SEISMIC PERFORMANCE ANALYSIS OF PILE-SUPPORTED WHARVES SUBJECTED TO LONG-DURATION GROUND MOTIONS

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Abstract

The impact of long-duration, design-level ground motions on the seismic performance of a pile-supported wharf has been evaluated using a practice-oriented 2D geomechanical model validated with case history data and supplemented with results from large-scale tests on representative pile-deck connections and pile-rockfill interaction. The modeling focuses on a wharf at Pier 400 Port of Los Angeles, the location of an extensive CGS SMIP strong motion instrumentation array. This paper provides a synthesis of modeling considerations and summary of the computational results for a subset of motions used in the investigation. The modeling has highlighted the impacts of pile kinematic loading due to foundation deformations associated with long-duration seismic loading. The phasing of inertial and kinematic loading on the pile foundations has been a primary consideration as well as approximate thresholds for pile damage due to displacement demand.

Introduction

This investigation addresses the effects of long-duration ground motions on the Soil-Foundation-Structure-Interaction (SFSI) and seismic performance of pile supported wharves in California. While pile supported wharves have been considered rather simple structures, dynamic SFSI of pile-supported wharves represent a complex geotechnical and structural interaction problem. The combination of inertial loading and kinematic effects due to seismically-induced ground displacement (i.e. displacement demand) imposes foundation loads that are commonly out-of-phase and quite variable depending on vertical and lateral location relative to the sloping face of terminal wharves. Observed failures to wharf foundations are often associated with geotechnical failures (liquefaction, cyclic degradation, slope instability). Field reconnaissance and inspection at ports after moderate to large earthquakes routinely finds that damage to waterfront structures and associated loss of operations are directly related to permanent ground deformation and large displacement demand on pile foundations, cutoff walls and anchor systems, and appurtenant structures (ASCE TCLEE 1998, PIANC WG34 2001, ASCE/COPRI 2014a).

In order to simulate the global movement of the waterfront slope, pile deformation, and possible wharf displacement in a coupled manner the 2D numerical dynamic SFSI modeling has been performed using the commercially available program FLAC, a geomechanical model wherein the soil profile is modeled as a continuum and the wharf, deck, and pile foundation are modeled using relatively simple structural elements. This program has been selected for application due to the wide usage in port engineering practice (e.g., Roth et al. 2003; Roth and Dawson 2003; Arulmoli et al. 2004; Dodds et al. 2004; Moriwaki et al. 2005, Yan et al., 2004) and the experience of the project team with this code for port and waterfront applications. The effort has included both the calibration of a practical dynamic SFSI modeling procedure and the application of the validated model for evaluating the impact of long-duration motions on the seismic performance of a modern wharf structure.

The adoption of performance-based seismic design provisions at major ports and marine oil terminals in California necessitates the reliance in engineering practice on numerical models for simulating dynamic SFSI of wharf and embankment structures. Recent investigations of the seismic performance of pile supported wharves have developed enhanced methods of analysis (e.g.; Chiaramonte et al., 2011; Shafieezadeh et al. 2012); however, the lack of well-documented, instrumented field case histories has precluded thorough validation of analysis methods for simulating dynamic SFSI of these structures. Berth 404 at Pier 400, Port of Los Angeles provides an extremely valuable test bed for this investigation. The wharf represents recent design and construction practices, and constitutes a very important terminal at the port. The wharf and embankment configuration is similar to other major terminals at the Port of Los Angeles and the adjoining Port of Long Beach, yet the Pier 400 site is particularly valuable due to the following;

