum extent possible. Exposed or painted reinforced concrete or CMU offers greater reliability (i.e., they have no coverings that can blow off and become wind-borne debris).

For all critical facilities located where the basic wind speed is 100 mph or greater that are not constructed using reinforced concrete or reinforced and fully grouted CMU (as is recommended in this manual), it is recommended that the wall system selected be sufficient to resist complete penetration of the wall by the "E" missile specified in ASTM E 1996.

For interior non-load-bearing masonry walls in critical facilities located where the basic wind speed is greater than 120 mph, see the recommendations given in Section 3.3.3.3.

3.4.3.4 Roof Systems

The following types of roof systems are recommended for critical facilities in hurricane-prone regions, because they are more likely to avoid water infiltration if the roof is hit by wind-borne debris, and also because these systems are less likely to become sources of wind-borne debris:

- In tropical climates where insulation is not needed above the roof deck, specify either liquid-applied membrane over cast-in-place concrete deck, or modified bitumen membrane torched directly to primed cast-in-place concrete deck.

- Install a secondary membrane over a concrete deck (if another type of deck is specified, a cover board may be needed over the deck). Seal the secondary membrane at perimeters and penetrations. Specify rigid insulation over the secondary membrane. Where the basic wind speed is up to 110 mph, a minimum 2-inch thick layer of insulation is recommended. Where the speed is between 110 and 130 mph, a total minimum thickness of 3 inches is recommended (installed in two layers). Where the speed is greater than 130 mph, a total minimum thickness of 4 inches is recommended (installed in two layers). A layer of ⅝-inch thick glass mat gypsum roof board is recommended over the insulation, followed by a modified bitumen membrane. A modified bitumen membrane is recommended for the primary membrane because of its somewhat enhanced resistance to puncture by small missiles compared with other types of roof membranes.

  The purpose of the insulation and gypsum roof board is to absorb missile energy. If the primary membrane is punctured or blown off during a storm, the secondary membrane should provide watertight protection unless the roof is hit with missiles of very high momentum that penetrate the insulation and secondary membrane. Figure 3-86 illustrates the merit of

specifying a secondary membrane. The copper roof blew off the hospital's intensive care unit (ICU). Patients and staff were frightened by the loud noise generated by the metal panels as they banged around during the hurricane. Fortunately there was a very robust underlayment (a built-up membrane) that remained in place. Since only minor leakage occurred, the ICU continued to function.

Figure 3-86:
Because this roof system incorporated a secondary membrane, the ICU was not evacuated after the copper roof blew off. Hurricane Andrew (Florida, 1992)

- For an SPF roof system over a concrete deck, where the basic wind speed is less than 130 mph, it is recommended that the foam be a minimum of 3 inches thick to avoid missile penetration through the entire layer of foam. Where the speed is greater than 130 mph, a 4-inch minimum thickness is recommended. It is also recommended that the SPF be coated, rather than protected with an aggregate surfacing.

- For a PMR, it is recommended that pavers weighing a minimum of 22 psf be specified. In addition, base flashings should be protected with metal (such as shown in Figure 3-93) to provide debris protection. Parapets with a 3 foot minimum height (or higher if so indicated by ANSI/SPRI RP-4, 2002) are recommended at roof edges. This manual recommends that PMRs not be used for critical facilities in hurricane-prone regions where the basic wind speed exceeds 130 mph.

- For structural metal roofs, it is recommended that a roof deck be specified, rather than attaching the panels directly to purlins as is commonly done with pre-engineered metal buildings. If panels blow off buildings without roof decking, as shown in Figure 3-17, wind-borne debris and rain are free to enter the building.

  Structural standing seam metal roof panels with concealed clips and mechanically seamed ribs spaced at 12 inches on center are recommended. If the panels are installed over a concrete deck, a modified bitumen secondary membrane is recommended if the deck has a slope less than ½:12. If the panels are installed over a steel deck or wood sheathing, a modified bitumen secondary membrane (over a suitable cover board when over steel decking) is recommend, followed by rigid insulation and metal panels. Where the basic wind speed is up to 110 mph, a minimum 2-inch thick layer of insulation is recommended. Where the speed is between 110 and 130 mph, a total minimum thickness of 3 inches is recommended. Where the speed is greater than 130 mph, a total minimum thickness of 4 inches is recommended. Although some clips are designed to bear on insulation, it is recommended that the panels be attached to wood nailers attached to the deck, because nailers provide a more stable foundation for the clips.

  If the metal panels are blown off or punctured during a hurricane, the secondary membrane should provide watertight protection unless the roof is hit with missiles of very high momentum. At the roof shown in Figure 3-87, the structural standing seam panel clips bore on rigid insulation over a steel deck. Had a secondary membrane been installed over the steel deck, the membrane would have likely prevented significant interior water damage and facility disruption.

Roofs over rooms used to store important records (such as police station evidence rooms) should incorporate secondary membranes to avoid water leakage damage. To preclude water infiltration damage from exterior walls, avoid locating important storage rooms adjacent to exterior walls.

- Based on field performance of architectural metal panels in hurricane-prone regions, exposed fastener panels are recommended in lieu of architectural panels with concealed clips. For panel fasteners, stainless steel screws are recommended. A secondary membrane protected with insulation is recommended, as discussed above for structural standing seam systems.

Figure 3-87:
Significant interior water damage and facility interruption occurred after the standing seam roof blew off. Hurricane Marilyn (U.S. Virgin Islands, 1995)

In order to avoid the possibility of roofing components blowing off and striking people arriving at a critical facility during a storm, the following roof systems are not recommended: aggregate surfacings either on BUR (shown in Figure 3-12), single-plies (shown in Figure 3-9), or SPF; lightweight concrete pavers; cementitious-coated insulation boards; slate; and tile (see Figure 3-88). Even when slates and tiles are properly attached to resist wind loads, their brittleness makes them vulnerable to breakage as a result of wind-borne debris impact. The tile and slate fragments can be blown off the roof, and fragments can damage other parts of the roof causing a cascading failure.

Figure 3-88:
Brittle roof coverings, like slate and tile, can be broken by missiles, and tile debris can break other tiles. Hurricane Charley (Florida, 2004)

Mechanically attached and air-pressure equalized single-ply membrane systems are susceptible to massive progressive failure after missile impact, and are therefore not recommended for critical facilities in hurricane-prone regions. At the building shown in Figure 3-89, a missile struck the fully adhered low-sloped roof and slid into the steep-sloped reinforced mechanically attached single-ply membrane in the vicinity of the red arrow. A large area of the mechanically attached membrane was blown away as a result of progressive membrane tearing. Fully adhered single-ply membranes are very vulnerable to missiles (see Figure 3-90) and are not recommended unless they are ballasted with pavers.

Figure 3-89:
Mechanically attached single-ply membrane progressively tore after being cut by wind-borne debris. Hurricane Andrew (Florida, 1992)

Figure 3-90:
Fully adhered single-ply roof membrane struck by a large number of missiles. Hurricane Marilyn (U.S. Virgin Islands, 1995)

Edge flashings and copings: If cleats are used for attachment, it is recommended that a "peel-stop" bar be placed over the roof membrane near the edge flashing/coping, as illustrated in Figure 3-91. The purpose of the bar is to provide secondary protection against membrane lifting and peeling in the event that edge flashing/coping fails. A robust bar specifically made for bar-over mechanically attached single-ply systems is recommended. The bar needs to be very well anchored to the parapet or the deck. Depending on design wind loads, spacing between 4 and 12 inches on center is recommended. A gap of a few inches should be left between each bar to allow for water flow across the membrane. After the bar is attached, it is stripped over with a stripping ply.

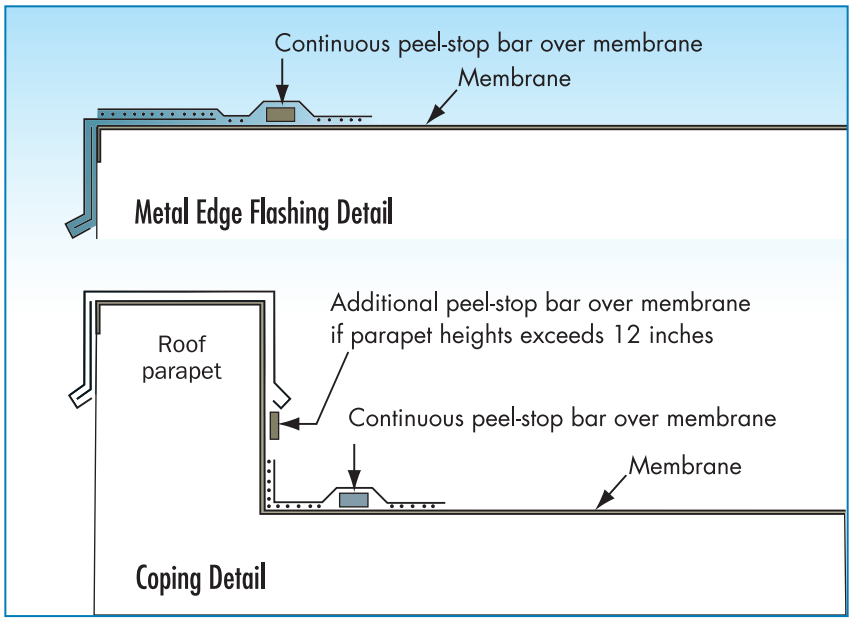


Figure 3-91:
A continuous peel-stop bar over the membrane may prevent a catastrophic progressive failure if the edge flashing or coping is blown off. (Modified from FEMA 55, 2000)

Walkway pads: Roof walkway pads are frequently blown off during hurricanes (Figure 3-92). Pad blow-off does not usually damage the roof membrane. However, wind-borne pad debris can damage other building components and injure people. Walkway pads are therefore not recommended in hurricane-prone regions.

Figure 3-92:
To avoid walkway pad blow off, as occurred on this hospital roof, walkway pads are not recommended. Hurricane Charley (Florida, 2004)

Parapets: For low-sloped roofs, minimum 3-foot high parapets are recommended. With parapets of this height or greater, the uplift load in the corner region is substantially reduced (ASCE 7 permits treating the corner zone as a perimeter zone). Also, a high parapet (as shown in Figures 3-78 and 3-118) may intercept wind-borne debris and keep it from blowing off the roof and damaging other building components or injuring people. To protect base flashings from wind-borne debris damage and subsequent water leakage, it is recommended that metal panels on furring strips be installed over the base flashing (Figure 3-93). Exposed stainless steel screws are recommended for attaching the panels to the furring strips because using exposed fasteners is more reliable than using concealed fasteners or clips (as were used for the failed panels shown in Figure 3-58).

Figure 3-93:
Base flashing protected by metal panels attached with exposed screws. Hurricane Katrina (Mississippi, 2005)

3.4.4 NONSTRUCTURAL SYSTEMS AND EQUIPMENT

Nonstructural systems and equipment include all components that are not part of the structural system or building envelope. Exterior-mounted equipment is especially vulnerable to hurricane-induced damage, and special attention should be paid to positioning and mounting of these components in hurricane-prone regions.

3.4.4.1 Elevators

Where interruption of elevator service would significantly disrupt facility operations, it is recommended that elevators be placed in separate locations within the building and be served by separate elevator penthouses. This is recommended, irrespective of the elevator penthouse enhancements recommended in Sections 3.3.3 and 3.3.4, because of the greater likelihood that at least one of the elevators will remain operational and therefore allow the facility to function as intended.

3.4.4.2 Mechanical Penthouses

By placing equipment in mechanical penthouses rather than leaving them exposed on the roof, equipment can be shielded from high-wind loads and wind-borne debris. Although screens (such as shown in Figure 3-77) could be designed and constructed to protect equipment from horizontally-flying debris, they are not effective in protecting equipment from missiles that have an angular trajectory. It is therefore recommended that mechanical equipment be placed inside mechanical penthouses. The penthouse itself should be designed and constructed in accordance with the recommendations given in Sections 3.4.2 and 3.4.3.

3.4.4.3 Lightning Protection Systems (LPS)

Lightning protection systems frequently become disconnected from rooftops during hurricanes. Displaced LPS components can puncture and tear roof coverings, thus allowing water to leak into buildings (see Figures 3-94 and 3-95). Prolonged and repeated slashing of the roof membrane by loose conductors ("cables") and puncturing by air terminals ("lightning rods") can result in lifting and peeling of the membrane. Also, when displaced, the LPS is no longer capable of providing lightning protection in the vicinity of the displaced conductors and air terminals.

Figure 3-94:
A displaced air terminal that punctured the membrane in several locations. Hurricane Marilyn (U.S. Virgin Islands, 1995)

Figure 3-95:
View of an end of a conductor that became disconnected. Hurricane Katrina (Mississippi, 2005)

Lightning protection standards such as NFPA 780 and UL 96A provide inadequate guidance for attaching LPS to rooftops in hurricane-prone regions, as are those recommendations typically provided by LPS and roofing material manufacturers. LPS conductors are typically attached to the roof at 3-foot intervals. The conductors are flexible, and when they are exposed to high winds, the conductors exert dynamic loads on the conductor connectors ("clips"). Guidance for calculating the dynamic loads does not exist. LPS conductor connectors typically have prongs to anchor the conductor. When the connector is well-attached to the roof surface, during high winds the conductor frequently bends back the malleable connector prongs (see Figure 3-96). Conductor connectors have also debonded from roof surfaces during high winds. Based on observations after Hurricane Katrina and other hurricanes, it is apparent that pronged conductor connectors typically have not provided reliable attachment.

Figure 3-96:
The conductor deformed the prongs under wind pressure, and pulled away from the connector. Hurricane Katrina (Mississippi, 2005)

To enhance the wind performance of LPS, the following are recommended:[10]

Parapet attachment: When the parapet is 12 inches high or greater, it is recommended that the air terminal base plates and conductor connectors be mechanically attached with #12 screws that have minimum 1¼-inch embedment into the inside face of the parapet nailer and are properly sealed for watertight protection. Instead of conductor connectors that have prongs, it is recommended that mechanically attached looped connectors be installed (see Figure 3-97).

Figure 3-97: This conductor was attached to the coping with a looped connector. Hurricane Katrina (Mississippi, 2005)

Attachment to built-up, modified bitumen, and single-ply membranes: For built-up and modified bitumen membranes, attach the air terminal base plates with asphalt roof cement. For single-ply membranes, attach the air terminal base plates with pourable sealer (of the type recommended by the membrane manufacturer).

In lieu of attaching conductors with conductor connectors, it is recommended that conductors be attached with strips of membrane installed by the roofing contractor. For built-up and modified bitumen membranes, use strips of modified bitumen cap sheet, approximately 9 inches wide at a minimum. If strips

10. Discussion is based on *Rooftop Attachment of Lightning Protection Systems in High-Wind Regions—Hurricane Katrina Recover Advisory* (May 2006, Revised July 2006).

are torch-applied, avoid overheating the conductors. For single-ply membranes, use self-adhering flashing strips, approximately 9 inches wide at a minimum. Start the strips approximately 3 inches from either side of the air terminal base plates. Use strips that are approximately 3 feet long, separated by a gap of approximately 3 inches (see Figure 3-98).

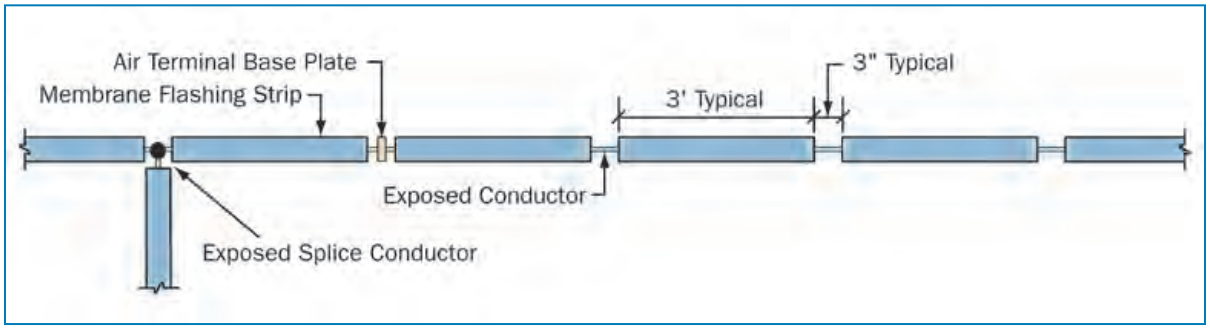


Figure 3-98: Plan showing conductor attachment

As an option to securing the conductors with stripping plies, conductor connectors that do not rely on prongs could be used (such as the one shown in Figure 3-99). However, the magnitude of the dynamic loads induced by the conductor is unknown, and there is a lack of data on the resistance provided by adhesively-attached connectors. For this reason, attachment with stripping plies is the preferred option, because the plies shield the conductor from the wind. If adhesive-applied conductor connectors are used, it is recommended that they be spaced more closely than the 3-foot spacing required by NFPA 780 and UL 96A. Depending on wind loads, a spacing of 6 to 12 inches on center may be needed in the corner regions of the roof, with a spacing of 12 to 18 inches on center at roof perimeters (see ASCE 7 for the size of corner regions).

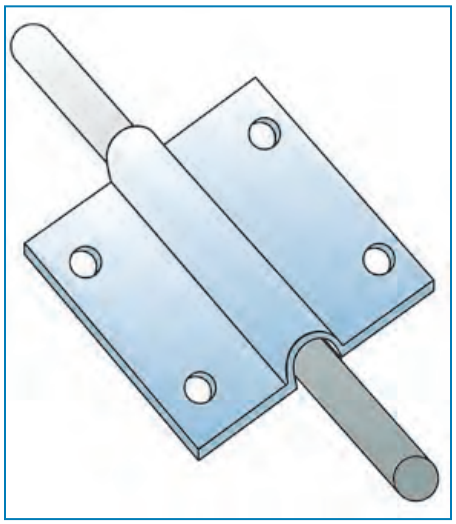


Figure 3-99:
Adhesively-attached conductor connector that does not use prongs

Mechanically attached single-ply membranes: It is recommended that conductors be placed parallel to, and within 8 inches of, membrane fastener rows. Where the conductor falls between or is perpendicular to membrane fastener rows, install an additional row of membrane fasteners where the conductor will be located, and install a membrane cover-strip over the membrane fasteners. Place the conductor over the cover-strip and secure the conductor as recommended above.

By following the above recommendations, additional rows of membrane fasteners (beyond those needed to attach the membrane) may be needed to accommodate the layout of the conductors. The additional membrane fasteners and cover-strip should be coordinated with, and installed by, the roofing contractor.

Standing seam metal roofs: It is recommended that pre-manufactured, mechanically attached clips that are commonly used to attach various items to roof panels be used. After anchoring the clips to the panel ribs, the air terminal base plates and conductor connectors are anchored to the panel clips. In lieu of conductor connectors that have prongs, it is recommended that mechanically attached looped connectors be installed (see Figure 3-97).

Conductor splice connectors: In lieu of pronged splice connectors (see Figure 3-100), bolted splice connectors are recommended because they provide a more reliable connection (see Figure 3-101). It is recommended that strips of flashing membrane (as recommended above) be placed approximately 3 inches from either side of the splice connector to minimize conductor movement and to avoid the possibility of the conductors becoming disconnected. To allow for observation during maintenance inspections, do not cover the connectors.

It is recommended that the building designer advise the building owner to have the LPS inspected each spring, to verify that connectors are still attached to the roof surface, that they still engage the conductors, and that the splice connectors are still secure. Inspections are also recommended after high-wind events.

Figure 3-100:
If conductors detach from the roof, they are likely to pull out from pronged splice connectors. Hurricane Charley (Florida, 2004)

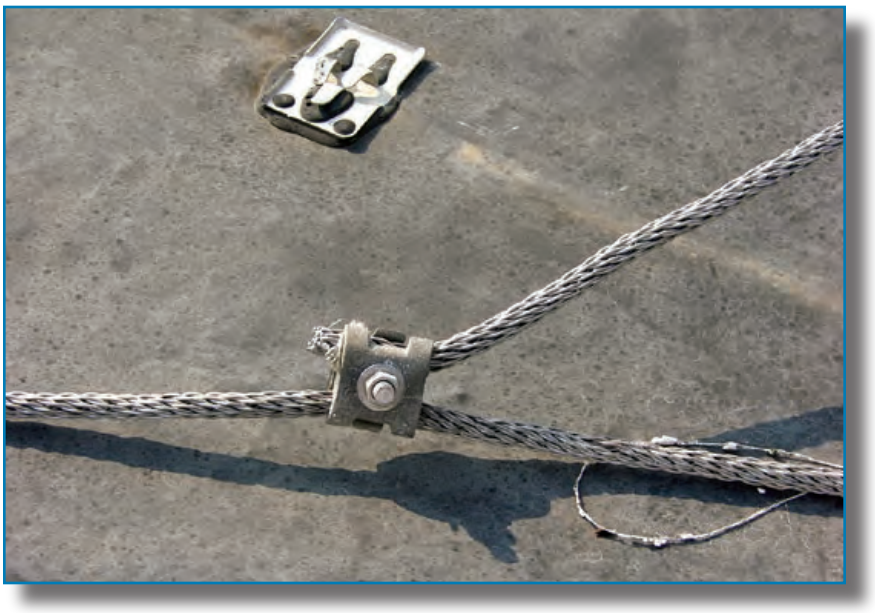


Figure 3-101:
Bolted splice connectors are recommended to prevent free ends of connectors from being whipped around by wind. Hurricane Katrina (Mississippi, 2005)

3.4.5 MUNICIPAL UTILITIES

Hurricanes typically disrupt municipal electrical service, and often they disrupt telephone (both cellular and land-line), water, and sewer services. These disruptions may last from several days to several weeks. Electrical power disruptions can be caused by damage

to power generation stations and by damaged lines, such as major transmission lines and secondary feeders. Water disruptions can be caused by damage to water treatment or well facilities, lack of power for pumps or treatment facilities, or by broken water lines caused by uprooted trees. Sewer disruptions can be caused by damage to treatment facilities, lack of power for treatment facilities or lift stations, or broken sewer lines. Phone disruptions can be caused by damage at switching facilities and collapse of towers. Critical facilities should be designed to prevent the disruption of operations arising from prolonged loss of municipal services.

3.4.5.1 Electrical Power

It is recommended that critical facilities that will be occupied during a hurricane, or will be needed within the first few weeks afterwards, be equipped with one or more emergency generators. In addition to providing emergency generators, it is recommended that one or more additional standby generators be considered, particularly for facilities such as EOCs, hospitals and nursing homes, shelters, and fire and police stations, where continued availability of electrical power is vital. The purpose of providing the standby generators is to power those circuits that are not powered by the emergency generators. With both emergency and standby generators, the entire facility will be completely backed up. It is recommended that the emergency generator and standby generator systems be electrically connected via manual transfer switches to allow for interconnectivity in the event of emergency generator failure. The standby circuits can be disconnected from the standby generators, and the emergency circuits can be manually added. The emergency generators should be rated for prime power (continuous operation).

Running generators for extended time periods frequently results in equipment failure. Thus, provisions for back-up generation capacity are important, because the municipal power system may be out of service for many days or even weeks. Therefore, it is recommended that an exterior box for single pole cable cam locking connectors be provided so that a portable generator can be connected to the facility. With a cam locking box, if one or more of the emergency or standby generators malfunction, a portable generator can be brought to the facility and quickly connected.

It is recommended that shelters be provided with an emergency generator to supply power for lighting, exit signs, fire alarm system, public address system, and for mechanical ventilation. A standby generator is also recommended in the event that the emergency generator malfunctions. A cam locking box is also recommended to facilitate connection of a back-up portable generator.

Back-up portable generators should be viewed as a third source of power (i.e., they should not replace standby generators), because it may take several days to get a back-up portable generator to the site.

Generators should be placed inside wind-borne debris resistant buildings (see recommendations in Sections 3.4.2 and 3.4.3) so that they are not susceptible to damage from debris or tree fall. Locating generators outdoors (see Figure 4-44) or inside weak enclosures is not recommended. It is recommended that wall louvers for generators be capable of resisting the test Missile E load specified in ASTM E 1996. Alternatively, wall louvers can be protected with a debris-resistant screen wall so that wind-borne debris is unable to penetrate the louvers and damage the generators.

It is recommended that sufficient onsite fuel storage be provided to allow all of the facility's emergency and standby generators to operate at full capacity for a minimum of 96 hours (4 days).[11] If at any time it appears that refueling won't occur within 96 hours, provision should be made to shut off part or all the standby circuits in order to provide longer operation of the emergency circuits. For remote facilities or situations where it is believed that refueling may not occur within 96 hours, the onsite fuel storage capacity should be increased as deemed appropriate. It is recommended that fuel storage tanks, piping, and pumps be placed inside wind-borne debris resistant buildings, or underground. If the site is susceptible to flooding, refer to Chapter 2 recommendations.

3.4.5.2 Water Service

It is recommended that critical facilities that rely on water for continuity of service (especially hospitals and nursing homes) be provided with an independent water supply—a well or onsite water storage. Facilities that only need drinking water for

11. The 96-hour fuel supply is based in part on the Department of Veterans Affairs criteria.

occupants can have bottled water provided instead.

If water is needed for cooling towers, the independent water supply should be sized to accommodate the system. It is recommended that the well or onsite storage be capable of providing an adequate water supply for fire sprinklers. Alternatively, it is recommended that the building designer should advise the building owner to implement a continual fire-watch and provide additional fire extinguishers until the municipal water service is restored. For hospitals, it is recommended that the well or onsite water storage be capable of providing a minimum of 100 gallons of potable water per day per patient bed for four days (the 100 gallons includes water for cooling towers).[12]

For critical facilities with boilers, it is recommended to store fuel onsite for a minimum of 96 hours (4 days). Storage tanks, piping, and pumps should be inside wind-borne debris resistant buildings or be placed underground (if site is susceptible to flooding, refer to Chapter 2).

It is recommended that pumps for wells or onsite storage be connected to an emergency power circuit, that a valve be provided on the municipal service line, and that onsite water treatment capability be provided where appropriate.

For hospitals and nursing homes, it is recommended that onsite storage of medical gases be sized to provide a minimum of 96 hours (4 days) of service.

3.4.5.3 Sewer Service

It is recommended that critical facilities that rely on sewer service for continuity of operations (especially hospitals and nursing homes) be provided with an alternative means of waste disposal, such as a temporary storage tank which can be pumped out by a local contractor. For facilities such as EOCs, fire and police stations, and shelters, portable toilets can be placed inside the facility before the onset of a hurricane. It is recommended that all critical facilities be provided with back-flow preventors.

12. This recommendation is based on the Department of Veterans Affairs criteria.

3.4.6 POST-DESIGN CONSIDERATIONS

In addition to adequate design, proper attention must be given to construction, post-occupancy inspection, and maintenance.

3.4.6.1 Construction Contract Administration

It is important for owners of critical facilities in hurricane-prone regions to obtain the services of a professional contractor who will execute the work described in the contract documents in a diligent and technically proficient manner. The frequency of field observations and extent of special inspections and testing should be greater than those employed on critical facilities that are not in hurricane-prone regions.

3.4.6.2 Periodic Inspections, Maintenance, and Repair

The recommendations given in Section 3.3.1.4 for post-occupancy and post-storm inspections, maintenance, and repair are crucial for critical facilities in hurricane-prone regions. Failure of a building component that was not maintained properly, repaired or replaced, can present a considerable risk of injury or death to occupants, and the continued operation of the facility can be jeopardized.

The recommendations given in Section 3.4 are summarized in Table 3-4.

Table 3-4: Recommendations for Design of Critical Facilities

EOCs, healthcare facilities, and shelters	Design very robustly.
Shelters	Refer to FEMA 361, *Design and Construction Guidance for Community Shelters*.
Walkways between campus buildings	If buildings will be occupied during a hurricane, provide enclosed walkways.
Structural systems	Use reinforced cast-in-place concrete. If the roof deck is not cast-in-place, use precast concrete or concrete topping over steel decking.
Exterior walls	Use reinforced concrete or fully grouted and reinforced CMU without wall coverings, other than paint.
Exterior doors	Use doors designed and tested to resist test missiles.
Exterior windows and skylights	Use laminated glass or shutters designed and tested to resist test missiles. If equipped with shutters, glazing is still required to resist wind pressure loads.
Roof covering	Design a roof system that can accommodate missiles as recommended in Section 3.4.3.4. Avoid aggregate surfacings, lightweight concrete pavers, cementitious-coated insulation boards, slate, and tile. Avoid single-ply membranes unless ballasted with heavy pavers.
Parapets	Use minimum 3-foot high parapets for low-sloped roofs.
Elevators	Place elevators in separate locations served by separate penthouses.
Mechanical penthouses	Place rooftop equipment in penthouses rather than exposed on the roof.
Lightning protection systems	Attach LPS to the roof as recommended in Section 3.4.4.3.
Emergency power	Provide emergency power as recommended in Section 3.4.5.1.
Water service	Provide a water supply independent of municipal supplies.
Sewer service	Provide a means of waste disposal independent of municipal service.
Construction contract administration	Construction executed by a professional contractor and subcontractors. Conduct more frequent field observations, special inspections and testing.
Periodic inspections, maintenance, and repair	After construction, conduct diligent periodic inspections and special inspections after storms. Ensure diligent maintenance and prompt repairs.

3.5 BEST PRACTICES IN TORNADO-PRONE REGIONS

Strong and violent tornadoes may reach wind speeds substantially greater than those recorded in the strongest hurricanes. The wind pressures that these tornadoes can exert on a building are tremendous, and far exceed the minimum pressures derived from building codes. Figure 3-102 shows a classroom wing in a school in Illinois. All of the exterior windows were broken, and virtually all of the cementitious wood-fiber deck panels were blown away. Much of the metal roof decking over the band and chorus area also blew off. The gymnasium collapsed, as did a portion of the multi-purpose room. The school was not in session at the time the tornado struck.

Figure 3-102:
A Northern Illinois school heavily damaged by a strong tornado in 1990

Strong and violent tornadoes can generate very powerful missiles. Experience shows that large and heavy objects, including vehicles, can be hurled into buildings at high speeds. The missile sticking out of the roof in the foreground of Figure 3-103 is a double 2-inch by 6-inch wood member. The portion sticking out of the roof is 13 feet long. It penetrated a ballasted ethylene propylene diene monomer (EPDM) membrane, approximately 3 inches of poly-isocyanurate roof insulation, and the steel roof deck. The missile lying on the roof just beyond is a 2-inch by 10-inch by 16-foot long wood member.

Figure 3-103:
A violent tornado showered the roof of this school with missiles. (Oklahoma, 1999)

There is little documentation regarding tornado-induced damage to critical facilities. Most of the damage reports available pertain to schools because schools are the most prevalent type of critical facilities and, therefore, are more likely to be struck. A 1978 report prepared for the Veterans Administration[13] identified four hospitals that were struck by tornadoes between 1973 and 1976. Table 3-5 (taken from that report) further illustrates the effects tornados can have on critical facilities.

13. *A Study of Building Damage Caused by Wind Forces*, McDonald, J.R. and Lea, P.A, Institute for Disaster Research, Texas Tech University, 1978.

Table 3-5: Examples of Ramifications of Tornado Damage at Four Hospitals

Location and Building Characteristics	Tornado Characteristics	Damage	Ramifications of Damage
Mountain View, Missouri (St. Francis Hospital). One-story steel frame with non-load bearing masonry exterior walls.	The tornado crossed over one end of the hospital.	Metal roof decking was blown off, some windows were broken, and rooftop mechanical equipment was displaced.	Patients were moved to undamaged areas of the hospital.
Omaha, Nebraska (Bishop Bergen Mercy Hospital). Five-story reinforced concrete frame.	Maximum wind speed estimated at 200 mph. Proximity to hospital not documented.	Windows were broken, and rooftop mechanical equipment was damaged and displaced. Communications and electrical power were lost (emergency generators provided power).	A few minor cuts; "double walled corridors" provided protection for patients and staff. Some incoming emergency room patients (injured elsewhere in the city) were rerouted to other hospitals. Loss of communications hampered the rerouting.
Omaha, Nebraska (Bishop Bergen Mercy Hospital – Ambulatory Care Unit). One-story load bearing CMU walls with steel joists.	See above.	The building was a total loss due to wall and roof collapse.	Patients were evacuated to the first floor of the main hospital when the tornado watch was issued.
Corsicana, Texas (Navarro County Memorial Hospital). Five-story reinforced concrete frame with masonry non-load bearing walls in some areas and glass curtain walls.	The tornado was very weak.	Many windows were broken by aggregate from the hospital's built-up roofs. Intake duct work in the penthouse collapsed.	Two people in the parking lot received minor injuries from roof aggregate. Electrical power was lost for 2 hours (emergency generators provided power).
Monahans, Texas (Ward Memorial Hospital). One-story load bearing CMU walls with steel joists. Some areas had metal roof deck and others had gypsum deck.	The tornado passed directly over the hospital, with maximum wind speed estimated at 150 mph.	The roof structure was blown away on a portion of the building (the bond beam pulled away from the wall). Many windows were broken. Rooftop mechanical equipment was damaged.	

For critical facilities located in tornado-prone regions (as defined in Section 3.2.2), the following are recommended:

- Incorporate a shelter within the facility to provide occupant protection. For shelter design, FEMA 361 criteria are recommended.

- For interior non-load-bearing masonry walls, see the recommendations given in Section 3.3.3.3.

- Brick veneer, aggregate roof surfacing, roof pavers, slate, and tile cannot be effectively anchored to prevent them from becoming missiles if a strong or violent tornado passes near a building with these components. To reduce the potential number of missiles, and hence reduce the potential for building damage and injury to people, it is recommended that these materials not be specified for critical facilities in tornado prone regions.

- For hospitals, nursing homes, and other critical facilities where it is desired to minimize disruption of operations from nearby weak tornadoes and from strong and violent tornados that are on the periphery of the facility, the following are recommended:

  1) For the roof deck, exterior walls, and doors, follow the recommendations given in Sections 3.4.2 and 3.4.3.

  2) For exterior glazing, specify laminated glass window assemblies that are designed to resist the test Missile E load specified in ASTM E 1996, and are tested in accordance with ASTM E 1886. Note that missile loads used for designing tornado shelters significantly exceed the missile loads used for designing glazing protection in hurricane-prone regions. Missiles from a strong or violent tornado passing near the facility could penetrate the laminated glazing and result in injury or interior damage. Therefore, to increase occupant safety, even when laminated glass is specified, the facility should also incorporate a shelter as recommended above.

Existing Facilities without Tornado Shelters

Where the number of recorded F3, F4, and F5 tornadoes per 3,700 square miles is one or greater (see Figure 3-2 and discussion of Fujita Scale in Section 3.1.1), the best available refuge areas should be identified if the facility does not have a tornado shelter. FEMA 431, T*ornado Protection, Selecting Refuge Areas in Buildings* provides useful information for building owners, architects, and engineers who perform evaluations of existing facilities.

To minimize casualties in critical facilities, it is very important that the best available refuge areas be identified by a qualified architect or engineer.[14] Once identified, those areas need to be clearly marked so that occupants can reach the refuge areas without delay. Building occupants should not wait for the arrival of a tornado to try to find the best available refuge area in a particular facility; by that time, it will be too late. If refuge areas have not been identified beforehand, occupants will take cover wherever they can, frequently in very dangerous places. Corridors, as shown in Figure 3-104, sometimes provide protection, but they can also be death traps.

Figure 3-104: View of school corridor after passage of a violent tornado (Oklahoma, 1999)

14. It should be understood that the occupants of a "best available refuge area" are still vulnerable to death and injury if the refuge area was not specifically designed as a tornado shelter.

Retrofitting a shelter space inside an existing building can be very expensive. An economical alternative is an addition that can function as a shelter as well as serve another purpose. This approach works well for smaller facilities. For very large facilities, constructing two or more shelter additions should be considered in order to reduce the time it takes to reach the shelter (often there is ample warning time, but sometimes an approaching tornado is not noticed until a few minutes before it strikes). This is particularly important for hospitals and nursing homes because of the difficulty of accommodating patients with different medical needs.

For small shelters within facilities such as fire and police stations, a designated storage room(s), office(s), or small conference room(s) can be economically retrofitted in accordance with FEMA 320 to protect the occupants. Where it is desired to provide a large shelter area, FEMA 361 criteria are recommended.

The recommendations given in Section 3.5 are summarized in Table 3-6.

Table 3-6: Critical Facilities Located in Tornado-Prone Regions

Proposed Facility	
Occupant protection	Refer to FEMA 361 for design guidance.
Interior non-load-bearing masonry walls	See recommendations in Section 3.3.3.3.
Wind-borne missiles	Avoid use of brick veneer, aggregate surfacing, roof pavers, slate, and tile.
Healthcare and other critical facilities where it is desired to minimize disruption of operations from nearby weak tornadoes	See recommendations in Section 3.5.
Existing facilities without specifically designed tornado shelters	
If one or more F3-F5 tornadoes per 3,700 square miles	Identify best available refuge areas. See Figure 3-2 for historical data on frequency, and refer to FEMA 431 (2003) for identification guidance.
If six or more F3-F5 tornadoes per 3,700 square miles	Consider incorporating a shelter within the building or inside a new building addition. Refer to FEMA 320 and FEMA 361 for design guidance.

3.6 REMEDIAL WORK ON EXISTING FACILITIES

Many existing critical facilities need to strengthen their structural or building envelope components. The reasons for this are the deterioration that has occurred over time, or inadequate facility strength to resist current design level winds. It is recommended that building owners have a vulnerability assessment performed by a qualified architectural and engineering team. A vulnerability assessment should be performed for all facilities older than 5 years. However, as illustrated by Figure 3-30 and the case of Garden Park Medical Center discussed in Section 4.2, an assessment is recommended for all facilities located in areas where the basic wind speed is greater than 90 mph (even if the facility is younger than five years). It is particularly important to perform vulnerability assessments on critical facilities located in hurricane-prone and tornado-prone regions.

American Red Cross (ARC) Publication 4496, *Standards for Hurricane Evacuation Shelter Selection* (2002) provides information regarding assessing existing buildings for use as hurricane shelters. Unless a facility has been specifically designed for use as a shelter, it should only be used as a shelter of last resort, and even then, only if it meets the criteria given in ARC 4496.

Components that typically make buildings constructed before the early 1990s vulnerable to high winds are weak non-load-bearing masonry walls, poorly connected precast concrete panels, long-span roof structures with limited uplift resistance (e.g., at gyms), inadequately connected roof decks, weak glass curtain walls, building envelope, and exterior-mounted equipment. Although the technical solutions to these problems are not difficult, the cost of the remedial work is typically quite high. If

funds are not available for strengthening or replacement, it is important to minimize the risk of injury and death by evacuating areas adjacent to weak non-load-bearing walls, weak glass curtain walls, and areas below long-span roof structures when winds above 60 mph are forecast.

As a result of building code changes and heightened awareness, some of the common building vulnerabilities have generally been eliminated for facilities constructed in the mid-1990s or later. Components that typically remain vulnerable to high winds are the building envelope and exterior-mounted mechanical, electrical, and communications equipment. Many failures can be averted by identifying weaknesses and correcting them.

Critical facilities sometimes occupy buildings that have changed their original use (see the case of Hancock County EOC, discussed in Section 4.4). Buildings that were not designed for a critical occupancy were likely designed with a 1.0 rather than a 1.15 importance factor, and hence are not as wind-resistant as needed. It is particularly important to perform a vulnerability assessment if a facility is located in a building not originally designed for a critical occupancy, especially if the facility is located in a hurricane- or tornado-prone region.

By performing a vulnerability assessment, items that need to be strengthened or replaced can be identified and prioritized. A proactive approach in mitigating weaknesses can save significant sums of money and decrease disruption or total breakdown in critical facility operations after a storm. For example, a vulnerability assessment on a school such as that shown in Figure 3-105 can identify weakness of exterior classroom walls. Replacing walls before a hurricane is much cheaper than replacing the walls and repairing consequential damages after a storm, and proactive work avoids the loss of use while repairs are made.

A comprehensive guide for remedial work on existing facilities is beyond the scope of this manual. However, the following are examples of mitigation measures that are often applicable.

Before beginning remedial work, it is necessary to understand all significant aspects of the vulnerability of a facility with respect to wind and wind-driven rain. If funds are not available to correct all identified deficiencies, the work should be systematically prioritized so that the items of greatest need are first corrected. For example, at a building such as that shown in Figure 3-105, had the windows been retrofitted with shutters, that effort would have been ineffective, because the walls themselves collapsed. Mitigation efforts can be very ineffective if they do not address all items that are likely to fail.

Figure 3-105:
Several walls at this school collapsed. Windows were located above a non-load-bearing masonry wall. Hurricane Katrina (Mississippi, 2005)

3.6.1 STRUCTURAL SYSTEMS

As discussed in Section 3.1.4.1, roof decks on many facilities designed prior to the 1982 edition of the SBC and UBC and the 1987 edition of the NBC are very susceptible to failure. Poorly attached decks that are not upgraded are susceptible to blow-off, as shown in Figure 3-106. Decks constructed of cementitious wood-fiber, gypsum, and lightweight insulating concrete over form boards were commonly used on buildings built in the 1950s and 1960s. In that era, these types of decks typically had very limited uplift resistance due to weak connections to the support structure. Steel deck attachment is frequently not adequate because of an inadequate number of welds, or welds of poor quality. Older buildings with overhangs are particularly susceptible to blow-off, as shown in Figure 3-107, because older codes provided inadequate uplift criteria.

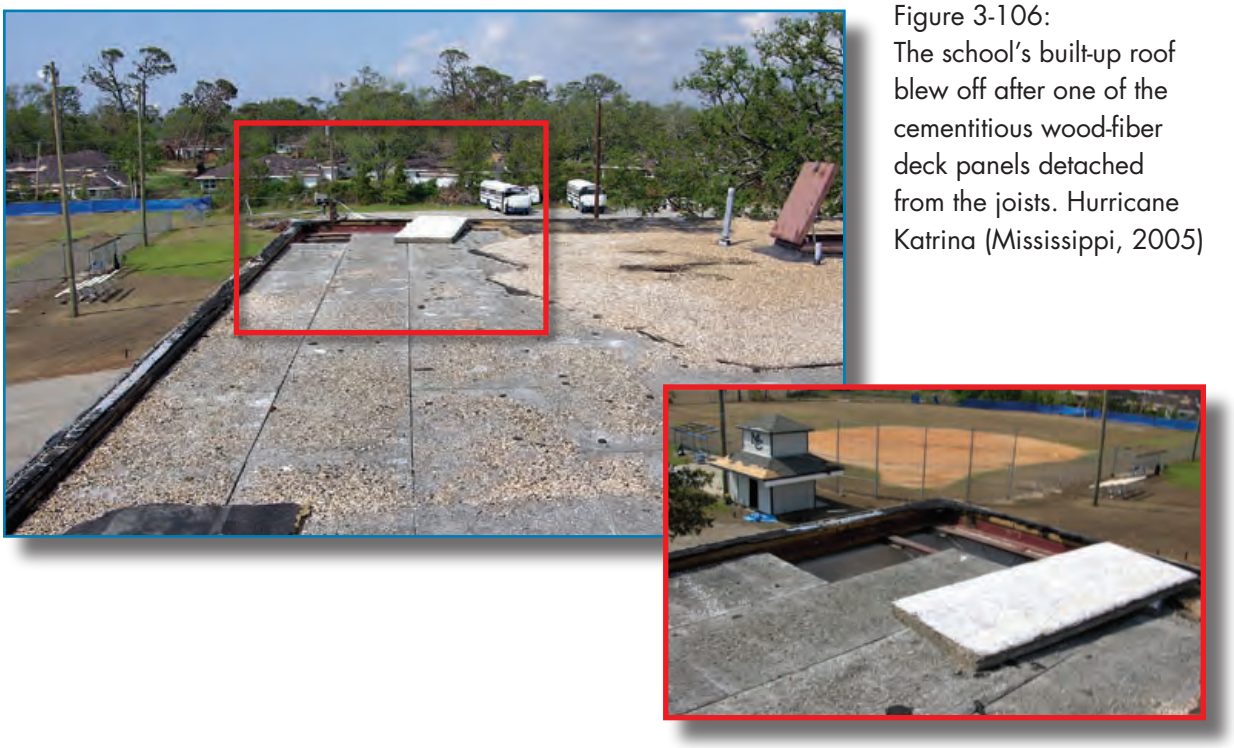


Figure 3-106:
The school's built-up roof blew off after one of the cementitious wood-fiber deck panels detached from the joists. Hurricane Katrina (Mississippi, 2005)

Figure 3-107:
The cementitious wood-fiber deck panels detached from the joists along the overhangs and caused the school's built-up membranes to lift and peel. Hurricane Katrina (Mississippi, 2005)

A vulnerability assessment of the roof deck should include evaluating the existing deck attachment, spot checking the structural integrity of the deck (including the underside, if possible), and evaluating the integrity of the beams/joists. If the deck attachment is significantly overstressed under current design wind conditions or the deck integrity is compromised, the deck should be replaced or strengthened as needed. The evaluation should be conducted by an investigator experienced with the type of deck used on the building.

If a low-slope roof is converted to a steep-slope roof, the new support structure should be engineered and constructed to resist the wind loads and avoid the kind of damage shown in Figure 3-108.

Figure 3-108:
The school's wood superstructure installed as part of a steep-slope conversion blew away because of inadequate attachment. Hurricane Katrina (Louisiana, 2005)

3.6.2 BUILDING ENVELOPE

The following recommendations apply to building envelope components of existing critical facilities.

3.6.2.1 Sectional and Rolling Doors

Sectional and rolling doors (e.g., at fire station apparatus bays and hospital loading docks), installed in older buildings before attention was given to the wind resistance of these elements, are very susceptible to being blown away. Although weak doors can be retrofitted, it is difficult to ensure that the door, door tracks, and connections between the door and tracks are sufficient. It is therefore recommended that weak doors and tracks be replaced with new assemblies that have been tested to meet the factored design wind loads. As part of the replacement work, nailers between the tracks and building structure should either be replaced, or their attachment should also be strengthened.

If a facility has more than one sectional or rolling door, all doors should be replaced, rather than just replacing one of the doors. The fire station shown in Figure 3-109 had six sectional doors. One door had been replaced before a hurricane. It performed very well, but three of the older doors were blown away and two of the older doors remained in place but had some wind damage.

Figure 3-109:
The new door in the center performed very well, but the older doors on either side of it were blown away. Hurricane Charley (Florida, 2004)

3.6.2.2 Windows and Skylights

Windows in older facilities may possess inadequate resistance to wind pressure. Window failures are typically caused by wind-borne debris, however, glazing or window frames may fail as a result of wind pressure (see Figure 3-110). Failure can be caused by inadequate resistance of the glazing, inadequate anchorage of the glazing to the frame, failure of the frame itself, or inadequate attachment of the frame to the wall. For older windows that are too weak to resist the current design pressures, window assembly replacement is recommended. Some older window assemblies have sufficient strength to resist the design pressure, but are inadequate to resist wind-driven rain. If the lack of water resistance is due to worn glazing gaskets or sealants, replacing the gaskets or sealant may be viable. In other situations, replacing the existing assemblies with new, higher-performance assemblies may be necessary.

Figure 3-110:
Wind pressure caused the window frames on the upper floor to fail (red arrow). Hurricane Katrina (Mississippi, 2005)

It is recommended that all non-impact-resistant, exterior glazing located in hurricane-prone regions (with a basic wind speed of 100 mph or greater) be replaced with impact-resistant glazing or be protected with shutters, as discussed in Section 3.4.3.2.

Shutters are typically a more economical approach for existing facilities. There are a variety of shutter types, all illustrated by Figures 3-111 to 3-113. Accordion shutters are permanently attached to the wall (Figure 3-111). When a hurricane is forecast, the shutters are pulled together and latched into place. Panel shutters (Figure 3-112) are made of metal or polycarbonate. When a hurricane is forecast, the shutters are taken from storage and inserted into metal tracks that are permanently mounted to the wall above and below the window frame. The panels are locked into the frame with wing nuts or clips. Track designs that have permanently mounted studs for the nuts have been shown to be more reliable than track designs using studs that slide into the track. A disadvantage of panel shutters is the need for storage space. Roll-down shutters (Figure 3-113) can be motorized or pulled down manually. Figure 3-113 illustrates the benefits of shuttering. Two of the unprotected window units experienced glass breakage and the third window unit blew in.

Figure 3-111:
This school has accordion shutters. Hurricane Ivan (Florida, 2004)

Figure 3-112:
Illustrates a metal panel shutter. Hurricane Georges (Puerto Rico, 1998)

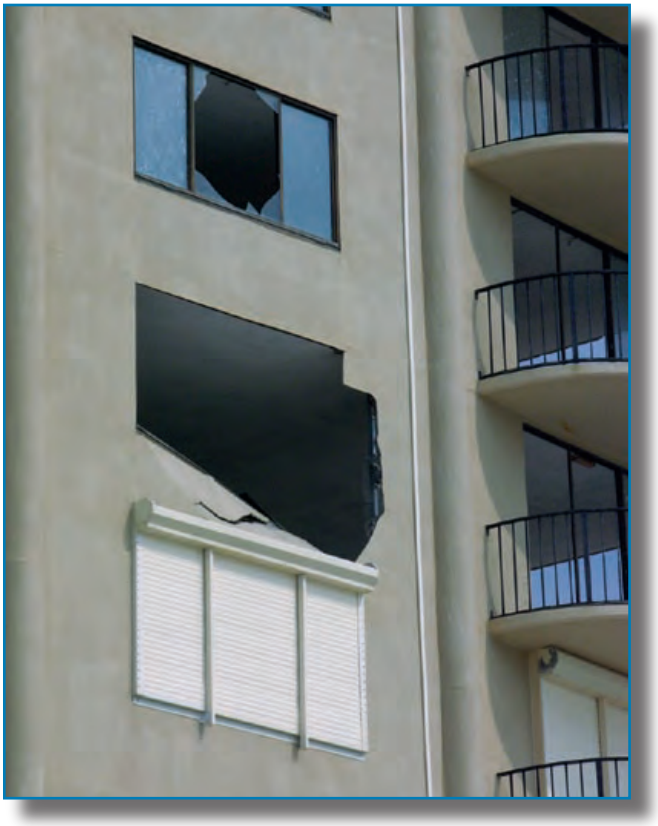


Figure 3-113:
The lower window assembly was protected with a motorized shutter. Hurricane Ivan (Florida, 2004)

Deploying accordion or panel shutters a few stories above grade is expensive. Although motorized shutters have greater initial cost, their operational cost should be lower. Other options for providing missile protection on upper levels include replacing the existing as-

semblies with laminated glass assemblies, or installing permanent impact resistant screens. Engineered films are also available for application to the interior of the glass. The film needs to be anchored to the frame, and the frame needs to be adequately anchored to the wall. The film degrades over time and requires replacement (approximately every decade). Use of laminated glass or shutters is recommended in lieu of engineered films.

3.6.2.3 Roof Coverings

For roofs with weak metal edge flashing or coping attachment, face-attachment of the edge flashing/coping (as shown in Figure 3-63) is a cost-effective approach to greatly improve the wind resistance of the roof system.

The vulnerability assessment of roofs ballasted with aggregate, pavers, or cementitious-coated insulation boards, should determine whether the ballast complies with ASNI/SPRI RP-4. Corrective action is recommended for non-compliant, roof coverings. It is recommended that roof coverings with aggregate surfacing, lightweight pavers, or cementitious-coated insulation boards on buildings located in hurricane-prone regions be replaced to avoid blow off (see Figure 3-114).

Figure 3-114: Aggregate from the hospital's built-up roofs broke several windows in the intensive care unit, which had to be evacuated during the hurricane. Hurricane Charley (Florida, 2004)

When planning the replacement of a roof covering, it is recommended that all existing roof covering be removed down to the deck rather than simply re-covering the roof. Tearing off the covering provides an opportunity to evaluate the structural integrity of the deck and correct deck attachment and other problems. For example, if a roof deck was deteriorated due to roof leakage (see Figure 3-115), the deterioration would likely not be identified if the roof was simply re-covered. By tearing off down to the deck, deteriorated decking like that shown in Figure 3-115 can be found and replaced. In addition, it is recommended that the attachment of the wood nailers at the top of parapets and roof edges be evaluated and strengthened where needed, to avoid blow-off and progressive lifting and peeling of the new roof membrane (see Figure 3-116).

Figure 3-115:
The built-up roof on this school was blown off after a few of the rotted wood planks detached from the joists. Hurricane Katrina (Mississippi, 2005)

Figure 3-116:
The edge nailer on top of an old brick wall at a hospital blew off because it was inadequately attached. Hurricane Ivan (Florida, 2004)

If the roof has a parapet, it is recommended that the inside of the parapet be properly prepared to receive the new base flashing. In many instances, it is prudent to re-skin the parapet with sheathing to provide a suitable substrate. Base flashing should not be applied directly to brick parapets because they have irregular surfaces that inhibit good bonding of the base flashing to the brick (see Figure 3-117). Also, if moisture drives into the wall from the exterior side of the parapet with base flashing attached directly to brick, the base flashing can inhibit drying of the wall. Therefore, rather than totally sealing the parapet with membrane base flashing, the upper portion of the brick can be protected by metal panels (as shown in Figure 3-93), which permit drying of the brick.

Figure 3-117: Failed base flashing adhered directly to the brick parapet. Hurricane Katrina (Louisiana, 2005)

3.6.3 EXTERIOR-MOUNTED EQUIPMENT

Fastening rooftop equipment to curbs, as discussed in Section 3.3.4.1, is a cost-effective approach to minimize wind-induced problems.

Exterior-mounted equipment on existing critical facilities should be carefully examined and evaluated.

3.6.3.1 Antenna (Communications Mast)

Antenna collapse is very common. Besides loss of communications, collapsed masts can puncture roof membranes or cause other building damage as shown in Figure 3-118. This case also demonstrates the benefits of a high parapet. Although the roof still experienced high winds that blew off this penthouse door, the parapet prevented the door from blowing off the roof.

In hurricane-prone regions, it is recommended that antennae strength be evaluated as part of the vulnerability assessment. Chapter 15 of ANSI/TIA-222-G provides guidance on the structural evaluation of existing towers. Appendix J of that standard contains checklists for maintenance and condition assessments. Additional bracing, guy-wires, or tower strengthening or replacement may be needed.

Figure 3-118:
The antenna at this hospital collapsed and was whipped back and forth across the roof membrane. Hurricane Andrew (Florida, 1992)

3.6.3.2 Lightning Protection Systems

Adhesively-attached conductor connectors and pronged splice connectors typically have not provided reliable attachment during hurricanes. To provide more reliable attachment for LPS located in hurricane-prone regions where the basic wind speed is 100 mph or greater, or on critical facilities in excess of 100 feet above grade, it is recommended that attachment modifications based on the guidance given in Section 3.3.4.3 be used.

The recommendations given in Section 3.6 are summarized in Table 3-7.

Table 3-7: Recommendations for Remedial Work on Existing Critical Facilities

Weakness	Recommended remedy
Critical facilities older than 5 years, or any age if located in an area with basic wind speed greater than 90 mph.	Perform vulnerability assessment with life-safety issues as the first priority, and property damage and interruption of service as the second priority.
A building with weak non-load-bearing masonry or curtain walls, poorly connected precast concrete panels, or weak long-span roof structures.	Implement remedial work on elements with insufficient strength to resist wind loads if the facility will be occupied during high wind events (e.g., strong thunderstorms).
Sectional and rolling doors.	Replace weak doors and tracks.
Worn window gaskets and sealants.	Replace with new gaskets and sealants, or replace window assembly.
Buildings in a hurricane-prone region where the basic wind speed is 100 mph or greater, with non-impact-resistant exterior glazing.	Replace with impact-resistant glazing or protect with shutters.
Inadequately attached edge flashings or copings.	Face-attach the vertical flanges. See Figure 3-63.
Ballasted single-ply roof membranes.	Take corrective action if non-compliant with ANSI/SPRI RP-4.
Buildings in a hurricane-prone region with aggregate roof surfacing, lightweight pavers, or cementitious-coated insulation boards.	Replace roof covering to avoid blow-off.
Rooftop equipment unanchored or poorly anchored.	Add screws or bolts to anchor equipment to curbs. Add cables to secure fan cowlings. Add latches to secure equipment access panels. See Section 3.3.4.1.
Weak roof deck connections or weak roof structure.	When planning replacement of roof covering, remove roof covering and strengthen attachment of deck and/or roof structure. See Section 3.6.2.3.
Emergency generators in a hurricane-prone region not adequately protected from wind-borne debris.	Build an enclosure to provide debris protection. See Section 3.4.5.1.
Antennae (communication masts) in hurricane-prone regions.	Evaluate wind resistance and strengthen as needed. See Chapter 15 and Appendix J of ANSI/TIA-222-G.
Lightning protection systems with adhesively-attached conductor connectors or pronged splice connectors located in hurricane-prone regions where the basic wind speed is 100 mph or greater, or on critical facilities in excess of 100 feet above grade.	Modify attachment according to recommendations in Section 3.4.4.3.

3.7 CHECKLIST FOR BUILDING VULNERABILITY OF CRITICAL FACILITIES EXPOSED TO HIGH WINDS

The Building Vulnerability Assessment Checklist (Table 3-8) is a tool that can help in assessing the vulnerability of various building components during the preliminary design of a new building, or the rehabilitation of an existing building. In addition to examining design issues that affect vulnerability to high winds, the checklist also examines the potential adverse effects on the functionality of the critical and emergency systems upon which most critical facilities depend. The checklist is organized into separate sections, so that each section can be assigned to a subject expert for greater accuracy of the examination. The results should be integrated into a master vulnerability assessment to guide the design process and the choice of appropriate mitigation measures.

Table 3-8: Checklist for Building Vulnerability of Critical Facilities Exposed to High Winds

Vulnerability Sections	Guidance	Observations
General		
What is the age of the facility, and what building code and edition was used for the design of the building?	Substantial wind load improvements were made to the model building codes in the 1980s. Many buildings constructed prior to these improvements have structural vulnerabilities. Since the 1990s, several additional changes have been made, the majority of which pertain to the building envelope. Older buildings, not designed and constructed in accordance with the practices developed since the early 1990s, are generally more susceptible to damage than newer buildings.	

Table 3-8: Checklist for Building Vulnerability of Critical Facilities Exposed to High Winds (continued)

Vulnerability Sections	Guidance	Observations
General (continued)		
Is the critical facility older than 5 years, or is it located in a zone with basic wind speed greater than 90 mph?	In either case, perform a vulnerability assessment with life-safety issues as the first priority, and property damage and interruption of service as the second priority.	
Site		
What is the design wind speed at the site? Are there topographic features that will result in wind speed-up?	ASCE 7 and Section 3.1.3.	
What is the wind exposure on site?	Avoid selecting sites in Exposure D, and avoid escarpments and hills (Section 3.1.3).	
Are there trees or towers on site?	Avoid trees and towers near the facility (Section 3.3.1.1). If the site is in a hurricane-prone region, avoid trees and towers near primary access roads (Section 3.4.1).	
Road access	Provide two separate means of access (Section 3.3.1.1).	
Is the site in a hurricane-prone region?	ASCE 7. If yes, follow hurricane-resistant design guidance (Section 3.4).	
If in a hurricane-prone region, are there aggregate surfaced roofs within 1,500 feet of the facility?	Remove aggregate from existing roofs (Section 3.6.2.3). If the buildings with aggregate are owned by other parties, attempt to negotiate the removal of the aggregate (e.g., consider offering to pay the reroofing costs).	
Architectural		
Will the facility be used as a shelter?	If yes, refer to FEMA 361.	
Are there interior non-load-bearing walls?	Design for wind load according to Section 3.3.3.3.	
Are there multiple buildings on site in a hurricane-prone region?	Provide enclosed walkways between buildings that will be occupied during a hurricane (Section 3.4.1).	
Are multiple elevators needed for the building?	Place elevators in separate locations served by separate penthouses (Section 3.4.4.1).	

Table 3-8: Checklist for Building Vulnerability of Critical Facilities Exposed to High Winds (continued)

Vulnerability Sections	Guidance	Observations
Structural Systems	Section 3.3.2	
Is a pre-engineered building being considered?	If yes, ensure the structure is not vulnerable to progressive collapse. If a pre-engineered building exists, evaluate to determine if it is vulnerable to progressive collapse.	
Is precast concrete being considered?	If yes, design the connections to resist wind loads. If precast concrete elements exist, verify that the connections are adequate to resist the wind loads.	
Are exterior load-bearing walls being considered?	If yes, design as MWFRS and C&C.	
Is an FM Global-rated roof assembly specified?	If yes, comply with FM Global deck criteria.	
Is there a covered walkway or canopy?	If yes, use "free roof" pressure coefficients from ASCE 7.Canopy decks and canopy framing members on older buildings often have inadequate wind resistance. Wind-borne debris from canopies can damage adjacent buildings and cause injury.	
Is the site in a hurricane-prone region?	A reinforced cast-in-place concrete structural system, and reinforced concrete or fully grouted and reinforced CMU walls, are recommended (Section 3.4.2).	
Is the site in a tornado-prone region?	If yes, provide occupant protection. See FEMA 361.	
Do portions of the existing facility have long-span roof structures (e.g., a gymnasium)?	Evaluate structural strength, since older long-span structures often have limited uplift resistance.	
Is there adequate uplift resistance of the existing roof deck and deck support structure?	The 1979 (and earlier) SBC and UBC, and 1984 (and earlier) BOCA/NBC, did not prescribe increased wind loads at roof perimeters and corners. Decks (except cast-in-place concrete) and deck support structures designed in accordance with these older codes are quite vulnerable.The strengthening of the deck attachment and deck support structure is recommended for older buildings.	

Table 3-8: Checklist for Building Vulnerability of Critical Facilities Exposed to High Winds (continued)

Vulnerability Sections	Guidance	Observations
Structural Systems	Section 3.3.2 (continued)	
Are there existing roof overhangs that cantilever more than 2 feet?	Overhangs on older buildings often have inadequate uplift resistance.	
Building Envelope	Section 3.3.3	
Exterior doors, walls, roof systems, windows, and skylights.	Select materials and systems, and detail to resist wind and wind-driven rain (Sections 3.3.3.1 to 3.3.3.4).	
Are soffits considered for the building?	Design to resist wind and wind-driven water infiltration (Section 3.3.3.3). If there are existing soffits, evaluate their wind and wind-driven rain resistance. If the soffit is the only element preventing wind-driven rain from being blown into an attic space, consider strengthening the soffit.	
Are there elevator penthouses on the roof?	Design to prevent water infiltration at walls, roof, and mechanical penetrations (Sections 3.3.3.3, 3.3.3.4, 3.3.4.1, and 3.4.4.1).	
Is a low-slope roof considered on a site in a hurricane-prone region?	A minimum 3-foot parapet is recommended on low-slope roofs (Section 3.4.3.4).	
Is an EOC, healthcare facility, shelter, or other particularly important critical facility in a hurricane-prone region?	If yes, a very robust building envelope, resistant to missile impact, is recommended (Section 3.4).	
Is the site in a tornado-prone region?	To minimize generation of wind-borne missiles, avoid the use of brick veneer, aggregate roof surfacing, roof pavers, slate, and tile (Section 3.5).	
Are there existing sectional or rolling doors?	Older doors often lack sufficient wind resistance. Either strengthen or replace. This is particularly important for fire station apparatus bay doors.	
Does the existing building have large windows or curtain walls?	If an older building, evaluate their wind resistance.	
Does the existing building have exterior glazing (windows, glazed doors, or skylights)?	If the building is in a hurricane-prone region, replace with impact-resistant glazing, or protect with shutters.	

Table 3-8: Checklist for Building Vulnerability of Critical Facilities Exposed to High Winds (continued)

Vulnerability Sections	Guidance	Observations
Building Envelope	Section 3.3.3 (continued)	
Does the existing building have operable windows?	If an older building, evaluate its wind-driven rain resistance.	
Are there existing exterior non-load-bearing masonry walls?	If the building is in a hurricane- or tornado-prone region, strengthen or replace.	
Are there existing brick veneer, EIFS, or stucco exterior coverings?	If the building is in a hurricane-prone region, evaluate attachments. To evaluate wind resistance of EIFS, see ASTM E 2359 (2006).	
Are existing exterior walls resistant to wind-borne debris?	If the building is in a hurricane-prone region, consider enhancing debris resistance, particularly if dealing with an important critical facility.	
Are there existing ballasted single-ply roof membranes?	Determine if they are in compliance with ANSI/SPRI RP-4. If non-compliant, take corrective action.	
Does the existing roof have aggregate surfacing, lightweight pavers, or cementitious-coated insulation boards?	If the building is in a hurricane- prone region, replace the roof covering to avoid blow-off.	
Does the existing roof have edge flashing or coping?	Evaluate the adequacy of the attachment.	
Does the existing roof system incorporate a secondary membrane?	If not, and if the building is in a hurricane-prone region, reroof and incorporate a secondary membrane into the new system.	
Does the existing building have a brittle roof covering, such as slate or tile?	If the building is in a hurricane-prone region, consider replacing with a non-brittle covering, particularly if it is an important critical facility.	
Exterior-Mounted Mechanical Equipment		
Is there mechanical equipment mounted outside at grade or the roof?	Anchor the equipment to resist wind loads (Section 3.3.4.1). If there is existing equipment, evaluate the adequacy of the attachment, including attachment of cowlings and access panels.	
Are there penetrations through the roof?	Design intakes and exhausts to avoid water leakage (Section 3.3.4.1).	
Is the site in a hurricane-prone region?	If yes, place the equipment in a penthouse, rather than exposed on the roof (Section 3.4.4.2).	

Table 3-8: Checklist for Building Vulnerability of Critical Facilities Exposed to High Winds (continued)

Vulnerability Sections	Guidance	Observations
Exterior-mounted Electrical and Communications Equipment		
Are there antennae (communication masts) or satellite dishes?	See Section 3.3.4.2. If there are existing antennae or satellite dishes and the building is located in a hurricane-prone region, evaluate wind resistance. For antennae evaluation, see Chapter 15 of ANSI/TIA-222-G-2005.	
Does the building have a lightning protection system?	See Sections 3.3.4.2 and 3.4.4.3 for lightning protection system attachment. For existing lightning protection systems, evaluate wind resistance (Section 3.6.3.1)	
Municipal Utilities		
Is the site in a hurricane-prone region?	See Section 3.4.5.1 for emergency and standby power recommendations.	
Is the emergency generator(s) housed in a wind- and debris-resistant enclosure?	If not, build an enclosure to provide debris protection in a hurricane-prone region (Section 3.4.5.1).	
Is the emergency generator's wall louver protected from wind-borne debris?	If the building is in a hurricane-prone region, install louver debris impact protection (Section 3.4.5.1).	
Is the site in a hurricane-prone region?	If yes, an independent water supply and alternative means of sewer service are recommended, independent of municipal services (Sections 3.4.5.2 and 3.4.5.3).	

3.8 REFERENCES AND SOURCES OF ADDITIONAL INFORMATION

NOTE: FEMA publications may be obtained at no cost by calling (800) 480-2520, faxing a request to (301) 497-6378, or downloading from the library/publications section online at http://www.fema.gov.

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4.1 INTRODUCTION

This chapter presents some observations on the performance of critical facilities during Hurricane Katrina that identify the various ways in which building and equipment damage, as well as loss of municipal services, can disrupt facility operations. These observations are intended to help people who own, operate, design, and build critical facilities to adjust their building designs, construction, and facility management practices to reflect the needs of comprehensive risk reduction.

During Hurricane Katrina, surging floodwaters and high winds caused considerable and often catastrophic building damage, forcing many critical facilities to cease operations and evacuate their premises even before the storm had passed. In many instances the continued operation of hospitals, police and fire stations, schools, and EOCs was severely compromised by relatively minor damage to the building or building-mounted equipment. Although the structural components of most critical facilities survived the hurricane, other building components performed less well, causing serious disruptions.

The observations highlighted in this chapter, made by a team of building professionals (architects and engineers) experienced in hazard mitigation, document the variety and severity of the building damage, and the corresponding effects on facility operations. Field inspections and discussions with facility managers and other personnel served as the basis for analysis of the experiences of individual facilities. The descriptions of these experiences are

accompanied with suggestions on possible mitigation measures that would improve hazard-resistance and protect the facility's functionality in the future. A comprehensive risk assessment of the facility's operation and building components and systems would be required before any specific mitigation measures were implemented. This chapter emphasizes the experiences and lessons learned from facility operation disruptions in particular circumstances, and may not be applicable to all situations.

4.2 HEALTH CARE FACILITIES

4.2.1 BACKGROUND

Health care facilities are at the front line of community protection, especially during and after a natural disaster event. Their capacity to continue to provide services to existing patients, and to respond to the needs of victims following a disaster, depends not only on protecting the integrity of the structure and the building envelope, but on the facilities' ability to carry out their intended functions with little or no interruption. Continued and uninterrupted operation of health care facilities, regardless of the nature of the disaster, is one of the most important elements of a natural disaster safety program.

Health care facilities, especially hospitals, are usually very complex building systems, because they accommodate diverse and highly specialized services in a strictly controlled environment. Hospital buildings must be designed to provide appropriate spatial arrangement for the flawless interaction between staff, patients, and visitors. They also require a complex network of technological infrastructure to support the hospital's functions. Even the smallest breakdown in this complex network can cascade into a serious disruption of operations.

Protecting the functionality of a hospital requires very careful facility planning and design that is in accordance with the most stringent flooding and high-wind mitigation requirements applicable to the site. The damage sustained by hospitals during

Hurricane Katrina emphasized the fact that many of the Nation's hospitals occupy old or inadequate buildings that may not be sufficiently protected against hazards. In particular, some of the hospitals were planned and built before mitigation against natural hazards became common practice.

Considering the expanding gap between the functionality of aging hospitals on the one side, and the requirements of new technologies and the needs of a growing and aging population on the other, it is expected that many of the country's hospitals will have to be replaced, upgraded, or rebuilt in the coming years. Some medical industry forecasts predict that by the end of the decade the United States will spend $20 billion annually for this purpose. Since new or rebuilt hospitals will have to last for a long time, the anticipated construction program provides an opportunity to rethink hospital planning and design, and to consider how to avoid hazard areas and reduce the vulnerability with improved hospital design.

The following observations are based on published sources at the time of Hurricane Katrina, and on the subsequent interviews with providers and regulators in both Louisiana and Mississippi. The damage assessments present a picture of common effects on the medical facilities, which are consistent across the region affected by Hurricane Katrina. Generous contributions by the following institutions and their staff are acknowledged:

- St. Tammany Parish Office of Homeland Security and Emergency Preparedness (Covington, Louisiana)
- Touro Infirmary (New Orleans, Louisiana)
- West Jefferson Medical Center (Jefferson Parish, Louisiana)
- Hancock Medical Center (Bay St. Louis, Mississippi)
- Garden Park Medical Center (Gulfport, Mississippi)
- Guest House of Slidell (Slidell, Louisiana)
- Slidell Memorial Hospital (Slidell, Louisiana)

- LSU Health Sciences Center (New Orleans, Louisiana)
- Charity Hospital (New Orleans, Louisiana)
- University Hospital (New Orleans, Louisiana)

4.2.2 EFFECTS OF FLOODING

The damage caused by Hurricane Katrina flooding was significantly more serious than the damage caused by wind. Along the Gulf Coast, the storm surge was higher than previously experienced, which caught many health care providers by surprise (see Figure 1-2). In most places, the storm surge flooding receded in several hours, but in New Orleans the floodwaters remained for more than a week. Apart from the damage and disruption caused by floodwaters that penetrated the facilities' lower levels, many hospitals in New Orleans were completely surrounded by floodwaters, which cut off all surface access.

As a result of the disrupted access, most hospitals had to manage on their own, without any assistance from the outside. Patients and visitors were stranded, along with staff that could not be relieved for days. The injured, and others in need of emergency care, could not be brought in for treatment. Family and friends of people stranded in the hospitals had no way of communicating with them. Food, water, medical, and other supplies could not be brought in except by small boats and helicopters, or in some instances, by military vehicles. The evacuation of the critically ill patients, and eventually others, was possible only by boat or by helicopter. Hospitals with dedicated or improvised elevated helipads managed the evacuation much better than others.

Hospitals and nursing homes that were inundated during Katrina experienced the greatest damage. Hospital functions located in the areas exposed to floodwaters had to be shut down. In many cases, the elevators and other mechanical and electrical services were shut down by the floodwaters.

Flooding caused considerable disruption of utility services in most hospitals in the New Orleans area. Sewers flooded or pumping stations shorted out, disabling sewage and liquid waste disposal.

The water supply was interrupted and, in many instances, onsite sources were contaminated and could not be used for drinking purposes. Emergency generators and electrical switchgear equipment, as well as underground transformers flooded and were put out of commission.

In several instances, communications panels and other controls were located on the first floor and shorted out. The heat and the buildup of humidity in New Orleans ruined the telephone connections and the fire alarm systems in some hospitals, even though floodwaters did not contact sensitive communications equipment directly.

4.2.3 EFFECTS OF HIGH WINDS

In general, three types of wind damage affected the hospitals: damaged roof coverings, rooftop equipment, and window breakage. Damaged roof coverings that were either peeled off or punctured by wind-borne debris exposed the interior to rainwater penetration and additional damage. Similarly, rooftop equipment displaced by wind left unprotected roof openings exposed to rainwater penetration. Water damage ranged from saturation of interior surfaces, like walls and ceilings, to ruined equipment and considerable mold growth.

Window breakage during the storm was particularly dangerous, because it allowed the penetration of rainwater, and wind that can cause pressurization in the interior. Hospital patient rooms, however, faced the greatest risks from window breakage, because many of their occupants could not be moved away from monitors, medical gases, and other equipment.

4.2.4 SITE DESIGN

Although most of New Orleans is located in the lowlands that are protected by a system of canals and levees, the hospitals were not built to resist the flooding caused by levee failures. As a result of Hurricane Katrina and the failure of levees, only 3 out of more than 20 of the city's hospitals remained open during the storm (see Figure 4-1). Only one of these, the Touro Infirmary, was not

completely surrounded by water, retaining access on one side. Access problems affected all hospitals. West Jefferson Medical Center had planned to deploy their staff in two shifts, to allow the second shift to stay at home until the storm had passed and then come to relieve the staff locked down in the hospital. Because of impeded access, the relief staff could not get to the hospital, which put an added burden on the initial staff, already nearing exhaustion.

Figure 4-1:
University Hospital surrounded by 4 feet of water

Figure 4-2: Evacuation of New Orleans by helicopter

Hancock Medical Center in Bay St. Louis, Mississippi, although located outside of the 500-year floodplain, experienced 3-foot deep flooding as a result of the storm surge (see Figure 4-3). Access to the hospital was disrupted long before Katrina made landfall because, prior to the storm arrival, there was a 33-mile traffic backup on the road leading out of town. Access was important because all the functioning hospitals needed relief supplies, medical gases, water, and fuel for their emergency generators. In many cases, the only way to resupply the facilities was by air, using large Chinook helicopters. These helicopters are too heavy for most roof structures, and to use them for emergencies in the future, a second helipad may be necessary on the site, requiring sufficient glide angles for landing and takeoff of the largest aircraft.

Access to emergency services was also blocked by the water and, in some cases, by trees and utility lines that were knocked down. Once the hospitals had access restored, they were deluged by the injured from nearby communities. Slidell Memorial Hospital administered 40,000 tetanus shots in the days after the storm. Hancock Medical Center saw 600 to 700 patients a day for up to 2 weeks after the surge water receded.

Figure 4-3:
The lobby of Hancock Medical Center was under 3 feet of water.

Perhaps the most serious consequence of the impeded access was the way it affected the evacuation of hospitals in New Orleans. Serious disruptions in hospital operations required immediate evacuation, which could not take place because the streets were not accessible for up to 5 days. There was a critical need for a helipad, either on the roof or an equivalent landing area on a parking structure, with emergency lighting for night operation. Elevated parking structures were a great asset, providing both a protection for the vehicles and a convenient helicopter landing site on the roof. They were especially useful if the parking structure had an elevated pedestrian bridge to the hospital.

4.2.5 ARCHITECTURAL DESIGN

A typical hospital configuration is based on access requirements that usually place the emergency department on the first floor in order to receive walk-in patients or those brought in by ambulances. Clinical laboratory and imaging are frequently on the first floor as well, as are surgery and intensive care units in many smaller hospitals. All of these are vital services in the event of an emergency and for providing routine patient care. Location on the first floor frequently exposes them to additional risks from natural hazards, especially flooding, as became evident during Hurricane Katrina.

Building configuration and general shape frequently contribute to high-wind damage. Protrusions and projections in walls and roofs cause additional wind turbulence that increases uplift pressures. The penthouse at West Jefferson Medical Center illustrates the vulnerability of projections and corners to high winds (see Figure 4-4). Large portions of metal cladding came loose because they were not designed or constructed to resist these loads.

Canopies, which most hospitals have over drop-off areas, are particularly susceptible to uplift and other damage, if not designed to resist the loads (see Figure 4-5). Glass-enclosed lobbies and atria, common to many hospitals, also proved to be a hazard, because of the large areas of usually unprotected glazing that could easily shatter under the impact of wind-borne debris. In many cases, these areas were closed during the storm, thereby cutting off a major point of access to the hospitals.

Figure 4-4:
Penthouse at the West Jefferson Medical Center

Figure 4-5:
Canopy soffit damage at West Jefferson Medical Center

At Louisiana State University Hospital, the emergency generators in the basement were flooded and shut down, which put the entire hospital out of commission. Similarly, all the major mechanical equipment in Charity Hospital in New Orleans was in the basement, including fifteen 5,000-watt emergency generators. The hospital had to be evacuated soon after the basement flooded and the emergency power supply failed. Touro Infirmary, however, had the emergency power generators located on the third floor. This allowed them to run most of the critical systems, including the

air conditioning, without interruption until the generators broke down from prolonged use and contaminated fuel supplies.

Critical operations such as emergency and surgical departments, recovery rooms, ICUs, and other patient bed units and laboratories should not be located in areas below ground or below the elevation of possible flooding and storm surge. These critical functions should be located on upper floors or in areas where communication between floors is easily accomplished. These areas should have no windows, or should have protected glazing to prevent window breakage and rain water penetration.

4.2.6 BUILDING ENVELOPE

Building envelope damage during Hurricane Katrina was widespread and included uplifted roof coverings and flashing, roofing punctured by flying debris or overturned roof-mounted equipment that led to extensive rainwater penetration, wall cladding separation, and window and door breakage.

The building envelope on the Garden Park Medical Center in Gulfport, a relatively new building opened in 2000, sustained considerable damage from 130-mph winds during Hurricane Katrina. The estimated wind speed may appear to have been close to the current design wind speed of 135 mph for this facility, but the actual pressures were below the current design pressures as a result of the 1.15 importance factor required for hospitals. Wall cladding consisting of EIFS was blown off in several areas, allowing water to penetrate wall cavities (see Figure 4-6). Extensive use of EIFS, despite a long track record of failure during hurricanes, contributed to significant damage from water penetration. EIFS is a popular wall cladding system, but not strong enough to prevent damage from wind-borne debris in cases where EIFS is applied over studs. In addition, EIFS design or construction deficiencies frequently make it insufficiently resistant to suction pressures caused by high winds. This is especially significant for hospitals, where such damage can allow water penetration and trigger serious disruptions in the mechanical and electrical systems and damage the building interior. Hospitals in hurricane-prone regions that have EIFS should have field testing performed to evaluate its attachment.

Figure 4-6:
Repair of EIFS wall covering on Garden Park Medical Center

Hancock Medical Center suffered damage to the wall sheathing behind the exterior brick veneer. This damage resulted from standing water at significant depths in various locations around the building. The sheathing retained moisture long enough to compromise its integrity through swelling, and to support the growth of mold and bacteria. There have been numerous examples of the failure of brick veneer. The reasons for this include corroded brick metal ties, insufficient number of metal ties installed, or ties not adequately embedded in the mortar.

Many hospitals have low-slope roofs. There have been many instances of failure of this type of roof, frequently beginning at the edge flashing and progressively spreading to other parts of the roof. Roofs are also susceptible to puncture, as happened at the Hancock Medical Center and Garden Park Medical Center (see Figure 4-7). Rubber walkway pads were blown away and the roof membrane was punctured in several places by displaced equipment and other flying debris. As a result, a substantial amount of water leaked through the roof openings into the top floor, causing considerable damage to the interior. In addition, the aggregate surfacing blew off, damaging glazing on surrounding buildings.

Figure 4-7:
Roof membrane starting to peel off at Hancock Medical Center.

Hancock Medical Center lost substantial portions of its metal roofing. Patients had to be relocated from the top floor to lower floors because of roof leakage (see Figure 4-8). West Jefferson Medical Center lost portions of roofing on the Psychiatry building, when the metal roof covering partially peeled off. The leading edge began to peel back, but did not go any further than the edge flashing, so a minimal amount of water penetrated through the roof. The psychiatry building was not occupied at the time because the hospital evacuated the building before the hurricane landfall.

Figure 4-8:
Roof damage on Hancock Medical Center

The primary cause of window breakage was wind-borne debris. (see Figure 4-9). West Jefferson Medical Center had 76 windows broken, mainly by flying aggregate. Intensive care patients had to be moved to the recovery room in the interior of the building and away from windows. The Psychiatry building had rainwater penetration through windows, even though they were not broken.

Although many important hospital functions are located away from the exterior windows, wind-blown rain can damage expensive equipment even when it is located some distance from the broken window (see Figure 4-10). At West Jefferson Medical Center, the fitness center building sustained $250,000 worth of damage that resulted from water driven through the broken windows and from 30 days of high humidity, before the air-conditioning was restored.

Exterior doors were often pushed in by floodwaters and blown open and damaged by wind pressure. Breakaway doors are particularly vulnerable to opening in high-wind conditions, as wind pressure can build up through the unsecured doors. Ground floor entrance doors at Hancock Medical Center had to be blocked by sandbags and two-by-fours, both on the inside and the outside, to stay closed (see Figure 4-11). The penthouse door at Garden Park Medical Center in Gulfport was blown off its hinges by strong winds.

Figure 4-9:
Broken windows at Touro Infirmary

Figure 4-10:
Broken windows at West Jefferson Medical Center

Figure 4-11:
Blocked doors at Hancock Medical Center

Hancock Medical Center lost numerous fan cowlings and other rooftop equipment, which left openings in the roof. Water penetrated the building through these openings, reaching the first floor, and damaged boilers and other equipment. Vent openings in the elevator penthouse in Touro Infirmary were blown off, allowing water penetration. The water damaged the electrical and mechanical equipment and controllers, shutting down the elevators.

West Jefferson Medical Center lost a number of cowlings and other covers causing water damage to equipment in the rooms below the roof. Some motors were sealed and continued to work during the storm.

All rooftop equipment should be safely secured to the curb and stay in place during a high-wind event. Equipment that has a high failure rate in high-wind events includes air-conditioning condensers, HVAC units, exhaust fans and air intake and exhausts.

4.2.7 UTILITY PLUMBING SYSTEMS

Hospitals depend heavily on municipal services and other utilities. While it is possible to go into a minimal function mode and still maintain patient safety, certain utility systems must be operable. In many cases, Hurricane Katrina and the subsequent flooding pushed hospitals beyond the limits of even a minimal function mode. Many had to be shut down, and the patients and staff had to be evacuated. In addition to the loss of electrical power, the most common problem in maintaining the operations proved to be the failure of water supply and sewer systems.

Most hospitals lost water within a day or two following Hurricane Katrina's landfall. Even when the water service was restored, it was suspected of contamination. Drinking water was in short supply in many hospitals. West Jefferson Medical Center received three truck loads of bottled water from a local retailer in the aftermath of the storm, but other hospitals suffered from serious shortages. A running water supply is critical for boilers to produce steam for sterilizing; for the chillers for the air-conditioning system; and for sanitary uses like toilets, washing dishes and linens, bathing patients, etc. The most successful hospitals had their own wells from which they pumped water using an emergency power supply. The hospitals that stayed without water and an emergency power supply had to evacuate patients as quickly as possible. As a consequence of this experience, West Jefferson Medical Center is planning to install two wells to provide both sufficient capacity and redundancy. One well will service the central plant boilers and chillers, and the other will be dedicated to hospital operations.

Potable water service will likely be shut down during and immediately after an event because of line breakages, lack of power to run pumping stations, or repair delays because of blocked roads and access routes. Hospitals, especially those in hurricane-prone regions, need a back-up water supply system in the form of:

- Wells for exclusive hospital use should be supplied with their own emergency generator to run the well pumps.

- Water storage or water recycling systems on site should serve as a back-up to the central water supply.

In many areas, the sewer system broke down shortly after the storm arrived, either because of loss of power required to run municipal pumping stations, sewage back-up or shut down of sewage treatment plants as a result of flooding, or because uprooted trees broke the sewer lines. When the water and sanitary waste systems were disabled, patients and staff were required to use hazardous waste bags (red bags) taped to buckets or toilet bowls to manage bodily waste. At West Jefferson Medical Center, used bags were stored in the hallways for several days and then buried on the hospital site for later disposal.

Hospital design in hurricane-prone regions should also take into consideration the need to provide an onsite sewage-diversion and storage system to prevent sewer backflows from disrupting hospital sanitary system operations. Field observations indicate that in situations where the sanitary sewer system may be damaged, causing a disruption in service, functionality may be protected by:

- Installation of an onsite septic tank as a sanitary back-up system

- Installation of backflow devices in the sanitary lines to prevent sewage from backing up into the hospital

- Designating, as part of the operations plan, a limited number of sanitary systems to use in the hospital, so the onsite storage tank does not fill too quickly

- Creating holding ponds for direct pumping of sewage, if sufficient site area is available

4.2.8 MECHANICAL AND ELECTRICAL SYSTEMS

Hurricane Katrina struck at a time when the temperatures during the daytime reached 95°F. Air-conditioning systems in hospitals were generally not connected to the emergency power supply, nor was there any flexibility to switch emergency power from one system to another to provide some relief. The heat build-up was a significant factor in creating difficult conditions for patients and staff (see Figure 4-12).

Lack of air-conditioning also allowed humidity to build up, which caused problems with switchgear, computers, and other electronic equipment. In many cases dehumidifying and cooling could not resume for months after the event. The excessive humidity damaged the electronic components of mechanical and electrical equipment, such as fire alarm systems, elevator switchgear, or the telephone switchgear, as happened in Touro Infirmary. Mold growth in conditions of high temperatures and humidity caused further damage. At West Jefferson Medical Center, rainwater and high humidity caused considerable damage to the fitness center, requiring all the electrical equipment to be replaced. The ground floor at Hancock Medical Center was flooded 3 feet deep; enough to cover all electrical outlets close to the floor. The damage was irreparable.

Figure 4-12:
Toppled HVAC equipment at Charity Hospital, New Orleans

Air-conditioning systems were also disrupted by a lack of water needed for water-cooled chillers, as happened in Garden Park Medical Center in Gulfport when the municipal water service was shut down. Loss of air-conditioning resulted in interior condensation, and warm and humid air interfered with the normal functioning of electronic medical equipment.

Measures to provide sufficient HVAC capacity for patient-occupied and support areas and to protect the operation of the hospital from humidity-related damages include the following:

- Install emergency electrical generators with capacity to run the air conditioning system. This might also require water for the chillers if this is the method of delivering cold air.

- Install sufficient controls in the hospital to be able to identify the areas with mold and mildew problems, and have them cordoned off quickly.

- Install HVAC duct cleanout locations, so that any mold or mildew that begins to grow in the ductwork can be easily cleaned out.

- Create air-conditioning zones (where practical), so that crucial operations can be cooled with a minimum amount of air-conditioning.

In a storm of Katrina's magnitude, it is expected that power would be lost almost immediately and for a considerable period of time thereafter. For example, Touro Infirmary lost power within minutes of being hit with wind gusts of about 60 mph. Since power is so vital to maintaining functionality in critical facilities during normal periods, it is of paramount importance to provide an emergency power supply for all critical hospital functions.

Most hospitals store up to 72 hours of emergency fuel on site. In the case of Hurricane Katrina, emergency generators needed to run continuously for 5 days or more. Many hospitals had to obtain fuel for their generators from outside sources, usually the military. West Jefferson Medical Center got fuel from a nearby Navy ship. Hancock Medical Center had one of their underground emergency generator fuel tanks flooded, with water

covering the fill cap and all vent openings. They could not use this tank, fearing that contaminated fuel would damage the power generators.

The capability to switch power to different locations at different points in time is one of the most important features of emergency power supply systems. However, most of the wiring systems in facilities affected by Hurricane Katrina were not set up with this capability. Emergency power in most cases was not available to run the air-conditioning systems. Redundancy in the emergency power systems that would have allowed maintenance and repair without disruption was lacking in most cases.

Portable generators, where they existed, were invaluable. They could be moved around as needed, but the problem of outside ventilation was difficult to overcome when the power was required away from vent openings. Use of generators indoors is extremely dangerous and should not be attempted in any circumstances.

More and more hospitals are dependent on computerized systems for patient medical records, transmission of X-ray film and other images, reporting laboratory results, patient physiological monitors, and a myriad of other uses. Once the power is lost, these systems are shut down unless they are on emergency power.

Piped oxygen and nitrous oxide supplies are essential for many patients. With electrical systems out, pumping of medical gas was not possible. Most of the hospitals maintained tanks of oxygen that could be brought to the bedside (Figure 4-13), but once the emergency supplies of medical gas ran out, the patients at serious risk had to be evacuated.

The experience of hospitals during Katrina indicates that the requirements for emergency power should have high priority. Generators should be located above the base flood elevation, be protected from flying debris, and have appropriate exhaust vent systems installed. Fuel storage tanks should be located above the flood elevation, or adequately anchored to ensure that the tank will not float off its foundation under pressure from rising floodwaters. Hospitals in hurricane-prone regions should store sufficient fuel to support running generators at full load for at

least 96 hours (4 days). Emergency power distribution systems should be able to provide power to every system in the hospital, and have switching capabilities to shift loads to different parts of the hospital as needed.

Figure 4-13:
Oxygen supplies at
Hancock Medical Center

4.2.10 COMMUNICATIONS SYSTEMS

Hospitals have a variety of communications systems in addition to the usual telephone and e-mail systems. Many of these are for emergency communications with ambulances and other emergency support services. These include satellite telephones, government line telephones, ham radios, internal radio systems, nurses' call systems, and wireless systems of various sorts. Communications systems are critical for medical uses, but during a storm like Katrina, the ability to contact family members and friends to check on their status meant a lot to both the staff and the patients. The importance of being able to get news from outside, to request restocking of supplies, and to arrange evacuation of patients became obvious in the aftermath of Katrina.

One consistent theme in damage reports was that satellite dishes were toppled, rendering satellite phones, one of the main sources of emergency communications, inoperable. Antennae used for

emergency communications frequently broke, disconnecting the hospital systems from the outside world. Hancock Medical Center lost not only the satellite dish, but also the dish for the education system and its television antennae. West Jefferson Medical Center lost only one antenna.

External communication isn't the only important link that can be broken during a hurricane. Internal communications (for example, from surgery to the nursing floor, or from the nursing floor to the supply storage areas) are critical to maintain functionality.

At Hancock Medical Center, the only means for internal communications were 5-Watt Motorola radios. These continued to work well, and were relied upon exclusively when the telephone switchgear flooded and broke down. West Jefferson Medical Center also used radio communication that continued to work throughout the storm. Cellular phones also worked, until the transmission tower was toppled. Hospitals that had ham radio operators bring their equipment in, or had a ham radio set-up in the hospital, found this system very important and useful. Several hospitals plan to have ham radios available for future emergencies.

Charity Hospital and West Jefferson Medical Center lost their computers because the computer network power supplies were not wired into the emergency power supply grid. John Hancock Medical Center also lost its computers when the ground floor flooded (see Figure 4-14).

Since many communication systems are dependent on external networks that may be out of commission, roof antennae should be well anchored, or mounted inside of penthouses. Satellite dish antennae may have to be taken down prior to a hurricane and put back after the storm had passed. Finally, redundancy of systems is important, as proven during Katrina—one system may work where others do not.

Figure 4-14: Damaged computer network equipment at Hancock Medical Center

4.2.11 NONSTRUCTURAL AND OTHER SYSTEMS

Nonstructural systems, among others, include interior non-load-bearing walls, ceilings, and floors. In hospitals flooded with storm surge, floors were destroyed and gypsum board walls were soaked. At Hancock Medical Center the water level reached 3 feet. The hospital must replace all the flooring material, rewire the outlets, and replace the wall sheathing at a level about 2 feet above the high water mark (see Figure 4-15).

In many hospitals humidity buildup damaged floor and ceiling tiles. Touro Infirmary had to replace 60,000 square feet of tile flooring and all of the ceiling tiles in the hospital because of humidity damage. The loss of air-conditioning at West Jefferson Medical Center caused the buildup of humidity in the air, leading to water condensation on terrazzo floors, making them very slippery.

Security was a major issue for most New Orleans hospitals. Members of the community naturally sought the hospital for shelter when their homes were destroyed. When the water rose, people trapped in the city followed the water's edge to the Touro Infir-

mary and West Jefferson Medical Center, where crowds of people congregated on the front doorsteps. Attempted break-ins and looting were reported, which prompted hospitals to barricade doors, post armed guards at entrances, and "lock down" so that no one could either enter or leave. At University Hospital, the doors were removed and the entrances sealed up.

Hospitals that allowed families to come in with the staff for the duration of the storm encountered numerous problems after a few days. Many of the family members became restless because hospitals had no appropriate accommodations for them. This became a security problem, as it was important to keep guests away from the patients and their caregivers.

Figure 4-15:
Drying of flooded ground floor at Hancock Medical Center

Fire safety should be a concern during a storm since the fire risks are usually greater than at other times. Although the fire alarms are normally connected to emergency power, other types of damage made them inoperable. At Touro Infirmary, the build-up of humidity knocked out the electronic components of the fire alarm system. After the storm, 470 points needed to be repaired before the system was fully operational again. At West Jefferson Medical Center, the fire alarm system was undamaged,

but the line to the fire station was cut. Heat-activated fire suppression systems continued to function where sufficient water pressure was available, but dry standpipes might not have been supplied with water in the event of a fire. Directional signs in many hospitals were blown away. In the rush for care after the hospitals were accessible, it was necessary to put up temporary signage to route the large volume of visitors and patients to the right destination.

4.2.12 SUMMARY

The General Accounting Office (GAO) report to Congress on the evacuation of hospitals and nursing homes noted that administrators should "consider several issues when deciding to evacuate or to shelter in place, including the availability of adequate resources to shelter in place, the risks to patients in deciding when to evacuate, the availability of transportation to move patients and of receiving facilities to accept patients, and the destruction of the facility's or community's infrastructure." For new facilities, most of these issues can and should be addressed during site selection, risk assessment, and application of the appropriate planning and design recommendations described in this manual. For existing facilities, careful evaluation and consideration of the recommendations provided in this manual should help ensure greater resilience during flooding and high-wind hazard events. In this way, communities can continue to depend on these facilities both for the care of existing patients and the care of people in disaster emergencies.

4.3 EDUCATIONAL FACILITIES

4.3.1 BACKGROUND

Educational facilities are considered critical facilities because, apart from their vital role in educating children, they are frequently used during and after a storm as emergency shelters, or as staging centers by the National Guard, law enforcement personnel, or critical infrastructure repair crews for emergency operations. Educational facilities, in this case mostly K-12 school buildings, are generally equipped with food preparation facilities, well distributed sanitary facilities, and ample space for personnel and equipment.

The primary function of a place of refuge shelter is the protection of the occupants during a storm. This function is dependent on preservation of the structural integrity of the building and the building envelope. The failure of a structural component or a breach in the building envelope can lead to casualties and can make the buildings unusable as a recovery shelter after the storm.

Educational facilities planned for use as place of refuge shelters must, above all, protect the lives and well-being of evacuees. To do so, they must be constructed to withstand storm surge or inland flooding and wind impact. This means that they must be located in areas that minimize these storm effects. If they are located in the areas subject to flooding and high-wind hazards, they need to be constructed to preserve full functionality, with uninterrupted electricity, communications, water, and sanitary service. This will

also allow them to serve as recovery shelters. Educational facilities that may not have served as shelters during the storm, because of their proximity to the coast or their susceptibility to inland flooding, may be useful as recovery shelters provided they are able to survive the hurricane intact.

The primary function of a recovery shelter is to provide a location for resources and manpower after the storm has passed, to centralize the command and distribution functions for recovery work in the community. Recovery shelters require that all building functions and services remain usable in the aftermath of a storm. They are often used by the American Red Cross (ARC), National Guard, or other government agencies as a distribution point of first aid, food, water, and other supplies. They also serve as management or information dissemination hubs concerning recovery efforts.

The following facilities were visited for the preparation of this manual:

- Charles P. Murphy Elementary School (Pearlington, Mississippi)
- Pass Christian Middle School (Pass Christian, Mississippi)
- St. Stanislaus High School (Bay St. Louis, Mississippi)
- Northbay Elementary School (Bay St. Louis, Mississippi)
- Hancock High School (Kiln, Mississippi)
- Pineville Elementary School (Pass Christian, Mississippi)
- D'Iberville High School (D'Iberville, Mississippi)
- D'Iberville Middle School (D'Iberville, Mississippi)
- Lyman Elementary School (Gulfport, Mississippi)
- East Hancock Elementary School (Hancock County, Mississippi)
- Saucier Elementary School (Saucier, Mississippi)
- Port Sulpher School (Port Sulpher, Louisiana)

4.3.2 EFFECTS OF FLOODING

The most devastating damage to educational facilities during Hurricane Katrina was caused by the storm surge. FEMA reported a storm surge of 20 to 30 feet above normal tide levels. In some places the flooding extended up to 6 miles inland. The effect on coastal communities was catastrophic, because most buildings within a quarter of a mile of the shore were virtually washed away, while most of the remaining buildings beyond this area were severely damaged. Virtually all of the educational facilities in Pass Christian, Mississippi, were heavily damaged as a result of their proximity to the coastline.

Inland flooding occurred in low-lying areas farther away from the coastline. D'Iberville Middle School had approximately 8 feet of water, but was not used as a shelter because the school district staff was aware of the potential for flooding from past experience. The structural integrity of the school was not compromised by the flooding, but the water damage to the interior was substantial (see Figure 4-16).

Figure 4-16:
D'Iberville Middle School was flooded to the depth of 8 feet

The experience of Hurricane Katrina shows that the damage caused by rising water generally renders shelters uninhabitable and useless, further aggravating the recovery process. In most cases the mechanical and electrical systems were disabled as a result of flooding, which allowed the internal temperatures to reach intolerable levels. Additionally, the flood-induced back-flow of plumbing systems created unsafe sanitary conditions in most of the facilities.

4.3.3 EFFECTS OF HIGH WINDS

Hurricane Katrina reached maximum gust wind speeds of 130 mph at landfall, with hurricane force winds extending outward 105 miles from the center of the storm (FEMA, 2006). Numerous tornadoes were also spawned by the hurricane, contributing to further damage as the storm moved northward. There were 11 tornadoes recorded in Mississippi, with 17 more in Georgia, and 4 in Alabama. Current model building codes in the areas affected by Hurricane Katrina generally require buildings to be designed to meet 120 to 150 mph design wind speeds. Some buildings, however, were constructed prior to the implementation and enforcement of the current model codes.

Wind damage was most evident on the building envelope, especially roofs, walls, doors, and windows, as well as other exterior elements, such as walkway canopies and exposed mechanical and electrical equipment. The extent of the damage was dependent on the force of the wind, the type of construction, and the configuration of the buildings. Once a building envelope was breached, the interior of the building sustained additional damage, both as a result of pressurization and rainwater penetration. The most severe damage in the interior was observed on the least durable finishes, such as acoustical ceilings, wood doors and trim, and building contents such as office equipment, furniture, and books.

Most of the educational facilities used as place of refuge shelters suffered wind damage to the roofs and windows. When a portion of the roof was lost or a window was broken, rainwater was able to enter the building, causing damage to the interior. In some instances, the occupants were forced to relocate to other buildings during the storm, because of the danger of progressive building failure (see Figure 4-17).

Figure 4-17:
Broken windows at
D'Iberville High School

4.3.4 SITE SELECTION

Most educational facilities that were destroyed or severely damaged by Katrina were located in the areas subject to storm surge flooding. Ideally, educational facilities used as shelters should not be located in floodplains, but since they must be located in proximity to the neighborhoods they serve, some are built on flood-prone sites. Even if buildings can be elevated to reduce the potential for damage, the surrounding area remains susceptible to flooding, which could prevent access and disrupt the delivery of emergency aid. This also underscores the need to provide multiple routes to and from a facility, in case of roadway blockage following a storm.

Pineville Elementary School near Pass Christian, Mississippi, was used as a place of refuge during the storm, but when the water rose to a depth of 2 feet, the people had to be moved by school buses to another shelter (see Figure 4-18). The buses were driven by school bus mechanics struggling to maintain control in rising water and 80 mph winds.

The schools in Pass Christian—mostly one-story buildings located in mapped flood hazard areas—sustained heavy damage from

storm surge flooding. The massive storm surge devastated parts of Northbay Elementary School in Bay St. Louis, Mississippi, a single-story building located in a mapped flood zone. Exterior walls collapsed, and most of the interior and contents were destroyed (see Figure 4-19).

Figure 4-18:
Pineville Elementary School shelter had to be evacuated when the floodwaters started to rise.

Figure 4-19:
Exterior walls at Northbay Elementary School

St. Stanislaus High School in Bay St. Louis, although located right on the coast, was on naturally higher ground, and only the first floors were affected by the storm surge. Within a few months after the storm, the second floors in some of the buildings had been repaired, including restored power, water, and sanitary service, and were back in use (see Figure 4-20).

Figure 4-20:
Lower floors washed away by the storm surge at St. Stanislaus campus

Considerable damage occurred in low-lying areas that were flooded by storm surge ranging in depth from 2 to 8 feet. Such was the case at D'Iberville Middle School in D'Iberville, Mississippi, which sustained severe damage, including the destruction of many exterior walls, when water rose to nearly 8 feet above the floor. Fortunately, it was not used as a shelter, unlike the nearby D'Iberville High School, which is a designated shelter. The high school is built on higher ground and was not affected by flooding.

4.3.5 ARCHITECTURAL DESIGN

Generally, educational facilities have centrally located corridors (with classrooms or other spaces on both sides) that have ready access to sanitary facilities, are at least 8 to 10 feet wide, and are free of all obstructions. These features make them ideally suited for

emergency shelter use. Although there are a few exterior openings that can be breached by a storm, the hallways are generally well protected from outside elements. It should be pointed out, however, that in situations where the roof or the exterior walls fall, interior corridor walls may collapse as well. The corridors are usually unfurnished and readily available for a variety of functions. They served well as safe areas during the storm, with only a few notable exceptions. At the D'Iberville High School, the corridors had unprotected windows at each end. These windows were broken during the storm and it was necessary to move people into another building. Unfortunately, there were no enclosed walkways or interior corridors connecting the buildings, and it was necessary to go outside into the storm to reach the other building. Similarly, when the roof structure at Lyman Elementary School shelter started to fail, it became necessary to take the refugees outside before they could reach safety in another building (see Figure 4-21).

For hurricane shelter safety, it would be beneficial to have all of the buildings connected with enclosed corridors, to allow safer movement between buildings during a storm. Other components of the building envelope would have to be sufficiently resistant to wind and wind-borne debris impact to protect the occupants and the services they need. School corridors in particular should either have impact-resistant (or protected) windows or none at all. All exterior doors should also be designed to meet hurricane wind loads and wind-borne debris impact requirements, as described in Section 3.4.3.1).

Figure 4-21:
Collapsed portion of the Lyman Elementary School Building

4.3.6 STRUCTURAL SYSTEMS

Most of the educational facilities observed were one-story buildings with concrete slab-on-grade foundations constructed using concrete masonry load-bearing walls and steel-framed roofs. The roof joists were supported either by masonry walls or steel beams. Some facilities, like the East Hancock Elementary School, used pre-engineered systems consisting of rigid steel frames supporting a standing seam metal roof. Pass Christian Middle School and Saucier Elementary School used structural steel frames with standing seam or built up roofs. St. Stanislaus High School, located on the coast, had a concrete structural frame where the first floor was heavily damaged by the storm surge, but the foundations, concrete structure, and the upper floors survived. This was because the buildings on campus were located on higher ground than the surrounding area.

The storm surge exerted tremendous forces on the buildings in its path, first as the water rose, and again when it receded. The forces were strong enough to knock down exterior masonry walls, as happened in the Northbay, Charles B. Murphy, Pass Christian, and Plaquemines Parish schools (see Figure 4-22). In most cases, the main structural components survived with little or no damage.

Figure 4-22: Damaged exterior walls at Charles B. Murphy Elementary School

Designing the connections between structural components according to loads is critical to maintaining the load path and the structural integrity of school buildings in hurricane-prone regions.

4.3.7 BUILDING ENVELOPE

Newer educational facilities located some distance from the coast sustained mainly wind-induced damage to the building envelope, primarily to roof coverings and windows. Facilities located along the coastline were hit by the storm surge that destroyed most of the building envelope on the ground floor, including large sections of exterior walls. The walls collapsed under flood loads that exceeded all expectations.

At Charles P. Murphy Elementary School, many of the infill masonry walls were displaced, and the concrete slabs on grade were uplifted and severely cracked in several classrooms (see Figure 4-23). A similar slab failure also occurred at D'Iberville Middle School. The cause of this type of failure was an increase in upward hydrostatic pressure below the slab, causing it to burst upward.

Figure 4-23:
Cracked concrete floor slab at Charles B. Murphy Elementary School

The St. Stanislaus High School campus consists of several buildings that used different exterior wall systems, including masonry and precast concrete panels. The masonry walls on the first floor were heavily damaged by the storm surge, but it appears that the precast concrete panels withstood the force of the hurricane much better (see Figure 4-24). Large sections of the exterior masonry walls at St. Stanislaus, as well as in Northbay Elementary in Bay St Louis and Port Sulphur schools in Plaquemines Parish, collapsed under the pressure from storm surge (see Figure 4-25).

Figure 4-24:
Precast concrete paneling at St. Stanislaus High School

Figure 4-25:
Collapsed exterior masonry wall at Port Sulphur school campus

The exterior brick veneer walls at the Pass Christian Middle School and at St. Stanislaus cracked and separated from the wall framing under the impact of storm surge (see Figure 4-26). This kind of damage usually results from a failure of brick ties that did not manage to hold the brick in place. In many cases these attachments can corrode and allow the brick veneer to move under lateral pressure. Such damage allows the penetration of water into the interior, where additional damage to building contents is inevitable.

The wind-induced damage was limited primarily to the roof components at the edge, such as flashing and coping. More substantial damage occurred on a building at St. Stanislaus campus and at Harrison 9th Grade gym, where large sections of metal roof covering were blown away (see Figure 4-27). The standing seam metal roof panels peeled back and away from the roof framing, exposing the interior to rainwater and additional wind damage.

Typically, glazed doors, windows, and roof coverings are the building envelope components most susceptible to damage caused by wind-borne debris. Although glass or shutters designed to resist such wind-borne debris are available, none of the educational facilities had protection on the windows and doors. The corridor windows at D'Iberville High School were broken during the storm, which prompted the complete evacuation of the building.

Figure 4-26:
Cracked brick veneer at St. Stanislaus High School

Figure 4-27:
Metal roof covering peeled away at a St. Stanislaus campus building

For buildings located in flood hazard areas, especially areas not subject to storm surge, elevation well above the predicted flood level is the most effective damage-reduction measure. Dry floodproofing may be used to provide some degree of protection, although if floodwaters rise higher than the designed level of protection, the damage can be catastrophic. This technique, generally feasible for flood depths of only 2 or 3 feet, is expensive and difficult to implement on existing buildings. Depending on the expected depth of water, local soil properties, and the size and location of openings, the facilities can be designed to limit water infiltration through the walls, openings, and conduits, and prevent envelope failure due to excessive hydrostatic pressure. Dry floodproofed buildings must have detailed emergency plans, with clear instructions for deployment of devices and other measures. Lack of periodic maintenance can render floodproofing measures ineffective.

Another alternative for minimizing structural damage to existing buildings is to provide wet floodproofing. This approach, which is not allowed for new construction, allows water to flood the lower floors protected with water-resistant materials and finishes that can be easily cleaned and restored.

4.3.8 UTILITY SYSTEMS

Educational facilities are typically not equipped with emergency back-up systems, but are mostly dependent on municipal water and sanitary systems. The breakdown of municipal water and sewer services, caused by power outages which shut down water treatment plants and pumping stations, adversely affected most of the facilities. Although a few of the facilities affected by Hurricane Katrina were served by onsite wells and some had septic systems, lack of power prevented their full use. The sanitary sewer system backed up under pressure from floodwaters and, together with the loss of water service, created great difficulties inside facilities used as shelters.

The sanitation problems, especially the unpleasant odors that permeated the facilities, combined with high humidity and heat, posed a serious health risk. At the D'Iberville High School, the loss of water prompted volunteers to haul buckets of water from a nearby ditch to be used for flushing the toilets. After the storm had passed, portable toilets and showers were brought in by the ARC.

4.3.9 MECHANICAL AND ELECTRICAL SYSTEMS

Most mechanical systems depend on the exterior equipment mounted on the roof or attached to the exterior walls. The exposed equipment is the most vulnerable element in the system, because it is commonly damaged by floodwaters, strong wind, and wind-borne debris. Rooftop equipment, such as air-handling units and exhaust fans, typically were not adequately anchored to meet the hurricane-force winds and were frequently damaged or toppled. Through-wall fan coil units installed below classroom windows at the Charles P. Murphy Elementary School in Pearlington were ruined by rising floodwaters (see Figure 4-28).

As a result of HVAC failures, conditions in shelters became very unpleasant because of the heat build-up and the lack of ventilation. The interior temperatures and humidity rose to unbearable levels because of the hot weather that followed Hurricane Katrina. Without the advantage of sufficient natural ventilation, the atmosphere quickly became stuffy. Undamaged mechanical equip-

ment, especially HVAC systems, was not operational because of the power outage and the lack of emergency power supply. At D'Iberville High School, the principal reported using a generator to run a large portable box fan in the corridors to provide some relief. The same generator also provided power for night-time lighting in the corridors.

The Harrison County School District office was one of the few buildings equipped with a generator, and for that reason it was used as an EOC. It served as a command center and provided sleeping and dining facilities for emergency crews. The success of this experience reinforces the importance of having emergency power generator systems and a sufficient fuel supply. Emergency generators should be in a protected enclosure and at an elevation high enough to prevent flooding.

Figure 4-28: Damaged HVAC unit at Charles B. Murphy Elementary School

4.3.10 NONSTRUCTURAL AND OTHER SYSTEMS

It is essential to the operation of educational facilities used as place of refuge and recovery shelters that communications systems remain operational. The place of refuge shelters had a

variety of communications systems at their disposal at the beginning of the storm. This included the phone system, cellular telephones, school system radios, and police and fire radios. These systems proved to be unreliable during the storm when antennae and utility lines were damaged. Immediately after the storm, all communications had to be handled through messengers, until portable antennae were brought to restore both cell phone and radio service. The Harrison County school board is now considering the purchase of satellite phones to be used during emergencies.

Nonstructural components and contents of educational facilities sustained the greatest damage from flooding. At D'Iberville Middle School, all interior wood doors, frames, trims, casework, fan coil units, and furniture typically suffered severe damage from water exposure.

The majority of interior walls were constructed of un-reinforced concrete masonry. Portions of these walls and some exterior non-load-bearing walls at Charles P. Murphy Elementary School were knocked down by the storm surge (see Figures 4-29 and 4-30). If the buildings had been occupied during the storm, people could have easily been injured by falling debris. Although many of these walls are used as partitions and are consequently not load-bearing, their mass poses a threat to life and property should they collapse.

Figure 4-29: Damaged non-load-bearing walls at Charles B. Murphy Elementary School

Figure 4-30:
Interior damage at Charles B. Murphy Elementary School

4.3.11 EQUIPMENT AND AUXILIARY INSTALLATIONS

School equipment is not necessarily important for the operation of emergency shelters, except for kitchen and dining facilities. Schools that sustained flooding damage usually had all their food preparation and refrigeration equipment ruined. The equipment stored outside consists mainly of buses and other vehicles.

In Pass Christian, many of the school buses stationed in the town were destroyed by the storm surge (see Figure 4-31), although some of them were used during the storm to move people from Pineville Elementary School to another shelter because of rising water.

The Harrison County School District moved all of its buses farther inland before the storm, which ultimately proved prudent and allowed the buses to remain available for use after Katrina. In the aftermath, Harrison County School District had no fuel supply to operate the buses or other vehicles. Several years ago, the school district opted to issue gasoline cards to the bus drivers to fill up at various gas stations rather than at a central depot. After the storm there were few gas stations operating, and it became a time-consuming process to get fuel for any vehicle. In the future, Harrison County School District intends to have its own gasoline supply available on generator power to ensure that school vehicles are kept operational.

Figure 4-31:
School bus washed away by the storm surge

4.3.12 SUMMARY

Educational facilities intended to serve as emergency shelters, together with accompanying parking lots and access roads serving these shelters, should be located inland, away from the coast, and on high enough ground to avoid flooding. Educational facilities that are built closer to the shore because of the school district boundaries should be constructed with the first floor above the highest expected storm surge level. These educational facilities should not be used as shelters during a hurricane, but could serve as relief centers after the storm, as long as they do not sustain significant damage. No educational facilities should be constructed within the immediate area along the shore, where waves are the strongest.

Based on observations, the most resilient school buildings are built using reinforced concrete structural systems, or reinforced masonry construction. The best performing roof decks were cast-in-place concrete, precast concrete, or concrete topping over metal deck. Reinforced concrete or reinforced concrete masonry exterior and interior walls, including precast concrete panels and tilt-up concrete wall panels, seem to have sustained the least flood damage. Other observations indicate that the following measures may help reduce the damage in hurricane-prone regions:

- Doors should be designed to meet the positive and negative design wind pressure and impact resistance, as recommended in Section 3.4.3.1. Windows should use impact-resistant glazing or shutters.

- Educational facilities that will be occupied during a hurricane or needed within a few weeks afterwards should be equipped with an emergency generator.

- All shelters should have protected communications systems linked to the EOC.

- Exterior corridors should be enclosed to allow full access between buildings, so that no one is forced to go outside during a storm to get to another area of a building.

- All educational facilities intended for use as evacuation shelters should be designed according to FEMA 361 guidelines.

4.4 EMERGENCY RESPONSE FACILITIES

4.4.1 BACKGROUND

Emergency response facilities include EOCs, police stations, and fire rescue stations. All of these facilities are considered critical because they must remain functional to manage response and recovery operations during and after a hazard event. EOCs function as incident command centers for coordination and support of all emergency activities. The command and response personnel must remain on duty, in full readiness for action both during and in the aftermath of a disaster. In addition to personnel and resources, EOCs house the information and communications systems that provide feedback to the emergency managers to help them make decisions about efficient and effective deployment of resources. They also relay information to local residents, shelters, media, and other first responders, while providing Continuity of Government (COG) and Continuity of Operations (COOP).

Police and fire rescue facilities are critical to disaster response, because an interruption in their operation as a result of building or equipment failure may prevent rescue operations, evacuation, assistance delivery, or general maintenance of law and order, which can have serious consequences for the community.

While each of the three types of emergency response facilities is used for different operational purposes and their needs are

different, most of them require and depend on the following facilities:

Back-Up Communications Equipment: Conventional communications that rely on radio towers with repeater systems cannot be relied upon during high-wind events, because of the high probability that the towers will lose power, as occurred in many jurisdictions in Mississippi and Louisiana. All facilities need back-up communications systems such as very high-frequency (VHF) and ham radios, and adequate back-up communications equipment.

Accommodation Space: Adequate space for all the essential personnel to work, with provisions made for continuous operations, such as sleep areas, kitchens (with supplies for all duty personnel), laundry facilities, and shower facilities for all such personnel. It should also be noted that many residents view the local fire station as a point of shelter, and will seek it as a refuge when winds and conditions become dangerous in their own homes.

Situation Rooms: "Meeting rooms" in which to conduct local government business as well as confer with community members, media, and government officials, and for press conferences and dissemination of information.

Safe Equipment Storage: Patrol and rescue vehicles and other equipment must be adequately protected from flood waters, wind-borne debris, and driving rain, and be accessible and readily available for emergency use.

The observations included in this section are based on the examination of the following facilities:

- Harrison County EOC (Gulfport, Mississippi)
- Jackson County EOC (Pascagoula, Mississippi)
- New Orleans EOC (New Orleans, Louisiana)
- Hancock County EOC (Bay Saint Louis, Mississippi)
- Orleans District Levee Board Police Department (New Orleans, Louisiana)

- New Orleans Police Department (New Orleans, Louisiana)
- Gulfport Police Department (Gulfport, Mississippi)
- Pass Christian Police Department (Pass Christian, Mississippi)
- New Orleans Fire Department (New Orleans, Louisiana)
- Gulfport Fire Department (Gulfport, Mississippi)
- Pass Christian Fire Department (Pass Christian, Mississippi)
- Cuevas Volunteer Fire Department (Pass Christian, Mississippi)
- Back Bay Fire Company #3 (Biloxi, Mississippi)

4.4.2 EFFECTS OF FLOODING

Flooding during Hurricane Katrina, especially the impact of the storm surge and levee failures, caused heavy damage that disrupted the long-term functional capabilities of the emergency response system.

The facilities damaged by rising water were generally rendered uninhabitable after the water receded. As a result, the emergency response teams were forced to relocate the operations elsewhere. Evacuation and relocation was common among damaged fire rescue and police facilities and affected their operations on many levels. Firstly, relocating farther away from their service areas meant that the response time to emergency calls increased substantially. Secondly, the response teams were forced to operate from temporary accommodations with inadequate facilities that were frequently overcrowded. Thirdly, new facilities lacked adequate supplies and services for the extra personnel, which required numerous improvisations, further hampering the operations.

