

Seismic Design of Piers and Wharves

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and customary units



American Society of Civil Engineers

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STANDARDS

In 2006, the Board of Direction approved the revision to the ASCE Rules for Standards Committees to govern the writing and maintenance of standards developed by the Society. All such standards are developed by a consensus standards process managed by the Society's Codes and Standards Committee (CSC). The consensus process includes balloting by a balanced standards committee made up of Society members and nonmembers, balloting by the membership of the Society as a whole, and balloting by the public. All standards are updated or reaffirmed by the same process at intervals not exceeding five years.

The following standards have been issued:

ANSI/ASCE 1-82 N-725 Guideline for Design and Analysis of Nuclear Safety Related Earth Structures
ASCE/EWRI 2-06 Measurement of Oxygen Transfer in Clean Water
ANSI/ASCE 3-91 Standard for the Structural Design of Composite Slabs and ANSI/ASCE 9-91 Standard Practice for the Construction and Inspection of Composite Slabs
ASCE 4-98 Seismic Analysis of Safety-Related Nuclear Structures
Building Code Requirements for Masonry Structures (ACI 530-13/ASCE 5-13/TMS 402-13) and Specifications for Masonry Structures (ACI 530.1-13/ASCE 6-13/TMS 602-13)
ASCE/SEI 7-10 Minimum Design Loads for Buildings and Other Structures
SEI/ASCE 8-02 Standard Specification for the Design of Cold-Formed Stainless Steel Structural Members
ANSI/ASCE 9-91 listed with ASCE 3-91
ASCE 10-97 Design of Latticed Steel Transmission Structures
SEI/ASCE 11-99 Guideline for Structural Condition Assessment of Existing Buildings
ASCE/EWRI 12-13 Standard Guidelines for the Design of Urban Subsurface Drainage
ASCE/EWRI 13-13 Standard Guidelines for the Installation of Urban Subsurface Drainage
ASCE/EWRI 14-13 Standard Guidelines for the Operation and Maintenance of Urban Subsurface Drainage
ASCE 15-98 Standard Practice for Direct Design of Buried Precast Concrete Pipe Using Standard Installations (SIDD)
ASCE 16-95 Standard for Load Resistance Factor Design (LRFD) of Engineered Wood Construction
ASCE 17-96 Air-Supported Structures
ASCE 18-96 Standard Guidelines for In-Process Oxygen Transfer Testing
ASCE 19-10 Structural Applications of Steel Cables for Buildings
ASCE 20-96 Standard Guidelines for the Design and Installation of Pile Foundations
ANSI/ASCE/T&DI 21-13 Automated People Mover Standards
SEI/ASCE 23-97 Specification for Structural Steel Beams with Web Openings
ASCE/SEI 24-05 Flood Resistant Design and Construction
ASCE/SEI 25-06 Earthquake-Actuated Automatic Gas Shutoff Devices
ASCE 26-97 Standard Practice for Design of Buried Precast Concrete Box Sections
ASCE 27-00 Standard Practice for Direct Design of Precast Concrete Pipe for Jacking in Trenchless Construction

ASCE 28-00 Standard Practice for Direct Design of Precast Concrete Box Sections for Jacking in Trenchless Construction
ASCE/SEI/SFPE 29-05 Standard Calculation Methods for Structural Fire Protection
SEI/ASCE 30-14 Guideline for Condition Assessment of the Building Envelope
SEI/ASCE 31-03 Seismic Evaluation of Existing Buildings
SEI/ASCE 32-01 Design and Construction of Frost-Protected Shallow Foundations
EWRI/ASCE 33-09 Comprehensive Transboundary International Water Quality Management Agreement
EWRI/ASCE 34-01 Standard Guidelines for Artificial Recharge of Ground Water
EWRI/ASCE 35-01 Guidelines for Quality Assurance of Installed Fine-Pore Aeration Equipment
CI/ASCE 36-01 Standard Construction Guidelines for Microtunneling
SEI/ASCE 37-02 Design Loads on Structures during Construction
CI/ASCE 38-02 Standard Guideline for the Collection and Depiction of Existing Subsurface Utility Data
EWRI/ASCE 39-03 Standard Practice for the Design and Operation of Hail Suppression Projects
ASCE/EWRI 40-03 Regulated Riparian Model Water Code
ASCE/SEI 41-06 Seismic Rehabilitation of Existing Buildings
ASCE/EWRI 42-04 Standard Practice for the Design and Operation of Precipitation Enhancement Projects
ASCE/SEI 43-05 Seismic Design Criteria for Structures, Systems, and Components in Nuclear Facilities
ASCE/EWRI 44-05 Standard Practice for the Design and Operation of Supercooled Fog Dispersal Projects
ASCE/EWRI 45-05 Standard Guidelines for the Design of Urban Stormwater Systems
ASCE/EWRI 46-05 Standard Guidelines for the Installation of Urban Stormwater Systems
ASCE/EWRI 47-05 Standard Guidelines for the Operation and Maintenance of Urban Stormwater Systems
ASCE/SEI 48-11 Design of Steel Transmission Pole Structures
ASCE/SEI 49-12 Wind Tunnel Testing for Buildings and Other Structures
ASCE/EWRI 50-08 Standard Guideline for Fitting Saturated Hydraulic Conductivity Using Probability Density Functions
ASCE/EWRI 51-08 Standard Guideline for Calculating the Effective Saturated Hydraulic Conductivity
ASCE/SEI 52-10 Design of Fiberglass-Reinforced Plastic (FRP) Stacks
ASCE/G-I 53-10 Compaction Grouting Consensus Guide
ASCE/EWRI 54-10 Standard Guideline for Geostatistical Estimation and Block-Averaging of Homogeneous and Isotropic Saturated Hydraulic Conductivity
ASCE/SEI 55-10 Tensile Membrane Structures
ANSI/ASCE/EWRI 56-10 Guidelines for the Physical Security of Water Utilities
ANSI/ASCE/EWRI 57-10 Guidelines for the Physical Security of Wastewater/Stormwater Utilities
ASCE/T&DI/ICPI 58-10 Structural Design of Interlocking Concrete Pavement for Municipal Streets and Roadways
ASCE/SEI 59-11 Blast Protection of Buildings
ASCE/EWRI 60-12 Guidelines for Development of Effective Water Sharing Agreement
ASCE/COPRI 61-14 Seismic Design of Piers and Wharves



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PREFACE

This is the first edition of this standard. 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. It is intended to reflect the current practice of seismic design of piers and wharves, aspects of which differ considerably from conventional design of buildings or building-like structures. For example, load combinations for piers and wharves are unique and include berthing and mooring loads that may govern the lateral load design in low seismic regions. Geotechnical issues are a prime concern for these types of structures, with pile foundations often penetrating through weak soil layers. Liquefaction, lateral spreading, and ground failures affecting slopes and earth structures are all primary considerations for the analysis and design of piers and wharves. It is often not practical to eliminate these hazards; rather, structures are designed to accommodate permanent ground deformations within specified performance requirements.