1. Extensive CSMIP strong motion array along a portion of the wharf, as shown in the Figure 1. CSMIP stations #14284 and #14256 provide 3 free-field and 15 structural accelerometers, respectively.
2. The type and configuration of the piles (24" octagonal prestressed concrete piles; seven piles per bent) are consistent with contemporary port design in California.
3. Large-scale structural modeling of representative pile-deck connections has been performed (Restrepo et al., 2007; Krier et al., 2008). The Force-Displacement and Moment-Rotation behavior of the pile-wharf deck connection has been very well characterized.
4. Large-scale modeling of pile-rockfill interaction (Kawamata, 2009) has been performed for conditions very similar to that at Pier 400. This work has provided very useful data for Force-Displacement and p-y behavior of piles in rockfill.
5. A geophysical investigation (MASW, ReMi) performed as a portion of this investigation has provided shear wave velocity profiles through zones of unimproved hydraulically-placed backland fill and in zones of fill treated with stone columns (Dickenson et al, 2013).
6. An extensive regional PSHA and port-wide ground motion characterization has been completed (EMI, 2006).
7. Ground motion data has been obtained at the instrumentation array for short-duration, weak to moderate motions recorded during the M_W 4.7 May 17, 2009 Inglewood Area Earthquake (PGA ≈ 0.10g in free-field and ≈ 0.20g on the edge of the wharf deck). These motions have been useful for validation of elastic properties used in FLAC.

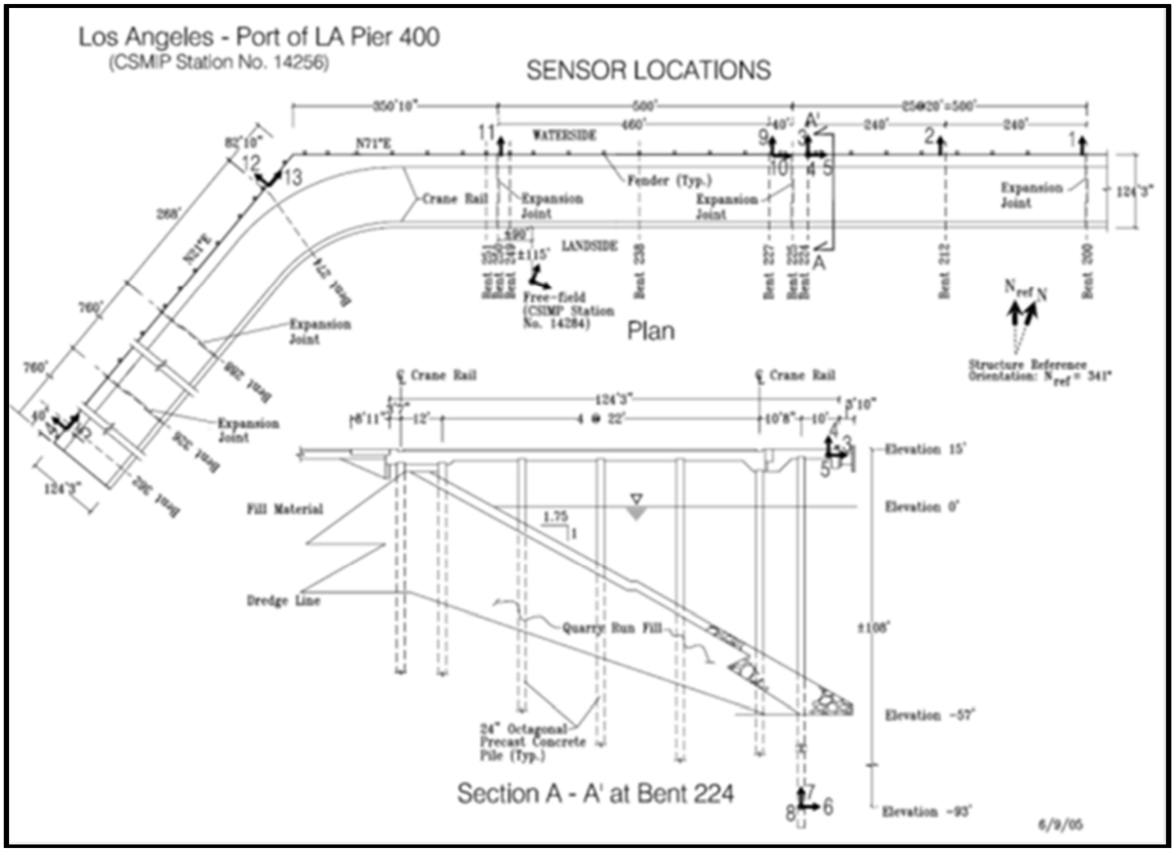

Figure 1: CSMIP Instrumentation Array at the Port of Los Angeles Pier 400 (CGS - CSMIP Station 14256), Center for Engineering Strong Motion Data (CESMD)

