Port of Long Beach
Wharf Design Criteria

POLB WDC Version 4.0

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A_{gross}	Gross cross-sectional area
A_{sc}	Total cross-sectional area of dowels in the joint
A_{sp}	Cross-sectional area of confining steel/ transverse reinforcement
B	Width of a wharf unit
BE	Berthing loads
BU	Buoyancy loads
C	Current loads on structure
D	Dead loads
D'	Diameter of confined core measured to the centerline of the confining steel
D_p	Pile diameter
DMF	Dynamic magnification factor
E	Earth pressure loads
E_c	Modulus of elasticity of concrete
E_{ps}	Modulus of elasticity for prestressing steel
E_s	Modulus of elasticity of steel
E_{sh}	Confining steel modulus of elasticity
EQ	Earthquake loads
F	Total lateral seismic force of the wharf strip considered at displacement demand
F_i	Lateral force per pile in row i from pushover analysis when seismic piles reach yield displacement
F_n	Lateral force determined from pushover curve for iteration n at $\Delta_{t,n-1}$
F_p	Prestress compressive force in pile
F_Δ	Total lateral seismic force of a wharf segment at displacement demand
H	The distance between the center of pile top plastic hinge and the center of pile in-ground plastic hinge
H'	The distance from the maximum in-ground moment to the center of gravity of the deck
I	Impact factor
I_{eff}	Effective moment of inertia
I_{gross}	Gross moment of inertia
J_{eff}	Effective polar moment of inertia
J_{gross}	Gross polar moment of inertia
k	Factor applied to dead load in earthquake load combination
K_e	Confinement effectiveness coefficient
L	Live loads
LB	Lower bound
L_c	The distance from the center of the pile top plastic hinge to the point of contraflexure
L_L	Length of the shorter exterior wharf unit
L_p	Plastic hinge length
L_s	Equivalent depth to point of fixity
L_u	Pile unsupported length
M	Mooring loads
M_n	Nominal moment capacity
M_o	Pile overstrength moment capacity

M_p	Pile idealized plastic moment capacity
$M_{p,in\text{-}ground}$	Pile plastic moment capacity at the in-ground plastic hinge including the effect of axial load due to crane dead load
$M_{p,\ top}$	Pile plastic moment capacity at the top plastic hinge including the effect of axial load due to crane dead load
M_y	Moment at first yield
N_u	External axial compression on pile including seismic load
P	Mooring line load
R	Creep loads
R_F	Force perpendicular to the fender panel due to berthing load
S	Shrinkage loads
T	Temperature loads
T_{crane}	Translational elastic period of the crane mode with the maximum participating mass
T_n	Effective period for iteration *n*
T_w	Effective elastic period of the wharf structure based on cracked section properties
T_{wi}	Transverse elastic period of a wharf segment
U	Total design load in moment, shear forces or axial forces
UB	Upper bound
V_a	Shear strength due to axial load
V_c	Concrete shear strength
V_F	Fender Shear Force
V_n	Nominal shear strength
V_o	Pile overstrength shear demand
V_p	Pile plastic shear
V_s	Transverse reinforcement shear strength
W	Wind loads on structure
W_{DL}	Effective dead load of the wharf strip considered
W_W	Waterside crane wheel load
W_L	Landside crane wheel load
X_1, X_2	Distance from the back of the wharf
c	Depth from the extreme compression fiber to the neutral axis at flexural strength
c_o	Clear concrete cover plus half the diameter of the transverse reinforcement
d_{bl}	Diameter of dowel reinforcement
d_{gap}	Distance between the top of the pile steel shell and the deck soffit
e	Eccentricity between the wharf center of mass and the center of rigidity
f_c	Concrete compression stress
f'_c	28-day unconfined concrete compressive strength
f'_{cc}	Confined concrete compressive strength
f'_{ce}	Expected compressive strength of concrete
f_l'	Effective lateral confining stress
f_{pu}	Maximum tensile strength of prestressing steel
f_{pue}	Expected maximum tensile strength of prestressing steel
f_{py}	Yield strength of prestressing steel
f_{pye}	Expected yield strength of prestressing steel

f_s	Steel tensile stress
f_{ue}	Expected maximum tensile strength of steel
f_y	Yield strength of longitudinal reinforcing steel or structural steel
f_{ye}	Expected yield strength of reinforcing steel/ structural steel
f_{yh}	Yield strength of confining steel/ transverse reinforcement
f_{yhe}	Expected yield strength of confining steel
i	Pile row
k	Curvature ductility factor determined as a function of μ_ϕ
k_e	System secant stiffness
$k_{e,n}$	Effective secant stiffness for iteration n at $\Delta_{t,n-1}$
k_i	Transverse elastic stiffness of a wharf segment
l_a	Actual embedment length of dowels anchored in the joint
l_{sp}	Strain penetration length
m	Seismic mass of a wharf segment
n	Iteration number (1, 2, 3,…n)
n_i	Total number of piles in row i for length L_L
p_1, p_2	Uniform backland load
s	Center-to-center spacing of confining steel/transverse reinforcement along pile axis
$v_\perp$	Approach velocity normal to fender line
α	Angle between the line joining the centers of flexural compression zones at the top and in-ground plastic hinges and the pile axis
β	Axial load shear strength factor
Δ_c	Displacement capacity
Δ_d	Displacement demand
$\Delta_{p,m}$	Pile plastic displacement capacity due to rotation of the plastic hinge at the OLE, CLE, or DE strain limits
Δ_t	Transverse displacement demand
$\Delta_{t,0}$	Assumed initial transverse displacement demand
$\Delta_{t,n}$	Transverse displacement for iteration n
$\Delta_{t,n-1}$	Transverse displacement for iteration $n-1$
Δ_{X1}, Δ_{X2}	Combined X-axis displacement demands from motions in the transverse and longitudinal directions
Δ_{XL}	X-axis displacement demand due to structure excitation in the longitudinal direction
Δ_{XT}	X-axis displacement demand due to structure excitation in the transverse direction
Δ_{Y1}, Δ_{Y2}	Combined Y-axis displacement demands from motions in the transverse and longitudinal directions
Δ_{YL}	Y-axis displacement demand due to structure excitation in the longitudinal direction
Δ_{YT}	Y-axis displacement demand due to structure excitation in the transverse direction
Δ_y	Pile yield displacement
Δ_{ys}	System yield displacement

ε_c	Concrete compression strain
ε_{cc}	Confined concrete compressive strain at maximum compressive stress
ε_{co}	Unconfined concrete compression strain at maximum compressive stress
ε_{cu}	Ultimate confined concrete compression strain
ε_p	Total prestressing steel tensile strain
ε_{pue}	Expected ultimate strain for prestressing steel
ε_{pye}	Expected yield tensile strain for prestressing steel
ε_s	Steel tensile strain
ε_{smd}	Strain at maximum stress of dowel reinforcement
ε_{sh}	Steel tensile strain at the onset of strain hardening
ε_{spall}	Ultimate unconfined compression (spalling) strain
ε_{ye}	Expected yield tensile strain for steel
ϕ	Reduction factor for nominal moment capacity according to ACI-318
ϕ_m	Total curvature at the OLE, CLE, or DE strain limits
$\phi_{p,dem}$	Plastic curvature at displacement demand
$\phi_{p,m}$	Plastic curvature at the OLE, CLE, or DE strain limit
ϕ_u	Ultimate curvature of the section
ϕ_y	Idealized yield curvature
ϕ_{yi}	Curvature at first yield
Φ	Strength reduction factor for shear
μ_n	System displacement ductility demand at iteration n
μ_ϕ	Curvature ductility demand
μ_f	Friction coefficient
θ	Angle of critical shear crack with respect to the longitudinal axis of the pile
θ_m	Total rotation at the OLE, CLE, or DE strain limits
$\theta_{p,m}$	Plastic rotation at the OLE, CLE, or DE strain limits
$\theta_{p,dem}$	Plastic rotation at displacement demand
θ_u	Ultimate rotation
θ_y	Idealized yield rotation
ρ	Volumetric ratio of longitudinal reinforcing steel
ρ_s	Effective volumetric ratio of confining steel
$\xi_{eff,n}$	Effective system damping for iteration n at $\Delta_{t,n-1}$

Acronyms/Definitions

AASHTO	American Association of State Highway and Transportation Officials
AC	Asphalt Concrete
ACI	American Concrete Institute
AISC	American Institute of Steel Construction
AF&PA	American Forest and Paper Association
ANSI	American National Standards Institute
AREMA	American Railway Engineering and Maintenance-of-Way Associates
ASD	Allowable Stress Design
ATC	Applied Technology Council
AWS	American Welding Society
CALTRANS	California Department of Transportation
CBC	California Building Code
CLE	Contingency Level Earthquake
Cooper E-80	Railroad load type per AREMA
CPT	Cone Penetration Test
CQC	Complete Quadratic Combination
c.g.	Center of gravity
DCR	Demand-to-capacity Ratio
DE	Code-level Design Earthquake
DMG	Division of Mines and Geology
e.g.	For example
FEMA	Federal Emergency Management Agency
FHWA	Federal Highway Administration
FOS	Factor of Safety
ft	Foot/ Feet
HL-93	Truck load type per AASHTO
in.	Inch/ Inches
Joint	Pile beam/deck joint
klf	Kips per foot
ksi	Kips per square foot
LOA	Length Overall
LRFD	Load Resistance Factor Design
MCE_G	Geometric Mean Maximum Considered Earthquake
MCEER	Multidisciplinary Center for Earthquake Engineering Research
MHHW	Mean Higher-High Water
MHW	Mean High Water
MLLW	Mean Lower-Low Water
MLW	Mean Low Water
MSL	Mean Sea Level
mph	Miles per hour
$M\text{-}\phi$	Moment-curvature analysis

NAVD 88	North American Vertical Datum of 1988
NAVFAC	Naval Facilities Engineering Command
NCEER	National Center for Earthquake Engineering Research
NCHRP	National Cooperative Highway Research Program
NDS	National Design Specification
NEHRP	National Earthquake Hazards Reduction Program
NGVD 29	National Geodetic vertical Datum of 1929
NSF	National Science Foundation
NTHA	Nonlinear Time-History Analysis
N/A	Not Applicable
OLE	Operating Level Earthquake
PCI	Prestressed Concrete Institute
PIANC	International Navigation Association
PGA	Peak ground acceleration
POLB	Port of Long Beach
pcf	Pounds per cubic foot
psf	Pounds per square foot
p-q	Pile tip soil springs
p-y	Pile lateral soil springs
RO-RO	Roll-on/Roll-off vessels
SDC	Seismic Design Criteria
SLC	State Lands Commission
SLD	Service Load Design
t-z	Pile axial soil springs
UCSD	University of California at San Diego
UFC	Unified Facilities Criteria
USACE	United States Army Corps of Engineers
WDC	Wharf Design Criteria
Wharf exterior unit	A wharf structure with an expansion joint at one end
Wharf interior unit	A wharf structure with expansion joints at both ends

1 Introduction

This document contains design guidelines and criteria for pile supported wharf construction, other structures may need to be considered differently. It is published by the Port of Long Beach (POLB or Port) to assist engineering staff of the POLB, as well as consulting firms providing consulting services related to the design of wharves for the POLB. Any deviation from the criteria listed herein will require specific prior written approval from the Port.

Design guidelines and reference materials cited throughout this document will be revised from time to time as required. Updates and revisions occurring during design shall be followed as directed by the Port.

This document is Version 4.0 of the "Port of Long Beach Wharf Design Criteria" and it supersedes the previous Version 3.0 that was published on February 29, 2012, Version 2.0 that was published on January 30, 2009, and Version 1.0 that was published in March 2007.

This document was prepared for the POLB under the leadership of Cheng Lai, P.E., S.E., Deputy Chief Harbor Engineer, POLB, and by a team of consultants consisting of Moffatt & Nichol (M&N), WKE, Inc., and Earth Mechanics, Inc. (EMI). The expert review team included Dr. Nigel Priestley, Emeritus Professor, Department of Structural Engineering, University of California, San Diego and Dr. Geoffrey Martin, Emeritus Professor, Department of Civil Engineering, University of Southern California.

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2 Geotechnical Considerations

Geotechnical evaluations identified in this section shall use methodologies that are considered acceptable standards of practice in the industry.

For seismic evaluations, ground motion criteria provided in Section 2.1 shall be used. Ground motions and response spectra are provided in the “Port-Wide Ground Motion Study, Port of Long Beach, California” (Ref. 20) and “Final Addendum No. 3 to Port-wide Ground Motion Study, Port of Long Beach, California” (Ref. 21). No deviation from these ground motions shall be allowed without prior written approval by the Port.

These guidelines are specific to pile-supported marginal wharves with engineered sloping ground conditions located under the wharf structure comprising dredged soils or cut slopes protected or stabilized by quarry run rock material. Applicability of these guidelines to other structures may be allowed upon written approval by the Port.

2.1 Ground Motions

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

Operating Level Earthquake (OLE)

The OLE is defined as the seismic event that produces ground motions associated with a 72-year return period. The 72-year return period ground motions have a 50% probability of being exceeded in 50 years. The OLE event occurs more frequently than the CLE and DE events and has a lower intensity. Recommended response spectra for OLE for different ground conditions are provided in “Port-Wide Ground Motion Study, Port of Long Beach, California” (Ref. 20).

Contingency Level Earthquake (CLE)

The CLE is defined as the seismic event that produces ground motions associated with a 475-year return period. The 475-year return period ground motions have a 10 percent probability of being exceeded in 50 years. The CLE event occurs less frequently than the OLE event, but more frequently than the DE event. The CLE event has a higher intensity than the OLE event, but lower intensity than the DE event. Recommended response spectra for CLE for different ground conditions are provided in “Port-Wide Ground Motion Study, Port of Long Beach, California” (Ref. 20).

Code-level Design Earthquake (DE)

The DE shall comply with the Design Earthquake requirements of the current California Building Code (Ref. 16). The DE event occurs less frequently than the OLE and CLE events and has a higher intensity than the other two events.

Recommended response spectra for DE for different ground conditions are provided in "Final Addendum No. 3 to Port-wide Ground Motion Study, Port of Long Beach, California" (Ref. 21). This reference also provides peak ground accelerations that should be used for geotechnical evaluations.

2.2 Site Characterization

Site characterization shall be based on site-specific information. Reviewing and cataloging available geotechnical information from past Port projects shall be performed to maximize the use of available data and to avoid conducting additional explorations where information already exists.

The presence of known active faults shall be verified using the available geological information such as the California Geological Survey (Ref. 24) or other appropriate documents. If a new fault is found at the project site, a peer review is required per Section 4.14.

Adequate coverage of subsurface data, both horizontally and vertically, shall be provided to develop geotechnical parameters that are appropriate for the project. An adequate number of explorations should extend to depths of at least 20 feet below the deepest anticipated foundation depths and should be deep enough to characterize subsurface materials that are affected by embankment behavior. Particular attention should be given during the field exploration to the presence of continuous low-strength layers or thin soil layers that could liquefy or weaken during the design earthquake shaking or cause embankment failure during dredging or other construction activities. Cone penetration tests (CPT) provide continuous subsurface profile and, therefore, should be used on large projects to complement exploratory borings. When CPTs are performed, at least one boring shall be performed next to one of the CPT soundings to check that the CPT-soil behavior type interpretations are reasonable for the project site. Any differences between CPT interpretations and subsurface conditions obtained from borings shall be reconciled prior to developing geotechnical design parameters.

An appropriate and sufficient number of laboratory tests shall be performed to provide the necessary soil parameters for geotechnical evaluations. Guidelines for site characterization can be found in "Soil Mechanics" (Ref. 34) and "Design and Construction of Driven Pile Foundations" (Ref. 22) or other appropriate documents.

2.3 Liquefaction Potential

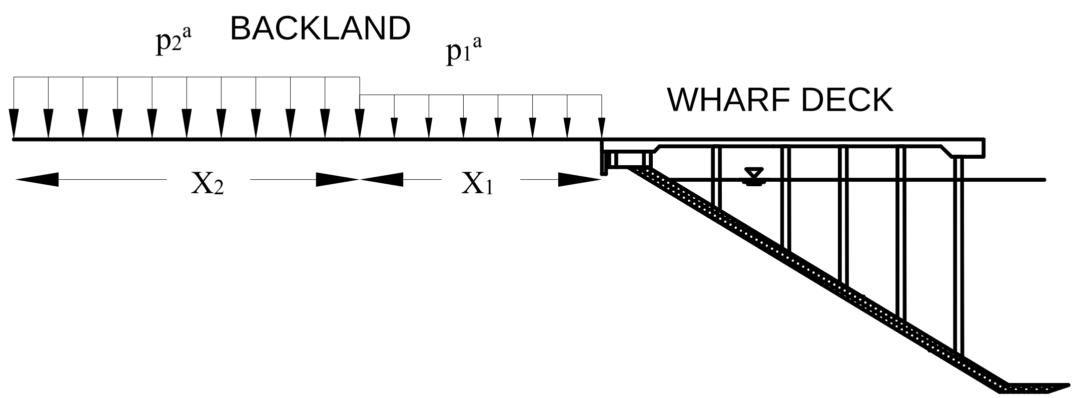
Liquefaction potential of the soils in the immediate vicinity of or beneath the wharf structure and associated embankment or rock dike shall be evaluated for the OLE, CLE, and DE. When performing geotechnical evaluations of wharf sites that are accessible to the general public, peak ground acceleration corresponding to geometric mean maximum considered earthquake (MCE_G) as provided in Final Addendum No. 3 to Port-Wide Ground Motion Study Report (Ref. 21) shall be used for liquefaction and associated strength loss evaluations, per current CBC (Ref. 16). For wharves that are not accessible to the general public, two-thirds of the MCE_G peak ground acceleration shall be used for liquefaction and associated strength loss evaluations. Liquefaction potential evaluation should follow the procedures outlined in "Liquefaction Resistance of Soils: Summary Report from the 1996 NCEER and 1998 NCEER/NSF Workshops on Evaluation of Liquefaction Resistance of Soils" (Ref. 45), "Recommended Procedures for Implementation of DMG Special Publication 117, Guidelines for Analyzing and Mitigating Liquefaction Hazards in California" (Ref. 29), "Chapter 31F, 2013 California Building Code" (Ref. 17), "Liquefaction Susceptibility Criteria for Silts and Clays" (Ref. 14), "Criteria for Liquefaction of Silty Soils," (Ref. 7), and "Assessment of the Liquefaction Susceptibility of Fine-Grained Soils" (Ref. 15).

If liquefaction is shown to be initiated in the above evaluations, the particular liquefiable strata and their thicknesses, including zones of liquefaction induced in the backland area, should be clearly shown on site profiles. Resulting hazards associated with liquefaction should be addressed, including translational or rotational deformations of the slope or embankment system and post liquefaction settlement of the slope or embankment system and underlying foundation soils. If such analyses indicate the potential for partial or gross failure of the embankment, adequate evaluations shall be performed to confirm such conditions exist. In these situations, and for projects where more detailed numerical analyses are performed, a peer review is required per Section 4.14.

2.4 Slope Stability and Seismically Induced Lateral Spreading

The surcharge loading values for different loading conditions and the required minimum factors of safety values are discussed in Sections 2.4.1, 2.4.2, and 2.4.3 and presented in Table 2-1. These recommended surcharge loading values may be revised based on project-specific load information, upon prior written approval by the Port.

Table 2-1: Minimum Requirement for Slope Stability Analyses



Load Condition	p_1[a] (psf)	X_1 (ft)	p_2[a] (psf)	X_2 (ft)	Min. FOS[b]
Static Condition	250	75	1,200	Remaining Backland	1.5
Temporary Condition (See Section 2.4.1)	250	Entire Backland	-	-	1.25
Pseudo-static Seismic Condition	250	75	800	Remaining Backland	-[c]
Post-earthquake Static Condition	250	75	800	Remaining Backland	1.1

[a] Load values may be revised based on project-specific information, upon prior written approval by the Port.

[b] FOS – Factor of Safety.

[c] Yield acceleration shall be obtained from the analysis to determine lateral deformations per Section 2.9.2.

2.4.1 Static Slope Stability

Static slope stability analysis shall be performed for the slope or embankment system. Backland loading shall be considered in the analyses. Slope stability analyses should follow guidelines outlined in "Recommended Procedures for Implementation of DMG Special Publication 117, Guidelines for Analyzing and Mitigating Landslide Hazards in California" (Ref. 12), or other appropriate documents. Backland loading shall be 250 psf for the first 75 feet from the back end of the wharf deck and 1,200 psf for the remaining backland area, see Table 2-1. The long-term static factor of safety of the slope or embankment shall not be less than 1.5.

For temporary conditions, the static factor of safety shall not be less than 1.25. The loading considerations shall be based on project-specific information (such as terminal operation, construction staging, etc.). The surcharge loading value shall not be less than 250 psf for the entire backland area, see Table 2-1.

2.4.2 Pseudo-static Seismic Slope Stability

Pseudo-static seismic slope stability analyses shall be performed to estimate the horizontal yield acceleration for the slope for the OLE, CLE, and DE. During the seismic event, the backland loading shall be 250 psf for the first 75 feet from the back end of the wharf deck and 800 psf for the remaining backland area, see Table 2-1.

If liquefaction and/or strength loss of the site soils is likely, residual strength of liquefied soils, strengths compatible with the pore-pressure generation of potentially liquefiable soils, and/or potential strength reduction of clays shall be used in the analysis. The residual strength of liquefied soils should be estimated using guidelines outlined in “Recommended Procedures for Implementation of DMG Special Publication 117, Guidelines for Analyzing and Mitigating Liquefaction Hazards in California” (Ref. 29), “SPT- and CPT-Based Relationships for the Residual Strength Shear Strength of Liquefied Soils,” (Ref. 25), “Liquefied Strength Ratio for Liquefaction Flow Failure Case Histories,” (Ref. 35), or other appropriate documents.

Using a seismic coefficient of one-half of the PGA or 0.15g, whichever is greater, in the pseudo-static seismic slope stability analyses the factor of safety shall be estimated without considering the presence of wharf piles. 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 2.4.4 and 2.9.2 is necessary.

2.4.3 Post-earthquake Static Slope Stability

The static factor of safety immediately following OLE, CLE or DE event shall not be less than 1.1 when post-earthquake residual strength of liquefied soils, strengths compatible with the pore-pressure generation of potentially liquefiable soils, and/or potential strength reduction of clays are used in the static stability analysis. The backland loading for post-earthquake stability analyses shall be 250 psf for the first 75 feet from the back end of the wharf deck and 800 psf for the remaining backland area, see Table 2-1.

2.4.4 Lateral Spreading – Free-Field

The earthquake-induced lateral deformations of the slope or embankment and associated foundation soils shall be determined for the OLE, CLE, and DE using the peak ground acceleration at the ground surface (not modified for liquefaction) based on the “Port-Wide Ground Motion Study, Port of Long Beach, California” (Ref. 20) and “Final Addendum No. 3 to Port-Wide Ground Motion Study, Port of Long Beach, California” (Ref.21). If liquefaction and/or strength loss of the site soils is likely, residual strength of liquefied soils, strengths compatible with the pore-pressure generation of potentially liquefiable soils, and/or potential strength reduction of clays should be used in the analysis. The wharf piles should not be included in the “free-field” evaluations.

Additional analyses may be performed with prior written approval by the Port.

2.5 Settlement

2.5.1 Static Consolidation Settlement

Long-term static consolidation settlement of sites that are underlain by continuous or large lenses of fine-grained soils shall be evaluated. The long-term static settlement should be estimated following guidelines outlined in “Foundation and Earth Structures” (Ref. 33) or other appropriate documents. If long-term settlement is anticipated, the resulting design impacts shall be considered, including the potential for development of downdrag loads on piles (See Section 2.7.1).

