NIST GCR 12-917-19

Program Plan for the Development of Seismic Design Guidelines for Port Container, Wharf, and Cargo Systems

NEHRP Consultants Joint Venture
A partnership of the Applied Technology Council and the Consortium of Universities for Research in Earthquake Engineering

National Institute of Standards and Technology
U.S. Department of Commerce

Disclaimers

This report was prepared for the Engineering Laboratory of the National Institute of Standards and Technology (NIST) under the National Earthquake Hazards Reduction Program (NEHRP) Earthquake Structural and Engineering Research Contract SB134107CQ0019, Task Order 68001. The contents of this publication do not necessarily reflect the views and policies of NIST or the U.S. Government.

This report was produced by the NEHRP Consultants Joint Venture, a partnership of the Applied Technology Council (ATC) and the Consortium of Universities for Research in Earthquake Engineering (CUREE). While endeavoring to provide practical and accurate information, the NEHRP Consultants Joint Venture, the authors, and the reviewers assume no liability, nor express or imply any warranty for the information contained herein. Users of information contained in this report assume all liability arising from such use.

Unless otherwise noted, photos, figures, and data presented in this report have been developed or provided by NEHRP Consultants, Joint Venture staff or consultants engaged under contract to provide information as works for hire. Any similarity with other published information is coincidental. Photos and figures cited from outside sources have been reproduced in this report with permission. Any other use requires additional permission from the copyright holders.

Certain commercial software, equipment, instruments, or materials may have been used in the preparation of information contributing to this report. Identification in this report is not intended to imply recommendation or endorsement by NIST, nor is it intended to imply that such software, equipment, instruments, or materials are necessarily the best available for the purpose.

NIST policy is to use the International System of Units (metric units) in all its publications. In this report, however, some information is presented in U.S. Customary Units (inch-pound), as this is the preferred system of units in the U.S. earthquake engineering industry.

Cover photo – Rendering of Container Wharf (Courtesy of Moffatt & Nichol)

NIST GCR 12-917-19

Program Plan for the Development of Seismic Design Guidelines for Port Container, Wharf, and Cargo Systems

Prepared for
U.S. Department of Commerce
National Institute of Standards and Technology
Engineering Laboratory
Gaithersburg, MD 20899

By
NEHRP Consultants Joint Venture
A partnership of the Applied Technology Council and the
Consortium of Universities for Research in Earthquake Engineering

September 2012

U.S. Department of Commerce
Rebecca M. Blank, Acting Secretary

National Institute of Standards and Technology
Patrick D. Gallagher, Under Secretary of Commerce
for Standards and Technology, and Director

NIST GCR 12-917-19

Participants

National Institute of Standards and Technology

John (Jack) R. Hayes, Jr., Director, National Earthquake Hazards Reduction Program
Steven L. McCabe, Deputy Director, National Earthquake Hazards Reduction Program

NEHRP Consultants Joint Venture

Applied Technology Council
201 Redwood Shores Parkway, Suite 240
Redwood City, California 94065
www.ATCouncil.org

Consortium of Universities for
Research in Earthquake Engineering
1301 S. 46th Street, Building 420
Richmond, California 94804
www.CUREE.org

Joint Venture Management Committee

James R. Harris
Robert Reitherman
Christopher Rojahn
Andrew Whittaker

Joint Venture Program Committee

Jon A. Heintz (Program Manager)
Michael Constantinou
C.B. Crouse
James R. Harris
William T. Holmes
Jack Moehle
Andrew Whittaker

Project Technical Committee

Richard Wittkop (Project Technical Director)
Omar Jarradat
Gayle Johnson
Geoffrey Martin
Glenn Rix
Ian Robertson

Project Manager

William L. Coulbourne

Preface

The NEHRP Consultants Joint Venture is a partnership between the Applied Technology Council (ATC) and the Consortium of Universities for Research in Earthquake Engineering (CUREE). In 2007, the National Institute of Standards and Technology (NIST) awarded a National Earthquake Hazards Reduction Program (NEHRP) "Earthquake Structural and Engineering Research" contract (SB1341-07-CQ-0019) to the NEHRP Consultants Joint Venture to conduct a variety of tasks, including Task Order 68001 entitled "Development of Seismic Design Guidelines for Port and Harbor Facilities: Phase I." The objective of this project was to develop a program plan for establishing nationally-accepted guidelines for assessing and mitigating seismic risks in critical port and harbor facilities, focusing primarily on containerized shipping activities.

Work on this project was intended to be an extension of a National Science Foundation (NSF), George E. Brown, Jr. Network for Earthquake Engineering Simulation (NEES) Grand Challenge project, "Seismic Risk Mitigation for Port Systems," conducted by the Georgia Institute of Technology. The Grand Challenge project was motivated by the observation that many large U.S. container ports are located in areas of significant seismic hazard, and that seismic risk management practices have not kept pace with the growing importance of container ports to the nation's economy. That effort resulted in a systems-based approach for seismic risk analysis and risk management for container port facilities, and demonstrated how the post-earthquake functionality of a port system is also dependent upon on the functionality of other key elements of the infrastructure including roadways, railways, power supply, natural gas, and telecommunications systems.

This report provides the basis for a multi-phase program for the development of nationally-accepted guidelines for seismic performance improvement of container cargo systems including cranes, wharves, and container storage yards. It summarizes the scope and content of a series of recommended guidance documents, outlines the studies necessary to support their development, and presents the estimated cost and schedule associated with the overall program.

The NEHRP Consultants Joint Venture is indebted to the leadership of Richard Wittkop, Project Director, and to the members of the Project Technical Committee, consisting of Omar Jarradat, Gayle Johnson, Geoffrey Martin, Glenn Rix, and Ian Robertson, for their contributions in developing this report and the resulting recommendations. The names and affiliations of all who contributed to this report are provided in the list of Project Participants.

The NEHRP Consultants Joint Venture also gratefully acknowledges Jack Hayes and Steve McCabe (NIST) for their input and guidance in the preparation of the report, Bill Coulbourne for ATC project management, and Peter N. Mork for ATC report production services.

Jon A. Heintz
Program Manager

Table of Contents

List of Figures

List of Tables

Executive Summary

Extensive ongoing work, including the development of seismic design guidelines for selected port components, is being carried out to increase knowledge on how to mitigate damage to ports and harbors from earthquakes. Organizations in both the United States and abroad have written design guidelines and design manuals, developed analytical tools, worked on soil improvement techniques, and created finite element methods, all to improve designs that meet crucial performance objectives. Although there are several differing performance objectives, most of them suggest a three- or four-tier approach to design, based primarily on the severity of the event as defined by mean recurrence intervals. None of the current methods, however, focus on the issues that this report addresses: the design guidelines required to improve earthquake performance of container handling facilities, specifically wharves, cranes, and container yards.

Based on experiences from past earthquakes that have affected ports, two illustrative case studies (Chapter 2) present example details about the impacts of those events on container handling facilities.

A state-of-practice review (Chapter 3) includes summary descriptions of current seismic design requirements for port facilities around the nation, in-depth reviews of existing seismic design guidelines for port facilities, and a literature review of seismic design issues and approaches that identifies earthquake-related shortcomings for (1) container wharves and embankment stability, (2) bulkheads, (3) structural design, (4) container storage yards, (5) cranes, and (6) tsunamis and sea level rise impacts. Also included are discussions about ground improvement methods important to both new and retrofit design scenarios and kinematic loading on piles and wharf structures. The chapter concludes with a summary of gaps and research needs that were identified during the state-of-practice review.

Chapter 4 provides an overview of the National Science Foundation-funded Network for Earthquake Engineering Simulation (NEES) Grand Challenge project awarded several years ago to a consortium led by the Georgia Institute of Technology on "Seismic Risk Mitigation for Port Systems." The project was motivated by the observation that many large U.S. container ports are located in areas of significant seismic hazard and that seismic risk management practices have not kept pace with the growing importance of container ports to the nation's economy. The project overview provides a focus on the issue of business interruption caused by seismic events affecting port facilities, and describes key project results, including findings pertaining to the seismic response and fragility of wharves, the seismic response and fragility of container cranes, and the development of potential earthquake repair models.

To address the needs identified in both the gap analysis and those articulated in the case studies, new guidance documents are recommended that deal with three important, and up to now, inadequately addressed issues. The guidance documents being recommended (Chapter 5) are as follows:

1. *Guidelines for Developing Seismic Performance Criteria for Container Cargo Systems*
2. *Guidelines for Seismic Retrofit Design for Container Wharves, Including Ground Improvement*
3. *Guidelines for Numerical Modeling of Container Wharves under Kinematic Embankment Loading*

Details for pursuing and completing each of these documents has been proposed by breaking the developmental work into finite tasks (Chapter 6), including the specialty skills and experience needed to complete these documents. Levels of effort of the various skill levels have been estimated and converted into a proposed schedule where total elapsed time, if the documents are not pursued simultaneously, is 7 years and 8 months. *Guidelines for Numerical Modeling* could be developed at the same time as *Guidelines for Seismic Performance Criteria*, which would shorten the total time to 5 years and 7 months. The preferred scheduling approach is to substantially complete *Guidelines for Seismic Performance Criteria* and *Guidelines for Numerical Modeling* before beginning *Guidelines for Seismic Retrofit Design* so the work on developing performance criteria and kinematic loading could be used in the retrofit design guidelines document.

The estimated total cost to develop these documents is $7.18 million. The cost for *Guidelines for Seismic Performance Criteria* is $1.62 million; for *Guidelines for Seismic Retrofit Design*, $4.44 million; and for *Guidelines for Numerical Modeling*, $1.12 million. There could be some economies of scale if the three-document effort was managed by one person continuously and some of the subject matter experts could serve on more than one document project. Some of the peer review panels might be able to serve on more than one document and help reduce costs. However, each document has been considered to be a standalone project funded without consideration for the other documents.

The vulnerability of damage to port and harbor facilities, especially those that are involved in cargo handling, is becoming increasingly evident. The advent of more containerized cargo handling systems becoming more automated with robotics and electronics makes finding solutions for reducing damage to cargo handling, cranes, piles and wharves, and storage yards a critical need and an important business interruption and loss prevention objective.

Chapter 1

Introduction

Significant damage to port and harbor facilities resulting from recent earthquakes in Japan, Chile, and Haiti have dramatically illustrated that strong earthquakes can severely affect port facilities, with impacts ranging from minor damage to complete crippling of ports and harbors, rendering them unable to handle any cargo. The destruction of such facilities can have immense impacts on the economic activity for the affected regions and countries.

Because of the growing recognition that port facilities can behave poorly in major earthquakes, there is an emerging critical need to further develop procedures for assessing the seismic performance of port structures and developing mitigation strategies. Without proactive steps to understand and address the performance of port and harbor structures during earthquakes, the risks they pose in these events will persist.

Ports and harbors consist of many components, each of which is vulnerable to earthquakes. These components include wharves, piers, navigation channels, cranes and other cargo handling equipment, container storage yards, buildings, utilities, electrical facilities, bridges, rail facilities, and roads. Meaningful reduction in the earthquake risks posed by these facilities is beset by many challenges, including the lack of available technical guidelines for seismic design and retrofit of port facilities and the need for seismic performance criteria specifically applicable to port facilities.

Recent developments in earthquake engineering have begun to produce needed new knowledge that will facilitate the development of needed seismic design guidelines. Several years ago the National Science Foundation (NSF) awarded a George E. Brown, Jr. Network for Earthquake Engineering Simulation (NEES) Grand Challenge project to a consortium led by the Georgia Institute of Technology to develop seismic remediation and retrofitting technologies to mitigate port and harbor facility vulnerabilities, focusing largely on containerized cargo (research results from this project are described in detail in Chapter 4). Additionally, the American Society of Civil Engineers (ASCE) Coasts, Oceans, Ports, and Rivers Institute formed a task force to develop a draft standard for seismic design of selected port components. Collectively, the decisions by NSF and ASCE to work in this area and the widely supported high-priority need for seismic design guidelines for port and harbor facilities have created an opportunity and impetus for planning and implementing a significant effort to develop seismic design guidelines for mitigating the potential effects of earthquakes on port and harbor facilities.

Recognizing this opportunity and need, the National Institute of Standards and Technology (NIST) initiated a program to develop nationally-accepted guidelines for the assessment and mitigation of seismic risk to port and harbor facilities. The preparation of this report, which describes a proposed Work Plan for development of seismic design guidelines for port container wharf and cargo systems, along with the basis for the recommended Work Plan, is the first step in NIST's overall program to develop needed guidelines.

Detailed information that influenced the formation of the Work Plan, and the Work Plan itself, are discussed in the following chapters:

- Chapter 2 presents case studies of the performance of selected port and harbor facilities in Oakland, California, during the 1989 Loma Prieta earthquake, and in Port-au-Prince during the 2010 Haiti earthquake.
- Chapter 3 summarizes findings from a state-of-practice review of available guidelines for port seismic design, available resources describing port seismic design issues and approaches, available information and perspectives obtained from engineers who work routinely for port authorities, and gaps and research needs uncovered during the state-of-practice review.
- Chapter 4 summarizes research conducted on the NEES Grand Challenge project along with key results, including findings pertaining to the seismic response and fragility of wharves, the seismic response and fragility of container cranes, and the development of potential earthquake repair models.
- Chapter 5 describes three needed guidance documents recommended for development that will close significant knowledge gaps: (1) guidance on developing seismic performance criteria for container storage systems, (2) guidelines for seismic retrofit design of container wharves, including ground improvement, and (3) guidelines for numerical modeling of container wharves under kinematic embankment loading.
- Chapter 6 summarizes recommended Work Plan tasks, schedule, and estimated costs for a multiyear program to develop the recommended guidance documents and lists key collaborators that should be involved in such a program.

The information and recommended Work Plan provided in this report all specifically pertain to container handling facilities, including wharves, cranes, and container yards. The seismic design of components not specifically unique to port and harbors, such as electrical systems, utilities, buildings, bridges and grade separations, rail facilities, and roadways, is the responsibility of other non-port entities and outside the scope of this document.

Chapter 2

Port Earthquake Performance Case Studies

This chapter presents two case studies describing the poor performance of port facilities in Oakland, California, and Port de Port-au-Prince, Haiti, resulting from earthquakes impacting these locations in 1989 and 2010, respectively. The case studies illustrate and underscore important seismic design issues and concerns that are addressed by the recommendations made in Chapter 5 of this report. The case studies are intended to be illustrative, and demonstrate impacts that have been observed elsewhere, including in Kobe, Japan, where severe ground shaking and widespread liquefaction caused by the destructive magnitude-6.8 earthquake in 1995 severely impacted port and harbor facilities, resulting in high direct and indirect economic losses.

2.1 Impacts of the 1989 Loma Prieta, California, Earthquake on the Port of Oakland

The 1989 magnitude-6.9 Loma Prieta, California, earthquake caused considerable damage to terminal facilities at the Port of Oakland (Werner, 1998; EERI, 1990), which was located approximately 90-km north of the earthquake epicenter. Ground acceleration levels at the Port of Oakland were in the 0.25 to 0.3 g range (EERI, 1990). Damage to the Port of Oakland, as compiled by Werner (1998), is summarized below.

The most severe damage to port facilities occurred at the 7th Street Terminal (shown schematically in Figure 2-1). Liquefaction of the hydraulic fill resulted in settlement, lateral spreading, and cracking of the pavement over large areas of the terminal. Maximum settlements of the paved container yards were about one foot. Several large cranes that operated along the edges of the fill, traversing laterally along the wharves on railroad tracks, were severely impacted. The outboard crane rail was pile supported on the concrete wharf and suffered no appreciable settlement. The inboard crane rail, however, which was supported on fill throughout much of the terminal, sustained damage as a result of differential settlements of the fill and pavement damage caused by general settlement and lateral spreading of the fill. Damage to the inboard rail rendered several of these major cranes immobile after the earthquake.

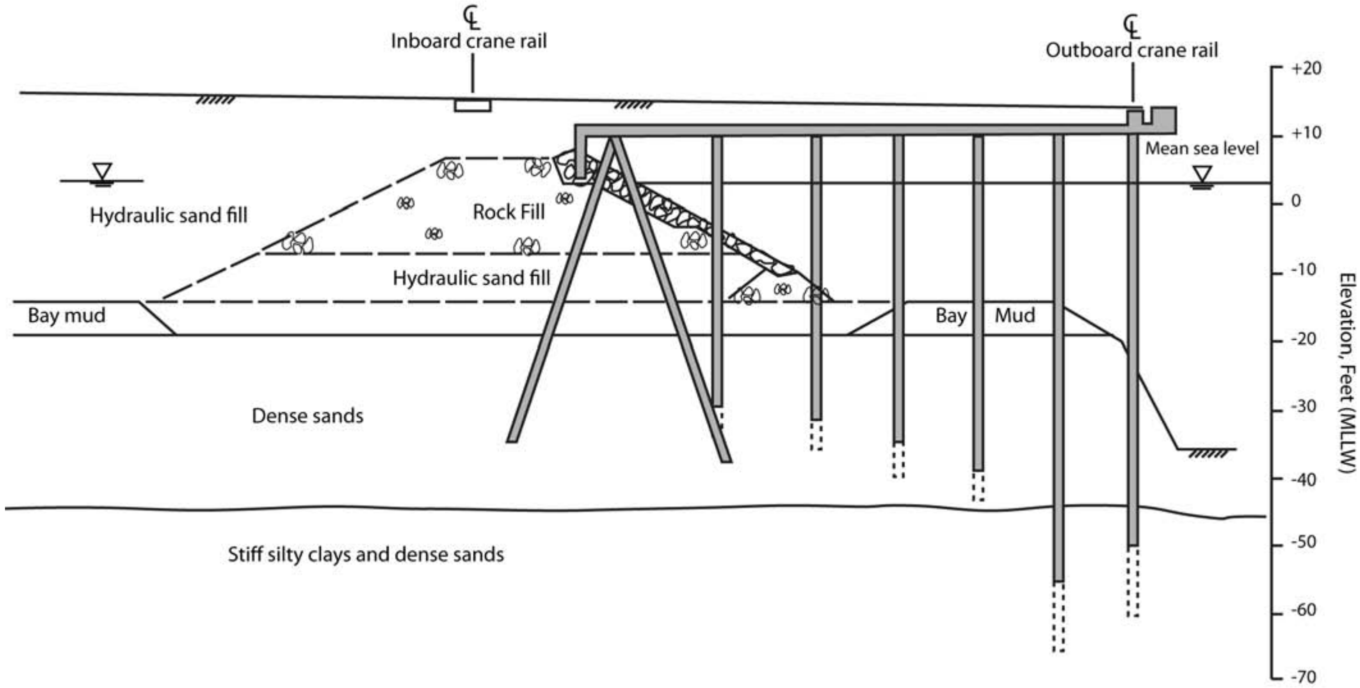


Figure 2-1 Cross-section schematic of 7th Street Terminal wharf, Port of Oakland, showing the foundation material and pile system (after Egan et al., 1992).

The lateral spread generated by the fill liquefaction and the soft clays beneath the dike caused damage at the tops of several piles supporting the wharves. Damage to piles occurred at the tops of the inboard battered piles and consisted primarily of tensile failures, though some piles also appeared to have been damaged in shear and compression. The vertical piles farther outboard were largely undamaged. The pile damage appears to represent an example of the damage that can occur as a result of the use of battered piles to resist lateral movements in relatively soft-soil foundations. These piles represent a "stiff" inclusion in the structural system, leading to large stress concentrations during earthquake shaking. The mode of failure observed, however, was predominantly tensile failure driven by outboard thrust of the fill, suggesting that liquefaction and associated lateral spreading were important factors.

In addition to the 7th Street Terminal, damage to other terminals was noted as follows:

- *Matson Terminal*: Piles and deck slabs separated and piles cracked. Container yard settlements were up to 2 feet deep. The terminal stayed in operation but with limited live loads during the crane operations.
- *Middle Harbor Terminal*: The yard areas had 0.5 to 2 feet of subsidence observed in the yard areas.
- *Howard Terminal*: The landslide crane rails had some lateral movement, although the crane remained operational. The vertical piles designed as ductile moment-frame systems performed well. The Marine Operations Building located

just behind the dike settled about 12 inches, but had no damage other than severed or displaced utility connections. This building was a small two-story building on a flat slab that floated on liquefied soils.

All terminals had pile-supported wharves at the edges of the terminal fills. In most areas, piles extend through perimeter sand and rock dikes, which serve as containment for the hydraulic fill forming terminal land inboard of the wharves. The hydraulic fill consists of dredged sands and silty sands.

2.2 Impacts of the 2010 Haiti Earthquake on Port de Port-au-Prince

The main port in Port-au-Prince suffered extensive damage during the 2010 magnitude-7.0 Haiti earthquake, inhibiting the delivery of relief supplies to affected areas of the city. At the time of the earthquake the port was operated by the Autorite Portuaire Nationale (APN) and consisted of two separate facilities designated as the North Wharf and South Pier. According to data provided by APN, the port handled 978,575 metric tons of cargo in 2005-2006 from 490 ship calls. An annotated aerial image of the port showing the earthquake impacts is shown in Figure 2-2.

Figure 2-2 Annotated aerial Google Earth image of Port de Port-au-Prince showing the impacts of the 2010 Haiti earthquake. Circles with embedded "F" denote locations of ground fissures. Light-colored areas depict locations where sand boils occurred.

North Wharf. The North Wharf (see Figure 2-2) consisted of a pile-supported marginal wharf[1] that was approximately 450 meters in length and 20 meters in width and was likely constructed on non-engineered fill of unknown origin. The water depth was 8 to 10 meters. Other information about the construction of the wharf, such as when it was constructed and the number and size of piles, is unknown. Immediately adjacent to the wharf were two steel-frame warehouses, each approximately 150 meters by 40 meters. Behind the warehouses was a container storage yard with mostly empty containers that were stacked two to four high at the time of the earthquake. There were also three cranes at the North Wharf, including one 15-meter gauge A-frame container crane and two rubber-tired mobile cranes.

The North Wharf collapsed, most likely because of liquefaction-induced lateral spreading. Numerous surface manifestations of the liquefaction were present in the vicinity of the North Wharf, including sand boils and large lateral spreading fissures (see Figure 2-2). Figure 2-3 shows an example of these fissures at the eastern end of the wharf area.

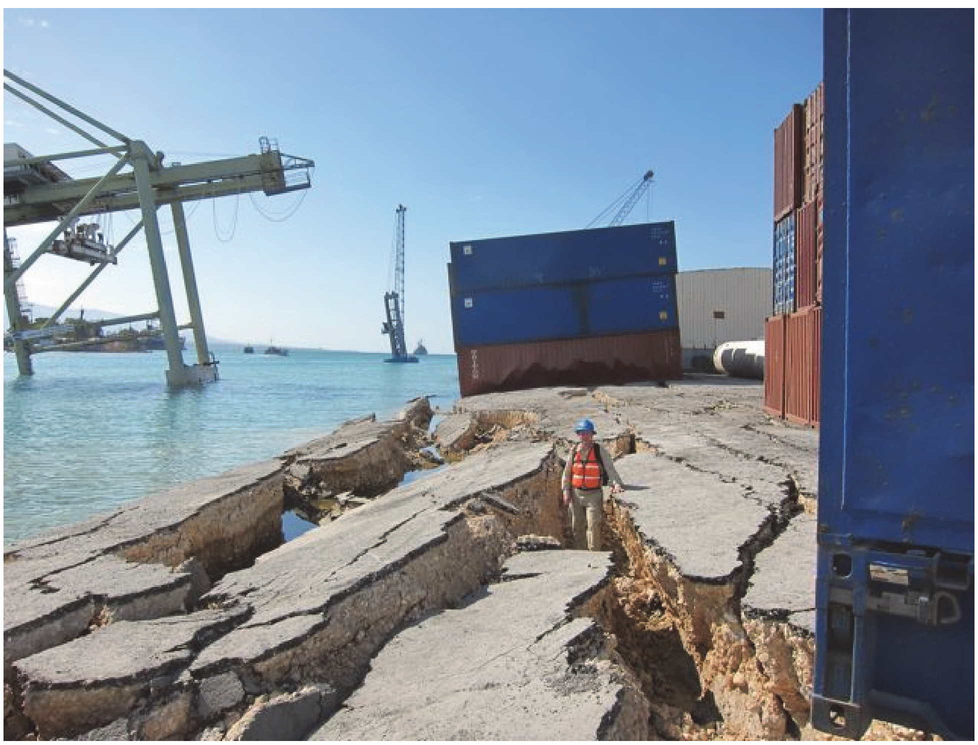


Figure 2-3 Fissures caused by lateral spreading, Port de Port-au-Prince, during the 2010 Haiti earthquake (S. Baldridge photograph).

Two of the three cranes on the North Wharf were partially submerged as a result of the earthquake and its aftershocks. Figure 2-4 shows the A-frame container crane in the foreground and a submerged mobile crane in the background. Though partially

[1] A marginal wharf is a wharf built as a continuation of the shoreline.

submerged, there was no obvious structural damage to the container crane. The second mobile crane, which was parked between the two warehouses, appeared to be undamaged (see Figure 2-4).

Interestingly, submergence of the cranes was apparently not caused by the main shock. Immediately after the earthquake the landside legs of the crane were observed to be above water. However, a U.S. Coast Guard photo taken during an overflight of the port at midday on January 13, 2010 clearly shows that the crane was in the same position as in Figure 2-4 (i.e., landside legs submerged). This raises the likelihood that a portion of the observed permanent displacements caused by liquefaction occurred as a result of aftershocks, which were numerous. (The National Earthquake Information Center reported no fewer than 45 aftershocks ranging from M_w 4.0 to 6.0 following the main shock until 1:54 p.m. EST on January 13, 2010.)

Figure 2-4 Submerged 15-meter gauge A-frame container crane (foreground) and mobile crane (left center, background), Port de Port-au-Prince, following the 2010 Haiti earthquake. A second, apparently undamaged, mobile crane is shown upright (right center, background).

South Pier. The South Pier was a pile-supported structure that was originally 380 meters long and 18 meters wide. A large bridge and a small pedestrian bridge, which were approximately perpendicular to the longitudinal axis of the pier, connected the pier to an island where the port security office was located. The western end of the pier was also connected to three dolphins by small pedestrian bridges. All of the bridges were pile-supported structures. An American or British contractor probably constructed the pier in about 1975. The vertical and battered concrete piles supporting the pier were approximately 51-centimeters square on 4.3 to 4.9-meter centers. The pile bents were 1.5 meters deep and 0.9 meter wide, and the deck was 45 centimeters thick (Brian Crowder, Naval Facilities Engineering Command, personal communication).

During the earthquake, the western-most 120 meters of the South Pier and portions of the pedestrian bridges linking the dolphins collapsed and subsequently submerged. One hypothesis is that the large bridge connecting the pier to the island and the abutment of the pier provided sufficient lateral restraint to prevent this portion of the pier from also collapsing. Nonetheless, the portion of the pier that was still standing after the earthquake was heavily damaged. U.S. Army divers who inspected the piles following the earthquake to determine whether the pier could support loads imposed by trucks carrying relief supplies found that approximately 40 percent of the piles were broken, 45 percent were moderately damaged, and 15 percent were slightly damaged (Brian Crowder, Naval Facilities Engineering Command, personal communication). The batter piles were, in general, more heavily damaged than the vertical piles. An aftershock on January 26, 2010, may have caused more damage, and the pier remained closed to traffic as of February 1, 2010.

In addition to the damage to the piles supporting the South Pier, the abutment also sustained liquefaction-induced lateral and vertical displacements. Approximately 1 meter of fill was required to re-level the approach to the pier. The piles supporting the small pedestrian bridge connecting the South Pier to the island were severely impacted (Figure 2-5), with extensive damage to the landward row of piles. In addition, one of the main entrance roads to the port was heavily damaged by liquefaction-induced lateral spreading.

Thor Marine Oil Terminal. At the Thor Marine Oil terminal, the approximately 230 meter-long embankment-supported portion of the pipelines used to offload petroleum products, which extended out into the water, sustained some damage because of vertical displacement of the embankment, as shown in Figure 2-6 (Scott Chodkiewicz, personal communication). The damage to the pipelines on the embankment was not serious enough to prevent usage of the pipelines following the earthquake. A second portion (approximately 120 meters in length) supported on piles was undamaged.

Varreux Terminal. At the Varreux terminal, a pile-supported pier collapsed during the earthquake killing approximately 30 employees who were working on the pier (Scott Chodkiewicz, personal communication). A photo of the remaining portion of the pier is shown in Figure 2-7.

Figure 2-5 Extensive damage to the landward row of piles, Port de Port-au-Prince, caused by the 2010 Haiti earthquake.

Figure 2-6 Embankment-supported petroleum pipelines, Port de Port-au-Prince, impacted by the 2010 Haiti earthquake (Source: Scott Chodkiewicz, U.S. Army Corps of Engineers).

Figure 2-7 Remaining portion of pile-supported pier at the Varreux terminal, Port de Port-au-Prince, following the 2010 Haiti earthquake (Source: Scott Chodkiewicz, U.S. Army Corps of Engineers).

Chapter 3

Summary of State of Practice

The state-of-practice review included (1) a review of regulatory requirements based on information obtained from port engineers and engineers who work routinely for port authorities at various locations throughout the nation, (2) a review of existing guidelines for port design, (3) a literature review to identify seismic design issues and approaches, and (4) the identification of technical gaps and research needs.

3.1 Regulatory Requirements

The intent of the review of regulatory requirements was to identify applicable codes and the structure of the building permit and regulatory authority at various locations throughout the United States. Input obtained from port authority representatives from across the nation are summarized in Table 3-1. For confidentiality reasons, only the regions in which each port authority (who provided input) is located are identified (i.e., the names of the information providers are not revealed). As indicated in Table 3-1, requirements and the regulatory authority structure and jurisdiction vary by location. For some locations/regions, there are no seismic design requirements for port facilities. For other locations, such as California and the Pacific Northwest, jurisdictions often depend on building code seismic design requirements, or special review procedures. In general, the review of existing regulatory requirements indicated there is a great deal of inconsistency in the structural design requirements and regulatory practices for marine structures in ports within the United States.

3.2 Review of Existing Port Design Guidelines

The review of existing port seismic design guidelines focused on the following documents:

- ASCE, 2012, *Seismic Design of Pile-Supported Piers and Wharves* (Draft), Standards Committee on Seismic Design of Piers and Wharves, American Society of Civil Engineers, Reston, Virginia.
- California State Lands Commission, 2010, *Title 24, California Code of Regulations*, Part 2, Chapter 31F, otherwise known as the Marine Oil Terminal Engineering and Maintenance Standards (MOTEMS).
- DOD, 2005, *United Facilities Criteria, Design: Piers and Wharves*, Report UFC 4-152-01, U.S. Army Corps of Engineers, Naval Facilities Engineering

Table 3-1 Summary of Input Obtained from Port Authorities Nationwide

U. S. Port Location	Regulator Authority	Comments/Input Obtained
Northeast	Port Authority	The port is effectively the building department for many, if not the majority, of major waterfront structures within the port. The port designs and builds many of their own facilities but also performs the reviews of waterfront structures built by their many tenants. These reviews are done by port design structural staff, who are familiar with marine detailing and issues. Waterfront facilities owned by the private sector and other government agencies in the port are reviewed by the corresponding building departments. The building department uses building code standards or American Association of State Highway Transportation Officials (AASHTO) standards for their review, if at all. (Note: it appears that many of these departments may defer to the consulting engineer on specific waterfront design issues.)
East	Chief Port Engineer	The State Engineer's office and local building officials have no jurisdiction over any of the port's properties, and the port can choose to follow any code or specification it deems appropriate.
East	State Construction Office	There are no seismic design requirements.
Southeast	Port Authority	There are no seismic design requirements.
Pacific Northwest	Port Authority	The municipality that owns the port has a building section that issues permits and provides inspections and code enforcement like most other U.S. cities. The municipality does not have anything to do with the port because it is not a building. The municipality brings in outside parties (e.g., Army Corps of Engineers) to do third-party reviews on a project-by-project basis.
Pacific Northwest	City Building Department	The City Building Department has recently created a Geotechnical Department to review geotechnical-related issues. The head of that department is quite knowledgeable about seismic and liquefaction issues but does not have a marine structures background. The city usually applies *International Building Code* (IBC) criteria for seismic load criteria, but for analysis and design, the city will accept any other recognized codes. The City Building Department reviews plans for marine structures because they are within the city limit.
California	Port: for piers and wharves without occupancy requirements per Building Code.	The city requires third-party review to approve "unconventional" designs that do not conform to conventional building codes.
	City Building Department: for all structures with occupancy requirements per Building Code	Structures involving any kind of occupancy require building code checks and building permits to be obtained from the city. No such code check is required for other structures (piers, wharves). Port staff "review" designs by tenants.
California	City Building Department	Designs are based on port-adopted standards, and the city generally accepts the port's adopted standards.
California	Port Authority	Port's criteria have been adopted into the city municipal code

Command, and Air Force Civil Engineer Support Agency, U. S. Department of Defense.