In order to validate the numerical model a major effort has been undertaken to collect supporting information and data required for robust nonlinear SFSI modeling. This has included; port reports on geotechnical site characterization, dynamic soil properties, geotechnical interpretation and design (Fugro West 2001a, b, c), structural seismic design and detailing (Priestley 2000, Weismair et al 2001), construction materials and methods (Degen et al. 2005, Fugro West 2004), as-built drawings (POLA 2002), and large-scale physical model testing of pile-wharf deck connections (Krier et al. 2008, Lehman et al. 2013, Restrepo et al. 2007) and pile-rockfill interaction (Kawamata 2009). The first phase of this project focused on the synthesis of geotechnical, structural, and strong motion data at three port sites instrumented by CSMIP. Background on the initial efforts and model validation has been presented by Dickenson and others (2013). A summary of pertinent aspects of the site characterization and considerations for modeling of Berth 404 are provided as follows.

Geotechnical Site Characterization

The geologic cross section and structural configuration at Berth 404 are provided in Figure 2. As defined by Fugro West (2001a, b, c); from youngest to oldest the soil profile consists of;

1. Hydraulic fill consisting of predominantly silty sand, with layers of sandy silt and silt with clay balls. The construction sequence associated with dredging, characteristics of fill based on borrow area, and the influence of placement techniques on density are addressed by Fugro West (2001a, b) and Foxworthy et al. (1998). A review of post-construction

boring logs in the area adjacent to the strong motion arrays at Berth 404 indicates that the SPT penetration resistances of the sand portions of the fill vary with location due to the cumulative influence of; fines content, method of placement, and deposition above or below water level). In the unimproved fill the 33-percentile $(N_1)_{60}$ above the water level (elevation 15 ft to 0 ft) is roughly 23 blows/ft, while the corresponding value below the water level (elevation 0 ft to -34 ft) is 13 blows/ft, indicative of sand vulnerable to liquefiable at design level ground motions.

2. A thin layer of soft harbor bottom sediments (Unit 1 – Harbor Bottom Sediments).
3. An approximately 15- to 35-ft thick layer of generally fine sand and find sand with silt of alluvial deposition (Unit 2 – Younger Channel Sands).
4. An approximately 15- to 20-ft thick layer of sand with silt or silty fine sand of marine deposition (Unit 3 – Marine Sands).
5. A 30- to 35-ft (maximum) thick sequence of paleochannel infill (Unit 4 – Older Paleochannel Infill) composed of very silty fine sand (Unit 4a) overlying silt and clayey silt.
6. A thick, highly layered (sands, silts, clays) sequence of transgressive marine deposits (Unit 7 – Undifferentiated Deposits).
7. An 80- to 100-ft thick sequence of alluvial fine to medium sand with gravel (Unit 8 – Older Allluvial Deposits) that correlates with the onshore Gaspur Aquifer.