2.5.2 Seismically Induced Settlement

Seismically induced settlement shall be evaluated. The seismically induced settlement should be based on guidelines outlined in “Recommended Procedures for Implementation of DMG Special Publication 117, Guidelines for Analyzing and Mitigating Liquefaction Hazards in California” (Ref. 29) or other appropriate documents. If seismically induced settlement is anticipated, the resulting design impacts shall be considered, including the potential development of downdrag loads on piles (See Section 2.7.1).

2.6 Earth Pressures

2.6.1 Earth Pressures under Static Loading

The effect of static active earth pressures on wharf structures resulting from static loading of backfill soils shall be considered where appropriate. Backfill sloping configuration, if applicable, and backland loading conditions shall be considered in the evaluations. The loading considerations shall be based on project-specific information, with a minimum assumed surcharge loading value of 250 psf. The earth pressures under static loading should be based on guidelines outlined in “Foundation and Earth Structures” (Ref. 33) or other appropriate documents.

2.6.2 Earth Pressures Under Seismic Loading

The effect of earth pressures on wharf structure resulting from seismic loading of backfill soils, including the effect of pore-water pressure build-up in the backfill, shall be considered. The seismic coefficients used for this analysis should be based on the earthquake magnitudes, peak ground accelerations, and durations of shaking provided in “Port-Wide Ground Motion Study, Port of Long Beach, California” (Ref. 20) and “Final Addendum No. 3 to Port-wide Ground Motion Study Report, Port of Long Beach, California” (Ref. 21). Backfill sloping configuration, if applicable, and backland loading conditions shall be considered in the evaluations. The loading considerations shall be based on project-specific information, with a minimum assumed surcharge loading value of 250 psf. Mononabe-Okabe equations may be used to estimate earth pressures under seismic loading, if appropriate. Refer to “Foundation and Earth Structures” (Ref. 33); “Seismic Analysis and Design of Retaining Walls, Buried Structures, Slopes, and Embankments” (Ref. 42). If Mononabe-Okabe equations are not appropriate, methods outlined in “Seismic Analysis and Design of Retaining Walls, Buried Structures, Slopes, and Embankments” (Ref. 42) or other appropriate methods may be used.

2.7 Pile Axial Behavior

These guidelines are based on the assumption that piles are driven into the dense to very dense soil layer that is generally present throughout the Port area at elevations approximately -80 feet to -100 feet MLLW and below. If piles are not embedded into this layer, additional guidelines may be applicable and the geotechnical engineer should provide recommendations for review and approval by the Port.

2.7.1 Pile Capacity

Axial geotechnical capacity of piles shall be evaluated using the load combinations in Table 3-4. Guidelines for estimating axial pile capacities are provided in "Foundation and Earth Structures" (Ref. 33), "Recommended Procedures for Planning, Designing, and Constructing Fixed Offshore Platforms" (Ref. 5), and other appropriate documents. A minimum factor of safety of 2.0 shall be achieved on the ultimate axial capacity of pile when using the largest of the service load combinations provided in Table 3-4. In addition, piles supporting the waterside crane rail girder should have a minimum safety factor of 1.5 on ultimate axial capacity of pile when using the broken pile load combinations provided in Table 3-1.

If long-term soil settlement is anticipated (See Section 2.5.1) above the pile tip, the effects of downdrag on axial geotechnical and structural capacity of piles shall be evaluated. The geotechnical capacity when evaluating the effects of downdrag loads should be estimated by considering only the tip resistance of the pile and the side friction resistance below the lowest layer contributing to the downdrag. Due to the short-term nature of transient loads, the factor of safety for the downdrag load evaluation may be reduced when downdrag loads are combined with transient loads. A minimum factor of safety of 1.5 should be achieved when combining the downdrag with the maximum of the service load estimated using load combination per Table 3-4.

For the earthquake load case, 10% of the design uniform live load should be included, per Section 4.5.2. However, the factor of safety should not be less than 2.0 when downdrag loads are combined with dead loads only. The geotechnical engineer should provide the magnitude of the downdrag load and its extent along the pile to the structural engineer.

An alternate approach to the evaluation of long-term settlement induced downdrag loads is to estimate the pile top settlement under the downdrag load plus service load and to design the structure to tolerate the resulting settlement.

If liquefaction or seismically-induced settlement are anticipated (See Section 2.5.2), the ultimate pile axial geotechnical capacity under seismic conditions shall be evaluated for the effects of liquefaction and/or downdrag forces on the pile. The ultimate geotechnical capacity of the pile during liquefaction should be determined on the basis of the residual strength of the soil for those layers where the factor of safety for liquefaction is determined to be less than or equal to 1.0. When seismically-induced settlements are predicted to occur during design earthquakes, the downdrag loads should be calculated, and the combination of downdrag load and earthquake load should be determined. Only the tip resistance of the pile and the skin friction resistance below the lowest layer contributing to the downdrag should be used in the capacity evaluation. The ultimate axial capacity of the pile should not be less than the combination of the seismically induced downdrag load and the maximum of the earthquake load combinations, refer to Section 4.5.2.

2.7.2 Axial Springs for Piles

The geotechnical engineer shall coordinate with the structural engineer and develop axial springs (T-z) for piles. The t-z springs may be developed either at the top or at the tip of the pile, see Figure 2-1. If the springs are developed at the pile tip, the tip should include both the skin frictional resistance along the pile (i.e., side springs [T-z]) and tip resistance at the pile tip (i.e., tip springs [q-w]), as illustrated in Figure 2-1. If T-z springs are developed at the pile top, the appropriate elastic axial stiffness of the pile should also be included in the springs. Linear or nonlinear springs may be developed if requested by the structural engineer.

Normally, it is assumed that the soil resistance along the side of the pile is developed at very small displacement (e.g., less than 0.5 inches) while the resistance at the tip of the pile will require large displacements (e.g., 5% of the pile diameter), (Ref. 23).

2.7.3 Upper and Lower Bound Springs

Due to the uncertainties associated with the development of axial springs (t-z), such as the axial soil capacity, load distributions along the pile, and the simplified spring stiffnesses used, both upper bound (UB) and lower bound (LB) limits should be used for the axial springs. The UB and LB springs should be developed by multiplying the load values estimated in Section 2.7.2 by 2 and 0.5, respectively, to be used in the structural analysis. Different values may be acceptable if supported by rational analysis and/or testing and upon written approval by the Port.

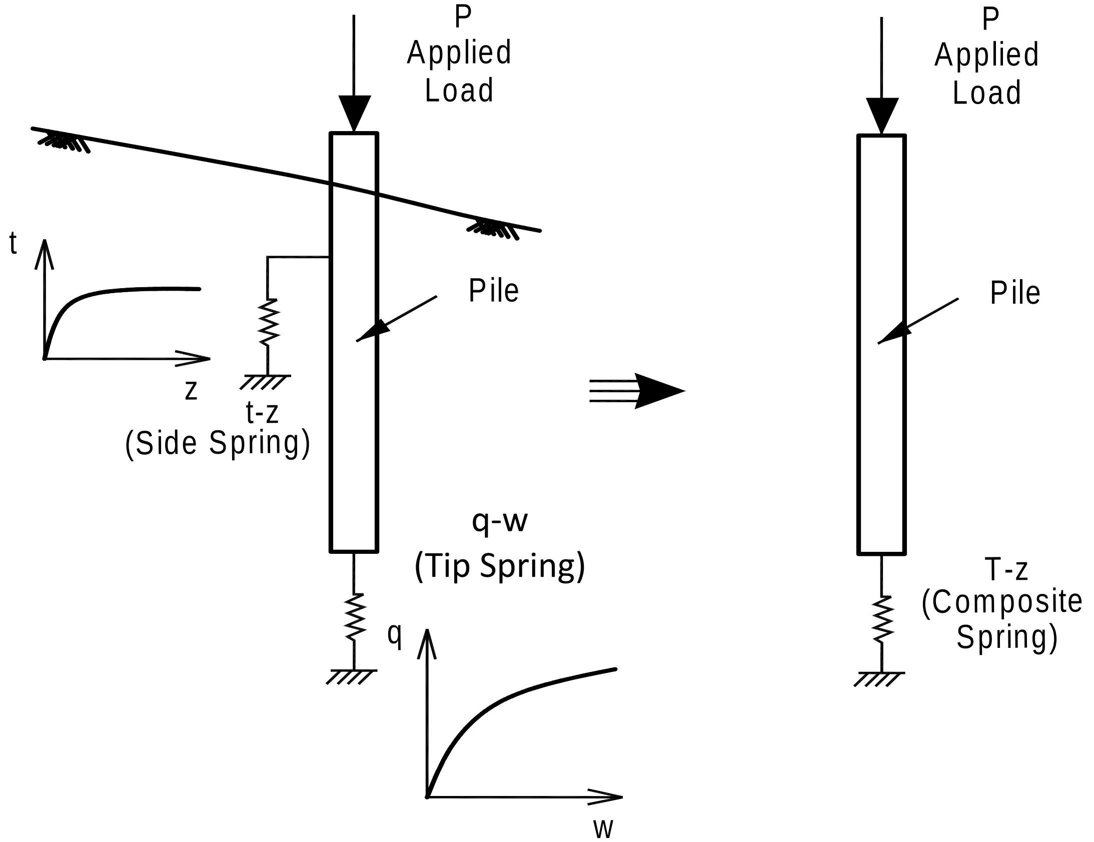


Figure 2-1: Axial Soil Springs

2.8 Soil Behavior under Lateral Pile Loading

2.8.1 Soil Springs for Lateral Pile Loading

For the design of piles under loading associated with the inertial response of the wharf structure, level-ground inelastic lateral springs (p-y) shall be developed. The lateral springs within the shallow portion of the piles (generally within 10 pile diameters below the ground surface) tend to dominate the inertial behavior. Geotechnical parameters for developing lateral soil springs may follow guidelines provided in "Recommended Practice for Planning, Designing, and Constructing Fixed Offshore Platforms" (Ref. 5) or other appropriate documents.

2.8.2 Upper and Lower Bound Soil Springs

Due to uncertainties associated with the development of lateral springs (p-y), such as uncertainties arising from rock properties, rock placement method, and sloping rock dike configuration, UB and LB p-y springs shall be developed for use in the wharf structure inertial response analyses. For level-ground configuration, the UB and LB springs shall use 1.25 times and 0.8 times the load values of the lateral spring developed per Section 2.8.1. For typical marginal container wharf slope/embankment/dike system at the Port, the UB and LB springs shall use 2 times and 0.3 times the load values of the lateral spring developed per Section 2.8.1. These UB and LB multipliers are intended to be used along the maximum slope of the dike for slopes between 1.5H:1V and 1.75H:1V. The range between UB and LB multipliers will be different with flatter and steeper slopes. For flatter slopes, the range between UB and LB multipliers is expected to be smaller. For steeper slopes, the range between UB and LB multipliers is expected to be larger. For dike slopes that are outside the range between 1.5H:1V and 1.75H:1V, slope-specific UB and LB multipliers should be developed and submitted to the Port for approval.

Upon written approval by the Port, rational analysis and/or testing may be performed to justify the use of different values. For other wharf slope/embankment/dike types, the UB and LB springs should be developed on a site-specific basis.

2.9 Soil-pile Interaction

Two separate load conditions for the piles analysis shall be considered: (1) Inertial loading under OLE, CLE and DE, and (2) Kinematic loading from lateral ground spreading. Inertial loading is associated with earthquake-induced lateral loading on the wharf structure, while kinematic loading refers to the loading on wharf piles from earthquake induced lateral deformations of the slope/embankment/dike system.

For typical marginal container wharves at the Port (vertical pile wharf configurations with typical slope/embankment/dike system), the inertial loading condition induces maximum moments in the upper regions of the pile, and the kinematic loading condition induces maximum moments in the lower regions of the pile. The locations of the maximum moments from these two load conditions are sufficiently far apart so that the effects of moment superposition are normally negligible. Furthermore, maximum moments induced by the two load conditions tend to occur at different times during the earthquake. Therefore, for typical marginal container wharves at the Port, these load conditions can be uncoupled (separated) from each other during design. For other wharf types, this assumption should be verified on a project-specific basis.

2.9.1 Inertial Loading Under Seismic Conditions

The evaluation of wharf structure response under inertial loading is discussed in Section 4. The lateral soil springs developed following the guidelines provided in Section 2.8 shall be used in the inertial loading response analyses. The wharf structure analysis under inertial loading can be performed by ignoring the slope/embankment/dike system deformations (i.e., one end of the lateral soil spring at a given depth is attached to the corresponding pile node and the other end is assumed fixed).

2.9.2 Kinematic Loading from Lateral Spreading

Kinematic loading from permanent ground deformation in the deep seated levels of the slope/embankment/dike foundation soils shall be evaluated. The lateral deformations shall be restricted to such amounts that the structural performance of wharf piles is not compromised as defined by pile strain limits outlined in Table 4-1. The lateral deformation of the embankment or dike and associated wharf piles and foundation soils shall be determined using proven analytical methods as outlined below (Figure 2-2).

Analysis for kinematic loading may not be required if it can be shown that a previously conducted dynamic soil-structure interaction analysis of a similar wharf representing a conservative upper bound solution results in higher pile curvature demands than the wharf under consideration, and still satisfies the strain limits for the pile.

Where analysis is required, initial estimates of free-field dike deformations (in the absence of piles) may be determined using the simplified Newmark sliding block method using the curves provided in “Port-Wide Ground Motion Study, Port of Long Beach, California” (Ref. 20) for the OLE and CLE, and “Final Addendum No. 3 to Port-wide Ground Motion Study, Port of Long Beach, California” (Ref. 21) for DE, as discussed in Section 2.4.4. For the 24-inch octagonal, precast, prestressed concrete piles and pile configurations that are typically used for Port container wharf structures, deformations are generally considered acceptable in terms of pile strain limits and performance criteria when the permanent free-field dike deformations are less than about 3 inches for the OLE, less than about 12 inches for the CLE and less than about 36 inches for DE conditions. Additional kinematic analysis is not required if the free-field dike deformations are less than these limits.

In cases where dike deformations estimated using the simplified Newmark sliding block method exceed the above displacement limits, site-response evaluations may be necessary to revise the free-field dike deformation analyses. Upon written approval by the Port, one-dimensional site response analyses may be performed to incorporate local site effects in developing site-specific acceleration time-histories at the base of the sliding block (“within motions”) for Newmark analyses. For the OLE and CLE, the firm-ground time-histories provided in “Port-Wide Ground Motion Study, Port of Long Beach, California” (Ref. 20) should be used as the basis for determining input in the site-response evaluations. For the DE, the firm-ground time-histories provided in “Final Addendum No. 3 to Port-wide Ground Motion Study, Port of Long Beach, California” (Ref. 21) should be used. Sensitivity analyses should also be performed on factors affecting the results. The site-specific time-histories representing the “within motions” should then be used in the simplified Newmark sliding block method to revise the dike deformation estimates. If the revised dike deformations still exceed the acceptable values, more detailed numerical soil-structure interaction evaluations may be necessary.

A full soil-structure interaction numerical analysis for kinematic loading may not be required if it can be shown by structural analysis that reduced displacement demands estimated by simplified Newmark evaluations incorporating pile “pinning” effects are structurally acceptable, as discussed in the following publications: “Recommended LRFD Guidelines for the Seismic Design of Highway Bridges” (Ref. 9) and “Seismic Analysis and Design of Pile Supported Wharves” (Ref. 13). The geotechnical engineer should provide the structural engineer with level-ground p-y springs for the weak soil layer and soil layers above and below the weak layer using appropriate overburden pressures for performing a simplified pushover analysis to estimate the OLE, CLE and DE displacement capacities and corresponding pile shear within the weak soil zone. For the pushover analysis, the estimated displacements may be uniformly distributed within the thickness of the weak soil layer (i.e., zero at and below the bottom of the layer to the maximum value at and above the top of the weak layer). At some distance above the weak soil layer (at least 15 Pile Diameter, 15 D_p), the pile may be fixed against rotation and at some distance below the weak layer, the pile should be fixed against rotation and translation (Figure 2-3).

Between these two points, lateral soil springs are provided, which allow deformation of the pile relative to the deformed soil profile. The geotechnical engineer should perform pseudo-static slope stability analysis (Section 2.4.2) with the "pinning" effects of piles arising from pile shear in the weak zone incorporated and estimate the displacement demands using simplified Newmark analysis. If the estimated displacement demands are less than the displacement capacities as defined by the structural engineer, no further analysis for kinematic loading will be necessary.

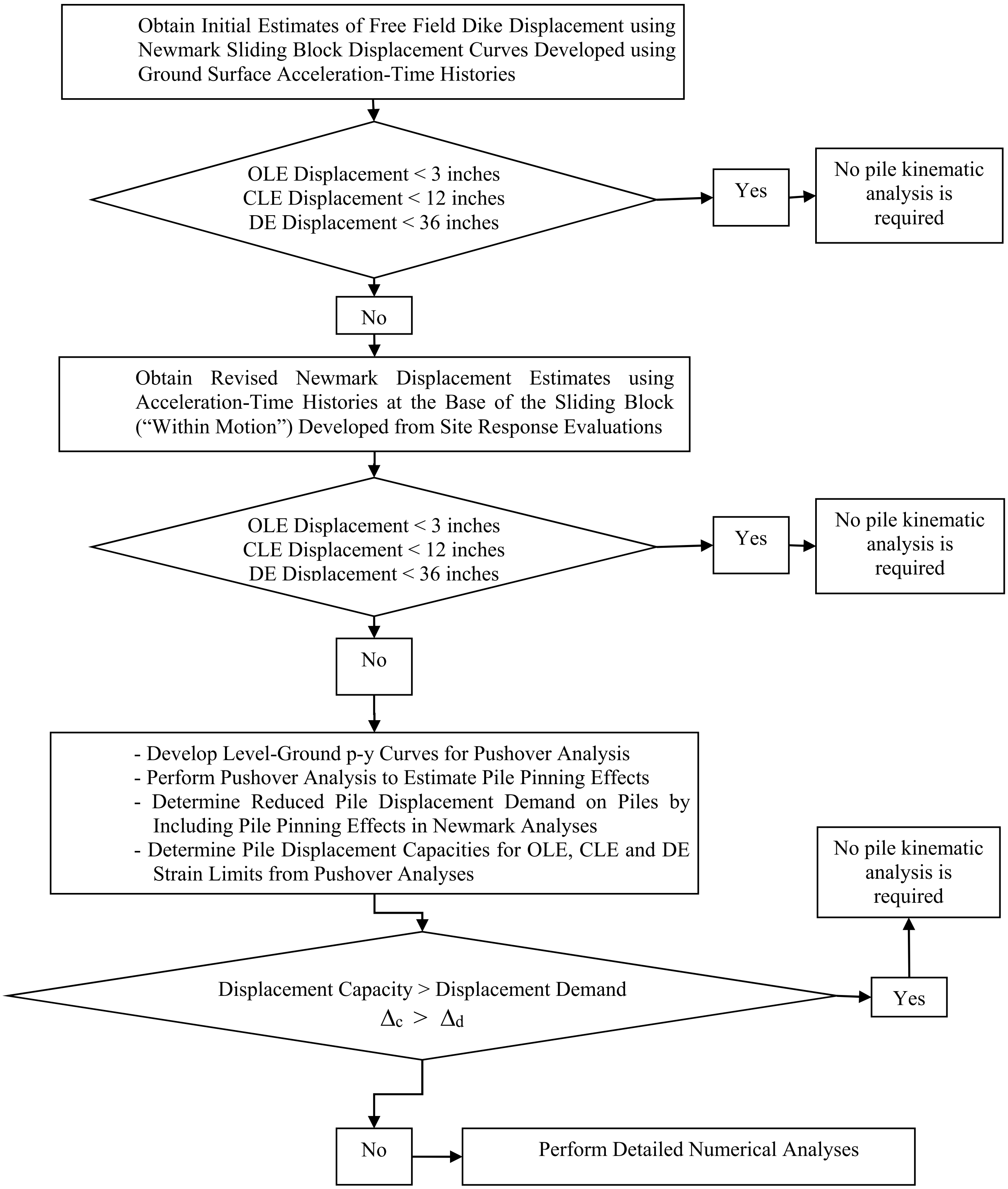


Figure 2-2: Flow Diagram for Evaluation of Kinematic Lateral Spread Loading for OLE, CLE and DE

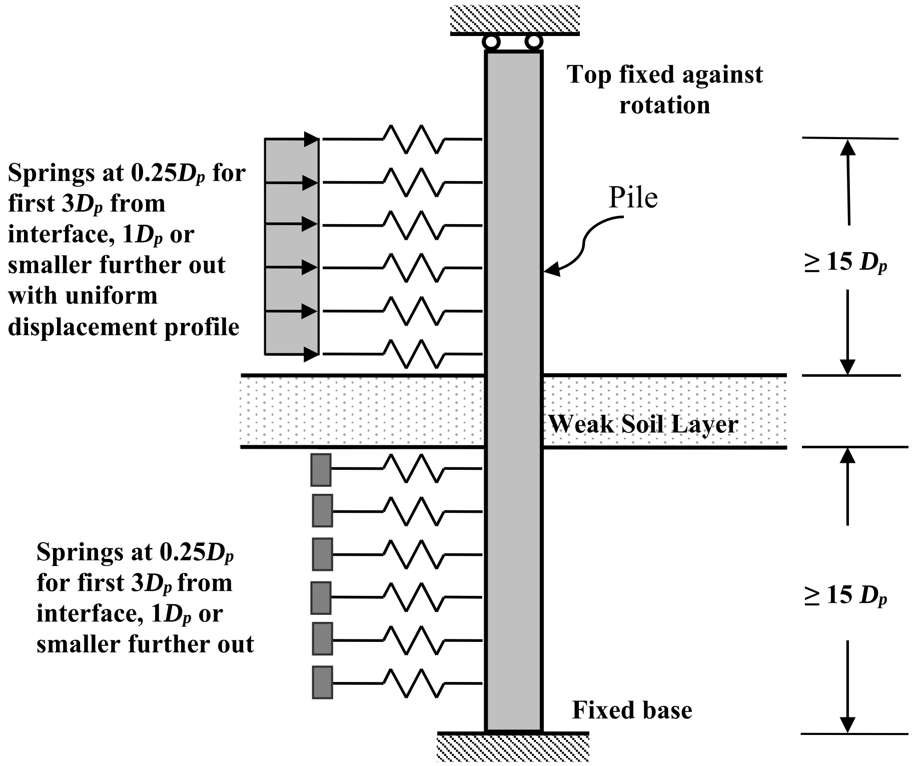


Figure 2-3: Sliding Layer Model

In cases where subsurface conditions indicate the presence of continuous, thin (less than 2 feet), liquefiable and/or soft soils beneath the dike that could result in concentrated deformations within these layers, more detailed numerical analyses may be necessary. Such analyses shall not be performed without prior written approval by the Port.

If more detailed numerical analyses are deemed necessary to provide input to the structural engineer, two-dimensional dynamic soil-structure interaction analysis of the wharf-pile-dike-soil system using numerical finite element or finite difference analyses should be performed. Sensitivity analyses should also be performed on factors affecting the results. As a minimum, deformation profiles along the length of the various pile rows should be provided to the structural engineer to estimate strains and stresses in the piles for the purpose of checking performance criteria. Such analyses should be coordinated with the structural engineer and shall not be performed without prior written approval by the Port.

2.10 Ground Improvement

In the event that all the requirements set forth in the above sections cannot be met for a project, ground improvement measures may be considered to meet the requirements. Prior written approval from the Port should be obtained before performing ground improvement evaluations. Ground improvement design recommendations should incorporate construction considerations including constructability, availability of contractors and equipment, schedule impact, and construction cost. Alternatives such as use of additional piles, or accepting greater damage due to larger displacements shall be considered and discussed with the Port.

