Performance based Design of Wharves with Steel Pipe Pile Performance based Design of Wharves with Steel Pipe Piles

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Abstract

Introdution-This paper reviews Performance Based approach (also called Direct Displacement Design method) pier structures, and is built as an extension of the standards developed by POLA/POLB1.The paper reviews performance design of the pier structures supported on steel pipe piles with steel pipe "shear plug" connectors, and benefits of steel pipe sections for design of piers in regions with a moderate to high seismic activity.

Index terms—

1 I. Introduction

his paper reviews Performance Based approach (also called Direct Displacement Design method) pier structures, and is built as an extension of the standards developed by POLA/POLB 1 .

The paper reviews performance design of the pier structures supported on steel pipe piles with steel pipe "shear plug" connectors, and benefits of steel pipe sections for design of piers in regions with a moderate to high seismic activity.

2 II. Performance-based Seismic Design Approach

The following is a review of two most reputable sources on the seismic event criteria utilized by a Direct Displacement Method:

? PIANC WG-34. PIANC reviews only two levels of seismic event:

L1 event -72 year RP L2 event

? POLA/POLB 2012:

-475 year RP

The current Port of Long Beach Wharf Design Criteria identifies three seismic events using Poisson equation: L1 event -72 year RP or 50% probability of exceedance in 50 years. (Operating Level Earthquake) 0.5 = 1-(1-P) ??0 or rewriting expression as a Log function

Log (1-P) 0.5 = 50 = Log 10 0.5 / Log 10 (1-P) = => P=0.0137, T=1/P = 72 years L2 event -475 year RP or 10% probability of being exceeded in 50 years. (Contingency Level or Design Basis Earthquake) 0.1 = 1-(1-P) ??0 Log (1-P) 0.9 = 50 = Log 10 0.9 / Log 10 (1-P) = => P=0.0021, T=1/P = 475 years L3 event -2475 year RP (Code Level Design Eartquake or MCE) 0.02 = 1-(1-P) ??0 Log (1-P) 0.98 = 50 = Log 10 0.98 / Log 10 (1-P) = => P=0.000404, T=1/P = 2475 years Where, P -annual exceedance probability T-mean recurrence interval In a Force Based design method, Design Level Earthquake is determined by scaling mapped M(aximum)C(onsidered)E(arthquake) by a factor of 2/3. In a stark contrast, Displacement Design Method places emphasis on the performance of the structure at different levels of seismic events, rather than on required structural strength corresponding to a single fictitious force of the Design Level Earthquake.

Unlike Force Based design approach based on a single 475 year R(eturn)P(eriod) seismic event, Displacement Design Method reviews structural performance at forces corresponding to 3 distinguished seismic cases: Level L1 (72 year RP case) -Operating level event. Structure should not experience any distress. L2 (475 year RP case) -Design level event. Structure shall stay in service and / or be economically repairable within a month. L3 (2475 year RP case) -Extreme level event. The structure should not collapse during or after seismic event. However, structure might be unsalvageable.

Performance of the pier structure is described by the pier bent diagram shown in Figures ??a and graph indicating sequence of plastic hinge development Figure ??b For proper results, pushover analysis model required for Direct Displacement Design should utilize only Effective Section Properties of the pile section.

The performance analysis of arbitrary concrete section presented by POLA, and deficiency of such analysis is explained below:

The well known relationship between the curvature and flexural moment in terms of Effective Section Moment of inertia for concrete section provides a true statement only for slowly propagating cracks typical for static load application.I eff = M y / (? y * E ce) (Formula 1)

Where, M y -Moment capacity of the section at first yield point.

? y = ? y / c y -curvature at a point where the first rebar or dowel in the concrete section yields ? y -strain in concrete at first yield point. The width of the crack in the concrete section grows with each cycle, displacing and moving fulcrum points formed by the split particles of the concrete jammed between the two plans of the crack.

It should be noted that prestressed strands at the pile top are not developed (strands were not shown in Fig. ??c for clarity), and flexural capacity of the pile at the pile to pile cap interface depends on yielding of the mild steel dowels (position 3) developed into the "shear plug" and into the pile cap.

3 Note:

The term "shear plug" detail (Figures 6a and 6b) denotes composite concrete section developed into the pile and pile cap. The shear plug is designed to provide pile to pile cap connection at the pile to pile cap interface.

Steel pipe shear plug detail more appropriate for high magnitude and intensity seismic loads is shown in Figure 6c.

The dowels of the shear plug (Figure 6a and 6b), however, yield not once, but multiple times during the cyclic movement. Predictability of the dowel elongation in such connections is next to impossible.

It is quite obvious that analysis and design for seismic events of Levels L2 and L3 relies on the cracked or partially plasrtisized concrete pile section, whilst rational design for seismic events of magnitude Level L1 must rely on the fully elastic reaction of the pile material. As it was stated above, predictability of the results based on POLA suggested pile to pile cap connection detail developed for precast prestressed concrete pile is questionable.

Therefore, discussion suggested below concentrates on analysis and design of the wharf framing with steel pipe pile sections only. It will be shown that design utilizing steel pipe sections for piles and shear plugs yields more predictable and accurate results.

Performance based analysis is based on performance (deflection) of the structure during the different seismic events. In turn performance based analysis allows 3 different design approaches: a) Design of RIgid pier b) Design of Flexible pier c) Design of Semi-Flexible pier ? First approach creates extremely rigid structure with relatively high natural frequency, and very high lateral force induced by a seismic event.

? Second approach leads to a structure with partially plastisized connection details or partially plastisized piles. Such piers are softer and experience lower lateral force acting on the pile bent, however, large seismic event frequently leaves residual deformations in the pier structure. ? The last approach is the most rational one. It allows design of the semi-flexible pier for Base Shear that is significantly lower than the Base Shear acting on the rigid structure but slightly higher than the Base Shear acting on the partially plastisized flexible pier.