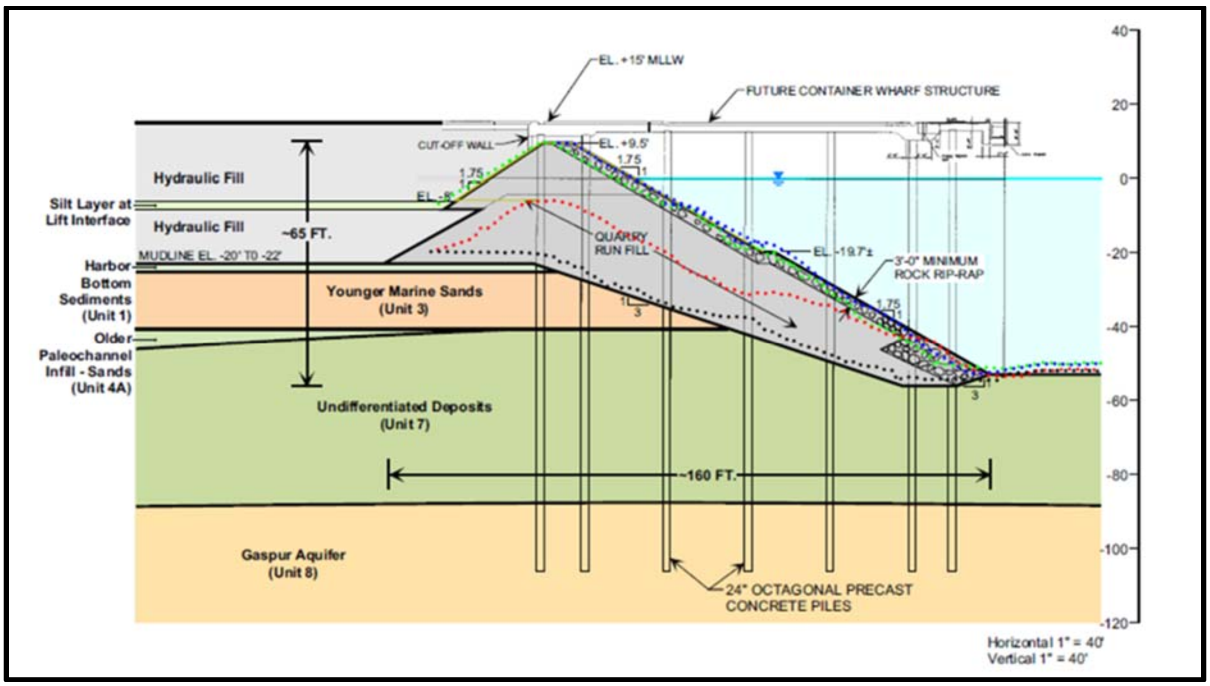

Figure 2: Geologic and structural section at Berth 404, Pier 400, Port of Los Angeles Los Angeles (Fugro West, 2004).

Geophysical Investigation at Berth 404

The research team, in partnership with POLA, CSMIP, and GEOVision, conducted a geophysical investigation using active and passive surface wave techniques (MASW, SASW, and ReMi) to develop the V_s profile across Berth 404 and in close proximity to the CSMIP free-field strong motion instrument station 14284. The geophysical survey provided useful data for seismic site characterization ($(V_s)_{30} \approx 207$ m/sec) and provided an opportunity to evaluate the V_s profiles through both unimproved fill and zones of fill treated with stone columns. The latter was considered a worthwhile effort for measuring "composite" low-strain behavior of the treated soil mass. The orientation and configuration of the surface wave arrays and results are provided by Dickenson and others (2013).

The results of the surface wave investigation are plotted in Figure 3. The agreement in the V_s profiles through native soils beneath the hydraulic fill layers is very good. The V_s trends in the unimproved and improved fill are highlighted in Figure 9a. As expected the "composite" V_s values are greater in the zone of treated soil, although the difference in the values is only roughly 7% to 12%. It is noted that the ground treatment was implemented in 2 zones adjacent to the waterfront each zone having a different spacing of stone columns and Area Replacement Ratio (ARR). Based on post-construction documentation the approximate average ARR values in the two zones were 14% and 18%, although the diameter of the stone columns was noted to change significantly between the sandy fill and layers of silt-rich soil (Degen et a 2005; Fugro West 2004).

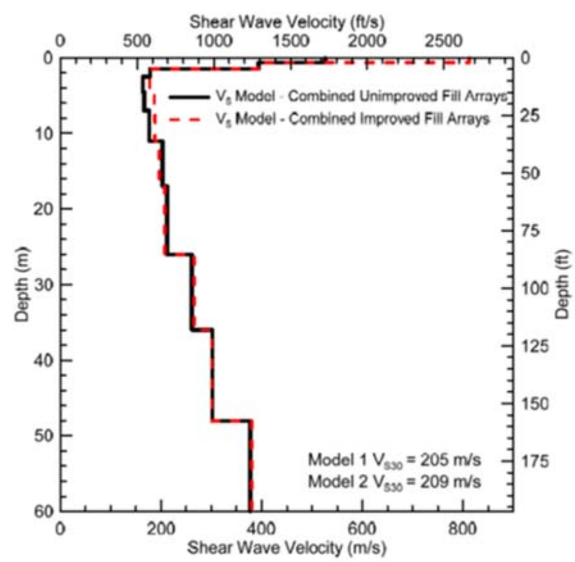

Figure 3: Comparison of shear wave velocity profiles across the Berth 404 site (GEOVision 2013).