3 Structural Loading Criteria

3.1 General

All container terminal wharves shall be designed for the loading requirements provided in Section 3, other structures may need to be considered differently. Where loading conditions exist that are not specifically identified, the designer should rely on accepted industry standards. However, in no case shall other standards supersede the requirements provided in this document.

3.2 Dead Loads (D)

3.2.1 General

Dead load consists of the weight of the entire structure, including all the permanent attachments such as mooring hardware, fenders, light poles, utility booms, brows, platforms, vaults, sheds, service utility lines, and ballasted pavement. A realistic assessment of all present and future attachments should be made and included.

3.2.2 Unit Weights

Actual and available construction material weights shall be used for design. The following are typical unit weights:

Steel or cast steel	490 pcf
Aluminum alloys	175 pcf
Timber (untreated or treated)	50 pcf
Concrete, reinforced (normal weight)	150 pcf
Concrete, reinforced (lightweight)	120 pcf
Compacted sand, earth, gravel, or ballast	150 pcf
Asphalt paving	150 pcf

3.3 Vertical Live Loads (L)

3.3.1 Uniform Loads

The wharf shall be designed for a uniform live load of 1,000 psf, except for areas outboard of the waterside crane rail, which shall be designed for 500 psf. When combined with crane loading, the uniform live load in all areas should be 300 psf with no uniform loading within 5 feet of either side of the crane rails. For the design of wharf piles, the uniform live load may be reduced by 20% (800 psf). All uniform live loads shall be distributed to produce maximum forces. At predetermined locations, the outboard deck slab will also be checked for the loads imposed during loading and unloading of container cranes or other large equipment from their transport vessel. This load will be obtained from the equipment manufacturer and/or transporting company. The wharf may have a specified “Heavy Load” area to be designed for a uniform live load of 2,000 psf.

3.3.2 Truck Loads

Truck loads shall be in accordance with the American Association of State Highway and Transportation Officials (AASHTO) Standard Specification for Highway Bridges (Ref. 1). The wharf structure shall be designed for HL-93 truck loads shown in AASHTO, increased by a factor of 1.25. Lane loads need not be considered for the deck structure. Impact will be in accordance with Section 3.4. When truck load is transferred through 2.0 feet or deeper ballast fill, the impact factor need not be considered in design.

3.3.3 Container Crane Loads

Crane Rail Loads

All crane rail beams and supporting substructures shall be designed for actual crane wheel loads. In the absence of actual crane wheel loads data, a crane wheel load analysis shall be performed. This analysis should be done to determine the design crane wheel loads due to crane dead, live, wind and earthquake loads. The crane wheel load analysis criteria including load combinations shall be submitted to the Port for approval prior to performing the analysis. The following design crane wheel loads shall be provided for the wharf design:

- Vertical uniform wheel loads.
- Lateral uniform wheel loads.
- Crane Stowage pin loads.
- Crane stop loads and point of application height.
- All wheel loads shall be provided for crane landside and waterside.
- All wheel loads shall be provided for Service Load Design (SLD) / Allowable Stress Design (ASD) and Load and Resistance Factor Design (LRFD) conditions.

Waterside Crane Beam Broken Pile Criteria

The waterside crane rail beam shall be designed to span over interior pile(s) that may be damaged or broken, refer to Figure 3-1. The design consideration associated with a crane moving over broken piles are shown in Table 3-1. The wharf shall be fully operational with one broken pile and no operational allowance for two adjacent broken piles. The crane shall be allowed to gantry without cargo load over the two adjacent broken piles.

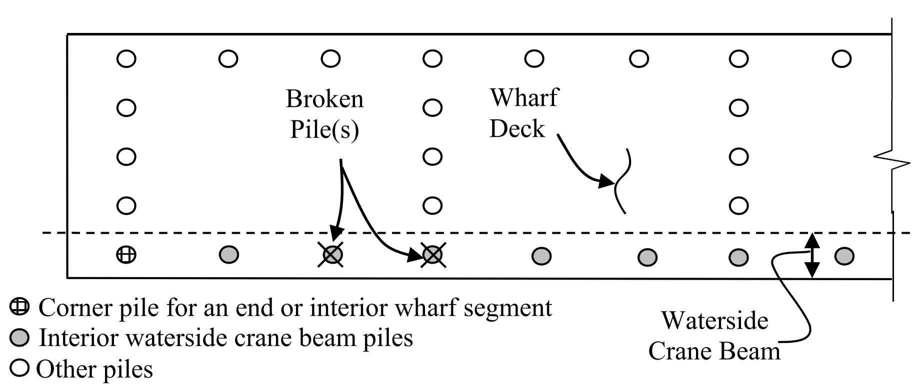


Figure 3-1: Waterside Crane Beam Broken Piles Layout

Table 3-1: Broken Pile Criteria

Load Case	Flexural Capacity[a]	Pile Soil Capacity Factor of Safety[b]
Normal operation	ϕM_n	2.0
One interior pile broken[c]	$1.1\phi M_n$	1.5
Two adjacent interior piles broken[c, d, e]	$1.1\phi M_n$	1.5

[a] ϕM_n is the reduced nominal moment capacity of the crane rail beam or supporting pile head, calculated based on ACI-318.
[b] This factor of safety is for service load design combinations.
[c] Use for exterior waterside crane girder only. If truck lane exists, the broken pile criteria are not applicable.
[d] Only wharf dead load and the waterside crane dead weight rail load specified above need to be considered for the case of two adjacent interior piles broken.
[e] Wharf design shall include the crane dead load only for moving over two adjacent broken piles. No cargo loads are permitted.

Crane Stowage Pin

Crane stowage pins shall be designed for the horizontal force provided in the crane wheel load analysis with a minimum of 250 kips service load (SL) per rail at each location under stowed wind condition.

Crane Stop Load

Crane stops shall be designed to resist a horizontal runaway wind-blown crane impacting force provided in the crane wheel load analysis with a minimum of 350 kips service load (SL) per rail. The force will be applied at the provided height at the crane wheel load analysis above the top of the rail, and in a direction parallel to the rail.

3.3.4 Container Handling Equipment Loads

Wharf deck slab shall be designed for container handler wheel loads shown in Figure 3-2. Wheel loads distribution shall be determined in accordance with AASHTO (Ref. 1). For equipment with hard rubber wheels or other wheels not inflated, the wheel contact area shall be designed as a point load. If handling equipment loading needs to be higher than the load shown in Figure 3-2, load values and distribution shall be provided to the port for approval.

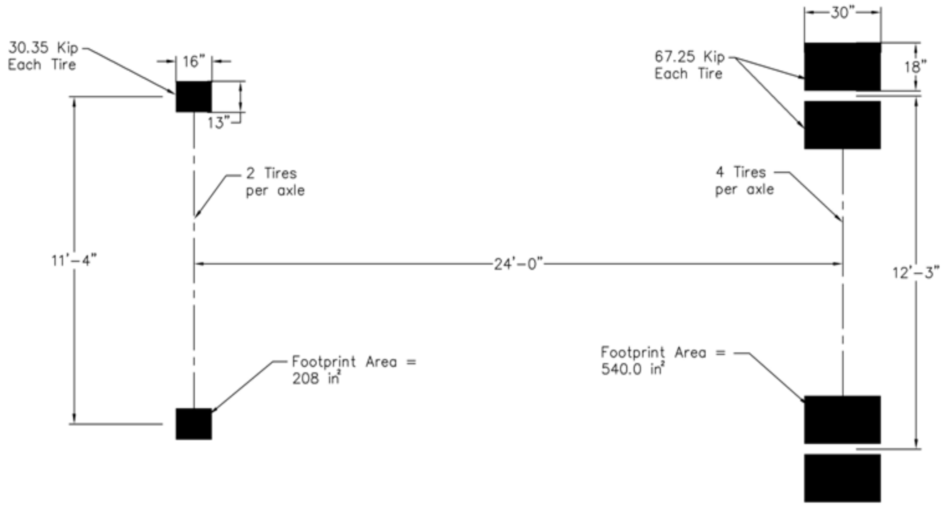


Figure 3-2: Container Handling Equipment Design Wheel Load

3.3.5 Railroad Track Loads

Wharves accessible by freight car shall be designed for railroad loads. Wheel loads shall correspond to Cooper E-80 designation of "American Railway Engineering and Maintenance-of-Way Association (AREMA) Manual" (Ref. 6).

3.4 Impact Factor (I)

The impact factors shown in Table 3-2 shall be applied to uniform live loads and wheel loads for the design of deck slab, crane beams and pile caps. Impact factors should not be used for the design of piles and other types of substructures.

Table 3-2: Impact Factors

Load	Impact Factor (I)
Uniform Loads	0%
Truck Loads	10%
Container Handling Equipment Loads	10%
Railroad Track Loads	20%

3.5 Buoyancy Loads (BU)

Typically, wharf decks are not kept low enough to be subjected to buoyancy forces. However, portions of the structure, such as utility lines and vaults and bent caps, may be low enough to be subjected to buoyancy forces. These are essentially uplift forces applied at the rate of 64 pounds per square foot of plan area for every foot of submergence below water level.

3.6 Berthing Loads (BE)

Berthing loads shall be based on the characteristics of design vessel as listed in Table 3-3. The berthing energy shall be determined by the deterministic approach according to "Guidelines for the Design of Fender Systems, 2002" (PIANC 2002) (Ref. 26) with "favorable" site condition.

Table 3-3: Design Vessel Parameters

Vessel Characteristic	Design Vessel
Length Overall (LOA)	Vessel Specific
Maximum Displacement	Vessel Specific
Beam	Vessel Specific
Draft	Vessel Specific
Allowable Hull Pressure	Per PIANC 2002 (Ref. 26)
Approach Velocity Normal to Fender Line, $v_{\perp}$	Per PIANC 2002 (Ref. 26)
Approach Angle, α	Per PIANC 2002 (Ref. 26)

Fender shear forces may be calculated using a friction coefficient, μ_f = 30%, at the fender face/ship hull interface. The berthing energy of the rubber fender shall be based on a fender panel deflected angle of 10˚. Vessel ship energy shall be resisted by one fender or dual fender system. If a dual fender system is used, each fender shall have the capacity for 75% of the total berthing energy.

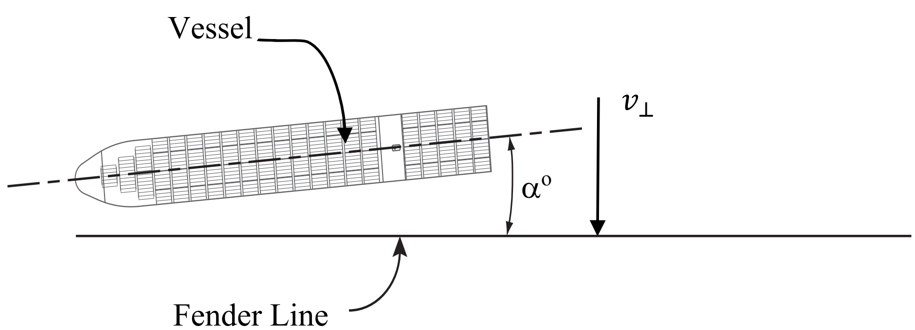


Figure 3-3: Vessel Berthing

$$V_F = \mu_f \times R_F \tag{3.1}$$

where:

V_F	=	Fender shear force (Horizontal and Vertical)
R_F	=	Force perpendicular to the fender panel due to berthing load

3.7 Mooring Loads (M)

For the design of the wharf structure, mooring line loads (P) shall be equal to the mooring hardware capacity. These line loads shall be applied at angles between horizontal and a maximum of 30° from horizontal in a vertical plane outboard of the wharf face, as shown in Figure 3-4. These load directions represent possible bow and stern breasting line loads.

In applying these loads to the wharf structure, consideration should be given to bow and stern breasting line separations as well as distances to possible adjacent vessel breasting lines. Where applicable, mooring line loads shall also be considered adjacent to expansion joints and/or the end of the structure.

Each mooring hardware for container ships shall have a minimum capacity of 200 metric tons. For other types of vessels, which may require higher mooring hardware capacities, a more detailed mooring analysis shall be performed. For mooring analysis use 75 mph design wind speed (30-second duration with 25-year return period), for more details refer to Current CBC Section 3103F.5 (Ref. 17).

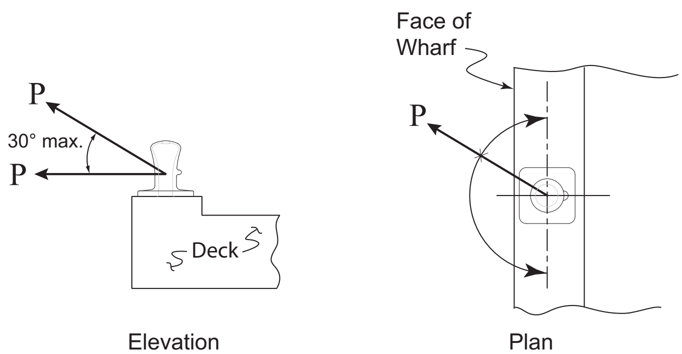


Figure 3-4: Mooring Line Force

3.8 Earth Pressure Loads (E)

Detailed requirements for static and dynamic earth pressure loads are discussed in Section 2.

3.9 Earthquake Loads (EQ)

Wharf structure shall be designed to resist earthquake motions by considering the relationship of the site to active faults, the seismic response of the soils at the site, and the dynamic response characteristics of the total structure and its individual components in accordance with the Seismic Design Criteria described in Section 4.

To account for the effect of vertical ground acceleration on the pile and deck, upper bound and lower bound dead load combinations shall be considered with seismic load.

3.10 Wind Loads on Structure (W)

The wind load calculations shall be based on Current CBC (Ref. 16) with basic wind speed of 110 mph (3-second gust with 7% probability of exceedance in 50 years).

3.11 Creep Loads (R)

Creep is a material-specific internal load similar to shrinkage and temperature, and is critical only to prestressed concrete construction. The creep effect is also referred to as rib shortening and shall be evaluated using the PCI Design Handbook (Ref. 38).

3.12 Shrinkage Loads (S)

Open wharf deck constructed from concrete components are subject to forces resulting from shrinkage of concrete due to the curing process. Shrinkage load is similar to temperature load in the sense that both are internal loads. For long continuous open wharf structures, shrinkage load is significant and should be considered. However, on pile-supported wharf structures, the effect is not as critical as it may seem at first because over the long time period in which shrinkage takes place, the soil surrounding the piles will slowly "give" and relieve the forces on the piles caused by the shrinking deck. The Prestressed Concrete Institute (PCI) Design Handbook (Ref. 38) is recommended for design of shrinkage.

3.13 Temperature Loads (T)

Thermal loads in structural elements shall be determined based on a temperature difference of 25° F whether increase or decrease.

3.14 Current Loads on Structure (C)

If site-specific current velocity data is not available, the current load on structure can be based on current velocity of 1.5 foot per second (Ref. 30). Loads due to tsunami-induced waves, wave heights in shallow water and particle kinematics can be determined based on current and wave heights presented in Ref. 31. Other structural considerations including uplift and debris impact shall be considered in the wharf design.

3.15 Loads Application

Concentrated Loads

Wheel loads and outrigger float loads from container handling equipment may be applied at any point on a wharf deck except outboard of the waterside crane rail. The equipment may be oriented in any direction, and the orientation causing the maximum forces on the structural members shall be used in the design. Trucks are permitted to operate outboard of the waterside crane rail. Power trench covers and utility vault covers outboard of the waterside crane rail shall be designed for wheel loads of trucks only; no other concentrated loads shall be used. Loaded containers shall not be stacked on the wharf deck. However, empties may be stacked inboard of the waterside crane rail, and the resulting corner casting compression or punching shear forces due to empty containers stacked six high should be checked.

Simultaneous Loads

Uniform and concentrated live loads should be applied in a logical, practical manner. Designated uniform live loads and concentrated live loads from pneumatic-tired equipment shall not be applied simultaneously in the same area. However, a uniform live load shall be used between crane rails as described in Section 3.3.1. When railroad tracks are present between crane rails, both crane and railroad track loads shall be applied simultaneously, and no uniform load between crane rails shall be applied.

Loads for Maximum Member Forces

For determining the shear forces and bending moments in continuous members, the designated uniform and concentrated loads shall be applied to produce the maximum effect.

Critical Loads

Concentrated loads are generally critical for punching shear and for the design of short spans such as deck slabs, power trench covers and utility vault covers. Uniform load, container handling equipment load, crane loads, and railroad track loads are generally critical for the design of beams, pile caps, and supporting piles.

3.16 Load Combinations

3.16.1 General

Wharf structures shall be proportioned to safely resist the load combinations represented in Table 3-4. Each component of the structure and the foundation elements shall be analyzed for all applicable combinations. For earthquake load combinations refer to Section 4.

Load Symbols

D = Dead Loads
L = Live Loads
I = Impact Factor
BU = Buoyancy Loads
BE = Berthing Loads
M = Mooring Loads
E = Earth Pressure Loads
W = Wind Loads on Structure
R = Creep Loads
S = Shrinkage Loads
T = Temperature Loads
C = Current on Structure Loads

3.16.2 Load and Resistance Factor Design (LRFD)

Load combinations and load factors used for load and resistance factor design are presented in Table 3-4. Concrete and steel structural members shall be designed using the load and resistance factor design method. However, concrete structural members shall also be checked for serviceability (i.e., creep, fatigue, and crack control as described in ACI-318 (Ref. 2), and temporary construction loads. Strength reduction factors shall follow ACI-318 (Ref. 2) for reinforced concrete design and AISC (Ref. 4) for structural steel design.

3.16.3 Service Load Design (SLD) / Allowable Stress Design (ASD)

Load combinations used for allowable stress design are presented in Table 3-4. The service load approach shall be used for designing vertical foundation capacity and long-term vertical wharf loads.

Table 3-4: Load Combinations[a]

LOAD AND RESISTANCE FACTOR DESIGN (LRFD)[b]									
Case	LOAD COMBINATION FACTORS								
	D	L+I[c]	E	W	BE	M	R+S+T	BU	C
I	1.20	1.60	1.60	1.00	—	—	1.20	1.20	1.20
II[d]	0.90	—	1.60	1.00	—	—	1.20	1.00	1.20
III	1.20	1.00	1.60	1.00	1.60	—	—	1.20	1.20
IV	1.20	1.60	1.60	1.00	—	1.60	—	1.20	1.20
SERVICE LOAD DESIGN (SLD) / ALLOWABLE STRESS DESIGN (ASD)[e]									
Case	LOAD COMBINATION FACTORS								
	D	L+ I[c]	E	W	BE	M	R+S+T	BU	C
I	1.00	1.00	1.00	0.60	—	—	1.00	1.00	1.00
II	1.00	0.75	1.00	0.45	1.00	—	—	1.00	1.00
III	1.00	1.00	1.00	0.60	—	1.00	—	1.00	1.00

[a] For earthquake load combinations, refer to Section 4.5.2

[b] The Load Resistance Factor Design require the strength reduction factors, ϕ as specified in ACI-318 (Ref. 2). Strength reduction factors shall follow ACI-318 (Ref. 2) for reinforced concrete design and AISC (Ref. 4) for structural steel design.

[c] The LRFD and SLD/ASD crane wheel loads determined according to Section 3.3.3 should be combined with other loads listed in this table without additional factor.

[d] Reduce load factor to 0.9 for dead load (D) to check members for minimum axial load and maximum moment.

[e] Increase in allowable stress shall not be used.

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4 Seismic Design Criteria

4.1 Introduction

The following criteria identify the minimum requirements for seismic design of wharves. The criteria, which are performance based, require the displacement capacities of the structural members to be greater than the displacement demand imposed by the seismic loads. Where required, structural members are intentionally designed and detailed to deform inelastically for several cycles without significant degradation of strength under earthquake demand.

4.2 General Design Criteria

Wharf design shall consider the following items:

Ductile Design

The wharf structure shall be designed as a ductile system. The pile-to-deck interface forms an integral part of the wharf structure, and shall be designed for ductile behavior.

Structural System

The structural system shall be based on the strong beam (deck), weak column (pile) frame concept. The pile-deck structural system shall be designed to develop plastic hinges in the piles and not in the deck. This concept is different from the strong column-weak beam structural system concept that is used for the design of buildings. Capacity design is required to ensure that the dependable strengths of the protected members exceed the maximum feasible demand based on high estimates of the flexural strength of piles plastic hinges.

Pile Connection

The pile shall be connected to the deck with mild steel dowels (Grade 60). Moment-resisting connection created by extending the prestressing tendons into the wharf deck shall not be permitted.

Vertical Piles

An all-vertical (plumb) pile system shall be used, with an appropriate connection at the pile-to-deck interface to ensure ductile performance of the structure. Battered piles shall not be used for the design of new wharves without prior written approval from the Port. Refer to Section 5.4.7 for the appropriate use of batter piles.

Crane Rails

Beams supporting crane rails shall be supported by vertical piles only. The gage between crane rails shall be maintained by structural members or a wharf deck that spans between the two rails to prevent spreading or loss of gage due to earth movements.

Bulkheads

Steel or concrete bulkheads shall be designed to resist DE demands to not exceed the strain limits of OLE presented in Table 4-1

Cut-off wall

Cut-off wall shall be used to prevent loss of soil from the backland and shall not be designed to provide seismic lateral resistance.

Slope Stability

A slope stability analysis, including seismic induced movements, shall be performed as outlined in Section 2.

Utilities & Pipelines

Utilities shall be designed with flexible connections between the backland area and the wharf capable of sustaining expected wharf movements under CLE response. Flexible connections shall also be provided across wharf deck expansion joints.

4.3 Performance Criteria

The ground motions levels provided in Section 2.1 shall be used for the seismic design. The permitted level of structural damage for each ground motion is controlled by the concrete and steel strain limits in piles defined in Section 4.4. The performance criteria of the three-level ground motions are defined below:

Operating Level Earthquake (OLE)

Due to an OLE event, the wharf should have no interruption in operations. OLE forces and deformations, including permanent embankment deformations, shall not result in significant structural damage. All damage, if any, shall be cosmetic in nature and located where visually observable and accessible. Repairs shall not interrupt wharf operations.

Contingency Level Earthquake (CLE)

Due to a CLE event, there may be a temporary loss of operations that should be restorable within a few months. CLE forces and deformations, including permanent embankment deformations, may result in controlled inelastic structural behavior and limited permanent deformations. All damage shall be repairable and shall be located where visually observable and accessible for repairs.

Code-level Design Earthquake (DE)

Due to a DE event, forces and deformations, including permanent embankment deformations, shall not result in the collapse of the wharf and the wharf shall be able to support the design dead loads in addition to cranes dead load. Life safety shall be maintained.