This standard contains displacement-based design methods that have become widely used in the industry for more than a decade and have been supported by a number of industry-sponsored testing programs.

This standard is intended to benefit designers and owners by providing a method for considering the economic importance of a structure in addition to meeting life safety requirements. It is also intended to benefit building officials who do not routinely deal with waterfront structures.

Compliance with this standard should not be considered a guarantee that the desired performance goals will be achieved. Variation between the desired performance levels and actual performance should be expected because of the inherent uncertainties in prediction of seismic ground motion, liquefaction, and other seismic effects and because of the inherent uncertainties in levels of workmanship, material strengths, levels of maintenance, structure condition, and other effects.

This standard has been prepared in accordance with recognized engineering principles and should not be used without the user's competent knowledge for a given application. The publication of this standard by ASCE is not intended to warrant that the information contained herein is suitable for any general or specific use, and ASCE takes no position respecting the validity of patent rights. The user is advised that the determination of patent rights or risk of infringement is entirely his or her own responsibility.

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Carlos E. Ospina, Ph.D., P.E., M.ASCE
Patrick M. Smith, Ph.D., P.E., M.ASCE

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CHAPTER 1

GENERAL

1.1 INTRODUCTION

1.1.1 Purpose The purpose of this standard is to provide minimum requirements for the seismic design of pile-supported piers and wharves.

1.1.2 Scope and Limitations This standard addresses the seismic design of new piers and wharves supported on concrete or steel piles.

This standard does not address the following:

- Structures that are subject to different operational or special regulatory requirements such as offshore platforms or liquefied natural gas (LNG) terminals, or
- Piers or wharves that are accessible to the general public.

This standard does not address all of the aspects of design of sheet pile walls, cellular walls, or similar structures that may be components of a pile-supported pier or wharf.

This standard is intended for use by qualified geotechnical and structural engineers with experience in the marine industry and officials engaged in preparing and administering local codes. This standard is not intended to replace sound engineering knowledge, experience, and judgment.

1.2 DEFINITIONS

The following definitions apply to the provisions of Chapter 1:

PIERS: In this standard, piers are pile-supported platform structures that extend outward perpendicular, or almost perpen-

dicular, from the shore into a body of water and allow water to flow underneath.

WHARVES: In this standard, wharves are pile-supported platform structures that are constructed parallel or almost parallel to the shore.

1.3 DOCUMENT LAYOUT

Chapter 1 presents the purpose, scope, limitations, and layout of this standard. Chapter 2 defines seismic design classifications, performance requirements, and seismic hazard levels. Chapter 3 provides the required steps and criteria for performing a seismic design of a pier or wharf. Chapter 4 introduces the geotechnical considerations required for a seismic design. Chapter 5 presents criteria for force-based design, and Chapter 6 presents those for displacement-based design. Chapter 7 provides provisions for design and detailing. Chapter 8 deals with design considerations for ancillary and nonstructural components. A commentary is provided for each chapter.

1.4 UNITS

Both English and SI units are presented in this document unless noted otherwise.

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CHAPTER 2

SEISMIC PERFORMANCE REQUIREMENTS

2.1 INTRODUCTION

2.1.1 Scope This chapter defines design classifications, performance requirements, and seismic hazard levels.

2.2 DESIGN CLASSIFICATIONS

Structures designed in accordance with this standard shall be assigned, by the authority having jurisdiction, one of the design classifications presented in Sections 2.2.1 through 2.2.3. An owner is permitted to assign a structure a more severe design classification.

2.2.1 High Structures shall be assigned a design classification of “high” if they are essential to the region’s economy or post-event recovery and they require a level of seismic performance beyond life safety protection.

2.2.2 Moderate Structures shall be assigned a design classification of “moderate” if they are of secondary importance to the regional economy and not essential to post-event recovery but they require a level of seismic performance beyond life safety protection.

2.2.3 Low Structures not assigned a design classification of “high” or “moderate” shall be assigned a design classification of “low.”

2.3 DESIGN CRITERIA

Minimum design criteria for structures in each design classification are shown in Table 2-1.

2.4 PERFORMANCE LEVELS

The required structural response for each performance level considered in this standard is presented in Sections 2.4.1 through 2.4.3.

2.4.1 Life Safety Protection A structure shall be classified as providing “life safety protection” when (a) the post-earthquake damage state is such that the structure continues to support gravity loads, (b) damage that does occur does not prevent egress, and (c) there is no loss of containment of materials in a manner that would pose a public hazard.

2.4.2 Controlled and Repairable Damage A structure shall be classified as having achieved “controlled and repairable damage” when (a) the structure responds in a controlled and ductile manner, experiencing limited inelastic deformations at locations where repair is possible; (b) the required repairs result in a loss of serviceability for no more than several months; and (c) there is no loss of containment of materials in a manner that would pose a public hazard.

2.4.3 Minimal Damage A structure shall be classified as having achieved “minimal damage” when (a) it exhibits near-elastic structural response with minor or no residual deformation, (b) there is no loss of serviceability of the structure, and (c) there is no loss of containment of materials in a manner that would pose a public hazard.

2.5 SEISMIC HAZARDS TO BE CONSIDERED

In addition to inertial loads caused by ground shaking, the potential for and extent of other seismic hazards that could occur at pier and wharf sites, including ground deformations associated with liquefaction and cyclic degradation of weak soils, shall be evaluated for each of the aboveground motion levels applicable in accordance with Table 2-1. Kinematic loading caused by ground deformations adjacent to piles and effects of fault rupture shall be considered in accordance with Chapter 4.