Overview of Modeling Parameters and Considerations

The 2D FLAC model has been used for nonlinear, coupled effective stress modeling. The stress-strain behavior of all soil units except for the submerged, untreated sand fill have been modeled using the Mohr-Coulomb constitutive model. The saturated, loose to medium dense sand fill layers (i.e., FILL 3 and FILL 4) have been modeled using the UBCsand model (Beaty 2009). The strength and low-strain stiffness values are provided in Table 1. Several modeling notes are provided as background;

1. The rockfill (quarry run and armor/rip-rap) have been modeled using a stress-dependent friction angle, which provides greater shearing resistance at low confining stress.
2. It is acknowledged that modeling large particles in rockfill dikes, rubble mound structures, and sloping armor layers using the Mohr-Coulomb relationship in a geomechanical continuum model has potentially significant limitations. Issues related to particle size to layer thickness, strength at low confining stress, and the interlocking behavior of the rockfill mass are not well replicated with the simple Mohr-Coulomb model. A practice-oriented model that accounts for the interlocking behavior of rockfill has been developed (Kawamata 2009); however, this material has been modeled using a simplification wherein interlocking is approximated using an artificial, or "pseudo-cohesion" as described by Martin (2005) and Dickenson and McCullough (2006). The shearing resistance of the rockfill has therefore been modeled with both the stress-dependent friction angle and the pseudo-cohesion. The influence of the pseudi-cohesion decreases rapidly with depth as the frictional strength of the rockfill dominates the mass behavior.
3. The stress-strain-strength behavior of the sand fill has been modeled on the basis of the 33-percentile $(N_1)_{60}$ values obtained for post-construction conditions. The percentile value was selected with consideration of mass behavior, generation of excess pore pressure in the submerged, untreated fill, and the variability in N-values (also CPT Qc trends) observed at Berth 404.
4. The static undrained shear strength of the fine-grained soil units was estimated using the stress-normalized relationship for loading in Direct Simple Shear;

   $$(Su)_{DSS} = 0.25 \text{ x } (\sigma_v') \text{ x } (OCR)^{0.8}$$

   The cyclic shearing resistance mobilized during seismic loading was increased to account for rate effects, as demonstrated in large-scale centrifuge tests by Brandenberg and others (2014). A multiplier of 1.35 was applied to account for rate effects.
5. The zones of sand fill treated with stone columns were modeled using a simple, mass "average" friction value and low-strain stiffness obtained in the surface wave geophysical investigation (with the geophone arrays aligned longitudinally along the zone of improved soil). It is acknowledged that the 2D continuum model cannot replicate the 3D nature of the interaction of stone columns in the layered sand and silt (Rayamajhi et al 2013). The influence of this approximation on the global embankment-wharf behavior can be evaluated by way of sensitivity analyses.

Table 1: General soil properties for modeling

UNIT	φ' (deg)	c' (psf)	S_u (psf)	$(V_s)_{avg}$ (fps)
A.C. Paving & C.M.B.	45	0	n/a	1725 to 2670
Compacted subgrade	45	0	n/a	1290
FILL 1: Emergent sand fill (untreated)	34	0	n/a	560
FILL 1: Emergent sand fill (treated)	40	0	n/a	600
FILL 2: Fill lift interface silt	n/a	n/a	[a]	545
FILL 3: Submerged sand fill (intermediate)	38	0	n/a	545
FILL 4: Submerged sand fill (base)	38	0	n/a	580
Quarry Run Fill (rock)	52[b]	250[c]	n/a	855
Armor/Rip-Rap (rock)	52[b]	250[c]	n/a	715
UNIT 1: Harbor bottom sediment (silt)	n/a	n/a	[a]	600
UNIT 3: Marine sands	38	0	n/a	666
UNIT 4A: Paleochannel fill sand	38	0	n/a	680
UNIT 7: Undifferentiated deposits (fine-grained)	n/a	n/a	[a]	696 to 776
UNIT 8: Gasper aquifer	40	0	n/a	865 to 1120