4.4 Strain Limits

The strain limits for the OLE, CLE and DE performance levels are defined by the following material strains for concrete piles and steel pipe piles. Strain values calculated in the analysis shall be compared to the following limits:

Table 4-1: Strain Limits

Component Strain		Design Level		
		OLE	CLE	DE
Solid Concrete Pile[a]	Top of pile hinge concrete strain	$\varepsilon_c \leq 0.005$	$\varepsilon_c \leq 0.005+1.1\rho_s \leq 0.025$	No limit
	In-ground hinge concrete strain	$\varepsilon_c \leq 0.005$	$\varepsilon_c \leq 0.005+1.1\rho_s \leq 0.008$	$\varepsilon_c \leq 0.005+1.1\rho_s \leq 0.012$
	Deep In-ground hinge (>10D_p) concrete strain	$\varepsilon_c \leq 0.008$	$\varepsilon_c \leq 0.012$	No limit
	Top of pile hinge reinforcing steel strain	$\varepsilon_s \leq 0.015$	$\varepsilon_s \leq 0.6\varepsilon_{smd} \leq 0.06$	$\varepsilon_s \leq 0.8\varepsilon_{smd} \leq 0.08$
	In-ground hinge prestressing steel strain	$\varepsilon_p \leq 0.015$	$\varepsilon_p \leq 0.025$	$\varepsilon_p \leq 0.035$
	Deep In-ground hinge (>10D_p) prestressing steel strain	$\varepsilon_p \leq 0.015$	$\varepsilon_p \leq 0.025$	$\varepsilon_p \leq 0.050$
Hollow Concrete Pile[b]	Top of pile hinge concrete strain	$\varepsilon_c \leq 0.004$	$\varepsilon_c \leq 0.006$	$\varepsilon_c \leq 0.008$
	In-ground hinge concrete strain	$\varepsilon_c \leq 0.004$	$\varepsilon_c \leq 0.006$	$\varepsilon_c \leq 0.008$
	Deep In-ground hinge (>10D_p) concrete strain	$\varepsilon_c \leq 0.004$	$\varepsilon_c \leq 0.006$	$\varepsilon_c \leq 0.008$
	Top of pile hinge reinforcing steel strain	$\varepsilon_s \leq 0.015$	$\varepsilon_s \leq 0.4\varepsilon_{smd} \leq 0.04$	$\varepsilon_s \leq 0.6\varepsilon_{smd} \leq 0.06$
	In-ground hinge prestressing steel strain	$\varepsilon_p \leq 0.015$	$\varepsilon_p \leq 0.020$	$\varepsilon_p \leq 0.025$
	Deep In-ground hinge (>10D_p) prestressing steel strain	$\varepsilon_p \leq 0.015$	$\varepsilon_p \leq 0.025$	$\varepsilon_p \leq 0.050$

Table 4-1: Strain Limits (Continued)

Component Strain		Design Level OLE	Design Level CLE	Design Level DE
Steel Pipe Piles[c]	Top of pile hinge concrete strain	$\varepsilon_c \leq 0.010$	$\varepsilon_c \leq 0.025$	No limit
	Top of pile hinge reinforcing steel strain	$\varepsilon_s \leq 0.015$	$\varepsilon_s \leq 0.6\varepsilon_{smd} \leq 0.06$	$\varepsilon_s \leq 0.8\varepsilon_{smd} \leq 0.08$
	In-ground hinge hollow pipe steel strain	$\varepsilon_s \leq 0.010$	$\varepsilon_s \leq 0.025$	$\varepsilon_s \leq 0.035$
	In-ground hinge pipe in-filled with concrete steel strain	$\varepsilon_s \leq 0.010$	$\varepsilon_s \leq 0.035$	$\varepsilon_s \leq 0.050$
	Deep In-ground hinge (>10D_p) hollow pipe steel strain	$\varepsilon_s \leq 0.010$	$\varepsilon_s \leq 0.035$	$\varepsilon_s \leq 0.050$

[a] For solid round or octagonal piles.

[b] If a hollow concrete pile is in-filled with concrete, the strain limits shall be identical to a solid concrete pile.

[c] Steel pipe pile deck connection shall be accomplished by concrete plug with dowel reinforcement.

Definitions:

D_p = Pile diameter

ε_c = Concrete compression strain

ε_s = Steel tensile strain

ε_{smd} = Strain at maximum stress of dowel reinforcement; see Section 4.6.2

ε_p = Total prestressing steel tensile strain

ρ_s = Effective volumetric ratio of confining steel

4.5 Seismic Analysis

4.5.1 Analysis Methods

Analysis of wharf structures shall be performed for each performance level to determine displacement demand and capacity. The capacity shall be based on the pile strain limits defined in Table 4-1. The following analysis methods may be used:

- Nonlinear Static Pushover
- Equivalent Lateral Stiffness Method
- Elastic Stiffness Method
- Substitute Structure Method
- Modal Response Spectra Analysis
- Nonlinear Time-History Analysis

The flow diagram in Figure 4-1 shows the typical steps a designer should follow to complete the seismic analysis and design for a wharf structure. After the design for service static loads has been completed, the performance design shall be performed for OLE, CLE and DE. The seismic design may require additional pile rows or a modified pile layout. A model including the effective section properties, seismic mass, and soil springs shall be prepared. An Equivalent Lateral Stiffness method may be used for preliminary design, if desired. Nonlinear static pushover analysis is always required, and will provide the displacement capacity based on strain limits for all methods. The structural analysis shall account for wharf torsional plan eccentricity, soil structure interaction, multi-directional effects of the ground motion and the interaction between adjacent wharf segments. Displacement demand for regular wharves shall be estimated by the Elastic Stiffness method, the Substitute Structure method, or Modal Response Spectra Analysis. For wharves with irregular geometry, special cases, or when demand/capacity ratios from Modal Response Spectra Analysis are too high, Nonlinear Time-History methods may be employed for the global model to verify the analysis results. Nonlinear Time-History analyses, however, shall not be conducted without prior written approval from the Port.

The maximum pile displacement shall be determined from the demand analysis, and compared to the displacement capacity. The demand determined using the Elastic Stiffness and Substitute Structure methods shall be adjusted for torsional effects using the Dynamic Magnification Factor. If the demand is greater than the capacity, the design must be revised. If the demand is less than the capacity, the pile shear, the beam/deck pile joint and P-Δ effects shall be checked. If the simplified kinematic loading and lateral spreading analysis performed per Section 2.9.2 requirements indicate that the anticipated pile strains for the estimated deformations are likely to exceed the strain limits per Section 4.4, kinematic analysis of the deep in-ground hinge shall be performed in accordance with Section 4.12.

4.5.2 Earthquake 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:

$$U = (1 \pm k)\, D + \gamma L + E + EQ \tag{4.1}$$

$$U = (1 \pm k)\, D + E + EQ \tag{4.2}$$

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
- D = Dead Loads
- L = Live Loads
- E = Earth Pressure Loads
- EQ = Earthquake Loads
- γ = *For container wharf structures use 0.1, all other structures need to be considered differently*

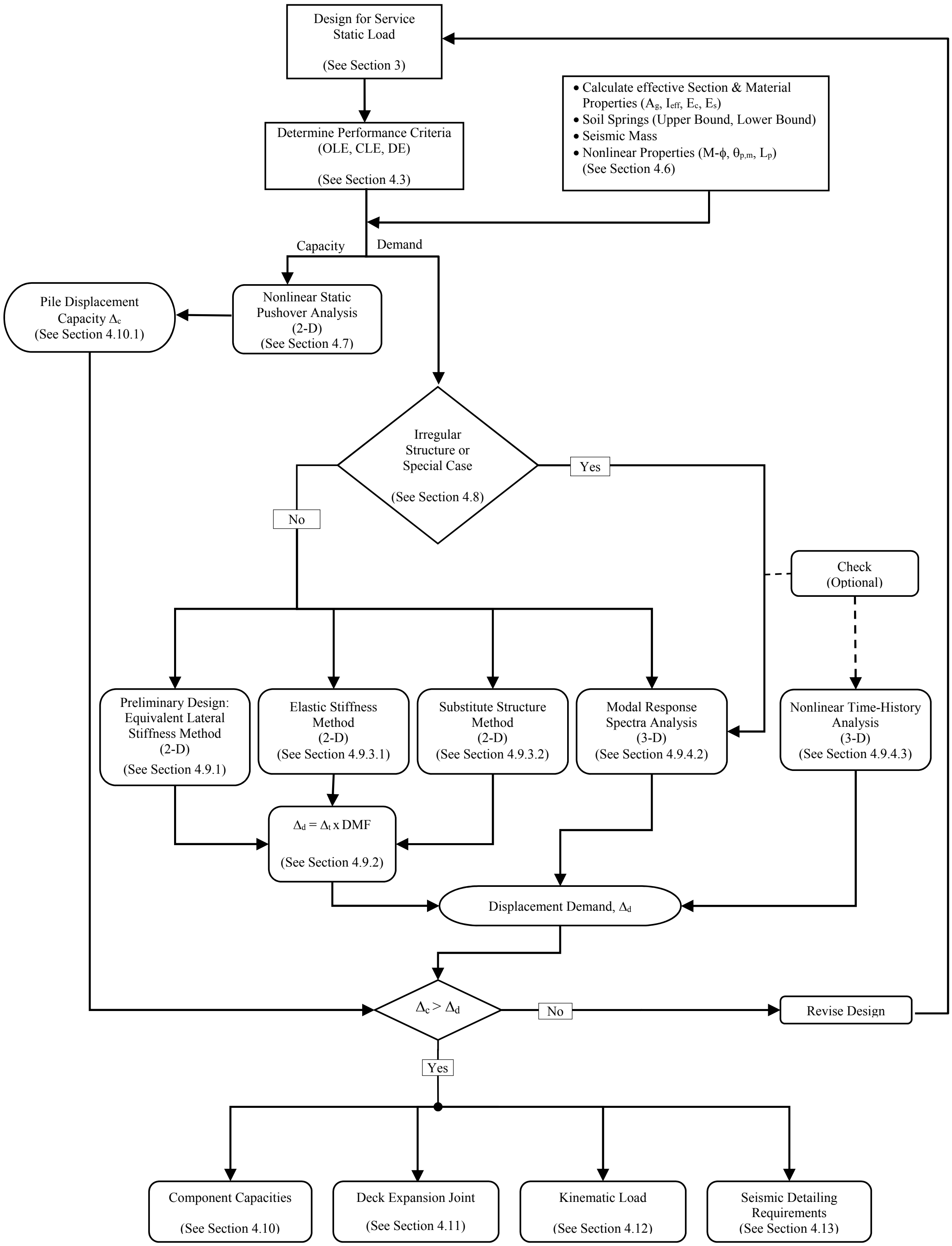


Figure 4-1: Flow Diagram for Seismic Analysis

4.6 Structural Model

4.6.1 Modeling

Due to the general uniformity and symmetry along the longitudinal axis of regular marginal wharves, the wharf may be modeled as a strip for pure transverse analyses. The number of piles considered in the strip should be modeled to reflect the pile spacing in each row, as shown in Figure 4-2.

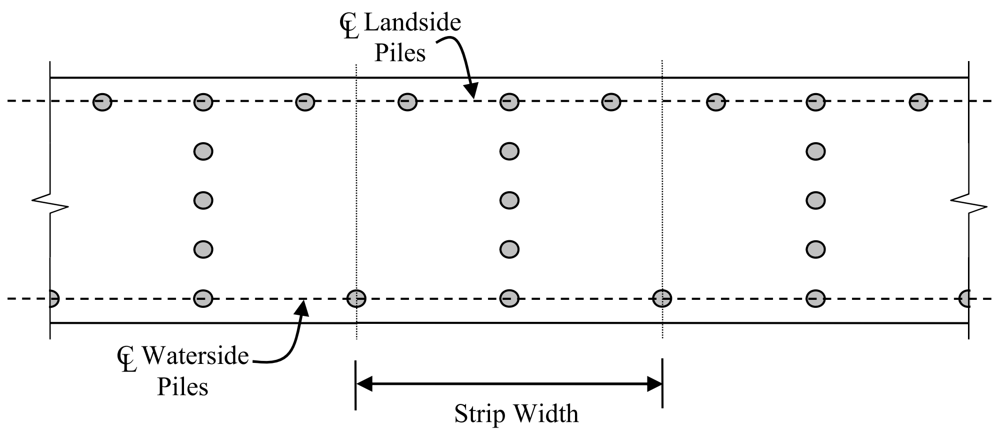


Figure 4-2: Pile Spacing for Modeling of Typical Wharf Strip

The structural model shall incorporate components for the lateral resisting system. All members shall be modeled at the center of gravity of the section. A minimum of two members for the pile unsupported length from the soffit to the first soil spring shall be used in the modeling. The ratio of the stiffness between the rigid links and the surrounding elements should be no more than 100 to stabilize the stiffness matrix. Soil springs shall be used to model soil-structure interaction, and shall be spaced at each layer to accurately capture the soil behavior. Two distinct models shall be created to model upper bound and lower bound soil springs; see Section 2.7.3.

The interface between the deck and the pile should not be considered entirely rigid. The effective top of the pile should be located a distance l_{sp} into the deck to account for strain penetration. This additional length applies only to displacements. The strain penetration of the pile section into the deck shall be modeled as a member with properties equivalent to the top of the pile. The member between the strain penetration and the center of gravity (c.g.) of the deck shall be a rigid link. The length of the strain penetration member shall be equal to:

$$l_{sp} = 0.1 f_{ye} d_{bl} \tag{4.3}$$

where,

l_{sp} = Strain penetration length (in.)

d_{bl} = The diameter of the dowel reinforcement (in.)

f_{ye} = Expected yield strength of the longitudinal reinforcement, ksi; see Section 4.6.2.

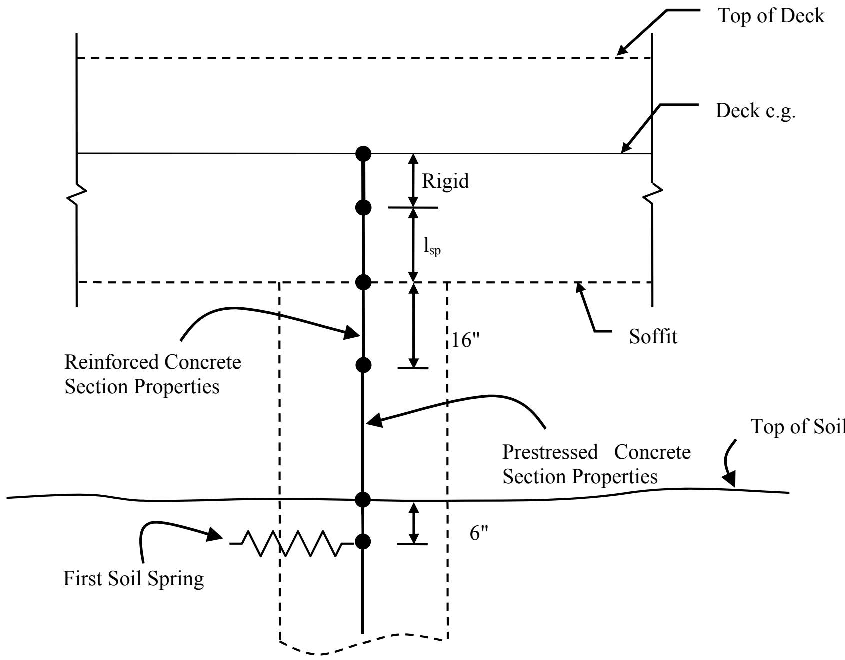


Figure 4-3: Pile-Deck Structural Model Schematic Showing Strain Penetration Length

For prestressed piles, the reinforced concrete effective section property per Section 4.6.3 shall be used for the first 16 inches of the pile below the soffit to account for development of the prestressing strands. Below the first 16 inches of the pile, the prestressed concrete effective section properties shall be used, see Section 4.6.3. Maximum pile moment shall be considered to develop at the soffit. Maximum in-ground moment will normally occur at between 50 and 100 inches below the dike surface for 24-inch diameter piles. This value depends on the soil stiffness and strength, and the clear height between the deck soffit and top of dike. To insure adequate precision in modeling the pile moment profile, it is important that the soil springs be closely spaced in the upper region of the pile. For typical 24-inch diameter piles it is recommended that the first soil spring be located 6 inches below the dike surface, then springs be spaced at 12 inches to a depth of about 126 inches. Below this, the spacing can be increased to 24 inches to a depth of about 246 inches, then to 48 inches to a depth of about 390 inches. It will not normally be necessary to model the soil below this depth and the pile can generally be considered fixed against displacement and rotation at a depth of about 500 inches.

4.6.2 Material Properties

The capacity of concrete components to resist all seismic demands, except shear, shall be based on the most probable (expected) material properties to provide a more realistic estimate for design strength.

The expected compressive strength of concrete, f'_{ce}, recognizes the typically conservative nature of concrete batch design, and the expected strength gain with age. The expected yield strength for reinforcing steel and structural steel, f_{ye}, is a "characteristic" strength and represents a low estimate of probable strength of the material, which is higher than the specified minimum strength. Expected material properties shall be used to assess capacity and demands for earthquake loads. Seismic shear capacity shall not be based on the expected material strength, see Section 4.10.3. For determining the demand on capacity-protected members, an additional overstrength factor shall be used on the capacity of pile plastic hinges as described in Section 4.10. Except for shear, the expected seismic material strengths shall be:

$$f'_{ce} = 1.3 f'_c \quad (4.4)$$

$$f_{ye} = 1.1 f_y \quad (4.5)$$

$$f_{yhe} = 1.0 f_{yh} \quad (4.6)$$

$$f_{pye} = 1.0 f_{py} \quad (4.7)$$

$$f_{pue} = 1.05 f_{pu} \quad (4.8)$$

$$E_c = 57{,}000\sqrt{f'_{ce}} \ \ (f'_{ce} \text{ is in psi}) \quad (4.9)$$

where,

f'_c = 28-day unconfined compressive strength

f_y = Yield strength of longitudinal reinforcing steel or structural steel

f_{yh} = Yield strength of confining steel

f_{py} = Yield strength of prestressing steel

f_{pu} = Maximum tensile strength of prestressing steel

f'_{ce}, f_{ye}, f_{yhe}, f_{pye}, f_{pue} = Expected material properties

E_c = Modulus of elasticity of concrete

The following stress-strain curves may be used to determine the deformation capacity of the structural members. Alternative stress-strain models are acceptable if adequately documented and supported by test results.

Concrete

The stress-strain curves for both confined and unconfined concrete are shown in Figure 4-4. This model is based on Mander's model for confined and unconfined concrete (Ref. 28).

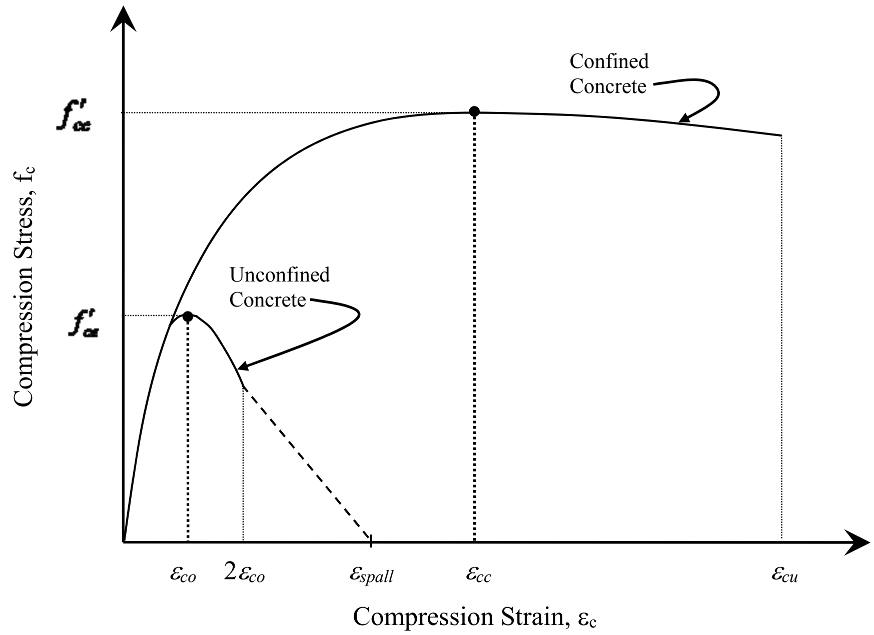


Figure 4-4: Stress-Strain Relationship for Confined and Unconfined Concrete

Unconfined Concrete:

Unconfined concrete either has no confinement steel or the spacing of the confinement steel exceeds 12 inches. For these cases:

ε_{spall} = Ultimate unconfined compression (spalling) strain, taken as 0.005

ε_{co} = Unconfined compression strain at the maximum compressive stress, taken as 0.002

Confined Concrete:

For confined concrete, the following are defined:

$$\varepsilon_{cu} = 0.005 + 1.1\rho_s \leq 0.025 \tag{4.10}$$

$$\varepsilon_{cc} = \varepsilon_{co}\left[1 + 5\left(\frac{f'_{cc}}{f'_{ce}} - 1\right)\right] \tag{4.11}$$

$$f'_{cc} = f'_{ce}\left(-1.254 + 2.254\sqrt{1 + \frac{7.94 f'_l}{f'_{ce}}} - 2\frac{f'_l}{f'_{ce}}\right) \tag{4.12}$$

where for circular core sections,

$$f'_l = \frac{1}{2} K_e \rho_s f_{yh} \tag{4.13}$$

$$\rho_s = \frac{4A_{sp}}{D's} \tag{4.14}$$

ε_{cu} = Ultimate concrete compression strain

ε_{cc} = Confined concrete compressive strain at maximum compressive stress

f'_{cc} = Confined concrete compressive strength

f'_{ce} = Expected compressive concrete strength of concrete

f_l' = Effective lateral confining stress

K_e = Confinement effectiveness coefficient, equal to 0.95 for circular core

ρ_s = Effective volumetric ratio of confining steel

f_{yh} = Yield stress of confining steel

A_{sp} = Cross-section area of confining steel

D' = Diameter of confined core, measured to the centerline of the confining steel

s = Center-to-center spacing of confining steel along pile axis

Figure 4-5 plots the ratio of confined concrete compressive strength to expected concrete compressive strength (f'_{cc} / f'_{ce}) with varying volumetric transverse steel ratios (ρ_s). This graph may be used to determine the confined concrete strength, f'_{cc} for circular core sections.

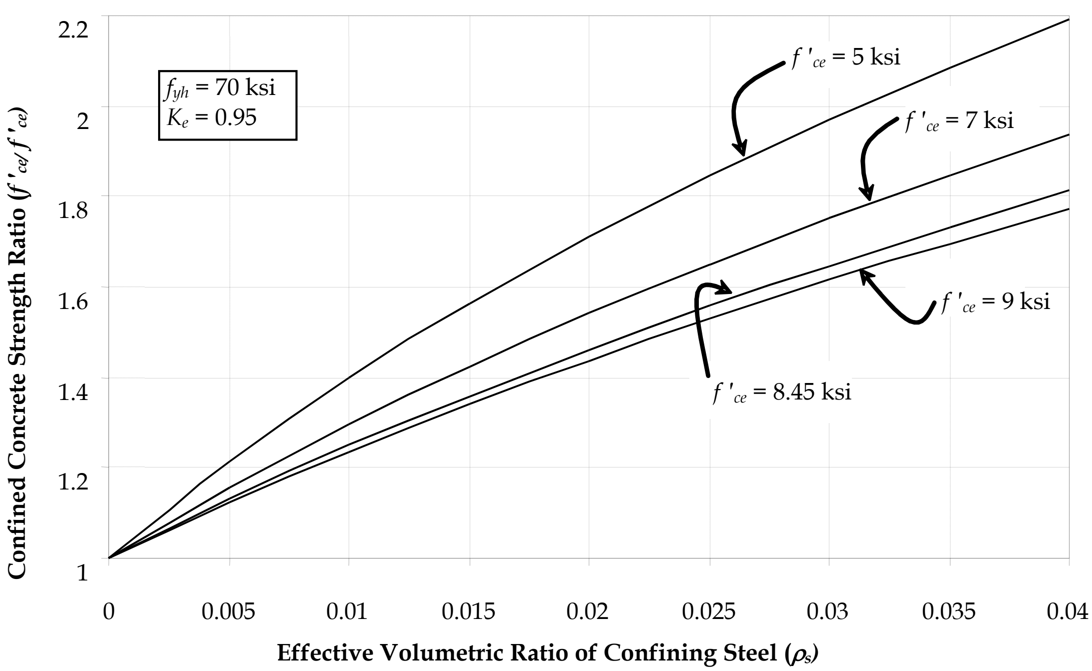


Figure 4-5: Concrete Strength Ratio versus Confining Steel Ratio

For pile sections with different transverse reinforcement strengths or shapes (i.e. rectangular stirrups), the confined concrete strength f'_{cc} may be approximated by 1.5 f'_{ce} or calculated according to Mander's model (Ref. 28).