- PIANC, 2002, *Seismic Design Guidelines for Port Structures*, World Association for Waterborne Transport Infrastructure, Brussels, Belgium.
- Port of Long Beach, 2009, *Wharf Design Criteria*, Version 2.0, Long Beach, California.
- Port of Los Angeles, 2004, *Code for Seismic Design, Upgrade and Repair of Container Wharves,* San Pedro, California.

Each review contains a summary of the document. In many instances, the review also identifies gaps requiring future development.

3.2.1 Review of ASCE, 2012, Seismic Design of Pile-Supported Piers and Wharves (Draft Standard)

Document Summary

This ASCE draft (2012) standard for the seismic design of new pile-supported piers and wharves specifically excludes those structures with public occupancy that would be governed by ASCE/SEI 7, *Minimum Design Loads for Buildings and Other Structures*. Because of its intent for industrial applications, the document explicitly provides provisions for concrete and steel structures, but does not address timber and other materials. The document also does not cover all aspects of sheet pile walls, cellular walls, or similar structures that might be a component of a pile-supported pier or wharf.

This standard has been developed by a group of practicing structural engineers, geotechnical engineers, owners, and academics with specific expertise and experience in the marine and waterfront industries. The standard indicates the current practice of seismic design of piers and wharves differs considerably from the design of building structures.

The standards document provides for a one-to-three-level seismic design, depending on the importance of the structure, as determined by the “Authority Having Jurisdiction”. The provisions address force-based design and define a limited set of conditions where force-based design can be used. Displacement-based design is permitted for all structures, and the design approach is intended to encourage its use.

The approach emphasizes the importance of geotechnical engineering being integral with the structural evaluation and design. The provisions recognize that design of marine structures may require accounting for the occurrence of liquefaction and slope instabilities, and the response of structures to kinematic loading of soil moving against piles.

Structural criteria are provided for the design of piles, connections, and decks. Strain limits are provided for concrete and steel piles and various types of connections. Other requirements for detailing of various connection types are provided. Requirements are included for the capacity protection of decks.

General modeling and analysis criteria are provided, as well as detailed analysis methods. Provisions are included for the design of ancillary equipment on the deck, such as piping and equipment, cranes, and marine loading arms.

Gaps Requiring Future Development

- Timber structures are not included.
- Evaluation or rehabilitation of existing structures or the addition of new structures or portions of a structure to an existing structure are not covered.
- Sheet pile walls, cell walls, or other types of structures that might be part of a pile-supported pier or wharf are not addressed.
- Kinematic loads (from soil slope movement that loads piles) combined with inertial loads (from ground shaking accelerations) are not included.
- A method to determine the importance and criticality of an individual structure is not provided to the Authority Having Jurisdiction.

3.2.2 Review of DOD, 2005, United Facilities Criteria, Design: Piers and Wharves, Report UFC 4-152-01

Document Summary

The Department of Defense (DOD, 2005) Report UFC 4-152-01, *United Facilities Criteria, Design: Piers and Wharves*, provides design criteria for pier and wharf operations for U.S. Navy vessels, and offers guidance for overall facility design, including facility planning, loads, structural design, fender system design, camels and separators, and access.

The section of the document that addresses selection of the structure type recommends using sheet pile bulkheads or walls because of the high lateral earth pressures that can develop on the sheet piling. In addition, when a pile-supported platform is used for a wharf structure with liquefaction-susceptible hydraulic fill, a rock dike is recommended with an engineered soil filter fabric.

The approach in this document requires determination of a Seismic Use Group (I, II, or III) and Seismic Design Category (A to F) from the 2003 *International Building Code* (IBC). Depending on the combination of Seismic Use Group and Seismic Design Category, the approach requires either the use of an equivalent lateral force design per ASCE/SEI 7-02 or performance based design per the Marine Oil Terminal

Engineering and Maintenance Standards (MOTEMS). For either case, the design ground motions are for the MOTEMS Level 2 event.

The section on dynamic fill loads suggests that if the soil surrounding the piles is susceptible to liquefaction or if slope failure occurs, the piles will move excessively, resulting in serious damage to the piles and structure. The document recommends removal and replacement of unstable materials.

For determination of stability of embankment and fills at solid wharves subject to earthquake forces, the document refers to another DOD document in the United Facilities Criteria (UFC) series, UFC (3-220-01N).

Tables are provided with load combinations for Allowable Stress Design (ASD) and Load Resistance Factor Design (LRFD). Earthquake loads are combined with dead load, live load, buoyancy load, current load, earth pressure load, berthing load, and current load on a ship. Earthquake loads are not combined with wind loads on a structure or ship, temperature-induced loads, or ice loads.

Advice on general framing including optimal design for seismic loads is provided. A plumb pile system is considered to be restricted to shallow waters and light lateral loads. A combined plumb and batter pile system is considered to be more cost-effective, although the pile caps are to be designed and detailed for the high axial forces transmitted from the batter piles. An all-batter pile system is considered to be both a compromise between the other two and a cost-effective method in some circumstances. Finally, a batter pile system with seismic isolation in the form of calibrated isolators or seismic fuses between the wharf deck and batter piles is identified as a possible configuration. In this case, a warning is given to consider the magnitude of lateral mooring and berthing forces so that they do not exceed the threshold lateral force of the isolator.

Gaps Requiring Future Development

- Container cranes are not addressed.
- Sheet pile walls, cell walls, and other types of structures that might be part of a pile-supported pier or wharf other than by reference to another DOD United Facilities Criteria document are not addressed.
- Kinematic loads (from soil slope movement that loads piles) combined with inertial loads (from ground shaking accelerations) is not covered.

3.2.3 Review of California State Lands Commission, 2010, Title 24, California Code of Regulations, Part 2, Chapter 31F (Marine Oil Terminal Engineering and Maintenance Standards)

Document Summary

The California State Lands Commission, 2010, *Title 24*, *California Code of Regulations*, Part 2, Chapter 31F, otherwise known as the Marine Oil Terminal Engineering and Maintenance Standards document (MOTEMS), provides comprehensive criteria for evaluating existing marine oil terminals and designing new marine oil terminals in California. Main topics include regulatory compliance; audit and inspection; structural loading; seismic analysis and structural performance; mooring and berthing analysis and design; geotechnical hazards and foundations; structural analysis and design of components; fire prevention, detection, and suppression; piping and pipelines; mechanical and electrical equipment; and electrical systems.

The document specifies a two-level earthquake design. The structure is expected to survive the lower-level earthquake with minor or no structural damage and temporary or no interruption in operations. At the higher-level earthquake, the structure can experience controlled inelastic structural behavior with repairable damage but should not collapse, and there should be no major oil spills.

Return periods associated with the two earthquake levels vary depending on the designated risk level. Design spectra are specified for certain port locations in California. Specific procedures are given for site-specific Probabilistic Seismic Hazard Assessments. No scaling procedures are provided to match generic code shapes.

All seismic criteria are based on displacement-based design. The required analytical procedures depend on materials, risk classification, and structural configuration. Detailed criteria are given for pushover analyses. The criteria cover steel, concrete and timber design.

Special criteria are provided for batter pile design. Tables are provided with load combinations for ASD and LRFD. Earthquake loads are combined with dead load and earth pressure.

Foundation and geotechnical criteria address site characterization, liquefaction assessment, stability of earth structures, and simplified and detailed ground movement analysis.

Gaps Requiring Future Development

- Container cranes are not addressed.

- Sheet pile walls, cell walls, and other types of structures that might be part of a pile-supported pier or wharf are not covered.
- Kinematic loads (from soil slope movement that loads piles) combined with inertial loads (from ground shaking accelerations) are not included.

This document was developed for California and does not necessarily address requirements or typical construction for other parts of the country.

3.2.4 Review of Port of Long Beach, 2009, Wharf Design Criteria, Version 2.0

Document Summary

The Port of Long Beach, 2009, *Wharf Design Criteria*, Version 2.0 document includes structural and geotechnical seismic design criteria for the design of new wharves. These guidelines are specific to pile-supported marginal wharves with engineered sloping ground conditions located under the wharf structure comprising dredged soils or cut slopes protected or stabilized by quarry run rock material.

Three earthquake levels are provided for the analysis and design of wharf structures: the Operational Level Earthquake (OLE), the Contingency Level Earthquake (CLE), and the Code-Level Design Earthquake (DE). The Operational Level Earthquake and the Contingency Level Earthquake correspond to different probabilities of occurrence (different average return periods). The Code-Level Design Earthquake corresponds to a larger and more rare earthquake than the Operational Level Earthquake and the Contingency Level Earthquake. The three levels of ground motions are defined below:

- The Operational Level Earthquake is defined as the seismic event that produces ground motions associated with a 72-year return period. The 72-year return period ground motions have a 50 percent probability of being exceeded in 50 years. The Operational Level Earthquake event occurs more frequently than the Contingency Level Earthquake and the Code-Level Design Earthquake events and has a lower intensity.
- The Contingency Level Earthquake is defined as the seismic event that produces ground motions associated with a 475-year return period. The 475-year return period ground motions have a 10 percent probability of being exceeded during 50 years. The Contingency Level Earthquake event occurs less frequently than the Operational Level Earthquake event but more frequently than the Code-Level Design Earthquake event. The Contingency Level Earthquake has a higher intensity than the Operational Level Earthquake but lower intensity than the Code-Level Design Earthquake.
- The Code-Level Design Earthquake shall comply with the design earthquake requirements of the 2010 *California Building Code* (California Building

Standards Commission, 2010) and ASCE/SEI 7-05 (ASCE, 2006). The Code-Level Design Earthquake event occurs less frequently than the Operational Level Earthquake and the Contingency Level Earthquake events and has a higher intensity than the other two events.

These criteria use a performance-based design approach for the design of new wharves. This approach is based on strain-limit criteria and performance objectives associated with three levels of structural damage at the Operational Level Earthquake, the Contingency Level Earthquake, and the Code-Level Design Earthquake. The permitted level of structural damage for each ground motion is controlled by concrete and steel strain limits in the piles. The performance criteria of the three-level ground motions are defined below:

- For an Operational Level Earthquake event, the wharf should have no interruption in operations. Operational Level Earthquake forces and deformations, including permanent embankment deformations, shall not result in significant structural damage. All damage, if any, shall be cosmetic and located where visually observable and accessible. Repairs shall not interrupt wharf operations.
- For a Contingency Level Earthquake event, there may be a temporary loss of operations that should be restorable within a few months. Contingency Level Earthquake forces and deformations, including permanent embankment deformations, may result in controlled inelastic structural behavior and limited permanent deformations. All damage shall be repairable and shall be located where visually observable and accessible for repairs.
- For a Code-Level Design Earthquake event, forces and deformations, including permanent embankment deformations, shall not result in the collapse of the wharf and the wharf shall be able to support the dead load including the cranes. Life safety shall be maintained.

The performance requirements for the seismic design of container wharf structures is satisfied if the specified strain limits are met in the pile design at three levels of ground motions. The strain limits were developed at specific damage states such as minimal damage at the Operational Level Earthquake, repairable damage at the Contingency Level Earthquake, and no major structural failure at the Code-Level Design Earthquake.

General seismic design criteria, load combinations, analytical and design requirements, detailing requirements, geotechnical requirements, and soil-structure requirements are provided.

3.2.5 Review of PIANC, 2002, Seismic Design Guidelines for Port Structures

Document Summary

The World Association for Waterborne Transport Infrastructure *Seismic Design Guidelines for Port Structures* (PIANC, 2002) document presents performance-based design guidelines that describe the expectation of some degree of damage during an earthquake event, depending on specific functions and response characteristics of a port structure and the probability of an earthquake occurrence in a particular region. The guidelines are intended to be user-friendly and offer designers a choice of analysis methods. These guidelines are also intended to be general enough to be useful around the world.

The document discusses earthquakes, including coverage of both liquefaction and tsunamis, performance-based methods, and ways to evaluate such performance. The primary text has information on damage criteria for sheet pile walls, pile-supported wharves, cranes, and breakwaters.

The document commentary extensively covers many subjects, including the following:

- Design codes and standards
- Case histories
- Earthquake motion, including size of earthquakes
- Geotechnical parameters
- Structural design of pile and deck systems
- Remediation of liquefiable soils
- Coverage of several different analysis methods
- Several examples of seismic performance evaluations

Gaps Requiring Future Development

- The document lacks focus, case histories, and supporting documentation on the seismic effects on ports and harbors in the United States.
- Liquefaction remediation is covered but not the specifics of this issue as it relates to storage yards or backland areas.
- Coverage of the seismic effect on cranes is limited.

3.2.6 Review of the Port of Los Angeles, 2004, Code for Seismic Design, Upgrade and Repair of Container Wharves

Document Summary

The Port of Los Angeles report, *Code for Seismic Design, Upgrade and Repair of Container Wharves* (2004), has provisions to safeguard life and protect against major structural failures. The code also provides seismic design provisions to limit damage and minimize economic losses because of earthquakes. The continuity of being able to use the container wharves after an earthquake event is vital to the economy at the local and national levels.

This code establishes minimum wharf design criteria to protect public safety by minimizing the earthquake-related risk to life in the event of rare, intense earthquake ground motions, defined as Design Level Earthquake ground motions. To reduce the risk of economic losses to the Port of Los Angeles, the code specifies performance criteria for wharf structures defining acceptable levels of damage in the event of moderate and large ground motions, called the Operational Level Earthquake and the Contingency Level Earthquake ground motions, respectively.

This code uses a performance-based design approach for the design of new wharves. This approach is based on strain-limit criteria and performance objectives associated with three levels of structural damage at the Operational Level Earthquake, the Contingency Level Earthquake, and the Design Level Earthquake:

- *Operational Level Earthquake*: There is to be no significant structural damage. The damage locations are to be visually observable and accessible for repairs. There is to be minimal or no interruption to wharf operations during repairs. After an Operational Level Earthquake event, the intent of the code is to limit repairable damages at pile top hinges. Minor plastic hinging in the pile top with occasional light spalling of the cover concrete is expected only directly below the deck, where the damage can be repaired, minimizing impact on operations. Although a limited amount of ductile behavior is expected for piles in the ground, at this location no spalling of concrete cover is expected.
- *Contingency Level Earthquake*: There is controlled inelastic structural behavior and limited permanent deformations. The damage locations are to be visually observable and accessible for repairs. There is to be only temporary or short-term loss of operations. After a Contingency Level Earthquake event, the intent of the code is to have controlled repairable damage at pile top hinges. Plastic hinges in piles with spalling of the cover concrete is expected below the deck and the damage needs to be repaired. Piles designed to the Contingency Level Earthquake strain limits have approximately 50 percent more reserved displacement capacity at the pile top hinge before the moment capacity of the pile deteriorates significantly.

The in-ground pile hinge has more stringent requirements than the pile top hinge. Although a limited amount of ductile behavior is expected for piles in the ground, at this section no spalling of concrete cover is expected. The temporary interruption of wharf operations may occur for a duration of time that depends on several factors, such as type of damage, damage locations, site access and availability of qualified repair contractors and engineers.

- *Design Level Earthquake*: This performance level safeguards life and guards against major structural failures. After the Design Level Earthquake, the intent of the code is to safeguard life and prevent major structure failures. It is expected that piles may be severely damaged, but this damage should not cause major structural failure to safeguard life. The loss of wharf operation is expected, and the duration of recovery and repair depends on several factors, such as type of damage, damage locations, site access, and availability of qualified repair contractors and engineers.

The scope of this code (Port of Los Angeles, 2004) is to provide performance-based provisions for the seismic design of new marginal container wharves at the specified earthquake. General seismic design criteria, load combinations, analytical and design requirements, detailing requirements, geotechnical requirements, and soil-structure requirements are provided.

There are several types of structures at the Port of Los Angeles including: container wharf structures, marine oil terminals structures, cruise terminals structures, buildings, bridges and grade separations, railroad bridges, cranes and other equipment. The scope of this code is for the seismic design of new marginal container wharves.

To achieve seismic performance goals, seismic performance criteria provided in terms of material strain limits for each earthquake level are specified. Complying with the specified strain limits will control and limit the damages correspondingly to meet the code purpose and intent. Because the code is subjective, it is not provided as a requirement.

The performance requirements for the seismic design of container wharf structures is satisfied if the specified strain limits are met in the pile design at three levels of ground motions. The strain limits were developed at specific damage states such as minimal damage at the Operational Level Earthquake, repairable damage at the Contingency Level Earthquake, and no major structural failure at the Design Level Earthquake.

This code (Port of Los Angeles, 2004) includes provisions for the upgrade and repair of existing container wharves damaged by seismic or other natural disasters or events.

3.3 Literature Review of Seismic Design Issues and Approaches

3.3.1 Container Wharves: Embankment Stability

The evaluation of the seismic stability and earthquake-induced deformations of embankments associated with pile container wharves is a critical component of container wharf design, as illustrated by the damages caused in past events and the currently available design standards summarized in Section 3.2. The state-of-practice literature review summarized in this section is divided into three subsections:

1. Embankment Deformation: Analysis Approaches

2. Pile Design: Soil-Pile Interaction

3. Deformation Mitigation: Ground Improvement

More detailed information about embankment deformation, pile design and deformation mitigation for container wharves and embankment stability is provided in Appendix A of this report.

Potential liquefaction of embankment foundation soils is highlighted in the discussion of the state of practice.

Embankment Deformation: Analysis Approaches

Analysis Methods. Current practice for the analysis of the performance of slopes and embankments during earthquake loading is to use one of two related methods:

1. A limit equilibrium method using a pseudo-static representation of the seismic forces, and

2. A displacement-based analysis method using either the Newmark sliding block concept shown schematically in Figure 3-1 or more rigorous numerical modeling methods.

The use of these methods for seismic slope design has been widely adopted both in the United States and in international practice, and has been adopted by the Port of Los Angeles and the Port of Long Beach. These methods have been described in detail in Transportation Research Board (TRB) National Cooperative Highway Research Program (NCHRP) Report 611 (Anderson et al., 2008) and in a Southern California Earthquake Center publication, *Recommended Procedures for Implementation of DMG Special Publication 117: Guidelines for Analyzing and Mitigating Landslide Hazards in California* (SCEC, 2002).

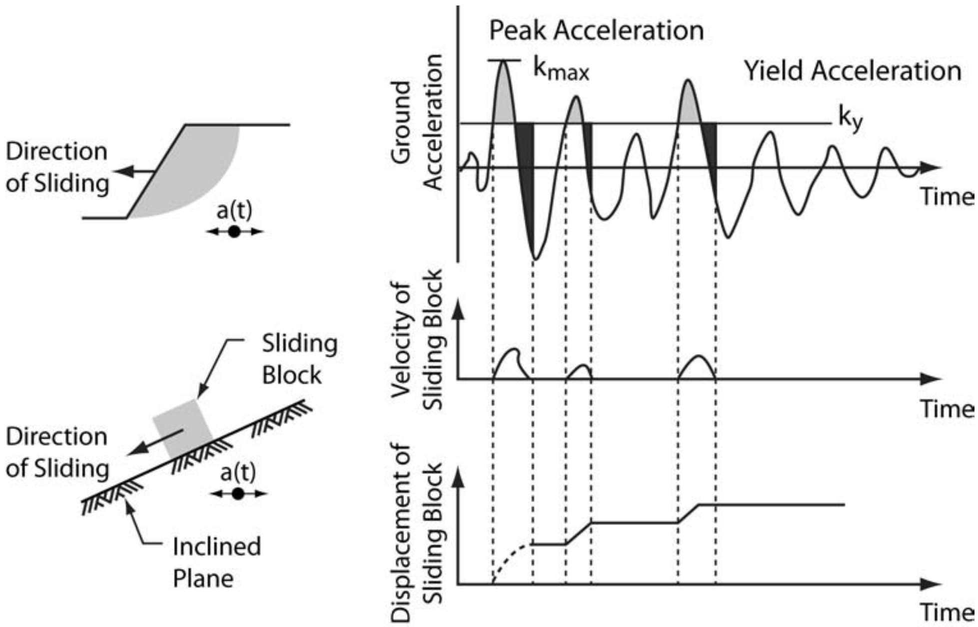


Figure 3-1 Schematic illustrating Newmark (1965) sliding block concept for slopes (from Anderson et al., 2008).

Most simplified methods of seismic analysis assume that the soil within the slope behaves as a rigid mass with all ground motions moving in phase. Although this assumption makes sense for small slope masses, it is not a likely case for higher slope masses where wave motion incoherence can reduce average lateral accelerations.

As part of the TRB NCHRP Report 611 ground-motion study (Anderson et al., 2008), updated methods for estimating permanent displacements of embankments were developed following the Newmark sliding block concept. The current AASHTO specification allows the peak seismic coefficient to be reduced by 50 percent if 10×A inches of permanent movement are permissible, where A is the peak ground acceleration (PGA) adjusted for site class. The work completed for the NCHRP 12-70 Project (Anderson et al., 2008) showed that the method in AASHTO for estimating permanent movement was conservative.

One of the conclusions from the NCHRP Report 611 (Anderson et al., 2008) analysis is that a common approach of using 50 percent of the PGA for pseudo-static design results in a permanent displacement estimate of less than 1 to 2 inches. The Port of Long Beach and the Port of Los Angeles adopt 33 percent of the PGA for pseudo-static design but use site-specific Newmark displacement curves (Newmark, 1965) established from design time-history data.

Liquefaction: The presence of potentially liquefiable sands are found at many port sites and have a significant influence in evaluation of potential earthquake-induced embankment deformations or lateral spreads. The consequences of liquefaction induced deformations have been documented and have a significant impact on wharf pile design.

Liquefaction potential of the soils in the immediate vicinity of the structure and associated embankment or rock dike, including soils in front of, beneath, and behind the pier or wharf, should be evaluated. As for the widely accepted state-of-practice, liquefaction potential evaluation follows the procedures outlined in "Liquefaction Resistance of Soils: Summary Report from the 1996 National Center for Earthquake Engineering Research (NCEER) and 1998 NCEER/NSF (National Science Foundation) Workshops on Evaluation of Liquefaction Resistance of Soils" (Youd et al., 2001), and *Recommended Procedures for Implementation of DMG Special Publication 117, Guidelines for Analyzing and Mitigating Liquefaction Hazards in California* (Martin, et al., 1999).

Slope Stability and Lateral Ground Deformation Analyses. In Port of Los Angeles and Port of Long Beach practice, the minimum static factor of safety of the slope or embankment for slope stability is 1.5. Pseudo-static slope stability analysis is performed using a horizontal seismic coefficient of 0.15 g or one-third of horizontal peak ground acceleration, whichever is greater. If the estimated factor of safety is greater than 1.1, then no further evaluation for deformations or kinematic analysis, as outlined below, is necessary. Under these conditions, some limited deformations are expected to occur; however, the seismic performance of the slope and structure under these deformations are considered satisfactory.

The static factor of safety immediately following a design earthquake is not less than 1.1. Shear strength values compatible with the appropriate residual strength of liquefied soils and sensitive clays are used in both pseudo-static and post-earthquake analyses.

Pile Design: Soil-Pile Interaction

Lateral soil-pile interaction under earthquake loading includes two loading conditions: (1) inertial loading associated with earthquake-induced lateral loading on the wharf structure and on piles due to the inertia of the structure and (2) kinematic loading on wharf piles from earthquake-induced lateral deformations of the slope/embankment/dike system.

For typical container wharves, the inertial loading condition tends to induce maximum moments and plastic hinges in the upper regions of the pile, whereas maximum moments and plastic hinges tend to be in the lower regions of the pile under kinematic loading conditions, as shown in Figure 3-2. However, this may not be the case for all wharves.

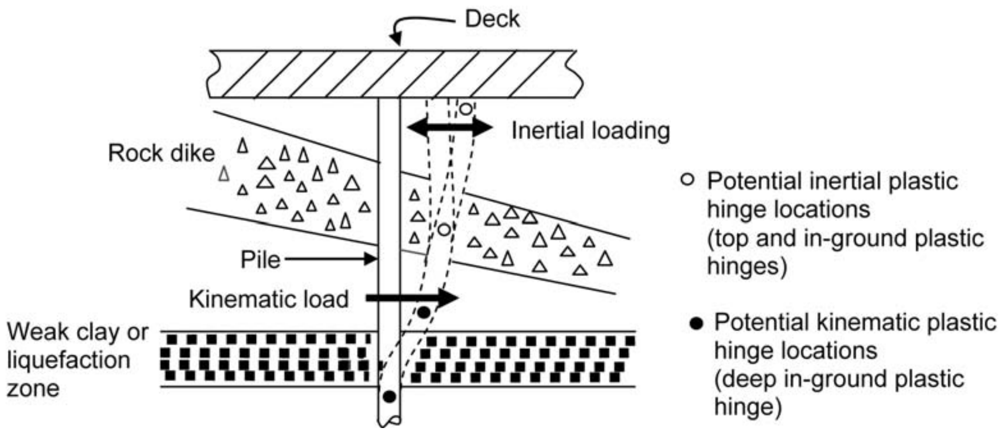


Figure 3-2 Schematic illustrating plastic hinge formation under inertial and kinematic loading conditions (Port of Long Beach, 2009).

Inertial Loading. In current practice adopted by the Port of Los Angeles and the Port of Long Beach, appropriate lateral soil springs (p-y curves) are used in the evaluation of the response of the soil-pile-structure under inertial loading of the structure. As discussed in various documents (Port of Los Angeles, 2004; Arulmoli et al., 2008; Port of Long Beach, 2009), the free-field deformation of the ground in the vicinity of the piles may be ignored in this analysis. For typical pile-supported piers and wharves, p-y curves at shallow depths (typically within 10 pile diameters from the ground surface) tend to dominate the inertial response. Best-estimate p-y curves are usually developed for baseline inertial loading evaluations. Because of uncertainties associated with the development of p-y curves, including sloping ground conditions, uncertainties in soil/rock parameters, pile group effects, and groundwater fluctuations, upper-bound and lower-bound p-y curves should be used to evaluate the inertial response of the structure.

Kinematic Loading. As noted, kinematic loading from permanent ground deformation in the deep seated levels of the slope/embankment/dike foundation soils should be evaluated to ensure that lateral deformations are kept within such amounts that the structural performance of wharf piles is not compromised, as defined by pile strain limits (Port of Los Angeles, 2004; Port of Long Beach, 2009).

Evaluation of kinematic pile loading can be performed using different methods ranging from simplified methods that use available charts to more complex one-dimensional site-response analyses and pile pushover analyses and finally to complex, two-dimensional dynamic soil-structure interaction analyses that use finite element or finite difference computer codes. The level of effort (hence the cost and time) increases as the analysis complexity increases. For the above reasons, a step-by-step evaluation process can be used for evaluation of kinematic loading. This step-by-step process has been adopted by the Port of Los Angeles and the Port of Long Beach (Port of Long Beach, 2009) as presented and described in Appendix A.

Deformation Mitigation: Ground Improvement

If structural or pile solutions to mitigate earthquake-induced embankment stability or slope deformations to acceptable levels are impractical or too expensive, geotechnical ground improvement methods may need to be evaluated. In most cases, liquefaction is the primary cause of embankment lateral spreads, albeit, in some cases, soft cohesive soils may be the source of large lateral deformations.

To prevent or reduce liquefaction induced embankment deformations to acceptable performance levels, site remediation using one or several ground improvement options may be deployed. Remediation methods commonly deployed in current practice include the following:

- in-situ densification of liquefiable soils in zones beneath embankments, underneath or adjacent to the toe of embankment fills or surrounding piers;
- deep soil mixing using cement, which creates stabilizing zones in the soil similar to those developed by ground densification techniques; and
- grouting techniques.

Overviews of the state-of-practice are documented in several publications (NCEL, 1990, ASCE, 1997; Andrus and Chung, 1995, PIANC, 2002). A comprehensive report on ground remediation measures for liquefaction at bridge sites (where embankment deformation concerns are similar to those at ports) was published by Cooke and Mitchell (1999). Brief summaries and examples of several remediation methods are provided in Appendix A.

3.3.2 Bulkheads

Bulkhead structures are rarely used for construction of new container wharf facilities in the United States, but are widely used in other countries, such as Japan. In the 1980s, the Port of Los Angeles considered the potential use of caisson structures for Pier 400, but concerns about seismic performance issues curtailed the study.

PIANC (2002) provides a good summary of the types of bulkhead structures, which fall into three main categories:

- gravity retaining walls such as caisson structures,
- sheet pile walls with tieback anchors, and
- cellular sheet pile closed or open structures.

Such structures are, in general, particularly vulnerable to liquefaction-induced failure or deformation-related damage. Although this report focuses on piled container wharf structures, literature review comments including a summary of the state-of-practice of seismic design for gravity caisson structures and sheet pile walls with tieback anchors, is given below. Additional information is provided in Appendix B.

Gravity Walls – Caisson Structures

The most difficult design issue for gravity walls, such as concrete caisson structures, which may be floated into position and founded on the seafloor, is the earthquake-induced displacement and tilt evaluation, particularly with liquefiable foundation or backfill soils.

Tilt performance criteria related to container crane operations make it difficult to use gravity caisson structures for container wharf operations subject to seismic loading. Conventional practice for evaluating seismic stability of gravity walls/caisson structures is based on pseudo-static approaches similar to those described for embankments

The stability of regular retaining walls is evaluated with respect to sliding, overturning, and bearing capacity. The state-of-practice assigns higher factors of safety for overturning and bearing capacity than for sliding, as these are more critical modes of failure. A key component in the above analyses is the evaluation of the seismic active earth pressure. Whereas the Mononabe Okabe equations are widely used in practice to predict active and passive earthquake pressures (Ebeling and Morrison, 1992; PIANC, 2002), the equations have limitations, and a generalized limit equilibrium approach has been recommended by the NCHRP 12-70 Project Report 611 (Anderson et al., 2008).

Also the onset of liquefaction and assignment of appropriate residual strength parameters for analysis leads to analysis difficulties.

Sheet Pile Walls with Tieback Anchors

Sheet pile walls with tieback anchors have been widely used for marginal wharf structures in the United States, but not for container wharf facilities. Because of liquefaction of backfill soils, past performance in earthquakes has been poor, as described in PIANC (2002).

Conventional practice for evaluating the seismic stability of tieback sheet pile walls is based on the use of pseudo-static approaches similar to those for caissons as previously described.

Whereas pseudo-static seismic design can address issues related to global stability and sheet pile and anchor seismic stress levels, they cannot be used to evaluate deformations related to performance requirements. As for the case of gravity caissons, more advanced numerical dynamic finite element or finite difference approaches have been used for this purpose.

3.3.3 Structural Design

The current seismic design practice for piers and wharves differs from that used for conventional buildings or building-like structures. For example, load combinations used in pier and wharf design must include berthing and mooring loads that may govern the lateral load design in low seismic regions. Because the configurations and construction procedures for pier and wharf structures are substantially different from those for buildings, seismic response characteristics will likewise differ. This underscores the need for the unique seismic design procedures for pier and wharf structures.

The performance goals of pile-deck structures, such as piers and wharves, are to primarily satisfy three performance levels for seismic design (see Section 3.2.4):

1. The Operational Level Earthquake, in which case the structure should be capable of withstanding moderate ground motions without requiring any immediate post-earthquake structural repair.

2. The Contingency Level Earthquake, in which case the pile-deck structures should be capable of withstanding a major earthquake, which has a recurrence interval of 475 years, and should result in controlled inelastic structural behavior that concludes in repairable damage. The pile-deck structure should be designed to resist a level two earthquake without collapse, allowing the structure to be repaired rather than replaced.

3. The Code-Level Design Earthquake (as established by the building code authority for the jurisdiction), which most structures typically are currently designed for but rarely experience. The structure should be able to maintain life safety and prevent major structural failures.

Additional information on issues important in structural design and those considered for both pile design and for pile-to-deck connections are addressed in Appendix C. No particular gaps in knowledge or research needs as they relate to structural design have been identified outside of those noted in Section 3.2.

3.3.4 Container Storage Yards

Container storage yards are behind and adjacent to terminal wharves and wharf cranes. These yards are critical to the movement of containers through a terminal by providing a buffer and storage area for containers waiting placement on ships for export cargo or awaiting pickup by trucks or placement on railcars for import cargo.

Conventional container yards either store containers on truck chassis or place the containers on the ground, stacking them up to six high. Stacked containers are placed and retrieved by large rubber-tired gantry cranes. Containers stored on chassis essentially use the yard as a parking lot similar to an automobile parking lot. For pickup of import cargo, a truck arrives, is directed to the parked location of the container to be picked up, attaches to the chassis, and leaves the terminal for delivery. For export cargo, the reverse occurs.

Because the high wheel loads of gantry cranes tend to rut and ravel asphaltic concrete pavements requiring frequent maintenance, concrete grade beams are the preferred option for gantry crane runways. Concrete is much more durable under the repetitive loads involved.