2.6 REFERENCES

American Society of Civil Engineers (ASCE). (2005). *Minimum design loads for buildings and other structures*, ASCE 7-05, American Society of Civil Engineers, Reston, VA.

Table 2-1. Minimum Seismic Hazard and Performance Requirements

	Seismic hazard level and performance level					
	Operating level earthquake (OLE)		Contingency level earthquake (CLE)		Design earthquake (DE)	
	Ground motion probability of exceedance	Performance level	Ground motion probability of exceedance	Performance level	Seismic hazard level	Performance level
High	50% in 50 years (72-year return period)	Minimal damage	10% in 50 years (475-year return period)	Controlled and repairable damage	Design earthquake per ASCE 7 (2005)	Life safety protection
Moderate	N/A	N/A	20% in 50 years (224-year return period)	Controlled and repairable damage	Design earthquake per ASCE 7 (2005)	Life safety protection
Low	N/A	N/A	N/A	N/A	Design earthquake per ASCE 7 (2005)	Life safety protection

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CHAPTER 3 DESIGN APPROACH

3.1 INTRODUCTION

3.1.1 Scope This chapter defines the steps required for the seismic design of pier and wharf structures.

3.2 DEFINITIONS

The following definitions apply to the provisions of Chapter 3:

CONTAINER WHARVES: Structures located in waterfront areas used for berthing and mooring of ships and for the transfer of containers from the ship to the land by means of movable cranes supported on a wharf structure.

DISPLACEMENT-BASED DESIGN: A design approach whereby an anticipated displacement of the structure is determined for design earthquake motions and the various elements are designed to perform to certain standards under the imposed displacements.

DYNAMIC MAGNIFICATION FACTOR (DMF): A factor to account for torsional response and simultaneous longitudinal and transverse excitations.

FORCE-BASED DESIGN: A design approach whereby structures and all components along the load path of the structure are designed to withstand prescribed design loads while remaining within prescribed stress limits.

STRAIN PENETRATION: The inelastic strain that occurs in the dowel on each side of the interface and provides additional flexibility at the connection.

SUBSTITUTE STRUCTURE METHOD: A nonlinear static demand analysis methodology that employs an iterative process using design spectra with effective secant stiffness at the response displacement to determine the displacement demand for each iteration.

3.3 SYMBOLS AND NOTATION

The following symbols and notation apply only to the provisions of Chapter 3:

A_c	Area of concrete test core as defined in ACI 318 (2008)
D	Dead loads, including all permanent fixed equipment and structures
D_p	Pile diameter
e	Eccentricity between center of mass and center of rigidity
E	Horizontal earthquake load
E_c	Modulus of elasticity of concrete
E_{h1}	Earthquake load in one primary orthogonal direction
E_{h2}	Earthquake load in second primary orthogonal direction
f'_{ce}	Expected strength of unconfined concrete
G_c	Modulus of rigidity of concrete

H	Soil pressure loads (e.g., soil pressure on end walls, concrete cutoff walls, and steel sheet pile walls on pier or wharf type structures and/or piles)
I	Moment of inertia
I_{eff}	Effective moment of inertia
J	Polar moment of inertia
J_{eff}	Effective polar moment of inertia
L	Uniform live loads
L_1	Length of pier or wharf segment
PGA	Peak ground acceleration
R	Response modification factor
S_{DS}	Design, 5% damped, spectral response acceleration parameter at short periods (taken as 0.2 s)
β_1	Factor relating depth of equivalent rectangular compressive stress block to neutral axis depth as defined in ACI 318 (2008)
Δ_d	Total displacement demand
Δ_r	Transverse displacement demand
ϵ_c	Extreme concrete fiber compressive strain
ϵ_p	Strain in prestressing tendons
ϵ_s	Strain in steel pipe pile (tension or compression)
ϵ_{smd}	Strain at peak stress of dowel reinforcement
ρ_s	Volumetric ratio of confining steel
ϕ	Capacity reduction factor per ACI 318 (2008)

3.4 OVERALL APPROACH AND ORGANIZATION OF THIS STANDARD

The list following identifies tasks that are necessary to satisfy the seismic design requirements of this standard and the location in the book of the requirements applicable to those tasks. Note that the steps listed are limited to those required for seismic design, assuming that the structure has already been designed for static and nonseismic lateral service loads before design for seismic loads.

1. Definition of the design classification in Section 2.2.
2. Based on the design classification, determination of which performance levels need to be satisfied per Table 2-1, along with associated hazard levels.
3. Determination of the design method (displacement-based and/or force-based) in Section 3.5.
4. Definition of ground motions in Section 4.3.
5. Determination of soil-structure modeling parameters (p - y and t - z springs) in Sections 4.8 and 4.10.
6. Determination of other special geotechnical considerations or loads in Chapter 4.
7. Development of the structural model, including general modeling considerations of Section 3.7.

8. Calculation of structural demands in Section 3.6 and either Section 5.4 for force-based design methods or Section 6.8 for displacement-based design methods.
9. Calculation of structural capacity in either Chapter 5 for force-based design methods or Chapter 6 for displacement-based design methods.
10. Development of design connection details in Chapter 7.
11. Design of ancillary components in Chapter 8.

Alternate approaches based on sound engineering principles, testing, or other bases are permitted if determined to be acceptable by the authority having jurisdiction.

3.5 DETERMINATION OF ANALYSIS AND DESIGN METHOD

Displacement-based and force-based design methods are permitted by this standard. Analysis and design provisions for force-based design and displacement-based design are provided in Chapters 5 and 6, respectively. Detailing requirements are provided in Chapter 7.

The following list summarizes requirements for selection of the design method:

- a. Displacement-based design is permitted for all design classifications.
- b. Force-based design is permitted for “low” design classifications.
- c. Force-based design is permitted for all design classifications when $S_{DS} < 0.33$ (per Section 11.4 of ASCE 7 [2005]).
- d. Force-based design is permitted for all design classifications where it can be demonstrated that the capacity of all primary structural members exceeds the elastic earthquake demand force when using a value of $R = 1$.
- e. Force-based design is permitted for the design of equipment and piping supports.