Steel

The stress-strain curve for reinforcing steel is shown in Figure 4-6. The strain-hardening equation for this curve is available in References 18, 39 and 40. To control the tensile properties, A706 reinforcing steel is preferred for pile dowels. The stress-strain curve for structural steel is similar to this curve (Ref. 18).

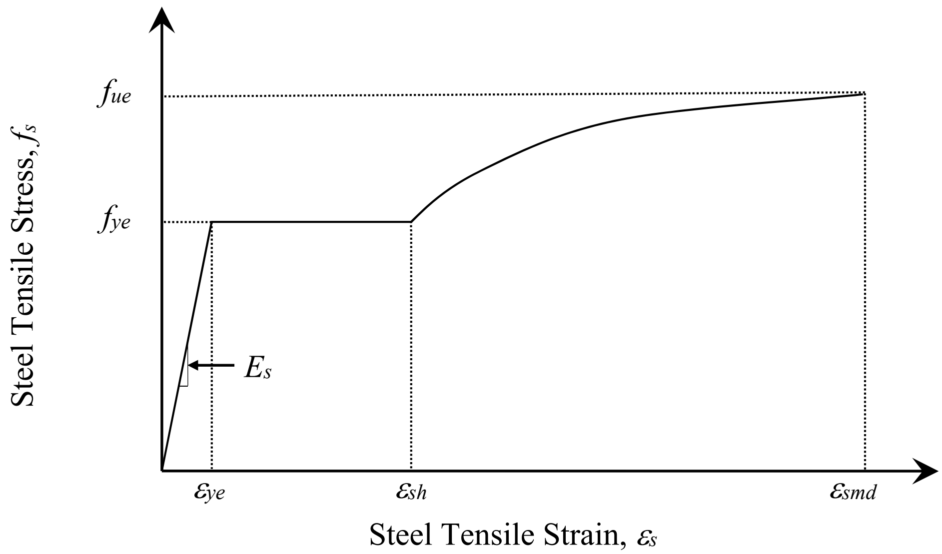


Figure 4-6: Stress-Strain Relationship for Reinforcing Steel

Where for ASTM A706 Grade 60 steel (Ref. 19):

Steel tensile strain at the onset of strain hardening

$$\varepsilon_{sh} = \begin{cases} 0.0150 & \text{\#8 bars} \\ 0.0125 & \text{\#9 bars} \\ 0.0115 & \text{\#10 \& \#11 bars} \\ 0.0075 & \text{\#14 bars} \\ 0.0050 & \text{\#18 bars} \end{cases}$$

Strain at maximum stress of dowel reinforcements

$$\varepsilon_{smd} = \begin{cases} 0.120 & \text{\#10 bars and smaller} \\ 0.090 & \text{\#11 bars and larger} \end{cases}$$

$f_{ue} = 1.4 f_{ye}$

f_{ue} = Expected maximum tensile strength of steel, equal to $1.4 f_{ye}$

E_s = 29,000 ksi

ε_{ye} = Expected yield tensile strain of steel

Prestressing Steel

The stress-strain curve for prestressing steel is shown in Figure 4-7.

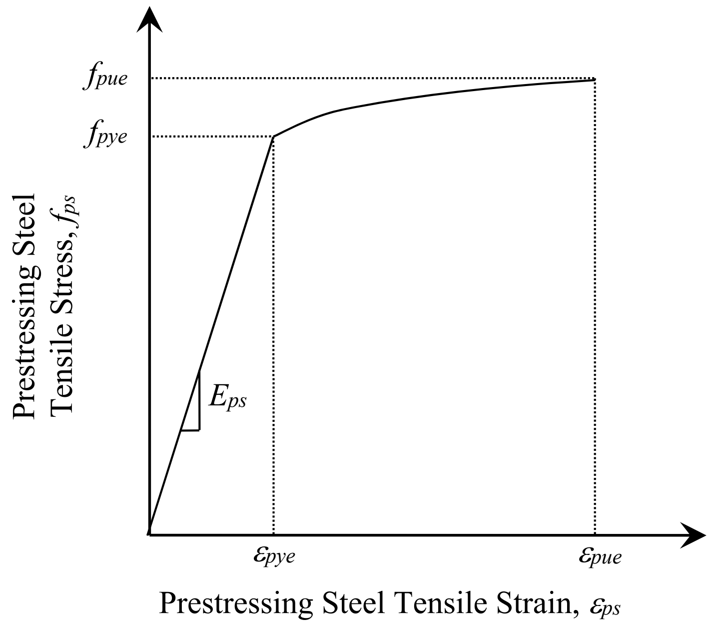


Figure 4-7: Stress-Strain Relationship for Prestressing Steel

E_{ps} = Modulus of elasticity for prestressing steel, taken as 28,500 ksi

ε_{pye} = Expected yield strain for prestressing steel

ε_{pue} = Expected ultimate strain for prestressing steel, taken as 0.060

f_{pye} = Expected yield strength of prestressing steel, equal to $0.85f_{pue}$

f_{pue} = Expected maximum tensile strength of prestressing steel

4.6.3 Effective Section Properties

Elastic analysis assumes a linear relationship between stiffness and strength of structural members. Concrete members display nonlinear response before reaching their idealized yield limit state. Section properties shall reflect the cracking that occurs before the yield limit state is reached. The effective section properties shall be used to determine 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} can be determined based on the value of the secant slope of the moment-curvature curve between the origin and the point of first yield:

$$E_c I_{eff} = \frac{M_y}{\phi_{yi}} \tag{4.15}$$

where:

E_c = Modulus of elasticity of concrete

M_y = Moment at first yield; see Section 4.6.6.1 for definition

ϕ_{yi} = Curvature at first yield; see Section 4.6.6.1 for definition

For reinforced concrete piles and the pile/deck connection, the effective moment of inertia ranges between 0.3-0.7I_{gross}, where I_{gross} is the gross moment of inertia. For prestressed concrete piles, the effective moment of inertia ranges between 0.6-0.75I_{gross}. The prestressing steel at the top of the prestressed pile near the pile/deck connection is not permitted to extend into the deck, therefore, it will not be developed at the deck soffit. Thus, I_{eff} of the dowel connection should be used. For the deck section, the effective moment of inertia is about 0.5I_{gross}. Sections that are expected to remain uncracked for seismic response should be represented by the gross section properties.

The polar moment of inertia of individual piles is typically an insignificant parameter for the global response of wharf structure. The effective polar moment of inertia, J_{eff}, could be assumed to be equal to 0.2 J_{gross}, where J_{gross} is the gross polar moment of inertia.

4.6.4 Seismic Mass

The seismic mass for the seismic analysis shall include the mass of the wharf deck, permanently attached equipment, and 10% of the design uniform live loads or 100 psf for container wharf structure. The live load percentage for other structures need to be considered differently. In addition, 1/3 of the pile mass between the deck soffit and 5D_p below the dike surface shall be considered additional mass lumped at the deck. Hydrodynamic mass associated with piles, where significant, should be considered. For 24-inch diameter piles or less, hydrodynamic mass may be ignored.

The seismic mass shall also include the larger of: 1) part of the crane mass positioned within 10 feet above the wharf deck or 2) 5% of the total crane mass.

4.6.5 Lateral Soil Springs

Upper and lower bound (UB and LB) lateral soil springs (p-y) shall be used to create two distinct models to determine the seismic demands and the corresponding capacities. This recognizes the inherent uncertainties associated with soil-structure interaction. The higher of the two demand-to-capacity ratios will provide a conservative estimate of compliance for displacement response. See Section 2 for further discussion on soil spring values.

4.6.6 Pile Nonlinear Properties

4.6.6.1 Moment-curvature Analysis

The plastic moment capacity of the piles shall be calculated by Moment-curvature (M-ϕ) analysis using expected material properties. The analysis must be modeling the core and cover concrete separately, and must model the enhanced concrete strength of the core concrete. The pile in-ground hinge section shall be analyzed as a fully confined section due to the soil confinement. Reinforcement and prestressing steel nonlinearity must also be modeled using material properties as specified in Section 4.6.2. Moment-curvature analysis provides a curve showing the moments associated with a range of curvatures for a cross-section based on the principles of strain compatibility and equilibrium of forces. The analysis shall include the pile axial load and the effective prestressing force. For most cases, the largest axial load need to be considered to obtain the highest moment capacity for the design of the capacity-protected members. While, the smallest axial load need to be considered to obtain the pile displacement capacity for the piles design.

The M-ϕ curve may be idealized by an elastic-perfectly plastic curve as follows:

Moment-curvature Curve Idealization - Method A:

The idealized plastic moment capacity, M_p, for typical concrete pile at the POLB corresponds to the moment associated with an extreme concrete strain of 0.004, as shown in Figure 4-8. Typically, the M-ϕ curve peaks around an extreme concrete strain of 0.004, has a reduction in moment, and peaks again, depending on confinement, spalling of concrete cover and strain-hardening of reinforcement. If the second peak on the curve is less than the M_p value, the moment at the lower second peak should be taken as M_p. However, for capacity protection analysis, the moment at the higher peak shall be used for M_p. The elastic portion of the idealized M-ϕ curve passes through the curvature at first reinforcing bar yield of the section or when concrete strain equals 0.002, whichever occurs first (ϕ_{yi}, M_y), and extends to meet M_p. The idealized yield curvature, ϕ_y, is determined as the curvature corresponding to the plastic moment value.

Moment-curvature Curve Idealization - Method B:

For other M-ϕ curves of concrete piles different than the typical POLB piles, the moment-curvature relationship may not exhibit the dramatic reduction in section moment capacity near the cover spalling strain. This may occur for larger diameter concrete piles, concrete-filled steel pipe piles with concrete plug connections, and hollow steel piles. For these types, an equal area approach to determine the idealized M-ϕ curve is more appropriate. For this approach, the elastic portion of the idealized M-ϕ curve should pass through the point marking the first reinforcing bar yield or when $\varepsilon_c = 0.002$, whichever comes first (ϕ_{yi}, M_y). The idealized plastic moment capacity is obtained by balancing the areas between the actual and the idealized M-ϕ curves beyond the first yield point (Figure 4-9).

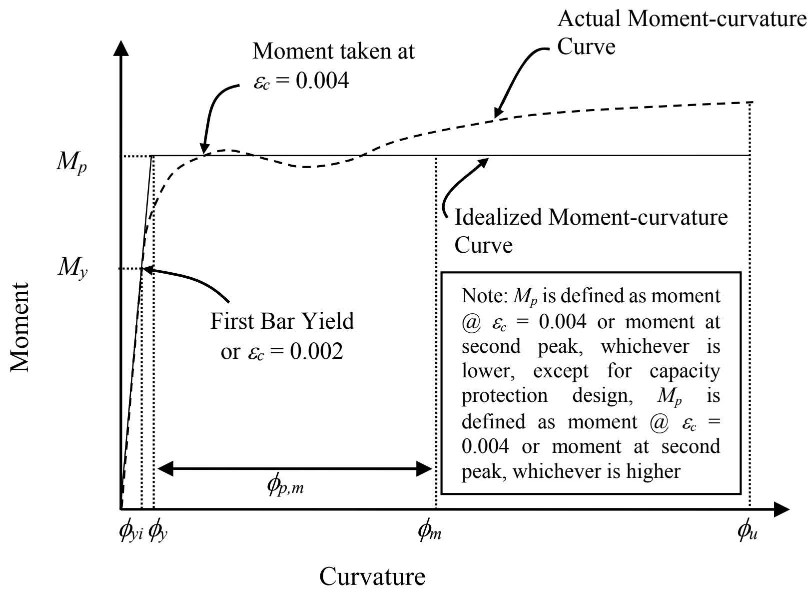


Figure 4-8: Moment–curvature Curve and Idealization for Method A

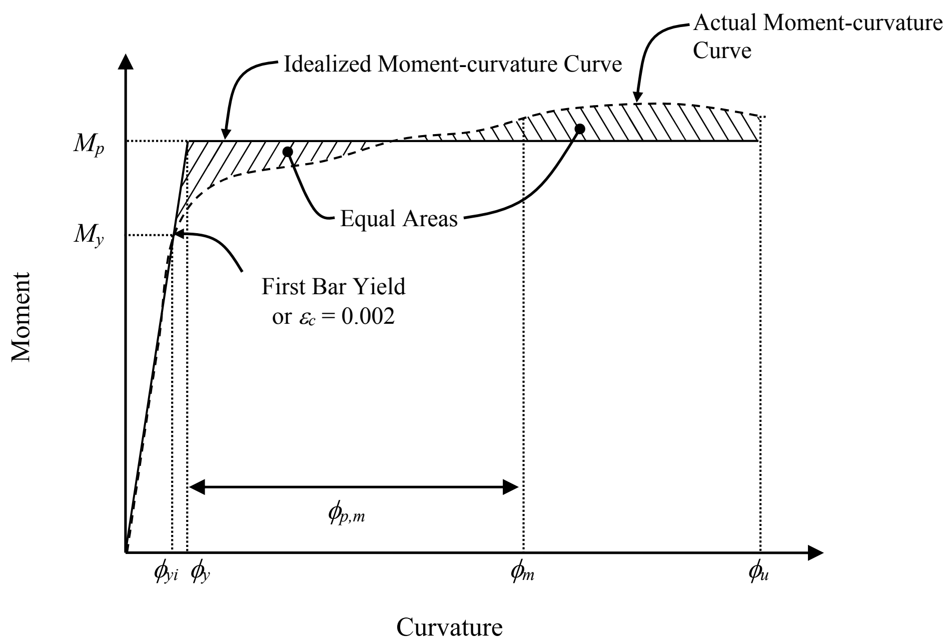


Figure 4-9: Moment-curvature Curve and Idealization for Method B

where:

M_y = Moment at first yield (corresponding to ϕ_{yi})

ϕ_{yi} = Curvature at first yield (first rebar yield or $\varepsilon_c = 0.002$)

ϕ_y = Idealized yield curvature

ϕ_m = Total curvature at the OLE, CLE or DE strain limits

$\phi_{p,m}$ = Plastic curvature at the OLE, CLE or DE strain limits

ϕ_u = Ultimate curvature of the section

4.6.6.2 Plastic Hinge Length

The plastic hinge length is needed to convert the moment-curvature relationship into a force-displacement or moment-rotation relationship for the nonlinear static pushover analysis. Table 4-2 cross references the equations that should be used to determine pile plastic hinge lengths for different pile sections.

Table 4-2: Plastic Hinge Length Equations

Section	Top	In-ground
Concrete Pile	4.16	4.18
Hollow Concrete Pile	4.16	4.18
Steel Pipe Pile (hollow with concrete plug connection)	4.17	4.18
Steel Pipe Pile (infilled with concrete)	4.17	4.18

For concrete pile dowel connections, the pile's plastic hinge length, L_p (above ground), when the plastic hinge forms against a supporting member, at deck soffits may be taken as:

$$L_p = 0.08L_c + 0.1 f_{ye} d_{bl} \geq 0.2 f_{ye} d_{bl} \quad (4.16)$$

where,

- L_c = The distance from the center of the pile top plastic hinge to the point of contraflexure in the pile (in.)
- d_{bl} = Diameter of dowel reinforcement (in.)
- f_{ye} = Expected yield strength dowel reinforcement (ksi)

For steel pipe sections connected to the deck by a concrete plug with dowels, the plastic hinge length for the top of pile hinge may be taken as:

$$L_p = 0.3 f_{ye} d_{bl} + d_{gap} \quad (4.17)$$

where,

- d_{gap} = The distance between the top of the pile steel shell and the deck soffit

The plastic hinge length for in-ground hinges may be calculated as defined in equation 4.18 for piles with 18 to 30 inches in diameter. For piles with larger diameter, reduced plastic hinge length for in-ground hinges may be used.

$$L_p = 2D_p \quad (4.18)$$

where,

- D_p = Pile diameter

4.6.6.3 Plastic Rotation

The pile plastic rotation can be determined as follows:

$$\theta_{p,m} = L_p \phi_{p,m} = L_p (\phi_m - \phi_y) \tag{4.19}$$

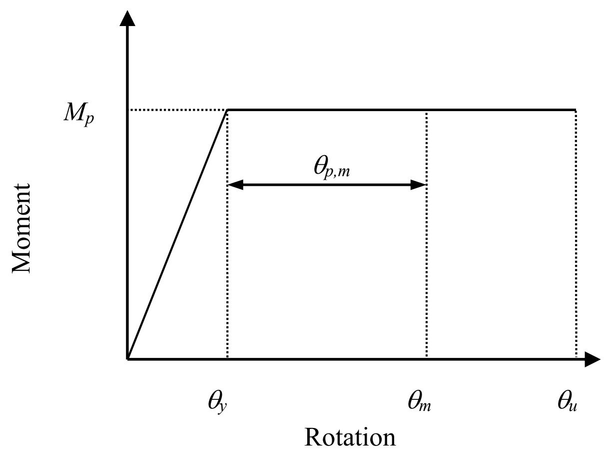
where,

$\theta_{p,m}$ = Plastic rotation at the OLE, CLE or DE strain limits

$\phi_{p,m}$ = Plastic curvature at the OLE, CLE or DE strain limits

The idealized moment-rotation (M-θ) curve is shown in Figure 4-10.

Figure 4-10: Idealized Moment-rotation Curve



θ_u = Ultimate rotation

θ_y = Idealized yield rotation ($\theta_y = \phi_y L_p$)

θ_m = Total rotation at the OLE, CLE or DE strain limits

4.7 Nonlinear Static Pushover Analysis

Two-dimensional (2-D) nonlinear static pushover analyses (pushover analysis) shall be performed for all wharf structures. The pushover curve shall have sufficient points to encompass the system's initial elastic response and predicted seismic demand. The pushover curve shall also encompass the OLE, CLE and DE displacement capacities. The yield displacements and OLE, CLE or DE displacement capacities may be obtained directly from the pushover analyses when plastic rotation and hinge proper definitions are included in the model. This analysis method incorporates soil deformation into the total displacement capacity of the pile. Pushover model shall use effective section properties and shall incorporate soil stiffness with nonlinear upper and lower bound *p-y* springs, see Figure 4-11. The results from the pushover analysis will provide the displacement capacities for OLE, CLE or DE, as well as the parameters needed for the Elastic Stiffness and Substitute Structure methods, see Figure 4-12. The pushover curve shall not experience a significant drop (greater than 20%) in total shear at the target-strain limits for OLE, CLE or DE.

Three dimensional (3-D) nonlinear static pushover analysis requires the proper modeling of the structure hinges' definitions and soil springs to reflect the varying conditions of the soil in all directions. This makes 3-D pushover analysis complex. Prior written approval by the Port is required before conducting 3-D pushover analysis.

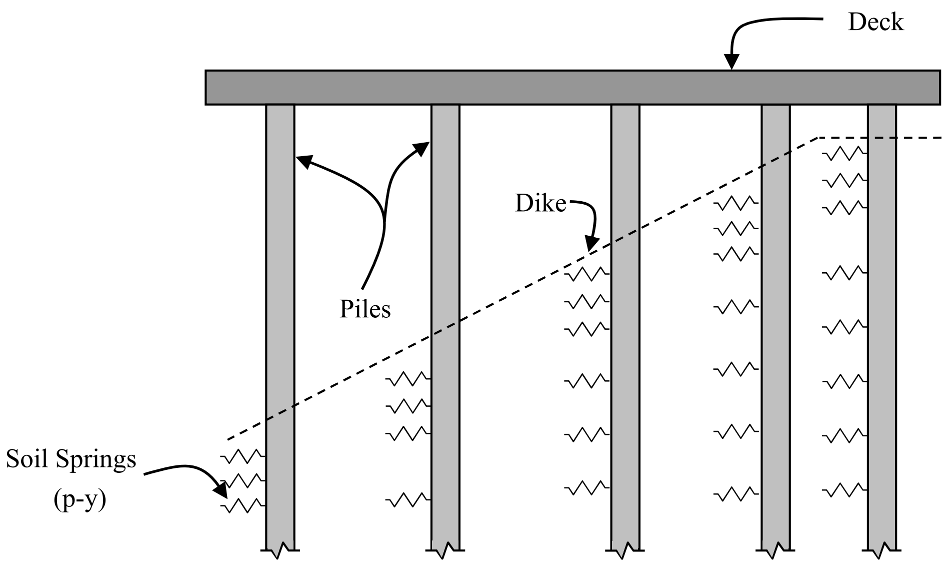


Figure 4-11: Pushover Model with p-y Springs

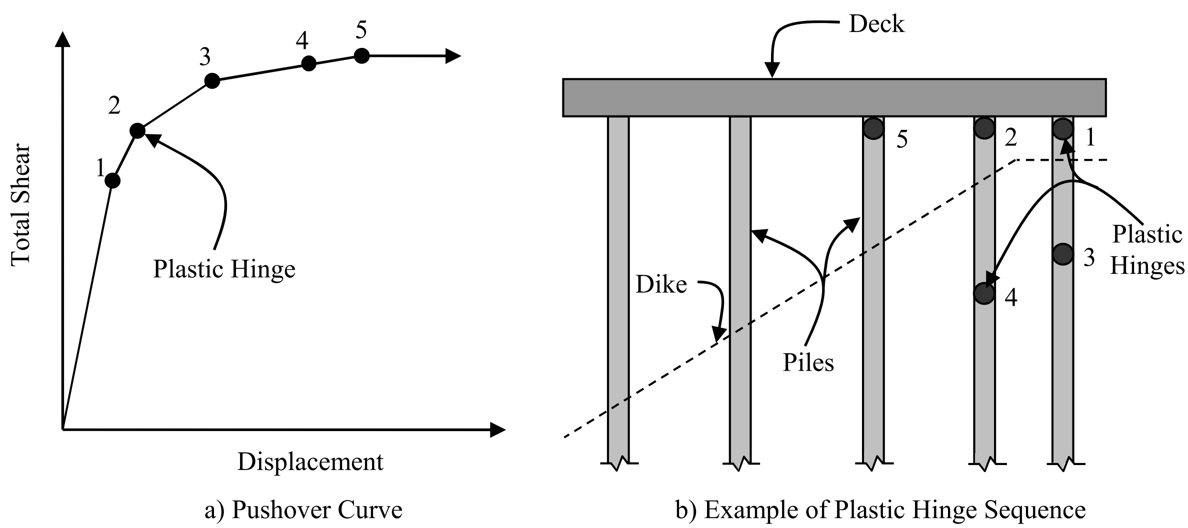


Figure 4-12: Example of Pushover Curve and Plastic Hinge Sequence

4.8 Irregular Structures and Special Cases

4.8.1 Irregular Structures

Horizontal irregularity occurs when wharves have unsymmetrical pile and/or dike layouts, and when wharves have an angle point; see Figure 4-13. Figure 4-13 a) shows a regular marginal wharf structure. The wharf in Figure 4-13 b) shows an irregular marginal wharf constructed with a partial dike. Figure 4-13 c) shows two adjacent wharves with large differences in stiffness, which may occur between two adjacent wharves with different pile or soil stiffnesses. Figure 4-13 d) shows an irregular wharf with an angle point.