Resulting structure might experience certain anticipated, but manageable and easily repairable damage within the secondary elements, the damage similar to the damage experienced by the flexible pier structure, but of smaller magnitude. And as always, "Devil is in the details".

4 Ductility of the connection detail.

The factor frequently neglected during the design stage of the project is investigation of the pile to pile cap connection ductility. Ductility of the pile connection and proper detailing allow better predictability of the framing system deformations during and after the seismic event.

Obviously, preferred design would dictate design of the semi-flexible structure. However, in certain cases flexible structure might provide a good alternative design leading to small and justifiable plastic deformations.

Figure 2a, 2b and 2c show plastic hinge geometry and analytical model utilized for Direct Displacement Design Method.

Plastic rotation at the Level L2 or L3 event can be determined from the following equation? p = L p * (? p -? y) (Formula 2)

Where, ? p -curvature corresponding to the plastic hinge at Level L2 or L3 seismic event Pile deflection after development of the first plastic hinge at the soffit.? p = ? p1 * (L -0.5L p) (Formula 5) Note 1:

Point of pile virtual fixity (PVF) approach may be used for preliminary analysis during the FEED study, but shall be avoided for final design.

5 PVF shall be taken as a point where full fixity of the pile produces the same deflection results as the deflection results obtained from the elastic foundation (EF) model. As a conservative approximation, the point of virtual fixity can be taken as a 0-deflection point in the elastic foundation model.

Pile displacement capacity should be determined using upper and lower bound p-y curve soil limits utilizing elasto-plastic behavior of the pipe section.

The displacement capacity of the pile at the level of the top or in ground plastic hinge, whichever is smaller shall be determined as follolws:? c = ? y + ? p (Formula 6)

Where, ? y -elastic displacement, or displacement developed between the initial position of the pile and formation of the plastic hinge.

6 ? p -plastic displacement

For reasonably short piles where ratio of inground plastic moment (M p in ground) to pile head plastic moment (M p head), M p in ground / M p head < 1.25 the distance from the point of contra flexure to the middle of the in-ground and top plastic hinges will be almost identical, and therefore plastic displacement for that condition can be reasonably accurately described by Formula 7:? p = 2? p * (0.5L 1 -0.5L p) (Formula 7)

Where, L 1 -the distance between the point of contra flexure and the pile head.

Both, ? y and ? p are determined from the pushover analysis with pipe section undergoing transformation from the fully elastic to partially plastisized section.

7 III. Basics of the Elasto-Plastic Behavior of the Pipe Sections

For calculating deflection within the elastoplastic mode, the designer shall calculate a new moment of inertia for the pipe pile section. I eff is a variable parameter depending on the extent of the plasticized extremities of the steel pipe section. The step dA i = 1/2 * (R+r) * t * d(?) (Formula 10)

5. Distance from the neutral axis to the elementary area,y i = y ? = 1/2 * (R+r) * sin(?) (Formula 11)

6. The moment of inertia of the pipe section confined by the central angle (?) in each of the 4 quadrants is,) (*) (sin * *) 2 /) ((2 2 2 3 2 ? ? ? + = = ? ? ? a i i eff d t r R dA y I I eff = 1/4 * (R+r) 3 * t * [0.5 * ? -0.25 * sin2(?)] over integration limits (Formula 11)

For checking formula, set integration limits between (?/2) and (-?/2) for fully elastic section:I ? =I a eff = 1/4 * (R+r) 3 * t * [0.5 * ? -0.25 * sin 2(?)] = 0.25 * (R+r) 3 * t * (1.57) (Formula 12)

7. Using Formula 11, designer can determine the central angle (?) corresponding to the flexural demand. 8. Elastic section modulus. (Elastic Section Modulus varies with central angle ?)S ? = I ? eff / y ? (Formula 13)

Where, I ? and y ? are effective moment of inertia (I ? eff) and (y) corresponding to a central angle (?) 9. Moment taken by elastic portion of the sectionM el = F y * S ? (Formula 14)

Plastic section modulus, Z= ?dA i * y i) (*) sin(* *) (* 5 . 0 * 2 * 4 2 / 0 2 ? ? ? ? d t r R dA y Z i i ? ? + = = Z ? = ?1.0 * (R+r) 2 * t * cos(?) over integration limits (Formula 15)

For checking formula, set integration limits between (?/2) and (0) (fully plastic section)Z ? = (R+r) 2 * t (fully plastic section) (Formula 16)

Moment taken by a plastisized portion of the sectionM pl = Fy * Z ? (Formula 17)

10. Total moment capacity of the section is determined from Formula 18M el-pl = F y * (S ? + Z ?) (Formula 18)

Step 10 concludes analysis of partially plastisized pipe section.

8 Example 1.

Example 1 shows analysis of the partially plastisized pipe section in a tabular format below. Pushover analysis should indicate moment demand at every plastic hinge under review (Figure **??** Pier performance shall be based on effective moments of inertia along the pile length, including moments of inertia based on partially plastisized sections. Considering that some length at the top of the pile and part of the pile above and below the point of virtual fixity will consist of composite telescopic sections, location of the plastic hinge shall be determined from the three side by side diagrams: Moment diagram, M; Composite Section Modulus diagram, S; and M / S diagram.

The boundaries of the plastic hinge were defined in Section II above.

9 Note 2:

The length of the hinge is defined by the length of the pile where stress exceeds steel Yield Stress, F y . Pile length within the effected plastic hinge area can be divided in several stepped sections for which designer can calculate composite pile moment capacity and effective moment of inertia using procedures outlined in Example 1.

Figure 2 indicates possible locations of the plastic hinges within the pile length. These areas can be effectively reinforced by a telescopic pile insert of smaller diameter extended into the pipe pile and into the soil socket at the bottom of the pile; and by a shear plug insert pipe at the level of the top plastic hinge. Such E e XV Issue III Version I details, if done properly (Figure **??**), may deliver pier structure with marginal level of plastification and very little residual deflection, if any.