3.6 LOAD COMBINATIONS

3.6.1 Load Combinations Including Seismic Each component of the structure shall be analyzed for the load combination of Eq. (3-1) (load and resistance factor design [LRFD]). Seismic loads need not be combined with mooring and berthing loads, or environmental loads such as wind, wave, and current.

$$(1.0 \pm 0.5 \text{ PGA})D + 0.1L + 1.0H + 1.0E \quad (3-1)$$

where D = dead loads, including all permanent fixed equipment and structures, and other items expected to be present for more than 50% of the time;

L = uniform live loads;

H = soil pressure loads (e.g., soil pressure on end walls, concrete cutoff walls, steel sheet pile walls on pier or wharf type structures, and/or piles);

E = horizontal earthquake loads as defined in Section 3.6.2; and

PGA = peak ground acceleration.

Exception: The load factor on H shall be set equal to zero if the structural action caused by H counteracts that caused by E . Where lateral earth pressure provides resistance to structural actions from other nonseismic forces, it shall not be included in H but shall be included in the design resistance.

Vertical earthquake loads are accounted for with the multiplier of 0.5 PGA in Eq. (3-1). Loads shall be applied in the direction that results in the most conservative response.

3.6.2 Orthogonal Loading and Torsional Effects Effects of simultaneous seismic excitation in orthogonal horizontal directions shall be considered by using both Eqs. (3-2) and (3-3). Directions shall be combined to produce the largest demand on the structure.

$$E = \pm 1.0E_{h1} \pm 0.3E_{h2} \quad (3-2)$$

$$E = \pm 0.3E_{h1} \pm 1.0E_{h2} \quad (3-3)$$

where E_{h1} and E_{h2} are the earthquake actions in the principal horizontal directions.

Torsional effects shall be considered where there is a significant eccentricity between the center of gravity and the center of rigidity.

In lieu of Eqs. (3-2) and (3-3), it is permitted to address both orthogonal and torsional loads using a dynamic magnification factor (DMF). For piers and wharves with an aspect ratio (length to width) greater than 3, it is permitted to use Eq. (3-4):

$$\Delta_d = \Delta_r \sqrt{1 + (0.3(1 + 20e/L_t))^2} \quad (3-4)$$

where Δ_d = total displacement demand;

Δ_r = transverse displacement demand;

e = eccentricity between center of mass and center of rigidity; and

L_t = length of pier or wharf segment.

3.7 GENERAL MODELING CONSIDERATIONS

3.7.1 General The analytical model shall incorporate all significant components of the lateral-force-resisting system for the purpose of determining member forces and structure displacements resulting from the applied loads and any imposed displacements or P-delta effects, where required. The model shall include the stiffness and strength characteristics of elements that are significant to the distribution of forces and deformations in the structure and represent the spatial distribution of mass and stiffness throughout the structure. At a minimum, the model shall include the deck, the pile-to-deck connection, and the piles.

The following shall be considered in the analysis:

- a. Uncertainties in soil properties (incorporated into the analysis in accordance with Section 4.8.2);
- b. Torsional effects caused by differences in plan location of the center of mass (CM) and center of rigidity (CR);
- c. Cracked section properties; and
- d. Mass of ancillary components (in accordance with Chapter 8).

3.7.2 Section Properties Section properties, flexural rigidity E_cI , and torsional rigidity G_cJ , shall reflect the cracking that occurs before the yield limit state is reached. The effective moments of inertia, I_{eff} and J_{eff} shall be used to obtain realistic values for the structure's elastic period and seismic demands.

The effective moment of inertia, I_{eff} , shall be used for the structural model. I_{eff} shall be estimated by the secant slope of the moment–curvature curve between the origin and the point of first yield, as defined in Section 6.6.2.

3.7.3 Seismic Mass The mass considered for the dynamic analysis shall include the structural self-weight of the entire pier or wharf, permanently attached equipment, and 10% of the design uniform live load. Hydrodynamic mass associated with piles, where significant, shall be considered. For 24-in. (0.610-m) diameter piles or less, hydrodynamic mass may be ignored. The mass of container cranes shall be determined in accordance with Section 8.5.2 and distributed over the length of the wharf.

For pile-supported piers, wharves, and platforms where containers are stacked on the structure, the effects of the stacked containers on the seismic mass of the structure shall be considered.

3.8 OTHER DESIGN CONSIDERATIONS

3.8.1 Capacity Protection The deck shall be designed as a capacity-protected element, such that all inelastic deformations occur in piles rather than in the deck elements. Yielding of the pile-to-deck connection is permitted.

Exception: Strain penetration of the pile connection dowels in the deck is permitted.

3.9 STRAIN LIMITS

The maximum plastic hinge zone strains for the operating level earthquake (OLE), contingency level earthquake (CLE), and design earthquake (DE) performance levels (as defined in Table 2-1) shall be calculated per Section 6.6.3 using the moment-curvature methods described in conjunction with the plastic hinge lengths provided in Table 6-1 and compared to the strain limits provided in this section.

3.9.1 OLE Strain Limits Table 3-1 provides the maximum material strains allowed within the performance level of “minimal damage” as per Section 2.4.3.

3.9.2 CLE Strain Limits Table 3-2 provides the maximum material strains within the performance level of “controlled and repairable damage” as per Section 2.4.2.

3.9.3 DE Strain Limits Table 3-3 provides the maximum material strains allowed within the performance level of “life safety protection” as per Section 2.4.1.

Concrete piles are also subject to the DE stability check in Section 3.10. Steel piles are subject to the DE stability check in Section 3.10.

Pile connections at the top of a pile may have additional requirements and limitations depending on the type of connection, as per Chapter 7.

3.10 DESIGN EARTHQUAKE STABILITY CHECK

The DE stability check shall be performed per Section 6.6.7, with the following modifications. All pile-to-deck connections with a reinforcement strain greater than 0.06 shall be assumed to be pinned at the top for the DE stability check. In addition, the strain limits in Table 3-3 shall not be exceeded.

