Port of Long Beach
Wharf Design Criteria

Version 2.0

1/30/2009

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List Symbols

A_e	Effective shear Area
A_{gross}	Gross cross-sectional area
A_{sc}	Total area of dowel bars in the connection
A_{sp}	Area of confining reinforcement
B	Width of a wharf unit
BE	Berthing Load
BU	Buoyancy Load
D	Dead Load
D'	Diameter of confining reinforcement core, measured to the centerline of the confinement
$D_{c.g.}$	The distance from the deck soffit to the center of gravity of the deck
D_p	Pile Diameter
DMF	Dynamic magnification factor
E	Earth Pressure Load
E_c	Modulus of elasticity of concrete
E_{ps}	Modulus of elasticity for prestressing steel
E_s	Modulus of elasticity of steel
EQ	Earthquake Load
F	Base shear of the wharf strip obtained from a pushover analysis
F_i	Lateral force per pile in row i from pushover analysis
F_p	Prestress compressive force in pile
G_c	Shear modulus (modulus of rigidity) for concrete
H	The distance between the center of the top hinge and center of the in-ground hinge
H'	The distance from the maximum in-ground moment to the center of gravity of the deck
I	Impact Load
I_{eff}	Effective moment of inertia
I_{gross}	Gross moment of inertia
J	Polar moment of inertia
J_{eff}	Effective polar moment of inertia
K	Factor applied to dead load to account for the effects of vertical ground acceleration
K_e	Confinement effectiveness coefficient
L	Live Load
L_c	The distance from the critical section of the plastic hinge to the point of contra-flexure
LB	Lower Bound
L_L	Length of the shortest exterior wharf unit
L_p	Plastic hinge length
L_s	Equivalent depth to fixity
M	Mooring Load
M_{dl}	Unfactored dead load moment

$M_{eq}^{R,L}$	The portion of M_{eq}^{pile} and $V_o \times D_{c.g.}$ (moment due to overstrength shear) distributed to the adjacent right or left deck span
M_{eq}^{pile}	The pile moment due to seismic loads that combines with the pile dead load and 10% live load moment to equal the pile overstrength moment, M_o.
M_{ll}	Unfactored live load moment due to 10% of the live load on the deck
M_n	Nominal moment capacity
$M_n^{deck,R,L}$	The nominal moment capacity of the adjacent right or left deck span
M_o	The pile overstrength moment capacity
M_p	Idealized plastic moment capacity
M_y	Moment at first yield
N	Number of pile rows
N_u	External axial compression on pile including seismic load
P	Mooring line loads
PGA	Peak ground acceleration
R	Creep/Rib Shortening Load
R_F	Force perpendicular to the fender panel due to berthing loads
S	Shrinkage Load
T	Temperature Load
T_{crane}	Period of the crane mode with the maximum participating mass
T_n	Effective period for iteration n
T_w	Effective elastic stiffness of the wharf system
T_{wi}	Initial period of the wharf based on cracked section properties
U	Pile unsupported length from the soffit to the ground
UB	Upper Bound
V_a	Shear strength due to axial load
V_c	Shear strength from concrete
V_{design}	Design shear, equal to V_o
V_F	Fender Shear Force
V_n	Nominal shear strength
V_o	The pile overstrength shear demand
V_p	The pile plastic shear
V_s	Transverse reinforcement shear strength
V_w	Wind speed at elevation 33 ft.
$V_\perp$	The ship approach velocity perpendicular to the wharf
W	Wind Load
W_{DL}	Dead load of the wharf segment
W_W	Waterside crane wheel loading
W_L	Landside crane wheel loading
c	Depth from extreme compression fiber to neutral axis at flexural strength
c_o	Concrete cover width to the center of hoop or spiral
d_{bl}	Diameter of longitudinal reinforcement
d_{gap}	Distance between the top of the steel shell pile and the soffit
e	Eccentricity between the center of mass and the center of rigidity
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} Specified maximum prestressing steel tensile strength
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 stress
f_u Specified maximum steel tensile strength
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 the longitudinal reinforcement steel
f_{yh} Yield strength of confining steel
f_{yhe} Expected yield strength of transverse reinforcement
g Acceleration of gravity
h Elevation above water surface of wind data in feet
i Pile row
k Curvature ductility factor as a function of μ_ϕ
k_e System secant stiffness
k_i Initial stiffness of the structure taken from the pushover analysis
$k_{1,2}$ Stiffness of the wharf
l_a Actual embedment length provided
l_{sp} Strain penetration length
m Seismic mass of the wharf segment
$m_{crane,deck}$ Part of the crane mass positioned close to wharf deck level
m_{wharf} Mass of wharf portion occupied by the crane
n_i Total number of piles in row i for length L_L
r Ratio of the second slope over the elastic slop of the pile stiffness curve.
Center to center spacing of confining reinforcement along pile axis
x_i Distance of row i from the landside pile row
x_L Distance of landside super-piles from the landside pile row
x_W Distance of the waterside super-piles from the landside pile row
α Angle between the line joining the centers of flexural compression in the deck/pile hinge and in-ground hinge and the pile axis
β Axial pile shear strength reduction factor
γ Displacement Capacity Factor
Δ_c Pile displacement capacity
Δ_d Pile demand displacement for three-dimensional response
$\Delta_{p,m}$ The plastic displacement capacity due to rotation of the plastic hinge at the OLE, CLE, or DE strain limits
Δ_t Displacement demand based on transverse response
$\Delta_{t,0}$ Initial assumed displacement demand for Substitute Structure method
$\Delta_{t,n}$ Displacement demand based on transverse response for iteration
Δ_{X1}, Δ_{X2} Combined X-axis displacement from motions in the transverse and longitudinal directions
Δ_{XL} X-axis displacement due to structure excitation in the longitudinal direction
Δ_{XT} X-axis displacement due to structure excitation in the transverse direction

Δ_{Y1}, Δ_{Y2}	Combined Y-axis displacement from motions in the transverse and longitudinal directions
Δ_{YL}	Y-axis displacement due to structure excitation in the longitudinal direction
Δ_{YT}	Y-axis displacement due to structure excitation in the transverse direction
Δ_y	Pile yield displacement
$\Delta_{y,i}$	Yield displacement of a pile in row *i* from pushover analysis
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_{pi}	Initial prestressing steel tensile strain after losses
ε_{pue}	Expected ultimate strain for prestressing steel
ε_{pye}	Expected yield strain for prestressing steel
ε_s	Total 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	Curvature at the OLE, CLE, or DE strain limit
$\phi_{p,dem}$	Plastic curvature at demand displacement
$\phi_{p,m}$	Plastic curvature for 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	Reduction factor for shear, taken as 0.85
$\mu_{\Delta,i,n}$	Displacement ductility for row i at iteration n, defined as $\frac{\Delta_{t,n-1}}{\Delta_{y,i}}$
μ_ϕ	Pile curvature ductility
μ_f	Coefficient of friction
θ	Angle of critical crack to the pile axis
θ_m	Total rotation at the OLE, CLE, or DE strain limit
$\theta_{p,m}$	Plastic rotation for the OLE, CLE, or DE strain limit
$\theta_{p,dem}$	Plastic rotation at demand displacement
θ_u	Ultimate rotation
θ_y	Idealized yield rotation
ρ	Effective volumetric ratio of longitudinal reinforcing steel
ρ_s	Effective volumetric ratio of confining steel
$\xi_{eff,i}$	Effective damping for a pile at row *i*
$\xi_{eff,system}$	Effective damping of the entire wharf system

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1 Introduction

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

Design guidelines and reference materials cited throughout this handbook will be revised from time to time as required. Updates and revisions occurring during design shall be followed as directed by the Port. The latest published editions of all references including all addenda shall be used in the design.

This document is Version 2.0 of the “Port of Long Beach Wharf Design Criteria” and it updates and supersedes the previous Version 1.0 that was published on March 20, 2007.

This document was prepared for the POLB under the leadership of Cheng Lai with the POLB and by a team of consultants consists of Moffatt & Nichol, PBS&J, Earth Mechanics, Inc. and P2S Engineering. The expert review team included Dr. Nigel Priestley, Emeritus Professor, Department of Structural Engineering, University of California, San Diego and Dr. Geoffrey Martin, Professor, Department of Civil Engineering, University of Southern California.

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. 21) and "Addendum to Port-wide Ground Motion Study, Port of Long Beach, California" (Ref. 22). No deviation from these ground motions shall be allowed unless prior approval by the Port is granted.

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 review and 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.

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 during 50 years. The CLE event occurs less frequently than the OLE event, but more frequently than the DE event. CLE has a higher intensity than OLE but lower intensity than DE.

Code-Level Design Earthquake (DE)

The DE shall comply with the Design Earthquake requirements by the 2007 California Building Code (Ref. 17) and ASCE 7-05 (Ref. 11). The DE event occurs less frequently than the OLE and CLE events and has a higher intensity than the other two events.

Recommended design acceleration response spectra for OLE, CLE and DE for different ground conditions are shown in Figure 2-1 and Figure 2-2. Further details are provided in References 21 and 22.

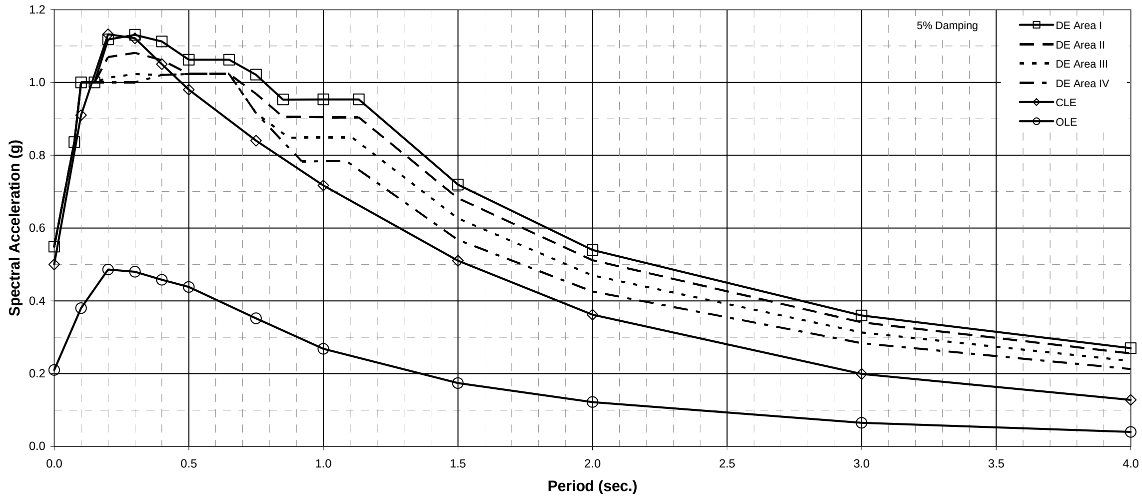


Figure 2-1: Design Acceleration Response Spectra for Unimproved Ground Conditions

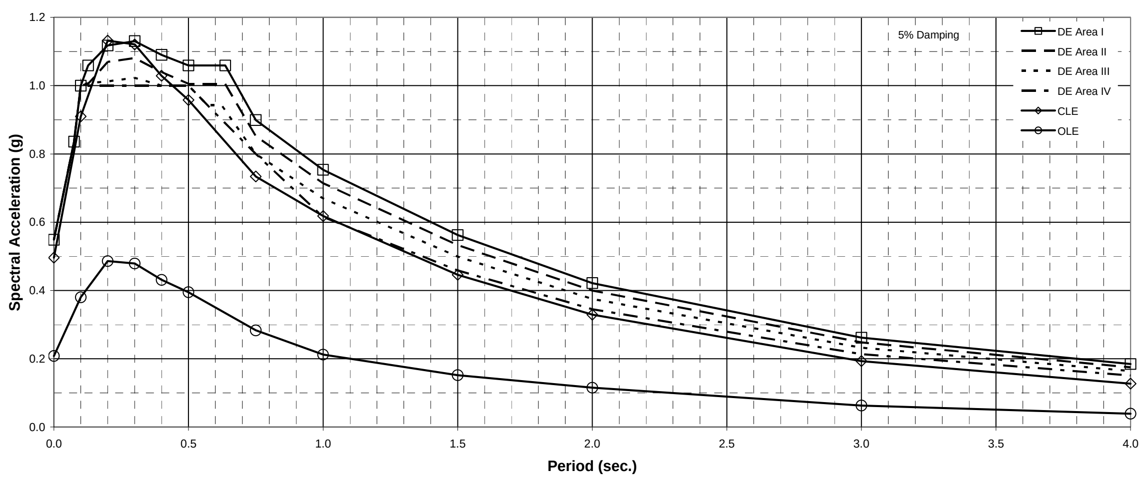


Figure 2-2: Design Acceleration Response Spectra for Improved Ground Conditions

2.2 Site Characterization

Site characterization shall be based on site-specific information. Reviewing and cataloging of 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. 25). If a known 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 ft. 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. 36) and “Design and Construction of Driven Pile Foundations” (Ref. 24) 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. 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. 43), “Recommended Procedures for Implementation of DMG Special Publication 117, Guidelines for Analyzing and Mitigating Liquefaction Hazards in California” (Ref. 34), “Chapter 31F, 2007 California Building Code” (Ref. 18), “Liquefaction Susceptibility Criteria for Silts and Clays” (Ref. 16), and “Soil Liquefaction During Earthquakes” (Ref. 26) or other appropriate documents.

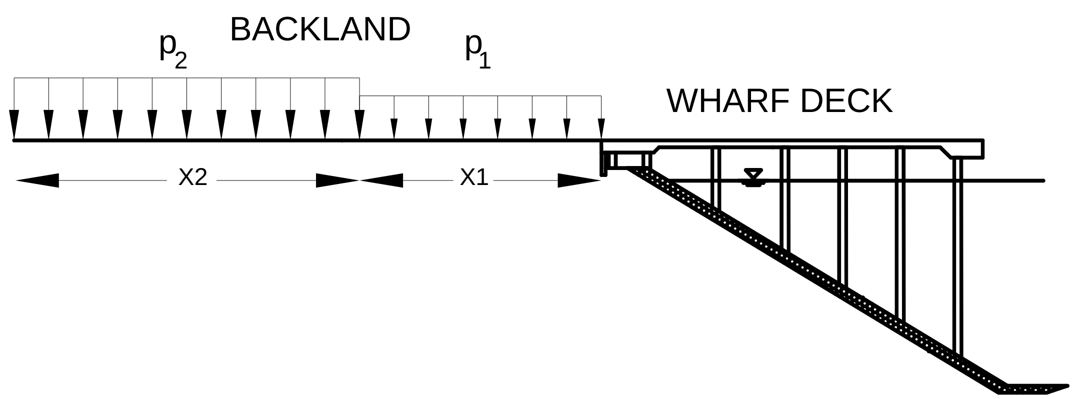
If liquefaction is shown to be initiated in the above evaluations, the particular liquefiable strata and their thicknesses (including zones of liquefaction induced in the backland area) should be clearly shown on site profiles. Resulting hazards associated with liquefaction should be addressed, including 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 may be required by an engineering team selected by the Port.

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 in Table 2-1.

The surcharge loading values recommended in the following subsections may be revised based on project-specific loading information, upon prior approval by the Port.

Table 2-1: Minimum Requirement for Slope Stability Analyses



Loading Conditions	p_1[a] (psf)	X_1 (ft)	p_2[a] (psf)	X_2 (ft)	Min. F.O.S[b]
Static Condition	250	75 ft	1,200	Remaining Backland	1.5
Temporary Condition (See Section 2.4.1)	250	Entire Backland	-	-	1.25
Pseudo-Static Seismic Condition	250	75 ft	800	Remaining Backland	- [c]
Post-Earthquake Static Condition	250	75 ft	800	Remaining Backland	1.1

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

[b] F.O.S. – 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 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. 13), or other appropriate documents. Backland loading shall be represented by 250 psf for the first 75 ft. from the back end of the wharf deck and 1,200 psf for the remaining backland area. 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.

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 represented by 250 psf for the first 75 ft. from the back end of the wharf deck and 800 psf for the remaining backland area.

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. 34), "Recent Advances in Soil Liquefaction Engineering: A Unified and Consistent Framework" (Ref. 41), "Soil Liquefaction During Earthquakes" (Ref. 26), or other appropriate documents.

Using a seismic coefficient of one-third 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 a design earthquake 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 represented by 250 psf for the first 75 ft. from the back end of the wharf deck and 800 psf for the remaining backland area.

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. 21) and "Addendum to Port-wide Ground Motion Study, Port of Long Beach, California" (Ref. 22). 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 presence of the wharf foundation system should not be included in the “free field” evaluations.

For the OLE and CLE, initial lateral spread estimates should be made using the Newmark curves provided in “Port-Wide Ground Motion Study, Port of Long Beach, California” (Ref. 21). For the DE, initial lateral spread estimates should be made using the Newmark displacement curves provided in “Seismic Analysis and Design of Retaining Walls, Buried Structures, Slopes and Embankments” (Ref. 43) or other appropriate documents. Additional analyses may be performed with prior 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. 35) 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. 34) 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. 35) or other appropriate documents.

2.6.2 Earth Pressures Under Seismic loading

The effect of earth pressures on wharf structures 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. 21) and "Addendum to Port-wide Ground Motion Study, Port of Long Beach, California" (Ref. 22).

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 [See "Foundation and Earth Structures" (Ref. 35); "Seismic Analysis and Design of Retaining Walls, Buried Structures, Slopes, and Embankments" (Ref. 43)]. If Mononabe-Okabe equations are not appropriate, methods outlined in "Seismic Analysis and Design of Retaining Walls, Buried Structures, Slopes, and Embankments" (Ref. 43) 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 ft. to -100 ft. MLLW. 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 3-3. Guidelines for estimating axial pile capacities are provided in "Foundation and Earth Structures" (Ref. 35), "Recommended Procedures for Planning, Designing, and Constructing Fixed Offshore Platforms" (Ref. 7), and other appropriate documents. A minimum factor of safety of 2.0 shall be achieved on the ultimate capacity of the pile when using the largest of the service load combinations provided in Table 3-1. In addition, piles supporting the waterside crane rail should have a minimum factor of safety of 1.5 on ultimate capacity 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 (loads other than dead load), 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 combinations estimated from Table 3-3. For the earthquake load case in Table 3-3, an additional 10% of the design uniform live load should be included, per Section 4.6.4. 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 plus service load combinations 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 axial geotechnical capacity of piles 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 1.0. When seismically-induced settlements are predicted to occur during design earthquakes, the drag loads should be computed, and the combination of drag load and service load should be determined. Only the tip resistance of the pile and the side friction resistance below the lowest layer contributing to the downdrag should be used in the capacity evaluation. The ultimate axial geotechnical capacity of the pile should not be less than the combination of the seismically induced downdrag force and the maximum of the service load combinations.

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 2-3. If the springs are developed at the pile tip, the tip should include both the 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 2-3. If t-z springs are developed at the pile top, the appropriate elastic shortening of the pile should also be included in the springs. Linear or nonlinear springs may be developed if requested by the structural engineer.

During development of the axial soil springs, the ultimate capacity of the soil resistance along the side of the pile and at the tip of the pile should be used. 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).

2.7.3 Upper and Lower Bound Springs

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

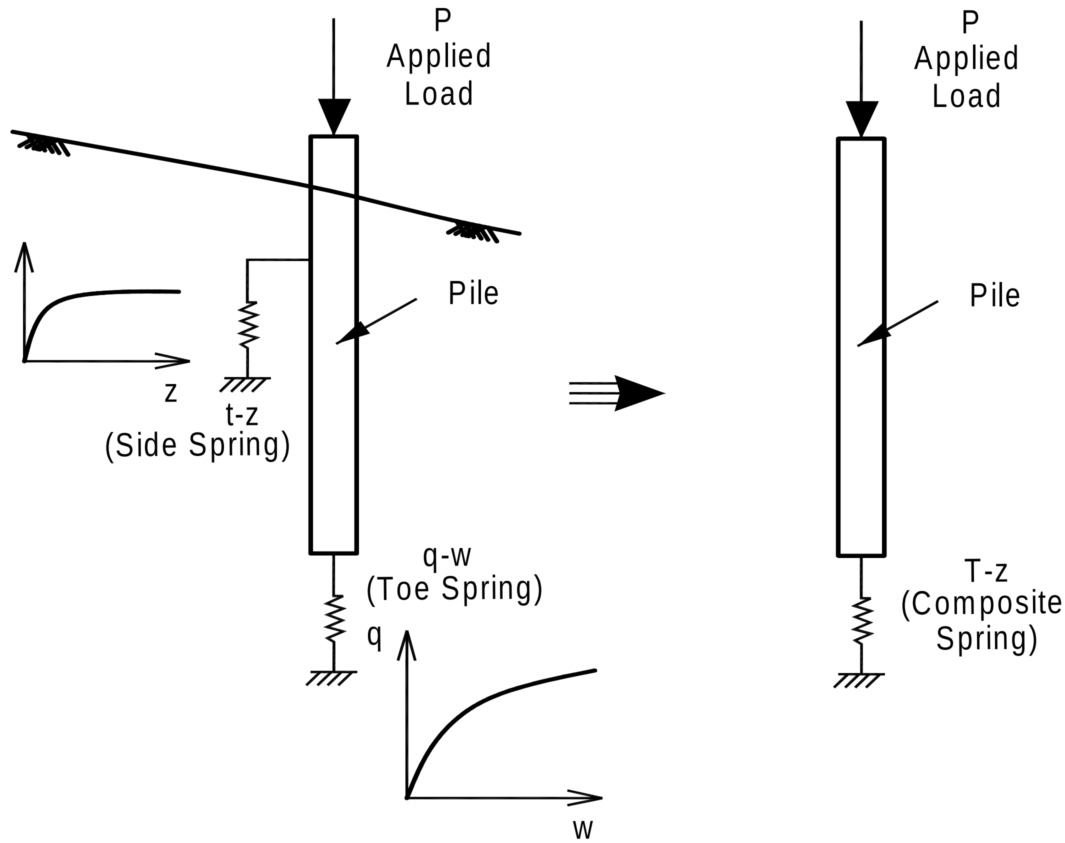


Figure 2-3: Axial Soil Springs

2.8 Soil Behavior under Lateral Pile Loading

2.8.1 Soil Springs for Lateral Pile Loading

For design of piles under loading associated with the inertial response of the superstructure, 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. 7) or other appropriate documents.

