The Port of Los Angeles
Code for Seismic Design, Upgrade and Repair of Container Wharves

City of Los Angeles Harbor Department

May 2010

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FOREWORD

In 2004, the City of Los Angeles Harbor Depart, also known as the Port of Los Angeles (POLA), adopted "The Port of Los Angeles Code for Seismic Design, Upgrade and Repair of Container Wharves" dated May 18, 2004 as the seismic code for container wharf structures at the Port of Los Angeles. That document is referred to as "POLA Seismic Code 2004".

Since 2004, POLA has endeavored to improve and update the POLA Seismic Code 2004. This endeavor has resulted in an updated document for the seismic code for marginal container wharf structures. Upon adoption by the City of Los Angeles Harbor Department, the updated document will come into effect and supersede the POLA Seismic Code 2004. The updated document will be referred to as "POLA Seismic Code 2010".

POLA Seismic Code 2010 provides updates and revisions to the POLA Seismic Code 2004. The POLA Seismic Code 2004 has been revised and reformatted to include changes in the seismic design requirements and to address review comments from the public during an industry-wide workshop held in 2005 cosponsored by POLA and the American Society of Civil Engineers (ASCE). The requirements in the POLA Seismic Code 2004 have been updated to reflect the conclusions of experimental research and technical studies conducted since its publication. Other additional revisions were made in a continued effort to improve the code requirements for marginal container wharf structures at the Port of Los Angeles.

Upon adoption of the POLA Seismic Code 2010, any variances and modifications not in compliance with the POLA Seismic Code 2010 will require explicit approval by POLA.

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STANDARDS AND GUIDELINES

Marginal container wharves analysis and design shall comply with the provisions of POLA Seismic Code 2010 and the following documents as applicable. The provisions of POLA Seismic Code 2010 shall supersede the requirements of all other documents if there are disagreements.

LABC	International Conference of Building Officials and California Building Standards Commission. *The City of Los Angeles Building Code.* 2008.
CBC	California Building Standards Commission. *California Building Code*. 2007.
ASCE 7-05	American Society of Civil Engineers. *Minimum Design Loads for Buildings and Other Structures*. 2005.
ACI-318	American Concrete Institute. *Building Code Requirements for Structural Concrete*. 2005.
AISC	American Institute of Steel Construction. *Specifications for Structural Steel Buildings*. 2005.
ASTM	American Society for Testing and Materials. *American Society for Testing and Materials (ASTM) Standards in Building Codes*. 2007.
NEHRP	National Earthquake Hazards Reduction Program. *NEHRP Recommended Provisions for Seismic Regulations for New Buildings and Other Structures*, *FEMA 450-1*. 2003.

DEFINITIONS

Capacity-protected: Structural elements such as pile caps, deck beams, and deck slabs that are designed to have a greater capacity than the adjacent ductile members such as piles. Refer to Section 1.6.6.

Code: The Port of Los Angeles Code for Seismic Design, Upgrade and Repair of Container Wharves (POLA Seismic Code 2010).

Contingency Level Earthquake (CLE): 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 in 50 years.

Design Earthquake (DE): Design earthquake as defined in ASCE 7-05 Section 11.2.

Diameter (Pile Diameter): Diameter of circular cross-section or diameter of circle inscribed within non-circular cross section.

Dike: Engineered assembly of rock material used to retain fill or cut slopes for container wharves.

Ductile Design: Design of structural elements that provide significant displacement and rotation capacity beyond yield strength through the use of detailing, such as confinement. Refer to Section 1.6.7.

Dynamic Magnification Factor (DMF): A factor to account for effects of higher order modes. Refer to Section 1.6.4.1f.

Embankment: Fill material or cut slopes protected or stabilized by dike.

Expansion Joint: A joint between two wharf units with a shear key that allows relative longitudinal movement (movement parallel to shore) but restricts relative transverse movement (movement perpendicular to shore).

Expected Strength: The strength of a structural member based on the most probable (expected) material properties. Refer to Section 1.6.2.

Hydrodynamic Mass: Mass of the water around the pile which is accelerated with the movement of the pile due to action of pressure under seismic load. Refer to Section 1.6.1.

Inertial Load: Loading on the piles from the response of the seismic mass due to seismic ground acceleration. Refer to Section 1.7.5.1.

Kinematic Load: Loading on the piles from permanent ground deformation. Refer to Section 1.7.5.2.

Linked Wharves: Two or more wharf units that are joined by one or more expansion joint(s).

Lower-bound Lateral Soil Spring: The lowest lateral soil spring representing the softest soil behavior. Refer to Section 1.7.5.1.

Marginal Container Wharves: Waterfront structures parallel to the shoreline that project from the land into a body of water used for transfer of containers. Typically, marginal wharves have a minimum of one row of piles located landside of or close to the dike crest.

Modal Response Spectrum Analysis: Spectral analysis that captures transverse, longitudinal and rotational modal responses. Refer to Section 1.6.4.1

Non-seismic Piles: Piles that resist no more than 10% of the total lateral seismic load. These piles are typically located waterside of the dike crest in deeper water and primarily carry vertical load.

Operational Level Earthquake (OLE): The seismic event that produces ground motions associated with a 72-year return period. The 72-year return period ground motions have a 50% probability of being exceeded in 50 years.

Performance-based Design: Design based on specific criteria and performance objectives associated with acceptable levels of damage at specified levels of seismic hazard.

Pile-deck Joint: The moment resisting connection between the top of the pile and the deck.

Plastic Hinge: The region of the pile where concrete or steel strain exceeds the strain associated with the yield strength. Refer to Section 1.6.4.2.

Pseudo-static Seismic Slope Stability Analysis: A slope stability evaluation where earthquake load is represented by an equivalent horizontal static load. Refer to Section 1.7.2.2.

Post-earthquake Static Slope Stability Analysis: A static slope stability evaluation using soil parameters following an earthquake to account for potential earthquake induced soil strength loss. Refer to Section 1.7.2.3.

Seismic Mass: The mass of the structure dead load and a portion of the design live load that contributes to the seismic response. Refer to Section 1.6.1.

Seismic Piles: Piles that resist most of the lateral seismic load. These piles are typically located landside of or close to the dike crest

Single-mode Transverse Analysis: Spectral analysis that captures the transverse modal response of the structure.

Soil-structure Interaction (SSI): The process in which the response of the soil influences the deformation of the structure and the deformation of the structure influences the response of the soil.

Upper-bound Lateral Soil Spring: The highest lateral soil springs representing the stiffest soil behavior. Refer to Section 1.7.5.1.

Wharf End Unit: A wharf structure with one expansion joint at one end.

Wharf Unit: A wharf structure between two expansion joints or an independent structure without expansion joints.

SYMBOLS AND NOTATIONS

Unless noted otherwise all units are in pounds, inches and seconds.