Table 3-2. Strain Limits for “Controlled and Repairable Damage” per Section 2.4.2

Pile type	Component	Hinge location		
		Top of pile	In ground	Deep in ground (>10D _p)
Solid concrete pile	Concrete	$\epsilon_c \leq 0.005 + 1.1\rho_s \leq 0.025$	$\epsilon_c \leq 0.005 + 1.1\rho_s \leq 0.008$	$\epsilon_c \leq 0.012$
	Reinforcing steel	$\epsilon_s \leq 0.6\epsilon_{smd} \leq 0.06$		
	Prestressing steel		$\epsilon_p \leq 0.025$	$\epsilon_p \leq 0.025$
Hollow concrete pile ^a	Concrete	$\epsilon_c \leq 0.006$	$\epsilon_c \leq 0.006$	$\epsilon_c \leq 0.006$
	Reinforcing steel	$\epsilon_s \leq 0.4\epsilon_{smd} \leq 0.04$		
	Prestressing steel		$\epsilon_p \leq 0.020$	$\epsilon_p \leq 0.025$
Steel pipe pile	Steel pipe		$\epsilon_s \leq 0.025^b$	$\epsilon_s \leq 0.035$
	Concrete	$\epsilon_c \leq 0.025$		
	Reinforcing steel	$\epsilon_s \leq 0.6\epsilon_{smd} \leq 0.06$		

^aIf the interior of the hollow pile is filled with concrete, all strain limits shall be the same as for solid piles.

^bIf the steel pipe pile is infilled with concrete, a value of 0.035 may be used.

Table 3-3. Strain Limits for “Life Safety Protection” per Section 2.4.1

Pile type	Component	Hinge location		
		Top of pile	In ground	Deep in ground (>10D _p)
Solid concrete pile	Concrete	No limit	$\epsilon_c \leq 0.005 + 1.1\rho_s \leq 0.012$	No limit
	Reinforcing steel	$\epsilon_s \leq 0.8\epsilon_{smd} \leq 0.08$		
	Prestressing steel		$\epsilon_p \leq 0.035$	$\epsilon_p \leq 0.050$
Hollow concrete pile ^a	Concrete	$\epsilon_c \leq 0.008$	$\epsilon_c \leq 0.008$	$\epsilon_c \leq 0.008$
	Reinforcing steel	$\epsilon_s \leq 0.6\epsilon_{smd} \leq 0.06$		
	Prestressing steel		$\epsilon_p \leq 0.025$	$\epsilon_p \leq 0.050$
Steel pipe pile	Steel pipe		$\epsilon_s \leq 0.035^b$	$\epsilon_s \leq 0.050$
	Concrete	No limit		
	Reinforcing steel	$\epsilon_s \leq 0.8\epsilon_{smd} \leq 0.08$		

^aIf the interior of the hollow pile is filled with concrete, all strain limits shall be the same as for solid piles.

^bIf the steel pipe pile is infilled with concrete, a value of 0.050 may be used.

Table 3-1. Strain Limits for “Minimal Damage” per Section 2.4.3

Pile type	Component	Hinge location		
		Top of pile	In ground	Deep in ground (>10D _p)
Solid concrete pile	Concrete	$\epsilon_c \leq 0.005$	$\epsilon_c \leq 0.005$	$\epsilon_c \leq 0.008$
	Reinforcing steel	$\epsilon_s \leq 0.015$		
	Prestressing steel		$\epsilon_p \leq 0.015$	$\epsilon_p \leq 0.015$
Hollow concrete pile ^a	Concrete	$\epsilon_c \leq 0.004$	$\epsilon_c \leq 0.004$	$\epsilon_c \leq 0.004$
	Reinforcing steel	$\epsilon_s \leq 0.015$		
	Prestressing steel		$\epsilon_p \leq 0.015$	$\epsilon_p \leq 0.015$
Steel pipe pile	Steel pipe		$\epsilon_s \leq 0.010$	$\epsilon_s \leq 0.010$
	Concrete	$\epsilon_c \leq 0.010$		
	Reinforcing steel	$\epsilon_s \leq 0.015$		

^aIf the interior of the hollow pile is filled with concrete, all strain limits shall be the same as for solid piles.

3.11 BATTER PILES

The use of batter piles is permitted provided that the structure satisfies all force, displacement, and compatibility requirements caused by seismic loading. In addition, the following restrictions apply:

- a. Yielding of the tension batter pile to deck connection is permitted in the CLE and DE provided the hysteretic performance of the connection is considered. If sleeved dowels are used as a fusing mechanism for either solid concrete piles or steel pipe piles with a plug connection, the permitted elongation in the dowels shall be calculated by multiplying the sleeve length times 50% of the strain limits in Section 3.9.
- b. The use of pile slip as a yielding mechanism for the compression pile is prohibited in the OLE.
- c. Pile slip in the CLE and DE is permitted as a fusing mechanism provided that the lower-bound geotechnical capacity is used to calculate displacement demand and the upper-bound geotechnical capacity is used to calculate force demand. The effects of permanent deck displacement caused by pile slip shall be considered.
- d. The deck shall be capacity protected and designed to resist 1.25 times the DE displacements associated with the lower-bound geotechnical capacity of the piles and 1.25 times the DE forces associated with the upper-bound geotechnical capacity of the piles.
- e. The use of pile buckling as a yielding mechanism for the compression pile is prohibited.

3.12 SEISMIC ISOLATION SYSTEMS

The use of seismic isolation systems is permitted provided that the isolation mechanism is suitable for use in a marine environment. The isolation mechanism shall meet the testing requirements of Section 17.8 of ASCE 7 (2005) and the following requirements:

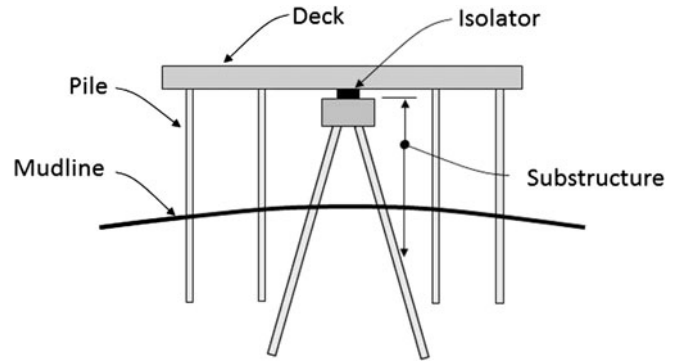


Fig. 3-1. Example of a pier with seismic isolation

3.12.1 Capacity Protection of Deck and Substructure The deck and substructure as shown in Fig. 3-1 shall be designed as capacity-protected elements using the calculated peak shear demand from the isolator(s) multiplied by 1.3 in the DE.