4.4.3 EFFECTS OF HIGH WINDS

The highest gust wind speeds during Hurricane Katrina were typically below the design wind speeds for this area, which range from 120 to 150 mph. Despite this, many emergency response facilities sustained damage that disrupted their operations, and in a few cases shut down the facility. Most of the damage was confined to the building envelope. Portions of metal roofing were lifted and peeled off; aggregate roof surfacing was blown off, becoming wind-borne debris; and roof-mounted equipment, including communication towers and antennae, were toppled or broken. Metal wall-cladding panels detached or peeled off on a number of facilities, exposing the interior to pressurization and rainwater penetration. Doors and windows were damaged by wind-borne debris, allowing the penetration of wind-driven rain.

4.4.4 SITE SELECTION

Ideally, the emergency response facilities should not be located in a floodplain or a site exposed to other types of hazards. However, emergency response facilities, especially fire rescue and police stations, must contend with geographic limitations pertaining to size and adequate coverage of their service areas that frequently place them in hazardous locations. Many of the facilities flooded during Katrina were located in designated floodplains. However, this fact alone does not explain the catastrophic damage sustained by facilities such as the police and fire stations in Pass Christian, Mississippi (see Figure 4-32), because most of the facilities damaged by flooding were built above the minimum required elevations. The primary reason for the widespread damage is the catastrophic nature of the flooding, caused by extremely high storm surge and the failure of levees in New Orleans.

In locations where the emergency response facilities were built to a higher standard than required by local regulations, the adverse effects of Katrina were substantially reduced. The Jackson County EOC occupies the second floor of a municipal building located in Pascagoula, Mississippi. An examination of the storm surge flooding associated with different hurricane scenarios indicated that the site would be inundated during a Category 3 hurricane. Consequently, the county decided to design the building to be

approximately 4.5 feet above BFE (see Figure 4-33). During Hurricane Katrina, floodwaters rose to about an inch below the level of the lowest floor. The building was not evacuated prior to the storm, because it was believed that flooding would not exceed the maximum levels recorded during Hurricane Camille. As floodwaters started to rise, the decision was made to evacuate most of the occupants to another building across the street. Despite this action, the EOC operations were not hampered significantly.

The Pass Christian police and west side fire station were less fortunate, as the storm surge flooding at these sites far exceeded the base flood elevation. It was estimated that the storm surge wave that hit the police building was at least 15 to 20 feet high, which rendered the building and the equipment stored inside totally unusable. The facility is currently looking at an alternative site on higher ground as a possible site for relocation and construction of a new station.

The headquarters of the New Orleans Police Department had approximately 2- to 3-foot deep water on the first floor, and a completely flooded basement (see Figure 4-34). The surrounding areas were also under water, isolating the facility and preventing its normal operation. All operations had to be transferred to other police facilities in the city.

Figure 4-32:
Police station in Pass Christian

Figure 4-33:
Jackson County EOC

Figure 4-34:
New Orleans Police Department Headquarters

The Hancock County EOC in Bay St. Louis, Mississippi, is located on naturally high ground outside mapped flood hazard areas. Nevertheless, the floodwaters inundated the ground level, damaging the interior finishes and practically destroying all equipment and contents (see Figure 4-35). Most of the EOC's operations were transferred to an alternate location prior to Katrina's landfall, most likely because the facility is mapped as part of an evacuation zone in case of a hurricane.

Figure 4-35:
Hancock County EOC

Site characteristics and landscaping contributed significantly to the extent and type of damage sustained by these facilities. The Gulfport Fire Station #5 in Gulfport, Mississippi, experienced significant disruption because of downed trees. One tree fell on the roof, causing minor damage, while two vehicles parked outside were severely damaged by the fallen trees. Furthermore, it took approximately 12 hours of cutting the trees before firefighters were able to open the access road and start responding to emergency calls. This experience also underscores the need to provide multiple routes into and away from a site, in order to have redundancy and minimize the possibility of isolation as a result of roadway blockage.

The experience during Hurricane Katrina proved the efficacy of preventive evacuation of equipment and personnel to a safe and secure location. It also proved wise to organize back-up facilities in other locations or in adjacent jurisdictions to serve as alternative command and operation centers. Emergency response facilities that did so were better equipped to respond to citizens' needs immediately after the storm.

4.4.5 ARCHITECTURAL DESIGN

Many buildings used as emergency response facilities were not initially designed for that purpose, or for operations under emergency conditions. During and after Hurricane Katrina, most of

them experienced significant problems, irrespective of the level of damage. Some of these facilities are located in existing buildings designed as regular office space. These facilities performed poorly, and although they were able to adapt to the circumstances, they did not operate as efficiently as those designed for their particular functions.

For example, EOC facilities in New Orleans were placed into civic center offices, which were not equipped with kitchens, showers, and other facilities essential for the smooth and continuous operation of an EOC (see Figure 4-36). As a consequence, the facility had to be relocated immediately after Katrina because the available accommodations were insufficient and poorly equipped. Some of the buildings occupied by first responders were originally designed for a different purpose and subsequently converted to their current use.

Figure 4-36:
New Orleans EOC in the City Hall building

Generally, any building, whether new or old, that is used as an emergency response facility should be carefully reviewed for compliance with the requirements for uninterrupted operation of the facility. Particular attention should be paid to issues that have

historically caused problems in building or operational performance, such as:

- Roof systems not designed for high winds and debris impact
- Rooftop equipment
- Unprotected exterior glazing
- Large, sectional and rolling doors not designed for high winds
- Communications towers
- Large roof overhangs
- Lack of facilities for an extended length of stay, especially emergency sanitary facilities and power supply

Basements are another design feature with a high damage potential, especially when important services and facility functions are located there. The basement at the New Orleans Police Department completely flooded, and all the essential equipment located there was severely damaged, crippling the facility for a long time. Observations indicate that essential functions and service equipment should be transferred from flood-prone basements to safer locations.

The relatively new Back Bay Fire Company #3 station in Biloxi, Mississippi, was built in 1996, and yet its design does not reflect the current needs of its occupants. The spaces are too small to accommodate the duty shifts. The kitchen is inadequate for longer stays, while the sleeping area has unprotected, large, storefront-type windows that represent a serious hazard in high-wind situations (see Figure 4-37). These minor architectural deficiencies may be amplified during the times of crisis and adversely affect the operation of the facility, especially if combined with other building component failures.

Figure 4-37:
Back Bay Fire Company
#3

4.4.6 STRUCTURAL SYSTEMS

Concrete and reinforced masonry have traditionally been the most robust structural systems for hurricane-prone areas, since they have a much higher reserve structural strength than other systems. During high-wind conditions, the added weight of the concrete helps counteract the uplift forces, while the mass and depth of concrete and masonry walls provides reserve structural strength that prevents the walls from being breached during high winds and flood conditions. However, with precast concrete elements, attention to design and construction of connections is important. This was generally confirmed during Hurricane Katrina.

The Jackson County EOC, located in Pascagoula, Mississippi, was built in 1977 and is an example of a structurally well-designed facility. The structure is composed of cast-in-place concrete, and performed remarkably well (see Figure 4-38). Although the EOC is located on the second floor of the building, water only came within 1 inch of the ground floor, which is approximately 4 to 5 feet above the surrounding grade. Other structural systems proven to be resistant to flood and wind loads are steel or concrete frames that are covered with precast concrete panels. These systems have very high reserve capacities that perform extremely well during high winds and storm surges.

Figure 4-38:
Reinforced concrete building for Jackson County EOC

In contrast, pre-engineered metal buildings performed less well. Although the main structural components of most of these buildings remained standing, suffering only light damage, the rest of the building components were not able to resist the forces of storm surge. Two prime examples of these buildings are the Pass Christian Police Department Headquarters and the Gulfport Fire Department Station #7 (see Figures 4-32 and 4-39). Both of these buildings were severely damaged and all equipment stored inside was destroyed.

Based on the observations, many existing structures can be retrofitted to perform better during high winds. Although such retrofits may be expensive and generally have limited capacity to strengthen the structure, they definitely increase the overall structural resistance. For example, roof decking can be retrofitted with additional connections to provide increased uplift resistance. Furthermore, roof decks should be attached securely to make the building diaphragm work as a unit and transfer loads adequately to the walls, while simultaneously preventing the deck from being pulled from the structure during high winds.

Figure 4-39:
Gulfport Fire Station #7

4.4.7 BUILDING ENVELOPE

When the building envelope is breached, the interior is no longer protected from the outdoor environment and the whole building is exposed to additional forces that may cause its progressive collapse.

The Gulfport Police Department Headquarters and the Pass Christian Police Station are the prime examples of building envelope failure. Both buildings, located near the coast, were exposed to the storm surge that crushed the lightly built exterior walls and destroyed everything in the interior. Fire Station #7 was breached by the storm surge, and practically the entire building envelope was washed away, except for the roof deck. What remained of the building was just a shell (see Figure 4-40). The duty personnel and most of the vehicles were relocated to other facilities until a trailer was provided as a temporary place of operations.

On the other hand, the Gulfport Police Department Headquarters building, constructed with heavy concrete masonry walls, sustained no significant damage to the building envelope. The building only required restoration of flooded building components.

Figure 4-40:
Exterior walls on the Gulfport Fire Station #7 washed away by the storm surge

Longbeach Police Station sustained heavy damage to its roof trusses, metal roof, and siding that allowed wind and rainwater to saturate the interior of the building. The police officers had to scramble to evacuate the prisoners and valuable records before they abandoned the building in the middle of the storm (see Figure 4-41).

Figure 4-41:
Longbeach Police Station

Third District Fire Station in New Orleans is a newer one-story, structural steel-framed building with brick veneer walls, metal fascia panels, steel roof deck, rigid plastic foam roof insulation, and metal roof panels. The estimated maximum wind speed at this location during Hurricane Katrina was significantly lower than the design wind speed. Nevertheless, a large portion of the metal roof covering was blown off the apparatus bay (see Figure 4-42). In some areas, the architectural metal wall panels with standing seams covered by snap-on battens were still in place, but the batten covers had broken away. In other areas, the batten covers were still attached, but they had lifted—it appeared that fatigue cracks occurred along the standing seams (see Figure 4-43). Battens like these are frequently susceptible to blow-off, which allows water infiltration and may lead to panel blow-off. Both the battens and separated panels may become dangerous wind-borne debris. This station was occupied at the time of the storm, but because of the extensive damage to the interior, apparatus bay doors, and the equipment, it could not be used and was consequently evacuated.

Figure 4-42:
Metal roof and wall panels peeled off the Third District Fire Station in New Orleans

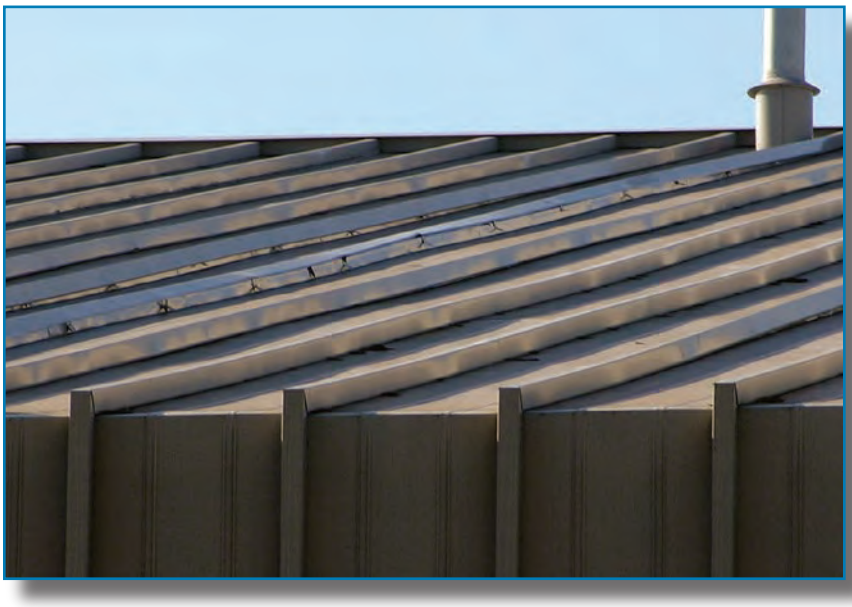

Figure 4-43:
Lifted batten covers on Third District Fire Station metal roof

Although the estimated wind speed in Bay St. Louis was slightly lower than the design wind speed, the wind blew off most of the roof membrane from the Hancock County EOC, located in the city (see Figure 4-44). The damage was initiated with the separation of metal edge flashing that had an uncleated vertical face. In addition to roof damage, the hardware on the exterior door failed and the door blew inward. As a result of rainwater penetration and flooding, most of the interior was ruined. Although most of the facility operations were moved before hurricane landfall, the remaining building occupants had to be evacuated during the event.

Jackson County EOC in Pascagoula experienced similar damage to its roofing when the metal edge flashing peeled off and lifted portions of the roof membrane. However, the roof damage did not cause water damage, because the cast-in-place reinforced concrete roof deck was capable of resisting rainwater penetration. The building's reinforced concrete walls and roof deck resisted the wind loads very well. The walls and roof deck were also extremely resistant to wind-borne debris, as was the exterior glazing retrofitted with shutters that protected the openings.

The edge flashing on low-slope roofs that usually initiates the peeling of roof membranes can be easily retrofitted with additional screws, to prevent it from uplifting and causing a progressive failure of the roofing system.

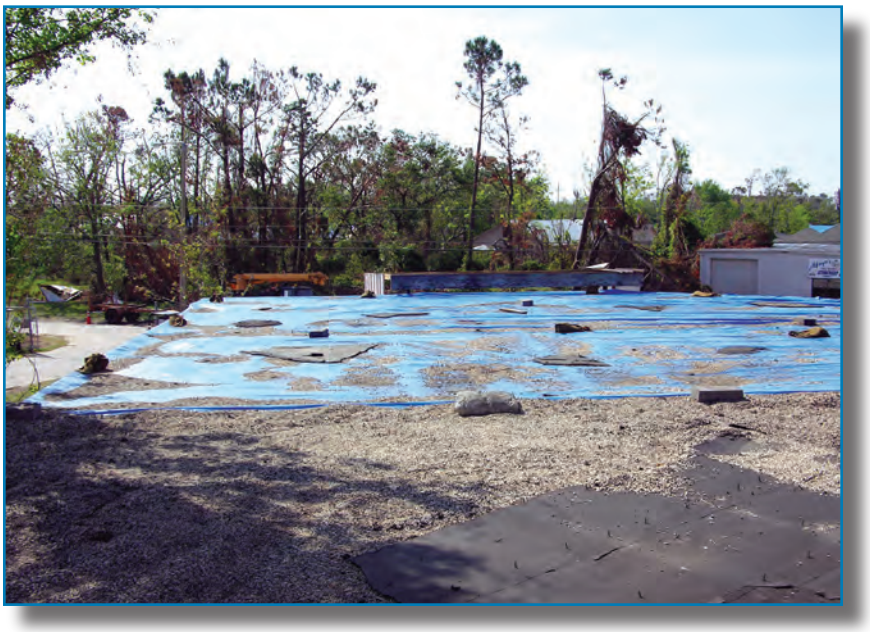

Figure 4-44:
Roof damage at Hancock County EOC

Fire stations are especially susceptible to breaches of the building envelope, because of their large sectional and rolling doors that are usually not strong enough to resist wind forces, and even less so the hydrodynamic forces of storm surge. The apparatus bay doors failed in many fire stations affected by flooding. (see Figures 4-39 and 4-45).

Water and wind from the storm were able to penetrate the buildings when the doors were breached, causing subsequent damage to other systems and equipment. Large doors should be designed to withstand wind pressures and windborne debris impact as recommended in Section 3.4.3.1.

All doors and windows can also be replaced with modern, impact-resistant systems that would reduce the chances of building pressurization and rainwater infiltration, which resulted in heavy losses to equipment and contents during Katrina. Rooftop units that were blown off and damaged the building envelope by puncturing the roof covering should be securely anchored to prevent such damage in the future.

Figure 4-45:
Damaged apparatus bay doors at Port Sulphur Volunteer Fire Department.

4.4.8 UTILITY PLUMBING SYSTEMS

Failures of public utility systems during Katrina were very common. Many first responder facilities were forced to improvise short-term solutions until public utilities were restored. The Gulfport Fire Department Headquarters was able to back-feed water into its lines by isolating the building's water supply lines from the public municipal supply system. They then fed water directly into the building's water lines in order to have water to shower and wash during the two weeks that they were without water in the facility. This capability should be considered for all emergency response facilities, as it minimizes disruption of basic sanitary functions.

In many facilities, flooding caused sewage to backflow into buildings, causing sanitary crises that directly affected their operations. Valuable time was spent cleaning up the facilities instead of helping others. To prevent this from occurring in future events the installation of backflow inhibitors (check valves) is recommended.

The Pass Christian Fire Department managed the loss of sanitary systems with plastic bags and buckets, while the staff at the Gulfport Police Department was able to acquire and use portable toilets and bottled water until public utility systems were restored. The Jackson County, Mississippi, EOC is equipped with a pressurized

underground tank for toilets and washing, which supported the occupants during the 3 to 4 weeks that the facility was without water.

The Cuevas Volunteer Fire Department in Pass Christian, Mississippi, was equipped with an underground septic tank, a drain field, and a well, and did not experience significant disruptions in its plumbing and fresh water systems. These independent septic and fresh water systems do not rely on public municipal systems, and are preferred where possible as they virtually eliminate the chances of disruption during widespread outages.

4.4.9 MECHANICAL AND ELECTRICAL SYSTEMS

Hurricane Katrina also affected facilities by damaging or destroying mechanical systems. Hurricane season occurs in the warmest months of the year, and many of these facilities were not designed to allow natural ventilation. For example, the New Orleans Police Department Headquarters Building is a multi-story building where the main circuitry for the HVAC system, which was located in the basement, was severely damaged by the flood. Since the building was designed as a closed structure, natural ventilation was a problem (see Figure 4-46). All the equipment located in the basement needed to be completely rebuilt or replaced before the building could be occupied again. In the interim, the entire department was forced to relocate its operations to other facilities in the city, placing a strain on facilities not intended to house additional personnel and take on additional responsibilities.

The inability to air-condition buildings because of damaged mechanical and electrical systems allowed internal temperatures and humidity to reach intolerable levels, and in many buildings mold began to form.

Loss of electrical power during and after Hurricane Katrina affected all other essential facility systems. Examples of this were evident at all of the sites visited. Utility, mechanical, and communications systems became partially or completely unusable, either because emergency power was not available, or it had to be rationed as a result of overload or breakdown of generators.

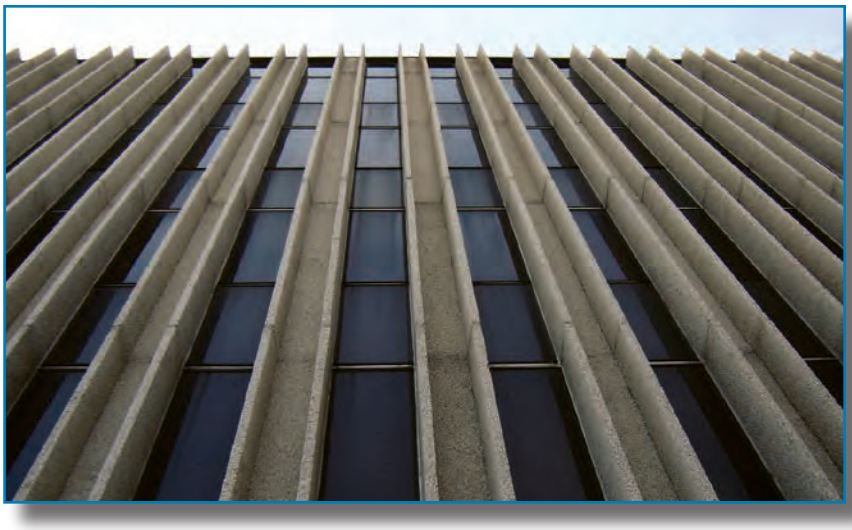

Figure 4-46:
Fixed windows on New Orleans Police Department Headquarters

Cuevas Volunteer Fire Department, located inland at Pass Christian, Mississippi, was without municipal power for approximately 2 to 3 days, but, since its generator functioned properly, their operations were only slightly affected by the storm. In contrast, the indoor generator and electrical panel at the Back Bay Fire Company #3 in Biloxi, Mississippi, became submerged in the flood, even though the generator was mounted several feet above the finished floor. The water flooded the building, putting all the mechanical and electrical equipment, including the generator, out of commission. As the water continued to rise, the firefighters and the local residents that sought refuge in the station had to climb to the top of fire engines until the floodwaters receded.

The New Orleans District 3 Fire Department Headquarters lost power as a result of flooding. The generator was located outside at grade level, and was ruined when approximately 2 feet of water flooded the area (see Figure 4-47). The firefighters were forced to relocate to the nearby West Bank facility for 4 months, until power was restored at the headquarters building.

Facilities that escaped deep flooding were typically operational immediately upon restoration of power. The Gulfport Fire Department Station #1 had water barely enter the station, and was able to get by on their generator for 3 days until municipal power was restored. Although their radios worked intermittently, they were able to perform their search and rescue duties and use their newly acquired chainsaws to help clear the roads of debris.

Figure 4-47:
Generator mounted at grade level damaged by flooding

The need to provide a back-up generator at a safe and elevated location is confirmed by the experiences during Hurricane Katrina. Many facilities were without power because of the low elevation and subsequent flooding of their generators. Jackson County EOC had its emergency power station elevated and protected in a separate enclosure, which allowed the facility to operate without interruption (see Figure 4-48).

Figure 4-48:
Jackson County EOC's elevated generator enclosure

4.4.10 COMMUNICATIONS SYSTEMS

It is essential for the operation of emergency response facilities to keep their communications systems intact. Loss of communication capability prevents their primary function of responding to community needs and adversely affects their ability to coordinate their actions. If the communications system malfunctions or becomes unavailable, the coordination between command centers will be hindered, affecting management of the response and recovery operations during and after an event. Many jurisdictions in New Orleans and along the Gulf Coast were cut off from each other and could not communicate, even with their own departments. For example, one police officer from the Orleans District Levee Board Police Department was in the heart of New Orleans when the city was flooded, and had no way of communicating with the senior officials in his department. He worked for days with the officers of the New Orleans Police Department (NOPD) assisting the remaining residents in the city. For a period of days, he and his partners from NOPD were operating without communications capabilities of any sort until the military arrived and issued them new radios. It took approximately 2 to 3 weeks for communications to be re-established in a manner that resembled normalcy.

The command and communications center for the police and fire rescue departments in Pass Christian, Mississippi, was located in the police department building, and was crippled as a result of the destruction of their headquarters (see Figure 4-49). The landline communications at the Jackson County, Mississippi, EOC continued to function for a day or two, and other communications systems only experienced minor problems. The building was used during and after the storm.

Hurricane Katrina experience indicates that alternative forms of communication need to be available during and in the aftermath of a storm. High-frequency and ham radios that do not rely on repeater systems should be in place, along with detailed instructions and plans for their use in emergencies. Emergency power supply for communications systems must be available at all times. This relatively low-cost solution would reduce the loss of communications during and after future events.

Figure 4-49:
Broken communications masts at Pass Christian Police Station

4.4.11 EQUIPMENT AND AUXILIARY INSTALLATIONS

Specialized equipment such as vehicles, rescue equipment, and fire pumps were the most common types of equipment lost during Hurricane Katrina. Damage to vehicles and other equipment seriously affected the operations, and frequently prevented speedy rescue and response efforts.

In many cases throughout Mississippi and Louisiana, the vehicles were stored immediately outside on facility parking lots while most of the specialized equipment was stored inside in apparatus bays. In other cases, as in Pass Christian, Mississippi, the police department stationed its vehicles in a remote location that was thought to be safe, but the entire area flooded and all vehicles were rendered useless. In Pass Christian the fire department, located approximately a quarter of a mile from the coastline, had four firefighting trucks and two rescue trucks ruined during the storm, hampering firefighters' efforts in responding to emergencies that required the use of their equipment. Gulfport Police Department and the New Orleans District Levee Board Police Department deployed their vehicles all over the area to ensure that the vehicles would be available on short notice after the storm.

For the 2006 hurricane season, Pass Christian emergency response plans include the evacuation of all vehicles to a staging site away from the coastline, and away from trees and other objects that may become wind-borne debris. This geographic distancing of vehicles from the coastline (and the area affected most by the storm and its surge) will protect key equipment and reduce the impact of a future storm by allowing the first responders to be mobile shortly after the event. It should be noted, however, that many jurisdictions in low-lying areas do not have safe staging sites at higher elevations or away from the coastline.

Based on conversations with many emergency response crews that were affected by Katrina, the protection of their equipment and vehicles was a main consideration prior to the storm. Many jurisdictions throughout Mississippi and Louisiana decided to spread their vehicles out to many locations thought to be safe, in order to be certain that at least some of them would remain operational. This practice proved prudent and enhanced their abilities to respond in the immediate aftermath of the disaster. The vehicles should be sheltered in an area that is safe from flooding hazard and easily accessible after the event.

4.4.12 SUMMARY

Emergency response services are at the backbone of a community's ability to protect and save the lives and property of its citizens in any crisis. The provision of these services depends on the uninterrupted operation of emergency response facilities during and after a hazard event. This means not only that the buildings that house the crews and equipment must survive the onslaught of flooding and high winds with minimal damage, but that all the equipment and systems necessary for emergency operations must remain fully functional.

Hurricane Katrina showed that emergency response facilities have a better chance of protecting their operations if they occupy solidly constructed buildings with sufficient reserve structural capacity that cannot be easily overwhelmed by a storm of this magnitude. It also showed that facilities with functional generators were better equipped to respond after the storm than those that were left completely without power. Since the facilities cannot op-

erate without adequate emergency power supply, all electrical systems should be connected to power back-ups, and emergency generators should be elevated and protected against floodwaters and wind-borne debris.

Mechanical systems need to be located in a sheltered area, where they will not be damaged by wind and flooding. Plumbing systems should be equipped with backflow inhibitors to prevent sewage from entering the structure during a flood. Provisions should also be made to allow the isolation of the building's water supply, so that it is possible to feed water directly into the building's water lines until municipal water supplies are restored.

Finally, the lines of communication between community command centers and individual facilities must remain functional at all times. The basic communications systems need to be protected against the effects of flooding and high-winds as much as possible, but as this storm showed, system redundancy is still the best policy. In the event of damage, the facilities should have alternative means of communication that do not depend on local systems and networks that could be damaged in the storm.

AASHTO	American Association of State Highway and Transportation Officials
ABFE	Advisory Base Flood Elevation
ADA	Americans with Disabilities Act
ARC	American Red Cross
ASCE	American Society of Civil Engineers
ASTM	American Society for Testing and Materials
ASCE/SEI	American Society of Civil Engineers' Structural Engineers Institute
BFE	Base Flood Elevation
BIA	Brick Industry Association
BUR	Built-Up Roof
C&C	Components and Cladding
CMU	Concrete Masonry Unit
DFE	Design Flood Elevation
EIFS	Exterior Insulation Finish Systems
EOC	Emergency Operation Center
EPDM	Ethylene Propylene Diene Monomer
FEMA	Federal Emergency Management Agency
FIRM	Flood Insurance Rate Map

FIS	Flood Insurance Study
FMA	Flood Mitigation Assistance
FMG	Factory Mutual Global
FMR	Factory Mutual Research
HMGP	Hazard Mitigation Grant Program
IBC	International Building Code
ICC	International Code Council
ICU	Intensive Care Unit
LPS	Lightning Protection System
MEPS	Molded Expanded Polystyrene
MOB	Medical Office Building
MWFRS	Main Wind-Force Resisting System
NBC	National Building Code
NEHRP	National Earthquake Hazard Reduction Program
NFIP	National Flood Insurance Program
NFPA	National Fire Protection Association
NSSA	National Storm Shelter Association
OSB	Oriented Strand Board
PA	Public Assistance
PDM	Pre-Disaster Mitigation
PMR	Protected Membrane Roof
PNP	Private Non-Profit
SBC	Standard Building Code
SEAW	Structural Engineers Association of Washington
SHMO	State Hazard Mitigation Officer
SPF	Sprayed Polyurethane Foam
UBC	Uniform Building Code

100-year flood. See "base flood."

A

Alluvial Fan. A fan-shaped deposit of alluvium formed by a stream where its velocity is abruptly decreased, as at the mouth of a ravine or at the foot of a mountain.

Astragal. The center member of a double door, which is attached to the fixed or inactive door panel.

B

Base flood. The flood having a 1-percent chance of being equaled or exceeded in any given year, commonly referred to as the "100-year flood." The base flood is the national standard used by the NFIP and all Federal agencies for the purposes of requiring the purchase of flood insurance and regulating new development.

Base flood elevation (BFE). The height of the base (1 percent or 100-year) flood in relation to a specified datum, usually the National Geodetic Vertical Datum of 1929, or the North American Vertical Datum of 1988.

Basic wind speed. A 3-second gust speed at 33 feet above the ground in Exposure C. (Exposure C is flat open terrain with scattered obstructions having heights generally less than 30 feet.) Note: Since 1995, ASCE 7 has used a 3-second peak gust measuring time. A 3-second peak gust is the maximum instantaneous speed with a duration of approximately 3 seconds. A 3-second peak gust speed could be associated with a given windstorm (e.g., a particular storm could have a 40-mph peak gust speed), or a 3-second peak gust speed could be associated with a design-level event (e.g., the basic wind speed prescribed in ASCE 7).

Building, enclosed. A building that does not comply with the requirements for open or partially enclosed buildings.

Building, open. A building having each wall at least 80 percent open. This condition is expressed by an equation in ASCE 7.

Building, partially enclosed. A building that complies with both of the following conditions:

1. The total area of openings in a wall that receives positive external pressure exceeds the sum of the areas of openings in the balance of the building envelope (walls and roof) by more than 10 percent.
2. The total area of openings in a wall that receives positive external pressure exceeds 4 square feet, or 1 percent of the area of that wall, whichever is smaller, and the percentage of openings in the balance of the building envelope does not exceed 20 percent.

These conditions are expressed by equations in ASCE 7.

Building, regularly shaped. A building having no unusual geometrical irregularity in spatial form.

Building, simple diaphragm. An enclosed or partially enclosed building in which wind loads are transmitted through floor and roof diaphragms to the vertical main wind-force resisting system.

C

Components and cladding (C&C). Elements of the building envelope that do not qualify as part of the main wind-force resisting system.

Coping. The cover piece on top of a wall exposed to the weather, usually made of metal, masonry, or stone, and sloped to carry off water.

D

Design flood. The greater of the following two flood events: (1) the base flood, affecting those areas identified as special flood hazard areas on a community's Flood Insurance Rate Map (FIRM); or (2) the flood corresponding to the area designated as a flood hazard area on a community's flood hazard map or otherwise legally designated.

Design flood elevation (DFE). The elevation of the design flood, including wave height, relative to the datum specified on a community's flood hazard map.

Dry floodproofing. An adjustment, modification, or addition of a feature, or any combination thereof, that eliminates or reduces the potential for flood damage by sealing walls and closing openings to keep water from entering a building.

E

Erodible soil. Soil subject to wearing away due to the effects of wind, water, or other geological processes during a flood or storm or long-term exposure.

Escarpment. Also known as a scarp. With respect to topographic effects, a cliff or steep slope generally separating two levels or gently sloping areas.

Exposure. The characteristics of the ground roughness and surface irregularities in the vicinity of a building. ASCE 7 defines three exposure categories—Exposures B, C, and D.

Extratropical storm. A cyclonic storm that forms outside of the tropical zone. Extratropical storms may be large, often 1,500 miles (2,400 kilometers) in diameter, and usually contain a cold front that extends toward the equator for hundreds of miles.

F

Federal Emergency Management Agency (FEMA). The Federal Emergency Management Agency is the Federal agency which administers the National Flood Insurance Program (NFIP).

Fetch. Distance over which wind acts on the water surface to generate waves.

Flashing. Any piece of material, usually metal or plastic, installed to prevent water from penetrating a structure.

Flood Insurance Rate Map (FIRM). The official map of a community on which FEMA has delineated both the special hazard areas, and the risk premium zones applicable to the community.

Flood Insurance Study (FIS). An engineering study performed by FEMA to identify flood hazard areas, flood insurance risk zones, and other flood data in a community; used in the development of the FIRM.

Floodplain. Any land area, including the watercourse, that is susceptible to partial or complete inundation by water, from any source.

Floodplain management regulations. Zoning ordinances, subdivision regulations, building codes, or special-purpose ordinances that set flood-resistant standards for new construction, land use, and development.

Flood profile. A graph of computed flood elevations at points located along a riverine waterway. A flood profile typically is available for a waterway that has Base Flood Elevations (BFEs) shown on the Flood Insurance Rate Map (FIRM). Flood profiles are usually found in the Flood Insurance Study (FIS) report.

Floodway. The channel and that portion of the floodplain that is to be reserved to convey the base flood, without cumulatively increasing the water surface elevation more than a designated height.

Floodway fringe. The area of the floodplain outside of the floodway, where floodwaters may be shallower and slower.

Freeboard. A factor of safety, usually expressed in feet above a flood level, for purposes of floodplain management. Freeboard also compensates for the many unknown factors that could contribute to flood heights greater than the height calculated for a selected size flood and floodway conditions, such as wave action, constricting bridge openings, and the hydrological effect of urbanization of the watershed. A freeboard of 1 to 3 feet is often applied to critical facilities.

Frontal dune. Ridge or mound of unconsolidated sandy soil, extending continuously alongshore landward of the sand beach and defined by relatively steep slopes abutting markedly flatter and lower regions on each side.

G

Glazing. Glass or a transparent or translucent plastic sheet used in windows, doors, and skylights.

Glazing, impact-resistant. Glazing that has been shown, by an approved test method, to withstand the impact of wind-borne missiles likely to be generated in wind-borne debris regions during design winds.

H

Hurricane-prone regions. Areas vulnerable to hurricanes; in the United States and its territories defined as:

1. The U.S. Atlantic Ocean and Gulf of Mexico coasts, where the basic wind speed is greater than 90 miles per hour.
2. Hawaii, Puerto Rico, Guam, U.S. Virgin Islands, and American Samoa.

Human intervention. The presence and active involvement of people necessary to enact or implement floodproofing measures prior to the onset of flooding.

Hydrodynamic load. Loads imposed by water flowing against and around an object or structure, including the impacts of debris and waves.

Hydrostatic load. Load (pressure) imposed on an object or structure by a standing mass of water; the deeper the water, the greater the load or pressure against the object or structure.

I

Impact-resistant covering. A covering designed to protect glazing, which has been shown by an approved test method to withstand the impact of wind-borne missiles likely to be generated in wind-borne debris regions during design winds.

Importance factor, I. A factor that accounts for the degree of hazard to human life and damage to property. Importance factors are given in ASCE 7.

L

Lowest floor. The lowest floor of the lowest enclosed area (including basement). An unfinished or flood resistant enclosure, usable solely for parking of vehicles, building access, or storage, in an area other than a basement area, is not considered a building's lowest floor, provided that the enclosure is compliant with flood-resistant requirements.

M

Main wind-force resisting system. An assemblage of structural elements assigned to provide support and stability for the overall structure. The system generally receives wind loading from more than one surface.

Mean roof height, (h). The average of the roof eave height and the height to the highest point on the roof surface, except that, for roof angles of less than or equal to 10 degrees, the mean roof height shall be the roof eave height.

Missiles. Debris that could become propelled into the wind stream.

N

National Flood Insurance Program (NFIP). A Federal program to identify flood-prone areas nationwide, and make flood insurance available for properties in communities that participate in the program.

Nor'easter. Nor'easters are non-tropical storms that typically occur in the eastern United States, any time between October and April, when moisture and cold air are plentiful. They are known for dumping heavy amounts of rain and snow, producing hurricane-force winds, and creating high surfs that cause severe beach erosion and coastal flooding. A nor'easter is named for the winds that blow in from the northeast and drive the storm up the east coast along the Gulf Stream, a band of warm water that lies off the Atlantic Coast.

O

Openings. Apertures or holes in the building envelope that allow air to flow through the building envelope. A door that is intended to be in the closed position during a windstorm would not be considered an opening. Glazed openings are also not typically considered openings. However, if the building is located in a wind-borne debris region and the glazing is not impact-resistant or protected with an impact-resistant covering, the glazing is considered an opening.

R

Racking. Lateral deflection of a structure resulting from external forces, such as wind or lateral ground movement in an earthquake.

Ridge. With respect to topographic effects, an elongated crest of a hill characterized by strong relief in two directions.

S

Scour. Removal of soil or fill material by the flow of floodwaters. The term is frequently used to describe storm-induced, localized erosion around pilings at building corners and other foundation supports where the obstruction of flow increases turbulence.

Seiche. A wave that oscillates in lakes, bays, or gulfs from a few minutes to a few hours as a result of seismic or atmospheric disturbances.

Sheetflow. Rainfall runoff that flows over relatively flat land without concentrating into streams or channels.

Stillwater elevation. The elevation that the surface of coastal flood waters would assume in the absence of waves, referenced to a datum.

Storm surge. Rise in the water surface above normal water level on the open coast due to the long-term action of wind and atmospheric pressure on the water surface.

Substantial damage. Damage of any origin sustained by a structure, whereby the cost of restoring the structure to its pre-damage condition equals or exceeds 50 percent of the market value of the structure before the damage occurred (or smaller percentage if established by the authority having jurisdiction). Structures that are determined to be substantially damaged are considered to be substantial improvements, regardless of the actual repair work performed.

Substantial improvement. Any reconstruction, rehabilitation, addition, or other improvement of a structure, the cost of which equals or exceeds 50 percent of the market value of the structure (or smaller percentage if established by the authority having jurisdiction) before the start of the improvement.

T

Tsunami. An unusually large sea wave produced by submarine earth movement or a volcanic eruption.

W

Wave runup. Rush of wave water up a slope or structure. The additional height reached by waves above the stillwater elevation.

Wet floodproofing. Permanent or contingent measures and construction techniques, applied to a structure or its contents, that prevent or provide resistance to damage from flooding while allowing floodwaters to enter the structure. Generally, this includes properly anchoring the structure, using flood-resistant materials below the BFE, protection of mechanical and utility equipment, and the use of openings or breakaway walls.

Wind-borne debris regions. Areas within hurricane-prone regions located:

1. Within 1 mile of the coastal mean high water line where the basic wind speed is equal to or greater than 110 mph and in Hawaii.
2. In areas where the basic wind speed is equal to or greater than 120 mph.

Federal funding for mitigation is available on a regular basis for pre-disaster mitigation activities and as federal assistance following a presidential disaster declaration. Table 1 provides a side-by-side comparison of available funding programs administered by FEMA. Although each program is constrained in a number of ways, as explained below, the following sources of federal funding may be available for mitigation projects for critical facilities:

Many other federal and state funding sources may be available to support planning and construction (or upgrades) of some critical facilities, but this manual does not identify or list such sources. Owners, planners, and community leaders are encouraged to contact appropriate state agencies to learn more.

- **Section 406 Public Assistance (PA)** is a post-disaster program established under Section 406 of the Stafford Act—it is jointly administered by FEMA and individual states. As part of the reimbursements made to restore damaged public facilities and certain private non-profit (PNP) facilities, public assistance funds may be made available for cost-effective mitigation measures undertaken as part of the recovery. The amount of Section 406 Mitigation funds made available in any given disaster is not computed by a formula, but is based on a project-by-project evaluation of the feasibility and cost-effectiveness of mitigation measures.

- **Section 404 Hazard Mitigation Grant Program (HMGP)** is a post-disaster program established under Section 404 of the Stafford Act. It offers funding to states, communities, and other eligible

grant recipients to invest in long-term measures that will reduce vulnerability to future natural hazards. The states have a strong role in administering HMGP, with FEMA providing oversight. Contact the State Hazard Mitigation Officer (SHMO) for state-specific information. General guidelines and resources for this program can be found on the FEMA website (www.fema.gov/fima/hmgp).

- **Pre-Disaster Mitigation (PDM)**, established under Section 203 of the Stafford Act, is a nationally competitive grant program designed to assist states and communities to develop mitigation plans and implement mitigation projects. PDM funds are appropriated annually. FEMA convenes national panels to evaluate eligible applications. Applications are submitted by states following the state selection process. Communities should contact the SHMO for state-specific PDM procedures. PDM program guidance and other information can be found on the FEMA website (www.fema.gov/government/grant/pdm/index.shtm).

- **Flood Mitigation Assistance (FMA)** is a grant program funded by the National Flood Insurance Program (NFIP) and focused on buildings that are insured by the NFIP, with particular attention to buildings that have received multiple claim payments. As with the HMGP, FMA is state-administered, and information and assistance is available from the SHMO. General guidelines and resources for this program can be found on the FEMA website (www.fema.gov/government/grant/fma/index.shtm).

As part of the Hurricane Katrina recovery in Mississippi and Louisiana, FEMA initiated a post-disaster Partnering Mitigation Programs initiative to combine funding from Section 406 and Section 404 where possible. Section 406 mainly funds the repair and restoration of storm damage; thus, some building elements may remain exposed to future damage. The initiative fosters a cooperative approach to making decisions to use both PA and HMGP funds to accomplish recovery as well as mitigation of facilities as whole buildings in a seamless fashion.

Table 1: Side-by-Side of FEMA Programs

Item	PA (406)	HMGP (404)	PDM (203)	FMA
Eligible Applicants and Sub-applicants for Project Grants	State and local governments; PNP organizations or institutions that own or operate a PNP facility (as defined in regulation); Indian tribes or authorized tribal organizations and Alaska Native villages or organizations, but not Alaska native corporations with ownership vested in private individuals.	State and local governments; PNP organizations or institutions that own or operate a PNP facility (as defined in regulation); Indian tribes or authorized tribal organizations and Alaska Native villages or organizations, but not Alaska native corporations with ownership vested in private individuals.	State-level agencies; local governments, Indian tribes, authorized Indian tribal organizations, and Alaska Native villages; public colleges and universities; tribal colleges and universities. PNP organizations and private colleges and universities are not eligible sub-applicants, but a relevant state agency or local government may apply on their behalf.	State agencies, NFIP-participating communities (have zoning and building code jurisdiction), and local authorities designated to plan and implement projects.
Eligible Activities	Basic assistance to repair the damaged elements of public facilities and infrastructure. Eligible mitigation activities are only those that protect against direct physical damages to structures, building, and contents.	Eligible activities include structural and nonstructural mitigation measures that are feasible and cost-effective. Projects may count benefits in terms of building and contents damages, displacement costs, loss of function, and casualties and loss of life.	Eligible measures must be cost-effective and designed to reduce injuries, loss of life, and damage and destruction of property, including damage to critical services and facilities under the jurisdiction of the states or local governments.	
Cost Share	75 percent federal 25 percent non-federal	75 percent federal 25 percent non-federal	75 percent federal 25 percent non-federal Small, impoverished communities may be eligible for 90 percent federal share.	75 percent federal 25 percent non-federal

Table 1: Side-by-Side of FEMA Programs (continued)

Item	PA (406)	HMGP (404)	PDM (203)	FMA
Funding Source	The Disaster Relief Fund after a presidential disaster declaration; public assistance must be authorized specifically as part of the disaster declaration.	The Disaster Relief Fund after a presidential disaster declaration; HMGP must be authorized specifically as part of the disaster declaration.	An annual appropriation by Congress.	Annual appropriation by Congress, by transfer of income from the National Flood Insurance Fund.
Planning Requirement	For categories C-G assistance, a state must have an approved mitigation plan.	For disasters declared after 11/1/04, state and local governments must have an approved mitigation plan.	State and local mitigation plans must be adopted and approved by FEMA prior to the beginning of the selection process, as a precondition for receiving project grants.	Recipient must have adopted a mitigation plan approved by FEMA prior to receipt of grant funds.
Application Deadline	Project worksheets usually are due no later than 60 days after a formal briefing presented by FEMA and the state (extensions may be granted in unusual circumstances).	Applications must be submitted to FEMA within 12 months of the disaster declaration date (extensions may be granted in unusual circumstances).	Varies by year; contact state agency for additional information.	Usually late spring; specific date varies by year, contact state agency for additional information.

Public Assistance (Section 406)

Following a presidential disaster declaration, the PA program provides assistance for debris removal, emergency protective measures, and permanent restoration of publicly owned infrastructure. Eligible recipients of this assistance include state and local governments, Indian tribes or authorized tribal organizations, and Alaska Native villages. Certain PNP organizations may also receive assistance. Eligible PNPs include educational, utility, irrigation, emergency, medical, rehabilitation, temporary or permanent cus-

todial care facilities (including those for the aged and disabled), and other PNP facilities that provide essential services of a governmental nature to the general public. Certain PNPs that provide "critical services" may apply directly to FEMA, including PNPs that provide power, sewer, wastewater treatment, communications, emergency medical care, and water (including water provided by an irrigation organization or facility).

As soon as practical after the declaration, the state, assisted by FEMA, conducts Applicants' Briefings for state, local, and PNP officials, to inform them of the assistance available and the application procedure. A request for public assistance must be filed with the state within 30 days after an area is designated eligible for assistance.

Following the Applicants' Briefing, a kickoff meeting is conducted with each eligible grant recipient, where specific damages are discussed, needs assessed, and a plan of action put in place. A combined federal/state/local team proceeds with project formulation—the process of documenting eligible facilities and determining the cost for fixing the identified eligible damages. The team prepares a project worksheet for each project. Eligible projects fall into the following categories:

Category A: Debris removal

Category B: Emergency protective measures

Category C: Roads and bridges

Category D: Water control facilities

Category E: Public buildings and contents

Category F: Public utilities

Category G: Parks, recreational, and other

The facility owner must pay for all costs that are not eligible for Section 406 reimbursement. The non-federal share can be a combination of cash and in-kind resources. Federal funding from other sources cannot be used as matching funds, with the exception of federal funding provided to states under the Community Development Block Grant program from the Department of Housing and Urban Development.

FEMA reviews and approves the project worksheets and obligates the federal share of the costs to the state, which, in turn, disburses the funds to local applicants. The federal share is not less than 75 percent of the eligible project costs (except in extraordinary circumstances, when some costs are eligible for 90 or 100 percent reimbursement).

Damaged buildings that are in a mapped floodplain and insured by NFIP may have access to an insurance payment to help cover the cost of mitigation, provided the damage caused by flooding is determined to be "substantial damage." A building is substantially damaged when the value of the work required to repair it to its pre-damaged condition equals or exceeds 50 percent of the market value of the building. In these cases, increased cost of compliance coverage in NFIP flood insurance policies provides to owners up to $30,000 to bring the building into compliance with floodplain management requirements. This payment may be used as part of the non-federal match of grant-funded mitigation projects designed to address flood hazards.

Section 406 provides FEMA with the authority to fund mitigation measures in conjunction with the repair of damaged facilities involving permanent restorative work (Categories C, D, E, F, and G projects). The mitigation measures must be related to the eligible disaster damages and must directly reduce the potential of future, similar damages to the eligible facility. In providing this discretionary authority, Congress recognized that the post-disaster period offers unique opportunities to prevent the recurrence of similar damage in future disasters. These measures are additional to any other measures undertaken for the purpose of compliance with applicable codes and standards, although such compliance, itself, could be considered a form of mitigation. The following are examples of the types of mitigation required for compliance with building codes that are eligible under Section 406:

- Improved building materials such as impact-resistant windows or flood-resistant materials
- Anchoring of rooftop equipment
- Improved installation methods or techniques
- New or higher elevation (for flood-prone facilities)

Owners of eligible public facilities often wish to repair and restore a damaged facility in ways that exceed the current building code requirements, in order to further reduce or eliminate future damage. Where it can be demonstrated that doing so is cost-effective, the added costs of such actions are eligible under FEMA Policy 9526.1 governing the implementation of the Section 406 program. Unfortunately, this approach rarely meets all the identified mitigation needs, because the funding can be used only for the damaged elements—mitigation of vulnerable but undamaged elements is not allowed. For example, a police station with some damaged windows can use Section 406 funds to replace the dam-

aged windows with impact-resistant windows, but cannot replace undamaged windows. To address this limitation, facility owners should seek other sources of mitigation funding, such as the Hazard Mitigation Grant Program described below.

All mitigation activities funded with Section 406 funds must be cost-effective. To facilitate recovery, FEMA can apply one of several tests to determine cost-effectiveness:

- Measures may cost to up to 15 percent of the total eligible cost of the eligible repair work on a particular project.

- Certain pre-approved mitigation measures identified in FEMA policies will be determined to be cost-effective, as long as the cost of the mitigation measure does not exceed 100 percent of the eligible cost of the repair project.

- For measures that do not meet the first or second test, the project applicant must demonstrate cost-effectiveness using an acceptable benefit/cost analysis. FEMA developed benefit/cost models that are specific to hurricanes, coastal flooding, riverine flooding, earthquakes, and tornadoes.

Hazard Mitigation Grant Program (Section 404)

Section 404, HMGP, is FEMA's primary hazard mitigation program to help implement long-term mitigation measures following major disaster declarations. Under HMGP, each state manages its own program, and eligible participants include state and local governments, tribes, and PNP organizations. The program funds up to 75 percent of the costs of individual FEMA-approved projects. The total amount of funding made available after specific disasters depends on several variables.

HMGP funds have been used for many types of projects, including the following:

- **Wind-resistant Retrofits.** Existing facilities that were built before current code requirements may be eligible to be retrofitted to resist high winds. For any given building, applicable measures are identified by conducting an evaluation of the building.

Critical facilities have been retrofitted with shutters for windows and doors, and anchoring of architectural features and rooftop equipment.

- **Floodproof Retrofits.** Depending upon the nature of the flood risk, elevating an existing flood-prone structure or incorporating dry floodproofing techniques may be the most practical approach to meet NFIP requirements and those administered by states and local governments.

- **Code Upgrades.** Certain measures intended to provide a level of protection that exceeds the minimum requirements of the applicable building code may be eligible. This approach to mitigation is especially applicable for critical facilities that serve vital post-disaster functions.

- **Relocation.** In some cases, it may be viable to physically move a structure to a new location outside of high-risk flood hazard areas. Relocated buildings must be placed on a site located outside of the 100-year floodplain and any regulatory erosion zones, in conformance with all applicable state or local land use regulations.

- **Acquisition and Demolition.** The community purchases the flood-damaged property and demolishes the structure. The acquired property must be maintained as open space in perpetuity.

Pre-Disaster Mitigation (Section 203)

PDM provides funds to state and local governments for hazard mitigation planning and the implementation of cost-effective mitigation projects unrelated to a specific disaster. PDM is administered directly by FEMA. Eligible applicants include states, tribes, territories, and local governments (PNPs can apply if sponsored by an eligible applicant). Grants are awarded on a nationwide competitive basis and without a formula-based allocation.

Similar to HMGP, PDM has funded numerous projects that improve wind- and flood-resistance of critical facilities such as emergency operations centers, hospitals, fire stations, police sta-

tions, and wastewater treatment plants. Specific measures include strengthening exterior walls, anchoring rooftop equipment, installing shutters, and elevation of facilities or equipment above the 100-year flood elevation. Other types of projects may be eligible, provided they meet the program requirements and are cost-effective. Each year, FEMA issues PDM program guidance with specific criteria.

Cost Share for Small, Impoverished Communities

PDM grants awarded to small, impoverished communities may receive a federal cost share of up to 90 percent of eligible costs. To qualify, a community must meet several eligibility criteria related to population, average annual income, unemployment rate, and other conditions.

Flood Mitigation Assistance Program

FMA is designed to fund mitigation projects that are cost-effective and in the best interest of NFIP. Funds are provided annually from income collected from flood insurance policyholders. Each state manages its own program. Eligible grant recipients must have a FEMA-approved mitigation plan. Eligible recipients include communities that have land use authority and participate in the NFIP, certain other local authorities, and state agencies. The program funds up to 75 percent of certain eligible costs for measures that reduce or eliminate flood damage to buildings that are insured by the NFIP, including buildings and facilities owned by public agencies and PNP entities.

Eligible activities under the FMA program include acquisition, increased elevation, relocation, demolition, floodproofing, and activities that bring nonresidential structures into compliance with minimum NFIP requirements and state and local codes, and minor physical activities such as drainage improvements. As with HMGP, if an NFIP-insured building is damaged by a flood and found to be eligible for the increased cost of compliance claim payment to bring the building into compliance, the payment can be used as part of the required non-federal match for FMA grants.

Guidelines for Design of Structures for Vertical Evacuation from Tsunamis

Second Edition

FEMA P-646 / April 2012

FEMA

Guidelines for Design of Structures for Vertical Evacuation from Tsunamis

Second Edition

Prepared by
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Cover Images: Photographs showing various examples of potential vertical evacuation structures. Clockwise from top left: (1) designated vertical evacuation building in Kesennuma Port, Japan, where numerous residents found safe refuge at the roof level during the 2011 Tohoku tsunami; (2) sports complex where large numbers of people could gain easy access to elevated concourse and seating levels; (3) multi-level cast-in-place reinforced concrete parking garage in Biloxi, Mississippi, that survived storm surge inundation during Hurricane Katrina; and (4) earthen mound with ramp access to a safe elevation. Photographs provided courtesy of Ian Robertson, University of Hawaii at Manoa and Magnusson Klemencic Associates, Seattle, Washington.

Foreword

This publication was equally funded by the National Oceanic and Atmospheric Administration (NOAA), which leads the National Tsunami Hazard Mitigation Program (NTHMP) and by the Federal Emergency Management Agency (FEMA), which is responsible for the implementation portion of the National Earthquake Hazard Reduction Program (NEHRP).

FEMA initiated this project in September 2004 with a contract to the Applied Technology Council. The project was undertaken to address the need for guidance on how to build a structure that would be capable of resisting the extreme forces of both a tsunami and an earthquake. This question was driven by the fact that there are many communities along our nation's west coast that are located on narrow spits of land and are vulnerable to a tsunami triggered by an earthquake on the Cascadia subduction zone, which could potentially generate a tsunami of 20 feet in elevation or more within 20 minutes. Given their location, it would be impossible to evacuate these communities in time, which could result in a significant loss of life. Many coastal communities subject to tsunami located in other parts of the country also have the same potential problem. In these cases, the only feasible alternative is vertical evacuation, using specially design, constructed and designated structures built to resist both tsunami and earthquake loads.

The significance of this issue came into sharp relief with the December 26, 2004 Sumatra earthquake, the Indian Ocean Tsunami, and the March 11, 2011 Tohoku Japan Tsunami. While these events resulted in a tremendous loss of life, this would have been even worse had not many people been able to take shelter in multi-story reinforced concrete buildings or been able to get to high ground sites after the tsunami warning was delivered. Without realizing it, these survivors were demonstrating the concept of vertical evacuation from a tsunami.

This publication presents the following information:

- General information on the tsunami hazard and its history;
- Guidance on determining the tsunami hazard, including the need for tsunami depth and velocity on a site-specific basis;
- Different options for vertical evacuation from tsunamis;

- Determining tsunami and earthquake loads and structural design criteria necessary to address them; and,
- Structural design concepts and other considerations.

This is the second edition of FEMA P-646, originally published in June 2008. In this second edition revisions were made throughout the document, but particularly to the following items:

- Inclusion of observations and lessons learned from the March 11, 2011 Tohoku tsunami;
- Revision and enhancement of the debris impact expression to remove over-conservatism in the prior edition; and
- Updating of all reference documents to the most current version.

FEMA also issued a companion document in 2009, FEMA P-646A, *Vertical Evacuation from Tsunamis: A Guide for Community Officials*, that presents information on how the use of this design guidance can be encouraged and adopted at the State and local levels.

FEMA is grateful to the original Project Management Committee of Steve Baldridge, John Hooper, Ian Robertson, Tim Walsh, and Harry Yeh. We are also grateful to the Project Review Committee, the members of which are listed at the end of the document, and to the staff of the Applied Technology Council. The updates included in this second edition were made thanks to Gary Chock, John Hooper, Ian Robertson, Tim Walsh, and Harry Yeh. Their hard work has provided this nation with a first document of its kind, a manual on how citizens may for the first time be able to survive a tsunami, one of the most terrifying natural hazards known.

– Federal Emergency Management Agency

Preface

In September 2004 the Applied Technology Council (ATC) was awarded a "Seismic and Multi-Hazard Technical Guidance Development and Support" contract (HSFEHQ-04-D-0641) by the Federal Emergency Management Agency (FEMA) to conduct a variety of tasks, including one entitled "Development of Design and Construction Guidance for Special Facilities for Vertical Evacuation from Tsunamis," designated the ATC-64 Project. This project included a review of available international research and state-of-the-practice techniques regarding quantification of tsunami hazard and tsunami force effects.

In 2008, this work resulted in the publication of the FEMA P-646 report, *Guidelines for Design of Structures for Vertical Evacuation from Tsunamis*, providing technical guidance and approaches for tsunami-resistant design, identification of relevant tsunami loads and applicable design criteria, development of methods to calculate tsunami loading, and identification of architectural and structural system attributes suitable for use in vertical evacuation facilities. In 2009, the companion FEMA P-646A report, *Vertical Evacuation from Tsunamis: A Guide for Community Officials*, was released providing information on how to use vertical evacuation design guidance at the state and local government levels.

Following its publication in 2008, FEMA P-646 was used in conceptual design studies as part of tsunami evacuation planning in Cannon Beach, Oregon. It was also used in ongoing research related to the development of Performance-Based Tsunami Engineering conducted at the University of Hawaii at Manoa, under the George E. Brown, Jr. Network for Earthquake Engineering Simulation (NEES). Based on findings from these activities, FEMA initiated a follow-up contract, designated the ATC-79 Project, to review the design guidance contained in FEMA P-646, and to consider updates, if needed, based on this new information.

As a result of this review, selected revisions were deemed necessary. Technical updates contained in this Second Edition of the FEMA P-646 report are related to: (1) inclusion of observations and lessons learned from the March 11, 2011 Tohoku tsunami; (2) revision of the debris impact expression to remove over-conservatism deemed to be present in the prior edition; (3) additional explanation of the definition of tsunami elevation as it

relates to runup elevation used in tsunami force equations; and (4) update of reference documents to the most current version.

ATC is indebted to the members of the ATC-79 Project Team responsible for the technical development of this Second Edition of the FEMA P-646 report. The Project Management Committee, including Ian Robertson (Project Technical Director), Gary Chock, John Hooper, Tim Walsh, and Harry Yeh, reviewed new technical information relative to guidance contained in the original report, and decided on the necessary updates.

ATC remains indebted to the members of the ATC-64 Project Team who participated in the development of the original FEMA P-646 report. The Project Management Committee, consisting of Steven Baldridge (Project Technical Director), Frank Gonzalez, John Hooper, Ian Robertson, Tim Walsh, and Harry Yeh, were responsible for the development of the technical criteria, design guidance, and related recommendations. Technical review and comment at critical developmental stages were provided by the Project Review Panel, consisting of Christopher Jones (Chair and ATC Board Representative), John Aho, George Crawford, Richard Eisner, Lesley Ewing, Michael Hornick, Chris Jonientz-Trisler, Mark Levitan, George Priest, Charles Roeder, and Jay Wilson. The affiliations of all individuals who participated in the development of the original and second edition reports are provided in the list of Project Participants.

ATC also gratefully acknowledges the input and guidance provided by Michael Mahoney (FEMA Project Officer), Robert Hanson (FEMA Technical Monitor), William Holmes (ATC Project Technical Monitor), William Coulbourne for ATC project management, and Peter N. Mork for ATC report production services.

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Chapter 1
Introduction

1.1 Objectives and Scope

Tsunamis are rare events often accompanied by advance warning. As such, strategies for mitigating tsunami risk have generally involved evacuation to areas of naturally occurring high ground outside of the tsunami inundation zone. Most efforts to date have focused on the development of more effective warning systems, improved inundation maps, and greater tsunami awareness to improve evacuation efficiency.

A Vertical Evacuation Refuge from Tsunamis is a building or earthen mound that has sufficient height to elevate evacuees above the level of tsunami inundation, and is designed and constructed with the strength and resiliency needed to resist the effects of tsunami waves.

In some locations, high ground may not exist, or tsunamis triggered by local events may not allow sufficient warning time for communities to evacuate low-lying areas. Where horizontal evacuation out of the tsunami inundation zone is neither possible nor practical, a potential solution is vertical evacuation into the upper levels of structures designed and detailed to resist the effects of a tsunami.

The focus of this document is on structures intended to provide protection during a short-term high-risk tsunami event. Such facilities are generally termed refuges. A *vertical evacuation refuge from tsunamis* is a building or earthen mound that has sufficient height to elevate evacuees above the level of tsunami inundation, and is designed and constructed with the strength and resiliency needed to resist the effects of tsunami waves.

This document is a resource for engineers, architects, state and local government officials, building officials, community planners, and building owners who are considering the construction and operation of tsunami-resistant structures that are intended to be a safe haven for evacuees during a tsunami event. It provides guidance on the design and construction of structures that could be used as a refuge for vertical evacuation above rising waters associated with tsunami inundation, and includes specific recommendations on loading, configuration, location, operation, and maintenance of such facilities. It is intended for use in areas of the United States that are exposed to tsunami hazard, but that should not preclude the use of this guidance for facilities located in other areas exposed to similar hazards.

1.2 Deciding to Construct a Vertical Evacuation Structure

Many factors influence the decision to construct a vertical evacuation structure, including:

- the likelihood of a region being affected by a tsunami event,
- the potential consequences of a tsunami event (e.g., damage, injury, and loss of life),
- the elements of a local emergency response plan, including available evacuation alternatives,
- the planned and potential uses for a refuge facility, and
- the cost of constructing a tsunami-resistant structure.

1.2.1 Tsunami Hazard versus Risk

Tsunami Hazard is a measure of the potential for a tsunami to occur at a given site.

Tsunami Risk is a measure of the consequences given the occurrence of a tsunami, which can be characterized in terms of damage, loss of function, injury and loss of life.

Hazard is related to the potential for an event to occur, while risk is related to consequences, given the occurrence of an event. Tsunami hazard is a measure of the potential for a tsunami to occur at a given site. It is also a measure of the potential magnitude of site-specific tsunami effects, including extent of inundation, height of runup, flow depth, and velocity of flow. Tsunami risk is a measure of the consequences given the occurrence of a tsunami, which can be characterized in terms of damage, loss of function, injury and loss of life. Risk depends on many factors including vulnerability and population density.

Similar to other hazards (e.g., earthquake and wind) structural design criteria for tsunami effects are based on relative tsunami hazard. The decision to build a vertical evacuation structure, however, may ultimately be based on real or perceived risk to a local population as a result of exposure to tsunami hazard.

1.2.2 Decision-Making and Design Process

A flowchart outlining the decision-making and design process for vertical evacuation structures is shown in Figure 1-1.

Given a known or perceived tsunami threat in a region, the first step is to determine the severity of the tsunami hazard. This involves identification of potential tsunami-genic sources and accumulation of recorded data on tsunami occurrence and runup. Chapter 3 provides guidance on the assessment of tsunami hazard, which can include a probabilistic assessment considering all possible tsunami sources, or a deterministic assessment considering the maximum tsunami that can reasonably be expected to affect a site. Once potential tsunami sources are identified, and the level of tsunami

hazard is known, site-specific information on the extent of inundation, height of runup, flow depth, and velocity of flow is needed. Some of this information may be obtained from available tsunami inundation maps, where they exist; however site-specific tsunami inundation studies should be performed to obtain reliable estimates of tsunami flow characteristics at the site of the proposed vertical evacuation structure.

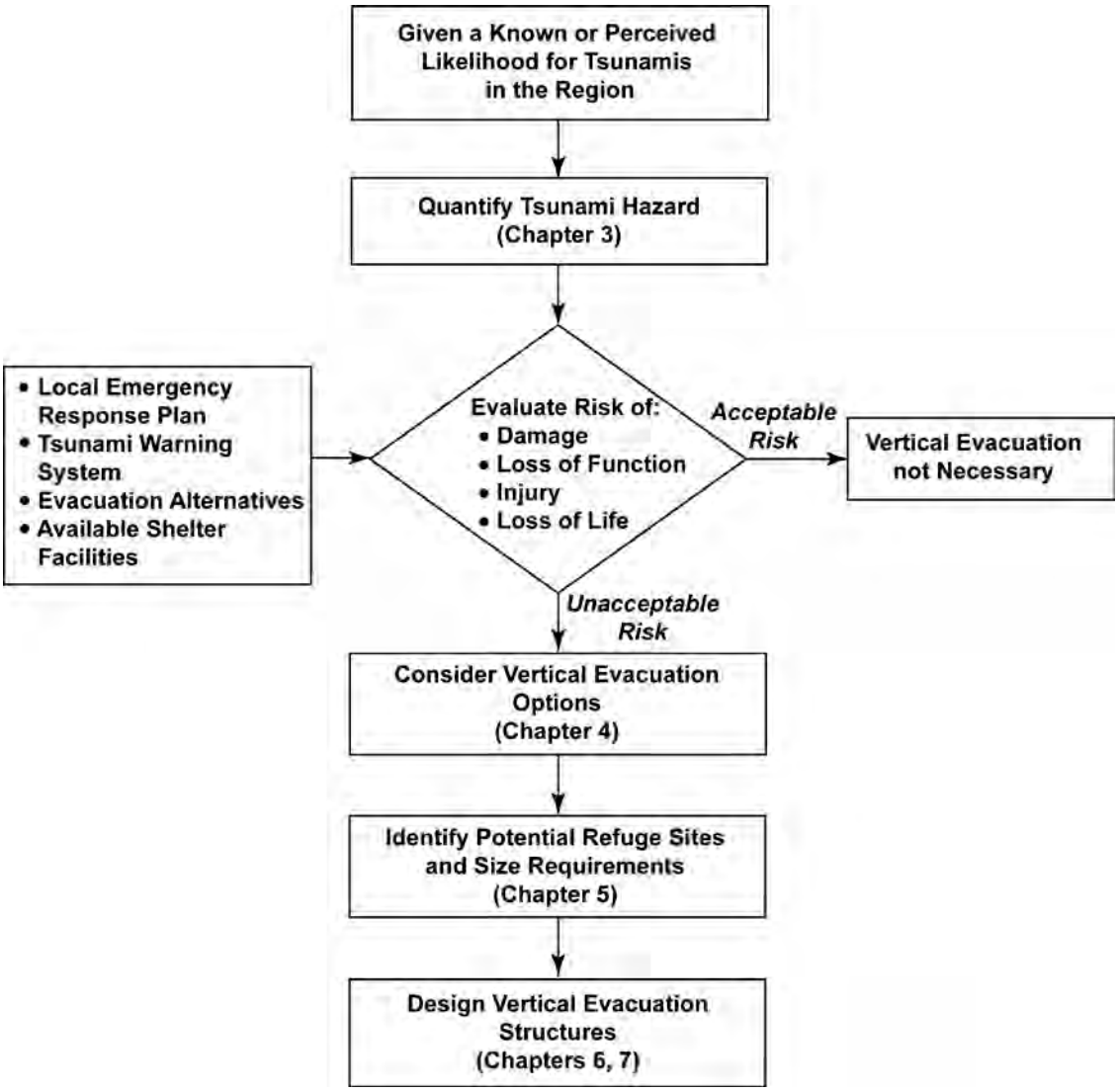


Figure 1-1 Decision-making and design process for vertical evacuation structures.

Given the tsunami hazard and extent of inundation, the potential risk of damage, injury, and loss of life in the region must then be evaluated. Explicit evaluation of tsunami risk is beyond the scope of this document, and will depend on a number of different factors including the presence of a tsunami warning system, existence of a local emergency response plan, availability of

various evacuation alternatives, vulnerability of the existing building stock, and locations of existing short- and long-term shelter facilities. The feasibility of evacuation to existing areas of refuge, as well as the tsunami-resistance of these areas, must be considered. Vertical evacuation structures will likely be most useful when there is not enough time between the tsunami warning and tsunami inundation to allow a community to evacuate out of the inundation zone or to existing areas of high ground. In most cases this will be in communities at risk for near-source-generated tsunamis.

Where the risk to a coastal community is deemed to be unacceptably high, vertical evacuation can be a possible solution for mitigating tsunami risk. Chapter 4 outlines a number of potentially viable options for design and construction of vertical evacuation structures.

Implementation of vertical evacuation requires a distribution of structures throughout the community that are suitable for providing refuge from the effects of tsunami inundation, and that are appropriately sized for the population. Chapter 5 provides guidance on locating and sizing vertical evacuation structures.

Once the decision to utilize vertical evacuation is made, structures must be designed and constructed to be tsunami-resistant. Loading and other criteria for the design of vertical evacuation structures are provided in Chapters 6 and 7. The 2012 *International Building Code*, Appendix M, may be adopted by local jurisdictions that have a tsunami hazard and that regulate the design and construction of structures placed in high-risk or high-hazard areas.

1.3 Limitations

This document is a compilation of the best information available at the time of publication. It provides guidance for design and construction of vertical evacuation structures that is currently not available in other design guides, building codes, or standards. It is not intended to supersede or replace current codes and standards, but rather to supplement them with guidance where none is otherwise provided. It is intended to provide specific recommendations and design criteria that are unique to tsunami loading conditions for vertical evacuation structures, once the decision has been made to build such a structure. It is not intended to mandate or imply that all structures in tsunami hazard areas should be made tsunami-resistant using these criteria. Such a decision would be cost-prohibitive, especially for light-frame residential structures.

Vertical evacuation structures designed in accordance with the guidance presented in this document would be expected to provide safe refuge under

the assumed design conditions. For these structures, multiple design assumptions are required, including the intensity of a local earthquake that could threaten the structure prior to a tsunami, the flow depths and velocities of the design tsunami at the site, and the type of waterborne debris that may be characteristic at the site. Maximum loading must therefore be considered uncertain, and conservative assumptions should be made, particularly since these structures are expected to provide security and safety to the public.

Large damaging tsunamis are rare events, and existing knowledge is based on limited historic information. Coastal inundation patterns are based on complex combinations of many parameters, and are highly uncertain. Proportioning a structure for a design tsunami event does not necessarily mean the structure will be able to resist every possible tsunami event. Selection of the design tsunami is therefore based on the tsunami hazard in a region, the risk tolerance of a local community, and economic considerations.

Critical to the design of a vertical evacuation structure is the height of the refuge area above the anticipated tsunami flow depth. Even if the structure survives inundation, overtopping of the refuge area will result in unacceptable loss of life of those who sought refuge in the designated evacuation structure. This is clearly unacceptable performance of a vertical evacuation refuge and every effort must be taken to avoid this outcome.

1.4 Organization

This document provides guidance on siting concepts, performance objectives, design loads, design concepts, and emergency management issues that should be considered in locating, designing, and operating vertical evacuation structures as a refuge from tsunamis. Examples are presented that illustrate how the criteria are used. Information contained in this document is organized as follows:

Chapter 1 defines the scope and limitations for the guidance contained in this document. Chapter 2 provides background information on tsunami effects and their potential impacts on buildings in coastal communities. Chapters 3 through 7 provide design guidance on characterization of tsunami hazard, choosing between various options for vertical evacuation structures, locating and sizing vertical evacuation structures, estimation of tsunami load effects, structural design criteria, design concepts, and other considerations.

Appendices A through E provide supplemental information, including examples of vertical evacuation structures from Japan, example tsunami load calculations, a community design example, development of impact load

equations, and background on maximum flow velocity and momentum flux in the tsunami runup zone.

A Glossary defining terms used throughout this document, and a list of References identifying resources for additional information, are also provided.

Chapter 2
Background

2.1 General

Tsunami is a Japanese word meaning "harbor" (tsu) and "wave" (nami). The term was created by fishermen who returned to port to find the area surrounding the harbor devastated. It is a naturally occurring series of waves that can result when there is a rapid, large-scale disturbance of a body of water. The most common triggering events are earthquakes below or near the ocean floor, but a tsunami can also be created by volcanic activity, landslides, undersea slumps, and impacts of extra-terrestrial objects. The waves created by this disturbance propagate away from the source. In deep water, the waves are gentle sea-surface slopes that can be unnoticeable. As the waves approach the shallower waters of the coast, however, the velocity decreases while the height increases. Upon reaching the shoreline the waves can have hazardous height and force, penetrating inland, damaging structures, and flooding normally dry areas.

A Tsunami is a naturally occurring series of ocean waves resulting from a rapid, large-scale disturbance in a body of water, caused by earthquakes, landslides, volcanic eruptions, and meteorite impacts.

In this document, tsunamis are categorized by the location of the triggering event and the time it takes the waves to reach a given site. A far-source-generated tsunami is one that originates from a source that is far away from the site of interest, and takes 2 hours or longer after the triggering event to arrive. A near-source-generated tsunami is one that originates from a source that is close to the site of interest, and can arrive within 30 minutes. Sites experiencing near-source-generated tsunamis will generally feel the effects of the triggering event (e.g., shaking caused by a near-source earthquake). A mid-source-generated tsunami is one in which the source is somewhat close to the site of interest, but not close enough for the effects of the triggering event to be felt at the site. Mid-source-generated tsunamis would be expected to arrive between 30 minutes and 2 hours after the triggering event.

A far-source-generated tsunami is one that originates from a source that is far away from the site of interest, and takes 2 hours or longer after the triggering event to arrive.

A near-source-generated tsunami is one that originates from a source that is close to the site of interest, and arrives within 30 minutes. The site of interest might also experience the effects of the triggering event.

A mid-source-generated tsunami is one in which the source is somewhat close to the site of interest, and would be expected to arrive between 30 minutes and 2 hours after the triggering event.

2.1.1 Historic Tsunami Activity

The combination of a great ocean seismic event with the right bathymetry can have devastating results, as was brought to the world's attention by the Indian Ocean Tsunami of December 26, 2004 and more recently the Tohoku Japan Tsunami of March 11, 2011. The Indian Ocean Tsunami was created by a magnitude-9.3 underwater earthquake and devastated coastal areas around the northern Indian Ocean. The tsunami took anywhere from 15

minutes to 7 hours to hit the various coastlines it affected. It is estimated that the tsunami took over 220,000 lives and displaced over 1.5 million people. The Tohoku Japan Tsunami was generated by the magnitude 9.0 Great East Japan Earthquake and led to inundation heights along the coast of the main Japanese island of Honshu that exceeded all historical records for that region. Breakwater and seawall defensive systems were overtopped or destroyed in almost all communities along the Tohoku coastline, leading to over 19,000 missing or dead, and extensive damage to ports, buildings, bridges and other coastal infrastructure.

Wave propagation times from far-source-generated tsunamis can allow for advance warning to distant coastal communities. Near-source-generated tsunamis, however, can strike suddenly and with very little warning. The 1993 tsunami that hit Okushiri, Hokkaido, Japan, for example, reached the shoreline within 5 minutes after the earthquake, and resulted in 202 fatalities as victims were trapped by debris from the earthquake and unable to flee toward higher ground and more secure places.

Although considered rare events, tsunamis occur on a regular basis around the world. Each year, on average, there are 20 tsunami-genic earthquake events, with five of these large enough to generate tsunami waves capable of causing damage and loss of life. In the period between 1990 and 1999 there were 82 tsunamis reported, 10 of which resulted in more than 4,000 fatalities. With the trend toward increased habitation of coastal areas, more populations will be exposed to tsunami hazard.

Relative tsunami hazard can be characterized by the distribution and frequency of recorded runups. Table 2-1 provides a qualitative assessment of tsunami hazard for regions of the United States that are threatened by tsunamis, as it has been characterized by the National Oceanic & Atmospheric Administration (NOAA) using the last 200 years of data on recorded runups.

Alaska is considered to have the highest potential for tsunami-generating events in the United States. Earthquakes along the Alaska-Aleutian subduction zone, particularly in the vicinity of the Alaskan Peninsula, the Aleutian Islands, and the Gulf of Alaska have the capability of generating tsunamis that affect both local and distant sites. The 1964 earthquake in Prince William Sound resulted in 122 fatalities, including 12 in California and 4 in Oregon. In 1994 a landslide-generated tsunami in Skagway Harbor resulted in one death and $21 million in property damage.

Table 2-1 Qualitative Tsunami Hazard Assessment for U.S. Locations (Dunbar, et. al., 2008)

Region	*Hazard Based on Recorded Runups*	*Hazard Based on Frequency of Runups*
Atlantic Coast	Very low to low	Very low
Gulf Coast	None to very low	None to very low
Caribbean	High	High
West Coast	High	High
Alaska	Very high or severe	Very high
Hawaii	Very high or severe	Very high
Western Pacific	Moderate	High

The Cascadia subduction zone along the Pacific Northwest coast poses a threat from northern California to British Columbia, Canada. An earthquake along the southern portion of the Cascadia subduction zone could create tsunami waves that would hit the coasts of Humboldt and Del Norte counties in California and Curry County in Oregon within a few minutes of the earthquake. Areas further north, along the Oregon and Washington coasts, could see tsunami waves within 20 to 40 minutes after a large earthquake.

Communities along the entire U.S. Pacific coastline are at risk for far-source-generated (trans-Pacific) tsunamis and locally triggered tsunamis. In southern California there is evidence that movement from local offshore strike-slip earthquakes and submarine landslides have generated tsunamis affecting areas extending from Santa Barbara to San Diego. The largest of these occurred in 1930, when a magnitude-5.2 earthquake reportedly generated a 20-foot-high wave in Santa Monica, California (California Geological Survey, 2006).

Hawaii, located in the middle of the Pacific Ocean, has experienced both far-source-generated tsunamis and locally triggered tsunamis (Pararas-Carayannis, 1968). The far-source 2011 Tohoku Japan Tsunami resulted in inundation of a number of coastal communities in Hawaii, causing structural and non-structural damage to homes, hotels and small boat harbors. Total damages were estimated at $40 million. The most recent near-source damaging tsunami in Hawaii occurred in 1975, the result of a magnitude-7.2 earthquake off the southeast coast of the island of Hawaii. This earthquake resulted in tsunami wave heights more than 20 feet and, in one area, more than 40 feet. Two deaths and more than $1 million in property damage were attributed to this local Hawaiian tsunami (Pararas-Carayannis, 1976).

Although the Atlantic and Gulf Coast regions of the United States are perceived to be at less risk, there are examples of deadly tsunamis that have occurred in the Atlantic Ocean. Since 1600, more than 40 tsunamis and tsunami-like waves have been cataloged in the eastern United States. In 1929, a tsunami generated in the Grand Banks region of Canada hit Nova Scotia, killing 51 people (Lockridge et al., 2002).

Puerto Rico and the U.S. Virgin Islands are at risk from earthquakes and underwater landslides that could occur in the Puerto Rico Trench subduction zone. Since 1530, more than 50 tsunamis of varying intensity have occurred in the Caribbean. In 1918, an earthquake in this zone generated a tsunami that caused an estimated 40 deaths in Puerto Rico. In 1867, an earthquake-generated tsunami caused damage and 12 deaths on the islands of Saint Thomas and Saint Croix. In 1692 a tsunami generated by massive landslides in the Puerto Rican Trench reached the coast of Jamaica, causing an estimated 2,000 deaths (Lander, 1999).

2.1.2 Behaviors and Characteristics of Tsunamis

Tsunami wave periods can range from a few minutes to over 1 hour, resulting in an increased potential for reflection, amplification, or resonance within coastal features.

Information from historic tsunami events indicates that tsunami behaviors and characteristics are quite distinct from other coastal hazards, and cannot be inferred from common knowledge or intuition. The primary reason for this distinction is the unique timescale associated with tsunami phenomena. Unlike typical wind-generated water waves with periods between 5 and 20 sec, tsunamis can have wave periods ranging from a few minutes to over 1 hour (FEMA, 2005). This timescale is also important because of the potential for wave reflection, amplification, or resonance within coastal features. Table 2-2 compares various coastal hazard phenomena.

Table 2-2 Comparison of Relative Time and Loading Scales for Various Coastal Hazard Phenomena

Coastal Hazard Phenomenon	*Time scale (Duration of Loading)*	*Loading Scale (Height of Water)*	*Typical Warning Time*
Wind-generated waves	Tens of seconds	1 to 2 meters typical	Days
Tsunami runup	Tens of minutes to an hour	1 to 10 meters	Several minutes to hours
Hurricane storm surge	Several hours	1 to 10 meters	Several hours to a few days
Earthquake shaking	Seconds	N/A	Seconds to none

There is significant uncertainty in the prediction of hydrodynamic characteristics of tsunamis because they are highly influenced by the tsunami

waveform and the surrounding topography and bathymetry. Although there are exceptions, previous research and field surveys indicate that tsunamis have the following general characteristics:

- The magnitude of the triggering event determines the period of the resulting waves, and generally (but not always) the tsunami magnitude and damage potential (FEMA, 2005).
- A tsunami can propagate more than several thousand kilometers without losing energy.
- Tsunami energy propagation has strong directivity. The majority of its energy will be emitted in a direction normal to the major axis of the tsunami source. The more elongated the tsunami source, the stronger the directivity (Okal, 2003; Carrier and Yeh, 2005). Direction of approach can affect tsunami characteristics at the shoreline, because of the sheltering or amplification effects of other land masses and offshore bathymetry (FEMA, 2005). A numerical example for the 2004 Indian Ocean Tsunami is shown in Figure 2-1.

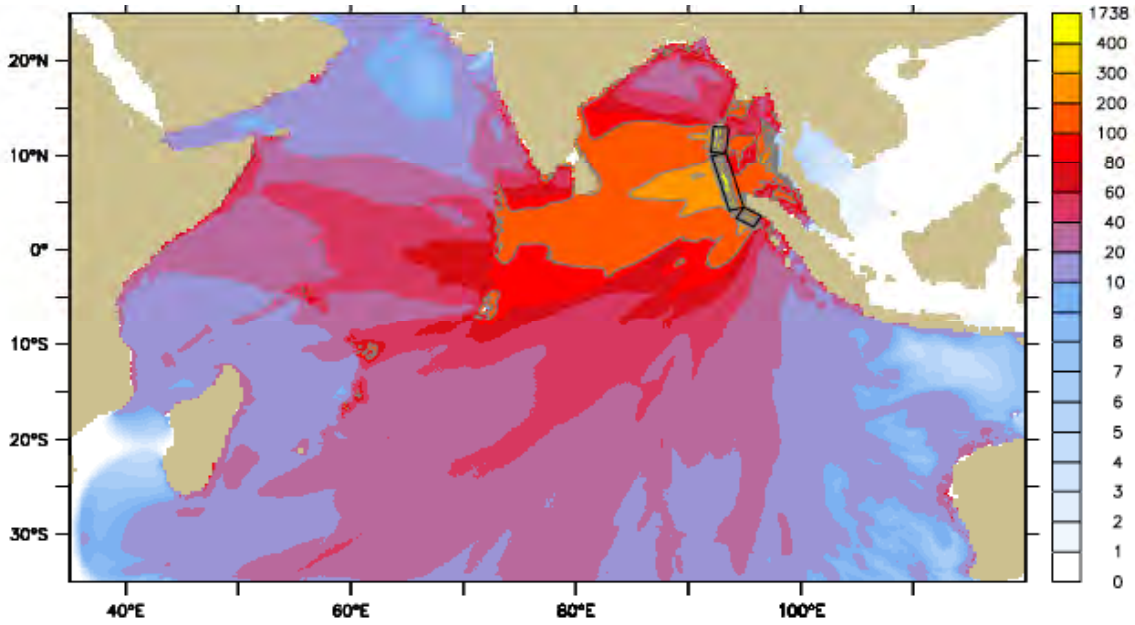


Figure 2-1 Maximum computed tsunami amplitudes (in centimeters) in the Indian Ocean (Titov, NOAA Center for Tsunami Research, http://nctr.pmel.noaa.gov/indo_1204.html)

- At the source, a tsunami waveform contains a wide range of wave components, from short to long wavelengths. Long wave components propagate faster than short wave components; therefore, a transoceanic tsunami is usually characterized by long-period waves (several to tens of

minutes). Shorter wave components are left behind and attenuated by radiation and dispersion.

- For a locally-generated tsunami, the first leading wave is often a receding water level followed by an advancing positive heave (an elevation wave). This may not be the case if the coastal ground subsides by co-seismic displacement. For far-source-generated tsunamis, the leading wave is often an elevation wave. This trend may be related to the pattern of sea floor displacement resulting from a subduction-type earthquake, shown schematically in Figure 2-2. Figure 2-3 shows a leading depression wave measured at a tide gage station in Thailand during the 2004 Indian Ocean Tsunami, in contrast with a leading elevation wave measured at the southern end of India.

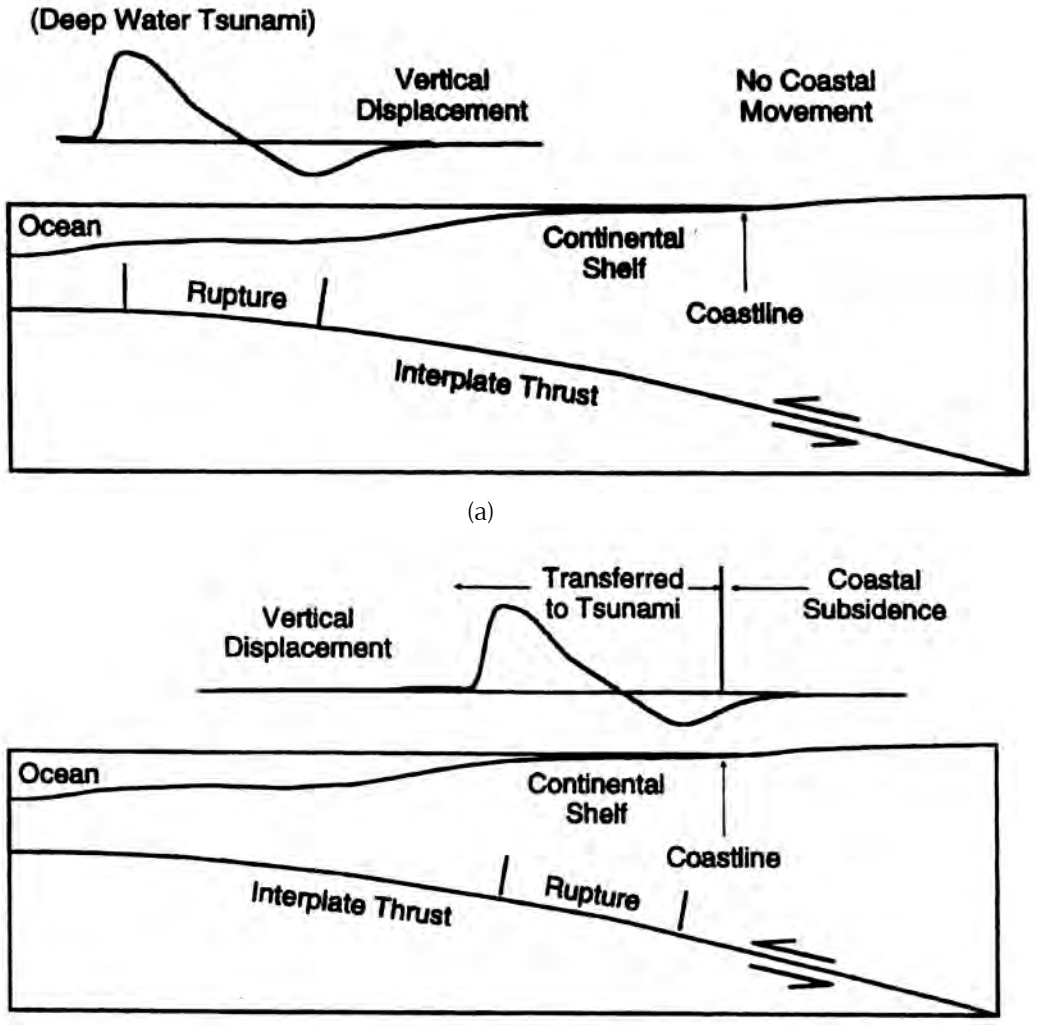
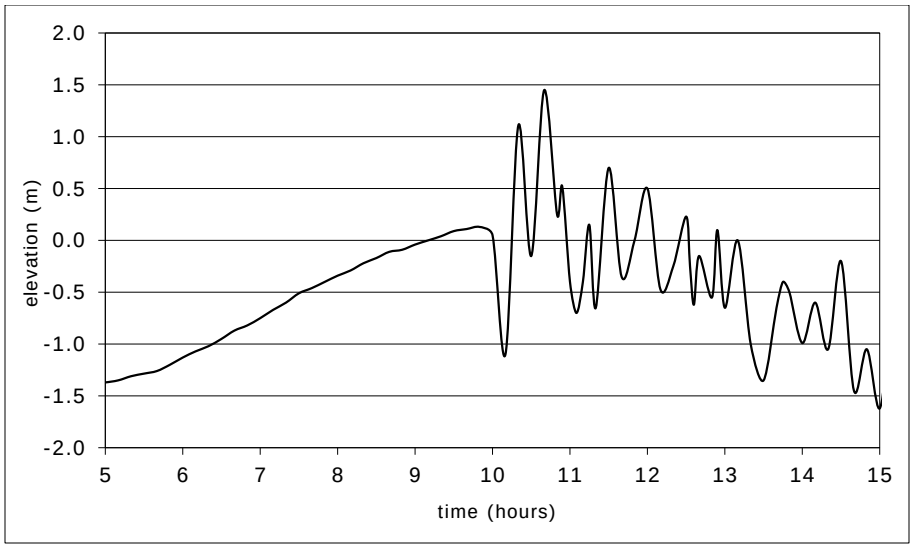
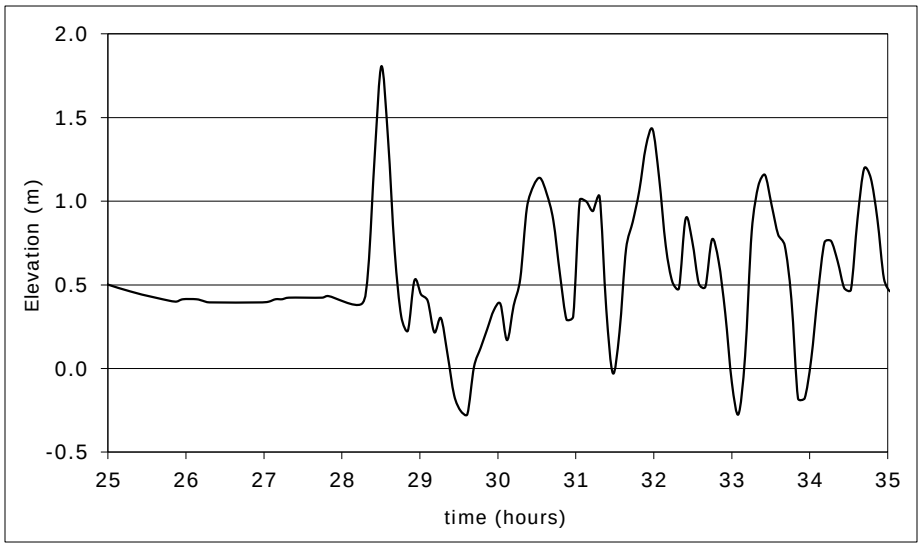


Figure 2-2 Schematic diagrams of the vertical displacement resulting from subduction-type fault dislocation: (a) rupture zone located far offshore; and (b) rupture zone adjacent to coastline with coastal subsidence (Geist, 1999).



(a)



(b)

Figure 2-3 Tide gage records (in meters) for the 2004 Indian Ocean tsunami at: (a) Ta Phao Noi, Thailand, showing the leading depression wave; and (b) Tuticorin, India, showing the leading elevation wave.

- Tsunamis are highly reflective at the shore, and capable of sustaining their motion for several hours without dissipating energy. Typically several tsunami waves attack a coastal area, and the first wave is not necessarily the largest. Sensitive instrumentation can detect tsunami activity for several days.
- Tsunami runup height varies significantly in neighboring areas. The configuration of the continental shelf and shoreline affect tsunami impacts at the shoreline through wave reflection, refraction, and shoaling. Variations in offshore bathymetry and shoreline irregularities

Tsunami runup heights vary significantly in neighboring areas due to variations in offshore bathymetry that can increase or decrease local tsunami impacts.

can focus or disperse tsunami wave energy along certain shoreline reaches, increasing or decreasing tsunami impacts (FEMA, 2005). Figure 2-4 shows significant variation in runup heights measured along the northwest coastline of Okushiri Island.

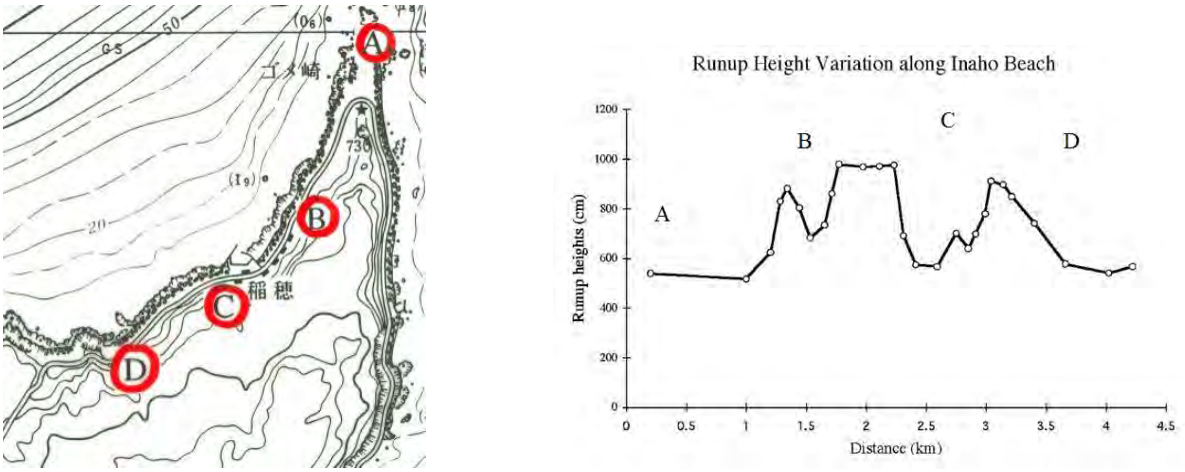


Figure 2-4 Measured runup heights of the 1993 Okushiri tsunami along Inaho Coast, demonstrating that runup height varies significantly between neighboring areas.

- The majority of eyewitness accounts and visual records (videos and photographs) indicate that an incident tsunami will break offshore forming a bore or a series of bores as it approaches the shore. A turbulent bore is defined as a broken wave having a steep, violently foaming and turbulent wave front, propagating over still water of a finite depth, as shown in Figure 2-5. These broken waves (or bores) are considered relatively short waveforms (although still longer than wind-generated waves) riding on a much longer main heave of the tsunami. Such bore formations were often observed in video footage of the 2004 Indian Ocean Tsunami and the 2011 Tohoku Japan Tsunami.

Figure 2-5 Sketch of a bore and photo of the 1983 Nihonkai-Chubu Tsunami showing the formation of a bore offshore (photo from Knill, 2004).

- After a bore reaches the shore, the tsunami rushes up on dry land in the formation of a surge, as shown in Figure 2-6. In some cases, especially when a long-wavelength, leading-elevation, and far-source-generated tsunami attacks land on a steep slope, the runup can be characterized as a gradual rise and fall of water (i.e., surge flooding) as shown in Figure 2-7. The impact of the 1960 Chilean tsunami at some Japanese localities and the 1964 Alaska tsunami at the town of Port Alberni, Canada are classic examples of surge flooding.

Figure 2-6 Sketch of a surge and photo of the 1983 Nihonkai-Chubu Tsunami showing the formation of a surge (photo courtesy of N. Nara).

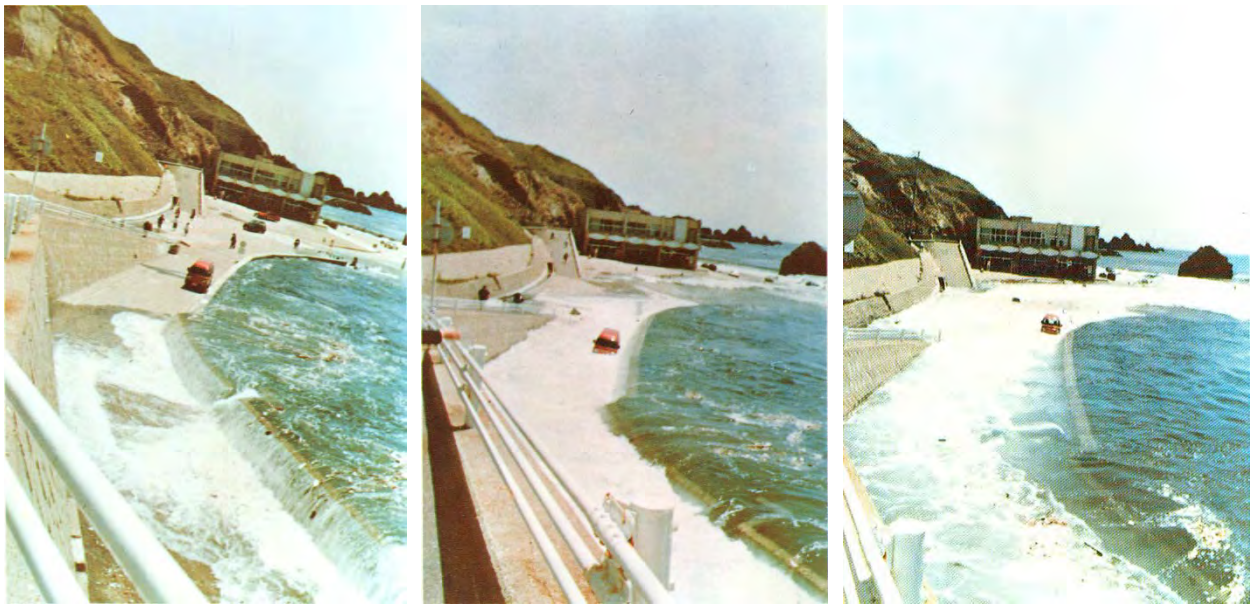


Figure 2-7 A sequence of photos of the 1983 Nihonkai-Chubu Tsunami showing surge flooding from tsunami runup (photo courtesy of S. Sato).

2.2 Tsunami Effects on Buildings

Damage studies from historic tsunami events, the 2004 Indian Ocean Tsunami and the 2011 Tohoku Japan Tsunami, and storm surge associated with Hurricane Katrina in 2005 have provided information on the response of the built environment to devastating tsunamis and coastal flooding. Although there is considerable damage to, and often total destruction of, residential and light-framed buildings during extreme coastal flooding, there are also numerous examples of mid- to high-rise engineered structures that survived tsunami inundation.

There are numerous examples of mid- to high-rise engineered structures that have survived tsunami inundation.

Structural damage from tsunamis can be attributed to: (1) direct hydrostatic and hydrodynamic forces from water inundation; (2) impact forces from water-borne debris; (3) fire spread by floating debris and combustible liquids; (4) scour and slope/foundation failure; and (5) wind forces induced by wave motion.

2.2.1 Historic Data on Tsunami Effects

Studies of damage from historic tsunamis have shown that building survivability varies with construction type and tsunami runup height (Yeh et al., 2005). Figure 2-8 shows data on damage for various types of construction resulting from the 1993 Okushiri Tsunami and earlier tsunamis.

For a given tsunami height, wood frame construction experienced considerably more damage and was frequently destroyed, while reinforced concrete structures generally sustained only minor structural damage. Recent data, including those of the 2004 Indian Ocean Tsunami, support this conclusion.

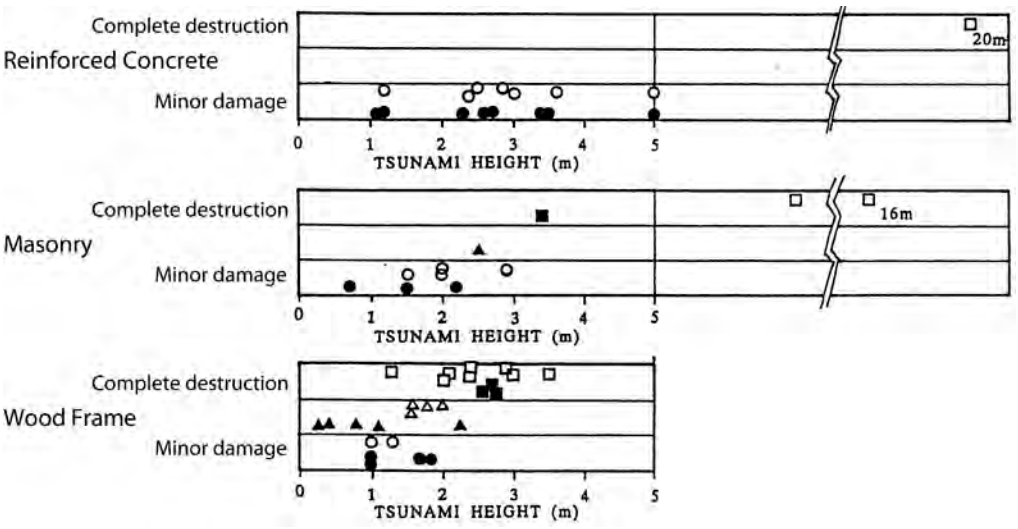


Figure 2-8 Degrees of building damage vs. tsunami runup height. Marks filled in black are data from the 1993 Okushiri tsunami; hollow marks are data from previous tsunami events (adapted from Shuto, 1994, Yeh, et al., 2005).

Note that the total destruction of one concrete structure is identified in Figure 2-8. This structure was the lighthouse at Scotch Cap, Unimak Island. The Scotch Cap lighthouse is shown in Figure 2-9, before and after the 1946 Aleutian Tsunami. There is some question as to how well the lighthouse was constructed, but it is possible that its destruction was the result of a wave breaking directly onto the structure, which was located right at the shoreline. The breaking wave could have been equivalent to a "collapsing" breaker, one of the classifications of wave breakers used in coastal engineering (Wiegel, 1964) that occurs at shorelines with steeply sloping beaches.

Figure 2-9 Scotch Cap Lighthouse destroyed by the 1946 Aleutian Tsunami.

The 1993 Okushiri Tsunami completely destroyed the entire town of Aonae. Figure 2-10 shows bare concrete foundations typically observed as remnants of wood-frame residential construction after the tsunami.

The 1992 Nicaragua Tsunami event provided other examples of variations in the performance of different structures. Figure 2-11 shows severe scour and complete destruction of a grade-level wood-frame house (left), and survival of an elevated wood frame and a grade-level rigid masonry structure (right). All three houses were located on a beach berm in the same vicinity, less than 200 meters apart.

Building failures have been observed when waterborne debris traveling at significant speeds impacts buildings. An example of the destruction caused by the impact of water-borne debris from the 1993 Okushiri Tsunami is shown in Figure 2-12. The debris in this case was a fishing boat that had broken free from its moorings. Waterborne debris is also known to collect

between structural supports creating a barrier that can significantly increase hydraulic forces on the building.

Figure 2-10 Total destruction of a group of wood-frame houses in Aonae Village, Okushiri Island, Japan (1993 Okushiri Tsunami).

Figure 2-11 Beach houses with varying levels of damage in El Popoyo, Nicaragua (1992 Nicaragua Tsunami). All three houses are in the same vicinity.

Figure 2-12 Damage caused by impact from water-borne debris (fishing boat) in Aonae, Japan (1993 Okushiri Tsunami) (photo courtesy J. Preuss).

In contrast to the many failures reported as a result of past tsunamis, many structures have been observed to survive tsunami inundation. Two structures that survived the 1993 Okishiri Tsunami are shown in Figure 2-13. Both are two-story reinforced concrete structures, and both were inundated by at least 3 meters of water.

Figure 2-13 Examples of reinforced concrete structures that survived the 1993 Okushiri Tsunami: vista house at Cape Inaho (left); and fish market in Aonae (right) (photo courtesy N. Shuto).

2.2.2 Observations from the Indian Ocean Tsunami

Damage observed as a result of the 2004 Indian Ocean tsunami confirmed observations from historic data on tsunami effects, and provided new evidence on observed effects.

Figure 2-14 shows a damaged unreinforced masonry house in Devanaanpattinam, India. Foundations experienced severe scour, and the rear walls were forced out by hydraulic pressure due to flooding inside the house. This type of damage is commonly observed in masonry buildings.

Figure 2-14 Damaged masonry beach house in Devanaanpattinam, India (2004 Indian Ocean Tsunami).

As observed in past tsunamis, numerous engineered buildings survived the 2004 Indian Ocean Tsunami. In some instances, there was damage to structural elements at the lower levels, but seldom to an extent that led to total collapse of the structure. One example of a surviving structure is a mosque located at the water's edge in Uleele, Banda Aceh, shown in Figure 2-15. The inundation depth at the mosque was about 10 m (just under the roof line), and the surrounding town was destroyed. The mosque suffered significant damage but was still standing.

Dalrymple and Kriebel (2005) commented that the survival of many hotel buildings in Thailand was due in part to the relatively open nature of the first floor construction, so that "these buildings suffered little structural damage as the force of the tsunami broke through all of the doors and windows, thus reducing the force of the water on the building itself."

The 2004 Indian Ocean Tsunami provided additional evidence of the effects of waterborne debris impact and scour on structural elements. Examples of waterborne debris included fishing boats and vehicles (Figure 2-16). Damage to structural elements of non-engineered reinforced concrete buildings was attributed to impact from such debris (Figure 2-17). Examples are also evident where debris damming resulted in damage to structural members (Figure 2-18). An example of observed scour below a shallow foundation is shown in Figure 2-19. From a review of available data taken by various survey teams, it appears that the maximum scour depth measured onshore was 3m in Khao Lak, Thailand.

Figure 2-15 Example of surviving reinforced concrete mosque in Uleele, Banda Aceh (photo courtesy J. Borerro).

Figure 2-16 Examples of waterborne debris from the 2004 Indian Ocean Tsunami (photos courtesy of M. Saatcioglu, A. Ghobarah and I. Nistor, CAEE, 2005).

A noteworthy structural failure encountered in the 2004 Indian Ocean Tsunami was uplift of precast concrete panels in buildings and docks (Figure 2-20). Uplift forces were sufficient to lift the concrete panels and break attachments between the panels and the supporting members. These failures cannot be explained by buoyancy effects alone, which reduce net downward gravity forces by the volume of water displaced. Net uplift forces sufficient to fail these elements have been attributed to additional buoyancy effects due to trapped air and vertical hydrodynamic forces caused by the rising water.

Figure 2-17 Damage to non-engineered concrete columns due to debris impact (photos courtesy of M. Saatcioglu, A. Ghobarah and I. Nistor, CAEE, 2005).

Figure 2-18 Damage to corner column due to debris damming (photo courtesy of M. Saatcioglu, A. Ghobarah and I. Nistor, CAEE, 2005).

Figure 2-19 Scour around shallow spread footing in Khao Lak area (Dalrymple and Kriebel, 2005).

Figure 2-20 Uplift damage to precast concrete floor panels and harbor piers (photo courtesy of M. Saatcioglu, A. Ghobarah and I. Nistor, CAEE, 2005).

Also, lack of adequate seismic capacity led to a number of collapses of multistory reinforced concrete buildings in Banda Aceh and other areas near the epicenter of the magnitude-9.3 earthquake that triggered the tsunami (Figure 2-21). These collapses occurred prior to inundation by tsunami waves, and highlight the importance of providing adequate seismic resistance in addition to tsunami resistance in regions where both hazards exist.

(a) Beam-column connection failures

(b) Soft story failure

Figure 2-21 Examples of structural collapse due to strong ground shaking in Banda Aceh prior to tsunami inundation (photos courtesy of M. Saatcioglu, A. Ghobarah and I. Nistor, CAEE, 2005).

2.2.3 Observations from the Tohoku Japan Tsunami

Along the Tohoku coast, tsunami inundation height was in the range of 5 to 30+ meters. In general, light-frame residential construction subject to about a story height or more of inundation will collapse. In this event, complete collapse of residential light-frame construction occurred in nearly 100% of all affected areas extending to the edge of the inundation limit. In commercial and industrial areas, 75-95% of the low rise buildings collapsed, with the higher collapse rate occurring as tsunami height reached the upper range (Figure 2-22). In these inundated coastal zones, buildings taller than 5 stories were uncommon. Despite this devastation, there were a number of these multi-story buildings that survived the tsunami without loss of structural integrity of their vertical load carrying system or foundation. In fact, a significant proportion of the surviving buildings did not appear to have significant structural damage. This provides some encouragement regarding the potential resilience of larger modern buildings having robust seismic designs and scour and uplift-resistant foundations, even when subjected to tsunami inundation greater than that for which they were designed.

Figure 2-22 Scene of near-total devastation of Minamisanriku (photo courtesy of I. Nistor, ASCE, 2012).

Under a 2005 Japanese Cabinet Office guideline, buildings to be designated as tsunami shelters should be made of concrete or other similarly robust materials. They should be at least three stories high in areas where flood levels are predicted to reach two meters, or at least four stories high if flood

levels are predicted to reach three meters. The 18 municipal governments in Aomori, Iwate, Miyagi and Chiba prefectures had designated a total of 88 buildings as vertical evacuation sites.

Figure 2-23 and Figure 2-24 show the designated evacuation area on the roof of a coastal building in Minamisanriku. This building was built as a residential structure, but with specific vertical evacuation attributes as part of the design. Access to the roof level evacuation area was provided by external elevator and staircase accessible without entering the rest of the building. The evacuation area measured a total of 660 square meters and was surrounded by a well-braced 2 meter high guard fence. Even though this building was overtopped by 0.7 meters, those who sought refuge on the roof survived the tsunami.

Figure 2-23 Minamisanriku designated coastal evacuation building – note tsunami trace on sign (photo courtesy of I. Robertson, ASCE, 2012).

Unfortunately, many of the designated vertical evacuation buildings were not tall enough for the flow depths encountered during this tsunami. An unknown number of people who sought refuge in these structures did not survive the inundation, even though the structures remained intact. It is therefore paramount that structures designated for vertical evacuation refuge be tall and strong enough to keep the refugees safe even during tsunami events that exceed the maximum considered event.

Figure 2-24 Exterior elevator and stairway access to large roof evacuation area protected by 2 meter high braced guard fence on Minamisanriku coastal evacuation building (photo courtesy of I. Robertson, ASCE, 2012).

Figure 2-25 shows a man-made earth mound in a park area at the West end of Sendai port that was only inundated to about half its height, allowing considerable area for refuge in an otherwise flat region. Similar mounds near the coastline in Natori were overtopped during the tsunami so would not have been suitable as evacuation sites. Only limited erosion was observed on the flanks of these earth mounds indicating that this concept can work, provided the evacuation site on the top of the mound is well above the inundation level.

Figure 2-25 Potential evacuation earth mound at West end of Sendai Port (photo courtesy of I. Robertson, ASCE, 2012).

As observed in prior tsunamis, the Tohoku Japan Tsunami created all loading and effects including hydrostatic forces, hydrodynamic forces, debris damming and debris impact forces, and scour effects.

Any of these effects alone, or in combination with the others, was observed to cause structural failures to low- to mid-rise building components of any structural material. Building performance was not guaranteed simply by generic choice of structural material and structural system. Lateral strength and element resistance to impact were critical to avoid local damage, while

resistance to progressive collapse was effective at preventing local member failures from precipitating disproportionate structural collapse.

A number of low-rise reinforced concrete buildings in Minamisanriku survived complete inundation (Figure 2-26). Many of these buildings had solid concrete walls facing the ocean, exposing them to the maximum possible hydrodynamic loading. A nearby reinforced concrete building with shear walls framing the lower two floors, and concrete cantilever columns supporting a steel truss roof, suffered complete collapse of the top story (Figure 2-27). The large quantity of trees as debris in the flow, and the susceptibility of cantilever columns to flexural failure, likely contributed to this failure.

Figure 2-26 Surviving and damaged reinforced concrete buildings in Minamisanriku (photo courtesy of I. Robertson, ASCE, 2012).

The harbor town of Onagawa experienced a tsunami surge of approximately 18+ meters that overtopped nearly all buildings in the area except for those on a central hillside. Outflow velocities following this initial tsunami run-up were particularly high. Despite this, many low-rise steel and concrete buildings survived. Among the failed structures were more than a half-dozen overturned and displaced whole buildings, nearly structurally intact from foundation to roof. These buildings were either floated by hydrostatic forces and carried away, or overturned by hydrodynamic forces of the tsunami inflow or outflow, or a combination of both effects. The contribution of these

effects to the failures depended on the degree of openness of the building structures.

Figure 2-27 Collapsed top floor of reinforced concrete building with steel truss roof (photo courtesy of I. Nistor, ASCE, 2012).

One illustration is a two-story reinforced concrete cold storage building, which had refrigerated storage on the ground floor and the refrigeration equipment on the second floor. Due to this function, the building consisted of a closed concrete shell except for doors and a few second floor windows for its administrative room and ventilation. Hydrostatic buoyancy lifted the building off its pile foundation, which did not have tensile capacity, and carried it over a low wall before being deposited about 15 meters inland from its original location (Figure 2-28).

Other overturned concrete and steel buildings were sufficiently open to relieve hydrostatic uplift but were still toppled by hydrodynamic forces of the incoming or returning flow. A four-story structural steel moment resisting frame lost many of its lightweight precast concrete cladding panels and had numerous window openings (Figure 2-29). Nevertheless, the building's spun-cast hollow precast piles were sheared off or extracted from the ground, and the office building displaced by about 15 meters.

Figure 2-30 shows a three-story reinforced concrete building frame with shear walls on a 0.9 meter-thick mat foundation which overturned toward Onagawa bay during the tsunami return flow.

Figure 2-28 Overturned cold storage building in Onagawa (photo courtesy of G. Chock, ASCE, 2012).

Figure 2-29 Overturned steel-framed office building in Onagawa (photo courtesy of G. Chock, ASCE, 2012).

Figure 2-30 Overturned three-story commercial building on mat foundation (photo courtesy of I. Robertson, ASCE, 2012).

2.2.4 Observations from Hurricane Katrina

The storm surge along the Mississippi Gulf coast was estimated to have been between 25 and 28 feet during Hurricane Katrina (FEMA 548, 2006). This resulted in extensive inundation of low-lying coastal regions from New Orleans, Louisiana to Mobile, Alabama.

While hurricane storm surge and tsunami inundation both result in coastal flooding, the characteristic behavior of this flooding can be quite different. Hurricane storm surge typically inundates coastal areas for a longer duration (several hours) with repeated pounding from wave action and gusting winds. Tsunami inundation generally takes place over a shorter time period (tens of minutes) with rapidly changing water levels and sweeping currents. Because of these differences, extrapolation of conclusions from hurricane storm surge effects to tsunami inundation effects is necessarily limited. In spite of these differences, however, observations from Hurricane Katrina appear to support many of the effects documented with tsunami inundation and the conclusions drawn from historic tsunami data.

Observations from **Hurricane Katrina** appear to support many of the effects documented with tsunami inundation and the conclusions drawn from historic tsunami data.

The worst storm surge in Hurricane Katrina was experienced between Pass Christian and Biloxi along the Mississippi coast, and thousands of light-framed single- and multi-family residences were totally destroyed or badly damaged by this surge (FEMA 549, 2006). However, consistent with

observations from past tsunamis, most multi-story engineered buildings along the coastline survived the surge with damage limited to nonstructural elements at the lower levels (Figure 2-31).

Figure 2-31 Pass Christian office building with cast-in-place concrete pan joist floor system that suffered non-structural damage at first two floors but no structural damage (Hurricane Katrina, 2005).

Also consistent with past tsunami observations, Hurricane Katrina illustrated the effects of debris impact and damming. At the parking garage structure shown in Figure 2-32, impact from a barge-mounted casino failed a lower level column resulting in progressive collapse of the surrounding portions of the structure. In Figure 2-33, damming effects were significant enough to fail a series of prestressed concrete piles at a construction site when a shipping container lodged between the piles and blocked the surge flow.

Similar to uplift failures observed in the 2004 Indian Ocean Tsunami, uplift loading applied to the underside of floor systems is blamed for the collapse of elevated floor levels in numerous engineered structures. Parking garages constructed of precast prestressed concrete double-tee sections, like the one shown in Figure 2-34, were susceptible to upward loading caused by additional buoyancy forces from air trapped below the double-tee sections and upward hydrodynamic forces applied by the surge and wave action. Although most failures of this type did not result in collapse of the entire structure, loss of floor framing can lead to column damage, increased unbraced lengths, and progressive collapse of a disproportionate section of the building.

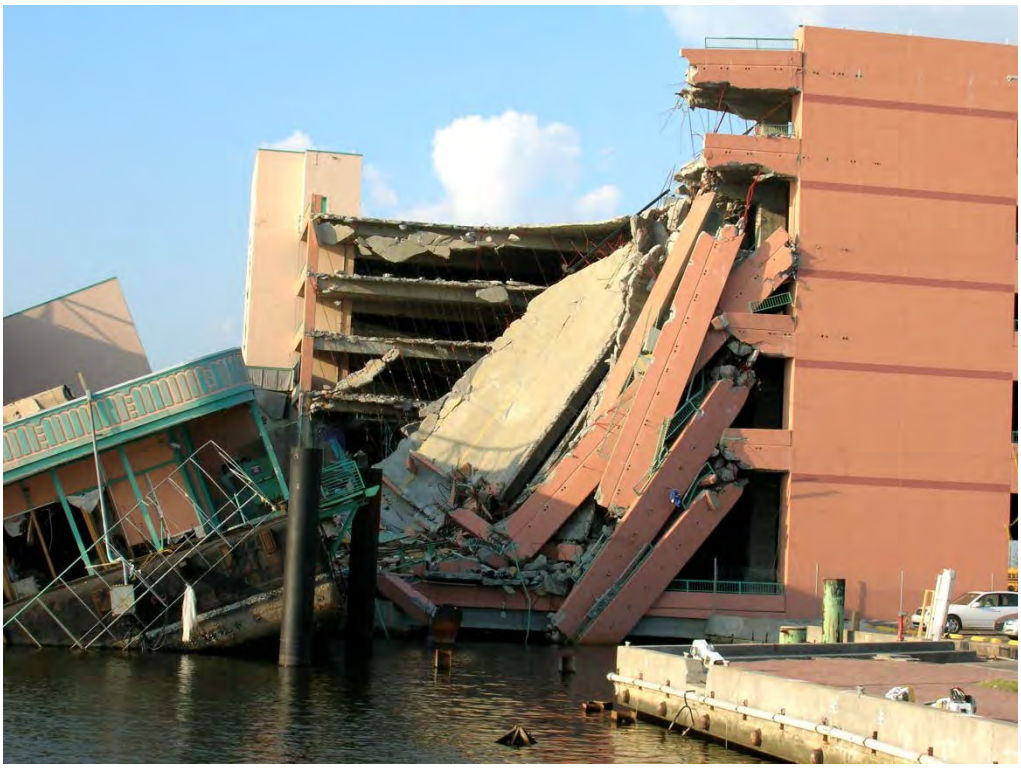


Figure 2-32 Progressive collapse of upper floors of a parking garage due to damage in lower level columns from impact of an adjacent barge-mounted casino (Hurricane Katrina, 2005).