[a] Su (DSS) = $(1.35)(0.25)(\sigma'_v)(OCR)^{0.8}$

[b] Stress-dependent friction angle (Charles and Watts 1980, Duncan 2004)

[c] Interlocking behavior of rockfill approximated with a "pseudo-cohesion" (Martin 2005, Dickenson and McCullough 2006)

The near-surface, lateral pile-soil response reflects the characteristics of the piles, nature of the inertial loading provided by the wharf deck and contributing loads, the embankment slope, and the nature of the soil and/or rock fill along the upper portion of the pile. Pile embedment through rock armor layers and quarry run fill presents issues related to particle size effects on pile-soil p-y behavior. Physical modeling studies of piles in rock fill have demonstrated the limitations of continuum models for lateral pile response (Boland et al 2001a, 2001b; McCullough 2003; Kawamata 2009). Two straightforward methods have been applied to model the dynamic p-y behavior of piles embedded in rockfill:

1. Kawamata (2009) describes the use of a simple constitutive relationship for the interlocking behavior of rockfill. The method has been calibrated in large-scale tests of instrumented piles in rockfill with very good agreement between computed and observed pile behavior (pile head deflection versus load and trends with depth of pile rotation, deflection, and curvature). The results of the investigation also demonstrated the applicability of practice oriented procedures for developing p-y curves with empirical adjustment. For the piles and rockfill used (POLA Pier 400 simulation) it appears that the use of the API procedures with $\varphi = 38°$ and a p-multiplier of 2.0 to 3.0 provides results that are considered worthwhile for practical applications.
2. A simple, practice-oriented procedure for modeling pile response in rockfill includes a nominal "pseudo-cohesion" for the rock to account for the individual rock particle interaction with the pile elements (McCullough 2003; Martin 2005; Dickenson and McCullough 2006). The approximation of $\varphi = 45°$ with an artificial cohesion of 200 to 300 psf provided reasonably good agreement for trends of pile bending moment and deflection with depth. The p-y behavior of the piles (i.e. normal spring stiffness) was modeled using the "pseudo-cohesion" procedure.

The structural detailing of the 24-inch octagonal concrete piles used at Berth 404 varies with row. The common terminology used to describe the piles is "seismic pile" and "non-seismic pile" based on the ductility demand imposed during seismic loading and the necessary design detail for the piles and pile-deck connections. The concept is illustrated in Figure 4 for the wharf configuration at POLA Pier 400. Large scale structural testing of the pile-deck connection (Restrepo et al. 2007, Krier et al. 2008) has been extremely useful for modeling the force-displacement and moment capacity of the piles. The results of a lateral load test on a "seismic pile" are provided in Figure 5. The moment capacity of the piles have been modeled on the basis of the large-scale testing test as; 550 k/ft for seismic piles in Rows F & G, and 400 k/ft for non-seismic piles in Rows A to E.

The mass of the gantry cranes has been incorporated in the modeling. While this investigation has not focused on the dynamic response characteristics of the crane a range of dynamic loads on the wharf representing the cranes has been evaluated. The range of loads used reflects the weight and dimensions of the cranes, as well as the number of cranes used along the wharf during operations. For example, at the time of the 2009 Inglewood Earthquake there were seven gantry cranes working along the vessel Columbine Maersk (approx. 1,200 ft long), which was at Berth 404. The Noell gantry cranes operating at Berth 404 weigh approximately 2,700 kips each and have a width between trolleys of 82 ft. A line load on each rail of roughly 17 k/ft represents a single crane. Alternatively, the wharf has been built with expansion joints spaced at 500-ft intervals. Three cranes can operate along this length of wharf therefore an equivalent line load of roughly 8 k/ft is applied on each rail. Five percent of the crane mass is added to the wharf in the dynamic analysis following the provisions of the POLA seismic code (2010).