10 IV. Shear Plug Function and Shear Analysis

Shear plug is a short pile element utilized as a transition connector between the pile and a pile cap. Shear plug analysis and design were discussed in "Seismic Design of Pile to Pile Cap Connections in Flexible Pier Structures." 2 Concept of the pile to pile cap connection modeling shall be based on the following assumptions: I. Shear plug shall be treated as a short inverted pile fixed within the pile and embedded into the rigid concrete medium of the pile cap. (Figure **??**) II. Concrete P-Y curves for concrete can be reasonably approximated by the P-Y curves for hard clay, the pile cap mid height. Effect of the strain penetration into the pile cap is negligent. Nevertheless, shear plug prying effect within the pile cap must be investigated.

Figure **??** shows Shear Plug Elastic Foundation model for upper (above the pile / pile cap interface) and lower (bodies of the Shear Plug separated by the plastic hinge. Due to the Shear Plug confinement within the pile cap and pile itself, it can be predicted that the plastic hinge develops at the section having the smallest section modulus: at the pile/pile cap interface. III. Shear Plug embedment into the pile must be treated as a beam on elastic foundation. Pile ovalization due to the shear plug prying action must be investigated and shear plug embedment into the pile must be determined from the model analysis.

Performance based Design of Wharves with Steel Pipe Piles

11 Note 3: Frame analytical model in that case is built on assumption that pile is directly attached to the pile cap at

Springs values for shear plug Elastic Foundation supports within the pile itself are determined from the half pipe model shown in Figure **??** k= P/? . Where, P -is a unit load. Unit load in that model is applied at the center of the section.

For convenience of analysis unit load can be of any arbitrary value that does not produce stress above the yield limit of the section material. ? -is elastic deformation of the section (elastic ovalization) If results of that analysis show that pile material yields or experiences excessive deformations, pipe section might require some form of reinforcement. One option for such reinforcement is shown in Fig. 6c where interior stiffening ring (12) is welded on the interior perimeter of the pipe pile. It would be advisable to weld such ring within 70 to 100 mm from the pile cut off.

12 Plug

Pipe Section Shear Plug vs. Caged Dowel Shear Plug.

Importance of the proper shear plug detailing is shown below. ? Satisfactory plastic moment capacity of the shear plug.

? Shear plug embedment into the pile shall be adequate for prevention of the pile ovalization at the pile /pile cap interface. Shear plug can be considered to be fully adequate if plastification angle (?) does not exceed 80 deg. The angle size was selected arbitrarily for maintaining marginal safety of the design. Importance of the shear plug detail cannot be underestimated.

It shall be explained that connection details of Type 1 and Type 2 can be successfully used in areas with mild to moderate seismic activity.

Type 3 shear plug connection was designed for regions with high PGA and seismic intensity. Shear plug confinement within the pile cap, in that connection, is provided by series of ?-stirrups (11) equally spaced along the height of the pile cap section, and pile ovalization at the top of the pile may be arrested by the circular donut stiffener (12) intermittently welded to the pile perimeter. Alternatively pile section geometry can be checked against plastic deformations using the half pipe model shown in Figure **??**.

Photograph 1 shows pile cap failure due to the lateral shear force. Such failure would be typical for pile caps inadequately reinforced in lateral direction. Type 3 connection detail shown in Figure 6c

13 E

For a neutral observer it is quite obvious that doweled shear plugs are less reliable than a shear plug formed from the pipe section of the comparable diameter, provided shear plug embedment length is adequately designed for prevention of section ovalization at the pile head.

14 V. Moment Capacity and Effective Moment of Inertia of Composite Pile Section

In telescopic pile details where smaller diameter pipe pile is overlapped with larger diameter starting pile the length of overlap shall extend at least 3 insert pile diameters beyond the point where I eff of the partially plastified starting pile combined with an elastic moment of inertia of the insert pile, I ins elast :I tot = I eff + I ins elast (Formula 19)

produce deflection of the pier of wharf structure that will be in compliance with performance requirements of the seismic event. The plastification angle (?) for starting pile shall not be taken less than 80 deg.

15 VI. Overload Factors and Ductility of the System

The following load factors for the limit state design method shall be used depending on the pile capacity to resist overloads by plastic yielding or by forming plastic hinge:

? No yielding possible, ? = 1.25

? Yielding possible until a displacement of at least two times the maximum elastic displacement, ? = 1.00 " For piles undergoing elasto-plastic deformations which are less than twice the elastic deflection based on gross moment of inertia of the affected piles, overload factor ? shall be interpolated.

16 Possibility of overload of an essentially elastic

Capacity Protected Element (CPE) is strong when pile material does not reach the yield point within the two times the max elastic deflection. Forces acting on the pile at the level of the pile cap soffit are than determined from the following equations 3 :M o pile = ? * M p pile (Formula 20) V o pile = 2 * M o pile / L c (Formula 21)

Where, M p -pile plastic moment capacity, at the location of the first plastic hinge. If the shear plug was designed as a composite reinforced concrete section, it is expected that the first plastic hinge will develop at, or slightly below, the soffit of the pile cap. L c -the distance between maximum moments in the pile (distance between the pile cap soffit and point of pile virtual fixity)

Figure **??** shows the Force vs. Deflection Graph where maximum ultimate deflection (? du) is limited by the ability of the single wharf bent to absorb plastic deformations without losing stability. The ratio of the max displacement (? du) to the elastic displacement of the bent (? de) is called bent ductility factor (µ D).µ D = ? du /? de (Formula 22)

Where, ? de -maximum deflection of the fully elastic section ? du -deflection of the fully plastic section prior to failure Note: ? du can be substituted for any arbitrary deflection corresponding to a selected partially plastisized section. That will artificially reduce full ductility to a performance ductility.