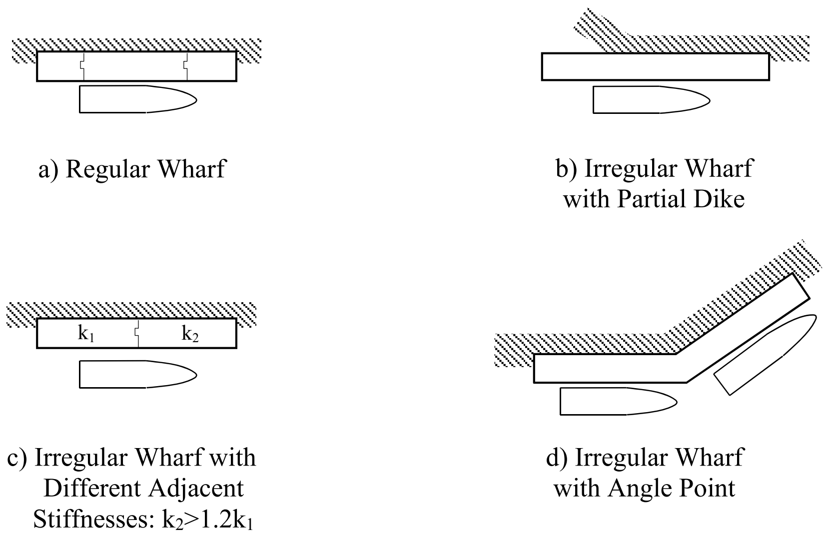


Figure 4-13: Horizontal Marginal Wharf Configurations

Vertical irregularity occurs when soil profiles below the wharf have sharp variations in lateral soil deformation over short vertical distances under seismic response.

4.8.2 Special Cases

4.8.2.1 Crane-wharf Interaction Analysis

A special case for crane-wharf interaction analysis shall be considered if the crane mass impacts the wharf behavior as follows:

$$T_{crane} < 2T_w \tag{4.20}$$

where:

T_{crane} = Translational elastic period of the crane mode with the maximum participating mass

T_w = Effective elastic period of the wharf structure based on cracked section properties

For crane-wharf interaction analysis, the displacement demand, Δ_d of the wharf shall be determined using Nonlinear Time-history Analysis per Section 4.9.4.3. This analysis requires prior written approval by the Port.

4.8.2.2 Linked-wharf Interaction Analysis

A special case for linked-wharf interaction analysis shall be considered for wharf structures if one of the following requirements is met:

1. L_L < 400 feet or L_L > 800 feet
2. B < 100 feet or B > 120 feet
3. More than 20% variation in the initial elastic stiffness of the wharf structure along the wharf length

where:

L_L = length of the shortest exterior wharf unit

B = width of a wharf unit

For linked-wharf interaction analysis, the displacement demand, Δ_d of the wharf shall be determined using Nonlinear Time-history Analysis per Section 4.9.4.3. This analysis requires prior written approval by the Port.

4.9 Demand Analysis

4.9.1 Equivalent Lateral Stiffness Method

The Equivalent Lateral Stiffness method uses a wharf model with piles fixed at the bottom without p-y lateral springs. In this method, the equivalent depth to point of fixity, L_s, is determined as the depth that produces the same top of pile displacement as that given by an individual lateral analysis for a given lateral load applied at top of pile. The equivalent pile length has all soil and associated lateral stiffness removed above its supported base, as shown in Figure 4-14. For different assumed displacements, different pile head conditions, free-head or fixed-head, and different subsurface conditions, L_s is expected to vary from approximately two times pile diameter to approximately twelve times pile diameter for typical container wharf piles.

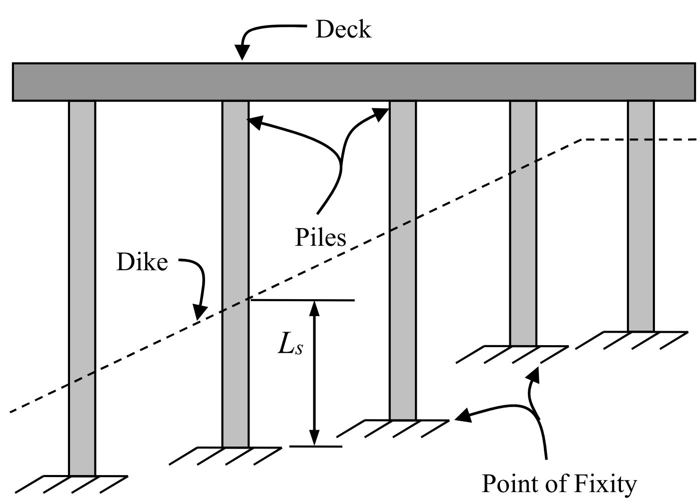


Figure 4-14: Depth to Point of Fixity

This method may not accurately predict pile top and in-ground hinge forces; therefore this method should only be used for preliminary design.

4.9.2 Dynamic Magnification Factor (DMF)

Most of the seismic lateral resistance of marginal wharves is provided by landward piles due to long embedment in soil. The seaward piles are mainly used for gravity loads and might provide about 10% of the overall seismic lateral resistance. This configuration creates eccentricity between the center of mass and the effective center of rigidity for the wharf, which will induce torsional response in the structure under longitudinal excitation. Displacement demand of the critical piles at the end of a segment can be determined by multiplying the displacement demand calculated under pure transverse excitation by Dynamic Magnification Factor, which accounts for torsional response and simultaneous longitudinal and transverse excitations, and interaction across expansion joints. An analytical study utilizing nonlinear time-history analysis was performed to calculate the DMF (Ref 13) using OLE and CLE ground motions with lower and upper bound soil springs conditions. The study was performed on 110-ft wide wharf with single segment, two linked segments and three linked segments. Segment lengths varied between 400 feet, 600 feet, and 800 feet. The study results show that DMF for CLE is always lower than DMF for OLE. Therefore, DMF for DE may conservatively be assumed to be equal to DMF for CLE.

For the single-mode transverse analysis, the displacement demand shall be multiplied by DMF values shown in equations 4.21 – 4.27 for straight wharf units only if all the following conditions are met, otherwise refer to Section 4.8.2.2 for the requirements of special case analysis:

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 4.8.2.1

Single Wharf Unit:

$$DMF = 1.80 - 0.05\, L_L / B \geq 1.10 \text{ for OLE} \tag{4.21}$$
$$DMF = 1.65 - 0.05\, L_L / B \geq 1.10 \text{ for CLE/DE, UB soil springs} \tag{4.22}$$
$$DMF = 1.50 - 0.05\, L_L / B \geq 1.10 \text{ for CLE/DE, LB soil springs} \tag{4.23}$$

Linked Wharf Exterior Unit:

$$DMF = 1.55 - 0.04\, L_L / B \geq 1.10 \text{ for OLE} \tag{4.24}$$
$$DMF = 1.35 - 0.02\, L_L / B \geq 1.10 \text{ CLE/DE, UB soil springs} \tag{4.25}$$
$$DMF = 1.16 - 0.02\, L_L / B \geq 1.10 \text{ for CLE/DE, LB soil springs} \tag{4.26}$$

Linked Wharf Interior Unit:

$$DMF = 1.10 \tag{4.27}$$

where:

L_L = length of the shortest exterior wharf unit

B = width of a wharf unit

LB = lower bound

UB = upper bound

Wharf Exterior Unit = a wharf structure with an expansion joint at one end

Wharf Interior Unit = a wharf structure with expansion joints at both ends

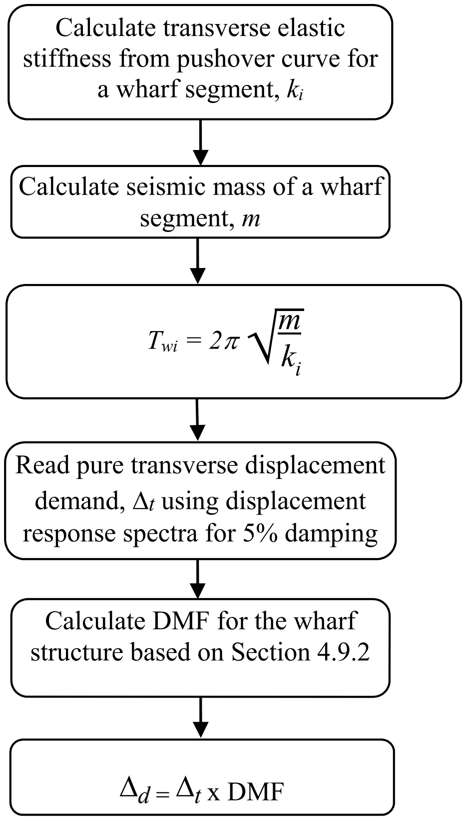
4.9.3 Transverse Single Mode Analysis

Reasonable estimates of displacement demand could be obtained from the Elastic Stiffness Method using cracked-section elastic stiffness of piles. However, improved representation of displacement demand could be obtained using the Substitute Structure Method. If the Elastic Stiffness Method described in Section 4.9.3.1 is used for the wharf design, the displacement demand–to-capacity ratio (DCR) shall be less than or equal to 0.85. If the DCR is larger than 0.85, the Substitute Structure Method described in Section 4.9.3.2 shall be used for verification.

4.9.3.1 Elastic Stiffness Method

The Elastic Stiffness Method is a single-mode pure transverse analysis of a typical wharf strip, refer to Figure 4-2. This method uses the transverse elastic stiffness, k_i, of wharf segment determined from the pushover curve to calculate the pure transverse displacement demand For this method, the damping ratio shall be 5%.

The pure transverse displacement demand shall then be modified with the DMF to include the influence of simultaneous longitudinal response, interaction across expansion joints, and torsional effects, to calculate the displacement demand Δ_d. The flow chart shown in Figure 4-15 demonstrates the analysis steps for the Elastic Stiffness Method.



k_i = Transverse elastic stiffness of a wharf segment determined from the pushover analysis

m = Seismic mass of a wharf segment per Section 4.6.4

T_{wi} = Transverse elastic period of a wharf segment

Δ_t = Transverse displacement demand

DMF = Dynamic Magnification Factor

Δ_d = Displacement demand

Figure 4-15: Flow Diagram for the Elastic Stiffness Method

4.9.3.2 Substitute Structure Method

The Substitute Structure Method is a single-mode pure transverse analysis, modified for simultaneous transverse and longitudinal response interaction across expansion joints and torsional effects by the DMF to calculate the displacement demand. Figure 4-16 demonstrates the analysis steps to calculate the displacement demand using the Substitute Structure Method.

This method is an iterative process that uses the effective secant stiffness, k_e, of a wharf segment at the demand displacement determined from the pushover curve, and an equivalent elastic damping representing the combined effects of elastic and hysteretic damping to determine the pure transverse displacement demand for each iteration, see Figure 4-17.

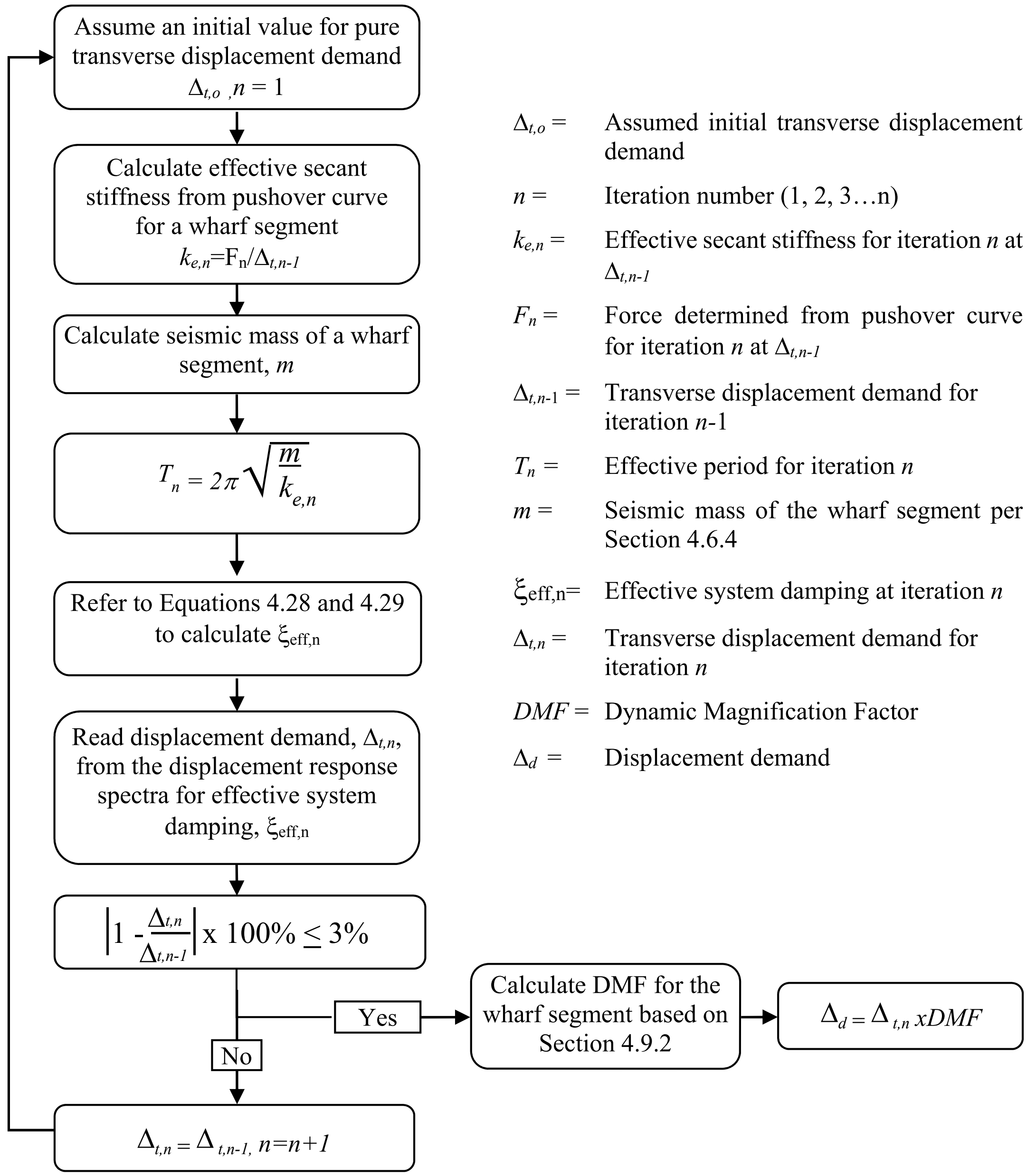


Figure 4-16: Flow Diagram for Substitute Structure Method

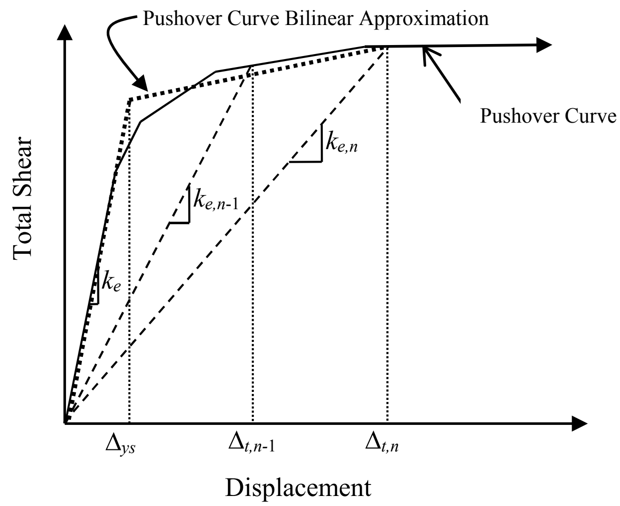


Figure 4-17: Effective System Stiffness for a Wharf Segment

The effective secant stiffness, k_e is the slope of the line that starts from the pushover curve origin point to the point of the first plastic hinge formed in a pile, refer to Figure 4-17. The system yield displacement, Δ_{ys}, is determined from the intersection of the elastic and post-yield branches of the bilinear approximation. The "Equal Energy" approach should be used to estimate the bilinear approximation of the system pushover curve. The bilinear curve should be determined at an estimated displacement demand, $\Delta_{t,n-1}$, for CLE. The system yield displacement will always be larger than the displacement at first yield of piles. The system displacement ductility demand at iteration n, μ_n, is determined as follows:

$$\mu_n = \frac{\Delta_{t,n}}{\Delta_{ys}} \tag{4.28}$$

The effective system damping at iteration n is then found as follows (Ref. 27):

$$\xi_{eff,n} = 0.10 + 0.565\left(\frac{\mu_n - 1}{\mu_n \pi}\right) \tag{4.29}$$

The wharf transverse displacement demand based on pure transverse excitation may be considered to have converged when $\left|1 - \frac{\Delta_{t,n}}{\Delta_{t,n-1}}\right| \times 100\% \leq 3\%$. Once the transverse displacement demand converges, the result shall be modified using the DMF.

4.9.4 Three-Dimensional (3-D) Analysis

Three-dimensional (3-D) demand analyses include Modal Response Spectra Analysis and Nonlinear Time-History Analysis. A typical wharf segment between expansion joints has a large number of piles, which may result in unacceptable matrix sizes for analysis. As an alternative, the structural characteristics of a wharf segment may be modeled by using the "Super-Pile" concept, as explained below.

4.9.4.1 Super-Pile Model

Four super-piles may be used to represent the combined properties and stiffness of piles in the model for a regular wharf segment between expansion joints. For the analysis of an irregular wharf, the super-pile concept should be used with special consideration of the irregular elements.

The super-pile locations are determined based on the locations of the gravity piles and the seismic piles, as shown in Figure 4-18. The gravity piles mainly carry vertical loads, usually carrying less than 10% of the total lateral seismic load, and have less stringent detailing requirements. Seismic piles also carry vertical loads and provide most of the lateral seismic resistance with stringent detailing requirements.

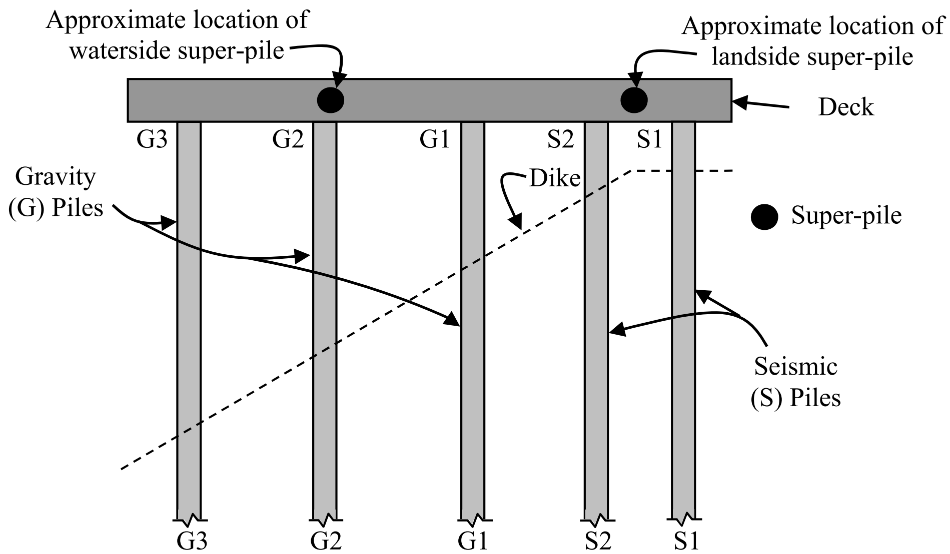


Figure 4-18: Elevation View of Transverse Wharf Segment

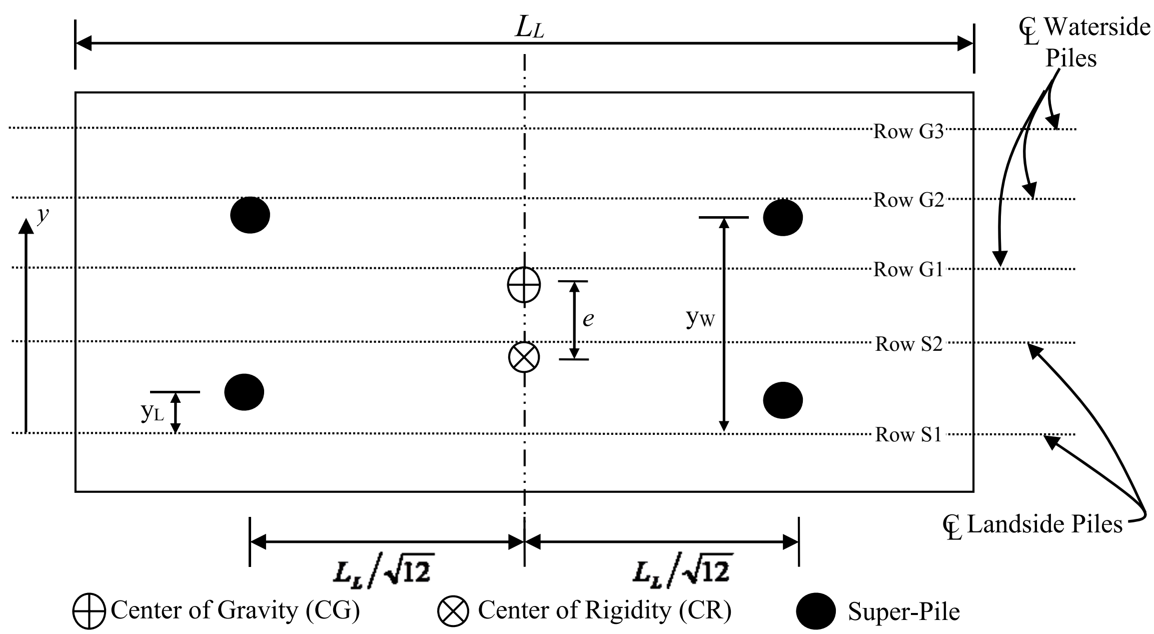


Figure 4-19: Super-pile Locations for a Wharf Segment

The super-piles shown in Figure 4-19 are located at distances y_L and y_W from the center line of landside pile row S1:

$$y_L = \frac{\sum_{i=S1}^{S2} n_i F_i y_i}{\sum_{i=S1}^{S2} n_i F_i} \text{ and } y_W = \frac{\sum_{i=G1}^{G3} n_i F_i y_i}{\sum_{i=G1}^{G3} n_i F_i} \tag{4.30}$$

where:

y_L = Distance of landside super-pile from centerline of landside pile row S1
i = Pile row (i.e. S1, S2, G1-G3 as shown in Figure 4-19 and Figure 4-18)
n_i = Total number of piles in row i for length L_L
F_i = Lateral force per pile in row i from pushover analysis when seismic pile yield reach displacement
y_i = Distance of row i from the landside pile row S1
y_W = Distance of waterside super-pile from centerline of waterside pile row S1

The super-pile stiffness is calculated from the pushover curve for the piles represented. The location of the super-pile should be determined based on the elastic response when the seismic piles reach yield displacement. For compatibility reasons, the gravity piles should have their stiffness determined at the same displacement. The landside super-pile stiffness is equal to the stiffness of all piles on the landside of the dike. The remainder of the total pile stiffness goes to the waterside super-piles. For a regular structure, the two landside super-piles should have equal stiffness, and the two waterside super-piles should have equal stiffness. In order to ensure the correct torsional stiffness under longitudinal response, the super-piles must be located at the center of gyration of the wharf segment. For a regular wharf segment the super-piles must be located at a distance of $L_L / \sqrt{12}$ from the segment centroid, as shown in Figure 4-19.

The simplified model described above is suitable for both Modal Response Spectral Analysis and Nonlinear Time-History Analysis.

4.9.4.2 Modal Response Spectral Analysis

This method is essentially a linear response spectrum analysis for a stand-alone wharf segment. When wharf segments are linked by shear keys at movement joints, Modal Response Spectral Analysis will not provide adequate representation of shear key forces or displacement of the movement joint. A three-dimensional (3-D) linear elastic modal response analysis shall be used with effective section properties to determine lateral displacement demands.

Super-pile model is recommended to perform 3-D modal response spectrum analysis. If the 3-D super-pile model is not used and a full 3-D model is utilized, the soil springs (p-y) need to be modeled as linear springs with effective stiffness, see Figure 4-20. The soil springs with effective secant stiffness based on iterative procedure shall not be used to determine demands.