2.8.2 Upper and Lower Bound Soil Springs

Due to uncertainties associated with the development of p-y curves, including uncertainties arising from rock properties, rock placement method, and sloping rock dike configuration, upper-bound and lower-bound p-y springs shall be developed for use in the superstructure inertial response analyses. For typical marginal container wharf slope/embankment/dike system at the Port, the stiffness of the upper-bound and lower-bound springs shall be 2 times and 0.3 times the stiffness of the lateral spring developed in Section 2.8.1. Upon approval by the Port, rational analysis and/or testing may be performed to justify the use of different values. For other wharf types, the upper-bound and lower-bound springs should be developed on a site-specific basis.

2.9 Soil-Pile Interaction

Two separate loading conditions for the piles shall be considered: (1) Inertial loading under seismic conditions, 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 loading conditions are sufficiently far apart so that the effects of moment superposition are normally negligible. Furthermore, maximum moments induced by the two loading conditions tend to occur at different times during the earthquake. Therefore, for typical marginal container wharves at the Port, these loading conditions can be uncoupled (separated) from each other during design. For other wharf types, this assumption should be checked on a project-specific basis.

2.9.1 Inertial Loading Under Seismic Conditions

The evaluation of inertial loading response under seismic conditions is discussed in detail in Section 1. The lateral soil springs developed following the guidelines provided in Section 2.8 shall be used in the inertial loading response analyses. The evaluation of 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 the pile strain limits outlined in Section 4.4 (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.

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. 21) for the OLE and CLE, as discussed in Section 2.4.4. For the DE, initial estimate of the free field dike deformations should be made using the curves provided in “Seismic Analysis and Design of Retaining Walls, Buried Structures, Slopes and Embankments” (Ref. 43) or other appropriate documents 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 wharf projects, 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 condition, less than about 12 inches for the CLE and less than about 36 inches for DE conditions.

In cases where dike deformations estimated using the simplified Newmark sliding block method exceed the displacement limits, site-response evaluations may be necessary to revise the free-field dike deformation analyses. Upon 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. 21) should be used as the basis for determining input in the site-response evaluations. 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. 10) and “Seismic Analysis and Design of Pile Supported Wharves” (Ref. 15). The geotechnical engineer should provide the structural engineer with level-ground p-y curves 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 and below the weak soil layer, see Figure 2-4, the pile should be fixed against rotation, and also against translation relative to the soil displacement. Between these two points (+/-10D from the soil layer), 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.

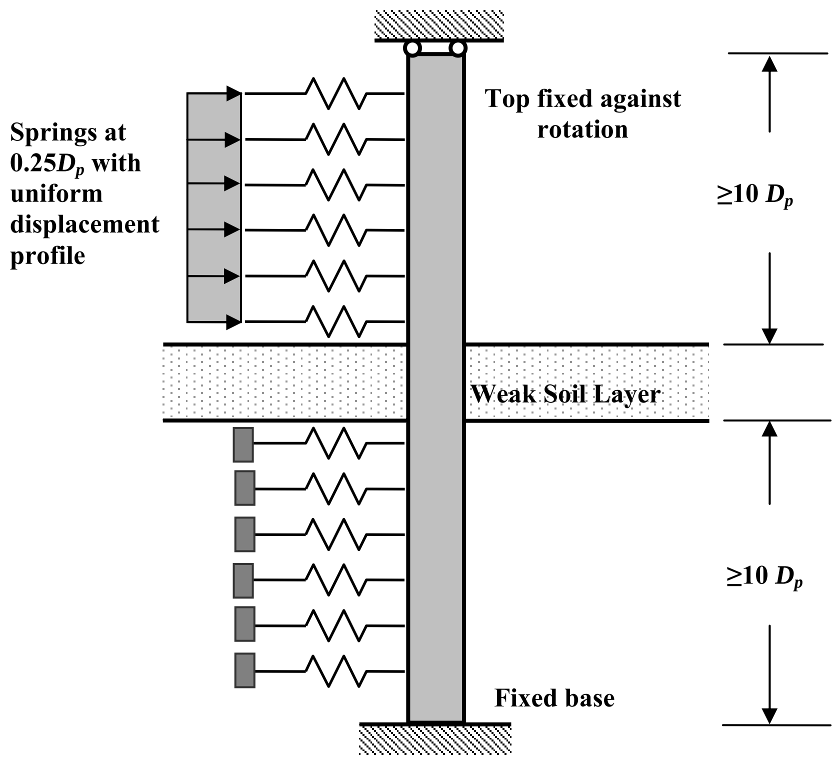


Figure 2-4: Sliding Layer Model

In cases where subsurface conditions indicate the presence of continuous, thin (less than 2 ft.), 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 approval of the Port.

If more detailed numerical analyses are deemed necessary to provide input to the structural engineers, 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 approval of the Port.

2.10 Ground Improvement

In the event that all the requirements set forth in the above sections cannot be met for the project, ground improvement measures may be considered to meet the requirements. Prior 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 discussed.

3 Structural Loading Criteria

3.1 General

All wharves shall be designed for the loading requirements provided herein. Where loading conditions exist that are not specifically identified herein, the designer should rely on accepted industry standards. However, in no case shall other standards supersede the requirements provided herein. For purposes of this document, the terms piers and wharves can be used interchangeably, and mean an engineered structure for the purpose of docking and mooring a vessel for cargo operations.

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 Loading

The wharf shall be designed for a uniform live load of 1000 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%. 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 loading will be obtained from the equipment manufacturer and/or transporting company. Under some loading circumstances, a specified area may be designed into the wharf structure to accommodate those extreme loads.

3.3.2 Truck Loading

Truck loading shall be in accordance with the American Association of State Highway and Transportation Officials (AASHTO) Standard Specification for Highway Bridges (Ref. 1). All piers and wharves shall be designed for HL-93 loading 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 loading is transferred through 2.0 feet or deeper ballast fill, the impact factor need not be considered in design.

3.3.3 Container Cranes

In the absence of actual crane load data from the manufacturer, the following values shall be used:

Crane Rail Loads

All crane rail beams and supporting substructures shall be designed for the container crane loads shown in Table 3-1 below. These rail loads are unfactored, and include both dead and live loads. The Table also indicates the load factors used for the various operating conditions, as well as the allowable stress and factors of safety for pile bearing in the soil. The uniform loading shown is based on eight wheels spaced at 5'-0" O.C. at each corner of crane.

The factored crane loads shall be used in combination with other loadings (Table 3-3) on the wharf deck for the design of the crane rail beam and piling.

The waterside crane rail beam shall be designed to span over interior pile(s) that may be damaged or broken. The load factors associated with a crane transiting over broken piles are shown in Table 3-1.

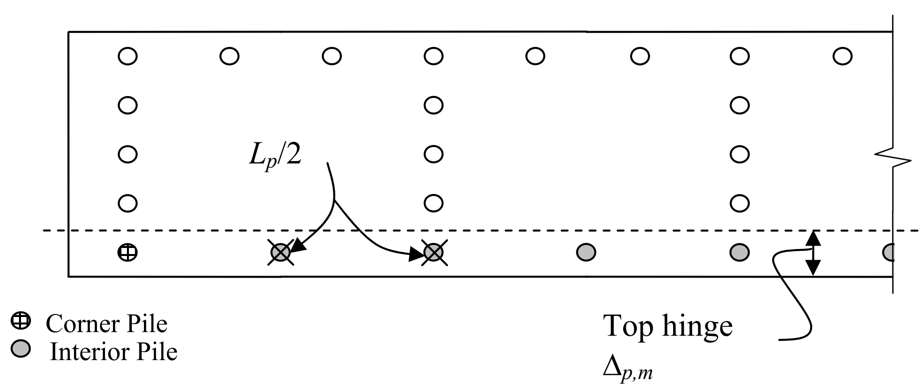


Figure 3-1: Broken Pile

Both waterside and landside crane rail beams shall be designed for a lateral load of 3.0 kips per linear foot applied at the top of rail.

Crane Stowage Pin

Crane stowage pins shall be designed for a horizontal force of 250 kips per rail at each location for strong wind conditions. For wind load see Section 3.10.

Crane Stop Loads

Crane stops shall be designed to resist a horizontal runaway wind blown crane impacting force of 350 kips per rail applied 6.0 feet above the top of the rail, and in a direction parallel to the rail.

Table 3-1: Vertical Container Crane Loading

Load Case	Crane Rail Loads		Load Factor[a]	Flexural Capacity[b]	Pile Soil Capacity Factor of Safety[c]
	W_W Waterside	W_L Landside			
Normal Operating[d]	50 klf	50 klf	1.3	ϕM_n	2.0
One interior pile broken[e]	50 klf	N/A	1.3	$1.1\phi M_n$	1.5
Two adjacent interior piles broken[e,f]	20 klf[g]	N/A	1.2	$1.1\phi M_n$	1.5

[a] These factors represent the combined dead and live load factors applied to the crane loading.

[b] ϕM_n is the reduced nominal moment capacity of the crane rail beam or supporting pile head, calculated based on ACI-318.

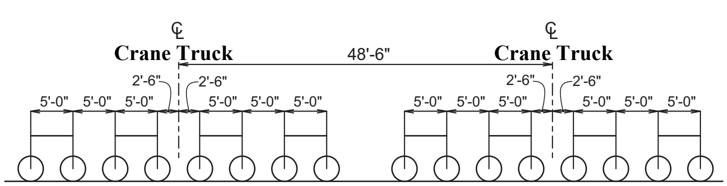
[c] This factor of safety is for service load design combinations.

[d] Crane rail loads are based on 3,000 kips crane dead load with 60 long ton lifting beam, servicing 22 box wide vessels.

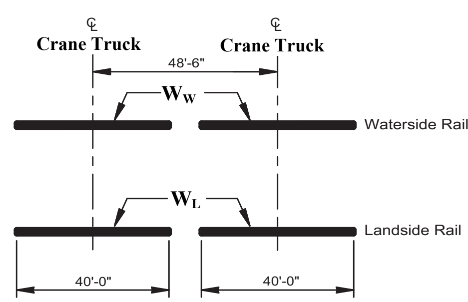
[e] Use for exterior waterside crane girder only. If truck lane exists the broken pile criteria are not applicable.

[f] Only wharf dead load and the waterside crane dead weight rail load specified above need be considered for the case of two adjacent interior piles broken.

[g] This value represents the crane dead load for transiting crane over broken piles only. No crane operations permitted.



Crane Wheel Spacing



Crane Loading Distribution

3.3.4 Container Handling Equipment Loading

Wharf decks slabs shall be checked for container handler wheel loads shown in Figure 3-2. Wheel load distribution shall be determined in accordance with AASHTO. 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 load needs to be higher than the load shown in Figure 3-2, load values and distribution shall be provided to the port for approval.

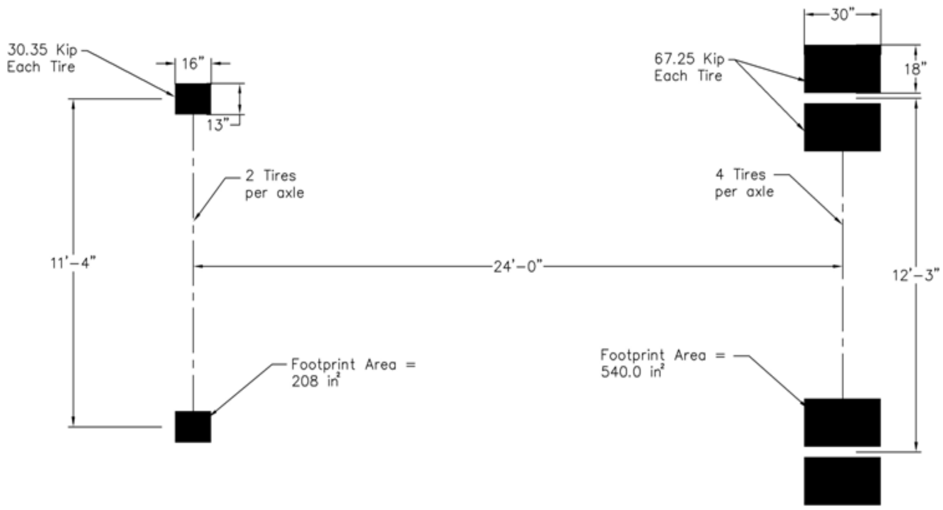


Figure 3-2: Design Wheel Loads

3.3.5 Railroad Track Loading

Piers and wharves accessible by freight car shall be designed for railroad loading. Wheel loads shall correspond to Cooper E-80 designation of "American Railway Engineering and Maintenance-of-Way Association (AREMA) Manual" (Ref. 33).

3.4 Impact (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 considered for the design of the piles and other types of substructures.

Table 3-2: Impact Factors

Loading	Impact
Uniform Load	0%
Truck Load	10%
Forklift & Container handler loading	10%
Railroad loading	20%

3.5 Buoyancy (BU)

Typically, piers and 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 following vessel characteristics, unless otherwise specified. The approach velocity called out below includes the factor for abnormal berthing and assumes a favorable site condition. The berthing energy shall be determined by the deterministic approach as shown in "Guidelines for the Design of Fender Systems, 2002" (Ref. 28).

LOA (Length Overall)	1,300 feet
Maximum Displacement	220,000 metric tons (1 metric ton ≈ 2,205 lbs)
Beam	185 feet
Draft (Max)	48 feet
Allowable Hull Pressure	4 ksf
Approach velocity normal to fender line, $V_\perp$	0.35 foot/second

Smaller container vessels may berth with increased approach velocity normal to the fender line, but the kinetic energy of the vessel should not exceed the energy of the vessel with the maximum displacement, as stated above. Fender shear forces may be computed 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˚.

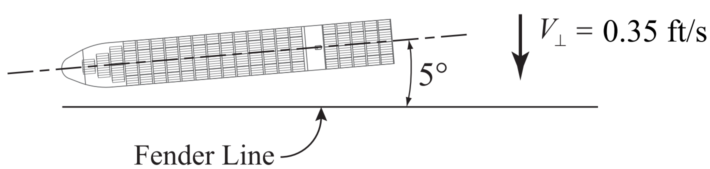


Figure 3-3: Berthing Load

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

where:

V_F	=	Fender shear force
R_F	=	Force perpendicular to the fender panel due to berthing

3.7 Mooring Loads (M)

For the design of the wharf or pier 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 or pier 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.

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 seconds duration with 25 years return period), for more details refer to 2007 CBC Section 3103F.5 (Ref. 18).

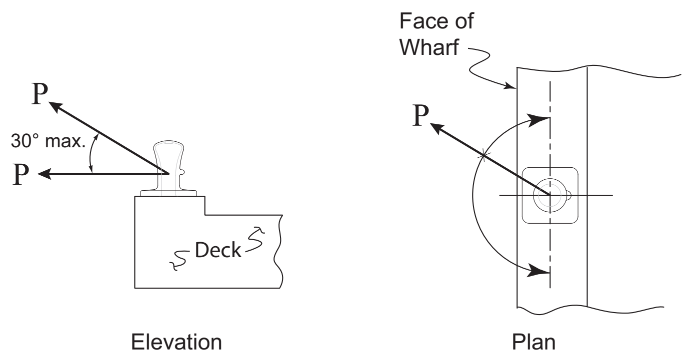


Figure 3-4: Mooring Lines Forces

3.8 Earth Pressure (E)

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

3.9 Earthquake (EQ)

All wharf structures 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 1.

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. This shall be accomplished using a "*K*" factor as a function of PGA (Peak Ground Acceleration).

$$D(1 \pm K) \tag{3.2}$$

$$K = 0.5(PGA) \tag{3.3}$$

The lower bound and upper bound of the dead load shall be applied to the deck, as in Figure 3-5.

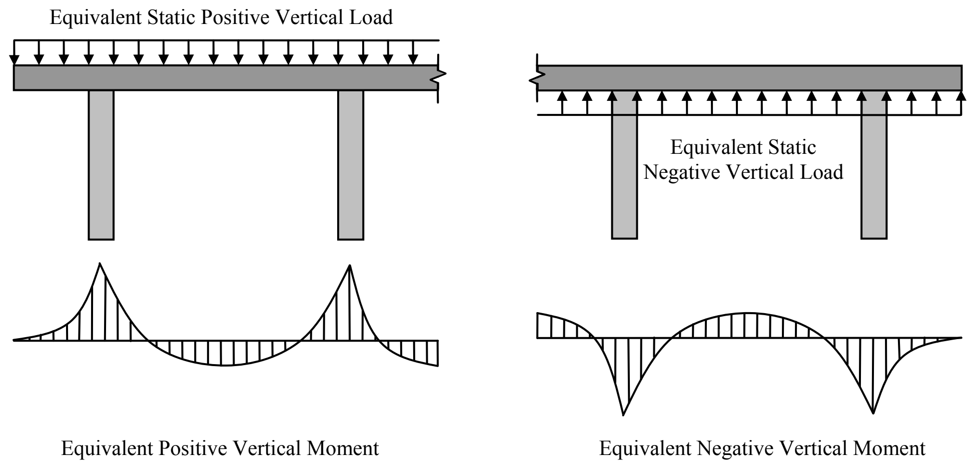


Figure 3-5: Equivalent Static Loads and Vertical Moments

3.10 Wind Load on Structure (W)

The wind load calculations should be based on 2007 CBC (Ref. 17) and ASCE7-05 (Ref. 11) with basic wind speed of 85 mph (3-second gust with 50 years return period).

3.11 Creep (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 (S)

Open pier and wharf decks, which are usually constructed from concrete components, are subject to forces resulting from shrinkage of concrete from the curing process. Shrinkage loads are similar to temperature loads in the sense that both are internal loads. For long continuous open piers and wharves and their approaches, shrinkage load is significant and should be considered. However, on pile-supported pier and 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 (T)

Thermal stresses in structural elements shall be based on a temperature increase or decrease of 25° F.

3.14 Application of Loadings

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 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 for empty containers stacked six high should be checked.

Simultaneous Loads

Uniform and concentrated live loads should be applied in a logical common sense 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.

Loading for Maximum Stress

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

Critical Loads

Concentrated loads are generally critical for punching shear and the design of short spans such as deck slabs, power trench covers and utility vault covers. Uniform loading, mobile crane floats, rail-mounted crane loading, and railroad loading are generally critical for the design of beams, pile caps, and supporting piles.

3.15 Load Combinations

3.15.1 General

Piers and wharves shall be proportioned to safely resist the load combinations represented in Table 3-3. Each component of the structure and the foundation elements shall be analyzed for all the applicable combinations.

Load Symbols

D	=	Dead Load
L	=	Live Load
I	=	Impact Load
BU	=	Buoyancy Load
BE	=	Berthing Load
E	=	Earth Pressure Load
EQ	=	Earthquake Load
W	=	Wind load
R	=	Creep/rib shortening Load
S	=	Shrinkage Load
T	=	Temperature Load
M	=	Mooring Load

3.15.2 Service Load Design

Load combinations used for service load design are presented in Table 3-3. The service load approach shall be used for designing and checking vertical foundation capacity and long-term vertical wharf loading, such as dead load. Timber structures for piers and wharves shall be designed using the service load combinations and allowable stresses. Mooring hardware and fittings (bolts and anchor plates) shall be designed using service load procedures.

3.15.3 Load Factor Design

Load combinations and load factors used for load factor design are presented in Table 3-3. Concrete and steel structures shall be designed using the load factor design method. However, they shall also be checked for serviceability (i.e., creep, fatigue, and crack control as described in ACI-318 (Ref. 2)), and temporary construction loads.

Table 3-3: Load Factors for LFD and LD

LOAD FACTOR DESIGN (LFD)[a]									
Case	LOAD COMBINATION FACTORS								
	D	L+I[b]	E	W	BE	R+S+T	EQ	BU	M
I	1.2	1.6	1.6	—	—	—	—	1.3	—
II	1.2	1.0	1.6	1.6	—	1.2	—	1.3	—
III[c]	0.9	—	1.6	1.6	—	1.3	—	1.3	—
IV	1.2	0.1[d]	1.6	1.0	1.6	—	—	1.3	—
V	1.2	1.0	1.6	1.3	—	—	—	1.3	1.3
VI	$1 \pm K$[e]	0.1[d]	1.0	—	—	—	1.0	—	—

SERVICE LOAD DESIGN (SLD)										
Case	LOAD COMBINATION FACTORS									Allowable Stress
	D	L+ I	E	W	BE	R+S+T	EQ	BU	M	
I	1.0	1.0	1.0	—	—	—	—	1.0	—	100%
II	1.0	1.0	1.0	1.0	—	1.0	—	1.0	—	133%
III	1.0	—	1.0	1.0	—	1.0	—	1.0	—	125%
IV	1.0	0.1[d]	1.0	0.3	1.0	—	—	1.0	—	100%
V	1.0	1.0	1.0	1.0	—	—	—	1.0	1.0	125%

[a] The Load Factor Design require the strength reduction factors, ϕ as specified in ACI-318 2005.

[b] For the load factor of crane load case see Table 3-1

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

[d] For uniform live load only.

[e] K = 0.50 (PGA), to account for the affects of the vertical component of ground acceleration. The K-factor shall be applied to the vertical dead load (D) only, not to the inertia mass of the wharf.

4 Seismic Design Criteria

4.1 Introduction

The following criteria identify the minimum requirements for seismic design of wharves and piers. 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

All wharf designs 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 (piles) 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 pile structural system concept that is used for the design of buildings. Capacity design is required to ensure that the dependable strengths of the protected locations and actions exceed the maximum feasible demand based on high estimates of the flexural strength of plastic hinges.

Pile Connections

The pile shall be connected to the deck with mild steel dowels. Moment-resisting connections 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 5.5.4 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 the two rails to prevent spreading or loss of gage due to earth movements.

Bulkheads

Bulkheads shall be designed for dynamic earth pressures induced during seismic events. Cut-off wall is mainly used to prevent loss of soil from backland and shall not be designed to provide seismic lateral resistant.