Equating the work done by the hypothetical external force (H) to the energy absorbed by the bent:H * ? du = 0.5H p * ? de + H p * (? du -? de) (Formula 23)

Where, H * ? du ? is work done by a hypothetical impact force (H) 0.5H p * ? de + H p * (? du -? de) ? Energy absorbed by a bent prior to being forced into instability. Rewriting Formula 19 in terms of H p /H:H p / H = 2µ D / (2µ D -1) (Formula 24)

Formula 24 establishes the relationship between the bent Capacity (H p) and Demand Load (H), Where H is the maximum anticipated load.

The ductility factor applies only to flexible partially plastisized pile supported systems, but does not have any physical meaning for semi-flexible systems exhibiting fully elastic behavior.

The Base Shear acting on the structure will be reduced by the ductility effect factor.V BS = C sm *W / µ D (Formula 25)

Where, C smi -is an Elastic Seismic Response Coefficient or Spectral Response Acceleration of the single transverse pile bent to the seismic event.

17 E e XV Issue III Version

I W -weight attributed to the pile bent during the seismic event.

C smi ? is magnified acceleration depending on the ratio of forcing frequency to first natural frequency of the structureC smi = PGA * Q

The amplitude of the Response or Force Magnification Factor, Q is described by Formula 26 4Q = 1 / [(1-? 2) 2 + (2? *?) 2)] 1/2

(Formula 26)

18 :

Where, ? = f f / f m ? ratio of the forcing frequency, (f f) to natural frequency of the wharf f m and ? ? is damping ratio. For properly detailed bent with steel piles the damping ratio, ? =0.015 If :? = f f / f m ====> 0 the structure response

approaches the static response where displacement is controlled by the stiffness of the spring, (k) rather than by mass or damping.

? = f f / f m = 1 structure starts to resonate, and if structural damping is zero, dynamic magnification attains infinity.

? = f f / f m > 1 the structure response starts to approach static response again, but in this case structure response is controlled by mass.

In other words, the acceleration of the structure will be scaled up or down from the Peak Ground Acceleration, PGA (horizontal acceleration of the absolutely rigid structure or structure having 0-sec Natural Period) depending on the softening or stiffening effect of the structure.

The damped Natural Frequency can be determined from Formula 27:f m = 0.5? * [k/m * (1-? 2)] 0.5 (Formula 27)

The explains the physics of the response spectra acceleration and how response spectra graphs are built by geotechnical engineers.

The following describes the steps necessary for estimating Fundamental Period of the wharf structure in longitudinal direction, T m2 and eccentricity of application of the orthogonal inertia force, e BS2 :

Step 1. Estimate the spring value of each longitudinal pile bent, k i = P/? Step 2. Calculate Fundamental Period of the whole wharf in longitudinal directionT m2 = 2?*(m tot / ?k i) 0.5 ====> Determine Spectral Response Acceleration C sm2

Where, (m tot) is the total mass of the wharf.

Step 3. Estimate average ductility of the sum of the longitudinal bents, µ a Total inertia force in longitudinal direction,V BS2 = C sm2 *W / µ a

The base shear attributed to each longitudinal pile bentV BSi = V BS2 * (k i / ?k i) Note 4:

It is recommended to design Fundamental Periods of adjacent longitudinal bents such that they satisfy the following requirement 3 : T i / T i+1 > 0.5 to 0.7 That provision was designed with the purpose of eliminating excessive twisting of the wharf deck Position of the inertia force in the transverse direction can be estimated from the following formula: Where, ?x i 2 = I p -polar moment of inertia of the wharf transverse pile bents. Each pile bent is treated as a line. y i -is the y-coordinate of the longitudinal pile bent. y C.L .? is the y-coordinate of the deck centerline.y BS = ?V BSi *y i / ?V BSi

x i -position of the transverse bent vs. deck centerline, taken as an absolute value. e 1accidental eccentricity of the transverse inertia force. e 2 -accidental eccentricity of the longitudinal inertia force.

?V BSi -is an inertia force increment due to the base shear eccentricity.

19 VII.

20 Gravity Component of the Inertia Force

The average live load on the deck (total live load divided by the area of the wharf deck) rarely exceeds 35 to 45% of the specified design live load.

Assuming, conservatively, the dynamic friction coefficient between the live load and the wharf deck, µ d = 0.3, the horizontal live load component of the inertia force acting on the pile bent should be based on 10% to 12% of the L.L. contribution. Gravity load acting on the pile bent shall include N= X% L.L. + D.L.,

Where, "X" can vary from 0 to 100% Whilst Inertia force acting on the same bent Free Field Dike Deformations Free Field Dike deformations in absence of piles can be determined utilizing simplified Newmark sliding block. Newmark method yields reasonably accurate results for short slopes where analytical assumption that all vertical slices of the dike are moving in the same direction is reasonable. For long slopes that method will be extremely conservative as different vertical slices along the slope will have different Natural Periods and might move in opposing directions at each instance.V BS = (45%

POLB recommends seismic coefficient of 0.33*PGA or 0.15 g, whichever is greater, for analyzing pseudo-static seismic slope stability. Pile pinning effect shall not be considered.

That assumption is explained by compatibility of slope lateral deformations and lateral forces exerted by the sliding dike on the pinned piles.

Where slope lateral deformation induces lateral force that displaces pile bent beyond the specified performance limits and / or moment or shear in the pile exceeds 90% of the pile ultimate capacity, the size of the piles and pile bent geometry will require revision.

POLB does not differentiate between the load in the backstage area at Operating Level Earthquake and Design Level Earthquake, whilst ASCE 7-10 treats these loads as transient loads applying reduction factor of 0.75 to the backstage surcharge loads.

analysis based on slope movement shall be decoupled from the pile flexural analysis based on the deck inertia forces.

Performance based Design of Wharves with Steel Pipe Piles Pseudo-static seismic slope stability analysis at the Design Level Earthquake (DLE) and Maximum Considered Earthquake (MCE) shall utilize only 75% of the surcharge load used in the static load analysis. Such reduction in the surcharge load within the backstage area at the time of the maximum seismic event is justified by the extremely low probability of both loads acting simultaneously.