A_e	Pile effective shear cross-sectional area (80% of gross cross-sectional area for solid circular or octagonal piles)
A_{sc}	Total cross-sectional area of dowel bars in the pile-deck joint
A_{sp}	Cross-sectional area of transverse reinforcement
B	Width of a wharf unit in feet
c	Depth from the extreme compression fiber to the neutral axis at flexural strength
c_o	Concrete cover plus half the diameter of the transverse reinforcement
D'	Diameter of the pile-deck joint core measured to the centerline of the confinement steel
d_b	Dowel diameter
DL	Dead load in moments, shear forces, or axial forces due to self-weight of the wharf deck, 1/3 of the pile weight between the deck soffit and $5D_p$ below the dike surface, crane self-weight and weight of any permanently attached equipment or fixtures
D_p	Diameter of the pile
E	Earth lateral pressure
e	Eccentricity between the wharf center of mass and center of rigidity
EQ	Earthquake lateral load due to OLE, CLE or DE
E_{sh}	Confining steel modulus of elasticity
F	Wharf total lateral seismic force of the wharf strip considered at Δ_d
f'_c	Specified compressive strength of unconfined concrete at 28 days
f'_{ce}	Expected concrete compressive strength
F_d	Wharf deck member, moment, shear and axial demands
F_o	Design moment, shear and axial forces for deck members
F_p	Prestress compressive force in pile taken as zero at top plastic hinge
f_{pu}	Ultimate strength of prestressing strands
f_{pue}	Expected prestressing strand ultimate strength
f_y	Nominal yield strength of longitudinal reinforcing steel, dowels, or structural steel
f_{ye}	Expected yield strength of longitudinal reinforcing steel, dowels, or structural steel
f_{yh}	Nominal yield strength of confining or transverse steel
f_{yhe}	Expected yield strength of confining or transverse steel
g	The distance between the top of the pile steel shell and the deck soffit
H	The distance between the center of the pile top plastic hinge and the center of the pile in-ground plastic hinge
j	Time step of the time-history record not more than 0.05-second interval
K	(0.5 x PGA / gravity) where PGA is the peak ground acceleration in feet/second^2 and gravity is 32.2 feet/second^2
k	Curvature ductility factor determined as a function of μ_ϕ
k_e	Secant wharf stiffness at seismic demand
k_i	Initial elastic stiffness of the wharf structure based on cracked section properties
k_s	Secant stiffness of the wharf structure at the considered seismic demand
$k_{s,n}$	k_s at iteration step n
$k_{s,n-1}$	k_s at iteration step n-1
k_y	Yield acceleration coefficient

L	Distance from the center of the pile top plastic hinge to the pile point of contra-flexure
l_a	Actual embedment length of dowels anchored in the pile-deck joint
LL	Live load in moments, shear forces or axial forces due to the design uniform live load
L_L	Length of the shortest exterior wharf unit in feet
L_p	Plastic hinge length
L_{sp}	Strain penetration length
m	Time-history record number
m_{crane}	Mass of crane
$m_{crane,deck}$	Part of the crane mass positioned within 10 feet above wharf deck
M_n	Nominal moment capacity
$M_{p,in\text{-}ground}$	Pile plastic moment capacity at the in-ground plastic hinge including effect of axial load on piles due to crane dead load
$M_{p,top}$	Pile plastic moment capacity at the top plastic hinge including the effect of axial load on piles due to crane dead load
$m_{seismic}$	Seismic mass
P_a	External axial load on pile (compression is taken as positive and tension as negative)
p-y	Inelastic lateral soil springs
T_{crane}	Translational elastic period of the crane mode with the maximum participating mass
T_{wi}	Initial elastic period of the wharf structure based on cracked section properties
T_{ws}	Secant period of the wharf structure
U	Total design load in moments, shear forces or axial forces
V_a	Shear strength due to the smallest axial load demand
V_c	Concrete shear strength
V_n	Pile nominal shear capacity
V_o	Pile shear demand
V_p	Pile plastic shear
V_s	Transverse reinforcement shear strength
V_{sk}	Expansion joint shear key force due to OLE, CLE or DE
V_Δ	Total wharf lateral seismic force at the displacement demand determined using pushover analysis
W	Effective dead load of the wharf strip considered
α	Angle between the line joining the centers of the compression zones at top and in-ground plastic hinges and pile axis
β	Factor determined as a function of wharf unit length
Δ_c	Displacement capacity corresponding to the performance level considered
Δ_d	Displacement demand corresponding to the earthquake level considered
$\Delta_{d,j}$	Δd at time step j
$\Delta_{d,j,m}$	Δd at time step j for time-history record number m
$\Delta_{p,m}$	The pile plastic displacement capacity due to rotation of the plastic hinge at OLE, CLE or DE specified strain limit
Δ_t	Displacement of wharf due to transverse excitation
Δ_{ti}	Spectral displacement demand for single-mode transverse response corresponding to wharf initial elastic period, T_{wi} using 0.05 damping ratio

Δ_{ts}	Spectral displacement demand for single-mode transverse response corresponding to wharf secant period, T_{ws} using effective damping ratio, ξ_{eff}
$\Delta_{ts,\,n}$	Δ_{ts} at iteration step n
$\Delta_{ts,\,n-1}$	Δ_{ts} at iteration step n-1
$\Delta_{ts,\,n-2}$	Δ_{ts} at iteration step n-2
Δ_{X1}, Δ_{X2}	Combined X-axis displacements due to excitations in the transverse and longitudinal directions
$\Delta_{X,\,j,\,m}$	Combined X-axis displacements due to excitation in the transverse and longitudinal directions for time step j and time-history record number m
Δ_{XL}	X-axis displacement due to longitudinal excitation
$\Delta_{XL,\,j,\,m}$	X-axis displacement due to longitudinal excitation for time step j and time-history record number m
Δ_{XT}	X-axis displacement due to transverse excitation
$\Delta_{XT,\,j,\,m}$	X-axis displacement due to transverse excitation for time step j and time-history record number m
Δ_{y}	Displacement when the considered pile plastic hinge develops
Δ_{Y1}, Δ_{Y2}	Combined Y-axis displacements due to excitations in the transverse and longitudinal directions
$\Delta_{Y,\,j,\,m}$	Combined Y-axis displacement from excitations in the transverse and longitudinal directions for time step j and time-history record m
Δ_{YL}	Y-axis displacement due to longitudinal excitation
$\Delta_{YL,\,j,\,m}$	Y-axis displacement due to longitudinal excitation for time step j and time-history record m
Δ_{ys}	Wharf structure system yield displacement determined from the intersection of the elastic and post-yield branches of the bilinear approximation of the force-displacement curve
Δ_{YT}	Y-axis displacement due to transverse excitation
$\Delta_{YT,\,j,\,m}$	Y-axis displacement due to transverse excitation for time step j and time-history record m
ε_{c}	Extreme fiber concrete compressive strain
ξ_{eff}	Effective damping ratio
ε_{p}	Prestressing strands tensile strain
ε_{s}	Steel shell extreme fiber strain
ε_{sd}	Dowel reinforcement tensile strain
ε_{smd}	Strain at peak stress of dowel reinforcement
ϕ	Strength reduction factor, 0.85 for OLE and CLE and 1.0 for DE
ϕ_{m}	Curvature at OLE, CLE, or DE specified strain limit
$\phi_{p,m}$	Pile plastic curvature at OLE, CLE or DE specified strain limit
ϕ_{y}	Yield curvature determined when the considered plastic hinge develops
μ_{Δ}	Displacement ductility
μ_{ϕ}	Curvature ductility
ρ_{s}	Effective volumetric ratio of confining steel = (volume of confining steel in one loop) / (volume of concrete core for a length equal to the confining steel spacing along the pile length)

θ	Angle of the critical shear crack with respect to the longitudinal axis of the pile
$\theta_{p,\,m}$	Pile plastic rotation at OLE, CLE or DE specified strain limit

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CHAPTER 1
SEISMIC DESIGN OF NEW CONTAINER WHARVES

1.1 PURPOSE

The purpose of this chapter is to provide seismic code provisions to safeguard life, protect against major structural failures, limit damages, and minimize economic losses due to seismic events for new marginal container wharves.

The intent of the provisions provided in this chapter is to achieve performance goals for the seismic design of new marginal container wharves at three levels of ground motions:

a. Operating Level Earthquake (OLE): No significant structural damage. Damage location to be visually observable and accessible for repairs. Minimum or no interruption to wharf operations during repairs may occur.

b. Contingency Level Earthquake (CLE): Controlled inelastic structural behavior and limited permanent deformations. Damage location to be visually observable and accessible for repairs. Temporary or short term loss of operations may occur.

c. Design Earthquake Level (DE): Safeguard life and against major structural failures.

1.2 SCOPE

The scope of this chapter 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 and soil-structure requirements are provided.

In order to achieve seismic performance goals, seismic performance criteria provided in terms of material strain limits for each earthquake level are specified.

1.3 PERFORMANCE REQUIREMENTS

The design of new marginal container wharf structures shall satisfy the strain limits at the three levels of ground motions provided in this section.