Slope Stability

A slope stability analysis, including seismic induced movements, shall be performed; see 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 expansion joints.

4.3 Performance Criteria

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

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 dead load including the cranes. Life safety shall be maintained.

4.4 Strain Limits

The strain limits for the OLE, CLE and DE performance levels shall be defined by the following material strains for concrete piles and steel pipe piles. Steel sheet piles and tie-back systems shall be designed to remain elastic under all three earthquake levels. Strain values computed 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	$\varepsilon_c \leq 0.005$	$\varepsilon_c = 0.005 + 1.1\rho_s \leq 0.025$	No limit
	In-ground hinge concrete strain	$\varepsilon_c \leq 0.005$	$\varepsilon_c = 0.005 + 1.1\rho_s \leq 0.008$	$\varepsilon_c = 0.005 + 1.1\rho_s \leq 0.025$
	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 tendon strain	$\varepsilon_p \leq 0.015$	$\varepsilon_p \leq 0.025$	$\varepsilon_p \leq 0.035$
	Deep In-ground hinge (>10D_p) prestressing tendon 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 tendon strain	$\varepsilon_p \leq 0.015$	$\varepsilon_p \leq 0.025$	$\varepsilon_p \leq 0.025$
	Deep In-ground hinge (>10D_p) prestressing tendon 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	CLE	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	$\varepsilon_s \leq 0.010$	$\varepsilon_s \leq 0.035$	$\varepsilon_s \leq 0.05$
	Deep In-ground hinge (>10D_p) hollow pipe steel strain	$\varepsilon_p \leq 0.010$	$\varepsilon_p \leq 0.035$	$\varepsilon_p \leq 0.050$

[a] For solid round or octagonal piles.
[b] If a hollow concrete pile is in-filled with concrete, the strain limits are identical to solid concrete piles.
[c] Steel pipe pile deck connection shall be accomplished by concrete plug with dowels.

Definitions:

ε_c = Concrete compression strain.
ε_s = Total steel tensile strain.
ε_{smd} = Strain at maximum stress of dowel reinforcement; see Section 4.6.2.
ε_p = Total prestressing steel tensile strain.
ε_{pi} = Initial prestressing steel tensile strain after losses.
D_p = Pile diameter.

4.5 Analysis Methods

Analysis of wharf structures shall be performed for each performance level to determine displacement demand and capacity. The capacity will 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
- Initial Stiffness Method
- Substitute Structure Method
- Modal Response Spectra
- Time-History Analysis.

The flow chart in Figure 4-1 shows the typical steps a designer should follow to complete the seismic design and analysis for a wharf structure. After the design for non-seismic loads has been completed, the performance design shall be completed 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 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 segments. Displacement demand for regular wharves shall be estimated by the Initial Stiffness method, the Substitute Structure method, or Modal Response Spectra. For wharves with irregular geometry, special cases, or when demand/capacity ratios from Response Spectrum Analysis are too high, Time-History methods may be employed for the global model to verify the analysis results. Time-History analyses, however, shall not be conducted without prior 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 Initial 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 geotechnical analyses indicate a potential for sliding failure of the dike, kinematic analysis of the deep in-ground pile hinge shall be performed in accordance with Section 4.12.

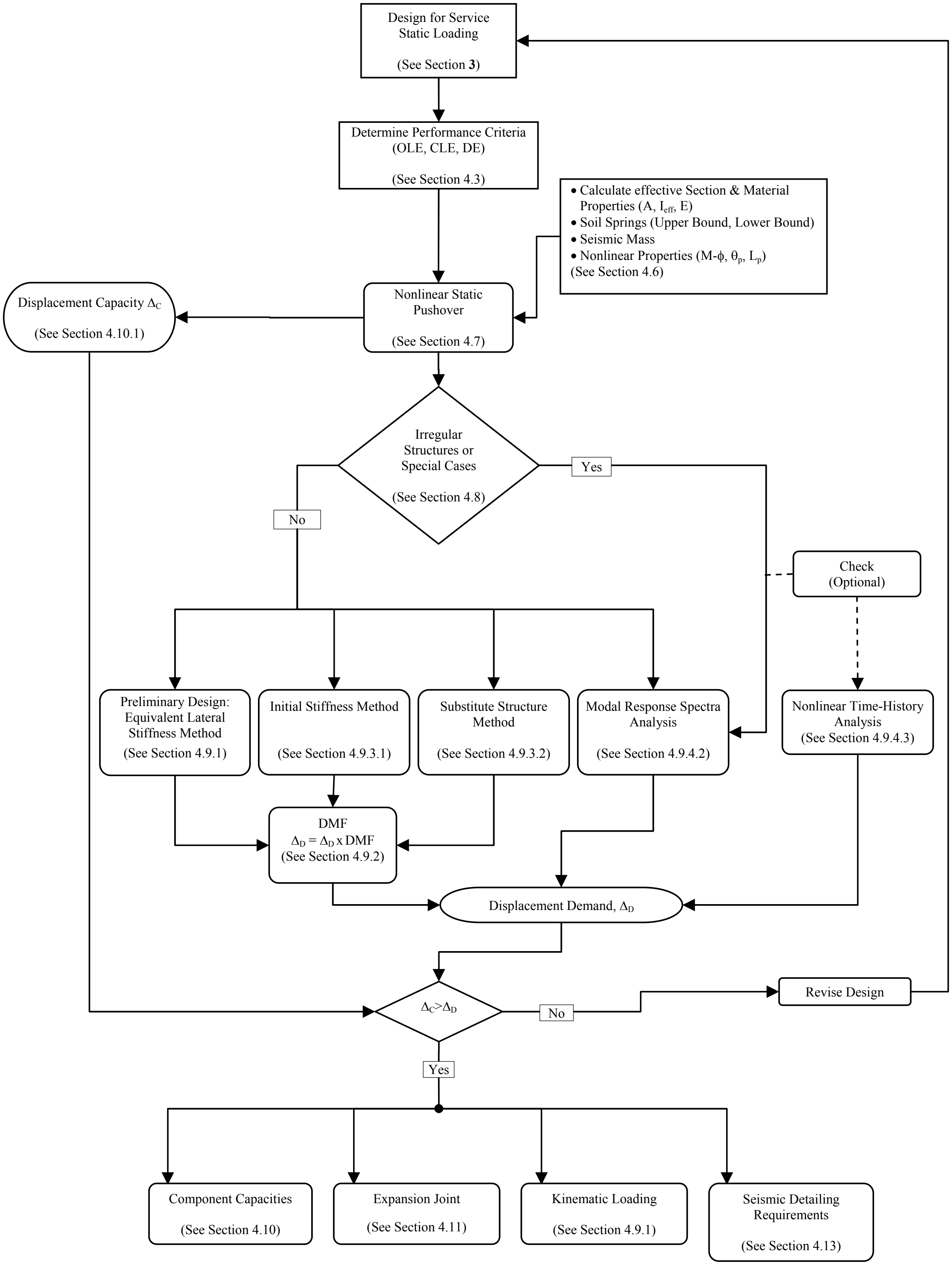


Figure 4-1: Flow Diagram for Seismic Analysis

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.

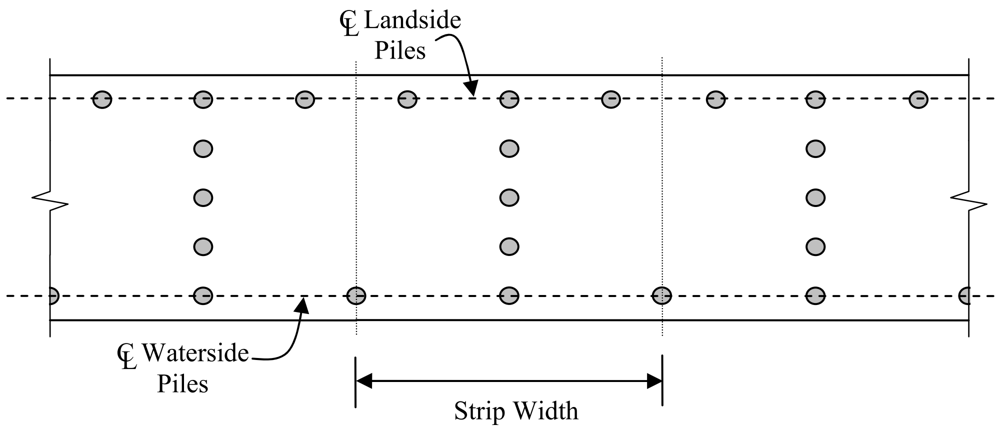


Figure 4-2: Pile Spacing for Typical Modeling Strip (Plan View)

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 0.

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:

$$l_{sp} = 0.12 f_{ye} d_{bl} \tag{4.1}$$

where,

d_{bl} = The diameter of the longitudinal reinforcement

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

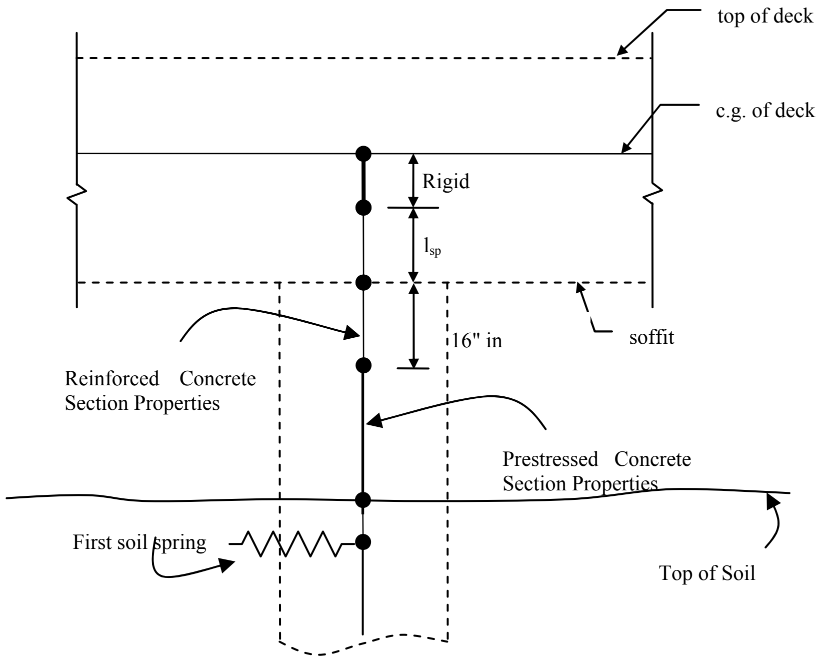


Figure 4-3: Pile Strain Penetration Length (Cross-Section)

For prestressed piles, the reinforced concrete effective section property shall be used for the first 16"of the pile below the soffit to account for development of the prestressing strands. Below the first 16" of the pile, the prestressed concrete effective section properties shall be used, see Section 4.10. Maximum moment shall be considered to develop at the soffit. Maximum in-ground moments 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 and dike. To insure adequate precision in modeling the pile moment profile, it is thus 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 inch 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 a bout 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 concrete compressive strength, f'_{ce}, recognizes the typically conservative nature of concrete batch design, and the expected strength gain with age. The expected yield stress 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 the evaluation demand on of capacity-protected members, an additional overstrength factor shall be used when determining the capacity of pile plastic hinges as described in Section 4.10. Except for shear, the expected seismic material strengths shall be:

$$f'_{ce} = 1.3 f'_c \tag{4.2}$$

$$f_{ye} = 1.1 f_y \tag{4.3}$$

$$f_{yhe} = 1.0 f_{yh} \tag{4.4}$$

$$f_{pye} = 1.0 f_{py} \tag{4.5}$$

$$f_{pue} = 1.05 f_{pu} \tag{4.6}$$

where,

f'_c = 28-day unconfined compressive strength

f_y = Yield strength of longitudinal reinforcing steel or structural steel

f_{yh} = Yield strength of spiral reinforcement

f_{py} = Yield strength of prestressing steel

f_{pu} = Ultimate strength of prestressing steel

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

The following stress-strain curves may be used to compute 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 below in Figure 4-4. This model is based on Mander's model for confined and unconfined concrete (Ref. 32).

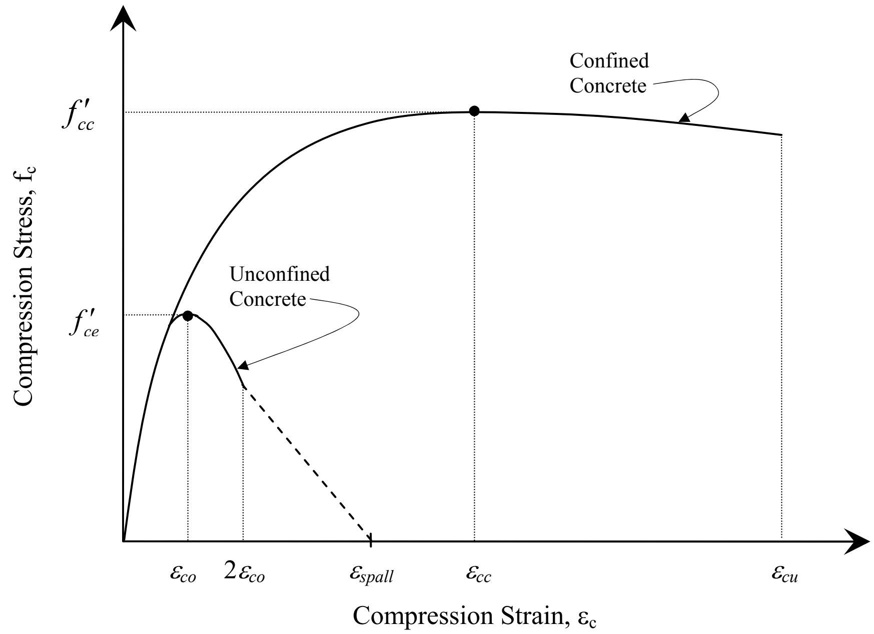


Figure 4-4: Stress-Strain Relationship for Confined and Unconfined Concrete

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:

$$\varepsilon_{cu} = 0.005 + 1.1\rho_s \leq 0.025 \tag{4.7}$$

$$\varepsilon_{cc} = \varepsilon_{co}\left[1 + 5\left(\frac{f'_{cc}}{f'_{ce}} - 1\right)\right] \tag{4.8}$$

$$f'_{cc} = f'_{ce}\left(-1.254 + 2.254\sqrt{1 + \frac{7.94 f'_l}{f'_{ce}}} - 2\frac{f'_l}{f'_{ce}}\right) \tag{4.9}$$

where for circular core sections,

$$f'_l = \frac{1}{2}K_e \rho_s f_{yh} \tag{4.10}$$

$$\rho_s = \frac{4A_{sp}}{D's} \tag{4.11}$$

ρ_s = Effective volumetric ratio of confining steel

A_{sp} = Area of confining reinforcement

D' = Diameter of confining reinforcement core, measured to the centerline of the confinement

s = Center to center spacing of confining reinforcement along pile axis

f_{yh} = Yield stress of confining steel

ε_{cu} = Ultimate concrete compression strain

ε_{cc} = Confined concrete strain at peak stress

f_l' = Effective lateral confining stress

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

f'_{ce} = Expected concrete strength

f'_{cc} = Confined concrete strength

Figure 4-5 plots the ratio of confined concrete strength to expected unconfined concrete 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.

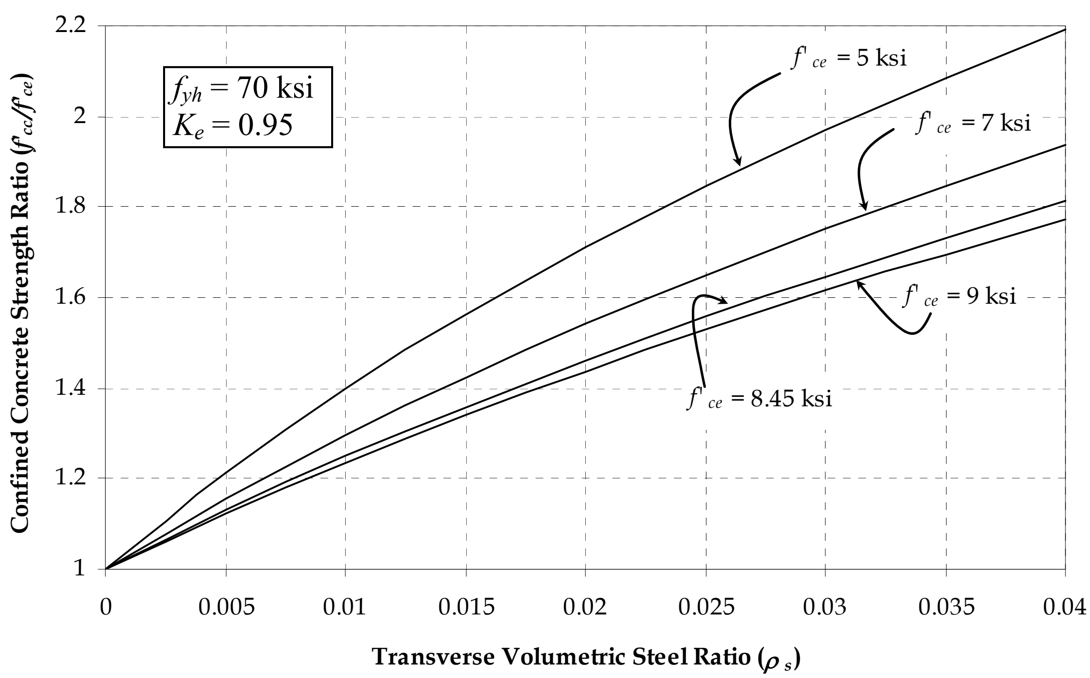


Figure 4-5: Confined Concrete Strength Ratio versus Transverse Steel Ratio

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. 32).

Steel

The stress-strain curve for reinforcing steel is shown in Figure 4-6. The stress-strain curve for structural steel is similar to the curve for reinforcing steel.

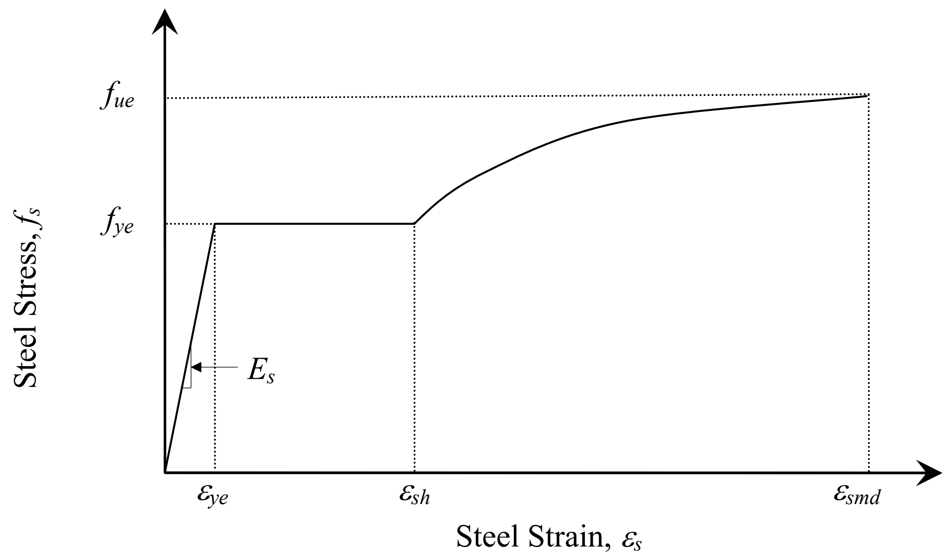


Figure 4-6: Stress-Strain Relationship for Reinforcing Steel

For ASTM A706 Grade 60 steel (Ref. 19):

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

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

f_{ue} = Expected maximum tensile strength of steel

E_s = 29,000 ksi

ε_{ye} = Expected yield strain of steel

Prestressing Tendons

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

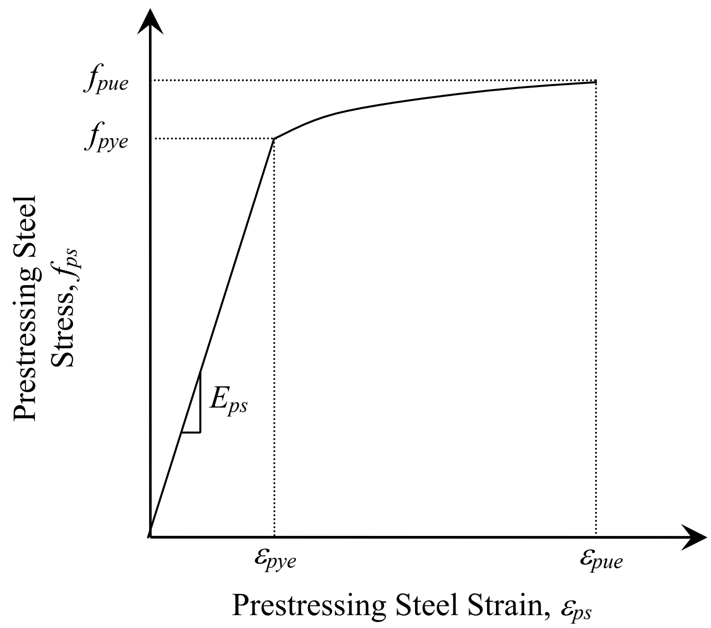


Figure 4-7: Stress-Strain Relationship for Prestressing Tendons

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 0.035

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. Concrete members display nonlinear response before reaching their idealized Yield Limit State.

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

The effective moment of inertia I_{eff} shall be used for the structural model. I_{eff} can be estimated by the secant slope of the moment-curvature curve between the origin and the point of first yield:

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

where:

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, see Figure 4-6 the effective moment of inertia ranges between 0.3-0.7I_{gross}. For prestressed concrete piles, the effective moment of inertia ranges between 0.6-0.75I_{gross}. The prestressing strands at the top of the prestressed pile near the pile/deck connection will not be developed, so I_{eff} of the dowel connection should be used. Use 0.5I_{gross} for the deck section. Sections that are expected to remain uncracked for seismic response should be represented by gross section properties.