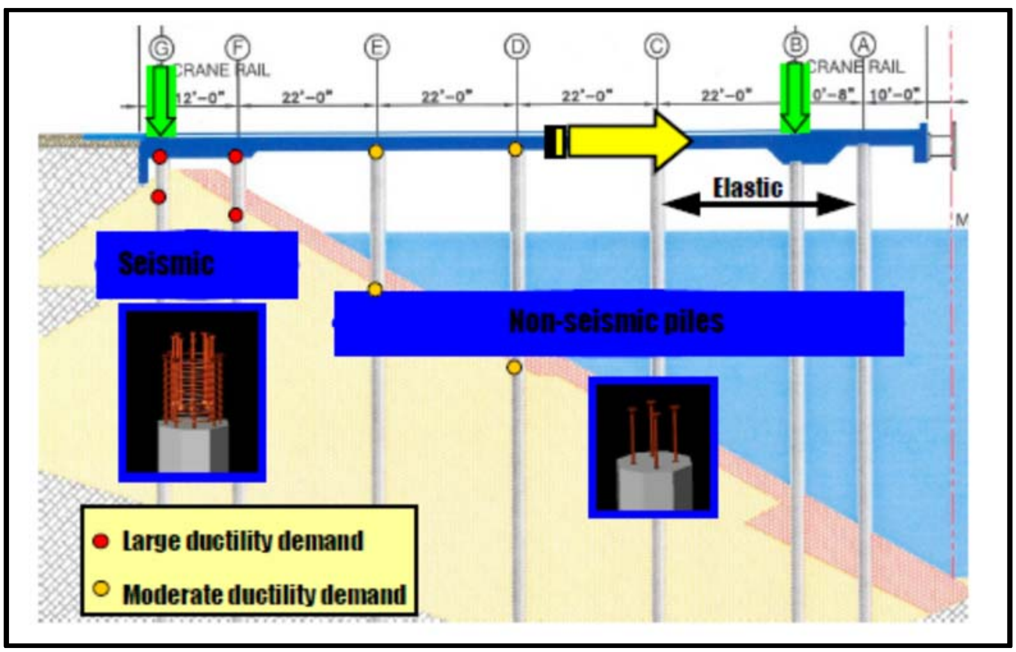

Figure 4: Schematic illustration of a pile-supported container wharf at POLA Pier 400 showing pile types and potential plastic hinge locations (Restrepo et al., 2007).