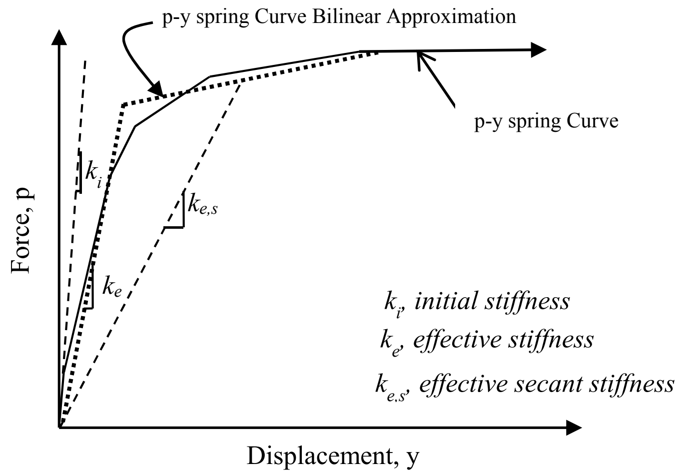


Figure 4-20: P-y Soil Springs

Sufficient modes shall be included in the analysis such that 90% of the participating mass is captured in each of the structure's principal horizontal directions. For modal combinations, the Complete Quadratic Combination (CQC) rule shall be used. A damping ratio of 5% for spectral analysis shall be used unless a higher ratio can be justified.

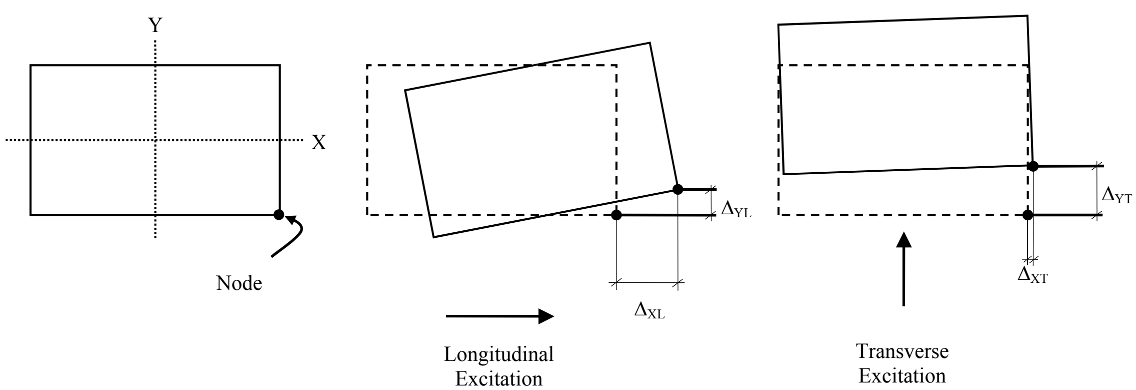


Figure 4-21: Wharf Response due to Longitudinal and Transverse Excitations

Input response spectra shall be applied separately along two orthogonal global axes (longitudinal and transverse), see Figure 4-21. Spectral displacement demand shall be obtained by the maximum of the following two load cases:

Case 1: Combine the displacement demand resulting from 100% of the longitudinal load with the corresponding displacement demand from 30% of the transverse load:

$$\Delta_{X1} = \Delta_{XL} + 0.3\Delta_{XT}$$
$$\Delta_{Y1} = \Delta_{YL} + 0.3\Delta_{YT}$$

Case 2: Combine the displacement demand resulting from 100% of the transverse load with the corresponding displacement demand from 30% of the longitudinal load:

$$\Delta_{X2} = 0.3\Delta_{XL} + \Delta_{XT}$$

$$\Delta_{Y2} = 0.3\Delta_{YL} + \Delta_{YT}$$

where,

Δ_{XL} = X-axis displacement demand due to structure excitation in the longitudinal direction

Δ_{XT} = X-axis displacement demand due to structure excitation in the transverse direction

Δ_{YL} = Y-axis displacement demand due to structure excitation in the longitudinal direction

Δ_{YT} = Y-axis displacement demand due to structure excitation in the transverse direction

Δ_{X1}, Δ_{X2} = Combined X-axis displacement demands from motions in the transverse and longitudinal directions

Δ_{Y1}, Δ_{Y2} = Combined Y-axis displacement demands from motions in the transverse and longitudinal directions

Pile seismic demand, Δ_d, is defined as follows:

$$\Delta_d = \max\left(\sqrt{\Delta_{X1}^2 + \Delta_{Y1}^2} \text{ or } \sqrt{\Delta_{X2}^2 + \Delta_{Y2}^2}\right) \quad (4.31)$$

Nonlinear time-history analysis has shown that the 100% + 30% spectral combination rule to be non-conservative for wharf structures (Ref. 13). If Modal Response Spectra Analysis method is used for the wharf design with soil initial lateral stiffness, the displacement demand to capacity ratio (DCR) shall be less than or equal to 0.85. If the DCR is larger than 0.85 other analysis methods shall be used.

4.9.4.3 Nonlinear Time-History Analysis

Nonlinear Time-History Analysis (NTHA) is the most accurate method for determining displacement demand. Since the inelastic characteristics of the piles can be directly incorporated in the response, the longitudinal and transverse excitation can be simultaneously applied, and the complexities of the movement joints can be directly modeled. NTHA must always be used in conjunction with another simplified analysis approach to verify results. The NTHA results should be within 20% of the results obtained from another simplified approach such as response spectral analysis. When modeling reinforced or prestressed concrete piles or steel piles with concrete plugs, degrading stiffness models such as the Modified Takeda rule (Ref. 41) should be adopted with α=0.3 and β=0.5. Elastic damping should be represented by tangent stiffness damping equivalent to 10% critical damping.

Displacement demands from NTHA shall be based on simultaneous orthogonal horizontal input motions, as defined in Section 2.1. Multiple time-history records will be required to achieve a representative displacement demand for the global model.

When three sets of spectrum-compatible time-history records are used, the envelope value of each response parameter shall be used in the design. When seven sets or more of spectrum-compatible time-history records are used, the average value of each response parameter shall be used.

When NTHA methods are used, a peer review shall be conducted per Section 4.14.

4.10 Structural Capacities

For the evaluation of capacity-protected members and actions, such as shear in piles, and shear and moment in deck beams, and deck slabs, the demand forces shall be determined from using an amplified strength (overstrength) of pile plastic hinges:

$$M_o = 1.25M_p \text{ and } V_o = 1.25V_p \quad (4.32)$$

where

- M_o = Pile overstrength moment capacity
- M_p = Pile idealized plastic moment capacity, which can be calculated by M-ϕ analysis
- V_o = Pile overstrength shear demand
- V_p = Pile plastic shear, which can be calculated based on pile plastic moments or as the maximum shear in the pile from both Upper Bound and Lower Bound pushover analyses

Deck beam and deck slab design moment and shear forces shall be in equilibrium with pile overstrength moment and shear demands.

The wharf structural elements shall be designed for the induced forces due to the lateral seismic deformations. For wharf deck, beam and deck slab, and pile beam/deck joint, the moment, shear and axial demands shall be determined using the load combinations per Section 4.5.2. Any moment demand caused by dead load and 10% live load need to be distributed to the entire frame. The pile earthquake moment represents the amount of moment induced by an earthquake, when coupled with the existing pile dead load moment and pile 10% live load moment, will equal the pile's overstrength moment capacity.

4.10.1 Pile Displacement Capacity

Pile displacement capacity, Δ_c, shall be determined at OLE, CLE and DE using strain limits provided in Table 4-1 for upper bound and lower bound soil conditions. The displacement capacity shall be the lesser of displacement capacity at pile top plastic hinge or at in-ground hinge, determined as follows:

$$\Delta_c = \Delta_y + \Delta_{p,m} \quad (4.33)$$

$$\Delta_{p,m} = \theta_{p,m} \times H \quad (4.34)$$

where:

Δ_c = Displacement capacity

Δ_y = Pile yield displacement, determined from pile initial position to the formation of the plastic hinge being considered (i.e. top hinge or in-ground hinge)

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

$\theta_{p,m}$ = Plastic rotation at LE, CLE, or DE strain limits, determined per equation 4.19

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

The pile yield displacements, Δ_y, of the top and in-ground hinges are obtained from the pushover analysis. Figure 4-22 shows a graphical representation of the displacement capacity calculation for a top plastic hinge. The concept is similar for an in-ground plastic hinge.

For piles with a large unsupported length, L_u and in-ground and top plastic hinges with a ratio $M_{p,\ in\text{-}ground}/M_{p,\ top} > 1.25$, the distance from the top and in-ground plastic hinges to the point of contraflexure becomes uneven. Therefore, the displacement capacity calculation becomes more complex, and the procedure used above will not provide accurate results. Thus, a detailed pushover analysis with proper definition of plastic curvature or rotation limits should be used to determine the displacement capacity.

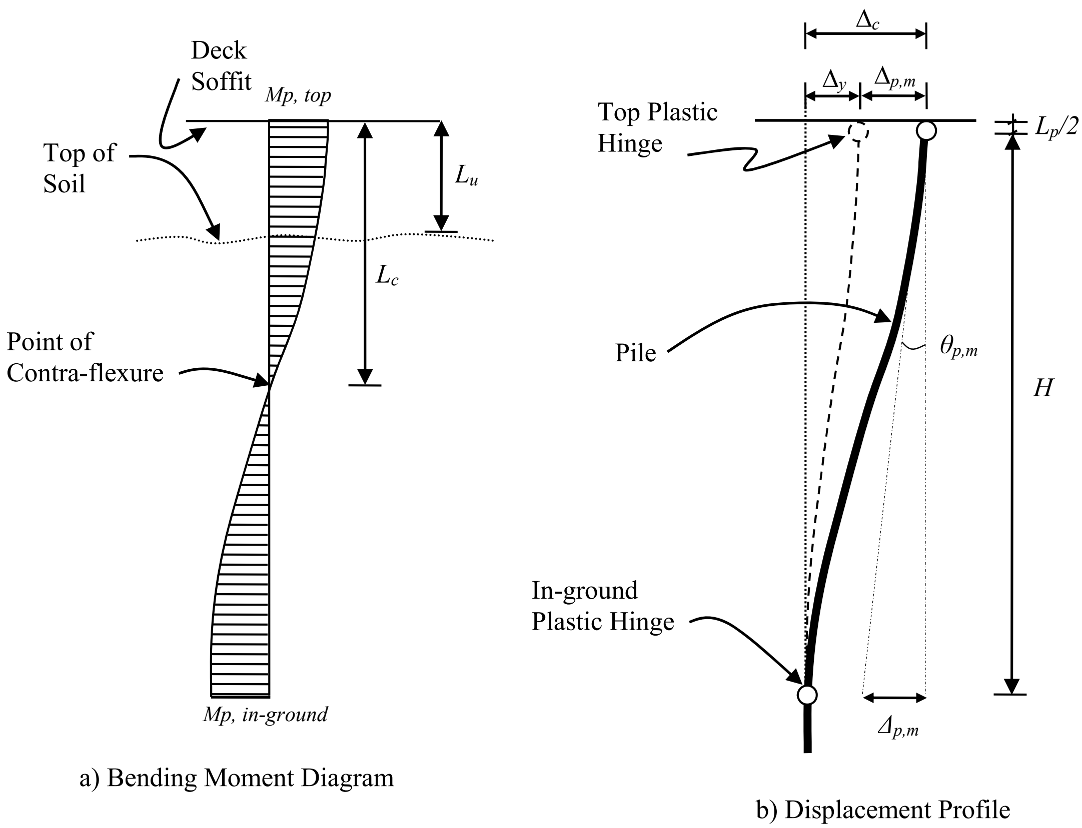


Figure 4-22: Pile Displacement Capacity

4.10.2 Pile Beam/Deck Joint

As previously stated, wharves are designed with weak column (pile), strong beam (deck beam or deck slab) concept. In this capacity, weak column (pile) is required to form plastic hinges and experience permanent deformation due to seismic load. The nominal strength capacity of the beam or deck shall be sufficient to ensure the piles have reached their plastic limit prior to the beam or deck reaching its expected nominal strength. The beam or deck shear and flexural capacities shall be determined based on ACI-318 using strength reduction factors. The superstructure flexural capacity shall be greater than the largest combination of deck dead load moment, deck moment due to 10% of live load, and pile overstrength moment distributed on each side of the pile beam/deck joint (joint). Any distribution factors shall be based on cracked section properties.

For the pile beam/deck joint details shown in Figure 4-28, joint shear requirements are satisfied by providing adequate confinement. The required effective volumetric ratio of confining steel, ρ_s, around the pile dowels anchored in the joint shall be:

$$\rho_s = \text{max of}\left[\frac{0.46A_{sc}}{D'l_a}\left[\frac{f_{ye}}{0.0015E_{sh}}\right] \text{ or } 0.016\right] \quad (4.35)$$

where:

A_{sc} = Total cross-section area of dowels in the joint

D' = Diameter of the confined core measured to the centerline of the confining steel

l_a = Actual embedment length of dowels anchored in the joint

f_{ye} = Expected yield strength of dowels

E_{sh} = Confining steel modulus of elasticity

Less conservative mechanisms for joint shear transfer are suggested in Ref. 39. If an alternate detail is proposed, joint shear principal stresses shall be checked according to ACI-318.

4.10.3 Pile Shear

Pile overstrength shear demand, V_o shall be determined by nonlinear pushover analyses using an overstrength factor of 1.25 including the effect of the axial load on piles due to crane dead load. In lieu of pushover analysis, V_o may be calculated as follows:

$$V_o = 1.25\,(M_{p,\,top} + M_{p,\,in\text{-}ground})/H \quad (4.36)$$

where

$M_{p,\,top}$ = Pile plastic moment capacity at the top plastic hinge including the effect of axial load due to crane dead load

$M_{p,\,in\text{-}ground}$ = Pile plastic moment capacity at the in-ground plastic hinge including the effect of axial load due to crane dead load

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

Steel Piles Shear Capacity

The shear capacity of steel piles shall be determined according to AISC-LRFD or API provisions, where applicable.

Concrete Piles Shear Capacity

The following applies to concrete piles and steel pipe piles with concrete plug and dowels connections to the deck. The shear capacity, ΦV_n, shall be calculated using the method described below.

This method is based on the modified UCSD three-parameter model (Ref. 40) with separate contributions to shear strength from concrete, transverse reinforcement and axial load:

$$\Phi V_n = \Phi(V_c + V_s + V_a) < \Phi\left(0.2 f'_{ce} A_e\right) \tag{4.37}$$

where,

- Φ = Strength reduction factor for shear, equal to 0.85 for OLE and CLE and equal to 1.0 for DE
- V_n = Nominal shear strength
- V_c = Concrete shear strength
- V_s = Transverse reinforcement shear strength
- V_a = Shear strength due to axial load
- f'_{ce} = Expected compressive strength of concrete
- A_e = Effective shear area, equal to 80% of gross cross-sectional area for solid circular and octagonal piles

Concrete Shear Strength, Vc:

$$V_c = k\sqrt{f'_c}A_e \tag{4.38}$$

where:

- k = Curvature ductility factor, determined as a function of μ_ϕ, refer to Figure 4-23
- f'_c = 28-day of unconfined concrete compressive strength (psi)
- A_e = Effective shear area, equal to 80% of gross cross-sectional area for solid circular and octagonal piles
- μ_ϕ = Curvature ductility demand

The curvature ductility demand, μ_ϕ shall be calculated at the demand displacement, and can be found using the formula below:

$$\mu_\phi = 1 + \frac{\phi_{P,dem}}{\phi_y} = 1 + \frac{\theta_{P,dem}}{L_p \phi_y} \tag{4.39}$$

where:

- $\phi_{P,dem}$ = Plastic curvature at displacement demand
- ϕ_y = Idealized yield curvature
- $\theta_{P,dem}$ = Plastic rotation at displacement demand
- L_p = Plastic hinge length

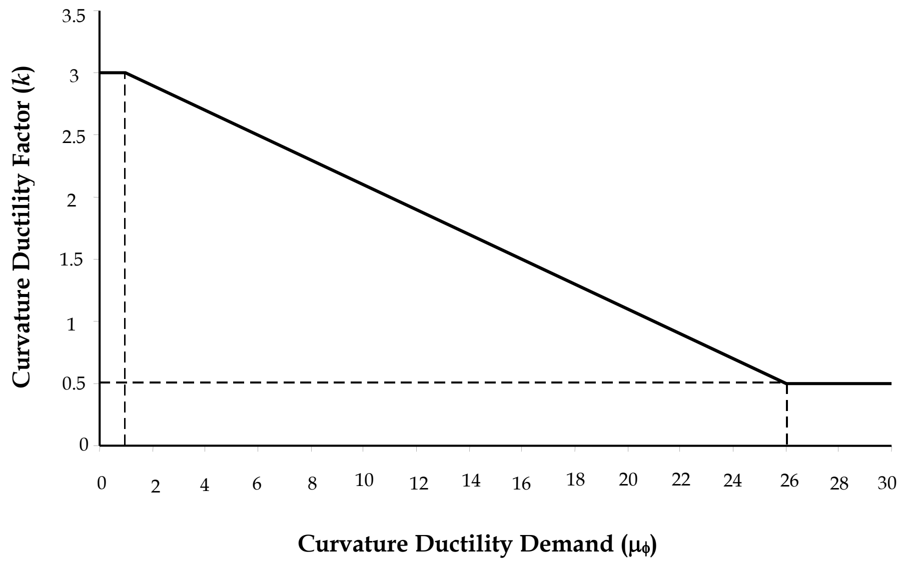


Figure 4-23: Curvature Ductility Factor versus Curvature Ductility Demand

Transverse Reinforcement Shear Strength, V_s:

$$V_s = \frac{\pi}{2}\frac{A_{sp}\, f_{yh}\,(D_p - c - c_o)\cot(\theta)}{s} \tag{4.40}$$

where:

- A_{sp} = Cross-section area of transverse reinforcement
- f_{yh} = Yield strength of transverse reinforcement
- D_p = Pile diameter
- c = Depth from the extreme compression fiber to the neutral axis at flexural strength, see Figure 4-24
- c_o = Clear concrete cover plus half the diameter of the transverse reinforcement, see Figure 4-24
- θ = Angle of critical shear with respect to the longitudinal axis of the pile, taken as 30^o for existing structures and 35^o for new design, see Figure 4-24
- s = Center-to-center spacing of transverse reinforcement along pile axis

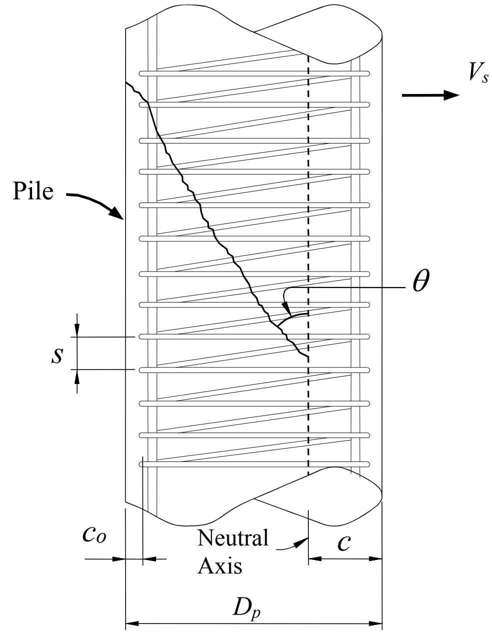


Figure 4-24: Transverse Shear Reinforcement Shear Strength Components

Shear Strength due to Axial Load, V_a:

$$V_a = \beta(N_u + F_p)\tan(\alpha) \tag{4.41}$$

where:

N_u = External axial compression on pile including seismic load; compression is taken as positive, and tension as negative

F_p = Prestress compressive force in pile, taken as zero for top plastic hinge

α = Angle between the line joining centers of flexural compression zones at top and in-ground plastic hinges and the pile axis, see Figure 4-25

β = Axial load shear strength factor, taken as 1.0 for existing structures, and 0.85 for new design

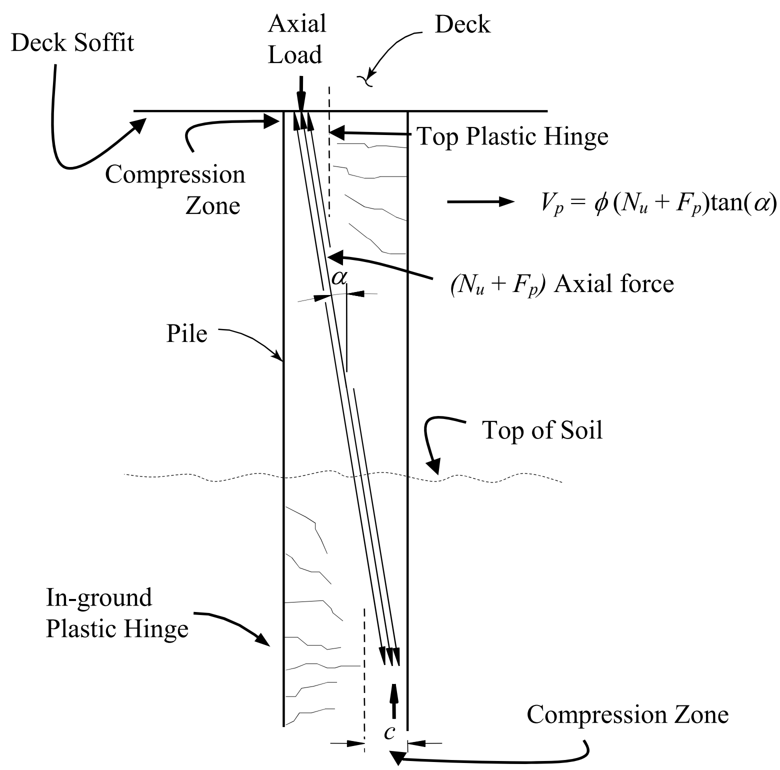


Figure 4-25: Axial Load Shear Strength Components

Alternatively, for piles with curvature ductility, $\mu_\phi < 2$, the pile shear strength may be calculated according to ACI-318 provisions.

4.10.4 P-Δ Effects

Additional secondary forces due to the effect of dead load and lateral seismic displacement demand (P-Δ) shall be included in the analysis for OLE, CLE and DE. The P-Δ effects may be ignored when:

$$\frac{F}{W_{DL}} \geq 4\frac{\Delta_d}{H'} \tag{4.42}$$

where:

- F = Total lateral seismic force of the wharf strip considered at displacement demand, determined from pushover curve
- W_{DL} = Effective dead load of the wharf strip considered
- Δ_d = Displacement demand
- H' = The distance from the maximum in-ground moment to the center of gravity of the deck

4.11 Deck Expansion Joint

Modal Response Spectral Analysis does not directly predict shear key forces between wharf segments at expansion joints. A series of time-history analyses were conducted as part of a research study (Ref. 13) to obtain shear key forces for different wharf configurations, soil properties and ground motion intensities. The results of the study are based on a 110-ft wide wharf section with wharf segment length combinations that varied from 400 feet, 600 feet, and 800 feet. The analysis was conducted using both lower and upper bound soil conditions and OLE and CLE ground motions.