Damage to conventional container storage yards resulting from earthquakes is typically related to liquefaction. Pavements are damaged by settlement and frequently by large lateral deformations. Concrete grade beams used to support the rails for gantry cranes can settle and crack to the extent that the cranes can no longer operate. Grade beams can be removed or paved over providing a surface for gantry cranes to operate. Damage to the wharves and other improvements in storage yards has a much longer repair time than the pavement.

In the last several years, partially and fully automated container terminals have been constructed and some are under development. Automated terminal container storage areas differ from conventional ones in ways that significantly impact their performance during seismic events. The potential damages and downtime that would occur are much more significant than for conventional terminals.

For automated terminals, gantry cranes that stack and retrieve containers typically run on rails mounted on concrete beams. These automated cranes must operate within tight tolerances because the operation is automated and the equipment robotic. The longitudinal and traverse directions tolerance of the crane pickup points is 3 inches or less. In addition, the difference in rail elevations cannot vary by more than 3 inches.

The beams that support the rail-mounted gantry cranes can be pile supported or not. Some are supported by tie and ballast systems. Pile-supporting these beams greatly increases costs and, therefore, concrete grade beams or ties and ballast are more frequently used. Because each of these support systems has its advantages and disadvantages and a careful evaluation of what is best for each terminal must be made. Each can be extensively damaged by a seismic event that induces liquefaction in the underlying supporting soils.

Seismic design or performance guidelines have not been developed for cargo storage yards. Each port has determined the criteria to be used for their facilities. Typically, the soils underlying cargo storage areas are not improved with methods to mitigate liquefaction. The primary reason for this is cost. The cost of installing stone columns or other liquefaction prevention methods is very high when done for storage areas 300 to 400 acres in size. As mentioned before, the other reason liquefaction mitigation measures are not implemented is that repairs can be made relatively quickly in comparison to repairs to other port facilities, such as wharves.

However, for automated terminals the stacking cranes run on rail and beams, and the operational tolerances are stringent. Any liquefying soils that cause the operational tolerances to be exceeded, either the rail foundations or paved stacking areas settling differentially, would prevent the cranes from operating. The rails and beams may be damaged and need replacement, which will require significant downtime for repairs to be completed. The downtime to make these repairs has been estimated to be between 4 to 8 months, which can be an intolerable amount of time for some terminals at major ports.

Crane beams can also be placed on piles. These measures can be implemented to mitigate liquefaction induced damage. Because these cargo storage areas can be hundreds of acres in size and a typical automated terminal can have 10 or more miles of crane beam, these measures are all expensive. Without any guidance, decisions as to what measures to implement can be difficult to make. Seismic risk analysis methods similar to those described in Section 4 of this report could be developed for this purpose.

3.3.5 Cranes

This section summarizes crane design issues. Appendix D has a more complete description of crane design parameters and current work that has been done on understanding crane design and the crane-wharf interaction. Proposed seismic design criteria for container cranes have been developed by Soderberg and Jordan (2007) and Soderberg et al. (2009). The recommended criteria were motivated by the observation that the development of seismic performance criteria for container cranes has lagged the development of similar criteria for wharf structures. Smaller, older cranes are able to tip prior to the occurrence of significant damage. As a result, service loads for seismic design were limited to 0.2 g to ensure sufficient strength. Newer, larger cranes are more stable, which will inhibit tipping and result in larger lateral seismic loads. However, the strength of the portal frame has not been increased in accordance with this higher seismic demand. The most important response is in the trolley travel direction, that is, the portal frame, which has a natural period of about 1.5 seconds. The O-frame in the gantry travel direction has a natural period of approximately 3 seconds. In general, the above-cited seismic design criteria documents recommend a comprehensive evaluation by the port community of the seismic performance criteria for container cranes. The proposed seismic design criteria are as follows:

- Elastic response and easily repaired damage for the Operational Level Earthquake (72-year return period).

- Tipping occurs when stresses are less than 90 percent of yield, and the portal frame that provides stability should yield in a ductile manner during the Contingency Level Earthquake (475-year return period).

The latter criteria is intended to prevent collapse and assure life safety.

Jacobs et al. (2011) examined the seismic response of jumbo container cranes as part of the Network for Earthquake Engineering Simulation (NEES) Grand Challenge project described in Chapter 4. The authors cite evidence that uplift of the landside legs of modern (i.e., larger) cranes increases demand on the waterside legs rather than limit seismic forces as is commonly assumed. The study focuses on understanding the onset and effect of uplift on the elastic response of cranes as a first step towards improving the seismic performance and design criteria for modern jumbo cranes. Jacobs et al. (2011) note that the effect of vertical ground motions on the response of the crane requires further study.

3.3.6 Tsunami Hazards / Sea Level Rise

Tsunami waves have caused significant damage to ports and harbors throughout history. Literally meaning “harbor wave”, recent damaging tsunami events have affected ports and harbors around the Indian Ocean (December, 2004), Samoa (September, 2009), Chile (February, 2010), and the Tohoku coastline of Japan as well as more distant small vessel ports in Hawaii, California, and Oregon (March 2011). An event similar to the Tohoku Tsunami is possible along the northwest U.S. coastline due to the potential for large magnitude tsunamigenic earthquakes along the Cascadia Subduction zone. Other tsunamigenic earthquake and landslide sources threaten Alaska, Hawaii, the southern California coastline, as well as the eastern seaboard and Gulf coast. All coastlines of the United States and many of its protectorates are at risk of damaging tsunamis, although the probability of occurrence is higher for certain regions than others.

Because port and harbor facilities must, of necessity, be at or near sea level and open to the ocean, it is clear that tsunami inundation is a threat that must be considered during port design. The effects of tsunami inundation are exacerbated by future sea level rise and the coastal subsidence that often accompanies large subduction zone earthquakes. Both of these effects reduce the freeboard for port facilities.

No U.S. codes currently address the design of port or coastal facilities for tsunami loading. An effort is underway to develop design information on “Tsunami Loads and Effects (TLE)” for inclusion in the 2016 edition of the ASCE/SEI 7 standard, *Minimum Design Loads for Buildings and Other Structures*.

Japan has a long history of damaging tsunamis, both from near-source and far-source earthquakes. The “Technical Standards and Commentaries for Port and Harbour

Facilities in Japan” (MLIT, 2007) provides detailed information on design of port facilities for earthquake, wave loading, and tsunami effects.

Pile-supported piers and marginal wharves can experience extremely large uplift forces during tsunami inundation. Uplift pressures exceeding 12 kPa (250 psf) are estimated to have occurred below access panels that failed at numerous ports in Japan during the Tohoku Tsunami (ASCE, 2012). Failure of these panels is believed to have relieved some of the uplift pressure on the remainder of the structure. However, some of the access panels and all of the pile-supported structure must be designed to resist these uplift pressures so that they can be restored to operation after the tsunami.

Sheet pile and slurry wall retaining systems around ports are susceptible to enhanced scour and tsunami-induced liquefaction due to elevated pore pressures during inundation that may not have time to dissipate during rapid tsunami drawdown. The resulting loss of backfill behind retaining wall systems has been observed to result in failure of dead-man tie-back systems (ASCE, 2012). Similar tsunami-induced liquefaction can lead to extraction of sands from below slabs-on-grade, leading to widespread failure of storage yard pavements.

Tsunami inundation of shipping container storage yards will result in containers becoming floating debris, leading to potential debris strikes to critical facilities, such as fuel storage tanks, cranes, warehouses and other port structures. Because of weight limitations required for container handling equipment, all shipping containers are buoyant, even when fully loaded. Impact forces depend on the size, shape, orientation, and velocity of the debris at the instant of impact. Research is underway to investigate the impact forces resulting from container strikes, though most studies focus on the worst case of axial impact at one of the stiff corner elements of the container (Paczkowski et al., 2012). Further research is required to determine the probability distributions of container weight, velocity, orientation, and impact alignment so as to develop a realistic design procedure for shipping container strikes.

Fuel storage tanks are also susceptible to flotation during a tsunami. If the tsunami inundation is sufficient to overtop the retaining berm built around the storage tank yard, empty and partially full tanks will be subjected to large uplift forces. In past tsunami events, tanks have often broken free of their foundations, drifted many hundreds of meters from their original locations and been ruptured through impact with other tanks or structures in their path. Rupture leads to environmentally damaging spills and potential fires that can spread quickly if the burning fuel floats on the incoming and outgoing tsunami flows.

Boats and ships that remain in the harbor during a tsunami may experience large currents, even when the tsunami does not cause inundation of port facilities. The resulting forces can fail the mooring system leading to these vessels becoming uncontrolled slow-moving debris, and thus leading to potentially damaging impact strikes on piers, wharfs, mooring systems, and other vessels. If the tsunami results in significant sea level rise, vessels may break free from their moorings and float over the inundated wharves to potentially strike cranes, buildings, storage tanks, and other port infrastructure. Even at low velocity, impact from a large ship will cause significant damage to structures in its path.

3.4 Summary of Gaps and Research Needs

This section summarizes the gaps and research needs that have been identified during the review of the state-of-practice.

3.4.1 Container Wharves

- Development of numerical models for nonlinear pile behavior and deck connections is a difficult process and not a task easily handled by practitioners. There are many issues that remain to be resolved to establish consensus on analysis guidelines for practical approaches to design. These include the following:
  - incorporation of ductile structural elements for different pile types in analyses;
  - consensus on a robust effective stress soil model for liquefaction analyses;
  - procedures for identifying when inertial and kinematic coupling is needed;
  - consensus on appropriate soil-pile coupling elements; and
  - establishment of conditions under which simplified analysis procedures may be used.

3.4.2 Container Storage Yards

- Automated storage yards require precision with rail placement and elevation not previously required. Mitigation measures are needed to reduce both vertical and horizontal displacements during a seismic event to ensure a faster start up after an event.
- Research is needed on less expensive methods for soil stabilization to solidify storage yards, which are subject to large displacements (usually caused by liquefaction) during earthquakes, and thereby reduce damage to containers. Research on less expensive methods than currently available could reduce rebuilding costs after an event.
- Seismic design and performance guidelines and performance metrics, which do not exist for container storage yards, are needed. Including performance guidelines for these facilities could reduce costs for rebuilding if the yards

performed in accordance with established performance criteria. The lack of performance metrics is a need for both new and existing yards.

3.4.3 Cranes

- The effect of vertical ground motions on the response of the crane requires further study.
- Further study is required to understand the crane-wharf interaction in terms of the fundamental period of these two components and the way the period will impact performance during a seismic event.
- Methods of crane design to resist excessive motion during earthquakes are needed for both new and existing systems.

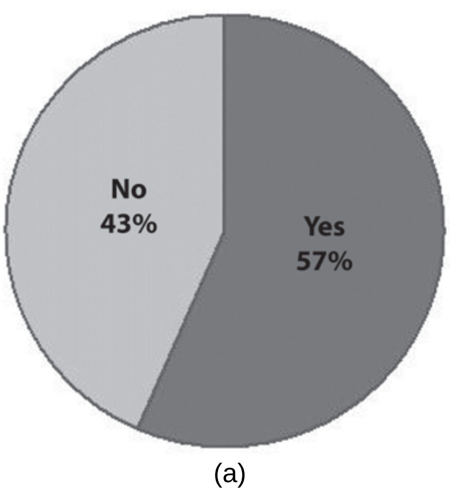
3.4.4 Tsunami Hazards / Sea Level Rise

- Tsunami forecast models should be leveraged for use in probabilistic tsunami hazard analysis for all critical ports and harbors on U. S. soil. Such forecast models include those developed by the National Oceanic and Atmospheric Administration (NOAA) for 75 coastal communities around the U.S. mainland, Alaska, Hawaii, Guam, American Samoa, Puerto Rico, and U.S. Virgin Islands (Tang et al., 2009).
- Methods are needed for the probabilistic analysis of debris strike potential, particularly for ports with large container handling facilities in close proximity to fuel storage tanks.
- Innovative mooring systems are needed that allow for rapid sea-level rise and high currents that may develop in ports during tsunamis.
- Research is needed to improve the understanding and mitigation of tsunami induced liquefaction and the enhanced scour below slab-on-grade pavements and behind wharf retaining wall systems.
- Design criteria for tsunami uplift of pile-supported piers and marginal wharves is needed.

Chapter 4
NEES Grand Challenge Project

In 2005 the National Science Foundation awarded a Network for Earthquake Engineering Simulation (NEES) Grand Challenge project to the Georgia Institute of Technology on “Seismic Risk Mitigation for Port Systems” (NSF Award CMMI-0530478, Glenn Rix, Principal Investigator). The project was motivated by the observation that many large U.S. container ports are located in areas of significant seismic hazard and that seismic risk management practices have not kept pace with the growing importance of container ports to the nation’s economy.

To better understand seismic risk perception and management practices within the ports community, the NEES Grand Challenge project participants sent questionnaires to 126 North American seaports (Scharks et al., under review). The questionnaires targeted the chief engineer or equivalent title at each port. The overall questionnaire return rate was 48.8%. Each port was assigned a Seismic Design Category (A through E) according to ASCE (2006), which was used to group responses according to the level of seismic hazard present at the port. Ports assigned Seismic Design Category A through C were considered to be in an area of “low” seismic hazard; those assigned Seismic Design Category D or E were considered to be “high” seismic hazard. One of the findings from the questionnaire effort was that many (43 percent) of the ports located in areas with high seismic hazard reported no plans to assess seismic vulnerability and a majority (61 percent) have no or only informal seismic risk mitigation plans (Figure 4-1).



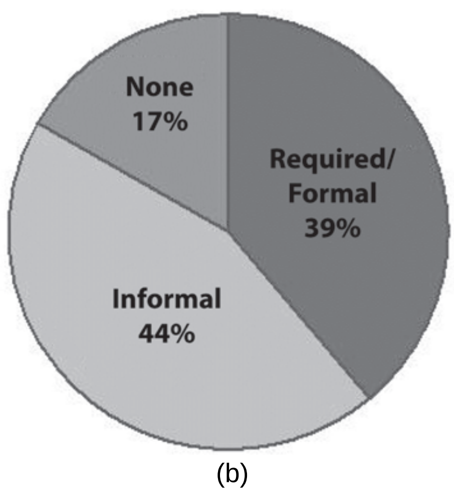


Figure 4-1 Questionnaire responses to (a) “are there plans to conduct a seismic vulnerability assessment?” and (b) “what is the nature of seismic risk mitigation plans?” for ports in areas of “high” seismic hazard.

When ports do address seismic risk, they usually focus on assuring that individual wharves meet displacement-based performance requirements during operating and contingency-level earthquakes, which are usually defined in terms of ground-motion intensities with return periods of 72 and 475 years, respectively (PIANC, 2002). This approach is limited in several respects:

- Potential business interruption losses are not explicitly considered.
- Other key physical and operational components of the port (e.g., container cranes) that are essential to operations following an earthquake are not addressed.
- Displacement-based performance requirements are intended to satisfy vaguely defined damage and operational criteria. For example, PIANC (2002) recommends that the operating-level earthquake (OLE) should cause “minimal damage” and “no downtime” and that the contingency-level earthquake (CLE) should result in “repairable/controllable damage” and “acceptable downtime.”
- These performance requirements are based on arbitrary return periods for ground motions rather than losses.

Lacking are the tools to conduct comprehensive seismic risk analyses that allow port stakeholders to estimate direct and indirect losses relevant to their business interests. Without such information, stakeholders cannot make informed decisions about managing seismic risk to assure business continuity following an earthquake. Thus, the central goal of the NEES Grand Challenge project was to develop a framework for conducting seismic risk analyses for container ports.

To develop the framework, research activities were focused in four primary areas (Figure 4-2):

1. dynamic response and fragility of pile-supported marginal wharves
2. dynamic response and fragility of container cranes
3. port operations and logistics during periods of disruption caused by earthquake-induced damage
4. decision making under uncertainty

Researchers from Decision Research, Inc., Drexel University, Georgia Institute of Technology, Massachusetts Institute of Technology, Seismic Systems & Engineering Consultants, University of California at Davis, University of Illinois, University of Southern California, University of Texas, and University of Washington performed the tasks illustrated by Figure 4-2. This chapter summarizes the key activities and products associated with the development of the seismic risk analysis framework and each of the supporting research tasks.

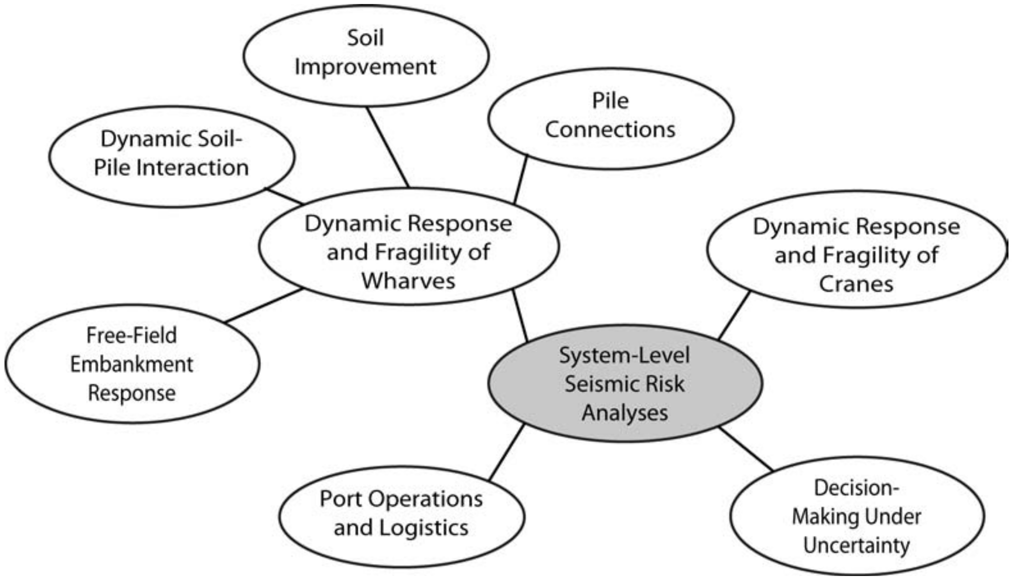


Figure 4-2 Primary research tasks, NEES Grand Challenge project.

4.1 System-Level Seismic Risk Analysis

Estimating earthquake-induced losses for container port systems is complicated by uncertainties in seismic hazard; structural response, damage, and repair; and operational matters. The most robust way to account for these sources of uncertainty is via a probabilistic seismic risk analysis (Woo, 1999; McGuire, 2004):

$$P\left[L > x\right] = \sum_{i=1}^{N_r} P\left[L > x \middle| E_i\right] P\left[E_i\right] \tag{4.1}$$

where $P\,[\,L > x\,]$ is the probability that the earthquake-induced loss, L, will exceed x; $P\,[\,L > x\,|\,E_i\,]$ is the probability that the loss will exceed x conditioned on the occurrence of an earthquake E_i, $P\,[E_i\,]$ is the probability of occurrence of earthquake E_i; and N_γ is number of potential earthquakes that may impact the port.

Implementation of Equation 4.1 involves three basic steps (Ivey, 2012): (1) earthquake hazard models are used to identify potential earthquakes in the vicinity of the port and the corresponding ground-motion intensities at each terminal within the port complex, (2) fragility and repair models are used to estimate the resulting physical damage to container wharves and cranes and the cost and downtime associated with repairing or replacing the damaged structures, and (3) port operations models are used to estimate the business interruption losses because of reduced container throughput and ship delays or re-routing. Each step is explained in more detail below.

4.2 Earthquake Hazard Models

The first step is to define a list of potential earthquake ruptures that may affect the port. In the Grand Challenge project, the Uniform California Earthquake Rupture

Forecast, Version 2 (Field et al., 2009) and the Open Seismic Hazard Analysis (OpenSHA) Event Set Data Calculator (Field et al., 2003) were used to generate a list of possible earthquake ruptures (including background seismicity) within approximately 200 km of the port under consideration and their corresponding annual rates of occurrence. The latter was used to calculate the probability of occurrence of each earthquake rupture. The complete list has N_r = 129,411 possible ruptures.

The ground motion intensity for each rupture was calculated using the ground-motion prediction equation developed by Boore and Atkinson (2008). For sites that are closely spaced (on the order of several kilometers), it is important to account for the spatial correlation of ground motions via the intra-event residuals (Park et al., 2007; Jayaram and Baker, 2009). Jayaram and Baker (2009) developed a correlation model that depends on the distance between two sites. The distance between terminals is generally less than about 5 km; thus, the ground-motion intensities will be highly correlated.

It is also important to account for the spectral correlation between ground motions at different periods. For example, the damage to pile-supported wharf structures was estimated using the peak ground velocity (PGV), while damage to container cranes was estimated using the spectral acceleration (S_a) at T = 1.5 sec. The correlation coefficient between the ground motion intensities at these two periods is calculated using the spectral correlation model developed by Baker and Jayaram (2008).

4.3 Dynamic Response and Fragility of Wharves

Several types of pile-supported marginal wharves that are representative of those currently in service at west-coast ports in the United States were modeled to assess their dynamic response and to develop fragility relationships. For example, Figure 4-3 shows the configuration of a wharf that is typical of design and construction practice in the late 1960s and early 1970s that employed batter piles and a separate landside crane rail. The soil profile includes loose, hydraulically placed sands that are prone to liquefaction.

The seismic response of the wharf structures was evaluated using two-dimensional (2-D) and three-dimensional (3-D) finite element models developed in OpenSees (McKenna et. al., 2010) that integrated contributions from project team members on the response of embankments containing liquefiable soils, dynamic soil-pile interaction in liquefiable soils, soil improvement methods for developed sites, the behavior of pile-deck connections, and seismic response of the wharf structure.

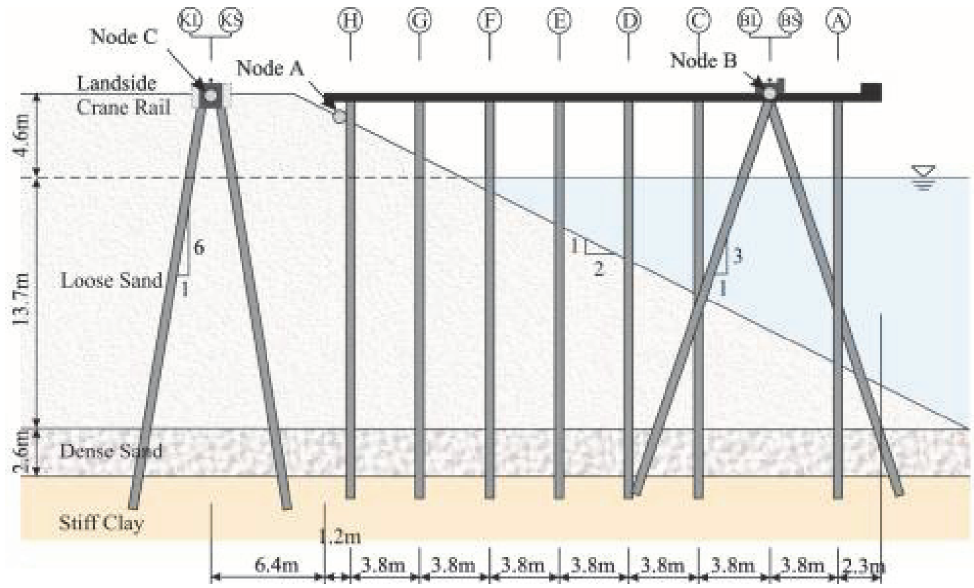


Figure 4-3 Schematic cross section of example pile-supported wharf configuration (Shafieezadeh et. al., 2011a).

4.3.1 Free-Field Response of Soil Embankments with Liquefiable Soils

Numerical simulations of the free-field response of soil embankments were performed using nonlinear, time-domain, finite element methods with coupled flow and deformations in OpenSees (Vytiniotis et. al., 2011). The output quantities include the free field displacements and pore pressures associated with the response of a sloped fill subject to vertically propagating shear waves.

The existing soil constitutive models used in OpenSees have limited predictive capabilities and must be recalibrated for each different initial density of the sand. To address these limitations, a more recent constitutive model presented by Dafalias and Manzari (2004) was selected for use in this study. The Dafalias and Manzari model includes void ratio as a separate state variable and shows realistic predictions of undrained cyclic behavior for laboratory element tests. The model has been successfully integrated into OpenSees for both 2-D and 3-D analyses. The elemental behavior of the model has been validated using a set of calibration parameters for Toyoura sand. The results confirm that the model produces a unique critical state locus for monotonic shearing and can provide realistic simulations of pore pressure accumulation in undrained cyclic shearing (through to the onset of the cyclic mobility phase).

4.3.2 Dynamic Soil-Pile Interaction in Liquefiable Soils

A key aspect of modeling the response of pile-supported marginal wharf structures in liquefiable soils is the dynamic soil-pile interaction. In these cases, empirical techniques based on quasi-static observations and widely implemented in practice for the analysis of dynamic soil-pile interaction problems may not be used to address the role of critical parameters, such as soil permeability, rate of loading and residual soil strength in the wharf performance, or simulate radiation damping phenomena for liquefiable soils in transient loading. On the other hand, very few experimental results exist on dynamic soil-pile interaction effects in liquefiable sites to justify the development of generic mechanical elements for this class of problems.

To address these limitations, a macroelement was developed for soil-structure interaction analyses of piles in liquefiable soils that captures efficiently the fundamental mechanisms of saturated granular soil behavior (Varun and Assimaki, 2010). The mechanical model comprises a nonlinear Winkler-type model that accounts for soil resistance acting along the circumference of the pile and a coupled viscous damper that simulates changes in radiation damping with increasing material non-linearity. Three-dimensional finite element simulations were conducted for a pile in radially homogeneous soil to identify the critical parameters governing the response. The identified parameters (i.e., hydraulic conductivity, loading rate of dynamic loading, dilation angle, and liquefaction potential) were expressed in dimensionless form. Next, the macroelement parameters were calibrated as a function of the soil properties and the effective stress. A semi-empirical approach that accounts for the effects of soil-structure interaction on pore pressure generation in the vicinity of pile was used to detect the onset of liquefaction. The predictions were compared with field data obtained using blast-induced liquefaction and centrifuge tests and found to be in good agreement. Finally, the macroelement formulation was extended to account for coupling in both lateral directions. Finite element simulations indicate that response, assuming no coupling between the two horizontal directions for biaxial loading, tends to overestimate the soil resistance and fails to capture features like negative apparent stiffness, strain hardening and “rounded corners.”

4.3.3 Soil Improvement Methods for Developed Sites

Traditional ground improvement methods to mitigate the effects associated with liquefaction damage are often not feasible in developed areas. Commonly used soil improvement methods can have adverse effects on the surrounding infrastructure and less invasive methods are therefore required. Research performed under the NEES Grand Challenge project has investigated two liquefaction mitigation techniques—prefabricated vertical drains and colloidal silica grout—that are well suited for remediating hydraulic fills prone to liquefaction at port facilities.

Colloidal Silica Grout—Passive site stabilization is a noninvasive grouting technique where a stabilizing material can be injected at the edge of a site and delivered to target locations through the groundwater. As the stabilizer flows through the subsurface, it displaces the pore water and subsequently forms a permanent gel that binds to soil particles, resulting in a more stable soil formation. Based on its unique characteristics, colloidal silica has been selected as an ideal material for passive site stabilization. The dynamic behavior of colloidal silica soils was studied through centrifuge model tests and a complementary, full-scale field test (Conlee, 2010). The centrifuge model tests and field test show colloidal silica soils to reduce pore pressure response and the shear strains induced when subjected to large dynamic loads.

Prefabricated Vertical Drains—A series of centrifuge tests demonstrated that prefabricated vertical drains were effective in dissipating the excess pore water pressures both during and after shaking and reducing the associated deformations (Howell et al., 2012). There was a 30 to 60 percent improvement in the horizontal deformations and a 20 to 60 percent improvement in the vertical settlements. The impact of the prefabricated vertical drains on the excess pore water pressure response was sensitive to the characteristics of the input motion. The drains were more effective in reducing the peak excess pore water pressures when the input motion built up gradually in intensity such that there was more time for the pore water to flow to the drains. However, even during those events in which the pore water did not have time to flow to the drains during shaking, the drains still increased the rate of excess pore water pressure dissipation, prevented the loosening of soil near the low-permeability interface and the associated localized shear deformations, and reduced the overall horizontal and vertical deformations.

A field test was also performed to investigate the performance of prefabricated vertical drains at a natural site (Marinucci et al., 2010). The recorded data show that the installation of the drains provided significant pore pressure generation and densification. Therefore, the prefabricated vertical drain installation provides an additional component of soil improvement: densification. After drain installation, large-scale shaking was performed, and a significant decrease in pore pressure generation was observed.

4.3.4 Pile-Deck Connections

Experience from previous earthquakes indicates that the connection between the pile and the wharf deck is a major source of damage during an earthquake. Previous studies indicate that although connections designed using current guidelines can maintain cyclic drift demands, they sustain damage and initiate strength deterioration, even at low levels of drift. Therefore, there is an interest in improving the performance of precast pile connections. A study to improve the performance of pile-wharf connections was undertaken to develop damage-resistant connections (Lehman et. al., 2009). To mitigate the pile and deck damage, several structural concepts were evaluated including (1) intentional debonding of the headed reinforcing bars, (2) supplemental rotation capacity through the addition of a cotton duck bearing pad, and (3) supplemental material to sustain the lateral deformations while minimizing deck damage. The test results show minimal damage and strength loss relative to current practice and achieving performance-based design objectives. These results have been combined with prior test results on connections. By separating the connection responses into several common categories, individual connection-rotation based fragility curves have been developed for three repair-specific damage states. These curves would be appropriate for performance-based evaluation of older, current, or innovative connections in port construction.

4.3.5 Seismic Response of Pile-Supported Wharves

Using the results of the tasks described above, a 2-D plane strain model of the wharf structure shown in Figure 4-3 was created in OpenSees (Shafieezadeh et. al., under review). The results of complex modal analysis of the wharf indicated large levels of damping ratios of the system ranging from 36.1 percent to 81.5 percent. The modal damping ratios found in this study are large compared to the modal damping ratios typically assumed in prior studies that do not include the nonlinear soil behavior. This can affect the procedures for simplification of the wharf models using equivalent linear elastic models.

Nonlinear time history analyses using a suite of 63 observed and simulated ground motions were also performed. For strong ground shaking, the results of these analyses demonstrate the following:

- Liquefaction-induced movements of the soil embankment impose large deformations in the piles and pile-deck connections that are sufficient to severely damage the wharf.
- Large relative horizontal displacements can develop between the structurally separate landside crane rail and the wharf deck (which supports the seaside crane rail).
- Severe damage to the wharf occurs in (1) pile sections near the interface of liquefiable and nonliquefiable sand layers, (2) pile sections close to the surface of the embankment, and (3) pile-deck connections as indicated in Figure 4-4, which shows the curvature ductility demand, deformed shape, and location of plastic hinges for the wharf subjected to a ground motion time history with a peak horizontal ground acceleration (PGA) of 0.64g.
- The damage patterns observed in this study coincide with damage patterns observed in historic cases of wharf damage in past earthquakes.
- Batter piles experience large axial forces that may exceed the tensile capacity of the piles.

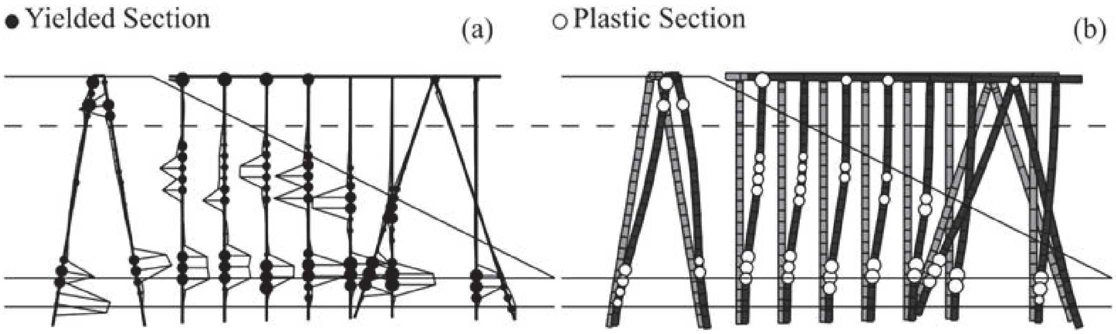


Figure 4-4 Example of maximum curvature ductility demand, deformed shape, and location of plastic hinges from a nonlinear time history analysis (Shafieezadeh et. al., under review).

For small-to-moderate levels of ground motion intensity, liquefaction of the embankment soils did not occur, and damage to the wharf structures was minor.