3.12.2 Effective System Damping The effective system damping of the structure shall be calculated based on the energy dissipated by the different structural elements. The effective damping of the substructure shall be taken as 5%.

3.12.3 Analysis Methods The substitute structure and the response spectrum methods of analysis are permitted provided that the total effective damping does not exceed 30%. If the total effective damping exceeds 30%, a nonlinear time history analysis is required.

3.13 REFERENCES

- American Concrete Institute (ACI). (2008). *Building code requirements for structural concrete* (ACI 318-08) and commentary (ACI 318R-08), American Concrete Institute, Farmington Hills, MI.
- American Society of Civil Engineers (ASCE). (2005). *Minimum design loads for buildings and other structures*, ASCE 7-05, American Society of Civil Engineers, Reston, VA.

CHAPTER 4

GEOTECHNICAL CONSIDERATIONS AND SOIL–STRUCTURE INTERACTION

4.1 INTRODUCTION

4.1.1 Scope This chapter describes the minimum requirements for evaluations of ground motions corresponding to the seismic hazard levels identified in Table 2-1, geotechnical hazards associated with these ground motions, and soil–structure interaction.

4.2 DEFINITIONS

The following definitions apply to the provisions of Chapter 4:

EQUIVALENT PILE DEPTH TO FIXITY: Depth at which a pile may be modeled as fixed with no lateral soil supports such that the same top of pile displacement results as that given by an individual lateral analysis for a given lateral load at the top of the pile.

KINEMATIC LOADING: The load imposed on a structure by movement of the surrounding soil.

PEAK GROUND ACCELERATION: Maximum ground surface acceleration for a level-ground condition.

4.3 EARTHQUAKE GROUND MOTIONS

Earthquake ground motions shall be based on seismic hazard levels defined in Table 2-1.

4.3.1 Design Earthquake (DE) Ground Motion Parameters DE ground motion parameters shall be developed using the procedures provided in ASCE 7 (2005).

4.3.2 Operating Level Earthquake (OLE) and Contingency Level Earthquake (CLE) Ground Motion Parameters OLE and CLE earthquake ground motion parameters shall be developed using one of the following probabilistic seismic hazard analysis (PSHA) methods:

- U.S. Geological Survey (USGS) published data, adjusted for local faulting and site conditions using the procedures provided in Chapter 11 of ASCE 7 (2005), or
- Site-specific PSHA, appropriate for local seismic sources and site conditions.

As a minimum, ground motion parameters shall include horizontal peak ground acceleration (PGA), acceleration response spectrum, and corresponding most probable magnitude of the controlling earthquake(s).

For Site Class F, as defined in ASCE 7 (2005), site-specific response analysis shall be performed.

Ground motion parameters shall be developed for damping levels as required for structural design.

If time history analyses are performed, spectrum-compatible orthogonal sets of time histories shall be developed. As a

minimum, three sets of time histories shall be developed. The minimum response parameters used in analysis and design shall be taken as the envelope of the response parameters when three sets of time histories are used or the average of the response parameters when seven or more sets of time histories are used.

4.4 LIQUEFACTION AND CYCLIC DEGRADATION POTENTIAL

The potential for liquefaction and cyclic degradation of soils in close enough proximity to affect waterfront slopes, embankments, and structures shall be evaluated. The effects of potential cyclic strength loss in foundation soils shall be addressed during the seismic design of the structure.

4.5 SLOPE STABILITY AND LATERAL GROUND DEFORMATION

4.5.1 Live Load Surcharge Live load surcharge shall be considered in slope stability and ground deformation analyses. The live load surcharge values for different analyses identified in Sections 4.5.2 through 4.5.5 shall be based on project-specific loading information.

4.5.2 Static Slope Stability The long-term static factor of safety of the slope or embankment shall not be less than 1.5.

4.5.3 Postearthquake Static Slope Stability The static factor of safety immediately after a DE event shall not be less than 1.1 when appropriate postearthquake shear strengths of soils are used.

4.5.4 Pseudostatic Slope Stability Pseudostatic slope stability analysis shall be performed using a horizontal seismic coefficient. The seismic coefficient shall be equal to one-half of the PGA. Shear strength values appropriate for the levels of earthquake loading shall be used in the analysis. If the estimated factor of safety is greater than or equal to 1.1, then no further evaluation for deformations or kinematic analysis as outlined in Sections 4.5.5 and 4.7.2 is required.

4.5.5 Lateral Ground Deformations If the factor of safety estimated in Section 4.5.4 is less than 1.1, the lateral ground deformations adjacent to the structure, including foundation soils, the slope or embankment, and backland areas affecting the structures shall be evaluated without considering the presence of the structure foundation system. Soil shear strength values appropriate for the levels of earthquake loading shall be used in the evaluation.

The foundation shall be evaluated for these lateral deformations. If the results indicate that the structural performance is

unacceptable, further kinematic analysis, as outlined in Section 4.7.2, is required.

4.6 FAULT RUPTURE

The effects of ground surface rupture caused by faults identified in the seismic hazard evaluation shall be addressed. Fault displacements, both in horizontal and vertical directions, shall be estimated.

4.7 LATERAL LOADINGS FOR SOIL-STRUCTURE INTERACTION

Two loading conditions for the foundation shall be considered for seismic design: (1) inertial loading and (2) kinematic loading caused by lateral ground deformations. Simultaneous application of these loading conditions shall be considered, taking into account the phasing and locations of these loads on foundation elements. The foundation shall be designed such that the structural performance is acceptable when subjected to both inertial and kinematic loadings.