The polar moment of inertia, J, of individual piles is normally an insignificant parameter on the global response of wharves and piers. Where the value of J is expected to affect the response, the following effective polar moment of inertia, J_{eff}, shall be used.

$$J_{eff} = 0.2\ J_{gross} \tag{4.12}$$

4.6.4 Seismic Mass

The mass considered for the dynamic analysis shall include the structural self-weight of the entire wharf, permanently attached equipment, and 10% of the design uniform live load. Also, 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” diameter piles or less, hydrodynamic mass may be ignored.

The wharf mass shall include that part of the mass of supported cranes that can be considered rigidly connected to the wharf at deck level, $m_{crane,deck}$ if:

$$m_{crane,deck} > 0.05 m_{wharf}$$

where:

$m_{crane,deck}$ = part of the mass of all cranes positioned close to wharf deck level on the wharf segment under consideration

m_{wharf} = mass of the wharf segment under consideration

The mass of the higher part of the cranes supported by flexible members above the deck need not be considered when determining $m_{crane,deck}$.

The translational elastic period of the crane mode with the maximum participating mass, T_{crane} should be at least twice the effective elastic period of the wharf based on cracked section properties and effective elastic stiffness of the wharf system, T_w ($T_{crane} \geq 2T_w$). If this requirement is not satisfied, crane-wharf interaction analysis shall be performed with prior approval by the Port.

When analysis is based on a reduced strip width, as illustrated in Figure 4-2, the crane mass used for analysis, if required by the above definition, should be reduced from the total effective mass of cranes on the wharf segment between movement joints, in proportion to the strip length divided by the segment length.

4.6.5 Lateral Soil Springs

Upper and lower bound soil springs shall be used in the model to determine the maximum displacement demands and the corresponding displacement capacities. This recognizes the inherent uncertainties associated with soil-structure interaction. The higher of the two demand-to-capacity ratios so determined 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, M_p, of the piles shall be calculated by Moment-Curvature (M-ϕ) analysis using expected material properties. The analysis must be capable of separately modeling the core and cover concrete, and must be capable of representing 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 nonlinearity must also be realistically modeled 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 prestress force.

The M-ϕ curve may be idealized by an elastic perfectly plastic response to estimate the plastic moment capacity as follows:

Moment-Curvature Curve Idealization: Method A

The idealized plastic moment capacity for typical concrete piles in the Port of Long Beach corresponds to the section moment associated with an extreme concrete fiber strain of 0.004, as shown in Figure 4-8. Typically, the M-ϕ curve peaks around 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 idealized 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, and extends to meet M_p.

Moment-Curvature Curve Idealization: Method B

For other M-ϕ curves, 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 may be more appropriate. For this approach, the elastic portion of the idealized M-ϕ curve should pass through the point marking the first reinforcement 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).

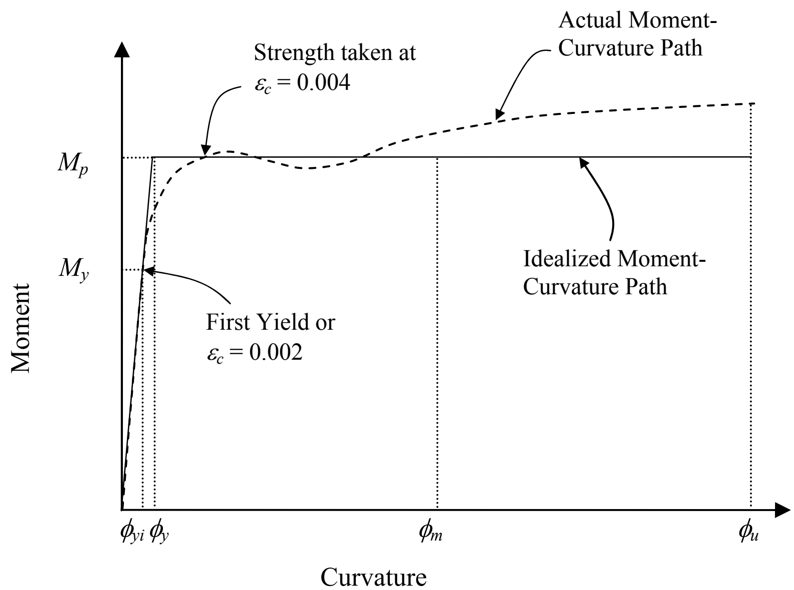


Figure 4-8: Moment–Curvature Curve and Idealization for Method A

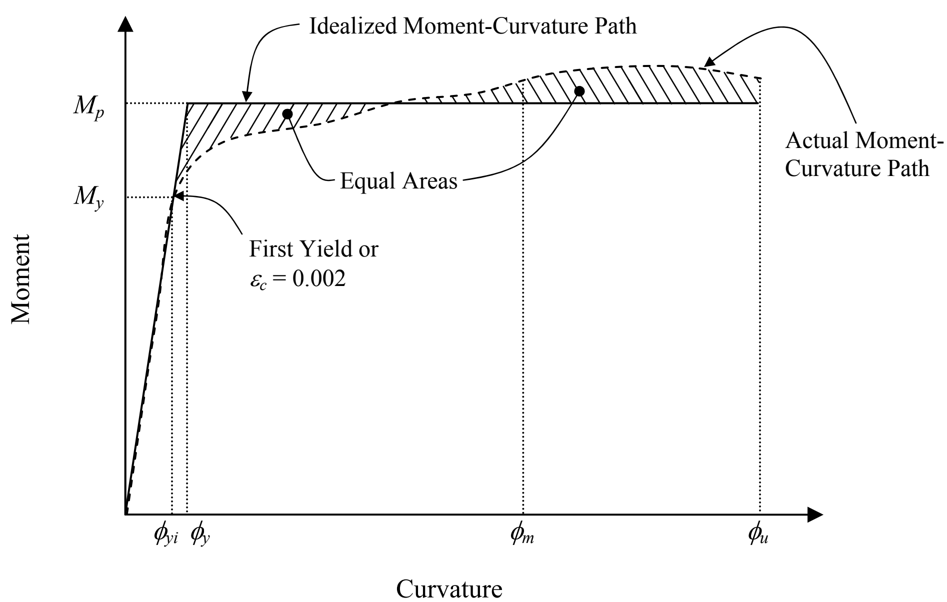


Figure 4-9: Moment-Curvature Curve and Idealization for Method B

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 = Curvature at the OLE, CLE or DE strain limit

ϕ_u = Ultimate curvature of the section

4.6.6.2 Plastic Hinge Length

The plastic hinge length needs to be determined to convert the moment-curvature relationship into a force-displacement or moment-plastic rotation relationship for the nonlinear pushover analysis.

Table 4-2: Plastic Hinge Length Equations

Section	Top	In-ground
Concrete Pile	4.13	4.15
Hollow Concrete Pile	4.13	4.15
Steel Pipe Pile (hollow with concrete plug connection)	4.14	4.15
Steel Pipe Pile (infilled with concrete)	4.14	4.15

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

$$L_p = 0.08L_c + 0.12 f_{ye} d_{bl} \geq 0.2 f_{ye} d_{bl} \tag{4.13}$$

where,

L_c = The distance from the critical section of the plastic hinge to the point of contra-flexure in the pile

d_{bl} = The diameter of the longitudinal reinforcement

f_{ye} = Expected yield strength of longitudinal 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:

$$L_p = 0.3 f_{ye} d_{bl} + d_{gap} \tag{4.14}$$

where,

d_{gap} = The distance from the top of the steel shell to the soffit

The plastic hinge length L_p for all in-ground hinges may be taken as:

$$L_p = 2D_p \tag{4.15}$$

where,

D_p = Pile diameter

4.6.6.3 Plastic Rotation

The plastic rotation can be determined from the following equations:

$$\theta_{p,m} = L_p\phi_{p,m} = L_p(\phi_m - \phi_y) \tag{4.16}$$

where,

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

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

The idealized Moment-Rotation (*M*-θ) diagram is shown below:

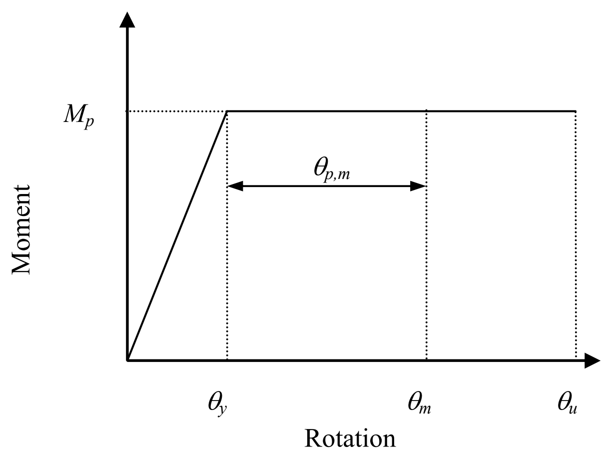


Figure 4-10: Idealized Moment-Rotation Curve

θ_u = Ultimate rotation

θ_y = Idealized yield rotation= $\phi_y L_{sp}$

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

4.7 Nonlinear Static Pushover Analysis

Two-dimensional nonlinear static (pushover) analyses shall be performed for all wharf structures. The pushover curve shall have sufficient points to encompass the system's initial elastic response and predicted earthquake inelastic 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 definitions are included in the model. This method incorporates soil deformation into the total displacement capacity of the pile. Pushover models shall use effective section properties and shall incorporate soil stiffness with nonlinear upper and lower bound *p-y* springs, as shown in Figure 4-11. The results from the pushover analysis will provide the displacement capacities for OLE, CLE or DE earthquake levels, as well as the tools for the initial stiffness and substitute structure methods, see Figure 4-12. The pushover curve shall not experience a significant drop (greater than 20%) in base shear at the target strain limits.

For the purpose of determining displacement demand for a specified limit state, the pushover curve may be approximated by a bilinear response, as illustrated in Figure 4-12

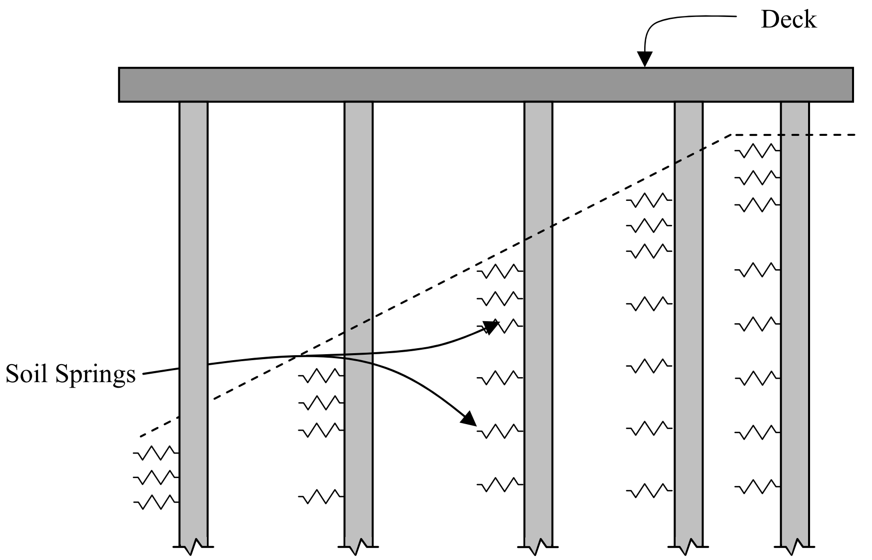


Figure 4-11: Pushover Analysis Model with P-y Springs

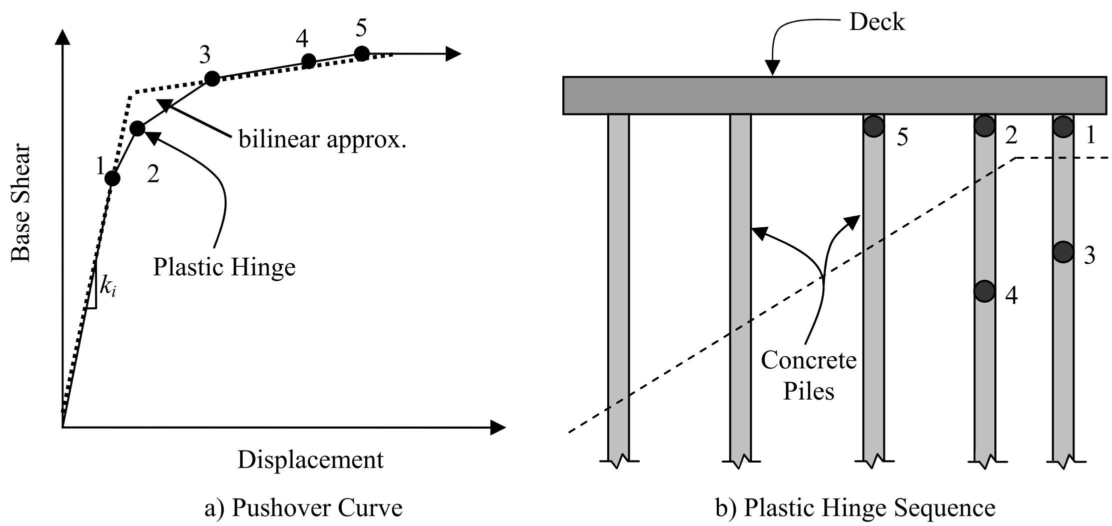
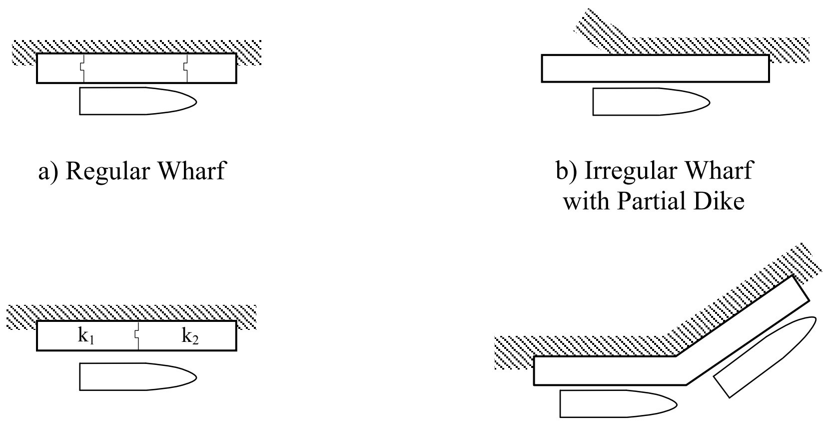


Figure 4-12: Example of Pushover Curve and Plastic Hinge Sequence

4.8 Irregular Structures and Special Case

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) depicts 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 structures with different pile stiffness or different soil stiffness. Figure 4-13 d) shows an irregular wharf with an angle point.



a) Regular Wharf

b) Irregular Wharf with Partial Dike

c) Irregular Wharf with Different Adjacent Stiffnesses: $k_2>1.2k_1$

d) Irregular Wharf with Angle Point

Figure 4-13: Horizontal 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 Case

A special analysis case shall be considered to exist if the crane mass impacts the wharf behavior according to Section 4.6.4.

4.9 Demand Analysis

4.9.1 Equivalent Lateral Stiffness Method

In this approach, the equivalent depth to the 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 at the top of pile. For different assumed displacements and different pile head conditions (free-head or fixed-head), L_s will vary. The equivalent pile length has all soil (and associated lateral stiffness) removed above its supported base, as shown in Figure 4-14.

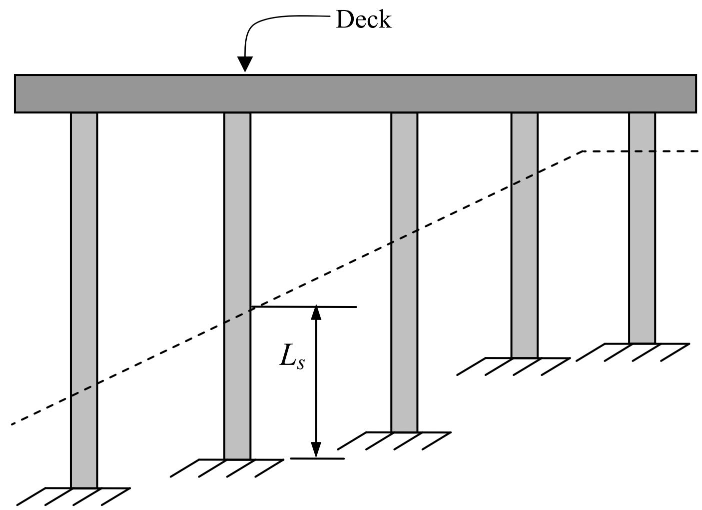


Figure 4-14: Equivalent Lateral Stiffness Method

This method produces adequate response results for the global elastic response. Pile-top moments will be underestimated and in-ground pile moments will be over-estimated, and hence the Equivalent Lateral Stiffness method may be used for preliminary design, but shall not be used for final design.

4.9.2 Dynamic Magnification Factor (DMF)

Most of the seismic lateral resistance of marginal wharves is provided by landward piles. The seaward piles are mainly for gravity loads and might provide about 10% of the overall lateral resistance. This 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. An upper bound of displacement demand of the critical piles at the end of a segment can be established by multiplying the displacement response calculated under pure transverse excitation by the Dynamic Magnification Factor, which accounts for torsional response and simultaneous longitudinal and transverse excitation. An analytical study utilizing non-linear time-history analysis was performed to calculate the DMF (Ref 15) 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 segment and three linked segments. The segment lengths varied between 400’, 600’, and 800’. The study results show that DMF for CLE case is always lower than DMF for OLE case. Therefore, DMF for DE case may conservatively be assumed to be equal to DMF for CLE case.

Based on the study results, the Dynamic Magnification Factor may be calculated as follows:

Single Wharf Unit:

OLE:

LB or UB springs: $$DMF = 1.80 - 0.05\, L_L / B \geq 1.10 \qquad (4.17)$$

CLE/DE:

UB springs: $$DMF = 1.65 - 0.05\, L_L / B \geq 1.10 \qquad (4.18)$$

LB springs: $$DMF = 1.50 - 0.05\, L_L / B \geq 1.10 \qquad (4.19)$$

Two or More Linked Wharf Units:

Exterior

OLE:

LB or UB springs: DMF = 1.55 - 0.04 L_L /B≥1.10 (4.20)

CLE/DE:

UB springs: DMF = 1.35 - 0.02 L_L /B≥1.10 (4.21)

LB springs: DMF = 1.16 - 0.02 L_L /B≥1.10 (4.22)

Interior

OLE/CLE/DE:

LB or UB springs: DMF = 1.10 (4.23)

Where:

L_L = length of the shortest exterior wharf unit

B = width of a wharf unit

LB = lower bound soil spring

UB = upper bound soil spring

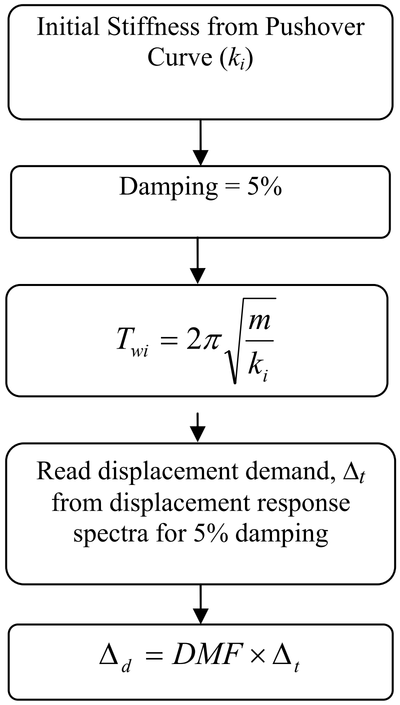
4.9.3 Two Dimensional Response Spectra Analysis

Reasonable estimates of displacement demand may be obtained from elastic analyses based on the Initial Stiffness method by using initial cracked-section elastic stiffness for the piles. However, improved representation of displacement demand will be obtained using the Substitute Structure approach. If the Initial 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 Initial Stiffness Method

The Initial Stiffness method is a pure transverse analysis of a typical wharf strip (see Figure 4-2). This method uses the initial stiffness, k_i, of the structure taken from the pushover curve to calculate the displacement demand (see Figure 4-12). This method assumes a damping ratio of 5%.

The results shall then be modified with the Dynamic Modification Factor (DMF) to include the influence of simultaneous longitudinal response, interaction across movement 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 Initial Stiffness method.