The study results show that for two linked wharf units, the shear key should be designed for a seismic shear key force demand, V_{sk}, as shown below:

$$V_{sk} = \beta_{sk}\left(\frac{F_{\Delta}e}{L_L}\right) \tag{4.43}$$

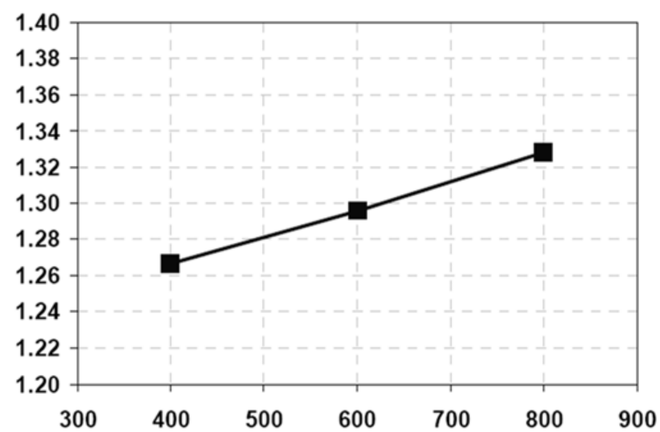
where,

F_{Δ} = Total lateral seismic force of a wharf segment at displacement demand, determined from the pushover curve of an entire wharf segment when the shear key joins two segments of different lengths, F_{Δ} refers to the shorter segment

e = Eccentricity between the wharf center of mass and center of rigidity

L_L = Length of the shorter exterior wharf unit

β_{sk} = Shear key factor, determined as a function of wharf segment length, refer to Figure 4-26



Shear Key Factor (β_{sk})

Figure 4-26: Share Key Factor versus Wharf Segment Length

For wharf section with configurations different than the wharf configurations used in the research study (Ref. 13), special case analysis per Section 4.8.2.2 needs to be performed with prior written approval by the port.

The wharf expansion joint shall be designed for the combined effect of seismic deformation, seismic forces and thermal expansion. For calculating expansion joint shear capacity according to ACI-318, a reduction factor of 0.85 should be used.

4.12 Kinematic Loads

Kinematic loads occurs in piles when the dike begins sliding on a weak soil layer during an earthquake, inducing bending moments in piles beneath the soil surface. Deep in-ground plastic hinges may form due to the dike movement, see Figure 4-27.

Section 2 provides screening criteria for kinematic analysis (nonlinear dynamic soil-structure interaction analysis) of the dike. If a kinematic analysis is required, the geotechnical engineer shall provide displacement profiles for the piles under kinematic load. The structural engineer shall analyze the piles for the given displacement profiles, and the material strains in the piles shall not exceed the strain limits provided in Table 4-1. In addition, the shear demand in piles shall not exceed shear capacity determined according to Section 4.10.3.

For the 24-inch octagonal, precast, prestressed concrete piles and dike configurations that are typically used at POLB and having an embedment length of at least 20 feet into the dike, kinematic load should not be considered when the permanent free field embankment or dike deformation determined per Section 2 are less than 3 inches for OLE, less than 12 inches for CLE and less than 36 inches for DE.

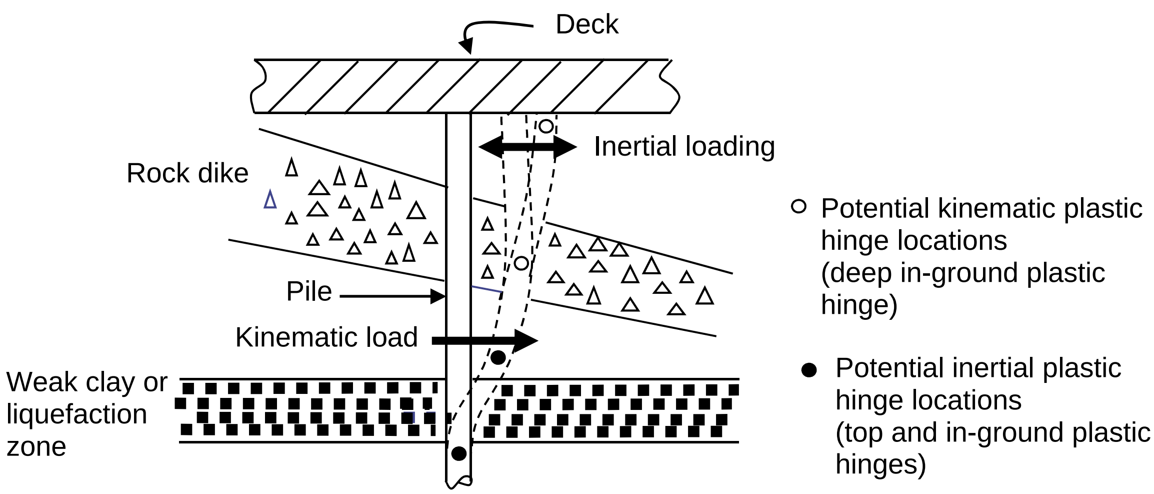
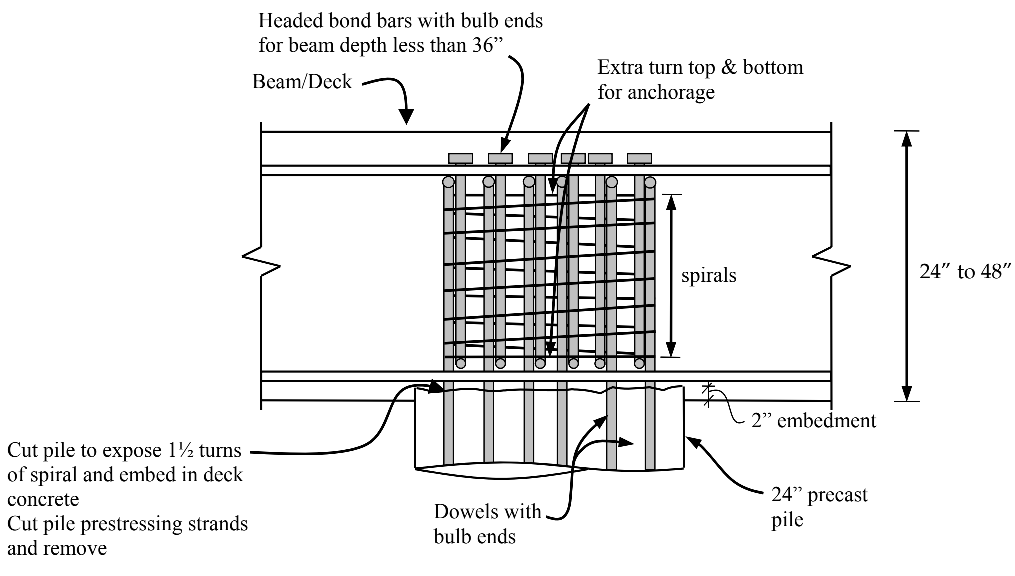


Figure 4-27: Plastic Hinge Formation due to Kinematic Loads

4.13 Seismic Detailing

The details shown in Figure 4-28 are acceptable confinement details for the pile beam/ deck connection. The volumetric ratio of longitudinal reinforcing steel (dowels), ρ shall be between 1% and 4%. The maximum dowel bar size should be No. 11. The dowels shall be developed into the pile according to ACI-318 requirements. The effective volumetric ratio of confining steal, ρ_s shall be provided according to Section 4.10.2. The pile prestressing steel shall be cut-off and removed at the top of the pile.



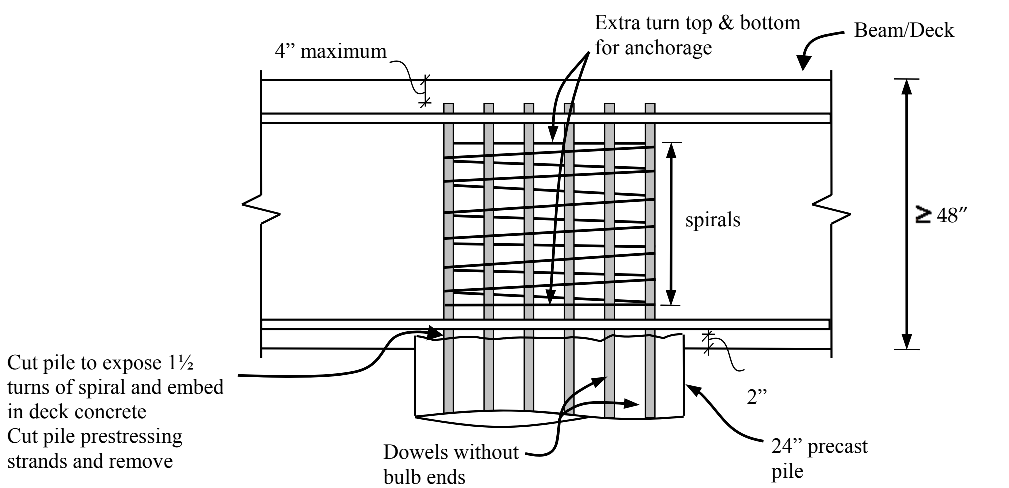


Figure 4-28: Anchorage Details for Pile Dowels

4.14 Peer Review

A peer review of the analysis and design shall be performed by an engineering team selected by the Port for:

1. Presence of new faults at the project site
2. Detailed numerical analysis for liquefaction potential
3. Irregular wharf structures
4. Nonlinear time-history analysis
5. Kinematic analysis (nonlinear dynamic soil-structure interaction analysis)

5 Structural Considerations

5.1 Design Standards

Wharf analysis and design shall comply with the provisions of POLB Wharf Design Criteria and the following codes and standards as applicable. The provisions of POLB Wharf Design Criteria shall supersede the requirements of all other documents if there are disagreements.

1. American Concrete Institute (ACI), "Building Code Requirements for Structural Concrete and Commentary," ACI-318, (Ref. 2).
2. American Forest and Paper Association (AF&PA), "National Design Specifications for Wood Construction and Supplement LRFD/ASD," (Ref. 3).
3. American Institute of Steel Constructions (AISC), "Code of Standard Practice for Steel Buildings and Bridges," (Ref. 4).
4. American Petroleum Institute (API), "Recommended Practice for Planning, Designing and Constructing Fixed Offshore Platforms – Working Stress Design," (Ref. 5).
5. American Society of Civil Engineers (ASCE/COPRI 61-14), "Seismic Design of Piers and Wharves," 2014. (Ref. 11).
6. ANSI/AWS D1.1, "Structural Welding Code – Steel," (Ref. 7).
7. ASCE 7, Standard, "Minimum Design Loads for Buildings and Other Structures," (Ref. 10).
8. California Building Code (CBC), "California Code of Regulations, Title 24," (Ref. 15).
9. California Building Code "Chapter 31F [For SLC], Marine Oil Terminals," also known as "Marine Oil Terminal Engineering Standards (MOTEMS)," (Ref. 17).

5.2 Wharf Geometrics

Controls

The wharf controls shall refer to the "Control" Section of the "Design Criteria and Standard Plans" under "General Criteria," (Ref. 36) for specific instructions as to survey controls.

Vertical Datum

The vertical datum for the POLB is based on NGVD 29 (National Geodetic vertical Datum of 1929), with MLLW elevation = 0.0 feet. The City of Long Beach uses NGVD 29 with MSL elevation = 0.0 feet. As a reference, tidal elevations are provided in Table 5-1 for NAVD 88 (North American Vertical Datum of 1988) and NGVD 29.

Monuments

The Project Plans shall show the location and type for installation of baseline monuments. The Port survey section shall provide the required locations and type of monuments.

Table 5-1: Tidal Elevations

Abbreviation	Description	Elevation (ft)	
		NGVD 29	NAVD 88
---	Highest Observed Water Level[a]	+7.54	+7.16
MHHW	Mean Higher-High Water	+5.43	+5.05
MHW	Mean High Water	+4.71	+4.33
MSL	Mean Sea Level	+2.80	+2.42
MLW	Mean Low Water	+0.95	+0.57
MLLW	Mean Lower-Low Water	0.00	-0.38
---	Lowest Observed Water Level	-2.56	-2.94

[a] The extreme elevations should be used with caution. Irregularities in the predicted tide (seiches) have been known to cause variations of up to 1.0 feet

Wharf Elevations

Wharf elevations shall be determined to maintain facility operations under all tidal conditions and the sea level rise (SLR) predicted by The National Resources Council (NRC 2012) report (Ref. 32). The NRC 2012 report predicts a 0.9 foot increase in SLR by 2050 and a 3.1 foot increase by 2100 for the Los Angeles area. Where applicable, the wharf elevation shall also match that of adjacent facilities, unless directed otherwise by project-specific criteria. Wharf elevations for RO-RO, barge loading and unloading, and special purpose docks are to be determined by project-specific criteria.

Crane Rail Elevations

The top of crane rails (except for wheel flange notches) shall be level with the adjacent deck surface. The top of rail elevation is dictated by drainage conditions for the wharf. This normally results in a relative elevation difference between the waterside and landside crane rails, due to deck transverse cross-slope. If cross-section elevations differ, crane design shall accommodate elevations differential by specifying crane legs to match. The longitudinal elevation of a crane rail shall be constant.

Typical rail elevations are at +15.0 feet for the waterside crane rail. The landside crane rail elevation is based on minimum grade requirements, typically 0.75%.

The allowable tolerances for the top of crane rail elevation shall be 1/8 inch, and 1/16 inch for any 10 feet along the rail length.

5.3 Construction Materials

Cement

Portland cement type II modified shall be used.

Reinforcing Steel

Grade 60 reinforcing steel shall be used. Epoxy coating is not permitted without prior written approval by the Port.

Prestressing Steel

270 ksi strands shall be used for piles prestressing steel.

Cast-in-place Concrete

Cast-in-place concrete strength (f_c') shall be a minimum of 5,000 psi at 28 days. Minimum concrete cover over reinforcing steel shall be 2 inches for the top of wharf face, and 3 inches for all other faces.

Non-prestressed Precast Concrete

Precast non-prestressed concrete strength (f_c') shall be a minimum of 5,000 psi at 28 days. Minimum concrete cover over reinforcing steel shall be 2 inches for the top face, and 3 inches for all other faces.

Prestressed Concrete Piles

Precast prestressed concrete piles strength (f_c') shall be a minimum of 6,500 psi at time of driving, and 4,500 psi at time of prestressing steel stress transfer. Minimum concrete cover over transverse reinforcing steel shall be 2½ inches.

Prestressed Precast Concrete (other than piles)

Precast prestressed concrete strength (f_c') shall be a minimum of 6,000 psi at 28 days. Minimum concrete cover over reinforcing steel shall be 2 inches for the top face, and 3 inches for all other faces.

5.4 Wharf Components

5.4.1 Wharf Deck

Beam/Slab

This system consists of a cast-in-place concrete slab supported by cast-in-place beams (pile caps) that are supported by piles. When beams (pile caps) exist both longitudinally and transversely, this system is also called a “waffle slab”.

Flat Slab

The flat slab system consists of a cast-in-place concrete deck supported by piles. The thickness of the deck slab is normally controlled by slab punching shear capacity to resist pile reactions. The slab depth in this case can be reduced by the use of capitals or shear caps under the deck at pile locations.

Flat slab system may have larger seismic mass when compared to a beam/slab system.

Precast Slab Panels

This system consists of precast deck slab panels placed on top of cast-in-place bent caps supported by piles. The entire system can also be covered with a reinforced cast-in-place topping slab for continuity. Precast deck slabs have the advantage of reducing the amount of required falsework, which lowers both the construction cost and construction duration. However, the bent cap beams reduce the construction tolerance of the pile placement (i.e. misalignment). This can be an important factor in locations of construction nearby or replacing existing structures, where submerged obstacles can be expected during pile driving. Additionally, the depth of the bent cap beams with this type of deck can become relatively large as the pile spacing is increased. This can place portions of the beam in the tidal zone, potentially increasing the corrosion potential of the superstructure.

Ballasted Decks

Ballasted decks are normally not a preferred system due to their high seismic mass and associated higher seismic demands. However, this type of system works well when deck accessories such as railroad tracks are necessary, and a large number of utilities and pipelines are required. A dropped deck or ballasted section is necessary in utility corridors, and can be combined with any of the above systems. Ballasted decks are also useful for non-container and general cargo (break-bulk) wharves where point loads from odd shaped equipment and freight are operated.

5.4.2 Expansion Joints

Expansion joints are joints 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). Expansion joints locations are determined by thermal forces, and are typically placed at a maximum of approximately 800 feet along the wharf.

The wharf expansion joints shall be designed for the combined effect of seismic deformation, seismic forces and thermal expansion.

5.4.3 Cut-off Wall

A cut-off wall is a vertical subsurface barrier designed to prevent erosion of backland materials under the wharf. It is normally constructed along the back edge of the wharf with a sufficient depth to maintain kick-out stability, while still providing erosion protection. It can be of either precast or cast-in-place construction. Cut-off wall shall not be relied on for seismic resistance of the wharf structure.

5.4.4 Crane Rails

Support System

Crane rails shall be supported by a continuous weight distributing sole plate with attached rail clips, a continuous flexible impact pad, and the appropriate crane rail. The assembly shall be galvanized and installed in a recessed pocket with an epoxy fill under the sole plate and asphalt concrete (AC) fill around the rail assembly to match the finished grade of the wharf deck, with block-outs for wheel flanges. Crane rails shall be continuously welded at expansion joint.

Crane Stops

Crane stops are provided at the ends of the wharf to restrict crane motion beyond their intended travel limits. The crane stop bumpers shall be positioned per crane manufacturer's recommendation. See Section 3.3.3 for further discussion on crane stops.

Crane stowage pins

The number of crane stowage pins and their location shall be based on operational considerations. They are typically placed at ends of wharf, and at intermediate points for long wharves. Consideration should be given to the number of cranes, length of wharf, location of power source, and distance between stowage pins.

5.4.5 Fenders and Mooring Hardware

Fenders and mooring hardware spacing shall be determined based on operational requirements and design vessels characteristics. Also, mooring hardware shall be located to not cause line interference with fenders. Due to the likelihood of bulbous bow vessels, a minimum distance of 8.5 feet shall be provided between the supporting structure piling and the face of a compressed fender. This requirement is not applicable to fender piling, if used.

To minimize additional crane boom reach, the maximum allowable stand off for fenders shall be considered per crane and vessel configurations. Fenders shall be located along the wharf face at a distance that will minimize the chance the vessel will contact the concrete face of the wharf. Vessel dimensions and allowable hull pressure shall also be considered in positioning and sizing fenders.

Mooring bollards shall be placed at intervals based on multiples of bent spacing, but no more than 60 feet to avoid hull/wharf strikes. Refer to Section 3.7 for mooring loads.

5.4.6 Safety Ladder

Safety ladders shall be provided at a maximum spacing of 400 feet along the face of the wharf.

5.4.7 Piling

Clearance

An approximate minimum of 4 feet clearance shall be used between the deck/ beam soffit and top of dike to allow for adequate post-earthquake inspection and repairs.

Concrete Piles

The Port's standard pile is a 24-inch octagonal precast prestressed concrete pile. Larger size solid or hollow piles may be proposed for situations where the 24-inch octagonal pile is not a cost effective solution. The Port prefers to use only one size pile for the entire structure, varying only the length and prestress level, unless project conditions and/or cost savings prove otherwise. The use of piles other than the standard 24-inch octagonal precast prestressed piles is not permitted without a prior written approval by the Port.

Steel Piles

The use of steel piles is strongly discouraged due to the corrosion potential and associated higher maintenance cost. Additionally, corrosion barrier coating systems and encasements impede routine visual pile inspections. Steel piles should only be used when project-specific criteria and site circumstances dictate.

Battered Piles

The use of battered piles is not permitted without a prior written approval by the Port. However, battered piles may be used for isolated structures with low seismic mass, such as landside anchors, mooring and breasting dolphins.

5.4.8 Guard Timber

On the waterside edges of the wharf deck, a curb or chemically treated guard timber 10-inch high by 12-inch wide shall be used. Notches shall be provided on the underside of the guard timber to permit drainage. The guard timber shall be anchored to the deck slab using recessed bolts or pins, and should include vessel's net anchor rings.

5.4.9 Trench Cover Plates

Galvanized steel checker plate shall be used for trench covers. Special consideration should be given to the hinge design due to the weight of the plates. The preferred location of the power trench is on the waterside of the waterside crane rail. The trench cover plates shall be designed using the applicable load specified in Section 3.

5.4.10 Cable Trench

Trench for crane power cables shall be covered with a continuous flexible material, fabricated from rubber with inlaid steel reinforcement. The trench shall be a minimum width and depth to accommodate the crane power cables anticipated at the facility.

5.4.11 Inclinometer Tubes/ Motion Instrumentation

The decision to install inclinometer tubes/ strong motion instrumentation in the wharf structure should be made during design, and should be coordinated with other instrumentations functioning within the Port.

5.4.12 Dike Scour

Submerged slopes shall be protected to withstand the effects of ocean waves, tidal currents, propeller wash, and vessels wakes. At a minimum, the slope protection shall consist of an under layer of quarry run rock and an armor layer consisting of nominal 500 pounds armor stone. The submerged slope protection shall at a minimum extend above all expected water levels and wave run-up elevations. Other approaches to slope protection shall require prior written approval by the Port.

Design current speed, wave height and other coastal hydrodynamic processes shall be defined and approved by the Port. Armor design and analysis shall consider the design water level including sea level rise, design wave conditions, design current speeds, design currents from propeller and bow thruster wash, design ship wake and any other potential sources of currents and waves such as tsunami (Ref. 43). The design vessel, for vessel related factors, is provided in Section 3.6. An approach for addressing sea level rise is given in Ref. 44.

5.5 Structural Analysis Considerations

Materials Properties

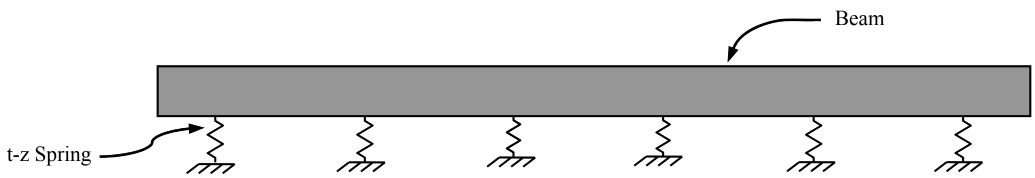
For service load analysis such as dead loads, live loads, and wind loads, the material properties shall be based on the relevant design code, see Section 5.1.

Section Properties

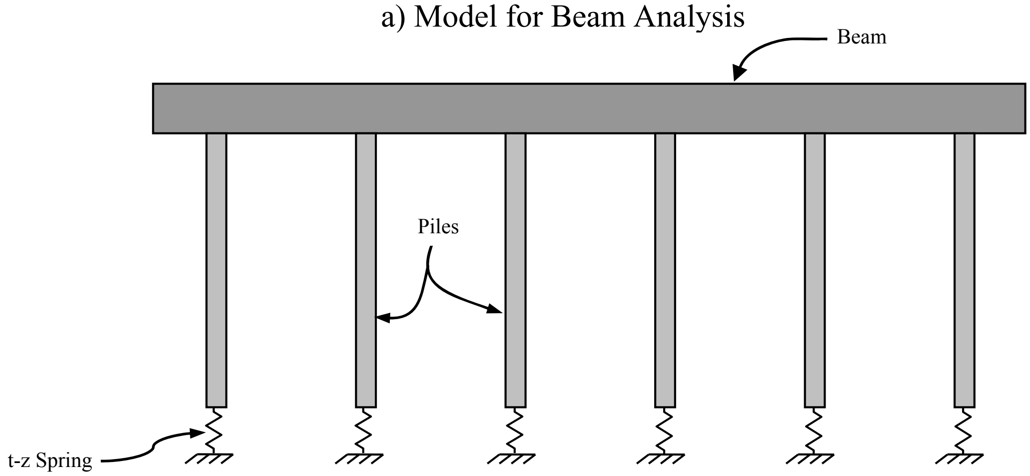
For temperature or creep loads, the effective moment of inertia (I_{eff}) should be used for piles, see Section 4.6.3. For all other service loads, gross moment of inertia (I_{gross}) shall be used.

Beam on Elastic Foundation Model

For modeling the wharf structure frame as beams on elastic foundation, UB and LB t-z springs shall be used for the analysis including the pile elastic shortening, see Section 2. To calculate moments in the beam and axial force in the piles, the t-z springs may replace modeling the piles, as shown in Figure 5-1-a). The piles should be included in the model to determine moments and shear in the piles, as in Figure 5-1-b).



a) Model for Beam Analysis



b) Model for Beam and Pile Analysis

Figure 5-1: Beam on Elastic Foundation

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