4.7.1 Inertial Loading Loading associated with the inertial response of the seismic mass, as outlined in Section 3.7.3, shall be considered in the analysis. The lateral soil springs developed following the requirements of Section 4.8 shall be used in the inertial loading analyses.

4.7.2 Kinematic Loading Kinematic loading from permanent lateral ground deformation on the foundation shall be evaluated. It is permitted to include the beneficial effects of piles on lateral ground deformations.

4.8 SOIL-PILE BEHAVIOR UNDER LATERAL LOADING

4.8.1 Lateral Soil Springs For design of the piles under loading associated with the inertial response of the superstructure, level-ground inelastic lateral soil (p - y) springs shall be developed and used to represent soil resistance in the analysis.

Exception: The use of an equivalent pile depth to fixity is permitted to calculate displacements as discussed in Sections 5.5.3 and 6.6.6.

4.8.2 Upper- and Lower-Bound Springs To account for the uncertainties associated with the characterization of soil stiffness as well as the inherent variability of the soils, both upper-bound

and lower-bound spring values (i.e., p - y relationships) shall be developed for use in the inertial response analyses of the structure.

4.9 SEISMICALLY INDUCED SETTLEMENT

Seismically induced settlement shall be evaluated for use in the design. Seismically induced settlement shall be estimated based on volumetric strains in both liquefied saturated soils and non-liquefied dry and unsaturated soils.

4.10 AXIAL PILE PERFORMANCE

4.10.1 Axial Pile Capacity Axial pile capacities shall be evaluated for compression and tension. Geotechnical pile capacity evaluation under axial loading shall include the effects of pile capacity reduction caused by downdrag forces induced by seismically induced settlement. The minimum factor of safety shall be greater than 1.0 when seismically induced downdrag loads are combined with the seismic structural loads on the pile.

4.10.2 Axial Pile Springs Axial pile performance evaluations shall include axial soil (t - z) springs for use in the analyses. The springs shall be developed either at the top of the pile or along the piles. Use of linear or nonlinear springs is permitted.

Exception: The use of an equivalent pile depth to fixity is permitted as discussed in Sections 5.5.3 and 6.6.6.

4.10.3 Upper- and Lower-Bound Springs To account for the uncertainties associated with the characterization of soil resistance along a pile and the development of t - z springs, both upper-bound and lower-bound spring values shall be developed.

4.11 EARTH PRESSURES

4.11.1 Seismic Earth Pressures The effect of earth pressures on pier and wharf structures resulting from seismic loading of backland soils shall be considered in addition to static pressures. The earth pressures shall include the effect of excess pore pressure generation in the backland soils.

4.12 REFERENCES

American Society of Civil Engineers (ASCE). (2005). *Minimum design loads for buildings and other structures*, ASCE 7-05, American Society of Civil Engineers, Reston, VA.

CHAPTER 5 FORCE-BASED DESIGN

5.1 INTRODUCTION

5.1.1 Scope Chapter 5 presents criteria for force-based design, which largely follows the requirements of ASCE 7 (2005).

5.2 DEFINITIONS

The following definitions apply only to the provisions of Chapter 5:

APPROVED: Acceptable to the authority having jurisdiction.

EQUIVALENT PILE DEPTH TO FIXITY: Depth at which a pile may be modeled as fixed with no lateral soil supports such that the same top of pile displacement results as that given by an individual lateral analysis for a given lateral load at the top of the pile.

5.3 SYMBOLS AND NOTATION

The following symbols and notation apply only to the provisions of Chapter 5:

A_{ch}	Confined core area of pile defined by outside diameter of reinforcement in in. ² (m ²)
A_g	Gross cross-sectional area in in. ² (m ²)
C_d	Deflection amplification factor
C_s	Seismic response coefficient
E	Modulus of elasticity in kip/in. ² (N/m ²)
E_s	Modulus of elasticity of steel in kip/in. ² (N/m ²)
E_c	Modulus of elasticity of concrete in kip/in. ² (N/m ²)
f'_c	Minimum 28-day strength of concrete in kip/in. ² (N/m ²)
f_{yh}	Nominal yield strength of the spiral reinforcement in kip/in. ² (N/m ²)
I	Importance factor (in accordance with Section 11.5.1 of ASCE 7 (2005))
I_c	Gross moment of inertia of the concrete section in in. ⁴ (m ⁴)
I_{eff}	Effective moment of inertia in in. ⁴ (m ⁴)
I_g	Gross moment of inertia in in. ⁴ (m ⁴)
I_s	Gross moment of inertia of the steel section in in. ⁴ (m ⁴)
L	Length between plastic hinges in in. (m), taken as the distance between the bottom of the deck to the center of the in-ground hinge
M_o	Overstrength moment capacity in kip-in. (N-m)
$M_{o,top}$	Overstrength moment capacity of the connection in kip-in. (N-m)
$M_{o,bot}$	Overstrength moment capacity of the in-ground hinge in kip-in. (N-m)
M_p	Plastic moment capacity in kip-in. (N-m)
N	Axial load in kips (N) (compression positive)
P	Factored axial load on the pile in kips (N)

R	Response modification factor
T	Fundamental period of structure (s)
V	Seismic base shear in kips (N)
V_{po}	Overstrength shear force in kips (N)
W	Total seismic weight in kips (N)
ρ_s	Volumetric ratio of transverse steel

5.4 FORCE-BASED DEMAND ANALYSIS

The following force-based demand analyses are permitted.

5.4.1 Equivalent Lateral Force Analysis Where an equivalent lateral force analysis is performed, the requirements of ASCE 7 (2005) shall be satisfied except as modified herein. The requirements described in Sections 12.8.1.2, 12.8.1.3, 12.8.2.1, 12.8.3, and 12.8.4.3 of ASCE 7 (2005) shall not be applied for pile-supported piers and wharves addressed in this standard.

5.4.1.1 Seismic Base Shear The seismic base shear shall be determined in accordance with Section 12.8.1 of ASCE 7 (2005), modified as follows:

$$V = C_s W \quad (5-1)$$

where V = seismic base shear;

C_s = seismic response coefficient determined in accordance with Section 5.4.1.2; and

W = total seismic weight (dead load and applicable portion of other loads as required in Section 3.6).