T_{wi} = Initial period

m = Seismic mass of the wharf segment per Section 4.6.4

g = Acceleration of gravity

Δ_t = Displacement demand based on transverse response

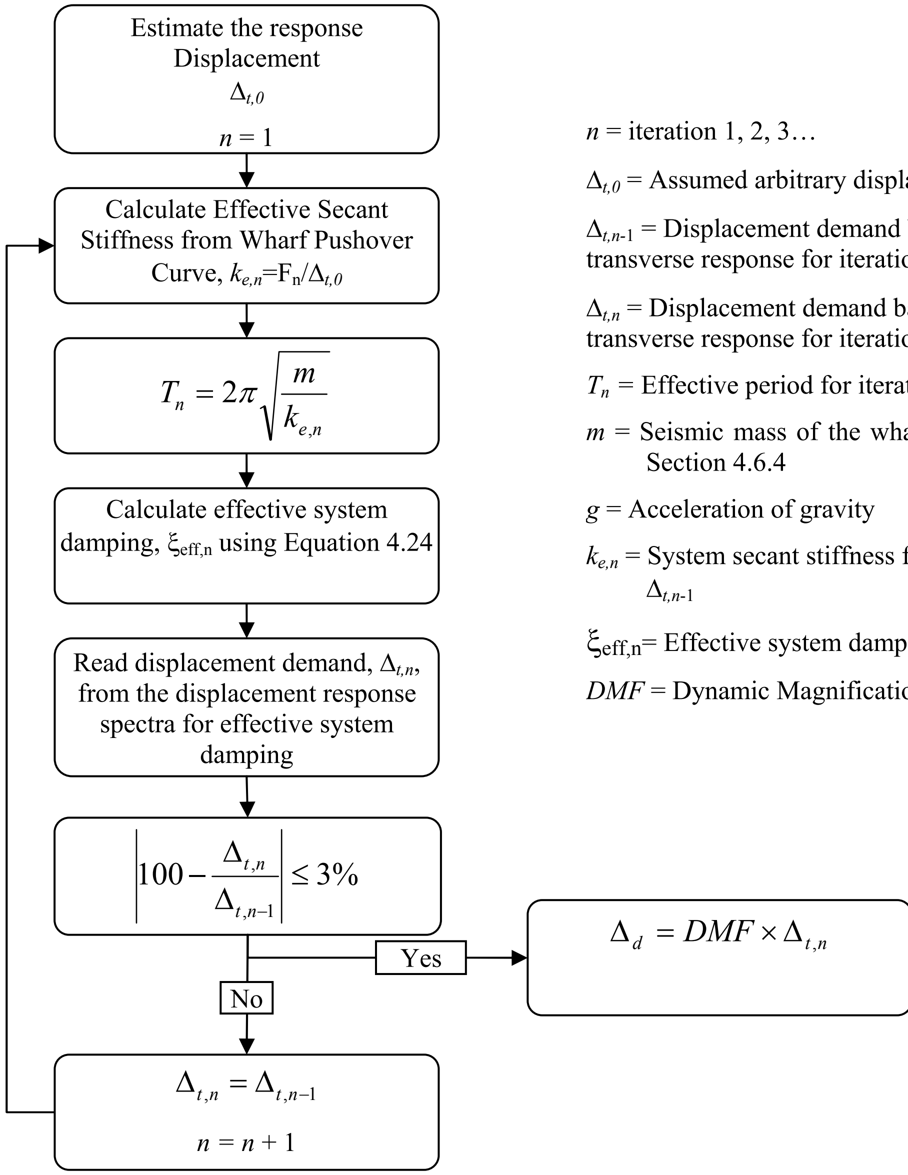
DMF = Dynamic Magnification Factor

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

4.9.3.2 Substitute Structure Method

The Substitute Structure method is an iterative process that uses the effective secant stiffness (k_e) at the response displacement, and an equivalent elastic damping representing the combined effects of elastic and hysteretic damping to determine the pure transverse displacement demand for each iteration, Δt,n (see Figure 4-17).

This approach is a single-mode pure transverse analysis, modified for simultaneous transverse and longitudinal excitation by the DMF. Figure 4-16 graphically shows the steps involved in computing the maximum displacement by the Substitute Structure method.



n = iteration 1, 2, 3…

$\Delta_{t,0}$ = Assumed arbitrary displacement

$\Delta_{t,n-1}$ = Displacement demand based on transverse response for iteration n-1

$\Delta_{t,n}$ = Displacement demand based on transverse response for iteration n

T_n = Effective period for iteration n

m = Seismic mass of the wharf segment per Section 4.6.4

g = Acceleration of gravity

$k_{e,n}$ = System secant stiffness for iteration n at $\Delta_{t,n-1}$

$\xi_{eff,n}$= Effective system damping

DMF = Dynamic Magnification Factor

Figure 4-16: Flow Diagram for Substitute Structure Method

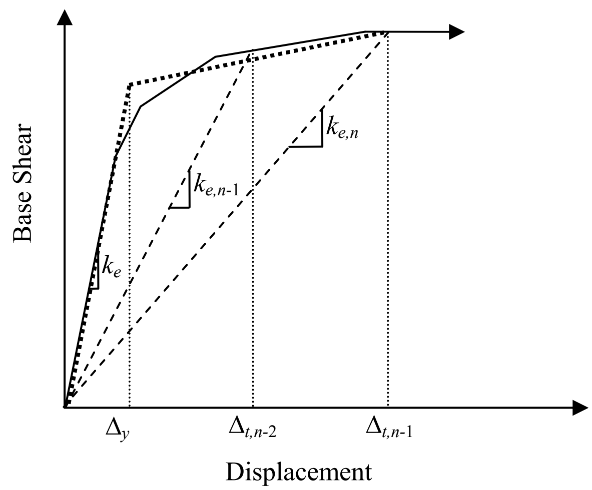


Figure 4-17: Effective Stiffness for Wharf System from Pushover Analysis

The yield displacement is the system yield displacement, found from the intersection of the elastic and post-yield branches of the bilinear approximation. This will always be larger than the first yield of piles. The "equal energy" approach may be used to estimate the bi-linear approximation of the system pushover curve. The system displacement ductility demand is found from the bilinear approximation to the system pushover curve (Figure Figure 4-12 and Figure 4-17) as:

$$\mu_n = \frac{\Delta_{t,n}}{\Delta_y} \tag{4.23}$$

The effective system damping is then found from Equation (4.24):

$$\xi_{eff,n} = 0.10 + 0.565\left(\frac{\mu_n - 1}{\mu_n \pi}\right) \tag{4.24}$$

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

4.9.4 Three Dimensional Analysis

Three dimensional demand analyses include Modal Response Spectra Analysis and Nonlinear Time-History Analysis. A typical wharf segment between movement joints has a large number of piles, which will 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 the piles in the model for a regular wharf segment between movement 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 by the locations of the gravity piles and the seismic piles. 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.

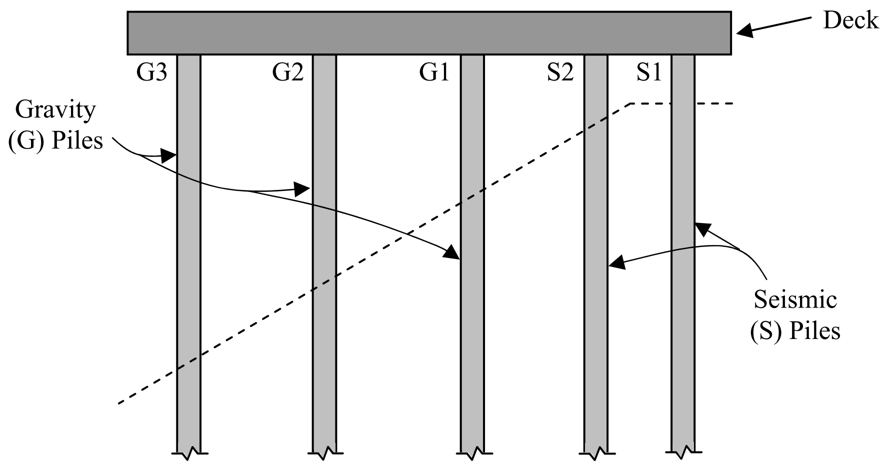


Figure 4-18: Elevation View of Transverse Wharf Segment

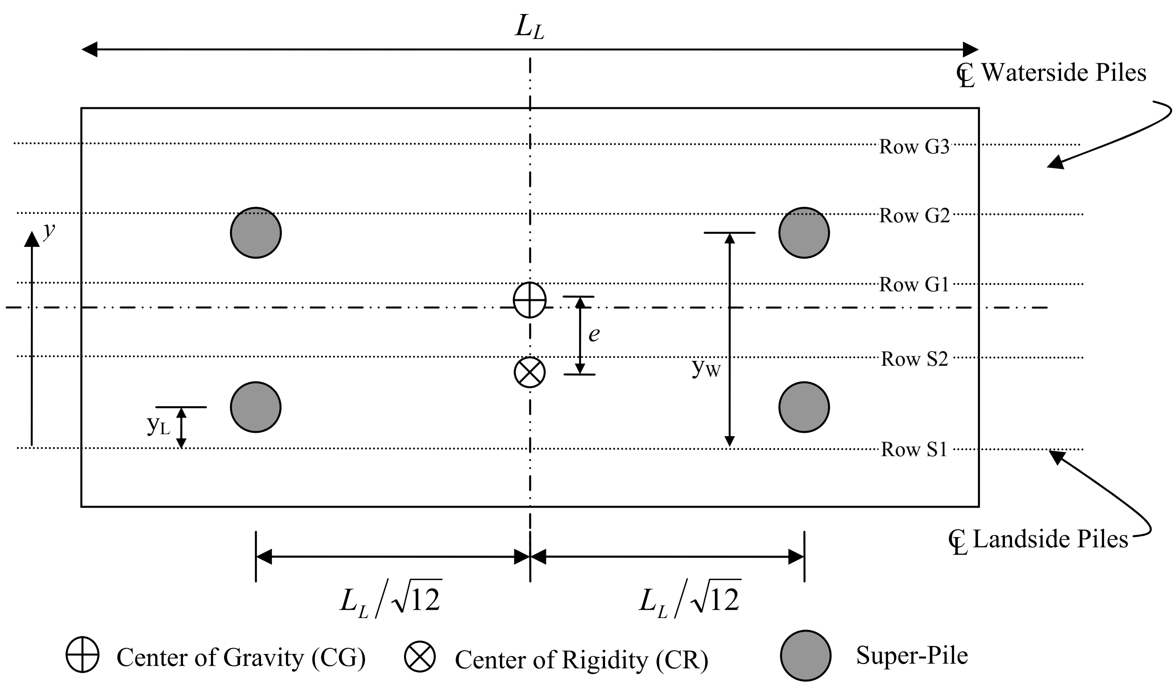


Figure 4-19: Plan View of Super-Pile Locations for a Wharf Segment

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

$$y_L = \frac{\sum_{i=S1}^{S2} n_i F_i y_i}{\sum_{i=S1}^{S2} n_i F_i} \text{ and } y_W = \frac{\sum_{i=G1}^{G3} n_i F_i y_i}{\sum_{i=G1}^{G3} n_i F_i} \tag{4.24}$$

where:

- 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
- y_i = Distance of row i from the landside pile row

The super-pile stiffness is calculated from the pushover curve for the piles represented. The landside super-pile stiffness is equal to the stiffness of the piles on the landside of the dike. The remainder of the 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; that is, for a regular wharf segment they 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 elastic modal analysis and inelastic time-history analysis.

4.9.4.2 Modal Response Spectra Analysis

This method is essentially a linear response spectrum analysis for a stand-alone wharf segment. A three dimensional linear elastic modal response analysis shall be used with effective section properties (Section 4.6.3) applied to components to establish lateral displacement demands.

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.

When wharf segments are linked by shear keys at movement joints, elastic modal analysis will not provide adequate representation of shear key forces or displacement of the movement joint.

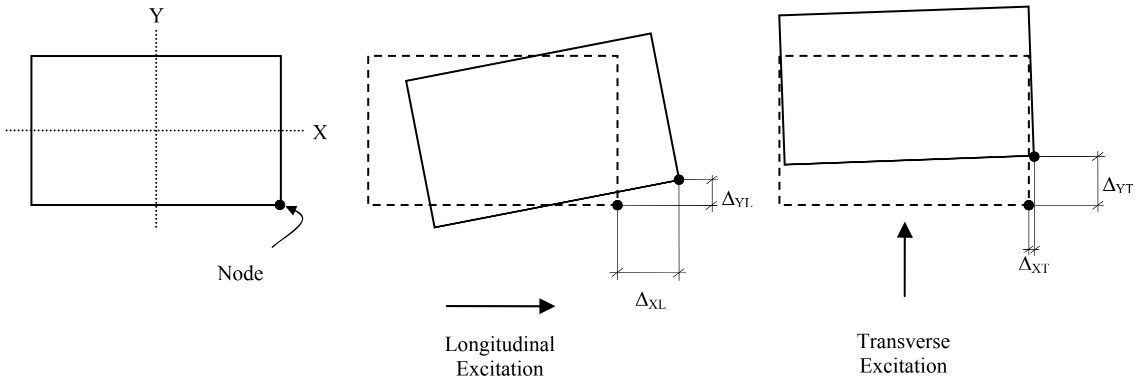


Figure 4-20: Wharf response from seismic motions

Input response spectra shall be applied separately along two orthogonal global axes (longitudinal and transverse). Spectral responses shall be obtained by the maximum of the following two loading cases:

Case 1: Combine the response resulting from 100% of the longitudinal loading with the corresponding response from 30% of the transverse loading.

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

Case 2: Combine the response resulting from 100% of the transverse loading with the corresponding response from 30% of the longitudinal loading.

$$\Delta_{X2} = 0.3\Delta_{XL} + \Delta_{XT}$$
$$\Delta_{Y2} = 0.3\Delta_{YL} + \Delta_{YT}$$

where,

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

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

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

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

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

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

The magnitude of seismic demand for a node (Δ_d) is defined as:

$$\Delta_d = \max\left(\sqrt{\Delta_{X1}^{\ 2} + \Delta_{Y1}^{\ 2}} \text{ or } \sqrt{\Delta_{X2}^{\ 2} + \Delta_{Y2}^{\ 2}}\right) \tag{4.25}$$

Nonlinear time-history analysis has shown that the 100% + 30% spectral combination rule to be non-conservative for wharf structures (Ref. 15). If Modal Response Spectra Analysis method described in Section 4.9.4.2 is used for the wharf design using 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 analysis. 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 (e.g. the Substitute Structure method) to check results. When modeling reinforced or prestressed concrete members or steel members with concrete plugs, degrading stiffness models such as the Modified Takeda rule (Ref. 42) should be adopted.

Displacement demands from NTHA shall be based on simultaneous orthogonal horizontal input motions, as defined in Section 2.1. Multiple time histories 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 Time-History 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 flexural demand deck beams, and deck slabs, the calculated demand on the members or action shall be determined from equilibrium considerations assuming an amplified flexural strength of pile plastic hinges together with gravity loads.

$$M_o = 1.25 M_p$$

.The pile shear demand can be determined by

$$V_o = 1.25 V_p$$

where

V_p = The 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

M_p = The idealized plastic moment capacity of the pile calculated by $M\text{-}\phi$ analysis

V_o = The pile overstrength shear demand

M_o = The pile overstrength moment capacity

Deck design moments and design shear forces shall be in equilibrium with the enhanced pile plastic moment capacity defined in this section.

4.10.1 Pile Displacement Capacity

For typical piles at the Port, the top and in-ground plastic hinge moment capacities are similar ($M_{p,\ in\text{-}ground} / M_{p,\ top} \leq 1.25$). For these cases, the distance L_c to the point of contraflexure is approximately the same for the top hinge and the in-ground hinge, so the displacement capacity, Δ_c, may be defined by:

$$\Delta_c = (\Delta_y + \Delta_{p,m}) \qquad (4.\ 26)$$

$$\Delta_{p,m} = \theta_{p,m} \times H \qquad (4.27)$$

where:

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

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

H = The distance between the deck soffit and the center of the in-ground hinge

The pile yield displacements, Δ_y, of the top and in-ground hinges are obtained from the pushover analysis. Δ_c shall be calculated for top and in-ground hinges, and the smaller value shall be used for the displacement capacity. Figure 4-21 shows a graphical representation of the displacement capacity calculation for a top hinge. The concept is similar for the in-ground hinge.

For piles with a large unsupported length, U and in-ground and top hinges with a ratio $M_{p,\ in\text{-}ground} / M_{p,\ top} > 1.25$, the displacement capacity calculation becomes more complex, and the procedure used above will not provide accurate results. Therefore, a more detailed analysis using software with hinge definition capabilities that include plastic curvature or rotation limits should be used to determine the displacement capacity.

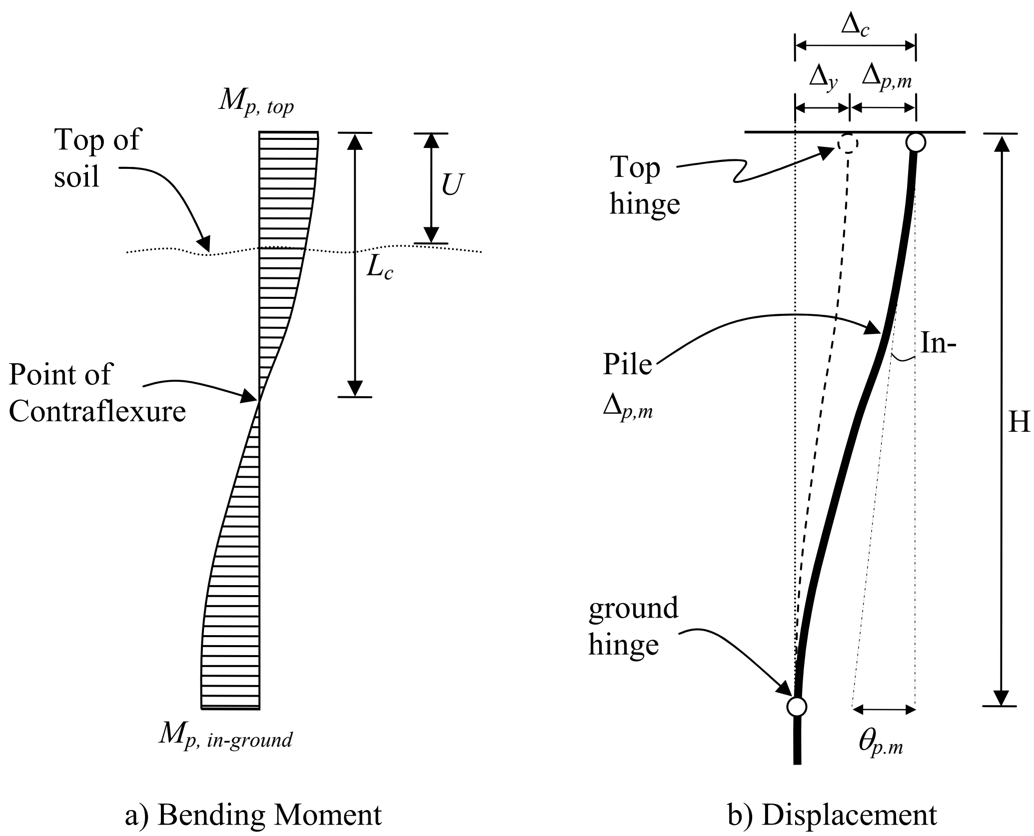


Figure 4-21: Schematic Pile Moment and Displacement Diagrams

4.10.2 Pile Beam/Deck Joint

The nominal strength capacity of the pile cap or deck shall be sufficient to ensure the piles have reached their plastic limit prior to the pile cap or deck reaching its expected nominal strength. The pile cap or deck shear and flexural capacities shall be 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 joint. Any distribution factors shall be based on cracked sectional properties.

For the joint details shown in Figure 4-27, joint shear requirements are satisfied by providing adequate confinement. The effective volumetric ratio ρ_s of confining steel around the pile dowels anchored in the joint shall be:

$$\rho_s \geq \frac{0.46 A_{sc}}{D' l_a}\left[\frac{f_{ye}}{f_s}\right] \tag{4.28}$$

where:

f_s = Permitted spiral steel stress taken as $0.0015E_s$, where E_s is the modulus of elasticity of the spiral reinforcement
f_{ye} = Expected yield strength of the dowels
l_a = Actual embedment length provided

A_{sc} = Total area of dowel bars in the connection

D' = Diameter of the connection core, measured to the centerline of the spiral confinement

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

Shear strength shall be based on nominal material strengths and shear strength reduction factors. Maximum overstrength shear demand, V_o established from nonlinear pushover analyses shall be used as the design shear:

$$V_{design} = V_o \tag{4.29}$$

where V_o is defined in Section 4.10.

Steel Pile

The shear capacity shall be established from AISC-LRFD or API where applicable.

Concrete Pile

The following applies to concrete piles and to the pile-deck connection of tubular steel piles with dowels and concrete plug. The shear capacity, V_n, shall be calculated using the method described below, and is dependent upon the curvature ductility, μ_ϕ:

$$\mu_\phi = \phi_m/\phi_y \tag{4.30}$$

where for shear strength calculations, ϕ_m is the curvature at the OLE, CLE or DE strain limits.

This method is based on the modified UCSD three-parameter model (Ref. 40) with separate contributions to shear strength from concrete to obtain the nominal shear strength, V_n:

$$V_n = V_c + V_s + V_a \tag{4.31}$$

where,

V_c = Shear strength from concrete, Equation (4.33)

V_s = Transverse reinforcement shear strength, Equations (4.35) and (4.36).

V_a = Shear strength due to axial load, Equation (4.37)

A shear strength reduction factor Φ = 0.85 shall be applied to the nominal strength for OLE and CLE conditions to determine the design shear strength. A value of Φ =1.0 may be used for the DE case:

$$V_o \le \Phi V_n \tag{4.32}$$

Concrete mechanism shear strength:

$$V_C = k\sqrt{f_c^{'}}A_e \tag{4.33}$$

where:

k = Curvature ductility factor as a function of μ_ϕ (see Figure 4-22).
μ_ϕ = Curvature ductility based on Equation 4.35.
A_e = Effective shear area (80% of gross cross-sectional area for solid circular and octagonal piles)
f'_{ce} = Expected strength of unconfined concrete strength (in psi)

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

$$\mu_\phi = 1 + \frac{\phi_{P,dem}}{\phi_y} = 1 + \frac{\theta_{P,dem}}{L_p\phi_y} \tag{4.34}$$

where:

$\phi_{P,dem}$ = plastic curvature at the demand displacement
ϕ_y = yield curvature of the pile
$\theta_{P,dem}$ = plastic rotation at the demand displacement
L_p = plastic hinge length, (see section 4.6.6.2)

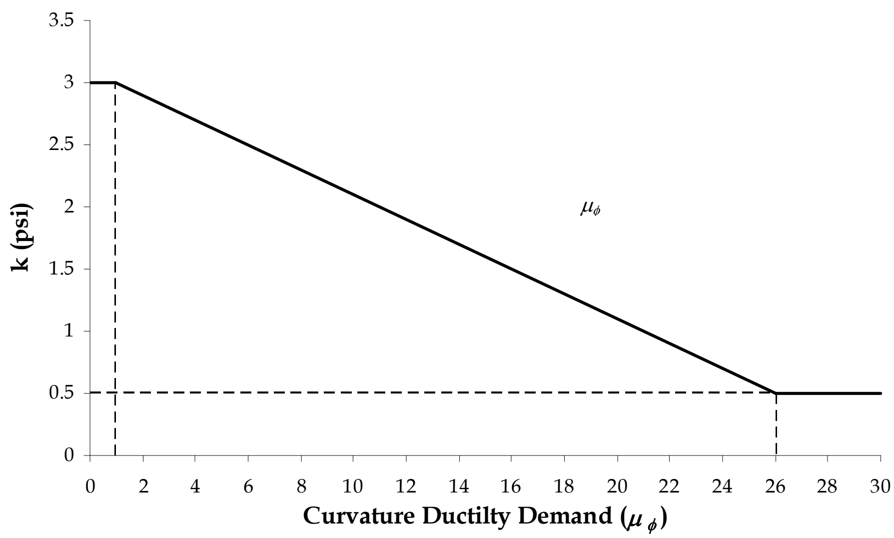


Figure 4-22: Relationship between Curvature Ductility and Strength of Concrete Shear Resisting Mechanism.