The effects of wharf-crane interaction were studied using a sliding/uplift capable model of a container crane. When the wharf was subjected to time histories of ground displacement and excess pore water pressures within the underlying soil embankment, and nonlinear time-history analyses were performed, it was found that, in contrast to the conclusions of earlier studies, the dynamic interaction between the wharf and crane may amplify the response of the wharf. These results suggest that wharf–crane interaction should be given more careful consideration than is currently required when evaluating the seismic response of a wharf system. Furthermore, simple analytical limits on the spectral acceleration response of the crane rails were derived to predict the occurrence of sliding, uplift, and yielding of the portal frame. The results of the numerical simulation show that the derived limits perform well in predicting the response of the crane.

Because of the complexity of the numerical modeling and analysis of wharf systems, very few studies investigated three-dimensional seismic response of wharves. Shafieezadeh et. al. (in press) developed a procedure for three-dimensional modeling of wharves in liquefiable soils. The framework included modeling structural elements and soil springs, and generating out-of-plane ground deformations. Modal analysis results of the wharf show that torsional modes have a predominant role in the inertial response of the wharf. However, the transverse seaward deformation of the wharf because of the transverse deformation of the liquefied sand layer was found to govern the overall wharf response.

Large computational times required for nonlinear dynamic analysis of the wharf system is a major drawback for this method of analysis to be applicable for probabilistic seismic demand analysis in which a large number of simulations are required. A simplified model of the three-dimensional wharf was introduced in this study by lumping the properties of wharf segments into a single-strip representative model. The simplified model was shown to closely capture linear and nonlinear dynamic behavior of wharves found by modal and time-history analyses. Furthermore, a simplified analysis technique was proposed to estimate the maximum response of the full wharf using nonlinear dynamic analysis results of a simplified wharf model combined with a nonlinear static pushover analysis of the wharf.

Probabilistic seismic demand models of critical response measures of the wharf were developed using either linear or bilinear regression models. The choice of intensity measure was made based upon a rigorous probabilistic analysis of various intensity measures and different demand parameters of the wharf. It was found that peak ground velocity is the most appropriate intensity measure for demand modeling of wharf systems in liquefiable soils.

Convolving the probabilistic demand models with component capacities, a series of fragility functions were developed for critical wharf components using two- and three-dimensional wharf models. The capacities or limit states used to develop fragility curves were derived from numerical simulations, experimental results, and expert judgment. Fragility analysis of wharf components showed that the relative movement of the wharf with respect to the landside rail is the most susceptible component to slight and moderate damage. However, pile sections are the most vulnerable components of the wharf to extensive damage primarily because of the large deformation demands on the piles at the interface of loose and dense sand layers. Furthermore, fragility curves of the three-dimensional wharf model were found to exhibit larger probabilities of failure compared to the corresponding quantities from the two-dimensional wharf model. This finding is not particularly surprising because longitudinal and torsional responses of the wharf in the three-dimensional wharf model contribute to component responses.

4.4 Dynamic Response and Fragility of Container Cranes

Since the 1960s, crane designers allowed and encouraged an uplift response from container cranes, assuming that this uplift would provide a “safety valve” for seismic loading (i.e., the structural response at the onset of uplift was assumed to be the maximum structural response). However, cranes are now much larger and more stable such that the port industry is beginning to question the seismic performance of their modern jumbo container cranes.

Research performed on the NEES Grand Challenge project has taken a step back and reconsidered the effect that uplift response has on the seismic demand of portal-frame structures, such as container cranes. One primary objective of this work was to develop methodologies for realistically modeling this effect and to serve as a foundation for the design and evaluation of new and existing container cranes.

The seismic behavior of container cranes was investigated by two large-scale experiments using a six-degree-of-freedom shake table (Jacobs, 2010; Jacobs et al., 2011). This six-degree-of-freedom table is located at the NEES site at the State University of New York at Buffalo. The characterization of uplift and derailment behavior was of particular interest. The first phase of testing was conducted on a 1/20th scale model and focused on the uplift and elastic behavior. The data collected confirmed that a simple tipping analysis is sufficient to predict when derailment will occur. The results from the Phase I test also indicate that torsion has little effect on the overall response of a jumbo container crane, suggesting that 2-D finite element models are sufficient for analysis.

The Phase II test was designed to be representative of a modern jumbo crane. It was also designed such that no inelastic action would develop prior to uplift (as is the common design practice). During testing, the crane yielded, buckled, and reached an unstable state after uplift, which challenges the conventional wisdom that uplift will provide a limit to the seismic forces. Two different boundary conditions were used, pinned and free to uplift, for the validation of finite element models with different boundary conditions. There were cases for which the pinned boundary condition led to higher forces, suggesting that cranes in earthquake zones should not be pinned except when the weather dictates otherwise. The test specimen was subjected to a suite of ground motions with various combinations of components. The data show that there are cases for which the inclusion of a vertical component will result in the most extreme load case. This result suggests that it would be prudent to run models with and without a vertical component to find the worst case.

A theoretical tool for estimation of seismic demand was developed that accounts for the uplift behavior and finds that the “safety valve” design assumption can be unconservative (Kosbab, 2010). This portal uplift theory is verified with detailed finite element models incorporating frictional contact elements and experimental shake-table testing of a scaled jumbo container crane allowed to uplift.

Using the verified models developed in this work, fragility curves and downtime estimates were developed for three representative container cranes (Figure 4-5). Throughout the analysis, appropriate levels of uncertainty and randomness were evaluated and propagated. Because the damage levels were defined globally and according to specific repair models, probabilistic estimates of the operational downtime, because of a given earthquake, were also developed. The results indicate that existing container cranes, especially stout cranes and those not specifically detailed for ductility, are not expected to achieve the seismic performance objectives of many ports.

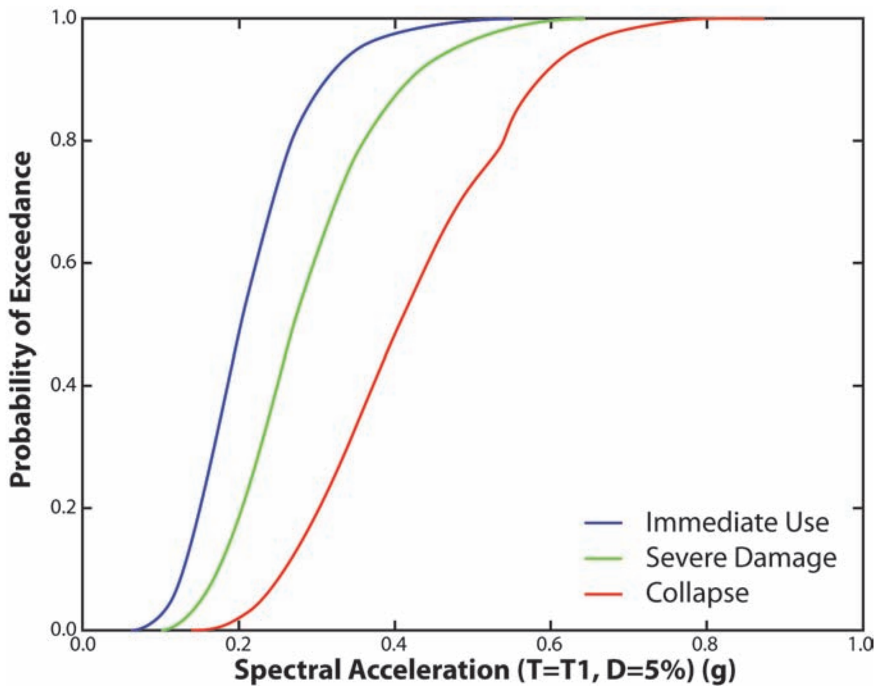


Figure 4-5 Fragility curves for a typical older 100-ft gauge container crane (Kosbab, 2010).

To address this deficiency, performance-based design recommendations were provided that encourage the comparison of demand and capacity in terms of the critical portal deformation, using the derived portal uplift theory to estimate seismic deformation demand. Simplified methods and basic design factors were proposed and demonstrated, which enable practitioners to conveniently design for reliable achievement of seismic performance objectives.

4.5 Repair Models

Once the extent of physical damage to the wharf and crane structures within the port complex is tabulated for each earthquake scenario, repair models are used to estimate the cost of repairing or replacing the damaged structures and the downtime associated with completing the repairs or replacements (Werner and Cooke, 2010). A considerable effort was made to develop detailed estimates of repair/replacement costs and downtime. An example for pile damage is shown in Table 4-1. Similar repair models were developed for other wharf and crane damage states.

4.6 Port Operations Models

The primary goals of the port operations models developed for the NEES Grand Challenge project were to simulate how terminal operators will manage operations during periods of disruption to port infrastructure, and to use this simulation to estimate key port performance metrics, such as container throughput, ship delays, and ship re-routing.

Table 4-1 Repair Model for Vertical Pile Damage (from Werner and Cooke, 2010)

Symptom	Crushed and badly damaged pile head. Rupture and displacement along inclined crack.		
Assessment	In shallow water: large lateral displacement. In deeper water: P-delta column effect near expansion joints or torsion		
Repair Method	Irreparable Damage; replace with new pile adjacent to broken pile.		
Repair Estimate	*Work Item*	*Repair Cost*	*Downtime*
	Post earthquake inspection design of repairs	$10,000	5 days
	Mobilization/demobilization of floating pile-driving equipment	$75,000	5 days (if $M_w < 7.0$)[1] 10 days (if $M_w \geq 7.01$)[1]
	Furnish 24-inch octagonal piles		
	Set up in casting yard.	$10,600	30 days[1]
	Cast 90-foot x 24-inch diameter pile @ $75/foot	$6,800	14 days
	Deliver to site.	$2,200	1 day
	Break hole in deck, cut rebar.	$8,500	2 days
	Remove riprap, replace.	$5,300	2 days
	Place and strip forms.	$6,900	3 days
	Splice rebar and add new rebar.	$5,900	2-days
	Concrete opening and cure.	$2,100	7 days
	Totals for first broken pile	$146,000[2,3]	67 days[2]

Note: [1] Mobilization and inspection/repair-design are assumed to occur concurrently with pile construction in casting yard.
[2] For next 2-3 broken piles, add 2 days of downtime and $51,000 of cost per pile. For 4 or more broken piles, assume 4 piles are driven per day, and add $45,000 in cost for each pile.
[3] Contingencies are to be added to these repair costs.

There are many ways to estimate performance metrics during periods of disruption. The simplest models would apply linear scaling factors to ship and container processing rates given the portwide system damage state. For example, if 30 percent of the wharf structures and 40 percent of the cranes were damaged at a particular terminal, the processing rate could be set at 60 percent of the nominal (i.e., pre-earthquake) rate and a simple queuing model could be used to estimate performance.

In this study, a different approach was employed to capture potential non-linearity. Using an optimization-based heuristic scheduling technique (Ak, 2008), the decision-making of terminal operators was simulated to determine: (1) what arriving vessels may be turned away or will go to another port, (2) what times arriving vessels will be berthed, (3) where they will be berthed within a particular terminal complex, and (4) which cranes will be assigned to the vessels and when. The scheduling technique makes these decisions by assigning berth space and cranes that are available at the time after the earthquake when the ship is scheduled to arrive at the port. These assignments maximize an objective function that seeks to mimic the objectives of the terminal operators, balancing the need to process containers quickly given available resources while avoiding excessive delay to any arriving vessels. The technique uses a rolling time horizon that considers the schedule of arriving vessels several days into the future; berth and crane assignments are updated daily based on the new information. The technique also has the ability to allow arriving ships to berth only at their planned destination terminal or to allow them to use available berths within other terminals as a result of possible "force majeure" agreements between terminal operators.

The inputs to the port operations model include the day-by-day, port-wide system damage state and the schedule of arriving ships within some period of time that extends from the time of earthquake occurrence up through the time when normal port operations are restored (i.e., when all of the damaged infrastructure is repaired). Models for estimating the number of arriving ships and their characteristics (e.g., length and capacity in terms of twenty-foot equivalent units) were based on data from the Marine Exchange of Southern California, which tracks ship arrivals and departures from the Port of Los Angeles and the Port of Long Beach.

An example of the output from the model is shown in Figure 4-6. The line identified as "No Damage" is the number of twenty-foot equivalent units (TEUs) handled per week for the case where the berth (i.e., wharves and cranes) is fully functional. The red lines denoted by (x,y) indicate the cumulative number of 600-foot wharf segments (x) and containers cranes (y) that are available following a large, damaging earthquake. The data in the figure indicate that it takes approximately 49 weeks to repair all five wharf segments and all six container cranes. The number of twenty-foot equivalent units processed while the damage is being repaired is indicated by the green line identified as "Earthquake." As the wharf segments and cranes are repaired, capacity for handling cargo is restored until normal operations are restored in about week 47.

From these results, the number of TEUs not able to be processed because of earthquake-induced damage can be tabulated for each terminal and berth. Business interruption losses may be estimated using financial information such as the revenue derived by the port and terminal operator for each TEU or the cost per hour for delayed ships. Adding these business interruption losses to the repair and replacement costs for wharves and cranes in each terminal yields the total port-wide losses because of damage from a given earthquake.

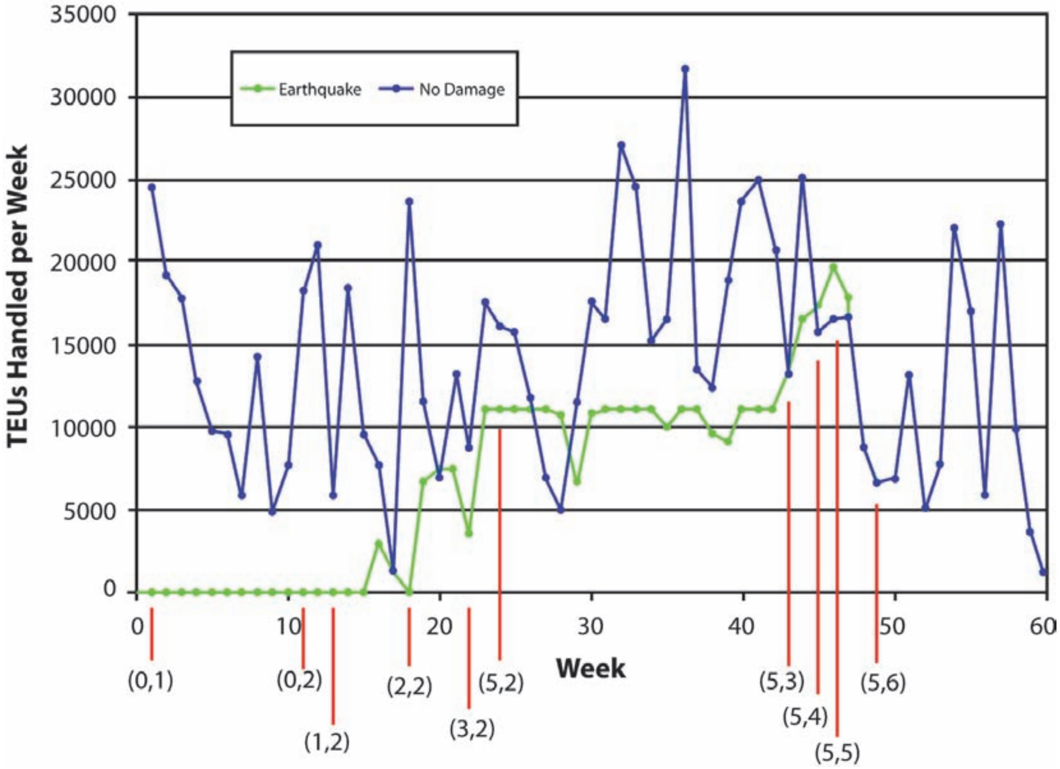


Figure 4-6 Sample output from port operations model (from Ivey, 2012).

4.7 Sample Results

The hazard, wharf and crane fragility, repair, and port operations models described in the previous sections have been integrated into the system-level seismic risk analysis framework described by Equation 4-1. The framework was applied to a hypothetical port located in Santa Cruz, California to illustrate the type of results that are generated (Ivey, 2012). The hypothetical port consists of four individual terminals; results are shown here for Terminal D, which has a length of 3,600 feet with eight 100-foot gauge cranes. The wharf configuration is shown in Figure 4-3 and is typical of design and construction practice in the late 1960s and early 1970s that employed batter piles and a separate landside crane rail. The soil profile includes loose, hydraulically placed sands that are prone to liquefaction. The container cranes are representative of older jumbo cranes, which were assumed to uplift and thus limit seismic forces. The portal frame elements are slender with very low ductility. The mean interarrival time between container ships at Terminal D is 1.47 days.

Figure 4-7 shows one of the basic results from the risk analyses—loss exceedance curves for (1) wharf and crane repair/replacement costs, (2) business interruption losses, and (3) the total loss that show the annual probability of the respective losses exceeding a specific value. The business interruption losses dominate the total loss, and thus not explicitly including them in seismic risk analyses may severely underestimate earthquake-induced losses at container ports.

Other information may be easily derived from the loss exceedance curves. For example, the mean annual total loss for Terminal D is \$4.0 million. Many ports have a long-term planning horizon; the results of the risk analyses allow one to calculate that the mean total loss for a 20-year planning period (assuming a 5 percent discount rate) is \$50 million. It is also possible to determine the losses associated with a specific return period. For example, the repair cost, business interruption loss, and

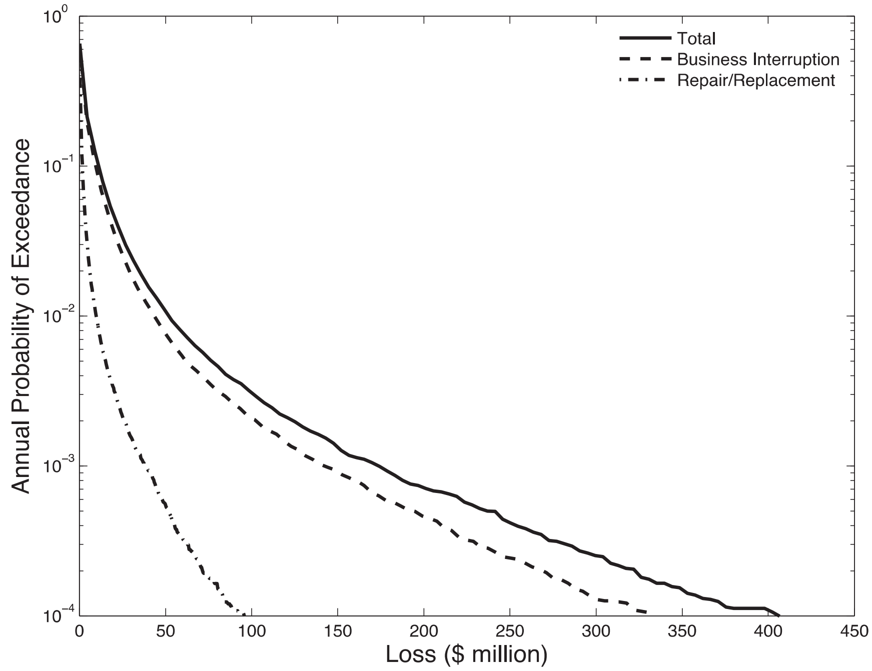


Figure 4-7 Loss exceedance curves for repair/replacement costs, business interruption losses, and total losses for a terminal at a hypothetical port (Ivey, 2012).

total loss for a return period of 475 years (annual probability = 2.1 x 10^{-3}) are approximately \$25 million, \$100 million, and \$120 million, respectively. This approach contrasts with current practice, which states that earthquake ground motions with a return period of 475 years should result in vaguely defined performance requirements that include "repairable/controllable damage" and "acceptable downtime."

The risk analyses can also be performed for a specific earthquake scenario (magnitude and distance) or specific ground motion intensity.

4.8 Applications

The seismic risk analysis framework developed in the NEES Grand Challenge project has many potential applications, including the following:

- performing benefit-cost analyses for investments in alternative seismic design and mitigation strategies
- prioritizing port construction projects by identifying facilities that are critical to business continuity following an earthquake
- calibrating current seismic design approaches based on operating-level and contingency-level earthquakes to determine whether they satisfy the desired performance requirements
- estimating average annual loss and probable maximum loss for insurance rate-making, portfolio management, and risk financing
- planning for using port facilities for emergency response operations

4.9 Summary

A systems-based approach for seismic risk analysis and management for container ports is a multidisciplinary process that incorporates technologies associated with the geosciences, geotechnical and structural engineering, construction engineering, logistics, and risk and decision analysis. It enables port stakeholders to consider the consequences of risk management options on the post-earthquake performance of the overall port system. In this sense, the procedure represents a new direction for reducing seismic risks to container ports based on better-informed decisions regarding preferred options.

The risk analysis procedure that is summarized herein was developed to assess waterfront infrastructure (i.e., wharves and cranes) only. However, experience has shown that the post-earthquake functionality of a port system will depend not only on these facilities but also on other key elements of the port system infrastructure. These include the roadways and rail systems that service the port, the lifeline infrastructure for providing the port with power, natural gas, and communications, and buildings that house key administration, computer, security, and other key port system elements. The future extension of this port system risk analysis procedure to also assess seismic risks to these other infrastructure represents an important goal for further development of the procedure. In addition, the procedure is not only applicable to the assessment and management of seismic risks, but can also be readily adaptable to the assessment of risks to a container port system from other natural hazards (such as extreme wind, tsunami, and flood) and accidental or deliberate manmade hazards.

Chapter 5
Recommended Guidance Documents

The state-of-practice review of existing port seismic design guidelines and concurrent literature review of seismic design issues and approaches for port container handling facilities, including wharves, cranes and container yards, uncovered a significant lack of seismic design guidance and information for these facility types. Seismic design guidance and related information for other important port facilities, however, do exist or are under development.

Container handling facilities are changing as the need for cost reduction and increased handling speed become more important issues for port owners and operators. The following three developmental needs have been identified (as a result of the state-of-practice review) that require significant research and development work to produce guidance for design practitioners and owners/operators:

1. development of guidance for developing performance criteria for container cargo systems
2. development of seismic retrofit design guidance for container wharves, including guidance for the design of ground improvements for embankments and cargo storage yards
3. development of guidelines for numerical modeling of container wharves under kinematic embankment loading

The proposed scope of each of these guidance documents is briefly described below. Work plans for preparing each document are discussed in Chapter 6.

5.1 Guidelines for Developing Seismic Performance Criteria for Container Cargo Systems

A container cargo system includes marginal wharf structures, embankments, container cranes, and cargo storage yards. Each element of the cargo system must be operational for containerized cargo to move through a port. If one or more elements are rendered unusable, then the entire system cannot function. The system is only as strong as its weakest link.

Marginal wharf structures and embankments have performance-specific criteria outlined under several current guideline documents, such as those of the Port of Los

Angeles (2004), Port of Long Beach (2009), American Society of Civil Engineers (ASCE, 2012, draft) and International Navigation Association (PIANC, 2002). Other components such as cranes and cargo storage yards have no specific performance criteria. In addition, the overall system performance is not adequately defined by the current practice.

To adequately define the overall system performance, the performance requirement definitions need to include requirements to maintain business continuity after an earthquake. Identifying the economic losses that will occur as a result of earthquake damage to some or all of the key port system components and developing acceptable business loss criteria are two of the key issues that need to be addressed. Economic losses and the maintenance of business continuity will be dependent on the severity and nature of the earthquake hazard, port component fragilities, repair scenarios, mitigation measures (such as optimizing facilities that are functional) and system downtimes for repair.

As described in Chapter 4, the Network for Earthquake Engineering Simulation (NEES) Grand Challenge project addressed (among other issues) the performance of wharves and wharf cranes (waterfront structures), and included an example risk analysis to wharves and wharf cranes (waterfront structures) of a hypothetical example port. The Grand Challenge project recognized, however, that the post-earthquake functionality of a port system will depend not only on these waterfront facilities but also on other key elements of the port system infrastructure. These include the roadways and rail systems that service the port, the lifeline infrastructure for providing the port with power, natural gas, and communications, and buildings that house key administration, computer, security, and other key port system elements.

The development of *Guidelines for Developing Performance Criteria for Container Cargo Systems* is recommended. This document should go beyond the wharf/crane systems analyzed under the NEES Grand Challenge project to include guidance for development of performance criteria for all components of port container cargo handling systems.

Challenges to develop these guidelines and apply them to each port require close collaboration between port owners and terminal operators to clearly define operational boundaries for each entity, develop fragility models for all the components, establish repair models, and develop port operations models that can be used to optimize port facilities that are functional after a seismic event.

5.2 Guidelines for Seismic Retrofit Design of Container Wharves, Including Ground Improvement

Container wharf components are the wharf structure, embankment, and cargo storage yards. Design guidelines for new container wharf structures and embankments are currently available. One reference (City of Los Angeles, 2004) includes brief design provisions for seismic upgrade and repair of existing container marginal wharves. There are available methodologies for the design of ground improvements for embankment and cargo storage yards, but there are no guidelines for the design of ground improvements for embankments and cargo storage yards. The proposed guidelines should address the following:

- Developing procedures to evaluate the seismic capacity and seismic performance expectations of existing container wharves
- Determining performance collapse criteria for container wharf structures
- Developing guidance to determine the need for seismic retrofit, or replacement of existing container wharves prior to a seismic event
- Defining seismic performance requirements for exiting container wharves
- Optimizing current methodologies and criteria for new designs for use in the development of seismic retrofit requirements for wharf structures and cargo storage yards
- Preparing guidance for the design of ground improvements for embankments and cargo storage yards
- Developing guidance to address the effect of tsunamis, and sea-level rise, in the seismic retrofit of existing container wharves and embankments

As part of the development of these guidelines, there is a need to review available international documents that may include retrofit and ground improvement guidelines.

Ultimately, the proposed guidance documents could be incorporated in an engineering design standard, or set of standards, by a group such as the American Society of Civil Engineers. Substantial research and development efforts will be required to prepare the information to be included in such standards.

5.3 Guidelines for Numerical Modeling of Container Wharves under Kinematic Embankment Loading

Seismic loading of piles include two components: inertial loading because of the wharf mass and kinematic loading because of to the embankment deformation. These loadings could be coupled or uncoupled depending on the configuration of the embankment and the piles. Kinematic interaction analyses need to be performed using the kinematic embankment loading to determine the stresses in the piles.

Current available guidelines focus on simplified methods for kinematic interaction analyses, such as the Newmark sliding block approach. Although more detailed numerical analysis, such as finite element methods, have been used, including the effects of pore pressure buildup and liquefaction, there is a lack of guidance on the appropriate approach for such analyses. In addition, there is a lack of guidance on the criteria on coupling or uncoupling the inertial and kinematic loadings. The approach to the development of these guidelines would include the following:

- reviewing available guidelines on kinematic interaction analyses, and assessing the applicability and appropriate use of numerical analyses, including the recent evaluation prepared by the NEES Grand Challenge project
- determining the appropriate and practical numerical analyses methods
- defining guidelines for the selection of appropriate soil parameters for analyses
- undertaking parametric studies to determine inertial and kinematic loading coupling and uncoupling criteria
- undertaking parametric studies to calibrate analysis methods from available case histories of embankment seismic deformations

Chapter 6

Work Plan

This chapter summarizes the recommended work plan tasks, schedule, and budget for a multiyear program to develop *Seismic Design Guidelines for Port Container Wharves and Cargo Systems*, a set of documents consisting of the following:

1. *Guidelines for Developing Performance Criteria for Container Cargo Systems*
2. *Guidelines for Seismic Retrofit Design of Container Wharves, Including Ground Improvement*
3. *Guidelines for Numerical Modeling of Container Wharves under Kinematic Embankment Loading*

Also included is a list of key collaborators for the overall work effort.

6.1 Work Plan Objectives

The objective of this suite of proposed documents is to provide comprehensive, state-of-the-art guidance for design practitioners and port owners/operators on the seismic design and retrofit of ports, filling a gap in current information as it relates to container cargo systems, container wharves, and embankments.

The proposed guidance documents are intended to be stand-alone so that each can be developed independently of the other. It is recommended that Document 1 on performance goals and Document 3 on numerical modeling be substantially completed prior to beginning work on Document 2 on seismic retrofit design, as the definition of performance goals and the results of the numerical modeling effort could be used in the development of the retrofit guidelines. This recommendation is a preference, however, not a proposed requirement.

6.2 Work Plan Overview

The Work Plan tasks address the development of the three above-specified guidance documents. The intent of each task is to address only features and components of ports that are unique to that use and not to attempt to address those features and components that are not unique. In other words, the tasks to develop the three recommended guidance documents only address marginal wharf structures and embankments, container cranes, and cargo storage yards.

Document 1 is a guidance document on the development of seismic performance criteria for container cargo systems. The Work Plan tasks to develop this document focus primarily on defining component performance for cargo storage yards and the other port facilities needed for cargo handling, for which criteria do not already exist. As indicated in Section 5.1, including requirements and criteria to maintain business continuity will be an important consideration during the development of this document.

Document 2 is a guidance document for the seismic retrofit of container wharves, including design guidance on ground improvement techniques for embankments and cargo storage yards. There is currently little guidance on what should be considered in a port seismic retrofit project as it applies to marginal wharf structures and embankments, container cranes, and cargo storage yards.

Document 3 is a guidance document on numerical modeling of container wharves under kinematic embankment loading. Widely-used approaches for determining these important loading criteria are based on simplified methods. This guidance document should include new robust and practical analysis tools for use by practicing engineers.

6.3 Work Plan Tasks

This section describes the recommended Work Plan tasks to develop each of the guidance documents.

6.3.1 Document 1—Guidelines for Developing Seismic Performance Criteria for Container Cargo Systems

The development of seismic performance criteria for container cargo systems necessarily encompasses consideration of the entire port system. Given that the focus of the work is on those features and components that are unique to ports, the guidance must consider the holistic nature of the "entire system." Of great utility will be the performance criteria information and guidance already developed for wharves and cranes (waterfront structures) under the NEES Grand Challenge project.

It is recommended that this document be substantially developed prior to the start of Document 2 on seismic retrofit design, as the procedures for defining seismic performance criteria should be considered in the development of the seismic retrofit guidance.

The Work Plan to develop this document includes the following tasks.

Task 1.1: Define the Boundaries/Scope of the Seismic Performance Criteria

This first technical task focuses on defining the scope of the seismic performance criteria. This requires the identification and description of the system components that are essential to cargo handling, including marginal wharf structures, embankments, container cranes, cargo storage yards, and other port facilities.

Task 1.2: Define Expected Performance of Components

A port cargo handling system is made of a series of components. The performance of some of these components depends on the performance of other components; others are independent of the performance of other components but are crucial to the overall acceptable performance of the system. This task focuses on the development of procedures to define performance criteria for port cargo handling system components that are dependent on the performance of other components and those that are independent.

Task 1.3: Define Expected System Performance

Using the information developed under Task 1.2, the purpose of Task 1.3 is to develop procedures to define the overall system performance in a way that provides meaningful information to maintain business continuity and to minimize business losses. Performance levels might be defined that are patterned after other seismic performance criteria that specify performance in terms of discreet levels: for example, in terms like serviceable (minor to no damage), repairable (controlled damage), near collapse (extensive damage), or collapse (complete loss). Another option is to define overall system performance criteria in specific terms of use in economic analysis, such as repair costs and time out of service (downtime).

Task 1.4: Conduct Peer Review

Peer review and evaluation of the procedures for developing performance criteria for components of port facilities and for combining that information to estimate overall system performance is a critically needed part of the developmental process. The process includes the identification and engagement of a broadly based panel of subject matter experts, who would be assigned the responsibility for reviewing and evaluating the technical, business and economic basis of the procedures for defining component seismic performance criteria and overall system performance. The Peer Review Panel should include port and facility operators, specialists in the seismic design of port facilities, and researchers specializing in the seismic performance of port facilities, including embankments and cargo yards.

Task 1.5: Develop Written Guidelines

This task focuses on the development of written guidelines for developing seismic performance criteria for container cargo systems. The guidelines are to be based on the criteria and procedures developed in the previous tasks, and should incorporate substantial and continuing peer review during the entire guidelines developmental process.

6.3.2 Document 2—Guidelines for Seismic Retrofit Design of Container Wharves, Including Ground Improvement

This guidance document will fill a significant gap in available knowledge and technical resources for seismic retrofit of existing container wharves and associated structures, embankments, and cargo storage yards. This work should be started after Document 1 on performance criteria and Document 3 on numerical modeling have been substantially completed so that the performance criteria selection procedures and numerical modeling analysis guidance defined in these documents can be considered in the development of seismic retrofit design guidance.

The Work Plan to develop this document includes the following tasks.

Task 2.1: Literature Review

The initial task is to conduct a literature search to identify and review available technical resources on seismic retrofit design and the seismic performance of retrofitted container wharves. The task should be undertaken by subject matter experts (a working group), including structural engineering and geotechnical engineering professionals experienced in the seismic design and performance of container wharves, researchers experienced in the investigation of the seismic performance of marginal wharves and embankments, and a container wharf operator or owner representative(s) who can address issues that arise pertaining to the operational aspects of container wharves.