Figure 2-33 Failure of prestressed piles due to damming effect of shipping container (Hurricane Katrina, 2005).

Figure 2-34 Negative bending failure of a prestressed double-tee floor system due to uplift forces (Hurricane Katrina, 2005).

2.2.5 Implications for Tsunami-Resistant Design

There is much evidence that appropriately designed structural systems can survive tsunami inundation.

This enables consideration of vertical evacuation as a viable alternative when horizontal evacuation out of the inundation zone is not feasible.

Building survivability varies with construction type and tsunami runup height. While observations from past tsunamis show that certain types of construction are largely destroyed by high-velocity water flow, there is much evidence that appropriately designed structural systems can survive tsunami inundation with little more than nonstructural damage in the lower levels, and can continue to support the levels of a building above the flood depth. This enables consideration of vertical evacuation as a viable alternative when horizontal evacuation out of the inundation zone is not feasible.

Observed effects from historic tsunami data, the 2004 Indian Ocean Tsunami, the 2011 Tohoku Japan Tsunami, and supporting evidence from extreme storm surge flooding associated with Hurricane Katrina result in the following implications for tsunami-resistant design:

- Vertical evacuation structures must be tall enough to ensure safety of those seeking refuge even if the tsunami event exceeds the maximum

considered tsunami. They should be well-engineered reinforced concrete or steel-frame structures.

- In the case of near-source generated tsunami hazards, vertical evacuation structures must be designed for seismic loading in addition to tsunami load effects.
- Vertical evacuation structures should be located away from the wave breaking zone.
- Impact forces and damming effects from waterborne debris are significant and must be considered.
- When elevated floor levels are subject to inundation, uplift forces from added buoyancy due to trapped air and vertical hydrodynamic forces on the floor slab must be considered.
- Scour around the foundations must be considered.

Because of uncertainty in the nature of water-borne debris and the potential for very large forces due to impact, progressive collapse concepts should be employed in the design of vertical evacuation structures to minimize the possibility of disproportionate collapse of the structural system.

Chapter 3
Tsunami Hazard Assessment

Tsunami hazard in a particular region is a combination of the presence of a geophysical tsunami source, exposure to tsunamis generated by that source, and the extent of inundation that can be expected as a result of a tsunami reaching the site. The consequences of that hazard to the population of a coastal community are a function of the time it takes a tsunami to propagate from a source to the site, maximum flood depth, maximum current velocity, integrity of the built environment, and the ability to evacuate to areas of refuge.

Inundation is a complex process influenced by many factors. These include the source characteristics that determine the nature of the initially generated waves, the bathymetry that transforms the waves as they propagate to the shoreline, the topography traversed, the structures and other objects encountered, and the temporal variation in bathymetry, topography, structures and other objects caused by the impact of successive waves. In general, the physics of tsunami inundation is time-dependent, three-dimensional, and highly nonlinear.

Modeling of tsunami inundation is a key component of tsunami hazard assessment. Progress has been made in the development of modeling tools, but theory is still under development. This chapter provides an overview of currently available modeling tools and associated products available through nationally-coordinated efforts such as the National Oceanic and Atmospheric Administration (NOAA) Tsunami Program and the U.S. National Tsunami Hazard Mitigation Program (NTHMP).

Modeling of tsunami inundation is a key component of tsunami hazard assessment. Current efforts to characterize tsunami hazard include:

The NOAA Tsunami Program: Forecast Modeling and Mapping

The National Tsunami Hazard Mitigation Program: Credible Worst-Case Scenarios

The FEMA Map Modernization Program: Probabilistic Tsunami Hazard Assessments

3.1 Current Tsunami Modeling and Inundation Mapping

Site-specific inundation models and model-derived products, including maps, are essential for reliable tsunami hazard assessment. The NOAA Tsunami Program and the NTHMP are engaged in closely related modeling efforts. The NOAA Tsunami Program is focused on the development of the NOAA Tsunami Forecast System (Titov and Synolakis, 2005). The NTHMP Hazard Assessment effort is working on the development of inundation maps for emergency management programs (González, et al., 2005a). Both efforts are fundamentally dependent on tsunami numerical modeling technology.

Tsunami modeling studies generally result in products that include a spatial mapping of the model output in either static or animated form. Primary tsunami wave parameters include the amplitude $\eta(x,y,t)$ and associated current velocity components $u(x,y,t)$ and $v(x,y,t)$. A Geographic Information System (GIS) database of these output parameters and associated input data (e.g., model computational grids and source parameters) can be used to derive parameters such as flood depth, velocity, acceleration, and momentum flux.

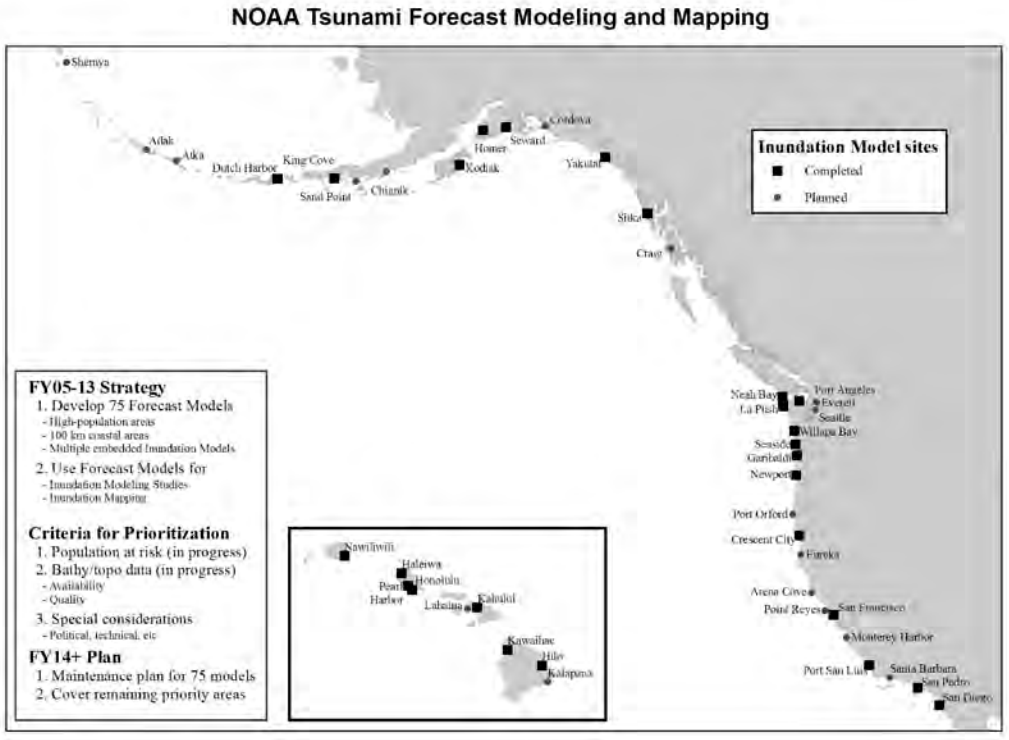
3.2 The NOAA Tsunami Program: Forecast Modeling and Mapping

As part of the Tsunami Forecasting System, NOAA is developing site-specific inundation models at 75 sites shown in Figure 3-1. The National Center for Tsunami Research (NCTR) at the Pacific Marine Environmental Laboratory (PMEL) in Seattle, Washington, has the primary responsibility for this forecast modeling and mapping effort. The first step at each site is the development of a Reference Model using a grid with the finest resolution available, followed by extensive testing against all available data to achieve the highest possible accuracy. The second step is development of the Standby Inundation Model (SIM), which is used as the forecast model. This is done through modification of the grid to optimize for speed, yet retain a level of accuracy that is appropriate for operational forecast and warning purposes.

The NCTR employs a suite of tsunami generation, propagation, and inundation codes developed by Titov and Gonzalez (1997). On local spatial scales, nonlinear shallow water (NSW) equations are solved numerically. Propagation on regional and transoceanic spatial scales requires equations that are expressed in spherical coordinates. Propagation solutions are obtained by a numerical technique that involves a mathematical transformation known as splitting (Titov, 1997). Consequently, this suite of models has become known as the Method of Splitting Tsunamis (MOST) (Tang, 2009) model.

Because life and property are at stake when tsunami warnings are issued, NOAA requires that models used in the Tsunami Forecasting System meet certain standards (Synolakis, 2006). Among the requirements are:

- *Peer-reviewed publication.* A peer-reviewed article must be published that documents the scientific and numerical essentials of the model and includes at least one model comparison study using data from an historical tsunami.



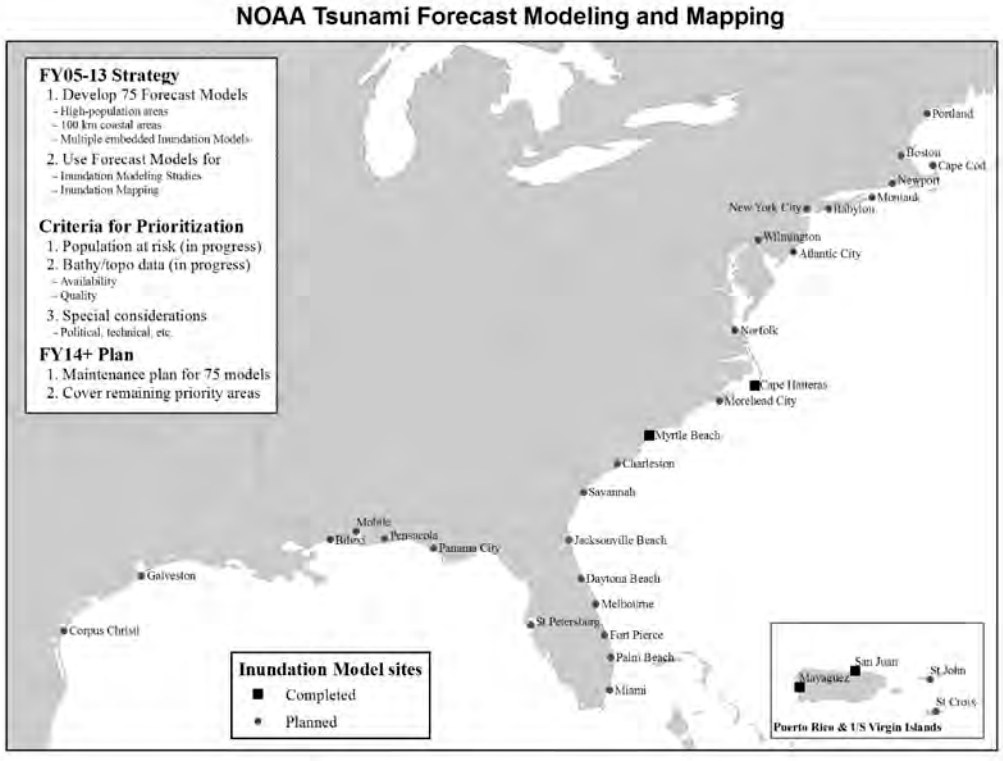


Figure 3-1 Coastal sites for site-specific tsunami inundation models for the Tsunami Forecasting System.

- *Benchmarking.* The model must be tested against other peer models in a benchmark workshop, and the results documented in a report. The National Science Foundation has supported two tsunami inundation modeling benchmark workshops (Yeh, et al., 1996; Liu, et al., 2006). The National Tsunami Hazard Mitigation Program (NTHMP) supported another benchmarking workshop in 2011 (NTHMP MMS Tsunami Inundation Model Validation Workshop, 3-28-2011 to 4-1-2011, Texas A&M Galveston campus) for which a peer-reviewed proceedings volume is in preparation (NTHMP, 2012).
- *Operational assessment.* Important factors to be assessed include the model speed, accuracy, special operating environment needs, ease of use, and documentation.

Models meeting these requirements include the ADvanced CIRCulation (ADCIRC) model (Luettich and Westerink, 1991, 1995a, and 1995b; Myers and Baptista, 1995), hydrodynamic models of Kowalik and Murty (1993a, 1993b) as applied and field-checked against observed inundation in Alaska by Suleimani and others (2002a; 2002b), and the MOST model (Titov and Synolakis, 1998).

The MOST model has been extensively tested against laboratory experimental data and deep-ocean and inundation field measurements, and by successful modeling of benchmarking problems through participation in NSF-sponsored tsunami inundation model benchmark workshops.

As of June 2008, reference inundation models and forecast models have been completed using the MOST model for seven sites in Alaska, four sites in Washington, three sites in Oregon, five sites in California, seven sites in Hawaii, one site in North Carolina, and one site in South Carolina. Planned and completed sites are shown in Figure 3-1.

The primary function of these models is to provide NOAA Tsunami Warning Centers with real-time forecasts of coastal community inundation before and during an actual tsunami event. However, these site-specific inundation models can be applied to inundation modeling studies and the creation of inundation parameter databases, digital products, and maps specifically tailored to the design process.

Models provisionally validated in the most recent benchmark workshop include the Alaska Tsunami Forecast Model from the West Coast and Alaska Tsunami Warning Center, the Alaska Tsunami Model from the University of Alaska, SELFE from the Oregon Health Sciences Institute, FUNWAVE, from the Universities of Delaware and Rhode Island, THETIS from the

Université de Pau et des Pays de l'Adour and University of Rhode Island, BOSZ and NEOWAVE from the University of Hawaii, TSUNAMI3D from University of Alaska and Texas A&M Galveston, GeoClaw from the University of Washington, and the MOST model from the Pacific Marine Environmental Laboratory. Most of these models are described elsewhere but the benchmark validation documentation is in preparation.

3.3 The National Tsunami Hazard Mitigation Program: Credible Worst-Case Scenarios

State mapping efforts performed as part of the National Tsunami Hazard Mitigation Program (NTHMP) are based on credible worst-case scenarios. Credible worst-case scenario maps are based on a geophysical tsunami source that can be scientifically defended as a worst-case scenario for a particular region or community, and a tsunami inundation model simulation for that scenario. The simulation output becomes the basis for maps that typically display maximum inundation depth and maximum current speed or velocity. Example worst-case scenario inundation model results for Seattle, Washington are shown in Figure 3-2. These products are provided to state geotechnical scientists, who then produce official state inundation maps such as the one for Seattle, Washington shown in Figure 3-3.

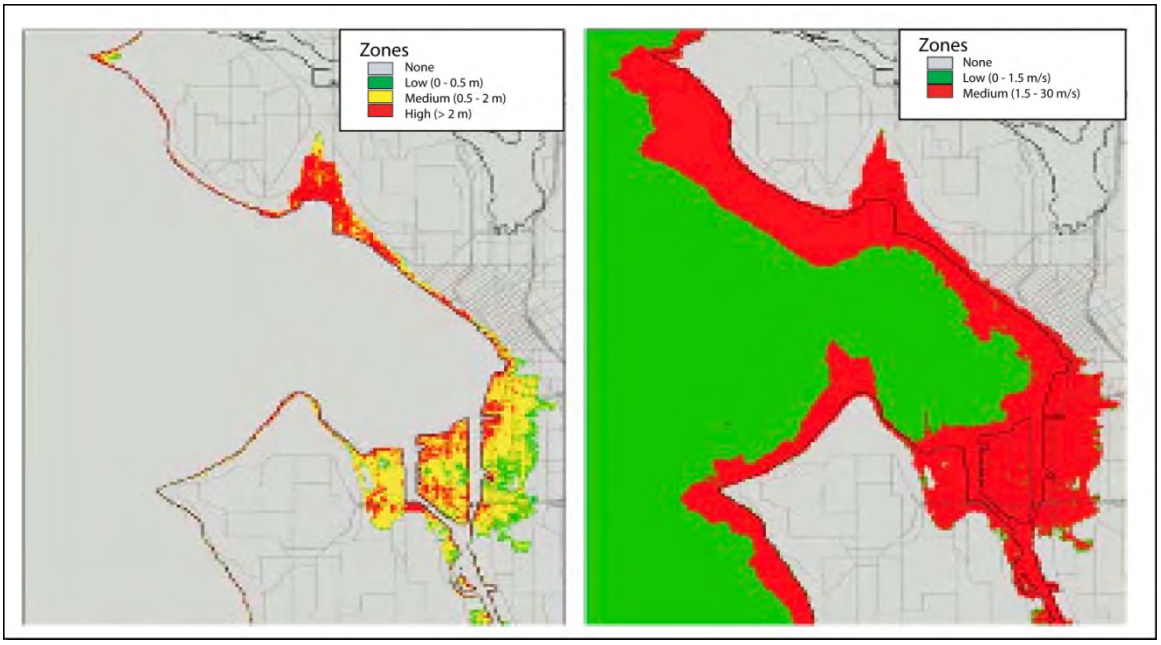


Figure 3-2 Tsunami inundation modeling products for Seattle, Washington. Left panel: zoned estimates of maximum inundation depth. Right panel: zoned estimates of maximum current (Titov, et al., 2003).

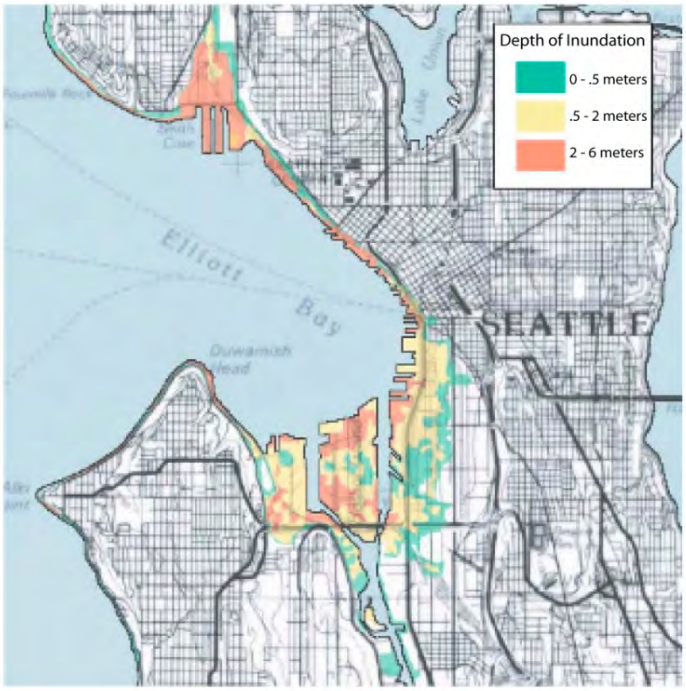


Figure 3-3 Tsunami inundation map for Seattle, Washington produced and published by the state of Washington, using modeling products as guidance (Walsh et al., 2003).

These maps are considered essential for effective disaster planning and development of emergency management products and programs. They guide the development of evacuation maps, educational and training materials, and tsunami mitigation plans. By 2004, the NTHMP Hazard Assessment component had completed 22 inundation mapping efforts and 23 evacuation maps covering 113 communities and an estimated 1.2 million residents at risk (González, et al., 2005a).

There are variations in state products because each state differs in its geophysical setting and the resulting tsunami regime including legislative goals, policies, agency structure, mission, scientific and technical infrastructure, and financial status. Differences between state mapping products include the following:

- Although most credible worst-case scenarios are based on seismic sources, maps generated for Alaska and California also include landslide sources in the tsunami hazard assessment.
- Oregon inundation maps, like the one for Yaquina Bay shown in Figure 3-4, display three inundation lines to depict the uncertainty in the hazard posed by tsunamis from the local Cascadia subduction zone.

- In addition to worst-case scenarios, maps in Alaska also depict inundation from a number of locally-generated scenario tsunamis.

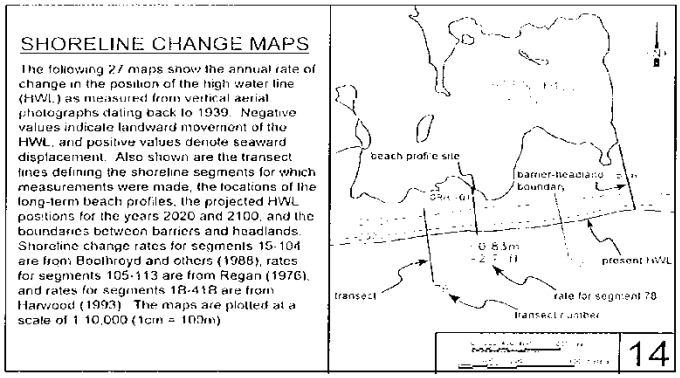


Figure G-19
Sample shoreline change map showing average annual shoreline change and projected future shoreline locations (RICRMC 1995).

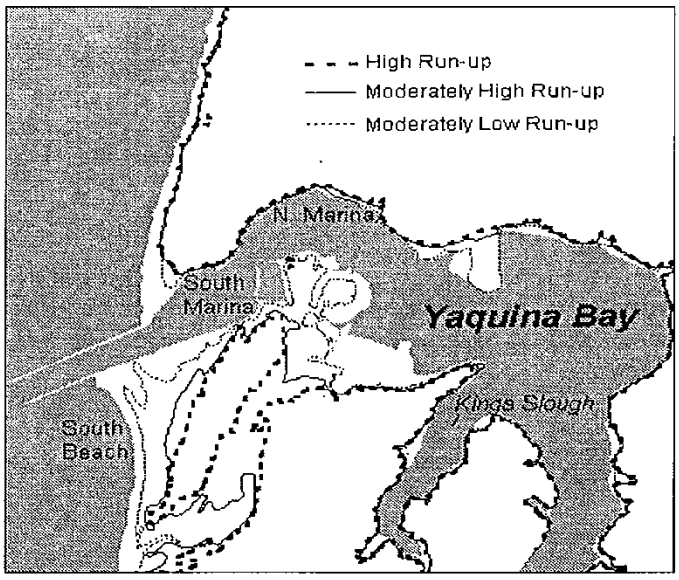


Figure G-20
Yaquina Bay, Oregon, tsunami inundation map (from Priest et al. 1997).

Figure 3-4 Yaquina Bay, Oregon tsunami inundation map with three inundation lines (Priest et al., 1997a; Priest et al.,1997b).

Detailed tsunami inundation simulations for credible worst-case scenarios can also be used to derive parameters such as flood depth, velocity, acceleration, and momentum flux, which are used to calculate forces for tsunami-resistant design. These data are archived with the state government hazard mapping agencies, cooperating academic institutions, and the NOAA Pacific Marine Environmental Laboratory. Currently, a central archive for all state mapping products does not exist. However, existing maps and

reports are available for viewing, download, or purchase from the following state web sites:

- Alaska: http://www.dggs.dnr.state.ak.us/pubs/publisher/dggs
- California: http://www.conservation.ca.gov/cgs/geologic_hazards /Tsunami/Inundation_Maps/Pages/Index.aspx
- Oregon: http://www.oregongeology.com/sub/publications/IMS/ims.htm
- Washington: http://www.dnr.wa.gov/Publications/ger_tsunami_inundation_maps.pdf

3.4 The FEMA Map Modernization Program: Probabilistic Tsunami Hazard Assessments

On the regional scale, FEMA (1997) presents a probabilistic estimate of the tsunami hazard for the West coast, Alaska, and Hawaii (Figure 3-5). On the local scale, FEMA Flood Insurance Rate Maps (FIRMs) present area-specific flooding scenarios for 100-year and, occasionally, 500-year events (i.e., events with a 1% and a 0.2% annual probability of exceedance, respectively).

The FIRMs provide a basis for establishing flood insurance premiums in communities that participate in the National Flood Insurance Program (NFIP), which is administered by FEMA. These maps were based on tsunami hazard assessment methods developed prior to 1990. To evaluate the underlying methodologies used to assess tsunami and other coastal flooding hazards, FEMA formed focused study groups for each of the flooding mechanisms. The Tsunami Focused Study Group found that the current treatment of tsunami inundation is inadequate, and recommended a joint NOAA/U.S. Geological Survey (USGS) pilot study to develop an appropriate methodology for Probabilistic Tsunami Hazard Assessments (PTHA) that could be used to update FIRMs (Chowdhury, et al., 2005).

In the joint NOAA/USGS/FEMA Seaside, Oregon Tsunami Pilot Study (Tsunami Pilot Study Working Group, 2006), USGS and academic colleagues developed a database of near- and far-field tsunami sources associated with a specified probability of occurrence, while NOAA developed a corresponding database of inundation model results based on the sources. The resulting PTHA methodology integrates hydrodynamics, geophysics, and probability theory to meet specific FEMA actuarial needs, and now represents the current state of the art in tsunami hazard assessment for emergency management and engineering design.

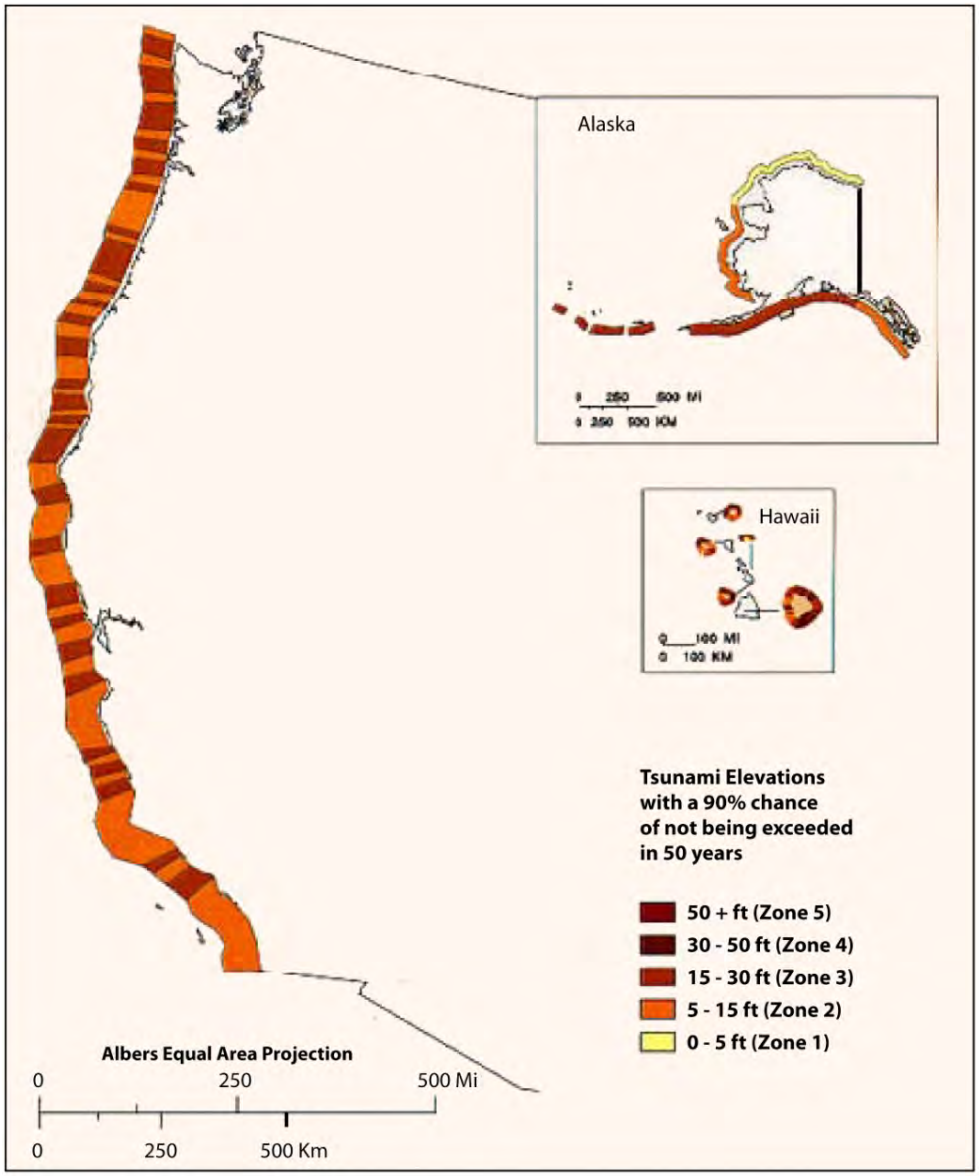


Figure 3-5 Tsunami elevations with a 90% probability of not being exceeded in 50 years (FEMA, 1997).

The 500-year maximum tsunami wave height map for Seaside, Oregon shown in Figure 3-6 is an example of the type of product that can be generated by such a study. The resulting GIS database of all model inputs, outputs, and related data can be used to conduct in-depth, site-specific probabilistic studies of tsunami hazard for design of vertical evacuation structures.

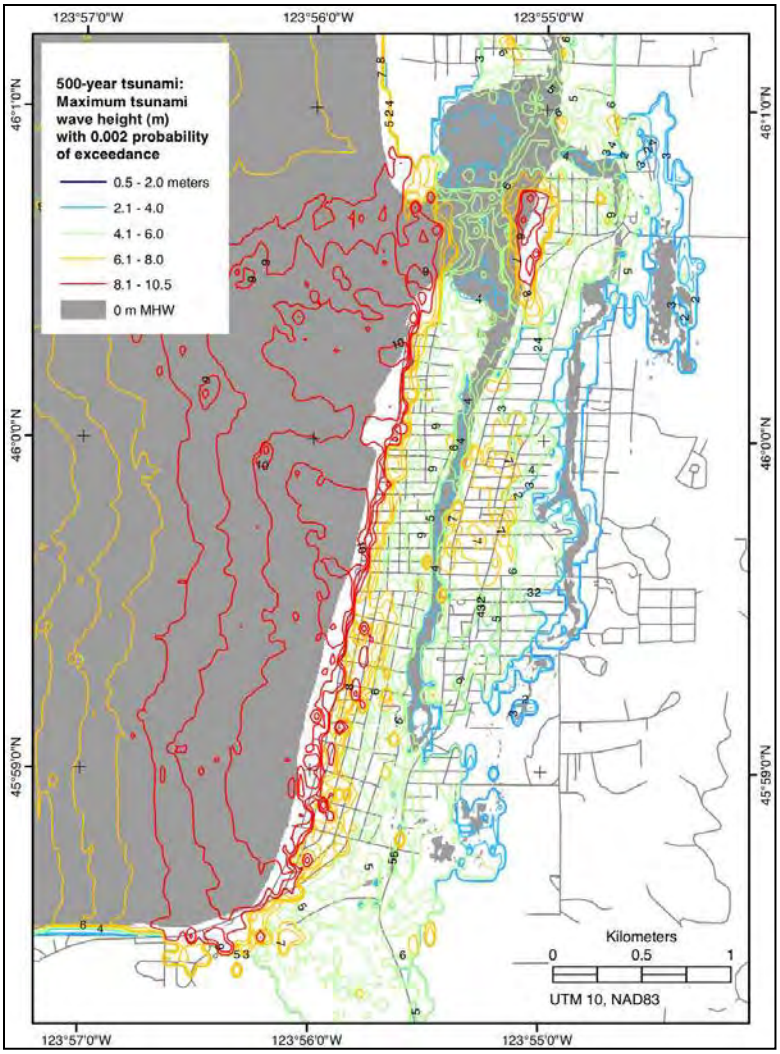


Figure 3-6 The 500-year tsunami map for Seaside, Oregon, depicting maximum wave heights that are met or exceeded at an annual probability of 0.2% (Tsunami Pilot Study Working Group, 2006).

3.5 Limitations in Available Modeling and Mapping Products

The quality, content, and availability of currently available modeling and mapping products are limited. Quality varies considerably and, in many cases, cannot be assessed because standard modeling and mapping procedures have not been adopted. Most maps do not provide estimates of currents, so their content is often inadequate for use in design. Digital model products are generally not available to derive the more relevant parameters needed for calculation of forces on structures. Availability of information is limited because a central repository for maps and other model products does not exist.

Limitations in bathymetric and topographic databases are being addressed through coordination of NOAA, USGS, and NTHMP to improve the coverage, quality and availability of the data, but this is an ongoing effort.

3.6 Hazard Quantification for Design of Tsunami Vertical Evacuation Structures

Given a known or perceived tsunami threat in a region, the first step is to determine the severity of the tsunami hazard. This involves identification of potential tsunami-genic sources and accumulation of recorded data on tsunami occurrence and runup. This can include a probabilistic assessment considering all possible tsunami sources, or a deterministic assessment considering the maximum tsunami that can reasonably be expected to affect a site.

Tsunami hazard can be characterized by:
(1) a probabilistic assessment considering all possible tsunami sources; or
(2) a deterministic assessment considering the maximum tsunami that can reasonably be expected to affect a site.

Once potential tsunami sources are identified, and the severity of the tsunami hazard is known, site-specific information on the extent of inundation, height of runup, and velocity of flow is needed. Some of this information can be obtained from available tsunami inundation maps, where they exist; otherwise site-specific tsunami inundation studies must be performed. In the absence of available maps or site-specific inundation studies, analytical solutions can be used to estimate tsunami inundation parameters for preliminary or approximate design. Analytical solutions for flow velocity, depth, and momentum flux are provided in Chapter 6 and Appendix E.

In this document, the design tsunami event is termed the Maximum Considered Tsunami (MCT). There is, however, no firm policy or methodology for setting a Maximum Considered Tsunami at a specified hazard level. For the design criteria contained within this document, it is anticipated that the hazard level corresponding to the Maximum Considered Tsunami will be consistent with a 2500-year return period. The hazard level for tsunamis is therefore similar to the return period associated with the Maximum Considered Earthquake used in seismic design. However, the Maximum Considered Tsunami is not defined to be the same as the Maximum Considered Earthquake because the tsunami source may be distant rather than local.

The Maximum Considered Tsunami (MCT) is the design tsunami event. For site-specific tsunami hazard assessments, the Maximum Considered Tsunami should be developed using the Deterministic Maximum Considered Earthquake as the initial condition of the tsunami model.

Existing tsunami hazard assessments in some areas may be adequate for the design of vertical evacuation structures. Even if published hazard maps do not include velocity and depth information, the underlying modeling might. Where the NTHMP has been producing tsunami inundation maps (Alaska, California, Hawaii, Oregon, Puerto Rico, and Washington), the state hazard assessment team (http://nthmp.pmel.noaa.gov) will provide details of the

appropriate modeling parameters and can either perform the assessment or provide a referral.

For site-specific tsunami hazard assessments, the Maximum Considered Tsunami should be developed using the tsunami-genic seismic events determined from a probabilistic hazard analysis. At a minimum, this analysis should also be checked for a Deterministic MCE for a near-source-generated tsunami in the United States evaluated as the largest potentially tsunami-genic earthquake reported in the "Quaternary Fault and Fold Database of the United States" http://earthquake.usgs.gov/regional/qfaults/.

Where the greatest threat is from a far-source-generated tsunami, selection of a Maximum Considered Tsunami is more difficult. At a minimum, it should be based on the largest event recorded in the National Geophysical Data Center (NGDC) Historical Tsunami Database (http://www.ngdc.noaa.gov/hazard/tsu_db.shtml) with allowance for the limited accuracy, quantity, and period of time covered by the historic record. It should also consider the largest earthquakes likely in all regions that have generated historic tsunamis affecting the site being considered. The NOAA forecast modeling program may be able to model a Maximum Considered Tsunami for these cases using the reference inundation models that have higher resolution and larger computational domains rather than the tsunami inundation models used for real-time forecasting.

Tsunami inundation modeling is not routinely available commercially, but is performed by a number of organizations including government laboratories (USGS, NOAA, Los Alamos National Laboratory), selected universities (Cornell University, Oregon Health and Science University, Texas A&M Galveston, University of Hawaii, University of Alaska Fairbanks, University of Rhode Island, University of Southern California, University of Washington), and some consulting companies. An extensive bibliography of past tsunami-related research in modeling is available in Wiegel (2005, 2006a, 2006b, and 2008). The NTHMP has suggested some minimum guidelines regarding tsunami hazard mapping and modeling.

- Models should meet the benchmark standards (Synolakis and others, 2007), which were recently updated at the benchmarking workshop in Galveston (Horillo and others, in preparation).
- Digital Elevation Models (DEMs) used to develop modeling grids near shore should be at a resolution of at least 1/3 arc second, or about ten meters, but should not be smaller than the spacing of the source topographic data unless necessary to resolve important morphologic features.

- DEMs should be based on the most accurate digital elevation model available. Lidar is becoming increasingly available and can achieve vertical accuracy of <1 foot.
- Model runtime should be sufficient to capture the maximum inundation and drawdown of the tsunami simulation.
- The computational grid developed from the DEM should be fine enough that any topographic or bathymetric feature that has an impact on inundation should be represented by more than three grid cells.
- The computational grid domain should be large enough to capture all important tsunami wave dynamics.
- A vertical datum of Mean High Water should be used to capture tidal conditions or an alternative maximum flooding condition should be used in modeling for tsunamis in lakes.

It should be noted that the above recommendations do not include modeling for tsunamis induced by landslides, volcanoes, or meteorite impacts.

3.7 Recommendations to Improve Tsunami Hazard Assessment

Similar to design for other hazards, a desirable goal for tsunami-resistant design of vertical evacuation structures is to achieve a uniform level of safety across all communities subjected to tsunami risk. ASCE 7, *Minimum Design Loads for Buildings and Other Structures*, is based on achieving structural reliability performance goals using probabilistic definitions of all hazards. In seismic and wind design, the starting point is probabilistic mapping of earthquake and wind hazard. The hazard is further refined by considering local effects such as soil type for seismic design, and topographic effects for wind design. Similar concepts can be used for tsunami design. Essential tools for tsunami hazard assessment are tsunami inundation models, maps, and comprehensive databases of tsunami inundation parameters.

Although more difficult for the public to interpret since they do not represent a single selected scenario, probabilistic maps for tsunami hazard can be made and are needed for reliable design of tsunami-resistant structures for uniform risk (Geist and Parsons, 2006). The probabilistic approach also provides a means to account for uncertainty. Probabilistic analysis of tsunamis is also important in explicitly defining the probability associated with individual deterministic scenarios.

Chapter 4
Vertical Evacuation Options

A *vertical evacuation refuge from tsunamis* is a building or earthen mound that has sufficient height to elevate evacuees above the level of tsunami inundation, and is designed and constructed with the strength and resiliency needed to resist the effects of tsunami waves. Vertical evacuation refuges can be stand-alone or part of a larger facility. They can be single-purpose refuge-only facilities, or multi-purpose facilities in regular use when not serving as a refuge. They can also be single-hazard (tsunami only) or multi-hazard facilities.

In concept, vertical evacuation options are applicable to new or existing structures, but it will generally be more difficult to retrofit an existing structure than to build a new tsunami-resistant structure using these criteria.

In concept, these options are applicable to new or existing structures, but it will generally be more difficult to retrofit an existing structure than to build a new tsunami-resistant structure using these criteria. This chapter describes the features of different vertical evacuation options that are available, and provides guidance to assist in choosing between various options.

It should be stressed that evacuation to high ground is always preferred where access to nearby high ground exists. This provides the option for refugees to move to even higher ground if the tsunami inundation is greater than anticipated, something that may not be possible in an evacuation building or earthen mound because of the height limitation of the refuge.

4.1 Vertical Evacuation Considerations

Vertical evacuation structures can be intended for general use by the surrounding population, or by the occupants of a specific building or group of buildings. Choosing between various options available for vertical evacuation structures will depend on emergency response planning and needs of the community, the type of construction and use of the buildings in the immediate vicinity, and the project-specific financial situation of the state, municipality, local community, or private owner considering such a structure.

4.1.1 Single-Purpose Facilities

The tsunami hazard assessment and inundation study may show that the best solution is to build new, separate (i.e., stand-alone) facilities specifically designed and configured to serve as vertical evacuation structures. Potential advantages of single-purpose, stand-alone facilities include the following:

Vertical evacuation facilities can be single-purpose, multi-purpose, or multi-hazard facilities.

- They can be sited away from potential debris sources or other site hazards.
- They do not need to be integrated into an existing building design or compromised by design considerations for potentially conflicting usages.
- They are structurally separate from other buildings and therefore not subject to the potential vulnerabilities of other building structures.
- They will always be ready for occupants and will not be cluttered with furnishings or storage items associated with other uses.
- Single-purpose, stand-alone structures will likely be simpler to design, permit, and construct because they will not be required to provide normal daily accommodations for people. They can have simplified prototypical structural systems, resulting in lower initial construction costs.

One example of a single-purpose facility is a small, elevated structure with the sole function of providing an elevated refuge for the surrounding area in the event of a tsunami. A possible application for such a facility would include low-lying residential neighborhoods where evacuation routes are not adequate, and taller safer structures do not exist in the area.

4.1.2 Multi-Purpose Facilities

A coastal community may not have sufficient resources to develop a single-purpose tsunami vertical evacuation structure or a series of structures, so creative ways of overcoming economic constraints are required. Possible solutions include co-location of evacuation facilities with other community-based functions, co-location with commercial-based functions, and economic or other incentives for private developers to provide tsunami-resistant areas of refuge within their developments. The ability to use a facility for more than one purpose provides immediate possibility for a return on investment through daily business or commercial use when the structure is not needed as a refuge.

Multi-purpose facilities can also be constructed to serve a specific need or function in a community, in addition to vertical evacuation refuge. Examples include elevated man-made earthen berms used as community open spaces. In downtown areas or business districts, they can be specially constructed private or municipal parking structures incorporating tsunami resistant design. On school campuses, vertical evacuation facilities could serve as gymnasiums or lunchrooms on a daily basis. In residential subdivisions, they can be used as community centers.

4.1.3 Multi-Hazard Considerations

Communities exposed to other hazards (e.g., earthquakes, hurricanes) may choose to consider the possible sheltering needs associated with these other hazards, in addition to tsunamis. This could include allowances for different occupancy durations, consideration of different post-event rescue and recovery activities, and evaluation of short- and long-term medical care needs.

Designing for multiple hazards requires consideration of the load effects that might be unique to each type of hazard. This can pose unique challenges for the resulting structural design. For example, the structural system for vertical evacuation structures exposed to near-source-generated tsunamis will likely need to be designed for seismic hazards. Such a structure might include break-away walls or open construction in the lower levels to allow water to pass through with minimal resistance. Open construction in the lower levels of a multi-story structure are contrary to earthquake engineering practice to avoid soft or weak stories in earthquake-resistant construction. Proper design and construction will need to include special consideration by the structural engineer of these and other potential conflicting recommendations.

4.2 Vertical Evacuation Concepts

To provide refuge from tsunami inundation, vertical evacuation solutions must have the ability to receive a large number of people in a short time frame and efficiently transport them to areas of refuge that are located above the level of flooding. Potential vertical evacuation solutions can include areas of naturally occurring high ground, areas of artificial high ground created through the use of soil berms, new structures specifically designed to be tsunami-resistant, or existing structures demonstrated to have sufficient strength to resist anticipated tsunami effects.

Vertical evacuation structures can be soil berms, parking garages, community facilities, commercial facilities, school facilities, or existing buildings.

Nonstructural systems and contents located in the levels below the inundation depth should be assumed to be a total loss if the design tsunami occurs. If the building is required to remain functional in the event of a disaster, the loss of lower level walls, nonstructural systems, and contents should be taken into account in the design of the facility and selection of possible alternative uses.

4.2.1 Existing High Ground

Naturally occurring areas of high ground may be able to be utilized or modified to create a refuge for tsunami vertical evacuation. Large open areas offer easy access for large numbers of evacuees with the added advantage of avoiding the possible apprehension about entering a building following an

earthquake. In addition, most coastal communities have educated their populations to "go to high ground" in the event of a tsunami warning. The topography of the existing high ground should be evaluated for the potential of wave runup or erosion. Some modification of the existing topography may be required to address these issues.

4.2.2 Soil Berms

If natural high ground is not available, a soil berm can be constructed to raise the ground level above the tsunami runup height, as shown in Figure 4-1. Although care must be taken to protect the sides of the soil berm from the incoming and outgoing tsunami waves, this option can be relatively cost-effective in comparison to building a stand-alone structure. The height of the berm must be sufficient to avoid becoming inundated, and the slope of the sides must allow for ingress. A maximum ramp slope in the range of one foot vertical rise to four feet horizontal run (1 in 4) is recommended. Soil berms have the added benefit that they are immune to damage from large debris strikes such as shipping containers, barges and ships, making them suitable for locations near port facilities (Figure 4-1).

Figure 4-1 Soil berm combined with a community park at Sendai Port, Japan. Concrete lining on the ocean face can deflect incoming waves while sloped sides provide for quick access. Graphic in the lower right side illustrates where the evacuation berm is located in Sendai Port.

4.2.3 Multi-Story Parking Garages

Parking garages are good candidates for use as vertical evacuation structures. Similar to the example shown in Figure 4-2, most parking garages are open structures that will allow water to flow through with minimal resistance. They can also be open for pedestrian access at any time of the day or night. Interior ramps allow ample opportunity for ingress, and easy vertical circulation to higher levels within the structure. Parking garages can also be used to provide additional community amenities on the top level, including parks, observation decks, and sports courts. They are also obvious revenue-generating facilities, especially in areas that attract large numbers of tourists.

Parking garages, however, tend to be constructed using low-cost, efficient structural systems with minimal redundancy. If designed with higher performance objectives in mind, and if subjected to additional code review and construction inspection by local jurisdictions, parking garages could be effective vertical evacuation structures.

Figure 4-2 Cast-in-place reinforced concrete parking garage in Biloxi, Mississippi after Hurricane Katrina. Open structural systems allow water to pass through with minimal resistance, and interior ramps allow for easy ingress and vertical circulation.

4.2.4 Community Facilities

Vertical evacuation structures could be developed as part of other community-based needs such as community centers, recreational facilities, sports complexes, libraries, museums, and police or fire stations. One such

example is shown in Figure 4-3. When not in use as a refuge, facilities such as these can be useful for a variety of functions that enhance the quality of life in a community. When choosing alternative uses for a vertical evacuation facility, consideration should be given to potential impacts that other uses might have on the vertical evacuation function. Potential negative impacts could include clutter that could become debris that disrupts ingress. Limited access after regular operating hours would make it difficult to use a facility for evacuation from a tsunami that could occur at any time of the day or night. Priority should be given to uses with complementary functions, such as accommodations for large numbers of people and 24-hour access.

4.2.5 Commercial Facilities

Vertical evacuation structures could be developed as part of business or other commercial facilities including multi-level hotels, restaurants, or retail establishments, as shown in Figure 4-4. For example, if the refuge area is part of a hotel complex, meeting rooms, ballrooms, and exhibit spaces that are located above the tsunami inundation elevation could be used to provide refuge when the tsunami occurs. The apartment building shown in Figure 4-5 was used successfully as a vertical evacuation structure during the Tohoku tsunami. Exterior stairs provided 24 hour access to the upper floors designated as the evacuation refuge.

Figure 4-3 Sports complex. Designed for assembly use, this type of structure can accommodate circulation and service needs for large numbers of people.

Figure 4-4 Hotel and convention complex. Meeting rooms, ballrooms, and exhibit spaces located above the tsunami inundation elevation can be used to provide areas of refuge.

Figure 4-5 Residential apartment building in Kamaishi, Japan, with designated refuge area at or above the fourth level.

4.2.6 School Facilities

Similar to community facilities, public and private school facilities have the benefit of providing useful and essential services to the communities in which they reside. Ongoing construction of schools provides an opportunity and potential funding mechanism for co-located tsunami vertical evacuation structures. This has the added benefit of possible additional public support for projects that increase the safety of school-age children. Obviously these buildings must be tall enough or sited on high ground so that they are useful as tsunami refuge areas.

4.2.7 Existing Buildings

Historic damage patterns suggest that many structures not specifically designed for tsunami loading can survive tsunami inundation and provide areas of refuge. It is possible that some existing structures could serve as vertical evacuation structures or could be made more tsunami-resistant with only minor modifications. An assessment of both the functional needs and potential structural vulnerabilities would be required to determine if an existing building can serve as a vertical evacuation structure.

In some situations, providing some level of protection is better than none. An example of this concept is shown in Figure 4-6. In a tsunami evacuation map for Waikiki, it is noted that "structural steel or reinforced concrete buildings of six or more stories provide increased protection on or above the third floor", and are identified as potential areas of refuge.

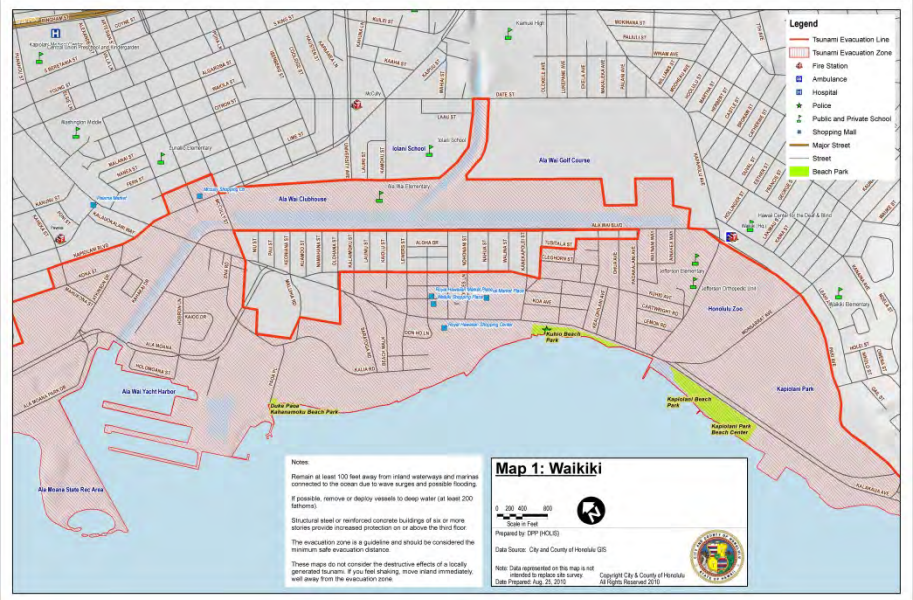


Figure 4-6 Evacuation map for Waikiki, Hawaii, indicating use of existing buildings for vertical evacuation.

Chapter 5
Siting, Spacing, Sizing, and Elevation Considerations

Tsunami risk is unique in that some communities may be susceptible to far-source-generated tsunamis (longer warning time), near-source-generated tsunamis (shorter warning time), or both. Far-source-generated tsunamis generally allow sufficient warning time so that emergency response plans can be based on evacuation out of the inundation zone. Near-source-generated tsunamis may not allow sufficient time for evacuation, so emergency response plans may need to include vertical evacuation refuge. This chapter provides guidance on how to locate vertical evacuation refuges within a community, and how to determine the size of a vertical evacuation structure.

Vertical evacuation structures should be located such that all persons designated to take refuge can reach the structure within the time available between tsunami warning and tsunami inundation.

5.1 Siting Considerations

Vertical evacuation structures should be located such that all persons designated to take refuge can reach the structure within the time available between tsunami warning and tsunami inundation. Travel time must also take into consideration vertical circulation within the structure to levels above the tsunami inundation elevation. Structures located at one end of a community may be difficult for some users to reach in a timely fashion. Routes to the structure should be easily accessible and well-marked.

Location of vertical evacuation structures within a community should take into account potential hazards in the vicinity of a site that could jeopardize the safety of the structure, and should consider that natural behaviors of persons attempting to avoid coastal flooding.

5.1.1 Warning, Travel Time, and Spacing

The West Coast and Alaska Tsunami Warning Center (WC/ATWC) in Alaska, and the Pacific Tsunami Warning Center (PTWC) in Hawaii monitor potential tsunamis, and warn affected populations of an impending tsunami. Table 5-1 summarizes approximate warning times associated with the distance between a tsunami-genic source and the site of interest. A far-source-generated tsunami originates from a source that is far away from the site, and could have 2 hours or more of advance warning time. A near-source-generated tsunami originates from a source that is close to the site,

and could have 30 minutes or less of advance warning time. Sites experiencing near-source-generated tsunamis will generally feel the effects of the triggering event (e.g., shaking caused by a near-source earthquake), and these effects will likely be the first warning of the impending tsunami. A mid-source-generated tsunami is one in which the source is somewhat close to the site of interest, but not close enough for the effects of the tsunami generating event to be felt at the site. Mid-source-generated tsunamis would be expected to have between 30 minutes and 2 hours of advance warning time.

Table 5-1 Tsunami Sources and Approximate Warning Times

Location of Source	*Approximate Warning Time (t)*
Far-source-generated tsunami	$t > 2$ hrs
Mid-source-generated tsunami	30 min $< t <$ 2 hrs
Near-source-generated tsunami	$t <$ 30 min

Recommended maximum spacing of vertical evacuation structures depends on warning time, ambulatory speed, and the surrounding population density.

Consideration must be given to the time it would take for designated occupants to reach a refuge. To determine the maximum spacing of tsunami vertical evacuation structures, the critical parameters are warning time and ambulatory capability of the surrounding community. Once maximum spacing is determined, size must be considered, and population becomes an important parameter. Sizing considerations could necessitate an adjustment in the number and spacing of vertical evacuation structures if it is not feasible to size the resulting structures large enough to accommodate the surrounding population at the maximum spacing. Sizing considerations are discussed in Section 5.2.

The average, healthy person can walk at approximately 4-mph. Portions of the population in a community, however, may have restricted ambulatory capability due to age, health, or disability. The average pace of a mobility-impaired population can be assumed to be about 2-mph.

Assuming a 2-hour warning time associated with far-source-generated tsunamis, vertical evacuation structures would need to be located a maximum of 4 miles from any given starting point. This would result in a maximum spacing of approximately 8 miles between structures. Similarly, assuming a 30 minute warning time, vertical evacuation structures would need to be located a maximum of 1 mile from any given starting point, or 2 miles between structures. Shorter warning times would require even closer spacing. Table 5-2 summarizes maximum spacing of vertical evacuation structures based on travel time associated with a mobility-impaired population.

Table 5-2 Maximum Spacing of Vertical Evacuation Structures Based on Travel Time

Warning Time	*Ambulatory Speed*	*Travel Distance*	*Maximum Spacing*
2 hrs	2 mph*	4 miles	8 miles
30 min	2 mph*	1 mile	2 miles
15 min	2 mph*	½ mile	1 mile

* Based on the average pace for a mobility-impaired population

5.1.2 Ingress and Vertical Circulation

Tsunami vertical evacuation structures should be spaced such that people will have adequate time not only to reach the structure, but to enter and move within the structure to areas of refuge that are located above the anticipated tsunami inundation elevation.

Increased travel times may need to be considered if obstructions exist, or could occur, along the travel or ingress route. Unstable or poorly secured structural or architectural elements that collapse in and around the entrance, or the presence of contents associated with the non-refuge uses of a structure, could potentially impede ingress. Allowance for parking at a vertical evacuation refuge may decrease travel time to the refuge, but could complicate access when the potential traffic jams are considered.

Stairs or elevators are traditional methods of ingress and vertical circulation in buildings, especially when designated users have impaired mobility. Ramps, such as the ones used in sporting venues, however, can be more effective for moving large numbers of people into and up to refuge areas in a structure. Estimates of travel time may need adjustment for different methods of vertical circulation. Disabled users may need to travel along a special route that accommodates wheelchairs, and those with special needs may require assistance from others to move within the structure.

When locating vertical evacuation structures, natural and learned behaviors of evacuees should be considered. Most coastal communities have educated their populations to "go to high ground" in the event of a tsunami warning. Also, a natural tendency for evacuees will be to migrate away from the shore. Vertical evacuation structures should therefore be located on the inland side of evacuation zones and should take advantage of naturally occurring topography that would tend to draw evacuees towards them. Figure 5-1 illustrates an arrangement of vertical evacuation structures in a community based on these principles.

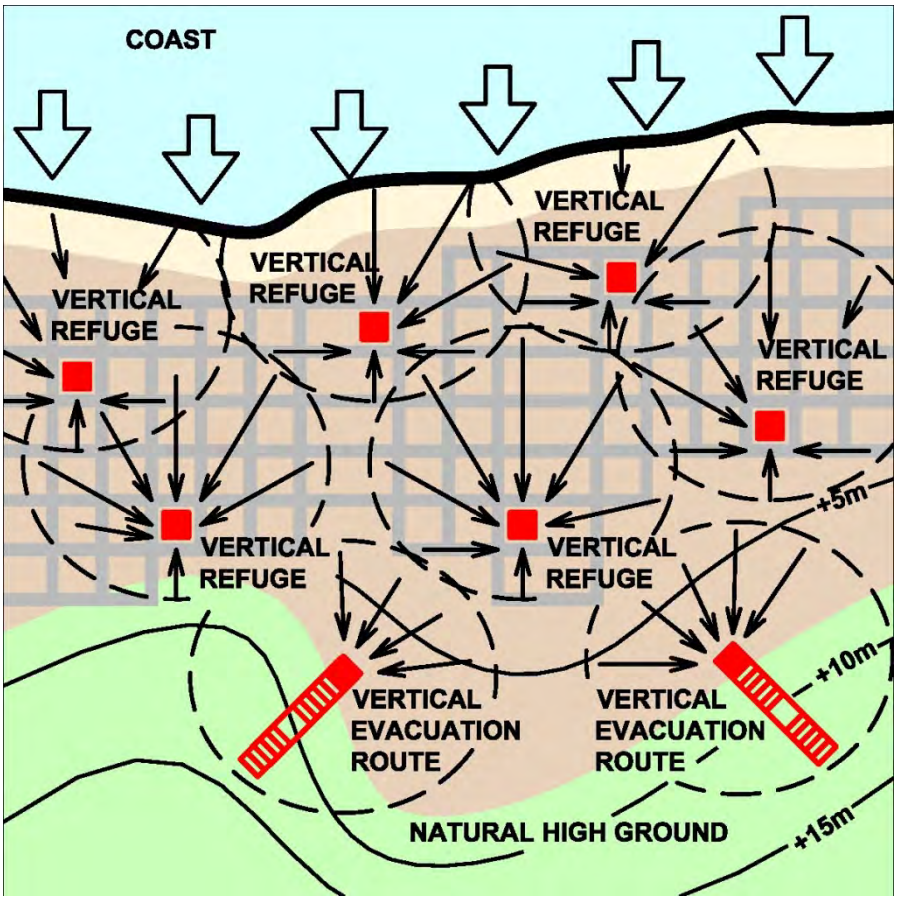


Figure 5-1 Vertical evacuation refuge locations considering travel distance, evacuation behavior, and naturally occurring high ground. Arrows show anticipated vertical evacuation routes.

5.1.3 Consideration of Site Hazards

Potential site hazards include breaking waves, sources of large waterborne debris, and sources of waterborne hazardous materials.

Special hazards in the vicinity of each site should be considered in locating vertical evacuation structures. Potential site hazards include breaking waves, sources of large waterborne debris, and sources of waterborne hazardous materials. When possible, vertical evacuation structures should be located away from potential hazards that could result in additional damage to the structure and reduced safety for the occupants. Due to limited availability of possible sites, and limitations on travel and mobility of the population in a community, some vertical evacuation structures may need to be located at sites that would be considered less than ideal. Figure 5-2 illustrates adjacent site hazards that could exist in a typical coastal community.

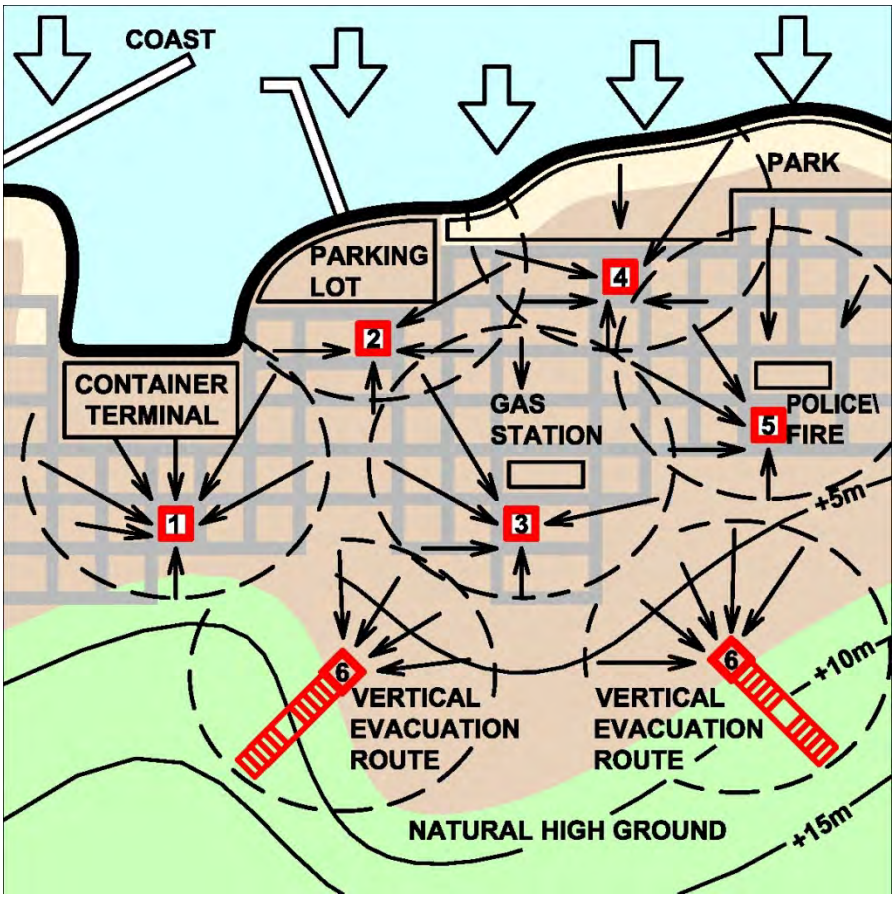


Figure 5-2 Site hazards adjacent to vertical evacuation structures (numbered locations). Arrows show anticipated vertical evacuation routes.

Wave breaking takes place where the water depth is sufficiently finite. In the design of usual coastal structures (e.g., breakwaters, seawalls, jetties), critical wave forces often result from breaking waves. In general, tsunamis break offshore. In the case of very steep terrain, however, they can break right at the shoreline, which is known as a collapsing breaker.

Forces from collapsing breakers can be extremely high and very uncertain. Location of vertical evacuation structures within the tsunami wave-breaking zone poses unknown additional risk to the structure. While the possibility of tsunami wave breaking at an on-shore location is not zero, it is considered to be very rare. For these reasons, recommended sites for vertical evacuation structures are located inland of the wave-breaking zone, and wave breaking forces are not considered in this document.

In Figure 5-2, vertical evacuation structures are located some distance inland from the shoreline. Structure No. 1 is located adjacent to a harbor and container terminal. Impact forces from ships, barges, boats, and other

waterborne debris have the potential to become very large. Locations with additional sources of large, possibly buoyant debris increase the chances of impact by one or more waterborne missiles, and increase the potential risk to the structure. If possible, it would be better if this structure was sited away from the harbor and container terminal. If there is no alternative location available to serve this area of the community, this structure would need to be designed for potential impact from the shipping containers and boats likely to be present during tsunami inundation.

Structure No. 2 is located off to the side of the harbor and adjacent to a parking lot. This structure would need to be designed for debris consistent with the use of the parking lot and surrounding areas, which could include cars, trucks, and recreational vehicles.

Structure No. 3 is immediately adjacent to a gas station. In past tsunamis, ignition of flammable chemicals or other floating debris has resulted in significant risk for fire in partially submerged structures. Depending on the potential for fuel leakage from this station in the event of a tsunami (or a preceding earthquake), this structure would need to be designed with fire resistive construction and additional fire protection.

Structure No. 4 is adjacent to a waterfront park facility. This location can be ideal, as the potential for waterborne debris can be relatively low. Possible hazards could include debris from park structures, naturally occurring driftwood, or larger logs from downed trees. This area has a higher potential for tourists and visitors unfamiliar with the area. It would require additional signage to inform park users what to do and where to go in the event of a tsunami warning.

Structure No. 5 is adjacent to an emergency response facility. Co-locating at such facilities can provide opportunities for direct supervision by law-enforcement and monitoring and support of refuge occupancies by other emergency response personnel.

At two locations, Structure No. 6 is intended to aid evacuees in taking advantage of naturally occurring high ground.

5.2 Sizing Considerations

Sizing of a vertical evacuation structure depends on the intended number of occupants, the type of occupancy, and the duration of occupancy. The number of occupants will depend on the surrounding population and the spacing and number of vertical evacuation structures located in the area.

Duration of occupancy will depend on the nature of the hazard and the intended function of the facility.

5.2.1 Services and Occupancy Duration

A vertical evacuation structure is typically intended to provide a temporary place of refuge during a tsunami event. While tsunamis are generally considered to be short-duration events (i.e., pre-event warning period and event lasting about 8 to 12 hours), tsunamis include several cycles of waves. The potential for abnormally high tides and coastal flooding can last as long as 24 hours.

A vertical evacuation structure must provide adequate services to evacuees for their intended length of stay. As a short term refuge, services can be minimal, including only limited space per occupant and basic sanitation needs. Additionally, a vertical evacuation structure could be used to provide accommodations and services for people whose homes have been damaged or destroyed. As a minimum, this would require an allowance for more space for occupants, supplies, and services. It could also include consideration of different post-event rescue and recovery activities, and evaluation of short- and long-term medical care needs. Guidance on basic community sheltering needs is not included in this document, but can be found in FEMA 361, *Design and Construction Guidance for Community Shelters* (FEMA, 2000a).

Choosing to design and construct a vertical evacuation structure primarily for short-term refuge, or to supply and manage it to house evacuees for longer periods of time, is an emergency management issue that must be decided by the state, municipality, local community, or private owner.

5.2.2 Square Footage Recommendations from Available Sheltering Guidelines

Square footage recommendations are available from a number of different sources, and vary depending on the type of hazard and the anticipated duration of occupancy. The longer the anticipated stay, the greater the minimum square footage recommended.

A shelter for mostly healthy, uninjured people for a short-term event would require the least square footage per occupant. A shelter intended to house sick or injured people, or to provide ongoing medical care, would require more square footage to accommodate beds and supplies. For longer duration stays, even more square footage is needed per occupant for minimum privacy and comfort requirements, and for building infrastructure, systems, and services needed when housing people on an extended basis.

Table 5-3, Table 5-4 and Table 5-5 summarize square footage recommendations contained in International Code Council/National Storm Shelter Association, ICC-500, *Standard on the Design and Construction of Storm Shelters* (ICC/NSSA, 2007), FEMA 361 *Design and Construction Guidance for Community Shelters* (FEMA, 2000a), and American Red Cross Publication No. 4496, *Standards for Hurricane Evacuation Shelter Selection* (ARC, 2002).

Table 5-3 Square Footage Recommendations – ICC-500 *Standard on the Design and Construction of Storm Shelters* (ICC/NSSA, 2007)

Hazard or Duration	*Minimum Required Usable Floor Area in Sq. Ft. per Occupant*
Tornado	
Standing or seated	5
Wheelchair	10
Bedridden	30
Hurricane	
Standing or seated	20
Wheelchair	20
Bedridden	40

Table 5-4 Square Footage Recommendations – FEMA 361 *Design and Construction Guidance for Community Shelters* (FEMA, 2000a)

Hazard or Duration	*Recommended Minimum Usable Floor Area in Sq. Ft. per Occupant*
Tornado	5
Hurricane	10

Table 5-5 Square Footage Recommendations – American Red Cross Publication No. 4496 (ARC, 2002)

Hazard or Duration	*Recommended Minimum Usable Floor Area in Sq. Ft. per Occupant*
Short-term stay (i.e., a few days)	20
Long-term stay (i.e., days to weeks)	40

The number of standing, seating, wheelchair, or bedridden spaces should be determined based on the specific occupancy needs of the facility under

consideration. When determining usable floor area, ICC-500 includes the following adjustments to gross floor area:

- Usable floor area is 50 percent of gross floor area in shelter areas with concentrated furnishings or fixed seating.
- Usable floor area is 65 percent of gross floor area in shelter areas with un-concentrated furnishings and without fixed seating.
- Usable floor area is 85 percent of gross floor area in shelter areas with open plan furnishings and without fixed seating.

5.2.3 Recommended Minimum Square Footage for Short-Term Refuge from Tsunamis

For short-term refuge in a tsunami vertical evacuation structure, the duration of occupancy should be expected to last between 8 to 12 hours, as a minimum. Because tsunami events can include several cycles of waves, there are recommendations that suggest evacuees should remain in a tsunami refuge until the second high tide after the first tsunami wave, which could occur up to 24 hours later.

Recommended minimum square footage is 10 square feet per occupant.

Based on square footage recommendations employed in the design of shelters for other hazards, the recommended minimum square footage per occupant for a tsunami refuge is 10 square feet per person. It is anticipated that this density will allow evacuees room to sit down without feeling overly crowded for a relatively short period of time, but would not be considered appropriate for longer stays that included sleeping arrangements. This number should be adjusted up or down depending on the specific occupancy needs of the refuge under consideration.

5.3 Elevation Considerations

In order to serve effectively as a vertical evacuation structure, it is essential that the area of refuge be located well above the maximum tsunami inundation level anticipated at the site. Determination of a suitable elevation for tsunami refuge must take into account the uncertainty inherent in estimation of the tsunami runup elevation, possible splash-up during impact of tsunami waves, and the anxiety level of evacuees seeking refuge in the structure. Unfortunately a number of designated evacuation structures in Japan were inundated during the Tohoku tsunami, leading to loss of life of many of the refugees. To account for this uncertainty, the magnitude of tsunami force effects is determined assuming a maximum tsunami runup elevation that is 30% higher than values predicted by numerical simulation modeling or obtained from tsunami inundation maps. Because of the high

Recommended minimum refuge elevation is the maximum anticipated tsunami runup elevation, plus 30%, plus 10 feet (3 meters).

consequence of potential inundation of the tsunami refuge area, it is recommended that the elevation of tsunami refuge areas in vertical evacuation structures include an additional allowance for freeboard above this elevation.

The recommended minimum freeboard is one story height, or 10 feet (3 meters) above the tsunami runup elevation used in tsunami force calculations. The recommended minimum elevation for a tsunami refuge area is, therefore, the maximum tsunami runup elevation anticipated at the site, plus 30%, plus 10 feet (3 meters). This should be treated as an absolute minimum, with additional conservatism strongly encouraged.

5.4 Size of Vertical Evacuation Structures

Given the number and spacing of vertical evacuation structures, and the population in a given community, the minimum size can be determined based on square footage recommendations for the intended duration and type of occupancy. Consideration of other functional needs, such as restrooms, supplies, communications, and emergency power, should be added to the overall size of the structure.

Given the maximum tsunami runup elevation anticipated at the site, the minimum elevation of the area of refuge within a vertical evacuation structure can be determined based on minimum freeboard recommendations.

Chapter 6

Load Determination and Structural Design Criteria

This chapter summarizes current code provisions as they may relate to tsunami load effects, describes intended performance objectives for vertical evacuation structures, specifies equations for estimating tsunami forces, and provides guidance on how tsunami forces should be combined with other effects.

Very little guidance is provided in currently available structural design codes, standards, and guidelines on loads induced by tsunami inundation.

Established design information focuses primarily on loads due to rising water and wave action associated with riverine flooding and storm surge.

6.1 Currently Available Structural Design Criteria

Very little guidance is provided in currently available structural design codes, standards, and guidelines on loads induced by tsunami inundation. Established design information focuses primarily on loads due to rising water and wave action associated with riverine flooding and storm surge. While little specific guidance was provided prior to this publication, the presumption heretofore had been that available flood design standards were to be adapted for designing for tsunami load effects. Therefore, it is important to understand those standards and how they differ from tsunami conditions.

6.1.1 Current U.S. Codes, Standards, and Guidelines

International Building Code. The International Code Council *International Building Code* (ICC, 2012) Section 1612 Flood Loads, Section 1804 Excavation, Grading and Fill, and Appendix G Flood Resistant Construction provides information on flood design and flood-resistant construction including by reference to ASCE/SEI Standard 24-05, *Flood Resistant Design and Construction* (ASCE 24, 2006a). Appendix M: Tsunami Generated Flood Hazard, provides tsunami regulatory criteria for those communities that have a recognized tsunami hazard and have developed and adopted a map of their Tsunami Hazard Zone, and is focused on keeping critical and high risk structures out of the tsunami inundation zone. However, buildings are permitted within the Tsunami Hazard Zone if designed as a Vertical Evacuation Refuge complying with the FEMA P-646 *Guidelines* or if designed to resist without collapse the hydrostatic, hydrodynamic, debris accumulation and impact, and scour effects of the Maximum Considered

Tsunami. Appendices G and M are non-mandatory unless adopted by a local jurisdiction having authority.

ASCE/SEI Standard 24-05. The American Society of Civil Engineers/Structural Engineering Institute (ASCE/SEI) Standard 24-05 *Flood Resistant Design and Construction* (ASCE, 2006a) provides minimum requirements for flood-resistant design and construction of structures located in flood-hazard areas. Topics include basic requirements for flood-hazard areas, high-risk flood-hazard areas, coastal high-hazard areas, and coastal A zones. This standard was formulated for compliance with FEMA National Flood Insurance Program (NFIP) floodplain management requirements.

ASCE/SEI Standard 7-10. ASCE/SEI Standard 7-10 *Minimum Design Loads for Buildings and Other Structures* (ASCE, 2010) provides expressions for forces associated with flood and wave loads on specific types of structural components. Chapter 5 of this standard, Flood Loads, covers important definitions that relate to flooding and coastal high-hazard areas related to tides, storm surges, and breaking waves. (In 2016 it is anticipated that a new Chapter 6, Tsunami Loads and Effects, will be added.)

FEMA P-55 Coastal Construction Manual. The fourth edition of the FEMA P-55 *Coastal Construction Manual* (FEMA, 2011) includes discussion of coastal seismic and tsunami loads. This *Manual* was developed to provide design and construction guidance for low-rise (less than three stories), one- and two-family residential structures built in coastal areas throughout the United States. The *Coastal Construction Manual* addresses seismic loads for coastal structures, and contains expressions for flood loads, wave loads, and load combinations for specific types of structural components.

The *Manual* also provides general information on tsunami hazard. Section 3.3.3 states that:

> "Tsunamis are long-period water waves generated by undersea shallow-focus earthquakes or by undersea crustal displacements (subduction of tectonic plates), landslides, or volcanic activity. Tsunamis can travel great distances, undetected in deep water, but shoaling rapidly in coastal waters and producing a series of large waves capable of destroying harbor facilities, shore protection structures, and upland buildings … Coastal construction in tsunami hazard zones must consider the effects of tsunami runup, flooding, erosion, and debris loads. Designers should also be aware that the "rundown" or return of water to the sea can also damage the landward sides of structures that withstood the initial runup."

The *Manual* also notes that tsunami effects at a particular site will be determined by the following four basic factors:

- the magnitude of the earthquake or triggering event,
- the location of the triggering event,
- the configuration of the continental shelf and shoreline, and
- the upland topography.

This *Manual* contains a warning statement in Chapter 8 that "This Manual does not provide guidance for estimating flood velocities during tsunamis. The issue is highly complex and site-specific. Designers should look for model results from tsunami inundation or evacuation studies."

With regard to designing to resist tsunami loads, Section 8.6 of the *Manual* states that:

> "Tsunami loads on residential buildings may be calculated in the same fashion as other flood loads; the physical processes are the same, but the scale of the flood loads is substantially different in that the wavelengths and runup elevations of tsunamis are much greater than those of waves caused by tropical or extratropical cyclones … When the tsunami forms a borelike wave, the effect is a surge of water to the shore. When this occurs, the expected flood velocities are substantially higher than in non-tsunami conditions … and if realized at the greater water depths, would cause substantial damage to all buildings in the path of the tsunami."

Although authors of the *Coastal Construction Manual* conclude that it is generally not feasible or practical to design normal structures to withstand tsunami loads, it should be noted that this study was for conventional single family residential construction, and did not take into account the possibility of special design and construction details that would be possible for vertical evacuation structures and other larger buildings.

City and County of Honolulu Building Code. The *City and County of Honolulu Building Code* (CCH, 2007), Chapter 16, Article 11, provides specific guidance for "*structural design of buildings and structures subject to tsunamis*" in Section 16-11.5(f). The loading requirements in this section are based on a January 1980 Dames & Moore report, *Design and Construction Standards for Residential Construction in Tsunami-Prone Areas in Hawaii,* specifically *Appendix A, Proposed Building Code Amendments.* Drag forces were based on a non-bore velocity of flow in feet per second roughly estimated as equal in magnitude to the depth in feet of water at the structure (inconsistent with a Froude number assumption that would relate to the

square root of the depth). The report states that "The adequacy of this approach ... has not been satisfactorily examined." However, at the same time prescriptive forces on walls were based on a bore flow velocity of $2\sqrt{gh}$. Rough estimates are also given for anticipated scour around piles and piers based on distance from the shoreline and the soil type at the building site. However, the basis for these scour values is not documented. These provisions have not been updated since they were first adopted in the 1980's, and are now largely archaic and primarily for historical reference.

6.1.2 Summary of Current Design Requirements

Coastal areas that are subject to high-velocity wave action from storms or seismic sources are designated Coastal High Hazard V-Zones (ASCE, 2010). In ASCE 7-2010 Chapter 5, Flood Loads, areas inland of Coastal V-Zones that are subject to smaller waves caused by storm surges, riverine flooding, seiches or tsunamis are designated Coastal A-Zones (ASCE, 2010). However, the Coastal Construction Manual defines the Coastal V-Zone as "an area subject to high-velocity wave action from storms or tsunamis", and the Coastal A-Zone as an area "in which the principal source of flooding is coastal storms, and where the potential base flood wave height is between 1.5 and 3.0 feet."

In design for coastal flooding due to storm surge or tsunamis, buildings or structures are proportioned to resist the effects of coastal floodwaters. Design and construction must be adequate to resist the anticipated flood depths, pressures, velocities, impact, uplift forces, and other factors associated with flooding, as defined by the code.

Habitable space in building structures must be elevated above the regulatory coastal storm flood elevation by such means as posts, piles, piers, or shear walls parallel to the expected direction of flow. Spaces below the base flood elevation must be free from obstruction. Walls and partitions in a coastal high-hazard area are required to break away so as not to induce excessive loads on the structural frame.

The effects of long-term erosion, storm-induced erosion, and local scour are to be included in the design of foundations of buildings or other structures in coastal high-hazard areas. Foundation embedment must be far enough below the depth of potential scour to provide adequate support for the structure. Scour of soil from around individual piles and piers must be provided for in the design. Shallow foundation types are not permitted in V-Zones unless the natural supporting soils are protected by scour protection, but are permitted in A-Zones subject to stability of the soil and resistance to scour.

The main building structure must be adequately anchored and connected to the elevating substructure system to resist lateral, uplift, and downward forces.

6.1.3 Limitations in Available Flood Design Criteria Relative to Tsunami Loading

Although many of the hydrostatic and hydrodynamic loading expressions in the above-referenced codes, standards and guidelines are well-established, there are significant differences between tsunami inundation and riverine or storm surge flooding. For a typical tsunami, the water surface fluctuates near the shore with amplitude that may range from several meters to over 10 meters during a period of a few minutes to tens of minutes. A major difference between tsunamis and other coastal flooding is increased flow velocity for tsunamis, which results in significant increases in velocity-related loads on structural components. Application of existing loading expressions to tsunami loading conditions requires an estimate of the tsunami flood depth and velocity, neither of which is provided with accuracy by the above referenced information on flood and tsunami design.

Although many of the hydrostatic and hydrodynamic loading expressions in currently available codes, standards and guidelines are well-established, there are significant differences between tsunami inundation and riverine or storm surge flooding.

Although impact of floating debris is required to be considered by the codes discussed in this chapter, impact force produced by a change in momentum is dependent on estimates of the debris mass, velocity, and the time taken for the mass to decelerate. No accommodation is made for added mass of the water behind the debris, or the potential for damming if debris is blocked by structural components. More significant forms of debris, such as barges, fishing boats, and empty storage tanks may need to be considered for tsunamis, depending on the location of the building under consideration. The size, mass, and stiffness of this type of debris are not considered in currently available criteria.

No consideration is given to upward loads on the underside of structures or components that are submerged by the flood or tsunami flow. These vertical hydrodynamic loads, different from buoyancy effects, are considered by the offshore industry in design of platforms and structural members that may be submerged by large waves.

There are two primary scour mechanisms that occur during a tsunami event. Shear-induced scour is similar to that observed during storm surge flooding, and consists of soil transport due to the flow velocity. Liquefaction-induced scour results from rapid drawdown as the water recedes. Without sufficient time to dissipate, pore pressure causes liquefaction of the soil resulting in substantially greater scour than would otherwise occur. Although the codes

discussed in this chapter require consideration of scour, little guidance (other than rough estimates) is given as to the potential extent of scour.

6.2 Performance Objectives

While specific performance objectives for various forms of rare loading can vary, acceptable structural performance generally follows a trend corresponding to:

- little or no damage for small, more frequently occurring events;
- moderate damage for medium-size, less frequent events; and
- significant damage, but no collapse for very large, rare events.

In the case of earthquake hazards, model building codes, such as the *International Building Code*, implicitly assign seismic performance objectives to buildings based on their inherent risk to human life (e.g., very large occupancies) or their importance after an earthquake (e.g., emergency operation centers or hospitals). Buildings and other structures are classified into Risk Categories I through IV, in order of increasing risk to human life or importance, and code prescriptive design criteria are correspondingly increased, with the intention of providing improved performance. For Risk Category IV, design rules are intended to result in a high probability of buildings remaining functional after moderate shaking, and experiencing considerably less damage than normal buildings in very rare shaking.

Currently available performance-based seismic design procedures are intended to explicitly evaluate and predict performance, instead of relying on the presumed performance associated with prescriptive design rules. However, performance-based design is an emerging technology and the targeted performance cannot be delivered with 100% certainty. The current standard-of-practice for performance-based seismic design contained in ASCE/SEI 41-06 *Seismic Rehabilitation of Existing Buildings* (ASCE, 2006b) defines discrete performance levels with names intended to connote the expected condition of the building: Collapse, Collapse Prevention, Life Safety, Immediate Occupancy, and Operational. Seismic performance objectives are defined by linking one of these building performance levels to an earthquake hazard level that is related to the recurrence interval (return period) and the intensity of ground shaking, as shown in Figure 6-1.

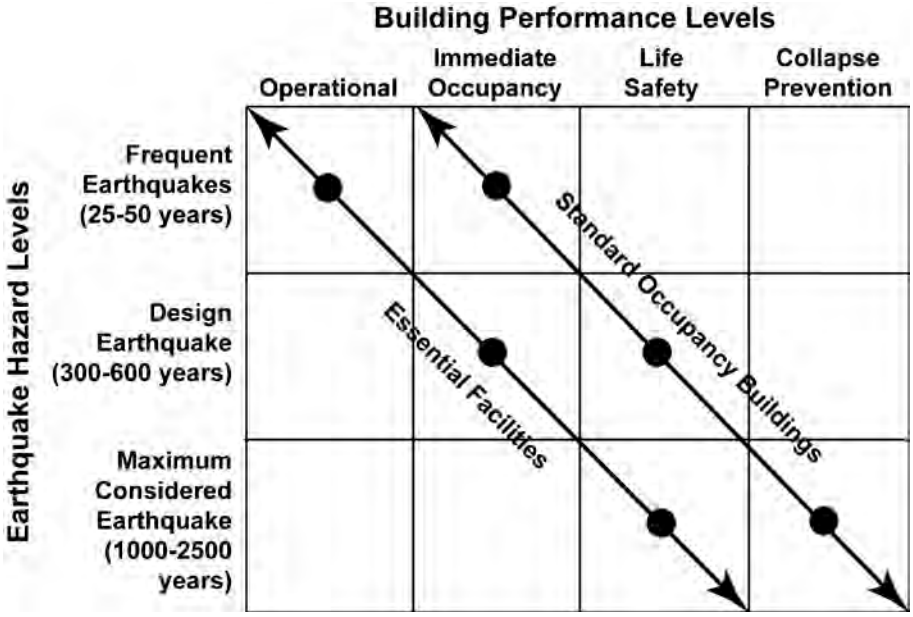


Figure 6-1 Seismic performance objectives linking building performance levels to earthquake hazard levels (adapted from SEAOC, 1995).

When determining performance objectives for natural hazards, the most difficult issue is deciding how rare (or intense) the design event should be. For seismic design in the United States, this issue has been resolved through the adoption of a national earthquake hazard map defining the risk-target Maximum Considered Earthquake (MCE) and the intensity of shaking associated with such an event (ASCE, 2010).

6.2.1 Tsunami Performance Objective

In this document, the design tsunami event is termed the Maximum Considered Tsunami (MCT). Unfortunately, there are no standardized national maps available for defining this hazard. In addition, due to the complexity of the tsunami hazard, which must consider near and distant tsunami-genic sources and highly uncertain relationships between earthquake events and subsequent tsunami, as of 2011 no firm policy has been established in the code defining a methodology for setting a Maximum Considered Tsunami at a consistent hazard level. Current methods for tsunami hazard assessment are described in Chapter 3.

The Tsunami Performance Objective includes the potential for significant damage while maintaining a reliable and stable refuge when subjected to the Maximum Considered Tsunami. Most structures would be expected to be repairable, although the economic viability of repair will be uncertain.

Vertical evacuation structures designed in accordance with the guidance presented in this document would be expected to provide a stable refuge when subjected to a design tsunami event consistent with the Maximum Considered Tsunami identified for the local area.

In general, the Maximum Considered Tsunami will be a rare, but realistic event with large potential consequences, generally to be taken as having a

collapse prevention design equivalent of a 2% probability of being exceeded in a 50-year period or a 2500 year average return period (similar to the probability level of seismic criteria). Consistent with the general trend of acceptable performance for "Maximum Considered" loadings, the performance of vertical evacuation structures in this event would include the potential for significant damage while maintaining a reliable and stable refuge above the inundation height, although the economics of repair versus replacement will be uncertain, depending on the specifics of the situation including the magnitude of the actual event, interaction with the local bathymetry, and the design and construction of the facility.

6.2.2 Seismic Performance Objectives

Seismic Performance Objectives are consistent with the code-defined performance of essential facilities such as hospitals, police and fire stations, and emergency operation centers.

The performance objective for vertical evacuation structures subjected to seismic hazards should be consistent with that of code-defined essential facilities such as hospitals, police and fire stations, and emergency operation centers. Following the prescriptive approach in the *International Building Code*, vertical evacuation structures are assigned to Risk Category IV, triggering design requirements that provide enhanced performance relative to typical buildings for normal occupancies.

In the specific case of earthquakes generating a near-source tsunami, design for enhanced performance is necessary to assure that the structure is still usable for a tsunami following a local seismic event. To obtain a higher level of confidence that a vertical evacuation structure will achieve enhanced seismic performance, the design developed by prescriptive code provisions can be evaluated using currently available performance-based seismic design techniques and verification analyses. Utilizing the approach in ASCE/SEI 41-06, the performance objective for code-defined essential facilities should be at least Immediate Occupancy performance for the Design Basis Earthquake (DBE) and Life Safety performance for the Maximum Considered Earthquake (MCE).

6.3 Earthquake Loading

The recommended basis for seismic design of vertical evacuation structures is the *International Building Code*, which references ASCE/SEI 7-10 *Minimum Design Loads for Buildings and Other Structures* for its seismic requirements. These requirements are based on the *NEHRP Recommended Provisions for Seismic Regulations for New Buildings and Other Structures* (FEMA, 2004a) and additional information provided in the *Commentary* (FEMA, 2004b). Vertical evacuation structures should be designed using rules for Risk Category IV buildings.

The recommended basis for seismic evaluation and rehabilitation of existing buildings that are being considered for use as vertical evacuation structures is the SEI/ASCE Standard 31-03 *Seismic Evaluation of Existing Buildings* (ASCE, 2003b), using the Immediate Occupancy performance objective, and ASCE/SEI Standard 41-06 *Seismic Rehabilitation of Existing Buildings*, using the performance objectives specified in Section 6.2.2.

6.3.1 Near-Source-Generated Tsunamis

A vertical evacuation structure located in a region susceptible to near-source-generated tsunamis is likely to experience strong ground shaking immediately prior to the tsunami. As a properly designed essential facility, it is expected that sufficient reserve capacity will be provided in the structure to resist the subsequent tsunami loading effects. The reserve capacity of the structure, which will be some fraction of the original, needs to be evaluated. It is recommended that the condition of the structure after the Design Basis Earthquake (DBE) be used to determine the adequacy for tsunami loading. If inadequate, the resulting design would then need to be modified as necessary to address tsunami effects. For areas that are subject to near-source-generated tsunamis, this sequential loading condition will clearly control the design of the structure. To help ensure adequate strength and ductility in the structure for resisting tsunami load effects, Seismic Design Category D, as defined in ASCE/SEI 7-10, should be assigned to the structure, as a minimum.

A vertical evacuation structure located in a region susceptible to near-source-generated tsunamis is likely to experience strong ground shaking immediately prior to the tsunami.

A properly designed essential facility is also expected to have improved performance of non-structural components including ceilings, walls, light fixtures, fire sprinklers, and other building systems. For evacuees to feel comfortable entering a vertical evacuation structure following an earthquake, and remaining in the structure during potential aftershocks, it is important that visible damage to both structural and non-structural components be limited. Particular attention should be focused on non-structural components in the stairwells, ramps, and entrances that provide access and vertical circulation within the structure.

6.3.2 Far-Source-Generated Tsunamis

Although a vertical evacuation structure is not likely to experience earthquake shaking directly associated with a far-source tsunami, seismic design must be independently included as dictated by the seismic hazard that is present at the site. Even in regions of low seismicity, however, it is recommended that Seismic Design Category D be assigned to the structure,

as a minimum, to help ensure adequate continuity, strength, and ductility for resisting tsunami load effects.

6.4 Wind Loading

The recommended basis for wind design of a vertical evacuation structure is the *International Building Code*, which references ASCE/SEI 7-10 *Minimum Design Loads for Buildings and Other Structures* for the majority of its wind requirements. In many locations affected by tsunami risk, earthquake loading will likely govern over wind loading, but this is not necessarily true for all regions.

At locations where wind loading controls the design, the use of special seismic detailing for structural components should be considered. It is recommended that Seismic Design Category D be assigned to the structure, as a minimum, to help ensure adequate strength and ductility for resisting tsunami load effects.

6.5 Tsunami Loading

Tsunami Load Effects include:
(1) hydrostatic forces;
(2) buoyant forces;
(3) hydrodynamic forces;
(4) impulsive forces;
(5) debris impact forces;
(6) debris damming forces;
(7) uplift forces; and
(8) additional gravity loads from retained water on elevated floors.

The following tsunami load effects should be considered for the design of vertical evacuation structures: (1) hydrostatic forces; (2) buoyant forces; (3) hydrodynamic forces; (4) impulsive forces; (5) debris impact forces; (6) debris damming forces; (7) uplift forces; and (8) additional gravity loads from retained water on elevated floors.

In this document, wave-breaking forces are not considered in the design of vertical evacuation structures. In general, tsunamis break offshore, and vertical evacuation structures should be located some distance inland from the shoreline. The term 'wave-breaking' is defined here as a plunging-type breaker in which the entire wave front overturns. When waves break in a plunging mode, the wave front becomes almost vertical, generating an extremely high pressure over an extremely short duration. Once a tsunami wave has broken, it can be considered as a bore because of its very long wavelength. Further justification for not considering wave-breaking forces can be found in Yeh (2008).

Wave-breaking forces could be critical for vertical evacuation structures located in the wave-breaking zone, which is beyond the scope of this document. If it is determined that a structure must be located in the wave-breaking zone, ASCE/SEI 7-10 *Minimum Design Loads for Buildings and Other Structures* and the *Coastal Engineering Manual*, EM 1110-2-1100, (U.S. Army Coastal Engineering Research Center, 2008) should be consulted for additional guidance on wave-breaking forces.

6.5.1 Key Assumptions for Estimating Tsunami Load Effects

Tsunami load effects are determined using the following key assumptions:

- Tsunami flows consist of a mixture of sediment and seawater. Most suspended sediment transport flows do not exceed 5% sediment concentration. Based on an assumption of vertically averaged sediment-volume concentration of 5% in seawater, the fluid density of tsunami flow should be taken as 1.1 times the density of freshwater, or ρ_s = 1,100 kg/m^3 = 2.13 slugs/ft^3.
- Tsunami flow depths vary significantly depending on the three-dimensional bathymetry and topography at the location under consideration. Figure 6-2 shows three possible scenarios where topography could affect the relationship between maximum tsunami elevation, T_E, at a particular location and the ultimate inland runup elevation, R. For the loading expressions presented in this chapter, it is assumed that Figure 6-2b applies, that is $T_E = R$. These expressions may be adjusted if numerical simulations of tsunami inundation provide more appropriate estimates of T_E at the location being considered.
- There is significant variability in local tsunami runup heights, based on local bathymetry and topographic effects, and uncertainty in numerical simulations of tsunami inundation. Based on empirical judgment from past tsunami survey data, it is recommended that the design runup elevation, R, be taken as 1.3 times the predicted maximum runup elevation, R*, to envelope the potential variability in the estimates of modeling. The inundation elevation from the runup point back towards the shoreline would then be scaled by the same factor. Figure 6-3 shows a typical numerical prediction (Yamazaki et al., 2011) made for the 2009 Samoa Tsunami, which demonstrates that the 1.3 safety factor for uncertainty is realistic.

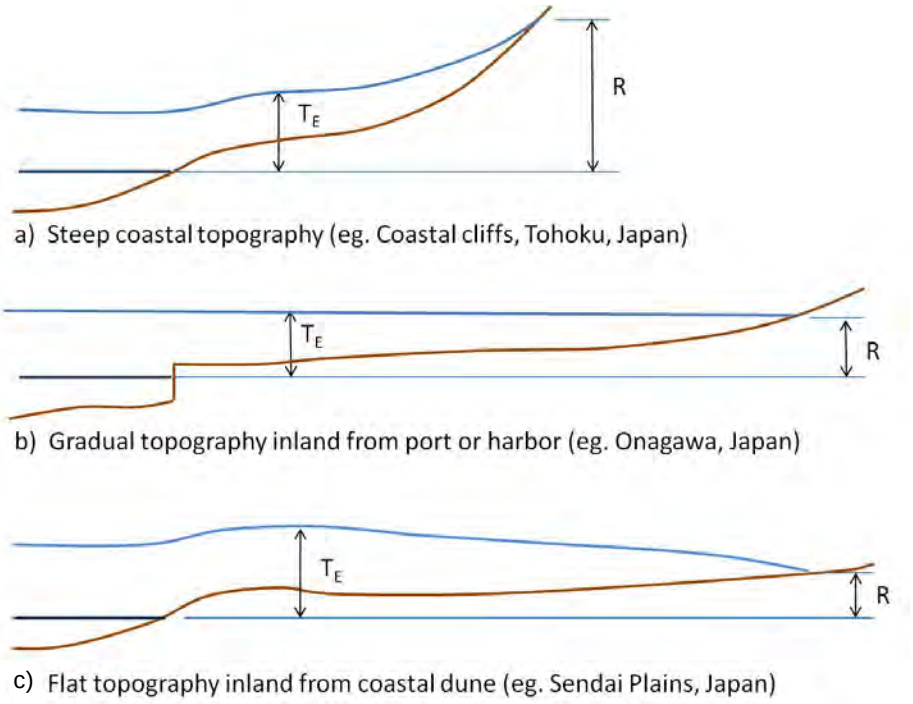


Figure 6-2 Three types of coastal inundation where the tsunami elevation (T_E) at a site of interest could be less than, equal to, or greater than the ultimate inland runup elevation (R)

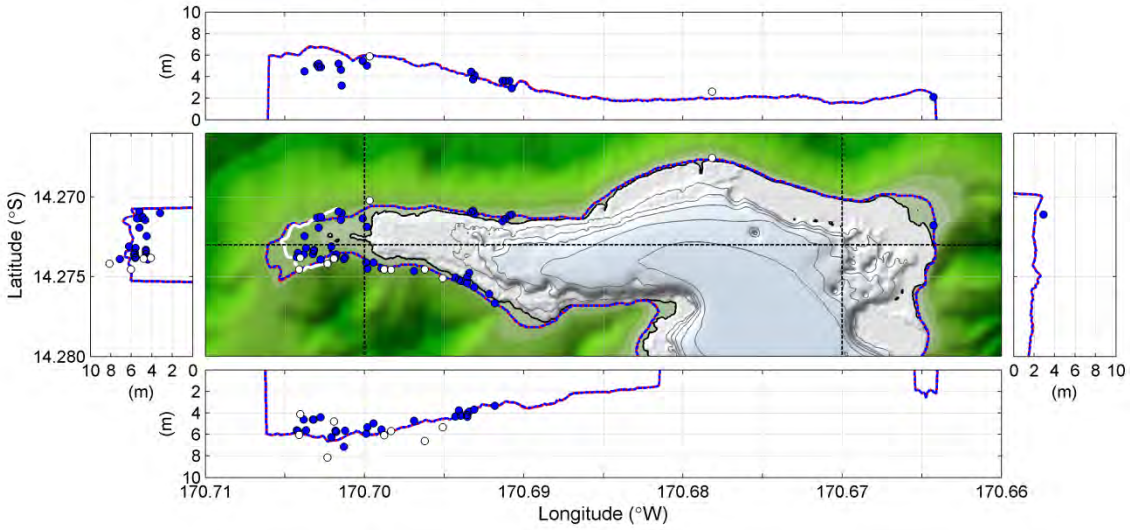


Figure 6-3 Comparison between numerical modeling (blue line) and field measurement of run-up (white dots) and flow elevations (blue dots) at Pago Pago Harbor, American Samoa (Yamazaki et al, 2011).

6.5.2 Hydrostatic Forces

Hydrostatic forces occur when standing or slowly moving water encounters a structure or structural component. This force always acts perpendicular to the surface of the component of interest. It is caused by an imbalance of

pressure due to a differential water depth on opposite sides of a structure or component. Hydrostatic forces may not be relevant to a structure with a finite (i.e., relatively short) breadth, around which the water can quickly flow and fill in on all sides. Hydrostatic forces are usually important for long structures such as sea walls and dikes, or for evaluation of an individual wall panel where the water level on one side differs substantially from the water level on the other side.

Hydrostatic and buoyant forces must be computed when the ground floor of a building is watertight, or is sufficiently insulated and airtight to prevent or delay the intrusion of water. In this situation, the hydrostatic force should be evaluated for individual wall panels. The horizontal hydrostatic force on a wall panel can be computed using Equation 6-1:

$$F_h = p_c A_w = \frac{1}{2}\rho_s g b h_{\max}^2, \tag{6-1}$$

where p_c is the hydrostatic pressure, A_w is the wetted area of the panel, ρ_s is the fluid density including sediment (1100 kg/m^3 = 2.13 slugs/ft^3), g is the gravitational acceleration, b is the breadth (width) of the wall, and h_{max} is the maximum water height above the base of the wall at the structure location. If the wall panel with height h_w is fully submerged, then the horizontal hydrostatic force can be written as Equation 6-2:

$$F_h = p_c A_w = \rho_s g\left(h_{\max} - \frac{h_w}{2}\right) b\, h_w \tag{6-2}$$

where $h_{\max}$ is the vertical difference between the design tsunami elevation R and the base elevation of the wall at the structure, z_w, as shown in Equation 6-3:

$$h_{\max} = 1.3R^* - z_w = R - z_w \tag{6-3}$$

where R^* is the estimated maximum inundation elevation at the structure from a detailed numerical simulation model, or the runup elevation at maximum horizontal penetration of the tsunami from available tsunami inundation maps. The design runup elevation, R, is taken as 1.3 times the predicted maximum runup elevation, R^*. The moment about the base of the wall can be evaluated using the line of action of the hydrostatic force resultant, as shown in Figure 6-4.

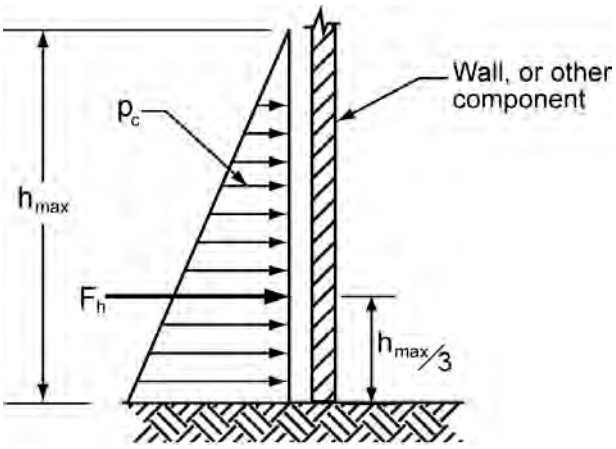


Figure 6-4 Hydrostatic force distribution and location of resultant.

6.5.3 Buoyant Forces

Buoyant or vertical hydrostatic forces will act vertically through the centroid of the displaced volume on a structure or structural component subjected to partial or total submergence. The total buoyant force equals the weight of water displaced. Buoyant forces on components must be resisted by the weight of the component and any opposing forces resisting flotation. Buoyant forces are a concern for structures that have little resistance to upward forces (e.g., light wood frame buildings, basements, empty tanks located above or below ground, swimming pools, components designed considering only gravity loads).

For a watertight structure, the total buoyant force is given by Equation 6-4:

$$F_b = \rho_s g V \tag{6-4}$$

where ρ_s is the fluid density including sediment (1100 kg/m^3 = 2.13 slugs/ft^3), and V is the volume of water displaced by the building, i.e., the volume below the level of h_{max} as determined by Equation 6-3. Buoyant forces on an overall building are shown in Figure 6-5. If there is insufficient building weight to resist buoyant forces, tension piles may be used to increase the resistance to flotation, but reduction in pile side friction due to anticipated scour around the tops of the piles must be considered.

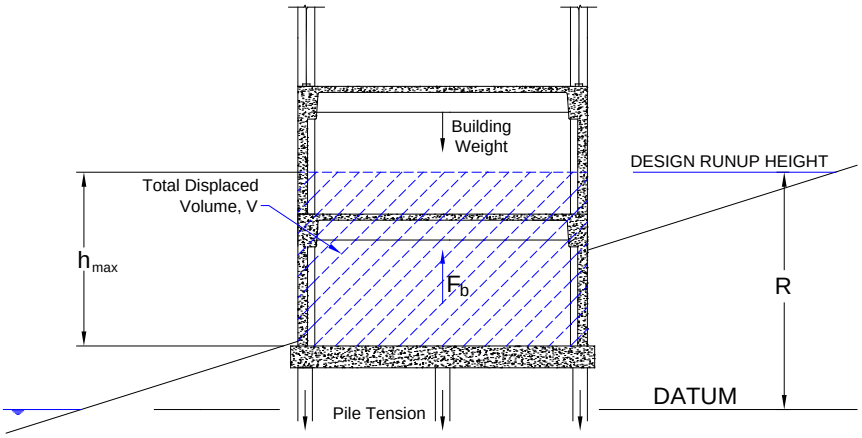


Figure 6-5 Buoyant forces on an overall building with watertight lower levels.

6.5.4 Hydrodynamic Forces

When water flows around a structure, hydrodynamic forces are applied to the structure as a whole and to individual structural components. These forces are induced by the flow of water moving at moderate to high velocity, and are a function of fluid density, flow velocity and structure geometry. Also known as drag forces, they are a combination of the lateral forces caused by the pressure forces from the moving mass of water and the friction forces generated as the water flows around the structure or component.

Hydrodynamic forces can be computed using Equation 6-5:

$$F_d = \frac{1}{2}\rho_s C_d B(hu^2)_{\max} \qquad (6\text{-}5)$$

where ρ_s is the fluid density including sediment (1100 kg/m^3 = 2.13 slugs/ft^3), C_d is the drag coefficient, B is the breadth of the structure in the plane normal to the direction of flow (i.e. the breadth in the direction parallel to the shore), h is flow depth, and u is flow velocity at the location of the structure. For forces on components, B is taken as the width of the component. The drag coefficient may be conservatively taken as $C_d = 2.0$; the actual value is shape-, orientation-, and size-dependent. The resultant hydrodynamic force is applied approximately at the centroid of the wetted surface of the component, as shown in Figure 6-6.

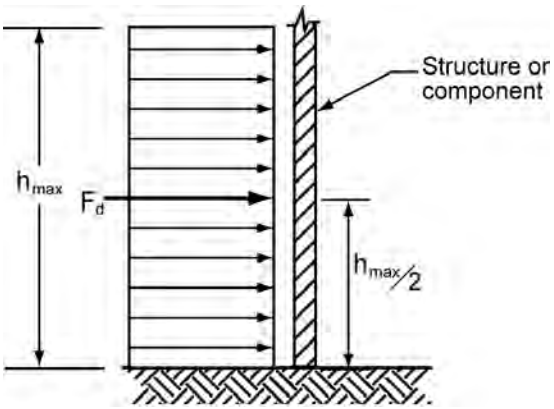


Figure 6-6 Hydrodynamic force distribution and location of resultant.

The combination hu^2 represents the momentum flux per unit mass per unit width. Note that $(hu^2)_{max}$ does not equal $h_{max}\, u^2_{max}$. The maximum flow depth, h_{max}, and maximum flow velocity, u_{max}, at a particular site may not occur at the same time. The hydrodynamic forces should be based on the parameter $(hu^2)_{max}$, which is the maximum momentum flux per unit mass per unit width occurring at the site at any time during the tsunami.

The maximum value of (hu^2) can be obtained by running a detailed numerical simulation model or acquiring existing simulation data. The numerical model in the runup zone must be run with a very fine grid size to ensure adequate accuracy in the prediction of hu^2.

When numerical simulation data are not available, the value $(hu^2)_{max}$ can be roughly estimated based on information in the inundation map, using Equation 6-6:

$$\left(hu^2\right)_{max} = gR^2\left(0.125 - 0.235\frac{z}{R} + 0.11\left(\frac{z}{R}\right)^2\right) \tag{6-6}$$

where g is the acceleration due to gravity, R is the design runup elevation taken as 1.3 times the maximum runup elevation, R^*, and z is the ground elevation at the base of the structure. To use this formula, the sea level datum must be consistent with that used in the inundation maps.

The basis of Equation 6-6 is described in Appendix E. Although this classical analytical solution is based on one-dimensional nonlinear shallow-water theory for a uniformly sloping beach, with no lateral topographical variation and no friction, the maximum value of (hu^2) obtained from Equation 6-6 can be used for: (1) preliminary design; (2) approximate design in the absence of other modeling information; and (3) to evaluate the reasonableness of numerical simulation results.

R^* and z can be obtained from tsunami inundation maps. Because of uncertainties in modeling tsunami inundation, it is recommended that numerically predicted values of (hu^2) should be compared with the values computed using Equation 6-6 to determine reasonableness.

6.5.5 Impulsive Forces

Impulsive forces are caused by the leading edge of a surge of water impacting a structure. Ramsden (1993) performed comprehensive experiments on impulsive forces. Laboratory data show no significant initial impact force (impulse force) in dry-bed surges, but an "overshoot" in force was observed in bores that occur when the site is initially flooded. The maximum overshoot is approximately 1.5 times the subsequent hydrodynamic force, consistent with some, but not all, of the independent laboratory data obtained by Arnason (2005). Further analysis of the conditions for the occurrence of this effect and high-speed video of similar test cases suggests it occurs when the surge depth to object width ratio is small so that a transient amount of additional "ponded" water depth accumulates against the forward side of the object before being eventually relieved by flowing around the sides. Since impact momentum increases with the sudden slam of the steep front of a bore (Yeh, 2007), the lack of overshoot in dry-bed surge can be attributed to the relatively mild slope of the front profile of the water surface. If the runup zone is flooded by an earlier tsunami wave, subsequent waves could impact buildings in the form of a bore.

For conservatism and especially for structural wall elements of significant width it is recommended that the impulsive forces be taken as 1.5 times the hydrodynamic force, as shown in Equation 6-7:

$$F_s = 1.5F_d \tag{6-7}$$

Impulsive forces may act on members at the leading edge of the tsunami bore, while hydrodynamic forces will certainly act on all members that have already been passed by the leading edge, as shown in Figure 6-7.

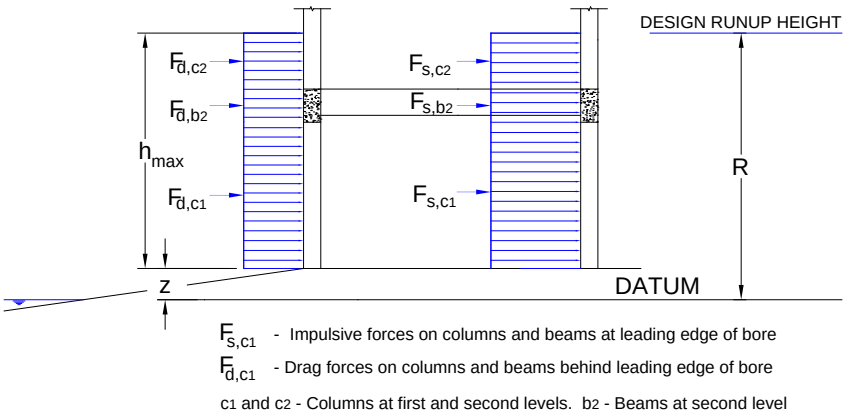


Figure 6-7 Hydrodynamic impulsive and drag forces on components of a building subjected to inundation by a tsunami bore.

6.5.6 Floating Debris Impact Forces

The impact force from waterborne debris (e.g., floating driftwood, lumber, boats, shipping containers, automobiles, buildings) can be a cause of building damage. Unfortunately, it is difficult to estimate this force accurately. Background information on the development of the recommended impact force calculation is provided in Appendix D.

The debris impact force can be estimated using Equation 6-8, which is a more direct generalized form of the ASCE 7 Chapter 5 equation for debris impacts during riverine flooding, without the reduction factors for random orientation:

$$F_i = 1.3 u_{\max} \sqrt{k m_d (1+c)} \tag{6-8}$$

where

1.3 is the Importance Coefficient for Risk Category IV structures that is specified by ASCE 7 Chapter 5 for debris impacts,

u_{max} is the maximum flow velocity carrying the debris at the site (the debris is conservatively assumed to be moving at the same speed as the flow), except for debris rolling along the bottom where the velocity may be reduced by 50%,

c is a hydrodynamic mass coefficient which represents the effect of fluid in motion with the debris (see Table 6-1). This coefficient depends on the size, shape, and orientation of the object with respect to the flow direction. Note that it

no longer represents the traditional added-mass term derived from potential flow hydrodynamics (see Appendix D).

k is the effective net combined stiffness of the impacting debris and impacted structural element(s) deformed by the impact (i.e. $1/k = 1/k_s + 1/k_d$). In this equation, the net stiffness is utilized to implicitly incorporate the impact duration to stop the debris. If the impact is large enough to cause inelastic behavior in the structure, this should be considered in determining the effective stiffness.

m_d is the mass of the debris.

Unlike other forces, impact forces are assumed to act locally on a single member of the structure at the elevation of the water surface, as shown in Figure 6-8. The probability of two or more simultaneous debris strikes is assumed to be low enough that it can be ignored.

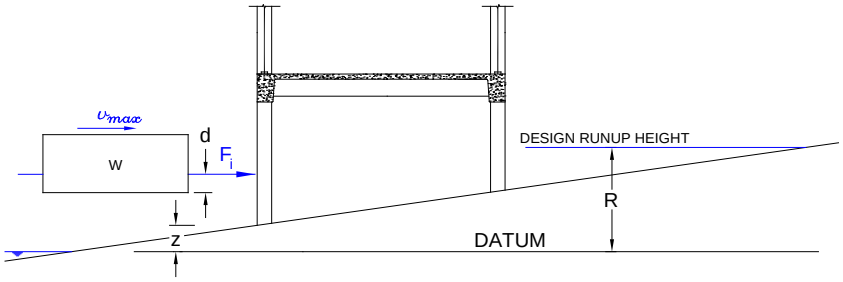


Figure 6-8 Waterborne debris impact force.

Debris impact forces should be evaluated considering the location of the vertical evacuation structure and potential debris in the surrounding area. For example, it is likely that floating debris would consist primarily of driftwood, logs and pier pilings for most coastal towns, whereas for some large port areas, the debris could be shipping containers. Locations near yacht marinas or fishing harbors should consider possible impact from boats that break their moorings.

Use of Equation 6-8 requires the mass, hydrodynamic mass coefficient, and stiffness properties of the debris. Approximate values of m_d, c, and k_d for common waterborne debris are listed in Table 6-1. The mass of contents in the shipping containers should only be included if they are rigidly attached to the container to prevent sliding during impact. Stiffness values for 20-ft standard shipping containers were determined using the secant stiffness corresponding to 25mm displacement for containers modeled numerically (Peterson and Naito, 2012). Values for the 20-ft heavy shipping containers

were increased by the proportion of container weight, while those for the 40-ft containers were adjusted based on differences in framing section properties. Mass and stiffness properties for other types of debris should be derived or estimated as part of the design process.

Table 6-1 Mass and Stiffness of Some Waterborne Floating Debris

Type of Debris	*Mass (m_d) in kg*	*Hydrodynamic Mass Coefft. (c)*	*Debris Stiffness (k_d) in N/m*
Lumber or Wood Log – oriented longitudinally	450	0	2.4×10^6 *
20-ft Standard Shipping Container – oriented longitudinally	2200 (empty)	0.30	85×10^6 **
20-ft Standard Shipping Container – oriented transverse to flow	2200 (empty)	1.00	80×10^6 **
20-ft Heavy Shipping Container – oriented longitudinally	2400 (empty)	0.30	93×10^6 **
20-ft Heavy Shipping Container – oriented transverse to flow	2400 (empty)	1.00	87×10^6 **
40-ft Standard Shipping Container – oriented longitudinally	3800 (empty)	0.20	60×10^6
40-ft Standard Shipping Container – oriented transverse to flow	3800 (empty)	1.00	40×10^6

* Haehnal and Daly, 2002; ** Peterson and Naito, 2012

The magnitude of the debris impact force depends on mass and velocity. Smaller (lighter) debris requiring little or no draft to float can travel at higher velocities than larger (heavier) debris requiring much larger depths to float. Use of maximum flow velocity without consideration of the depth required to float large debris would be unnecessarily conservative. The appropriate maximum flow velocity u_{max} for a given flow depth can be obtained by running a detailed numerical simulation model or by acquiring existing simulation data. It is noted, however, that numerical predictions of flow velocities are less accurate than predictions of inundation depths, and the grid size for numerical simulations in the runup zone should be very fine in order to obtain sufficient accuracy in velocity predictions. Because of the uncertainty involved in even 'accurate' numerical simulations, it is suggested that a margin of safety be applied to the computed flow velocity, depending on the level of confidence in the numerical model simulations.

When a suitable numerical simulation model is unavailable, the maximum flow velocity carrying lumber or a wooden log (with essentially no draft) can be estimated using the analytical solution for tsunami runup on a uniformly sloping beach with no lateral topographical variation, given by Equation 6-9:

$$u_{\max} = \sqrt{2gR\left(1 - \frac{z}{R}\right)}. \qquad (6\text{-}9)$$

where g is the acceleration due to gravity, R is the design runup height that is 1.3 times the ground elevation R^* at the maximum tsunami penetration, and z is the ground elevation at the structure (the datum must be at the sea level). Background information on the development of this equation is provided in Appendix E.

For a shipping container or other similar large debris with draft d, the ratio of the draft d to the maximum runup height R can be computed, and Figure 6-9 can be used to estimate the maximum flow velocity. Draft d can be estimated using Equation 6-10:

$$d = \frac{W}{\rho_s g A_f} \qquad (6\text{-}10)$$

where W is the weight of the debris, ρ_s is the fluid density including sediment (1100 kg/m^3 = 2.13 slugs/ft^3), g is the acceleration due to gravity, and A_f is the cross-sectional area parallel to the water surface such that the product $d \times A_f$ represents the volume of water displaced by the debris.

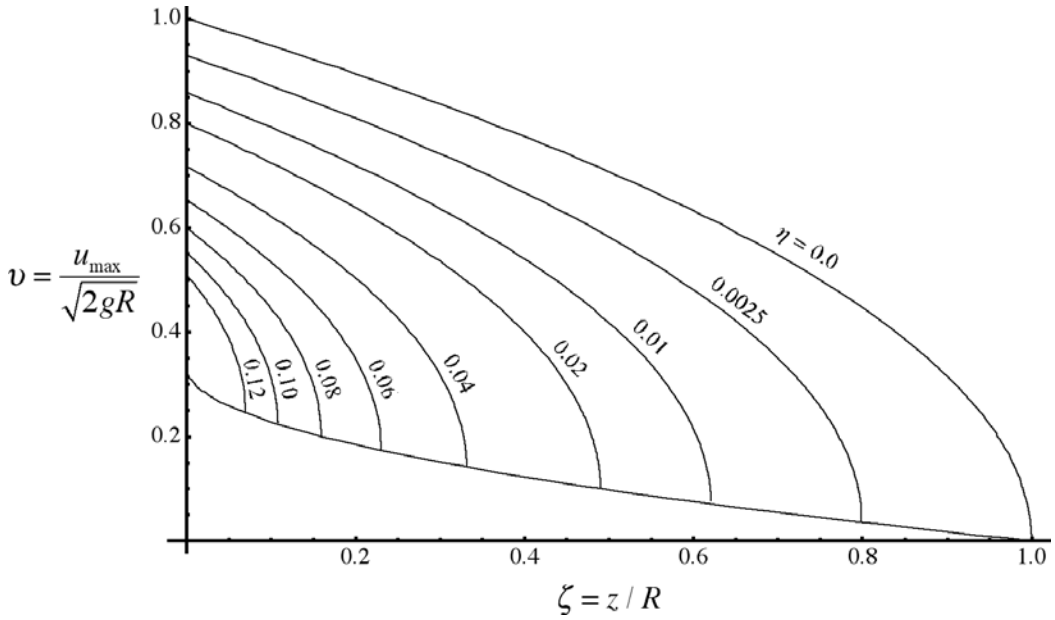


Figure 6-9 Maximum flow velocity of depth, d, at the ground elevation, z, and maximum runup elevation, R. The bottom curve represents the lower limit of maximum flow velocity.