***Transverse Reinforcement (Truss) Mechanism Shear Strength**:*

Spirals:

$$V_s = \frac{\pi}{2} \frac{A_{sp} f_{yh} (D_p - c - c_o) \cot(\theta)}{s} \tag{4.35}$$

where:

A_{sp}	=	Spiral cross section area
f_{yh}	=	Yield strength of transverse reinforcement
D_p	=	Pile diameter or gross depth (in the case of a rectangular pile with spiral confinement)
c	=	Depth from extreme compression fiber to neutral axis (N.A.) at flexural strength (see Figure 4-23)
c_o	=	Concrete cover thickness to the center of hoop or spiral (see Figure 4-23)
θ	=	Angle of critical crack to the pile axis (see Figure 4-23) taken as 30^{o} for existing structures and 35^{o} for new design
s	=	Spacing of hoops or spiral along the pile axis

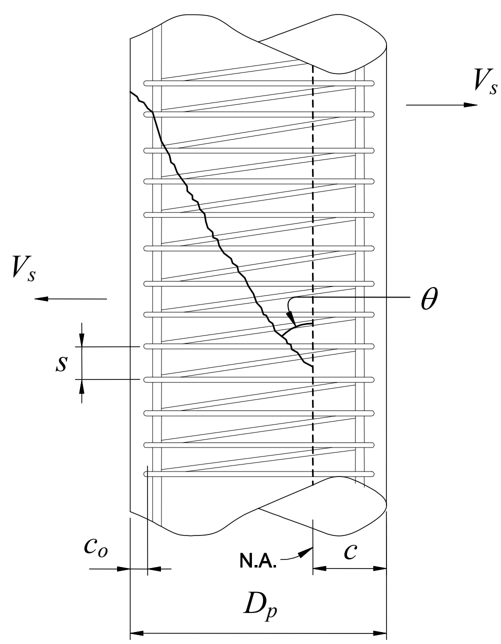


Figure 4-23: Transverse Shear Mechanism

Rectangular stirrups:

$$V_s = \frac{A_h f_{yh} (D_p - c - c_o) \cot(\theta)}{s} \tag{4.36}$$

where:

A_h = Cross-sectional area of transverse reinforcement

***Shear strength from axial mechanism*:**

$$V_a = \beta(N_u + F_p)\tan(\alpha) \tag{4.37}$$

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

α = Angle between line joining centers of flexural compression in the deck/pile and in-ground hinges and the pile axis (see Figure 4-24)

β = 1.0 for existing structures, and 0.85 for new design

Prestress compressive force F_p in the top hinge shall be taken as zero.

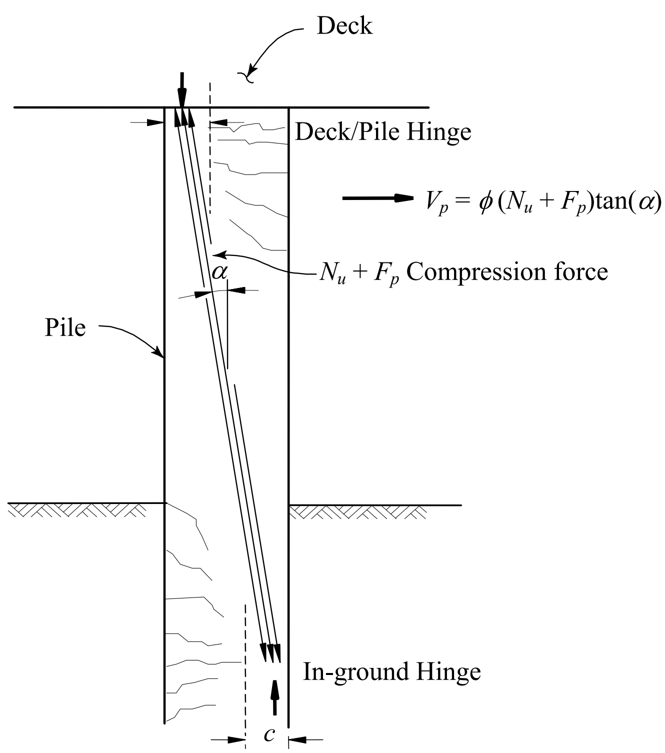


Figure 4-24: Axial Force Shear Mechanism

As an alternative to the above method, the shear strength may be calculated in accordance with the provisions of ACI-318 for piles with curvature ductility $\mu_\phi < 2$.

4.10.4 P-Delta Effects

The effects of P-Δ may be ignored when:

$$\frac{F}{W_{DL}} \geq 4\frac{\Delta_t}{H'} \tag{4.38}$$

where:

- F = Base shear strength/total lateral force of the wharf strip (see Figure 4-2) obtained from a pushover analysis
- W_{DL} = Dead load of the wharf strip considered
- Δ_t = Displacement demand in the transverse direction
- H' = Distance from the maximum in-ground moment to center of gravity of the deck

4.11 Expansion Joint

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 at UCSD (Ref. 15) 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' wide wharf section. Different wharf segment length combinations were used for the analyses. The segment lengths were varied between 400', 600', and 800', and the analysis was conducted using both lower and upper bound soil characteristics 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:

$$V_{sk} = \beta\left(\frac{V_{\Delta}e}{L_L}\right) \tag{4.39}$$

where,

- V_{Δ} = Lateral force at the maximum displacement calculated from the pushover analysis of a wharf transverse section
- e = Wharf eccentricity between the center of mass and center of rigidity
- L = Wharf segment length
- β = Factor calculated as a function of wharf segment length, see Figure 4-25, (Ref. 15).

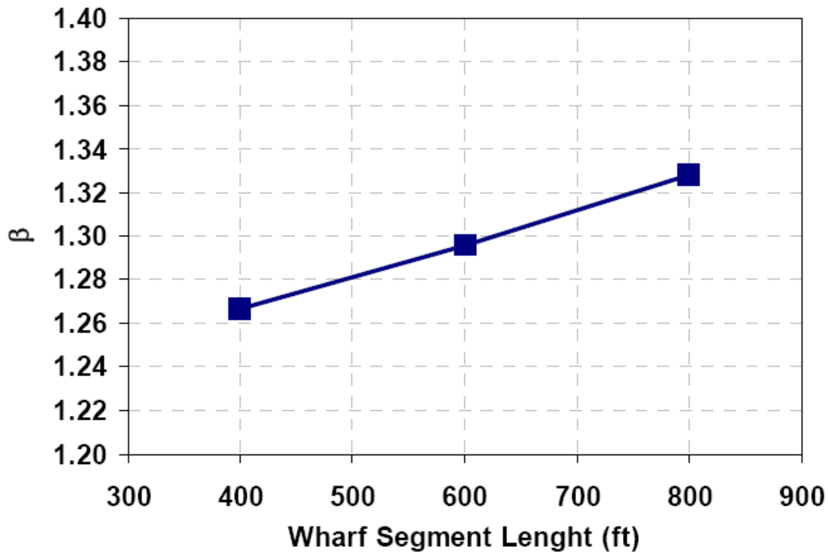


Figure 4-25: β variation

For wharf sections with different configurations, special analysis needs to be performed with prior approval by the port.

For calculating expansion joint shear capacity according to ACI-318, a reduction factor of 0.85 should be used.

4.12 Kinematic Loading

Kinematic loading occurs when the dike begins at depth on a weak soil layer sliding in an earthquake, inducing bending moments beneath the soil surface in the pile. Deep in-ground hinges may form due to the dike movement, as shown in Figure 4-26.

Section 2 provides screening criteria for kinematic analysis of the dike. If a kinematic analysis is required, the Geotechnical Engineer shall provide displacement profiles for the piles under kinematic loading. The piles shall be analyzed for the given displacement profiles, and the material strains in the piles shall be checked according to the strain limits provided in Table 4-1. The shear in the pile shall also be checked.

For the 24-inch octagonal, precast, prestressed concrete piles and pile configurations that are typically used for port wharf projects, kinematic loading at shallow pile embedments (≤ 20 ft.) need not be analyzed. For shallow embedments, lateral soil springs are expected to be softer and piles are expected to have higher displacement capacities than for deeper embedments. Therefore, kinematic loading at shallow embedments is not expected to be a controlling case.

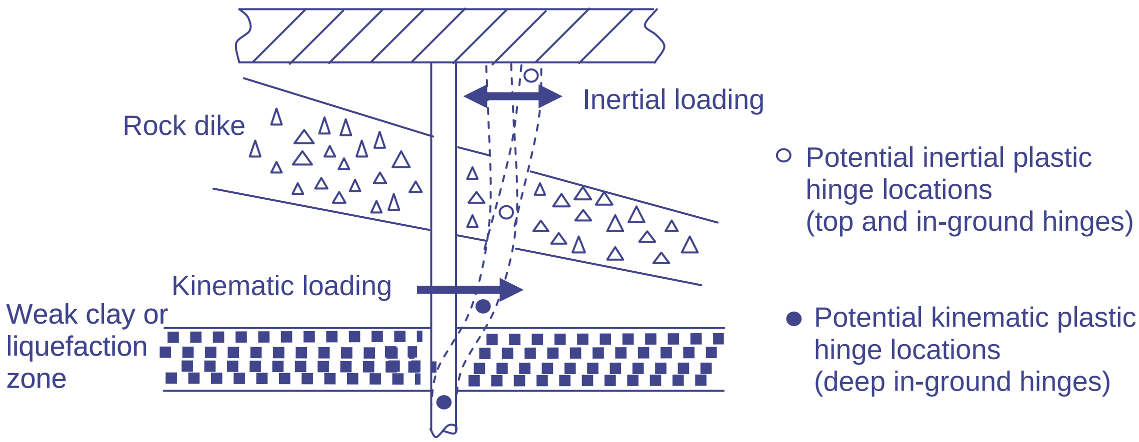


Figure 4-26: Hinge Formation for Kinematic Loading

4.13 Seismic Detailing

The details shown in Figure 4-27 are acceptable confinement details for the pile/deck connection. The volumetric ratio of longitudinal dowel reinforcement (ρ) shall be between 1% and 4%. The maximum dowel bar size should be No. 11. The dowels shall be developed into the pile to satisfy ACI-318 requirements. The joint transverse reinforcement volumetric ratio shall be provided according to Section 4.10.2. The pile prestressing strands shall be cut-off and removed at the top of the pile.

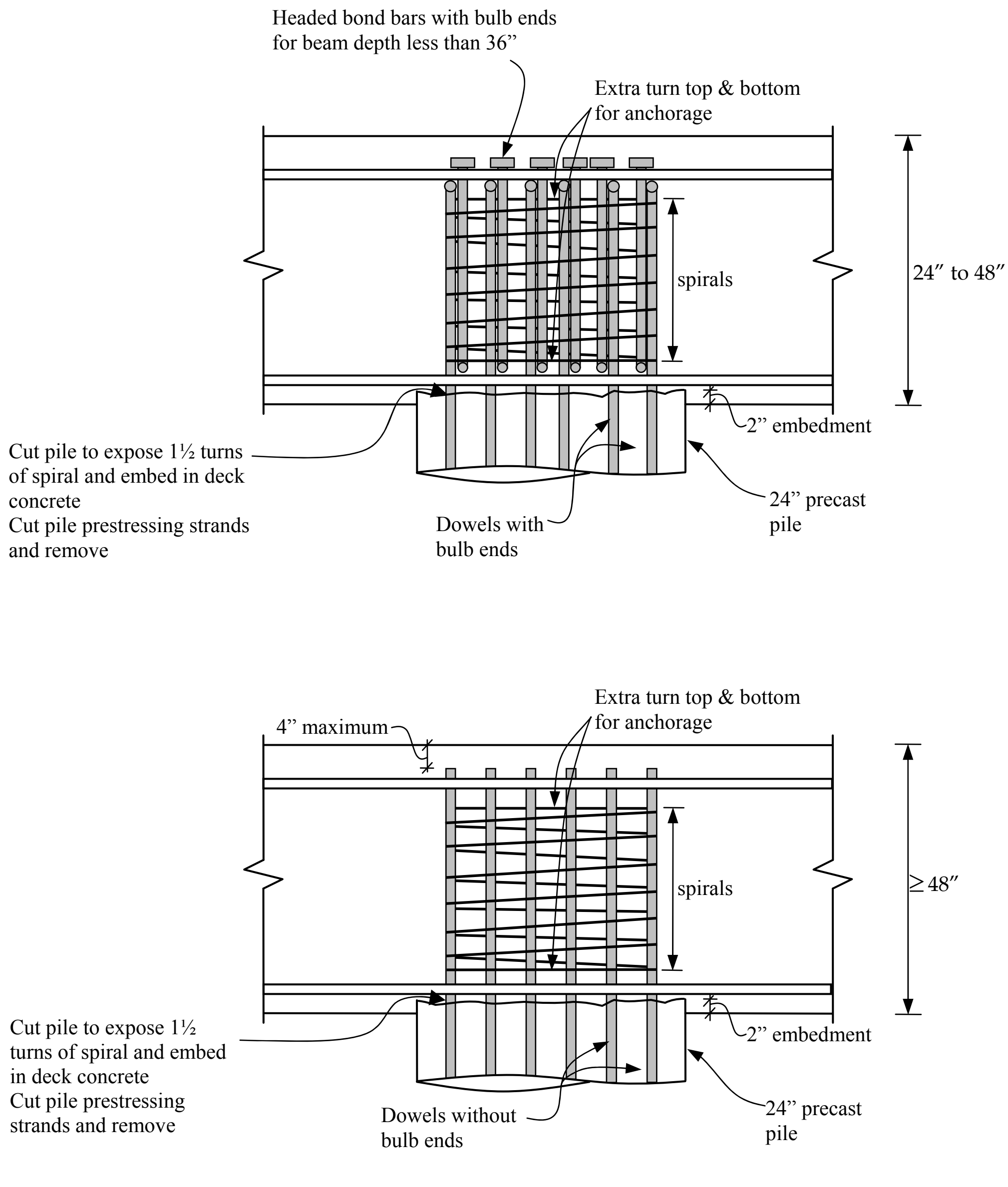


Figure 4-27: Anchorage Details for Dowels

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. Irregular wharf structures
2. Time-History Analyses
3. Presence of faults at the project site
4. Kinematic Analysis

5 Structural Considerations

5.1 Design Standards

The following design codes shall be used where appropriate. The latest published editions of all references with all addenda shall be used in the design.

1. California Building Code (CBC), California Code of Regulations, Title 24 (Ref. 17).
2. American Concrete Institute, Building Code Requirements for Structural Concrete and Commentary, ACI 318 (Ref. 2)
3. ASCE 7-05, Standard, Minimum Design Loads for Buildings and Other Structures (Ref. 11)
4. American Institute of Steel Constructions, Code of Standard Practice for Steel Buildings and Bridges (Ref. 6)
5. ANSI/AWS D1.1, Structural Welding Code – Steel (Ref. 8)
6. California Building Code "Chapter 31F [For SLC], Marine Oil Terminals", also known as Marine Oil Terminal Engineering Standards (MOTEMS) (Ref. 18)
7. American Petroleum Institute, Recommended Practices 2A (Ref. 7)

5.2 Wharf Geometrics

5.2.1 Controls

The Engineer shall refer to the CONTROL Section of the Design Criteria and Standard Plans under General Criteria for specific instructions as to survey controls.

Vertical Datum

The tidal range for the Port of Long Beach is based on NGVD 29 (National Geodetic vertical Datum of 1929), with MLLW = 0.0'. The City of Long Beach uses NGVD 29 with MSL=0.0'. As a reference, NAVD 88 (North American Vertical Datum of 1988) is provided in Table 5-1:

Monuments

The Engineer shall indicate on the Project Plans the location and type for installation of baseline monuments. The Port survey section shall provide to the Engineer 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	0.0	-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 ft.

5.2.2 Structure Elevations

In general, wharf elevations shall maintain facility operations under all tidal conditions. Where applicable, the wharf elevation shall also match that of adjacent facilities, unless directed otherwise by project criteria. Wharf elevations for RO-RO, barge loading and unloading, and special purpose docks are to be determined by project criteria.

5.2.3 Crane Rails

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 designer shall accommodate by fabricating crane legs to match. The longitudinal elevation of a crane rail shall be constant.

Typical rail elevations are +15.0 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 elevations shall be 1/8” for the design elevation, and 1/16” in any 10’ of rail length.

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 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 pin pockets

The number of crane pin pockets and their location is 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 pin pockets. The number and placement of stowage pin pockets are also based on operational considerations.

5.2.4 Fenders and Mooring Hardware

Fender spacing will be as required by evaluation of the prescribed design vessels. Mooring devices will be located so as to not cause line interference with the 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 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 is 5 feet. Fenders will 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 should also be considered in positioning and sizing fenders.

Mooring bollards will be placed at intervals based on multiples of bent spacing, but no more than 60’ to avoid hull/ wharf strikes. Refer to Section 3 for mooring hardware loads.

Safety Ladder Spacing

Maximum safety ladder spacing shall be 400’ along the face of wharf.

5.3 Construction Materials and Types of Construction

5.3.1 Construction Materials

Portland Cement

Type II modified

Reinforcing

Grade 60 reinforcing, no epoxy coating unless approved by the Port.

Prestressing Tendons

270 ksi strands

Admixtures

Refer to specifications.

5.3.2 Cast-in-place concrete

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

5.3.3 Precast concrete

Non-prestressed concrete

Precast, non-prestressed concrete strength (f'_c) shall be 5,000 psi minimum at 28 days. Minimum concrete cover over reinforcing bars shall be 2" for the top face, and 3" for all other faces.

Prestressed concrete (other than piles)

Precast, prestressed concrete strength (f'_c) shall be 6,000 psi minimum at 28 days. Minimum concrete cover over reinforcing bars shall be 2" for the top face, and 3" for all other faces.

Prestressed concrete piles

Precast, prestressed concrete pile strength (f'_c) shall be a minimum of 6,500 psi at time of driving, and 4,500 psi at time of tendon stress transfer. Minimum concrete cover over spiral reinforcing bars for piles shall be 2½".

5.4 Structural Systems and Components

5.4.1 Wharf Deck

Beam / Slab

This system consists of a wharf slab supported by pile caps (beams) that are supported by piling. When 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 punching shear from the pile reactions. The slab depth in this case can be reduced by the use of capitals or shear caps under the deck at the pile locations.

Flat slab systems need to consider the larger associated seismic mass when compared to a beam/slab configuration.

Precast Panels

This system consists of precast deck slabs placed on top of cast-in-place bent caps, which are supported by the 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 falsework required, 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 existing construction, 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 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 structural 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

Transverse expansion joints are determined by thermal forces, and are typically placed at a maximum of approximately 800' along the wharf.

5.4.3 Cut-off walls

Cutoff walls are vertical subsurface barriers designed to prevent erosion of yard materials under the wharf. They are normally constructed along the back edge of wharf, and are of sufficient depth to maintain kick-out stability, while still providing erosion protection. They can be of either precast or cast-in-place construction. Cut-off walls should not be relied on for seismic resistance.

5.5 Piling

5.5.1 Clearance

An approximate minimum of 4'-0" clearance shall be used between the deck/ beam soffit and top of dike to allow for adequate post earthquake inception and repairs.

5.5.2 Concrete Piles

The standard pile is a 24" octagonal precast prestressed concrete pile. Larger size solid or hollow piles may be proposed for situations where the 24" 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" octagonal precast prestressed piles shall be approved by the Port.

5.5.3 Steel Pipe 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 criteria and site circumstances dictate.

5.5.4 Battered Piles

The use of battered piles to support the wharf or pier shall not be used without prior written approval from the Port; see Section 4.2. However, battered piles may be used for isolated structures with low seismic mass, such as landside anchors, mooring and breasting dolphins.

5.6 Structural Analysis Considerations

Materials

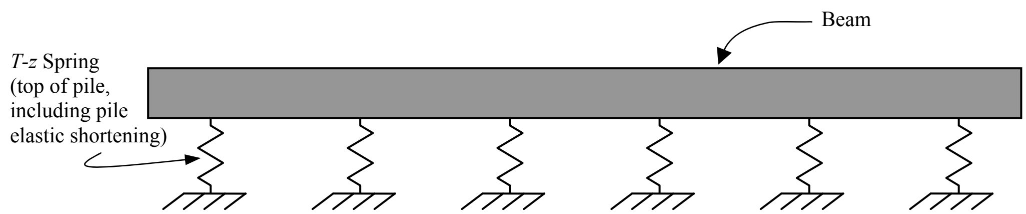
For service load analysis (dead load, live load, wind, etc.), the material properties shall be based on the relevant design code (Section 5.1):

Section Properties

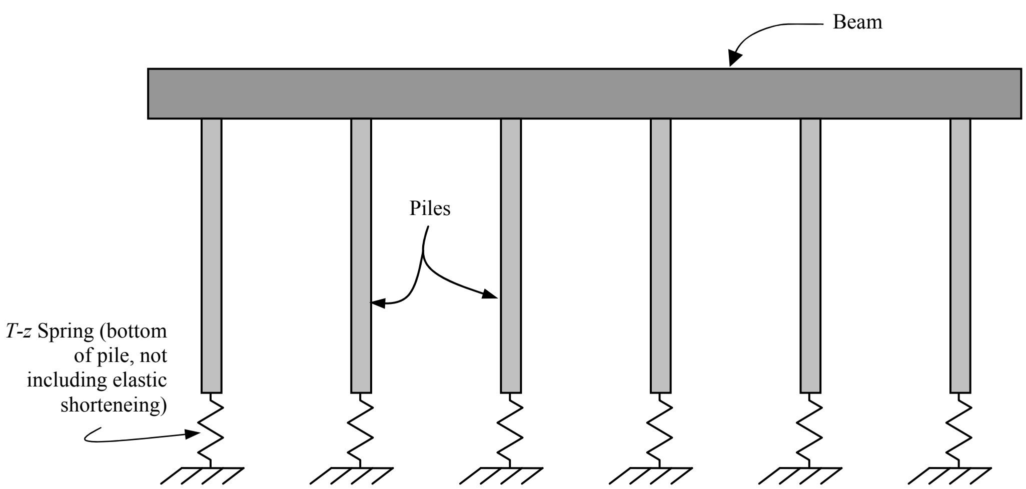
For temperature or creep/rib shortening loads, the effective moment of inertia (I_{eff}) should be used for piles; see Section 4.6.3. For all other service loads, gross moment of inertia (I_{gross}) shall be used.