Mononobe-Ocabe formula coupled with modified Boussinesq equations shall be utilized for estimating additional pressure on the cut off wall from the seismic effect of the backstage area. The load from the cut off wall shall be traced to the wharf framing structure. Note 5:

Factor of Safety, F.O.S. for static slope stability shall not be less than 1.5 Whilst pseudo-static seismic slope stability shall be not less than 1.1

If the estimated F.O.S. for pseudo-static seismic slope stability exceeds 1.1, no pile -slope interaction kinematic analysis is required. The following support boundaries shall be used for kinematic model.

21 Modeling Kinematic Loading on the Piles

? Fully fixed base at the bottom. Fixity at the bottom shall be placed at a distance not less than 10 pile diameters from the bottom of the weak layer. ? Rotational fixity at the top shall be placed at a distance not less than 3 to 5 pile diameters from the top of the weak layer (3 pile diameters for pile diameters less than 762mm, and 5 pile diameters for piles with diameter up to 1524mm) POLA/POLB sets the following criteria for concrete piles:

If the estimated Displacement Demand of the slope calculated by the Geotechnical Engineer is less than Displacement Capacity of the pile, no further analysis is required. Otherwise, the pile size or pile bent framing should be modified.

That statement is irrelevant for structures supported on steel pipe piles.

Modified statement rewritten for wharves supported on steel pipe piles will be significantly more relaxed:

? Fully elastic response of the wharf structure to seismic events of level L1 shall be expected. ? Development of full or partially developed plastic hinges in the piles during seismic events of magnitude L2 are governed by performance requirements set for designed structure. ? The forces exerted by the spreading of the dike soil on the piles shall not exceed 80% of the ultimate capacity of the piles providing residual stability of the wharf framing. This requirement is mostly irrelevant for seismic events of level L2, but important for seismic event of level L3, setting a single structural requirement: wharf structure should not collapse during or after extreme seismic event.

In other words, extreme seismic event shall not create fully developed plastic hinges endangering wharf stability.

22 IX. Liquefaction as a Surge Protector

It is important to remember that liquefaction frequently works as a ”surge protector”:

While it increases pile effective length, it simultaneously reduces bent lateral stiffness, k i = H / ? increasing Natural Period of the structure, T m = 2? *(m/k i) 0.5 That in turn reduces Spectral Response Acceleration C sm and corresponding Base Shear,V BS = C sm * W / µ D ’

Where, µ D ’-modified ductility of the pile bent.

Performance based Design of Wharves with Steel Pipe Piles X. Deck Span. Effect of Vertical Acceleration

The effect of the vertical acceleration becomes significant only when the induced force frequency is comparable with the span fundamental frequency. That is not the case for short and rigid spans of the wharf deck having fundamental frequencies, (f m) 3 to 5 times higher than the frequencies of the dominant seismic waves, (f f) Dynamic Magnification in that case is between 4 and 12%: Q= 1 / [(1-0.2 2) 2 + (2*0.01* 0.2) 2)] 1/2 = 1. 04 when? = f f / f m = 1/5 =0.2

Q= 1 / [(1-0.33 2) 2 + (2*0.01* 0.33) 2)] 1/2 = 1.12 when? = f f / f m = 1/3 =0.33

It would be conservative to include 10% weight increase for analysis of the deck structure for total gravity load.

23 XI. Summary. why Steel Pipe Piles?

Steel piles have well defined hysteresis curves and well defined plastic hinges with high level of ductility. That makes them a perfect material for construction in regions with high seismic forces.

24 Corrosion Protection of Steel Piles.

Typical line of defense against corrosion is epoxy coating coupled with cathodic protection. However, cathodic protection works only under submergence. The cons of cathodic protection are frequently neglected. Cathodic protection compatibility with coating must be always investigated. Cases of coating disbondment caused by effects of cathodic protection are well known.

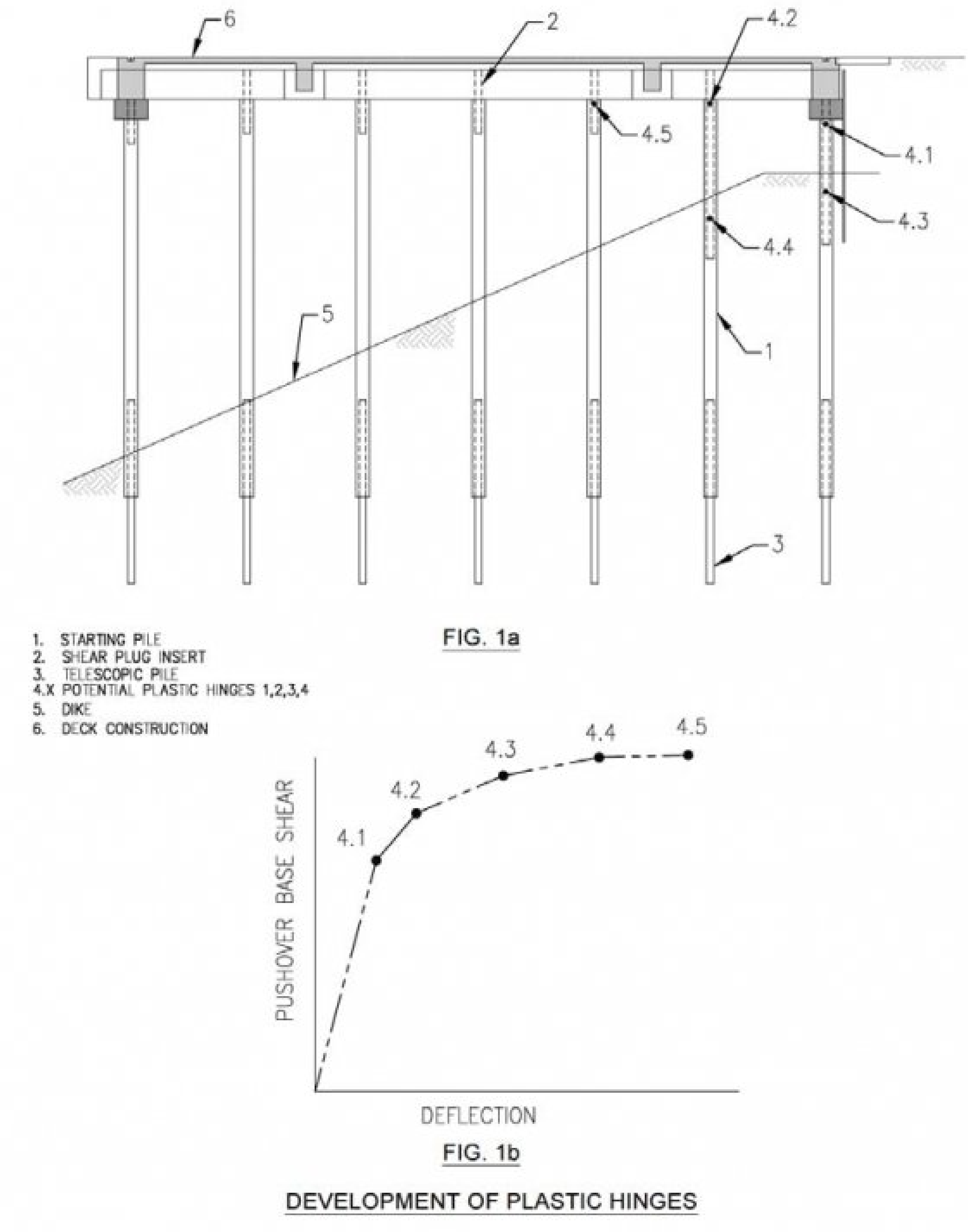
The following is the list of products which showed excellent results in the offshore construction: ? Denso Shield Marine Pile Protection System.