Based on the appropriate curve for d/R, and ratio between the elevation of the structure relative to the design runup elevation (z/R), Figure 6-9 will provide an estimate of the maximum flow velocity. It should be understood that

Figure 6-9 is based on an analytical solution valid only for the flow in the vicinity of the runup tip on a uniformly sloping beach, with no lateral topographical variation, and no friction. Computed values may differ from the actual velocities, and additional engineering evaluation and judgment should be considered. Background information on the development of Figure 6-9 is provided in Appendix E.

Impacts by Floating Vehicles. The impact of vehicles has been studied and codified for the case of vehicles impacting safety guardrails in parking structures. Vehicles are designed to resist impacts with significant inelastic deformation in order to reduce the forces experienced by passengers. It is recommended that the prescriptive code force of 6,000 lbs. used for safety barriers in parking structures be utilized to consider this effect on structural members immersed during the tsunami (ASCE 7, 2010). Alternatively, a work-energy approach similar to that discussed in Appendix D can be used.

6.5.7 Damming of Accumulated Waterborne Debris

The damming effect caused by accumulation of waterborne debris can be treated as a hydrodynamic force enhanced by the breadth of the debris dam against the front face of the structure. Equation 6-11 is a modification of Equation 6-5 to include the breadth of the debris dam:

$$F_{dm} = \frac{1}{2}\rho_s C_d B_d (hu^2)_{\max} \tag{6-11}$$

where ρ_s is the fluid density including sediment (1100 kg/m^3 = 2.13 slugs/ft^3), C_d is the drag coefficient, B_d is the breadth of the debris dam, h is flow depth, and u is flow velocity at the location of the structure. It is recommended that the drag coefficient be taken as $C_d = 2.0$.

The maximum momentum flux per unit width $(hu^2)_{\max}$ should be obtained by running a detailed numerical simulation model or acquiring existing simulation data. If no numerical simulation results are available, an estimate of $(hu^2)_{\max}$ can be determined using Equation 6-6.

Since debris damming represents an accumulation of debris across the structural frame, the total debris damming force will likely be resisted by a number of structural components, depending on the framing dimensions and the size of debris dam. The debris damming force, F_{dm}, should be assumed to act as a uniformly distributed load over the extent of the debris dam. It should be assigned to each resisting structural component by an appropriate tributary width, and distributed uniformly over the submerged height of each resisting component. The recommended minimum debris dam width is the larger of $B_d = 40$ feet (or 12 m), representing a sideways shipping container,

or a full structural bay width. The effects of debris damming should be evaluated at various locations on the structure to determine the most critical location. In addition, it has been observed that internal building contents may generate accumulated debris dammed against the exterior wall. The exterior wall may have partially failed to allow water flow, but structural studs and girts may be capable of holding contents in, thus generating hydrodynamic drag forces on the captured internal debris as the water flows through the structure. Accordingly, a full structural bay of debris dam is the minimum recommended width.

6.5.8 Uplift Forces on Elevated Floors

Uplift forces will be applied to floor levels of a building that are submerged by tsunami inundation. In addition to standard design for gravity loads, these floors must also be designed to resist uplift due to buoyancy and hydrodynamic forces. When computing the buoyant forces on a floor slab, consideration must be given to the potential for increased buoyancy due to the additional volume of water displaced by air trapped below the floor framing system. In addition, exterior walls at the upper floor level will exclude water until their lateral resistance is exceeded by the applied hydrostatic pressure. This can significantly increase the displaced volume of water contributing to the buoyancy, as shown in Figure 6-10.

The total upward buoyant force exerted on a floor system can be estimated using Equation 6-12:

$$F_b = \rho_s \, g \, A_f \, h_b \tag{6-12}$$

where ρ_s is the fluid density including sediment (1100 kg/m^3 = 2.13 slugs/ft^3), g is the acceleration due to gravity, A_f is the area of the floor panel or floor framing component, and h_b is the water height displaced by the floor (including potentially entrapped air). The value of h_{max} indicated in Figure 6-10 should be determined using Equation 6-3.

The upward buoyant force per unit area exerted to the floor system can be estimated using Equation 6-13:

$$f_b = \rho_s g h_b \tag{6-13}$$

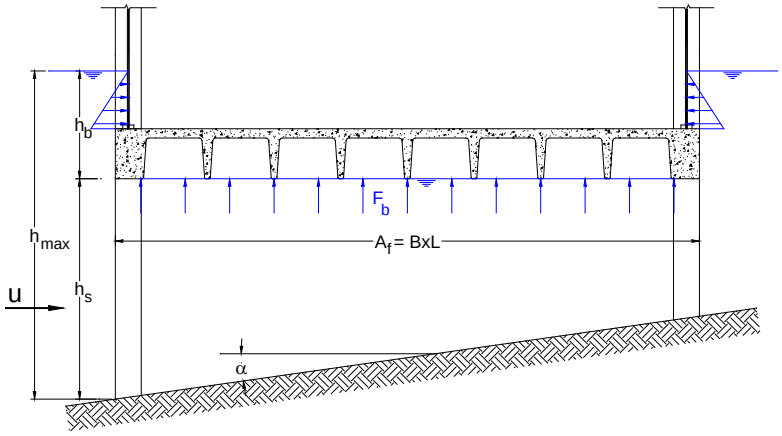


Figure 6-10 A definition sketch for upward buoyant force exerted on an elevated floor.

Hydrodynamic forces can also act vertically on floor slabs. During rapid inundation, rising water will apply uplift to the soffit of horizontal structural components, adding to the buoyancy uplift. The presence of structural walls and columns in a building will obstruct the tsunami flow passing through the building, and recent experiments have shown that this can result in significant uplift forces on the floor slab immediately in front of the obstruction. It is recommended that the building structural layout be designed to minimize obstruction of tsunami flow through the lower levels of the building.

Until further research results become available, the total uplift force on the floor system can be estimated using Equation 6-14:

$$F_u = \frac{1}{2} C_u \rho_s A_f u_v^2 \tag{6-14}$$

where C_u is a coefficient (taken as 3.0), ρ_s is the fluid density including sediment (1100 kg/m^3 = 2.13 slugs/ft^3), A_f is the area of the floor panel or floor framing component, and u_v is the estimated vertical velocity or water rise rate (adapted from American Petroleum Institute, 1993).

The hydrodynamic uplift per unit area can be determined from Equation 6-15:

$$f_u = \frac{1}{2} C_u \rho_s u_v^2 \tag{6-15}$$

Unless a detailed hydrodynamic study is performed, the value of u_v for the condition of sloping terrain below the building can be estimated using Equation 6-16:

$$u_v = u \tan \alpha \tag{6-16}$$

where u is the horizontal flow velocity corresponding to a water depth, h_s equal to the elevation of the soffit of the floor system, and α is the average slope of grade at the site, as shown in Figure 6-10. Using the maximum horizontal flow velocity, u_{max}, in Equation 6-15 would be unnecessarily conservative since it may not correspond to a flow depth equal to the floor soffit elevation. The maximum horizontal velocity u in Equation 6-16 can also be estimated using Figure 6-9 by replacing d/R with h_s/R.

6.5.9 Additional Retained Water Loading on Elevated Floors

During drawdown, water retained on the top of elevated floors, as shown in Figure 6-11, will apply additional gravity loads that can exceed the loads for which the floor system was originally designed. The depth of water retained, h_r, will depend on the maximum inundation depth at the site, h_{max}, and the lateral strength of the wall system at the elevated floor. It should be assumed that the exterior wall system will be compromised at some point so that water will inundate submerged floor levels. Because of the rapid rate of drawdown, it is likely that much of this water will be retained in the upper levels (at least temporarily) resulting in significant additional gravity load on the floor system. The maximum potential downward load per unit area, f_r, can be estimated using Equation 6-17:

$$f_r = \rho_s g h_r \tag{6-17}$$

where ρ_s is the fluid density including sediment (1100 kg/m^3 = 2.13 slugs/ft^3), g is the acceleration due to gravity, and h_r is the maximum potential depth of water retained on the elevated floor determined using Equation 6-18:

$$h_r = h_{\max} - h_1 \le h_{bw} \tag{6-18}$$

where h_{max} is the maximum inundation level predicted at the site, h_1 is the floor elevation above grade, and h_{bw} is the maximum water depth that can be retained before failure of a significant portion of the wall due to internal hydrostatic pressure of the retained fluid.

For elevated floors without walls (such as a parking structure with open guardrails) water may remain on elevated floors until it has had time to drain off the structure. Drainage systems should be provided to ensure that the weight of retained water does not exceed the live load for which the floor is designed if the floor is necessary for structural stability.

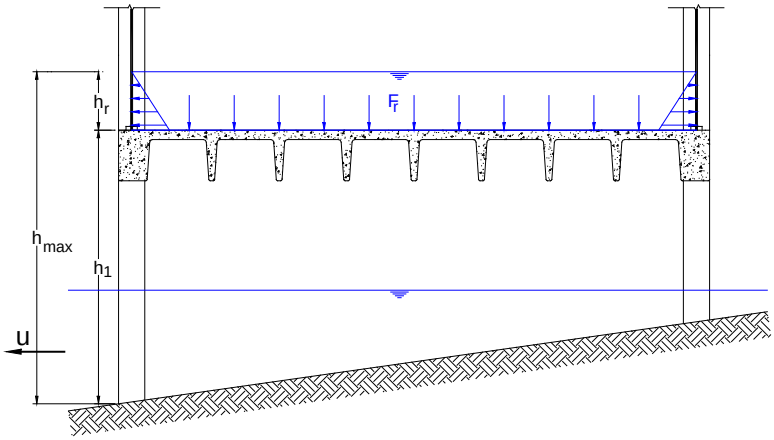


Figure 6-11 Gravity loads exerted on an elevated floor with water retained by exterior walls during rapid drawdown.

6.6 Combination of Tsunami Forces

Not all tsunami load effects will occur simultaneously, nor will they all affect a particular structural component at the same time. This section describes combinations of tsunami forces that should be considered for the overall structure and for individual structural components. Other potential combinations should be considered as needed, based on the particular siting, structural system, and design of the structure under consideration.

6.6.1 Tsunami Force Combinations on the Overall Structure

Not all tsunami load effects will occur simultaneously, nor will they all affect a particular structural component at the same time.

Tsunami forces are combined on the overall structure as follows:

- Uplift due to buoyancy, F_b, and hydrodynamic uplift, F_u, have the effect of reducing the total dead weight of a structure, which may impact the overturning resistance. Buoyancy and hydrodynamic uplift appropriate for the design inundation level should be considered in all load combinations.
- Impulsive forces, F_s, are very short duration loads caused by the leading edge of a surge of water impinging on a wall-like structure. As the surge passes through a structure, impulsive forces will be applied sequentially to all structural components, but not at the same time. Once the leading edge of the surge has passed a structural component, it will no longer experience the impulsive force, but rather a sustained hydrodynamic drag force, F_d. The total horizontal hydrodynamic force on a structure will therefore be a combination of impulsive forces on members at the leading edge of the surge, and drag forces on all previously submerged members behind the leading edge. Figure 6-12 shows how this

combination would apply to a building with multiple columns and shear walls. The worst case lateral load will likely occur when the leading edge of the surge fully impacts the most closed off section of the building.

- Debris impact forces, F_i, are short duration loads due to impact of large floating objects with individual structural components. Since large floating objects are not carried by the leading edge of the surge, the effect of debris impact is combined with hydrodynamic drag forces, F_d, but not impulsive forces, F_s. Although many floating objects may impact a building during a tsunami event, the probability of two or more impacts occurring simultaneously is considered small. Therefore, only one impact should be considered to occur at any point in time. Both the individual structural component and the overall structure must be designed to resist the impact force in combination with all other loads (except impulsive forces).
- Debris damming has the effect of increasing the exposed area for hydrodynamic loading. The debris damming force, F_{dm}, should be considered to act in the most detrimental location on a structure while hydrodynamic forces act on all other components of the structure. Figure 6-13 shows typical debris dam locations that could be considered in conjunction with drag forces on all other submerged structural components. It is conservative to ignore any shielding effect provided by the debris dam for components downstream of the dam.

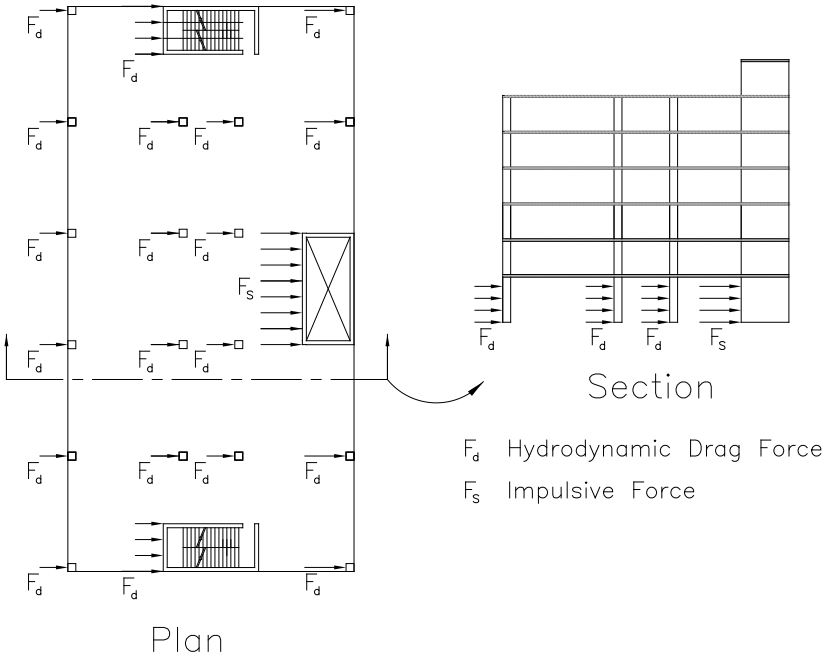


Figure 6-12 Impulsive and drag forces applied to an example building.

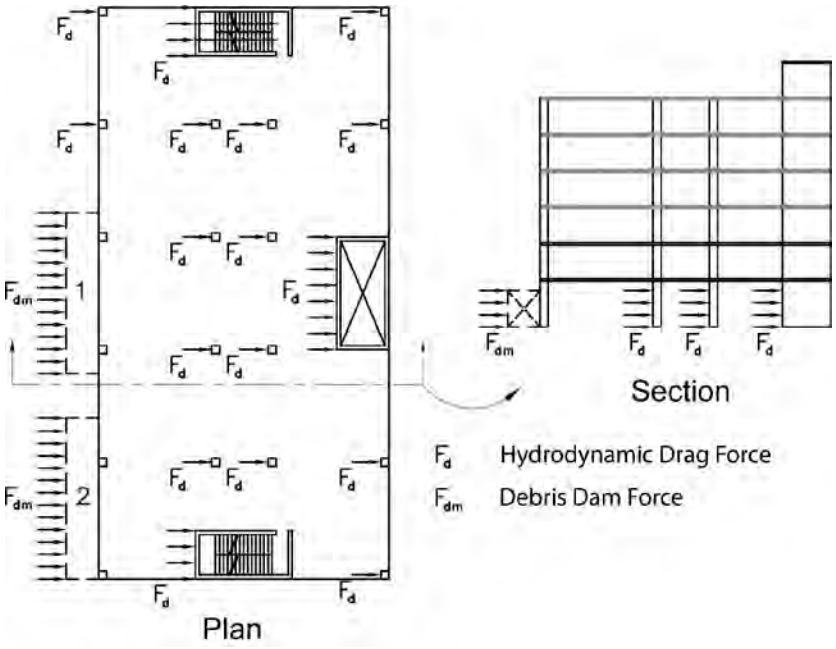


Figure 6-13 Debris dam and drag forces applied to an example building.

- Breakaway walls are not part of the structural support of the building, and are intended, through design and construction, to fail under specific lateral loading. If lower level infill walls are designed as breakaway walls, the maximum lateral load will be the load at which the walls will "fail," and the overall structure, as well as the structural components supporting these walls, must be designed to resist this failure load. Guidance on the design of break-away walls is provided in Chapter 7.
- Design of floor systems to withstand the effects of potential retained water, F_r, can be performed independently of the lateral loading on the structure.

6.6.2 Tsunami Force Combinations on Individual Components

Tsunami forces are combined on individual structural components (e.g., columns, walls, and beams), as follows:

- Impulsive force, F_s, applicable to wall and pier structural elements due to the leading edge of the tsunami bore, for maximum hu^2.
- Hydrodynamic drag force, F_d, plus debris impact, F_i, at the most critical location on the member, for maximum hu^2.
- Debris damming, F_{dm}, due to a minimum 40-foot wide or structural bay width debris dam causing the worst possible loading on the member, for maximum hu^2.

- Hydrostatic pressure, F_h, on walls enclosing watertight areas of a structure, for maximum h.

For uplift on floor framing components, the following combinations of tsunami forces should be considered:

- Buoyancy, F_b, of submerged floor framing components including the effects of entrapped air and upturned beams or walls, for maximum h.
- Hydrodynamic uplift, F_u, due to rapidly rising flood waters, for flow velocity at a depth equal to the soffit of the floor system, h_s.
- Maximum uplift case: The larger of the above uplift loads combined with 90% dead load and zero live load on the floor system, for design against uplift failure of floor slabs, beams, and connections.

For downward load on floor framing components due to retained water, the following force combination should be considered:

- Downward load due to water retained by exterior walls, f_r, combined with 100% dead load.

6.7 Load Combinations

The load combinations presented herein are based on the guidance given in the *Commentary* of ASCE/SEI 7-10 *Minimum Design Loads for Buildings and Other Structures* (ASCE, 2010), but are modified from those used in Section 2.5, Load Combinations for Extraordinary Events, of ASCE/SEI Standard 7-10. The modification is based on the presumption that only the refuge floor areas will be occupied during a tsunami event. They have been reviewed in the development of this document, but have not been extensively studied. They should be considered in addition to all other load combinations required by the current building code in effect, or Section 2 of ASCE/SEI 7-10.

Tsunami Load Combinations should be considered in addition to all other load combinations provided by the current building code in effect, or ASCE/SEI 7-05.

Tsunami forces that will act on the entire structure and on individual structural components should be calculated in accordance with Section 6.5 and Section 6.6. The resulting member forces (T_s) should then be combined with gravity load effects using the following Strength Design Load Combinations:

Load Combination 1: $1.2D + 1.0T_s + 1.0L_{REF} + 0.25L$

Load Combination 2: $0.9D + 1.0T_s$

where D is the dead load effect, T_s is the tsunami load effect, L_{REF} is the live load effect in the refuge area (assembly loading), and L is the live load effect outside of the refuge area.

A load factor of 1.0 is used in conjunction with tsunami forces calculated in accordance with this document for the following reasons: (1) it is anticipated that the tsunami hazard level corresponding to the Maximum Considered Tsunami will be consistent with the 2500-year return period associated with the Maximum Considered Earthquake used in seismic design; and (2) potential variability in tsunami runup elevations is explicitly considered by applying a 30% increase to runup elevations used in tsunami force calculations.

Load Combination 1 considers the refuge area in the vertical evacuation structure to be fully loaded with assembly live load (i.e., 100 psf). The assembly live load represents a practical upper limit for the maximum density of evacuees standing in the refuge area. In combination with tsunami inundation, it is expected that all other floor areas will experience a reduced live load equal to 25% of the design live load. This reduced live load is consistent with live load reductions used in combination with earthquake forces. When gravity load effects oppose tsunami load effects, Load Combination 2 applies.

No additional importance factor, I, is applied to tsunami loads in this document. These design guidelines have been developed specifically for tsunami evacuation structures, and the critical nature of these structures has been considered throughout.

Seismic loads are not considered to act in combination with tsunami loads. While aftershocks are likely to occur, the probability that an aftershock will be equivalent in size to the design level earthquake, and will occur at the same time as the maximum tsunami loading, is considered to be low. However, since seismic design in the U.S. does utilize post-elastic ductility, seismically damaged components may have less available ductility for a subsequently arriving local tsunami.

Member Capacities and Strength Reduction Factors should be applied to design for tsunami loading in the same way they are currently applied to design for earthquake and wind loading.

6.8 Member Capacities and Strength Design Considerations

Model building code provisions and engineering standards for Strength Design, also known as Load and Resistance Factored Design (LRFD), provide material-specific member capacity calculations and strength reduction factors for various force actions and different structural components. Until further research shows otherwise, it is recommended that

capacity calculations and strength reduction factors be applied to design for tsunami loading in the same way they are currently applied to design for earthquake and wind loading.

6.9 Progressive Collapse Considerations

Reducing the potential for disproportionate (i.e., progressive) collapse due to the loss of one or more structural components will increase the likelihood that a vertical evacuation structure will remain standing if a column is severely damaged due to waterborne debris. The decision to include progressive collapse considerations in the design for a particular structure will depend on the site and the nature of the debris that could potentially impact the structure. Because the potential exists for localized severe damage due to debris impact, design for progressive collapse prevention is strongly encouraged. In the United States, primary design approaches for progressive collapse include measures to implement "tie force", "enhanced local resistance" and "alternative load path" mitigation measures. For essential facility occupancies including emergency shelters, the Department of Defense requires the application of all three measures. The General Services Administration requires the alternative load path design technique to span over a missing vertical load carrying column or wall element.

6.9.1 Department of Defense Methodology

The Department of Defense (DOD) has adopted occupancy-dependent requirements for progressive collapse prevention to address the potential for progressive collapse in the design of facilities using UFC 4-023-03, *Design of Buildings to Resist Progressive Collapse* (DOD, 2009). For Risk Category IV, the designer provides:

1. Internal, peripheral, and vertical tie force capacities so that the building is mechanically tied together to enhance the development of alternative load paths.
2. Enhanced Local Resistance of the first two stories on the building perimeter, with flexural capacities of columns and walls increased by factors of 2 and 1.5, respectively, over the design flexural strength determined from the alternative load path procedure. The shear capacities of these elements shall be greater than the flexural capacities. For design of vertical evacuation structures it is proposed that these measures be applied to all levels anticipated to be submerged by the tsunami, but not less than the first two stories.

3. Alternative Load Path to enable the structure to bridge over vertical load-bearing elements that are notionally removed one at a time along the exterior.

The tie force strategy is illustrated in Figure 6-14.

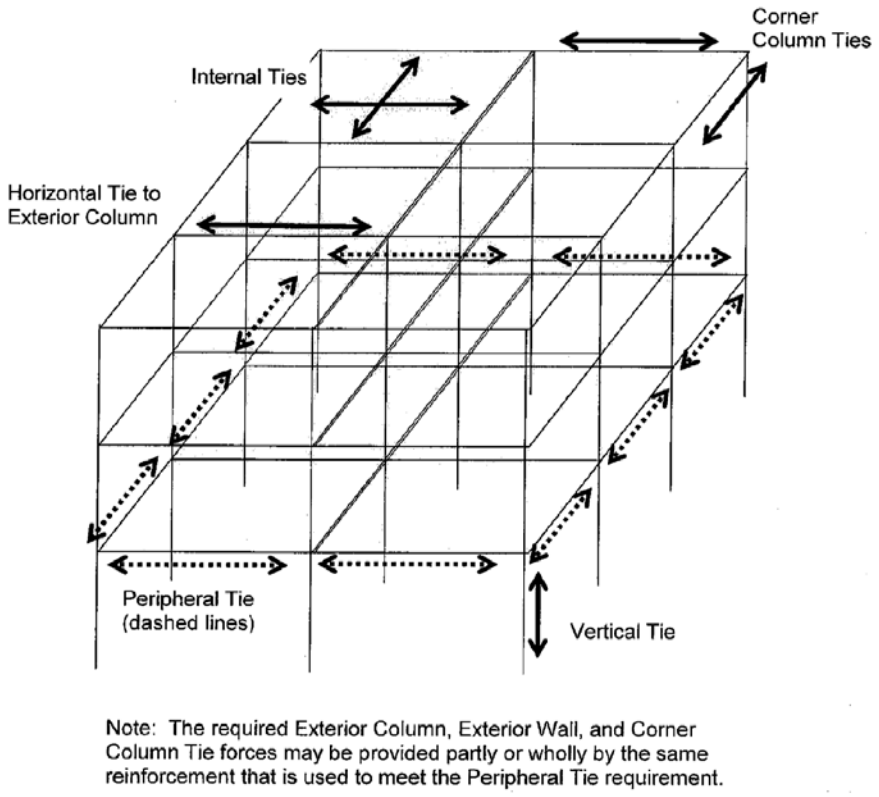


Figure 6-14 Tie force strategy in a frame structure.

Tension ties in reinforced concrete structures typically consist of continuous reinforcing steel in beams, columns, slabs, and walls, as shown in Figure 6-15. Reinforcement required for tension ties can be provided in whole, or in part, by steel already sized to resist other actions, such as shear or flexure. In many cases, the quantity of steel provided to resist gravity and lateral forces for typical reinforced concrete structures is also sufficient to develop the necessary tie forces.

It is reasonable to check tie force compliance after a structure is initially designed for gravity and lateral loading. Ties must be properly spliced and adequately anchored at each end in order to develop their full capacity and perform as anticipated. Reinforcing steel used as tension ties must have lapped, welded, or mechanically joined (Type 1 or Type 2) splices per ACI 318, *Building Code Requirements for Structural Concrete* (ACI, 2011). Splices should be staggered and located away from joints and regions of high stress.

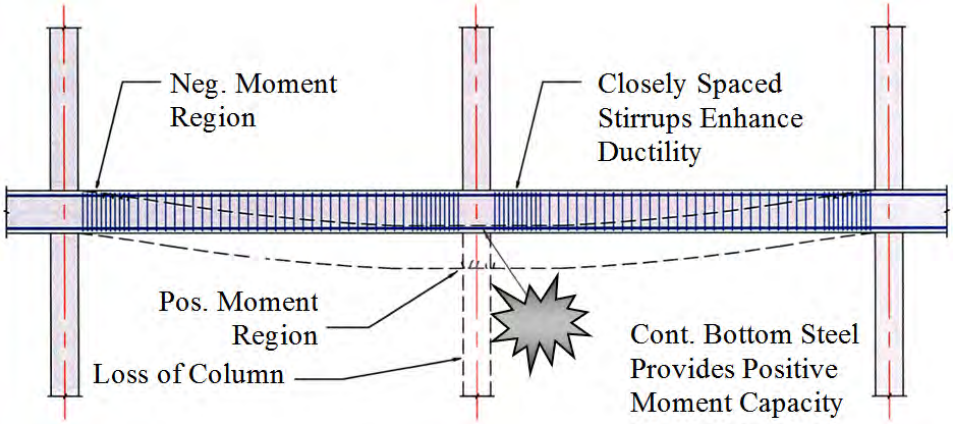


Figure 6-15 Detailing of reinforcing steel for potential loss of a supporting column.

Anchorage is critical to the performance of ties and must be carefully assessed, particularly in cases where building layout may be non-typical. Seismic detailing should be used to anchor ties to other ties, or at points of termination (such as at the perimeter of a building). This includes providing seismic hooks and seismic development lengths, as defined in ACI 318.

6.9.2 General Services Administration Methodology

The General Services Administration (GSA) missing column strategy is an independent check performed without consideration of other loads. This approach is based on the concept that loss of a single column, in this case due to impact from waterborne debris, should not result in progressive collapse of the surrounding structural components.

Current progressive collapse criteria are found in *Progressive Collapse Analysis and Design Guidelines for New Federal Office Buildings and Major Modernization Projects* (GSA, 2003). As illustrated in Figure 6-16, this strategy requires evaluation of surrounding structural components to continue to support anticipated gravity loads in a series of missing column scenarios. Live loads on the building are reduced to simulate those in place at the time the column is damaged. In the case of vertical evacuation structures, full live loads should be considered in the refuge area while reduced live loads can be considered elsewhere in the building.

The missing column approach utilizes plastic design concepts in evaluating the capability of surrounding structural components to continue to support gravity loads, so some damage in these components is permitted as a result of a missing column scenario. Given that waterborne debris is most likely to impact an exterior or corner column, missing column scenarios should

consider the potential loss of any single exterior column. Loss of interior columns need not be considered.

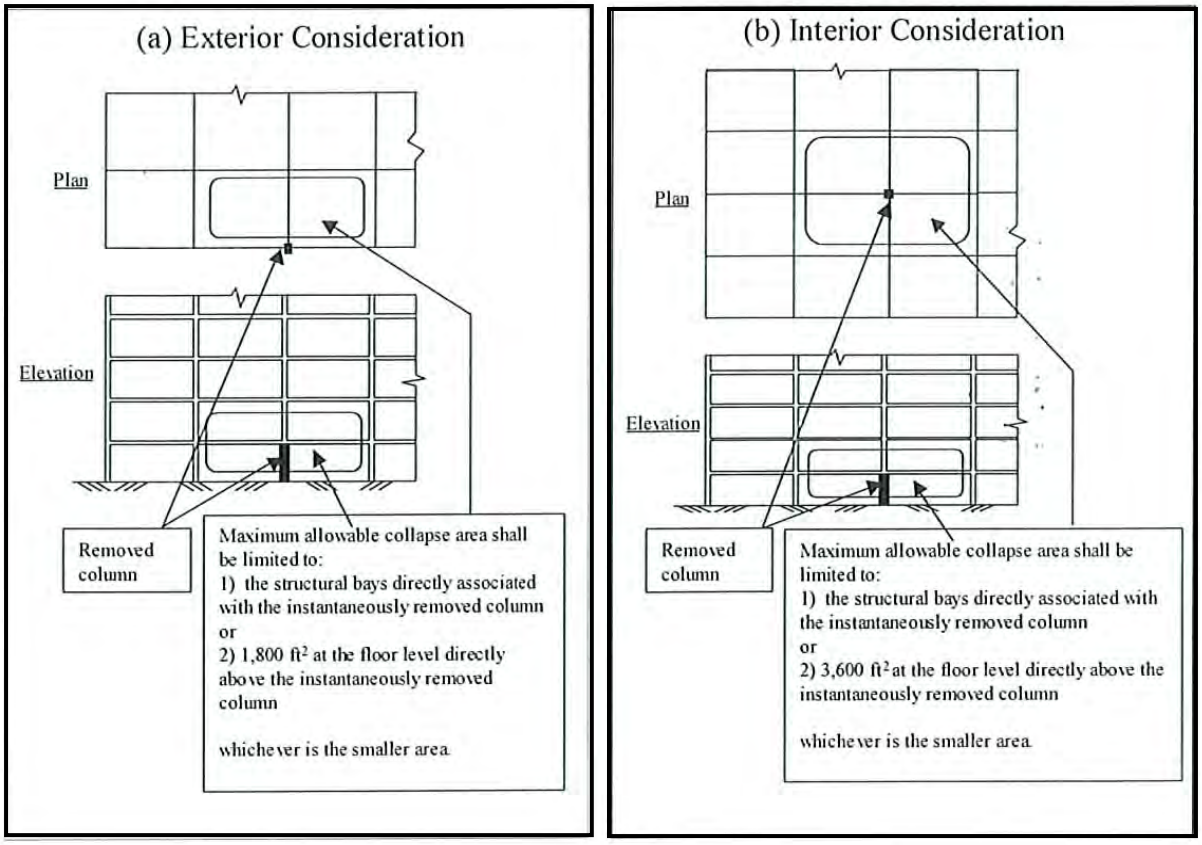


Figure 6-16 Missing column strategy.

Chapter 7
Structural Design Concepts and Additional Considerations

This chapter summarizes structural design concepts and other considerations relevant to the design of vertical evacuation structures, including retrofit of existing structures, permitting, peer review, quality control, planning issues, and potential cost impacts.

Tsunami-Resistant Structures have:
(1) strong systems with reserve capacity to resist extreme forces;
(2) open systems that allow water to flow through with minimal resistance;
(3) ductile systems that resist extreme forces without failure; and
(4) redundant systems that can experience partial failure without progressive collapse.

7.1 Attributes of Tsunami-Resistant Structures

Structural system selection and configuration, from foundation to roof framing, can have a significant effect on the ability of a vertical evacuation structure to withstand anticipated tsunami, earthquake, and wind loading. Many common structural systems can be engineered to resist tsunami load effects.

Structural attributes that have demonstrated good behavior in past tsunamis include: (1) strong systems with reserve capacity to resist extreme forces; (2) open systems that allow water to flow through with minimal resistance; (3) ductile systems that resist extreme forces without failure; and (4) redundant systems that can experience partial failure without progressive collapse. Systems exhibiting these attributes include reinforced concrete and steel moment frame systems, and reinforced concrete shear wall systems.

7.2 Structural Considerations for Tsunami Load Effects

Foundation design must consider the local effects of scour and liquefaction. In many cases foundation support will consist of deep foundations (piles). Pile design must consider increased demands due to downdrag and additional lateral forces, and increased unbraced pile length due to scour. Potential uplift from the overall buoyancy of the structure and overturning moments due to hydrodynamic and unbalanced hydrostatic loads need to be accounted for in the foundation design.

Design of individual columns for tsunami lateral loads should be performed assuming the appropriate degree of fixity at the column base and at each floor level. For example, a reinforced concrete column in a multi-story building supported by pile foundations can be assumed fixed at the base and

at each floor level. A steel column forming part of a moment-resisting frame can be assumed pinned or fixed at the base and at each floor level.

Column shape is also important. Round columns will result in lower drag forces than square or rectangular shapes. In addition, waterborne debris will be less likely to fully impact round columns.

If shear walls are used, the plan orientation of the walls is important. It is recommended that the shear walls be oriented parallel to the anticipated direction of tsunami flow to reduce associated hydrodynamic forces and impact forces from waterborne debris.

Design of reinforced concrete walls for tsunami forces should consider the full load on the wall, including hydrodynamic and debris impact forces, spanning vertically between floor levels. Reinforced concrete beams poured integral with the floor will be braced by the slab. Design of beams for horizontal tsunami forces should take into account the lateral bracing provided by the floor slab. Isolated beams must be designed for horizontal shear and bending induced by tsunami loads.

Floor systems must be designed for the effects of buoyancy and hydrodynamic uplift, which will induce shear and bending effects that are opposite to those resulting from gravity loads. Even though lower levels of a vertical evacuation structure are not intended for use during a tsunami, failure could result in damage or collapse of columns supporting upper levels, including the tsunami refuge area.

In structural steel floor systems, lateral torsional buckling of beam bottom flanges must be considered when subjected to uplift loading. In reinforced concrete floor systems, continuity of reinforcement should be provided in beams and slabs for at least 50% of both the top and bottom reinforcement.

Prestressed concrete floor systems must be carefully checked for buoyancy and hydrodynamic uplift effects when submerged. Internal prestressing forces used to oppose dead loads add to these effects. Web elements of typical prestressed joist systems are susceptible to compression failure under uplift conditions, and many typical bearing connections are not anchored for potential net uplift forces. Localized damage to the concrete in a prestressed floor system can result in loss of concrete compressive capacity, and release of the internal prestressing forces.

7.2.1 Foundation / Scour Design Concepts

Scour around shallow foundations can lead to failure of the supported structural element. Foundations consisting of drilled shafts or driven piles

can be designed to avoid this failure; however, they must be able to resist all applied loads after scouring has exposed the pile cap and top of the shafts or piles.

Dames and Moore (1980) suggests that scour depth is related to distance from the shoreline and soil type. As indicated in Table 7-1, scour depth is estimated as a percentage of the maximum tsunami flow depth, *d*.

Table 7-1 Approximate Scour Depth as a Percentage of Flow Depth, *d* (Dames and Moore, 1980)

Soil Type	*Scour depth (% of d) (Shoreline Distance < 300 feet)*	*Scour depth (% of d) (Shoreline Distance > 300 feet)*
Loose sand	80	60
Dense sand	50	35
Soft silt	50	25
Stiff silt	25	15
Soft clay	25	15
Stiff clay	10	5

Observations after the Indian Ocean Tsunami indicate that scour can occur significantly farther inland than 300 feet from the shoreline. Scour depths of 10 to 13 feet (3 to 4 meters) were observed in locations of high velocity flow during the Tohoku tsunami. Conservative engineering judgment should be exercised in categorizing the soil type at the site into the broad categories listed above.

7.2.2 Breakaway Wall Concepts

Solid enclosure walls below the tsunami inundation level will result in large tsunami loads on the overall building. These walls will also increase the potential for wave scour at grade beams and piles. Non-structural walls below the anticipated tsunami flow depth can be designed as breakaway walls to limit the hydrostatic, buoyancy, hydrodynamic, and impulsive forces on the overall building and individual structural members. Breakaway wall requirements are described in detail in the FEMA 55 *Coastal Construction Manual* (FEMA, 2005), which complies with National Flood Insurance Program (NFIP) requirements for construction in the mapped V-Zone. Breakaway walls can create wave reflection and runup prior to failure as indicated in Figure 7-1.

In accordance with ASCE/SEI Standard 24-05 *Flood Resistant Design and Construction* (ASCE, 2006a), walls, partitions, and connections to the structure that are intended to break away are designed for the largest of the following loads acting perpendicular to the plane of the wall:

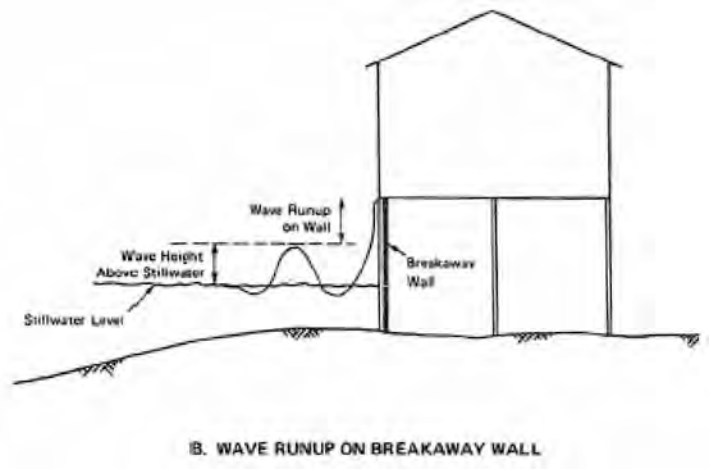


Figure 7-1 Effect of breakaway walls on waves (FEMA, 2005).

- The wind load specified in ASCE/SEI Standard 7-05 *Minimum Design Loads for Buildings and Other Structures* (ASCE, 2006b).
- The earthquake load specified in ASCE/SEI Standard 7-05.
- 10 psf (0.48kN/m^2).
- Not more than 20 psf (0.6 kN/m^2) unless the design meets the following conditions: (1) breakaway wall collapse is designed to result from a flood load less than that which occurs during the base flood; and (2) the supporting foundation and the elevated portion of the building is designed to resist collapse, permanent lateral displacement, and other structural damage due to the effects of flood loads in combination with other loads.

Standard engineering practice can often result in considerable design overstrength, which would be detrimental to a breakaway wall system and the supporting structure. Care should be taken to avoid introducing unnecessary conservatism into the design. All components, including sheathing, siding, and window frame supports, must be considered in determining the actual strength of the breakaway wall system, and the resulting maximum load on the supporting structure. The most desirable fusing mechanism includes failure of the top and side connections while the bottom connection remains intact, allowing the wall panel to lay down under the tsunami flow without becoming detached and part of the debris flow.

Metal Stud Walls. Metal stud infill walls are commonly used as part of the building envelope. Unless properly galvanized, metal studs will corrode rapidly in the coastal environment. Recent lateral load testing of typical metal stud wall configurations shows that ultimate failure occurs when the studs separate from either the top or bottom tracks. However, the load

required to produce this failure is as much as four times the wind load for which the studs were initially designed. It is therefore necessary to introduce some sort of a "fuse" at the top track connection to ensure that the wall fails at a predictable load. Such a fuse might include a reduced stud section at the top of the studs. Testing of fuse mechanisms would be required to verify that they have the capacity needed to resist design loads, but will fail at predictably higher load levels.

Masonry Walls. Masonry walls are commonly used as enclosures in lower levels of larger buildings. They can be restrained with the use of a dowel pin fuse system around the top and sides of the wall, without bonded contact to the structure. Such a system should be tested to verify that it will fail at predictable load levels that exceed design loads. If properly fused, the masonry wall will cantilever from the foundation and load will no longer be applied to the surrounding structural frame, upon failure of the dowel pins. To allow wall failure due to foundation rotation without damage to the remaining structure, separation of the wall foundation from the building foundation should be considered.

7.3 Concepts for Modifying and Retrofitting Existing Structures

It may not always be feasible to construct new buildings in an area that requires vertical evacuation refuge. Although retrofitting existing buildings to perform as a vertical evacuation structure could be expensive and disruptive to current users of the building, it may be the most viable option available. Existing buildings considered for use as vertical evacuation structures should possess the structural attributes listed in Section 7.1 that are associated with tsunami-resistant structures, and should be evaluated for tsunami load effects in accordance with Chapter 6. In the case of near-source-generated tsunamis, existing buildings should also be evaluated for seismic effects. Because of the importance of vertical evacuation structures, and the need for these facilities to function as a refuge when exposed to extreme tsunami and seismic loading, reduced loading criteria for existing buildings, as is the current state-of-practice for seismic evaluation of existing buildings, is not recommended for evaluation of potential tsunami vertical evacuation structures.

Existing buildings considered for use as vertical evacuation structures should possess the attributes of tsunami-resistant structures listed in Section 7.1

The following concepts can be considered in the modification and retrofit of existing buildings for use as vertical evacuation structures:

- **Roof system.** Upgrade roof systems to support additional live loads associated with refuge occupancy. Protect or relocate existing building functions at the roof level (e.g., mechanical equipment) that would be at

risk or unsafe in the immediate vicinity of high occupancy areas. Modify existing roof parapets for fall protection of refuge occupants.

- **Wall system.** Consider modifying walls and wall connections in the lower levels of the building to perform as breakaway walls to minimize tsunami hydrostatic, hydrodynamic, and surge forces on the building.
- **Access.** Modify ingress into the building and improve vertical circulation through the use of new entrances, ramps, and stairs. Consider placing access points on the outside of the building for ease of construction and high visibility.
- **Potential Debris.** Remove or relocate building ground level functions that may become potential water-borne debris.
- **Existing hazards at the site.** Consider and protect against other hazards that might exist at the building site, including other adjacent buildings that could collapse, and the presence of hazardous or flammable materials near the site.

7.4 Permitting and Quality Assurance for Vertical Evacuation Structures

7.4.1 Permitting and Code Compliance

The unique nature of vertical evacuation structures may require special allowances for:
(1) permitting and code compliance;
(2) peer review; and
(3) quality assurance.

Before construction begins, all necessary state, local, building, and other permits should be obtained. Because model building codes and engineering standards do not address the design of a tsunami refuge specifically, design professionals should meet with building officials to discuss possible design requirements.

In general, mechanical, electrical, and plumbing systems should be designed for the normal daily use of the facility, unless otherwise directed by the authority having jurisdiction. Designing these systems for the high occupancy load that would occur only when the structure is serving as a vertical evacuation refuge may not be necessary.

7.4.2 Peer Review

A vertical evacuation structure is a unique structure that must withstand special loads and load combinations. While earthquake, wind, and flood loading effects are well understood in the design and permitting process, consideration of tsunami load effects includes some new concepts and approaches. Considering the importance of vertical evacuation structures and the extreme nature of tsunami loading, peer review by a qualified individual or team is recommended.

7.4.3 Quality Assurance / Quality Control

Because a vertical evacuation structure must perform well during extreme loading conditions, quality assurance and quality control for the design and construction of the structure should be at a level above that for normal building construction. Design calculations and drawings should be thoroughly scrutinized for accuracy.

The quality of both construction materials and methods should be ensured through the development and application of a quality control program. A quality assurance plan should be based on the Special Inspection Requirements listed in Chapter 17 of the *International Building Code* (ICC, 2006). Special inspections and quality assurance provisions for primary seismic- and wind-resisting systems should be applied to tsunami-resisting elements of vertical evacuation structures. Exceptions that waive the need for quality assurance when elements are prefabricated should not be allowed.

In addition to the building elements that are normally included special inspection programs, the following items require special attention:

- Breakaway walls and their connections to structural components to avoid unintended conservatism in construction.
- Other special components or details that are used to minimize tsunami-loading effects.
- Piles, pilecaps and grade beams that will potentially experience the effects of scour.

7.5 Planning Considerations for Vertical Evacuation Structures

In addition to structural design, planning for vertical evacuation facilities should consider a number of issues, including access, parking, pets, occupancy limitations, and protection of critical functions.

Planning for vertical evacuation facilities should allow for:
(1) access and entry;
(2) Americans with Disabilities Act;
(3) parking;
(4) pets;
(5) occupancy limitations; and
(6) protection of critical functions.

- **Access and Entry.** Confusion and panic will occur if evacuees arrive at a refuge facility, but cannot enter. Provisions should be made to ensure access in the event of a tsunami, while providing adequate security during times when the facility is unoccupied. Ideally, a vertical evacuation refuge should be configured so that it is always accessible, or can be entered without emergency personnel.
- **Americans with Disabilities Act (ADA).** Vertical evacuation structures, when not operating as a refuge, must comply with Federal, state, and local ADA requirements and ordinances for the normal daily use of the facility. Design of ingress and vertical circulation within a

vertical evacuation structure should consider the needs of disabled occupants to the extent possible, and the extent required by law, in the case of emergency evacuation. Given potential limitations on functionality of power sources and vertical conveyance systems (e.g., elevators and escalators) in the event of a near-source earthquake, disabled occupants may need assistance accessing refuge areas in vertical evacuation structures.

- **Parking.** Parking at evacuation facilities can be a problem. Traffic congestion can adversely affect access to the facility, and parked vehicles can become waterborne debris that can damage the structure. Planning for vertical evacuation facilities should consider parking limitations.
- **Pets.** Refuge facilities are typically not prepared to accommodate pets. Many people, however, do not want to leave their pets behind during a disaster. Planning should carefully consider the policy regarding pets.
- **Occupancy Limitations.** Population density can be non-uniform, and can vary by time of day, week, or year. In the event of a tsunami, evacuation behavior of the surrounding population may result in an unequal distribution of evacuees among available refuge facilities. In determining the maximum occupancy for a refuge facility, the time of day, day of the week, or season of the year that will result in the largest number of possible evacuees should be considered. The maximum occupancy might need to be increased in order to accommodate unexpected additional occupants or visitors in the area.
- **Protection of Critical Functions.** A vertical evacuation facility must be operational to serve its intended function in the event of a tsunami. Functions that are critical for operation as a short-term refuge, emergency response, medical care, or long-term sheltering facility must be protected from tsunami inundation, or located within the area of refuge. These might include emergency power, electrical equipment, communications equipment, basic sanitation needs, medical and pharmaceutical supplies, and emergency provisions (e.g., food, water, and supplies).

7.6 Cost Considerations for Vertical Evacuation Structures

Design of vertical evacuation structures for tsunami load effects will require more strength, ductility, and robustness than is necessary for normal-use structures. As recommended in this document, this can include the use of seismic detailing provisions, progressive collapse preventative measures, customized breakaway wall details, and deeper foundation systems. As such,

it is expected that structural construction costs will be higher for vertical evacuation structures than for other structures. While there are no direct comparisons between the cost of a conventional structure versus the cost of a tsunami-resistant structure, order-of-magnitude information on potential structural construction cost increases can be obtained from currently available information.

Structural costs, however, are only a fraction of total construction costs for a building. Depending on the nature of building occupancy and use, structural construction costs can range between 5% and 40% of total construction costs. Structural costs are a lower percentage of the total for occupancies with special uses (e.g., hospitals) requiring more expensive nonstructural systems and contents, and are higher percentage of the total for occupancies with standard uses (e.g., offices).

Structural construction costs are only a fraction of total construction costs for a building.

Anecdotal evidence from design and construction of essential facilities (e.g., hospitals) in California, Oregon, and Washington indicate that the cost premium for seismic design requirements associated with essential facilities versus ordinary occupancy facilities is on the order of 10% to 20% of structural construction costs. This would represent an increase on the order of 1% to 8% in terms of total construction costs.

In a recent study funded by the National Institute of Standards and Technology (NIST), *Engineering Design and Cost Data for Reinforced Concrete Buildings for Next Generation Design and Economic Standards for Structural Integrity* (NIST, 2007), the cost premium for progressive collapse-resistant design was on the order of 10% to 20% of structural construction costs. Similar to seismic design, this would represent an increase on the order of 1% to 8% in terms of total construction costs.

Considering additional allowances for added strength to resist tsunami load effects, it is reasonable to expect that a tsunami-resistant structure, including seismic-resistant and progressive collapse-resistant design features, would experience about a 10% to 20% order-of-magnitude increase in total construction costs over that required for normal-use buildings. While each project will be unique, and relative costs will depend on the specific tsunami hazard and site conditions, it should not be assumed that incorporation of tsunami-resistant design features in a vertical evacuation structure will be cost prohibitive.

Tsunami-resistant structures could experience about a 10% to 20% order-of-magnitude increase in total construction costs over that required for normal-use buildings.

Appendix A
Vertical Evacuation Structure Examples from Japan

In Japan there are examples of structures that were designed and constructed specifically for the purpose of tsunami refuge. The Government of Japan, Director-General for Policy Planning, published *Guidelines for Tsunami Evacuation Buildings* in Japanese in June 1995 (DGPP, 1995). Okada, et al (2005) of the Building Center of Japan, Building Technology Research Institute, provide an English explanation in *SMBTR - Structural Design Method of Buildings for Tsunami Resistance,* which has been used for design of vertical evacuation structures such as the apartment building in Minamisanriku, shown in Figure 2-23.

A number of multi-story reinforced concrete and structural steel buildings in Japan were designated as vertical evacuation buildings prior to the Tohoku tsunami. All performed well structurally, though many were too low for the actual inundation depth, resulting in loss of life (Murakami et al, 2012). Figure A-1 shows such a building in Kesennuma Port that was successfully used by refugees during the tsunami. Over 4000 buildings and other structures in Japan are now officially designated for use as vertical evacuation refuges (Yomiuri Shimbun, 2012).

Life-Saving Tower: The Life-Saving Tower (Tasukaru Tower) developed by Fujiwara Industries Company, Limited, Japan, is shown in Figure A-2. This is a simple and economical structure that enables a temporary high refuge for evacuees. The structure has a 5.4-meter span between the supporting posts, a refuge elevation of 5.8 meters from ground level, and a capacity of 50 people.

Figure A-1 Successful designated vertical evacuation building in Kesennuma Port, Japan.

Figure A-2 Life-Saving Tower

Nishiki Tower: The Nishiki Tower, shown in Figure A-3, was constructed in the town of Kise, Mie Prefecture, Japan. The five-story, 22-meter tall reinforced concrete structure resembles a lighthouse, and has a spiral staircase winding up the outside of the building. It was specifically designed to serve as a tsunami refuge, but is used for other (non-refuge) purposes on normal days. The first floor is used for public toilet and storage space for fire equipment; the second floor for a meeting room; and the third floor for an archival library for natural disasters. The fourth and fifth floors have 73 square meters of refuge space for evacuees.

Figure A-3 Nishiki Tower.

Nishiki Tower is a well-engineered structure that is designed to withstand a seismic event commensurate to JMA VII on the Japanese earthquake intensity scale that is comparable to a MMI XII (modified Mercalli scale). The building is founded on a 4-meter deep sand-and-gravel layer, and is supported on concrete piles extending 6 meters below grade. The possibility of liquefaction is remote, considering the large particle size of the sand-and-gravel layer. Elastic design was employed for consideration of tsunami forces. Based on historical data from the 1944 Tou-Nankaido Earthquake, a design tsunami of 6 meters in height was used for design. It is designed to withstand the impact of a 10-ton ship at a velocity of 10 m/sec. This

criterion was based on size of ships moored in the neighboring port. The intended performance level allows for partial damage of the building without incurring loss of life.

Elevated Shelter at Shirahama Beach Resort: A rather aesthetic tsunami refuge was constructed at a beach resort in the town of Shirahama, Tokushima Prefecture, shown in Figure A-4. It is designed to accommodate 700 refugees in the area of 700 square meters. The design inundation elevation is 7.5 meters, based on historical data from the 1854 Ansei-Tokai Earthquake (M 8.4) and resulting tsunami. With a planned freeboard of 4 meters, the evacuation platform is located at elevation of 11.5 meters. This reinforced concrete structure is designed to withstand a maximum base acceleration of 780 gal. Because of a potential for soil liquefaction, pipe piles were driven approximately 20 meters deep into bedrock. The facility is also equipped with a solar-powered lighting system.

Figure A-4 Refuge at Shirahama Beach Resort (photo courtesy of N. Shuto).

Other Tsunami Refuge Structures: There are other structures in Japan specifically designed as tsunami refuges. A reinforced concrete structure in the town of Kaifu, Tokushima Prefecture, Japan is shown in Figure A-5. An artificial high ground (berm), shown in Figure A-6, was constructed in Aonae, Okushiri-Island, Japan, where the 1993 tsunami struck the hardest. After the 1993 Okushiri Tsunami, Aonae elementary school, shown in Figure A-7, was reconstructed as a tsunami resistant structure. The upper floor can

be used as a tsunami refuge space. The ground floor of the school is constructed with breakaway walls to relieve tsunami forces.

Figure A-5 Tsunami refuge in Kaifu, Japan.

Figure A-6 Berm constructed for tsunami refuge in Aonae, Japan.

Figure A-7 Aonae Elementary School. Upper floor is intended for use as tsunami refuge space.

Appendix B

Community Design Example

A hypothetical community is indicated in Figure B-1 below. In this appendix, the initial design and configuration of a series of vertical evacuation structures is illustrated.

The community has evaluated public and private sites that might be appropriate for construction of new vertical evacuation structures and identified existing facilities for possible renovation for use as vertical evacuation structures. This evaluation includes consideration of the number of sites required based on travel time and population, as discussed in Chapter 5.

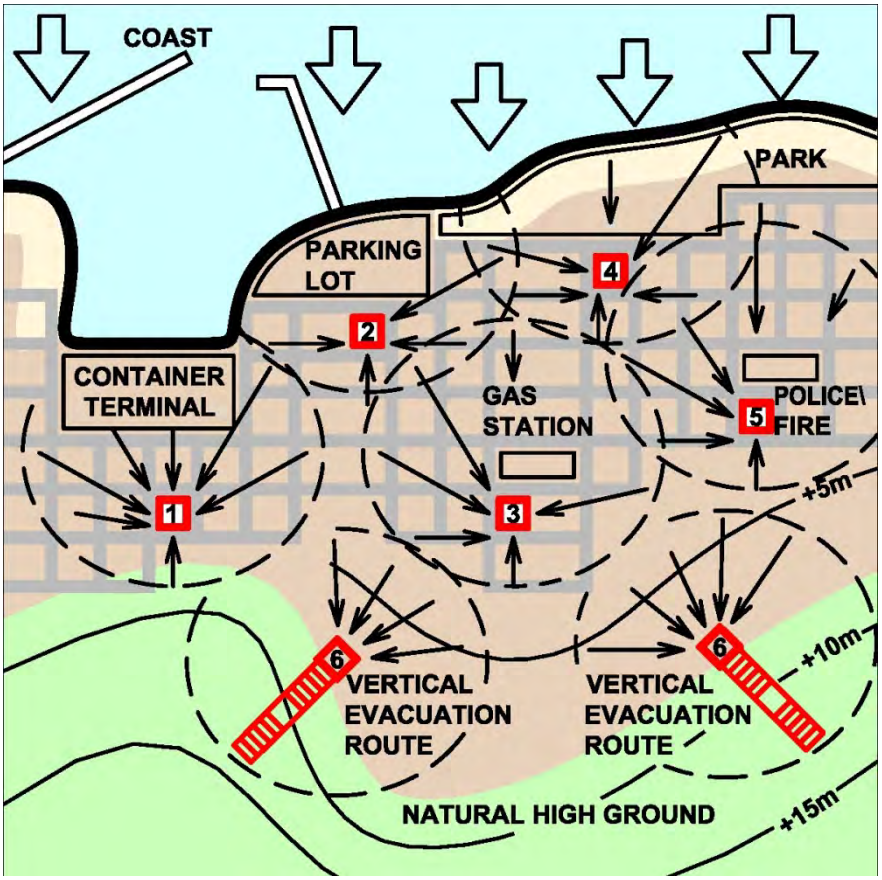


Figure B-1 Hypothetical sketch of example community showing potential vertical evacuation structure sites and evacuation routes.

An assessment of the tsunami inundation depths and flow velocities is necessary for assessing tsunami effects within the community and determining tsunami design parameters. Predicted tsunami inundation depths for this example community are shown in Figure B-2.

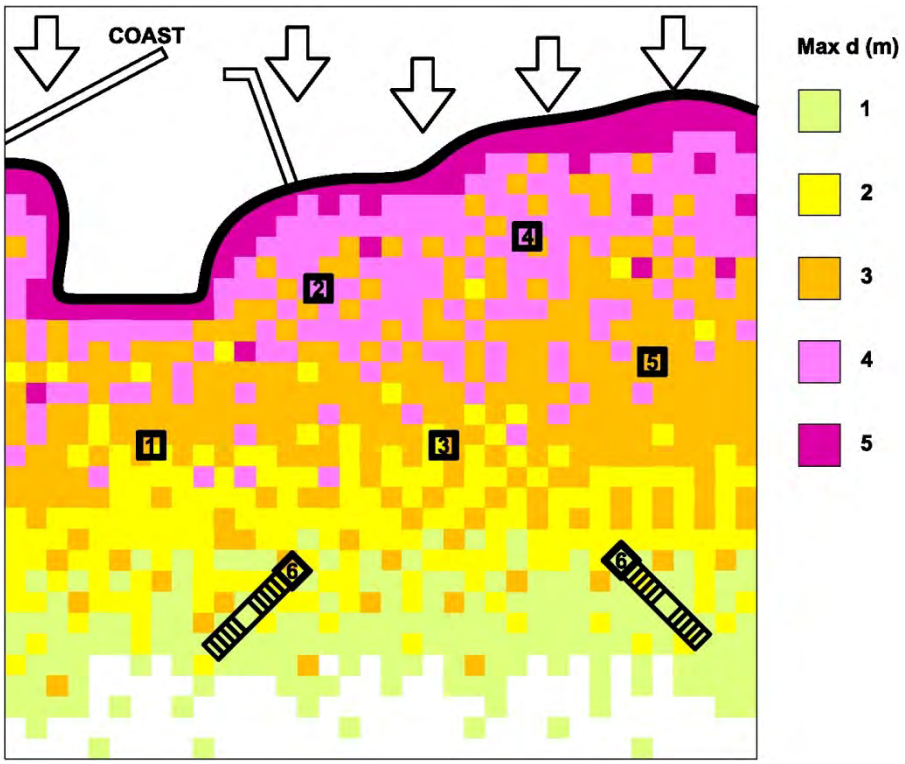


Figure B-2 Example community inundation map. Shaded areas show various predicted tsunami inundation depth, *d*.

In this example community, the area of refuge at each site would need to be elevated as indicated in Table B-1.

Table B-1 Design Elevations for Areas of Refuge

Site	*Predicted Inundation Depth*	*Freeboard (3 meters plus 30%)*	*Design Elevation*
Site 1	3 m	3 m + 0.9 m	6.9 m
Site 2	4 m	3 m + 1.2 m	8.2 m
Site 3	3 m	3 m + 0.9 m	6.9 m
Site 4	4 m	3 m + 1.2 m	8.2 m
Site 5	3 m	3 m + 0.9 m	6.9 m

Tsunami inundation depths indicated in Figure B-2 are increased by 30% to account for local variability in numerical simulations. An additional minimum freeboard of 3 meters (or one-story height) is recommended to ensure that the area of refuge is not inundated from splash or wave action.

The velocity at a particular site is affected by the surrounding topography as well as natural and man-made obstructions to flow. Predicted flow velocities for this example community are shown in Figure B-3 and summarized in Table B-2.

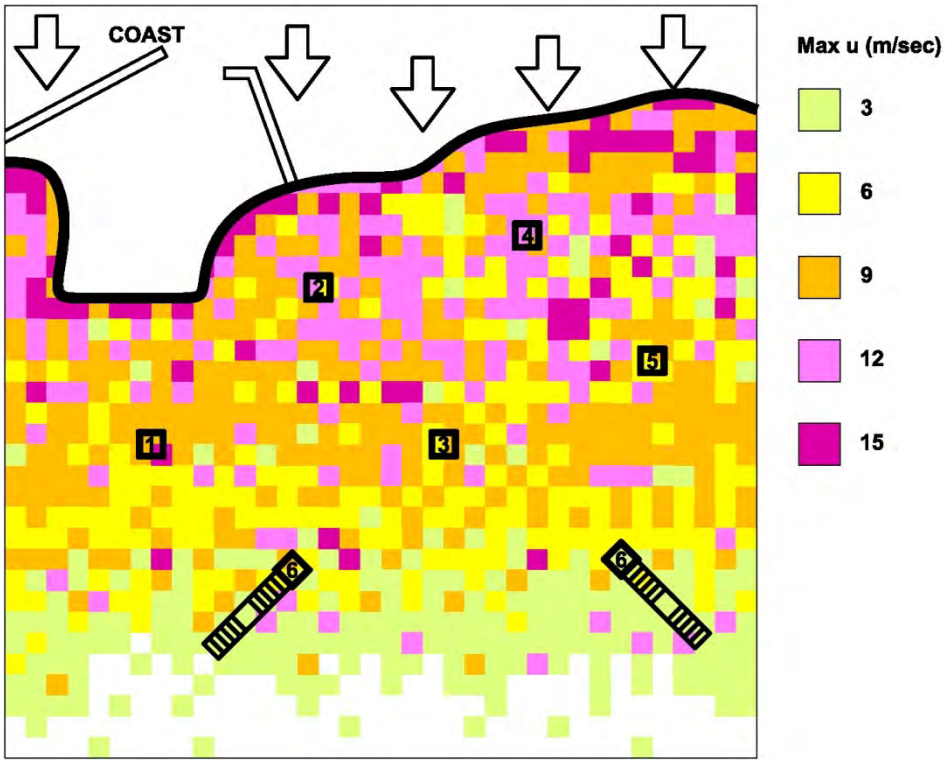


Figure B-3 Example community inundation flow velocity map. Shaded areas show various predicted tsunami flow velocities, *u*.

Table B-2 Tsunami Flow Velocity at Each Site

Site	*Tsunami Flow Velocity*
Site 1	9 m/s
Site 2	12 m/s
Site 3	9 m/s
Site 4	12 m/s
Site 5	9 m/s

B.1 Site 1 Example: Escape Berm

Site 1 has several unique conditions to consider. The waterfront in this area is somewhat industrial in nature and includes a container terminal facility at the harbor. Areas adjacent to the site contain some residential development. The evacuation population at this site would include both employees of the harbor industrial area and adjacent residences.

The community has been struggling with finding ways to address other social issues in this area, which have included a lack of recreational facilities for the residents, some neglected and deteriorating properties, and a need to revitalize and enhance the area. At this site a man-made berm, as shown in Figure B-4, provides an opportunity to add new public open space in addition to vertical evacuation refuge. This solution creates a unique elevated park setting for the community, which addresses recreational needs, and provides a scenic overlook for the waterfront.

With a location adjacent to a container terminal facility, there is a potential for shipping containers to become waterborne debris. Construction of the berm utilizing a sheet piles to contain the fill addresses this issue.

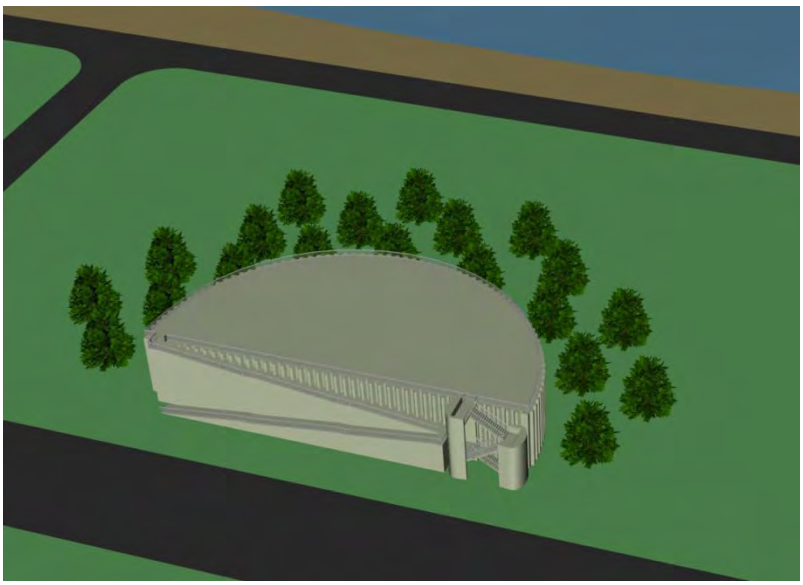


Figure B-4 Example escape berm design.

The features of this escape berm, illustrated in Figure B-5, include the following:

- *Location 1 (Figure B-5).* The semicircular configuration was selected to help divert tsunami flood waters and potential waterborne debris around the facility and away from the access stairs and ramp. The elevated area is over 31,000 square feet, and can handle over 3,000 evacuees at 10 square feet per person. There is sufficient space in the elevated area to accommodate a comfort station that could be used for both day to day recreational purposes and emergency use.

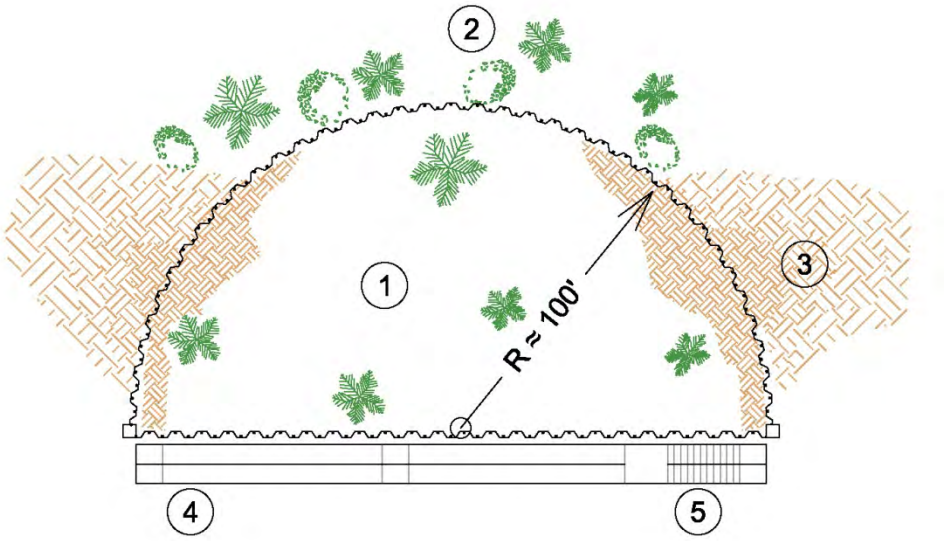


Figure B-5 Example escape berm plan layout.

- *Location 2 (Figure B-5).* The ocean facing side of the berm is essentially vertical to prevent tsunami flood waters and potential floating debris from moving upslope into the area of refuge. Trees and other landscaping can be used to hide the vertical face and create an aesthetically appealing feature.
- *Location 3 (Figure B-5).* The sides of the berm can be sloped to provide additional access to the area of vertical refuge. Care should be taken to orientate the slope so that water and debris are not inadvertently channeled upslope.
- *Locations 4 and 5 (Figure B-5).* Stairs and ramps provide primary access for both recreational and emergency purposes.

Additional considerations are illustrated in Figures B-6 and B-7 and described below.

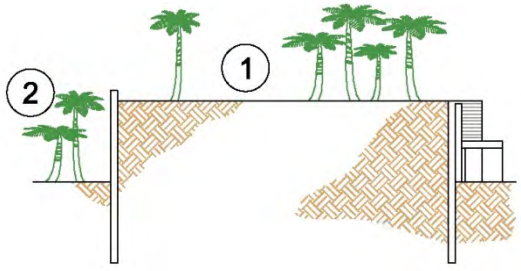


Figure B-6 Example escape berm section.

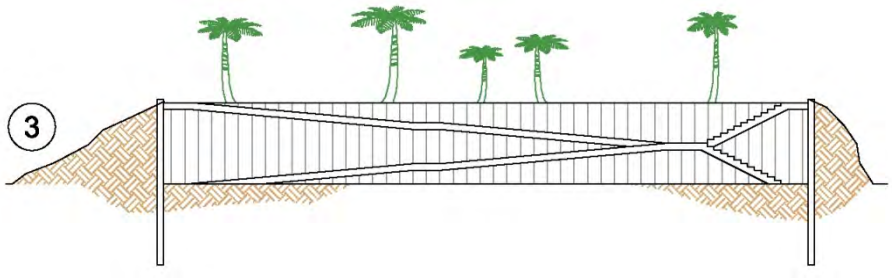


Figure B-7 Example escape berm rear elevation.

- *Location 1 (Figure B-6).* Where the elevated area is adjacent to a steep drop off, guard rails or walls of appropriate size and height should be provided for fall protection. Using a solid wall for the guardrail will have the added benefit of providing additional protection from tsunami runup or splash onto the area of refuge. Walls can be configured to divert splash away from the wall.
- *Location 2 (Figure B-6).* Materials used to help create the berm will need to be constructed deep enough below existing grade to ensure that retaining system is not undermined by scour around the perimeter of the berm.
- *Location 3 (Figure B-7).* With sufficient length, both ADA compliant ramps and stairs can be provided. This will address both the day to day recreational use of the facility as well as emergency evacuation needs. Sloped surfaces on the sides of the berm can be used to provide additional access, and can also help channel floating debris away from the base of the ramps and stairs to minimize the risk of blockage.

B.2 Site 2 Example: Multi-Use Structure

Site 2 is situated on property managed by the school district. The site is located adjacent to an existing school and the surrounding area contains a combination of residential and business use. The existing school is located well within the inundation zone. The waterfront in this area includes an on-grade parking lot that services businesses in the area, and a nearby oceanfront park. The evacuation population at this site would include children attending the school, neighbors in the adjacent residences, employees of nearby businesses, and nearby users of the oceanfront park.

The school district has had an ongoing need for a covered gymnasium. At this site, the community has decided to incorporate the roof of the proposed gymnasium into its emergency planning. It is decided that this new structure will be designed to meet the requirements for a vertical evacuation structure

to serve two important community needs. The structure is illustrated in Figure B-8.

Located adjacent to an on-grade parking lot, the structure will need to be designed for potential impacts from floating vehicles. If the community is located in a climate that requires the gymnasium to be enclosed, special attention should be paid to the design of the exterior wall system. Walls should be detailed as breakaway walls to minimize tsunami loading on the overall structure. Otherwise the structure will need to be designed to for the corresponding increased hydrostatic, hydrodynamic, and impulse loads.

As a school facility, the building must also be designed to address typical health and safety requirements for school facilities in normal use (when not serving as a vertical evacuation refuge).

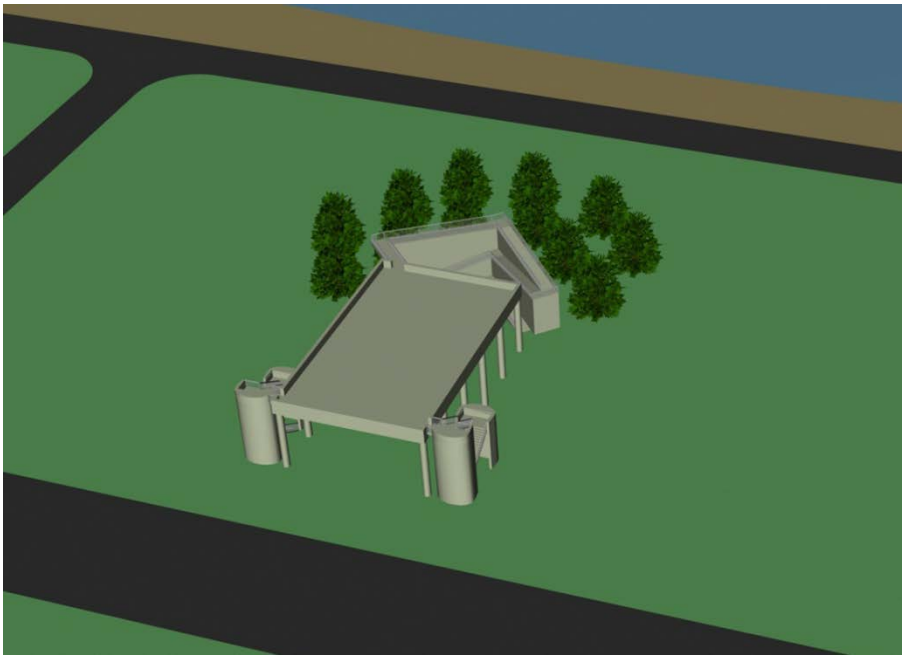


Figure B-8 Example gymnasium.

Features of this multi-use structure, illustrated in Figure B-9 and Figure B-10, include the following:

- *Location 1 (Figure B-9).* The rectangular layout is selected based on the gymnasium requirements for the school. The elevated area is over 10,000 square feet in size, and can handle over 1,000 evacuees at 10 square feet per person. Using available census information, it has been determined that this should be sufficient for the surrounding area this facility is intended to serve.

- *Location 2 (Figure B-9).* Stair access is designed using a concrete encased stair structure that will have its own inherent strength. The shape is intended to channel tsunami flow and potential debris away from both the structure and the stair system.
- *Location 3 (Figure B-9).* An additional ADA accessible ramp system is considered for a future phase of the project. This could utilize sheet piles and fill to further channel tsunami flow and waterborne debris away from the structure.

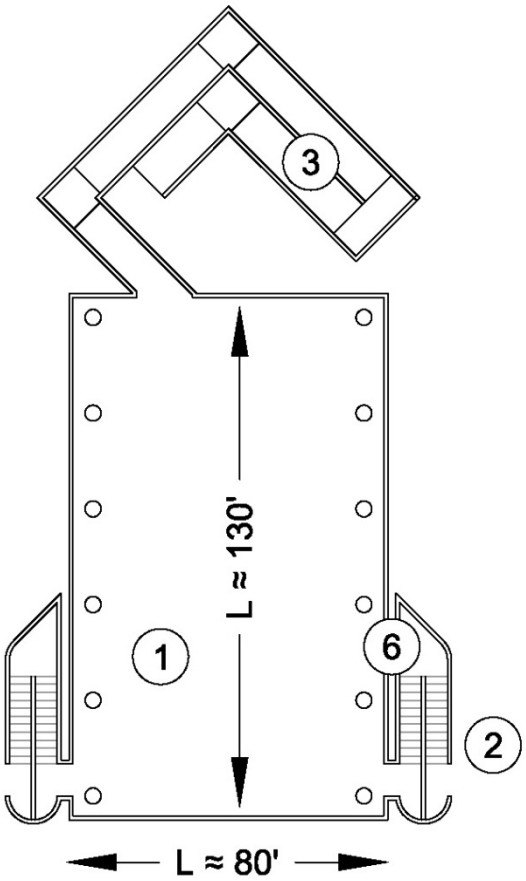
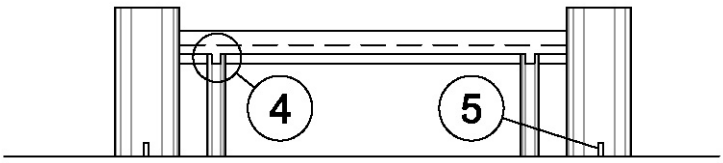


Figure B-9 Example gymnasium plan.



- *Location 4 (Figure B-10).* The structural system utilizes a concrete moment frame to create an open lower level that will keep hydrodynamic loads on the structure to a minimum. This includes using circular shaped columns.
- *Location 5 (Figure B-10).* Additional strength can be provided in the system by using walls that parallel the anticipated direction of the tsunami inundation flow.
- *Location 6 (Figure B-9).* The stairs structures can be integrated with the primary structure to provide additional strength, or they can be made structurally independent.

Appendix C
Example Calculations

A rectangular-shaped tsunami evacuation structure, 10 m wide, is constructed at a site 200 m from the shoreline, where the elevation is 4 m from the sea level. The local beach slope is 1/50 and there is no significant alongshore variation in the topography. The tsunami inundation map indicates the elevation $R^* = 10$ m at the maximum inundation point (runup height of 10 m at the location 500 m from the shoreline). A log (8.53 m long, 0.35 m in diameter, and 450 kg mass) is considered as the design waterborne missile for the impact loading. In addition, the impact loading of a 40-ft shipping container (40 ft L x 8 ft W x 8-1/2 ft H, or 12.2 m x 2.44 m x 2.59 m) is estimated. A definition sketch for these example calculations is provided in Figure C-1.

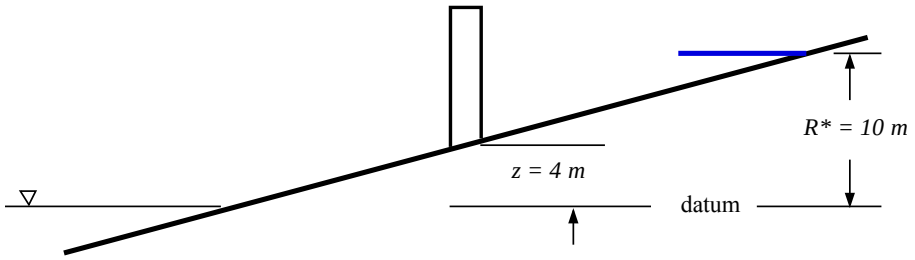


Figure C-1 Definition sketch for example calculations: R^* is the maximum runup elevation (the maximum inundation distance is 500 m) and z is the elevation at the location of the tsunami evacuation structure (located 200 m from the shoreline). Two horizontal lines represent the initial water level and the maximum inundation level, respectively.

If a reliable and accurate tsunami inundation numerical model satisfying the criteria in Chapter 3 has been used to estimate flow depth and velocity at the building location, then the numerical data should be used for the force evaluations. At a site of interest, the following parameters should be extracted from the numerical simulation: the maximum inundation depth h_{max}, the maximum flow speed of the depth greater than the debris draft u_{max}, and the maximum value of the product, $(hu)^2_{max}$. The local effects of the tsunami flows are difficult to predict due to nonlinear interactions of three-dimensional flows. It is recommended that the design inundation elevation be increased at the building site by 30% over the computed inundation elevation, that the design flow velocity be increased by 15%, and that the

momentum flux (hu^2) be increased by 70% over the computed values for conservatism. These safety factors are simply a guideline based on the 30% error band in modeled tsunami runup heights compared with observed runup heights from past tsunami survey data. In practice, the safety factors should be determined based on the confidence in accuracy of the numerical simulations. Once the design flow parameters are determined, then the forces can be calculated using the methods described below.

The following calculations are for situations where no detailed numerical simulation data are available. In such cases, it is assumed that the only information available at a site of interest is an inundation map, and the forces are intended to be conservatively estimated.

C.1 Inundation Depth

The recommended design runup height, R, is 30% greater than the predicted maximum runup elevation, R^*, to account for local amplification and uncertainty in the predicted value, i.e., $R = 1.3\ R^* = 13$ m. Therefore, the design inundation depth at the structure is 13 – 4 = 9 m. A minimum freeboard of 3 m (10 ft) or one story height is recommended. If the typical floor height is 4 m, the refuge area must be located higher than 9 + 4 = 13 m above the ground level. This would imply that the refuge area should be located on the 4th floor or higher. Note that when numerical simulation data are available, the inundation depth at the site can be obtained directly. However, it is still recommended that a 30% safety factor be applied to the computed inundation elevation at the site.

C.2 Hydrostatic and Buoyant Forces

It is recommended that all nonstructural walls at the lower levels of the building be designed as breakaway walls. In that case, the hydrostatic forces and potential uplift of the overall building are significantly reduced. However, if the structure, or any portion of the structure, is constructed watertight at the lower levels, then the wall panels must be designed for the anticipated hydrostatic pressure. The maximum force acting on a wall panel of 4 m wide and 3 m tall on the ground floor can be computed using Equation 6-2. Since the wall panel on the ground floor is fully submerged:

$$F_h = \rho_s g\left(R - (z + \Delta z) - \frac{h_W}{2}\right) h_W\, b$$

$$= \left(1100\ \text{kg/m}^3\right)\left(9.81\,\text{m/sec}^2\right)\left(1.3 \times 10\,\text{m} - (4\,\text{m} + 0.5\,\text{m}) - \frac{3\,\text{m}}{2}\right)(3\,\text{m})(4\,\text{m})$$

$$= 906\,\text{kN}$$

or an average lateral pressure of

$$P_h = 906 \text{ kN}/12 \text{ m}^2 = 75.5 \text{ kPa}$$

where Δz is the height at the toe of the wall panel from the ground level, assumed to be 0.5 m. Note that the fluid density $\rho = 1.1\ \rho_{water}$ is used assuming a mixture of seawater and sediment.

With the water level at 9 m at the building location, the first and second floors will be submerged. Assuming the nonstructural walls have broken away at these two levels, but not yet at the third level, then the uplift due to buoyancy acting on the third floor should be evaluated. Assuming plan dimensions of 5 m by 5 m for a typical floor panel on the third floor, and a floor elevation of 7 m above the ground level, as shown in Figure C-2, then the upward buoyant force can be computed using Equation 6-4:

$$\begin{aligned} F_b &= \rho_s g A_f h_b \\ &= (1100 \text{ kg/m}^3)(9.81 \text{m/sec}^2)(5 \text{ m} \times 5 \text{ m})((1.3 \times 10 \text{ m} - 4 \text{ m}) - 7 \text{ m}) \\ &= 540 \text{ kN} \end{aligned}$$

or an upward pressure of

$$P_h = 540 \text{ kN}/25 \text{m}^2 = 21.6 \text{ kPa}$$

where h_b is the water height displaced by the floor including the effect of air trapped below the floor, as shown in Figure C-2.

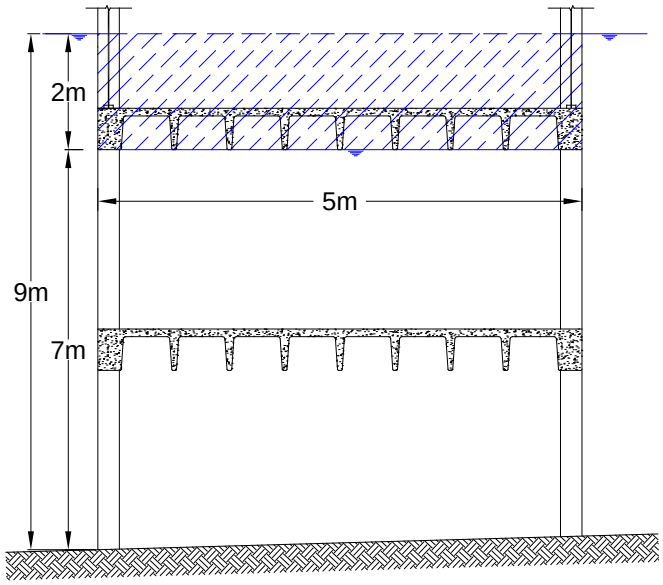


Figure C-2 Condition resulting in buoyant forces.

C.3 Hydrodynamic and Impulsive Forces

Hydrodynamic drag and impulse forces are exerted on the building as a whole, assuming no breakaway walls at the lower levels. The maximum value of $h\,u^2$ at the site can be computed using Equation 6-6, with z = 4 m, R = 13 m and g = 9.81 m/sec^2:

$$\left(hu^2\right)_{\max} = gR^2\left(0.125 - 0.235\frac{z}{R} + 0.11\left(\frac{z}{R}\right)^2\right) = 105\ \text{m}^3/\text{sec}^2$$

Hence, from Equation 6-5 the fluid force is:

$$\begin{aligned} F_d &= \frac{1}{2}\rho_s C_d (hu^2)_{\max} \\ &= \frac{1}{2}(1100\ \text{kg/m}^3)(2.0)(10\ \text{m})(105\ \text{m}^3/\text{sec}^2) \\ &= 1155\ \text{kN} \end{aligned}$$

where B = 10 m (shelter width), and C_d = 2.0. If the worst-case tsunami arrives at a previously flooded site, then the tsunami front may form a bore. The impulsive force for this condition would be 1.5 times the hydrodynamic force, as in Equation 6-7:

$$F_s = 1.5F_d = 1730\ \text{kN}$$

If the nonstructural walls at the lower level are designed to break away during a tsunami, then the hydrodynamic drag and impulse forces would be computed for all individual structural members (e.g., columns, shear walls) and combined as shown in Figure 6-12.

C.4 Impact Force

The maximum flow velocity at the site can be estimated using R = 13 m in Equation 6-9:

$$\begin{aligned} u_{\max} &= \sqrt{2gR\left(1 - \frac{z}{R}\right)} \\ &= \sqrt{2g(13\ \text{m})\left(1 - \frac{4\,\text{m}}{13\,\text{m}}\right)} = 13.3\ \text{m/sec}. \end{aligned}$$

Note that this flow velocity is at the leading tongue of the flow where the flow depth is nil. Hence, this value of approximately 48 km/hr (30 mph) will be conservative. Using this conservative velocity estimate, the impact force due to a floating log can be computed by Equation 6-8, with c= 0, k = 2.4 x 10^6 N /m, and m = 450 kg:

$$F_i = 1.3u_{\max}\sqrt{km_d(1+c)}$$

$$= 1.3(13.3 \text{ m/sec})\sqrt{(2.4\times10^6 \text{ N/m})(450 \text{ kg})(1+0)}$$

$$= 568 \text{ kN}$$

This force would be applied locally at the assumed point of impact.

If the assumed draft, d, of the log is 0.25m, then the velocity is evaluated using Figure 6-9. Using the ratios $\zeta = z/R = 0.31$, and the flow depth, $d/R = 0.019$, at the location of the site:

$$\frac{u_{\max}}{\sqrt{2gR}} = 0.53$$

$$u_{\max} = 0.53\sqrt{2(9.81)(13)} = 8.5 m/\text{sec}$$

The impact force is then:

$$F_i = 1.3u_{\max}\sqrt{km_d(1+c)}$$

$$= 1.3(8.5 \text{ m/sec})\sqrt{(2.4\times10^6 \text{ N/m})(450 \text{ kg})(1+0)}$$

$$= 363 \text{ kN}$$

which is more realistic than the previous estimate (568 kN). The total force on the structure at the time of the impact can be determined conservatively by combining this impact force with the hydrodynamic drag force determined earlier:

$$F_i + F_d = 363 + 1155 = 1518 \text{ kN}$$

To compute the impact force due to a floating shipping container, the draft, d, must be estimated:

$$d = \frac{m_d g}{\rho_s g A_h}$$

$$= \frac{(3800 \text{ kg})g}{(1100 \text{ kg/m}^3)g(12.2 \text{ m}\times 2.44 \text{ m})} = 0.116 \text{ m}$$

where m_d is the weight (Table 6-1) and A_h is the cross sectional area of the box in the horizontal plane, and the constant g cancels out. Considering the configuration of the support frame at the bottom of the container and the large horizontal dimension, the container is assumed to float freely at $d = 0.5$ m. The maximum flow velocity that supports a draft, $d = 0.5$ m, can be found from Figure 6-9. At the location of the site, $\zeta = z/R = 0.31$, and the flow

depth, $d/R = 0.039$. Figure 6-9 shows $\upsilon = 0.31$. Hence, the maximum velocity is:

$$u_{\max} = 0.31\sqrt{2gR} = 5.0 \text{ m/sec}$$

Note that Figure 6-9 is only valid near the leading tip of the runup, therefore, use of a numerical model to estimate inundation flow depth and velocity is encouraged for large and heavy debris objects.

With a debris velocity of 5 m/s at impact, the impact force due to a longitudinal strike by the shipping container is computed by Equation 6-8 with $c = 0.2$, $k = 60 \times 10^6$ N /m, and $m_d = 3800$ kg (Table 6-1):

$$F_i = 1.3u_{\max}\sqrt{km_d(1+c)}$$

$$= 1.3(5.0 \text{ m/sec})\sqrt{(60\times10^6 \text{ N/m})(3800 \text{ kg})(1+0.2)}$$

$$= 3400 \text{ kN}$$

The impact force due to a transverse strike by the shipping container is computed by Equation 6-8 with $c = 1.0$, $k = 30 \times 10^6$ N /m, and $m_d = 3800$ kg (Table 6-1):

$$F_i = 1.3u_{\max}\sqrt{km_d(1+c)}$$

$$= 1.3(5.0 \text{ m/sec})\sqrt{(30\times10^6 \text{ N/m})(3800 \text{ kg})(1+1)}$$

$$= 3100 \text{ kN}$$

These are large forces compared with the hydrodynamic drag determined earlier. They represent a conservative assumption that the container is traveling at high velocity and applies a direct strike to the building. Unless the site is located adjacent to a container storage yard, this is a low probability event. The incorporation of progressive collapse prevention in the building design is intended to protect against failure of an exterior column or wall element due to this low probability impact.

C.5 Damming Effect of Waterborne Debris

The damming effect of debris can be computed using Equation 6-11, which is readily obtained from the hydrodynamic force computed earlier, substituting the recommended debris dam width of 12 m (40 ft):

$$F_{dm} = (1155 \text{ kN}) \times \left(\frac{12 \text{ m}}{10 \text{ m}}\right) = 1386 \text{ kN}$$

If the building were wider than 12 m, then the damming effect should be considered at various locations as shown in Figure 6-13 to determine the

worst condition for loading on the structure as a whole, and on individual structural elements.

C.6 Hydrodynamic Uplift Forces

The hydrodynamic uplift force can be computed using Equation 6-14. Assuming that the water depth at the soffit of the second floor is h_s = 3 m, and at the location of the shelter site, $\zeta = z/R = 0.31$, and the flow depth, $d/R = h_s/R = 0.23$, Figure 6-9 shows υ along the limit curve at $\zeta = 0.31$. Hence, the maximum velocity is:

$$u = 0.15\sqrt{2gR} = 2.4\,\text{m/sec}.$$

The vertical velocity can be computed using Equation 6-16, assuming the slope at the site is 1/20:

$$u_v = u\tan\alpha = (2.4)(1/20) = 0.12\ \text{m/sec}$$

Hence, the hydrodynamic uplift force given by Equation (6-14) is:

$$\begin{aligned} F_u &= \frac{1}{2} C_u \rho_s A_f u_v^2 \\ &= \frac{1}{2}(3)\left(1100\ \text{kg/m}^3\right)(5\,\text{m} \times 5\,\text{m})(0.12\,\text{m/sec})^2 \\ &= 594\ \text{N} \end{aligned}$$

which is insignificant for the beach slope assumed in this example. If a beach slope of 1/5 is assumed, the hydrodynamic uplift force increases to 9.5 kN or an uplift pressure of 0.38 kPa on the bottom of the floor slab.

Appendix D
Background Information on Impact Load Calculations

D.1 Available Models for Impact Loads

The impact force from waterborne debris (e.g., floating driftwood, lumber, boats, shipping containers, automobiles, and other buildings) can be a cause of structural damage or even building destruction. Unfortunately, it is difficult to estimate this force accurately. Unlike the other forces, the impact force occurs locally at the point of contact when the debris is smaller than the building. Impact forces can be assumed to act at or near the water surface level when the debris strikes the building. Most available models are based on the impulse-momentum concept, in which the impulse of the resultant force acting for an infinitesimal time is equal to the change in linear momentum:

$$I = \int_0^{\tau} F\,dt = d(mu); \quad \tau \to 0 \qquad \text{(D-1)}$$

where:

I = impulse

F = resultant force

m = mass of waterborne debris

u = velocity of the debris

t = time

For actual computations, a small but finite time, Δt (not infinitesimal), and the average change in momentum are used as an approximation. There is significant uncertainty in evaluating the duration of impact, Δt. The following are available formulae for debris-impact force estimation.

Matsutomi (1999). Matsutomi experimentally investigated the impulse forces of driftwood. He performed two sets of experiments: one in a small water tank and the other for full-scale impact in air. In his small water tank, a bore and a surge were generated (a bore is a moving hydraulic jump onto a quiescent shallower water in front of it, while a surge is a moving water body onto a dry bed). A scaled-down driftwood model was placed 2.5 m upstream from the receiving wall. The model driftwood was picked up by the generated bore (or surge) and impacted onto the receiving vertical wall. His

full-scale impact experiments were conducted to compensate for potential scale effects in his small-scale experiments. A full-scale log was tied at the end of a pendulum and was swung against a stationary stop equipped with a load cell. It is noted that this impact condition in the air may significantly differ from an actual waterborne case because of the absence of the added mass effect of water: prior to the impact, the waterborne debris is carried by the surrounding water flow and the momentum of the water may increase or decrease the impact force. Matsutomi then compensated for the added mass effect with the data obtained from the small-scale water tank experiments. Based on a regression analysis of the large amount of data, Matsutomi proposed Equation D-2 for the impact force, F:

$$\frac{F}{\gamma_w D^2 L} = 1.6 C_M \left(\frac{u}{\sqrt{gD}} \right)^{1.2} \left(\frac{\sigma_f}{\gamma_w L} \right)^{0.4} \quad \text{(D-2)}$$

where:

γ_w = the specific weight of the log,

D and L = the diameter and the length of the log, respectively,

$C_M = 1 + C_a$, is an inertia coefficient,

C_a is the added mass coefficient based on the displaced fluid volume,

u = the velocity of the log at impact, and

σ_f = the yield stress of the wood.

Matsutomi recommended $\sigma_f = 20 \times 10^6$ Pa for a wet log. The equation applies when driftwood collides at almost right angles with "rigid" structures such as reinforced concrete buildings.

From small-scale experimental data, he recommended a value of $C_M \approx 1.7$ for driftwood located at the tip of the inundation flow or strong bore condition, and $C_M \approx 1.9$ for a steady flow or if the log is located behind the tip of the inundation flow or strong bore. Note that the recommended values of C_M are the upper limit when more than 60% of the receiving wall is open and permeable. The value of C_M is smaller when the receiving wall does not allow the flow to pass through. For a solid (impermeable) receiving wall, Matsutomi found that $C_M = 0.5$ for a bore and $C_M = 1.1$ for a surging flow. Note that in the case of a bore striking an impermeable wall (i.e., no flow-through), C_M is less than unity (= 0.5). This is because the flow reflection at the wall actually reduces the impact force.

In spite of a thorough study with a large amount of laboratory data, the derived form of Equation D-2 is inconvenient due to the particular choice of the scaling parameters, and it is only applicable to driftwood or logs.

Ikeno et al. (2001, 2003). Laboratory experiments similar to Matsutomi (1999) were performed to examine the impact forces of objects other than driftwood or logs. They used cylindrical, square column, and spherically-shaped drift bodies. Note that unlike Matsutomi's experiments, Ikeno et al. only examined the impact onto an impermeable vertical wall. The following empirical formula was derived based on small-scale experiments (approximately 1/100 model):

$$\frac{F}{gm} = SC_M \left(\frac{u}{\sqrt{g\sqrt{DL}}} \right)^{2.5} \tag{D-3}$$

where:

S = a constant (equal to 20 for a bore case),

C_M = $1 + C_a$ is the inertia coefficient

C_a is the added mass coefficient based on the displaced fluid volume

m = the mass of the drift body.

C_M = 0.5 was used regardless of the shape of the objects for a bore impact onto an impermeable wall, which was adopted from Matsutomi's results. For a dry-bed surge, Ikeno and Tanaka (2003) suggested S = 5 and C_M = 0.8 for spherical-shaped objects and C_M = 1.5 ~ 2.0 for cylinders and square-shaped columns. The results by Ikeno et al. are valid only for the condition of an impermeable wall (i.e., the entire incident flow reflects back to the offshore direction). This is why the inertia coefficient has a value less than unity.

Haehnel and Daly (2002). At the U.S. Army Cold Regions Research and Engineering Laboratory (CRREL), Haehnel and Daly performed experiments similar to Matsutomi (1999). They considered reduced-scale logs in steady flow in a small flume, and prototype logs in a large towing basin. It must be noted that the condition in the towing basin differs from the actual impact condition of a waterborne object. In the towing basin the water is stationary while in the actual condition moving water carries the debris. Instead of the impulse-momentum approach, Haehnel and Daly analyzed the data using linear dynamic analysis with one degree of freedom. Since the collision occurs over a short duration, damping effects are neglected. Assuming the overall structural system has a period much greater than the impact duration, approximating a rigid structure, the model can be formulated by Equation D-4:

$$m\ddot{x} + kx = 0 \tag{D-4}$$

where:

m = the mass of the log,

x = the summation of the compression of the building and the log during impact and rebound, with the dot denoting the time derivative, and

k = the effective stiffness associated with both the log and the building.

The effective stiffness of the collision is $1/k = 1/k_s + 1/k_d$ where k_s is the local stiffness of the structure at the impact zone; and k_d is the stiffness of the debris, and other nonstructural elements deformed at impact. The structure will be rigid if the structure stiffness is much greater than the stiffness of the target zone nonstructural elements or the debris. The structure will also act as if it is rigid if the mass of the structure is so great that it does not move appreciably in response to the impact.

Solving Equation D-4 yields the maximum force given by Equation D-5:

$$F_{max} = \mathrm{Max}(kx) = u\sqrt{km} \tag{D-5}$$

where: u is the impact velocity.

Based on their laboratory experiments, the effective stiffness k between a log and a rigid building was estimated to be 2.4×10^6 N/m.

Haehnel and Daly demonstrated that the impulse-momentum approach could be reduced to the constant-stiffness approach shown in Equation D-5 by setting $\Delta t = \frac{\pi}{2}\sqrt{\frac{m}{k}}$ (note that, to be consistent to Equation D-4, the force is considered a sinusoidal function in time). The work-energy approach can also be made equivalent to Equation D-5 by setting the stopping distance as $S = u\sqrt{\frac{m}{k}}$. The work-energy approach is an impact force estimation that equates the work done on the building with available kinetic energy of the floating debris object. Based on their laboratory data, the following formulae were suggested by Haehnel and Daly:

Constant-stiffness approach:

$$F_{max} = \mathrm{Max}(kx) = u\sqrt{km} \approx 1550u\sqrt{m} \tag{D-6}$$

Impulse-momentum approach:

$$F_{max} = \frac{\pi}{2}\frac{u\,m}{\Delta t} \approx 90.9\,u\,m \qquad \text{(D-7)}$$

Equivalent to $\Delta t = 0.0173$ sec.

Work-energy approach:

$$F_{max} = \frac{u^2\,m}{\Delta x} \approx 125\,m\,u^2 + 8000 \qquad \text{(D-8)}$$

Note that in Equations D-6, D-7, and D-8, the velocity, *u*, is in m/sec and the mass, *m*, is in kg. It is emphasized that errors associated with the use of a towing tank (instead of the realistic condition of a log being carried with flow) may be significant in the results by Haehnel and Daly (2002), since the added mass effect of water flowing with the debris is not included. To include the added mass effect that opposes the direction of acceleration, the single degree of freedom equation of motion D-4 above (Equation D-4) would be modified to:

$$m\ddot{x} + kx = -m_a\ddot{x}$$

and therefore,

$$(m + m_a)\ddot{x} + kx = 0 \qquad \text{(D-9)}$$

where m_a is the hydrodynamic mass given by;

$$m_a = cm_{disp} = c\rho(\text{displaced volume})$$

where *c* is the hydrodynamic mass coefficient.

Note that this application of hydrodynamic mass to the solution of the structural dynamics equation of debris impacts in water no longer represents the traditional added-mass term derived from potential flow hydrodynamics. However, both are similar transitory impulsive effects and are more related to the shape and orientation of the body than a property of its true mass. To avoid confusion, the term "hydrodynamic mass" is used in Equation D-9 instead of the term "added mass."

Solving Equation D-9 yields the maximum force given by Equation D-10:

$$F_{\max} = u\sqrt{k(m + m_a)} = u\sqrt{km(1 + c)} \qquad \text{(D-10)}$$

SEI/ASCE Standard 7-10 (ASCE, 2010). ASCE gives the following modified design formula based on Equation D-1:

$$F = \frac{\pi\,m\,u\,C_I\,C_O\,C_D\,C_B\,R_{max}}{2\,\Delta t} \qquad \text{(D-11)}$$

where:

m = the water-borne-debris mass,

u = the impact velocity of the debris,

C_I = the importance coefficient,

C_O = the orientation coefficient, (statistically based)

C_D = the depth coefficient,

C_B = the blockage coefficient,

R_{max} = the maximum response ratio for impulsive load, and

Δt = the impact duration.

The C coefficients are based on results of laboratory testing and on engineering judgment. R_{max} is a coefficient to compensate for the effect of the degree of flexibility of the building. A single value of the impact duration, Δt = 0.03 sec, is recommended (Kriebel, et al, 2000), although there is wide variation in the impact duration owing to, for example, the object material and deformability, the flow blockage condition, and the flexibility of the building element being struck. It is worth noting that the *City and County of Honolulu Building Code* (CCH, 2000) recommends Δt values for wood construction as 1.0 sec, steel construction as 0.5 sec, and reinforced concrete as 0.1 sec; these values are unsubstantiated. Furthermore, the FEMA 55 *Coastal Construction Manual* (FEMA, 2005) provides Δt values shown in Figure D-1. Such an excessive variation in Δt could make Equation D-11 unreliable if used with various prescriptive impact duration values. Improved results may result with explicit calculation of the duration per

$$\Delta t = \frac{\pi}{2}\sqrt{\frac{m}{k}}.$$

Type of Construction	Duration (t) of Impact (sec)	
	Wall	Pile
Wood	0.7 – 1.1	0.5 – 1.0
Steel	NA	0.2 – 0.4
Reinforced Concrete	0.2 – 0.4	0.3 – 0.6
Concrete Masonry	0.3 – 0.6	0.3 – 0.6

NA - Not Applicable

Figure D-1 Ranges of duration of impact (FEMA, 2005).

D.2 Summary and Discussion

Review of previous work clearly demonstrates the uncertainty of the present understanding of waterborne debris-impact forces. The form of Equation D-11 exhibits a struggle to obtain an engineering estimate of the forces by

adjusting five coefficients based on engineering judgment, together with a single estimate for Δt. All of the prediction formulae are based on small-scale laboratory data by compensating with the full-scale measurements in compromised conditions. For example, Matsutomi's full-scale data were obtained by the impact study in air, and Haehnel and Daly's data were obtained in a towing tank. Since the added mass effect appears important at the impact (the impact halts not only the waterborne debris itself but also decelerates a portion of the water caused to flow around it), the results derived from the compromised experimental conditions may contain significant errors.

Even if the impact velocity, u, and the debris mass, m, were given, each formula yields a different functional relation to predict the forces, which indicates complexity and uncertainty inherent in the problem. For each of the available methods, proportionality between the impact force, debris velocity and mass are:

$$\begin{aligned}
&\text{Constant-stiffness approach} \Rightarrow F \propto u\sqrt{m}\,, \\
&\text{Impulse-momentum approach} \Rightarrow F \propto u\,m\,, \\
&\text{Work-energy approach} \Rightarrow F \propto u^2\,m\,, \\
&\text{Ikeno and Tanaka (2003)} \Rightarrow F \propto u^{2.5} m^n\,,\ n \approx 0.58, \text{ and} \\
&\text{Matsutomi (1999)} \Rightarrow F \propto u^{1.2} m^n\,,\ n \approx 0.66.
\end{aligned} \qquad \text{(D-12)}$$

Although Equation D-2 by Matsutomi is based on his substantial analyses of a large set of laboratory data, the form of Equation D-2 is physically ambiguous in terms of the choice of the scaling parameters, is limited only to cylindrical shaped debris, and is inconvenient for use in actual practice. The empirical Equation D-3 by Ikeno et al. is based on their small-scale laboratory experiments with an impermeable wall; hence, its extrapolation is unreliable for most real-world applications. Proper estimates of Δt and Δx are uncertain for the impulse-momentum and work-energy approaches, respectively. The value of the effective constant stiffness, k, should be evaluated when using Haehnel and Daly's Equation D-5. In reality, k is not constant; it is likely a function of x during the impact.

Until more comprehensive studies can be made, an effective stiffness approach given in Equation D-10, based on Haehnel and Daly, is recommended because of its simple but rational formulation. In addition, as shown in the foregoing comparisons in Equations D-12, the functional relation of m and u to the force F is similar to Matsutomi's empirical Equation D-2, which was derived based on a very large amount of experimental data. Considering that Matsutomi's empirical treatment was

based on the impulse-momentum approach, the coincidental similarity with the constant-stiffness approach provides additional confidence in the formulation. With the introduction of the hydrodynamic mass parameter, *c*, the hydrodynamic mass effect is already included in Equation D-10. Applying an importance factor of 1.3 per ASCE 7-10 for critical facilities results in the recommended impact expression as shown in Equation D-13:

$$F_i = 1.3u_{\max}\sqrt{km_d(1+c)} \tag{D-13}$$

In this expression, *k* must be determined along the direction of the impact based on the modeled debris (e.g., as mentioned earlier, $k = 2.4 \times 10^6$ N/m was recommended for a log by Haehnel and Daly). Note that a proper estimate of *k* is the key for this method. In reality, *k* may not be the elastic stiffness; it is likely a function of *x* during inelastic impacts of structural significance. Hence, the linearized equation D-4 may be inadequate. Engineering judgment and iterative analysis may be necessary to determine the most appropriate secant stiffness to be used for a particular magnitude of impact. The values for *k* suggested in Table 6-1 for shipping containers were developed based on computer models of standard containers (Peterson and Naito, 2012). An added advantage for the use of Equation D-13 is that *k* is not as sensitive as Δx in the work-energy approaches, which can be shown from the fact that Δx is proportional to $\sqrt{1/k}$, as discussed earlier.

The hydrodynamic mass coefficient, *c*, is due to the fact that the decelerating body must also momentarily decelerate or disturb some volume of the surrounding fluid flow. It depends greatly on the size, shape, and orientation of the object with respect to the surge direction. Estimated values for *c* are provided in Table 6-1 for the various debris strike conditions considered.

Appendix E

Maximum Flow Velocity and Momentum Flux in the Tsunami Runup Zone

E.1 Flow Velocity

For prediction of flow velocities and depths at a site of interest for a given design tsunami, the best practice available is to run a detailed numerical simulation model with a very fine grid size (less than 10 meters) in the tsunami runup zone. Such a numerical model is usually run with a nested grid system with a grid size of several kilometers in the abyssal plain, a few hundreds of meters on the continental shelf, a few tens of meters near the shore, and less than 10 meters in the runup zone. A numerical simulation can provide the complete time history of flow velocity and depth at the site of interest.

Alternatively, the use of analytical solutions can be considered. Although some simplifications and assumptions must be imposed, the results are useful as a guideline for checking the reasonableness of results, or as estimate of approximate values in the absence of other information. Available analytical solutions are based on one-dimensional, fully nonlinear shallow-water-wave theory for the condition with a uniformly sloping beach. With those assumptions, the exact solution for the runup of an incident bore was given by Shen and Meyer (1963), based on Ho and Meyer (1962). The maximum fluid velocity occurs at the leading runup tip as calculated by Equation E-1:

$$u = \sqrt{2 g x \tan \alpha} \ , \qquad \text{(E-1)}$$

where:

α = the beach slope,

g = the gravitational acceleration, and

x = the distance from the maximum runup location to the location of interest; the location of interest must be above the initial shoreline.

Results indicate that the flow at the leading runup tip moves up the beach under gravity, just like a particle with simple energy transfer between its kinetic and potential energies. According to Yeh (2006), Equation E-1

provides the upper-limit envelope of the flow velocity for all incident tsunami forms. Because a real beach is not uniformly sloped, it is more convenient to present Equation E-1 as a function of the ground elevation, instead of distance as follows:

$$u_{\max} = \sqrt{2gR\left(1 - \frac{z}{R}\right)} \tag{E-2}$$

where:

R = the ground elevation at the maximum penetration of tsunami runup, measured from the initial shoreline, and

z = the ground elevation of the location of interest, measured from the initial shoreline level.

It is emphasized that the model does not include the effects of friction and the maximum flow velocity occurs at the leading runup tip, where the flow depth is zero. Since debris requires some finite flow depth in order to float (draft), use of Equations E-1 and E-2 to estimate velocity for impact load calculations is overconservative.

Based on Shen and Meyer's (1963) results, Peregrine and Williams (2001) provided the formulae for the temporal and spatial variations in fluid velocity and flow depth of the incident bore runup in the vicinity of the leading runup tip. With slightly different scaling, Yeh (2007) expressed Peregrine and Williams' formulae for the flow depth and velocity, respectively as follows:

$$\eta = \frac{1}{36\tau^2}\left(2\sqrt{2}\,\tau - \tau^2 - 2\zeta\right)^2 \tag{E-3}$$

and

$$\upsilon = \frac{1}{3\tau}\left(\tau - \sqrt{2}\,\tau^2 + \sqrt{2}\,\zeta\right) \tag{E-4}$$

where, in the above equations:

$$\eta = \frac{d}{R};\quad \upsilon = \frac{u}{\sqrt{2gR}};\quad \tau = t\tan\alpha\sqrt{\frac{g}{R}};\quad \zeta = \frac{z}{R}$$

d = the water depth,

R = the ground elevation at the maximum penetration of tsunami runup, measured from the initial shoreline,

u = the flow velocity,

g = the gravitational acceleration,

α = the beach slope,

t = the time: 0 when the bore passes at the initial shoreline, and

z = the ground elevation of the location of interest, measured from the initial shoreline: this identifies the location of interest along a uniformly sloping beach.

For a given maximum runup penetration, a bore formation should yield the fastest flow velocity among all the incident tsunami formations. Gradual flooding of non-breaking tsunamis should result in slower flow velocity than that caused by the bore runup. Therefore, Equations E-3 and E-4 can be used to estimate the maximum flow velocity at a given location for a given flow depth. Combining Equations E-3 and E-4 and eliminating τ, Figure E-1 can be derived. Each curve in the figure represents the dimensionless flow velocity υ versus the location ζ (in terms of ground elevation, z) for a given local flow depth, d. This figure can be used to evaluate the maximum flow velocity that can carry floating debris with finite draft depth, since draft of the debris must be greater than the flow depth to make the debris float. Equations E-3 and E-4 are valid only for the flows very close to the leading runup tip. Therefore, the velocity estimated for a case with a sufficiently large draft depth can be overly conservative.

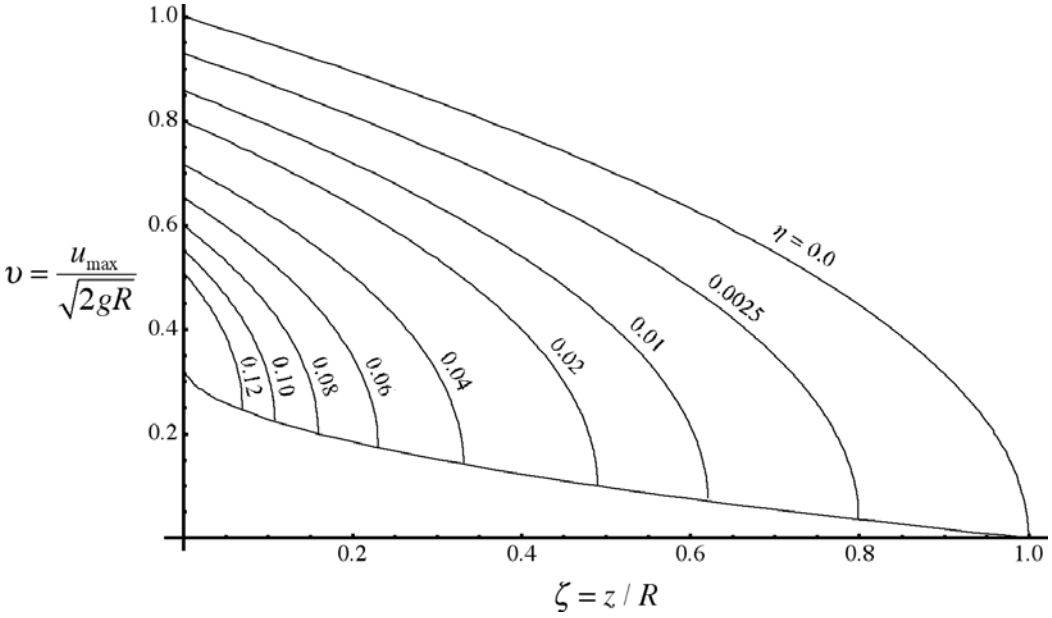


Figure E-1 Maximum flow velocity of depth, d, at the ground elevation, z, and maximum runup elevation, R. The bottom curve represents the lower limit of maximum flow velocity.

The bottom curve in Figure E-1 is the lower limit of the maximum flow velocity for a given depth, d. Note that the results in Figure E-1 are based on the runup condition of uniform incident bore. Local inundation depth of other tsunami forms usually exceeds that of a bore runup, and the maximum flow velocity is lower than the limit curve in Figure E-1. Hence when a floating-debris has a draft that exceeds the flow depth of the bore runup, the

design velocity u_{max} can be estimated conservatively with the lower limit curve.

E.2 Momentum Flux

When a detailed numerical simulation model is available, the critical values of forces can be evaluated directly for a location of interest. The maximum value of the product of water depth and the square of flow velocity hu^2 is needed to compute the hydrodynamic forces.

Even in the case of no numerical simulation data, the maximum momentum flux per unit water mass per unit width hu^2 can be estimated conservatively once the maximum runup height (or distance) is determined. Using the exact solution algorithm, Yeh (2006) developed an envelope curve of hu^2, expressed in Equation E-5:

$$\frac{hu^2}{g\alpha^2\ell^2} = 0.11\left(x/\ell\right)^2 + 0.015\left(x/\ell\right) \qquad \text{(E-5)}$$

where:

hu^2 = the momentum flux per unit mass per unit width,

α = the beach slope,

g = the gravitational acceleration,

x = the distance from the maximum runup location to the location of interest (the location of interest must be above the initial shoreline), and

ℓ = the maximum runup distance.

Once the maximum runup distance, ℓ, is determined (e.g., from an available inundation map), the momentum flux, ρhu^2 per unit breadth at a given location x, can be computed by Equation E-5. It is emphasized that Equation E-5 is for a uniform beach slope; therefore, some adjustments need to be made to evaluate realistic conditions. Because a real beach is not uniformly sloped, it is more convenient to express Equation E-5 as a function of ground elevation instead of distance, as follows:

$$\frac{hu^2}{gR^2} = 0.125 - 0.235\frac{z}{R} + 0.11\left(\frac{z}{R}\right)^2 \qquad \text{(E-6)}$$

where:

hu^2 = the momentum flux per unit mass per unit width,

g = the gravitational acceleration,

R = the ground elevation at the maximum penetration of tsunami runup, measured from the initial shoreline, and

z = the ground elevation of the location of interest, measured from the initial shoreline: this identifies the location of interest along a uniformly sloping beach.

Although a real beach is not uniformly sloped and tsunami runup is not a one-dimensional motion, Figure E-1 and Equations E-2 and E-6 provide an analytical basis for runup conditions.

Glossary

The following definitions are provided to explain the terms and acronyms used throughout this document. Many have been taken directly from the FEMA 55, *Coastal Construction Manual* (FEMA, 2005).

A

ADA – Americans with Disabilities Act. Law requiring that design accommodations be made for persons with certain disabilities.

Armor – Material used to protect slopes from erosion and scour by floodwaters, such as riprap, gabions, or concrete.

ASCE – American Society of Civil Engineers.

ATC – Applied Technology Council.

A-Zone – Under the National Flood Insurance Program, the area subject to inundation by a 100-year flood where waves are less than 3 feet high [designated Zone A, AE, A1-A30, A99, AR, AO, or AH on a Flood Insurance Rate Map (FIRM)].

B

Base flood – Flood that has a 1% probability of being equaled or exceeded in any given year, also known as the 100-year flood.

Base Flood Elevation (BFE) – Elevation of the base flood in relation to a specified datum, such as the National Geodetic Vertical Datum or the North American Vertical Datum. The Base Flood Elevation is the basis of the insurance and floodplain management requirements of the National Flood Insurance Program.

Bathymetry – Underwater configuration of a bottom surface of an ocean, estuary, or lake.

Berm – A mound of soil or other earthen material.

Bore – A long, broken wave propagating into a quiescent body of water, with an abrupt increase in water depth at its front face covered with turbulent, tumbling water.

Breakaway wall – Under the National Flood Insurance Program, a wall that is not part of the structural support of the building and is intended, through its design and construction, to collapse under specific lateral loading forces without causing damage to the elevated portion of the building or supporting foundation system. Breakaway walls are required by the National Flood Insurance Program regulations for any enclosures constructed below the Base Flood Elevation beneath elevated buildings in coastal high-hazard areas (also referred to as V-Zones). In addition, breakaway walls are recommended in areas where floodwaters flow at high velocities or contain ice or other debris.

Building codes – Regulations adopted by local governments that establish standards for construction, modification, and repair of buildings and other structures.

Building official – An officer or other designated authority charged with the administration and enforcement of the code, or a duly authorized representative such as a building, zoning, planning, or floodplain management official.

Bulkhead – A wall or other structure, often of wood, steel, stone, or concrete, designed to retain or prevent sliding or erosion, and occasionally used to protect against wave action.

C

CAEE – Canadian Association for Earthquake Engineering.

Cast-in-place concrete – Concrete that is formed, placed, and cured in its final location in the structure.

Cladding – Exterior surface of the building envelope.

Coastal A-Zone – The portion of the Special Flood Hazard Area landward of a V-Zone or landward of an open coast without mapped V-Zone in which the principal sources of flooding are astronomical tides, storm surge, seiches, or tsunamis (not riverine sources). The flood forces in coastal A-Zones are highly correlated with coastal winds or coastal seismic activity. Coastal A-Zones may therefore be subject to wave effects, velocity flows, erosion, scour, or combinations of these forces. (Note: National Flood Insurance Program regulations do not differentiate between coastal A-Zones and non-coastal A-Zones.)