Beam on Elastic Foundation Model

For modeling the wharf structure frame as beams on elastic foundations, vertical (*T-z*) springs shall be used for the analysis. To find moments in the girder and axial force in the piles, the *T-z* springs may replace modeling the pile, as shown in Figure 5-1-a). The piles should be modeled to determine moments and shear in the piles, if needed, as in Figure 5-1-b). For more discussion on vertical springs, see Section 2.



a) Model for Beam Analysis



b) Model for Beam and Pile Analysis

Figure 5-1: Beam on Elastic Foundation

5.7 Miscellaneous Considerations

5.7.1 Guard Timber

On the waterside edges of piers and wharves, a curb or guard timber (chemically treated) 10” high by 12” wide shall be provided. Notches shall be provided on the underside of the guard timber to permit drainage. The guard timber shall be anchored to the deck slab via recessed bolts or pins, and should include ship’s net anchor rings.

5.7.2 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 design loading on the cover plate shall be the applicable loadings specified in Section 3.3.

5.7.3 Cable Slot

Slots for Cable Reel cable shall be covered with a flexible material, such as reinforced rubber, and shall be a minimum width.

5.7.4 Inclinometer Tubes/ Motion Instrumentation

The decision to instrument wharf structures and provide inclinometers tubes is made at the time of design, and is based on the other instrumentation functioning within the Port.

6 Electrical Considerations

6.1 General

Verify and confirm as to which edition of the California Electrical Code shall be used for checking plans and electrical standards specifications and clearly state this requirement on contract documents.

6.2 Electrical System

6.2.1 Underground Electrical Work

During the design stage of low voltage and high voltage cable systems, the cable manufacturer shall provide a set of their calculations and criteria for pulling the cables through the conduits. Also, the contract shall require the Contractor to provide this criteria and calculations as part of their submittal. The sidewall pressure of the conductors shall be monitored when pulling them through the conduit bends to ensure that the recommended limits for the cables are not exceeded. For trailing cable system, coordinate plans and requirements with the Port.

6.2.2 Crane System

Electrical power supply to the cranes shall be provided from crane substations supplying 12,000V power via a bus collector or trailing cable system installed below grade in a trench along the entire length of the wharf. The trench shall be located adjacent to and on the waterside of the waterside crane rail.

An analysis shall be provided for the crane bus bar and feeder calculations, voltage drop, and short circuit withstand rating. Isolation bars, expansion joints, and feeder connection points shall be staggered on all three phases, as opposed to side by side and in line. A weld shall be placed around the entire bracket when it is welded to the anchor plate

Crane substations shall be designed with redundant equipment in order to provide a highly reliable power supply to the cranes. Redundant components of a crane substation (or each substation where separate redundant substations are designed) should be sized to carry the load of all the available cranes and those anticipated for use in the future.

6.2.3 Shore-to-Ship Power System

The electrical power supply for the shore-to-ship power system shall be provided from a 7.5 MVA 6.6kV Unit Substation. If a 7.5MVA transformer is deemed inappropriate for the application, any deviation must be approved by the Port. The shore power feeder shall have a medium voltage circuit breaker with overcurrent protective relays for each ungrounded conductor installed in the switchgear. Protective relays shall be set to the maximum rating of the shore power outlet and shore power feeder. The voltage drop to shore power outlets shall be limited to two percent. The available fault from shore power shall be below the withstand rating of the ship service distribution.

The shore power feeder shall have a grounding switch to discharge the induced voltage before being disconnected from the shore power outlets. A grounding switch shall have a mechanical interlocking scheme between the shore power circuit breakers and shore power outlets to prevent the mating or un-mating of a plug and receptacle while the shore power feeder is energized. An indicating light shall be illuminated when power is available from the shore, and a multifunction meter should be present to read shore power voltage, current, power (kVA, kW, & kVAR), harmonics, 15-minute demand and energy consumption.

A mechanical or electrical interlocking system should be implemented between the shore power circuit breaker and ship generator circuit breakers unless load transfer paralleling capability is provided. A ground check relay shall be provided to automatically open the shore power circuit breaker once the plug is released from the shore power outlet or once the ship circuit breaker provided for the shore power feeder is opened.

A shore-to-ship connection box mounted in exposed locations shall be rated for NEMA 4X enclosures, 14-gauge, and 304 stainless steel. It shall be properly sized to accommodate conductor bending radius and have provisions for bottom and end entrances. The Shore-to-Ship Connection Box assembly shall be rated for the maximum available fault and shall be designed to prevent moisture or water entrance.

Shore-to-ship power outlets shall have current and voltage ratings. Kirk Key interlocks shall be provided in accordance with the approved procedure. A ground switch shall be provided to discharge the induced voltage of the shore power feeder before being disconnected from the shore power outlets. The minimum clearance for live parts shall be per the latest National Electrical Code. Terminals shall be properly sized and shaped to facilitate satisfactory connections, and the phase sequence shall be marked and arranged per National Electrical Code.

6.2.4 Power Systems

The Contractor shall submit the short circuit analysis and coordination study on the electrical system. A review of the short circuit analysis and coordination study with the Port's engineer shall be required when the coordination study is completed and after the report is submitted to the Port for evaluation.

The National Electrical Code 2002 Article 110.16 requires flash protection. Arc flash calculations shall be provided in accordance with IEEE Std. 1584, stamped and signed by a licensed electrical engineer in the State of California. Labels for arc flash and shock hazard shall be provided and attached to equipment in accordance with the National Electrical Code.

6.3 Detailing

6.3.1 General

After contract drawings and the electrical drawings are considered 99% complete, the plans should be checked for interference with conduit runs, gas lines, water lines, manholes and pull box locations, fire hydrants, lighting poles and similar structures.

When an "existing" design is shown on plans, the reference drawing number from which the design originated shall be provided on the plans. This reference drawing number should be listed on the title sheet also. The title of a sheet listed on the title sheet shall be identical to the title on the individual sheet itself. Each electrical drawing shall be identified with an appropriate title, and titles on each drawing should be unique. Similarly, each detail shall have a unique title. For details, the word "Detail" should not be used in the title.

Repeated callouts and details on the drawings should be avoided. All drawings that have the same detail shall be referenced to one sheet with the complete detail.

The following shall be included in the Electrical Plans under "General Notes".

1. When references are made to specific code sections, standards and other similar guidelines, they are intended to add emphasis only. They are not in any way intended to relieve the Contractor of the responsibility of following other applicable references.
2. The Contractor shall be responsible for the testing of the ground-fault protection system after installed on site, in accordance with the latest National Electrical Code. A written record of this test shall be made and shall be available to the Engineer.
3. Conduit that is to be abandoned shall have wires removed. The conduit ends shall be removed to a depth of at least 12" below the finished surface. Both ends of abandoned conduits shall be crimped if the conduit is metallic; conduits made of non-metallic materials shall be capped at both ends with concrete.
4. All electrical equipment and materials shall be U.L. listed and labeled.

Instructions for removals shall clearly identify what is to be removed. The notes shall provide instructions on whether the item will be removed and disposed of by the Contractor or whether it will be salvaged. If the item is to be disposed of, clear instructions shall be provided on the contract drawings that the Contractor shall be responsible for disposal. If it is to be salvaged, clear instructions shall be provided on the contract drawings, including exactly where the item will be delivered and the name and telephone number of the contact person who has agreed to accept it.

To be considered equal, a note shall be included in the specifications or drawings, such as:

"The manufacturer to be considered as equal for a product must first submit all that is specified for similar work they have performed within the past five years. This submittal shall be complete and delivered to the Engineer in one package. This submittal will be first evaluated to determine if all that is listed in the contract documents is included in the submittal package; any missing item will be adequate reason to not approve the submittal. If the package is determined to be complete, it will next be reviewed to determine if the information it contains can meet the requirements of the project specifications and industry standards. After successfully passing these two reviews, only then can the manufacturer be considered equal by the Engineer. The Engineer will make this final determination. Only after this approval is granted by the Engineer may the Contractor proceed with submitting shop drawings by the manufacturer determined to be equal in accordance with the plans and specifications."

If there is a charge for a service connection by the utility company that must be paid by the Contractor during construction, the note below shall be completed and added to the applicable drawing:

> "Contractor shall arrange to obtain the electrical service connection from Southern California Edison. Contractor shall be responsible for paying Southern California Edison charges for this service connection. Bids shall include a sum of $___ for each of these services by the Contractor. If this Southern California Edison charge is less than $___, the Contractor shall reimburse the balance to the Port. If it is more than $___, then the Port will reimburse this extra cost to the Contractor. The addition or reduction of the charge is strictly the difference between the Southern California Edison billing and the $___ included in this note. There will be no other considerations for profit, supervision, overhead, management or ANY OTHER similar items."

Note: *For $___ above, inquire what the approximate cost is from Southern California Edison and use that figure.*

The following note shall be added to drawings when a project requires new electrical service from the Utility Company:

> "Contractor shall be responsible for coordinating the electrical service connection. This includes:
>
> a. Two months prior to the required connection date, notify the Port of Long Beach project manager that a formal request to provide this service connection must be made to the utility company.
>
> b. Make sure the Port of Long Beach project manager follows Port of Long Beach procedures in notifying Port of Long Beach Accounting if Port of Long Beach is responsible for billing. Otherwise, notify the responsible party to request billing from the utility company."

For new electrical services, the Port of Long Beach Design Group can issue a street address shown on the drawing where the new electrical service meter is shown. For existing electrical services, the assigned address must be shown on the drawing where the electrical meter is shown.

Southern California Edison requires a note on the plans to state: "The Contractor shall install a service feed conduit from the existing Southern California Edison Transformer to the Southern California Edison service pole near the transformer per Southern California Edison requirements. The Contractor performing this work shall be a Southern California Edison approved Contractor."

Southern California Edison requires that manhole covers offer provisions for lifting the lids. Reproduce drawings that Southern California Edison has provided the Port onto the contract drawings. Reproduce only the manhole cover, not the lifting device, onto Port of Long Beach contract documents. Verify both with Southern California Edison and manufacturer that what is on the drawing is still valid, available, acceptable and applicable to the project in consideration. See "UNDERGROUND STRUCTURES - 30" ROUND CAST IRON MANHOLE COVER AND FRAME HS-20 LOADING" from the SCE Electric Distribution Manual. (An electronic copy of the drawing is filed under "SCEManholeCover.")

City of Long Beach, Planning and Building, requires separate drawings for separately addressed sewer lift pumps.

6.3.2 Electrical System

6.3.2.1 Underground Electrical Work

For underground conduits, Schedule 40 PVC conduits shall be used in backlands and wharves. Type EB conduits should not be used. The top of the underground conduit bank shall be a minimum of 3'0" below the finished surface. Pull ropes shall be provided in all empty conduits, including stubbed conduits or dead end conduits. For conduits for communications between buildings, manholes, and structures, a minimum of 4" conduits should be provided. For a typical conduit at expansion joints, see Figure 6-1.

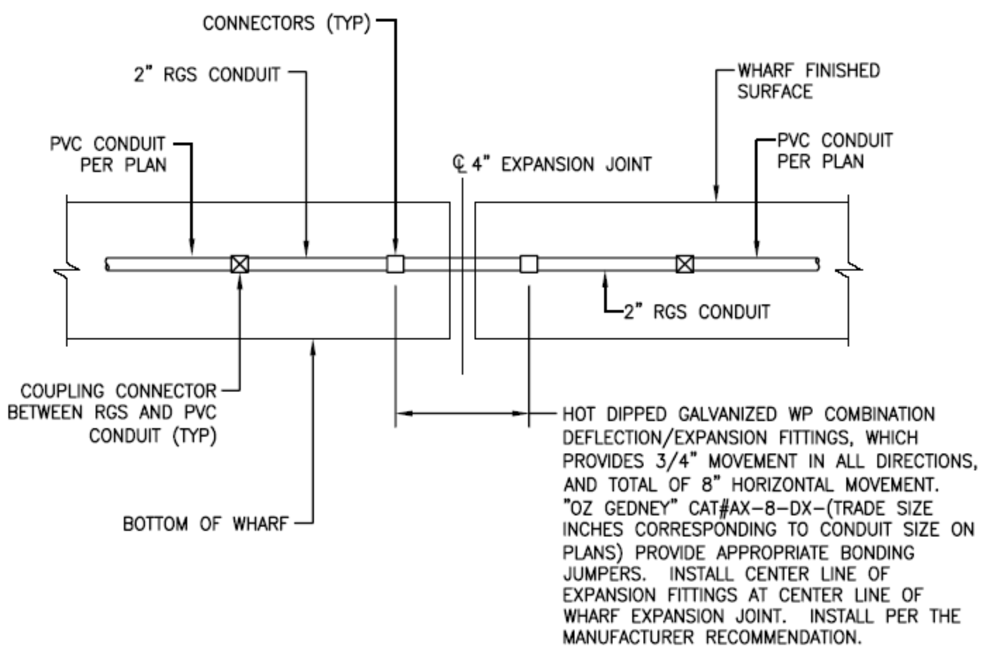


Figure 6-1: Conduit at Expansion Joints

Conduit stubs shall be capped with plastic caps, not with metal caps. For conduit stub detail, see Figure 6-2. The brass cap shall be engraved, indicating E or T, conduit quantity, and size. All actual stub-out locations shall be shown on record drawings. One 90 degree elbow conduit and engraved brass cap shall be provided per duct bank.

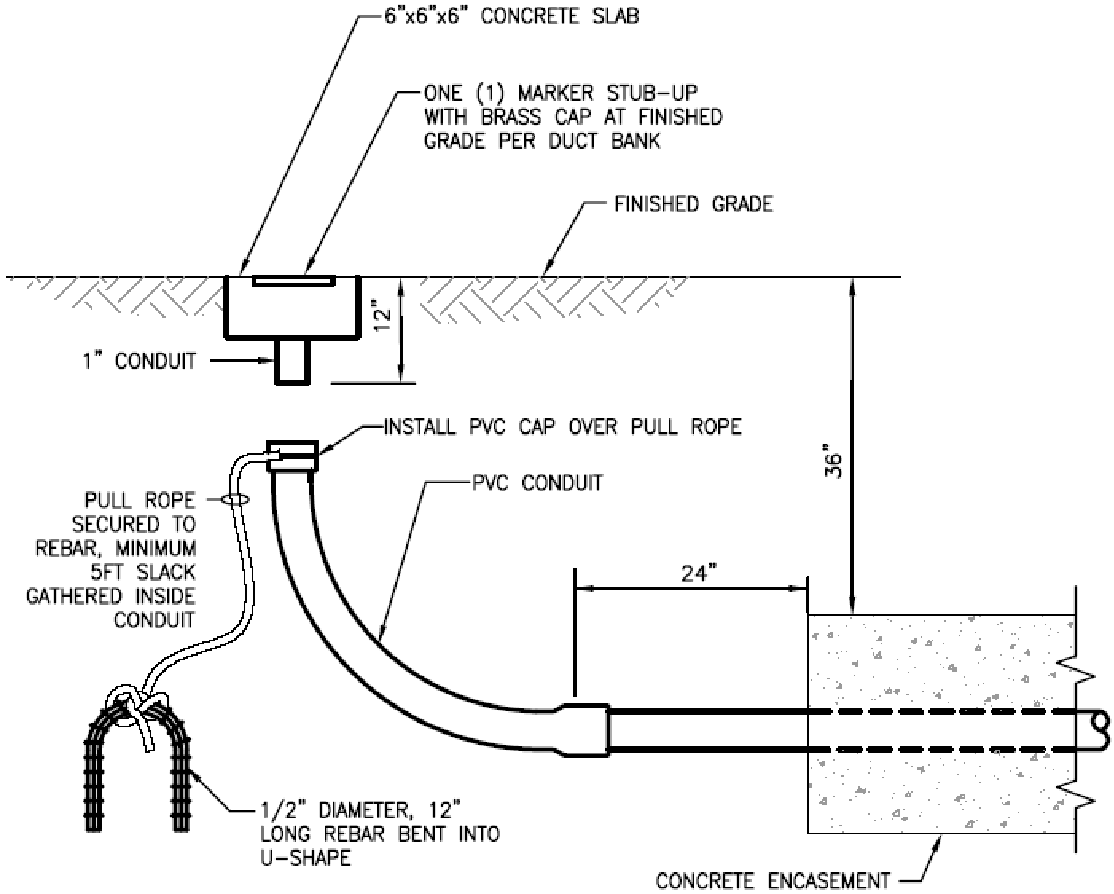


Figure 6-2: Conduit Stub

The following note shall be added to the drawings for underground duct banks to specify the slurry encasement:

> "Construct underground duct lines of individual conduits encased in red concrete slurry. The cement slurry shall consist of three sacks of Portland cement per cubic yard of sand with 10 pounds of red oxide or one gallon of red color. The slurry mix shall be allowed to solidify sufficiently to support a man's weight before beginning the backfill. Compaction of the backfill shall not begin until at least 36 hours after placement of the slurry. No deformed or broken pieces of conduit shall be used. The slurry encasement surrounding the bank shall be rectangular in cross-section and shall provide at least 3-inches of slurry covering the ducts. Separate conduits using plastic spacers. Provide red color admixture in concrete to indicate the duct bank as electrical, except Southern California Edison duct banks. Southern California Edison design mix is required in all Southern California Edison duct banks."

Duct banks shall be separated and the space between them shall be filled with slurry. From a point no more than 60 ft away from a manhole, conduits in a duct bank may begin deflecting to terminate into the manhole. Ampacities of conductors used for electrical ducts shall be calculated per the latest National Electrical Code adopted by the City of Long Beach. A typical duct bank section is shown on Figure 6-3.

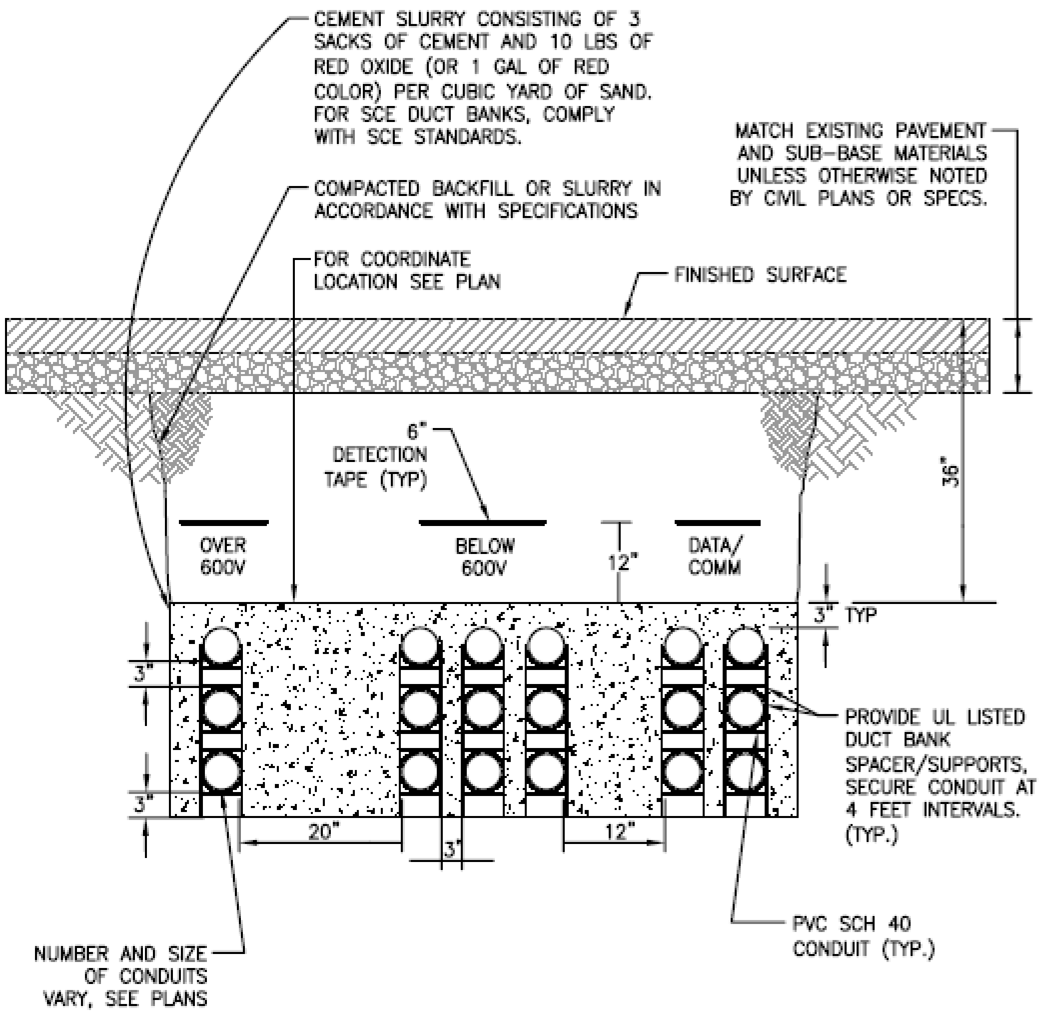


Figure 6-3: Duct Bank Section

The Contractor shall be required to attach a permanent waterproof tag to all conductors in outdoor pull boxes and manholes. Tags shall be marked appropriately.