Task 2.2: Develop Performance Criteria

This task should consider the procedures and performance definitions developed for Document 1, with a special focus on developing performance criteria for seismic retrofit projects. The performance criteria for retrofit projects may not be the same as that developed for new installations, given the restrictions with which retrofit projects must contend. This task should be carried out by a working group of subject matter experts with qualifications similar to those specified for the conduct of Task 2.1. Review of the proposed performance criteria by a broadly-based panel of specialists (see Task 2.6) is an important aspect of this task.

Task 2.3: Develop Seismic Retrofit Approaches for Structures

The seismic retrofit approaches to be identified and developed in Task 2.3 should be based on the performance criteria identified in Task 2.2. The approaches, and in some cases techniques, must consider the age and condition of the existing facility, the current operation and any restrictions on operation, and some method for evaluating the operational impact on the retrofitted structure immediately following a seismic event. As suggested in Section 5.2, the seismic retrofit approach developmental process should include the review and optimization of current methodologies and criteria for the seismic design of new wharf facilities. In addition, guidance is needed on the most appropriate means to address the effect of tsunamis, and sea-level rise, in the seismic retrofit of existing container wharves and embankments.

This task should be conducted by one or more working groups with similar skills and experience to specialists involved in the conduct of Tasks 2.1 and 2.2.

Task 2.4: Develop Hypothetical Seismic Retrofits and Conduct Structural Engineering Evaluations to Determine if the Retrofits Meet the Selected Performance Criteria

The task begins with the development of hypothetical seismic retrofits of container wharves based on the performance criteria and retrofit approaches developed under Tasks 2.2 and 2.3. The task also includes the conduct of structural engineering evaluations of these hypothetical seismic retrofits to determine if the hypothetical retrofit measures provide the strengthening and resiliency needed to meet the selected performance criteria. The task should be conducted by a working group consisting of structural and geotechnical engineers experienced in the seismic design of wharf structures, with appropriate input by qualified researchers.

Task 2.5: Development and Evaluation of Embankment-Related Ground Improvements

This task involves the development and evaluation of ground improvement techniques for embankments and cargo storage yards. Consideration should be given to the practicality and costs of proposed ground improvement measures. A working group of geotechnical engineers experienced in the development and evaluation of ground improvement techniques should be engaged to conduct this task.

Task 2.6: Conduct Peer Review

This task involves the identification and engagement of a broadly based panel of subject matter experts, who would be assigned the responsibility for reviewing and evaluating the retrofit guidance, including ground improvement techniques, developed under Tasks 2.1 through 2.5. The Peer Review Panel should include port and facility operators, specialists in the seismic design of port facilities, and researchers specializing in the seismic performance of port facilities, including ground improvement for embankments and cargo yards.

Task 2.7: Develop Written Guidelines

This task focuses on the development of written guidelines for the seismic retrofit design of container wharves, including ground improvement techniques for embankments and cargo storage yards. The guidelines are to be based on the criteria and procedures developed in the previous tasks, and should incorporate substantial and continuing peer review during the entire guidelines developmental process.

6.3.3 Document 3—Guidelines for Numerical Modeling of Container Wharves under Kinematic Embankment Loading

The purpose of this document is to provide for numerical modeling of container wharves under kinematic embankment loading. It is recommended that this document be substantially developed prior to the start of Document 2 on seismic retrofit design, as the numerical modeling procedures should be considered in the development of the seismic retrofit design guidance.

The Work Plan to develop this document includes the following tasks.

Task 3.1: Literature Review

This task involves the review of available literature and guidelines on kinematic interaction analyses and the assessment of the applicability and appropriate use of numerical analyses, including the recent evaluation prepared by the NEES Grand Challenge project. This task should be undertaken by geotechnical and structural engineers and researchers experienced in numerical modeling of embankments.

Task 3.2: Develop Analysis Procedures

This task involves determining appropriate and practical numerical analyses methods and defining guidelines for the selection of appropriate soil parameters for analyses. Of utility will be the results from the review of guidelines on kinematic interaction analyses conducted under Task 3.1 It is also recommended that this task involve the development of problem statements that numerical modeling procedures should solve, followed by testing of proposed procedures to determine their adequacy in solving the problem statements.

This task should be undertaken by geotechnical and structural engineers and researchers experienced in numerical modeling of embankments.

Task 3.3: Conduct Parametric Studies

This task involves the conduct of parametric studies to (1) determine inertial and kinematic loading coupling and uncoupling criteria; and (2) calibrate analysis methods from available case histories of embankment seismic deformations. Following these studies, the analysis procedures developed under Task 3.2 should be updated to reflect the parametric study findings.

This task should be undertaken by geotechnical and structural engineers and researchers experienced in numerical modeling of embankments.

Task 3.4: Conduct Peer Review

As in the case of Tasks 1.4 and 2.6 above, this task involves the identification and engagement of a broadly based panel of subject matter experts, who will have responsibility for reviewing and evaluating the guidance developed under Tasks 3.1 through 3.3, and Task 3.5. In this case, the Peer Review Panel should include geotechnical and structural engineers and researchers experienced in numerical modeling of embankments.

Task 3.5: Develop Written Guidelines

This task involves the development of written guidelines for the numerical modeling of container wharves under kinematic embankment loading. The guidelines will be based on the procedures developed in the previous tasks. The developmental process should incorporate substantial and continuing peer review throughout its duration.

6.4 Recommended Schedule

Table 6-1 is a summary of the estimated durations of each of the individual tasks described above. The total overall time frames include consideration of those tasks that can be done simultaneously with other tasks, as noted in the Comments section of the table.

6.5 Estimated Budget

The estimated budgets to prepare the three guidelines documents are provided in Table 6-2. Budget estimates for developing each document are based on the estimated duration and level of effort for each task for each document. The estimated level of effort is based on comparable work undertaken on projects with similar scope, duration, and level of effort; the cost estimates are also based on the collective experience of the project team. Dollar values have been assigned based on a weighted average of the several skill levels of the needed personnel. Costs include an allowance for oversight, expenses, and overhead charges.

Table 6-1 Recommended Guideline Development Schedule

Task	Duration (months)	Comments
1.1	6	
1.2	12	Starts after Task 1.1 completion
1.3	6	Starts after Task 1.2 completion
1.4	6	Overlaps with all tasks
1.5	6	Starts after completion of Task 1.3
Total	30	
2.1	6	
2.2	6	Starts after 25% of Task 2.1 is complete
2.3	12	Starts after 25% of Task 2.2 is complete
2.4	12	Starts after 75% of Task 2.3 is complete
2.5	6	Starts with Task 2.4
2.6	6	Overlaps with all tasks
2.7	6	Starts after completion of Task 2.6
Total	37	
3.1	3	
3.2	12	Starts after 25% of Task 3.1 is complete
3.3	6	Starts after completion of Task 3.2
3.4	3	Overlaps with all tasks
3.5	6	Starts after completion of Task 3.4
Total	25	

Table 6-2 Estimated Guidance Development Budget

Guideline No.	Duration (years)	Direct Technical Development	Direct Mgmt. & Oversight	Direct Expenses	Overhead	Total
1	2.5	$810,000	$162,000	$226,800	$421,200	$1,620,000
2	3.1	$2,220,000	$444,000	$621,600	$1,154,400	$4,440,000
3	2.1	$561,000	$112,200	$157,080	$291,720	$1,122,000
Total	7.7	$3,591,000	$718,200	$1,005,480	$1,867,320	$7,182,000

6.6 Key Collaborators

Key collaborators in this work include the following:

- Structural and geotechnical engineering professionals experienced in the seismic design and seismic performance of port facilities and in ground improvement techniques

- Researchers experienced in the investigation of the seismic performance of port facilities, including embankments and marginal wharves, and in the use and development of procedures for numerical modeling of container wharves under kinematic embankment loading
- American Society of Civil Engineers (ASCE)
- Technical Council on Lifeline Earthquake Engineering
- American Association of Port Authorities – Facilities Engineering Committee
- World Association for Waterborne Transport Infrastructure
- Terminal Operators

Some of the potential collaborators, such as ASCE, might have ongoing activities in one of the task areas that should be considered in the proposed work, or results from the proposed work might be considered for a design standard or guidance published by ASCE (or others).

The involvement and active participation of terminal operators are crucial to making sure that operational aspects of container cargo systems are thoroughly considered during the development of performance criteria for those systems. Involvement of operator/owner representatives will also be important to understanding business interruption caused by earthquakes and the way retrofit activities might be proposed to minimize downtime by maximizing performance improvements.

Appendix A
Container Wharves: Embankment Stability

This appendix provides additional information on embankment deformation (Section A.1), pile design and soil pile interaction (Section A.2), and deformation mitigation for ground improvement (Section A.3). The information presented here supplements the literature review findings described in Section 3.3.1.

A.1 Embankment Deformation: Analysis Approaches

Analysis Methods. Earthquake-induced ground accelerations can result in significant inertial forces in slopes or embankments, and these forces may lead to instability or permanent deformation. Current practice for the analysis of the performance of slopes and embankments during earthquake loading is to use one of two related methods:

1. A *Limit Equilibrium Method* using a pseudo-static representation of the seismic forces. In this approach, induced seismic loads are modeled as a static horizontal force in conventional limit equilibrium stability analysis to determine the factor of safety for comparison to an acceptable value. The seismic load is defined by a horizontal seismic coefficient, *k*, that is determined on the basis of the peak ground acceleration, the geometry of the mass of soil being loaded, and a performance criterion (i.e., the allowable displacement).

2. A *Displacement-Based Analysis Method* using either the Newmark sliding block concept (Newmark, 1965) or more rigorous numerical modeling methods. In this approach, which is schematically illustrated in Figure A-1 (and discussed in Section 3.3.1), deformations accumulate, leading to permanent ground displacement, when the average acceleration of a potential failure mass exceeds its yield acceleration (that is, when the acceleration exceeds the acceleration at which the Factor of Safety is equal to 1.0).

Even though the seismic coefficient may be less than the peak average acceleration of the failure mass, the cumulative deformation that occurs over the entire earthquake could be small, provided yield acceleration of the failure mass is not exceeded repeatedly. This concept not only provides a basis for selection of an appropriate seismic coefficient but also forms the basis of the displacement based analysis approach discussed further below.

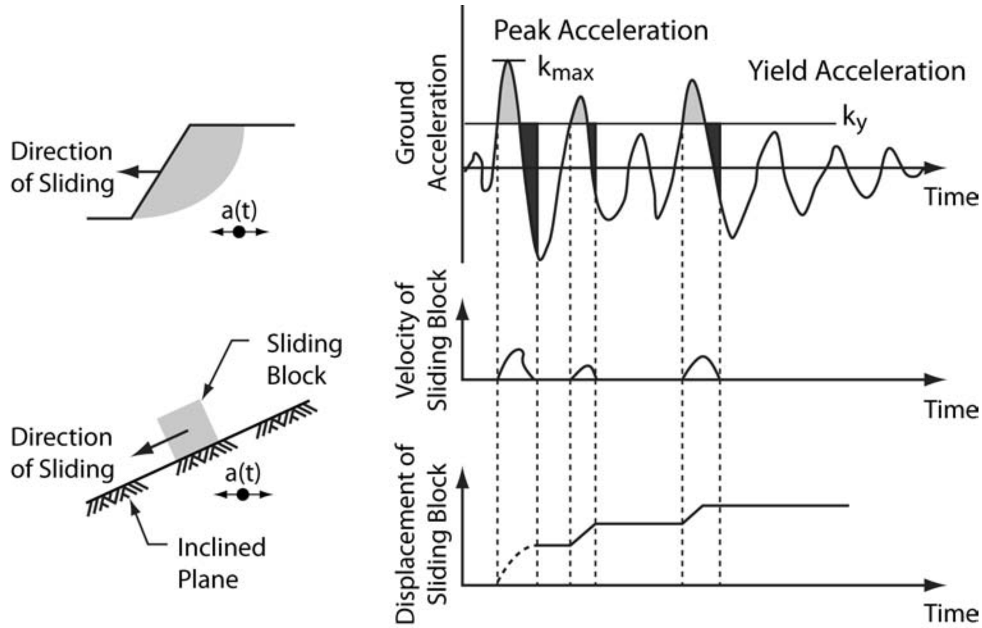


Figure A-1 Schematic illustrating Newmark (1965) sliding block concept for slopes (from Anderson et al., 2008).

Based on studies that have shown that vertical accelerations have a minor effect on a limit equilibrium seismic stability evaluation, the vertical acceleration is normally set equal to zero. The seismic coefficient used in the analysis is based on the peak average acceleration of the failure mass which, in turn, is based upon the site-adjusted peak ground accelerations (PGA) potentially reduced for spatial incoherence effects, as discussed below.

Most simplified methods of seismic analysis assume that the soil within the slope behaves as a rigid mass with all ground motions moving in phase. Although this assumption makes sense for small slope masses, it is not a likely case for higher slope masses where wave motion incoherence can reduce average lateral accelerations.

Figure A-2 shows the form of this reduction based on National Cooperative Highway Research Program (NCHRP) Project 12-70, where finite element numerical studies were conducted to establish revised Newmark methods of analysis for the American Association of State Highway and Transportation Officials (AASHTO) (Anderson et al., 2008). In this figure, the parameter (β) reflects earthquake frequency characteristics and is defined as $F_V S_1$/PGA, where $F_V S_1$ is the U.S. Geological Survey Site Class factor for S_1 (the design spectral ordinate for a 1-second period). For design purposes, curves shown in Figure A-2 can be defined by the following equations.

$$k_{av} = \alpha\, k_{max} \tag{A-1}$$

where k_{max} = PGA from the AASHTO maps (modified for Site Class) and α = height dependent reduction factor. For AASHTO Site Classes C, D, and E foundations soils, the α value is given by

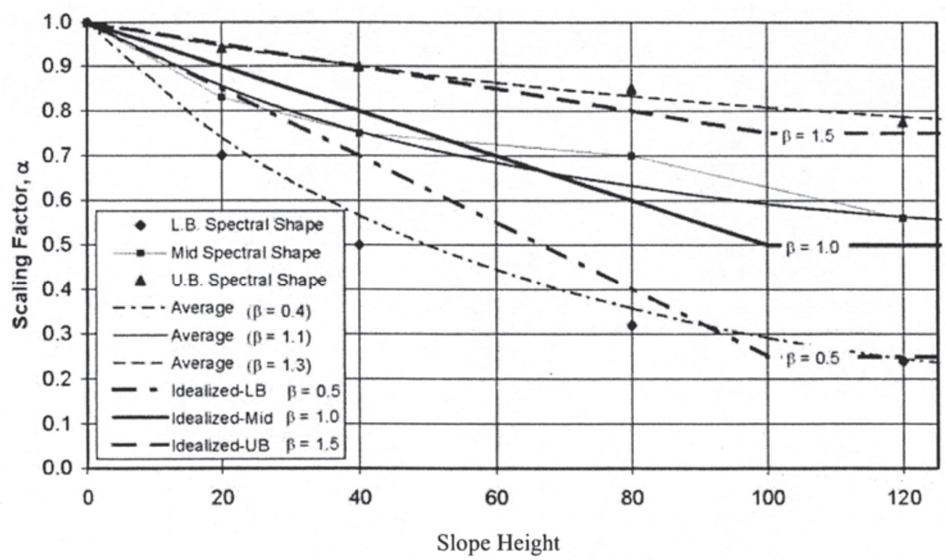


Figure A-2 Simplified height-dependent scaling factor recommended for design. L.B. = lower bound; U.B. = upper bound. (Anderson et al., 2008).

$$\alpha = 1 + 0.01H\,[(1.5\,\beta) - 1] \qquad \text{(A-2)}$$

where H = fill height. For Site Class A and B foundation conditions (i.e., hard rock and soft rock), the α value is increased by 20 percent. Slopes with a height of 20 feet or less have an α value of approximately 1.0; special studies should be conducted for fill heights greater than 100 feet. Most current practice for port embankments assume α = 1.

A database of over 1,800 earthquake records was evaluated as part of the NCHRP 12-70 Project. Two equations were derived from this evaluation, one for soil sites in the central and eastern United States (CEUS) and Western United States (WUS) and another for rock sites in the CEUS. The equations relate displacement (d), the ratio of yield acceleration (k_y) to peak acceleration (k_{max}), and the peak ground velocity (PGV). The evaluation also showed that PGV could be reasonably estimated on the basis of the spectral acceleration at 1-second, adjusted by the site factor using the following equation:

$$\text{PGV} = 55\,F_V S_1. \qquad \text{(A-3)}$$

Liquefaction. Representative embankment profiles at the Port of Los Angeles are shown in Figure A-3. The presence of potentially liquefiable sands in such profiles, which are found at many port sites, has a significant influence in evaluation of potential earthquake-induced embankment deformations or lateral spreads.

The design for liquefaction evaluations at the Port of Long Beach and Port of Los Angeles, as adopted for the proposed ASCE (2012) design standards for wharves, are described by Arulmoli et al. (2008), together with lateral embankment deformation methods of analysis.

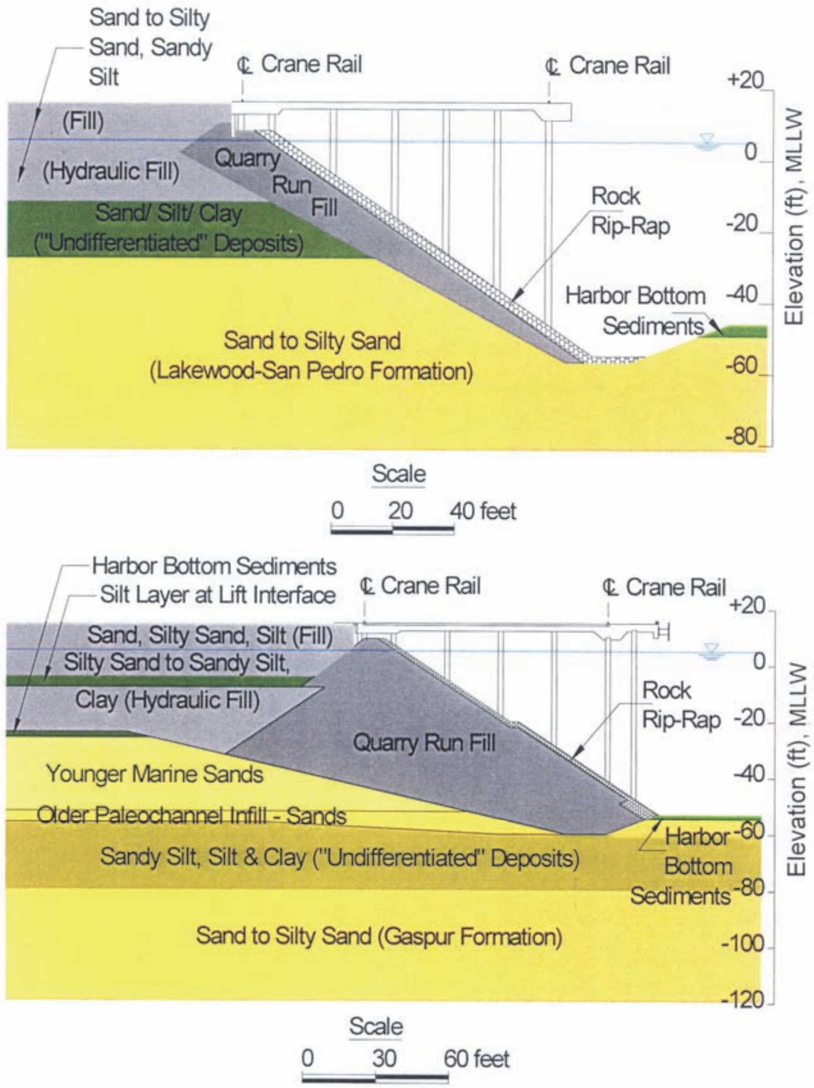


Figure A-3 Representative embankment profiles, Port of Los Angeles (Arulmoli and Martin, 2005).

The cyclic resistance of fine-grained soils are evaluated using the current method of analysis (Andrews and Martin, 2000; Seed et al., 2003; Bray and Sancio, 2006; Boulanger and Idriss, 2006).

If liquefaction is shown to be initiated in the above evaluations, specific liquefiable strata and their thickness (including zones of liquefaction induced in the backland area) are clearly shown on site profiles. Potential effects of hazards associated with liquefaction evaluation include the following:

- flow slides or large translational or rotational failures mobilized by the static driving stresses associated with the dike/embankment system
- limited lateral spreading of the dike/embankment system of a few feet or less triggered and sustained by the earthquake ground shaking

- post-liquefaction settlement of the dike/embankment system and underlying foundation soils

Slope Stability and Lateral Ground Deformation Analyses. The "free field" lateral deformations of the slope or embankment and associated foundation soils are first evaluated without considering the presence of the foundation system. Initial estimates of "free field" seismic dike deformations are determined using the simplified Newmark sliding block method. Strength values compatible with the post-earthquake residual strength of liquefied soils (Seed and Harder, 1990; Olson and Stark, 2002; Seed et al., 2003) and sensitive clays (Skempton, 1964; Duncan and Wright, 2005; Hsai-Fang and Daniels, 2006) should be used in the evaluation. If the "free field" lateral deformations do not compromise the structural performance of the pile foundations, further kinematic analysis, as outlined, is not required.

A.2 Pile Design and Soil-Pile Interaction

Inertial Loading: For typical container wharves at the Port of Los Angeles and Port of Long Beach, upper-bound and lower-bound *p*-multiplication factors of 2 and 0.3 are used for the quarry rock dike slopes (Port of Long Beach, 2009). These factors are based on the evaluation of full-scale pile load tests at the Port of Los Angeles (Diaz et al., 1984; Harding-Lawson Associates, 1983 and 1991), performed as a part of the Port of Los Angeles Pier 400 container wharf design project (EMI, 1998a). The recommended best-estimate reaction-displacement (*p*-*y*) curves for the quarry rock are shown in Figure A-4 (EMI, 1998a).

University of California, San Diego Full-Scale Lateral Pile Load Test Program. Recently, the Port of Los Angeles, with the support of the Port of Long Beach, commissioned a series of full-scale lateral pile load tests at the University of California, San Diego (UCSD) Englekirk Center. These pile load tests are discussed in detail in a UCSD report (Kawamata et al., 2008). The primary goal of this UCSD experimental program was to evaluate structural aspects of the current Port of Los Angeles and Port of Long Beach pile designs and pile-to-deck connections and improve design procedures and details. A secondary goal was to check if the *p*-*y* curves used for quarry rock in pile evaluations are appropriate and improve them if necessary.

The total pile-head shear as a function of pile-head deflection for a system test is shown in Figure A-5. The system consists of two 24-inch octagonal precast, prestressed concrete piles, which are standard container wharf piles used at the Port of Los Angeles and Port of Long Beach, embedded in a quarry rock with a sloping ground surface. Both push (in the downslope direction) and pull (in the upslope direction) test results are shown in Figure A-5.

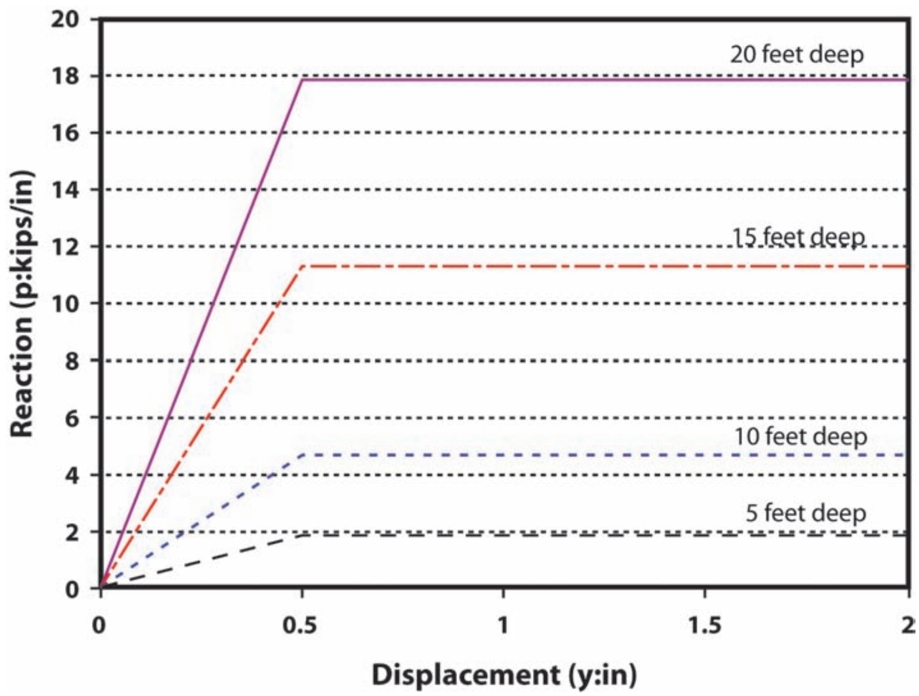


Figure A-4 Recommended best estimate *p-y* curve for quarry run rock dikes (EMI, 1998a).

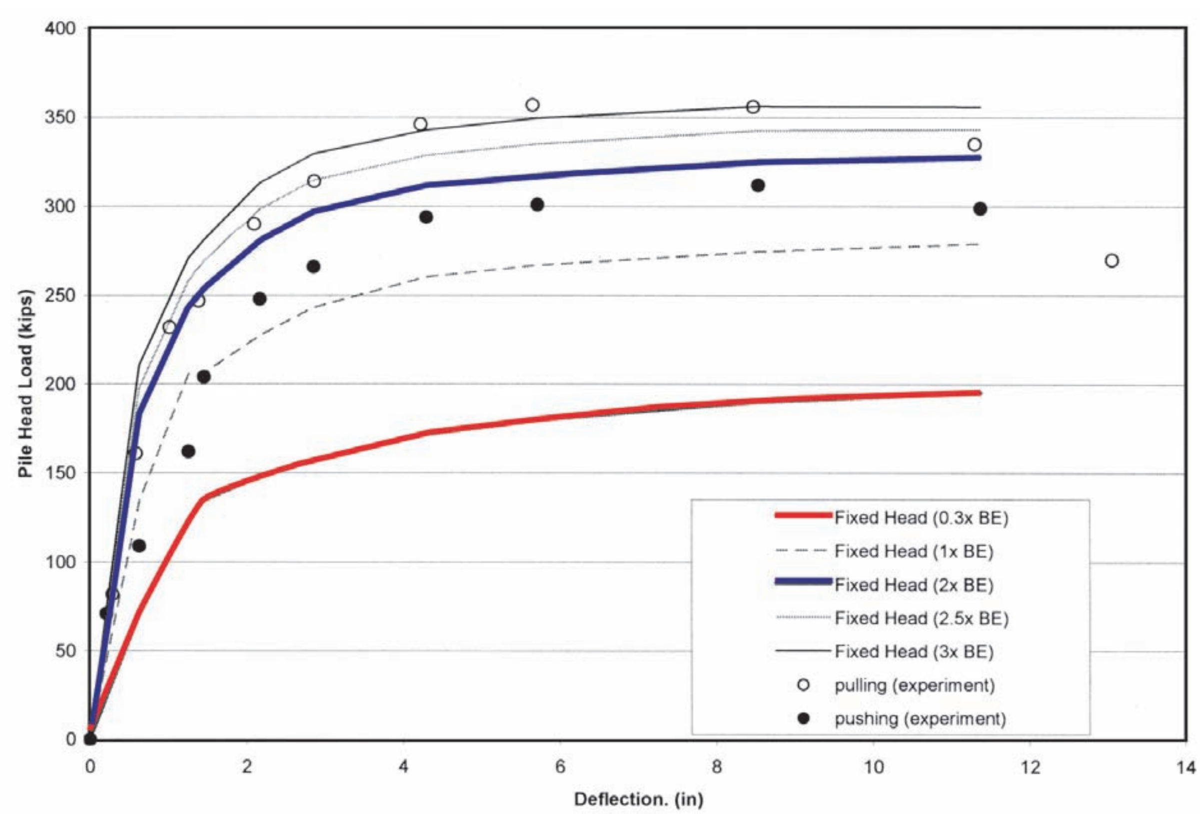


Figure A-5 Pile head load-deflection curves from experiment and analysis (Kawamata et al., 2008).

Independent lateral loading analyses using *p*-*y* curves were conducted by EMI (Arulmoli et al., 2009) using the computer program LPILE 5 (Reese et al., 2004), which includes the capability of nonlinear moment-curvature behavior. Results are also plotted in Figure A-5. The LPILE results, when compared with the UCSD test results, indicate that *p*-multipliers between 2 to 2.5 give good correlation for up to about a 2-to-3 inch pile-head deflection for the pull (upslope) test, while a factor of 3 provides a better correlation for larger displacements corresponding to pile yield. For the push (downslope) test, factors between approximately 1 and 2 provide the best correlation with the test data, especially for larger displacements associated with softer downslope behavior.

As mentioned earlier, the rock dike *p*-*y* curves recommended for the Port of Los Angeles and Port of Long Beach container wharf design use lower-bound and upper-bound *p*-multipliers of 0.3 and 2 for best-estimate design curves shown in Figure A-6. The UCSD test data suggest that although the design lower-bound *p*-multiplier is conservative for design, the upper-bound design *p*-multiplier could be unconservative. However, there are a number of differences between the UCSD test condition and actual field conditions at the Port of Los Angeles and Port of Long Beach, where these wharves are constructed. Some of these differences include the following:

- UCSD tests were performed on dry rock, while in actual field situations much of the rock dike is always submerged. Use of dry materials in the UCSD tests would lead to greater effective confining pressures, which in turn would lead to stiffer pile behavior.
- UCSD tests were performed on a single row of two piles where typical container wharf piles are spaced closer in both perpendicular and parallel directions to the wharf. Pile group effects inherent in port container wharf configurations also contribute to reduced effective *p*-*y* curves in actual field cases.

In addition, the difference in maximum rock sizes used in the experiment and rock placement method could also have led to the stiffer pile behavior in the experiment.

The lower-bound and upper-bound *p*-multipliers of 0.3 and 2 used in the design of the Port of Los Angeles and Port of Long Beach container wharves with sloping rock dikes are conservative, and no changes were recommended to these values. While these *p*-multiplier values are specific to the Port of Los Angeles and Port of Long Beach container wharves, similar values can be developed for other regions based on local conditions and construction practices.

Kinematic Loading. As indicated in Section 3.3.1, a step-by-step evaluation process can be used for the evaluation of kinematic loading. The step-by-step process

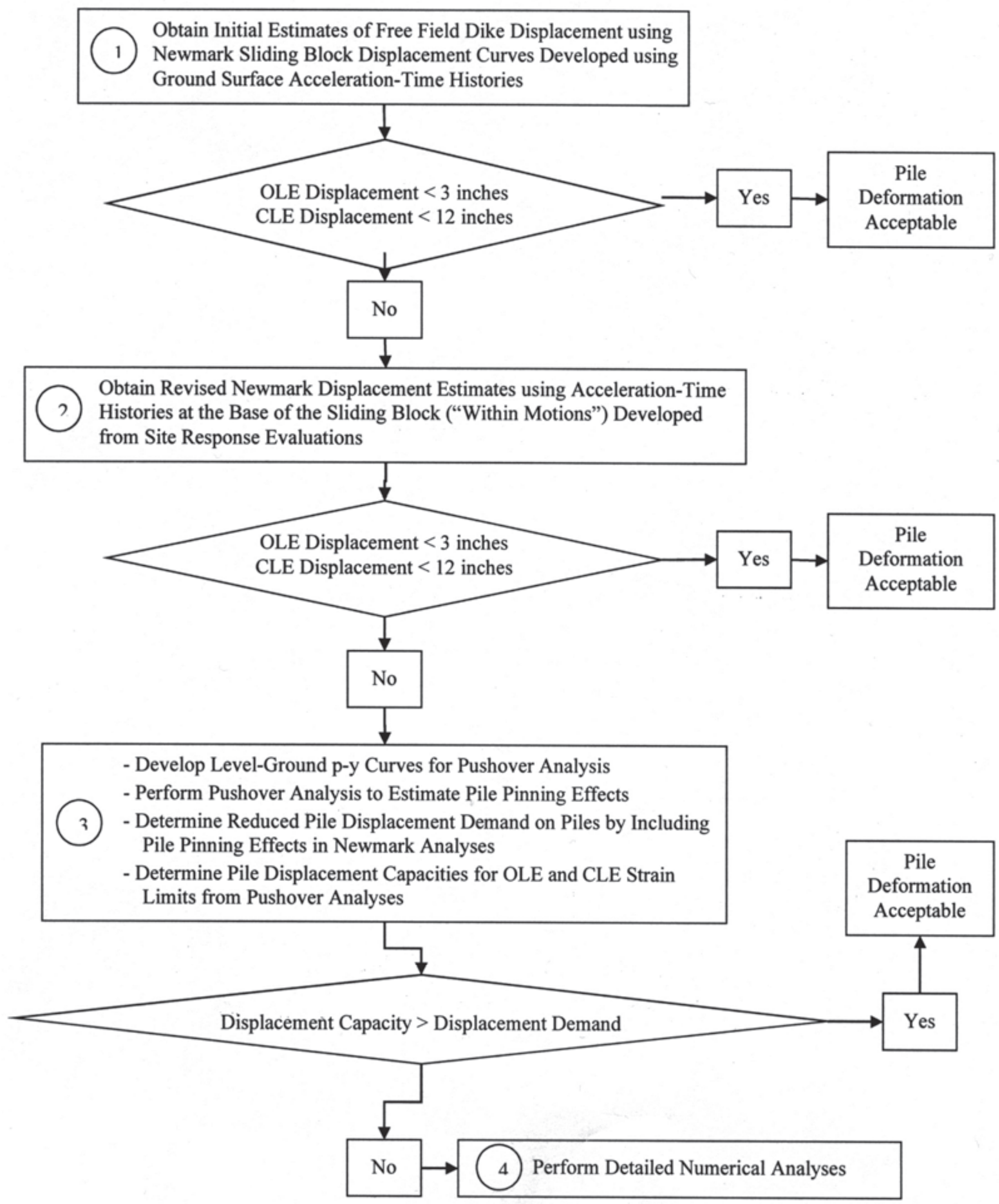


Figure A-6 Flow diagram for evaluation of kinematic lateral spread loading for Operational Level Earthquakes (OLE) and Contingency Level Earthquakes (CLE) at the Ports of Long Beach and Los Angeles (Arulmoli and Martin, 2009).

adopted by the Port of Los Angeles and the Port of Long Beach (Port of Long Beach, 2009), as presented in Figure A-6, is described below:

1. Initial estimates of free-field dike deformations (in the absence of piles) are determined using the simplified Newmark sliding block method. These estimates may be made using available charts (e.g., Anderson et al., 2008) or using site-specific acceleration time histories. For the 24-inch octagonal, precast, prestressed concrete piles that are typically used for Port of Los Angeles and Port

of Long Beach container wharf projects, studies have shown that deformations are generally acceptable (in terms of pile strain limits and performance criteria) when the permanent free-field dike deformations are less than about 3 inches for the Operating Level Earthquake (OLE) condition and less than about 12 inches for the Contingency Level Earthquake (CLE) condition (Port of Los Angeles, 2004; Port of Long Beach, 2009).