5.4.1.2 Calculation of Seismic Response Coefficient The seismic response coefficient C_s shall be determined according to Section 12.8.1.1 of ASCE 7 (2005). However, design coefficients shall be chosen from Table 5-1 of this standard. The importance factor, I , shall be determined in accordance with Section 11.5.1 of ASCE 7 (2005).

5.4.1.3 Fundamental Structural Period The fundamental period of the structure, T , for any given direction shall be determined in accordance with Section 12.8.2 (except Section 12.8.2.1) of ASCE 7 (2005) using an appropriate analytical model. The effects of soil-structure interaction shall be considered in accordance with Section 4.8 of this standard.

5.4.1.4 Design Displacement Determination The design displacements shall be determined by multiplying the calculated displacements by the deflection amplification factor C_d given in Table 5-1. Drift limits specified in ASCE 7 (2005) need not be considered for structural design purposes but may be required for design of ancillary components, as described in Chapter 8.

5.4.2 Modal Response Spectrum Analysis Where a modal response spectrum analysis is performed, the requirements of

Table 5-1. Design Coefficients for Various Elements

Ductile element	<i>R</i>	<i>C_d</i>
Solid prestressed concrete piles	2	2
Steel pipe piles	2	2
Connections not meeting Chapter 7 provisions	1	1
Batter piles	1	1
Other tested connections	To be determined by experimental testing and/or finite element modeling and must be approved by the authority having jurisdiction	To be determined by experimental testing and/or finite element modeling and must be approved by the authority having jurisdiction

Table 5-2. Elastic Section Properties for Pier and Wharf Components

Pier or wharf component	<i>EI_{eff}</i> /(<i>EI_g</i>)
Reinforced concrete pile	$0.3 + N/(f'_c A_g)$
Pile/deck dowel connection ^a	$0.3 + N/(f'_c A_g)$
Prestressed pile ^a	$0.6 < EI_{eff}/(EI_g) < 0.75$
Steel pile	1.0
Concrete pile with steel casing	$(E_s I_s + 0.25 E_c I_c)/(E_s I_s + E_c I_c)$
Deck	0.5

^aThe pile/deck connection and prestressed pile may also be approximated as one member with an average stiffness of $EI_{eff}/EI_g = 0.42$.

Section 12.9 of ASCE 7 (2005) shall be satisfied, except as modified herein. The response modification factor *R* and the deflection amplification factor *C_d* shall be taken from Table 5-1. The importance factor, *I*, shall be determined in accordance with Section 11.5.1 of ASCE 7 (2005).

5.5 MODELING

5.5.1 Section Properties It is permitted to use section properties given in Table 5-2 for force-based design.

5.5.2 P-delta and Slenderness Effects P-delta and slenderness effects shall be considered.

5.5.3 Pile Stiffness The use of an equivalent pile depth to fixity is permitted provided that the pile length is suitably adjusted for the unsupported length above the soil surface (i.e., above the mudline, sloping rock-fill armor, or embankment) per Section 6.6.6.

5.6 FORCE-BASED CAPACITY ANALYSIS

5.6.1 Applicable Provisions The demand obtained in Section 5.4 shall not exceed the capacity calculated in accordance with Section 5.6. The capacity of the member to resist a given action shall be the nominal capacity as determined from the applicable nationally recognized code. Strength reduction factors (i.e., ϕ factors) shall be in accordance with the applicable nationally recognized codes.

5.6.2 Shear Strength of Concrete Piles Shear strength of concrete piles shall be determined in accordance with ACI 318 (2008).

5.6.3 Confinement for Prestressed Piles In the ductile region of the piling, volumetric spiral ratio ρ_s shall not be less than

$$\rho_s = 0.25 \left(\frac{A_g}{A_{ch}} - 1 \right) \frac{f'_c}{f_{yh}} \left(0.5 + 1.4 \frac{P}{f'_c A_g} \right) \quad (5-2)$$

$$\rho_s \geq 0.12 \frac{f'_c}{f_{yh}} \left(0.5 + 1.4 \frac{P}{f'_c A_g} \right)$$

where *A_g* = gross cross-sectional area of the piling,

A_{ch} = confined core area,

f'_c = 28-day compressive strength of the concrete,

f_{yh} = nominal yield strength of the spiral,

P = factored axial load on the pile, and

ρ_s need not be taken as larger than 0.02.

The length of the ductile region requiring this spiral ratio shall be measured three pile diameters downward from the bottom of the pile deck and shall be taken as three pile diameters either side of the point of maximum moment below ground, including cutoff allowance. Piles that are expected to respond elastically to the design earthquake (DE) are permitted to be designed for a spiral ratio that is 50% of that obtained from Eq. (5-2). However, ρ_s shall always be at least 0.005 over the entire length of all piles.

At least one-half the spiral ratio required in the ductile region shall be provided along the rest of the pile length.

In the pile's ductile region, the maximum center-to-center spacing of the spirals shall be taken as the smaller of one-fifth the least pile dimension, six times the longitudinal strand diameter, or 8 in. Spiral shall be developed per Section 7.5.2.

5.6.4 Shear Strength of Steel Pipe Piles Shear strength of steel pipe piles shall be determined in accordance with ANSI/AISC (2010), including the effects of corrosion.

5.6.5 Overstrength Shear Force Shear design shall consider the overstrength capacity of the pile-to-deck connection. The overstrength moment demand, *M_o*, shall be calculated from moment curvature analysis using expected material properties as defined in Section 6.5.1.

The overstrength shear force shall be determined as follows unless indicated otherwise by a detailed inelastic soil-pile interaction analysis:

$$V_{po} = \frac{M_{o,top} + M_{o,bot}}{L} \quad (5-3)$$

where *M_{o,top}* = overstrength moment capacity of the connection,
M_{o,bot} = overstrength moment capacity of the in-ground hinge, and

L = length between plastic hinges, taken as the distance between the bottom of the deck to the center of the in-ground hinge.

5.6.5.1 Capacity Protection of the Deck All pile-to-deck connections shall be strong deck-weak pile connections. The deck shall be capacity protected by designing and detailing the deck to resist the design plastic hinge mechanism based on expected material properties defined in Section 6.5 times an overstrength factor of 1.0 for all structures.