Coastal barrier – Depositional geologic features such as a bay barrier, tombolo, barrier spit, or barrier island that consists of unconsolidated

sedimentary materials; is subject to wave, tidal, and wind energies; and protects landward aquatic habitats from direct wave attack.

Coastal High-Hazard Area – Under the National Flood Insurance Program, 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. On a Flood Insurance Rate Map, the coastal high-hazard area is designated Zone V, VE, or V1–V30. These zones designate areas subject to inundation by the base flood where wave heights or wave runup depths are greater than or equal to 3 feet. In Hawaii, the VE-Zones are generally determined where the depth of water from a 100-year event (as determined from tsunami and/or hurricane data) is greater than 4 feet.

Collapsing breaker – A type of breaking wave associated with a steep beach slope and flat incident wave, which occurs right at the instantaneous shoreline.

D

Dead load – Weight of all materials of construction incorporated into the building, including but not limited to walls, floors, roofs, ceilings, stairways, built-in partitions, finishes, cladding, and other similarly incorporated architectural and structural items and fixed service equipment. See Loads.

Debris – Solid objects or masses carried by or floating on the surface of moving water.

Debris impact loads – Loads imposed on a structure by the impact of waterborne debris.

Debris line – Markings on a structure or the ground caused by the deposition of debris, indicating the height or inland extent of floodwaters.

Design Basis Earthquake (DBE) – The earthquake hazard level that structures are specifically proportioned to resist, taken as two-thirds of the Maximum Considered Earthquake (MCE) hazard level.

DoD – Department of Defense.

Draft – The depth of water that a body needs in order to float.

F

Far-source-generated tsunami – Tsunami resulting from a source located far from the site such that it arrives in excess of a 2-hour timeframe.

FEMA – Federal Emergency Management Agency.

FEMA MAT Report – FEMA Mitigation Assessment Team Report.

Fill – Material such as soil, gravel, or crushed stone placed in an area to increase ground elevations or change soil properties. See Structural Fill.

FIRM – Flood Insurance Rate Map.

500-year flood – Flood that has a 0.2% probability of being equaled or exceeded in any given year.

Flood elevation – Height of the water surface above an established elevation datum such as the National Geodetic Vertical Datum, the North America Vertical Datum, or mean sea level.

Flood-hazard area – The greater of the following: (1) the area of special flood hazard, as defined under the National Flood Insurance Program, or (2) the area designated as a flood-hazard area on a community's legally adopted flood-hazard map, or otherwise legally designated.

Flood Insurance Rate Map – Under the National Flood Insurance Program, an official map of a community upon which the Federal Emergency Management Agency has delineated both the special hazard areas and the risk premium zones applicable to the community. (Note: The latest FIRM issued for a community is referred to as the effective FIRM for that community.)

Footing – The enlarged base of a foundation wall, pier, post, or column designed to spread the load of the structure so that it does not exceed the soil bearing capacity.

G

Grade beam – Section of a concrete slab that is thicker than the slab and acts as a footing to provide stability, often under load-bearing or critical structural walls.

GSA – General Services Administration.

H

Hydrodynamic loads – Loads imposed on an object, such as a building, by water flowing against and around it. Among these loads are positive frontal pressure against the structure, drag effect along the sides, and negative pressure on the downstream side.

Hydrostatic loads – Loads imposed on a surface, such as a wall or floor slab, by a standing mass of water. The water pressure increases linearly with the water depth; hence, the hydrostatic loads increase with the square of the water depth.

I

Impact forces – Loads that result from waterborne debris transported by tsunami waves striking against buildings and structures or parts thereof.

Impulsive forces – Force induced against a vertical obstruction subjected to the leading edge of a tsunami during runup, also termed "surge" forces.

Ingress – The act of entering a building.

Inland zone – For the purposes of this report, the area that is inland of the A- and X-Zones (the limit of the 500-year flood).

L

Liquefaction – A phenomenon that occurs in saturated soils when the net pore pressure exceeds the gravity force holding soil particles together. Soil strength and stiffness decrease dramatically as the soil behaves similar to a fluid.

Loads – Forces or other actions that result from the weight of all building materials, occupants and their possessions, environmental effects, differential movement, and restrained dimensional changes.

M

Masonry – Built-up construction of combination of building units or materials of clay, shale, concrete, glass, gypsum, stone, or other approved units bonded together with or without mortar, grout, or other accepted methods of joining.

Maximum Considered Earthquake (MCE) – The most severe earthquake effects considered by seismic design codes and standards. The MCE is based on the United States Geological Survey seismic hazard maps, which are based on a combination of: (1) 2500-year probabilistic earthquake ground motion hazards; and (2) deterministic ground motion hazards in regions of high seismicity, with the appropriate ground motion attenuation relationships defined for each region.

Maximum Considered Tsunami (MCT) – A design tsunami event based on a probabilistic assessment considering all possible tsunami sources, or a

deterministic assessment considering the maximum tsunami that can reasonably be expected to affect a site.

Mid-source-generated tsunami – Tsunami generated by a source that is near the site of interest, but not close enough so that the effects of the triggering event is felt at the site.

Mitigation – Any action taken to reduce or permanently eliminate the long-term risk to life and property from natural hazards.

N

National Flood Insurance Program (NFIP) – The federal program created by Congress in 1968 that makes flood insurance available in communities that enact and enforce satisfactory floodplain management regulations.

National Geodetic Vertical Datum (NGVD) – Datum established in 1929 and used as a basis for measuring flood, ground, and structural elevations; was previously referred to as Sea Level Datum or Mean Sea Level. The Base Flood Elevations shown on most of the Flood Insurance Rate Maps issued by the Federal Emergency Management Agency are referenced to NGVD or, more recently, to the North American Vertical Datum.

Near-source-generated tsunami – Tsunami generated by a source located near the site such that it arrives within a 30-minute timeframe, and the effects of the triggering event are felt at the site.

Nonstructural wall – A wall that does not support vertical loads other than its own weight.

North American Vertical Datum (NAVD) – Datum used as a basis for measuring flood, ground, and structural elevations. NAVD, rather than the National Geodetic Vertical Datum, has been used in many recent flood insurance studies.

P

Pier foundation – Foundation consisting of isolated masonry or cast-in-place concrete structural elements extending into firm materials. Piers are relatively wide in comparison to their length, and derive their load-carrying capacity through skin friction, end bearing, or a combination of both.

Pile foundation – Foundation consisting of concrete, wood, or steel structural elements driven or jetted into the ground, or cast in place. Piles are relatively slender in comparison to their length, and derive their load-

carrying capacity through skin friction, end bearing, or a combination of both.

Plain concrete – Structural concrete with no reinforcement or with less reinforcement than the minimum amount specified for reinforced concrete.

Plunging Breaker – A type of breaking wave when the wave front curls over, forming a tube; it usually happens on beaches where the slope is moderately steep.

Post foundation – Foundation consisting of vertical support members, usually made of wood, set in holes and backfilled with compacted material.

Precast concrete – Concrete, usually a discrete structural member, that is formed, placed, and cured at one location, and subsequently moved and assembled into a final location in a structure.

Probabilistic maps – Maps of predicted tsunami effects including for inundation zone, flood depths, and flow velocities, based on a method involving probability and uncertainty.

Progressive collapse – ASCE/SEI Standard 7-02 defines progressive collapse *as "the spread of an initial local failure from element to element resulting eventually, in the collapse of an entire structure or a disproportionately large part of it."*

R

Rapid drawdown – A sudden reduction in water level immediately prior to the first tsunami wave, or between tsunami waves.

Reinforced concrete – Structural concrete reinforced with steel.

Retrofit – Any change made to an existing structure to reduce or eliminate potential damage to that structure from flooding, erosion, high winds, earthquakes, or other hazards.

S

Scour – Removal of soil or fill material by the flow of floodwaters, frequently used to describe storm-induced, localized conical erosion around pilings and other foundation supports where the obstruction of flow increases turbulence.

Sea wall – Solid barricade built at the water's edge to protect the shore and to prevent inland flooding.

SEI – Structural Engineering Institute of ASCE.

Shearwall – Load-bearing or non-load-bearing wall that transfers in-plane forces from lateral loads acting on a structure to its foundation.

Special Flood Hazard Area (SFHA) – Under the National Flood Insurance Program, an area having special flood, mudslide (i.e., mudflow), and/or flood-related erosion hazards, and shown on a Flood Hazard Boundary Map or Flood Insurance Rate Map as Zone A, AO, A1-A30, AE, A99, AH, V, V1-V30, VE, M, or E.

Stillwater elevation – Projected elevation that floodwaters would assume, referenced to the National Geodetic Vertical Datum, the North American Vertical Datum, or some other datum, in the absence of waves resulting from wind or seismic effects.

Storm surge – Rise in the water surface above normal water level on an open coast due to the action of wind stress and atmospheric pressure on the water surface.

Structural fill – Fill compacted to a specified density to provide structural support or protection to a structure.

T

Topography – Configuration of a terrain, including its relief and the position of its natural and man-made features.

Tsunami – A naturally occurring series of ocean waves resulting from a rapid, large-scale disturbance in a body of water, caused by earthquakes, landslides, volcanic eruptions, and meteorite impacts.

Tsunami inundation elevation – The elevation, measured from sea level, at the location of the maximum tsunami penetration

Tsunami inundation zone – The region flooded by tsunami penetration inland.

Tsunami runup – Rush of tsunami waves up a slope, terrain, or structure.

Tsunami runup height – The difference between the elevation of maximum tsunami penetration and the elevation of the shoreline at the time of tsunami attack.

Tsunami water level – The difference between the elevation of the highest local water level and the elevation of the shoreline at the time of tsunami attack.

U

Undermining – Process whereby erosion or scour exceeds the depth of the base of a building foundation, or the level below which the bearing strength of the foundation is compromised.

Uplift – Vertical hydrostatic pressure caused by the volume of displaced water under a building.

V

V-Zone – See Coastal High-Hazard Area.

VE-Zone – Coastal High-Hazard Areas where the Base Flood Elevations have been determined through a detailed study.

Vertical Evacuation Refuge from Tsunamis – A building or earthen mound that has sufficient height to elevate evacuees above the tsunami inundation depth, and is designed and constructed with the strength required to resist the forces generated by tsunami waves.

W

Waterborne debris – Any object transported by tsunami waves (e.g., driftwood, small boats, shipping containers, automobiles).

Wave crest – The point of highest elevation in a wave profile.

Wave height – Vertical distance between the successive local maximum and minimum elevations in a wave profile.

Wave zone – Area that coincides with V, VE, or V1–V30 Zones or Coastal High-Hazard Areas.

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THE IMPACTS OF SEA-LEVEL RISE ON THE CALIFORNIA COAST

A Paper From:
California Climate Change Center

Prepared By:
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FINAL PAPER

DISCLAIMER

This paper was prepared as the result of work funded by the California Energy Commission, the California Environmental Protection Agency, Metropolitan Transportation Commission, California Department of Transportation, and the California Ocean Protection Council (collectively "the funding agencies"). It does not necessarily represent the views of the funding agencies, their respective officers, agents and employees, or the State of California. The funding agencies, the State of California, and their respective officers, employees, agents, contractors, and subcontractors make no warrant, express or implied, and assume no responsibility or liability for the results of any actions taken or other information developed based on this paper; nor does any party represent that the uses of this information will not infringe upon privately owned rights. This paper is being made available for informational purposes only and has not been approved or disapproved by the funding agencies, nor have the funding agencies passed upon the accuracy, currency, completeness, or adequacy of the information in this paper. Users of this paper agree by their use to hold blameless each of the funding agencies for any liability associated with its use in any form. This work shall not be used to assess actual coastal hazards, insurance requirements or property values, and specifically shall not be used in lieu of Flood Insurance Studies and Flood Insurance Rate Maps issued by the Federal Emergency Management Agency (FEMA).

Arnold Schwarzenegger, *Governor*

May 2009
CEC-500-2009-024-F

Acknowledgments

Major funds for this report were made through the California Energy Commission's Public Interest Energy Research (PIER) Program. Additional support was provided by the Metropolitan Transportation Commission and the Ocean Protection Council. We thank them for their generosity and foresight.

The scientists and engineers at Philip Williams and Associates provided us with information and analysis on coastal flood and erosion hazards. Thanks to Dr. David L. Revell, Robert Battalio, Jeremy Lowe, Justin Vandever, Brian Spear, and Seungjin Baek. For additional information about their work on this project, please see www.pwa-ltd.com/resources/resource_publications.html.

Many individuals, organizations, and agencies helped make this work possible by providing data, information, and input and review of the final report. We owe thanks to Will Travis, director of the Bay Conservation and Development Commission, for initiating the study and suggesting our involvement and to staff members Leslie Lacko, Tim Doherty, Adam Parris, and Steve Goldbeck, who worked closely with us as we prepared this report.

We thank Dr. Noah Knowles, Dr. Dan Cayan, Mary Tyree, and Dr. Peter Bromirski of Scripps Institution of Oceanography for much of the oceanographic data. Dr. Reinhard Flick at Scripps also provided useful data on historical tide trends.

Thanks to Doug Kimsey and his staff at the Metropolitan Transportation Commission for providing accurate transportation data. Thanks to Reza Navai, Vahid Nowshiravan, and Barry Padilla at the California Department of Transportation for many helpful conversations.

Special thanks to the staff at the National Oceanic and Atmospheric Administration's (NOAA) Coastal Services Center, Kirk Waters and Keil Schmid, for helping us obtain several gigabytes of LIDAR data. Abby Sallenger at the United States Geological Survey gave additional advice and insights about the Coastal Change program's LIDAR data. Thanks to Mark Sanchez, geographic information system (GIS) wizard at the State of Oregon, for help in figuring out how to handle all those gigabytes!

Brian Fulfrost at the University of California (UC) Santa Cruz, now at DCE Planning, helped us locate several helpful GIS datasets. We thank Robert Colley, GIS Manager for Santa Clara County, for providing data and for recognizing that the free and open sharing of public data is so valuable to researchers and the public. Ray McDowell, GIS Data Coordinator at the California Resources Agency, helped locate and obtain still more GIS data. At the Federal Emergency Management Agency (FEMA), Eric Simmons and Ray Lenaburg engaged us in helpful discussion and gave pointers to the spatial data from recent FEMA mapping studies.

Staff at California's Resources Agency engaged us in a number of insightful and provocative discussions. Thanks to Sam Schuchat, Brian Baird, Tony Brunello, John Ellison, and Abe Doherty. Special thanks to Christine Blackburn at the Ocean Protection Council (OPC) for

seeing the importance of this issue to the entire state, for helping to coordinate OPC's participation in the project, and for many valuable conversations.

Johanna Fenton, formerly head of the Earthquake and Tsunami Program in the Governor's Office of Emergency Services, provided early guidance and advice. Leslie Ewing, Mark Johnsson, and Greg Benoit of the California Coastal Commission provided data or suggestions. We were especially thankful to discover the excellent work of Jennifer Dare, a NOAA Coastal fellow, who compiled the Coastal Armoring Database.

Thanks to Philip Pang in the South Pacific Division of the U.S. Army Corps of Engineers for his work estimating levee construction costs. Thanks to Jos Dijkman, flood management engineer at Deltares/Delft Hydraulics in the Netherlands, for a great deal of detailed information on the construction of dikes and flood defenses. Walt Crampton, principal engineer at TerraCosta Consulting Group, also provided seawall construction costs for California.

We wish to thank ESRI. A grant to the Pacific Institute in 2007 through their Conservation Grants program allowed us to expand the range and sophistication of our analysis.

Special thanks go to the leader of the PIER Research Team, Guido Franco, not only for skillfully overseeing this complex set of studies, but also for a number of insightful comments and suggestions. Technical editor Susie Moser provided insightful comments on an early draft. Thanks to editor Mark Wilson for skill and patience in making all of this readable.

Finally, we are especially grateful for our reviewers: Michael Hanemann, Arlene Wong, June Gin, and two anonymous reviewers, who provided thoughtful and insightful comments. We also received several public comments during the open comment period that helped improve the final report. All conclusions and errors are, of course, our own.

Preface

The California Energy Commission's Public Interest Energy Research (PIER) Program supports public interest energy research and development that will help improve the quality of life in California by bringing environmentally safe, affordable, and reliable energy services and products to the marketplace.

The PIER Program conducts public interest research, development, and demonstration (RD&D) projects to benefit California's electricity and natural gas ratepayers. The PIER Program strives to conduct the most promising public interest energy research by partnering with RD&D entities, including individuals, businesses, utilities, and public or private research institutions.

PIER funding efforts focus on the following RD&D program areas:

- Buildings End-Use Energy Efficiency
- Energy-Related Environmental Research
- Energy Systems Integration
- Environmentally Preferred Advanced Generation
- Industrial/Agricultural/Water End-Use Energy Efficiency
- Renewable Energy Technologies
- Transportation

In 2003, the California Energy Commission's PIER Program established the **California Climate Change Center** to document climate change research relevant to the states. This center is a virtual organization with core research activities at Scripps Institution of Oceanography and the University of California, Berkeley, complemented by efforts at other research institutions. Priority research areas defined in PIER's five-year Climate Change Research Plan are: monitoring, analysis, and modeling of climate; analysis of options to reduce greenhouse gas emissions; assessment of physical impacts and of adaptation strategies; and analysis of the economic consequences of both climate change impacts and the efforts designed to reduce emissions.

The California Climate Change Center Report Series details ongoing center-sponsored research. As interim project results, the information contained in these reports may change; authors should be contacted for the most recent project results. By providing ready access to this timely research, the center seeks to inform the public and expand dissemination of climate change information, thereby leveraging collaborative efforts and increasing the benefits of this research to California's citizens, environment, and economy.

For more information on the PIER Program, please visit the Energy Commission's website at www.energy.ca.gov/pier/ or contract the Energy Commission at (916) 654-5164.

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Abstract

Over the past century, sea level has risen nearly eight inches along the California coast, and general circulation model scenarios suggest very substantial increases in sea level as a significant impact of climate change over the coming century. This study includes a detailed analysis of the current population, infrastructure, and property at risk from projected sea-level rise if no actions are taken to protect the coast. The sea-level rise scenario was developed by the State of California from medium to high greenhouse gas emissions scenarios from the Intergovernmental Panel on Climate Change (IPCC) but does not reflect the worst-case sea-level rise that could occur. We also evaluate the cost of building structural measures to reduce that risk. If development continues in the areas at risk, all of these estimates will rise. No matter what policies are implemented in the future, sea-level rise will inevitably change the character of the California coast.

We estimate that a 1.4 meter sea-level rise will put 480,000 people at risk of a 100-year flood event, given today's population. Among those affected are large numbers of low-income people and communities of color, which are especially vulnerable. Critical infrastructure, such as roads, hospitals, schools, emergency facilities, wastewater treatment plants, power plants, and more will also be at increased risk of inundation, as are vast areas of wetlands and other natural ecosystems. In addition, the cost of replacing property at risk of coastal flooding under this sea-level rise scenario is estimated to be nearly $100 billion (in year 2000 dollars). A number of structural and non-structural policies and actions could be implemented to reduce these risks. For example, we estimate that protecting some vulnerable areas from flooding by building seawalls and levees will cost at least $14 billion (in year 2000 dollars), with added maintenance costs of another $1.4 billion per year. Continued development in vulnerable areas will put additional areas at risk and raise protection costs.

Large sections of the Pacific coast are not vulnerable to flooding, but are highly susceptible to erosion. We estimate that a 1.4 meter sea-level rise will accelerate erosion, resulting in a loss of 41 square miles (over 26,000 acres) of California's coast by 2100. A total of 14,000 people currently live in the area at risk of future erosion. Additionally, significant transportation-related infrastructure and property are vulnerable to erosion. Statewide flood risk exceeds erosion risk, but in some counties and localities, coastal erosion poses a greater risk. This report also provides a comprehensive set of recommendations and strategies for adapting to sea-level rise.

Keywords: sea-level rise, coastal impacts, climate change, California, San Francisco Bay, flood, erosion, climate adaptation, climate impacts, levees, seawalls, greenhouse effect

1.0 Introduction

California's coastline, which includes more than 2,000 miles of open coast and enclosed bays, is vulnerable to a range of natural hazards, including storms, extreme high tides, and rising sea levels resulting from global climate change. Development along California's coast is extensive. In 2000, 26 million Californians lived in coastal counties, and by 2003, this number had grown to nearly 31 million (U.S. Census Bureau 2000; NOAA 2004). Indeed, six of the ten fastest growing coastal counties in the United States between 1980 and 2003 were in California (NOAA 2004). Major transportation corridors and other critical infrastructure are found along the California coast, including oil, natural gas, and nuclear energy facilities, as well as major ports, harbors, and water and wastewater plants. The California coast is also an extraordinary cultural and ecological resource and offers extensive tourism and recreational opportunities.

Flooding and erosion already pose a threat to communities along the California coast and there is compelling evidence that these risks will increase in the future. Based on a set of climate scenarios prepared for the California Energy Commission's Public Interest Energy Research (PIER) Climate Change Research Program, Cayan et al. (2009) project that, under medium to medium-high emissions scenarios, mean sea level along the California coast will rise from 1.0 to 1.4 meters (m) by the year 2100.[1] Rising seas put new areas at risk of flooding and increase the likelihood and intensity of floods in areas that are already at risk. In areas where the coast erodes easily, sea-level rise will likely accelerate shoreline recession due to erosion. Erosion of some barrier dunes may expose previously protected areas to flooding.

National studies on the economic cost of sea-level rise suggest that while adapting to climate change will be expensive, so are the costs of doing nothing, as substantial investments are already at risk and vulnerable.[2] Because the economic costs of flooding are highly site-specific, regional analyses are critical for guiding land-use decisions and evaluating adaptive strategies.

The Pacific Institute published one of the earliest comprehensive regional assessments of sea-level rise (Gleick and Maurer 1990), concluding that a one-meter sea-level rise would threaten existing commercial, residential, and industrial structures around San Francisco Bay valued at $48 billion (in year 1990 dollars). Building or strengthening levees and seawalls simply to protect existing high-value development was estimated to require an immediate capital investment of approximately $1 billion (in year 1990 dollars) and would require an additional $100 million per year in ongoing maintenance.[3] The report also noted that substantial areas of the San Francisco Bay, especially wetlands and marshes, could not be protected and would likely be damaged or lost.

---

[1] It is important to note that most climate models fail to include ice-melt contributions from the Greenland and Antarctic ice sheets, and as a result, the potential increase in mean sea level may be much higher.

[2] See, for example, Titus et al. (1992) and Yohe et al. (1996).

[3] This estimate does not include the cost of protecting and restoring wetlands, groundwater aquifers, etc.

This assessment updates and expands our 1990 analysis using more comprehensive data, new climate scenarios, and modern computerized analytical tools. We made extensive use of geographic information system (GIS) software and updated sea-level rise scenarios from the Scripps Institution of Oceanography to estimate the population, infrastructure, ecosystems, and property at risk. We also estimate some of the cost of armoring the coast, one potential adaptation strategy to reduce that risk. This work is part of a larger set of research projects by the California Climate Action Team to understand the impacts of climate change to Californians, funded by the California Energy Commission's Public Interest Energy Research (PIER) program. The Pacific Institute also received significant financial support from two other state agencies: the Ocean Protection Council and the Metropolitan Transportation Commission, part of the Department of Transportation.

1.1. Key Findings

Over the past century, sea level has risen nearly eight inches along the California coast, and general circulation model scenarios suggest very substantial increases in sea level as a significant impact of climate change over the coming century. This study includes a detailed analysis of the current population, infrastructure, and property at risk from projected sea-level rise if no actions are taken to protect the coast, and the cost of building structural measures to reduce that risk. We find the following:

- Under medium to medium-high greenhouse-gas emissions scenarios, mean sea level along the California coast is projected to rise from 1.0 to 1.4 meters (m) by the year 2100. Maps for the entire coast of California demonstrating the extent of the areas at risk are posted at www.pacinst.org/reports/sea_level_rise.[4]

- A 1.4 meter sea-level rise will put 480,000 people at risk of a 100-year flood event, given today's population. Populations in San Mateo and Orange Counties are especially vulnerable. In each, an estimated 110,000 people are at risk. Large numbers of residents (66,000) in Alameda County are also at risk.

- A demographic analysis identified large numbers of people at risk with heightened vulnerability, including low-income households and communities of color. Additionally, adapting to sea-level rise will require tremendous financial investment. Given the high cost and the likelihood that individuals, the State, and local agencies will not protect everything, adaptation raises additional environmental justice concerns.

- A wide range of critical infrastructure, such as roads, hospitals, schools, emergency facilities, wastewater treatment plants, power plants, and more will also be at increased risk of inundation in a 100-year flood event. This infrastructure at risk includes:

[4] These maps are not the result of detailed site studies and were created to quantify risk over a large geographic area. They should not be used to assess actual coastal hazards, insurance requirements or property values, and specifically shall not be used in lieu of Flood insurance Studies and Flood Insurance Rate Maps issued by the Federal Emergency Management Agency (FEMA). Local governments or regional planning agencies should conduct detailed studies to better understand the potential impacts of sea-level rise in their communities.

  - nearly 140 schools;
  - 34 police and fire stations;
  - 55 healthcare facilities;
  - more than 330 U.S. Environmental Protection Agency (U.S. EPA)-regulated hazardous waste facilities or sites, with large numbers in Alameda, Santa Clara, San Mateo, and Los Angeles counties;
  - an estimated 3,500 miles of roads and highways and 280 miles of railways;
  - 30 coastal power plants, with a combined capacity of more than 10,000 megawatts;
  - 28 wastewater treatment plants, 21 on the San Francisco Bay and 7 on the Pacific coast, with a combined capacity of 530 million gallons per day; and
  - the San Francisco and Oakland airports.
- Vast areas of wetlands and other natural ecosystems are vulnerable to sea-level rise. An estimated 550 square miles, or 350,000 acres, of wetlands exist along the California coast, but additional work is needed to evaluate the extent to which these wetlands would be destroyed, degraded, or modified over time. A sea-level rise of 1.4 m would flood approximately 150 square miles of land immediately adjacent to current wetlands, potentially creating new wetland habitat if those lands are protected from further development.
- We estimate that nearly $100 billion (in year 2000 dollars) worth of property, measured as the current replacement value of buildings and contents, is at risk of flooding from a 100-year event with a 1.4 m sea-level rise if no adaptation actions are taken. An overwhelming two-thirds of that property is concentrated on San Francisco Bay. The majority of this property is residential.
- Coastal armoring is one potential adaptation strategy. Approximately 1,100 miles of new or modified coastal protection structures are needed on the Pacific Coast and San Francisco Bay to protect against coastal flooding. The total cost of building new or upgrading existing structures is estimated at about $14 billion (in year 2000 dollars). We estimate that operating and maintaining the protection structures would cost approximately 10% of the initial capital investment, or around another $1.4 billion per year (in year 2000 dollars).
- Large sections of the Pacific coast are not vulnerable to flooding, but are highly susceptible to erosion. We estimate that a 1.4 m sea-level rise will accelerate erosion, resulting in a loss of 41 square miles of California's coast by 2100. A total of 14,000 people live in areas at risk of erosion. In addition, significant transportation-related infrastructure and property are also at risk. Throughout most of the state, flood risk exceeds erosion risk, but in some counties, coastal erosion poses a greater risk.

- Continued development in vulnerable areas will put additional areas at risk and raise protection costs.

2.0 Methods

Numerous studies have attempted to quantify the cost of sea-level rise and have been based primarily on a framework developed in Yohe (1989) and refined in Yohe et al. (1996) and Yohe and Schlesinger (1998). That framework employs a cost-benefit model to evaluate the property at risk and the cost of protecting or abandoning that property. Property is protected if the value of the property exceeds the protection cost at the time of inundation, and the protection cost is equal to the construction cost of the protective structure. If the value of the property does not exceed the cost of protection, then the property is abandoned, with the cost equal to the value of the land and structure at the time of inundation. The total economic cost is then the sum of the protection cost plus the value of the lost property.

To determine the value of lost property, the Yohe approach considers land and structure values separately. In most locations, coastal land commands a premium price, with the price declining as one moves inland. With inundation, the Yohe method assumes that land values will simply migrate inland, and thus, the economic value of lost land is equal to the economic value of interior land. The value of structures is calculated under two conditions: with and without foresight. With perfect foresight, the economic value of structures is assumed to depreciate over time as the "impending inundation and abandonment become known" (Yohe and Schlesinger 1998), approaching $0 at the time of inundation. Without foresight, the structure value does not depreciate.

Despite its wide application, the Yohe method has a number of limitations, many of which are discussed in Hanemann (2008):

- First, it ignores any transfers among property owners and looks only at the net social cost. In reality, there will be winners (those who had inland property that is now closer to the coast and thus more valuable) and losers (those who have lost their property), and the gross social cost "could be enormous" (Yohe et al. 1996).
- Second, it assumes that coastal protection will be constructed just in time to avoid damage from flooding. This is unlikely. If coastal protection is constructed too late, then the property would incur some damage, thereby increasing the cost. If constructed too early, then the discounted net present value of the cost of building the structure would be higher (Hanemann 2008).
- Third, it only examines changes in mean sea level (eustatic change), thereby ignoring damage from storm surge and extreme events.
- Fourth, by focusing on property values, it ignores other potentially expensive costs. For example, the flooding of transportation infrastructure essential for moving people or goods, e.g., highways and ports, could cause major interruptions to the local economy. Flooding also causes impacts on the health and well-being of the affected individuals and environmental damage, including erosion, oils spills, and discharge of pollution

from coastal industry (Hanemann 2008). Over the long-term, flooding can lead to the loss of wetlands.

- Fifth, prioritization of protection based on property value may directly undermine an environmental justice framework for protection.

This study used a different approach to estimate the economic impact of sea-level rise. We adopted the scenarios developed for the PIER studies and mapped the extent of inundation from a 100-year flood event that is likely to occur with rising sea levels. We also identified areas at increased risk from erosion as a result of rising seas. The inundation and erosion geodata were overlaid with other geospatial data using GIS to produce quantitative estimates of the population, infrastructure, and replacement value of property at risk from sea-level rise, as well as the impacts on harder-to-quantify coastal ecosystems. We also produced an initial estimate of the cost of adaptation measures, specifically building seawalls and levees in high-valued coastal zones to protect against future flooding. Greater detail on the methods is provided below.

2.1. Study Area

The study area spans approximately 1,100 miles of California's Pacific coast and 1,000 miles of shoreline along the perimeter of the San Francisco Bay. The San Francisco Bay study area extends from the Golden Gate in the west to Pittsburg, California, in the east and San Jose in the south. The eastern boundary of the San Francisco Bay study was set according to where United States Geological Survey (USGS) researchers were able to extract reliable flood elevations from the Bay hydrodynamic model. We provide estimates for a number of scenarios for San Francisco Bay due to the ready availability of high-resolution geographic data provided by the USGS.

The study area of the erosion analysis extended from Santa Barbara to the Oregon border, covering about 930 miles (1,450 kilometers, km). Much of the Southern California coast was excluded from the erosion analysis due to myriad ongoing initiatives focused on climate change and hazards mapping.

2.2. Sea-Level Rise Projections

2.2.1. Mean Water Levels and Extreme Events

Sea levels are constantly in flux, subject to the influence of astronomical forces from the sun, moon, and earth, as well as meteorological effects like El Niño. A worldwide network of more than 1,750 tidal gages continuously collects data on water levels relative to a nearby geodetic reference, and new satellite-based sensors are extending measurements. Tide gage data indicate that the global mean sea level is rising. Water level measurements from the San Francisco gage (CA Station ID: 9414290), shown in Figure 1, indicate that mean sea level rose by an average of

2.01 millimeters (mm) per year from 1897 to 2006, equivalent to a change of eight inches in the last century.[5]

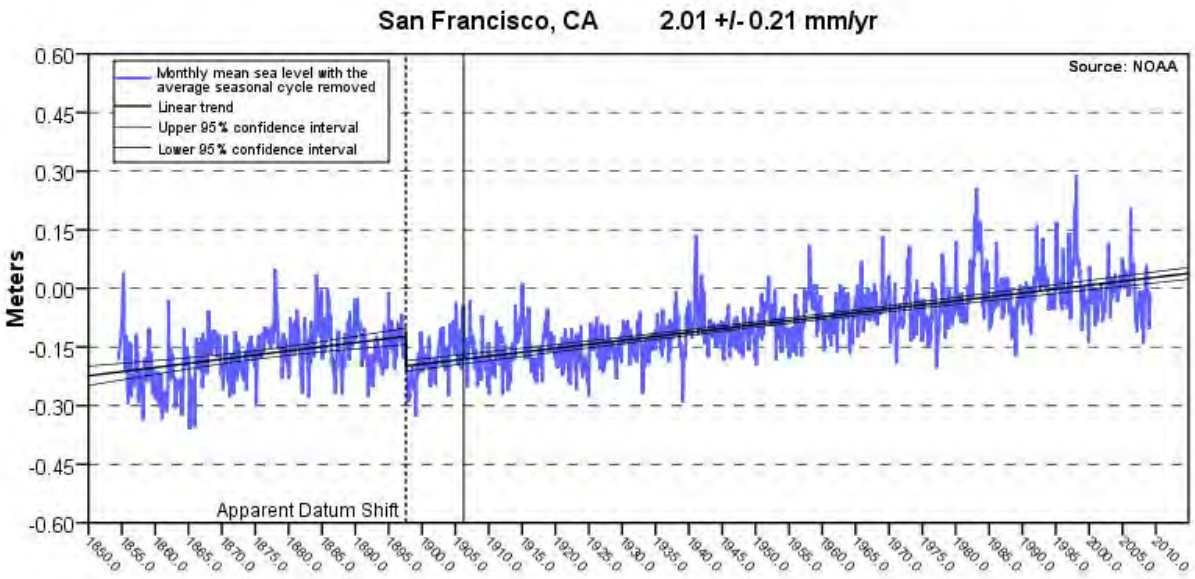

Figure 1. Trend in monthly mean sea level at the San Francisco tide station from 1854–2006

Source: NOAA Sea Levels Online, http://tidesandcurrents.noaa.gov/sltrends/sltrends_station.shtml?stnid=9414290

Sea levels are expected to continue to rise, and the rate of increase will likely accelerate. In order to evaluate climate change impacts, the Intergovernmental Panel on Climate Change (IPCC) developed future emission scenarios that differ based on assumptions about economic development, population, regulation, and technology (see Box 1 for a description of the scenarios). Based on these scenarios, mean sea level was projected to rise by 0.2 m to 0.6 m by 2100, relative to a baseline of 1980–1999, in response to changes in oceanic temperature and the exchange of water between oceans and land-based reservoirs, such as glaciers and ice sheets (Meehl et al. 2007).

More recent research by leading climate scientists, which includes more accurate sea-level measurements by satellites, indicates that sea-level rise from 1993–2006 has outpaced the IPCC projections (Rahmstorf et al. 2007). The authors suggest that the climate system, particularly sea levels, may be responding to climate changes more quickly than the models predict. Additionally, most climate models fail to include ice-melt contributions from the Greenland and Antarctic ice sheets and may underestimate the change in volume of the world's oceans.

[5] The solid vertical line shows the earthquake of 1906. NOAA researchers fit separate trendlines before and after an apparent datum shift (vertical movement of the land surface) that occurred in 1897, disrupting consistent measurements.

To address these new factors, the PIER projects used sea-level rise forecasts developed by a team at the Scripps Institution of Oceanography led by Dr. Dan Cayan. Using a methodology developed by Rahmstorf (2007), Cayan et al. (2009) produced global sea-level estimates based on projected surface air temperatures from global climate simulations for both the IPCC A2 and B1 scenarios using the output from six global climate models: the National Center for Atmospheric Research (NCAR) Parallel Climate Model (PCM); the National Oceanic and Atmospheric Administration (NOAA) Geophysical Fluids Dynamics Laboratory (GFDL) version 2.1; the NCAR Community Climate System Model (CCSM); the Max Planck Institute ECHAM3; the MIROC 3.2 medium-resolution model from the Center for Climate System Research of the University of Tokyo and collaborators; and the French Centre National de Recherches Météorologiques (CNRM) models.

Box 1: IPCC Climate Change Scenarios

The impacts of climate change will ultimately depend on future greenhouse gas concentrations. Future greenhouse gas emissions remain uncertain and are influenced by a variety of demographic, socio-economic, and technological factors. Scenarios can be a useful tool for examining how changes in these driving factors affect greenhouse gas concentrations. These scenarios can be useful for evaluating impacts associated with climate change as well as assessing adaptation and mitigation activities. The Special Report on Emissions Scenarios (SRES) outlines four storylines that differ according to demographics, social, economic, environmental, and technological factors and lead to different levels of greenhouse gas emissions. Each storyline has a number of different scenarios, referred to as a family. A total of 40 scenarios have been developed.

The four storylines are described below:

The **A1** storyline is characterized by "a future world of very rapid economic growth, global population that peaks in mid-century and declines thereafter, and the rapid introduction of new and more efficient technologies. Major underlying themes are convergence among regions, capacity building, and increased cultural and social interactions, with a substantial reduction in regional differences in per capita income" (IPCC 2000). The A1 family is further divided into three subgroups that are differentiated according to energy source: fossil intensive (**A1FI**), non-fossil sources (**A1T**), and a mix of fossil and non-fossil sources (**A1B**).

The **A2** storyline is characterized by "self-reliance and preservation of local identities" (IPCC 2000). Population is expected to continuously increase, but economic growth and technological development are expected to be slow.

The **B1** storyline has the same population projections as the A1 storyline but "rapid changes in economic structures toward a service and information economy, with reductions in material intensity, and the introduction of clean and resource-efficient technologies" (IPCC 2000).

The **B2** storyline is characterized by "a world with continuously increasing global population at a rate lower than A2, intermediate levels of economic development, and less rapid and more diverse technological change than in the B1 and A1 storylines" (IPCC 2000).

Additionally, Cayan et al. (2009) modified the sea-level rise estimates to account for water trapped in dams and reservoirs that artificially reduced runoff into the oceans (Chao et al. 2008). Absolute sea-level rise along the California coast was assumed to be the same as the global estimate. Based on these methods, Cayan et al. (2009) estimate an overall projected rise in mean sea level along the California coast for the B1 and A2 scenarios of 1.0 m and 1.4 m, respectively, by 2100 (Figure 2). The more severe A1FI scenario, which assumes a continued high level use of fossil fuels, was not used in this analysis, but is shown for comparative purposes.

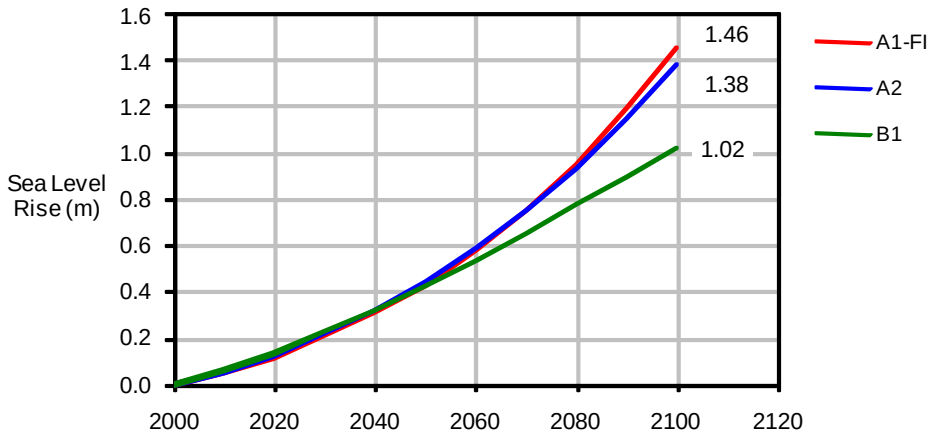

Figure 2. Scenarios of sea-level rise to 2100

Source: Dan Cayan, Scripps Institution of Oceanography, NCAR CCSM3 simulations, Rahmstorf method.

The majority of studies on climate change have emphasized changes in average conditions, yet the greatest socio-economic impacts tend to occur as a result of extreme events. Coastal flooding is often caused by storm surges, which are caused by high winds and pressure differentials associated with storms. Along the California coast, wave-induced storm surge can exceed 1.5 m (Cayan et al. 2006), flooding low-lying areas and eroding coastal bluffs. Increases in mean sea level are expected to increase the frequency and intensity of these extreme events. Although this study does not explicitly account for changes in storm surge, we do account for higher flood elevations associated with extreme events, as described below in Section 2.3.

2.3. Expected Risk to the Coast

2.3.1. Coastal Inundation Risk

Sea-level rise increases the risk of flooding in low-lying areas. For this study, we considered coastal flood risks only, e.g., flooding caused by rising seas along the Pacific Ocean and San Francisco Bay. Higher sea levels, however, can also worsen flooding in nearby rivers as higher water surface elevations at the downstream end of a river causes water to back up and increase upstream flooding. These impacts are not evaluated here.

For the California coast, we used GIS to produce maps of the areas at risk of inundation from a 1.4 m sea-level rise. For the San Francisco Bay, we produced maps of the areas at risk of inundation under three different sea-level rise scenarios: 0.5 m, 1.0 m, and 1.4 m. Below, we describe the methods used to determine the areas at risk of flooding along the Pacific coast and in the San Francisco Bay. Erosion is discussed in Section 2.3.2.

Pacific Coast

A flood is often described by its recurrence interval, which is the period of time between floods of a particular intensity that is based on historic conditions for a given area. The terminology used to describe the recurrence interval, however, can be misleading and is often misinterpreted. A "100-year flood" does not refer to a flood level that occurs every 100 years. Rather, it refers to a flood that has a 1/100, or 1%, chance of occurring in any year. Thus, over a typical 30-year mortgage period, a 100-year flood has a 1-in-4 chance of occurring (see Box 2).

For the Pacific coast, we approximate the potential future flood impact by adding projected sea-level rise estimates to water levels associated with a 100-year flood event; that is, current flood elevations for the 100-year flood are increased by 1.4 meters, the projected increase in sea level by 2100 under the A2 scenario (Figure 3).

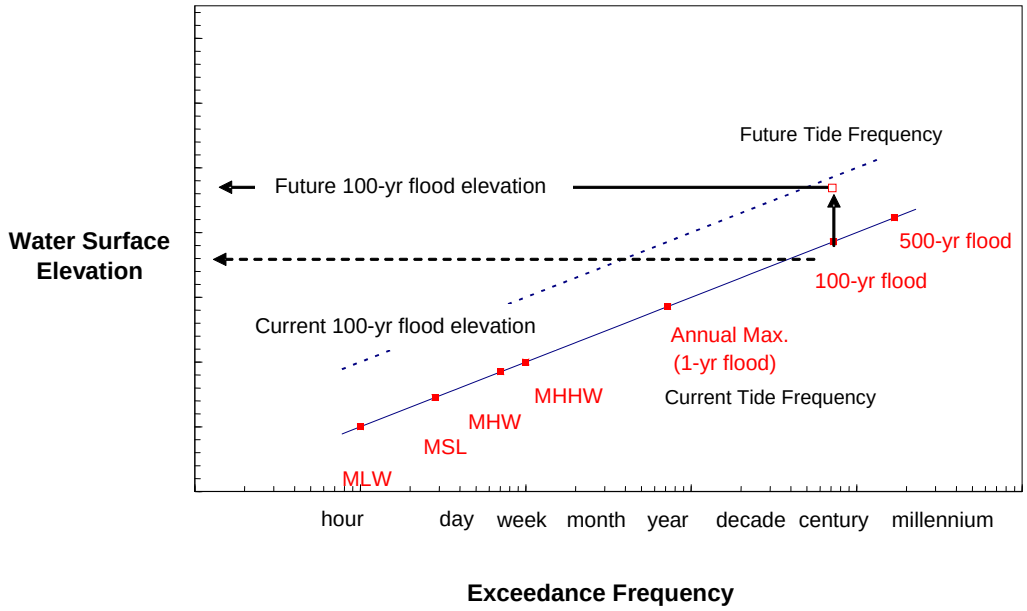

Figure 3. Determining future flood elevations

Note: The solid line represents the current tide frequency. The dotted line represents the future flood frequency. As can be seen, an increase in water surface elevation increases the frequency and intensity of flood events. For example, a 100-year flood event could become an annual flood event. The flood frequency estimates shown are for demonstration purposes only and are not based on actual data. See the Glossary for definitions of the abbreviations MLW, MSL, MHW, and MHHW.

This approach assumes that all tide datums, e.g., mean high tide and flood elevations, will increase by the same amount as mean sea level. There is some evidence that this assumption may not always hold true. Flick et al. (1999) found that in San Francisco, mean higher high water (MHHW) was increasing at a rate of 258 mm per century, while the mean sea level

increased at a lower rate of 217 mm per century (Figure 4). Thus, while the overall trend is one of rising seas, the intertidal range, i.e., the difference between MHHW and mean lower low water (MLLW), also seems to be widening. In addition, an increase in storminess due to climate change might cause more frequent storm surges and an increase in the frequency of high water events, although there is not yet consensus among climate scientists on changes in storm intensity or frequency, and such changes are not included here explicitly.

Box 2: Estimating Flood Risk

What are the chances that a 100-year flood will occur during a 30-year period?

To make this determination, we must apply basic probability theory. Flooding is a random event, i.e., the odds of it occurring in any year are independent of past conditions. Thus the odds of a storm not occurring over a 30-year period can be calculated using the following methodology.

If an event has an X percent chance of occurring in a given year, then the odds that the event will **not** occur in a given year are

$$1-X$$

The odds that an event will not occur in two successive years is

$$(1-X)(1-X) = (1-X)^2$$

And the odds of an event not occurring over y number of years is

$$(1-X)^y$$

Let's now calculate the odds that a 100-year flood event will not occur over 30 years.

In this case,

$$X = 1/100 = 0.01 \text{ and } y = 30$$

$$(1-X)^y = (1-0.01)^{30} = 0.74$$

Thus there is a 74% change that a 100-year storm will not occur over a 30-year period; and a 26%, or approximately a 1 in 4 chance that it will occur.

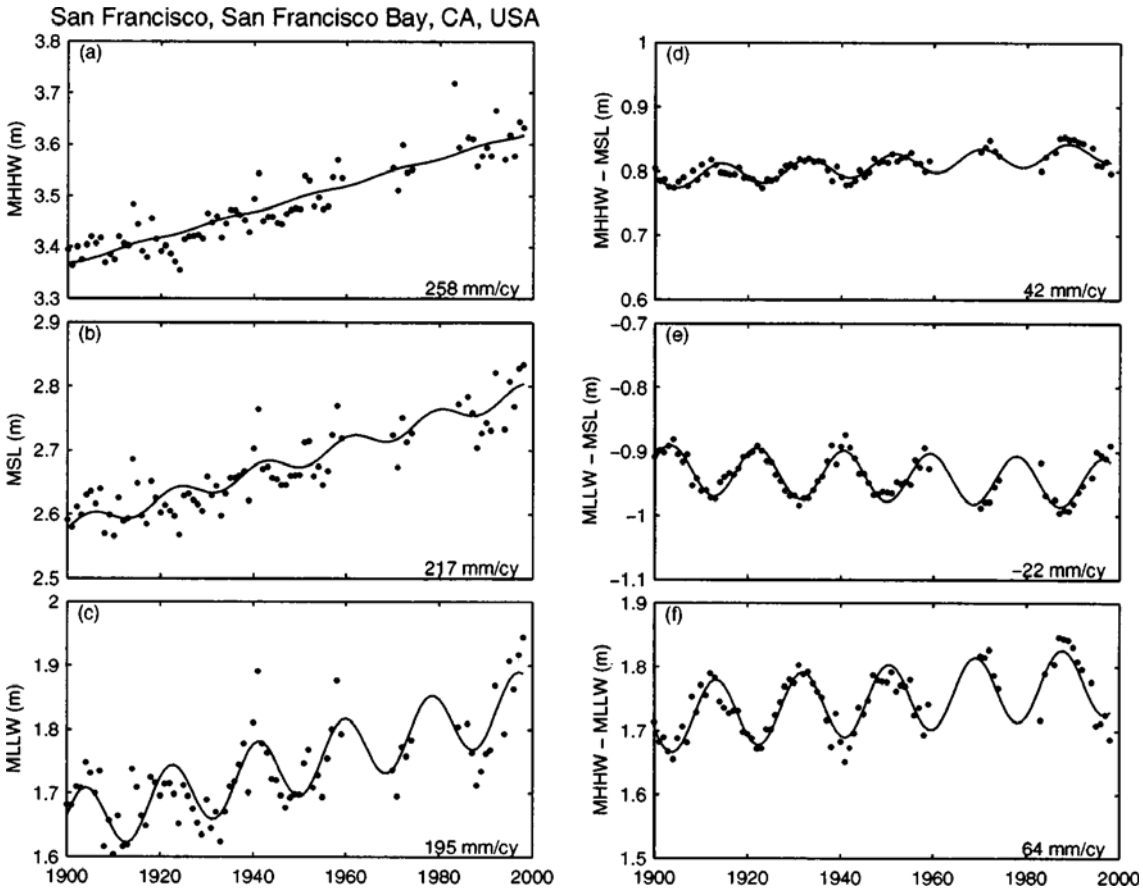

Figure 4. Rates of change of tidal datums, San Francisco from 1900–2000

Source: Flick et al. 1999

Existing flood levels were based on estimates of the 100-year flood elevation (also called the *base flood elevation* or BFE) from Flood Insurance Studies published by the Federal Emergency Management Agency (FEMA). The FEMA BFEs, however, only cover a part of the coast. We contracted with Philip Williams and Associates (PWA) to provide estimates of BFEs where none exist. Their work consisted of the following:

1. Compiled available coastal flood BFEs published by FEMA for the California coast.
2. Estimated BFEs where FEMA estimates are not available using professional judgment.
3. Converted elevations to the North American Vertical Datum of 1988 (NAVD88).
4. Adjusted elevations to nearest half foot based on observed sea-level rise to present day.

Further information on the methods used by PWA is available in a separate technical memorandum (Battalio et al. 2008).

We used automated mapping methods in GIS to delineate areas inundated by the current and future flood elevations. The key inputs to this analysis are digital elevation models (DEMs), gridded datasets that contain values representing elevations of the Earth's surface. We used the most accurate, high-resolution terrain data available. NOAA's Coastal Service Center assisted

us in processing and obtaining each of these data sets. The elevation datasets used for this project are summarized in Table 1.

For much of the Central and Northern California coast, high-accuracy Light Detection and Ranging (LIDAR) data were available from Airborne LIDAR Assessment of Coastal Erosion (ALACE) project, a partnership between NOAA, the National Aeronautics and Space Administration (NASA), and USGS. The ALACE project emphasized shoreline change, and so the data were available for a relatively narrow swath of the coast. The coverage did not always extend inland far enough to fully map the coastal floodplain. In addition, there were several gaps in coverage along the entire coast.

We supplemented the LIDAR data, filling in gaps in coverage with topographic information from the USGS National Elevation dataset. Although these data are at a lower resolution and accuracy, they allowed us to map the entire coast. For portions of the Southern California coast, Interferometric Synthetic Aperture Radar (IFSAR) data were available from NOAA. The IFSAR data are of coarser resolution than the LIDAR data described above (i.e., they are 3-meter pixel resolution compared to 2-meter resolution), and they have less vertical accuracy (i.e., ±2.2 m compared to ±0.07 m for the LIDAR data).

Table 1. Elevation datasets used for mapping coastal flood risks

Dataset	National Elevation Dataset	ALACE 1998	ALACE 2002	So. Cal. IFSAR
Source/Mission	USGS	NASA, NOAA, USGS	NASA, NOAA, USGS	NOAA
Geographic Coverage	National	Stinson Beach to Santa Barbara	Northern border of California to Stinson Beach	Santa Barbara to Mexican border
Data Collection Method	Various	LIDAR	LIDAR	IFSAR
Resolution	10 m	3 m	2 m	3 m
Year Collected	Various	1998	2002	2003
Stated Vertical Accuracy	± 7.5 m	± 0.07 m	± 0.07 m	± 2.2 m

GIS raster math tools were used to compare the elevation of land surfaces with the adjacent flood elevation to determine the extent of flooding. Because of the large file sizes, and the large area being studied, we worked with the terrain datasets in over 600 tiles. Pacific Institute researchers wrote scripts to automate the processing steps on each of these tiles. The resulting inundation grids were boundary-smoothed and small isolated ponds and islands were removed. The raster datasets were then converted to vector polygons and merged so they could be used in the social and economic analyses. A separate technical memorandum is available at www.pacinst.org/reports/sea_level_rise that describes the GIS flood delineation methodology in greater detail.

San Francisco Bay

While our study looks at the entire California coastline, we also produced more detailed estimates of coastal flood risk in San Francisco Bay. In total, we estimated impacts along approximately 1,100 miles of Pacific Coast from Oregon to Mexico, and an additional 1,000 miles inside of San Francisco Bay. Inundation maps generated from the climate scenarios were provided to the Pacific Institute by Dr. Noah Knowles of the United States Geological Survey (Knowles 2008). These estimates are described in Knowles 2009.

To estimate inundated areas in the Bay, "the highest resolution elevation data available were assembled from various sources and mosaicked to cover the land surfaces of the San Francisco Bay region. Next, to quantify high water levels throughout the Bay, a hydrodynamic model of the San Francisco Estuary was driven by a projection of hourly water levels at the Presidio. This projection was based on a combination of climate model outputs and empirical models and incorporates astronomical, storm surge, El Niño, and long-term sea level rise influences" (Knowles 2009). The Bay computer model simulates the water surface elevation for each hour from 2000–2009. Inputs to the model include both upstream inflows and downstream water surface elevations (Figure 5).

Dr. Knowles performed statistical analyses on the Bay model output to determine flood quantiles for various years and provided outputs in the form of GIS raster files to the Pacific Institute. These files were provided for five flood recurrence intervals (Table 2) for each of four years between 2000 and 2099, for a total of 20 files. Based on this information, we estimated risks due to inundation with a 0.5 m, 1.0 m, and 1.4 m sea-level rise, which for the A2 scenario correspond to 2050, 2081, and 2099, respectively.

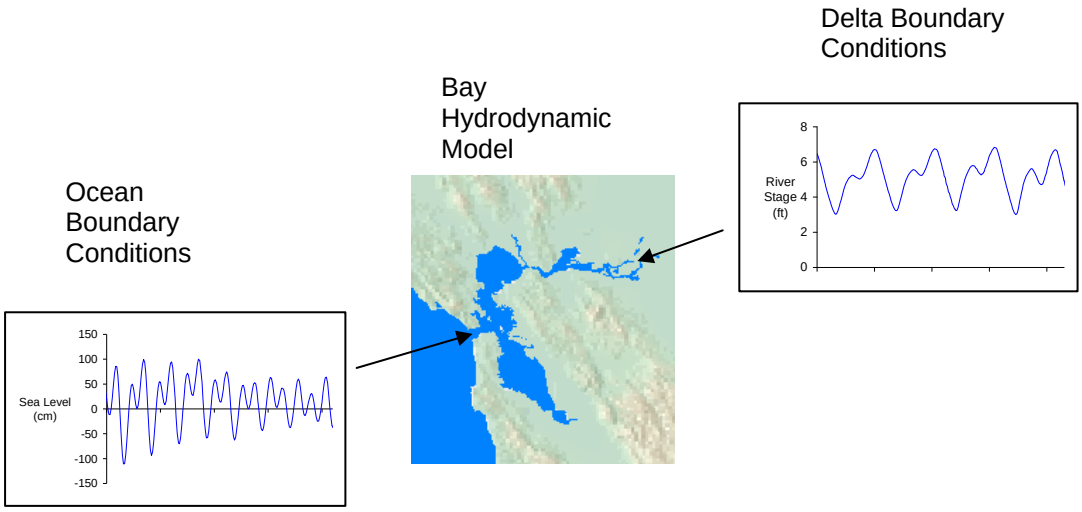

Figure 5. Simple schematic of USGS San Francisco Bay hydrodynamic model

It is important to note that we report results based on the vertical rise in sea level rather than a particular year in which the rise is projected to occur. As shown in Table 3, the year in which a 0.5 m sea-level rise is projected to occur under the A2 and B1 scenarios differs by only three years. Additionally, sea-level rise estimates are continuously updated as climate science advances and greenhouse gas emissions change over time. Indeed, carbon dioxide emissions in 2005 and 2006 were well above even the highest future emissions scenario, as shown in Figure 6 (Raupach et al. 2007). Because the results of this analysis are driven by sea levels and are not directly tied to any set of scenarios, the results of this study will be relevant even when climate projections change.

Table 2. Recurrence intervals of inundation estimates

Flood Interval	Annual probability
1-year	1
10-year	0.1
50-year	0.02
100-year	0.01
500-year	0.002

Table 3. Year and estimated mean sea-level for inundation estimates under the A2 and B1 scenarios

Mean Sea-Level Rise (m)	Year Reached	
	A2	B1
0	2000	2000
0.5	2054	2057
1.0	2083	2098
1.4	2100	2125

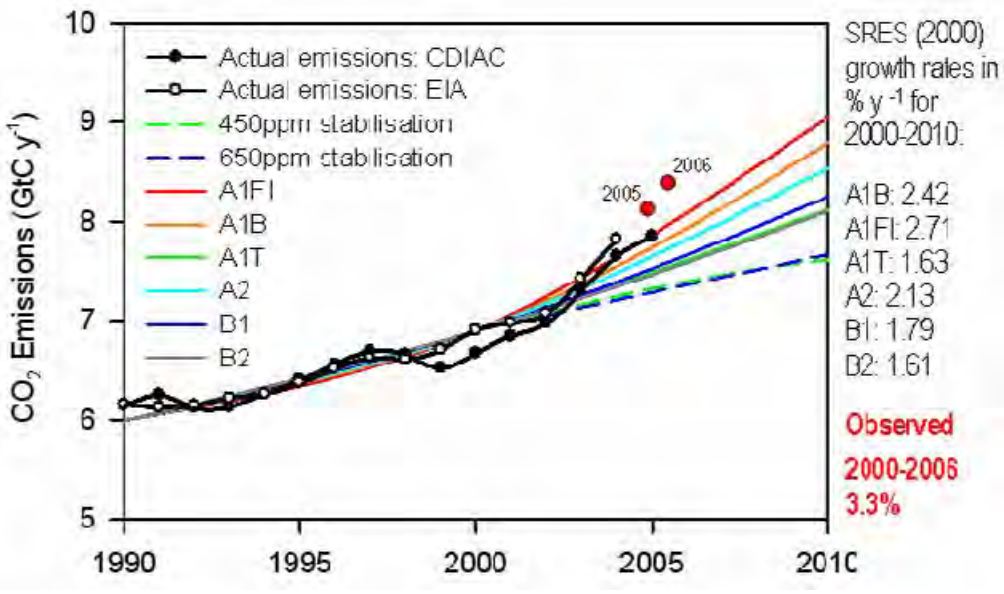

Figure 6. Historical and projected carbon dioxide emissions scenarios, 1990–2010

Note that actual emissions for 2004–2006 exceed the highest IPCC scenarios.

Source: Raupach et al. 2007

2.3.2. Erosion Risk

Large sections of the Pacific coast, especially those with rocky headlands or sea cliffs, are not vulnerable to flooding, but are highly susceptible to erosion. In areas where the coast erodes easily, higher sea levels are likely to accelerate shoreline erosion due to increased wave attack. In addition, erosion of some sand spits and dunes may expose previously protected areas to flooding.

The amount of erosion can be estimated by several methods. The most widely applied method of predicting shoreline recession based on a sea-level rise was developed by Bruun in 1962. This is based on the concept that the depth of water near the coast remains constant with sea-level rise, that the basic beach profile will remain the same, and that there is a well-defined offshore limit of sediment transport. The sediment required to maintain the beach profile through water-level changes is derived from erosion of the shore material. Based on this, an approximate estimate of the shoreline recession due to readjustment of the beach profile to an equilibrium state is 1.0-to-1.5 meters of shore recession per centimeter of sea-level rise.

Although once widely used, the Bruun rule has been largely abandoned because it makes several assumptions that may not be accurate (Pilkey and Cooper 2004). The formulation is based on a two-dimensional concept, while the sediment transport along a shoreline is a three-dimensional process. The Bruun rule assumes a shoreline profile in equilibrium, which is difficult to confirm at any site. Another problem is that this approach always predicts shoreline recession with offshore sediment transport as sea-level rises, yet there are several cases where shorelines have accreted as a result of sea-level rise due to the movement of sand onshore from offshore deposits. Depending on local sources and sinks of sediment, wave climate, topography, and other conditions governing sediment transport mechanisms, the predictions of shoreline

recession obtained using the Bruun rule can significantly over- or underestimate the future recession. More specific methods are needed for particular sites, and should be conducted to better evaluate the impact of sea-level rise on a given region.

A team of scientists and engineers at Philip Williams and Associates (PWA) developed an alternative approach to evaluate erosion risk. They evaluated potential future erosion by examining changes to a time series of total-water level (TWL) elevations. TWL is a water elevation determined by the sum of mean sea level, tides, waves and wave run-up, other storm components (including surge), and El Niño (Ruggiero et al. 1996; Ruggiero et al. 2001). Studies suggest that erosion will accelerate as sea levels rise and the coast is exposed to higher waves. Higher water levels result in greater wave energy being dissipated higher up on the shoreline and directly onto the face of cliffs and dunes. The exceedance of TWL above the elevation of the toe junction has been related to erosion (Sallenger et al. 2002; Ruggiero et al. 2001; Hampton and Griggs 2004; FEMA 2005).

To generate the TWL predictions, PWA used a 100-year time series of "measured tides" and deepwater waves from Dr. Dan Cayan and colleagues at Scripps (Cayan et al. 2009). The deepwater wave heights were transformed to 140 near-shore locations by the Coastal Data Information Program to account for differences in wave exposure and shoreline orientation. Finally wave run-up was calculated using the relationship between wave height, wave period, and beach slope (Stockdon et al. 2006). The combination of sea levels and wave run-up were evaluated over time to estimate future elevations of TWL, which were then intersected with the land elevations along 4,100 segments of the coast.

California's coastline is geologically and morphologically complex and each major geologic unit will exhibit differential response to rising sea levels. PWA classified the shoreline based on geologic formations and type, such as sea cliffs and dunes. For each type of coast, slightly different methods were used to project the response to rising seas. For sea cliffs, which accounted for 720 miles of the study area, erosion was estimated based on an acceleration of the historic erosion rate and a percent increase in TWL exceeding the elevation of the toe of the sea cliffs. The historic sea cliff erosion data were obtained from the USGS National Shoreline Change Assessment (Hapke and Reid 2007). The data were averaged by geologic unit with an additional factor of safety (two standard deviations) included to account for subtle changes in geology along the coast.

For the dune classified shorelines, which covered about 170 miles of the study area, erosion rates were based on the following information:

- Recession based on changes in TWL from sea level-rise.
- Historic shoreline change trends from the USGS National Shoreline Change Assessment (Hapke et al. 2006).
- The impact of a "100-year storm event" extracted from the TWL time series and estimated using a storm-response geometric model of dune erosion (Komar et al. 1999).

Based on this approach, PWA developed digital GIS shapefiles representing future coastal erosion hazard zones for cliff-backed and dune-backed coastal areas for 2025, 2050, and 2100 under a low (1.0 m) and a high (1.4 m) sea-level rise scenario. For this analysis, we evaluate the socio-economic impacts of erosion under the 1.4 m sea-level rise scenario for 2100. Note that for erosion, the year is important because it includes a background erosion rate plus accelerated erosion rates resulting from sea-level rise.

The study area of the erosion analysis extended from Santa Barbara to the Oregon border, covering about 930 miles (1,450 km). Much of the Southern California coast was excluded due to the myriad of ongoing initiatives focused on climate change and hazards mapping. Due to insufficient data, however, PWA was only able to include 80% of the 930 mile study area (see Section 2.4 for additional discussion of the limitations).

The erosion analysis represents a first-order evaluation of coastal hazards based on currently available projections of water levels and wave conditions and interpretations of sea-level rise, shoreline change rates, and geomorphic conditions. Available methods and data are not sufficient to model coastal erosion with high confidence. While the methodology used to develop the hazard zones was kept relatively simple and modular to facilitate understanding and future application with minimal effort, it represents one of the most comprehensive erosion hazard assessments under conditions of climate change ever completed for the California coast. For additional information, see PWA (2008).

2.3.3. Limitations of the Analysis

Researchers at Scripps Institution of Oceanography and USGS performed hydrographic modeling of the San Francisco Bay Estuary to determine the flood elevations under climate change scenarios. All models are subject to errors and inaccuracies. It was not possible to directly calibrate or verify a model that predicts flood frequencies. We performed an independent evaluation of USGS-predicted San Francisco Bay flood elevations and found that the model estimates of the 100-year water surface elevation for the year 2000 were generally similar to flood elevations predicted by the U.S. Army Corps of Engineers (USACE 1984a). We compared all 52 points on the San Francisco Bay shoreline shown on the 1984 Corps maps and found that 75% of the flood elevations were within 0.25 feet of those predicted by USGS. Most of the new estimates were slightly lower than the heights estimated by the Corps, as shown in Figure 7.

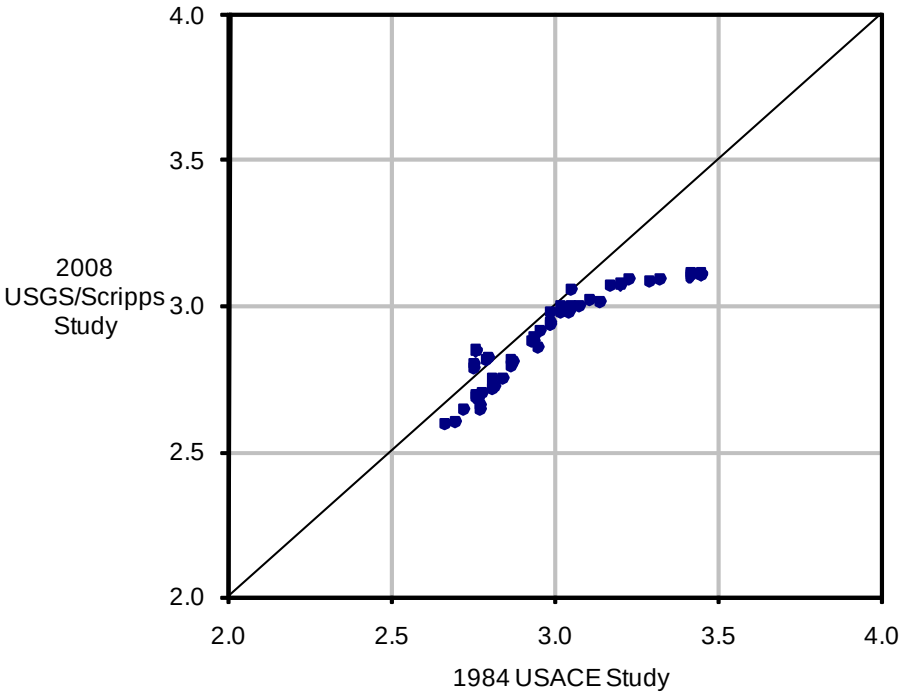

Figure 7. Comparison of 100-year flood elevations (in meters NAVD88)

The location of the shoreline is inexact and probably subjective. Knowles used a "mask" of open water as a filter, so as to report only land areas that are flooded. However, the shoreline is constantly in flux and difficult to map precisely. Further, there are errors and inaccuracies in the terrain data. The digital terrain model creates a smoothed or average surface from the raw elevation data, and it does not accurately depict breaks in elevation that occur at a vertical wall such as a cliff or a curb.

Another limitation is that the automatic, computerized method classifies flooding by depth only. The algorithm using depth alone to determine flooding does not factor in the presence of a flow pathway. In some cases, the high ground may be a levee specifically designed to protect adjacent low-lying areas. In other locations, there are simply depressions, but they are not really at risk because there is no path for seawater to flow into them. This means low-lying objects or features such as ditches, stormwater detention basins, subway tunnels, and empty swimming pools are filled in inappropriately at times, as shown in Figure 8.

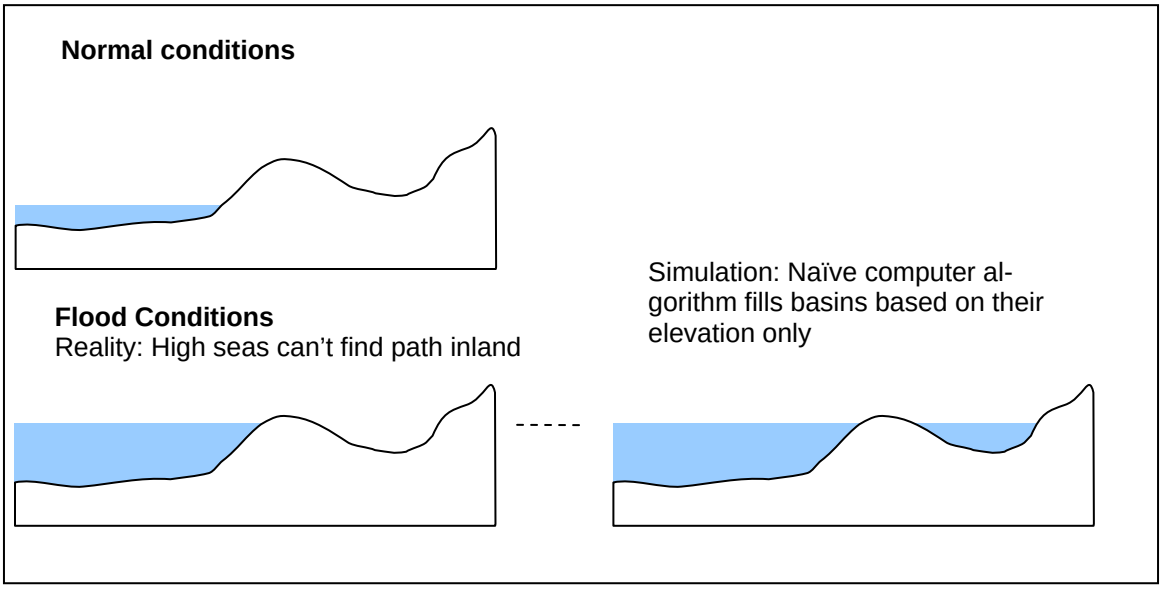

Figure 8. Limitations of the computer's ability to accurately map coastal flooding in areas protected by seawalls or levees or natural barriers

The study area for the erosion analysis was constrained by data availability. The erosion analysis covered only the 11 counties north of Santa Barbara County. Furthermore, data limitations limited the analysis to only 81% of the coast in the 11 counties (Table 4). The three counties with the least coverage include Humboldt County, Monterey, and Santa Barbara. Humboldt County included the Kings Range and the Lost Coast, public lands with no development. The Monterey County analysis was limited along the Big Sur coast where high levels of erosion currently affect the major transportation corridor of Highway 1 and are expected to continue. In Santa Barbara, missing data along the region between Pt. Conception and Goleta and the ending of the erosion analysis south of Santa Barbara harbor explain the missing erosion analysis. As a result, the vulnerability assessments underestimate the actual economic impact from erosion. Note that the flood analysis covered the entire Pacific coast of California and results for the erosion analysis were not adjusted to account for missing segments of the coast.

Table 4. Miles and fraction of coastline studied for the erosion hazard study, by county

County	Studied	Total	% Studied
Del Norte	42.7	49.7	86
Humboldt	72.9	123.3	59
Marin	69.5	75.2	93
Mendocino	145.5	151.4	96
Monterey	94.4	132.0	71
San Francisco	7.5	8.8	85
San Luis Obispo	77.0	102.6	75
San Mateo	57.8	59.6	97
Santa Barbara	84.4	116.5	72
Santa Cruz	46.0	46.0	100
Sonoma	63.0	68.9	91
Total	**760.7**	**934.1**	**81**

2.4. Resources Threatened by Sea-Level Rise

In any given area, rising seas pose a threat to many different types of resources. Among the vulnerable coastal systems are transportation facilities such as roadways, airports, bridges, and mass transit systems; electric utility systems and power plants; stormwater systems and wastewater treatment plants and outfalls; groundwater aquifers; wetlands and fisheries; and many other human and natural systems from homes to schools, hospitals, and industry. Any impacts on resources within the affected area may lead to secondary impacts elsewhere. Determining the types of resources threatened by sea-level rise is a crucial step toward choosing an appropriate level of response and method of protection.

2.4.1. Population

Sea-level rise and increased coastal flooding will lead to disruption due to evacuations, displacement from destruction of homes and property, and possibly the loss of lives. To determine populations at risk if no adaptation actions are taken, we overlay the inundation and erosion hazard maps with year 2000 census block data. We use current population data aggregated by census block, the highest resolution available for California. We make an assumption common in regional GIS analyses that the population is distributed evenly within a block's boundaries. So if our mapping shows that 50% of a 500-person census block is inundated by a flood, we estimate that 250 people are at risk. This method may underestimate (where the houses are clustered on the coast) or overestimate (when the houses are set back from the coast) the actual risk.

While disasters do not discriminate, the existing societal and environmental conditions before, during, and after a disaster produce differences in vulnerability among groups within the population affected.

It is critical to understand that our estimates of populations at risk are based on current population data, not a projection of populations that might be at risk in the future. If no policies are put in place to limit new exposure in areas at risk of rising seas, our estimates will be low—perhaps substantially low. If, however, policymakers are proactive about reducing coastal risks in coming decades, the levels of risk could be substantially reduced.

We also evaluate potential environmental justice impacts of sea-level rise.[6] As seen during Hurricane Katrina, flooding and other natural disasters often do the greatest harm to low-income communities and communities of color. Hurricane Audrey, for example, struck the coast of Louisiana in 1957 and had a death rate of 38 per thousand among whites and 322 per thousand among blacks (Bates et al. 1963, cited in Pastor et al. 2006). A study of all U.S. disasters between 1970 and 1980 found that white households had $2,370 less of a financial burden following a disaster than other racial groups (Rossi et al. 1983). One year after Hurricane Katrina, the black population of New Orleans had decreased 57% while the white population had fallen 36% (Frey 2007). Racial disparities are mirrored in economic disparities where low-income communities have shouldered a disproportionate burden of harm resulting from disasters: reports following Hurricanes Hugo and Katrina pointed to a range of problems related to the "invisibility" of low-income communities before the disasters (Pastor et al. 2006).

The uneven distribution of the harms of natural disasters highlights the same racial and economic inequities present in the distribution of other environmental risks and benefits, which in the 1980s catalyzed affected communities to develop the framework of "environmental justice." This framework was ultimately affirmed by the United States Environmental Protection Agency (U.S. EPA) in its 1992 creation of what is now called the Office of Environmental Justice, which holds that

> "no group of people, including racial, ethnic, or socioeconomic groups, should bear a disproportionate share of the negative environmental consequences resulting from industrial, municipal, and commercial operations or the execution of federal, state, local, and tribal environmental programs" (U.S. EPA).

Presidential Order 12898 of 1994 expanded the application of environmental justice principles in its decree that "each Federal agency shall make achieving environmental justice part of its mission" (Presidential Executive Order 12898).

We use the environmental justice framework in two analyses that are relevant to understanding the full costs of sea-level rise in California. The first is a simple analysis looking for potential inequities in who is likely to be directly exposed to sea-level rise, within the geographic units at which relevant political decisions are made. In this case these geographic units include the state of California as a whole and each county affected by sea-level rise. We urge further studies looking at possible inequities at different spatial scales, e.g., within cities, neighborhoods, and metropolitan regions. Our second environmental justice analysis focuses on the factors of

[6] Here, we evaluate the environmental justice impacts of flooding but not erosion. Additional analysis should examine erosion as well.

vulnerability and the differential vulnerability to the impacts of sea-level rise of people from different demographic groups.

A third analysis, which is beyond the scope of this study, should focus on potential inequities in the distribution of the resources invested to protect and adapt to sea-level rise. Here we focus on completing a part of the first and second analyses, and leave the third analysis for future studies.

Any analysis of populations affected by sea-level rise should include a broader discussion of vulnerability to these events. According to the IPCC, "Vulnerability to climate change is the degree to which these systems are susceptible to, and unable to cope with, adverse impacts" (Schneider et al. 2007). Vulnerability is a function of the magnitude of the impact, the sensitivity of the system to that impact, and the system's ability to adapt. Vulnerabilities, like lack of access to a vehicle or other means of transportation, are shaped by "intervening conditions" that are not tied to a specific hazard but will greatly determine the human impact of the disaster and the specific needs for preparedness, response, and recovery (Hewitt 1997).

Here, we report key population characteristics that increase vulnerability to the adverse impacts of flood events and disasters for low-income people and communities of color. We sort the types of vulnerabilities and key demographics correlated with increased vulnerability, according to the three phases of a disaster event: preconditions, disaster, and recovery and reconstruction (Hewitt 1997). Figure 9 offers a conceptual model of the relationship between demographics, vulnerabilities, and human impact. Our analysis is limited to two factors: the distribution of race and income. A more comprehensive analysis of the human impact of sea-level rise is needed for all vulnerable subgroups, including children, elderly, homeless, and incarcerated residents.

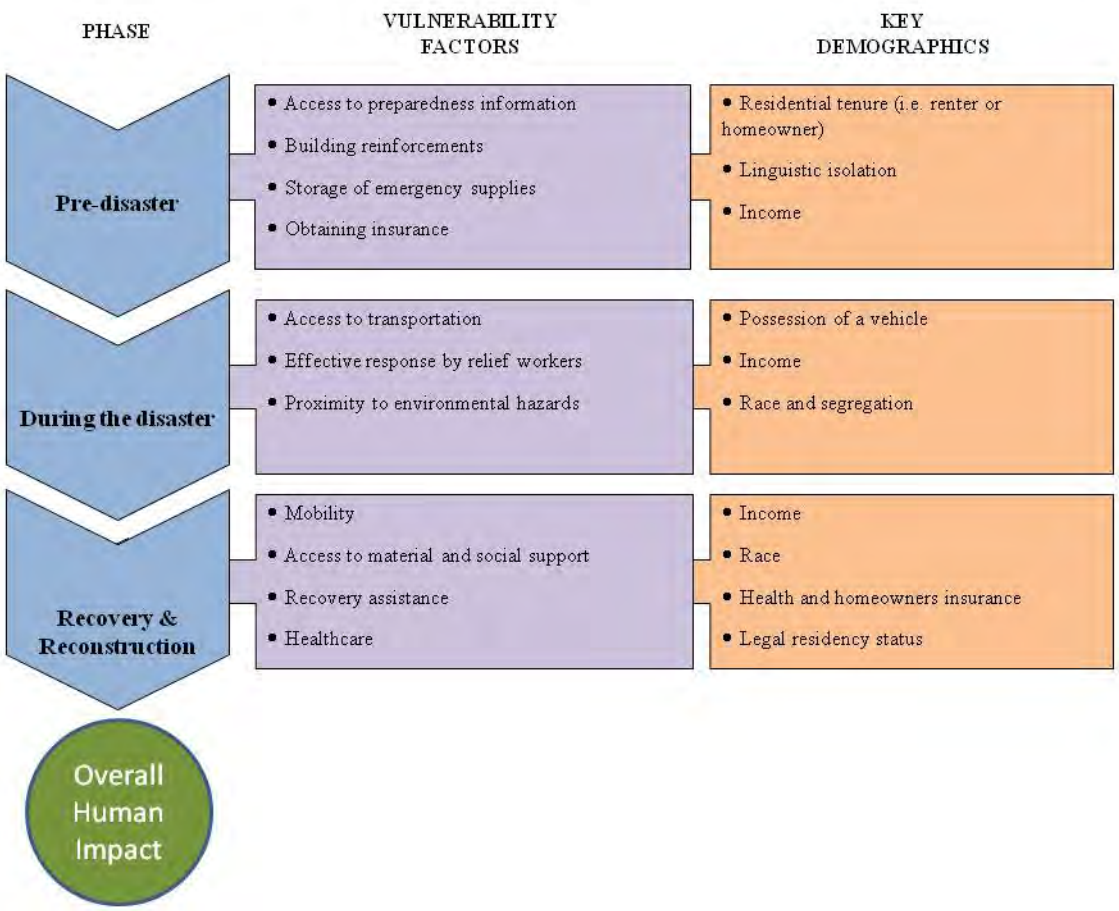

Figure 9. Relationship between demographics and vulnerabilities

2.4.2. Impacts on the Built Environment

Extensive development has occurred in areas already threatened by erosion and floods along the California coast. Residential homes along the California coast often draw a premium price as a result of their location. Some homes in coastal zones are protected by levees and revetments; many are not protected at all. Additionally, high-value commercial, industrial, and transportation facilities are also located along the coast. Such facilities make use of the waterfront for waste disposal, movement of goods or people, or commercial activities. Among the most common coastal facilities are airports, railroad tracks and terminals, highways, power plants, waste-disposal sites, waste-treatment plants, ports and docks, warehouses, salt ponds, and marinas. Existing forms of protection for these facilities vary greatly, from bulkheads and engineered seawalls to riprap and non-engineered levees. An increase in sea level will increase the severity of possible damages in threatened areas and will expand the size of flood and erosion zones.

Data on the replacement value of buildings and contents were taken from datasets supplied with the HAZUS model, which was developed for FEMA's Mitigation Division by the National Institute of Building Sciences. HAZUS was designed to help planners estimate the potential

losses from natural disasters such as earthquakes, floods, and hurricane winds. HAZUS uses a database called the "General Building Stock Inventory" that contains the value of buildings and contents based on data from a number of sources including the U.S. Census Bureau, Dun & Bradstreet (a business listing service), and the U.S. Department of Energy. HAZUS estimates direct economic losses based on the repair and replacement of damaged or destroyed buildings and their contents, and includes the following:

- Cost of repair and replacement of damaged and destroyed buildings
- Cost of damage to building contents
- Losses of building inventory (contents related to business activities)

Replacement values are provided for residential, commercial, industrial, agricultural, religious, governmental, and educational developments and are compiled at the census block level. See Section 14.2 of the HAZUS technical manual for additional detail (FEMA 2006). To determine the replacement value for the areas at risk, we overlay the inundation maps with year 2000 census block data. We assume that if 50% of an area is affected, then 50% of its assets are at risk. For inundation risks, we use replacement value, as described in more detail below, because flooding does not usually destroy property and land value completely. In contrast, erosion often completely destroys the property. As a result, replacement value is not appropriate for evaluating the economic cost of erosion and was not used for that part of the study. For the erosion analysis, we assume that the value of the average coastal property is about $1.4 million (Heinz Center 2000).

We compared replacement costs and the market value of homes at a few locations along the California coast and found that the replacement costs in HAZUS can substantially underestimate actual market values for residential properties. According to the HAZUS database, the median home replacement values range from $63,000 in Del Norte County to $135,000 in San Mateo County (Figure 10). In comparison, the median home price in California was $286,000 in November 2008. In Northern California, the median price was $307,000, and in the San Francisco Bay Area, the median price was $474,000. Of course, homes on the coast are usually much more expensive.

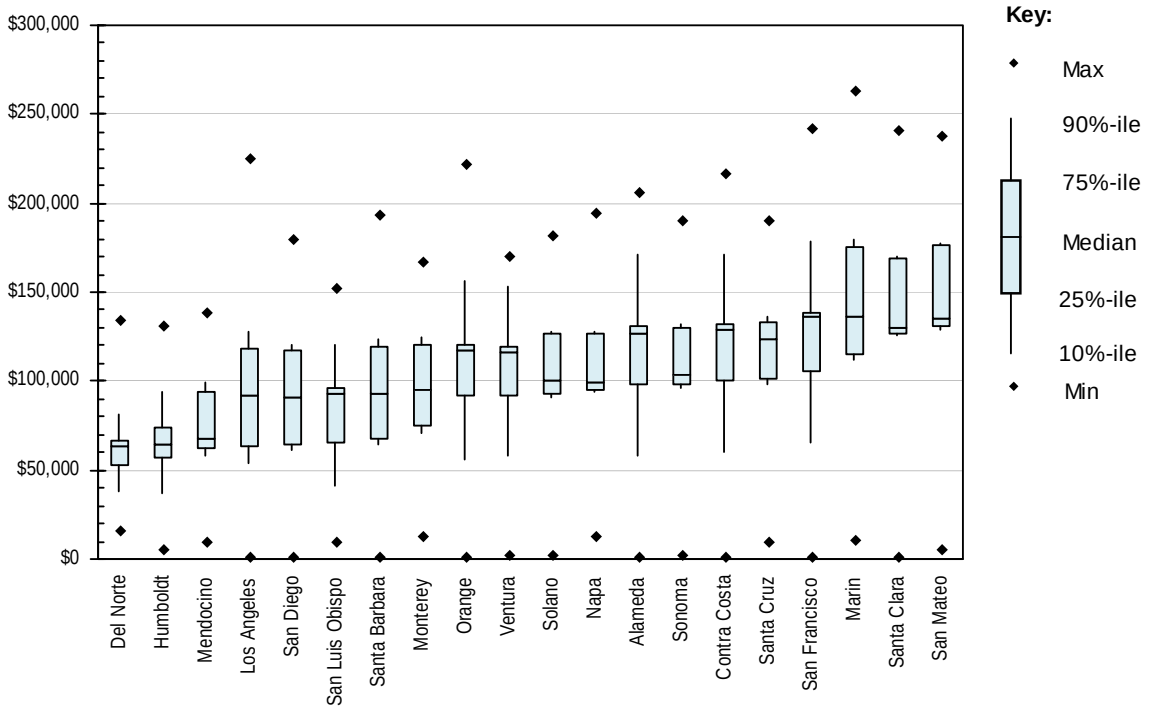

Figure 10. Distribution of census-block average replacement costs for single-family homes from HAZUS

The difference between the replacement value and the market value of a home is likely due to several factors. Home values are determined by more than the cost to build the house, including land value, neighborhood, school district, and dozens of other tangible and intangible factors. In addition, the HAZUS documentation warns that replacement value is based on national-average construction costs, which are much lower than construction costs in California. Future studies should include more detailed estimates of California construction costs.

Parcel data from each county assessor's office provide higher spatial resolution, but there are some significant limitations to using these data. First, we were unable to obtain complete coverage for all coastal counties. In some counties, parcel data have not been converted to a digital format, while others claimed that sharing these data was a threat to Homeland Security. Second, even where parcel boundary files are available, these must be linked to the value of the property. While obtaining a list of affected parcels is straightforward, most counties do not readily share their tax rolls or tables with assessed value. This information is part of the public record, and can legally be requested in person or by phone from a county assessor's office, but this approach is not feasible for a regional analysis where hundreds or thousands of parcels are affected. Third, even if assessed value were readily available to us, it often bears little relationship with the actual market value of a property. Finally, assessed value will not include any publicly owned buildings, so it would exclude many police and fire stations, government buildings, park buildings, schools, water treatment plants, and others.

Important transportation infrastructure is also at risk of flooding and erosion from projected increases in sea-level rise (Figure 11). We estimate the miles of roadways and railroads at risk by overlaying the GIS inundation and erosion hazard layers with transportation data from Tele Atlas. We note that because there are not elevations associated with the roadways, it is difficult to infer the extent to which the roadway is at risk from flooding. Additionally, the railroad data do not provide information on the number of tracks, e.g., single, double. We also do not provide estimates of the value of this infrastructure because adequate data are not available. Thus, the information on roads and railways is presented as miles of structures at risk rather than value, but it provides an indication of the areas at risk and those warranting additional analysis.

Figure 11. Flooding of a coastal road in Santa Cruz, California

Photo courtesy of David L. Revell

A number of other facilities along the coast are also at risk of flooding and erosion. We evaluate the sites and facilities at risk by overlaying the GIS inundation layer with the relevant spatial data. Data on the locations of schools and emergency facilities come from the HAZUS geographic database (FEMA 2006). Data on licensed healthcare facilities come from the California Office of Statewide Health Planning and Development (2006). Data on coastal power plants were provided by the California Energy Commission.

Data on U.S. EPA-monitored hazardous materials sites were from the U.S. EPA Geospatial Data Access Project 2008 and included Superfund sites, hazardous waste generators, facilities required to report emissions for the Toxics Release Inventory, facilities regulated under the National Pollutant Discharge Elimination System (NPDES), major dischargers of air pollutants with Title V permits, and brownfield properties.[7] The Pacific Institute developed a geographic database of wastewater treatment plants based on data in the U.S. EPA's Permit Compliance System (PCS) database, by interpreting aerial photos and by telephone and Internet research.

2.4.3. Natural Resources

Wetlands are among the Earth's most productive ecosystems. Once abundant across the United States, wetlands have been extensively drained and filled to make way for agricultural, industrial, commercial, and residential development. Pollution and invasive species threaten the health of the remaining areas. The U.S. EPA estimates that more than 220 million acres of wetlands existed in the lower 48 states in the 1600s. By 2000, only 100 million acres of wetlands remained (U.S. EPA 2001). In some parts of the United States, wetland loss was even more severe. In California, for example, more than 90% of the historic wetlands have been lost to development. Growing recognition of their importance and concern about their rapid decline has prompted wetland restoration efforts across the United States, including the San Francisco Bay. A recent U.S. Fish and Wildlife Service report suggests that the net wetland acreage actually increased between 1998 and 2004 for the first time as a result of restoration efforts and the construction of engineered wetlands (Dahl 2006).

While legislation has helped to protect wetlands from further destruction, rising seas threaten to substantially modify or destroy remaining wetland habitat. Most coastal wetlands in the United States are within one tidal range of mean sea level (Titus 1988), i.e., between mean high tide and mean low tide. Thus, as noted by Titus (1988), if sea levels rose by one tidal range overnight, "then all of the existing wetlands in an area would drown." Rising seas, however, may also inundate land that is now dry, thereby creating new wetlands. Wetlands may also be able to adapt to rising water levels over time by trapping sediment or building on the peat the sediment creates, a process referred to as vertical accretion. These compensatory mechanisms may be hindered by coastal development that limits wetland migration or rates of sea-level rise that exceed natural accretion rates.

Spatial Extent of Wetlands

In this analysis, we use GIS data from the National Wetlands Inventory (NWI) to determine the current spatial extent of wetlands along the California coast and the San Francisco Bay. While there is currently no single source that contains the boundaries of all existing wetlands, the NWI is the best dataset available. It is important to note that all datasets likely underestimate the actual wetland area. Wetland delineation is a time- and labor-intensive task requiring extensive field work by experts; vast areas have never been subject to detailed study.

[7] A *brownfield* is an abandoned industrial site available for redevelopment, often with environmental contamination.

The NWI does not make a clear distinction between coastal and upland wetlands. The datasets are distributed in tiles, with each tile containing a mix of marine, estuarine, and freshwater wetlands. We used a simple rule-based approach to decide which wetlands are coastal, or "coast-dependent"" we assume that coastal wetlands are generally limited to within 100 feet (horizontally) of the mean higher-high water line (Figure 12).

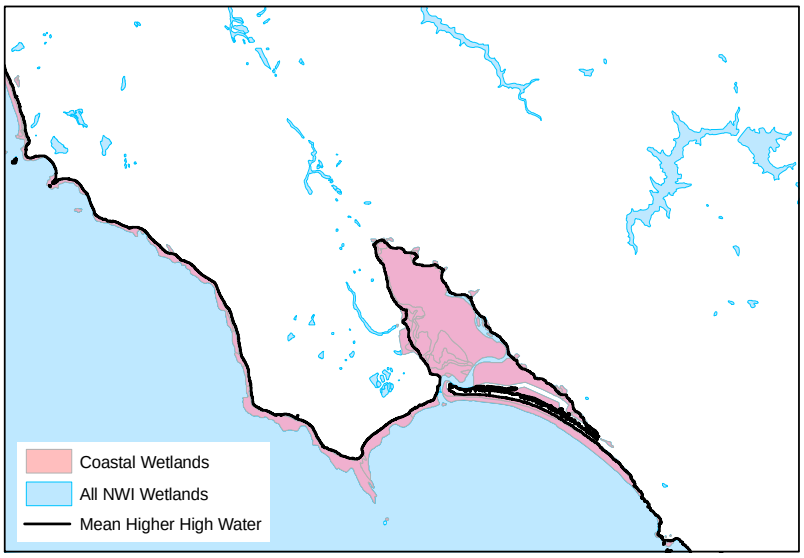

Figure 12. National Wetlands Inventory wetlands classified as "coastal" are below or adjacent to the MHHW line

Economic Value of Wetlands

Wetlands are highly diverse ecosystems that provide a variety of goods and services, including flood protection, water purification, wildlife habitat, recreational opportunities, and carbon sequestration. While there are rarely any direct market values for services provided by wetlands, such as biodiversity and flood control, there is a growing recognition that these services have real economic values and should be included in decision-making processes.

Methods for estimating the economic value of an ecosystem, including wetlands, can be done in one of three ways: direct, indirect, and proxy (Table 5). Each of these methods has strengths and weaknesses; each fails to fully capture the value of ecosystems. The unacceptable alternative, however, is to assign an economic value of $0—clearly acknowledged to be wrong. To put it simply, "we don't protect what we don't value" (Myers and Reichert 1997).

In recent years, a number of studies have attempted to estimate the economic value of wetlands. Based on a literature review and some original calculations, Costanza et al. (1997) estimate that the value of tidal marshes is around $5,700 per acre per year (in year 2007 dollars). In a meta-analysis of 39 wetland valuation studies, Woodward and Wui (2001) found that wetland values varied considerably according to the methods used, the type and location of wetlands evaluated, and the study characteristics. While the valuation method affected the value

obtained, the method was not the primary determinant of value. However, study quality was not a strong determinant either; weak studies yielded wetland values similar to strong studies, but with more error, suggesting that the quality of the study affects precision. The authors conclude: "From our analysis it is clear that the prediction of a wetland's value based on previous studies is, at best, an imprecise science. The need for site-specific studies remains" (Woodward and Wui 2001).

For this analysis, we estimate the economic value of wetlands in California using recent cost estimates for restoring wetlands. Numerous wetland restoration projects have been initiated in the San Francisco Bay, with the cost of restoring these tidal marshes ranging from $5,000 to $200,000 per acre (Hutzel 2008). The South Bay wetland restoration project, for example, is estimated to cost about $67,000 per acre (Hutzel 2008). We note that these estimates represent the public's willingness to pay for these ecosystems rather than their actual value, but without a more detailed site-specific analysis, the restoration costs are the best estimates available. We do not evaluate the ability of wetlands to adapt to these changes through vertical accretion or landward migration, but note that these processes could reduce damage to wetlands. We urge more detailed wetland valuation studies be conducted to improve these estimates.

Table 5. Approaches for estimating ecosystem values

Approaches	Description	Example	Weaknesses	Strengths
Direct	Surveys can be used to ascertain people's willingness to pay for benefits provided by the wetland or the level of compensation they would expect for the loss of those benefits. Such surveys measure the value of specific benefits.	A survey that asks users what they would be willing to pay to retain a recreational area.	This approach requires sophisticated survey design, analysis and interpretation.	This approach can measure relatively subtle changes in value and can also be used to calculate the value of non-use benefits.
Indirect	Economists use mathematical models to estimate wetland values based on the market demand for related goods and services.	Expenditures and the distance traveled by people visiting a wetland are used as indicators of the value of the wetland for recreational purposes. Similarly, real-estate price differences could be used to estimate the value of the wetland's aesthetic benefits.	This approach cannot measure non-use benefits (e.g., option or bequest benefits) or benefits that do not currently exist (e.g., the benefits of an enlarged wetland).	This approach is usually faster and less expensive, as it can be based on easily accessible data.
Proxy	The values of other goods and services are used to approximate the values of wetland benefits.	The replacement cost for a wetland benefit (e.g., water filtration), such as the cost of installing a buffer strip or building a water treatment plant, is used as a measure of the value of the benefit.	This approach may confuse costs and benefits. For example, using the cost of a water treatment plant estimates the cost rather than the value of water filtration, (i.e., people's willingness to pay for clean water).	This approach can be more quickly calculated, but the result is only a very rough estimate of value.

Source: Environment Canada 2001

Impact of Sea-Level Rise on Wetlands

Evaluating the impacts of sea-level rise on a particular coastal wetland area requires site-specific data on various physical and biological factors, as described above. While this information is clearly important for developing adaptation strategies, it is beyond the scope of this analysis. A simple method to estimate wetland loss is to compare wetland elevations to future tide elevations. If the areas are permanently inundated in the future, they will be converted to open

water and lose their value as wetland habitat. Data limitations, however, prevent us from performing even this simple analysis: the existing digital elevation models (DEMs) do not include data below the shoreline and the modeled mean lower low water mark, even with 1.4 m of sea-level rise, falls below this elevation. This means there are no data in the critical area where the boundary must be drawn. We recommend additional work in this area to create a DEM for the California coast that combines land surface elevations with accurate bathymetry to allow for more detailed study of potential wetland responses to sea-level rise. Given these data limitations, we evaluate the land cover *adjacent* to existing wetlands and the potential for these areas to support suitable wetland habitat. We note that this simplified analysis does not take into account erosion or accretion due to sediment movement, which is difficult to predict with any accuracy.

Wetlands exist in areas that are frequently, but not permanently, inundated. In *The Effects of Sea Level Rise on US Coastal Wetlands*, Park et al. (1989) assumed that all areas between mean lower water (MLW) and mean higher water springs (MHWS) are tidal wetlands (Figure 13). The MHWS is only a few centimeters from the mean higher high water (MHHW) datum, which is more readily calculated and tabulated in tide reports. We assume that wetlands will migrate to land areas that are below the future MHHW, which we estimate as current MHHW plus the projected 1.4 m sea-level rise.

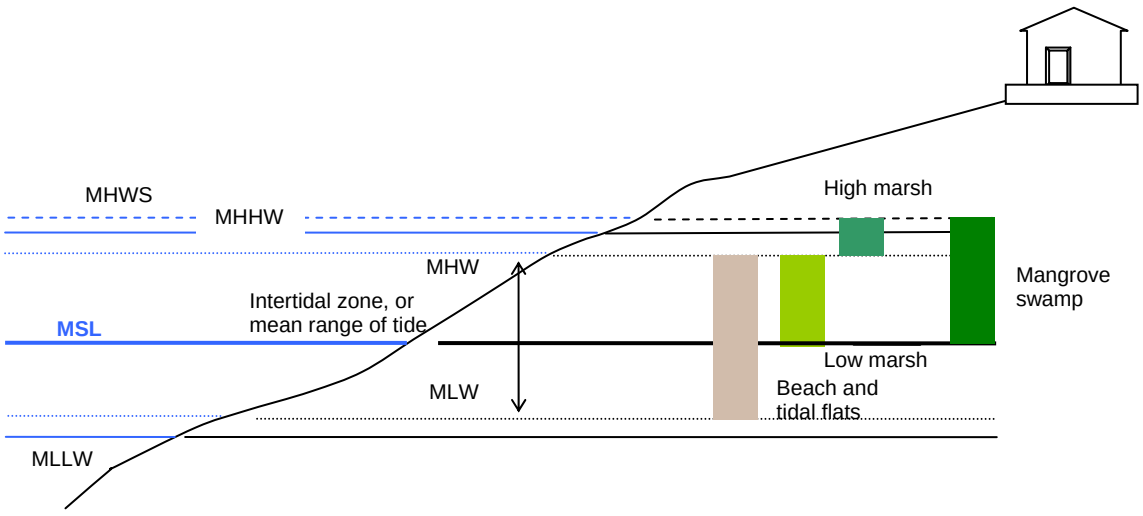

Figure 13. Assumed wetland area defined by the intertidal range

Adapted from Park et al. 1989.

The National Oceanic and Atmospheric Administration maintains tide stations along the California coast that provide measurements of MHHW. We interpolated the high-water elevation for the entire California Pacific coast using data from 12 long-term coastal tide gages. Each of these NOAA tide stations has been in continuous operation for over 25 years. The MHHW elevation for each of these stations is listed in Table 6. Using spatial interpolation tools

available in ArcGIS software, we developed a continuous grid or "surface" of MHHW elevations in year 2000.[8] To estimate MHHW elevations with a 1.4 m sea-level rise for the Pacific coast of California, we created a second surface by adding 1.4 m to each pixel in the year 2000 MHHW surface. The difference between the high water lines is the "wetland migration zone": the land into which wetlands must migrate to survive.

Table 6. Mean higher high water (MHHW) for long-term tide stations on California's Pacific coast

NOAA Station ID	Station Name	MHHW
9410170	San Diego, CA	1.61
9410230	La Jolla, CA	1.57
9410660	Los Angeles, CA	1.61
9410840	Santa Monica, CA	1.60
9411340	Santa Barbara, CA	1.61
9412110	Port San Luis, CA	1.60
9413450	Monterey, CA	1.67
9414290	San Francisco, CA	1.80
9415020	Point Reyes, CA	1.75
9416841	Arena Cove, CA	1.76
9418767	North Spit, CA	1.99
9419750	Crescent City, CA	1.98

Note: Elevations in meters above NAVD88 vertical datum. Tide datums calculated by NOAA for the 1983–2001 epoch.

Source: http://tidesandcurrents.noaa.gov/

We analyzed the land cover in the potential wetland migration zone using 2001 land cover data from NOAA's Coastal Change Analysis Program (C-CAP).[9] We rated each land cover type according to its suitability to support wetland habitat in the future. We assume that natural lands such as woodland, grassland, or shrub could provide suitable habitat for wetland plants and animals in the future when they are in the new intertidal zone and are intermittently wetted. Other land cover types may be viable for conversion to wetlands, but at a loss of some direct value to humans, e.g., farmland or parks. The third and final category represents built-up

[8] In some areas of Southern California, however, the available digital terrain data were not sufficiently detailed to complete the analysis. The terrain data do not include points below an elevation of 1.5 m NAVD88, and we could not map the current MHHW inundation extent for the entire coast. We mapped about 49% of Santa Barbara County, 23% of Los Angeles County, and 65% of Orange County. The coverage was 100% in the other 11 counties on the Pacific coast.

[9] The C-CAP data layer classifies land cover based on an adapted version of the Anderson et al. (1976) classification scheme and is estimated to have an accuracy of 85% (NOAA Land Cover Analysis website www.csc.noaa.gov/crs/lca/ccap.html).

areas that will likely provide unsuitable habitat for wetlands in the future due to the presence of buildings and other paved areas.

2.4.4. Limitations

Our analysis also has limitations related to the economic valuation methodology. For the flood analysis, we estimate the economic cost of sea-level rise based on estimates of the replacement value of buildings and their contents. We do not include estimates of the property or land value, which are much higher and should be included if inundation is permanent or leads the abandonment of property. Replacement values are also not appropriate for estimating the cost of erosion because it typically results in the total loss of property and land. We make a rough estimate of land values along the coast but note that additional study is needed.

Flooding and erosion can cause serious economic and social disruptions that are not captured in estimates of the buildings and infrastructure. For example, flooding events can cause deaths and injuries. Flooding or erosion of a major highway can prevent people from getting to work. Thus, estimating the replacement value and even some wetland values substantially underestimates the total cost of flood impacts and as a result, our findings should be considered conservative. A more detailed analysis would include transportation risks, lost work days, health issues, impacts on migratory bird habitat, and others.

We also do not factor in any expected changes in population density or the level of development in the regions at risk over the next century: these are largely unknown and will be determined by future policies. If policies are put in place to reduce development in regions of future flooding, society could over time reduce the risks. While limiting coastal development (an institutional adaptation) is likely the most effective way to reduce risk, this approach can also incur costs. Development permits designed to provide flexibility for future generations to address sea-level rise (e.g., development permits that allow development but stipulate that the area reverts to nature if seas rise a specified amount) may reduce today's cost. Conversely, if current development in coastal areas continues unchecked, a far larger population and far more infrastructure will be vulnerable than at present. We make no estimates of these changes, but future research could look at different scenarios for growth and coastal development and integrate them into the assessment tools developed here.

2.5. Determine the Protective Responses Appropriate for the Region

Each of the resources and facilities described in Section 2.4 can be protected by some combination of structural and non-structural measures. Some of the possible structural measures include building or improving coastal defenses such as dikes and dunes, seawalls, bulkheads, and other structures. Non-structural measures include abandoning property and land and moving to less threatened areas and beach nourishment. Perhaps the most effective non-structural response is to prohibit development in regions likely to be threatened in the future. This choice, however, requires the most forethought and planning. Below, we describe some of the structural measures and their associated costs.

2.5.1. Structural Coastal Protection Measures

Beach Nourishment

The addition of beach sand to a shoreline has been used to construct beaches where none had previously existed and to replenish eroded sand. As a response to the expected increase in erosion due to sea-level rise, the purpose of beach nourishment is to restore the width of an eroding beach on a temporary basis, although nourishment can also provide long-term restoration in certain types of areas. The rate at which the replenished beach erodes is a function of wave action, the uniformity of placement of the sand, and the grain size (U.S. Army Corps of Engineers 1984b). The sand used for a beach nourishment project usually comes from offshore dredging and pumping to the desired site; less frequently material is imported from an off-site location. The cost of the material can vary greatly depending on its origin and associated transportation costs.

Groins

One type of structure designed to lessen the impact of coastal processes on a shoreline is a groin —a structure oriented perpendicular to the shore that serves to reduce the flow of sediment along a shore (the local littoral drift rate). Sand collects on the updrift side of the groin until it is filled to capacity, when longshore drift is allowed to pass. Groins are often used in fields (sets of more than one groin) to protect a long section of coastline. The shoreline immediately downfield of the groin field, however, is often subjected to accelerated erosion, especially when the groins are not filled with sand during construction (National Research Council 1987).

Sea-level rise can affect a groin by reducing its effectiveness due to "flanking" or "submergence." A groin typically extends landward to the dune line, and the dune line may retreat due to sea-level rise, leaving the groin susceptible to flanking during high or storm tides, allowing sand to bypass the groin. Submergence of the groin can lead to overtopping by the longshore current, further decreasing the structures' efficiency at stabilizing the area (National Research Council 1987).

Seawalls, Bulkheads, and Revetments

There are three principal forms of vertical shoreline walls used to protect upland areas from storm surges and high tides: seawalls, bulkheads, and revetments. The differences between seawalls, bulkheads, and revetments are in their protective function. Seawalls are designed to resist the forces of storm waves; bulkheads are to retain the fill; and revetments are to protect the shoreline against the erosion associated with light waves (U.S. Army Corps of Engineers 1984b). These structures tend to fix the position of the coast. While this strategy may protect upland development, there are two kinds of adverse consequences of these types of structures. *Placement loss* refers to the loss of beach due to the footprint of the structure. For seawalls this is not as great as a revetment, which is usually built at a 2:1 (horizontal:vertical) slope. The other impact of these structures is called *passive erosion.* As sea level rises, and the structure fixes the position of the shoreline, the beach in front of the structures can be "drowned," resulting in a loss of recreation opportunities and habitat (Griggs 2005).

Breakwaters

Offshore breakwaters are above-water structures parallel to the shore that reduce both wave heights at the shoreline and littoral drift. Sea-level rise will reduce the protective capacities of breakwaters in two ways: rising water levels will effectively move the shoreline farther from the breakwater, increasing the ability of the waves to diffract behind the structure and reducing the sheltering and efficacy of the device; and the increased frequency of overtopping will diminish the ability of the breakwater to reduce the wave energy in the sheltered region (National Research Council 1987).

Dikes and Levees

Dikes or levees are embankments to protect low-lying land. A sea-level rise can result in reduced stability and increased overtopping of existing levees. New levees may be constructed to protect developed areas (National Research Council 1987). Whether existing levees can be modified for a rise in sea level depends on the availability of material for raising the levee, the suitability of the foundation material to support the additional weight of the material, the stability of the levee with the increased water level, and the accessibility of additional area for widening the base of the levee. Considerations for new levees also include issues such as land condemnation and interference of the levee with navigation (National Research Council 1987).

Raise Existing Structures (Roadways, Railroads, and Other Structures)

In some regions, building levees or seawalls to protect a small number of structures may not be cost effective. In these instances, raising the structures may be a better alternative. Roadways, railroads, and other structures may be raised so as to avoid damage from flooding. Over time, for example, we think it likely that important economic assets such as airports, transmission lines, or roadways will be raised rather than protected with levees or seawalls.

2.5.2. Cost of Structural Protection Measures

The cost of flood defenses is site-specific and little reliable information is available to generalize these costs. Gleick and Maurer (1990) developed cost estimates for building new coastal protection structures and raising existing ones, as well as raising roadways, railroads, and individual structures. We update these costs for this analysis based on a literature review (Table 7). Costs are converted to year 2000 dollars. Given the site specificity of construction costs, we relied on cost information from California where possible.

Data suggest that a new levee between 10 and 20 feet in height with a waterside slope of 3:1 would cost about $1,500 per linear foot (in year 2000 dollars). This represents a 320% increase over the 1990 estimate, much higher than the rate of inflation. The increase is likely due to large increases in construction and material costs in recent years. We estimate that raising existing levees would cost about $530 per linear foot (in year 2000 dollars). Seawalls, while providing significant protection, are among the most expensive option, estimated at about $5,300 per linear foot (in year 2000 dollars).

Table 7. Costs (in year 2000 dollars) for building new levees, raising existing levees, and building new seawalls

	Cost ($ per linear foot)	Location	Sources
New Levee	$725–$2,228	San Francisco, CA	Pang (2008)
Average New Levee	**$1,500**		
Raise Levee	$319	Central Valley, CA	Mount and Twiss (2005)
	$223–$1,085	San Francisco, CA	Moffatt and Nichol Engineers (2005)
	$278–$944	Central Valley, CA	Mount and Twiss (2005)
Average Levee Upgrade	**$530**		
New Seawall	$1,292	New England	Kanak (2008)
	$3,828	Southern California	Gustaitis (2002)
	$2,646–$6,173	Northern California	Stamski (2005)
	$5,654–$8,078	Philadelphia	PennPraxis (2008)
	$4,847	California	Crampton (2008)
Average New Seawall	**$5,300**		

Note: All costs are shown in year 2000 dollars. Costs shown for a new levee are based on a U.S. Army Corps of Engineers cost-estimation model, for a levee between 10 and 20 feet in height with a waterside slope of 3:1 and built using local materials.

In addition to the construction costs of the various structures described above, maintenance costs are often significant. In general, the greater the engineering employed in the construction of a shore protection scheme, the lower the proportion of maintenance costs. The maintenance cost of engineered riprap-revetment, for example, can amount to 2%–4% of the construction cost per year over the life of the project. This can be compared with the maintenance cost for a non-engineered revetment of 5%–15% of the construction cost per year (Fulton-Bennett and Griggs 1986). Average maintenance costs for levees are about 10% per year of the costs of construction. The estimated maintenance costs for seawalls run from 1%–4% per year, reflecting the higher level of engineering that goes into their construction. Because the majority of structures in our study are levees, we assume here an annual operation and maintenance cost equal to 10% of the capital cost of construction.

Levees, seawalls, and other structural methods have a number of environmental and social costs that are not reflected in the cost estimates shown in Table 7. Armoring the coast prevents natural movement and migration of the beach and associated ecosystems. In some areas, beaches may disappear completely, as shown in Figure 14. Structural measures can also increase vulnerability by encouraging development in flood-prone areas and giving those who live behind the structure a false sense of security. According to the United Nations,

"protective works have a tendency to increase the level of development in floodprone areas, as the assumption is made that it is now safe to build and invest in areas that are protected. However, it must be recognized that at some point in the future the design event will likely be exceeded and catastrophic damages will result" (United Nations 2004).

In addition, structural measures require regular maintenance, a task that is often overlooked due to budgetary constraints. Failure to maintain protective structures can lead to structural failures and catastrophic damage.

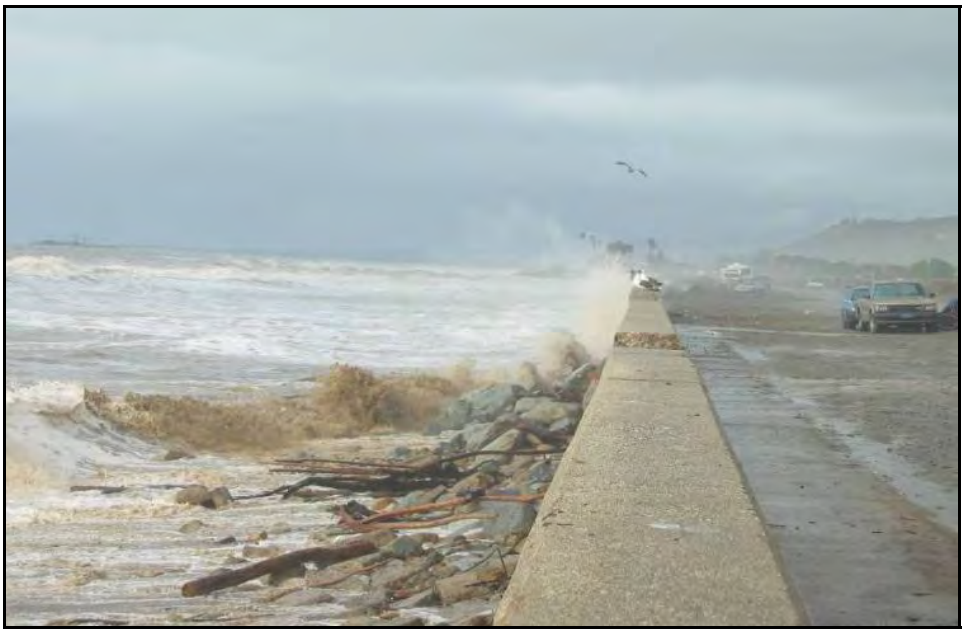

Figure 14. An example of coastal armoring leading to the disappearance of beach
Source: David L. Revell

2.5.3. Estimating Needed Coastal Defenses

Details about what level of protection to choose are a function of the perception of the value of the threatened property, the cost of alternative measures, and political and societal factors. In this analysis, we evaluate one scenario: the cost associated with raising the height of existing structures to maintain current flood protection levels and building new structures to protect some development that will be at risk of flooding with a 1.4 m sea-level rise. We do not evaluate coastal protection costs for erosion and urge additional studies on this topic.

In order to determine the cost of protecting development along the San Francisco Bay and California coast, we first needed to determine the location and type of existing coastal protection structures. Unfortunately, neither the U.S. Army Corps of Engineers nor any other agency maintains a comprehensive database with this information. The California Coastal Commission, however, recently compiled spatial data on the location and type of protective structure along the Pacific coast, e.g., groins, revetments, levees, and seawalls. Similar data were not available for the San Francisco Bay. Digital Flood Insurance Maps (DFIRMs) that showed

the presence of protective structures in the San Francisco Bay, however, were available in some areas. We supplemented the DFIRMS with a visual assessment of aerial imagery of the region. Because the DFIRMs do not distinguish between the types of structure, we assumed that seawalls were located around high-density, highly valued areas and levees were located around all other areas.

Geospatial data on the existing coastal protection structures were overlaid with the inundation maps to determine where existing structures needed to be raised and new structures built. To make this determination, we made the following assumptions:

- Existing coastal protection structures are strengthened and raised by 1.4 m with no change in the type of protection, e.g., levees are raised but are not replaced by a seawall.
- New coastal protection structures are needed wherever built structures are at risk of flooding. Agricultural land was not protected, unless a levee already existed.
- Seawalls are used in areas along the Pacific coast that are currently not protected but will need protection in the future and in areas where space limitations due to development prohibit the construction of new levees.
- Levees are used within enclosed areas, like the San Francisco Bay, that are currently not protected but will need protection in the future. These bays are protected from wave action, and we assume that levees will provide sufficient protection.

3.0 Results

Here we report on the results of our analyses for San Francisco Bay and the Pacific coast. In particular, we report on the population, infrastructure, and property at risk from sea-level rise, as well as the impacts on harder-to-quantify coastal ecosystems. We also provide an estimate of the economic costs of building coastal protections of different types to protect lives and property from flooding. All economic values are reported in year 2000 dollars. Results are reported separately for the flood and erosion risks.

3.1. Flood-Related Risks

In this analysis, we use the 100-year flood levels to evaluate the vulnerability to inundation. The 100-year flood is used as a standard for planning, insurance, and environmental regulations. It is important to note that people, infrastructure, and property are already located in areas vulnerable to flooding from a 100-year event. Sea-level rise will cause more frequent and more damaging floods to those already at risk and will increase the size of the coastal floodplain, placing new areas at risk where there were none before. In Figure 15, for example, those areas shown in light blue are currently vulnerable to a 100-year flood event in the Santa Cruz area. With a 1.4 m sea-level rise, additional areas (shown in dark blue) will be at risk. Thus, the damage attributed to a 1.4 m sea-level rise is equal to the area currently vulnerable to a 100-year flood event (but now protected by levees, seawalls, etc.) plus new inundated areas, i.e., the areas shown in light blue and dark blue in Figure 15.

A series of maps for the entire coast of California demonstrating the extent of the areas at risk are posted at www.pacinst.org/reports/sea_level_rise. It should be noted again that these maps are not the result of detailed site studies, and were created to quantify risk over a large geographic area. **These maps should not be used to assess actual coastal hazards, insurance requirements or property values, and specifically shall not be used in lieu of Flood Insurance Studies and Flood Insurance Rate Maps issued by the Federal Emergency Management Agency (FEMA).** Local governments or regional planning agencies should conduct detailed studies to better understand the potential impacts of sea-level rise in their communities.

Figure 15. Estimated current and future 100-year coastal flood risk areas around Santa Cruz

3.1.1. Population at Risk

Major population centers are located all along California's coast. Nearly 26 million people lived in coastal counties in 2000. Of these, 74% lived along the Pacific coast and the remaining 26% lived along the San Francisco Bay. An estimated 260,000 people, or 1% of the population in

California's coastal counties, live in areas that are currently vulnerable to a 100-year flood event. As discussed in Section 2.3.3, the inundated area does not adequately take into account existing flood barriers. It is likely that most existing coastal protection structures are sufficient to protect people living in these areas against the present-day flood risk. Most existing defenses, however, will not be adequate to protect inhabitants following significant sea-level rise.