2. In cases where dike deformations estimated using the simplified Newmark sliding block method exceed the above displacement limits, site-response evaluations may be necessary to revise the free-field dike deformation analyses. One-dimensional site response analyses may be performed to incorporate local site effects in developing site-specific acceleration-time histories at the base of the sliding block ("within motions") for Newmark analyses. If the revised dike deformations are below the above acceptable values, more detailed numerical evaluations are not necessary.

3. In cases where dike deformations estimated from Step 2 exceed the displacement limits, additional analyses may be performed to estimate the reduced simplified Newmark displacements by incorporating pile "pinning" effects (MCEER/ATC, 2003; FHWA, 2006). Pseudo-static slope stability analysis with the "pinning" effects of piles incorporated are used to estimate the displacement demands using the simplified Newmark analysis. If the estimated displacement demands are less than the pile displacement capacities, no further analysis for kinematic loading will be necessary.

4. In cases where dike deformations estimated from Step 2 exceed the pile displacement limits, more detailed numerical analyses may be necessary to provide input to structural engineers. For these cases, two-dimensional dynamic soil-structure interaction analysis of the wharf-pile-dike-soil system using numerical finite element or finite difference analyses may be necessary. Sensitivity analyses should also be performed on factors affecting the results. As a minimum, deformation profiles along the length of various pile locations should be provided to the structural engineer to estimate strains and stresses in the piles for the purpose of checking pile performance criteria.

Although the specific values recommended in this approach are specific to the Port of Los Angeles and Port of Long Beach container wharf configurations, a similar approach can be followed for pile-supported wharves in other regions, and specific recommendations can be developed based on local conditions and construction practice. The procedure could also be used to check pile performance under the Design Level Earthquake (see Chapter 3).

Combining Inertial and Kinematic Load Conditions. As previously noted, for typical piers and wharves, the inertial loading condition tends to induce maximum strains (hence maximum moments) in the upper part of the piles while kinematic loading tends to induce maximum strains (and maximum moments) in deeper regions of the piles. In addition, the two loading conditions for certain cases tend to induce maximum moments at somewhat different times during the earthquake shaking (for example, typical container wharf slopes at the Port of Los Angeles and Port of Long Beach (EMI, 1998b)). For these situations, inertial and kinematic loading conditions can be uncoupled (separated) from each other during design. This assumption, however, should be checked on a project-specific basis. Additional evaluations, such as combining the results of separate pile push-over analyses for inertial and kinematic loading conditions, using simplified procedures (e.g., Boulanger et al., 2007), or using a coupled analysis, may be needed if these two loading conditions cannot be separated. Close coordination between the geotechnical engineer and the structural engineer is recommended during these evaluations.

Batter Piles. The performance of batter piles with conventional pile-to-deck connections has been poor during past earthquakes (Ferrito et al., 1999; Werner, 1998). They result in stiffer behavior than vertical piles during earthquakes, thus attracting higher forces including high earthquake-induced axial forces. High axial forces in piles induce large shear forces in the deck. Pile pull out could also be anticipated, and piles are also susceptible to damage due to kinematic loads. For these reasons, batter piles with conventional pile-to-deck connections should not be used where seismic actions control the design. Properly designed and detailed special connections (such as a "seismic fuse" or other details) are needed in these circumstances. Vertical piles are considered a preferred, simple, and cost-effective solution to batter piles for seismic loading conditions. Both the Port of Los Angeles seismic code and Port of Long Beach wharf design criteria prohibit the use of batter piles for new wharf design without prior approval. The proposed ASCE (2012) standard is also expected to adopt similar restrictions on the use of batter piles for seismic design.

A.3 Deformation Mitigation for Ground Improvement

A summary of several ground improvement methods for liquefaction remediation is provided in Table A-1, extracted from Cooke and Mitchell (1999). Brief summaries and examples of several of the remediation methods, which have found applications at port sites, are given below.

Densification. For new construction, particularly in open areas some distance from existing structures, insitu-densification using vibratory techniques or dynamic compaction are the most commonly deployed methods for improving the density and hence strength and stiffness of loose cohesionless soils. For saturated cohesionless soils, vibratory methods have been widely used to increase liquefaction resistance under earthquake loading. However, these methods would, in most cases, not be suitable for use in improving soils around existing piled foundations. Equipment

Table A-1 Ground Improvement Methods (after Cooke and Mitchell, 1999)

Method	Principle	Suitable Soil Types	Treated Soil Properties	Relative Costs
Compaction Grout	Highly viscous grout acts as spherical hydraulic jack when pumped under high pressure resulting in densification.	Compressible soils with some fines	Increased D_r SPT: $(N_1)_{60}$ 25 to 30 CPT: q_{cl}=80 To 150 tsf (Kg/cm^2)	Low material cost; high injection cost.
Particulate Grouting	Penetration grouting fills soil pores with cement, soil or clay.	Clean, medium to coarse sand and gravel	Cement- grouted soil: high strength	Lowest of grouting systems
Chemical Grouting	Solutions of two or more chemicals react in soil pores to form a gel or solid precipitate.	Sands and Silts	Low to high strength	High to very high
Jet Grouting	High-speed jets at depth excavate, inject and mix stabilizer with soil to form column or panels	Sands, silts, and clays	Solidified columns and walls	High
Vibro-Compaction	Densification by vibration and compaction of backfill at depth.	Sand (<20% passing No. 200 sieve)	D_r: up to 85+% SPT: $(N_1)_{60}$ 25 to 30 CPT: q_{cl}=80 To 150 tsf (Kg/cm^2)	Moderate
Vibro-replacement/ Stone Columns	Densely compacted gravel columns provide densification, reinforcement, and drainage	Sands and silts	Increased D_r SPT: $(N_1)_{60}$ 25 to 30 CPT: q_{cl}=80 To 150 tsf (Kg/cm^2)	Moderate to high
Drains: Gravel Sand Wick	Drains relieve excess pore water pressure to prevent liquefaction and intercept and dissipate excess pore water pressure plumes from adjacent liquefied soil.	Sands and silts	Improved drainage	Low to moderate
Mini-Piles	Piles carry loads through liquefiable soils to firm stratum.	All soils		High

CPT: cone penetration test; SPT: standard penetration test.

such as vibratory rollers or tampers could be used, but densification effects would occur to only relatively shallow depths surrounding footings. To be effective to deeper depths surrounding piles, deep dynamic compaction or vibro probes, as described below, would be necessary. Each method has its own technology, effectiveness and suitability for different soil types. Experience indicates that vibratory techniques can increase relative densities to values greater than 80 percent.

Vibro-Replacement Method. The most widely used densification method for port projects is the vibro-replacement technique, which has been shown to be effective in remediating potentially liquefiable soil in past earthquakes (Mitchell et al., 1995). This method involves the replacement of liquefiable soil with a column of dense granular soil formed by repeated insertion and withdrawal of a large vibrating probe into the soil to the desired depth of improvement. As the vibrating probe is plunged up and down, crushed stone backfill is placed around the vibrator, leading to the development of a dense stone column that is approximately 1 meter (3 feet) in diameter. The procedure is repeated at a grid spacing of 2.4 to 3.7 meters (8 to 12 feet) to form an improved zone.

Vibro-replacement includes wet methods, where the insertion of the probe is aided by jetting and dry methods where the probe is penetrated into the ground typically using a vibrating hammer. The wet methods rely more on replacement then displacement and are more suitable for ground improvement at nonliquefiable soft soil sites. Dry methods displace the soil, densifying the adjacent soil and providing liquefaction mitigation by densification as well as through reinforcement and enhanced drainage. Vibro-replacement is effective if the sands to be densified contain less than 15 to 20 percent fines.

Examples of stone column mitigation approaches for port embankments are described by Vazifdar and Kaldveer (1995), Egan et al. (1992), and Mageau and Chin (2009). Vazifdar and Kaldveer (1995) describe the reconstruction of a container wharf in Guam following the 1993 Guam earthquake, where liquefiable soils were stabilized using vibro-replacement stone columns on an excavated slope as shown in Figure A-7. Egan et al (1992) describe the use of vibro-replacement stone columns to retrofit the Seventh Street Wharf of the Port of Oakland following a liquefaction induced lateral spread caused by the 1989 Loma Prieta earthquake.

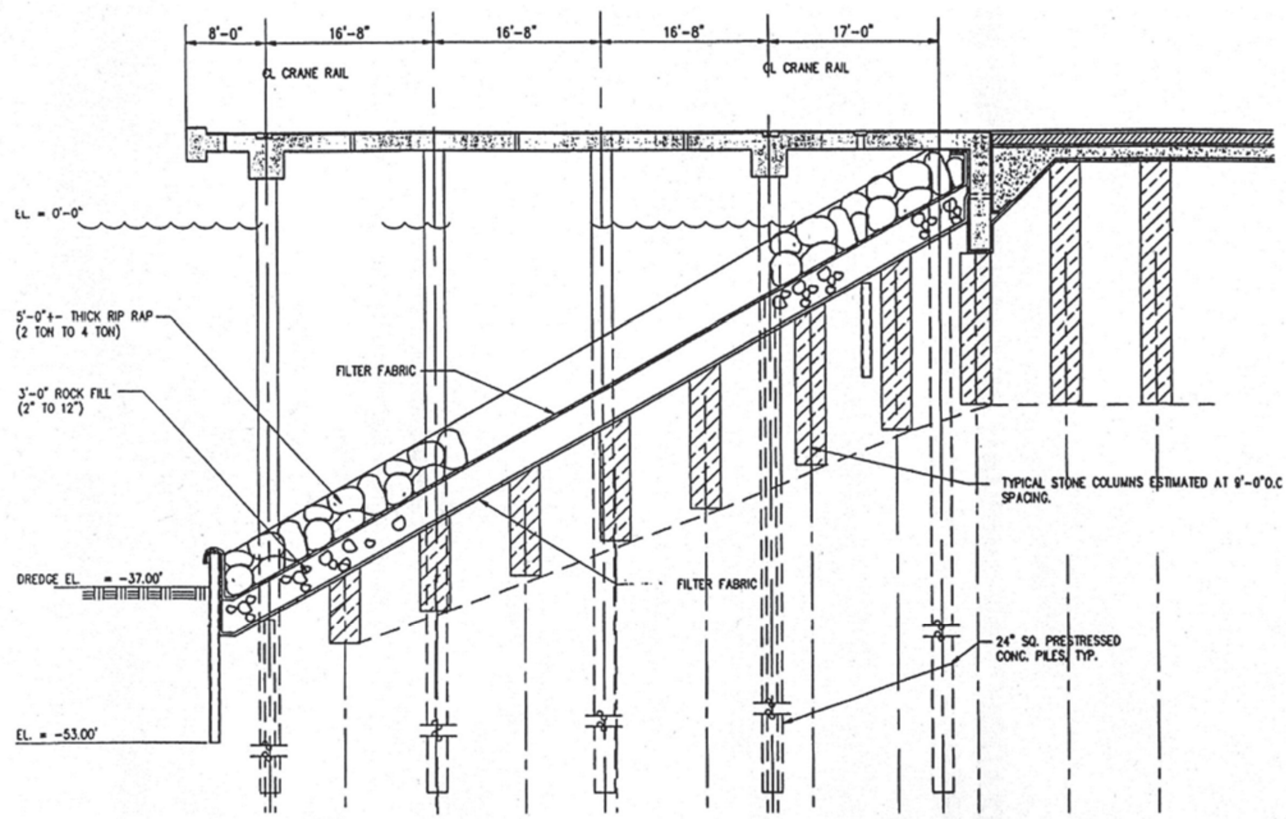


Figure A-7 Schematic profile showing Guam container wharf reconstruction using stone columns (Vazifdar and Kaldveer,1995).

Figure A-8 illustrates the retrofit concept, where the extent of ground improvement to limit deformations from a future earthquake (arising from liquefaction in the sand fill) was evaluated using the Newmark sliding block analysis method. Mageau and Chin (2009) describe the use of numerical methods to evaluate the effectiveness of stone column ground improvement to limit deformations of slopes containing liquefiable soil layers at a new pile-supported wharf at the Port of Tacoma.

Compaction Grouting. For sites where vibratory techniques may be impractical, because of silt contents greater than 20 percent, compaction grouting can be used. Compaction grouting involves pumping a stiff mix of soil, cement, and water into the

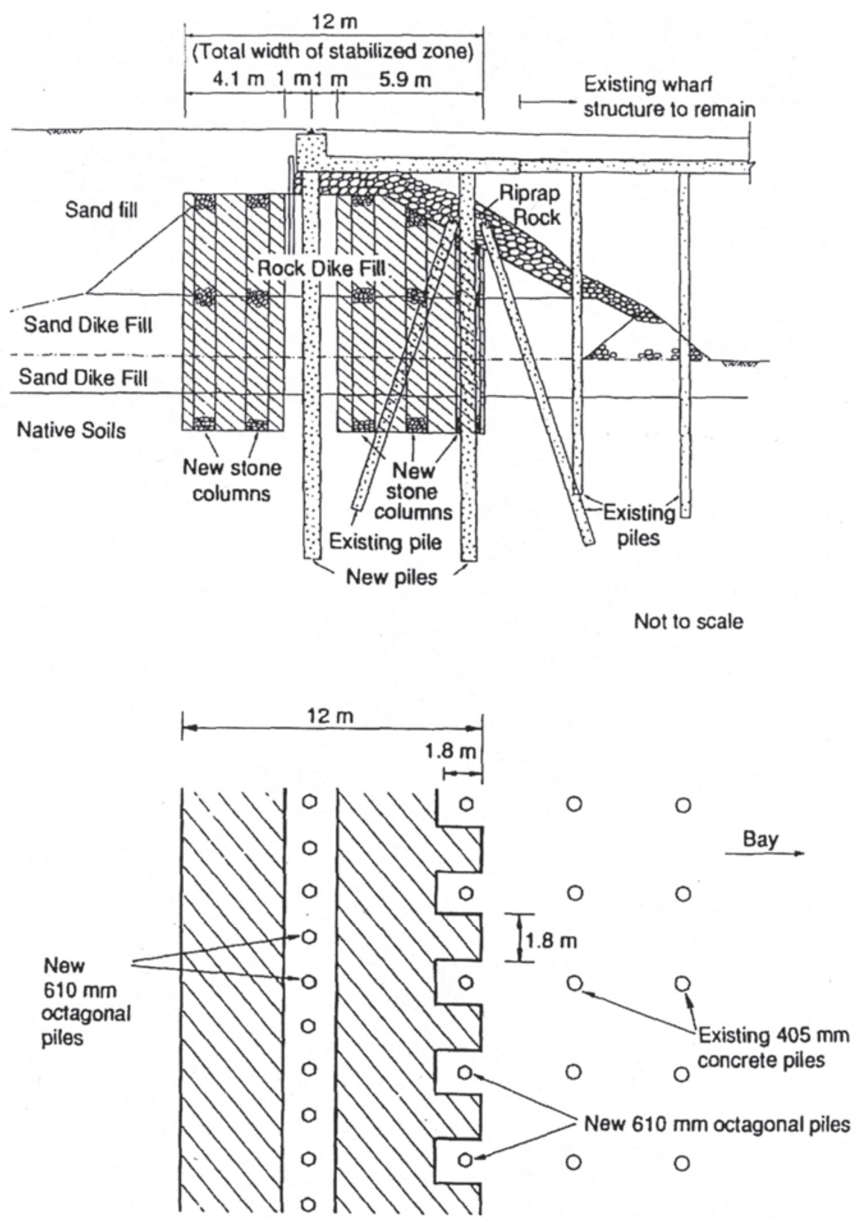
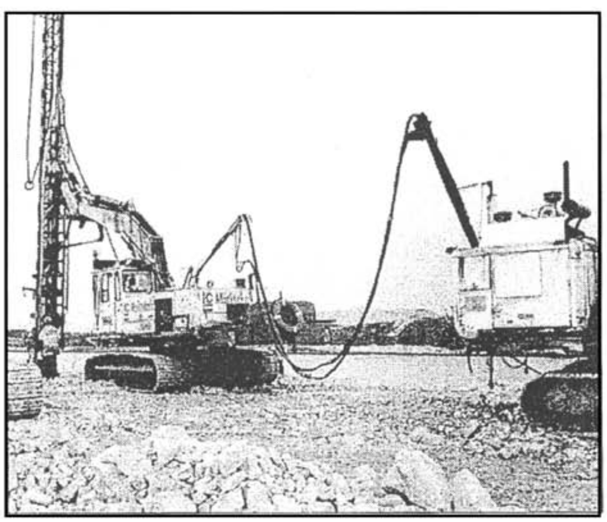


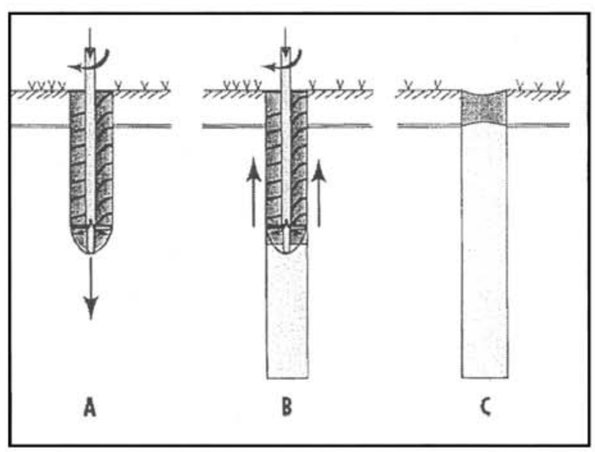
Figure A-8 Profile and plan drawings showing Oakland container wharf repair (Egan, 1992).

ground under high pressure to compress or densify the soil. A very stiff soil-cement and water mixture is injected into the soil forming a grout bulb, which displaces and potentially densifies the surrounding ground without penetrating the soil pores. A grid or network of grout columns is formed by grouting from the bottom up, resulting in improved liquefaction resistance over the required surface area, similar to a network of stone columns (Boulanger and Hayden 1995). A theoretical study of the mechanics of ground improvement in sands (Mace and Martin, 2000) has shown that increased liquefaction resistance arises primarily from increased lateral stresses rather than from densification. Consequently, increases in liquefaction strength are best measured using cone penetration test (CPT) correlations (Salgado et al., 1997).

Deep Soil Mixing. The deep soil mixing method encompasses a group of technologies where cementitious material (usually cement or lime) is introduced and blended into the soil through a hollow totaling shaft or shafts equipped with cutting tools and mixing paddles or augers. The materials may be injected under pressure in either slurry (wet) or dry form. Figure A-9 shows a typical rig used for the dry mixing method with a schematic diagram of the mixing process. The process leads to vertical stabilized columns of about 1 meter diameter. Multiple augers are often used in the wet methods. For dry methods (used beneath the water table or in high moisture content clays), typically 100-300 kilograms of cementitious material is injected per cubic meter of soil, while for wet methods, 100-500 kilograms is injected. The strength gain of the soil depends on the physical properties of the soil and the quantity of cementitious material injected. Typically, values of unconfined compressive strengths of 5-50 tsf (tons per square foot) are achieved in treated granular soils and 2-20 tsf in cohesive soils.



Dry Soil Mixing rig with mobile powder silo.



(A) Mixing tool is inserted to prepare the soil. (B) Lime and cement are injected and mixed as tool is rotated and raised. (C) Depths of completed columns are predetermined and controlled by on-board computer.

Figure A-9 Illustrations depicting dry method column installation (from publicly available Keller Corporation brochure).

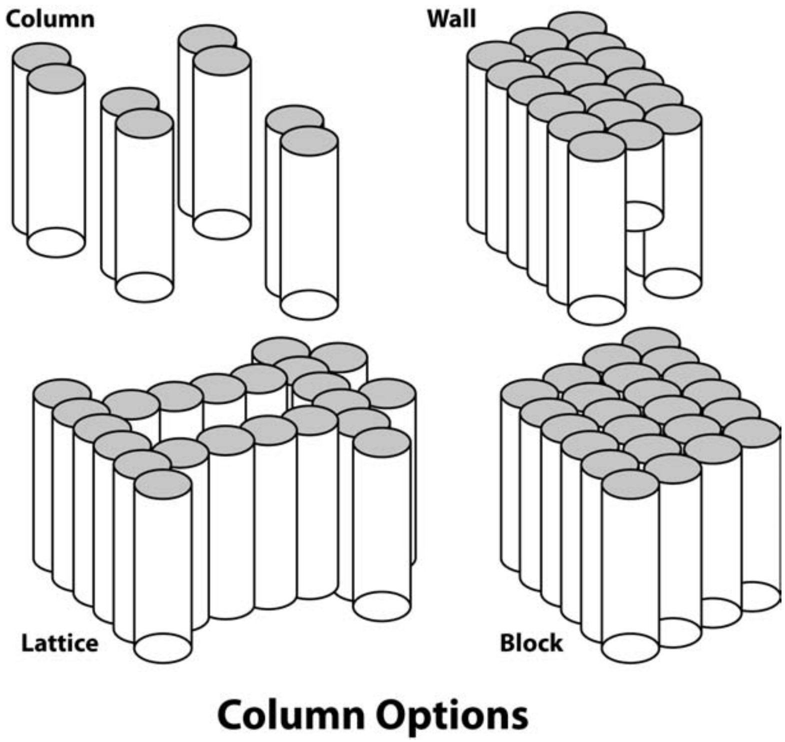


Figure A-10 Schematics of deep mixing column patterns (after Porbaha et al., 1999).

The versatility of the deep mixing construction technique allows columns to overlap to form blocks, walls, or lattice configurations, as shown in Figure A-10. The choice of pattern depends on the specific application. Structural walls are typically used for resisting lateral earth pressures in construction of deep excavations while solid blocks may be used to strengthen large volumes of weak soil. Lattice or cellular structures may be used to support lightly loaded structures or to control embankment stability. For the latter case as applied to port embankments, the lattice structure provides a buttress in a liquefiable soft soil zone.

Mitigation Related to Soft Cohesive Soils. Where earthquake-induced embankment deformations or lateral spreads are soft cohesive soil zones or layers, mitigation methods include deep soil mixing (as described above) to form strengthened or buttress zones or jet grouting approaches. A further innovative concept is the use of driver displacement piles installed at close spacing in soft zones, where consolidation of induced pore pressures leads to increased soil strength. The piles also provide additional "pinning" effects in soft layers. The concept is illustrated by Zmuda and Arulmoli (2004) where displacement piles were used to improve slope stability of Berth 100 at the Port of Los Angeles. Figure A-11 shows a wharf cross section where displacement piles (14-inch square prestressed concrete) were driven at 6-foot centers along the entire 1000-foot length of the wharf.

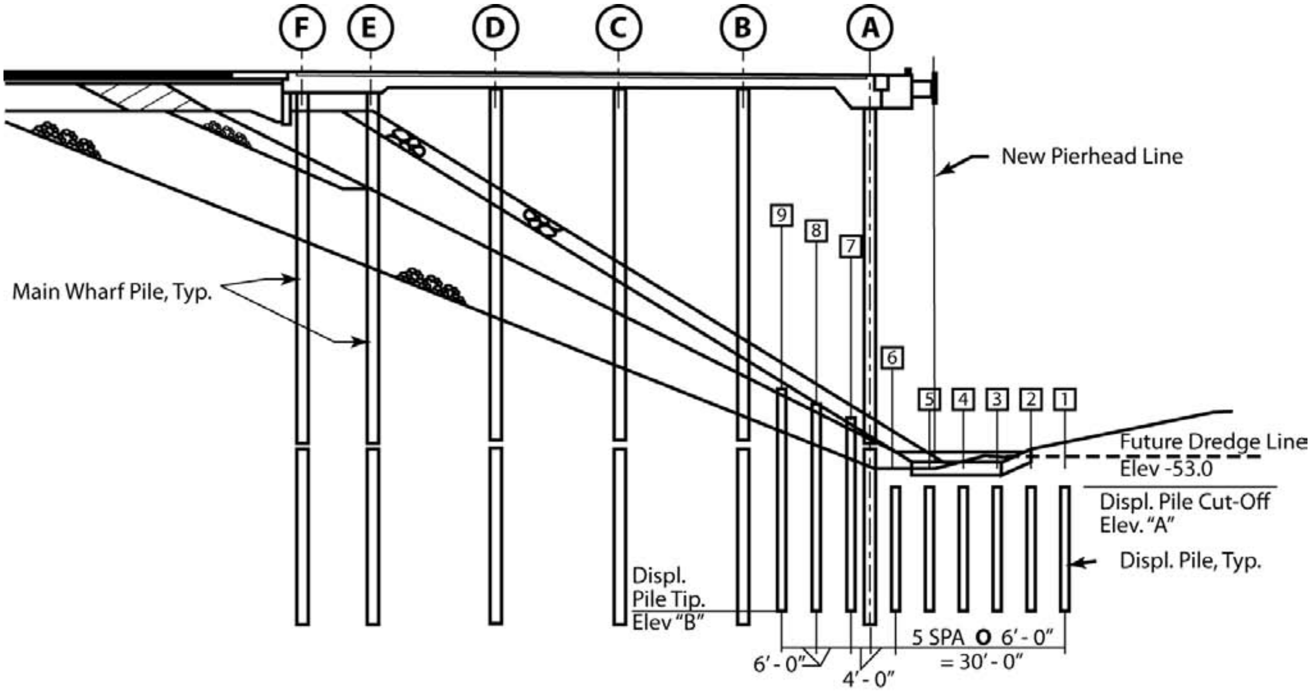


Figure A-11 Wharf cross section showing displacement pile buttress (after Zmuda and Arulmoli, 2004).

Appendix B

Bulkheads

This appendix provides additional information on gravity walls and caisson structures (Section B.1), and sheet pile walls with tieback anchors (Section B.2). The information presented here supplements the literature review findings described in Section 3.3.2.

B.1 Gravity Walls—Caisson Structures

Earthquake-induced displacement and tilt evaluation, particularly with liquefiable foundation or backfill soils, is the most difficult design issue for gravity walls, such as concrete caisson structures that may be floated into position and founded on the seafloor. This is illustrated by the damage to a caisson quay wall at Kobe Port during the 1995 Hanshin earthquake, as shown in Figure B-1.

As indicated in Section 3.3.2, conventional practice for evaluating seismic stability of gravity walls/caisson structures is based on pseudo-static approaches, similar to those described for embankments. Ebeling and Morrison (1992) and PIANC (2002) provide a comprehensive discussion of this approach. The forces acting for pseudo-static limit equilibrium design are shown in Figure B-2, where P_{ae} is the seismic active earth pressure and P_{dw} is the Westergaard suction force.

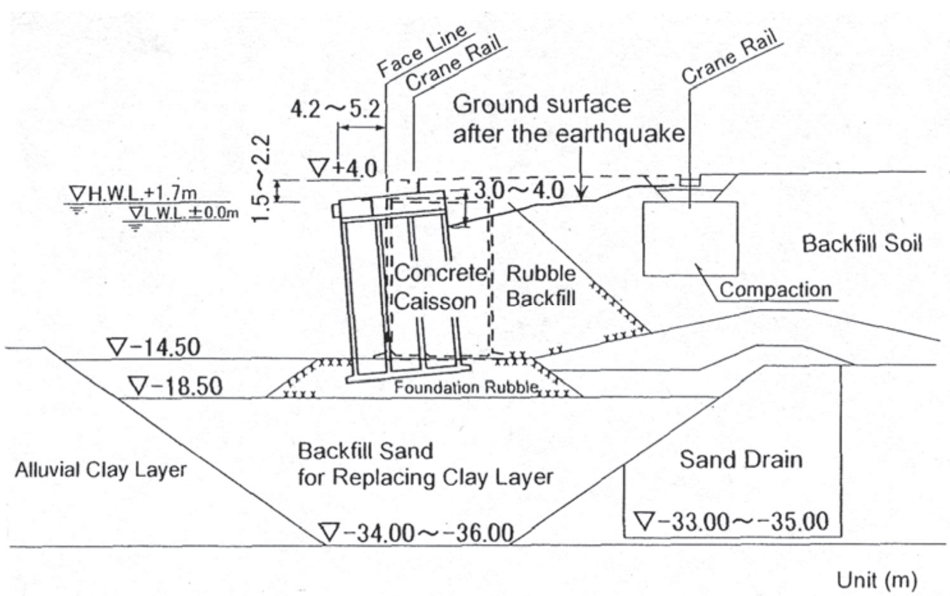


Figure B-1 Cross section of caisson wall at Kobe Port showing effects of 1995 Hanshin earthquake (PIANC, 2002).

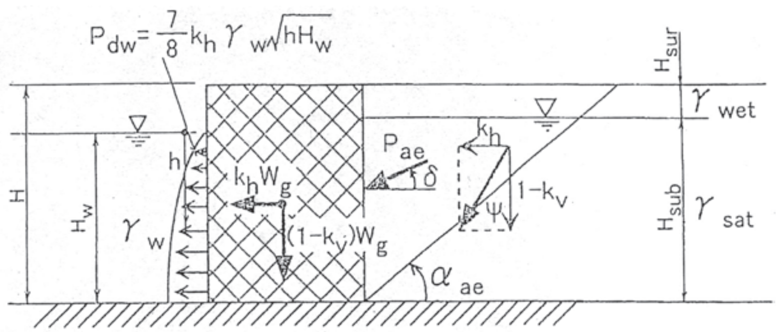


Figure B-2 Schematic showing forces in pseudo-static analyses (after PIANC 2002)

In determining lateral accelerations for analyses, peak ground accelerations are conventionally reduced to reflect ground motion incoherence in backfill soils as for the case of embankment slopes (Anderson et al., 2008), or to reflect allowable displacements of the caisson (sliding) evaluated using the Newmark sliding block concept as for embankment slopes (Anderson et al., 2008). PIANC (2002) describes reduced equivalent seismic coefficients for design based on back analyses of damaged and nondamaged quay wells (129) during 12 earthquakes. The studies completed for the NCHRP 12-70 Project (Anderson et al., 2008) for retaining walls and slopes, document displacement-based recommended reductions in peak ground acceleration (PGA) for design of walls and supersede the displacement-based design charts noted in PIANC (2002) and the equation given in Ebeling and Morrison (1992).

Effective stress finite element studies have been conducted by Iai et al. (1999) and Dickenson and Yang (1998) and are described in PIANC (2002). Analyses clearly indicate the necessity of ground improvement in the case of liquefiable soils to prevent displacement-related damage. The model approach described by Dickenson and Yang (1998) used the program FLAC and an effective stress based Mohr Coulomb constitutive model allowing pore pressure increases during dynamic response analyses. The model was successfully calibrated against the Kobe Port case history, as shown by the deformed mesh in Figure B-3.