1.3.1 Ground Motions

Three levels of site-specific ground motions shall be determined for the design of wharf structures as defined in Table 1-1.

Table 1-1: Ground Motions

Earthquake	Probability of Exceedance	Return Period
Operating Level Earthquake (OLE)	50% in 50 years	72
Contingency Level Earthquake (CLE)	10% in 50 years	475
Design Earthquake Level (DE)	"Design Earthquake" as defined in ASCE 7-05 Section 11.2.	

1.3.2 Strain Limits

The strain limits for piles at OLE, CLE and DE are provided in Table 1-2, Table 1-3, and Table 1-4.

Table 1-2: Seismic Piles Top Plastic Hinge Strain Limits

Pile	Design Level		
	OLE	**CLE**	**DE**
Solid Concrete Piles	$\varepsilon_c \leq 0.005$ $\varepsilon_{sd} \leq 0.015$	$\varepsilon_c \leq (0.005 + 1.1\ \rho_s) \leq 0.025$ $\varepsilon_{sd} \leq 0.6\varepsilon_{smd} \leq 0.06$	ε_c [(b)] $\varepsilon_{sd} \leq 0.8\varepsilon_{smd} \leq 0.08$
Round Hollow Concrete Piles [(a)]	$\varepsilon_c \leq 0.004$ $\varepsilon_{sd} \leq 0.015$	$\varepsilon_c \leq 0.006$ $\varepsilon_{sd} \leq 0.4\varepsilon_{smd} \leq 0.04$	$\varepsilon_c \leq 0.008$ $\varepsilon_{sd} \leq 0.6\varepsilon_{smd} \leq 0.06$
Steel Pipe Piles: Concrete Plug with Dowels	$\varepsilon_c \leq 0.010$ $\varepsilon_{sd} \leq 0.015$	$\varepsilon_c \leq 0.025$ $\varepsilon_{sd} \leq 0.6\varepsilon_{smd} \leq 0.06$	ε_c [(b)] $\varepsilon_{sd} \leq 0.8\varepsilon_{smd} \leq 0.08$

[(a)] If the interior of hollow concrete piles is filled with concrete, all strain limits shall be the same as for solid concrete piles.
[(b)] No limit.

Table 1-3: Non-seismic Piles Top Plastic Hinge Strain Limits

Pile	Design Level	
	OLE and CLE	**DE**
Solid Concrete Piles	$\varepsilon_c \leq 0.005$ $\varepsilon_{sd} \leq 0.015$	ε_c [(b)] $\varepsilon_{sd} \leq 0.8\varepsilon_{smd} \leq 0.08$
Round Hollow Concrete Piles [(a)]	$\varepsilon_c \leq 0.004$ $\varepsilon_{sd} \leq 0.015$	$\varepsilon_c \leq 0.008$ $\varepsilon_{sd} \leq 0.6\varepsilon_{smd} \leq 0.06$
Steel Pipe Piles: Concrete Plug with Dowels	$\varepsilon_c \leq 0.010$ $\varepsilon_{sd} \leq 0.015$	ε_c [(b)] $\varepsilon_{sd} \leq 0.8\varepsilon_{smd} \leq 0.08$

[(a)] If the interior of hollow concrete piles is filled with concrete, all strain limits shall be the same as for solid concrete piles.
[(b)] No limit.

Table 1-4: In-ground Plastic Hinge Strain Limits for Seismic and Non-seismic Piles

Pile	In-ground Plastic Hinge Location	Design Level		
		OLE	CLE	DE
Solid Concrete Piles	Hinge form at depth ≤ 10 D_p	$\varepsilon_c \leq 0.005$ $\varepsilon_p \leq 0.015$	$\varepsilon_c \leq (0.005 + 1.1\ \rho_s) \leq 0.008$ $\varepsilon_p \leq 0.025$	$\varepsilon_c \leq (0.005 + 1.1\ \rho_s) \leq 0.025$ $\varepsilon_p \leq 0.035$
	Hinge form at depth >10 D_p	$\varepsilon_c \leq 0.008$ $\varepsilon_p \leq 0.015$	$\varepsilon_c \leq 0.012$ $\varepsilon_p \leq 0.025$	ε_c [(b)] $\varepsilon_p \leq 0.050$
Round Hollow Concrete Piles[(a)]	Hinge form at depth ≤ 10 D_p	$\varepsilon_c \leq 0.004$ $\varepsilon_p \leq 0.015$	$\varepsilon_c \leq 0.006$ $\varepsilon_p \leq 0.025$	$\varepsilon_c \leq 0.008$ $\varepsilon_p \leq 0.025$
	Hinge form at depth >10 D_p	$\varepsilon_c \leq 0.004$ $\varepsilon_p \leq 0.015$	$\varepsilon_c \leq 0.006$ $\varepsilon_p \leq 0.025$	$\varepsilon_c \leq 0.008$ $\varepsilon_p \leq 0.050$
Steel Pipe Piles	Hinge form at depth ≤ 10 D_p	$\varepsilon_s \leq 0.010$	$\varepsilon_s \leq 0.025$	$\varepsilon_s \leq 0.035$
	Hinge form at depth >10 D_p	$\varepsilon_s \leq 0.010$	$\varepsilon_s \leq 0.035$	$\varepsilon_s \leq 0.050$
Steel Pipe Piles Filled with Concrete	Hinge form at depth ≤ 10 D_p	$\varepsilon_s \leq 0.010$	$\varepsilon_s \leq 0.035$	$\varepsilon_s \leq 0.050$
	Hinge form at depth >10 D_p	$\varepsilon_s \leq 0.010$	$\varepsilon_s \leq 0.035$	$\varepsilon_s \leq 0.050$

[(a)] If the interior of hollow concrete piles is filled with concrete, all strain limits shall be the same as for solid concrete piles.
[(b)] No limit.

Where:

ε_c = Extreme fiber concrete compressive strain

ρ_s = Effective volumetric ratio of confining steel = (volume of confining steel in one loop) / (volume of concrete core for a length equal to the confining steel spacing along the pile length)

ε_{sd} = Dowel reinforcement tensile strain

ε_{smd} = Strain at peak stress of dowel reinforcement

D_p = Diameter of the pile

ε_p = Prestressing strand tensile strain

ε_s = Steel shell extreme fiber strain

1.4 General Seismic Criteria

1.4.1 Wharf

a. The seismic design of a new marginal container wharf structure shall comply with provisions in Section 1.6.

b. The structural system shall be based on the strong beam (deck) and weak column (pile) frame concept. All plastic hinges shall be designed to occur in the piles. All elements of the deck shall be capacity-protected as defined in Section 1.6.6.

c. The wharf shall be designed as a moment-resisting frame consisting of a reinforced concrete deck supported by vertical piles. Pile plastic hinge region and pile-to-deck ductile connection shall be detailed according to Section 1.6.7. Battered piles shall not be used.

d. For non-seismic piles, if a plastic hinge is developed the non-seismic piles shall comply with the detailing requirements of the seismic piles according to Section 1.6.7.

e. The pile-deck joint forces determined based on the maximum induced moments, shears and axial forces in the pile shall be in equilibrium.

f. The design of concrete elements shall comply with the provisions of this Code and ACI-318-05. The design of steel elements shall comply with the provisions of this Code and AISC 13th edition.

g. Crane-wharf interaction shall be evaluated per Section 1.6.4.3.

1.4.2 Embankment and Dike

a. The embankment and dike shall be designed according to Section 1.7.

b. The clearance between the deck soffit and the top of the dike or embankment shall be a minimum of 3.5 feet.

1.4.3 Utilities and Pipelines

Utilities and pipelines connections shall be designed to accommodate the maximum seismic deformation including ground surface rupture at the following locations:

a. Where utilities and pipelines pass from the backland through the cutoff wall or other rigid structure at the back of the wharf.

b. Where utilities and pipelines span across expansion joints of individual wharf units.