As sea levels rise, the area and the number of people vulnerable to flooding will also rise. Rising sea levels will overwhelm the existing protection structures, putting the 260,000 people currently living in vulnerable areas at increased risk. In total, we estimate that a 1.4 m sea-level rise will put around 480,000 people (nearly half a million) at risk from a 100-year flood event (Figure 16). Continued development in these regions could put additional people at risk.

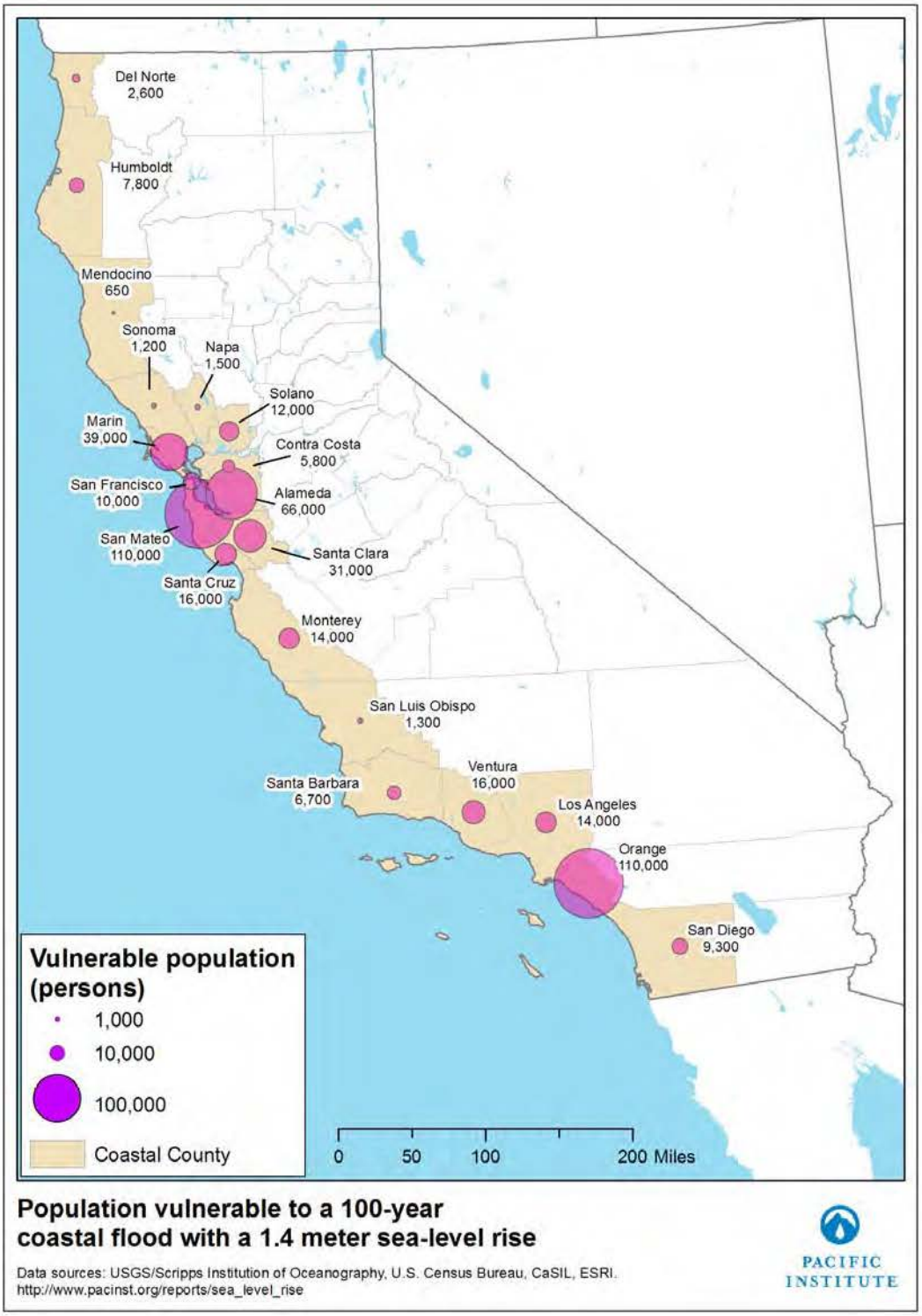

Figure 16. Population vulnerable to a 100-year coastal flood with a 1.4 m sea-level rise, by county

Table 8 shows the population vulnerable to a 100-year flood event along the Pacific coast by county. In 2000, an estimated 130,000 people lived in areas vulnerable to a 100-year flood event. A 1.4 m sea-level rise will increase the number of people vulnerable to a 100-year flood event to 210,000. Half of these residents live in Orange County, although significant numbers of people are also at risk in Los Angeles, Monterey, San Mateo, Santa Cruz, and Ventura Counties.

Table 8. Population vulnerable to a 100-year flood along the Pacific coast, by county

County	Current Risk	Risk with 1.4 m sea-level rise	Percent increase
Del Norte	1,800	2,600	47%
Humboldt	3,700	7,800	110%
Los Angeles	3,700	14,000	270%
Marin	530	630	20%
Mendocino	530	650	22%
Monterey	11,000	14,000	36%
Orange	72,000	110,000	55%
San Diego	3,000	9,300	210%
San Francisco	4,800	6,500	35%
San Luis Obispo	670	1,300	98%
San Mateo	4,700	5,900	24%
Santa Barbara	3,400	6,700	94%
Santa Cruz	11,000	16,000	49%
Sonoma	580	700	21%
Ventura	7,300	16,000	120%
Total	**130,000**	**210,000**	**67%**

Note: Counties with borders on the Pacific coast and the San Francisco Bay (e.g., San Mateo) were separated based on the shoreline affected. Numbers may not add up due to rounding.

In San Francisco Bay, the population vulnerable to flooding is even greater. Table 9 shows the population vulnerable to a 100-year flood event in 2000 and with a 0.5 m, 1.0 m, and 1.4 m sea-level rise. In 2000, an estimated 140,000 people lived in areas at risk from a 100-year flood event. An increase in sea levels of 0.5 m has only a modest effect on the number of people at risk. With a 1.4 m increase in sea levels, however, the number of people at risk of a 100-year flood event nearly doubles to 270,000. Populations in San Mateo County are especially vulnerable, accounting for about 40% of those at risk with a 1.4 m sea-level rise. Large numbers of residents in Alameda, Marin, and Santa Clara counties are also at risk.

Table 9. Population vulnerable to a 100-year flood along the San Francisco Bay, by county

County	Current risk	Risk with sea-level rise			Percent increase (with 1.4 m rise)
		0.5 m	1.0 m	1.4 m	
Alameda	12,000	22,000	43,000	66,000	+ 470%
Contra Costa	840	1,600	3,400	5,800	+ 590%
Marin	25,000	29,000	34,000	39,000	+ 55%
Napa	760	830	970	1,500	+ 99%
San Francisco	190	600	1,600	3,800	+ 1900%
San Mateo	80,000	88,000	99,000	110,000	+ 34%
Santa Clara	13,000	17,000	24,000	31,000	+ 140%
Solano	3,700	5,500	8,800	12,000	+ 230%
Sonoma	250	300	420	540	+ 110%
Total	**140,000**	**160,000**	**220,000**	**270,000**	**+ 98%**

Note: Counties with borders on the Pacific coast and the San Francisco Bay (e.g., San Mateo) were separated based on the shoreline affected. Numbers may not add up due to rounding.

Workplaces as well as residences will be vulnerable. We estimate that statewide, the number of employees in the hazard area will increase from 190,000 at present to 410,000. In this group, 80% are employed in commercial settings, and 20% in the industrial sector. Of those affected, 80% are in areas vulnerable to flooding from San Francisco Bay, with the other 20% along the Pacific coast.

Environmental Justice Concerns

The analysis of the potential environmental justice impacts of sea-level rise considers the population within the areas at risk and their vulnerability to the potential adverse impacts. There is little difference between the overall racial and income demographics of Californians affected by a 1.4 m sea-level rise and those of the state as a whole. However, we do find some important differences between the racial and income demographics of those affected and those of the total population of each county.

Table 10 and Figure 17 show a simplified racial breakdown of the flood-affected population and the population of the counties as a whole. Sea-level rise induced flooding may disproportionately affect whites in the majority of counties along the California coast. In Los Angeles County, for example, 72% of those affected are white, while only 31% of the population in the county is white. Conversely, along the San Francisco Bay, however, communities of color are disproportionately impacted by sea-level rise. In total, communities of color are disproportionately impacted in 10 of the 20 counties studied. The greater proportion of people of color in areas affected by a 1.4-meter sea-level rise highlights the need for these counties to take concerted efforts to understand and mitigate potential environmental injustice.

The results presented above highlight the importance of conducting socio-economic analyses and comparisons at various geographic scales. It is significant to note that these numbers only reflect exposure to the hazard. In the next section, we also evaluate other vulnerability factors, such as access to transportation and ability to speak English.

Table 10. Total county population and population vulnerable to a 100-year flood with a 1.4-meter sea-level rise along the Pacific coast, by race

County	White		Asian, Black, Latino, Native American, or Other Race	
	Affected population (%)	County population (%)	Affected population (%)	County population (%)
Alameda	35	41	60	55
Contra Costa	28	58	69	39
Del Norte	75	70	21	26
Humboldt	82	82	15	15
Los Angeles	72	31	26	67
Marin	59	79	38	19
Mendocino	74	75	23	22
Monterey	29	40	69	57
Napa	63	69	35	29
Orange	80	51	18	46
San Diego	73	55	25	42
San Francisco	51	44	46	53
San Luis Obispo	85	76	13	22
San Mateo	46	50	51	47
Santa Barbara	68	57	30	41
Santa Clara	49	44	47	53
Santa Cruz	43	66	54	32
Solano	38	49	58	46
Sonoma	70	75	28	23
Ventura	56	57	41	41
All coastal counties	56	44	41	53

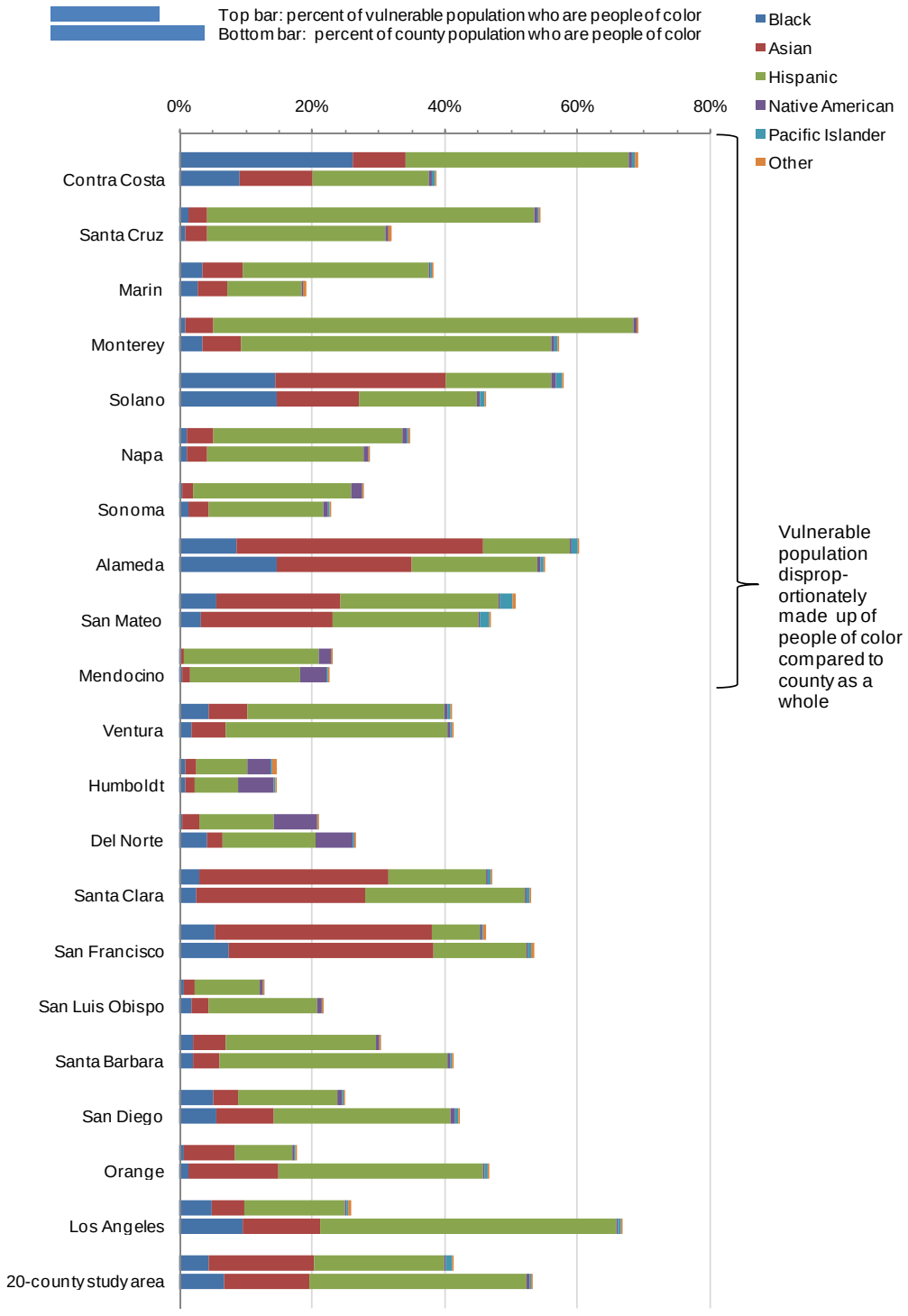

Figure 17. Total county population and population vulnerable to a 100-year flood with a 1.4 meter sea-level rise along the Pacific coast, by race

Note: The lower bar shows the percentage of the county's population that is classified as people of color, and the top bar shows the percentage of the population at risk of a 100-year flood with a 1.4 m sea-level rise that is classified as people of color. A county for which the top bar is longer indicates that there is a disproportionate impact on communities of color.

Preconditions

The period preceding a disaster is the key phase for taking action to reduce vulnerabilities and proactively prevent harm. For example, reinforcing residential buildings, obtaining insurance, and storing emergency supplies can reduce injury and loss. Studies show that those who are the most vulnerable are the least likely to adopt these preventive measures. Below, we evaluate key demographic factors affecting vulnerability during the pre-disaster phase, including residential tenure (renter or homeowner), income, and linguistic isolation.

Preventive measures such as reinforcing buildings and buying insurance are adopted at lower rates by people with low income levels (Bolin and Bolton 1986; Blanchard-Boehm 1997). In California, 31% of households earn less than 150% of the federal poverty threshold ($30,000). Low-income households make up 29% of the 20-county study area, slightly less than the statewide total (Figure 18).

An estimated 56,000 people along the Pacific coast, or about 27% of those vulnerable to a 100-year flood with a 1.4 m sea-level rise, earn less than $30,000. Likewise, an estimated 51,000 people along the San Francisco Bay, or about 19% of the affected population, earn less than $30,000 (Table 11). Income demographics vary markedly among the vulnerable populations and counties in this study (Figure 18). Our analysis indicates that there is a disproportionate impact on low-income households in 13 of the 20 coastal counties. These households are less likely than their counterparts to be able to afford emergency preparedness materials, buy insurance policies, and obtain needed building reinforcements.

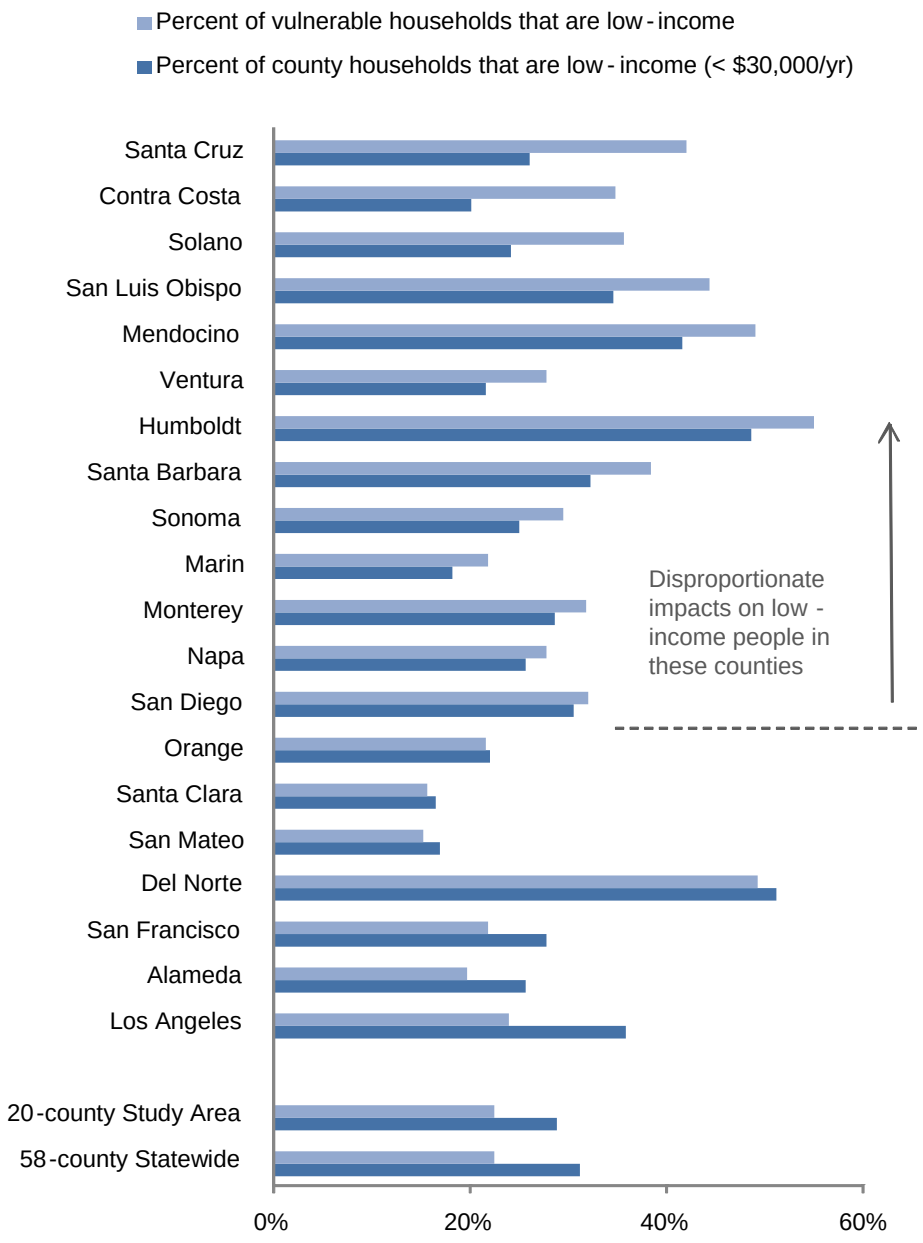

Figure 18. Percentages of low-income households among the population vulnerable to a 100-year flood with a 1.4 m sea-level rise compared with the county total

Note: The lower bar shows the percentage of low-income households in the county, and the top bar shows the percentage of low-income households within the population at risk of a 100-year flood with a 1.4 m sea-level rise. A county for which the top bar is longer indicates that there is a disproportionate impact on low-income households.

Table 11. Key demographics of populations vulnerable to a 100-year flood event with a 1.4 m sea-level rise

	Pacific Coast		San Francisco Bay	
	Number in 100-year flood zone	Percent of total in flood zone	Number in 100-year flood zone	Percent of total in flood zone
Households				
Linguistically isolated	4,700	4%	9,700	9%
With no vehicle	7,600	7%	8,200	7%
Who rent (not own) their home	45,000	43%	47,000	41%
People				
Earn less than 150% the federal poverty threshold ($30,000)	56,000	27%	51,000	19%
People of color	60,000	29%	148,000	55%

Data source: U.S. Census Bureau 2000

Renters are also less likely to reinforce buildings and buy insurance because the decision to make major improvements and financial gains typically lies with the property owner. Of those vulnerable to a 100-year flood event with a 1.4 m sea-level rise, about 45,000 households along the Pacific coast and 47,000 households along the San Francisco Bay rent their homes. These households comprise 43% and 41%, respectively, of the homes within the areas at risk.

Language ability is also an important factor in assessing vulnerability (Wang and Yasui 2008). Earthquake preparedness materials following the 1987 Whittier-Narrows earthquake in California, for example, were available only in English, despite other language needs of the victims (Tierney 1993, cited in Pastor et al. 2006). Additionally, emergency response crews may be unable to communicate with non-English speakers. A recent study of 148 emergency preparedness and public health entities found that only 72% provided links on their website to translated materials, and only 14% offered courses for service providers that addressed potential language issues and cultural competence (Andrulis et al. 2008). Among the population at risk from a 100-year flood event with a 1.4 m sea-level rise, 9,700 households along the San Francisco Bay and 4,700 households along the Pacific coast are "linguistically isolated," meaning no one over age 14 speaks English well (Table 11). These 14,000 households are the most likely to need preparedness materials and outreach strategies suitable for non-English speakers of various backgrounds.

Even among those for whom language is not a barrier, cultural factors can influence the effectiveness of preparedness outreach. Numerous studies show that black and Latino communities prefer neighborhood meetings as a way of receiving information about hazards (Blanchard-Boehm 1997; Perry and Mushkatel 1986; Phillips and Ephraim 1992, cited in Pastor et al. 2006). The historic role of African-American churches in providing disaster planning and

response provides a unique asset and partner to public efforts in these communities (Trader-Leigh 2008).

The representation of low-income and people of color in the groups with heightened vulnerabilities during the pre-disaster phase are higher than these communities' representation in the overall population. In 2000, 65% of white Californian heads of households were homeowners, while 55% of Asian, 46% of Native American, 44% of Latino, and 39% of black heads of household owned their home (U.S. Census Bureau 2000). Eighty-one percent (81%) of Californians who cannot speak English "well" or "well at all" are people of color, while people of color are 31% of the California population (U.S. Census Bureau 2000). Additionally, people of color tend to earn less than white wage earners. The median household income of black, Latino, and Native households in California was $15,000 less than white and Asian households (U.S. Census Bureau 2000). These factors raise vulnerability to a disaster and increase the likelihood that communities of color and low-income Californians will share a disproportionate burden of harm.

During a disaster

The ability to remain safe and/or evacuate high-risk areas during a flood event is shaped by factors such as quality of residential structures, access to transportation, availability of emergency supplies, effective service by emergency responders, and exposure to environmental hazards. Key demographics associated with these vulnerabilities are income, possession of a vehicle, race, and proximity to environmental hazards that compound health risk, such as toxic waste facilities.

Low-income communities have been unable to evacuate during disasters like Hurricane Andrew due to lack of financial means to buy supplies or transportation (Morrow and Enarson 1996). In a survey after Hurricane Katrina, 55% of respondents who did not evacuate said one of the main reasons was that they did not have a car or other means of transportation (Brodie et al. 2006). Our study shows that nearly 16,000 households in areas vulnerable to a 100-year flood event with a 1.4 m sea-level rise do not have a vehicle (Table 11). Half of these households are located along the San Francisco Bay and the remaining half along the Pacific coast. These households will be more vulnerable to the adverse effects of sea-level rise due to their increased chance of lacking the transportation means necessary to evacuate.

Race has been an important factor influencing the effectiveness of past emergency response efforts. Perceptions of emergency response workers toward neighborhoods that are predominantly people of color can increase the vulnerability of these communities. In a recent report, the International Federation of Red Cross and Red Crescent Societies (IFRCC) found that "stereotypical views of a specific group can overwhelm the scientific methods employed to prioritize the order of relief works, even if some of those involved are professionally trained, such as disaster managers and relief workers" (Klynman 2007). Along the Pacific coast, we estimate that 60,000 Asian, black, and Latino residents live in areas vulnerable to a 100-year flood event with a 1.4 m sea-level rise. The numbers are even higher along the San Francisco Bay, where an estimated 150,000 Asian, black, and Latino residents live in vulnerable areas. The

areas with the highest concentrations of people of color are more likely to be subject to problems with stereotypes that may result in less effective emergency services.

Section 3.1.3, below, describes the number of U.S. EPA-regulated facilities that are at risk of flooding. These facilities contain a range of toxic chemicals that result in increased risk during a flood event due to the possibility that environmental hazards could be released and nearby residents exposed. In California as a whole, the population living within 3 kilometers (1.8 miles) of a commercial hazardous waste facility is disproportionately (81%) people of color compared to communities without such facilities (51% people of color) (Bullard et al. 2007). The same national study concluded that "race continues to be an independent predictor of where hazardous wastes are located, and it is a stronger predictor than income, education, and other socioeconomic indicators" (Bullard et al. 2007). The combination of higher concentrations of environmental hazards and higher rates of demographic characteristics that increase vulnerability has been termed "double jeopardy" by the Institute of Medicine (1999).

This disproportionate representation of people of color living near hazardous waste facilities is coupled with an overrepresentation among households with no vehicle. While black and Latino households comprised 7% and 22% of California's households in 2000, respectively, they comprised 13% and 32% of the households with no vehicle (U.S. Census Bureau 2000), and, as noted above, people of color are also over-represented among low-income Californians. Their higher rates of characteristics associated with vulnerabilities during the time of a disaster raise the possibility that communities of color and low-income people will be disproportionately affected.

Recovery and reconstruction

Following a flood event or other disaster, a range of conditions determines the victims' ability to recover and reconstruct their homes and lives. Important vulnerability factors include the ability to move where opportunities arise, obtain insurance compensation for losses, and receive medical care and public services. The demographic characteristics of income, insurance coverage, legal residency status, and race affect the vulnerability of individuals living in potential flood areas.

White and upper middle-class groups have been found to receive more disaster recovery assistance than black and low-income groups (Bolin and Bolton 1986; Fothergill 2004). For example, following the 1995 flooding of New Orleans, low-income elderly women were one-third as likely as other elderly victims to receive FEMA low-interest loans (Childers 1999). Disaster recovery services have often targeted homeowners to the disadvantage of renters and residents of public housing (Pastor et al. 2006). Reconstruction efforts of the past have inadequately rebuilt housing suitable for low-income families. Four years after the Loma Prieta earthquake, half of the affected multifamily units remained uninhabitable (Comerio et al. 1994). Government agencies explicitly denied housing assistance to those who were homeless before the earthquake (Tierney 2007).

The loss of wealth to homeowners resulting from a disaster is greater for those whose home equity comprises a greater proportion of their wealth. This effect is particularly problematic for

black homeowners, whose home equity accounts for 20% more of their wealth than white homeowners (Oliver and Shapiro 1995; Gittleman and Wolff 2000).

Legal residency status influences recovery efforts as well. Undocumented residents fear that participating in recovery assistance programs will put them at risk of deportation (Subervi-Velez et al. 1992; Yelvington 1997). Data on the number of undocumented immigrants are elusive, but the Public Policy Institute of California (2008) estimates that 8% of Californians are undocumented. The number and distribution of undocumented immigrants in areas vulnerable to current and future flood events deserves further study.

Recovery for disaster victims suffering adverse health effects is dependent upon their access to health insurance. The uninsured get about half as much medical care as the insured, are less likely to receive preventive screening and care, and overall have worse health outcomes (Bovbjerg and Hadley 2007). Race is a predictor of rates of health insurance coverage in California: 34% of California Latinos did not have health insurance in 2005, while 22% of Native Californians, 18% of Asians, 15% of black Californians, and 13% of whites were not insured, according to the California Health Interview Survey (Brown et al. 2007).

The correlation of lower income and race, and the over-representation of communities of color among those without legal residency and without health insurance, increases these communities' vulnerability to the harms of sea-level rise even in the period following a disaster. The history of disparate treatment of people of color in recovery assistance services suggests another level of increased vulnerability.

Summary of Environmental Justice Concerns

The adverse impacts of sea-level rise on Californians will depend upon the population's vulnerabilities, which are heightened for certain demographic groups. Race and income cut across many of the key vulnerabilities, with low-income and communities of color overly represented in the most vulnerable segments of the population. Additionally, adapting to sea-level rise will require tremendous financial investment. Given the high cost and the likelihood that we will not protect everything, adaptation raises additional environmental justice concerns. Specifically, what we choose to protect and how we pay for it may have a disproportionate impact on low-income neighborhoods and communities of color. Decisions about how to use public funds can lead to inequitable distribution of costs and benefits, whether they are based on economics (protect the most valuable assets) or utility (protect the largest number of people). We urge, therefore, that policy makers planning responses to sea-level rise understand and address environmental justice concerns carefully and proactively.

3.1.2. Emergency and Healthcare Facilities at Risk

Table 12 shows the schools and emergency and healthcare facilities along the Pacific coast that are currently at risk from a 100-year flood event and that will be at risk with a 1.4 m sea-level rise. Numerous schools are vulnerable to flooding along the Pacific coast. In 2000, 30 schools were vulnerable to a 100-year flood event. With a 1.4 m sea-level rise, however, the number of schools at risk nearly doubles, rising to 56 schools. Emergency and healthcare facilities are also at risk.

Table 12. Schools and emergency and healthcare facilities along the Pacific coast that are at risk from a 100-year flood event in 2000 and with a 1.4 m sea-level rise

Facility	Current risk	Risk with 1.4 m sea-level rise
Schools	30	56
Healthcare facilities	5	13
Fire stations and training facilities	2	6
Police stations	4	8

Note: Healthcare facilities include clinics, long-term care facilities, hospitals, and home health agencies/hospices. Counties with borders on the Pacific coast and the San Francisco Bay (e.g., San Mateo) were separated based on the shoreline affected.

Table 13 shows the schools and emergency and healthcare facilities along San Francisco Bay that are currently at risk of a 100-year flood event and that will be at risk with a 0.5 m, 1.0 m, and 1.4 m sea-level rise. The risk for each of these facilities is greater than along the remainder of the Pacific coast. Schools in particular are at significant risk. In 2000, 35 schools were at risk of a 100-year flood event. With a 1.4 m sea-level rise, the number of schools at risk more than doubles, to 81. Significant numbers of healthcare facilities are also at risk. In 2000, there were 15 healthcare facilities at risk of a 100-year flood. With a 1.4 m sea-level rise, however, the number of healthcare facilities at risk rises to 42.

Table 13. Schools and emergency and healthcare facilities along San Francisco Bay that are at risk of a 100-year flood event in 2000 and with a 0.5 m, 1.0 m, and 1.4 m sea-level rise.

Facility	Current risk	Risk with sea-level rise		
		0.5 m	1.0 m	1.4 m
Schools	35	41	60	81
Healthcare facilities	15	19	29	42
Fire stations and training facilities	6	7	10	11
Police stations	5	6	8	9

Note: Healthcare facilities include clinics, long-term care facilities, hospitals, and home health agencies/hospices. Counties with borders on the Pacific coast and the San Francisco Bay (e.g., San Mateo) were separated based on the shoreline affected.

3.1.3. Hazardous Materials Sites

The presence of land or facilities containing hazardous materials in areas at risk of inundation increases the risk of exposure to toxic chemicals for nearby residents and ecosystems. For example, sediment samples in New Orleans taken one month after Hurricane Katrina found

excess levels of arsenic, lead, and the gasoline constituent benzene, all considered toxic pollutants by the U.S. EPA (Adams et al. 2007). Those living or working near these facilities may be affected by the potential release and spreading of contamination through floodwaters or through flood-related facility malfunctions.

We evaluated sites containing hazardous materials at risk of flooding along the Pacific coast and the San Francisco Bay. Here, we report on a range of sites monitored by the U.S. EPA, including Superfund sites; hazardous waste generators; facilities required to report emissions for the Toxics Release Inventory; facilities regulated under the National Pollutant Discharge Elimination System (NPDES); major dischargers of air pollutants with Title V permits; and brownfield properties. An estimated 130 U.S. EPA-regulated sites are currently vulnerable to a 100-year flood event (Table 14). Nearly 60% of these facilities are located in San Mateo and Santa Clara counties. Sea-level rise will put additional facilities, people, and the environment at risk. The number of facilities at risk increases by 250% with a 1.4 m sea-level rise, with more than 330 facilities at risk of a 100-year flood event. San Mateo, Alameda, and Santa Clara counties have the highest numbers of U.S. EPA-regulated sites within future flood areas.

Table 14. U.S. EPA-regulated sites within areas vulnerable to 100-year flood event in 2000 and with a 1.4 m sea-level rise

County	Sites currently at risk	Risk with 1.4 m sea-level rise
Alameda	6	63
Contra Costa	4	22
Del Norte	1	3
Humboldt	10	13
Los Angeles	13	26
Marin	1	6
Monterey	1	1
Napa	1	2
Orange	4	16
San Diego	-	13
San Francisco	-	4
San Luis Obispo	-	1
San Mateo	39	78
Santa Barbara	1	5
Santa Clara	41	53
Santa Cruz	5	6
Solano	2	5
Sonoma	-	2
Ventura	5	13
Total	**134**	**332**

Data Source: EPA Geospatial Data Access Project 2008
Note: Table combines risk for those counties along the San Francisco Bay and Pacific coast.

3.1.4. Infrastructure at Risk

Roads and Railways

California's transportation infrastructure is vulnerable to flooding under current conditions, and those risks will be greater in the future due to sea-level rise (Tables 15, 16, and 17 and Figures 19 and 20). Under current conditions, we estimate that 1,900 miles of roadway are at risk of a 100-year flood event. With a 1.4 m sea-level rise, 3,500 miles of roads will be at risk of flooding, nearly a doubling of current risk. Of the total, about 430 miles are highways (12% of the total mileage), while the remainder are neighborhood and local streets. About half of the roads at risk are around San Francisco Bay, and another half on the Pacific Coast.

Railways are also at risk. In 2000, 140 miles of railways were at risk of flooding. With a 1.4 m sea-level rise, the length of railways at risk doubles to 280 miles. About 60% of the vulnerable railway lines are in the San Francisco Bay area.

Table 15. Miles of roads and railways vulnerable to a 100-year flood in 2000 and with a 1.4 m sea-level rise along the Pacific coast, by county and type

County	Highways (miles)		Roads (miles)		Railways (miles)	
	Current risk	Risk with 1.4 m sea-level rise	Current risk	Risk with 1.4 m sea-level rise	Current Risk	Risk with 1.4 m sea-level rise
Del Norte	6.6	8.2	59	80	-	-
Humboldt	37	58	120	190	21	28
Los Angeles	14	31	42	140	5.6	14
Marin	1.2	4.1	22	27	-	-
Mendocino	5.6	7.9	28	41	2.7	4.0
Monterey	27	31	85	110	19	23
Orange	32	48	340	490	5.3	6.6
San Diego	0.62	8.0	12	57	3.0	9.8
San Francisco	0.20	0.37	17	22	-	-
San Luis Obispo	5.3	7.4	10	21	0.019	0.31
San Mateo	3.4	5.0	23	30	-	-
Santa Barbara	1.5	8.0	9.1	25	3.4	7.0
Santa Cruz	9.4	11	52	67	4.2	5.5
Sonoma	4.5	5.9	14	20	-	-
Ventura	2.4	11	69	150	3.7	10
Total	150	250	910	1,500	68	110

Note: Counties with borders on the Pacific coast and San Francisco Bay (e.g., San Mateo) were separated based on the shoreline affected. Numbers may not add up due to rounding.

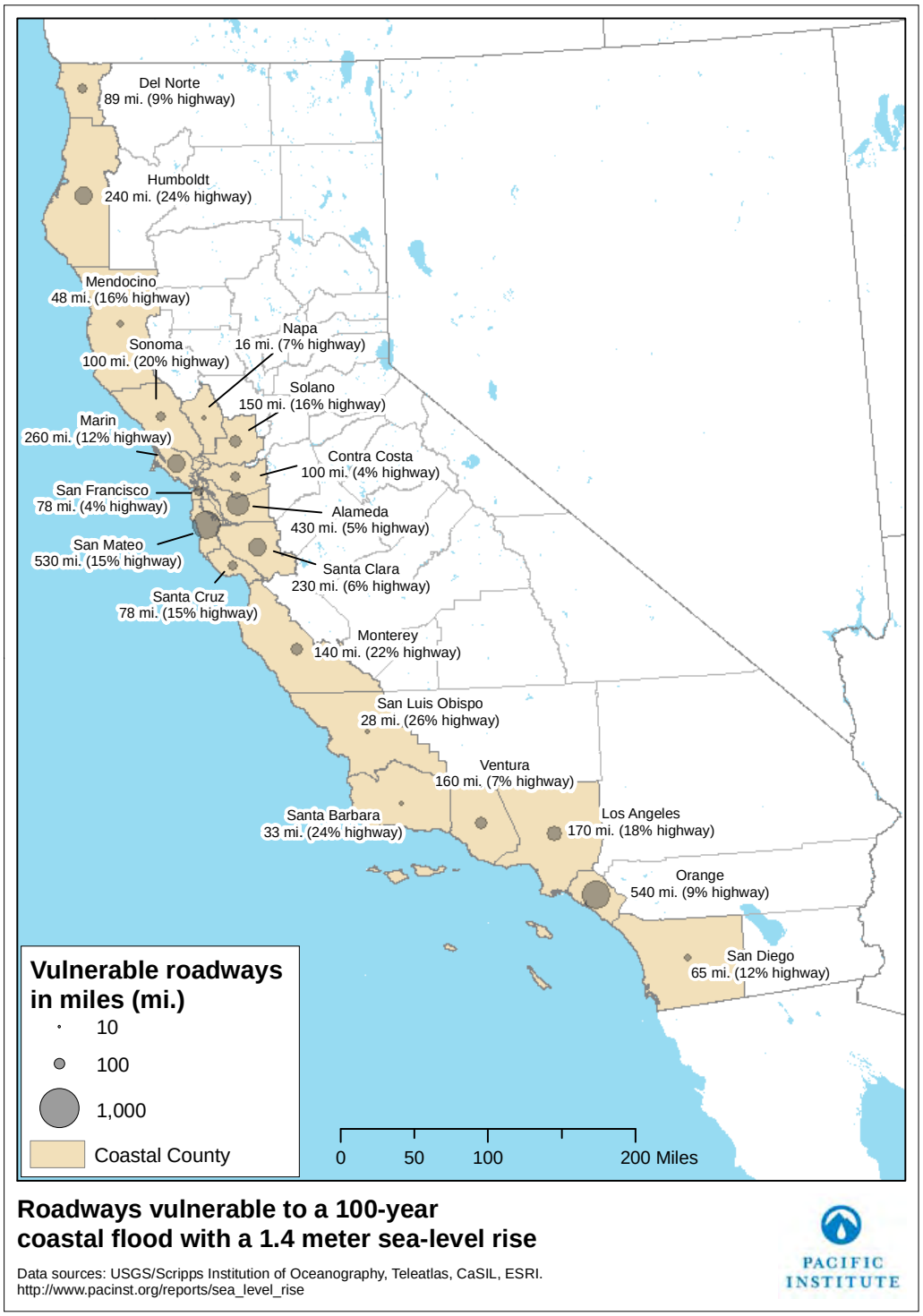

Figure 19. Roadways vulnerable to a 100-year coastal flood with a 1.4 m sea-level rise

Figure 20. Railroads vulnerable to a 100-year coastal flood with a 1.4 m sea-level rise

Table 16. Miles of roads vulnerable to a 100-year flood along San Francisco Bay, by county and type

County	Current Risk		Risk with sea-level rise					
			0.5 m		1.0 m		1.4 m	
	Highways (miles)	Roads (miles)	Highways (miles)	Roads (miles)	Highways (miles)	Roads (miles)	Highways (miles)	Roads (miles)
Alameda	1.1	76	4.8	160	14	280	23	410
Contra Costa	2.4	20	2.7	42	3.4	67	4.5	96
Marin	16	110	20	150	24	180	28	200
Napa	0.70	7.0	0.70	9.0	0.80	11	1.2	15
San Francisco	0.30	3.4	0.60	11	1.5	29	3.1	53
San Mateo	27	300	49	360	66	390	72	420
Santa Clara	9.4	110	12	150	14	180	15	220
Solano	5.7	53	14	78	19	100	23	120
Sonoma	11	53	12	57	13	59	14	61
Total	**72**	**730**	**120**	**1,000**	**160**	**1,300**	**180**	**1,600**

Note: Counties with borders on the Pacific coast and San Francisco Bay (e.g., San Mateo) were separated based on the shoreline affected. Numbers may not add up due to rounding.

Table 17. Miles of railways vulnerable to a 100-year flood along San Francisco Bay, by county

County	Current risk	Risk with sea-level rise			Percent increase (with 1.4 m rise)
		0.5 m	1.0 m	1.4 m	
Alameda	9.1	17	35	49	+ 81%
Contra Costa	10	17	25	37	+ 73%
Marin	12	15	16	17	+ 29%
Napa	6.0	7.0	7.9	8.2	+ 27%
San Francisco	0.26	0.56	0.91	1.6	+ 84%
San Mateo	3.7	5.2	7.8	10	+ 65%
Santa Clara	5.9	7.2	8.9	10	+ 43%
Solano	9.3	12	17	21	+ 56%
Sonoma	11	14	17	18	+ 39%
Total	**68**	**94**	**140**	**170**	**+ 61%**

Note: Counties with borders on the Pacific coast and San Francisco Bay (e.g., San Mateo) were separated based on the shoreline affected. Numbers may not add up due to rounding.

We do not attempt to quantify the cost of flooding on roads and railways. In some cases, damages may be minor, resulting in temporary closures and modest repairs. As the frequency and intensity of flooding increases, however, closures may become longer and the cost of repair may rise. Eventually, roads and railways may need to be raised or rerouted. The cost of repairing, moving, or raising roads and railways is highly site-specific and dependent on the level of damage that is sustained.

Furthermore, flooding and closure of roads and railways can have significant impacts on the local, state, and national economy. Railways are particularly important for the conveyance of goods shipped to and from California ports. In addition, road closures can prevent people from getting to work, causing major economic disruptions. Additional research is needed to improve our understanding of specific transportation risks.

Power Plants

Figures 21, 22, and 23 show California's coastal power plants vulnerable to a 100-year flood event with a 1.4 m sea-level rise. In some cases, actual power generating infrastructure is at risk; in others, intake or other peripheral structures are vulnerable. Specific site assessments are needed for each coastal plant. In total, 30 coastal power plants, with a combined capacity of more than 10,000 megawatts (MW), are at risk from a 100-year flood with a 1.4 m sea-level rise. The capacities of the vulnerable power plants range from a relatively small 0.2 MW plant to one that is more than 2,000 MW. The majority of vulnerable plants are located in Southern California and along the San Francisco Bay.

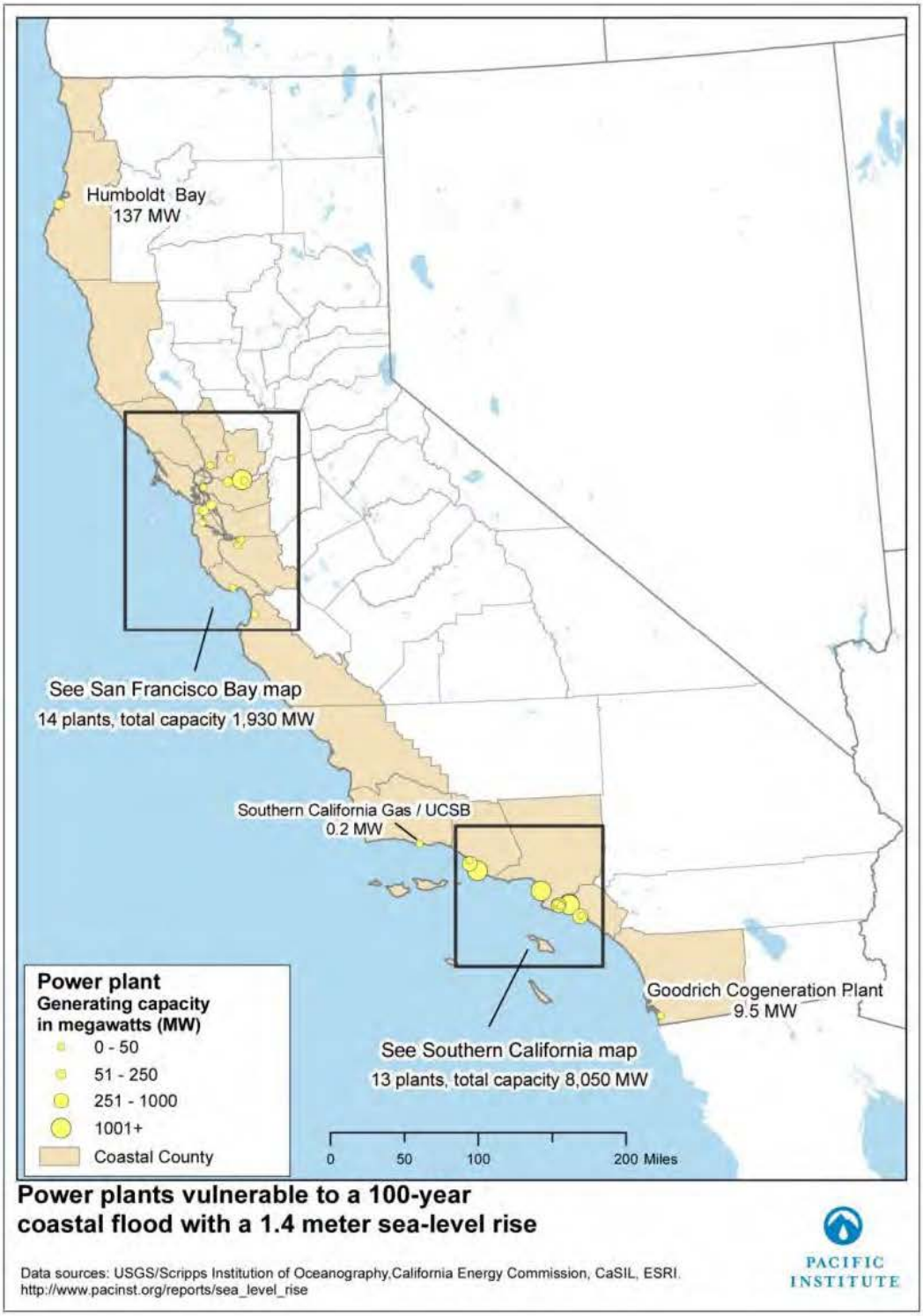

Figure 21. Power plants vulnerable to a 100-year coastal flood with a 1.4 m sea-level rise

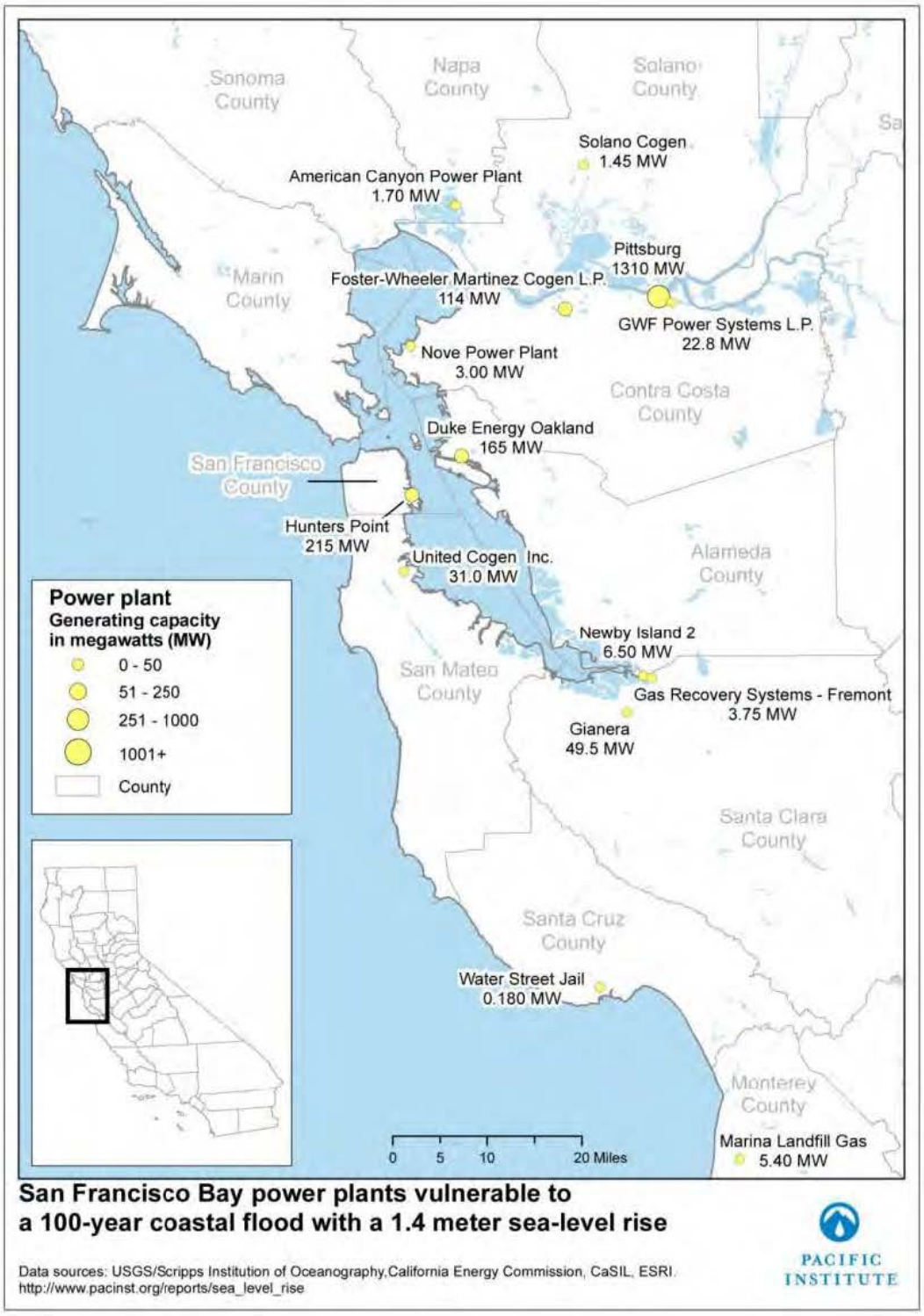

Figure 22. San Francisco Bay power plants vulnerable to a 100-year coastal flood with a 1.4 m sea-level rise

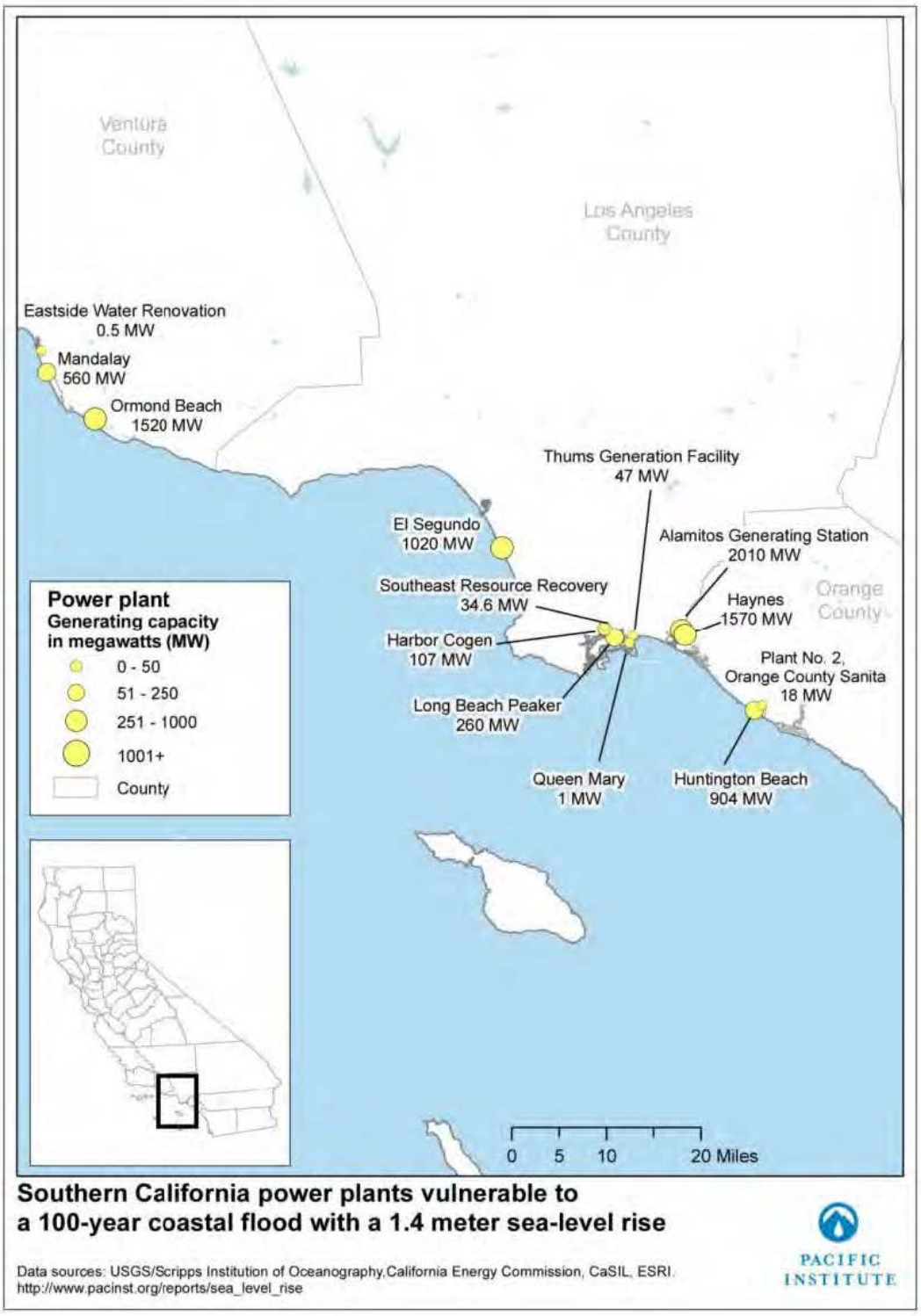

Figure 23. Southern California power plants vulnerable to a 100-year coastal flood with a 1.4 m sea-level rise

Wastewater Treatment Plants

Figures 24 and 25 show the wastewater treatment plants vulnerable to a 100-year flood event with a 1.4 m sea-level rise. We identified a total of 28 vulnerable wastewater treatment plants: 21 on the San Francisco Bay and 7 on the Pacific coast. The combined capacity of these plants is 530 million gallons per day (MGD). Inundation from floods could damage pumps and other equipment, and lead to untreated sewage discharges. Besides the flood risk to plants, higher water levels could interfere with discharge from outfalls sited on the coast. Cities and sanitation districts should begin to assess how higher water levels will affect plant operations and plan for future conditions.

Ports

Goods movement in California, and especially the San Francisco Bay Area, is critically important to the state's economy. A recent report by the Metropolitan Transportation Commission (MTC) stated that "over 37 percent of Bay Area economic output is in manufacturing, freight transportation, and warehouse and distribution businesses. Collectively, these goods-movement-dependent businesses spend approximately $6.6 billion on transportation services. The businesses providing these services also play a critical role as generators of jobs and economic activity in their own right" (MTC 2004).

Our assessment of future flood risk with sea-level rise shows significant flooding is possible at California's major ports in Oakland, Los Angeles, and Long Beach. These ports are central to the economy of California, the nation, and the world. The Port of Los Angeles-Long Beach, for example, handles 45%–50% of the containers shipped into the United States. Of these containers, 77% leave the state—half by train and half by truck (Christensen 2008).

Many port managers have already experienced how disasters can affect their operations. Following the Loma Prieta earthquake in 1989, for example, the Port of Oakland sustained damages that interrupted business for 18 months. These disruptions have economic implications for the nation and the world, as evident by a 2002 contract dispute that resulted in a work slowdown at West Coast ports and cost the U.S. economy an estimated $1 billion to $2 billion per day. Others speculated that Japan and China would lose several percentage points off their gross domestic product if the ports closed for longer than a week (Farris 2008).

In addition to directly affecting port operations, sea-level rise may cause other interruptions to goods movement at ports. Sea-level rise can reduce bridge clearance, thereby reducing the size of ships able to pass or restricting their movements to times of low tide. Higher seas may cause ships to sit higher in the water, possibly resulting in less efficient port operations (National Research Council 1987). These impacts are highly site specific, and somewhat speculative, requiring detailed local study. We also note the connection between possible direct impacts of sea-level rise on the ports themselves and possible flooding of transportation (rail and road) corridors to and from the ports.

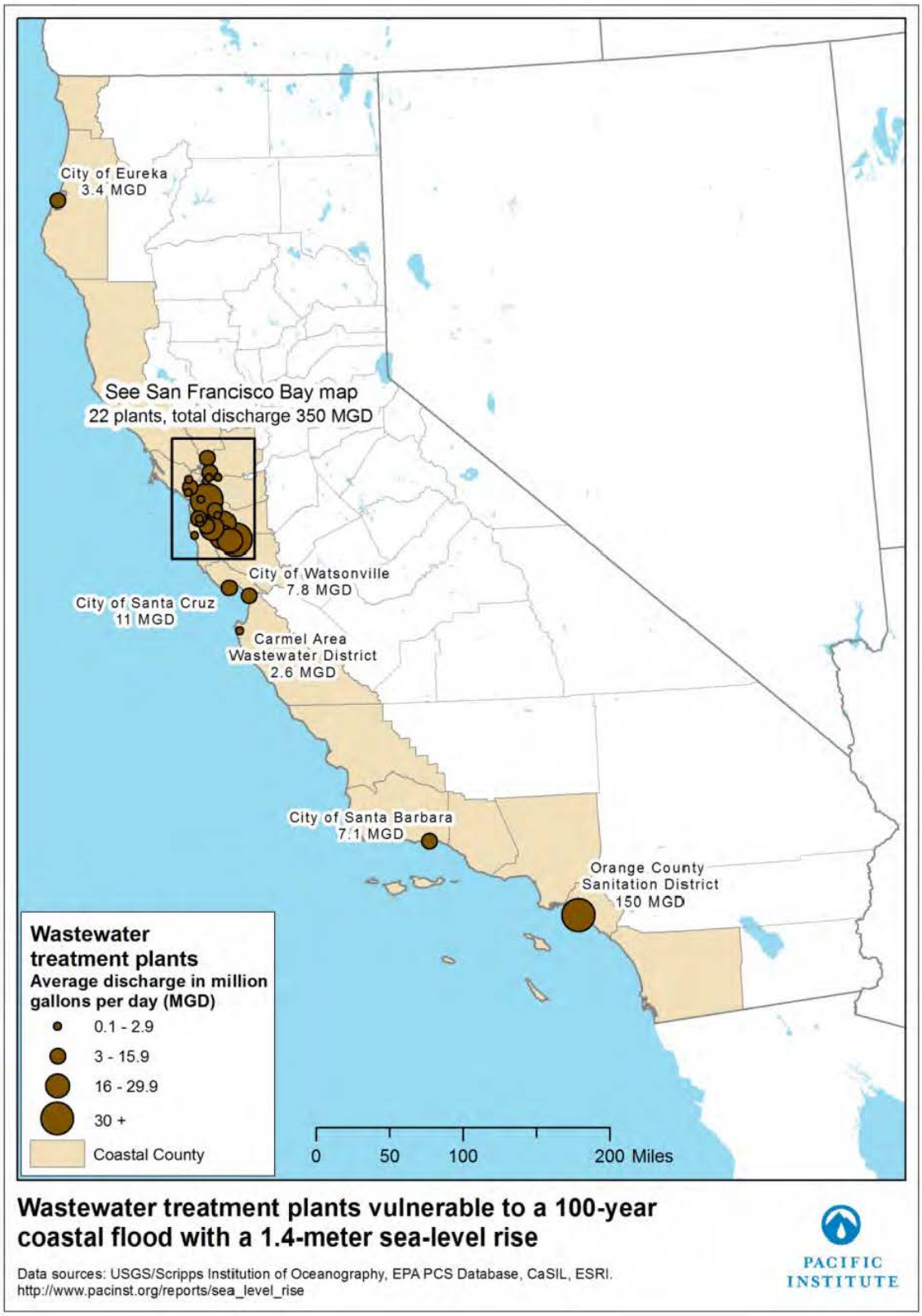

Figure 24. Wastewater treatment plants on the Pacific coast vulnerable to a 100-year flood with a 1.4 m sea-level rise

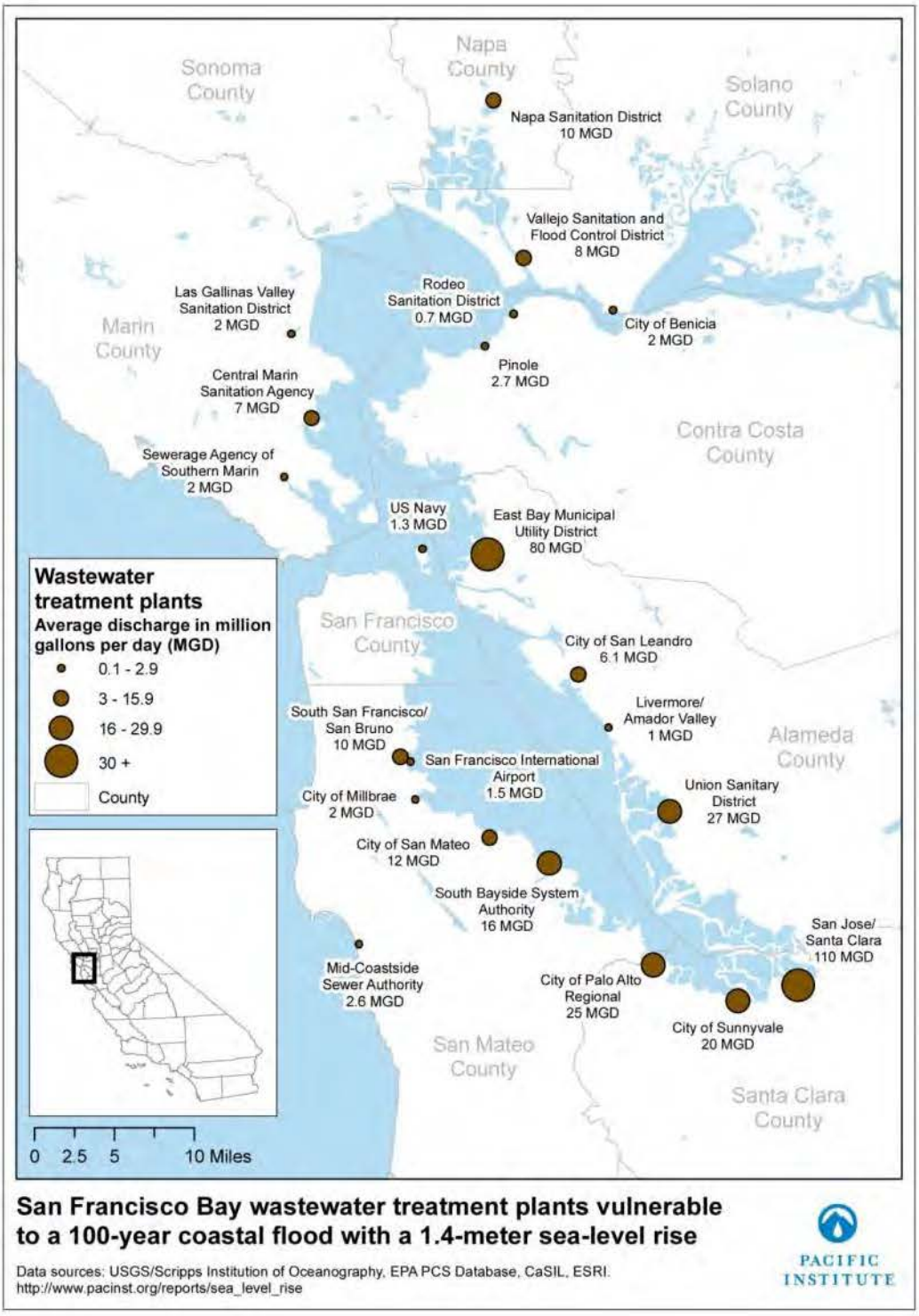

Figure 25. Wastewater treatment plants on the San Francisco Bay vulnerable to a 100-year flood with a 1.4 m sea-level rise

Airports

The San Francisco and Oakland airports are vulnerable to flooding with a 1.4-meter sea-level rise. Other major airports near the coast, such as the San Diego, San Jose, and Los Angeles airports, were not identified as vulnerable in our analysis.

The economic impact of a disruption in airport traffic in San Francisco and Oakland is potentially large, and it would have significant effects on the state and regional economy. In 2007, the Oakland International airport transported 15 million passengers and 647,000 metric tons of freight. Activity at the San Francisco International airport is even greater than in Oakland. The San Francisco International Airport is the nation's thirteenth busiest airport, transporting 36 million people in 2007 (Airports Council International 2007). It also plays a significant role in the movement of goods regionally and internationally. In 2007, the San Francisco airport handled 560,000 metric tons of freight. San Francisco Airport ranked twelfth among foreign trade freight gateways by value of shipments in 2005, handling $25 billion in exports and $32 billion in imports (Bureau of Transportation Statistics 2006), more than double that of the $23.7 billion handled by vessels at the Port of Oakland.

3.1.5. Wetlands

Today, there are approximately 350,000 acres, or 550 square miles, of coastal wetlands in California (Figure 26). Based on an approximated wetland value of $5,000 to $200,000 per acre, we estimate that California's coastal wetlands are worth from $1.8 billion to $70 billion. Large wetland areas are found in almost every county on the California coast (Table 18). The vast majority of coastal wetlands are in San Francisco Bay and the Sacramento-San Joaquin Delta.

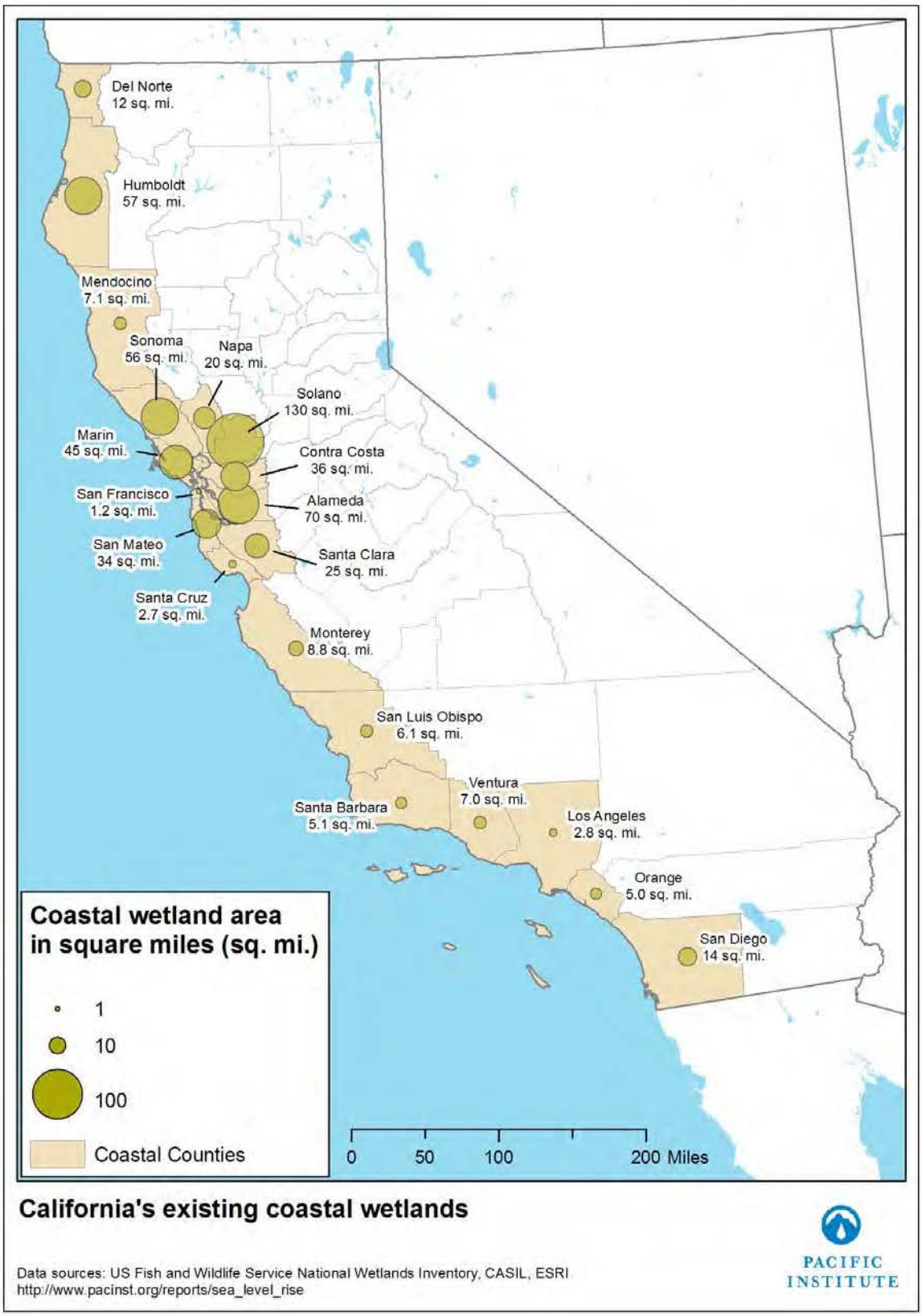

Figure 26. Existing coastal wetlands

There are also significant and important coastal wetlands in Northern California, especially in and around Humboldt and Eureka. Much of the Central California coast, from the Lost Coast in Mendocino County to Big Sur in Monterey, San Luis Obispo, and Santa Barbara counties, is dominated by rugged hills and cliffs plunging into the sea. In these areas, there are very few coastal wetlands. There are critically important wetlands that may be of small size, but that serve vital ecological functions—we understand that size is not the only measure of wetland value. We note that adequate wetland delineation has not been performed on vast areas of California and the actual wetland area may be larger.

Table 18. Existing California coastal wetland area by county

County	Area (square miles)	Area (acres)	Percent of state total
Alameda	70	45,000	13%
Contra Costa	36	23,000	6.5%
Del Norte	12	7,700	2.2%
Humboldt	57	37,000	10%
Los Angeles	2.8	1,800	0.5%
Marin	45	29,000	8.3%
Mendocino	7.1	4,500	1.3%
Monterey	8.8	5,600	1.6%
Napa	20	13,000	3.6%
Orange	5	3,200	0.9%
San Diego	14	8,900	2.5%
San Francisco	1.2	760	0.2%
San Luis Obispo	6.1	3,900	1.1%
San Mateo	34	22,000	6.2%
Santa Barbara	5.1	3,300	0.9%
Santa Clara	25	16,000	4.5%
Santa Cruz	2.7	1,700	0.5%
Solano	130	86,000	24%
Sonoma	56	36,000	10%
Ventura	7	4,500	1.3%
Total	**550**	**350,000**	**100%**

Note: Numbers may not add up due to independent rounding.

Evaluating the impacts of sea-level rise on a particular coastal wetland area requires site-specific data on various physical and biological factors. A simple method to estimate wetland loss is to compare wetland elevations to future tide elevations. Data limitations, however, prevent us from performing even this simple analysis, i.e., there are no data in the critical area where the boundary must be drawn. Given these data limitations, we evaluated the land cover *adjacent* to existing wetlands and the potential for these areas to support suitable wetland habitat. We note that this simplified analysis does not take into account erosion or accretion due to sediment movement, which is difficult to predict with any accuracy.

We estimate that wetlands require approximately 150 square miles of accommodation space, or land into which they must migrate to survive a sea-level rise of 1.4 m. Of this amount, 83 square miles, or 55%, would make viable wetland habitat (Table 19). These areas should be protected to ensure their viability as wetland habitat is maintained. Twenty-three square miles, or 15%, is land that is viable for wetland migration but at some loss of value, including parks, orchards, and agricultural land. The remaining 30% of the available accommodation space is unsuitable for wetland migration because it is built up; covered with roads, buildings, and pavement.

Table 19. Wetland migration frontier area classified by land cover type and conversion potential

Land cover type	Total frontier area (square miles)
Not viable for wetland migration	
High Intensity Developed	12
Medium Intensity Developed	12
Low Intensity Developed	21
Subtotal	**45**
Viable for wetland migration, but will cause property loss	
Developed Open Space	4.7
Pasture/Hay	11
Cultivated	7.0
Subtotal	**23**
Viable for wetland migration	
Evergreen Forest	0.28
Deciduous Forest	0.040
Mixed Forest	0.27
Scrub/Shrub	1.3
Grassland	16
Bare Land	0.89
Palustrine Scrub/Shrub Wetland	0.85
Palustrine Forested Wetland	0.47
Palustrine Emergent Wetland	4.7
Estuarine Scrub/Shrub Wetland	42
Estuarine Forested Wetland	2.4
Estuarine Emergent Wetland	0.11
Estuarine Aquatic Bed	0.046
Unconsolidated Shore	4.0
Water	10
Subtotal	**83**
Total	**150**

Note: Numbers may not add up due to rounding.

Figures 27, 28, 29, and 30 and Table 20 summarize the potential wetland migration area by county. Solano County has the largest wetland migration area, totaling 22 miles, and 85% of that area is currently viable wetland habitat. Of the potential 20 miles of wetland migration area in Humboldt County, only 39% is viable wetland habitat, although an additional 54% is viable but with some economic loss. San Francisco and Los Angeles Counties have only small potential wetland migration areas, in part because there are few wetlands in these counties. Unfortunately, those that do exist are at high risk because 70% and 60% of the potential wetland migration area in San Francisco and Los Angeles Counties, respectively, is not viable wetland habitat.

Table 20. Land area available for wetland migration, by county, in square miles, with percent of county total in italics

County	Wetland migration viable		Migration viable with loss of value		Migration not viable		Total	Percent of State Total
Alameda	8.5	*49%*	0.94	*5%*	8.1	*46%*	17	10%
Contra Costa	8.1	*72%*	0.68	*6%*	2.5	*22%*	11	6.7%
Del Norte	2.1	*81%*	0.39	*15%*	0.13	*5%*	2.6	1.6%
Humboldt	7.7	*39%*	11	*54%*	1.2	*6%*	20	12%
Los Angeles*	0.10	*35%*	0.012	*4%*	0.17	*60%*	0.28	0.17%
Marin	5.7	*54%*	0.29	*3%*	4.7	*44%*	11	6.3%
Mendocino	1.3	*93%*	0.035	*2%*	0.059	*4%*	1.4	0.8%
Monterey	4.1	*56%*	2.6	*36%*	0.60	*8%*	7.3	4.3%
Napa	2.9	*80%*	0.24	*6%*	0.51	*14%*	3.7	2.2%
Orange*	0.72	*22%*	0.20	*6%*	2.4	*72%*	3.3	2.0%
San Diego	3.7	*64%*	0.33	*6%*	1.7	*30%*	5.8	3.4%
San Francisco	0.20	*18%*	0.15	*13%*	0.80	*70%*	1.1	0.7%
San Luis Obispo	0.78	*69%*	0.081	*7%*	0.27	*24%*	1.1	0.7%
San Mateo	2.9	*20%*	0.54	*4%*	11	*76%*	15	8.7%
Santa Barbara*	0.87	*86%*	0.023	*2%*	0.12	*12%*	1.0	0.6%
Santa Clara	2.2	*29%*	0.81	*11%*	4.6	*60%*	7.6	4.5%
Santa Cruz	0.98	*40%*	1.1	*43%*	0.42	*17%*	2.5	1.5%
Solano	19	*85%*	0.87	*4%*	2.5	*11%*	22	13%
Sonoma	7.6	*87%*	0.53	*6%*	0.59	*7%*	8.8	5.2%
Ventura	**3.4**	*45%*	2.2	*29%*	2.0	*26%*	7.6	4.5%
Total	**83**	***55%***	**23**	***15%***	**45**	***30%***	**150**	**100%**

*Given data limitations, we mapped about 49% of Santa Barbara County, 23% of Los Angeles County, and 65% of Orange County. The coverage was 100% in the other 11 counties on the Pacific coast.

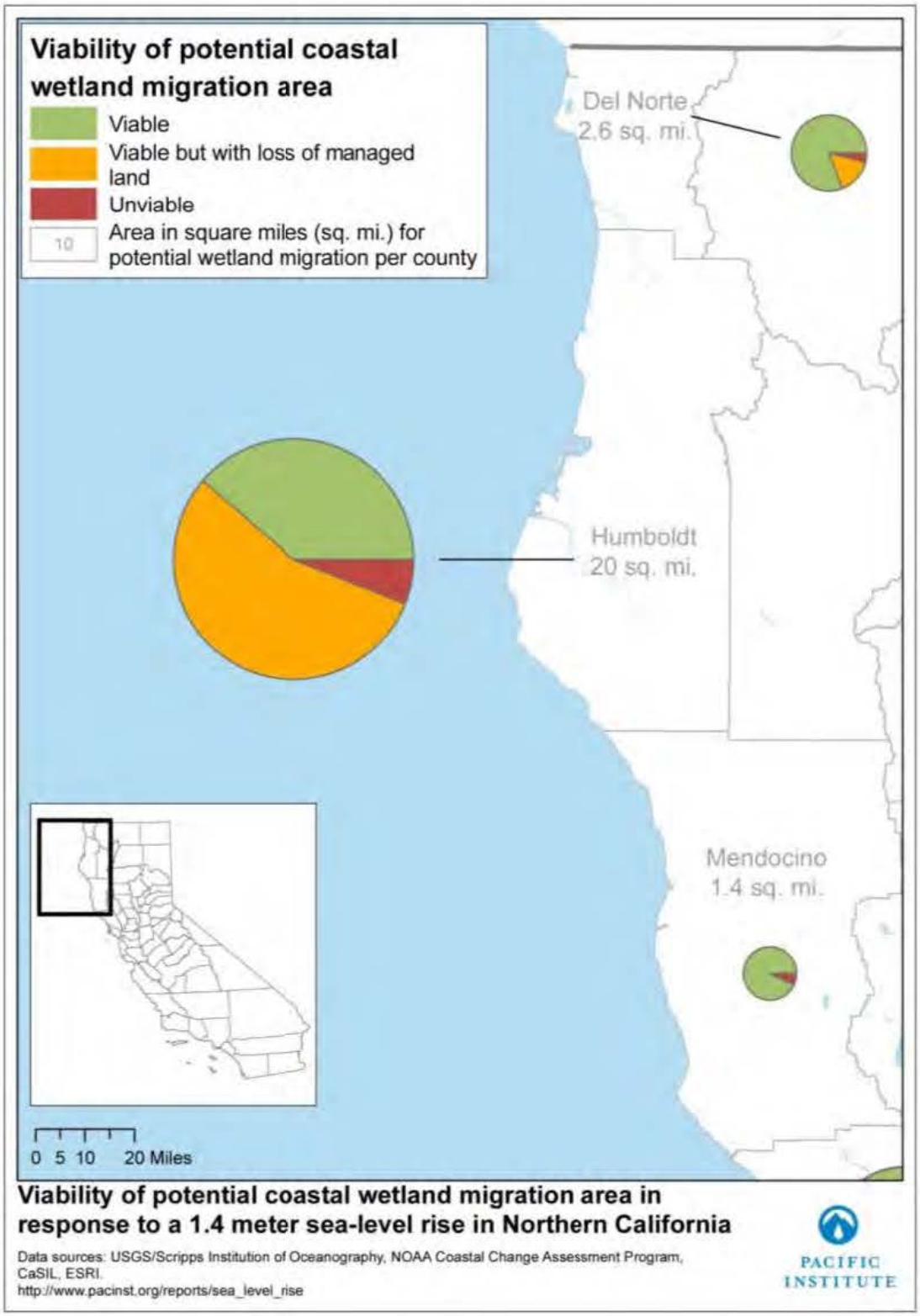

Figure 27. Viability of potential wetland migration area in response to a 1.4 m sea-level rise in Northern California

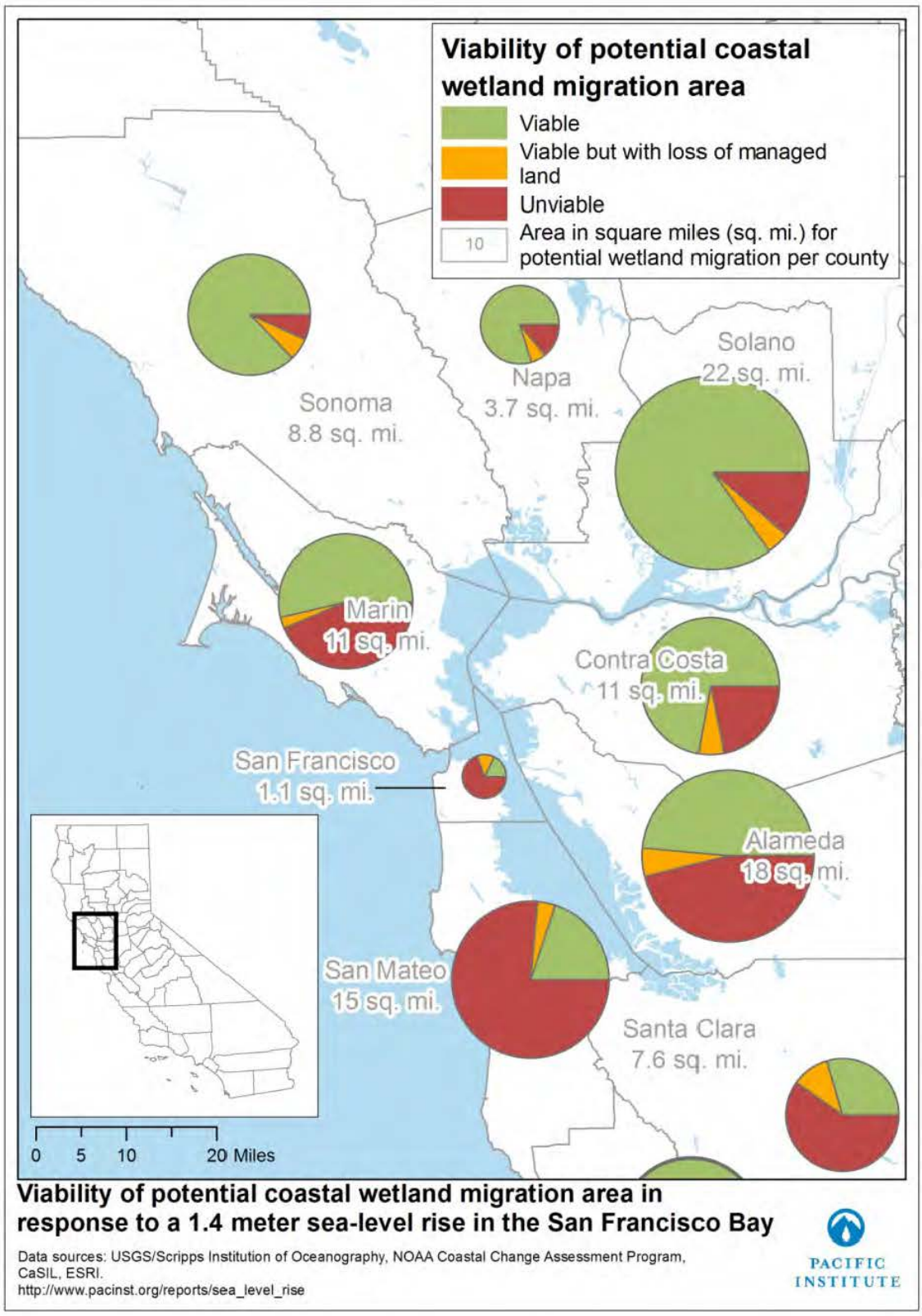

Figure 28. Viability of potential wetland migration area in response to a 1.4 m sea-level rise in the San Francisco Bay

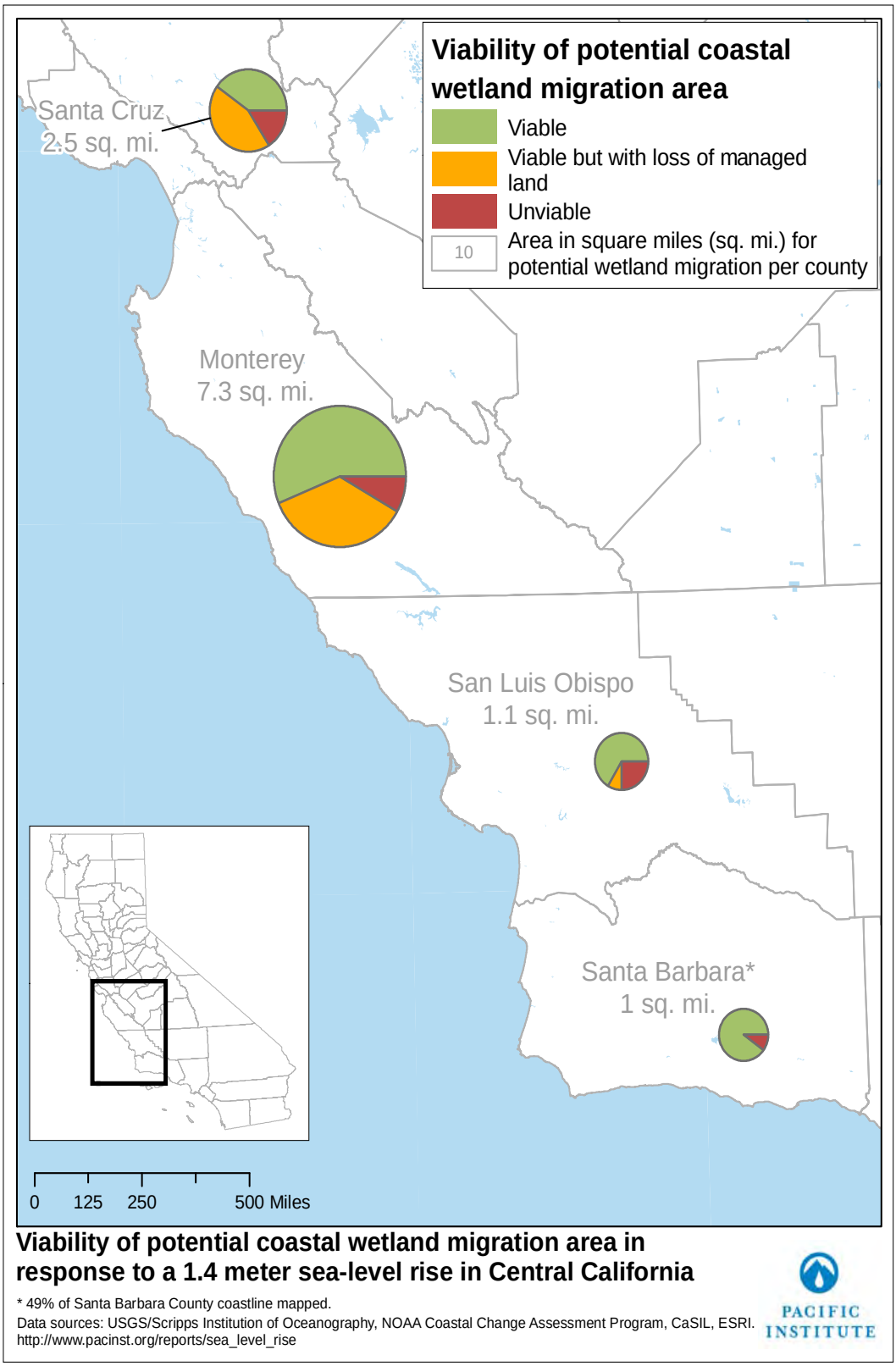

Figure 29. Viability of potential wetland migration area in response to a 1.4 m sea-level rise in Central California

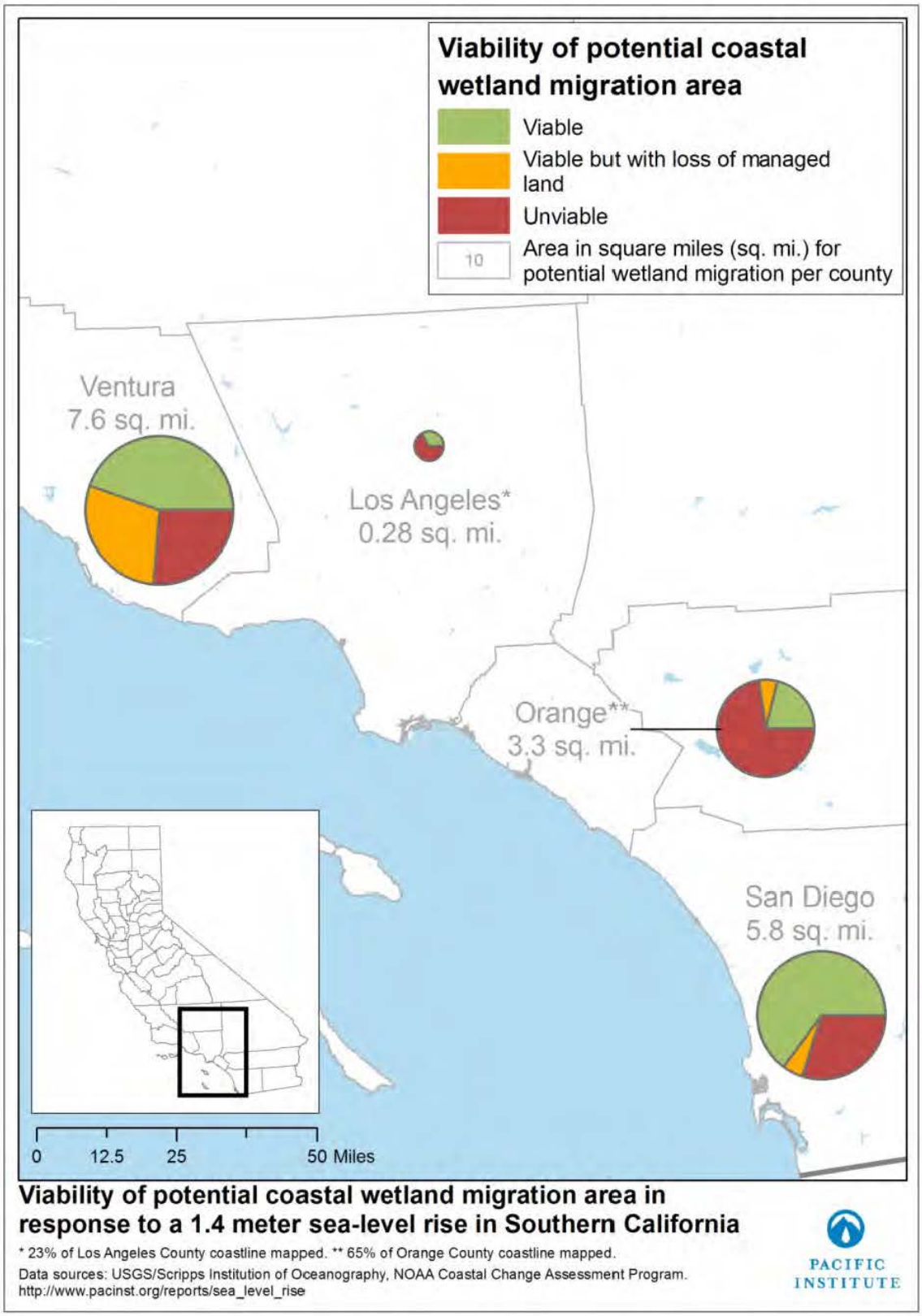

Figure 30. Viability of potential wetland migration area in response to a 1.4 m sea-level rise in Southern California

3.1.6. Property at Risk

Significant property is at risk of flooding from 100-year flood events as a result of a 1.4 m sea-level rise. In total, we estimate that the replacement value of this property totals nearly $100 billion (Figure 31). An overwhelming two-thirds of that property is concentrated on San Francisco Bay, indicating that this region is particularly vulnerable to impacts associated with sea-level rise due to extensive development on the margins of the Bay (Figure 32).

Replacement value of buildings and contents vulnerable to a 100-year coastal flood with a 1.4-meter sea-level rise

* Values are in year 2000 dollars

Data sources: USGS/Scripps Institution of Oceanography, FEMA HAZUS model, CaSIL, ESRI.
http://www.pacinst.org/reports/sea_level_rise

PACIFIC INSTITUTE

Figure 31. Replacement value of buildings and contents vulnerable to a 100-year coastal flood with a 1.4 m sea-level rise

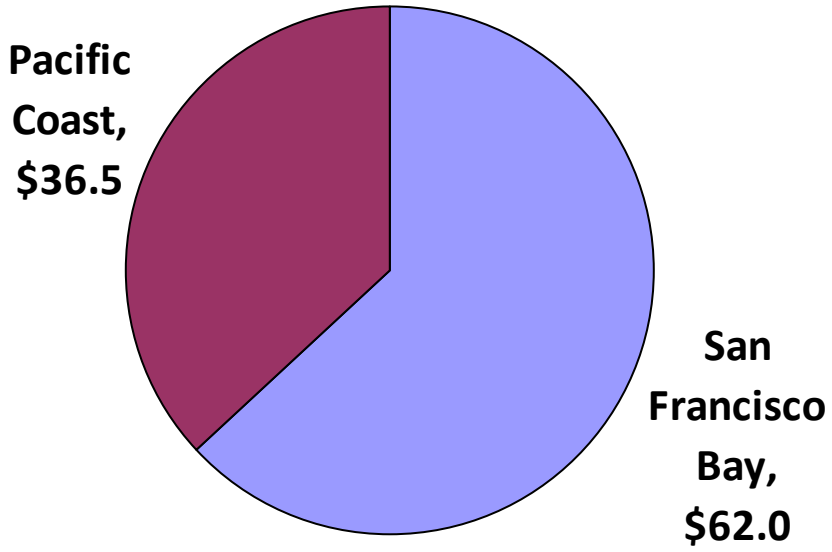

Figure 32. Replacement value (in billions of year 2000 dollars) of buildings and contents at risk of a 100-year flood event with a 1.4 m sea-level rise, by region

Note: Counties with borders on the Pacific coast and San Francisco Bay (e.g., San Mateo) were separated based on the shoreline affected.

Pacific Coast

Within each region, vulnerability to sea-level rise is highly variable. Table 21 shows the replacement value of buildings and their contents at risk of a 100-year flood event with a 1.4 m sea-level rise for the Pacific coast by county. Property at risk during a 100-year flood increases from about $21 billion in 2000 to $37 billion (in year 2000 dollars) with a 1.4 m sea-level rise. About $17 billion of property, or about 50% of the total property at risk, is in Orange County. Los Angeles, Santa Cruz, Monterey, and Ventura Counties also have significant assets at risk, totaling in excess of $2 billion each.

Table 21. Replacement value of buildings and contents (millions of year 2000 dollars) at risk of a 100-year flood event along the Pacific coast, by county

County	Current risk	Risk with 1.4 m sea-level rise	Percent increase
Del Norte	240	350	+ 43%
Humboldt	680	1,400	+ 110%
Los Angeles	1,400	3,800	+ 180%
Marin	220	260	+ 16%
Mendocino	120	150	+ 22%
Monterey	1,700	2,200	+ 36%
Orange	11,000	17,000	+ 63%
San Diego	690	2,000	+ 190%
San Francisco	670	890	+ 33%
San Luis Obispo	220	360	+ 67%
San Mateo	730	910	+ 26%
Santa Barbara	460	1,100	+ 140%
Santa Cruz	2,400	3,300	+ 34%
Sonoma	170	200	+ 20%
Ventura	980	2,200	+ 120%
Total	**21,000**	**37,000**	+ **71%**

Note: All values are shown in millions of year 2000 dollars. Counties with borders on the Pacific coast and San Francisco Bay (e.g., San Mateo) were separated based on the shoreline affected.

All economic sectors are vulnerable to impacts associated with sea-level rise. Figure 33 shows the breakdown of the buildings and contents at risk of 100-year flood by major economic sector for the Pacific coast (specific sectors, such as transportation, are discussed in Section 3.2). More than 70% of the assets at risk are residential. The commercial sector, accounting for nearly 20% of the value at risk, will also likely encounter significant costs. Agriculture, education, religion, and government each account for about 1% of the assets at risk, thus, their exposure to risk is relatively small.

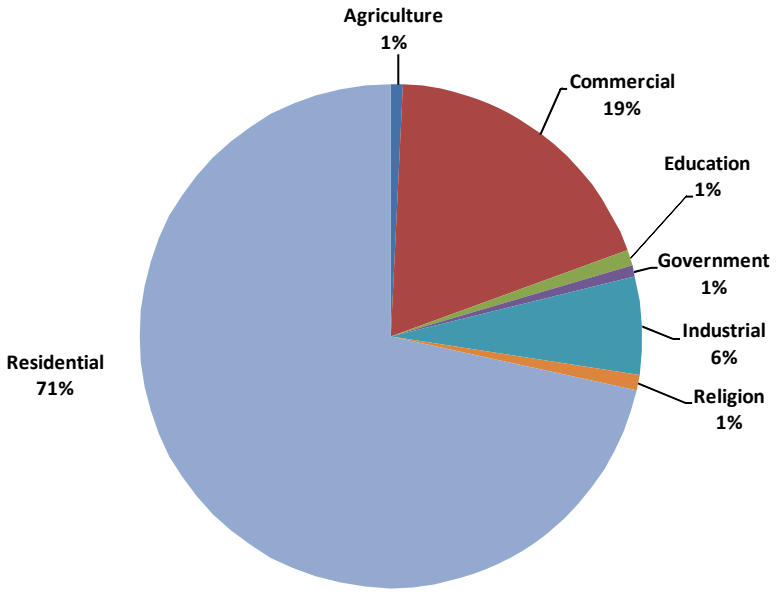

Figure 33. Replacement value of buildings and contents at risk of 100-year flood event with a 1.4 m sea-level rise along the Pacific coast, by major economic sector

San Francisco Bay

The value of assets at risk on San Francisco Bay is substantially higher than along the Pacific coast. Table 22 shows the replacement value of buildings and their contents vulnerable to a 100-year flood event with a 0.5 m, 1.0 m, and 1.4 m sea-level rise. Note that the model used to develop inundation maps for San Francisco Bay allows us to determine the property at risk from any flood intensity. Assets at risk during a 100-year flood increase from about $29 billion in 2000 to $36 billion, $49 billion, and $62 billion (in year 2000 dollars) with a 0.5 m, 1.0 m, and 1.4 m sea-level rise, respectively.

The assets at risk are not evenly distributed among the counties on San Francisco Bay (Table 22). San Mateo and Alameda counties have the greatest assets at risk, accounting for about 60% of the total assets at risk with sea-level rise. Marin, Santa Clara, and San Francisco counties are also exposed to a high degree of risk; exposure to risk in these counties is higher than in all other counties along the Pacific coast, with the exception of Orange County. Exposure to risk in Sonoma and Napa counties is relatively modest.

Table 22. Replacement value of buildings and contents at risk of a 100-year flood on San Francisco Bay, by county (in millions of year 2000 dollars)

County	Current Risk	Risk with sea-level rise			Percent Increase (1.4 m)
		0.5 m	1.0 m	1.4 m	
Alameda	3,300	5,300	10,000	15,000	+ 370%
Contra Costa	190	330	620	980	+ 430%
Marin	4,700	5,900	7,400	8,500	+ 79%
Napa	220	260	320	410	+ 89%
San Francisco	110	370	1,400	4,000	+ 3400%
San Mateo	16,000	18,000	21,000	23,000	+ 41%
Santa Clara	3,700	4,700	6,400	7,800	+ 110%
Solano	620	940	1,400	1,900	+ 210%
Sonoma	150	180	240	280	+ 82%
Total	29,000	**36,000**	**49,000**	**62,000**	+ **110%**

Note: Counties with borders on the Pacific coast and San Francisco Bay (e.g., San Mateo) were separated based on the shoreline affected.

As it is along the Pacific coast, the residential sector on San Francisco Bay faces the greatest risk. Figure 34 shows the buildings and contents at risk of a 100-year flood by major economic sector on San Francisco Bay (specific sectors, such as transportation, are discussed in Section 3.1.4). Of the $62 billion of property at risk with a 1.4 m sea-level rise, about 50% of the assets at risk are residential, substantially smaller than along the Pacific coast. The commercial and industrial sectors face much greater risk on San Francisco Bay than on the Pacific coast. Agriculture, education, religion, and government each account for about 1% of the assets at risk, thus their exposure to risk is fairly small.

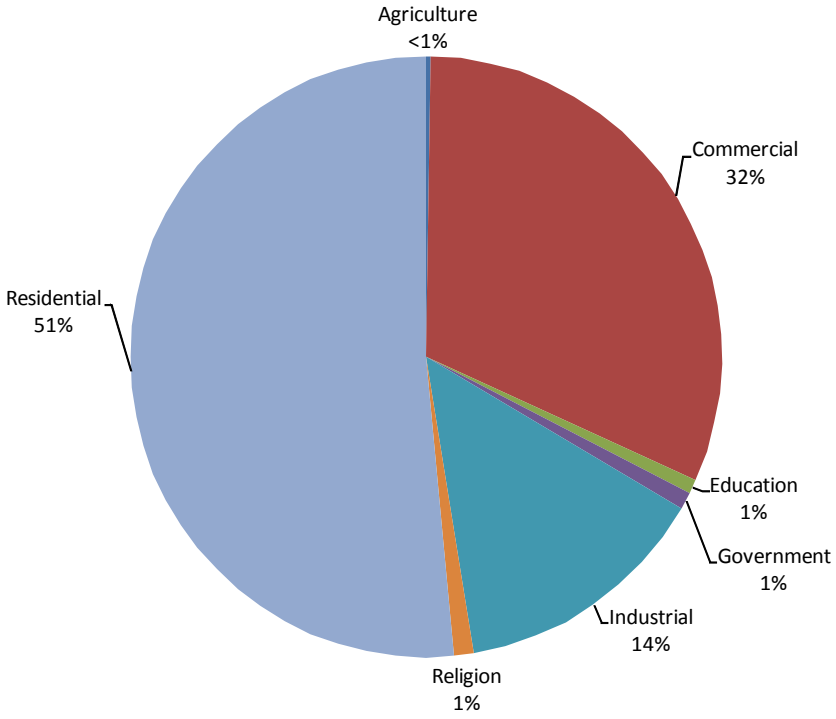

Figure 34. Replacement value of buildings and contents at risk of a 100-year flood with a 1.4 m sea-level rise on San Francisco Bay, by major economic sector

3.1.7. Saltwater Intrusion to Groundwater Aquifers

Saltwater intrusion into aquifers is a man-made problem in many places in California, resulting from over-pumping, but it will be accelerated and made worse by sea level rise. It occurs where saline water moves inland into a freshwater aquifer, contaminating it with salts and making it unsuitable for water supply or irrigation. Pumping coastal aquifers in excess of natural recharge rates draws down the surface of the aquifer. When the ocean has a higher "potentiometric surface," or water elevation, it causes the saltwater wedge to intrude further inland (Figure 35). Seawater intrusion is already problematic in California's coastal aquifers throughout Central and Southern California, including the Pajaro and Salinas Valleys and aquifers in Orange and Los Angeles Counties.

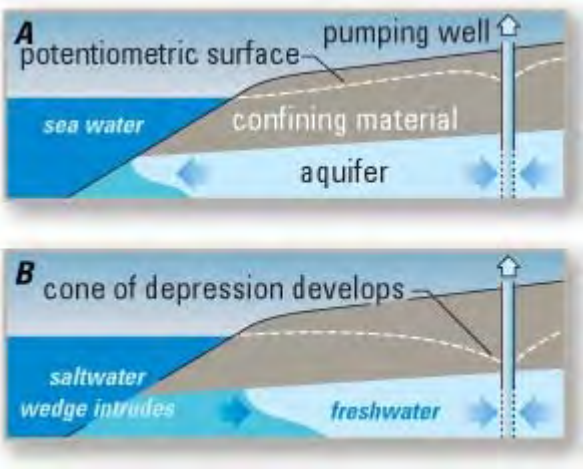

Figure 35. Saltwater intrusion

Source: Edwards and Evans 2002

Sea-level rise will increase saltwater intrusion into coastal aquifers. This in turn will increase water insecurity, as communities that rely on groundwater for all or part of their water supply will be forced to search for alternatives, which could include costly surface water transfers from outside their basin or desalination. Farmers who irrigate with groundwater may be unable to secure another reasonably-priced water source, forcing them to retire the land. None of these costs are quantified here, but they are not zero and should be studied in future work.

A number of actions can help mitigate saltwater intrusion. Significant investments in water conservation and efficiency improvements can allow water managers to reduce pumping. Enhancing natural recharge by limiting impervious areas (pavement), adopting low-impact development techniques, and building infiltration basins can also reduce intrusion. Another option is to artificially recharge the aquifer with freshwater imported from outside the basin. Additional work is needed to understand the impacts of sea-level rise on California's coastal aquifers and to develop ways to mitigate those impacts.

3.1.8. Cost of Protection

Approximately 1,100 miles of new or modified coastal protection structures are needed on the Pacific Coast and San Francisco Bay (Table 23). The total cost of building new or upgrading existing structures is estimated at about $14 billion (in year 2000 dollars). The majority of the investment is needed in Southern California. Nearly 20% of that investment would be needed in Los Angeles County alone. Significant investments would also be needed in Orange and San Diego counties. Mendocino would need the least amount of coastal armoring, although this area is particularly vulnerable to erosion, which is not reflected in this analysis. We estimate that operating and maintaining the protection structures would cost approximately 10 percent of the initial capital investment, or around another $1.4 billion per year.

Table 23. Estimated length (in miles) and capital cost of required defenses needed to guard against flooding from a 1.4 m sea-level rise, by county.

County	Raise levee (miles)	New levee (miles)	New seawall (miles)	Total (miles)	Capital Cost (millions of year 2000 dollars)
Alameda	45	49	16	110	950
Contra Costa	26	29	8.0	63	520
Del Norte	-	38	1.0	39	330
Humboldt	-	36	6.6	42	460
Los Angeles	0.88	2.5	94	97	2,600
Marin	43	77	7.7	130	930
Mendocino	-	0.29	1.2	1.4	34
Monterey	27	6.4	19	53	650
Napa	2.8	62	-	64	490
Orange	-	11	66	77	1,900
San Diego	-	-	47	47	1,300
San Francisco	-	10	21	31	680
San Luis Obispo	-	7.4	5.4	13	210
San Mateo	35	29	9.2	73	580
Santa Barbara	2.4	5.6	4.5	13	180
Santa Clara	47	4.0	-	51	160
Santa Cruz	3.9	1.6	9.3	15	280
Solano	2.7	63	8.0	73	720
Sonoma	30	15	1.3	47	240
Ventura	-	0.35	28	29	790
Total	**270**	**450**	**350**	**1,100**	**14,000**

3.2. Erosion-Related Risks

3.2.1. Population at Risk from Erosion

The erosion hazard zone totals 41 square miles within the 11 coastal counties evaluated in this analysis (Table 24). There is significant variation in the areas at risk of erosion. In Humboldt County, for example, 6.1 square miles of coast would be lost by 2100 under a sea-level rise scenario of 1.4 meters. In San Francisco, however, the erosion-related risk is small.

Table 24. Erosion with a 1.4 m sea-level rise, by county.

County	Dune erosion (sq. miles)	Cliff erosion (sq. miles)	Total erosion (sq. miles)
Del Norte	1.9	2.6	4.5
Humboldt	3.7	2.4	6.1
Marin	1.0	3.7	4.7
Mendocino	0.74	7.5	8.3
Monterey	1.9	2.5	4.4
San Francisco	0.23	0.30	0.53
San Luis Obispo	1.4	1.5	2.9
San Mateo	0.82	2.4	3.2
Santa Barbara	0.62	1.9	2.6
Santa Cruz	0.87	0.9	1.8
Sonoma	0.60	1.6	2.2
Total	**14**	**27**	**41**

As discussed in Section 2.3.2, dunes and cliffs will exhibit differential responses to rising sea levels. Our results indicated that cliffs will erode an average distance of about 66 m by the year 2100 (Table 25). In some areas, however, erosion is projected to be much higher. In Del Norte County, for example, cliffs erode a maximum distance of 520 m. Cliff erosion is much less severe in the other counties along the coast, although still significant. Dunes exhibit much less resistance to erosion. On average, dunes will erode about 170 m by 2100. In Humboldt County, however, dunes are projected to erode nearly 600 m by 2100.

Table 25. Average and maximum erosion distance in 2000 for cliffs and dunes, by county

	Dune erosion		Cliff erosion	
County	Average distance (m)	Maximum distance (m)	Average distance (m)	Maximum distance (m)
Del Norte	180	400	160	520
Humboldt	160	600	61	260
Marin	140	270	110	240
Mendocino	190	440	33	160
Monterey	180	400	37	220
San Francisco	150	230	90	220
San Luis Obispo	140	330	78	280
San Mateo	230	430	31	220
Santa Barbara	190	320	54	240
Santa Cruz	170	340	36	130
Sonoma	150	320	41	190
Average	**170**	**370**	**66**	**240**

Table 26 shows the population at risk from erosion with a 1.4 m sea-level rise in 2100. Flood-related risk is shown for comparative purposes. In the 11 coastal counties north of Santa Barbara, a total of 14,000 people live within areas at risk of erosion. In comparison, 63,000 people are vulnerable to a 100-year flood event within these counties. In most counties, the flood-related risk is substantially higher than the erosion-related risk. In Mendocino and Sonoma Counties, however, erosion poses a greater threat than flooding. In Marin and San Luis Obispo Counties, the flood-related and erosion-related risks are comparable. In addition to those who live in areas vulnerable to erosion, approximately 6,600 people are employed in facilities located there, of which 95% are employed in the commercial sector and the remaining 5% are employed in the industrial sector.

Table 26. Population vulnerable to flood and erosion from a 1.4 m sea-level rise along the Pacific coast, by county

County	Flood-related Risk	Erosion-related Risk
Del Norte	2,600	620
Humboldt	7,800	580
Marin	630	570
Mendocino	650	930
Monterey	14,000	820
San Francisco	6,500	1,200
San Luis Obispo	1,300	1,100
San Mateo	5,900	2,900
Santa Barbara	6,700	2,100
Santa Cruz	16,000	2,600
Sonoma	700	300
Total	**63,000**	**14,000**

Note: Numbers may not add up due to rounding.

3.2.2. Emergency and Healthcare Facilities at Risk from Erosion

Emergency and healthcare facilities at risk from erosion along the California coast are limited. The analysis identified a single health care facility near Pacifica that is vulnerable to erosion. There are no schools or fire and police stations within the erosion hazard zone.

3.2.3. Infrastructure at Risk from Erosion

Roads and Railways

Significant transportation-related infrastructure is vulnerable to erosion. Nearly 240 miles of highways and roads and 10 miles of railways are at risk of erosion in the 11 coastal counties north of Santa Barbara (Table 27). This is far fewer than the transportation-related infrastructure at risk from flooding but as mentioned previously, erosion causes far greater and potentially more permanent damage than flooding. In addition, areas such as Big Sur already have significant routine highway maintenance costs due to existing erosion conditions and these costs are likely to increase as erosion rates increase (Figure 36).

Other than roads and railways, little critical infrastructure is located within the erosion hazard zone. We identified no wastewater treatment plants, power plants, schools, police, or fire stations within the area at risk of erosion.

Table 27. Miles of roads and railways vulnerable to erosion and flood from a 1.4 m sea-level rise along the Pacific coast, by county and type

	Highways (miles)		Roads (miles)		Railways (miles)	
County	Erosion-risk	Flood-risk	Erosion-risk	Flood-risk	Erosion-risk	Flood-risk
Del Norte	4.3	8.2	14	80	-	-
Humboldt	6.0	58	20	190	-	28
Marin	2.1	4.1	19	27	-	-
Mendocino	13	7.9	25	41	-	4.0
Monterey	11	31	15	110	2.1	23
San Francisco	0	8.0	17	25	-	-
San Luis Obispo	2.5	0.4	18	22	-	0.3
San Mateo	9.8	11	18	67	-	-
Santa Barbara	0.74	7.4	12	21	6.4	7.0
Santa Cruz	2.4	5.0	20	30	1.6	5.5
Sonoma	6.2	8.0	8.4	57	-	-
Total	**58**		**180**		**10**	

Note: Numbers may not add up due to rounding.

Figure 36. Road erosion along Highway 1 with deployment of erosion mitigation strategy

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3.2.4. Property at Risk from Erosion

Land on or near the coast is highly desirable and often commands a premium price. Homes lost to erosion cannot be replaced because the land will have disappeared. As a result, the replacement values reported in the HAZUS database cannot be used to evaluate erosion. A detailed estimate of the value of land and homes that would be completely lost was beyond the scope of this analysis. In order to bound the problem, however, we sought to determine the number of parcels at risk by overlaying the erosion hazard zone layer with the available parcel data. Note that the erosion hazard zone was identified for portions of 11 of California's coastal counties. Eight of these 11 counties had parcel data in digital format.

Parcels are used by counties to levy property taxes. Assessor's offices divide entire counties into parcels, which can represent publicly-owned land, roads, lakes, and other features. A single parcel may also contain an apartment building with many hundreds of residences. Thus, this is an imprecise way of estimating how much property may be lost to coastal erosion. This is an area of study that can and should be pursued in more detail by local governments and regional planning agencies.

We estimate that approximately 10,000 parcels lie within the erosion hazard zone, as summarized in Table 28. Of these parcels, 66%, or two-thirds, lie entirely in the erosion hazard zone, meaning the property would be lost completely. The remaining third are partially eroded. If we assume that the value of the average coastal parcel is $1.4 million (Heinz Center 2000), then the economic cost to property of erosion from a 1.4 m sea-level rise would total $14 billion. More work on the economic consequences of erosion is needed.

Table 28. Number of properties within the erosion zone hazard zone with a 1.4 m sea-level rise, by county

County	Number of parcels
Del Norte	No data
Humboldt	570
Marin	1,300
Mendocino	No data
Monterey	1,600
San Francisco	850
San Luis Obispo	No data
San Mateo	1,900
Santa Barbara	580
Santa Cruz	3,000
Sonoma	500
Total	**10,000**

Note: Numbers may not add up due to rounding.

4.0 Conclusions and Recommendations

4.1. Conclusions

Rising sea levels will be among the most significant impacts of climate change to California. Sea level will rise as a result of thermal expansion of the oceans and an increase in ocean volume as land ice melts and runs off. Over the past century, sea level has risen nearly eight inches along the California coast and general circulation model scenarios suggest very substantial increases in sea level due to climate change over the coming century. This study evaluates the current population, infrastructure, and property at risk from projected sea-level rise if no actions are taken to protect the coast. The sea-level rise scenario was developed by the State of California from medium to medium-high greenhouse gas emissions scenarios from the Intergovernmental Panel on Climate Change (IPCC) but does not reflect the worst case sea-level rise that could occur.

We estimate that a 1.4 m sea-level rise will put 480,000 people at risk of a 100-year flood event. Among those affected are large numbers of low-income people and communities of color, which are especially vulnerable. A wide range of critical infrastructure, such as roads, hospitals, schools, emergency facilities, wastewater treatment plants, power plants, and wetlands is also vulnerable. In addition, $100 billion (in year 2000 dollars) worth of property is at risk of coastal flooding. A number of structural and non-structural policies and actions could be implemented to reduce these risks. For example, we estimate that protecting vulnerable areas from flooding by building seawalls and levees will cost $14 billion (in year 2000 dollars), along with an additional $1.4 billion per year (in year 2000 dollars) in maintenance costs. Continued development in vulnerable areas will put additional assets at risk and raise protection costs. Determining what to protect, how to pay for it, and how those choices are made raises concerns over equity and environmental justice.

Large sections of the Pacific coast are not vulnerable to flooding, but are highly susceptible to erosion. We estimate that a 1.4 m sea-level rise will accelerate erosion, resulting in a loss of 41 square miles of California's coast by 2100. A total of 14,000 people live in areas at risk of erosion. In addition, significant transportation-related infrastructure and property are also at risk. Throughout most of the state, flood risk exceeds erosion risk, but in some counties, coastal erosion poses a greater risk. We also provide, below, a set of recommendations for actions and policies that can reduce future risks and vulnerabilities.

4.2. Recommendations

Climate changes are inevitable, and adaptation to unavoidable impacts must be evaluated, tested, and implemented. Sea levels have risen observably in the past century, and scientists forecast that sea-level rise will continue for centuries, even if we stop emitting greenhouse gases immediately. As a result, coastal areas will be subject to increasing risk of inundation and erosion. Below, we provide a series of recommendations and principles to guide the adaptation process.

4.2.1. Principles for Adaptation

The decisions about what to protect, how to protect it, and who will have to pay will be both challenging and controversial. Given the complexity of these issues, it is important to develop an open and transparent process involving all affected stakeholders. Below, we provide some general principles to guide this process:

- Human life must be protected.
- Critical ecological systems should be preserved.
- Development and protection of the coast should be governed by the principles of sustainability. Simply stated, this means "meeting the needs of the present without compromising the ability of future generations to meet their own needs" (WCED 1987).
- Equal and full participation must be a central element of any decision-making process. No social or economic group should be excluded from decision-making that will affect its well-being.
- Communities must determine the resources and features they value, e.g., beaches, public access, fisheries, etc., and develop plans to protect those resources.
- Consideration should be given to equitable distribution and apportionment of costs and benefits of adaptation measures.
- Adaptation strategies should account for the distinct vulnerabilities of potentially affected subpopulations.
- Local and regional planning processes must begin early to incorporate estimates of sea-level rise and strategies for adaptation.

4.2.2. Recommended Practices and Policies

Climate change must be integrated into the design of all coastal structures.

Current efforts to build, maintain, or modify structures in coastal areas at risk of sea-level rise must now be based on estimates of that rise. The costs of modifying structures in the design phase are often far lower than the costs of later reconstruction or flood damage.

The federal government and the insurance industry should develop and implement a methodology for integrating climate change into insurance policies and strategies.

Properly designed insurance policies are vital for helping landowners choose whether to protect or abandon risky property. The design, availability, and cost of flood insurance will be a key instrument in implementing floodplain policy. For example, the government should not continue to subsidize flood insurance for properties that have suffered repetitive losses. Nor should insurance be available for properties highly likely to be inundated under future conditions.

We do not propose that flood insurance policies be taken away from business or homeowners. We do suggest, however, that policyholders in areas that are exposed to increased risk should not be allowed to rebuild or make major repairs to a structure after it has been damaged, nor should they be allowed to expand or make significant improvements, and that long-term insurance policies take into account changes in coastal risks.