25 ? Archo Rigidon Coating & Linings

The first system consist of the complete wrapping of the effected pile surface, cutting exposure oxygen and salts; and second system consist of special coating which allows up to 40 mils of coating application in one coat. The Archo Rigidon Coating showed high sea water resistance, high temperature tolerance and abrasion resistance and showed excellent compatibility with cathodic protection (low disbondment results). Some cementitious epoxy

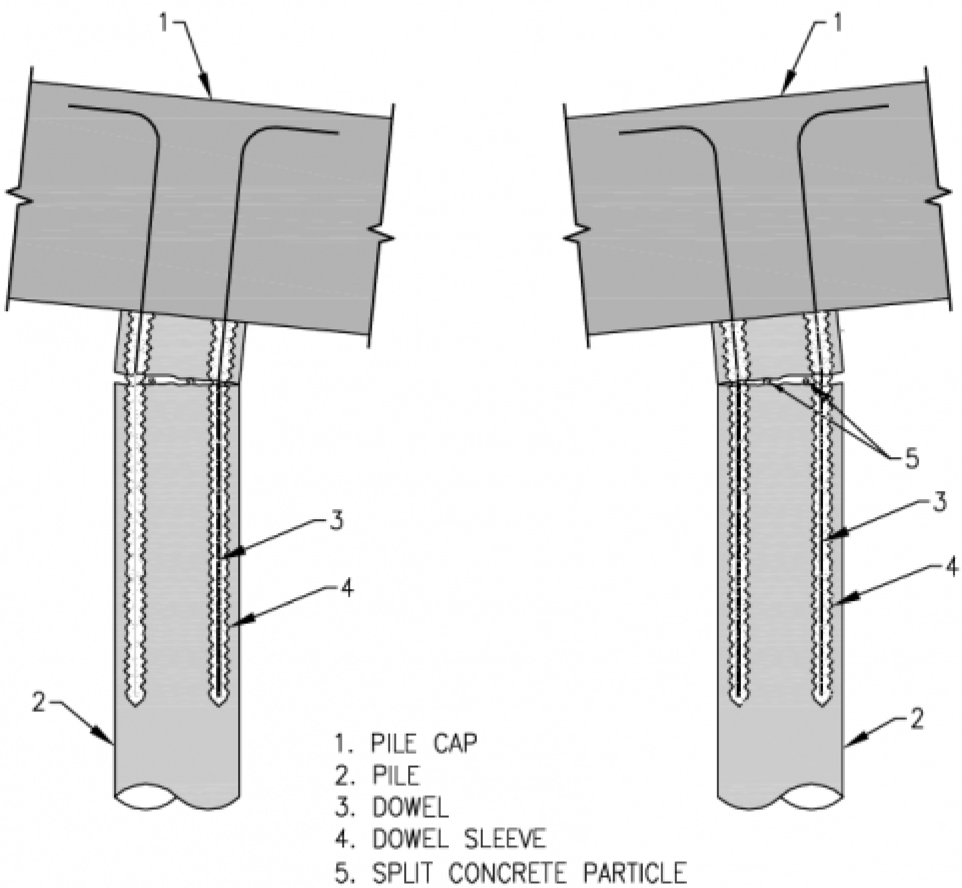
Figure 1:

coatings containing aluminum powder showed excellent results as the stand alone systems, but indicated very poor compatibility with cathodic protection. XII. [1] [2]



3

Figure 2: Figure 3



PLASTIC HINGE DEVELOPMENT
POLA / POLB CONNECTION DETAIL

FIG. 1c

3

Figure 3: ? 3 .