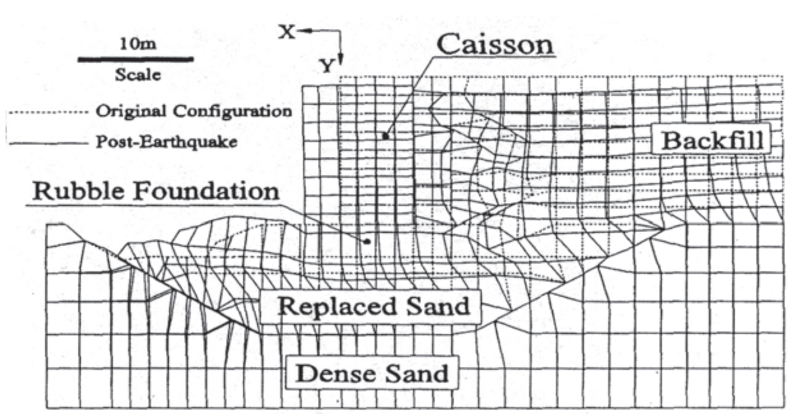


Figure B-3 Deformed mesh from finite element analysis of Kobe Port response to the 1995 Hanshin earthquake (Dickenson and Yang, 1998).

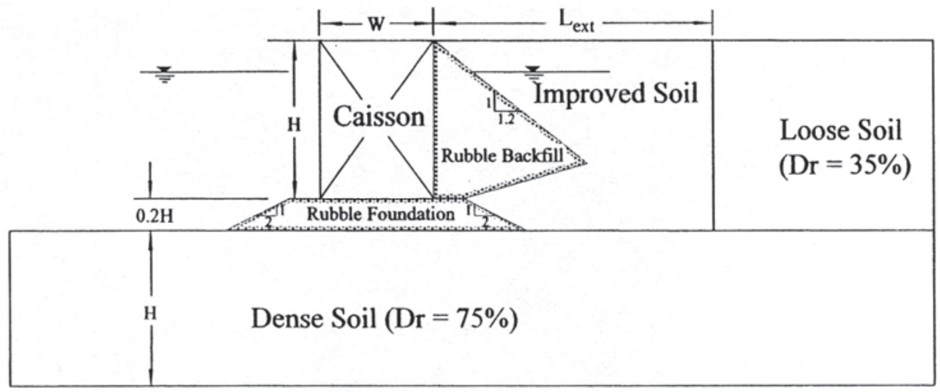


Figure B-4 Cross section used in analyses by Dickenson and Yang (1998) to illustrate the beneficial effects of ground improvement in reducing displacements.

Additional parametric studies showing the beneficial effects of ground improvement in reducing displacements were also performed using the concept shown in Figure B-4, leading to parametric design curves.

The results of the parametric study illustrated the influence of ground motion characteristics, geotechnical properties, and caisson geometry on wall deformations. In terms of a practical example of ground improvement, Koelling and Dickenson (1998) describe the use of vibro-compaction to improve liquefiable foundation and backfill soils for a caisson structure for the Port of Vancouver, British Columbia.

Further detailed examples of simplified and finite element analysis methods are provided in the Technical Commentary of PIANC (2002) and by Brodbaek and Laursen (2010). The latter analyses modeled both the caisson and soils using the program PLAXIS and assumed compacted backfill and dense nonliquefiable foundation soils. The model configuration is shown in Figure B-5.

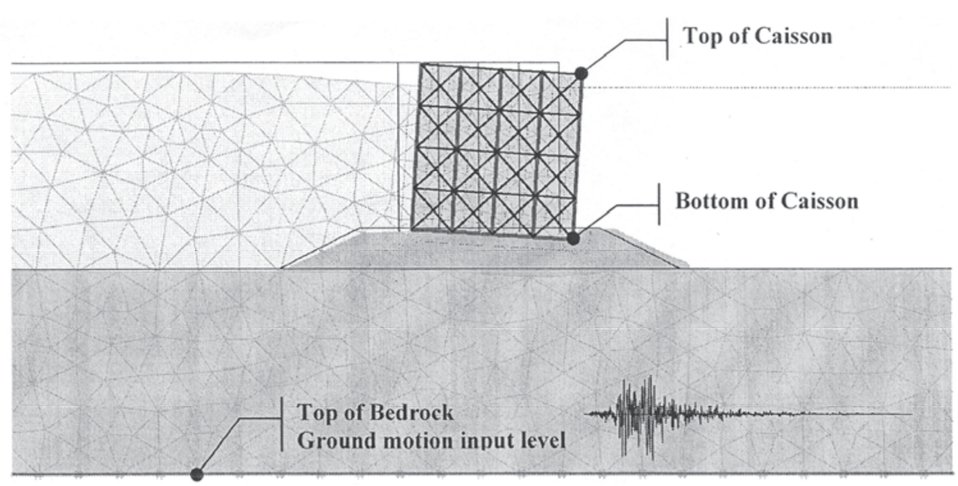


Figure B-5 Model configuration for analyses of caisson and soils by Brodbaek and Laursen (2010).

Yield accelerations computed from the dynamic finite element analyses were also used to compute horizontal displacements based on a Newmark sliding block approach and agreed reasonably well with the finite element analysis results.

B.2 Sheet Pile Walls with Tieback Anchors

Sheet pile walls with tieback anchors have been widely used for marginal wharf structures in the United States, but not for container wharf facilities. Because of liquefaction of backfill soils, past performance in earthquakes has been poor, as described in PIANC (2002). Representative failure modes and related damage criteria are illustrated in Figure B-6.

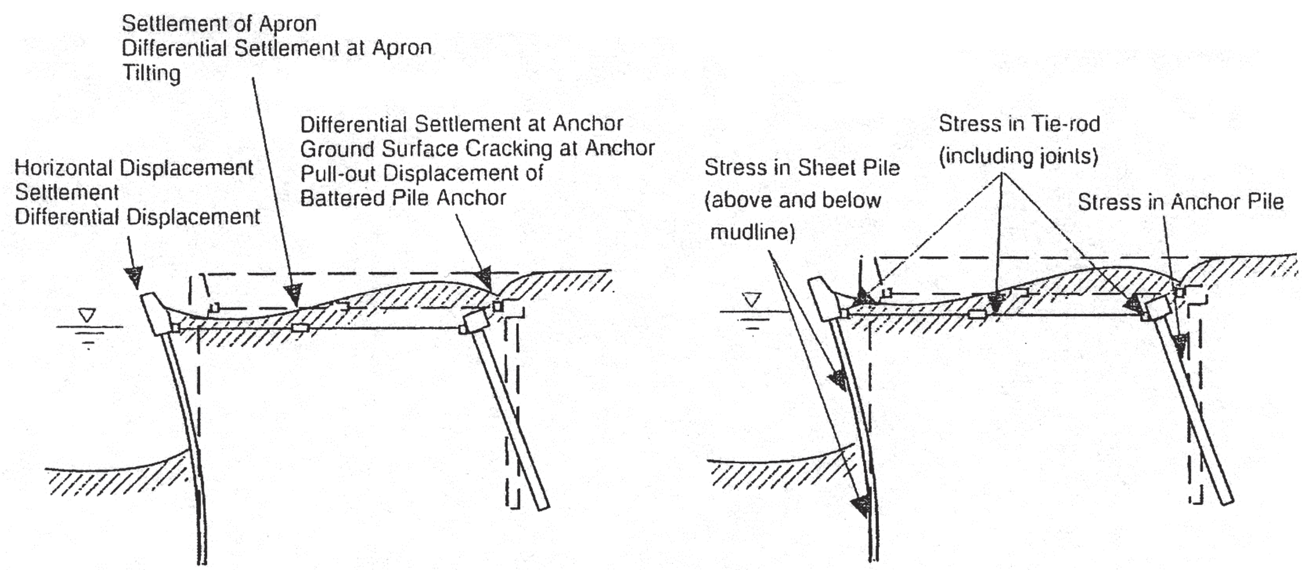
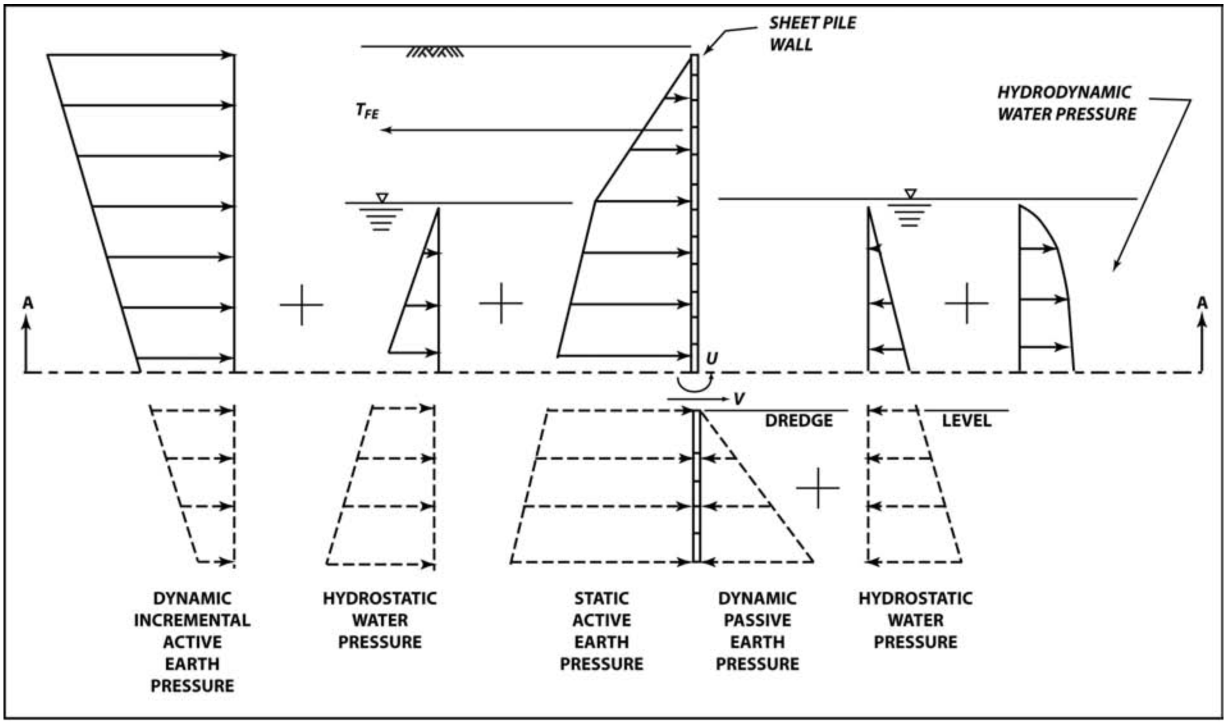


Figure B-6 Schematic cross sections illustrating earthquake damage modes (left) and damage criteria (right) for sheet pile walls with tieback anchors (PIANC, 2002).

Conventional practice for evaluating the seismic stability of tieback sheet pile walls is based on the use of pseudo-static approaches similar to those for caissons as previously described. The forces acting for pseudo-static limit equilibrium design are shown in Figure B-7. Detailed descriptions of this approach can be found in PIANC (2002) and in Ebeling and Morrison (1992).

Whereas pseudo-static seismic design can address issues related to global stability and sheet pile and anchor seismic stress levels, they cannot be used to evaluate deformations related to performance requirements. More advanced numerical dynamic finite element or finite difference approaches have been used for gravity caissons.

A deformation approach using the finite difference computer program FLAC has been presented by McCullough and Dickenson (1998). The deformations observed at the Ohama Wharf at the Akita Port in Japan during the M7.7 Nihonkai-Chubu earthquake in 1983 were simulated with reasonable success, as shown in Figure B-8. Additional parametric studies were performed to illustrate the extent of backland ground improvement needed to limit horizontal displacements.



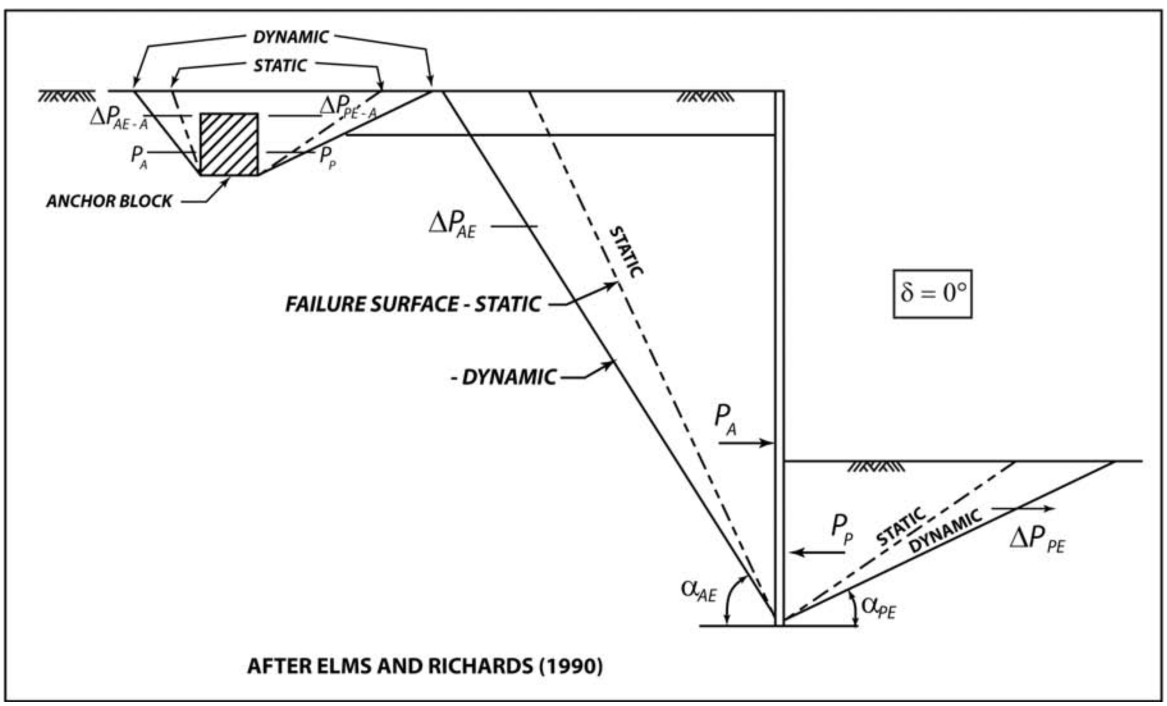


Figure B-7 Schematics illustrating forces in pseudo-static analysis of tieback sheet pile walls (Ebeling and Morrison, 1992).

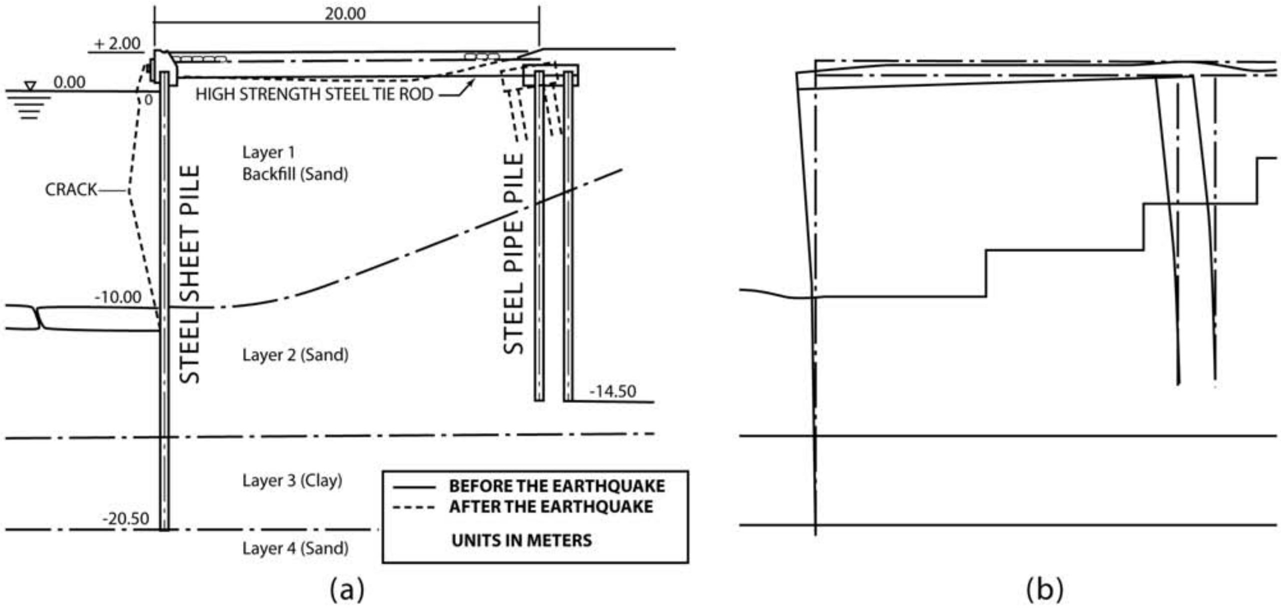


Figure B-8 Schematics showing (a) observed wharf deformations caused by the 1983 Nihonkai-Chubu earthquake versus (b) finite element simulation calculated using FLAC (McCullough and Dickenson, 1998).

A further example of a finite element approach using the computer program PLAXIS to compute deformations is described by Christie (2010), where both pseudo-static and dynamic finite element analysis methods were compared in the case of a non-liquefiable backfill. Figure B-9 shows deformation patterns under dynamic loads, while Figure B-10 shows a comparison between pseudo-static stress distributions and those from finite element analyses.

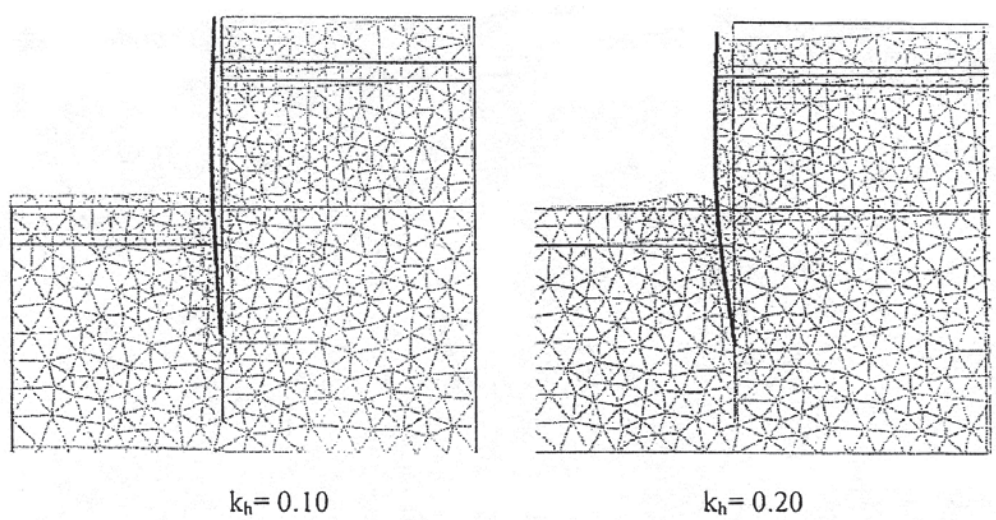


Figure B-9 Deformed mesh from finite element analysis showing deformation patterns of tieback sheet pile walls under dynamic loads (Christie, 2010).

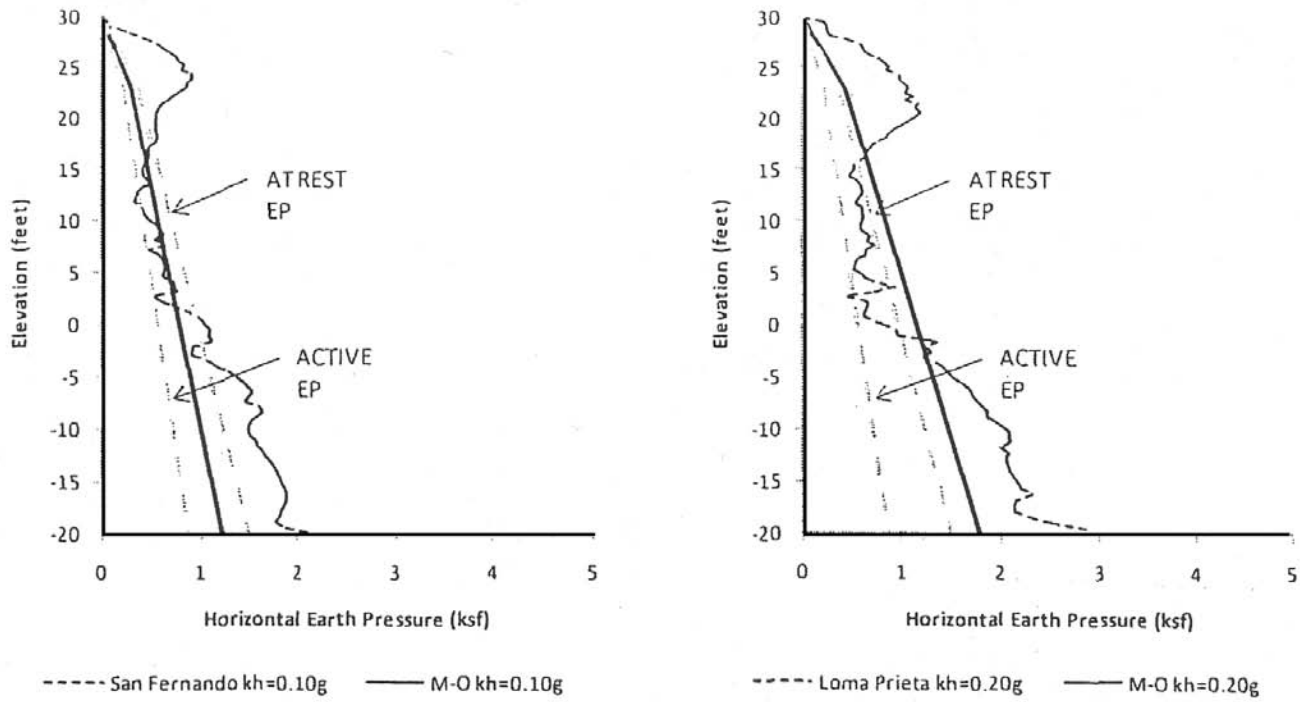


Figure B-10 Schematic cross sections showing pseudo-static and dynamic horizontal earth pressure distributions on tieback sheet pile wall systems during two different earthquakes (Christie, 2010).

Appendix C

Structural Design

This appendix provides additional information pertaining to the structural design of piles (Section C.1), and connections (Section C.2).

As indicated in Section 3.3.3, the current seismic design practice for piers and wharves differs from that used for conventional buildings or building-like structures. Geotechnical hazards are a prime consideration in the seismic design of pier and wharf structures, which are often constructed on slopes at sites with hydraulic fills and other weak soil materials that may be prone to liquefaction, lateral spreading, and slope instability during an earthquake (see Figure C-1).

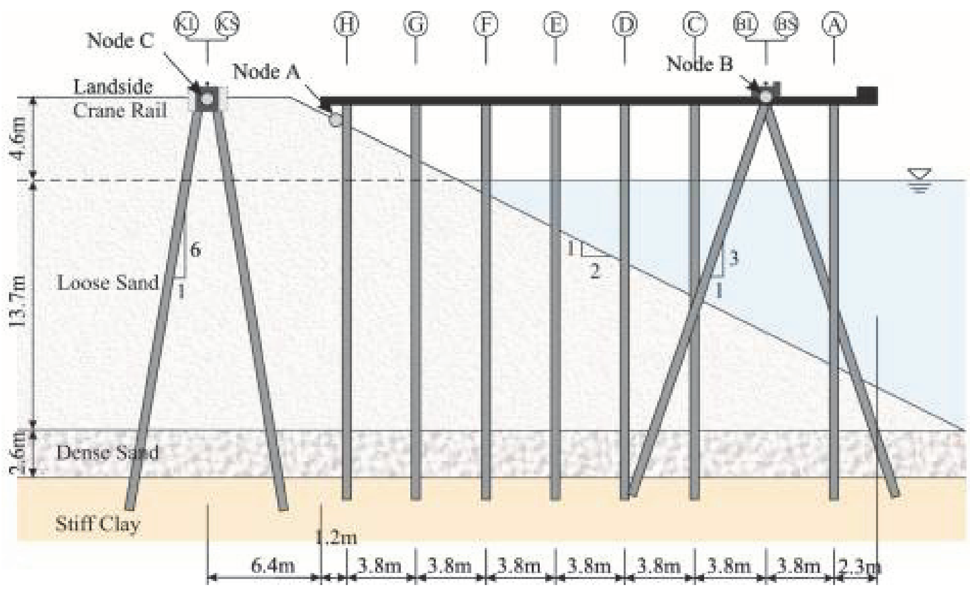


Figure C-1 Example cross-section of pile-supported wharf configuration (Shafieezadeh et. al., 2011a).

For piers and wharves to withstand a wide range of earthquake magnitudes, the structural design should emphasize three important parts of the structure: piles, pile-to-deck connections, and the deck. The pile-deck structural system should be designed as a moment-resisting frame resulting in a ductile system. Ideally, the moment-resisting frame concept should consist of the deck acting as a strong beam and the pile acting as a weak column. Plastic elastic hinges should be developed in the piles only. Capacity design (also known as capacity protected design) is the determination of the behavior and location of inelastic actions and the protection of other locations, such as decks and pile shear, to guarantee these remain elastic. Based on this concept, the deck must be designed as a capacity-protected member, requiring it to have greater capacity than the piles. To provide a capacity-protected design, the design forces for a deck member should be 25 percent greater than the demand forces. The design required strength for capacity protected members should be determined in a pushover analysis at displacements corresponding to the Contingency Level Earthquake limit state. For the evaluation of demand on capacity-protected members, an additional over-strength factor should be used when determining the capacity of pile plastic hinges.

The objective of seismic design is to verify that the displacement capacity of the pile-deck structure exceeds the displacement demand for the three earthquake performance levels. Seismic loads need not be combined with mooring and berthing loads or environmental loads. The current design methods for individual structural elements are summarized below, based on information provided in the following resource documents:

- Port of Long Beach (2009), *Wharf Design Criteria*, Version 2.0;
- Port of Los Angeles (2010), *Code for Seismic Design, Upgrade and Repair of Container Wharves*;
- Port of Long Beach (2012), *Wharf Design Criteria*, Version 3.0;
- International Navigation Association, *Seismic Design Guidelines for Port Structures* (PIANC, 2002);
- California Building Standards Commission (2010), *California Building Code*;
- American Society of Civil Engineers Standards; and
- Department of Defense (2005), *Unified Facilities Design Criteria: Piers and Wharfs.*

A majority of the codes and published documents differ slightly but generally follow the same design guidelines.

C.1 Piles

Wharf and pier structures are generally analyzed either by displacement-based design or force-based design. Displacement-based design is a common design method permitted for all structures. Force-based design is typically permitted for “Low” design classifications of structures or buildings. Force-based design is permitted for design classifications where the nominal capacity of all primary members can be demonstrated to be 20 percent larger than the elastic earthquake demand forces. Pile-deck structures are designed using the displacement-based design method.

Displacement-based design is based on the assumption that the sizing and configuration of primary structural members has been previously defined based on service loads. Analysis and design for seismic loads is to be performed for each seismic hazard and performance level. For each seismic hazard and performance level, the displacement capacity should exceed the displacement demand. Seismic analysis needs to include soil-pile interaction. The ability of pile-deck structures to respond to seismic activity depends on the displacement capacity of pile plastic hinges. The displacement capacity varies in accordance with the material properties, length, cross-section area, and axial loading of the pile. The performance of the potential plastic hinges in the piles should be checked with the performance levels to confirm that the maximum pile material strain limits do not exceed the limits defined in the structural performance limit states. The Operating Level Earthquake, Contingency Level Earthquake, and Design Level Earthquake limit states should be verified for top of pile, in-ground, and deep in-ground plastic hinge strain limits. Typically, depending on the material of the pile, Operating Level Earthquake strain limits for minimal damage range from 0.004 to 0.015, Contingency Level Earthquake strain limits for controlled and repairable damage range from 0.005 to 0.025, and Design Level Earthquake strain limits for life safety protection range from 0.008 to 0.050. Operating Level Earthquake, Contingency Level Earthquake and Design Level Earthquake limit states may vary based on each code requirement and should be verified accordingly.

Displacement-based design is a compilation of several analysis methods. The Nonlinear Static Pushover Analysis Method provides the displacement capacity based on material strain limits. This method incorporates soil deformation into the total displacement capacity of the pile. Pushover analysis consists of inelastic pushover analyses in both the transverse and longitudinal directions, where two-dimensional sections of the wharf or pier are subjected to incremental increases in displacement. This method determines an appropriate stiffness for a stiffness method, damping levels for elastic analysis, and peak plastic rotations of critical hinges at the maximum displacement. The pushover model should use effective material and section properties while incorporating the soil stiffness with nonlinear upper- and lower-bound p-y springs. From the analysis, displacement demand should not exceed the displacement capacity. If the demand exceeds the capacity, the design must be revised. Otherwise, for the pile-to-deck connection, the pile shear can be checked.

In addition to the nonlinear static pushover analysis method, the supporting documents mention the use of other analysis methods that can be used for displacement-based design. However, a nonlinear static analysis method is more common. The other analyses include but are not limited to the following: equivalent lateral stiffness method, elastic stiffness method, substitute structure method, modal spectra analysis, and nonlinear time-history analysis.

Kinematic loads occur in piles when the soil or dike below the deck begins to move as a result of sliding on a weak soil layer during a seismic event. Induced moments in the pile will develop beneath the soil, causing deep in-ground plastic hinges to form (see Figure C-2). In the case of kinematic loads, additional nonlinear dynamic soil-structure interaction analysis will be necessary. Material strain at pile plastic hinges should stay within code-defined strain limits because of inertial and kinematic loads.

C.2 Connections

Wharf and pier structures are designed as moment resisting frames, as previously stated. The deck acts as a strong beam and the piles act as weak columns. To ensure

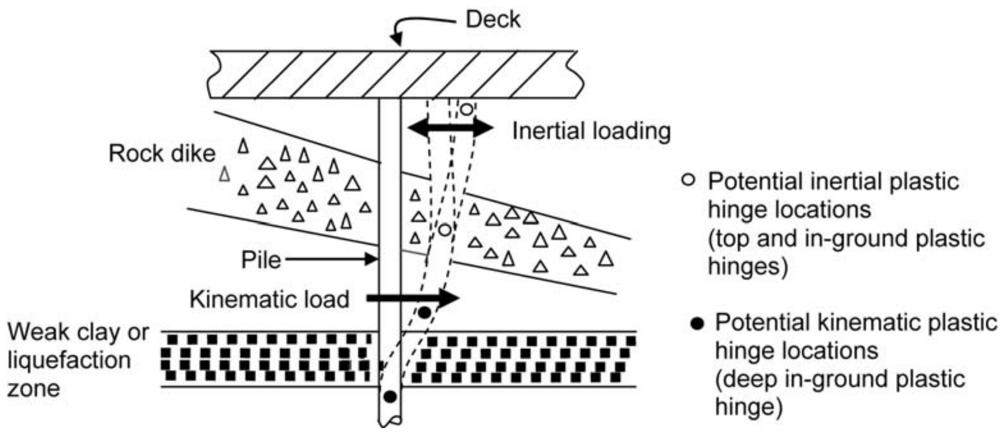
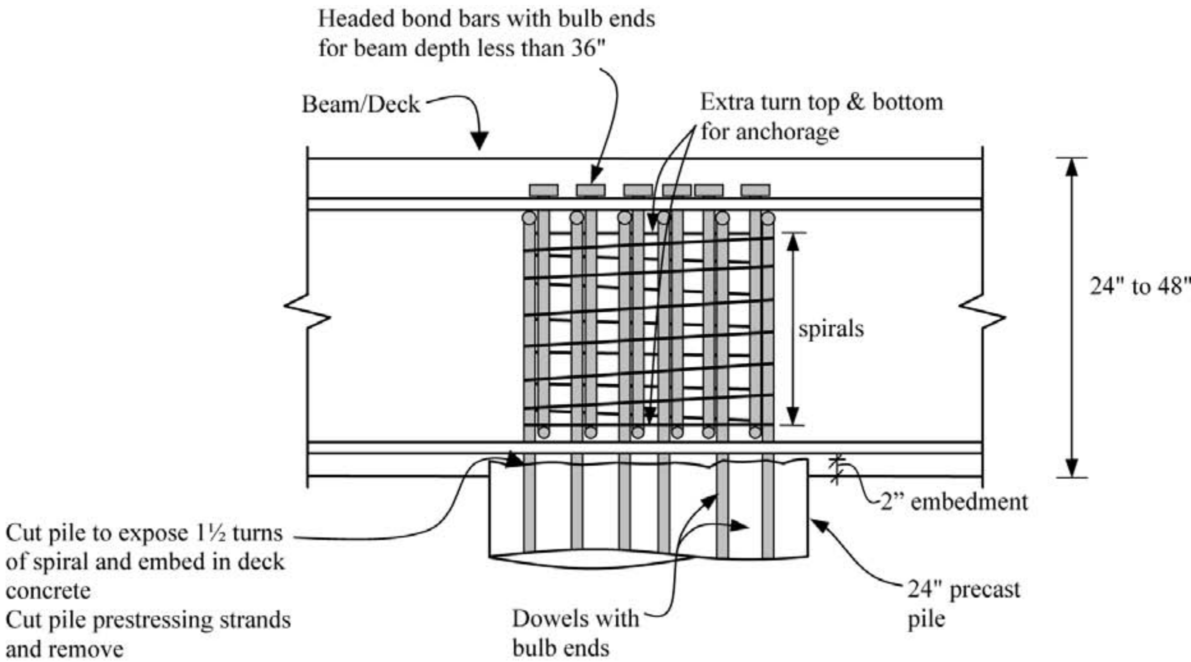


Figure C-2 Schematic cross section of pile-supported deck and foundation showing plastic hinge formation due to kinematic and inertial loads (Port of Long Beach, 2009)

that the system works properly, it is vital that the nominal strength capacity of the deck be sufficient to ensure the piles plastic limit is reached prior to the deck attaining its expected nominal strength. The deck should not form a plastic hinge or any type of permanent deformation. In the connection between the deck and piles, ductile moment-resisting connections are required. At a typical pile-deck joint location the moment, axial and shear demands as a result of the plastic hinge capacity of the piles should be determined using the load combinations provided in the code.