Federal flood insurance maps should include information on future flood risks due to sea-level rise.

The Federal Emergency Management Agency's official flood insurance studies show hazard zones that reflect past or present flood risks. Because these are the *de facto* planning documents used by most local governments, they should be updated to show the *future* hazard areas and include the current science on climate change and sea-level rise.

Wetlands and the potential migratory paths should be protected.

Development should be prohibited on natural lands that are immediately adjacent to wetlands at risk. These buffer areas may be the only areas suitable for future wetland restorations projects.

Future development should be limited in areas that are at risk from rising seas.

In regions at risk that are not yet heavily developed, local communities and coastal planning agencies have the opportunity to limit development and reduce future threats to life and property. Policies that maintain such low-lying areas will help to accommodate rising seas. In addition to insurance policies, discussed above, such policies may include local ordinances, statewide coastal development policies, and explicit purchases of land for conservation purposes. This is often the least expensive option for currently undeveloped areas.

While limiting coastal development is the most effective way to reduce risk, this approach can incur costs today. Development permits designed to provide flexibility for future generations to address sea-level rise will reduce today's cost. For example, permits might allow development but stipulate that the area reverts to nature if seas rise by a specified amount.

Local planning processes need to involve communities most vulnerable to harm when developing appropriate preparation and adaptation strategies.

The particular needs of vulnerable communities, and appropriate adaptation policies, are best identified and developed through processes in which the affected communities are at the center of decision making. The vulnerabilities to sea-level rise created by access to transportation, legal residency, income, and language abilities can only be fully understood and protected when members of these communities are directly involved in the process.

Consider phased abandonment of low- and medium-density areas at high risk.

In some low- and medium- density areas, the monetary and environmental cost of holding back the sea may become unacceptably high. The lowest-cost option may be to allow natural processes take place. Policies that prevent flood-damaged homes or businesses from rebuilding may help ease this transition.

Protect vital societal resources, especially those that are "coastal-dependent."

In many cases, the value of an area's infrastructure far exceeds the cost to raise structures or build protective barriers. For example, the San Francisco airport and the Port of Long Beach are extremely important to the state and national economy. In choosing what to protect, we should favor infrastructure that necessarily belongs on the coast, such as ports, bridges, and marinas.

Cost-benefit analyses should explicitly evaluate the social and environmental costs of building coastal protection structures.

Armoring the coastline can save lives and property, but it also comes at a cost. The natural dynamics that occur between water and land are disrupted. Beaches and wetlands disappear and habitat is lost. Traditional cost-benefit analyses, such as those required for all US Army Corps of Engineers projects, do not adequately account for these inherent tradeoffs.

Coastal emergencies are inevitable. Coastal communities should improve disaster response and recovery.

In this analysis, we have focused on increased risk of coastal flooding and erosion as a result of sea-level rise. California is also subject to tsunamis, earthquakes, wildfires, terrorist attack, and other hazards. Improving community preparedness provides benefits for responding to any type of emergency. Before a disaster strikes, communities must plan for evacuation routes, emergency action plans, and shelters, and take into account the specific needs of vulnerable populations. In addition, roles and responsibilities must be clearly defined among local, state, and federal agencies.

Coastal managers should consider adopting the principles of "No Adverse Impact" when designing and permitting flood protection, beach nourishment, and other coastal protection projects.

Current coastal protection projects are often done with no regard for how they will affect adjacent portions of the coast. According to the Association of State Floodplain Managers (ASFM): "Over the past 50 years a system has developed through which local and individual accountability has been supplanted by federal programs for flood control, disaster assistance, and tax incentives that encourage and subsidize floodplain occupation and development." We recommend that coastal managers consider adopting a policy similar to "No Adverse Impact"

where the "actions of one property owner are not allowed to adversely affect the rights of other property owners" (ASFM 2008).

4.2.3. Additional Research and Analysis

Local governments or regional planning agencies should conduct detailed studies to better understand the potential impacts of sea-level rise in their communities.

The analysis presented here provides an initial estimate of the impacts of sea-level rise along the California coast. More detailed assessments of local impacts and potential response strategies are needed. While the effects of sea-level rise, responses, and threatened resources must all be evaluated at a local level, broader regional effects must also be incorporated into final protection strategies.

Our analysis was hindered by inadequate data on existing coastal structures. Existing levees and other flood defenses should be surveyed, assessed, and cataloged.

The U.S. Congress passed the Water Resources Development Act of 2007, creating a National Levee Safety. The act requires the establishment and maintenance of an inventory of the nation's levees and inspection of all federally owned, operated, or constructed levees. This program should be fully funded and quickly implemented, and the information it compiles should be made readily available to residents, local government, and others.

Conduct further research focused on all vulnerable subpopulations, including children, elderly, homeless, physically disabled, and people with limited mobility (e.g., incarcerated residents and healthcare facility patients), accurately measuring and analyzing the potential human costs of sea-level rise and adaptation measures.

This analysis does not include various demographic groups that can be expected to have unique vulnerabilities to potential disasters. For pre-disaster, disaster response, and recovery efforts to effectively safeguard all Californians, further study is needed to identify all vulnerable populations and assess the unique vulnerabilities of each group.

Assess the environmental justice implications of potential mitigation measures, and develop strategies to effectively safeguard all communities.

The measures taken to adapt to sea-level rise must not distribute costs and benefits of protection in ways that place a disproportionate burden on the low-income households and communities of color who are most vulnerable to a potential disaster. The means of prioritizing protection measures must be analyzed with and held to the principles of environmental justice.

Natural ecosystems are at serious risk from sea-level rise, but are undervalued or ignored in traditional economic analyses. Improved methods for incorporating them into future studies are needed.

Wetlands are highly diverse ecosystems that provide a variety of goods and services, including flood protection, water purification, wildlife habitat, recreational opportunities, and carbon sequestration. Large tracts of wetlands along the California coast are vulnerable to sea-level rise. No satisfactory method for incorporating their environmental values has been developed, and we thus risk ignoring them when we make policy decisions. This would be a serious mistake. Additional work is needed to evaluate the costs and values of natural ecosystems.

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6.0 Acronyms and Abbreviations

ALACE	Airborne LIDAR Assessment of Coastal Erosion
ASFM	Association of State Floodplain Managers
BFE	Base flood elevation; elevation of floodwaters with an annual probability of 1%. Also referred to as the 100-year flood.
CALSIM	A computer simulation model of river basins developed by California's Department of Water Resources
CASCADE	Computational Assessments of Scenarios of Climate Change in the Delta Ecosystem; a suite of computer models of the hydrology and biology of California's Sacramento/San Joaquin river delta developed by the US Geological Survey
C-CAP	Coastal Change Analysis Program, a NOAA initiative
CCC	California Coastal Commission
CCSM	Community Climate System Model

CNRM	Centre National de Recherches Météorologiques (France's National Center for Meteorological Research)
DEM	Digital Elevation Model, a digital database of land surface elevations
DFIRM	Digital Flood Insurance Map, electronic maps and databases published by FEMA
FEMA	Federal Emergency Management Agency
GFDL	Geophysical Fluids Dynamics Laboratory
GIS	Geographic Information System
HAZUS	Hazards U.S. Multi-Hazard, a computer model for estimating damages from natural disasters
IFRCC	International Federation of Red Cross and Red Crescent Societies
IFSAR	Interferometric Synthetic Aperture Radar
IPCC	Intergovernmental Panel on Climate Change
LIDAR	Light Detection and Ranging, a remote sensing technology used to collect terrain elevation data
MGD	million gallons per day
MHHW	Mean higher-high water
MHW	Mean high water
MHWS	Mean high water springs
MIROC	The Model for Interdisciplinary Research on Climate
MLLW	Mean lower-low water
MLW	Mean low water
MSL	Mean sea level
MTC	Metropolitan Transportation Commission
MW	Megawatt
NASA	National Aeronautics and Space Administration
NAVD88	North American Vertical Datum of 1988; modern reference system for measuring heights above the earth's surface
NCAR	National Center for Atmospheric Research
NPDES	National Pollutant Discharge Elimination System

NGVD29	National Geodetic Vertical Datum of 1929; a reference system for measuring heights above the earth's surface, superseded by NAVD88
NOAA	National Oceanic and Atmospheric Administration
NPDES	National Pollutant Discharge Elimination System; an EPA program to track and regulate pollutants discharged to surface waters of the United States
NRC	National Research Council
NWI	National Wetlands Inventory, a geographic database of US wetlands published by the US Fish and Wildlife Service
PCM	Parallel Climate Model
PCS	Permit Compliance System; an EPA database of licensed discharges to the surface waters of the United States
PIER	Public Interest Energy Research
PWA	Philip Williams and Associates
SRES	Special Report on Emissions Scenarios
TWL	Total water level
U.S. EPA	United States Environmental Protection Agency
USACE	United States Army Corps of Engineers
USFWS	United States Fish and Wildlife Service
USGS	United States Geological Survey
WCED	World Commission on Environment and Development

CITY OF OXNARD CALIFORNIA

LAND USE APPLICATION FORM

APPLICATIONS ARE ACCEPTED BY APPOINTMENT ONLY - PLEASE TYPE OR WRITE LEGIBLY

Type of Permit Requested

- ☐ Annexation
- ☐ Coastal Admin Modification to CDP, DRP
- ☒ Coastal Development Permit (CDP)
- ☐ CBD Design Review Permit
- ☐ Development Design Review Permit (DDR)
- ☐ Final Parcel Map
- ☐ Final Subdivision Map
- ☐ General Plan Amendment
- ☐ Lot Line Adjustment
- ☐ Major Modification to SUP or PD
- ☐ Minor Modification to SUP or PD
- ☐ Planned Development Permit (PD)
- ☐ Pre-Application
- ☐ Special Use Permit (SUP)
- ☐ Specific Plan Review/Amendment
- ☐ Tentative Parcel Map
- ☐ Tentative Subdivision Map
- ☐ Zone Change
- ☐ Zone Variance
- ☐ Other _____

Description of Proposed Project

(Include type of development, number of residential units, number of affordable units/request for payment of in-lieu fee, parcel size, square feet of building area, etc. If this application is for a modification, describe the requested change. Attach more pages if required.)

Periodic removal of sand barriers which accumulate and obstruct the proper flow into the ocean from the saltwater discharge system identified as Ventura County Assessor's Parcel ID#183002301. Periodic fence repairs as required for safety and security purposes. There is no new development or changes to structures.

Property Information

Name of Project: Sand barriers/fence repairs (optional)

Property Location: 393 North Harbor Blvd. Oxnard, Ca. 93035

Assessor's Parcel Number(s): 183002301 (9.34 ac.) Beach area with saltwater discharge from power plant only (no other structures)

Additional Info: Maintenance only, no new development

Current Zoning: EC Proposed Zoning: _____

Current General Plan: _____ Proposed GP: _____

Property Owner Information

Name: NRG California South LP

Address: 211 Carnegie Center Princeton, NJ 08540

Phone: 415-627-1639

Email: sean.beatty@nrg.com

NRG California South LP is

I hereby certify that I am the owner of record of the subject project property(ies) described on this application and that I approve of the action requested herein.

Signature: S P Beatty

For additional property owners & properties, please use the back of this form and additional sheets if necessary.

Designated Agent (Attorney-in-Fact)

Designation of Agent (Attorney-In-Fact)

I, NRG California South LP *(property owner)*

hereby designate Scott Warnock *(agent) as the Attorney-in-Fact for the Property Owner for all purposes of processing this application with the City of Oxnard.*

Signature: S P Beatty

For multiple property owners, use additional Land Use Application forms.

Primary Contact/Designated Agent (Attorney-in-Fact)

Name: Scott Warnock

Address: 393 North Harbor Blvd. Oxnard, Ca. 93035

Tel: 805-984-5217 Fax: 805-984-5295

Signature: Scott Warnock

Email: scott.warnock@nrg.com

Other Persons to be Notified

_____

_____

_____

_____

_____

OFFICE USE ONLY

Fees	Amount
Permit Fee	
Env. Fees	
Total	
Verified By	

Date Received _____
Permit No. _____
Env Det./No. _____
Final Action _____
Expiration Date _____

Related / Concurrent Permits

_____

_____

Rec'd By _____
Assigned to: _____

Mandalay Generating Station
393 North Harbor Blvd.
Oxnard, Ca. 93035

May 4, 2015
Ashley Golden
Planning Manager
City of Oxnard Planning Division
214 South C Street
Oxnard, Ca. 93030

Attention: Chris Williamson, Principal Planner

Re: Coastal Development Permit

Dear Ms. Golden,

Please see attached Land Use Application, Project Questionnaire, and other applicable documents for a Coastal Development Permit to replace Coastal Emergency Permit (CEP) #PZ 15-000-17 dated April 16, 2015. The CDP is being requested for the maximum allowable period of five (5) years. Included is the required permit fee deposit of $7,851.58.

For clarification, the permit title 'new development or change to an existing development requiring construction' is not applicable. This application is submitted for the necessary maintenance of the existing outfall location. We are providing all documents that apply to our continued maintenance only associated with Ventura County Assessor Parcel #183002301 that is located west of the NRG Mandalay Generating Station, 393 North Harbor Blvd., Oxnard, CA 93035. We respectfully request that the CDP utilize the same language as the CEP and the previous City standing authorization, under an exempt status, since there are no changes. We also request that all items indicated by asterisk* on the Discretionary Permit Submittal Document be provided by the City as needed.

Our review has determined that the following documents are not applicable since we are doing maintenance and not requesting a new development or changes to the only structure, within the subject parcel, which is a permitted power plant cooling water discharge to the Pacific Ocean:

1. Engineering Site Plan Items 1-18
2. Floor Plan Items 1-6
3. Building Elevations Items 1-13
4. Conceptual Landscape Plan Items 1-12
5. Colored Elevations Items 1-3
6. Preliminary Drainage Report Items 1-4
7. Post-Construction Storm Water Quality Report Items 1-4
8. Materials Sample Board Items 1-4

Also, from our review a Hazardous Waste Site Affidavit is not required, because the proposed maintenance is not on a site that is included on any of the hazardous waste lists compiled by the State Office of Planning and Research (OPR).

Access to the work site, parcel 183002301, is provided along the beach road as noted on the maps to the cooling water discharge area. Every reasonable precaution will be taken to avoid impact to dune grass and other habitat in traveling to and from the work site. The periodic relocation of sand that naturally berms and partially or completely blocks the permitted power plant cooling water discharge is required for the safe operation of the power plant and to prevent ponding of the discharge laterally on the beach to the north and south. Berming can occur within days depending on the offshore littoral current, storm surge, and other naturally occurring events. The power plant does not operate continuously but as needed and may be called on to run at any time which requires the discharge channel to be open to the ocean. It is also requested that the CDP allow periodic repairs to fencing for public safety and power plant security.

A current title report is also required with this application and has been contracted to Fidelity National Title Company. A preliminary title report is attached with the final report forthcoming when available.

You made a separate request for a biologist to provide a report for the area surrounding the saltwater discharge structure and maintenance area. The report has been contracted to AECOM and will be provided as soon as available.

We authorize the City or its designated contractor to install on-site signage for public notification of public meetings if required. The City or its authorized contractor may enter, with advanced notification, the subject property to install, maintain, and remove such signs. The best access point to the subject parcel is through the main entrance to Mandalay Generating Station, 393 North Harbor Blvd., Oxnard, CA. 93035.

We appreciate your attention to this CDP. Please contact George Murr (805)212-2853 or Scott Warnock (805)276-1765 with questions or for access to the property.

Best Regards,

Thomas Di Ciolli
Plant Manager
NRG Mandalay Generating Station

CITY OF
OXNARD
CALIFORNIA

PROJECT INFORMATION QUESTIONNAIRE

PLANNING DIVISION
214 SOUTH C STREET, OXNARD, CA 93030
(805) 385-7858
FAX: (805) 385-7417

PROJECT INFORMATION QUESTIONNAIRE

General Information

1. Applicant (name, address, e-mail, phone, fax):
NRG Mandalay - Attention: Scott Warnock (email:scott.warnock@nrg.com)
393 North Harbor Blvd.
Oxnard, Ca. 93035
Phone: 805-984-5217 Fax: 805-984-5295

2. Project Location (address and cross streets):
393 North Harbor Blvd. (Between 5th St. and Gonzales Rd.)
Oxnard, Ca. 93035

3. Project Name (if any):
Discharge Area Sand Relocation and Fence Repairs

4. Types of permits and/or applications being applied for at this time:
Coastal Development Permit

5. Previous permits and/or land divisions approved for this site:
Exempt Dev. Section 37-5.3.2 and Coastal Emergency Permit PZ 15-000-17

6. Has a soils exploration, market study, traffic study, parking study or other report been prepared which would aid in the evaluation of this project? If so, list here and attach:
NO

7. Is this a modification or renewal of a previously approved project? If yes, list permit number here. If renewal, how long is renewal time period?
YES - Emergency Permit PZ 15-000-17 (CDP for 5 years)

If yes, have changes been made in the plans? NO

Project Description

1. Proposed use of site: Discharge of saltwater from power plant to ocean per NPDES Permit

2. Size of site (acreage and square feet): Parcel size: 9.34 acres / 406,900 sf
Estimated size of area where maintenance occurs is < one acre.

3. Number and type of structures or units to be constructed: Not applicable

4. Total square footage of each structure(s) Not applicable

5. Height and number of stories of proposed structures: Not applicable

(If more than one building, attach a table with the information in questions 3, 4 & 5.)

6. Percentage of land coverage, by type, for total site:
   a. Building 0%
   b. Paving 0%
   c. Interior Yard Space 0%
   d. Landscape Areas 0%
   e. Other 100%

7. Project phasing and anticipated time schedules (including phasing map):
   Periodic over 5 year permit time allowance

Existing Conditions

1. Existing land use: Beach and saltwater discharge from power plant
   Existing zoning: EC

2. Adjacent zoning and land uses:
   North: RC - Coastal Recreation
   South: RC - Coastal Recreation
   East: EC - Coastal Energy Facility
   West: RC - Coastal Recreation

3. Describe any special characteristics of the soil, topography, native trees, wildlife habitats, historic or aesthetic aspects of the site: (Attach separate sheet if necessary.)
   Beach, sand dunes, some natural vegetation, and near various avian habitat.

   a. Does the site include more than one existing parcel? If so, how many? NO
   b. Will the project require subdivision? NO
   c. What is the existing tract or parcel map number? 183002301
   d. To what depth will the site be excavated ? Minus 0.8 ft. at low tide

6. Will any of the following effects occur if the project is constructed? Please elaborate (on additional pages) on items checked "Yes."

	Yes	No	
a.	☐	☒	Use or disposal of potentially hazardous materials, toxic substances, flammable, or explosives
b.	☐	☒	Creation of dust, ash, smoke, fumes or odors
c.	☐	☒	Substantial or unusual demands for fossil fuels or electricity
d.	☐	☒	Substantial demand for municipal service (police, fire, water, sewage, etc.)
e.	☐	☒	Creation of significant amount of solid waste or litter
f.	☐	☒	Noise generation

7. Provide the following information on a separate sheet of paper:
   a. List all raw materials used, stored, transported or otherwise handled, including hazardous materials, solvents and catalysts.
   b. List all products provided, stored, manufactured, generated or otherwise handled, including waste, hazardous waste products and other byproducts.
   c. Describe the manufacturing processes, including the sizes and types of machinery used, and utilization of heat, microwaves, electrolysis, extreme pressures or liquefied gases.
   d. Describe the method of delivery, handling and disposal of raw materials, finished projects, wastes and hazardous wastes.

List proposed actions and features of the project that will minimize environmental impacts (e.g., noise, air pollution, water and energy conservation, etc.) (Attach separate pages if necessary.)

Movement of sand in designated areas and along designated paths to the cooling water discharge structure at the beach.

Fence repairs along the property line and jettys for safety and security purposes.

Certification

I hereby certify that the statements furnished above, and in the attached exhibits, present the data and information required for this initial evaluation to the best of my ability, and that the facts, statements and information presented are true and correct to the best of my knowledge.

5-5-2015
Date

NRG Energy
Signature
Company/Firm

NRG Mandalay Generating Station

RANCHO EL RIO DE SANTA CLARA O' LA COLONIA
T.2N, R.23W, S.B.B.&M

Tax Rate Area
03040

183–02

1" = 300'

Bk. 138
01
BEACH
ROAD
HARBOR
BLVD.
147
148
146
145
53RS86
19PM7
Par. A
Par. B
Par. C
022
023
021
1 9.34Ac.
2 36.00Ac.
SBE 1114–56–1 Par. Par.1
1 4.58Ac.
1 26.38Ac.
SBE 148–56–56 Par. 4
2 8.58Ac.
SBE 1114–56–1 Por. Par. 1
3 10.61Ac.
3 16.11Ac.
SBE 148–56–56 Par. 5
(A) L.L.A. 970074360
(B) L.L.A. 970074360
(C) L.L.A. 970074360
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N88°40'54"W
1602.67'
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3559.32'
1956.72'
1143.16'
N11°53'38"E
1359.03'
N88°47'4"W
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299.89'
474.34'
1391.47'
N66°25'10"E
314.85'
300.06'
N23°34'50"W
476.34'
L=369.35'
R=400'
1125.21'
1264.76'
265.97'
N11°53'38"E 43.58'
N66°38'22"W
N66°59'0"E
257.24'
N66°14'49"E
83.58'
N63°58'46"W
N68°39'31"E
163.67'
N63°58'46"W
N66°29'12"E
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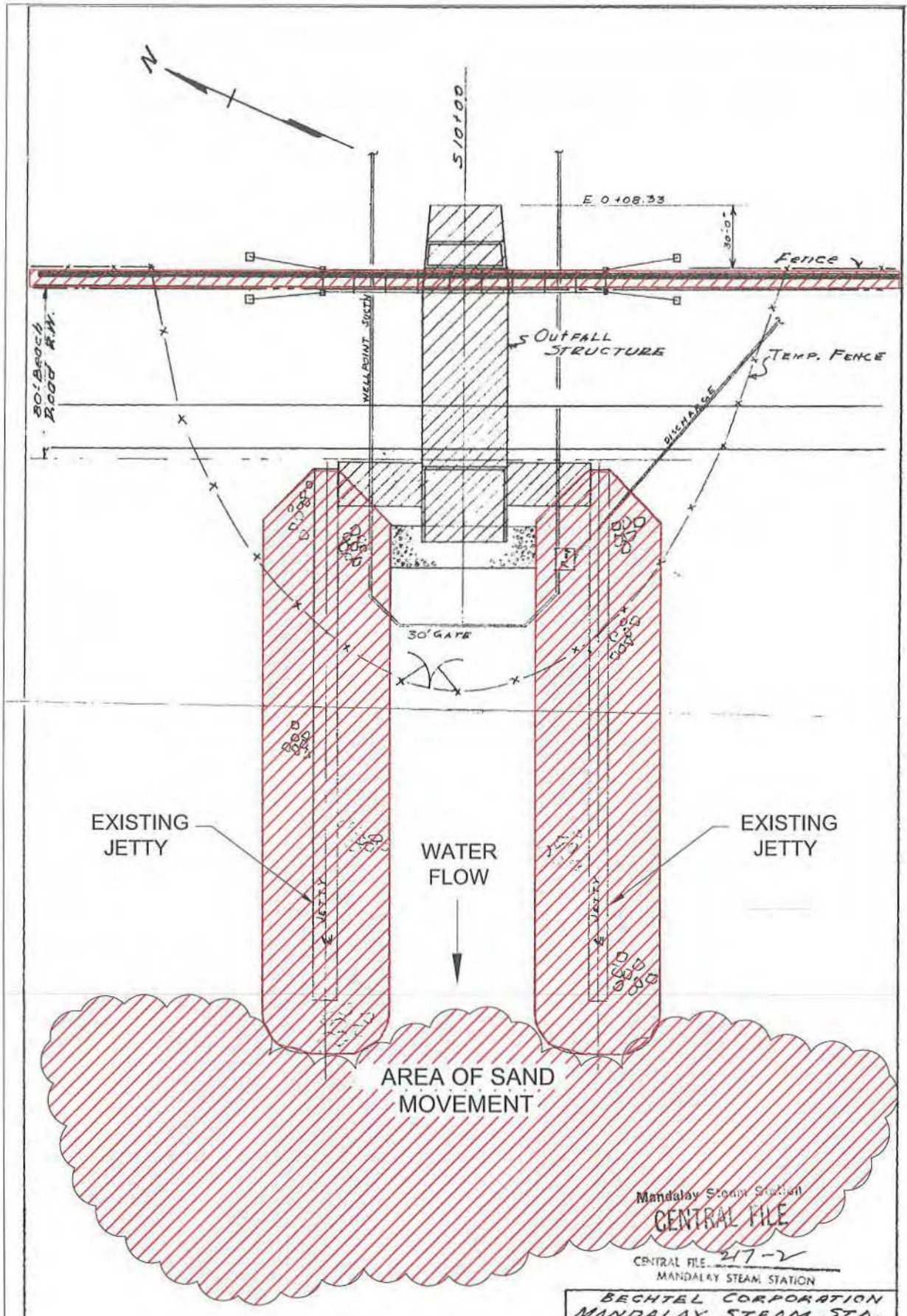
N66°25'10"E
N23°34'50



Mandalay Steam Station
CENTRAL FILE
217-V
CENTRAL FILE
MANDALAY STEAM STATION

BECHTEL CORPORATION
MANDALAY STEAM STA.
OUTFALL STRUCTURE

FILE: SK2640-F-101

REV. 1 BY: E.L.H.	BY: M.R.	SK2640-F-101
	DATE: 2015	

ATTACHMENT 1

FIGURES

Figure 1. Proposed Limits of Activity

April 2015	Mandalay Generating Station Outfall Channel Mantenance Project

ATTACHMENT 2

SITE PHOTOGRAPHS

Photograph 1: South side of MGS outfall channel. View to west.

Photograph 2: North side of MGS outfall channel. View to west.

All photos April 24, 2015

Photograph 3: North side of MGS outfall channel, including sand relocation area. View to east.

Photograph 4: MGS outfall channel, with sand in foreground blocking normal outfall flow. View to east.

Photograph 5: MGS outfall channel deflected southward by accumulated sand. View to south.

Photograph 6: Alignment of Beach Road, southern access route. View to south, away from outfall channel.

Photograph 7: Alignment of Beach Road, southern access route. View to north from MGS, towards outfall channel.

Photograph 8: Alignment of Beach Road, southern access route. View to north, away from outfall channel.

All photos April 24, 2015

Photograph 9: Alignment of Beach Road, southern access route. View to south from MGS gate, towards outfall channel.

Photograph 10: View to from northern MGS fenceline towards fenced snowy plover nesting habitat on McGrath State Beach, distance of approximately 200 feet. View to west.

All photos April 24, 2015

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May 7, 2015

Mr. George Murr
Station Environmental Support
Mandalay Generating Station
393 Harbor Blvd.
Oxnard, Ca. 93035

Re: **Biological Technical Letter – Mandalay Generating Station Outfall Channel Maintenance Project**

Dear Mr. Murr:

Per your request, AECOM Technical Services, Inc. (AECOM) has conducted an analysis of impacts to biological resources associated with proposed maintenance and sediment movement activities at the outfall canal of the Mandalay Generating Station (MGS) in Oxnard, California. AECOM understands that a biological evaluation has been requested by City of Oxnard, to inform the City's issuance of a Coastal Development Permit for the project. The study methods and conclusions are presented below.

INTRODUCTION AND BACKGROUND

The MGS is an existing, natural gas–fired steam electric generating facility consisting of two conventional steam turbine units and one gas combustion turbine unit with a combined generating capacity of 560 MW. The facility includes a cooling water intake/outfall system, which provides cooling water to the two conventional steam turbine units. Cooling water is obtained from Channel Islands Harbor via the Edison Canal, a 2.5 mile manmade canal specifically constructed to provide cooling water to the station, and is returned to the Pacific Ocean through a single shoreline outfall. The outfall channel is approximately 100 feet wide, and is lined on both sides with rock riprap for stability and chain link fencing for safety. Once flows exit the stabilized portion of the outfall channel, water reaches the Pacific Ocean by cutting its own path through the beach sand. This path is affected by natural processes of sand deposition and movement, and can at times become blocked or diverted laterally along the beach. When this occurs, flows do not reach the ocean and flooding of the beach areas adjacent to the outfall channel can result.

Based on information provided by MGS personnel, AECOM understands that MGS has been performing routine maintenance of the flow path from the outfall channel to the Pacific Ocean through mechanical sand movement on an as-needed basis throughout the life of the facility. This work has essentially entailed moving sand to create a straightened channel, affording outfall flows a direct path to reach the ocean. In the past, the City of Oxnard, which has land use jurisdiction over the MGS facility site, has considered the periodic flow channel maintenance work to be exempt from Coastal Development Permit requirements. However, AECOM understands that the City's position regarding the exempt status of these activities has changed, and that MGS is seeking a Coastal Development Permit from the City that would authorize sand movement and channel straightening on an as-needed basis in the future. This letter report summarizes the expected effects of these activities on biological resources that occur in the work area, and recommends measures to minimize adverse effects.

PROJECT DESCRIPTION

The proposed project entails the mechanical movement of accumulated beach sand to straighten the outfall channel of the MGS, on an as-needed basis in a manner similar to what has been performed for many years. The work would be performed by a single operator using a front-end loader or similar piece of equipment, and would be confined to the smallest practical disturbance footprint. During the proposed work activities, any sand blocking the direct path of water from the MGS outfall to the ocean would be removed by the equipment bucket to form a straight channel. The material would be placed on the sides of the channel, above the high tide line, and leveled to prevent the appearance of piles or topographic disruption to the beach. Side channels or relict flow paths would not be filled, except as needed to ensure flows adopt the newly created primary channel, and would be allowed to fill with sand naturally.

The estimated maximum limits of disturbance would encompass an area extending approximately 150 feet laterally to each side of the outfall channel, and extending towards the ocean approximately 175 feet from the seaward end of the rock riprap along the edges of the outfall channel. The proposed limits of disturbance, including site access (see below) are illustrated graphically on Figure 1 in Attachment 1. The aerial photograph on this figure was taken shortly after a prior outfall channel maintenance effort, and helps to illustrate the nature of the activities proposed. The newly straightened channel extending directly from the outfall to the ocean is visible, and the pattern of displaced sand adjacent to the channel is evident.

During the proposed maintenance work, activities would be limited to movement of existing beach sand, and no cut or fill material would be imported or exported to/from the work site. Work would be confined to periods of low tide, to minimize the need for equipment to operate in wetted areas. In the event that the outfall channel has been blocked and ponding on the beach has occurred, effort will be made to avoid placing sand into ponded areas.

Access to the work location would be achieved by travelling from one of the two southern gates of the MGS along Beach Road, a roadway that is no longer maintained and has become covered with sand deposits (see Figure 1 in Attachment 1 and photos 6 through 9 in Attachment 2). Upon reaching the outfall channel, equipment would travel along existing vehicle travel paths along the outer channel edges (outside the existing safety fencing) to access the mouth of the channel. Access along one or both banks may be necessary, depending on beach sand patterns and the location of the flow channel along the beach.

EXISTING CONDITIONS

Existing biological conditions within the proposed work location were assessed based on a review of publicly-available literature and a reconnaissance-level biological survey of the site conducted by AECOM biologist Christopher Julian on April 24, 2015.

Physical Setting

The MGS outfall channel is situated along a long, sandy beach and dune system. The slope of the beach is moderate, and the beach is approximately 100 to 200 feet wide during the summer months. The coastline itself is straight, extending from northwest to southeast, with the Pacific Ocean situated to the west. Hummocks of sand dunes separate the beach area from the MGS facility, and also shield

the proposed access routes along Beach Road from the beach. The dunes vary in height, with the highest being approximately 30 feet above mean sea level.

Vegetation

Within the area of proposed disturbance and along the proposed access routes, the predominant habitat is unconsolidated sand. Vegetation is largely absent, except in areas where plants have colonized the adjacent dunes. Vegetation and habitat occurring within and adjacent to the work site and access routes is described below.

Sandy Beach

Sandy beach occurs throughout the proposed disturbance footprint, and is the primary type of habitat that would be affected by the proposed activities. Sandy beaches generally lack vegetation, due to the loose and fluctuating nature of the sandy substrate. Although they are separated from the beach by the adjacent sand dunes, the access routes along Beach Road are more similar to the sandy beach habitat found near the outfall channel than to the habitats found on the adjacent dunes. The access road is generally flat and unvegetated, rather than sloped and occupied sand dune plant species (see below). Despite the absence of vegetation, sandy beaches can provide valuable habitat features for some species. Beach sand supports an abundance invertebrate fauna, and these small animals form the prey base for a variety of shorebirds.

Dune Mats (*Abronia latifolia-Ambrosia chamissonis* Herbaceous Alliance)

Dune mats do not occur within the proposed disturbance footprint, but are present in areas adjacent to the northern access route along Beach Road. This vegetation community is dominated by sand verbena (*Abronia latifolia*) and/or beach bur (*Ambrosia chamissonis*) mixed with other perennial herbs, grasses, and low shrubs to form a low canopy. Emergent shrubs of coyote brush (*Baccharis pilularis*), coastal bush lupine (*Lupinus arboreus*), and others may be present (Sawyer *et al.* 2009). In the vicinity of the northern access route, this community inhabits the inner dunes inland from the leading edge of the beach. These areas are rarely impacted by saltwater overwash during storms (ESA 2003). Dune mats are identified by the California Department of Fish and Wildlife (2010) as a sensitive natural community.

Ice Plant Mats (*Carpobrotus edulis* or Other Ice Plants Semi-Natural Herbaceous Stands)

Ice plant mats do not occur within the proposed disturbance footprint, but occur on both sides of Beach Road along the southern access route. Ice plant mats are an invasive, nonnative vegetation community dominated by ice plant species. A total of eight ice plant species occur in California, and all can form this community. In the vicinity of the MGS facility, ice plant mats are dominated by freeway ice plant (*Carpobrotus edulis*), a ground-hugging, perennial succulent that forms impenetrable mats covering large areas. Freeway iceplant has been identified as a highly invasive species by the California Invasive Plant Council (Cal-IPC 2006). Stands of this species are common on coastal dune systems and along bluffs on the southern California coast (Sawyer *et al.* 2009). The value of ice plant mats for native wildlife is generally low, as ice plants stabilize the substrate and render it unsuitable for species adapted to the loose sand of dunes and beaches.

Sensitive Species

The proposed disturbance area has the potential to support certain types of sensitive biological resources. These plants and animals are discussed below.

Western Snowy Plover (*Charadrius nivosus nivosus*)

The western snowy plover is a federally listed threatened species and a California Species of Special Concern. The beaches in the project area support colonies of nesting western snowy plovers, and the stretch of beach in the project vicinity, including the proposed work site, is within designated critical habitat for the species. This designation indicates that the U.S. Fish and Wildlife Service regards the beach to be essential to the conservation of the snowy plover. The beaches and sand dunes within Mandalay State Beach and McGrath State Beach in the immediate vicinity of the project site support both wintering populations and breeding populations of this species (California State Parks 2013), and it is therefore probable that the species is likely to breed and winter in the proposed maintenance area as well. During a biological reconnaissance of the site vicinity on April 24, 2015, a fenced enclosure with snowy plover protective signage was observed around a dune habitat on the southern edge of McGrath State Beach. This location is approximately 750 feet north of the proposed disturbance limits of the outfall channel maintenance work, but is within approximately 200 feet west of the northern gate where maintenance vehicles would enter Beach Road if using the northern access route. Though no snowy plovers or other nesting birds were observed during the field reconnaissance, it is presumed that this area is likely to support nesting snowy plovers.

California Least Tern (*Sterna antillarum browni*)

The California least tern is a federally and state-listed endangered bird species. This species nests on relatively open beaches where vegetation is limited by tidal scouring and forages over open water. California least terns have been documented nesting in the vicinity of the project between the Santa Clara River Mouth and McGrath Lake, and on Ormond Beach between Ormond Beach Generating Station and Perkins Road (CDFW 2015). Because California least terns are known to nest in proximity to the project site, and because the project site contains suitable nesting habitat for this species, there is potential for California least terns to nest within the proposed disturbance area.

Nesting Birds

In addition to the protected birds described above, the sandy beach in the proposed disturbance area has the potential to support other common species of nesting birds. The unauthorized destruction of native non-game birds, including their eggs and nestlings, is a violation of federal and state law.

Rare Plants

A variety of rare plants are known to occur in the vicinity of the proposed project site, some of which are listed as threatened or endangered by federal and/or state agencies. Some examples include:

- Ventura Marsh Milk-Vetch (*Astragalus pycnostachyus* var. *lanosissimus*) – This federally and state-listed endangered plant occurs on coastal dunes and is known within 1 mile of the project site (CDFW 2015).

- Coulter's Saltbush (*Atriplex coulteri*) – This shrub occurs coastal sand dunes, and has been documented approximately five miles north of the project site (CDFW 2015).
- Salt Marsh Bird's Beak (*Chloropyron maritimum*) – This species is found in the upper areas of salt marshes, on beaches, and on alkali flats, and has been documented at McGrath State Beach (CCH 2015).
- Red (Sticky) Sand Verbena (*Abronia maritima*) – This species occurs on coastal dunes, and is known to occur on McGrath and Mandalay State Beaches (CCH 2015).
- Dunedelion (*Malacothrix incana*) – This plant occurs in coastal dunes, and has been reported occurring in the vicinity of the project site (California Coastal Commission 2009).

None of these sensitive plants were observed during the reconnaissance-level site visit conducted on April 24, 2015. However, because the project site contains suitable habitat for these species, and because they are known from the project vicinity, it is possible that they may occur within the project site in the future.

IMPACTS OF THE PROPOSED PROJECT

The project's impacts would be short-term and localized, and would be associated with the movement and operation of a single piece of construction equipment (front-end loader or similar).

Impacts Along Access Routes

Movement of the loader along Beach Road from the MGS facility to the work site could result in injury or mortality of plants and animals within the loader's path. However, based on field observations, vegetation along the access route is minimal and does not currently include sensitive plants. If sensitive plants are identified along access routes in the future, it is expected that these resources would be avoidable due to the availability of two alternative access routes from the MGS to the work site. Injury or mortality of vertebrates due to driving along the access routes is not expected, as these animals are generally mobile and would vacate the path of construction equipment. Bird nests or nestlings, which are not as mobile and would not be able to move to safety, could potentially be crushed by equipment. This type of impact is prohibited by federal and state law, and could feasibly be avoided by ensuring that the selected access path is free of nesting birds.

In addition to potential impacts resulting from contact with construction equipment, the use of Beach Road to access the work site could result in impacts to wildlife in adjacent habitats due to noise, dust, and human presence. These effects are expected to be minor due to the short duration of exposure, the fact that only a single construction vehicle would be used, and because Beach Road is isolated from the adjacent beaches by a dune system that would serve to block lines of sight and attenuate noise and dust.

All proposed access routes are currently unvegetated, and accessing the work site would not remove substantial amounts of vegetation. Further, any impacts along the proposed routes would be short-term and temporary.

Impacts Within the Proposed Area of Disturbance

Sediment management activities at the outfall channel would occur sporadically, on an as-needed basis, and the effects of these activities would be largely dependent on the season when activities occur. Potential impacts would be most severe during the bird nesting season (March 1 through September 30, based upon USFWS snowy plover restrictions), as there is potential for bird nests with eggs or nestlings to be present in the disturbance footprint during this period. These resources can be difficult to detect, and bird nests or nestlings could potentially be crushed by equipment if activities were to proceed in a nesting area. In addition, nests in adjacent habitat areas could be adversely affected if noise, vibration, dust, or human presence at the work site were to disrupt essential behaviors such as the incubation of eggs or the feeding or sheltering of young. The take of western snowy plover or other active bird nests is prohibited by federal and state law, and could be avoided through pre-construction surveys, worker environmental awareness, and presence of a biological monitor at the work site.

During the period outside the bird nesting season (October 1 through February 28), the severity of potential impacts would be significantly reduced. Birds would not be using the beaches for nesting during this period, and it is expected that any wildlife present would be mobile enough to vacate the work site. The limited extent of the area to be disturbed (see Figure 1 in Attachment 1) would allow wildlife to escape without necessitating long-distance movements or substantial deviations from normal behavior patterns. Birds may opt to avoid foraging within the outfall channel during maintenance activities, but would be expected to resume this activity once the work is complete.

Interstitial invertebrates living in the sand may be adversely affected by the proposed activities, which would manipulate the substrate and could crush or displace these animals. Because beach invertebrates are abundant and widely distributed, and due to the small area to be disturbed, the effects of short-term, localized disturbances to these species are not expected to be substantial.

CONCLUSIONS AND RECOMMENDATIONS

The following avoidance and minimization measures are recommended to reduce the project's effects on biological resources occurring within and adjacent to the work site and associated access routes:

- The disturbance footprint should be no larger than is necessary to complete the proposed activities.
- For each maintenance event, the least impactful access route (northern or southern route, along Beach Road) should be utilized. A qualified biologist should be consulted in making this determination.
- Prior to the operator beginning maintenance work, a qualified biologist should discuss with the operator the following: 1) sensitive resources with the potential to occur on-site; 2) the protection afforded these resources; 3) procedures to follow in the event that sensitive species are observed during the maintenance work; and, 4) penalties for non-compliance.
- Prior to conducting maintenance, a qualified biologist should survey the access routes and proposed disturbance footprint for rare plants. If found, these species should be avoided.
- If work activities occur during the western snowy plover nesting season (March 1 through September 30), a qualified biologist should survey the disturbance footprint and proposed

access routes, plus an appropriate buffer, for plover nests. The biologist should remain on-site during the proposed activities, and should have the authority to stop work if snowy plovers or other sensitive species are observed in or near the work site. If plover nests are present in the work site, the biologist should work with MGS personnel to develop an impact avoidance strategy. As required by the Endangered Species Act, the proposed work must be implemented in a manner that does not result in the take of western snowy plovers.

- If nesting birds are present in an area that would be affected by the project, work activities should be halted until a qualified biologist has determined that the nest has fledged. Destruction of birds, eggs, or nestlings is a violation of federal and state law.

With implementation of measures similar to those listed above, the proposed maintenance activities could be conducted in a manner that minimizes impacts to biological resources and maintains consistency with federal and state endangered species laws.

We are grateful for the opportunity to support the Mandalay Generating Station on this important project. Should you have any questions regarding this submittal, please contact me at 805-692-0626.

Sincerely,
AECOM

Christopher Julian
Principal Environmental Scientist

Attachments:
Attachment 1 – Figures
Attachment 2 – Site Photographs

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ATTACHMENT 1

FIGURES

Figure 1. Proposed Limits of Activity

April 2015	Mandalay Generating Station Outfall Channel Maintenance Project

ATTACHMENT 2

SITE PHOTOGRAPHS

Photograph 1: South side of MGS outfall channel. View to west.

Photograph 2: North side of MGS outfall channel. View to west.

All photos April 24, 2015

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All photos April 24, 2015

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All photos April 24, 2015

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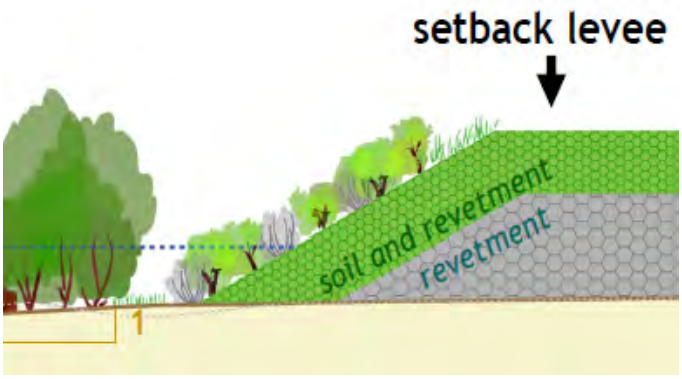
Photograph 10: View to from northern MGS fenceline towards fenced snowy plover nesting habitat on McGrath State Beach, distance of approximately 200 feet. View to west.

All photos April 24, 2015

Santa Clara River Parkway

Levee Setback Assessment of the Lower Santa Clara River, Ventura County, California - Implications for Flood Risk Management and Ecological Benefit



September 2011

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Acknowledgements:
Stillwater Sciences and cbec would like to thank the Ventura County Watershed Protection District for providing access to the watershed HSPF hydrologic model and for information pertaining to flood dynamics along the lower Santa Clara River. We would also like to thank Dr. Lee Harrison (NOAA NMFS) for providing a thorough review of the MIKE FLOOD modeling effort conducted for this project.

Preferred citation:
Stillwater Sciences. 2011. Santa Clara River Parkway: Levee Setback Assessment of the Lower Santa Clara River, Ventura County, California. Prepared by Stillwater Sciences and cbec, Inc. for the California State Coastal Conservancy.

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APPENDICES

Appendix A. Levee Setback Modeling Technical Memorandum (cbec Inc. 2011)

1 INTRODUCTION

1.1 Background and Study Context

The Santa Clara River flows from the San Gabriel Mountains in Los Angeles County, through Ventura County, and eventually into the Pacific Ocean near the City of Ventura (Figure 1-1). Since the onset of Eurpoean American settlement over 150 years ago, the contributing watershed and mainstem floodplain have experienced sustained impacts from both development and agricultural influences. Over the past several decades, the lower 64 km (40 miles) of the Santa Clara River and its floodplain have been significantly altered due to flood protection infrastructure (including reinforced levees), water diversions and flow regulation, roads, agriculture, aggregate mining, and urbanization. The flood protection structures have constrained or disrupted natural geomorphic and hydrologic processes, often causing riparian and aquatic habitat loss or degradation. Despite the historical alterations to the riparian system, the lower Santa Clara River (LSCR, reach downstream of the Los Angeles County Line) presents a unique opportunity to conserve and restore riparian functions and ecosystems compared with other coastal southern California rivers, most of which are highly degraded. As the watershed is one of the least altered rivers in southern California, it continues to support a variety of natural aquatic and terrestrial communities and native species. It also provides a regionally important north-south corridor between protected terrestrial wildlife areas in the southern California coastal ecoregion, and the river itself provides an important aquatic habitat linkage from the coast and estuary to upstream habitats in the mainstem channel and tributaries.

The Santa Clara River Parkway project, which is lead by the California State Coastal Conservancy (Coastal Conservancy) and The Nature Conservancy (TNC), seeks to ameliorate historical ecological impacts in the LSCR and conserve existing riparian habitats. The primary goal of the Parkway project is to create, protect and restore 64 km (40 miles) of continuous river and floodplain corridor from the Los Angeles County Line downstream to the Santa Clara River mouth. Other goals of the Parkway project are to: 1) conserve and restore aquatic and riparian habitat for native species, 2) provide enhanced flood protection, and 3) provide public access and environmental education within the Parkway. The Parkway is being created through the acquisition of river channel, floodplain, and agricultural lands that are vulnerable to flooding under current or restored conditions (i.e., without levees) and that do not contain vital infrastructure, and conversion of those lands back to riparian and upland habitats. Land acquisition is being conducted on a willing seller basis and is focused on the lower river, where a number of parcels have already been acquired (Figure 1-2).

Figure 1-1. The Santa Clara River watershed.

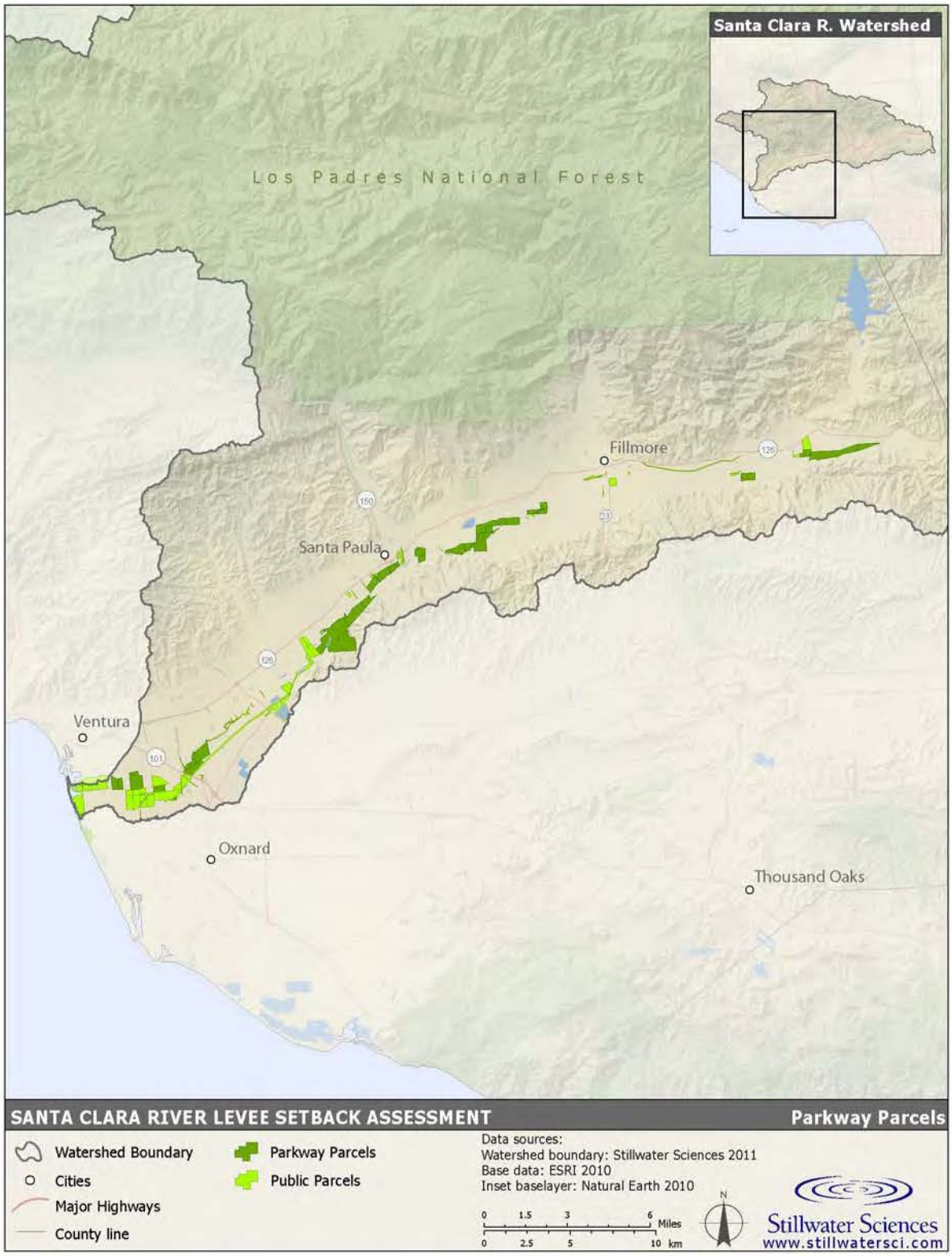

Figure 1-2. Santa Clara River Parkway parcels.

The Santa Clara River Parkway Floodplain Restoration Feasibility Study (Stillwater Sciences 2008) was undertaken to assist with the acquisition, management, and eventual restoration of lands within the Parkway. Through a thorough investigation of historical and contemporary geomorphic and ecological functioning of the Santa Clara River and floodplain, the Feasibility Study identified levee removal and setback as one of the primary restoration strategies that should be considered for the Parkway in order to improve riparian habitat conditions and geomorphic processes affected by channel-confining levees. However, the Feasibility Study also recognized a number of important uncertainties associated with levee setback and removal that would need to be resolved before pursuing the restoration strategy further. One of the primary uncertainties identified was the local change in channel hydraulics and sedimentation resulting from a parcel-based strategy of levee removal or setback. To resolve this uncertainty, the Feasibility Study recommended modeling several flood flows and resultant water depths and velcities as a guide to erosion and deposition trends to ascertain any potentially negative upstream or downstream effects. In addition, a regional-level evaluation using hydraulic models was recommended for the Parkway area to assess the overall potential flood management benefit of this restoration strategy. The attached levee setback modeling (Appendix A) and this levee setback assessment were undertaken in direct response to the recommendations in the Feasibility Study.

1.2 Goals and Objectives

The overall goal of the levee setback modeling and assessment study presented here is to elucidate key hydraulic, geomorphic, flood risk, and ecosystem benefits and uncertainties associated with levee setback and removal along the LSCR. More specifically, the main goal is to characterize the effects of a range of levee setback scenarios on: (1) flood control/risk management; (2) local, upstream, and downstream hydrogeomorphic conditions; (3) riparian vegetation and habitat conditions; and (4) Parkway acquisition efforts.

The objectives of this modeling and assessment study are as follows:

- Development of a hydraulic model of the LSCR capable of gaming the impacts of levee set-back on flow hydraulics at a resolution necessary to assess potential flood-control, geomorphic, and ecological benefits.
- Modeling flood flow water depths and velocities along the LSCR with existing levees and for a suite of levee setback scenarios under current and hypothetical future hydrologic conditions (as impacted by climate change)
- Assessing potential flood-control, geomorphic, and ecologic benefits associated with each levee setback scenario through analysis of modeling results.

For this study, we investigated the potential benefits associated with levee setbacks during an extreme flood event (100-year flood, used to calibrate against the existing FEMA hydraulic modeling of the LSCR) and a more frequently occuring flood event (25-year flood, similar to the peak flow during the January 2005 storm). We also used the most up-to-date knowledge regarding climate change impacts on sea level and flood flows to assess the impact levee removal could have on flood flow water depth and velocity several decades into the future.

1.3 Study Area

The Santa Clara River Parkway Floodplain Restoration Feasibility Study and this levee setback modeling and assessment study focus on the lower 64 km (40 miles) of the mainstem Santa Clara River downstream of the Los Angeles/Ventura County Line (see Figure 1-1). The Santa Clara River originates on the northern slopes of the San Gabriel Mountains in Los Angeles County (approximately 2,700 m [9,000 ft] above mean sea level) and flows through the Santa Clara River Valley and the Oxnard Plain in Ventura County, finally emptying into the Pacific Ocean near the City of Ventura (Figure 1-1). The river has one of the largest watersheds on the southern California coast, draining an area of approximately 4,140 km^2 (1,600 mi^2).

Consistent with other rivers in the region, the Santa Clara River watershed experiences highly variable annual rainfall and peak river flows. Generally, flows in the river are relatively low: 75% of the time flows are less than 4.2 m^3s^{-1} (150 cfs) at the Montalvo gage (approximately 7.2 km [4.5 miles] upstream of the mouth) and 50% of the time flows are less than 0.3 m^3s^{-1} (10 cfs) (URS 2005). However, large peak flows associated with winter storm events exceed 2,800 m^3s^{-1} (100,000 cfs) once every 10 years on average (URS 2005), generally during years with a strong positive El Niño - Southern Oscillation (ENSO) signal (or, ENSO years). In general, ENSO years are characterized by relatively high rainfall intensities and higher annual peak flow magnitudes than in non-ENSO years (Cayan *et al*. 1999, Andrews *et al*. 2004). Flows in the mainstem Santa Clara River can increase, peak, and subside rapidly in response to high intensity rainfall, with the potential for severe flooding under saturated or near-saturated watershed conditions. Between winter rainfall events in wet years, the river may exhibit continuous baseflow to the ocean from residual watershed discharge; in dry years, flow may be intermittent in the mainstem channel. During the dry summer season, flows in the mainstem and tributaries are intermittent or non-existent, depending primarily on areas of rising groundwater or inflows from dam releases or other anthropogenic sources, such as irrigation runoff and treated wastewater effluent.

The Santa Clara River watershed is located within the San Andreas Fault system, a geologically active region that forms the dynamic boundary between the Pacific and North America tectonic plates. Rapid tectonic uplift rates and relatively erodible bedrock lead, combined with frequent high-intensity storm events, lead to extremely high rates of hillslope sediment production. As would be expected, sediment transport processes in the Santa Clara River are dominated by extreme events associated with the river's highest flows. For instance, an estimated 55% of the roughly 57.6 million tonnes (63.5 million tons) of sediment that passed the USGS gage at Montalvo near Highway 101 between 1968 and 1975 was transported during high flows in just two days during two separate floods of record in January and February 1969 (Williams 1979).

In planform, the LSCR is characterized by a wide, relatively straight floodway with one or more low-flow channels that are reconfigured after each flood event. The full mainstem channel bed is occupied only during higher magnitude floods, typically a 5-year event or larger. Erosion of alternate outer banks of the active floodway in some reaches following the large floods in January and February 2005 suggests that the entire floodway of the contemporary lower river behaves in a manner similar to a broad, single thread meandering channel at very high flows. As floods recede, the river becomes more braided in character, with multiple flow courses. There is insufficient perennial flow to retain multiple flowing channels in a majority of the LSCR and, in general, a single dominant channel defines the channel thalweg.

Since the 1950s, a total of 53 levees have been constructed along over 40 km (25 miles) of the LSCR (URS 2005), amounting to approximately 33% of the total river bank length in Ventura County (Figure 1-3). The

levees include both public levees (approximately 32 km [20 miles] total length) and private levees (approximately 12 km [7.5 miles] total length) along both left and right banks, and were designed to protect agricultural lands, urban development, and floodplain mining pits. Many of the private levees are composed of riverbed materials and are designed to protect agricultural land from flooding; these levees typically have to be repaired or re-constructed after large floods. Several of these levees are themselves protected by earthen or stone groins projecting perpendicular to river flow and designed reduce the velocity of near-bank flood flows that might otherwise undermine the levee. Notable public levee construction began in 1961 with the completion of a U.S. Army Corps of Engineers levee designed to protect agricultural land along the south side (left bank) of the LSCR between South Mountain and Highway 101. The levee is approximately 7.4 km (4.6 miles) long and was built to prevent floodplain inundation for flows of up to the defined standard project flood (6,370 m^3s^{-1}, 225,000 cfs), or an approximate 100-year flood event (according to watershed-wide hydrologic modeling, see AQUA TERRA Consultants 2009). The Ventura County Watershed Protection District (VCWPD) now manages this stone-revetted levee, along with an adjoining reinforeced levee structure from Highway 101 to Victoria Avenue. Stone-protected or soil cement-cored levees have been constructed for flood and erosion protection of urban developments, including some that project into the historical river course.

Over the past several decades, major flood events have caused damage to the levees along the LSCR. In 1969, for example, a 610 m (2,000 ft) reach of the South Mountain–Highway 101 levee failed due the combined effect of the January and February flood events. Further flood damage to the downstream levee and to the Saticoy "dike" (which protects Cabrillo village) occurred during the 1978 events (Simons, Li & Associates 1983). The damage was attributed primarily to undercutting brought about by channel incision associated with the effects of aggregate mining.

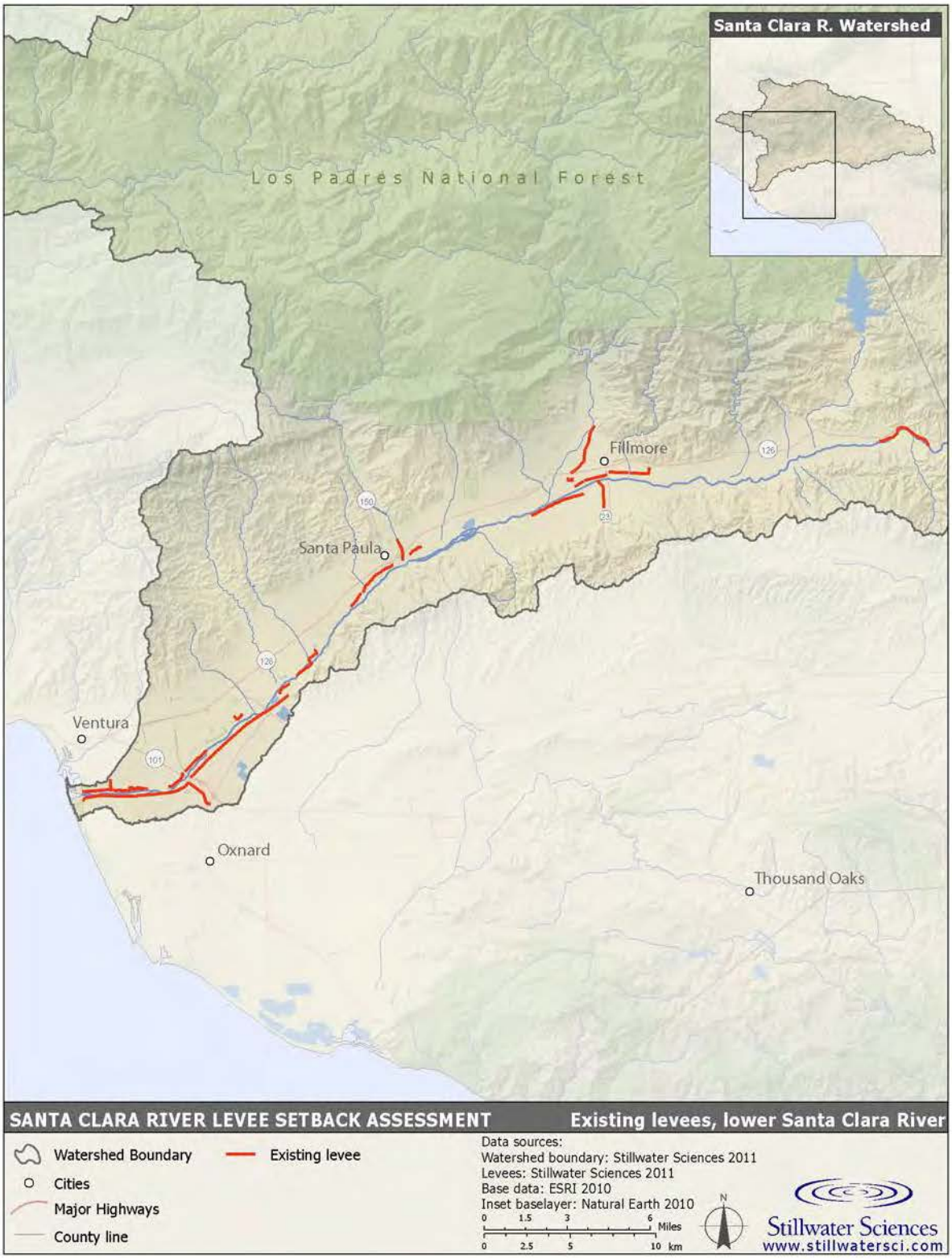

Figure 1-3. Existing levees on the lower Santa Clara River.

2 LEVEE IMPACTS AND POTENTIAL BENEFITS OF LEVEE SETBACK

2.1 Levee Impacts on Fluvial and Floodplain Processes

Channel confinement by privately and publicly maintained levees is one of the most severe impacts limiting geomorphic and hydrologic functioning of the LSCR, especially below the confluence of Santa Paula Creek (Simons, Li & Associates 1983, Stillwater Sciences 2007a). Flood flows from the LSCR historically spilled onto the Oxnard Plain and flowed towards the Pacific Ocean. By design, the levees constructed along the Santa Clara River have confined high flows to the active channel width and have significantly reduced the riparian area historically inundated by large floods. The levees have also reduced the effective flow width during floods and stabilized the river's planform, resulting in an alteration of channel morphologic development and sediment transport. Furthermore, the levees have effectively reduced the flood water storage capacity of the river, thus forcing the majority of high flows to be conveyed solely within the active channel rather than being allowed to spread out upon the floodplain (Stillwater Sciences 2007a).

During high flows, the narrowing of the active flow width combined with the increase in flood water volume moving through the river channel due to levee confinement has increased flood stages, velocities, and shear stresses. Thus, there has been greater potential for sustained and systemic bed and bank scour. Since the levees were first constructed in 1961, a pattern of channel bed lowering, or incision, has developed particularly in the reach downstream Freeman Diversion, which is confined by the majority of the river's levees (see Stillwater Sciences 2007a). Channel incision has likely led to the lowering of adjacent floodplain water table elevations and decreased groundwater storage in some reaches compared to historical conditions.

Where levees are used in conjunction with bank protection to "train" the channel to a particular planform there is the risk that, if the imposed channel planform does not align with the natural planform tendency during flood events (or if the channel is simply too narrow), the flood thalweg will flow directly towards the levee in certain locations. This can lead to high near-bank flow velocities and the potential for levee erosion. This effect is accentuated in incising channels wherein the levee toe can be prone to failure and lead to levee breaches. The 1969 flood apparently produced just such an effect, with flow spilling out through a left bank breach downstream of Victoria Avenue in the direction of historical flood overflows (see Simons, Li & Associates 1983 for more details). Many levees have been reinforced with exterior stone revetments and soil-cement cores that can help defend against such erosion and earthen and stone groins projecting into the flow are used to deflect high velocity flows before they attack the banks. However, notable erosion of some levees and other protected banks still occurred in the 2005 floods (Stillwater Sciences 2007a). An additional impact of protected levees is that flood flows can be reflected towards an opposing, unprotected bank that would not otherwise be prone to substantial erosion.

Although the levee system is an integral part of the flood control efforts along the river, its integrity is regularly threatened by winter high flow events, which, in conjunction with alterations to the channel morphology, has led to significant levee failures requiring costly repairs (Simons, Li & Associates 1983, URS 2005).

2.2 Levee Impacts on Aquatic and Riparian Ecology

In addition to altering the river's morphology, the levees have reduced floodplain habitat use opportunities for a variety of species within the lower watershed. For example, Bozkurt *et al*. (2000) outlined levee impacts on ecological systems whereby a decrease in floodplain inundation and channel migration generally leads to reductions in habitat formation and maintenance, and ultimately to loss of biodiversity with a potential decline of many species populations. In recent years, the ecological and societal benefits (e.g. ecosystem services) provided by intact or restored (reconnected) floodplains have received increased attention in river corridor conservation planning and management (Brauman *et al*. 2007, Stillwater Sciences 2007b, Opperman *et al*. 2009 and 2010).

The presence of multiple flood-control structures and reinforced levees has interrupted lateral habitat connectivity, decreasing the degree of linkage between instream aquatic habitat and riparian vegetation and/or freshwater wetlands in the floodplain. The construction of levees in the lower reaches of the river and estuary, together with development on the floodplain, has dramatically reduced the area available for floods to inundate and, thus, for riparian forests to recruit and grow. Simons, Li & Associates (1983) report that the Santa Clara River floodplain was historically as much as 3.2 km (2 miles) wide in the lowermost reaches. Regular flood inundation over a wide floodplain supported the recruitment of riparian trees over a vast area, while groundwater sustained plants through the summer, allowing mature forests to develop in many reaches. The riparian area likely supported dense, multi-storied stands of broadleaf trees, including cottonwood, sycamore, and various willows, that extended from a few to several miles wide in gaining reaches with rising groundwater (Schwartzberg and Moore 1995, Briggs 1996, Boughton *et al*. 2006, Stillwater Sciences 2007c, Beller *et al*. 2011, Orr *et al*. 2011), while drier conditions in losing reaches supported alluvial scrub vegetation (Stillwater Sciences 2007c, Beller *et al*. 2011, Orr *et al*. 2011). In addition, levee construction and land development within and adjacent to the historical Santa Clara River Estuary footprint have decreased the quality and quantity of aquatic habitat (e.g., rearing habitat for native steelhead, see Stillwater Sciences 2011 for a more detailed discussion).

Currently, the riparian corridor of the LSCR is much narrower compared to historical accounts. The 2005 flood (3,850 m^3s^{-1} [136,000 cfs], approximate 17-year recurrence interval based on a flood frequency analysis at Montalvo gage [USGS 11114000]) inundated just over 2,800 hectares (7,000 acres) along 55 km (34 miles) of the lower river. For comparison, the 1938 flood (3,400 m^3s^{-1} [120,000 cfs], 14-year recurrence interval) inundated over 4,900 hectares (12,000 acres) in this same longitudinal area (Figure 2-1). This difference represents a nearly 40% loss in the extent of the riparian corridor since 1938 (Stillwater Sciences 2007c). This loss is most acute in the lowest reaches of the river where nearly 70% of the 1938 riparian corridor has been lost. The loss of is riparian and floodplain habitats int the LSCR is even greater, closer to 60% overall, when current conditions are compared to pre-1850 historical conditions (Beller *et al*. 2011, Orr *et al*. 2011).

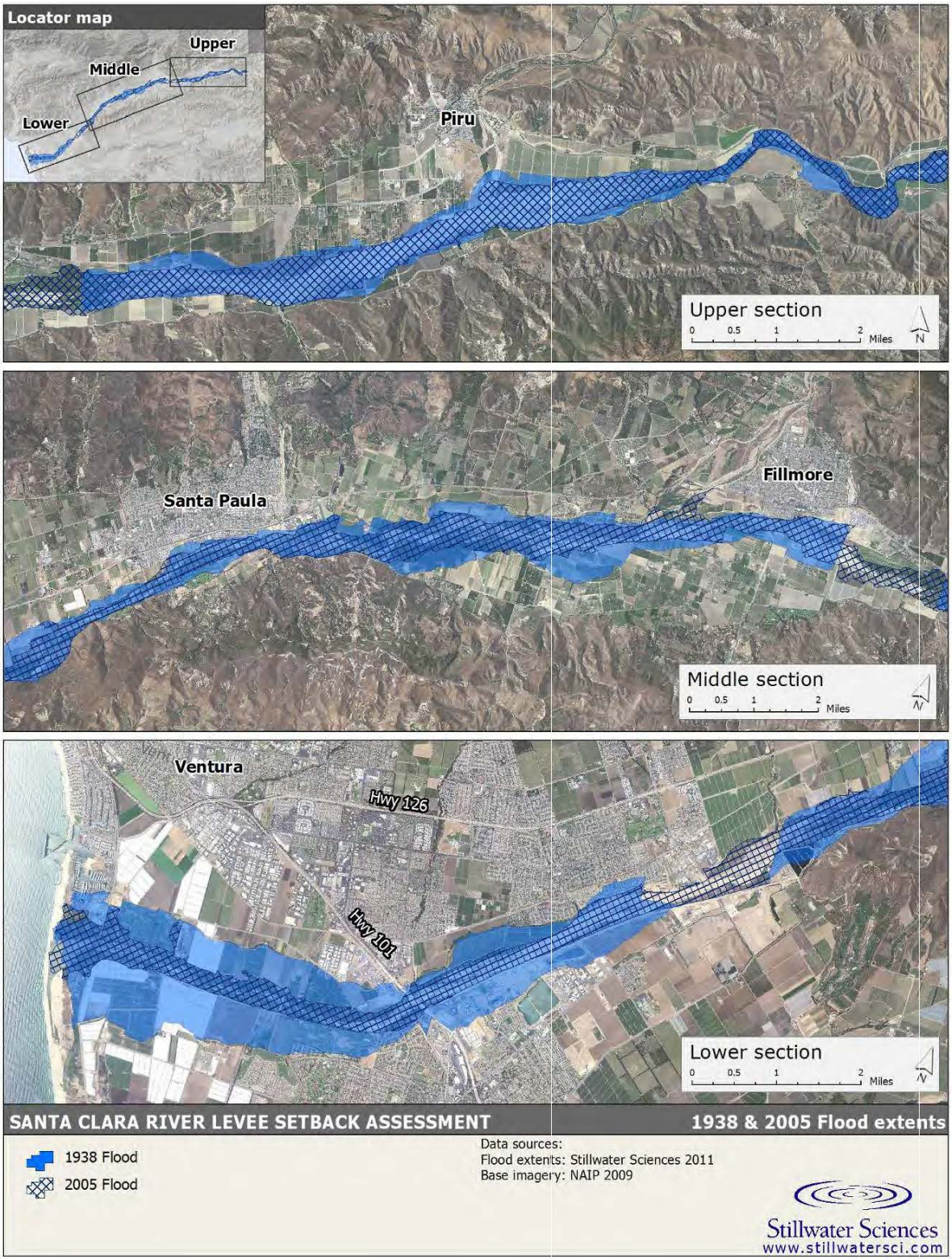

Figure 2-1. Extent of the 1938 and 2005 floods.

2.3 Levee Setback as a Restoration Tool

Levee setback or removal involves some combination of the following actions either singularly or in combination: the active or passive removal of existing channel-confining levees, construction of setback levees away from the river channel, re-contouring of the restored floodplain area to bring the floodplain elevation closer to the channel elevation, and removal or modification of infrastructure.

On properties acquired for conservation uses, floodplain restoration can be accomplished by either passive or active levee removal. Passive removal would be appropriate primarily for privately maintained agricultural levees that are not highly engineered, as are common along the LSCR, particularly in the upper reaches. Passive removal implies allowing failure or breaching to occur naturally during large flood events but without subsequent repair. Active removal requires heavy equipment to dismantle the levee. In locations where levees are allowed to fail or breach, consideration should be given to whether restoration would benefit from the subsequent removal of the remaining levee structure.

In areas that require continued flood protection of development or agriculture on the floodplain, bank-edge levees could be replaced by setback levees constructed to provide the same level or better flood protection while still providing lateral connection between the river and its floodplain, thus encouraging natural fluvial processes and habitat development. A setback levee is placed landward some distance away from the active channel margin, which allows the restored floodplain area between the setback levee and the river's edge to be occasionally inundated during seasonal high flow events (Mount 1995, USACE 2002) (Figure 2-2). In some instances, setback levees are not necessary when there are existing natural or man-made topographic features that will contain flood water within the targeted floodplain parcel.

Along the LSCR, levee setback strategies should be focused downstream of the Vern Freeman Diversion Dam, where existing levees severely constrain the floodplain width and opportunities for setback exist. Some opportunities also exist in the reaches upstream of the Diversion Dam, however the reaches downstream should be the priority becuase they have the greatest extent of channel confinement and have shown a trend of channel incision over the past several decades (see Stillwater Sciences 2007a). The recent FEMA floodplain mapping of the Santa Clara River presents a strategic opportunity for setting back levees: when levees are reconstructed to meet FEMA certification standards, they can be simultaneously setback. Because they increase the floodway width and so inherently increase the river's flood conveyance capacity, setback levees need not be as high as bank-edge levees and are much less disruptive to high-flow processes.

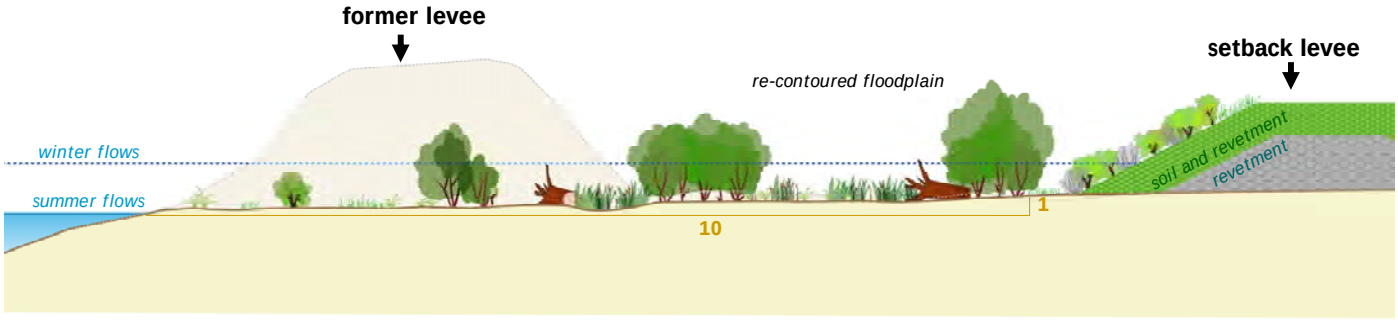

Figure 2-2. Conceptual diagram of levee setback strategy.

Following removal of existing levees or construction of setback levees, some areas may require re-grading of the floodplain surface that is now part of the active floodway. This procedure, also called floodplain re-contouring, may involve filling in any abandoned gravel-mining pits, or other man-made depressions that could present stranding issues for salmonids and other native fishes, and increased risk of predation by avian or fish species. Lowering or sloping of the floodplain towards the river channel may also be required to increase the potential for seasonal inundation, especially along incised reaches where the elevation of the floodplain in relation to the channel bed has dramatically changed from pre-development conditions. Similar to other restoration projects involving floodplain re-contouring, the restored riparian corridor could be passively or actively revegetated with native plant species to provide habitat for wildlife. Floodplain re-contouring may also be necessary in the few areas where levee removal or setback would connect former mining or industrial areas with the floodplain.

Another activity associated with levee removal and/or setback is the modification or removal of infrastructure in the floodway. As the floodplain is widened and the channel is allowed to migrate, existing infrastructure may have an increased risk of damage by bank erosion or flood flows. In most cases, infrastructure is modest, consisting primarily of fencing, concrete debris, power lines, and pumping facilities. In other cases, such as McGrath State Beach, it would be prudent to relocate camping and day use facilities farther away from the river mouth to allow the river to move away from the Ventura Water Reclamation Facility and reduce maintenance costs following high flows. In more extreme cases, modifying bridges or relocating wastewater treatment facilities would be necessary to improve fluvial processes and reduce flood risk to these structures.

2.4 Potential Benefits Associated with Levee Setback

Implementation of levee setback or removal has the potential to provide numerous benefits to riparian habitat conditions and geomorphic processes that have been substantially altered due to the presence of the existing channel-confining levees. The primary potential ecological benefit of this restoration strategy is the re-establishment of a seasonally inundated floodplain. The river's re-connection to its floodplain would allow for an exchange of water, sediment, and nutrients between the river and its floodplain, and an increase in riparian habitat patch size and quality.

Constructing setback levees would, where necessary, maintain flood protection for the surrounding developments outside of the Parkway area. An enlarged river corridor would enhance landscape linkages, providing movement corridors for wildlife between protected lands. Additionally, a wider floodway should increase the residence time of flood waters on the floodplain and so increase groundwater recharge (Poole *et al*. 2002, Kazama *et al*. 2007). It should be noted that while levee setback and removal may need to be implemented in a parcel-by-parcel fashion due to land acquisition and funding constraints, the larger benefits of this restoration strategy, particularly flood control and landscape linkages, as well as cost efficiencies, will not be fully realized until larger extents of levees are setback or removed.

Re-initiation of fluvial processes, such as bank erosion, bar growth, channel migration and width increases to the active channel bed, would be an expected outcome following implementation of this restoration strategy. During flood events in a braided-meandering river such as the Santa Clara River, bank erosion naturally occurs at the outer banks of a river bend where velocities are greatest, or near perturbations on the channel bed, such as midchannel bars, that can topographically steer the flow against the adjacent bed or banks causing higher shear stresses to scour the channel boundaries (Leopold

et al. 1964). Deposition of sediment occurs in slower portions of the channel and contributes to the formation of point bars, mid-channel bars, and natural sedimentation processes. The re-initiation of this process would not only restore a dynamic physical characteristic of the Santa Clara River but also would benefit in-channel and riparian habitat diversity (Bozkurt *et al*. 2000, Stillwater Sciences 2007b, Opperman *et al*. 2010).

A long-term trend of channel incision downstream of the Highway 118 bridge (as illustrated in Stillwater Sciences 2007a) could potentially slow or cease following implementation of this restoration strategy, because the flood waters would be allowed to spread out upon the reconnected floodplain, thus increasing the river's flood capacity and effective flood width. Also, because floodplain discharge has low velocities due to frictional resistance from vegetation and other roughness features on the floodplain, sufficiently broad floodways can attenuate flood flows, thus diffusing the potential for deep, high velocity flows to scour the channel bed.

An additional benefit from the seasonal inundation of the restored floodplain is recharge of groundwater into the basin's aquifers, which are a major source of fresh water for the many land use activities in the valley, especially agriculture. The amount of groundwater recharge by inundated floodplains depends on several factors, the most critical of which are the residence time of the water on the floodplain, the permeability of the floodplain substrates, and depth to the water table. For these reasons, arid-region rivers dominated by sporadic high flow events such as the LSCR, have greater potential for groundwater recharge because their floodplains are often composed of coarse sediments with high permeability, and the groundwater table is usually well below the channel (G. Wallace, Pacific Groundwater Group, pers. comm., 2008). Short residence time, low porosity, and a shallow groundwater table inhibit groundwater recharge. For example, the Oxnard Plain groundwater basin has several clay strata that inhibit effective infiltration of floodwater. However, the majority of the groundwater basins underlying the LSCR occur in recent and relatively deep alluvial deposits that are very porous and allow easy infiltration of floodwater.

A final benefit from this action is the potential for improved flood protection for the various developments located throughout the LSCR valley. Hydraulic modeling of levee setback scenarios suggests that water surface elevations and velocities for high-magnitude flow events can be greatly reduced. Flood protection can be further enhanced by new setback levees by building them according to the latest engineering and FEMA certification standards. Setback and properly constructed levees should also reduce levee maintenance and other public works and private property costs associated with flood damage.

3 ASSESSMENT OF LEVEE SETBACK MODELING

3.1 Modeling Approach

To assess the potential flood risk management, geomorphic, and ecologic benefits associated with levee setback, and to assist with restoration project design, a sophisticated numerical model of the LSCR was developed. Previous hydraulic models of the LSCR were 1-dimensional (1D) HEC-RAS models capable of simulating flood velocity and stage. These models provide a snapshot of flood hydraulics (i.e., river stage and velocity) at individual cross sections for one discrete flow and do not adequately simulate the complex nature of flow movement between the river and adjacent floodplains over an entire flood hydrograph necessary for examining the impacts of levee setback. A-MIKE FLOOD model (coupled 1D/2D hydraulic model) of the LSCR was therefore developed to assess impacts of levee setback on both in-channel and floodplain flow hydraulics throughout an individual flood. The model topography is based on the 2005 LiDAR data and its domain extends from the Los Angeles County Line downstream to the Santa Clara River mouth. A key benefit of this model is the ability to visualize a flood moving through the LSCR by tracking changes in in-channel and floodplain flow depth, velocity, and inundation extent as the flood pulse moves downstream. For this project, the model output of interest for assessing the impact of levee setback for each scenario included a time series of flood stage/water depth and average velocity values within the main river channel and adjacent floodplain. Appendix A provides a complete and detailed description of the MIKE FLOOD modeling approach.

Prior to modeling the impacts of levee setback, the MIKE FLOOD model was calibrated for existing conditions using a combination of results from a pre-existing hydraulic model and floodplain inundation observations. The flood stage and floodplain inundation extent derived from the recent FEMA 100-year floodplain 1D hydraulic modeling effort along the LSCR was used to calibrate the model and help guide refinement of model inputs. It should be noted that the modeled 100-year flood event is very large (i.e., peak discharge at Montalvo of over 5,700 m^3s^{-1} [200,000 cfs]) and has not been experienced in the LSCR since European American settlement in the watershed over 150 years ago. The model was calibrated further for a 25-year flood event with information regarding spatial patterns of floodplain inundation during the January 2005 flood event, which had a recorded peak discharge in the mainstem LSCR that was somewhat higher than that for a modeled "watershed-wide" 25-year flood event used in this analysis. In this instance, the watershed-wide 25-year flood event is defined as the flood event where all tributaries to the LSCR are experiencing a 25-year flood (with values derived from watershed-wide hydrologic modeling, see AQUA TERRA Consultants 2009). The floodplain inundation information was therefore used as a check to identify areas where levee elevations needed to be incrased to better reflect actual condtions (i.e., where modeling showed floodplain inundation but no inundation was observed).

Following calibration, the MIKE FLOOD model was set-up to simulate the hydraulic effects of levee setback at several locations for a range of hydrologic conditions. The levee setback areas consisted of four floodplain parcels (Vulcan, Camp, Lower North Bank [NB], and Lower South Bank [SB]) that the Coastal Conservancy and TNC are interested in including in the Santa Clara River Parkway (see Figure 3-1). Several other floodplain parcels were initially considered for inclusion within the levee setback assessment (e.g., south bank floodplain area across from the Sespe Creek confluence with the LSCR), however the relatively high restoration potential of these four parcels made them the current assessment priorities. The model was modified to reflect a suite of levee setback scenarios: levee setback at individual floodplain parcels and levee setback at all four floodplain parcels combined. These scenarios were then run for the 25-year and 100-year flood events to allow for comparison with the calibration (or, existing

conditions) model runs. In an effort to determine the levee setback impacts under hypothesized future hydrologic conditions as compared to current conditions, the model scenarios were run with an increased 25-year flood hydrograph and downstream sea level elevation derived from climate change projections for 2050 (see Carollo 2011 and references therein).

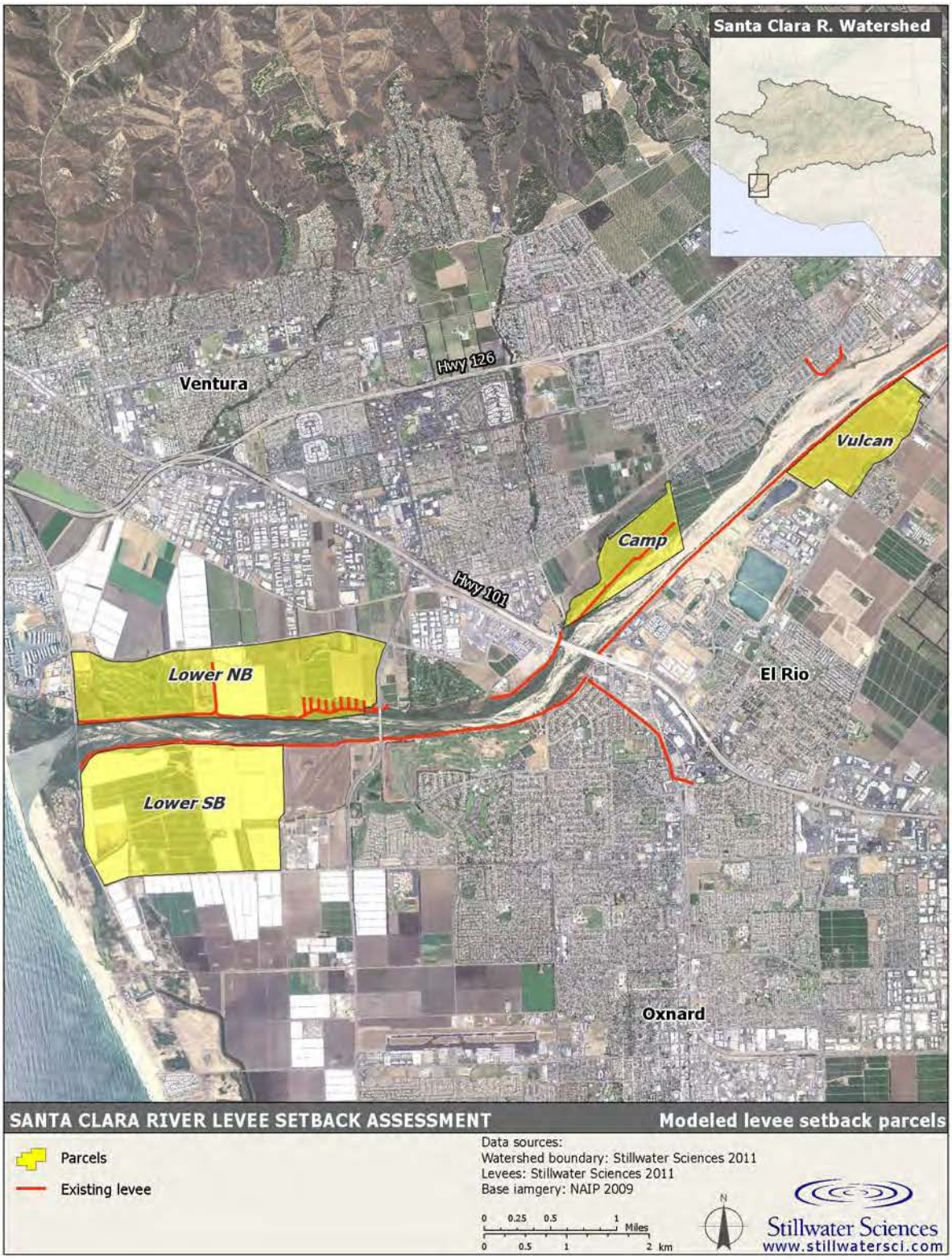

Figure 3-1. Modeled levee setack parcels along the lower Santa Clara River

3.2 Key Findings of the Levee Setback Modeling

The most salient results asscociated with the levee setback modeling effort are described in brief below. We concentrate the discussion on the results from the current and hypothesized future 25-year flood event model runs for the suite of levee setback scenarios as they are most applicable to understanding potential benefits associated with lower flood flows (i.e., the flows at the beginning and end of the flood hydrogrpah) and the largest peak flood flow of recent record (i.e., the peak discharge for the January 2005 flood event). A comprehensive discussion of the modeling results for all the model runs can be found in Appendix A.

Modeling the levee setback scenarios for current hydrologic conditions illustrated both the flows that initiate floodplain inundation and the likely flood risk management benefits associated with individual and combined levee setback approaches at present. The modeling effort showed that floodplain "activation," or the initation of floodplain inundation, occurs at the setback levee locations between 480 and 710 m^3s^{-1} (17,000 and 25,000 cfs, approximate 2.5- to 3-year flood events) and that floodplain areas become inundated (i.e., the entire floodplain area shows some degree of saturation) between 1,400 to 2,000 m^3s^{-1} (50,000 to 70,000 cfs, approximate 5- to 7-year flood events). The floodplain activation flow is lower for the Vulcan and Lower SB parcels (480 m^3s^{-1} [17,000 cfs] and 570 m^3s^{-1} [20,000 cfs] respectively) and higher for the Camp and Lower NB parcels (710 m^3s^{-1} [25,000 cfs] for both). With regard to effects on flood hydraulics, levee setback at the Vulcan and Camp parcels had similar limited benefits due to relatively small floodplain areas (0.2 m [0.7 ft] maximum river stage decrease and 0.2 to 0.5 m s^{-1} [0.7 to 1.6 ft s^{-1}] average river velocity decrease). Levee setack at the Lower NB parcel showed increased benefits (0.6 m [2.0 ft] maximum stage decrease and 0.5 m s^{-1} [1.6 ft s^{-1}] maximum velocity decrease), while levee setback at the Lower SB parcel had the greatest benefits (1.3 m [4.3 ft] maximum stage decrease and 0.8 m s^{-1} [2.6 ft s^{-1}] maximum velocity decrease). The impact is greatest at Lower SB in part because of the relatively low floodplain elevation and the southern model domain extent allowing for flooding of both Harbor Boulevard and McGrath Lake. There was no apparent difference in benefits at each levee location when comparing individual levee setback with the comprehensive levee setback scenario because the decrease in stage and velocity associated with levee setback were localized around each levee setback area. This localized nature of levee setack effects is due to three primary factors: 1) limited overall flood water storage and rapid floodplain 'filling' during the first half of the modeled storm; 2) increase in flood stage due to floodwater impoundment behind bridges downstream of levee setback parcels; and 3) increase in flood stage at the downstream extent of the model due to a static water surface elevation (set at mean higher high water [MHHW] in the model). The first two factors controlled the impacts at the Vulcan and Camp pacels while the third factor was the primary control on the impacts at the Lower NB and Lower SB parcels.

For hypothesized future hydrologic conditions, levee setback at the selected parcels has a somewhat greater beneficial effect compared to current conditions. Assuming that the LSCR river bed elevation remains static, the localized flood stage decrease associated with the future 25-year flood event and MHHW elevation is slightly higher than for current conditions (0.3 m [1 ft] maximum stage decrease at the Vulcan and Camp parcels and 1.6 m [5.2 ft] maximum stage decrease at the Lower NB and Lower SB parcels). Although there are data suggesting a trend of modest bed aggradation adjacent to the Vulcan and Camp parcels and modest bed incison adjacent to the Lower NB and Lower SB parcels between 1993 and 2005 (see Figure 5-9e in Stillwater 2007a), a static bed elevation assumption for future conditions seemed appropriate given all other modeling assumptions and caveats. As discussed in Appendix A, the future 25-year flood event was estimated based on the concept that large storm frequency will increase as climate changes (e.g., a future 25-year flood event will be similar to the present-day 38-year flood event).

Following this logic, and assuming a static bed elevation, it can be concluded that the flows required to activate and inundate the selected floodplain parcels will also occur more frequently in the future.

3.3 Model Implications for Lower Santa Clara River and Floodplain Management

3.3.1 Flood dynamics

The modeling exercise effectively illustrates the anticipated flood risk management benefits associated with the levee setback scenarios. The modeling results show that there can indeed be a considerable reduction in local flood stage and in-channel flow velocity for large storm events, however these effects have been shown to be localized and maximized at the largest parcels (over 1 m [3 ft] decrease in stage and greater than 0.5 m s^{-1} [0.16 ft s^{-1}] decrease in velocity at the Lower NB and Lower SB parcels). In general, the decreases in stage and velocity are maximized at the center of the setback location and extend less than 2 km (1.2 miles) upstream or downstream, or to the next upstream or downstream bridge. The modeling further shows that flood risk management benefits associated with the levee setback scenarios would increase somewhat under future flood flows conditions due to a proportional relationship between floodplain storage potential and maximum flood discharge (i.e., there is more flood risk management benefit associated with levee setback for a 100-year flood than a 25-year flood).

With regard to management implications, the modeling results highlight two factors that should be considered when developing either an integrated or targeted levee setback plan for flood risk management purposes along the LSCR. The spatial extent of the floodplain area intended to store floodwater exerts a primary control on the relative degree of flood risk management benefit and should therefore be considered an initial factor in plan development. Modeling results show that setting back levees at the larger, downstream parcels had a greater effect than setting back levees at the smaller, upstream parcels. Although this control may be apparent, it emphasizes the need to seek opportunites for restoring the largest floodplain areas where ever possible, and suggests that obtaining adjacent floodplain properties and setting back contiguous levees could be an effective means of maximizing flood risk management benefit. The modeling results also highlight the impacts of bridge structures on flood hydraulics and associated flood risk management benefits. Bridges along the LSCR appear to have a strong control on the upstream and downstream propogation of flood risk management benefit (as shown by the effects of the Union Pacific Railroad, Highway 101, and Victoria Avenue bridges on flood stage). Because of this, any plan developed to setback levees for flood risk management purposes should either seek to avoid levees in the vicinity of bridges or seek opportunities to include modification of the nearby bridge structures as part of the flood risk management plan.

3.3.2 Geomorphic and ecological functioning

The changes to flood hydraulics associated with the levee setback scenarios translate to changes in geomorphic and ecological functioning of the channel and adjacent floodplain. These changes are a result of alterations to flood flow velocity, floodplain inundation frequency, and floodplain inundation depth and extent. From a management perspective, it is important to understand the relative effect on geomorphic and ecologic functioning associated with each scenario as a means of helping to develop levee setback priorities for the LSCR.

Levee setback allows for increased floodplain inundation frequency and decreased average in-channel flow velocity for flood events. In general, increased floodplain inundation leads to increased fine sediment deposition and the creation of suitable habitat for riparian vegetation species. The more

frequent and deeper the inundation flow, the more frequent the fine sediment deposition and habitat creation and resetting of vegetation successional processes, which can create and maintain or more diverse mosaic of natural riparian habitat. Decreased in-channel flow velocity during flood flows results in decreased channel and bank scour, which can cause increased in-channel sediment deposition, channel stabilization, and subsequent in-channel habitat maintenance. In-channel sediment depositon downstream of Victoria Avenue is of particular importance as it could help offset the recent trend of incision. As climate change continues and sea level and flood frequency increase, levee setback essentially allows for more frequent floodplain inundation, thereby increasing the potential for improved channel and floodplain functioning into the future.

In considering management approaches for the LSCR, modeling results suggest that setting back levees at all four parcels considered provides the greatest benefit to geomorphic and ecological functioning, under both current and predicted future hydrologic conditions. Setting back levees at all four parcels has the greatest total effect on decreasing in-channel velocity and increasing floodplain inundation, albeit from a combination of localized changes at individual levee setback locations. With regard to which specific parcels have the greatest potential benefits associated with levee setback, Lower NB and Lower SB should be considered the highest priorities due to their relatively large floodplain area and relatively high floodplain inundation frequency. As with the flood risk management benefit, seeking opportunities to increase levee setback areas by combining adjacent floodplain properties would be an effective means of increasing the potential geomorphic and ecological benefits.

3.4 Recommendations

Overall, the approach to levee setback along the LSCR should be developed with consideration of improved flood risk management and geomorphic/ecologic benefits for both current and future conditions. The modeling exercise described in brief above (and presented in detail in Appendix A) has provided some answers on how best to accomplish this goal and has also highlighted some data and analysis gaps that would need to be addressed when developing an approach. The primary recommendations drawn from this modeling excercise for moving forward with developing a robust levee setback approach that addresses all pertinent considerations are as follows:

- The modeling effort clearly illustrates potential flood risk management and ecologic/geomorphic benefits associated with levee setback at all four floodplain parcels. In consideration of all modeling results, we suggest that acquisition and restoration priorities should be focused on the Lower NB and Lower SB parcels. Combined, these parcels provide the greatest flood risk management benefit and the largest floodplain extent available for restoration of natural geomorphic and ecologic processes. Setting back levees along the Lower SB parcel would, however, need to be combined with flood protection or other flood risk management considerations along Habor Blvd. and McGrath Lake.
- Prior to implementation of any levee setback approach, improving the modeling framework, to the extent possible, should be done to help improve the prediction of the magnitude and extent of flood risk management and geomorphic/ecologic benefits. Improvements can be gained through such action as additional calibration data collection (e.g., collection of water surface data in the LSCR for known flow discharge values), better defining the extent of floodplain restoration (e.g., adding levees to protect infrastructure at the Lower SB parcel southern extent), and modeling additional parameters (e.g., sediment transport and bed scour). Including sediment transport and bed scour is of particular importance and could help better define the maximum in-channel flood stage and associated floodplain inundation extent associated with storm events.

- Assessing the impact of both levee setback and bridge modification on flood stage and velocity should be included in future modeling efforts to determine the potential flood risk management and ecologic/geomorphic benefits given less hydraulic constraint. The current modeling suggests that the Highway 101 bridge, the Union Pacific Rail Road bridge, and the Victoria Avenue bridge have the greatest impact on flood hydraulics. The openings beneath these bridges could be modified in the model and a sensitivity analysis could be conducted to determine the opening characteristics that provide the optimal amount of flood water passage.
- Moving forward, levee setback efforts along the LSCR should be focused on parcels with willing sellers that have a relatively large potential inundation area (i.e., large floodwater storage and floodplain inundation area), have a relatively low average ground surface elevation compared to the adjacent river bed (i.e., high inundation frequency), and have levees that can be setback without adversely affecting adjacent, protected properties or the upstream and downstream river channel stability. Combining an assessment of these factors with an analysis of restoration-related costs would be an effective means of prioritizing and targeting LSCR levee setback opportunities.

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Appendix A

Levee Setback Modeling Technical Memorandum

(cbec, Inc. 2011)

MEMORANDUM

Date:	August 24, 2011
To:	Scott Dusterhoff, Zooey Diggory (Stillwater Sciences)
From:	Chris Campbell, April Sawyer
Project:	09-1005 – Santa Clara River Levee Setbacks
Subject:	Appendix of Model Results

1 INTRODUCTION

A 2D hydrodynamic model was developed for the lower Santa Clara River (SCR), from the Ventura-Los Angeles County line to the Ocean in Ventura County, to assess the potential flood benefits afforded to the SCR system as a result of specific levee setback scenarios. The 2D model was used as a tool to specifically assess how the setbacks, in part and in combination, changed water levels and inundation extents within the lower SCR. Previous hydraulic studies have modeled this reach of the lower SCR, but those studies have been limited to 1D model platforms (i.e., HEC-RAS). However, these existing 1D models provided several model input parameters (e.g., bridge details), which made for efficient 2D model development.

This study was undertaken in collaboration with the California State Coastal Conservancy (SCC) and Stillwater Sciences.

2 EXISTING CONDITIONS MODEL DEVELOPMENT

2.1 MODEL DOMAIN

The 2D hydrodynamic modeling platform MIKE FLOOD (DHI, 2009) was used to simulate the potential flood benefits of specific levee setback scenarios. MIKE FLOOD is a dynamically coupled 1D/2D (MIKE 11/MIKE 21) model that can simulate the complex interplay between and amongst the river and adjacent floodplains. This model includes robust methods to accommodate wetting and drying of the floodplain and can readily accommodate hydraulic structures in both the 1D and 2D components of the model. Furthermore, due to the dynamic nature of the model, it can be used to effectively characterize the transient storage effects of the levee setback scenarios by accounting for the volume of water moving through the system and the effect they have on inundation levels and infrastructure.

A MIKE FLOOD model was constructed for a 40-mile long reach of the SCR from the Los Angeles County Line to the Ocean (see Figure **1**). The entire river and floodplain were represented in 2D, with only the hydraulic structures (i.e., bridge crossings) represented in 1D (see Section 2.4 for details). An unstructured 2D mesh consisting of triangular and quadrilateral elements was created for the channel and floodplain areas (see Figure **2** for example). Areas within the FEMA designated 100-year floodway were defined with triangular elements of finer resolution (15 m to 30 m element faces). Areas outside the floodway on the floodplains, upper floodplain terraces, and adjacent valley hillslopes were defined with triangular elements of coarser resolution (up to 100 m element faces). The model was registered to UTM 11N WGS84 meters (horizontal datum) and NAVD88 meters (vertical datum).

2.2 TOPOGRAPHY AND BATHYMETRY

Channel bathymetry and floodplain topography was derived from a 2005 LiDAR dataset developed for the FEMA restudy sponsored by the Ventura County Watershed Protection District (VCWPD) and registered to CASP V NAD83 feet and NAVD88 feet. LiDAR for the lower watershed was flown in mid February 2005 with river flows in the 10 – 18 cms range following a major flood event with a peak occurring on January 9, but before a second major flood event in February of 2005 with a peak occurring on February 22 (see Figure **3** for details). LiDAR for the upper part of the SCR above Vern Freeman Diversion Dam was flown after the February 2005 event in early March 2005 when the river flows were in the 40 – 55 cms range. At the seam between the two flights, the channel is slightly misaligned due to changes as a result of the February 2005 flood as well as differences in flow due to LiDAR returns off of the water surface. Overall, the flows in the SCR varied during these multi-week LiDAR flights, and as such, variability was introduced into the topography and bathymetry used for this modeling effort, which then introduces some uncertainty in the model results, as discussed in Section 4.1.2.

Project levees (i.e., FEMA certified), non-project levees (i.e., not FEMA certified), agricultural berms, and road embankments within the SCR model domain were identified using levee alignments provided by Stillwater Sciences and by thoroughly examining the LiDAR contours to identify streamwise and lateral topographic features that would impede flow onto floodplain surfaces. Quadrilateral elements were defined at these locations to ensure that the topographic maxima were represented within the model domain along the entire length of the features.

2.3 BOUNDARY CONDITIONS

2.3.1 Existing Conditions Hydrology

Design hydrographs for the 25-year (Q25) and 100-year (Q100) recurrence interval flood events were derived from the calibrated and validated HSPF (Hydrologic Simulation Program Fortran) model, a US Environmental Protection Agency (EPA) watershed hydrology model, developed for the VCWPD and the Los Angeles County Department of Public Works (LACDPW) by AQUA TERRA Consultants (2009). Thirty four Q100 design hydrographs were extracted from the HSPF model (see Table 1 for a list of inputs). To obtain the Q25 design hydrographs, the Q100 design hydrographs were individually scaled based on

published scaling factors (AQUA TERRA, 2009) provided in Table 1. Q100 hydrographs from this HSPF model were also used as inputs to the FEMA (2009) restudy HEC-RAS (RAS) model used as a tool for model parameterization and validation throughout this analysis.

Note: the 50-year (Q50) design hydrographs were originally going to be analyzed in this study; however, after review of preliminary model results, cbec and Stillwater staff decided to analyze the Q25 flood event because a) the January 2005 flood event was a flood event of recent memory (with anecdotal evidence) with a peak discharge similar to the Q25 flood event, and b) the modeled Q50 flood event overwhelmed more than half of the levees chosen for setback analysis under existing conditions, resulting in little to no flood benefits under setback conditions.

Table 1. Hydrologic summary table

Stream Name	Reach Node Name	Q100 Peak Discharge (cms)[1,2]	Q25 Peak Discharge (cms)[1]	Q25 Scale Factor [1,3]	Q25 Future Peak Discharge (cms)[1]	Q25 Future Scale Factor[1,4]
Piru Creek	RCH529	1163.8	528.4	0.454	665.7	1.26
Salt Canyon	RCH322	165.9	79.9	0.481	100.6	1.26
Tapo Canyon	RCH401	124.6	56.6	0.454	71.3	1.26
Edward	RCH603	61.2	27.8	0.454	35.0	1.26
Warring Real Canyon	RCH605	83.8	40.5	0.483	51.0	1.26
Hopper Canyon	RCH614	552.2	250.7	0.454	315.9	1.26
Basolo Ditch	RCH631	45.9	22.0	0.479	27.7	1.26
Pole Creek	RCH634	209.3	95.0	0.454	119.7	1.26
Sespe Creek	RCH728	3794.4	2083.1	0.549	2624.8	1.26
Reimer Ditch	RCH806	124.6	60.0	0.482	75.6	1.26
Balcom Canyon	RCH812	130.0	62.6	0.481	78.9	1.26
Orcutt Canyon	RCH821	150.1	72.2	0.481	91.0	1.26
Timber Canyon	RCH822	142.4	68.5	0.481	86.3	1.26
Santa Paula Creek	RCH835	1115.7	490.9	0.440	618.5	1.26
Fagan Canyon	RCH837	128.8	62.0	0.481	78.1	1.26
Peck Drain	RCH838	51.8	34.0	0.656	42.8	1.26
Adams Barranca	RCH842	194.8	94.0	0.483	118.5	1.26
Haines Barranca	RCH844	83.5	40.2	0.481	50.7	1.26

Stream Name	Reach Node Name	Q100 Peak Discharge (cms)[1,2]	Q25 Peak Discharge (cms)[1]	Q25 Scale Factor [1,3]	Q25 Future Peak Discharge (cms)[1]	Q25 Future Scale Factor[1,4]
Todd Barranca	RCH852	188.3	90.6	0.481	114.2	1.26
Briggs Rd Drain	RCH853	34.8	16.7	0.480	21.1	1.26
Cummings Rd Drain	RCH854	51.0	24.6	0.483	31.0	1.26
Ellsworth Barranca	RCH862	269.6	130.0	0.482	163.8	1.26
Franklin Wason	RCH874	111.9	53.8	0.481	67.8	1.26
El Rio Drain	RCH881	29.7	19.5	0.657	24.6	1.26
Brown Barranca	RCH882	77.0	37.1	0.482	46.7	1.26
Sudden Barranca	RCH885	38.8	18.7	0.482	23.5	1.26
Clark Barranca	RCH886	43.6	28.6	0.656	36.0	1.26
Patterson Rd Drain	RCH891	41.1	26.9	0.655	33.9	1.26
Fairview	RCH619	37.7	18.1	0.481	22.8	1.26
Grimes	RCH641	126.6	60.9	0.481	76.7	1.26
Bear	RCH807	85.8	41.3	0.482	52.1	1.26
O'Leary	RCH809	106.5	51.3	0.481	64.6	1.26
Harmon	RCH883	131.1	63.1	0.482	79.6	1.26
Santa Clara River at LA County Line	RCH320	1877.4	852.3	0.454	1073.9	1.26

[1] Q25 = 25-year; Q100 = 100-year; EX = existing conditions; FUT = future conditions
[2] Data extracted directly from VCWPD HSPF model output (AQUA TERRA, 2009).
[3] Scaling factor adopted from AQUA TERRA (2009) Appendix L, Table 5.
[4] Scaling factor was calculated by the method outlined in Section 3.2.1.

2.3.2 Existing Conditions Tidal Boundary

At the ocean, a constant water surface elevation derived from NOAA tidal datums was used as the downstream tidal boundary condition. Since the mouth of the SCR is located between the NOAA tide stations at Santa Barbara and Santa Monica, the average of the mean higher high water (MHHW) values was used in the model (see Table 2). This assumption was considered reasonable considering that a) limited sensitivity testing demonstrated that the effect of lower tidal elevations did not propagate past Harbor Blvd, and b) the FEMA (2009) restudy RAS model assumed critical depth conditions at the downstream boundary, which resulted in WSEs higher than MHW for the 10-year (Q10) flood event and larger.

Table 2. Existing conditions tidal datums

Datum (m, NAVD88)	Santa Barbara (ID 9411340)	Santa Monica (ID 9410840)	Average Value
MHHW	1.615	1.597	1.606
MHW	1.384	1.371	1.378
MTL	0.827	0.798	0.813
MLW	0.271	0.226	0.249
MLLW	-0.029	-0.057	-0.043

2.4 HYDRAULIC STRUCTURES

Nine bridges were included in the simulation throughout the study reach. Head losses through these structures were calculated in 1D in the MIKE 11 component of the MIKE FLOOD software. Hydraulic structure geometry, loss factors, hydraulic roughness, and other coefficients were extracted from the FEMA (2009) restudy RAS model and used as inputs to the MIKE 11 setup. The Vern Freeman Diversion Dam was also included in the model. The 2005 LiDAR dataset included the dam crest elevation, so the structure was represented in the model as a broad crested weir.

2.5 HYDRAULIC ROUGHNESS

Hydraulic roughness or Manning's n for the MIKE FLOOD model was derived from Stillwater's 2005 vegetation mapping (Stillwater Sciences & URS, 2007) by comparing the 2005 vegetation habitat types to Manning's n values from the FEMA (2009) restudy RAS model. The RAS model hydraulic roughness also relied on an independent field verified land use classification derived from aerial imagery. RAS cross section locations were compared to underlying 2005 polygons to associate habitat type to Manning's n based on station along the cross section. Once it was established that certain habitat types correlated well with RAS Manning's n values, the Manning's n values were assigned to the individual vegetation types (see Figure **4** and Table 3).

Table 3. Santa Clara River hydraulic roughness coefficients

Habitat Type	Manning's n
Water (channel bed)	0.035
Riverwash	0.035
Beach	0.035
Herbaceous (native and non-native)	0.040
Sand dune	0.040
Freshwater wetland	0.045
Tidal Marsh	0.045
Riparian Shrub (desert and mixed/willow)	0.055

Giant reed (*Arundo donax*)	0.055
Disturbed	0.055
Mixed non-native trees	0.075
Agriculture	0.085
Cottonwood/willow forest	0.115
Mixed riparian forest	0.115
Coastal sage scrub	0.115
Developed	0.130

3 FORMULATION OF SCENARIOS

3.1 SETBACK SCENARIOS

Several locations within the study reach were chosen to assess the potential flood attenuation benefits of levee setbacks (see Figure **5**). These setback locations would also facilitate improved ecological services to the system as a whole by increasing channel to floodplain connectivity. Setback locations were chosen by SCC and Stillwater based on current land ownership and potential opportunities to utilize the parcels as seasonal floodplain habitat with minimal risk to existing infrastructure or nearby development (see Table 4). Levees proposed for removal included non-project levees with the exception of the levee at the Vulcan property. Levees were typically set back to existing road embankments surrounding the parcels of interest. See Figure **5** for modeled setback levee locations relative to existing levee alignments. Also, as shown by Table 4, some of the Q100 setback scenarios were grouped together. The reason for doing so was to demonstrate maximum benefits for adjacent setback areas when existing conditions results for Q100 already showed these areas as fully inundated.

Table 4. Levee setback scenario catalog

Levee Scenario	Description	Hydrology		
		$Q_{25,EX}$	$Q_{100,EX}$	$Q_{25,FUT}$
1	Existing Conditions	X	X	X
2	Camp Prop[1]	X	X	
3	Vulcan Prop[2]	X		
4	Lower NB[3]	X	X	
5	Lower SB[4]	X		
6	No Constraints[5]	X	X	X

[1] Levee on the north bank of the SCR upstream of Highway 101
[2] Levee on the south bank of the SCR upstream of Highway 101
[3] Levee on the north bank of the SCR between Victoria Ave and Harbor Blvd
[4] Levee on the south bank of the SCR between Bailard Landfill and Harbor Blvd
[5] Combination of Scenarios 2 through 5

3.2 BOUNDARY CONDITIONS

3.2.1 Future Conditions Hydrology

To represent increased storm intensity in this system and sea level rise due to predicted climate change, future conditions hydrology was developed assuming 2050 condition using existing conditions (i.e., 2005) as a baseline. A relatively recent analysis of the frequency of extreme precipitation events in the US identified a 50% increase in such events for coastal southern California (Madsen & Figdor, 2007). For simplicity, it was assumed that a 50% increase in extreme precipitation frequency directly translated into a 50% increase in flood recurrence interval. As such, the peak flow for a future Q25 flood event would be equivalent to the peak flow of an existing 37.5-year (Q37.5) flood event. Based on this reasoning, data from four (4) US Geological Survey (USGS) gages on the SCR were analyzed by Stillwater using the Log-Pearson distribution to identify Q25 and Q37.5 peak flows based on historic flow data. Table 5 shows the results of this analysis and suggests that a scaling factor of 1.26 (as also shown in Table 1) should be used to convert the existing Q25 flood hydrographs into future Q25 flood hydrographs.

Table 5. Future conditions hydrology

USGS Gage	Existing Q_{25} (cms)	Existing $Q_{37.5}$ or Future Q_{25} (cms)	$Q_{37.5}/Q_{25}$
SCR @ LA Co Line (11108500/11109000)	967.0	1251.7	1.29
Sespe @ Fillmore (11113000)	2173.6	2617.8	1.20
SPC @ Santa Paula (11113500)	482.8	620.0	1.28
SCR @ Montalvo (11114000)	4852.9	6036.3	1.24

3.2.2 Future Conditions Tidal Boundary

To represent predicted increases in sea level rise, an increase of 0.41 meters was added to the MHHW elevation shown in Table 2. The increase of 0.41 meters (relative to MSL) was derived from a recent study in the SCR (Carollo Engineers, 2011).

4 MODEL RESULTS

4.1 EXISTING CONDITIONS

4.1.1 Steady-State Comparison to FEMA 100-Year Model Results

Following model development, the existing conditions model was run for steady-state Q100 conditions and compared to the FEMA restudy RAS Q100 inundation mapping and peak water surface profile given that both models relied upon the same input data. These 2D model results generally showed good agreement with the spatial inundation patterns (see Figure **6**a and 6b) and maximum water surface profile (see Figure **7**) predicted by the 1D FEMA restudy RAS model. However, the 2D results did differ from the 1D results in a few locations:

1. Near the town of Santa Paula, more extensive flooding was predicted behind the Santa Paula Freeway embankment on the north bank of the SCR because the Santa Paula Creek flood control channel was included in the 2D model.
2. Downstream of the Vern Freeman Diversion Dam, more extensive flooding was predicted with the filling of various aggregate mining pits on the south bank. These areas did not show inundation in the FEMA model due to the presence of a project levee along the south bank. While the 2D model also included this levee, flow entered these areas at much localized low points in the model terrain.
3. Near Wagon Wheel Rd on the south bank of the SCR between Highway 101 and the Union Pacific Rail Road train bridge, flow overtopped the levee in the 2D simulation causing more extensive inundation because these low lying areas were modeled as ineffective areas in the FEMA restudy RAS model.
4. The historic lagoon near McGrath Lake in the southern estuary showed more inundation than in the FEMA model. Slight dips in the topography not captured in the reaches between cross sections of the 1D model likely resulted in these differences.

4.1.2 Validation to the January 2005 Flood

The January 2005 flood event was a flood event of recent memory with a peak discharge similar to the Q25 flood event in the lower river (approximately 3850 cms at USGS gage #11114000 at Montalvo). Anecdotal evidence for this event (e.g., Stillwater (2007); aerial photographs; on-the-ground photographs; and VCWPD post-event damage estimates) was used to verify the extent of inundation under existing conditions.

The first round of modeling showed inundation at the Camp property, Lower North Bank and Lower South Bank setback locations (Figure **5**), yet the anecdotal evidence showed that these areas were not inundated during the January 2005 flood event. To correct these inundation errors, levee heights were adjusted as needed in order to show the proper level of floodplain inundation and to demonstrate the flood attenuation benefits of the proposed setbacks (under Q25 only):

1. At the Camp property, levee elevations were increased up to 2 m to restrict floodwaters from inundating the property.
2. On the lower south bank, levee elevations were increased up to 3 m to restrict floodwaters from inundating the low lying areas.
3. On the north bank between Victoria Ave and Harbor Blvd, an east-west berm that excluded floodwaters from agricultural fields was likely scoured away during the January 2005 flood (Figure **9**). This berm was replaced in the 2D model in its original location with 2 m of relief above the surrounding topography.

Levee height adjustments were used as a means to compensate for key factors, as listed below, that likely contributed to higher than observed flood stages:

1. USGS gage data from the January 2005 storm event was compared to the HSPF model outputs for the Q25 flood event at two locations, (1) the SCR at the LA County Line, and (2) Sespe Creek, a significant tributary to the SCR (Figure **8**). While peak flows were very similar, the HSPF model outputs under-predicted flood volumes at both locations, on the rising limb on the SCR at the LA County Line and on the falling limb at Sespe Creek. Differences in flood volumes influence transient flood storage, and in this instance, under-predicting volumes can lead to under-predicting flood stages when using a design flood as a surrogate for an observed flood.
2. Channel bathymetry at the peak of the flood was likely different from the post-flood condition captured in the LiDAR since the LiDAR was flown after this event. Moving bed conditions during the flood likely scoured the channel bed, resulting in a lower bed profile during the peak of the flood, a phenomenon that was not simulated in the 2D model since sediment transport functionality was not enabled (or scoped). A lower bed profile increases the cross sectional area, and hence, the conveyance capacity of the reach under moving bed conditions. Evidence that moving bed conditions were important during the January 2005 flood was corroborated by the significant amount of riparian vegetation that was removed from the reach between Highway 101 and Harbor Blvd as shown by the scour patterns in this area (Figure **9**). As mentioned in Section 2.2, LiDAR water returns used as bed bathymetry in the shallow braided channels of the lower reach likely had negligible effects on stage due to the relative magnitude of flight-time base flow (in the 10 – 55 cms range) compared to the peak of the Q25 flood event (approximately 3300 cms).