1.5 Load Combinations

The following load combinations shall be used to determine seismic moment, shear and axial demands for wharf deck and pile cap, and seismic shear and axial force demands for piles:

$$U = (1 \pm K)\ DL + 0.1\ LL + E + EQ \qquad (1\text{-}1)$$

$$U = (1 \pm K)\ DL + E + EQ \qquad (1\text{-}2)$$

Where:

U = Total design load in moments, shear forces or axial forces

K = (0.5 x PGA / gravity) where PGA is the peak ground acceleration in feet/second^2 and gravity is 32.2 feet/second^2

DL = Dead load in moments, shear forces, or axial forces due to self-weight of the wharf deck, 1/3 of the pile weight between the deck soffit and $5D_p$ below the dike surface, crane self-weight and weight of any permanently attached equipment or fixtures

LL = Live load in moments, shear forces or axial forces due to the design uniform live load

E = Earth lateral pressure

EQ = Earthquake lateral load due to OLE, CLE or DE

1.6 ANALYTICAL AND DESIGN REQUIREMENTS

1.6.1 Seismic Mass

The seismic mass, $m_{seismic}$ shall include:

a. The mass of the wharf deck.
b. 1/3 of the mass of piles with the tributary length measured from the bottom of deck soffit to $5D_p$ below the surface of pile embedment.
c. The mass of any permanently attached equipment/ fixtures.
d. Mass due to the minimum of 10% of the design uniform live load or 100 pounds per square foot.
e. The hydrodynamic mass, if the pile diameter is greater than 2 feet.
f. The part of crane mass not less than $m_{crane,deck}$ or 0.05 m_{crane}

where,

$m_{crane,deck}$ = Part of the crane mass positioned within 10 feet above wharf deck

m_{crane} = Mass of crane

1.6.2 Material Properties

The capacity of ductile elements, except shear, shall be based on the following material properties:

Expected concrete compressive strength	$f'_{ce} = 1.3f'_c$
Expected yield strength of longitudinal steel, dowels, or structural steel	$f_{ye} = 1.1f_y$
Expected yield strength of confining or transverse steel	$f_{yhe} = 1.0f_{yh}$
Expected prestressing strand ultimate strength	$f_{pue} = 1.05f_{pu}$

Where:

f'_c= Specified compressive strength of unconfined concrete at 28 days

f_y = Nominal yield strength of longitudinal reinforcing steel, dowels, or structural steel

f_{yh} = Nominal yield strength of confining or transverse steel

f_{pu} = Ultimate strength of prestressing strand

1.6.3 Modeling Requirements

The analytical model shall represent all significant structural components including, but not limited to structural configurations, seismic mass, material and section properties, and soil-structure interaction properties as follows:

a. The analytical model shall accurately represent distribution of seismic mass, structural member properties, joint and boundary conditions and contain sufficient nodes and elements to capture the critical structural seismic responses.

b. The analytical model shall include soil-structure interaction using upper bound and lower bound lateral soil springs. See Section 1.7. The contribution of soil passive pressure at the cut-off wall shall not be used to reduce wharf displacement demand or to increase wharf displacement capacity.

c. Pile cracked section proprieties shall be used based on the expected material properties provided in Section 1.6.2.

d. The pile effective stiffness shall be determined using the expected material properties provided in Section 1.6.2.

e. The wharf deck-to-concrete pile connection shall be modeled as shown in Figure 1-1, which includes, but is not limited to, the following:

1. A node to capture the pile plastic moment capacity at the deck soffit.
2. The length of the first pile element below the soffit shall have reinforced concrete section properties and be at least 16 inches in length.
3. For piles connected to the deck with dowels, a pile element with the strain penetration length, L_{sp} shall be provided as follows:

$$L_{sp} = 0.12 f_{ye} d_b \tag{1-3}$$

Where:

f_{ye} = Expected yield strength of dowels in kips per square inch
d_b = Dowel diameter

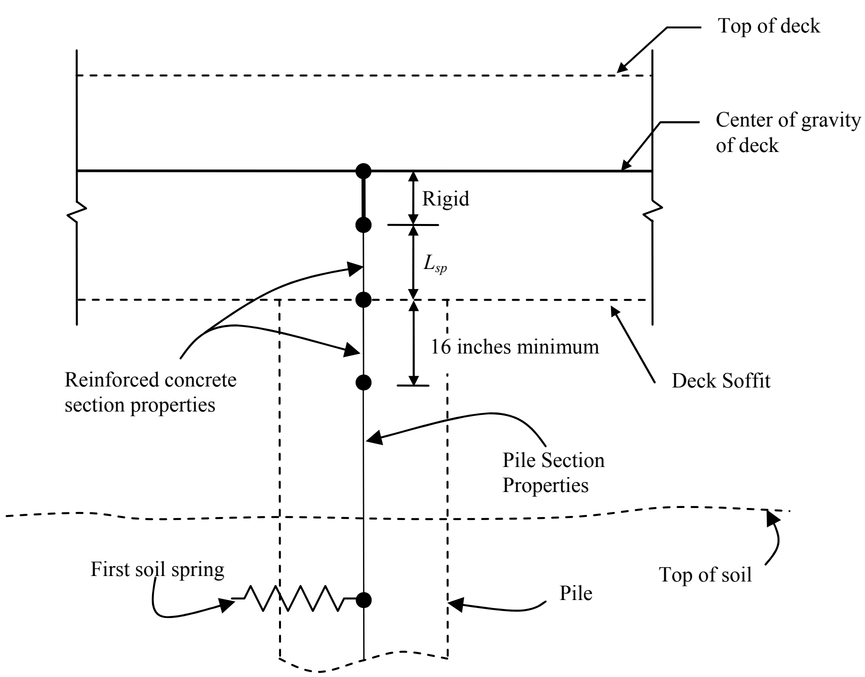

Figure 1-1: Modeling of Pile-to-Deck Connection (Not-to-Scale)

1.6.4 Displacement Demand and Capacity

Displacement demand, Δ_d of the wharf shall not be greater than the displacement capacity, Δ_c. of the wharf:

$$\Delta_d \leq \Delta_c \tag{1-4}$$

1.6.4.1 Displacement Demand of Wharf

a. Displacement demand, Δ_d shall be determined at OLE, CLE, and DE per Section 1.3.1 using upper bound and lower bound soil spring strength and stiffness values per Section 1.7.5.1.

b. The displacement demand shall include the effect of kinematic load per Section 1.7.5.2 and combined effect of inertial and kinematic loads per Section 1.7.5.3.

c. Displacement demand shall be determined using Multi-modal Spectral Analysis per Section 1.6.4.1e.

   Exceptions:

   Single-mode Transverse Analysis may be used to determine displacement demand per Section 1.6.4.1f for straight wharf units if all the following conditions are met:

   1. 400 feet < L_L < 800 feet
   2. 100 feet < B < 120 feet
   3. Less than 20% variation in the initial elastic stiffness of the wharf structure along the wharf length.
   4. Crane-wharf interaction analysis is not required per Section 1.6.4.3.