In areas of low seismicity, other lateral service loads, such as waves and mooring, may be significantly greater than the seismic forces and the deck will be stronger than the seismic demand forces. In the event that elastic seismic forces and other lateral forces are similar in magnitude, the pile overstrength forces should not exceed the nominal capacity of the deck.

The connection shear principal stresses because of the demand forces are determined according to American Concrete Institute (ACI) standards or formulas used for concrete design in other codes. Once the stresses are determined to be satisfactory, the pile-deck connection needs to be detailed properly to ensure ductile behavior. Figures C-3 provides examples of pile-to-deck connection details.



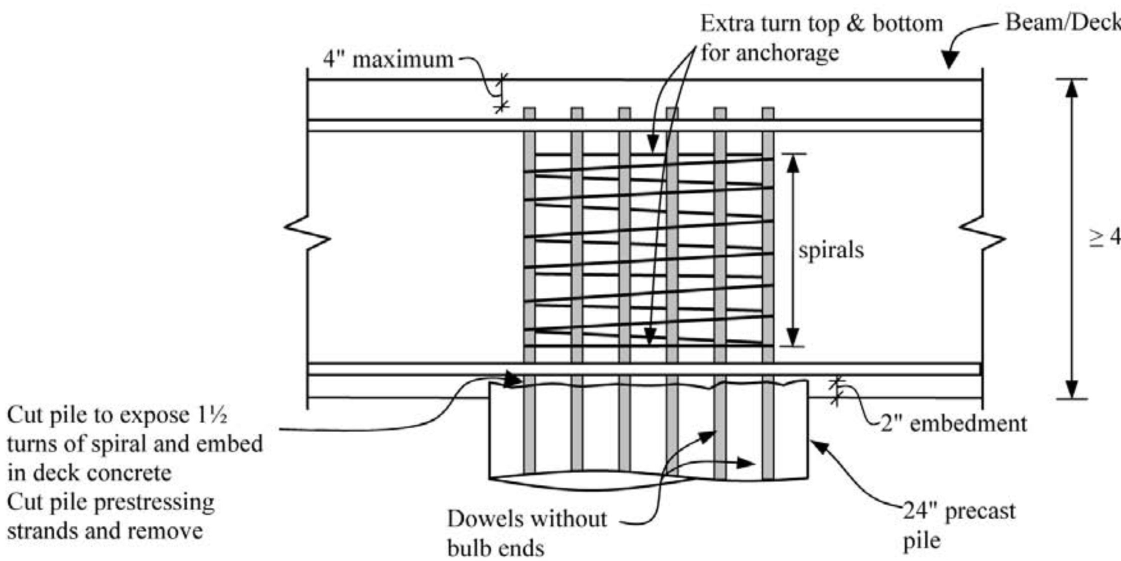


Figure C-3 Port of Long Beach (2009) wharf design criteria pile-deck connection details.

Appendix D
Cranes

The information on cranes presented in this appendix supplements the literature review findings described in Section 3.3.5

Soderberg et al. (2009) examined several retrofit options for existing cranes, including (a) modifying the crane to tip by installing diagonal bracing in the portal frame, (b) modifying the crane to yield in a ductile fashion by adding external stiffeners, and (c) modifying the crane by installing an isolation mechanism to the portal frame legs. Implementing the proposed seismic design criteria is expected to cost approximately $150,000 to $180,000 for new cranes and $300,000 to more than $500,000 for existing cranes.

The Port of Los Angeles has developed requirements governing the seismic evaluation of container cranes on existing container wharves (Port of Los Angeles, 2010). Existing container cranes on existing wharves do not require evaluation. The following requirements apply when the crane is modified or replaced. For cranes supported on wharves designed in accordance with either the 2004 or 2010 Port of Los Angeles *Code for Seismic Design, Upgrade and Repair of Container Wharves*:

- If the new crane is identical or similar to the existing crane, no evaluation is required.
- If the new crane is not identical or similar:
    - If $T_{crane} > 2\ T_{wharf}$, no evaluation is required.
    - If $T_{crane} < 2\ T_{wharf}$, an evaluation via nonlinear time history analysis to determine if the wharf-crane interaction increases the *displacement* demand on the wharf must be performed. If the increased demand exceeds the capacity of the wharf, the crane and wharf must be modified.

For cranes supported on wharves prior to the use of the 2004 Port of Los Angeles *Code for Seismic Design, Upgrade and Repair of Container Wharves*:

- If the new crane is identical or similar to the existing crane, no evaluation is required.
- If the new crane is not identical or similar:
    - If $T_{crane} > 2\ T_{wharf}$, no evaluation is required.

- If $T_{crane} < 2\ T_{wharf}$, an evaluation via nonlinear time history analysis to determine if the wharf-crane interaction increases the *seismic* demand on the wharf must be performed. If the increased demand exceeds the capacity of the wharf, the crane and/or wharf must be modified.

In the Jacobs et al. (2011) examination of the seismic response of jumbo container cranes as part of the NEES Grand Challenge project described in Chapter 4, a simple pseudo-static model was developed to predict the threshold horizontal acceleration at which the crane will uplift, which results in derailment. A frictional contact element was developed to simulate the response of crane wheels. Two-dimensional (2-D) and three-dimensional (3-D) numerical models of the crane were validated using a large-scale shake table experiment. The 2-D analysis was found to be sufficient for most cases.

Shafieezadeh (2011) has studied the effect of dynamic interaction between pile-supported wharves and container cranes. The conventional wisdom is that cranes will act as tuned-mass dampers with respect to the response of the wharf, and thus it is conservative to neglect crane-wharf interaction. The study considers several models of a modern jumbo crane to assess wharf-crane interaction:

- two-degree-of-freedom system
- five percent of the mass of the wharf rigidly attached to the deck
- constant vertical forces equal to the gravity loads exerted by the crane
- pinned-based portal frame model
- portal frame model with contact base conditions allowing sliding and uplift (i.e., tipping).

Ten ground motions with Peak Ground Accelerations (PGAs) ranging from 0.05 to 0.83 g were used to perform nonlinear time-history analyses. The fundamental natural period of the wharf in the transverse direction was 0.26 second; the natural period of the crane in the trolley-travel direction was 1.5 seconds. The analyses showed that the presence of the crane considerably increases deformation demands on the wharf. The maximum response in different response measures of the wharf, including the curvature response of pile sections and pile-deck connections as well as the horizontal and vertical response of crane rails, increased considerably when a detailed crane model was included. This behavior is a consequence of the large contribution of the crane gravity loads on the response of the wharf, the factor lacking in the formulation of the simplified approach. In addition, a number of common simplified crane models were also analyzed with the nonlinear wharf model, and their performance in response prediction of the wharf with sliding/uplift crane model was evaluated. The main conclusions of the set of analyses are as follows:

- Increasing the mass of the wharf deck by 5 percent cannot appropriately capture the effect of wharf-crane interaction on the response of the wharf.
- The pinned crane model well represents the sliding/uplift crane model especially for small earthquakes where the intensity of the ground shaking is not large enough for sliding or uplift to occur.
- The wharf with crane gravity captures the effect of the wharf-crane interaction on the response of the wharf properly for low- and high-intensity earthquakes. However, it does not adequately capture the deformation response of pile-deck connections.

References

Ak, A., 2008, *Berth and Quay Crane Scheduling: Problems, Models, and Solution Methods*, Ph.D. dissertation, Georgia Institute of Technology, Atlanta, Georgia.

Anderson, D. G., Martin, G.A., Lam, I. (Po), and Wang, T. N., 2008, *Seismic Analysis and Design of Retaining Walls, Buried Structures, Slopes, and Embankments*, Report 611, National Cooperative Highway Research Program (NCHRP) Transportation Research Board, Washington, DC.

Andrews, D. C. A., and Martin, G. R., 2000, "Criteria for Liquefaction of Silty Soils," *Proceedings of the 12th World Conference Earthquake Engineering*, Auckland, New Zealand.

Andrus, R. D., and Chung, R. M., 1995, *Ground Improvement Techniques for Liquefaction Remediation Near Existing Lifelines*, NISTIR Report #5714, Building and Fire Research Laboratory, National Institute of Standards and Technology, Gaithersburg, Maryland.

Arulmoli, K., Johnson, G. S., Yin, P., Jarradat, O. A., and Mays, T. W., 2008, "Geotechnical Considerations and Soil-Structure Interaction: Proposed ASCE Standards for Seismic Design of Piers and Wharves," *Proceedings of Geotechnical Earthquake Engineering and Soil Dynamics IV*, American Society of Civil Engineers, Reston, Virginia.

Arulmoli, A.K., and Martin, G.R., 2005, Presentation on Geotechnical Aspects (unpublished), Port of Los Angeles Container Wharf Seismic Code Workshop, Los Angeles.

Arulmoli, A.K., and Martin, G.R., 2009, "Seismic Soil Structure Interaction Issues for Pile-Supported Piers and Wharves," *Proceedings, Conference on Lifeline Earthquake Engineering in a Multihazard Environment*, American Society of Civil Engineers, Reston, Virginia.

ASCE, 1997, *Ground Improvement, Ground Reinforcement, Ground Treatment, Developments 1987-1997*, Geotechnical Special Publication No. 69, American Society of Civil Engineers, New York, New York, 616 pp.

ASCE, 2006, *Minimum Design Loads for Buildings and Other Structures*, ASCE/SEI 7-05 Report, American Society of Civil Engineers, Reston, Virginia, 388 pp.

ASCE, 2012, *Seismic Design of Pile-Supported Piers and Wharves*, Draft Report, American Society of Civil Engineers, Reston, Virginia.

Baker, J.W., and Jayaram, N., 2008, “Correlation of Spectral Acceleration Values from NGA Ground Motion Models,” *Earthquake Spectra*, Vol. 24, No. 1, pp 299-317.

Boore, D.M., and Atkinson, G.M., 2008, “Ground-Motion Prediction Equations for the Average Horizontal Component of PGA, PGV, and 5%-Damped PSA at Spectral Periods between 0.01s and 10.0s,” *Earthquake Spectra*, Vol. 24, No. 1, pp. 99-138.

Boulanger, R.W., and Hayden R.F., 1995, “Aspects of Compaction Grouting of Liquefiable Soils,” *Journal of Geotechnical Engineering*, Vol. 121, No. 12, pp. 844-855.

Boulanger, R. W., and Idriss, I. M., 2006, “Liquefaction Susceptibility Criteria for Silts and Clays,” *Journal of Geotechnical and Geoenvironmental Engineering*, ASCE, Vol. 132, No. 11, November.

Bray, J. D. and Sancio, R. B., 2006, “Assessment of the Liquefaction Susceptibility of Fine-Grained Soils,” *Journal of Geotechnical and Geoenvironmental Engineering*, ASCE, Vol. 132, No. 9, September.

Brodbaek, C., and Laursen, P., 2010, *Seismic Design of Marine Caisson Waterfront Structures*, Proceedings, Ports 2010 Conference, Florida.

California State Lands Commission, 2010, *California Code of Regulations*, Title 24, Part 2, Chapter 31F, otherwise known as the Marine Oil Terminal Engineering and Maintenance Standards (MOTEMS).

California Building Standards Commission, 2010, *California Building Code*, Title 24, Part 2, Sacramento, CA.

Christie, S.R., 2010, *Seismic Design Methods for Anchored Sheet Pile Bulkheads*, Proceedings, Ports 2010 Conference, Florida.

Conlee, C.T., 2010, *Dynamic Properties of Colloidal Silica Soils using Centrifuge Model Tests and A Full-Scale Field Test*, PhD Dissertation, Drexel University, Philadelphia, Pennsylvania 186 pp.

Cooke, H.G., and Mitchell, J.K., 1999, *Guide to Remedial Measures for Liquefaction Mitigation at Existing Highway Bridge Sites*, Technical Report MCEER-99-0015, Multidisciplinary Center for Earthquake Engineering Research, University at Buffalo, 176 pp.

Dafalias, Y.F. and Manzari, M.T., 2004, “Simple Plasticity Sand Model Accounting for Fabric Change Effects,” *Journal of Engineering Mechanics*, Vol. 130, No. 6, pp 622-634.

DOD, 2005, *United Facilities Criteria, Design: Piers and Wharves*, UFC 4-152-01 Army Corps of Engineers, Naval Facilities Engineering Command, Air Force Civil Engineer Support Agency, Department of Defense.

Diaz, G. M., Armstrong, B. W., and Joolazadeh, M., 1984, “Lateral Load Test of Piles in Sloping Rockfill,” *Proceedings of the Session on Analysis and Design of Pile Foundations*, ASCE National Convention, San Francisco, California.

Dickenson, S. E., and Yang, D. S., 1998, *Seismically-Induced Deformations of Caisson Retaining Walls in Improved Soils*, Geotechnical Earthquake Engineering and Soil Dynamics III, ASCE Geotechnical Special Publication No. 75.

Duncan, J. M., and Wright, S. G., 2005, *Soil Strength and Slope Stability*, John Wiley, Hoboken, New Jersey.

EERI, 1990, “Loma Prieta, California, Earthquake of October 15, 1989,” *Earthquake Spectra*, Vol. 06, Supplement, Earthquake Engineering Research Institute, Oakland, California

Ebeling, R.M., and Morrison, E. E., 1992, *The Seismic Design of Waterfront Retaining Structures*, US Navy Technical Report NCEL TR-939, U.S. Army Engineer Waterways Experiment Station, Vicksburg, MS.

Egan, J.A., Hayden, R.E., Scheibel, L.L., Otus, M., and Serventi, G.M., 1992, “Seismic Repair at Seventh Street Marine Terminal,” *Grouting, Soil Improvement and Geosynthetics*, Vol. 2, ASCE Geotechnical Special Publication No. 30, Reston, Virginia.

EMI, 1998a, “Bi-Linear p-y Curves for Pier 400 Piles and Pendent Wall,” Memorandum prepared by Earth Mechanics, Inc. for the Port of Los Angeles, March 25, Appendix to “Geotechnical Study, Pier 400, Stage 2, Phase 1 Container Wharf, Port of Los Angeles,” prepared by Fugro West, Inc. for the Port of Los Angeles, 1999.

EMI, 1998b, “Soil-Structure Interaction at Ground Surface Region for Unimproved Ground Displacement Scenario,” Memorandum prepared by Earth Mechanics, Inc. for the Port of Los Angeles, July 1, Appendix to “Geotechnical Study, Pier 400, Stage 2, Phase 1 Container Wharf, Port of Los Angeles,” prepared by Fugro West, Inc. for the Port of Los Angeles, 1999.

Fang, H.Y., and Daniels, J.L., 2006, *Introductory Geotechnical Engineering, An Environmental Perspective*, Taylor and Francis, New York and London, pp. 306-313.

Ferrito, J., Dickenson, S., Priestley, N., Werner, S., and Taylor, C., 1999, *Seismic Criteria for California Marine Oil Terminals*, Technical Report TR-2103-SHR, Naval Facilities Engineering Service Center, Port Hueneme, California.

FHWA, 2006, *Seismic Retrofitting Manual for Highway Structures: Part 1 – Bridges*, Publication No. FHWA-HRT-06-032, Federal Highway Administration, U.S. Department of Transportation, Washington, DC.

Field, E.H., Dawson, T.E., Felzer, K.R., Frankel, A.D., Gupta, V., Jordan, T.H., Parson, T., Petersen, M.D., Stein, R.S., Weldon, R.J., and Wills, C.J., 2009, "Uniform California Earthquake Rupture Forecast, Version 2 (UCERF 2)," *Bulletin of the Seismological Society of America*, Vol. 99, No. 4, pp. 2053-2107.

Field, E.H., Jordan, T.H., and Cornell, C.A., 2003, "OpenSHA: A Developing Community-Modeling Environment for Seismic Hazard Analysis," *Seismological Research Letters*, Vol. 74, No. 4, pp. 406-419.

Harding-Lawson Associates, 1983, *Pile Lateral Load Test, Berths 216-218, Port of Los Angeles, California*, Report to Daniel, Mann, Johnson & Mendenhall, Los Angeles, California.

Harding-Lawson Associates, 1991, *Pile Lateral Load Test, Berths 212-215, Port of Los Angeles, Terminal Island, California*, Report to Moffatt & Nichol Engineers, Long Beach, California.

Howell, R., Rathje, E., Kamai, R., and Boulanger, R., 2012, "Centrifuge Modeling of Prefabricated Vertical Drains for Liquefaction Remediation," *Journal of Geotechnical and Geoenvironmental Engineering*, Vol. 138.

Iai, S., Ichii, K., Sato, Y., and Liu, H., 1999, "Residual Displacement of Gravity Quaywalls – Parameter Study through Effective Stress Analysis," *7th Proceedings of the U.S.-Japan Workshop on Earthquake Resistant Design of Lifeline Facilities and Countermeasures against Soil Liquefaction*, Technical Report MCEER-99-0019, Multidisciplinary Center for Earthquake Engineering Research, State University of New York, University at Buffalo.

Ivey, L., 2012, *Forecasting Earthquake Losses in Port Systems*, Ph.D. Dissertation, Georgia Institute of Technology, Atlanta, Georgia, 391 pp.

Jacobs, L.D., 2010, *Shake Table Experiments for the Determination of the Seismic Response of Jumbo Container Cranes*, Ph.D. Dissertation, Georgia Institute of Technology, Atlanta, Georgia, 306 pp.

Jacobs, L.D., Kosbab, B.D., Leon, R.T., and DesRoches, R., 2011, "Seismic Behavior of a Jumbo Container Crane Including Uplift," *Earthquake Spectra*, Vol. 27, No. 3, pp. 745-773.

Jayaram, N., and Baker, J.W., 2009, "Correlation Model for Spatially Distributed Ground-Motion Intensities," *Earthquake Engineering and Structural Dynamics*, Vol. 38, pp. 1687-1708.

Kawamata, Y., Blandon, C. A., Ashford, S. A., and Restrepo, J. I., 2008, *Seismic Performance of Container Wharf Piles: Test Results*, Report No. TR-2007/6, Department of Structural Engineering, University of California, San Diego, California.

Koelling, M.A., and Dickenson, S.E., 1998, "Ground Improvement Case Histories for Liquefaction Mitigation of Port and Near-Shore Structures," *Proceedings, Geotechnical Earthquake Engineering and Soil Dynamics III Conference*, ASCE Geotechnical Special Publication No. 75, American Society of Civil Engineers, Reston, Virginia.

Kosbab, B.D., 2010, *Seismic Performance Evaluation of Port Container Cranes Allowed to Uplift*," Ph.D. Dissertation, Georgia Institute of Technology, 326 pp.

Lehman, D.E., Brackmann, E., Jellin, A., and Roeder, C.W., 2009, "Seismic performance of pile-wharf connections", *Proceedings, TCLEE 2009: Lifeline Earthquake Engineering in a Multihazard Environment*, A.K.K. Tang and S.D. Werner (Eds.), American Society of Civil Engineers, Reston, Virginia, pp. 865-877.

Mace, N., and Martin, G.R., 2000, *An Investigation of Compaction Grouting for Liquefaction Mitigation*, Unpublished Report, Multidisciplinary Center for Earthquake Engineering Research, State University of New York at Buffalo, Buffalo, New York.

Mageau, D., and Chin, K., 2009, "Effectiveness of Stone Columns on Slope Deformations Beneath Wharves," *ASCE TCLEE Proceedings, Conference on Lifeline Earthquake Engineering in a Multihazard Environment*, Oakland, California.

Marinucci, A.M., Rathje, E., Ellington, J.S., Cox, B.R., Menq, F.Y., and Stokoe II, K.H., 2010, "Evaluation of the Effectiveness of Prefabricated Vertical Drains using Full-Scale In Situ Staged Dynamic Testing," *Art of Foundation Engineering Practice*, M.H. Hussein, J.B. Anderson, and W.M. Camp, (editors), Geotechnical Special Publication 198, American Society of Civil Engineers, Reston, Virginia.

Martin et al.,1999, *Recommended Procedures for Implementation of DMG Special Publication 117: Guidelines for Analyzing and Mitigating Liquefaction Hazards in California*, Southern California Earthquake Center, Los Angeles, California.

McCullough, N.J., and Dickenson, S.E., 1998, "Estimation of Seismically Induced Lateral Deformations for Anchored Sheetpile Bulkheads," *Proceedings, ASCE Geotechnical Earthquake Engineering and Soil Dynamic Conference III*, Seattle, Washington, American Society of Civil Engineers, Reston, Virginia.

McGuire, R., 2004, *Seismic Hazard and Risk Analyses*, Monograph MNO-10, Earthquake Engineering Research Institute, Oakland, California, 221 pp.

McKenna, F., Scott, M.H., and Fenves, G.L., 2010, "Nonlinear Finite Element Analysis Software Architecture Using Object Composition," *Journal of Computing in Civil Engineering*, Vol. 24, pp. 95-107.

MLIT, 2007, *Technical Standards and Commentaries for Port and Harbour Facilities in Japan*, Ministry of Land, Infrastructure, Transport and Tourism, 998 pp.

Mitchell, J. K., Baxter, C. D. P., and Munson, T.C., 1995, "Performance of Improved Ground During Earthquakes," *ASCE Proceedings on Soil Improvement for Earthquake Hazard Mitigation*, GSP. 49, R.D. Hryciw (editor), American Society of Civil Engineers, Reston, Virginia, pp. 1-36.

MCEER/ATC, 2003, *Recommended LRFD Guidelines for the Seismic Design of Highway Bridges – Part I: Specifications; Part II: Commentary and Appendices; Liquefaction Study Report*, prepared by MCEER/ATC Joint Venture under National Cooperative Highway Research Program Project 12-49, published as MCEER/ATC Report 49, Parts I and II, and MCEER/ATC Report 49-1 (available through the Applied Technology Council), and as IIMCEER Report Numbers MCEER-03-SP03 (Part I and Part II) and MCEER-03-SP08 (available through MCEER, Statue University of New York at Buffalo).

NCEL, 1990, *Technical Note on Liquefaction Mitigation Technology*, Naval Civil Engineering Laboratory, Port Hueneme, California.

Newmark, N.M., 1965, "Effects of earthquakes on dams and embankments". *Geotechnique*, Vol. 15, No. 2, London, England, pp. 139-160.

Olson, S. M., and Stark, T. D., 2002, "Liquefaction Strength Ratio from Liquefaction Flow Failure Case Histories," *Canadian Geotechnical Journal*, Vol. 39, May.

Paczkowski, K., Riggs, H. R., Naito, C. J., and Lehmann, A., 2012, "A One-Dimensional Model for Impact Forces Resulting From High Mass, Low Velocity Debris, *Structural Engineering and Mechanics*, Vol. 42, No. 6.

Park, J., Bazzurro, P., and Baker, J.W., 2007, "Modeling Spatial Correlation of Ground Motion Intensity Measures for Regional Seismic Hazard and Portfolio Loss Estimation," *Proceedings, Tenth International Conference on Application of Statistics and Probability in Civil Engineering* (ICASP10), Tokyo, Japan.

PIANC, 2002, *Seismic Design Guidelines for Port Structures*, World Association for Waterborne Transport Infrastructure, A. A. Balkema Publishers, Brussels, Belgium.

Porbaha, A., Zen, K,. and Kobayashi, M., 1999, "Mixing Technology for Liquefaction Mitigation," *Journal of Infrastructure Systems*, Vol. 5, No. 1, pp. 21-34.

Port of Long Beach, 2009, *Wharf Design Criteria*, *Version 2.0*, Long Beach, California.

Port of Los Angeles, 2004, *Code for Seismic Design, Upgrade and Repair of Container Wharves*, Los Angeles, California.

Port of Los Angeles, 2010, *Code for Seismic Design, Upgrade and Repair of Container Wharves*, Los Angeles, California.

Salgado, R., Boulanger, R., and Mitchell, J. K., 1997, "Lateral Stress Effects on CPT Liquefaction Resistance Correlations." *Journal of Geotechnical and Geoenvironmental Engineering*, Vol. 123, No. 8, American Society of Civil Engineers, Reston, Virginia, pp. 726-735.

SCEC, 2002, *Recommended Procedures for Implementation of DMG Special Publication 117: Guidelines for Analyzing and Mitigating Landslide Hazards in California*, Southern California Earthquake Center, Los Angeles, California.

Scharks, T., Bostrom, A., Reimann-Garretson, L., and Rix, G.J., (under review), "Risk Decision Making and Seismic Risk Preparedness at North American Seaports: Analysis of a System-Wide Survey," under review for *Earthquake Spectra*, Earthquake Engineering Research Institute, Oakland, California.

Seed, R. B., and Harder, L. F. Jr., 1990, "SPT-Based Analysis of Cyclic Pore Pressure Generation and Undrained Residual Shear Strength," *Proceedings, H. Bolton Seed Memorial Symposium*, BiTech Publishers, Ltd.

Seed, R. B., et al., 2003, "Recent Advances in Soil Liquefaction Engineering: A Unified and Consistent Framework," *26th Annual ASCE Los Angeles*

Section, Spring Seminar, Long Beach, California, American Society of Civil Engineers, Reston, Virginia.

Shafieezadeh, A., DesRoches, R., and Rix, G.J., (under review), "Three-Dimensional Wharf Response to Far-Field and Impulsive Near-Field Ground Motions in Liquefiable Soils," under review for *Journal of Structural Engineering*.

Shafieezadeh, A., DesRoches, R., Rix, G.J., and Werner, S.D., (in press), "Seismic Performance of Pile-Supported Wharf Structures Considering Soil-Structure Interaction in Liquefied Soil," *Earthquake Spectra*, Earthquake Engineering Research Institute, Oakland, California.

Shafieezadeh, A., 2011, *Seismic Vulnerability Assessment of Wharf Structures*, Ph.D. Dissertation, Georgia Institute of Technology, Atlanta, Georgia, 234 pp.

Skempton, A.W., "1964 Long Term Stability of Clay Slopes," *Geotechnique*, Vol. 14, No. 2, pp. 71-102.

Soderberg, E., and Jordan, M., 2007, *Dockside Ship-to-Shore Cranes: Seismic Risk and Recommended Design Criteria, Revision 2*, Liftech Consultants, Inc., Oakland, California.

Soderberg, E., Hsieh, J., and Dix, A., 2009, "Seismic Guidelines for Container Cranes," *TCLEE 2009: Lifeline Earthquake Engineering in a Multihazard Environment*, A.K.K. Tang and S.D. Werner, (editors), American Society of Civil Engineers, Reston, Virginia, pp. 919-929.

Tang, L., Titov, V.V., and Chamberlin, C.D., 2009, "Development, Testing, and Applications of Site-Specific Tsunami Inundation Models for Real-Time Forecasting," *Journal of Geophysical Research*, Vol. 114, American Geophysical Union.

Varun, and Assimaki, D., 2010, "A Generalized Hysteresis Model for Biaxial Response of Pile Foundations in Sands," *Soil Dynamics and Earthquake Engineering*, Vol. 32, No. 1, pp. 56-70.

Vazifdar, F.R., and Kaldveer, P., 1995, "The Great Guam and Hanshin Earthquake, Profiles in Port Disasters," Liftech Consultants, Inc. Report, Oakland, California.

Vytiniotis, A., Whittle, A.J., and Kausel, E., 2011, " Effects of Seismic Motion Characteristics on Cyclic Mobility and Liquefaction", *Proceedings, 5th International Conference on Earthquake Geotechnical Engineering*, Santiago, Chile.

Werner, S.D., 1998, *Seismic Guidelines for Ports*, Technical Council on Lifeline Engineering, Monograph No. 12, American Society of Civil Engineers, Reston, Virginia.

Werner, S.D., and Cooke, W.C., 2010, *Wharf Repair Estimates for Use in Demonstration Seismic Risk Analysis of Port Systems*, unpublished report.

Woo, G.,1999, *The Mathematics of Natural Catastrophes*, Imperial College Press, London, England, 292 pp.

Youd, T.L., et al., 2001, "Liquefaction Resistance of Soils: Summary Report from the 1996 NCEER and 1998 NCEER/NSF Workshops on Evaluation of Liquefaction Resistance of Soils," *Journal of Geotechnical and Geoenvironmental Engineering*, Vol. 127, No. 10, American Society of Civil Engineers, Reston, Virginia.

Zmuda, R., and Arulmoli, K., 2004, "Displacement to Improve Slope Stability at Port of Los Angeles Berth 100 Wharf," *Proceedings Ports 2004 Conference,* Houston, Texas.

Project Participants

National Institute of Standards and Technology

John (Jack) R. Hayes
National Institute of Standards and Technology
Engineering Laboratory (MS8604)
100 Bureau Drive
Gaithersburg, Maryland 20899

Steven L. McCabe
National Institute of Standards and Technology
Engineering Laboratory (MS8603)
100 Bureau Drive
Gaithersburg, Maryland 20899

NEHRP Consultants Joint Venture

APPLIED TECHNOLOGY COUNCIL
201 Redwood Shores Parkway, Suite 240
Redwood City, California 94065
www.ATCouncil.org

CONSORTIUM OF UNIVERSITIES FOR RESEARCH IN EARTHQUAKE ENGINEERING
1301 S. 46th Street, Building 420
Richmond, California 94804
www.CUREE.org

Joint Venture Management Committee

James R. Harris
J.R. Harris & Company
1775 Sherman St, Suite 1525
Denver, Colorado 80203

Christopher Rojahn
Applied Technology Council
201 Redwood Shores Parkway, Suite 240
Redwood City, California 94065

Robert Reitherman
Consortium of Universities for Research in Earthquake Engineering
1301 S. 46th Street, Building 420
Richmond, California 94804

Andrew Whittaker
University at Buffalo
Dept. of Civil, Structural, and Environmental Engineering, 230 Ketter Hall
Buffalo, New York 14260

Joint Venture Program Committee

Jon A. Heintz (Program Manager)
Applied Technology Council
201 Redwood Shores Parkway, Suite 240
Redwood City, California 94065

William T. Holmes
Rutherford and Chekene
55 Second Street, Suite 600
San Francisco, California 94105

C.B. Crouse
URS Corporation
1501 4th Avenue, Suite 1400
Seattle, Washington 98101

Jack Moehle
University of California Berkeley
325 Davis Hall, MC1792
Berkeley, California 94720

Michael Constantinou
University at Buffalo
Dept. of Civil, Structural, and Environmental Engineering, 132 Ketter Hall
Buffalo, New York 14260

James R. Harris (ex-officio)
Andrew Whittaker (ex-officio)

Project Technical Committee

Richard Wittkop (Project Director)
Moffatt & Nichol
3780 Kilroy Airport Way, Suite 600
Long Beach, California 90806

Omar Jarradat
Moffatt & Nichol
3780 Kilroy Airport Way, Suite 600
Long Beach, California 90806

Gayle Johnson
Halcrow
500 12th Street, Suite 310
Oakland, California 94607

Geoffrey Martin
Consultant
17660 Newhope Street, Suite E
Fountain Valley, California 92708

Glenn Rix
Georgia Institute of Technology
School of Civil and Environmental Engineering
790 Atlantic Drive
Atlanta, Georgia 30332

Ian Robertson
University of Hawaii at Manoa
Department of Civil and Environmental Engineering
2540 Dole Street, Holmes Hall 383
Honolulu, Hawaii 96822

Project Manager

William L. Coulbourne
Applied Technology Council
19266 Coastal Highway
Unit 4, #19
Rehoboth Beach, Delaware 19971