As part of the validation for the January 2005 flood event, limited sensitivity testing was performed to understand how downstream tide levels affect upstream water levels. To assess whether the MHHW tidal boundary was affecting upstream inundation, the Q25 flood hydrograph was modeled with MTL and the actual low water that occurred during the January 2005 flood event. Reductions in the downstream tide level did not affect the hydrodynamics upstream of Harbor Blvd, which suggests that channel geometry downstream of Harbor Blvd buffers inaccuracies in the downstream tide level and perhaps even the effects of sea level rise.

4.2 PROPOSED ALTERNATIVE SCENARIOS

4.2.1 100-year Storm Event

Setback Scenarios 2 and 3 – Camp and Vulcan Property Setbacks Combined

For the Q100 flood event, the Camp and Vulcan properties were combined into a single scenario because Camp was inundated under existing conditions and it was assumed that any additional storage at the Camp property would augment flood attenuation benefits at Vulcan. As shown by Figure **10**, there were localized stage reductions of up to 0.3 m in the centerline water surface profile at the Vulcan property at approximately station 9100 m to 12000 m. Further downstream at the Camp property, stage reductions were less than 0.15 m. This pattern of localized stage reduction was constant for all scenarios, as setback benefits do not propagate significantly far upstream or downstream of the setback extents. As shown by Figure **11**, inundation mapping for the combined Camp/Vulcan setbacks relative to existing conditions resulted in full inundation of the Vulcan setback to E Vineyard Rd with very similar inundation extents elsewhere. Thus, for the Q100 flood event, Vulcan provides the only real flood storage benefit in the reach between Los Angeles Ave and Highway 101.

Setback Scenarios 4 and 5 – Lower North and South Bank Setbacks Combined

Similar to above, the Lower North and Lower South setback areas were combined because portions of these areas already showed inundation under existing conditions. As shown by Figure **12**, there were localized stage reductions between Victoria Ave and Harbor Blvd of up to about 1.3 m. As shown by Figure **13**, inundation mapping for the combined Lower North/South Bank setbacks relative to existing conditions resulted in inundation of the Olivas Links golf course. It was difficult to discern whether the main stage reduction benefits were attributed mostly to the Lower North Bank or Lower South Bank setbacks, but it was likely that removal of the levee around the golf course contributed significantly to the rather large stage reduction midway between Victoria Ave and Harbor Blvd by removal of the pinch point and that removal of the South Bank levee allowed for the relatively unobstructed passage of floodwaters across Harbor Blvd.

Setback Scenario 6 – No Constraints

Under the no constraints alternative, all setbacks were combined to assess the maximum potential benefits of all identified setback areas. As shown by Figure **14**, there were very similar localized stage reductions as previously identified in the combined scenarios. As shown by Figure **14** and Figure **15**, there was no noticeable system-wide increase in benefit as a result of combining all setbacks.

4.2.2 25-year Storm Event

As mentioned in Section 4.1.2, the January 2005 (or Q25) flood event was contained within the levees in the lower reach of the SCR from Los Angeles Ave to the Ocean. The Q25 flood benefits of each scenario listed in Table 4 are described below.

Setback Scenario 2 – Camp Property Setback

As shown by Figure **16**, there were localized stage reductions of up to 0.25 m when the Camp levee was removed. The extent of inundation (as shown by Figure **17**) was limited by natural topography or an elevated terrace on the property. Stage reductions were accompanied by localized reductions in maximum channel velocity up to 0.3 m/s adjacent to the removed levee segment, especially in the lower half approximately 1 km upstream of the Highway 101 bridge.

Setback Scenario 3 – Vulcan Property Setback

As shown by Figure **16**, there were localized stage reductions at the Vulcan property of up to 0.25 m when the Vulcan levee was removed. The extent of inundation (as shown by Figure **18**) on the Vulcan property was limited due to higher ground behind the levee; hence, the entire area of the property was not utilized. Decreases in maximum channel velocity occurred in the lower portion of the levee setback area, ranging from 0.2 – 0.5 m/s with the largest reductions occurring in the lower third of the setback area.

Setback Scenario 4 – Lower North Bank Setback

As shown by Figure **19**, there were localized stage reductions of up to 0.6 m due to removal of The Nature Conservancy (TNC) and Olivas Links golf course levees. The extent of inundation to the north between Victoria Ave and Harbor Blvd (as shown by Figure **20**) was maximized to the limits of the floodplain terrace. Reductions in maximum channel velocity were greatest in the lower third of the levee setback length (up to 0.5 m/s), moderate in the middle third (up to 0.3 m/s) and lowest in the upper third (up to 0.2 m/s).

Setback Scenario 5 – Lower South Bank Setback

As shown by Figure **19**, there were localized stage reductions of up to 1.2 m with removal of the south levee downstream of the landfill. The extent of inundation (as shown by Figure **21**) did not extent as far south as W Gonzales Rd, but did overtop Harbor Blvd, flowing into McGrath Lake behind the sand dunes. Reductions in maximum channel velocity were greatest in the lower half of the setback area, ranging from 0.7-0.8 m/s and decreasing to 0.2-0.3 m/s in the upper portion.

Setback Scenario 6 – No Constraints

As shown by Figure **22**, and in comparison to Figure **16** and Figure **19**, there was no appreciable increase in flood benefit during the Q25 flood event as a result of combining all of the setbacks into a single scenario. Reductions in WSE were still fairly local to each setback area and did not propagate upstream or downstream considerably. One reason for this is that the setback areas were typically filled with floodwaters during the rising limb of the flood hydrograph, thereby allowing the flood peak to travel through the system more-or-less not attenuated. A second reason is that the setbacks were not contiguous, so floodwaters still had to be conveyed through constricted reaches to include the bridges. In addition, the overall extent of inundation (as shown by Figure **23**) for the setback areas in combination did not differ considerably from the individual setback areas. One small exception occurred in the Olivas Links golf course where there was a slight reduction in inundation due to the hydraulic relief afforded by the Lower South Bank levee removal. When all setback areas are implemented in combination, maximum channel velocity reductions are commensurate in magnitude with the reductions reported in the previous sections.

4.2.3 25-year Storm Event – Future Conditions

To understand the effects of climate change on the potential flood benefits afforded by setback areas, a future Q25 flood event coupled with sea level rise was modeled for 1) existing topographic conditions and 2) all four setbacks in combination. As shown by Figure **24**, the future flood hydrology under existing levee conditions resulted in 1) the Camp Property inundating, 2) the Lower North Bank in the vicinity of TNC landholdings inundating, and 3) small isolated breakouts downstream of Highway 101 along the south levee. These areas inundated with the increase in flood flows under future conditions, which generally resulted in WSE increases over the entire study reach by 0.3 m to 0.6 m.

With implementation of all four setbacks, the combined flood benefits of the setbacks under a future Q25 flood event were slightly greater relative to an existing Q25 flood event. As shown by Figure **25**, 1) flood depths within Vulcan increased relative to existing Q25 levels, 2) inundation extents within the Camp Property increased slightly due to levee removal, 3) Olivas Links golf course fully inundated due to levee removal, and 4) flood depths on the Lower South Bank near Harbor Blvd and McGrath Lake increased with the increase in flood relief afforded by the levee removal. As shown by Figure **26**, slightly greater reductions in stage under future conditions resulted in 1) up to 0.3 m of stage reduction near Vulcan, and 2) up to 1.5 m of stage reduction downstream of Victoria Ave, which was 0.3 m more than seen under current Q25 conditions (see Figure **22**).

5 CONCLUSIONS AND RECOMMENDATIONS

Based on the Q25 and Q100 model runs for existing conditions, proposed setback conditions, and proposed setbacks with future climate change and sea level rise, the 2D model results generally showed that each setback area resulted in localized reach scale flood benefits with little to no system wide flood benefits propagating beyond the setback areas and general vicinity. The following are a summary of 2D model findings that lead to this overall conclusion:

- The 2D model results generally showed good agreement with the spatial inundation patterns and maximum water surface profile predicted by the 1D FEMA restudy RAS model for steady state Q100 conditions. Slight differences in extents and profile were likely due to the 2D versus 1D spatial resolution in capturing the hydraulic effects of low lying topography and lateral berms or levees.
- The 2D model required levee height adjustments at various locations under the January 2005 validation, which were an interim fix for other factors that may have contributed to differences in expected inundation patterns. Such factors included differences in flood hydrograph volumes (which could not be corroborated at the downstream end since the USGS Montalvo gage was inactive during January 2005) and use of post-flood bed topography (which was likely lower during the passage of the flood peak due to bed scour).
- Under the existing Q100 setback scenarios, whether in part or combination, there was a consistent pattern of localized stage reduction, which did not propagate significantly far

downstream or upstream of the setback extents. This resulted because most of the setback areas, apart from Vulcan and Olivas Links golf course, were already inundated under existing Q100 conditions. The area with the greatest water level reduction occurred in the Lower North and South Bank areas with floodwaters escaping across the south floodplain, across Harbor Blvd, to McGrath Lake.

- Under the existing Q25 setback scenarios, there was no appreciable increase in flood benefit as a result of combining all of the setbacks into a single scenario. Again, water level reductions were fairly local to each setback area with the greatest reductions occurring in the Lower North and South Bank areas. Possible reasons for this were that 1) the setback areas typically filled with floodwaters during the rising limb of the flood hydrograph, thereby allowing the flood peak to travel through the system more-or-less not attenuated, and 2) the setbacks were not contiguous, so floodwaters still had to be conveyed through constricted reaches to include the bridges.
- Under the existing Q25 setback scenarios, reductions in maximum channel velocities typically occurred to the greatest extent in the lower half of reaches directly adjacent to the setback areas with diminishing deceases moving upstream. For example, decreases up to 0.8 m/s were simulated in the lower reach adjacent to the Lower South Bank with typical decreases in the upper reaches adjacent to individual setbacks up to 0.3 m/s.
- Under the future Q25 setback scenarios, which were done to understand the effects of climate change on the potential change in flood benefits afforded by setback areas, there was an overall increase in water levels across the entire study reach by 0.3 m to 0.6 m. As a result of these water level increases, the Camp Property and TNC landholdings upstream of Olivas Links golf course inundated. Again, water level reductions were fairly local to each setback area with the greatest reductions occurring in the Lower North and South Bank areas whereby there was an increase in the relative flood benefit with 0.3 meters of additional drop when compared to existing Q25 conditions.

If the 2D model will continue to be used in the future to evaluate additional scenarios or enhance system understanding, the following recommendations are put forth to improve model performance (i.e., no interim fixes) and perhaps demonstrate greater flood benefits resulting from the various setback scenarios:

- Update the model calibration for a period in time that includes measured flow and water level data within the area of interest.
- Perform additional sensitivity tests to evaluate other model parameters (e.g., hydraulic roughness, flow inputs).
- Include sediment transport modeling to assist in validating the January 2005 flood condition and to more realistically simulate the effects of bed scour (and bed profile changes) associated with passage of a flood wave.
- Develop concepts to address flooding of Harbor Blvd and the McGrath Lake area since the Lower South Bank setbacks downstream of Victoria Ave could be interpreted to exacerbate flooding in this area.

- Consider performing a cost-benefit-risk analysis for each setback scenario based on land acquisition costs, levee degradation and hauling costs, potential flood benefits and impacts, as well as ecological functions and values resulting from more frequent floodplain inundation.

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Notes: background image courtesy of BingMaps.

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Location map and model domain

Project No. 09-1005	Created By: AMS	**Figure 1**

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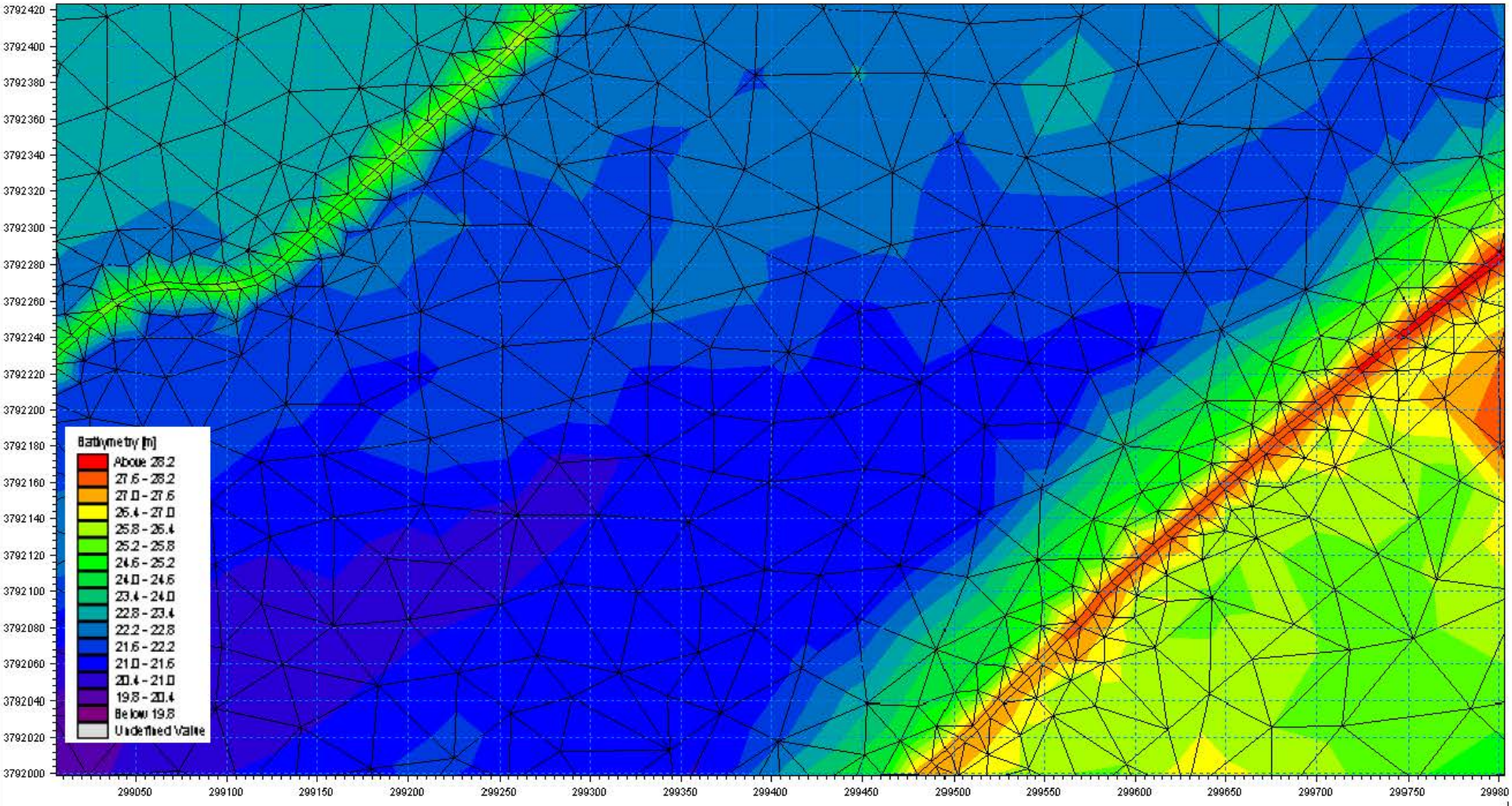

Notes: existing conditions topography; levee at top left is the Camp property levee; projected coordinates in WGS1984 UTM11N.

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Unstructured mesh and topography

Project No. 09-1005	Created By: AMS	**Figure 2**

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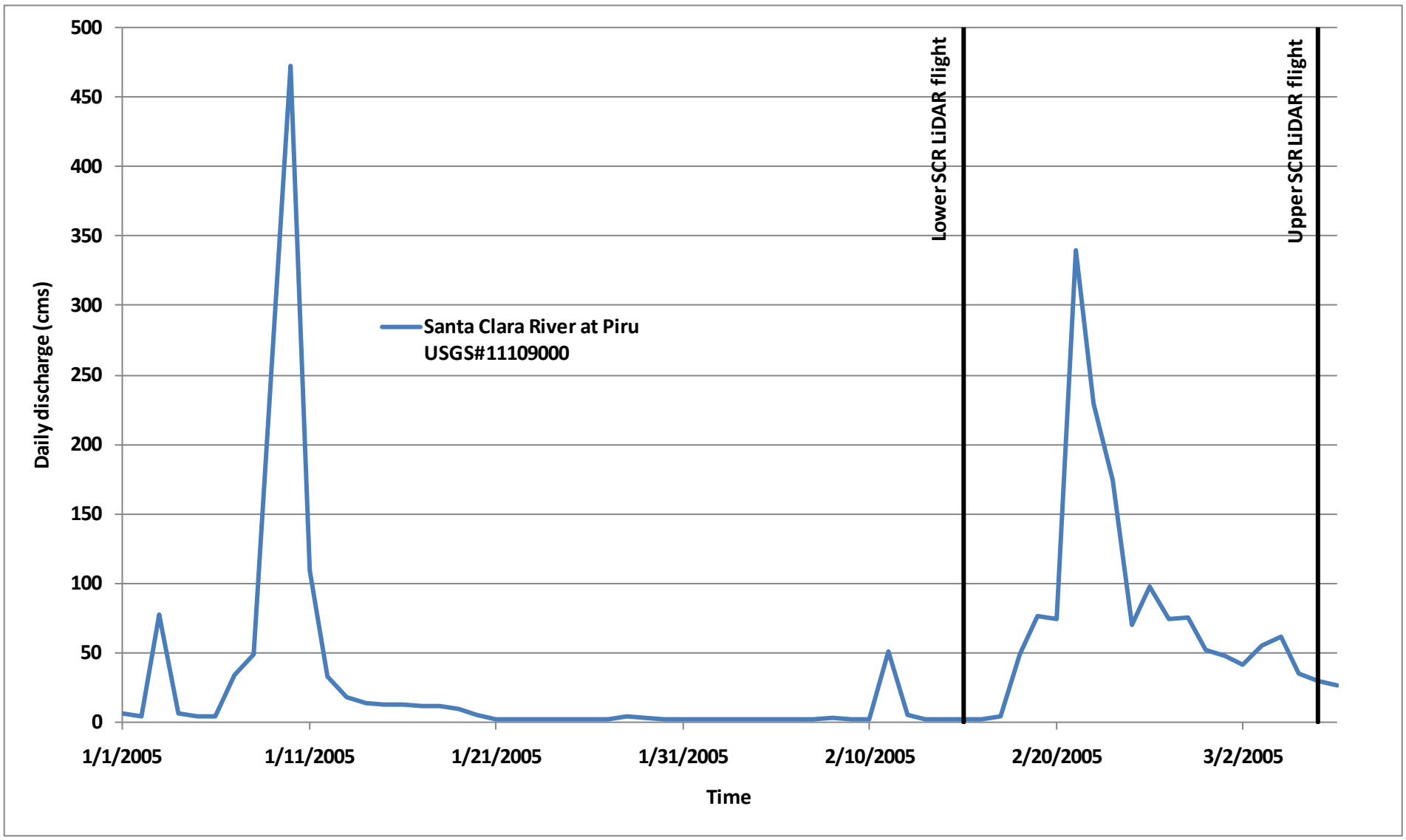

Notes: flight dates are approximate.

Santa Clara River Levee Setback Assessment

LiDAR flight timing

Project No. 09-1005 | Created By: AMS | **Figure 3**

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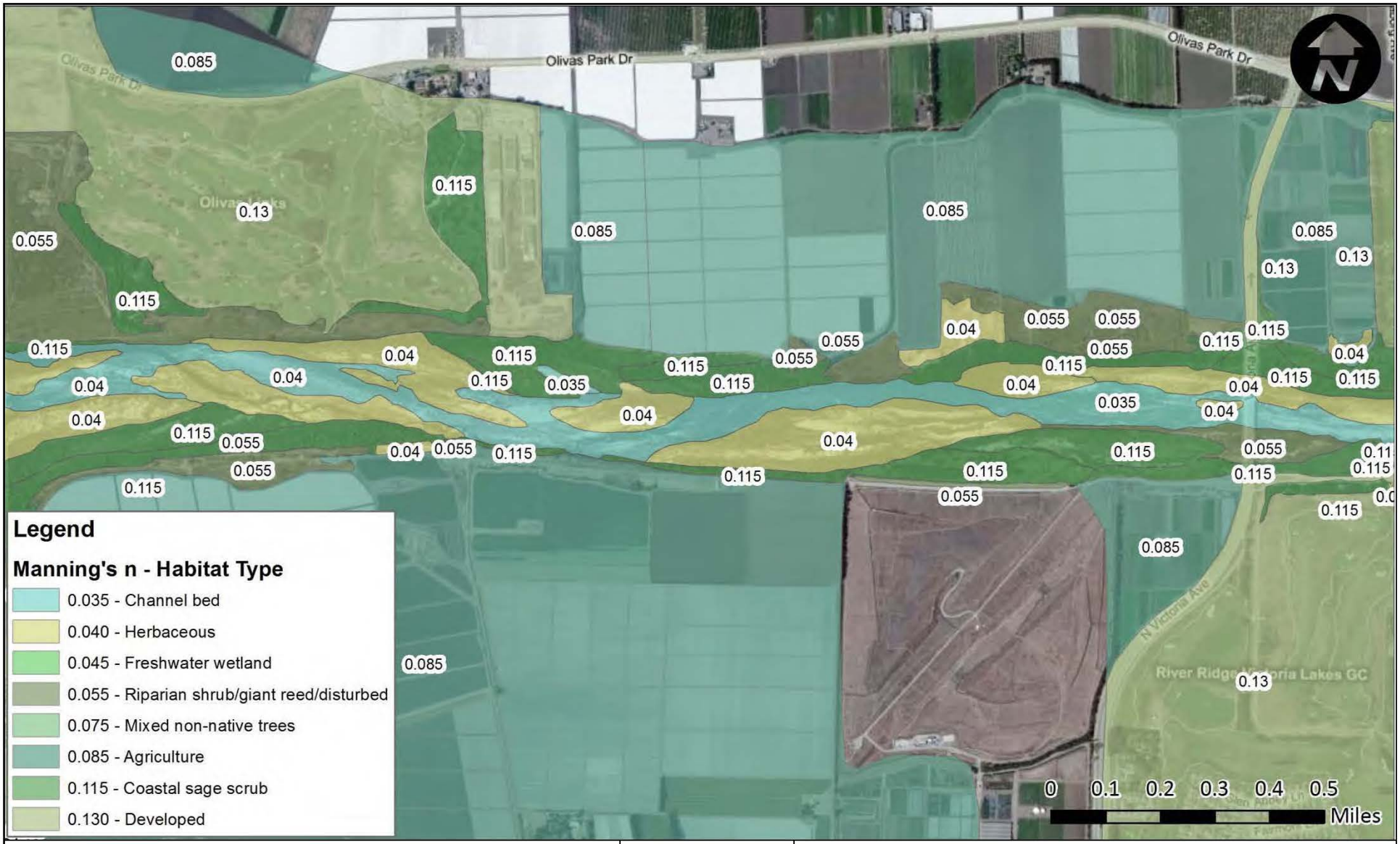

Notes: see Table 3 for habitat types corresponding to Manning's n values.
Vegetation mapping from Stillwater Sciences (2007); background image courtesy of BingMaps.

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Hydraulic roughness mapping

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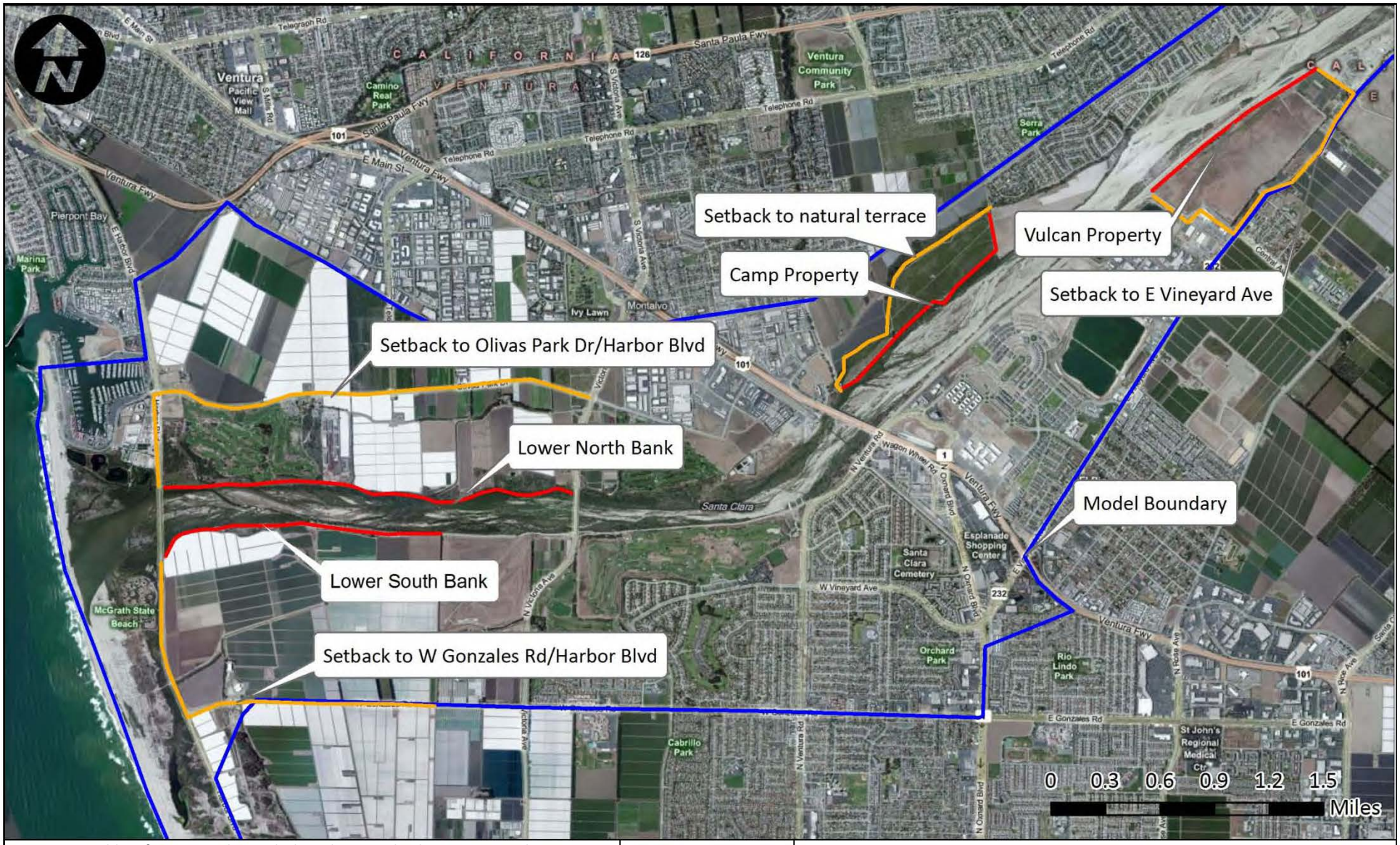

Notes: see Table 4 for run catalog with these levee setback scenarios; red are existing levees to be set back; orange are location of setback levees.

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Levee setback scenarios

Project No. 09-1005	Created By: AMS	**Figure 5**

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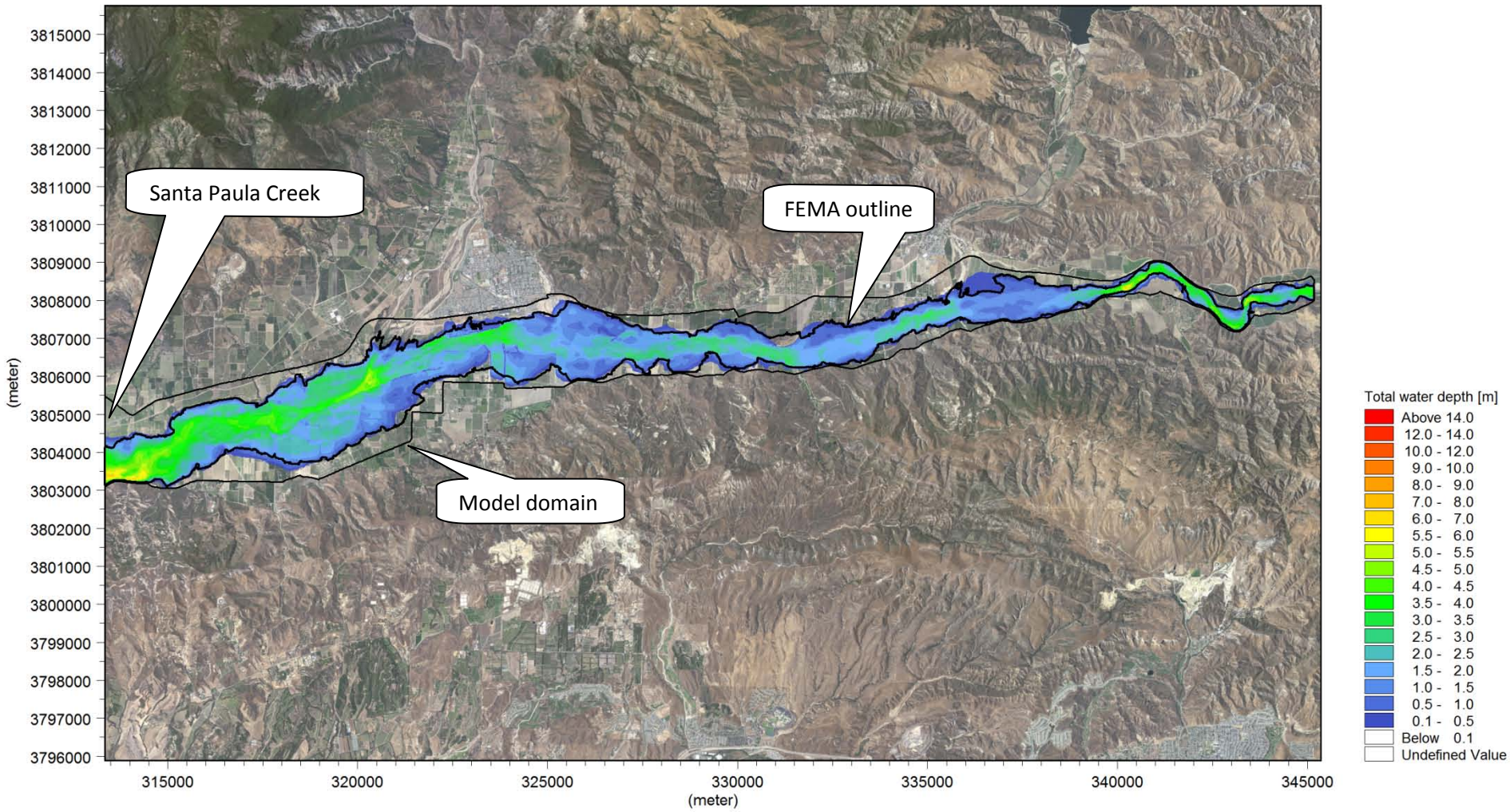

Notes: upper reach; inner black outline denotes the FEMA restudy RAS predicted inundation extents; projected coordinates in WGS1984 UTM11N.

Santa Clara River Levee Setback Assessment

Existing conditions Q100 comparison to FEMA mapping

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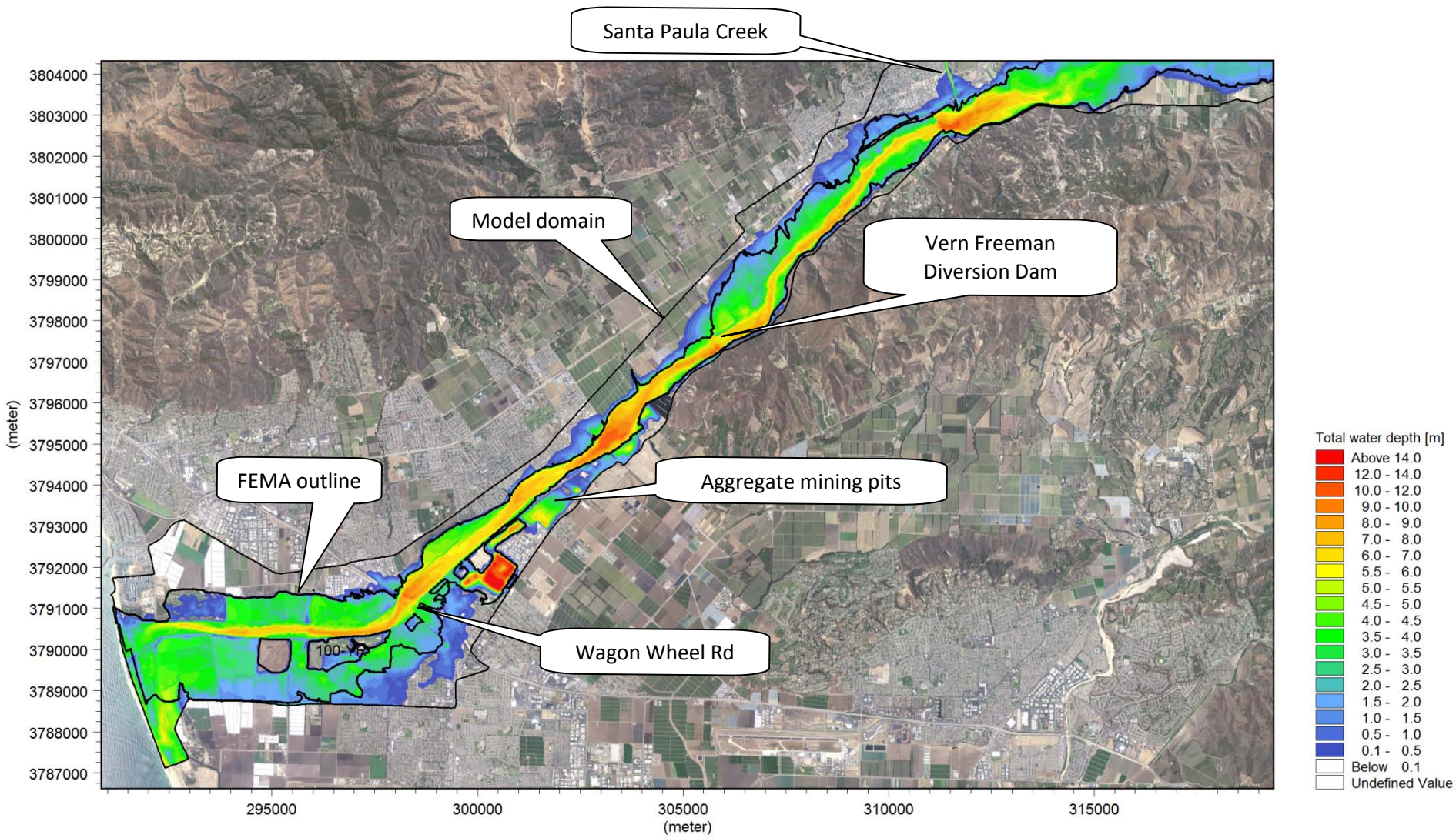

Notes: lower reach; inner black outline denotes the FEMA restudy RAS predicted inundation extents; projected coordinates in WGS1984 UTM11N.

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Existing conditions Q100 comparison to FEMA mapping

Project No. 09-1005	Created By: AMS	**Figure 6b**

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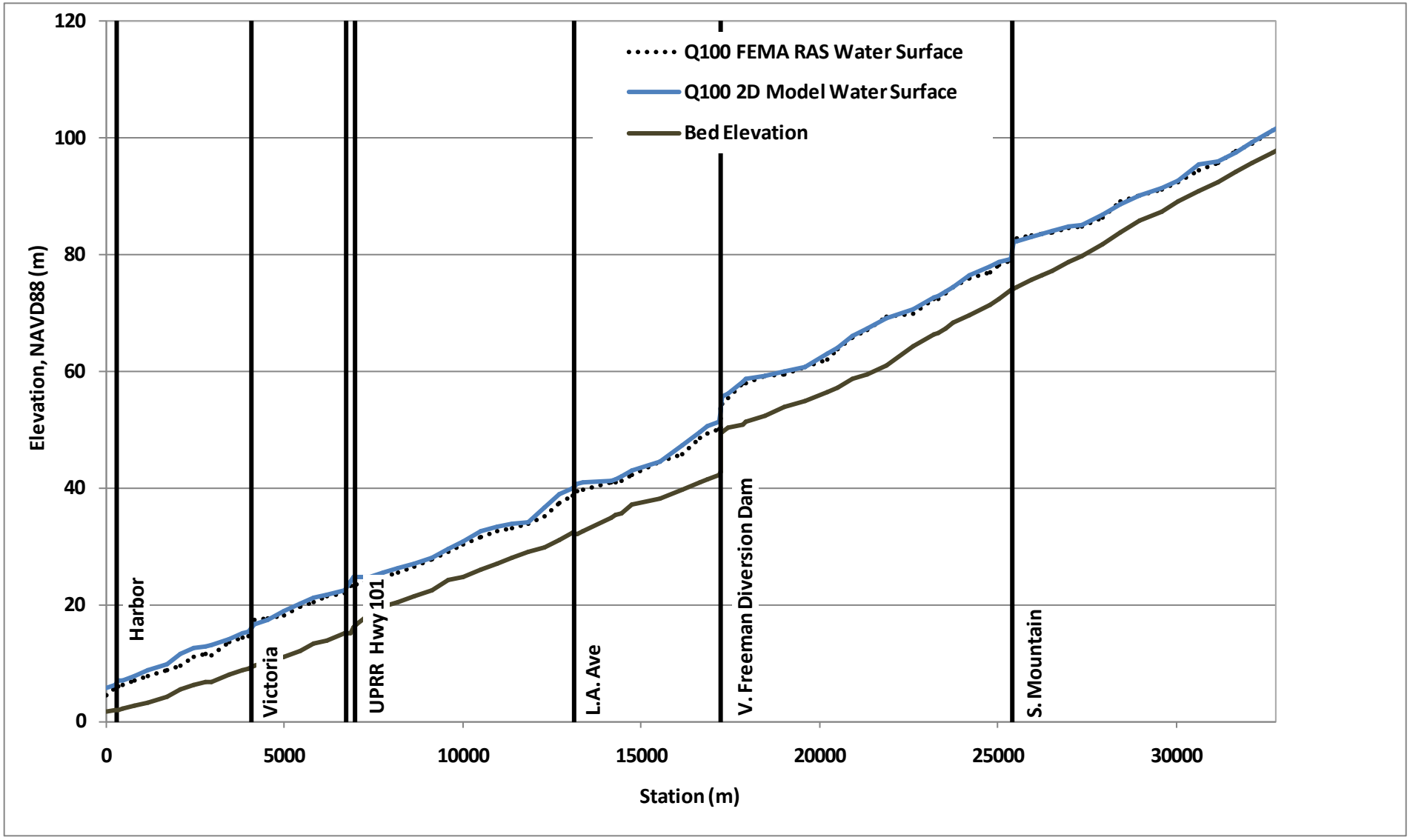

Notes:

Santa Clara River Levee Setback Assessment

Existing conditions Q100 comparison to FEMA profile

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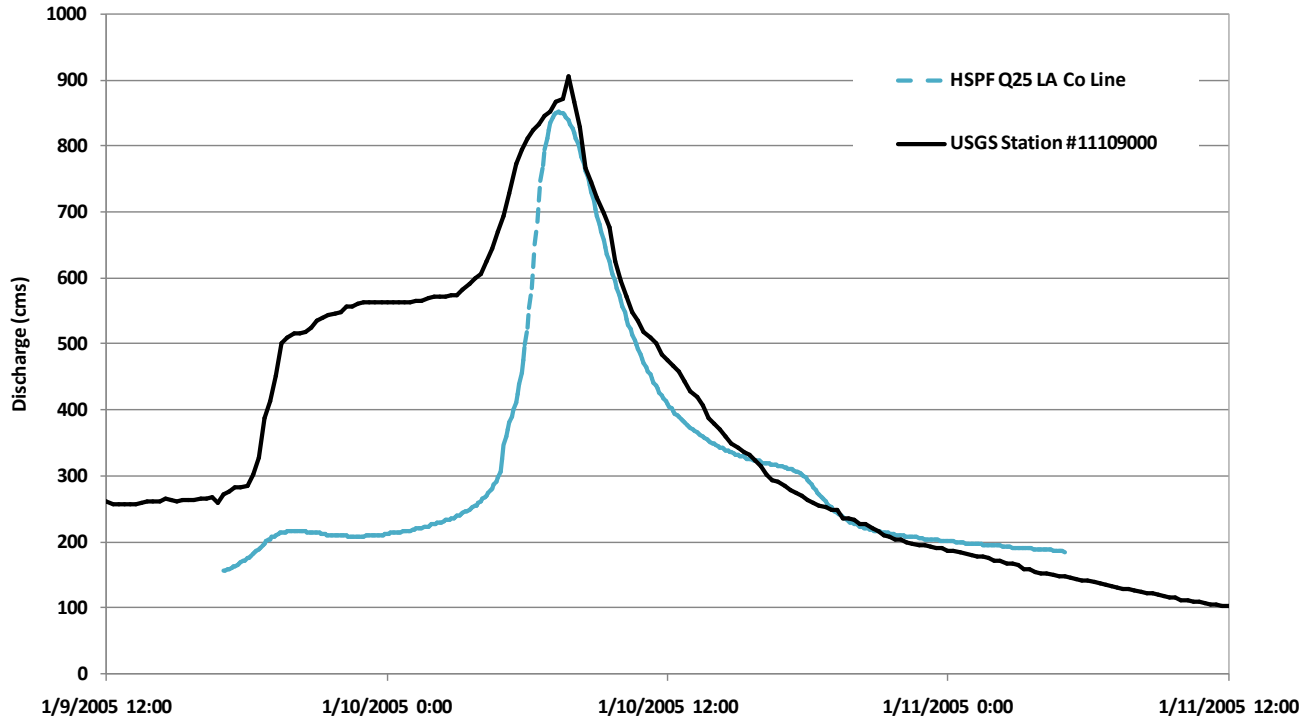

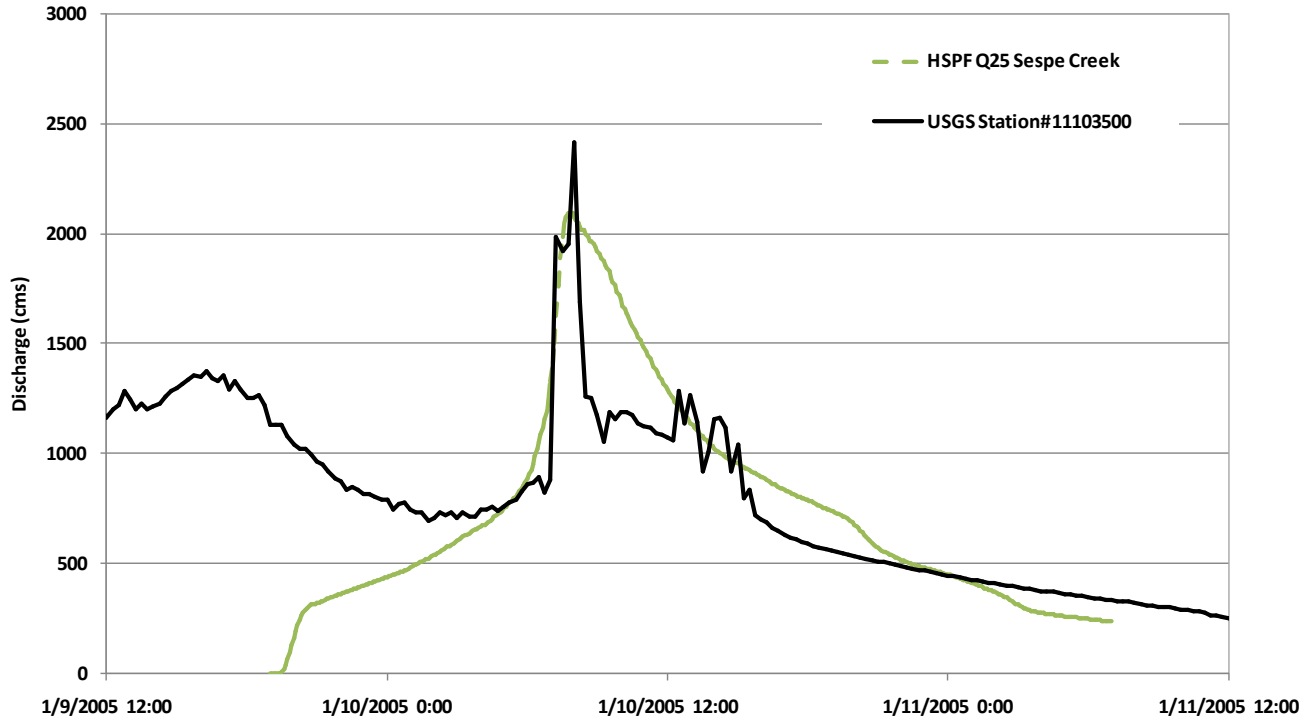

Notes:		*Santa Clara River Levee Setback Assessment*		
	cbec eco engineering	**January 2005 event-based hydrology**		
		Project No. 09-1005	Created By: AMS	**Figure 8**

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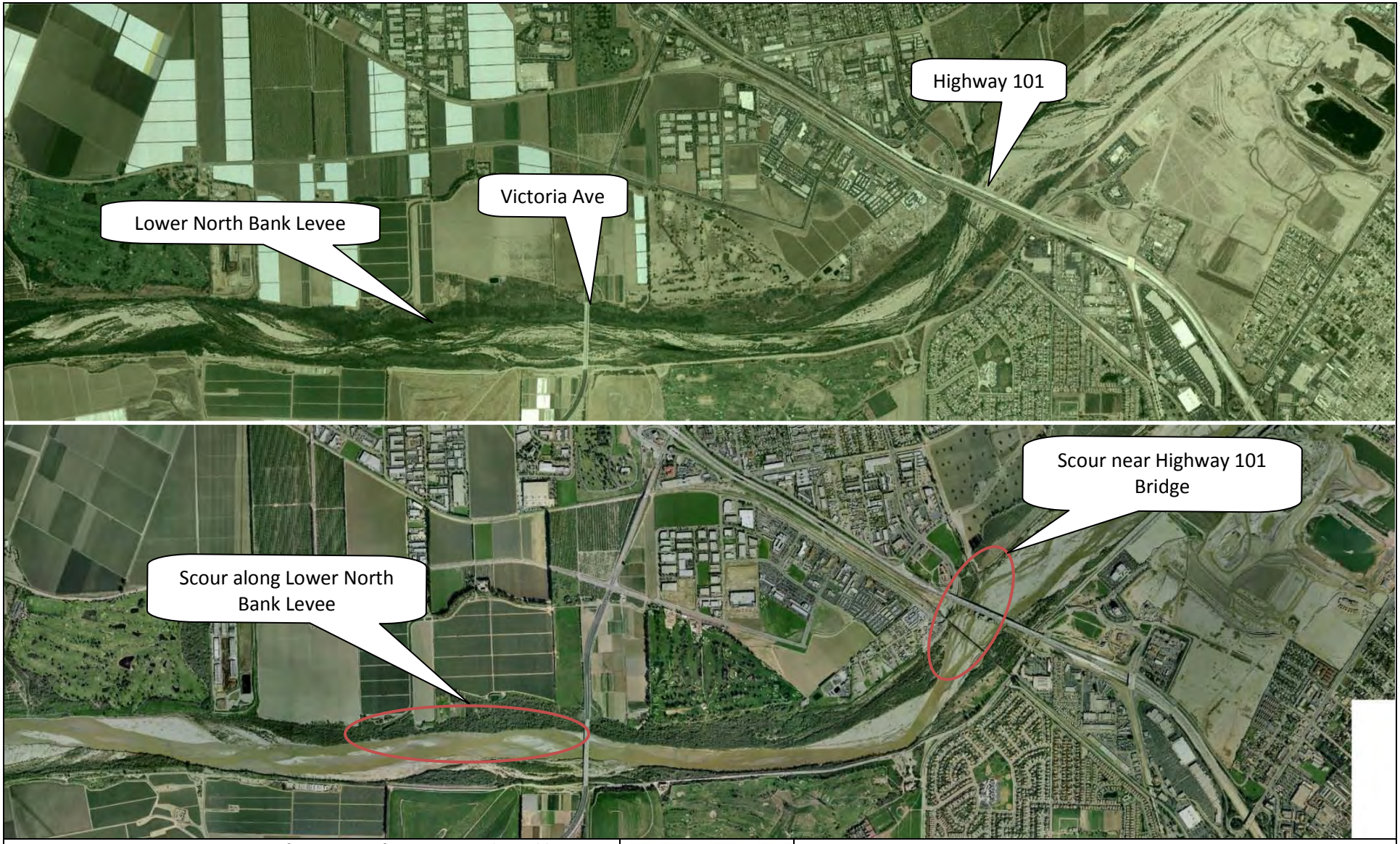

Notes: 2004 – 2005 NAIP images; significant scour of vegetation and possibly some berms in the reach from Highway 101 to Harbor Blvd.

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2004 to 2005 aerial photo comparison

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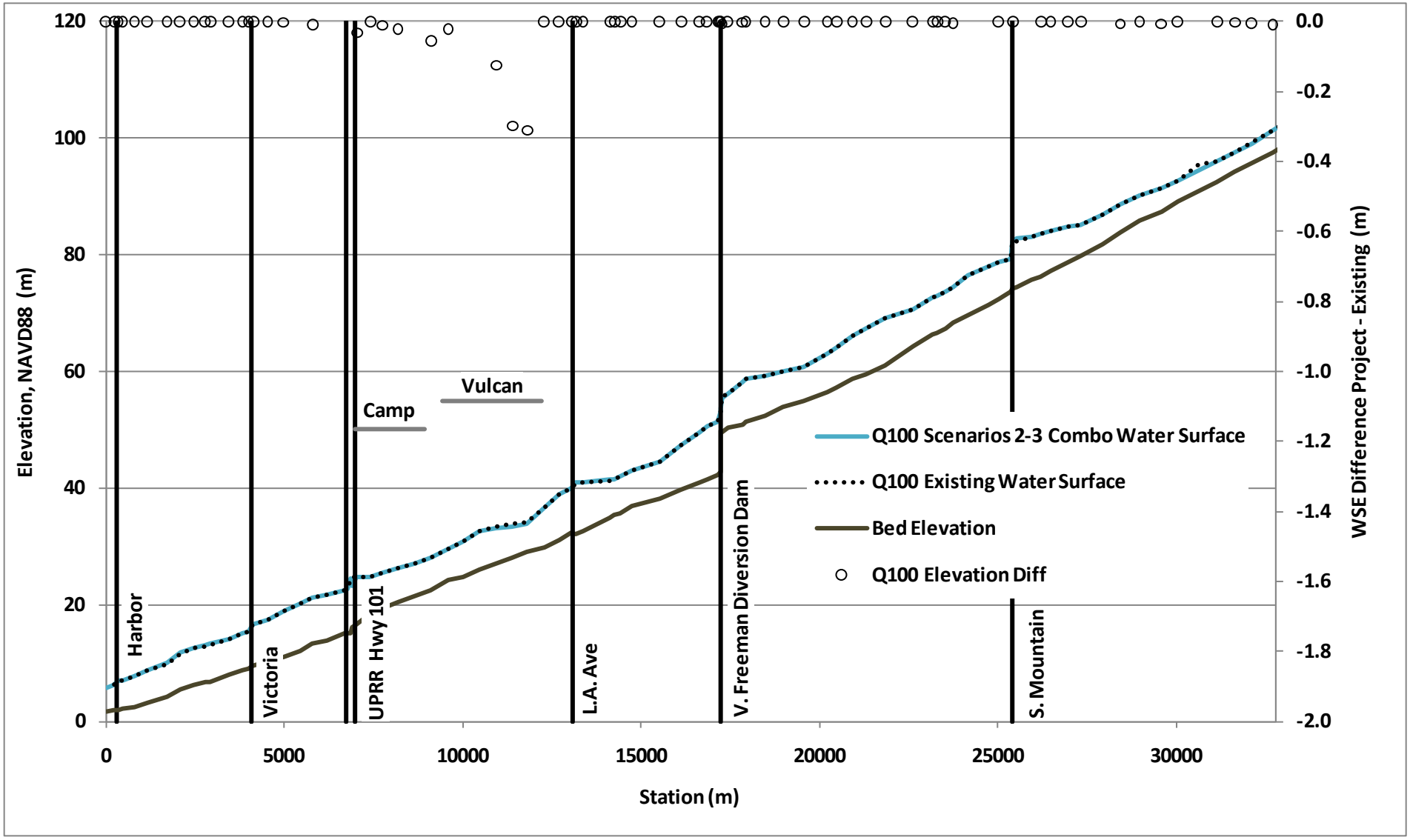

Notes: water surface elevation (WSE).

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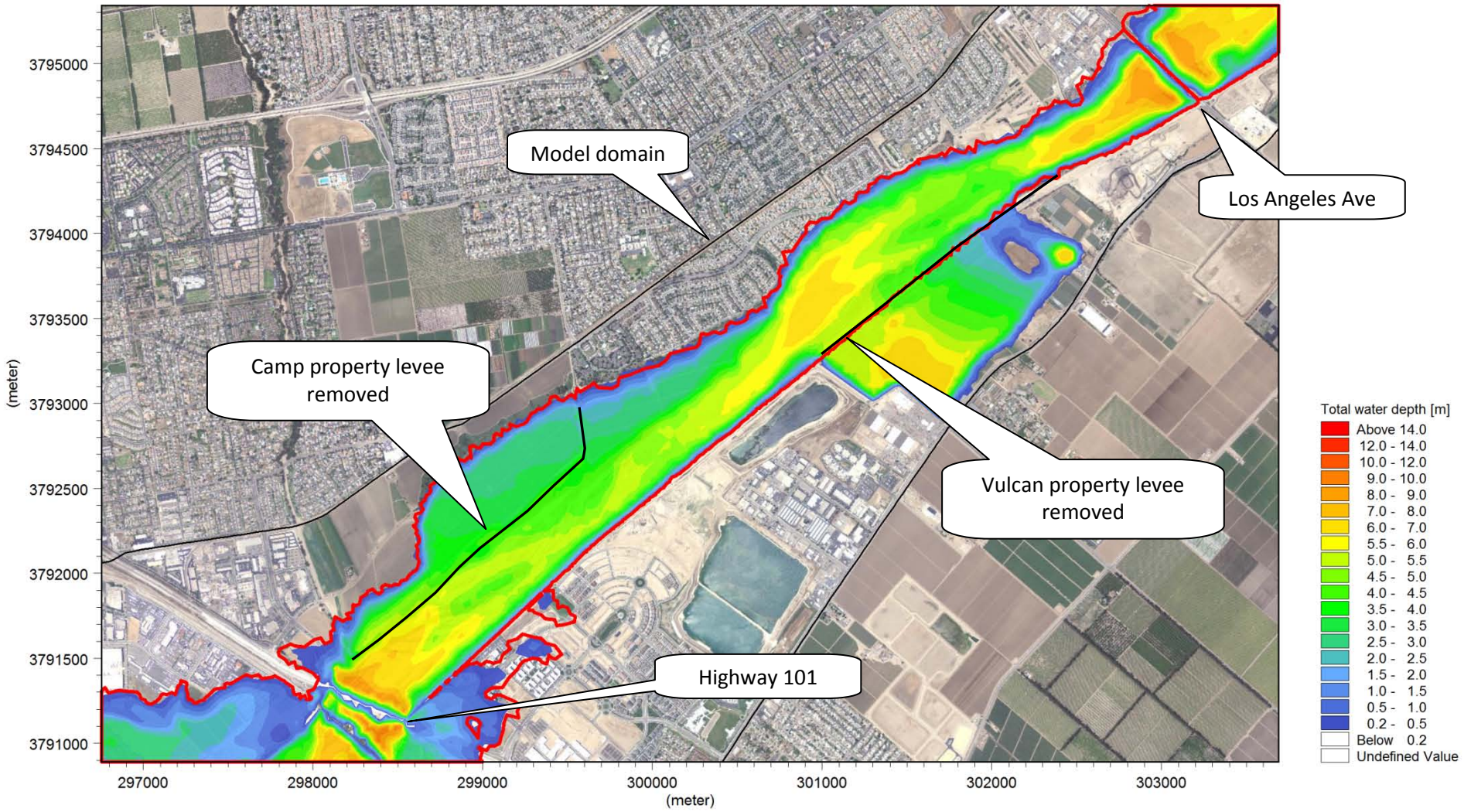
Q100 WSE comparison – Scenarios 2 and 3

Project No. 09-1005	Created By: AMS	**Figure 10**

R:\Projects\09-1005_Santa_Clara_River\Reporting\Figures\parts\Fig10_Q100_LevScen2-3_WSE.docx
8/24/2011



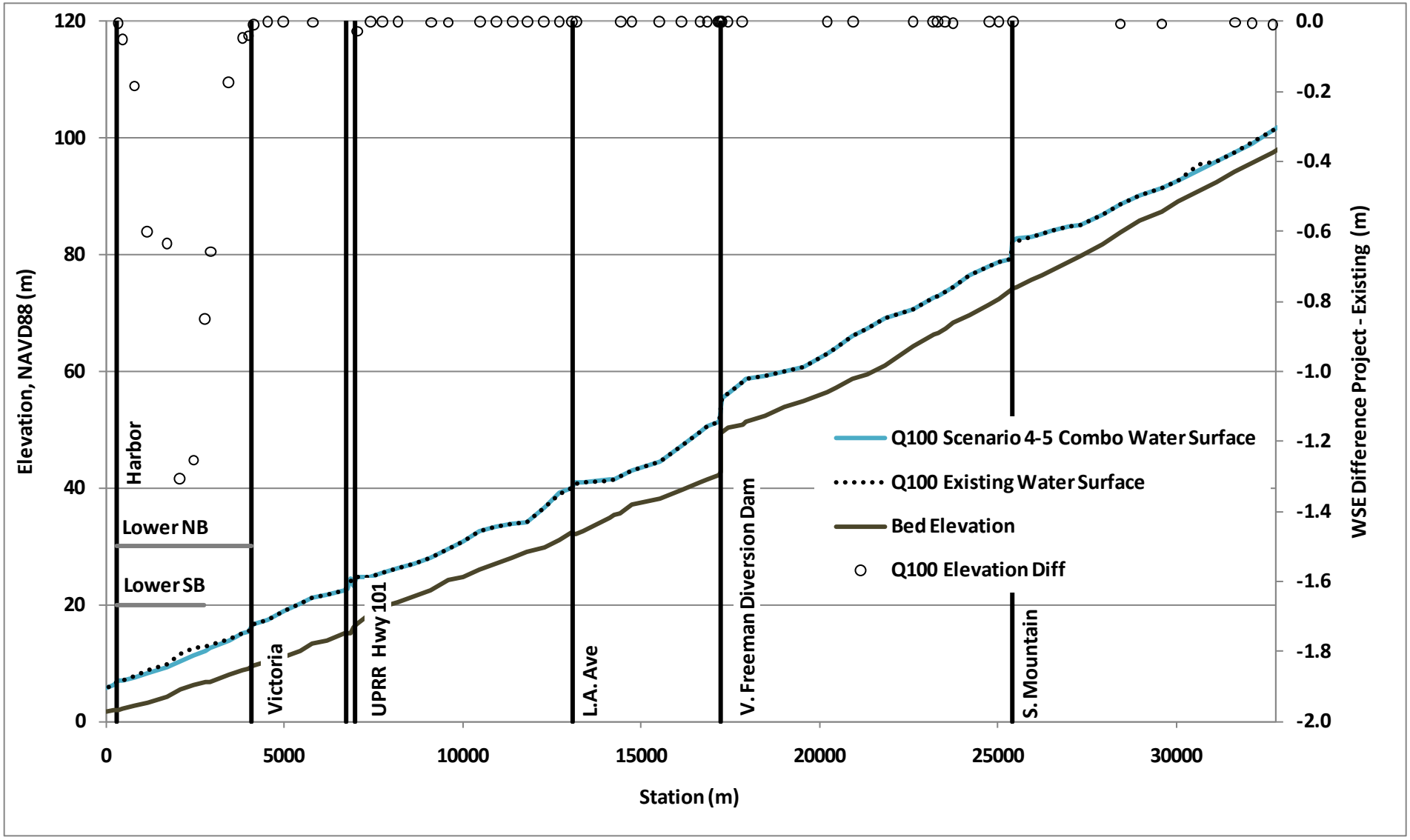
Notes: red outline denotes the Q100 existing conditions inundation extents; projected coordinates in WGS1984 UTM11N.

Santa Clara River Levee Setback Assessment

Q100 inundation mapping – Scenarios 2 and 3

Project No. 09-1005	Created By: AMS	**Figure 11**

R:\Projects\09-1005_Santa_Clara_River\Reporting\Figures\parts\Fig11_Q100_LevScen2-3_extents.docx
8/24/2011



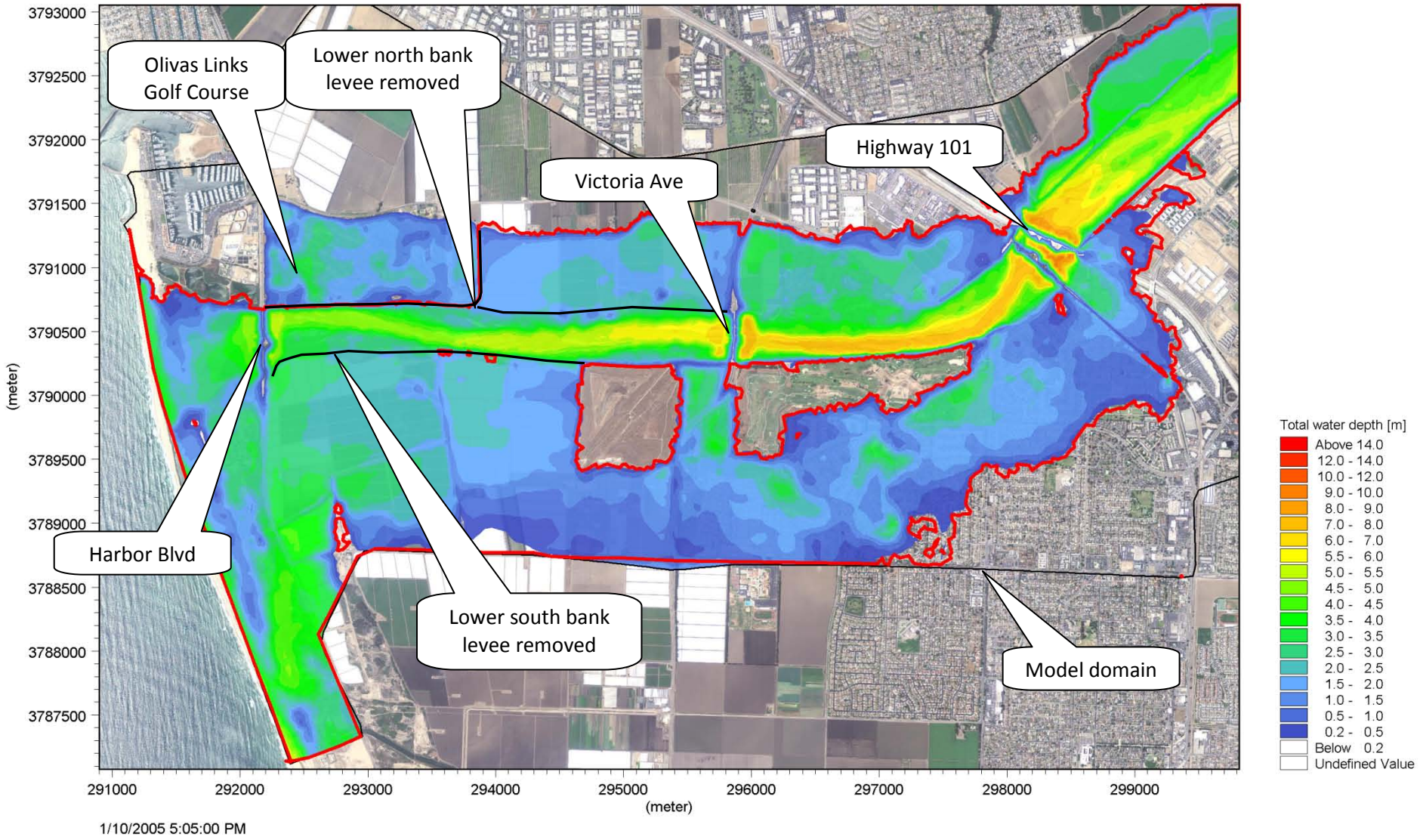
Notes: water surface elevation (WSE).

Santa Clara River Levee Setback Assessment

Q100 WSE comparison – Scenarios 4 and 5

Project No. 09-1005	Created By: AMS	**Figure 12**

R:\Projects\09-1005_Santa_Clara_River\Reporting\Figures\parts\Fig12_Q100_LevScen4-5_WSE.docx
8/24/2011



Notes: red outline denotes the Q100 existing conditions inundation extents; projected coordinates in WGS1984 UTM11N.

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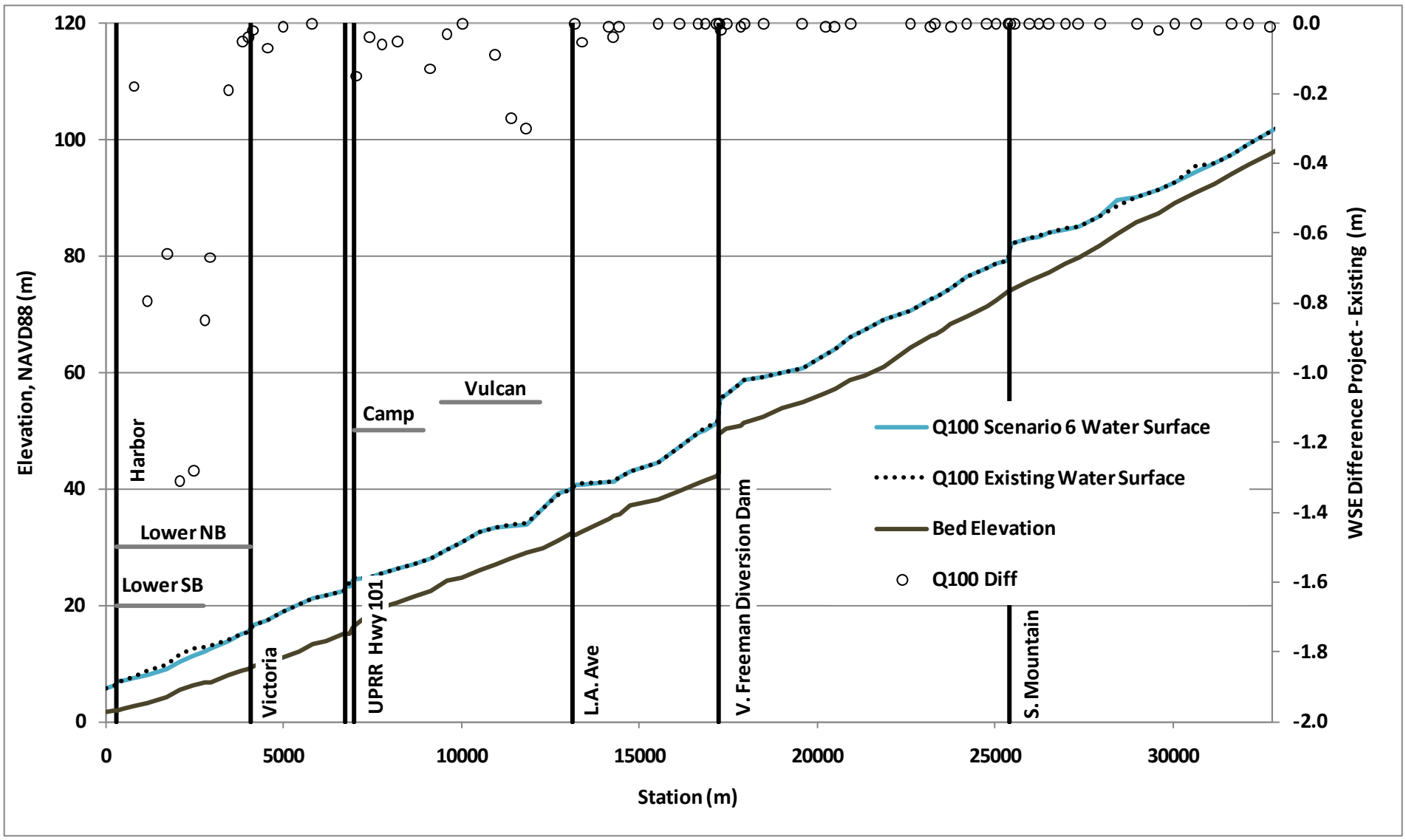
Santa Clara River Levee Setback Assessment

Q100 inundation mapping – Scenarios 4 and 5

Project No. 09-1005	Created By: AMS	**Figure 13**

R:\Projects\09-1005_Santa_Clara_River\Reporting\Figures\parts\Fig13_Q100_LevScen4-5_extents.docx
8/24/2011



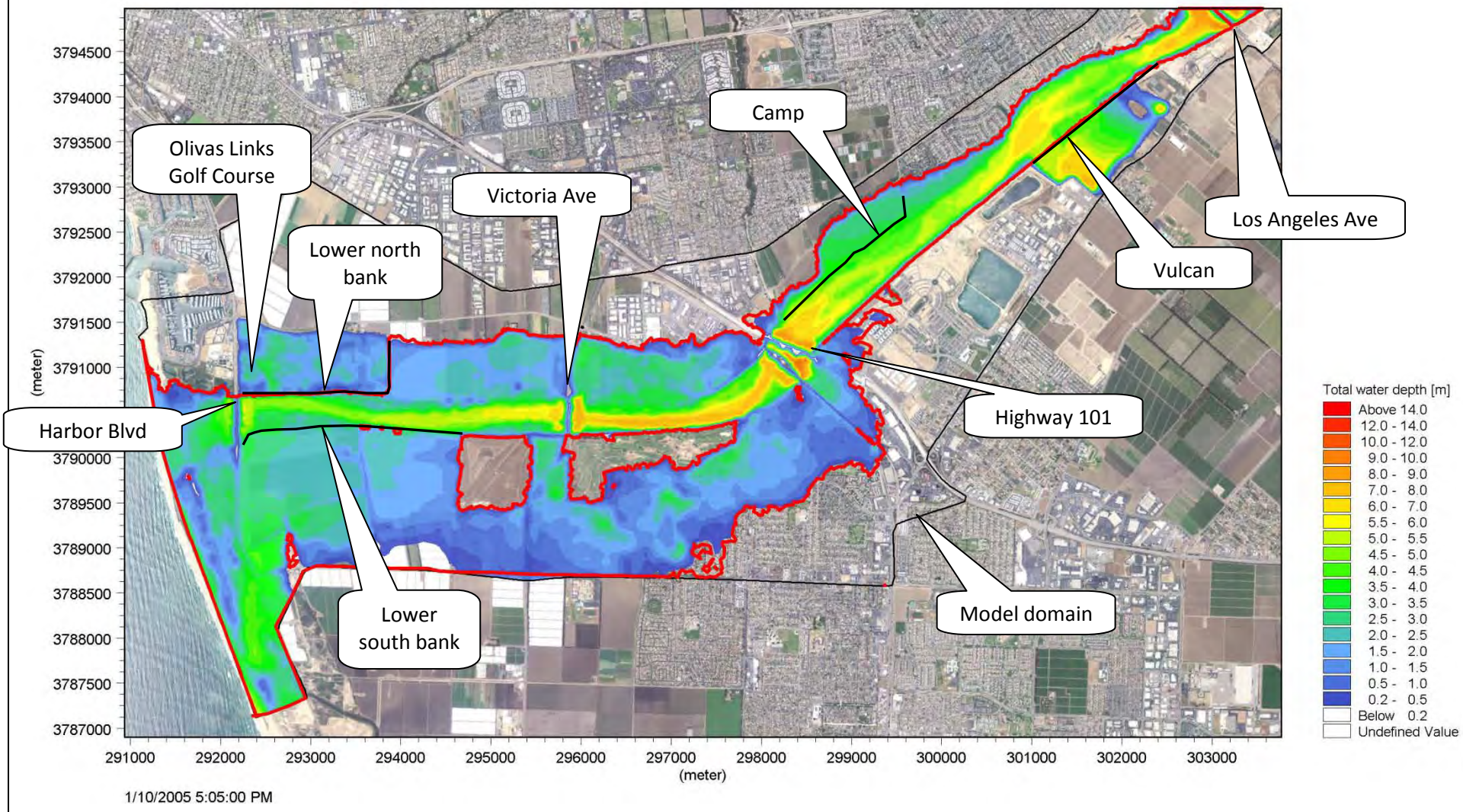
Notes: water surface elevation (WSE).

Santa Clara River Levee Setback Assessment

Q100 WSE comparison – Scenario 6

Project No. 09-1005	Created By: AMS	**Figure 14**

R:\Projects\09-1005_Santa_Clara_River\Reporting\Figures\parts\Fig14_Q100_LevScen6_WSE.docx
8/24/2011



Notes: red outline denotes the Q100 existing conditions inundation extents; projected coordinates in WGS1984 UTM11N.

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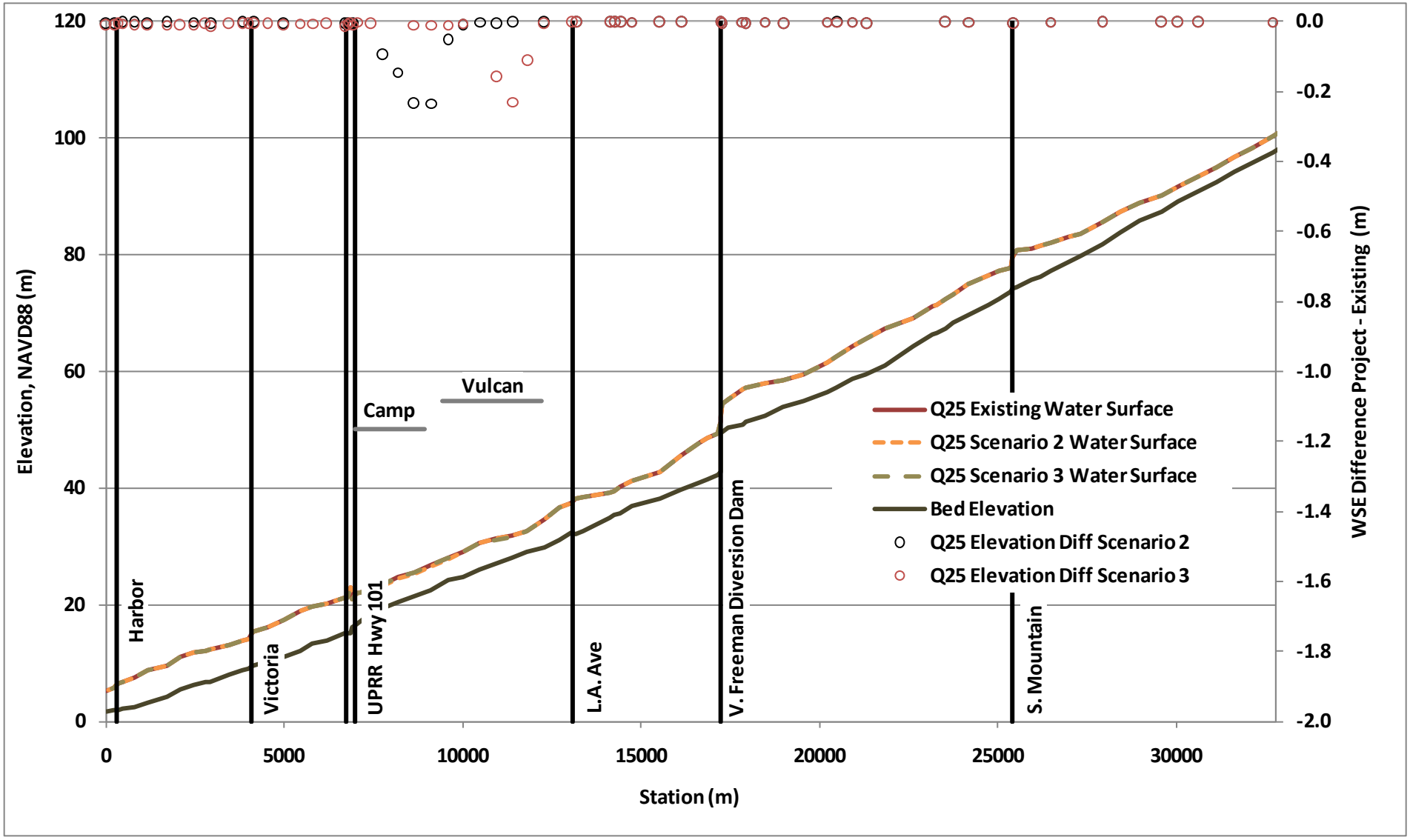
Q100 inundation mapping – Scenario 6

Project No. 09-1005	Created By: AMS	**Figure 15**

R:\Projects\09-1005_Santa_Clara_River\Reporting\Figures\parts\Fig15_Q100_LevScen6_extents.docx
8/24/2011



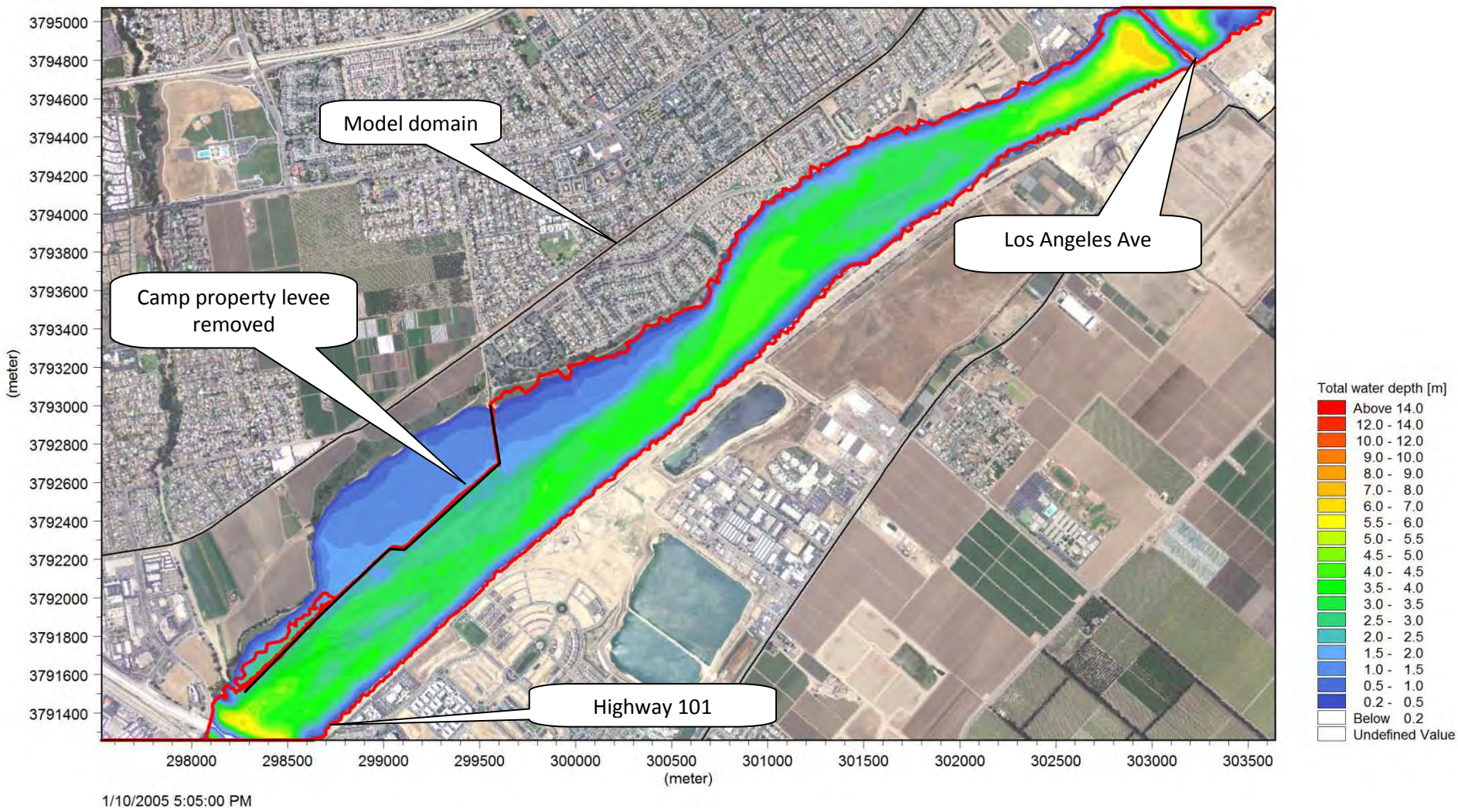
Notes: water surface elevation (WSE).

Santa Clara River Levee Setback Assessment

Q25 WSE comparison – Scenarios 2 and 3

Project No. 09-1005	Created By: AMS	**Figure 16**

R:\Projects\09-1005_Santa_Clara_River\Reporting\Figures\parts\Fig16_Q25_LevScen2-3_WSE.docx
8/24/2011



Notes: red outline denotes the Q25 existing conditions inundation extents; projected coordinates in WGS1984 UTM11N.

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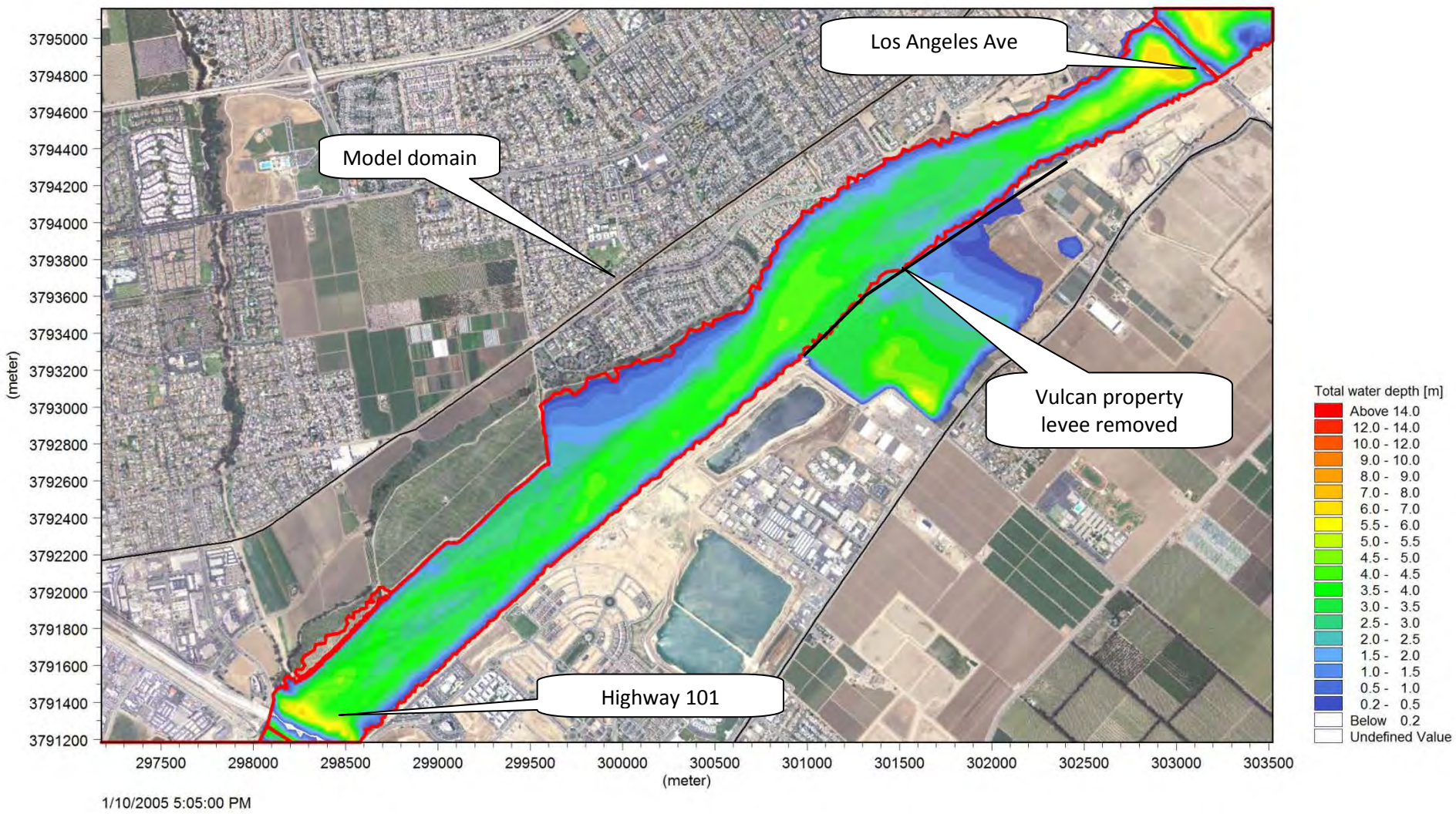
Q25 inundation mapping – Scenario 2

Project No. 09-1005	Created By: AMS	**Figure 17**

R:\Projects\09-1005_Santa_Clara_River\Reporting\Figures\parts\Fig17_Q25_LevScen2_extents.docx
8/24/2011



Notes: red outline denotes the Q25 existing conditions inundation extents; projected coordinates in WGS1984 UTM11N.

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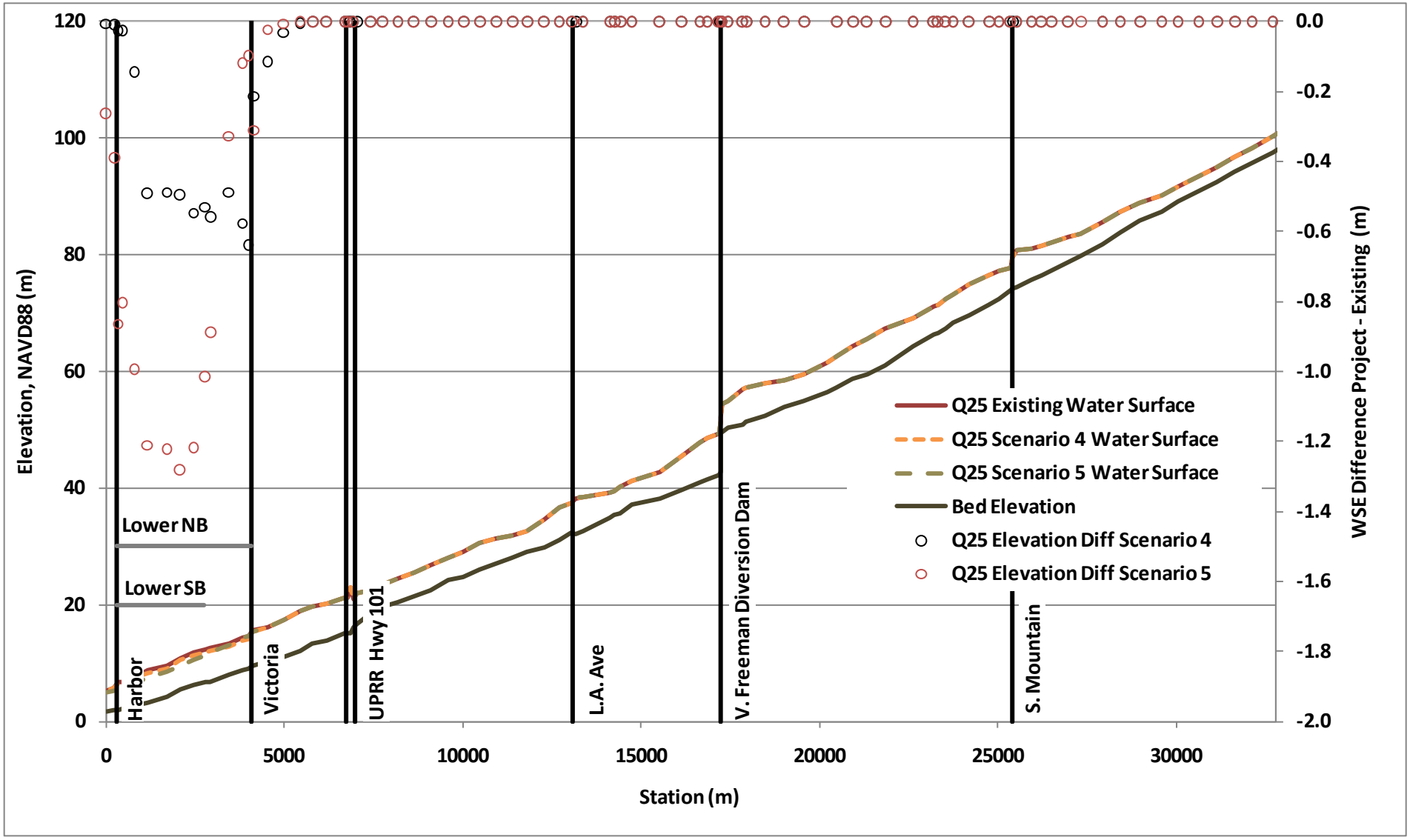
Q25 inundation mapping – Scenario 3

Project No. 09-1005	Created By: AMS	**Figure 18**

R:\Projects\09-1005_Santa_Clara_River\Reporting\Figures\parts\Fig18_Q25_LevScen3_extents_.docx
8/24/2011



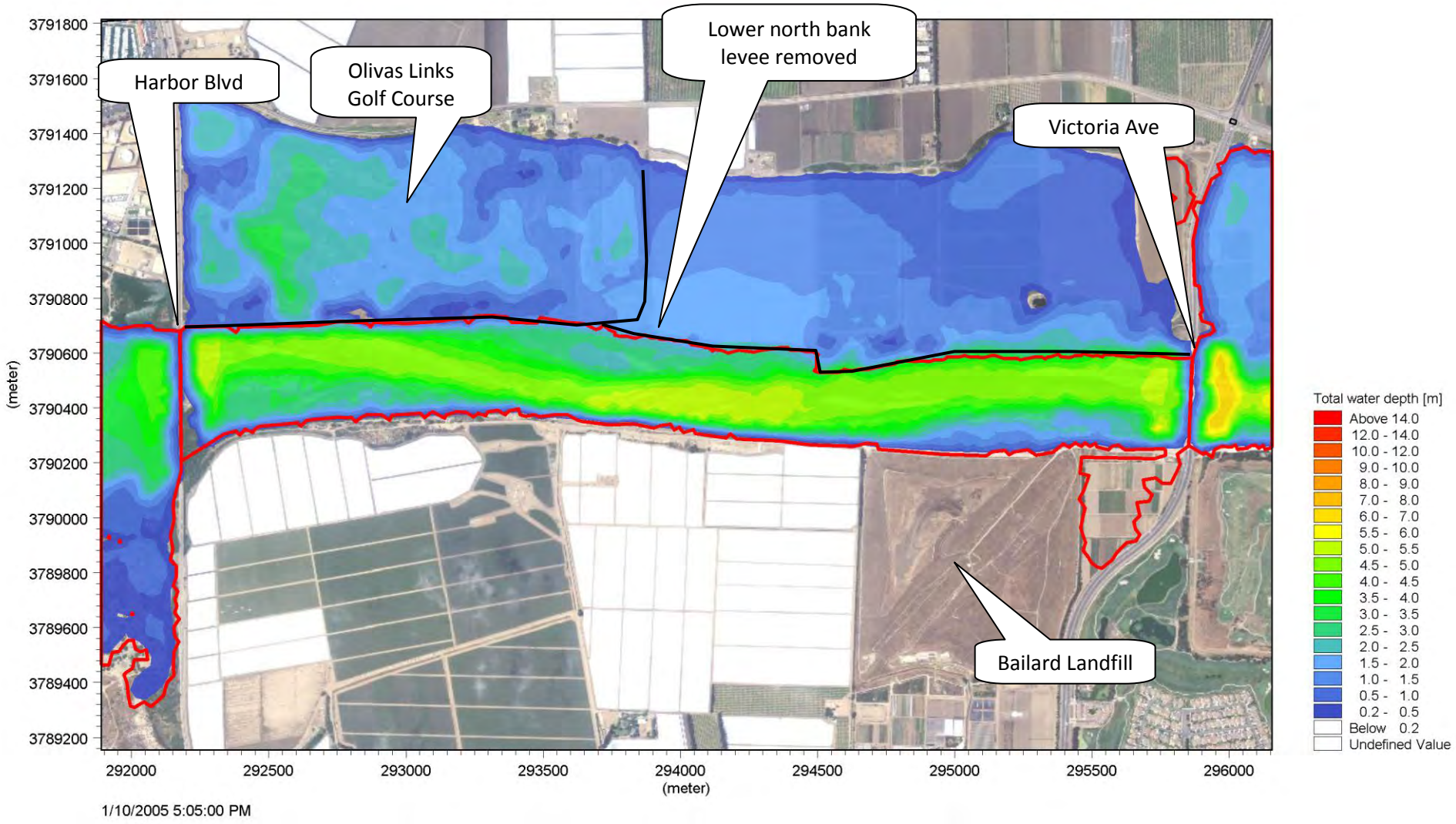
Notes: water surface elevation (WSE).

Santa Clara River Levee Setback Assessment

Q25 WSE comparison – Scenarios 4 and 5

Project No. 09-1005	Created By: AMS	**Figure 19**

R:\Projects\09-1005_Santa_Clara_River\Reporting\Figures\parts\Fig19_Q25_LevScen4-5_WSE.docx
8/24/2011



Notes: red outline denotes the Q25 existing conditions inundation extents; projected coordinates in WGS1984 UTM11N.

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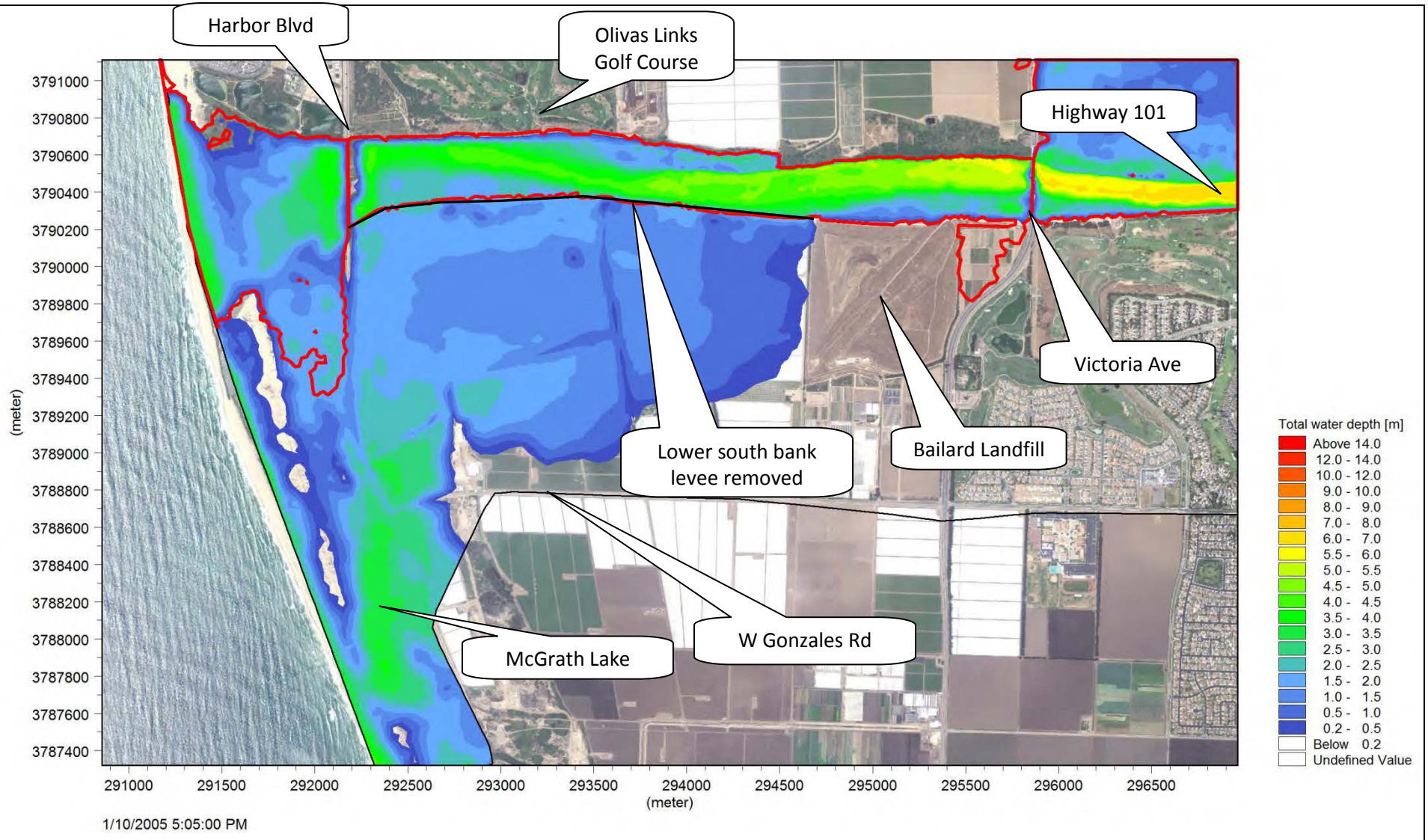
Q25 inundation mapping – Scenario 4

Project No. 09-1005	Created By: AMS	**Figure 20**

R:\Projects\09-1005_Santa_Clara_River\Reporting\Figures\parts\Fig20_Q25_LevScen4_extents.docx
8/24/2011



Notes: red outline denotes the Q25 existing conditions inundation extents; projected coordinates in WGS1984 UTM11N.

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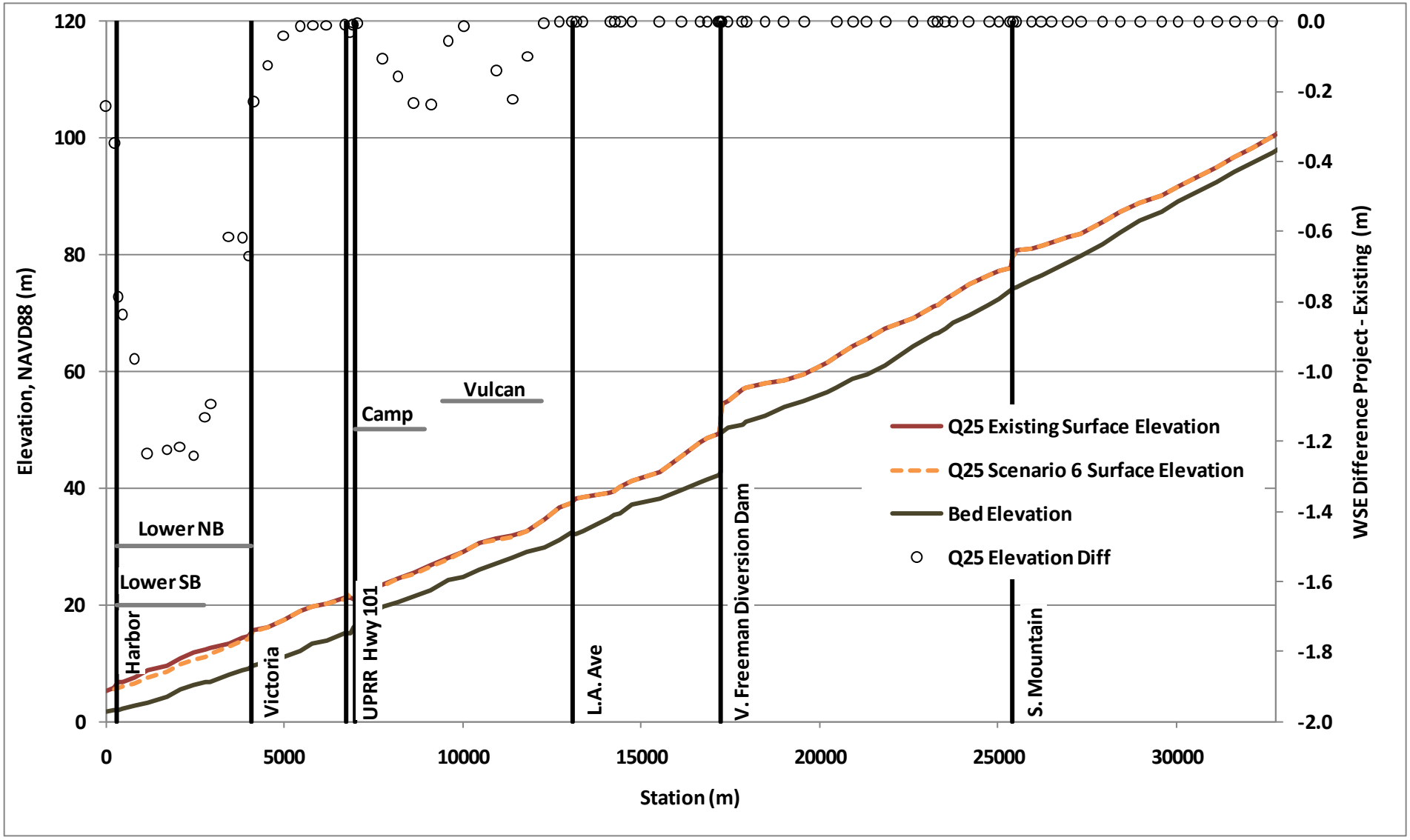
Q25 inundation mapping – Scenario 5

Project No. 09-1005	Created By: AMS	**Figure 21**

R:\Projects\09-1005_Santa_Clara_River\Reporting\Figures\parts\Fig21_Q25_LevScen5_extents.docx
8/24/2011



Notes: water surface elevation (WSE).

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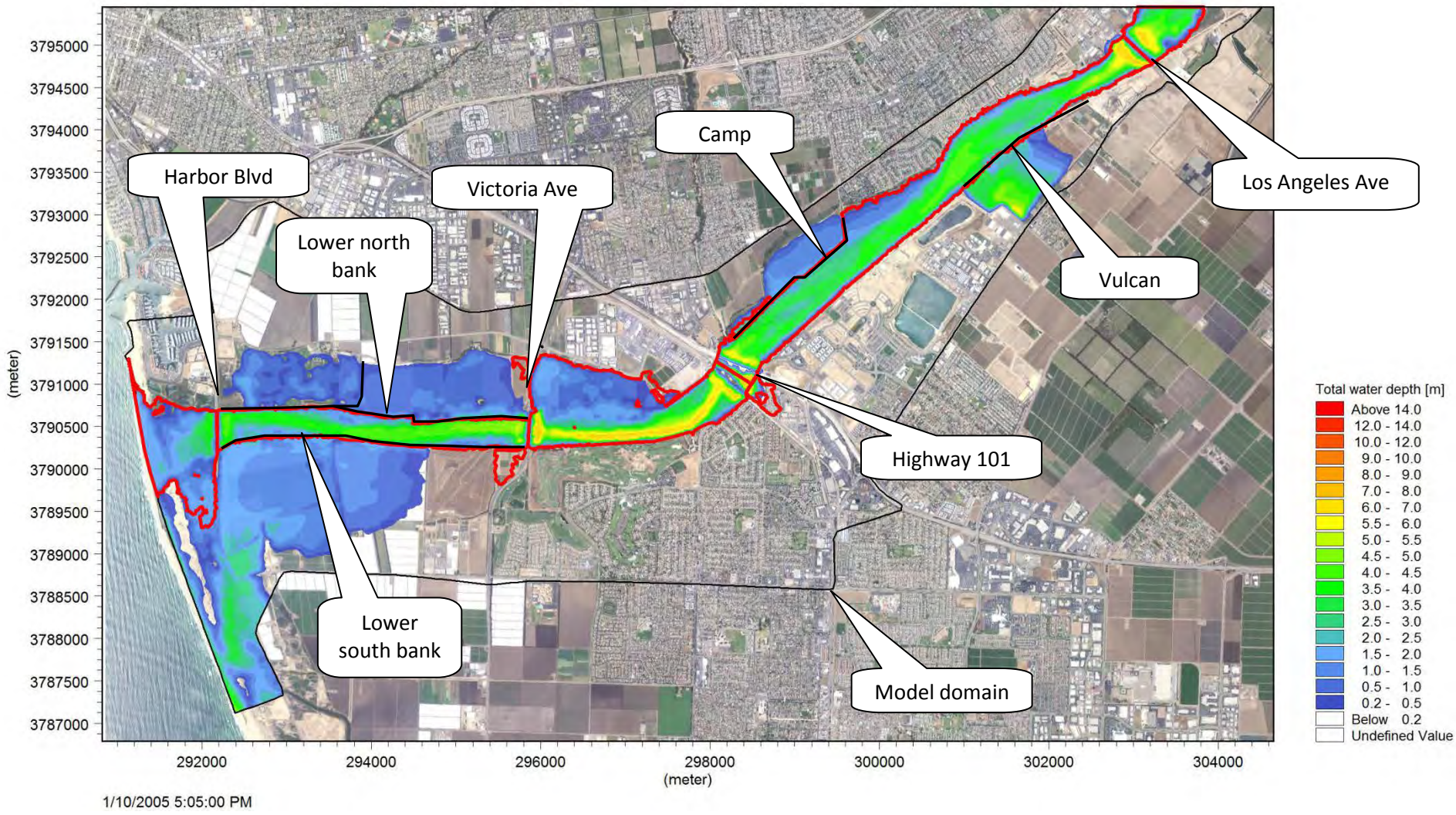
Santa Clara River Levee Setback Assessment

Q25 WSE comparison – Scenario 6

Project No. 09-1005	Created By: AMS	**Figure 22**

R:\Projects\09-1005_Santa_Clara_River\Reporting\Figures\parts\Fig22_Q25_LevScen6_WSE.docx
8/24/2011



Notes: red outline denotes the Q25 existing conditions inundation extents; projected coordinates in WGS1984 UTM11N.

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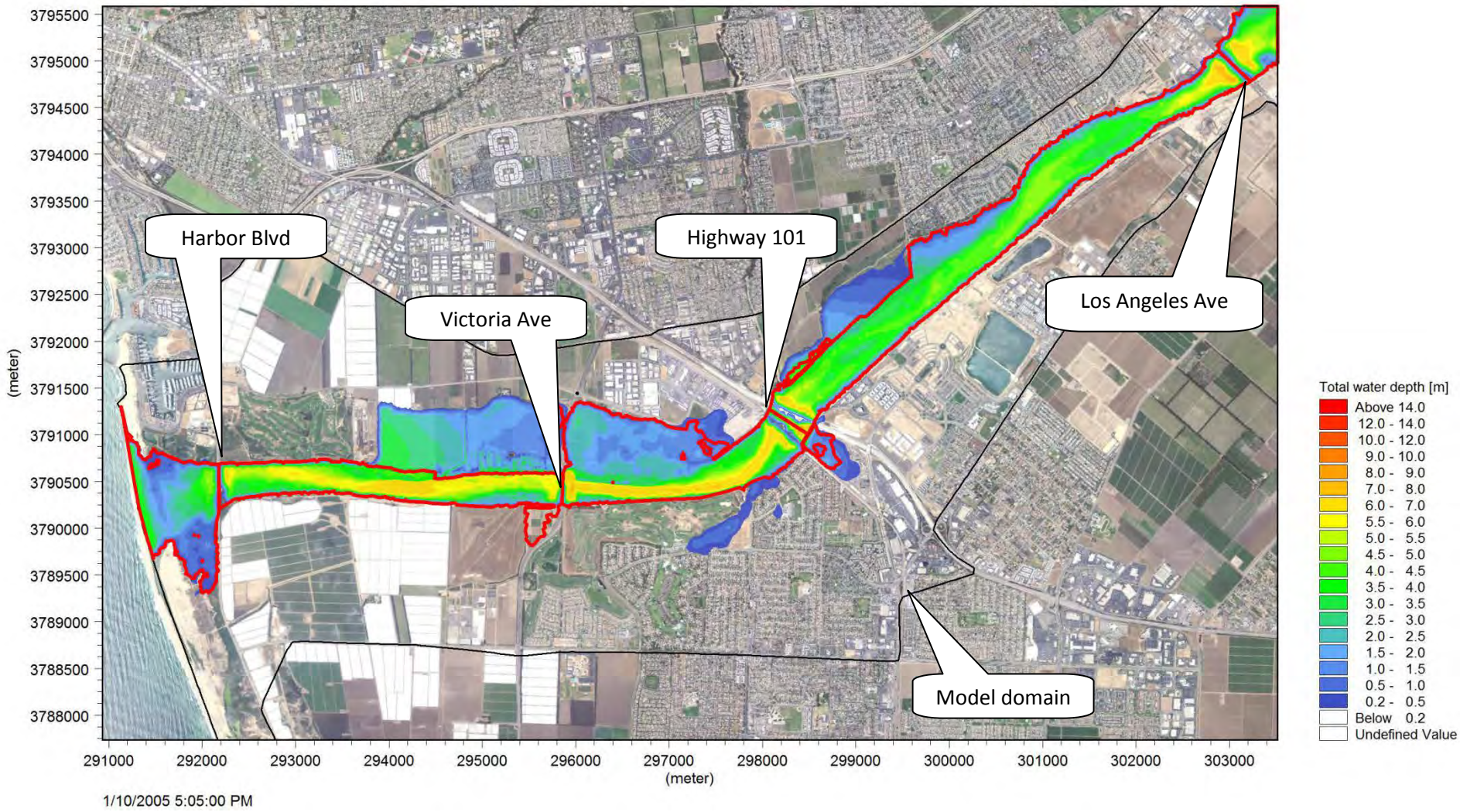
Santa Clara River Levee Setback Assessment

Q25 inundation mapping – Scenario 6

Project No. 09-1005	Created By: AMS	**Figure 23**

R:\Projects\09-1005_Santa_Clara_River\Reporting\Figures\parts\Fig23_Q25_LevScen6_extents.docx
8/24/2011



Notes: red outline denotes the Q25 existing hydrology and topography conditions inundation extents; projected coordinates in WGS1984 UTM11N.

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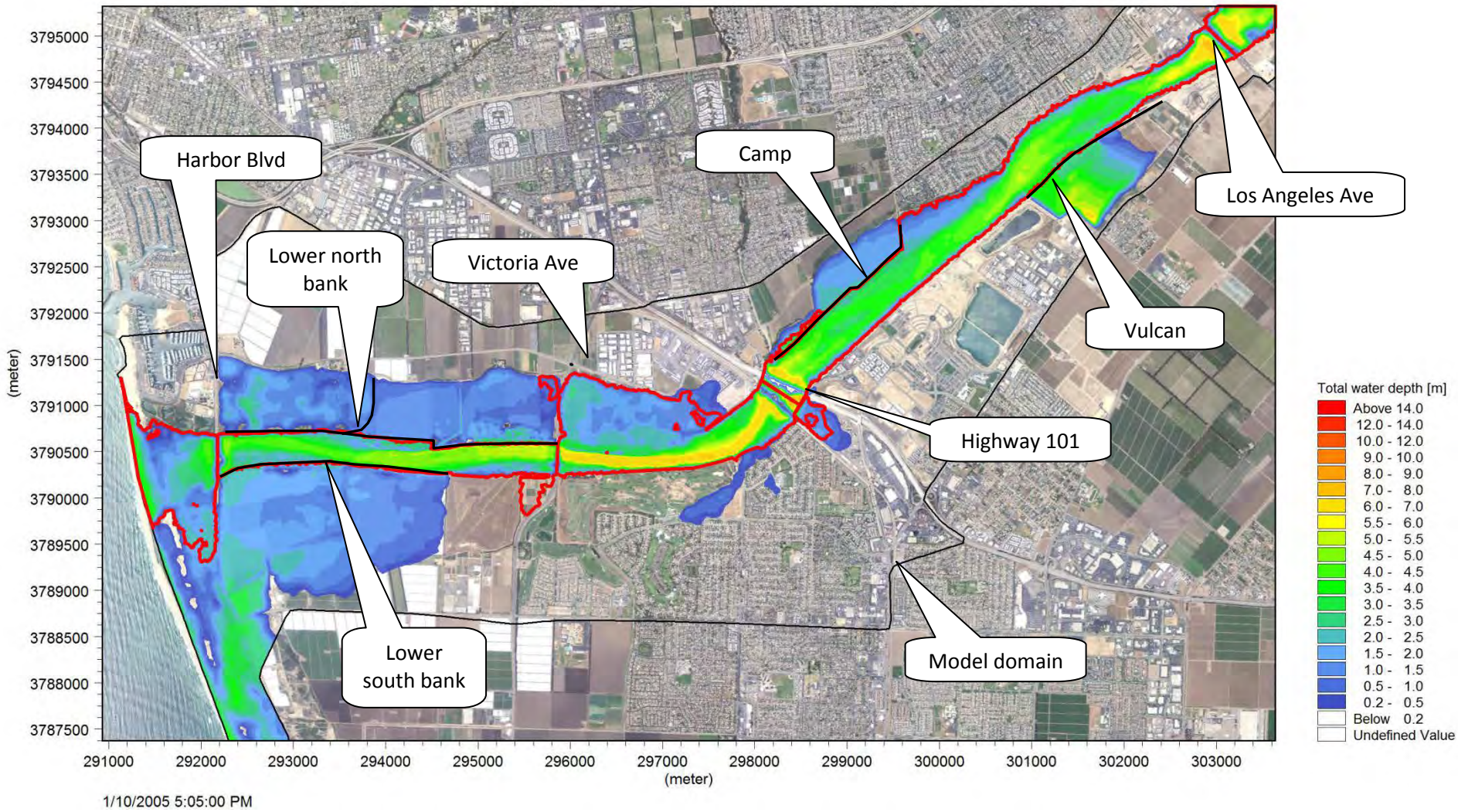
Santa Clara River Levee Setback Assessment

Future Q25 inundation mapping - Existing

Project No. 09-1005	Created By: AMS	**Figure 24**

R:\Projects\09-1005_Santa_Clara_River\Reporting\Figures\parts\Fig24_Q25_FUT_Ex_extents.docx
8/24/2011



Notes: red outline denotes the Q25 existing hydrology and topography conditions inundation extents; projected coordinates in WGS1984 UTM11N.

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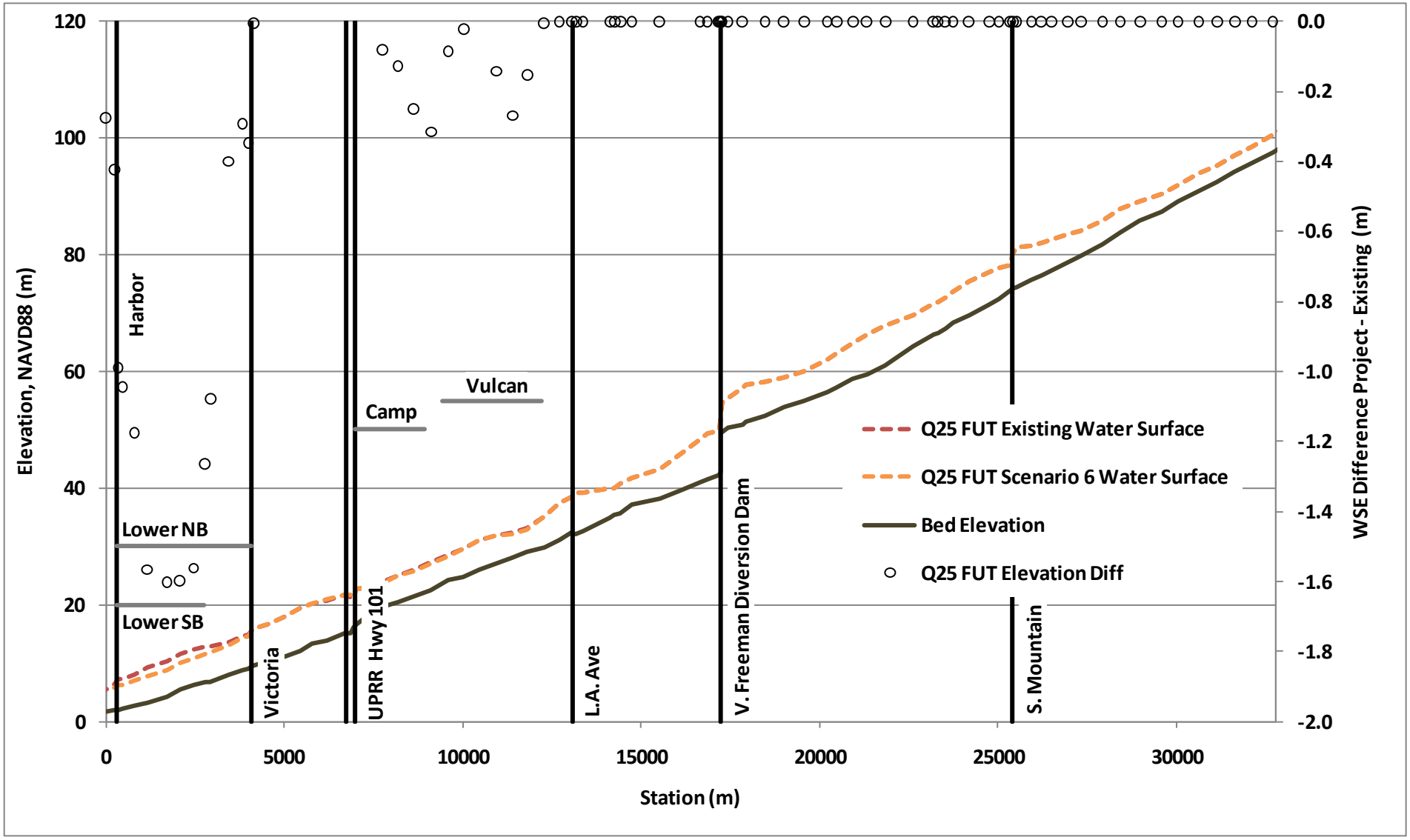
Santa Clara River Levee Setback Assessment

Future Q25 inundation mapping – Scenario 6

Project No. 09-1005	Created By: AMS	**Figure 25**

R:\Projects\09-1005_Santa_Clara_River\Reporting\Figures\parts\Fig25_Q25_FUT_NC_extents.docx
8/24/2011



Notes: water surface elevation (WSE).

Santa Clara River Levee Setback Assessment

Future Q25 WSE comparison – Scenario 6

Project No. 09-1005	Created By: AMS	**Figure 26**

R:\Projects\09-1005_Santa_Clara_River\Reporting\Figures\parts\Fig26_Q25_LevScen6_WSE.docx
8/24/2011