   Where:

   L_L= Length of the shortest exterior wharf unit in feet
   B = Width of a wharf unit in feet

d. Nonlinear Time-history Analysis per Section 1.6.4.1g shall be used to verify Multi-modal Spectral Analysis or Single-mode Transverse Analysis for special conditions as required by the POLA.

e. For Multi-modal Spectral Analysis, sufficient modes shall be included such that 90% of the participating mass is captured, a damping ratio of 5% shall be used, and wharf displacement demand, Δ_d, shall be determined as follows:

$$\Delta_d = \max\left(\sqrt{\Delta_{X1}^2 + \Delta_{Y1}^2} \text{ or } \sqrt{\Delta_{X2}^2 + \Delta_{Y2}^2} \right) \tag{1-5}$$

$$\begin{aligned} \Delta_{X1} &= \Delta_{XL} + 0.3\Delta_{XT} \\ \Delta_{Y1} &= \Delta_{YL} + 0.3\Delta_{YT} \end{aligned} \tag{1-6}$$

$$\begin{aligned} \Delta_{X2} &= 0.3\Delta_{XL} + \Delta_{XT} \\ \Delta_{Y2} &= 0.3\Delta_{YL} + \Delta_{YT} \end{aligned} \tag{1-7}$$

Where:

Δ_{XL} = X-axis displacement due to longitudinal excitation, refer to Figure 1-2

Δ_{XT} = X-axis displacement due to transverse excitation, refer to Figure 1-2

Δ_{YL} = Y-axis displacement due to longitudinal excitation, refer to Figure 1-2

Δ_{YT} = Y-axis displacement due to transverse excitation, refer to Figure 1-2

Δ_{X1}, Δ_{X2} = Combined X-axis displacements due to excitations in the transverse and longitudinal directions

Δ_{Y1}, Δ_{Y2} = Combined Y-axis displacements due to excitations in the transverse and longitudinal directions

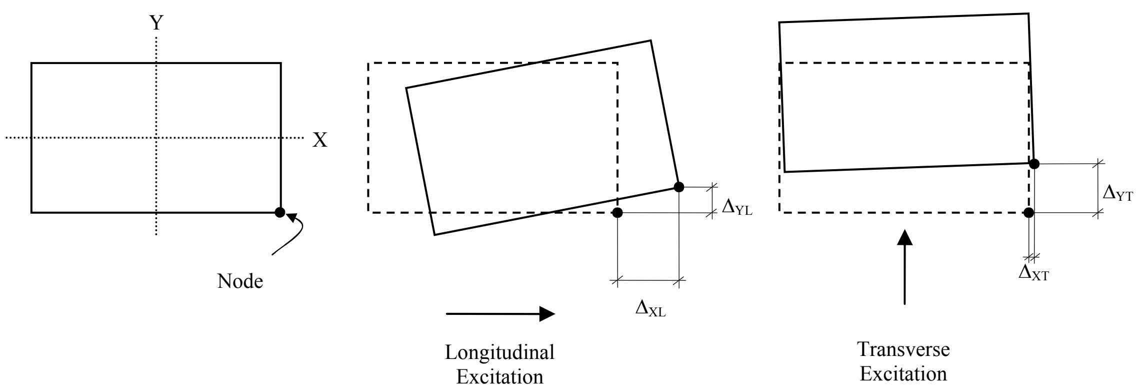

Figure 1-2: Multi-modal Spectral Analysis

f. For Single-mode Transverse Analysis, the displacement demand Δ_d, shall be determined as follows:

$$\Delta_d = \Delta_t \times \text{DMF} \tag{1-8}$$

Where:

Δ_t = Maximum of Δ_{ti} or Δ_{ts}

Δ_{ti} = Spectral displacement demand for single-mode transverse response corresponding to wharf initial elastic period, T_{wi} using 0.05 damping ratio

$$T_{wi} = 2\pi\sqrt{\frac{m_{seismic}}{k_i}} \tag{1-9}$$

k_i = Initial elastic stiffness of the wharf structure based on cracked section properties

Δ_{ts} = Spectral displacement demand for single-mode transverse response corresponding to wharf secant period, T_{ws} using effective damping ratio, ξ_{eff}

$$T_{ws} = 2\pi\sqrt{\frac{m_{seismic}}{k_s}} \tag{1-10}$$

k_s = Secant Stiffness of the wharf structure at the considered seismic demand

$$\xi_{eff} = 0.10 + 0.565\left(\frac{\mu_\Delta - 1}{\mu_\Delta \pi}\right) \quad (1\text{-}11)$$

$$\mu_\Delta = \frac{\Delta_{ts}}{\Delta_{ys}} \quad (1\text{-}12)$$

Δ_{ys}= Wharf structure system yield displacement determined from the intersection of the elastic and post-yield branches of the bilinear approximation of the force displacement curve, refer to Figure 1-3

Δ_{ts} shall be determined using an iterative procedure with convergence tolerance such that $\left|100 - \frac{\Delta_{ts,n}}{\Delta_{ts,n-1}}\right| \leq 3\%$

DMF =

Single Wharf Unit:

DMF = 1.80 - 0.05 LL / B ≥ 1.10 for OLE (1-13)

DMF = 1.65 - 0.05 LL / B ≥ 1.10 for CLE/DE, UB springs (1-14)

DMF = 1.50 - 0.05 LL / B ≥ 1.10 for CLE/DE, LB springs (1-15)

Linked Wharf Exterior Unit:

DMF = 1.55 - 0.04 LL /B ≥ 1.10 for OLE (1-16)

DMF = 1.35 - 0.02 LL /B ≥ 1.10 for CLE/DE, UB springs (1-17)

DMF = 1.16 - 0.02 LL /B ≥ 1.10 for CLE/DE, LB springs (1-18)

Linked Wharf Interior Unit

DMF = 1.10 (1-19)

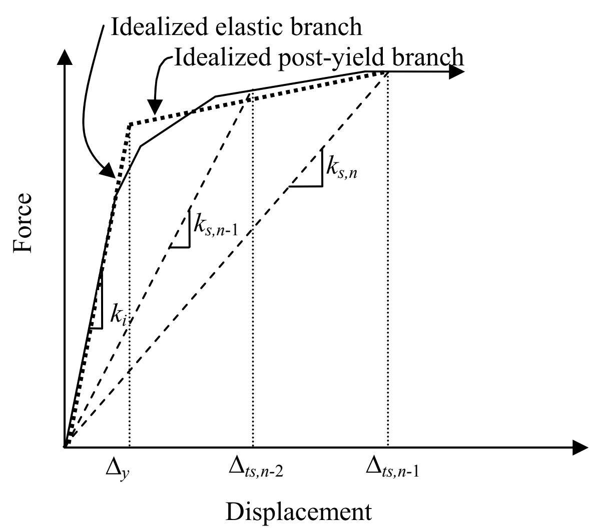

Figure 1-3: Force-displacement Curve

g. For Nonlinear Time-history Analysis, seven orthogonal sets of spectrum-compatible time-histories and nonlinear material properties shall be used. The displacement demand shall be determined using the maximum of the average displacements for each response parameter as follows:

$$\Delta_d = \max \left[\Delta_{d,j}\right]_{j=1}^{j=\max\ time\ step} \tag{1-20}$$

$$\Delta_{d,j} = \text{average}\left[\Delta_{d,j,m}\right]_{m=1}^{m=7\ records} \tag{1-21}$$

$$\Delta_{d,j,m} = \sqrt{\Delta_{X,j,m}^{\ \ 2} + \Delta_{Y,j,m}^{\ \ 2}} \tag{1-22}$$

$$\begin{aligned} \Delta_{X,j,m} &= \Delta_{XL,j,m} + \Delta_{XT,j,m} \\ \Delta_{Y,j,m} &= \Delta_{YL,j,m} + \Delta_{YT,j,m} \end{aligned} \tag{1-23}$$

Where:

$\Delta_{XL,j,m}$= X-axis displacement due to longitudinal excitation, refer to Figure 1-2, for time step j and time-history record number m

$\Delta_{XT,j,m}$= X-axis displacement due to transverse excitation, refer to Figure 1-2, for time step j and time-history record number m

$\Delta_{YL,j,m}$ = Y-axis displacement due to longitudinal excitation, refer to Figure 1-2, for time step j and time-history record number m

$\Delta_{YT,j,m}$= Y-axis displacement due to transverse excitation, refer to Figure 1-2, for time step j and time-history record number m

j = Time step of the time-history record not more than 0.05-second interval

m = Time-history record number

1.6.4.2 Displacement Capacity of Wharf

Displacement capacity, Δ_c of the wharf shall be determined at OLE, CLE and DE based on the strain limits provided in Section 1.3.2 using two lateral soil spring conditions: upper bound and lower bound. Displacement capacity shall be the lesser of displacement capacity at pile top plastic hinge or displacement capacity at pile in-ground plastic hinge determined as follows:

$$\Delta_c = \Delta_y + \Delta_{p,m} \tag{1-24}$$

$$\Delta_{p,m} = \theta_{p,m} \times H \tag{1-25}$$

$$\theta_{p,m} = L_p \phi_{p,m} = L_p(\phi_m - \phi_y) \tag{1-26}$$

Where:

Δ_y = Displacement when the considered pile plastic hinge develops

$\Delta_{p,m}$ = The pile plastic displacement capacity due to rotation of the plastic hinge at OLE, CLE or DE specified strain limit

H = The distance between the center of the pile top plastic hinge and the center of the pile in-ground plastic hinge

$\theta_{p,m}$ = Pile plastic rotation at OLE, CLE or DE specified strain limit

$\phi_{p,m}$ = Pile plastic curvature at OLE, CLE or DE specified strain limit

ϕ_m = Curvature at OLE, CLE, or DE specified strain limit
ϕ_y = Yield curvature determined when the considered plastic hinge develops
L_p = Plastic hinge length determined based on Table 1-5

Table 1-5: Plastic Hinge Length

Pile	Top Hinge [(a)]	In-ground Hinge
Solid Concrete Piles	$L_p = 0.08L + 0.12 f_{ye} d_b \geq 0.2 f_{ye} d_b$	$L_p = 2\ D_p$
Round Hollow Concrete Piles	$L_p = 0.08L + 0.12 f_{ye} d_b \geq 0.2 f_{ye} d_b$	$L_p = 2\ D_p$
Steel Pipe Piles with Concrete Plug and Dowels	$L_p = 0.3 f_{ye} d_b + g$	Not applicable
Steel Pipe Piles	Not applicable	$L_p = 2\ D_p$

[(a)] Alternatively, for solid concrete piles $L_p=0.08L+0.15 f_{ye} d_b \geq 0.3\, f_{ye} d_b$ may be used.

Where:

L = Distance from the center of the pile top plastic hinge to the pile point of contra-flexure
f_{ye} = Expected yield strength of dowels in kips per square inch
d_b = Dowel diameter
g = The distance between the top of the pile steel shell and the deck soffit
D_p = Diameter of the pile

1.6.4.3 Crane-wharf Interaction

a. Crane-wharf interaction analysis shall be required if:

$$T_{crane} \leq 2T_{wi} \qquad (1\text{-}27)$$

Where:

T_{crane} = Translational elastic period of the crane mode with the maximum participating mass
T_{wi} = Initial elastic period of the wharf structure based on cracked section properties

b. If crane-wharf interaction analysis is required, the displacement demand, Δ_d of the wharf shall be calculated using Nonlinear Time-history Analysis per Section 1.6.4.1g.

1.6.5 Piles

1.6.5.1 Moment Capacity

Pile plastic hinges moment capacities shall be determined using the following:

a. Expected material properties as defined in Section 1.6.2.

b. Largest axial load to obtain highest moment capacity for the design of capacity protected elements.

c. Smallest axial load to obtain the smallest pile displacement capacity for the design of piles.

1.6.5.2 Shear

Pile shear demand, V_o shall not be greater than the pile shear capacity ϕV_n :

$$V_o \leq \phi V_n \tag{1-28}$$

Where:

V_o = Pile shear demand
V_n = Pile nominal shear capacity
ϕ = Strength reduction factor, 0.85 for OLE and CLE and 1.0 for DE

1.6.5.2.1 Shear Demand

a. The pile shear demand, V_o shall be calculated as follows:

$$V_o = 1.25 \times V_p \tag{1-29}$$

b. Pile plastic shear, V_p shall be determined based on load combinations per Section 1.5 using nonlinear static pushover analysis with upper bound soil springs and including the effect of the axial load on piles due to crane dead load.

c. In lieu of Section 1.6.5.2.1a, pile shear demand, V_o may be calculated as follows:

$$V_o = 1.25\left(M_{p,top} + M_{p,in-ground}\right)/H \tag{1-30}$$

Where:

$M_{p,top}$ = Pile plastic moment capacity at the top plastic hinge including the effect of axial load on piles due to crane dead load
$M_{p,bottom}$ = Pile plastic moment capacity at the in-ground plastic hinge including effect of axial load on piles due to crane dead load
H = The distance between the center of the pile top plastic hinge and the center of the pile in-ground plastic hinge

1.6.5.2.2 Shear Capacity

a. Pile shear capacity ϕV_n, shall be calculated as follows:

$$\phi V_n = \phi\left(V_c + V_s + V_a\right) \tag{1-31}$$

$$V_c = k\sqrt{f_c'} \times A_e \tag{1-32}$$

$$\mu_\phi = 1 + \frac{\phi_{p,dem}}{\phi_y} = 1 + \frac{\theta_{p,dem}}{L_p \phi_y} \tag{1-33}$$

$$V_s = \frac{\pi}{2} \times \frac{A_{sp} \times f_{yh}\left(D_p - c - c_o\right) \times \cot(\theta)}{s}, \theta = 35° \tag{1-34}$$

$$V_a = 0.85(P_a + F_p)\tan(\alpha) \tag{1-35}$$

$$\tan(\alpha) = \frac{D_p - c}{H} \tag{1-36}$$

Where:

- V_c = Concrete shear strength
- V_s = Transverse reinforcement shear strength
- V_a = Shear strength due to the smallest axial load demand
- k = Curvature ductility factor determined as a function of μ_ϕ, refer to Figure 1-4
- A_e = Pile effective shear cross-sectional area (80% of gross cross-sectional area for solid circular or octagonal piles)
- μ_ϕ = Curvature ductility
- A_{sp} = Cross-sectional area of transverse reinforcement
- c = Depth from the extreme compression fiber to the neutral axis at flexural strength, refer to Figure 1-5
- c_o = Clear concrete cover plus half the diameter of the transverse reinforcement
- s = Center-to-center spacing of transverse reinforcement
- θ = Angle of the critical shear crack with respect to the longitudinal axis of the pile, refer to Figure 1-5
- P_a = External axial load on pile (compression is taken as positive and tension as negative)
- F_p = Prestress compressive force in pile taken as zero at top plastic hinge
- α = Angle between the line joining the centers of the compression zones at top and in-ground plastic hinges and the pile axis, refer to Figure 1-6
- β= 1.0 for existing structures, and 0.85 for new design