If pull boxes are rectangular, a detail of the orientation of the box with respect to the conduits entering the box shall be clearly shown. The respective conduits shall allow proper bending radius for the conductors to be pulled through.

Sump drains in pull boxes shall be opened, and clear notes shall be provided on the drawing for the Contractor.

A narrative describing the steps of operation and the function of the components of the control wiring shall be included whenever control wiring is included in the contract drawings.

6.3.2.2 Crane System

The Contractor shall complete installation, include bus bar alignments and elevations, include torque connections and bolts based on Conductix Insul-8 provided data, secure all caps and covers, and clean the bar system including the power trench. Inspection should be coordinated with the Port.

Detail of the wharf trench and crane rail are shown in Figure 6-4, through Figure 6-7.

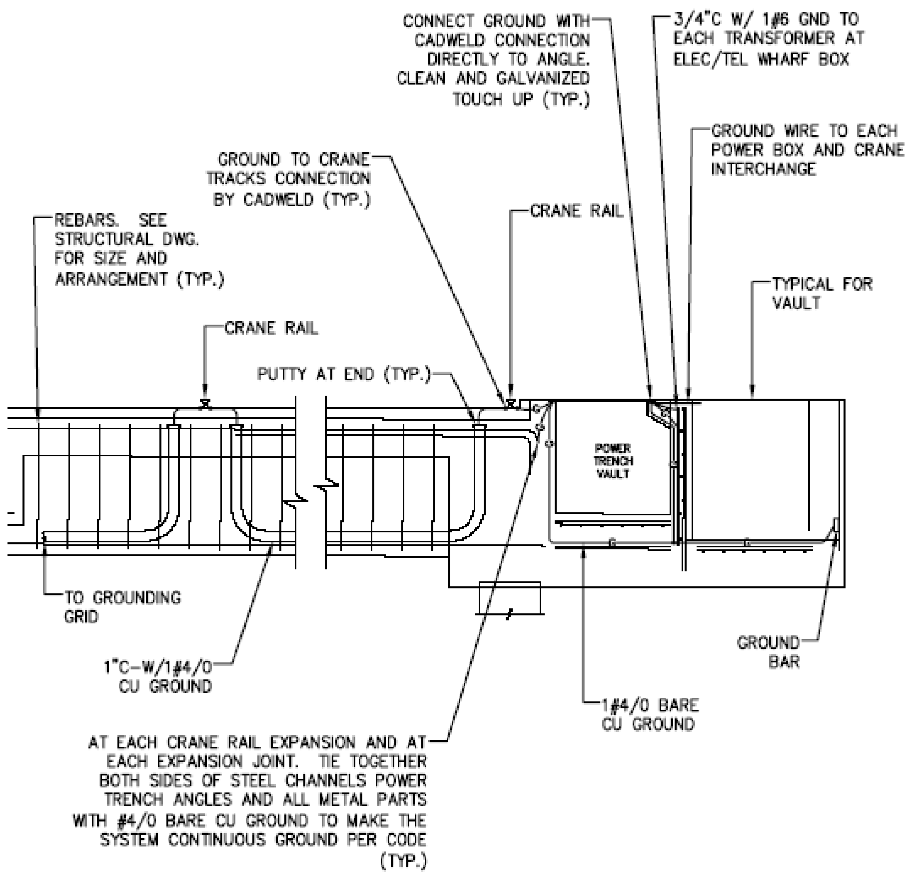


Figure 6-4: Wharf Trench and Crane Rail Cross Section

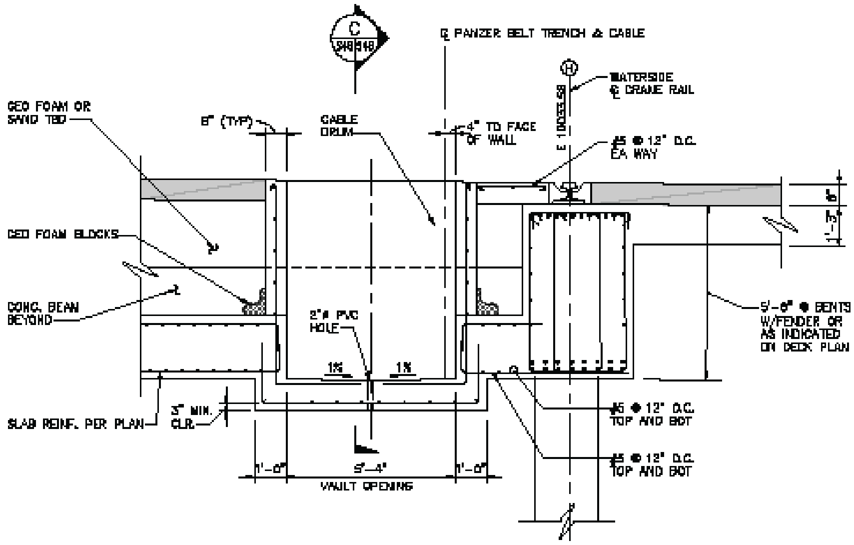


Figure 6-5: Crane Power/ Cable Vault Cross Section

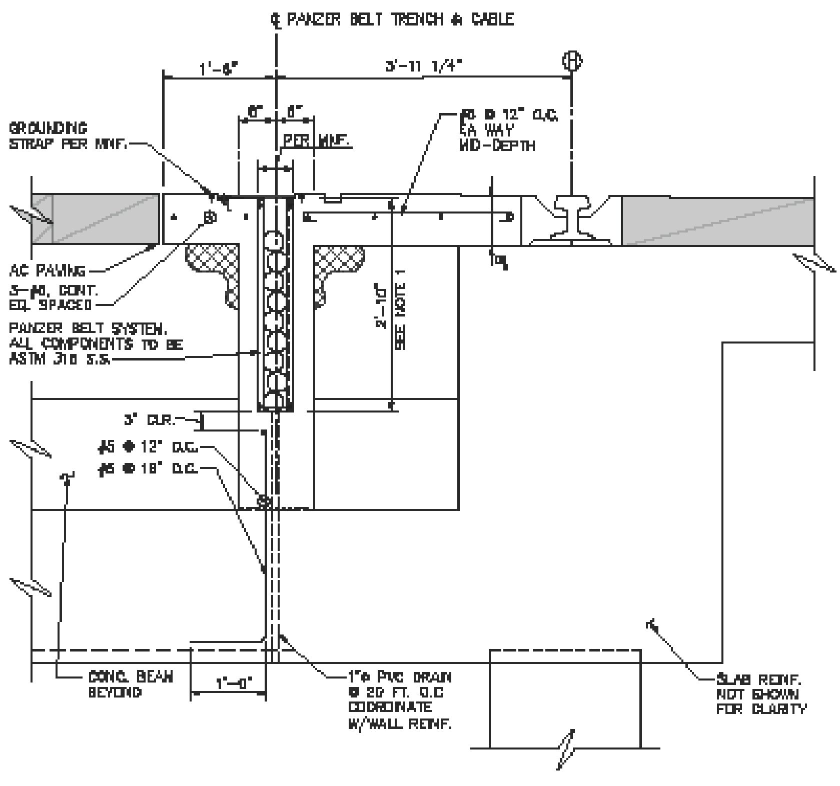


Figure 6-6: Panzer Belt Trench and Cable Detail

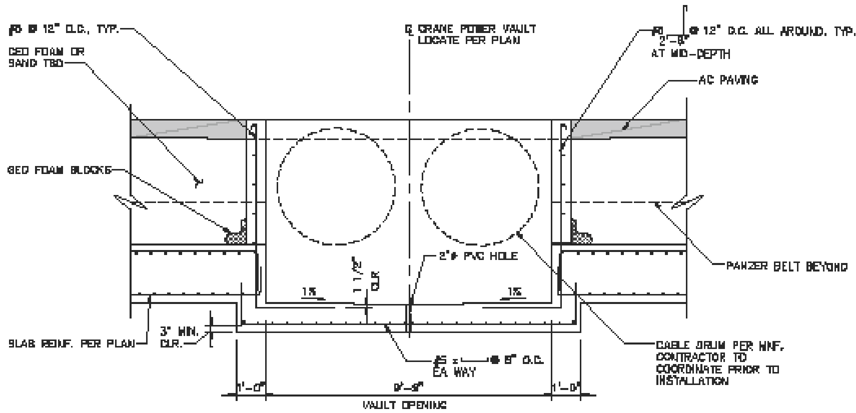


Figure 6-7: Cable Drum Vault Detail

6.3.2.3 Shore-to-Ship Power System

For the shore-to-ship connection box, green and red lights shall be provided to indicate whether the shore power feeder is energized. A nameplate shall be provided stating that when the green indicator light is on, the system is de-energized; a red light indicates that the system is energized. An instruction plate or a sheet indicating connection procedures shall be provided. Warning signals shall be provided that read "Do not engage or disengage plug and receptacle while energized." A nameplate indicating supply voltage, maximum current per outlet, and matching plug shall be provided. The shore-to-ship connection box assembly shall be constructed as shown on Figure 6-8 and a fiber optic box shall be provided.

A minimum of two 5" and two 2" PVC 40 conduits terminating at shore-to-ship connection box shall be provided. Watertight PVC fittings shall be provided, and all conduits shall be sealed with approved foam. Provide approved drop-in anchor, threaded hook. Exposed non-current carrying metal parts of equipment, conductor supports or racks, conduits and other metal appurtenances, including any metal cover and its supporting ring, shall be bonded together and connected to a common ground and to the incoming ground conductors per the National Electrical Code.

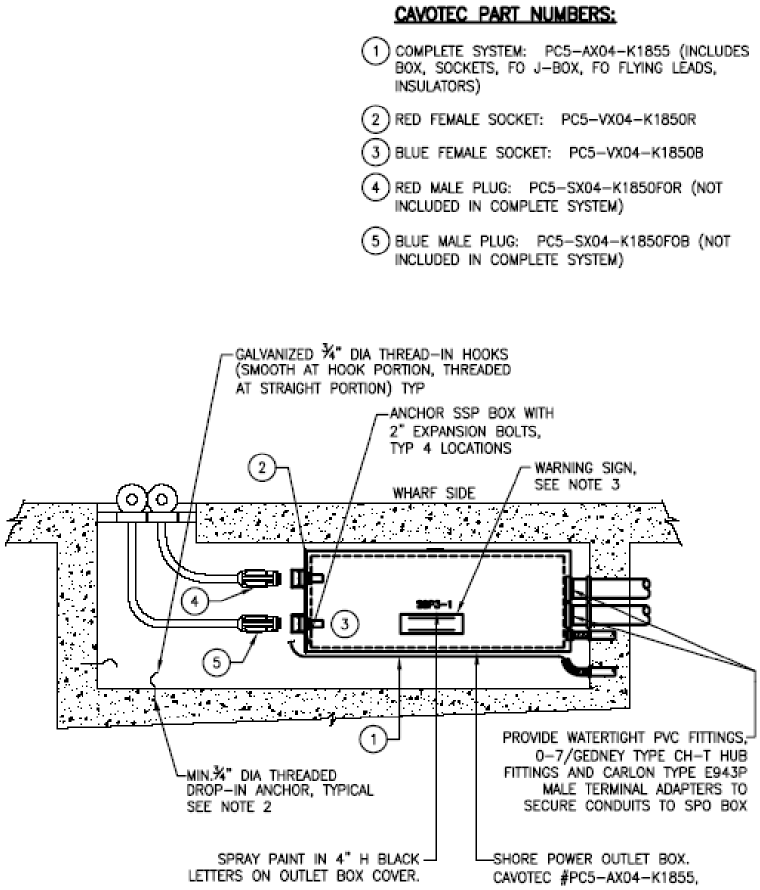


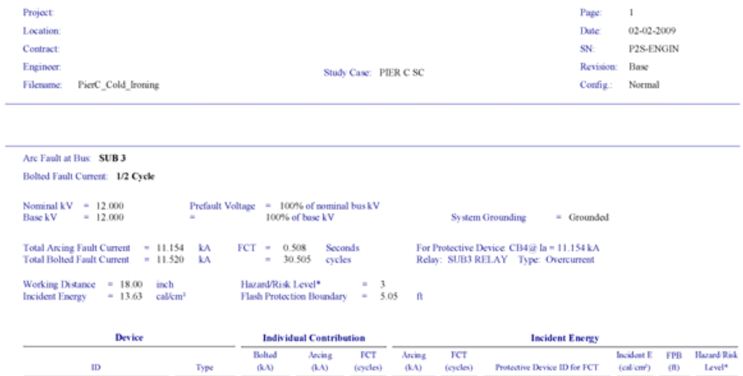
Figure 6-8: Shore-to-Ship Connection Box Assembly

6.3.2.4 Power Systems

The note on the drawings should read as follows:

> "The latest National Electrical Code requires flash protection. The Contractor shall be responsible to provide arc flash calculations in accordance with IEEE Std. 1584, stamped and signed by a licensed electrical engineer in the State of California. The Contractor shall be responsible to provide labels for arc flash and shock hazard, which shall be attached to each piece of electrical equipment, in accordance with the National Electrical Code."

The Arc Flash label and Calculation Results shall be followed as shown on Figure 6-9.



Project:
Location:
Contract:
Engineer:
Filename: PierC_Cold_Ironing

Study Case: PIER C SC

Page: 1
Date: 02-02-2009
SN: P2S-ENGIN
Revision: Base
Config.: Normal

Arc Fault at Bus: **SUB 3**

Bolted Fault Current: **1/2 Cycle**

Nominal kV = 12.000 Prefault Voltage = 100% of nominal bus kV
Base kV = 12.000 = 100% of base kV System Grounding = Grounded

Total Arcing Fault Current = 11.154 kA FCT = 0.508 Seconds For Protective Device: CB4@ Ia = 11.154 kA
Total Bolted Fault Current = 11.520 kA = 30.505 cycles Relay: SUB3 RELAY Type: Overcurrent

Working Distance = 18.00 inch Hazard/Risk Level* = 3
Incident Energy = 13.63 cal/cm² Flash Protection Boundary = 5.05 ft

Device		Individual Contribution			Incident Energy					
ID	Type	Bolted (kA)	Arcing (kA)	FCT (cycles)	Arcing (kA)	FCT (cycles)	Protective Device ID for FCT	Incident E (cal/cm²)	FPB (ft)	Hazard/Risk Level*

NOTES:

1. THIS IS A SAMPLE REPORT, SHOWING THE MINIMUM DATA AS A RESULT OF ARC FLASH CALCULATIONS, IN ACCORDANCE WITH IEEE STD. 1584.
2. CONTRACTOR SHALL BE RESPONSIBLE TO PROVIDE THIS DATA AND SUPPORTING CALCULATIONS PRIOR TO SUBMITTING LABELS FOR ARC FLASH WARNING.

ARC FLASH CALCULATION RESULTS - TYP

SCALE: NTS

NOTES:

1. CONTRACTOR SHALL BE RESPONSIBLE TO PROVIDE LABELS, COMPLETE WITH DATA BASED ON AND SUPPORTED BY CALCULATIONS, AND MOUNT ON APPROPRIATE EQUIPMENT.
2. SUBMIT PROPOSED LABELS TO THE CITY AND OBTAIN APPROVAL OF THE CITY PRIOR TO RELEASING ORDER TO LABEL MANUFACTURER.

ARC FLASH LABEL - TYP

SCALE: NTS

Figure 6-9: Arc-Flash and Calculation Results

6.4 Specifications

6.4.1 General

The manufacturer's name and part numbers should be on the contract drawings, not in the specifications. Specifications should describe the features of the product specified on the plans. New products that have not been used for a Port of Long Beach project shall be approved by the Port and the Port tenant/operator, and shall satisfy building and safety requirements

If instruments or special equipment are specified, provisions shall be added for appropriate training by the manufacturer for the use and maintenance of the item specified. This applies especially to switchgear, lighting controls, sewer pump controls, high voltage splices, and terminations. For specialty systems or equipment such as Fire Warning Systems, cranes and hoists, or dimming systems, the model number and manufacturer's name shall be provided. Furthermore, the branch circuit conductor and overcurrent protection shall also be provided. If the Contractor makes a substitution, the Contractor shall be required to submit the whole system as a shop drawing.

The Port of Long Beach Electrical Standard Specifications shall be used as a reference for design. If changes to these specifications are necessary, the Port of Long Beach Administrative Control Section shall revise and adopt the changes before they are included in the final design specifications. If the electrical standard specification is not available on a particular item from the Port of Long Beach, the Administrative Control Section shall be informed of the proposed specification for their review and adoption before it is included in the final design specifications.

Two letters from Southern California Edison should be received and filed before the project is advertised. The first shall state that Southern California Edison will provide the new service, and it should be received from a Southern California Edison representative. The second letter shall state what the Southern California Edison available interrupting current (AIC) is at the service point.

Southern California Edison requires the name of a contact person for correspondence. The contact's name, address and telephone number should be listed on the drawing where the electrical service meter is shown. The cement around the duct banks shall be the natural gray color. Southern California Edison requires a minimum of 2-1/2" for the service conduit size. The electrical service design shall always be approved by Southern California Edison.

The size of the service riser conduit and the weatherproof entrance cap shall be shown on the service poles. The detail shall be reproduced in contract drawings, and Southern California Edison standards shall be referenced. The risers to the poles shall be PVC schedule 80.

When a wharf/terminal is developed, the address should be posted by the main gate of the terminal. An appropriate sign shall be provided and installed, and the sign shall meet the requirements and be approved by the Fire Department.

The name and phone number of the Contractor/manufacturer, years of warranty, related drawing and manual/specification shall be included in the warranties.

The Operations and Maintenance Manual shall have a cover page that includes as a minimum: Title, specification number of the Port of Long Beach project, the Port of Long Beach base drawing number of the contract set, date the manual was prepared, person to contact regarding the manual and his/her telephone number.

6.4.2 Electrical System

6.4.2.1 Underground Electrical Work

Megger tests for low voltage cables shall be required in the specifications. During the design stage, the manufacturer of the specified cables shall provide a written statement that cables have a 40 year warranty. High voltage cables shall be installed in the presence of the manufacturer's representative. The specifications and drawings shall clearly describe the cable splices and terminations required for the project.

For outdoor enclosed disconnect switches, the enclosure shall be NEMA 3R. NEMA 3R finish shall be specified as follows:

> “The finish for outdoor, weatherproof, NEMA 3R enclosures shall have all covers and doors thoroughly cleaned using a phosphate wash. Apply a zinc-rich corrosion resistant primer and then a polyester powder coat suitable for a marine environment. Exterior surfaces shall be given a final finish coat of ANSI 61 light grey air-dried acrylic enamel, covered with a clear polyurethane top coat.”

The specifications shall state that the Contractor will provide narratives describing the steps of operation and the function of the components of the control wiring with the submittal of such shop drawings.

When alarms are designed, the consultant shall coordinate with the Port and any other interested parties to transmit the alarm signal to the proper area and to design an appropriate sign to be posted at the alarm equipment that reads “(insert telephone number) CALL FOR ALARM”.

6.4.2.2 Crane System

High potential dielectric tests of the new bus system shall be performed. Results of these tests are to be provided to the Port. Tests shall be conducted by a qualified high voltage testing Contractor using a 20kVDC (15kVAC) high potential test instrument for a 5kVAC conductor bus and a 38kVDC (27kVAC) high potential test instrument for a 15kVAC conductor bus. Test results shall be followed with certified test report that includes the make and model of the test instrument, the date of the last calibration of the instrument, a statement on the weather conditions, including temperature, the signature and electrical license number of the high voltage test technician, and the date of the test.

Table 6-1: Maximum Acceptable Leakage

Conductor Length (ft)	Maximum Acceptable Leakage (total based on system length) (mA)	Test Unit Size Required (for AC test unit, increase size by 40% for DC units) (VA)	
		5kVAC Conductor Bus	15kVAC Conductor Bus
100	7.5	110	200
200	15	220	410
300	22.5	330	620
400	30	440	830
500	37.5	550	1040
600	45	660	1250
700	52.5	770	1460
800	60	880	1670
900	67.5	990	1880
1000	75	1100	2090
1500	112.5	1210	3130
2000	150	1320	4170
2500	187.5	1430	5210

Prior to testing, people in the vicinity of the high voltage test shall be instructed to clear the area. The system shall be free of trash and debris. Each phase of the conductor system shall be tested to ground and to each of the other phase conductors. The test leads shall be connected according to the High-Pot test unit instructions. The voltage shall be gradually increased to the required level and held for one minute. If the system leakage is within the levels outlined in Table 6-1, then the field test is a PASS. After the test has been passed, power shall be energized to the bus.

6.4.2.3 Power systems

The review period for the short circuit analysis and coordination study should take place in one day and should be eight hours long, and the Contractor must have the engineer who prepared the report available during this review period. For smaller projects, a four hour review session shall be required.

Available short circuit current shall be indicated on the plans at the service point and at all panelboards and switchboards. The characteristics of the first upstream protective device shall be indicated on the one-line diagram. If this device is a fuse, the manufacturer model number and ampacity shall be provided. If a relay is applied, the manufacturer model number and CT setting shall be provided. The one-line diagram shall also indicate the time margin Southern California Edison requires between its own protective device and the Port of Long Beach main overcurrent device at the maximum fault level.

6.4.2.4 Grounding

Copper-clad steel grounding electrodes, each with ¾" diameter and 10 ft length, shall be used for grounding where needed. Where a ground rod is needed, a ground well set flush with the finished surface shall be provided as required by the City of Long Beach Building and Safety Department. Otherwise, the ground rod and the attached grounding conductor may be buried under the finished surface or terminated above the finished surface in accordance with the National Electrical Code. Where multiple ground rods are to be installed, rods shall be placed at least 15 ft from any adjacent rods.

The ground resistance shall be measured by employing the "fall-of-potential" method using the Biddle "Megger Earth Tester" with two electrodes. This method shall be required in the specifications. The specifications shall also require that any grounding system test reports include the soil temperature at the time the test was conducted. The specifications shall require the Contractor to provide a copy of the test reports certified by the testing technician and the Engineer's representative authorized to witness the test.

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