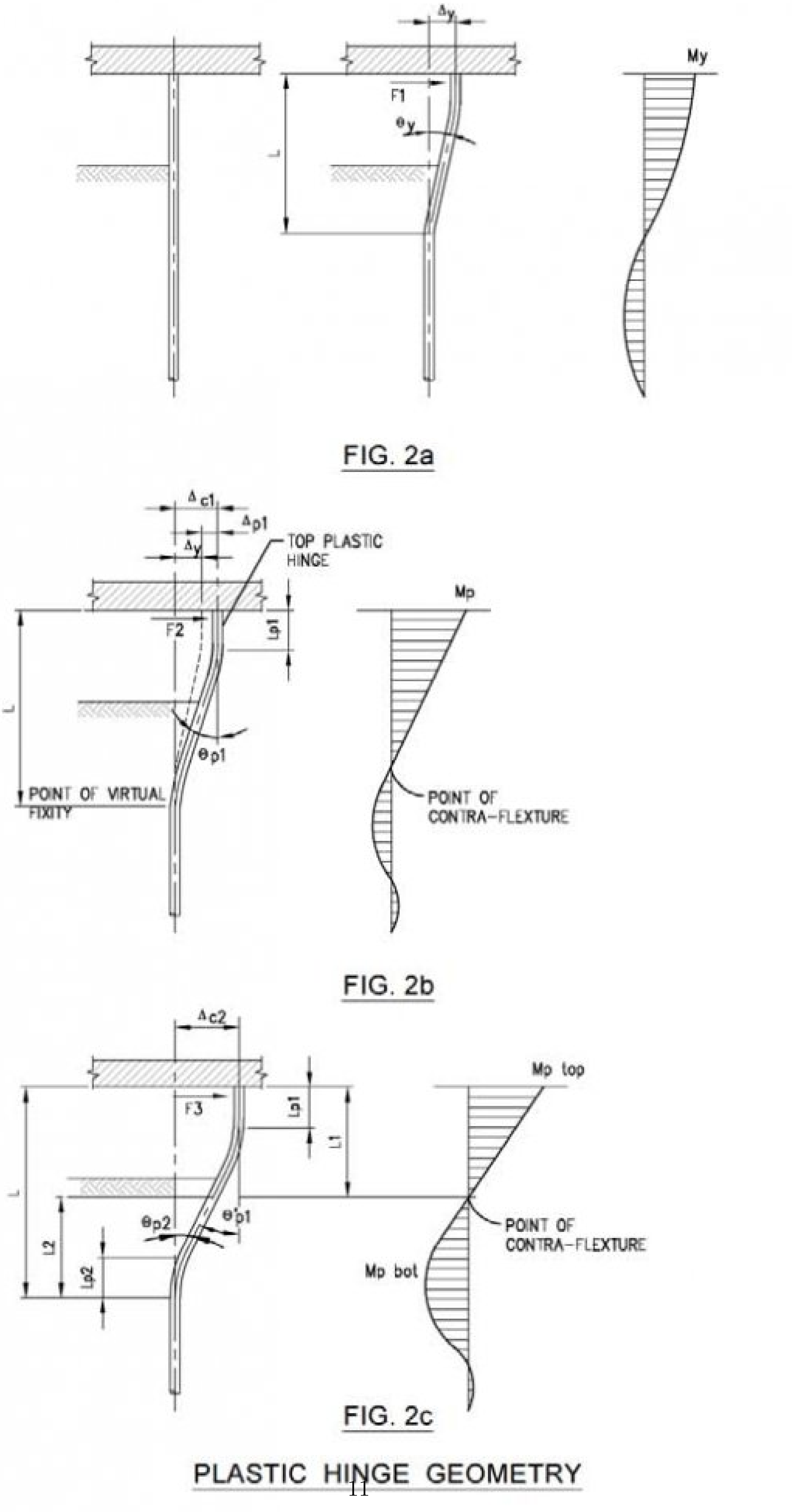
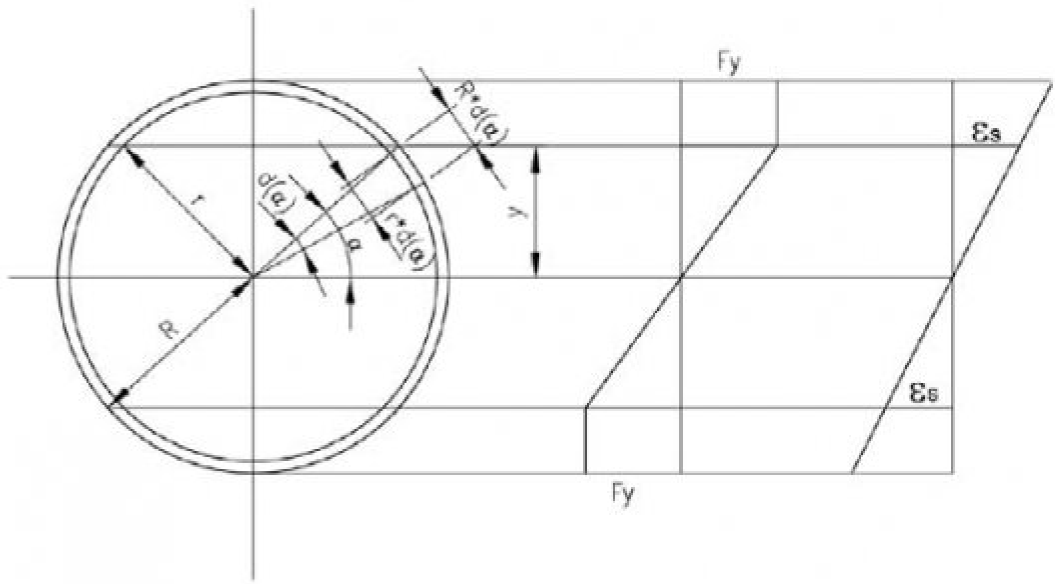

Δy
My
F1
θy
L
FIG. 2a
Δc1
Δp1
Δy
TOP PLASTIC HINGE
Mp
F2
Lp1
L
θp1
POINT OF VIRTUAL FIXITY
POINT OF CONTRA-FLEXTURE
FIG. 2b
Δc2
Mp top
F3
Lp1
L1
L
L2
θp2
θ'p1
POINT OF CONTRA-FLEXTURE
Mp bot
Lp2
FIG. 2c
PLASTIC HINGE GEOMETRY