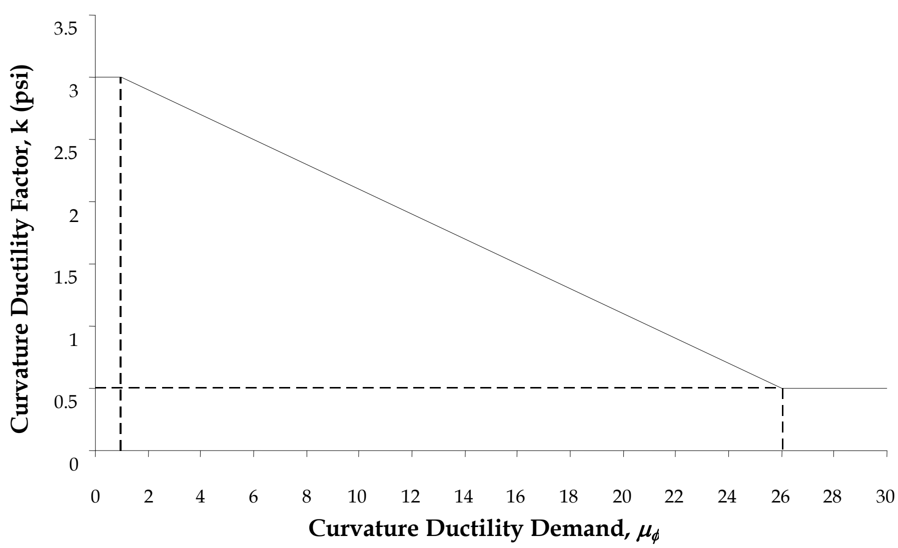

Figure 1-4: Curvature Ductility Factor Versus Curvature Ductility Demand

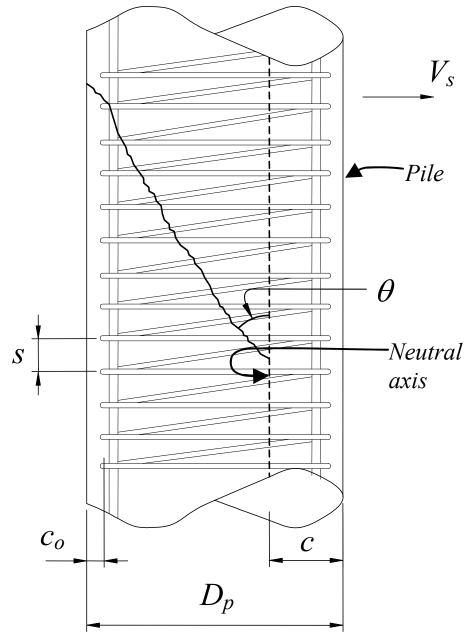

Figure 1-5: Pile Transverse Reinforcement Shear Strength Component

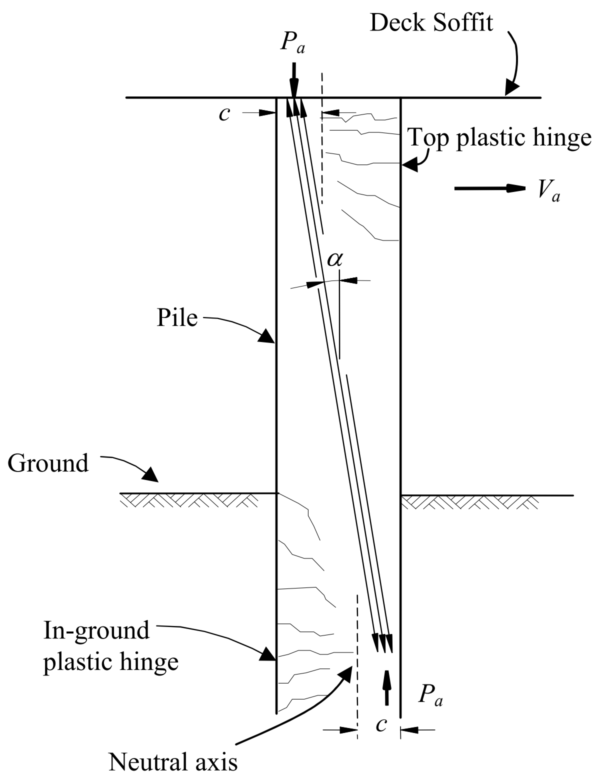

Figure 1-6: Pile Axial Force Shear Strength Component

1.6.5.3 P-Δ Effects

Additional secondary forces due to the effective dead load and the lateral seismic displacement demand (P-Δ) shall be calculated at OLE, CLE and DE. P-Δ effects may be ignored if the following is satisfied:

$$\frac{F}{W} \geq 4\frac{\Delta_d}{H} \tag{1-37}$$

Where:

F = Total lateral seismic force of the wharf strip considered at Δ_d
W = Effective dead load of the wharf strip considered
Δ_d = Displacement demand corresponding to the earthquake level considered
H = Distance from the center of the pile top plastic hinge to the center of the pile in-ground plastic hinge

1.6.5.4 Pile Transverse Reinforcement

The pile plastic hinge region shall comply with the following requirements:

a. The core of seismic piles in the plastic hinge region shall be confined over a minimum length of $2L_p$, with a minimum $\rho_s = 0.016$.
b. The core of non-seismic piles shall have a minimum $\rho_s = 0.008$.
c. Pile transverse reinforcement shall comply with the maximum requirements of Sections 1.6.5.1, 1.6.5.2.2 and 1.6.6.2.

1.6.6 Deck

1.6.6.1 Deck Members

a. Design moment, shear and axial forces for wharf deck members, F_o shall be calculated as follows:

$$F_o = 1.25 \times F_d \tag{1-38}$$

b. Wharf deck member, moment, shear and axial demands, F_d shall be determined based on load combinations per Section 1.5 using nonlinear static pushover analysis with upper bound soil springs and including the effect of the axial load on piles due to crane dead load.

1.6.6.2 Pile-Deck Joint

The pile-deck joint design shall comply with the following requirements:

a. Joint shear principal stresses due to maximum joint forces using load combinations per Section 1.5 shall comply with ACI-318.

b. The principal tension stresses shall be less than $\sqrt{12f_c'}$ and the principal compression stresses shall be less than $0.3f_c'$, where f_c' is the concrete compressive strength of the deck.
c. The effective volumetric ratio of confining steel, ρ_s around the pile dowels anchored in the pile-deck joint shall comply with the following:

$$\rho_s = \max \text{ of} \left[\frac{0.46 A_{sc}}{D' l_a} \left[\frac{f_{ye}}{0.0015 E_{sh}} \right] \text{ or } 0.016 \right] \tag{1-39}$$

Where:

A_{sc} = Total cross-sectional area of dowel bars in the pile-deck joint
f_{ye} = Expected yield strength of the dowels
D' = Diameter of the pile-deck joint core measured to the centerline of the confinement steel
l_a = Actual embedment length of dowels anchored in the pile-deck joint
E_{sh} = Confining steel modules of elasticity

1.6.6.3 Expansion Joint

a. The wharf expansion joints shall be designed for the combined effect of seismic deformation, seismic forces and thermal expansion.
b. The expansion joint shear key force, V_{sk} due to OLE, CLE or DE shall be calculated as follows:
   1. For wharf units with 400 feet ≤ L_L ≤ 800 feet and 100 feet ≤ B ≤ 120 feet

$$V_{sk} = \beta \left(\frac{V_\Delta e}{L_L} \right) \tag{1-40}$$

   Where:

   L_L = Length of the shortest exterior wharf unit
   B = Width of wharf unit
   β = Factor determined as a function of wharf unit length, refer to Figure 1-7
   V_Δ = Total wharf lateral seismic force at the displacement demand determined using pushover analysis
   e = Eccentricity between the wharf center of mass and center of rigidity

   2. For wharf units with 800 feet < L_L ≤ 950 feet or 120 feet < B ≤ 140 feet, use β=1.5.
   3. For L_L > 950 feet or B > 140 feet, V_{sk} shall be determined using nonlinear time-history linked wharf analysis.
c. For determining wharf expansion joint shear capacity according to ACI-318, a reduction factor, ϕ of 0.85 shall be used.

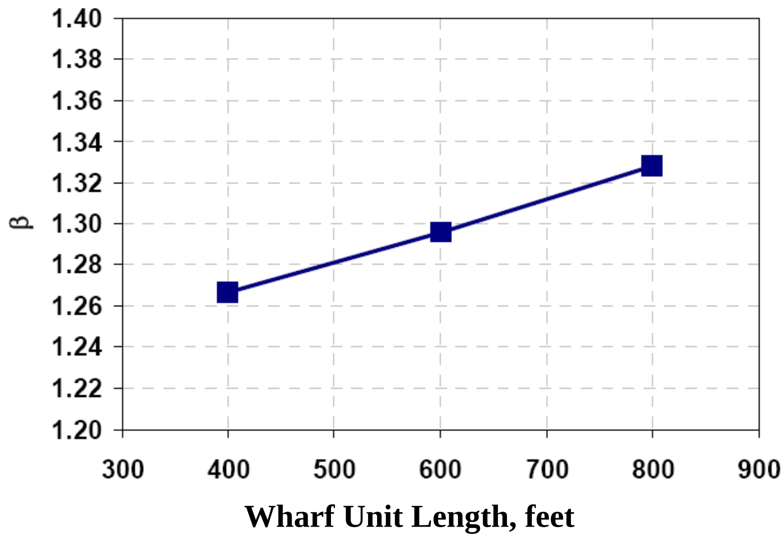

Figure 1-7: β Versus Wharf Unit Length

1.6.7 Detailing Requirements

a. The minimum concrete cover shall be 3 inches.