Figure 4:



PIPE PILE PLASTIFICATION

FIG. 3

7

Figure 5: Figure 7 :I

PLASTISIZED PIPE SECTION

R =	744	mm	=O.D/2	I_{eff} = 1/4 * (R+r)^3 * t * [0.5 * a – 0.25 * sin2(a)]
r =	725	mm	=I.D./2	a_1 =0.25*t(R+r)
t =	19	mm	wall thickness	k_1 = [0.5 * a – 0.25 * sin2(a)] integration limits a to -a
F_y=	344	Mpa		I_{eff} = a_1 * k_1

ELASTIC PART OF THE PIPE SECTION

α (deg)	α (rad)	k_1	a_1= = 0.25*t*(R+r)^3	I_{eff} (mm4)	y (mm)	S_α (mm^3)	$M_{el} = F_y*S_\alpha$ (kN-m)
90	1.57	1.571		2.3653E+10	744	31,791,128	10,936
89	1.55	1.536		2.3127E+10	744	31,089,465	10,695
88	1.54	1.501		2.2602E+10	744	30,397,280	10,457
87	1.52	1.466		2.2077E+10	743	29,714,378	10,222
86	1.50	1.431		2.1554E+10	742	29,040,578	9,990
85	1.48	1.397		2.1031E+10	741	28,375,712	9,761
84	1.47	1.362		2.0510E+10	740	27,719,623	9,536
83	1.45	1.328		1.9992E+10	738	27,072,165	9,313
82	1.43	1.293		1.9475E+10	737	26,433,203	9,093
81	1.41	1.259		1.8961E+10	735	25,802,612	8,876
80	1.40	1.225		1.8450E+10	733	25,180,276	8,662

2

Figure 6: 2

10	0.17	0.004	53,046,115	129	410,592	141
9	0.16	0.003	38,715,419	116	332,643	114
8	0.14	0.002	27,219,258	104	262,874	90
7	0.12	0.001	18,251,451	91	201,294	69
6	0.10	0.001	11,502,733	78	147,909	51
5	0.09	0.000	6,661,137	65	102,726	35
4	0.07	0.000	3,412,373	52	65,750	23
3	0.05	0.000	1,440,209	39	36,987	13
2	0.03	0.000	426,859	26	16,440	6
1	0.02	0.000	53,367	13	4,110	1
0.001	0.00	0.000	0	0	0	0

Figure 7:

6

$Z_\alpha = -1.0*(R+r)2 *t *\cos(\alpha)$
$k_2=\cos(\alpha)$ integr limits $\pi/2$ to 0
$b = -1.0*t*(R+r)$^2

Figure 8: Note 6 :

PLASTISIZED PART OF THE PIPE SECTION

α (deg)	α (rad)	k_2	b= $-1.0*t*(R+r)^2$	Z_α (mm^3)	$M_{pl} = Fy*Z_\alpha$ (kN-m)
90	1.57	-1.000		41,001,259	14,104
89	1.55	-0.983		40,285,688	13,858
88	1.54	-0.965		39,570,336	13,612
87	1.52	-0.948		38,855,419	13,366
86	1.50	-0.930		38,141,156	13,121
85	1.48	-0.913		37,427,764	12,875
84	1.47	-0.895		36,715,460	12,630
83	1.45	-0.878		36,004,462	12,386
82	1.43	-0.861		35,294,987	12,141
81	1.41	-0.844		34,587,249	11,898
80	1.40	-0.826		33,881,465	11,655

Figure 9:

10	0.17	-0.015	622,901	214
9	0.16	-0.012	504,794	174
8	0.14	-0.010	399,021	137
7	0.12	-0.007	305,617	105
6	0.10	-0.005	224,609	77
5	0.09	-0.004	156,022	54
4	0.07	-0.002	99,877	34
3	0.05	-0.001	56,191	19
2	0.03	-0.001	24,977	9
1	0.02	0.000	6,245	2
0.001	0.00	0.000	0	0

Figure 10:

EXAMPLE:

Calculate maximum moment capacity of the partially plastisized section with (a) = 80 deg = 1.40 rad

α =	1.4	rad	plastification angle
M_{el} =	8,662	kN-m	@ α = 1.40
M_{pl} =	2,449	kN-m	$= \mathbf{M}_{pl\ @\pi/2} - \mathbf{M}_{pl\ @1.40}$
$M_{el\text{-}pl}$ =	11,111	kN-m	$= \Sigma(\mathbf{M}_{el} + \mathbf{M}_{pl})$
I eff =	1.8450E+10	mm^4	= I eff @ α =1.40

Figure 11:

1

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Figure 12: Table 1 :

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Figure 13: Table 2 :

3

Figure 14: Table 3 :

4

Figure 15: Table 4 :

VIII. Slope and Wharf Stability

Figure 16:

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Dedication: This article is dedicated to a memory of late Ron Joseph Mancini, P.E. of Mancini Forces in the wharf and wharf performance after projected liquefaction must be recalculated.

[California Department of Transportation Dynamic of Marine Structures: Methods of calculating the dynamic response of fixed st *California Department of Transportation Dynamic of Marine Structures: Methods of calculating the dynamic response of fixed structures subject to wave and current action*, November 2010. CALTRAN ; Atkins Research and Development. Ciria Underwater Engineering Group. London SW1P 3AU (Report UR8) (Seismic Design Criteria. Second Edition)

[Seismic Design of Pile -to-Pile Cap Connections in Flexible Pier Structures. Vitaly B. Feygin. Structure, SEI (2012)] *Seismic Design of Pile -to-Pile Cap Connections in Flexible Pier Structures. Vitaly B. Feygin. Structure, SEI*, March 2012.

[Port and Beach (2013)] *Wharf Design Criteria. POLB WDC Version 3.0*, Long Port , Beach . February 29, 2013.