   Exception: For headed reinforcing bars such as pile dowels or shear stirrups, the cover may be reduced to 2 ½ inches at the top surface only.

b. All piles shall use ASTM A706 dowels to connect to the deck.

c. The pile-deck joint region for seismic pile shall be confined according to Section 1.6.6.2.

d. Dowels that are extended from piles into wharf deck or beam shall not be bent outwards.

e. If the principal tensile stress in the pile-deck joint region exceeds 3.5 $\sqrt{f_c'}$ where f_c' is the concrete compressive strength of the deck, additional joint shear reinforcements are required.

f. The extension of pile prestressing strands into the deck shall not be used for the pile-deck joint.

1.7 GEOTECHNICAL AND SOIL-STRUCTURE INTERACTION REQUIREMENTS

1.7.1 Liquefaction

Liquefaction potential of the soils in the immediate vicinity of or beneath the wharf structure and associated embankment or dike shall be evaluated. The strains in the piles induced by liquefaction effects shall not exceed the strain limits provided in Section 1.3.2.

1.7.2 Slope Stability and Seismically Induced Lateral Spreading

1.7.2.1 Static Slope Stability

a. Static slope stability analysis shall be performed for the embankment or dike.

b. Backland surcharge load shall be included in the analysis.

c. The minimum backland load shall be 250 pounds per square feet for the first 75 feet from the back end of the wharf, and 1,200 pounds per square feet for the remaining backland area.

d. The long-term static factor of safety of the embankment or dike shall not be less than 1.5.

e. For temporary conditions during construction, the static factor of safety shall not be less than 1.25 and the surcharge load value shall not be less than 250 pounds per square feet for the entire backland area.

1.7.2.2 Pseudo-static Seismic Slope Stability

a. Pseudo-static seismic slope stability analysis shall be performed for the embankment or dike to determine the yield acceleration coefficient, k_y.

b. Backland surcharge load shall be included in the analysis.

c. The minimum backland load shall be 250 pounds per square feet for the first 75 feet from the back end of the wharf and 800 pounds per square feet for the remaining backland area.

d. If liquefaction and/or strength loss of the site soils is expected, the residual strength of liquefied soils, strength compatible with the pore pressure generation of potentially liquefied soil and/or potential strength reduction of clays shall be used in the analysis.

1.7.2.3 Post-earthquake Static Slope Stability

a. Post-earthquake static slope stability analysis shall be performed for the embankment or dike.

b. Backland surcharge load shall be included in the analysis.

c. The minimum backland load shall be 250 pounds per square feet for the first 75 feet from the back end of the wharf and 800 pounds per square feet for the remaining backland area.

d. The static factor of safety immediately following OLE, CLE, or DE shall not be less than 1.1.

e. If liquefaction and/or strength loss of the site soils is expected, the residual strength of liquefied soils strength compatible with the pore pressure generation of potentially liquefied soil and/or potential strength reduction of clays shall be used in the analysis.

1.7.3 Lateral Spreading-free Field

a. The earthquake-induced lateral deformations of the embankment or dike and associated foundation soils shall be determined for OLE, CLE, and DE using the peak ground acceleration at the ground surface. The effects of liquefaction on soil properties shall not be included in the determination of peak ground acceleration.

b. If liquefaction and/or strength loss of the site soils is expected, residual strength of liquefied soils strength compatible with the pore pressure generation of potentially liquefied soil and/or potential strength reduction of clays shall be used in the analysis.

c. The presence of piles shall not be included in the "free field" evaluations.

1.7.4 Seismically Induced Settlement

Seismically induced settlement shall be addressed in the analysis and design for both unsaturated and saturated soils and its effects on piles.

1.7.5 Soil-structure Interaction

Inertial and kinematic load conditions shall be analyzed for the pile design as follows:

1.7.5.1 Inertial Load

a. Level ground inelastic lateral soil springs (p-y springs) shall be developed for the site specific soil conditions.

b. Upper bound estimates of the spring strength and stiffness shall be determined by multiplying the level ground, p-y springs values by a factor of 2.0.

c. Lower bound estimates of the spring strength and stiffness shall be determined by multiplying the level ground, p-y springs values by a factor of 0.3.

1.7.5.2 Kinematic Load

a. Kinematic load on seismic piles shall be calculated based on the site-specific conditions.

   Exception: For seismic piles with 24-inch diameter and having an embedment length of at least 20 feet into the dike, kinematic load need not be considered when the permanent free field embankment or dike deformations determined per Section 1.7.3 are less than 3 inches for OLE, less than 12 inches for CLE and less than 36 inches for DE.

b. Deformations shall be restricted so that the pile strains comply with Section 1.3.2.

1.7.5.3 Combination of Inertial and Kinematic Loads

a. The inertial load and kinematic load on seismic piles shall be combined.

   Exception: For seismic piles with 24-inch diameter and having an embedment length of at least 20 feet into the dike, inertial and kinematic pile loads need not be combined.

1.7.6 Earth Pressure

The earth pressure on the wharf structure resulting from static and seismic load conditions including the effect of pore water pressure in the backfill shall be calculated.

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CHAPTER 2
UPGRADE AND REPAIR CRITERIA FOR EXISTING WHARVES

2.1 PURPOSE AND SCOPE

This chapter provides requirements for the upgrade and repair of existing container wharves damaged by seismic or other natural disasters or events.

2.2 SEISMIC UPGRADES

a. Existing wharf seismic upgrade shall comply with the performance requirements of this Code for the design of a new wharf or as approved by POLA.

b. The overall seismic capacity of existing wharf shall not be reduced by the seismic upgrade.

c. Existing wharf seismic upgrade also includes embankment and dike.

2.3 STRUCTURAL REPAIRS

a. The damage to existing container wharves caused by seismic or other natural disasters or events shall be repaired in compliance with the requirements provided in this chapter based on the level of damage determined by the "Damage Ratio". A "Damage Ratio", expressed in a fraction or percent, shall be calculated as follows:

$$\text{Damage Ratio} = \frac{\text{Estimated Repair Cost}}{\text{Estimated Replacement Cost}}$$

Where:

Estimated Repair Cost is equal to an estimated cost of the repairs required to restore the damaged wharf members and components to comply with the requirements of this Code for the damaged wharf unit. Wharf members and components include decks, beams, piles, cut-off walls, embankments, dikes, all connections, and other supporting elements.

Estimated Replacement Cost is equal to an estimated cost of replacing the entire wharf unit.

b. When the Damage Ratio for structural damage does not exceed 0.1 (10%), the structural damages shall be repaired such that the existing wharf is restored, at a minimum, to the pre-event condition.

c. When the Damage Ratio for structural damage exceeds 0.1 (10%) but does not exceed 0.5 (50%), the damaged wharf members and components shall be repaired and strengthened such that all repaired and strengthened structural members, all connections associated with the damaged structural members, all structural members supported by the damaged members, and all structural members supporting the damaged members comply with the performance requirements of this Code for the design a new wharf.

d. When the Damage Ratio for structural damage exceeds 0.5 (50%), the entire existing wharf shall be repaired and strengthened as necessary such that the entire wharf complies with the performance requirements of this Code for the design of a new wharf.

e. Portions of existing wharf may be replaced with a new wharf which complies with the performance requirements of this Code for the design of a new wharf to satisfy wharf strengthening requirements.

f. The overall seismic capacity of existing wharf shall not be reduced by the repairs or replaced portions.

g. Wharf components also include embankments and dike.

2.4 NONSTRUCTURAL REPAIRS

a. For all Damage Ratios determined according to Section 2.3.a nonstructural repairs that do not adversely affect any structural member or any part of the existing wharf may be repaired with the same materials of which the wharf was constructed.

b. The overall seismic capacity of existing wharf shall not be reduced by the nonstructural repairs.

END OF THE CODE