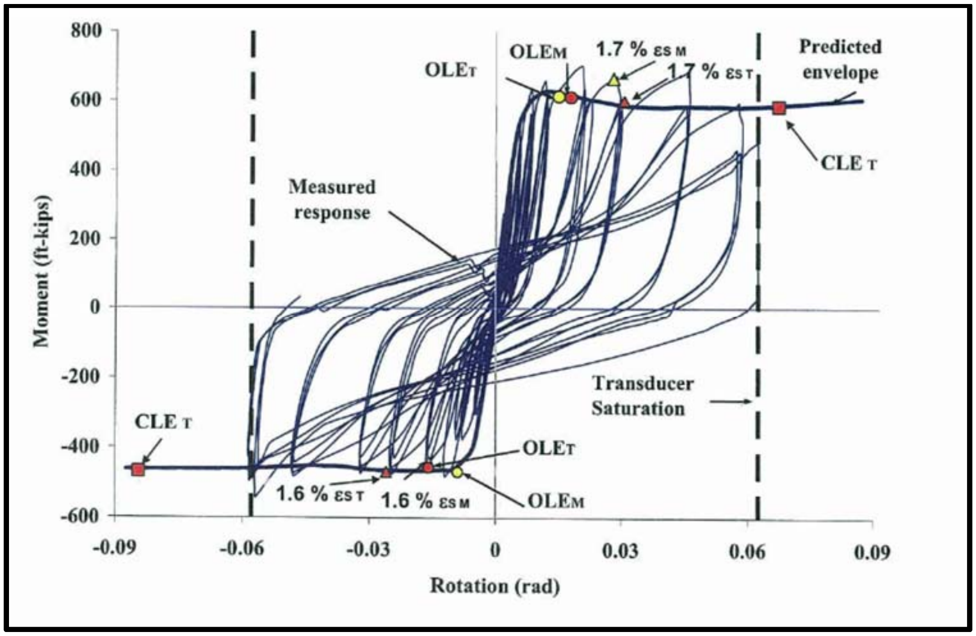

Figure 5: Moment-rotation hysteretic response for a test specimen of a "seismic pile" at POLA Pier 400 (Krier et al. 2008).

Ground Motions

The ground motions used in this investigation reflect the multi-level seismic design requirements provided in the POLA seismic code (2010) and recommendations presented in the "Port-Wide Ground Motion and Palos Verdes Fault Study, Port of Los Angeles, California" (EMI 2006). Three levels of site-specific ground motions are determined for the design of wharf structures as defined in Table 2.

Table 2: Basis for Ground Motions used in Design at the Port of Los Angeles

Earthquake	Probability of Exceedance	Return Period (years)	PGA (g)[a]	Magnitude[c]
Operating Level Earthquake (OLE)	50% in 50 years	72	0.23	6.5
Contingency Level Earthquake (CLE)	10% in 50 years	475	0.52	7.0
Design Earthquake Level (DE)	"Design Earthquake" as defined in ASCE 7-05 Section 11.2[b]			

a Peak Ground Acceleration for "firm ground condition" having Vs = 1,000 ft/sec.

b Refer also to ASCE/COPRI Standard 61-14 Seismic Design of Piers and Wharves (2014).

c Magnitude of the dominant source identified in PSHA deaggregation.

The port-wide ground motion investigation included the development of a collection of ground motions for the firm base condition (V_S 1,000 ft/sec) spectrally-matched to the OLE and CLE Uniform Hazard Spectra. The UHS for various return periods is provided in Figure 6. This investigation has focused on the OLE and CLE ground motion levels, consistent with port design requirements. The modeling proceeded with baseline analyses performed for lower ground motions levels, specifically;

1. Ground motions recorded at, and in proximity, to the Berth 404 array during the M_w 4.7 May 17, 2009 Inglewood Area Earthquake, and
2. One OLE motion from the EMI (2006) collection (Set 5, 1979 Imperial Valley Earthquake, Calexico Fire Station, Fault Normal).

Subsequent analyses were then performed using a collection of the motion spectrally-matched to the CLE UHS (Figure 6). This included motions from the EMI investigation (2006) and long-duration motions spectrally-matched to the CLE UHS. For the sake of brevity this paper summarizes the results of the modeling for a subset of the motions used in the investigation. The time histories include;

3. Two CLE motions from the EMI (2006) collection (Set 1, 1999 Hector Mine Earthquake, Hector Station, Fault Normal; Set 4, 1999 Duzce Earthquake, Lamont 1059 Station, Fault Normal).
4. Two long-duration motions obtained from subduction zone sources (1985 Michoacan Earthquake, La Union Station, E-W component; 2011 Tohoku Earthquake, TCG005 Station, E-W component).

It must be noted that the subduction zone motions are being used as "seed motions" for evaluating the influence of long-duration motions on the test bed wharf structure (i.e., representative of design and construction at the Port of Los Angeles). Although the long-duration motions have been spectrally-matched to the CLE UHS they are not considered representative of ground motions associated with the regional seismic hazard due to their Significant Durations (T_{95} –T_5) and Arias Intensities (La Union 19.6 sec and 8.9 ft/sec; TCG005 70.8 sec and 36.1 ft/sec, respectively). These analyses are therefore conducted to provide an "index" of possible performance at various ground motion levels and not intended to be indicative of predicted performance during an earthquake generating motions at, or exceeding, the CLE at the Port of Los Angeles. Again, the motions are being used to support the primary research goals of; examining the influence of long-duration motions on inertial and kinematic loading, and evaluating the relationship between ground motion characteristics across a broad range of motions with damage thresholds for a pile supported wharf.

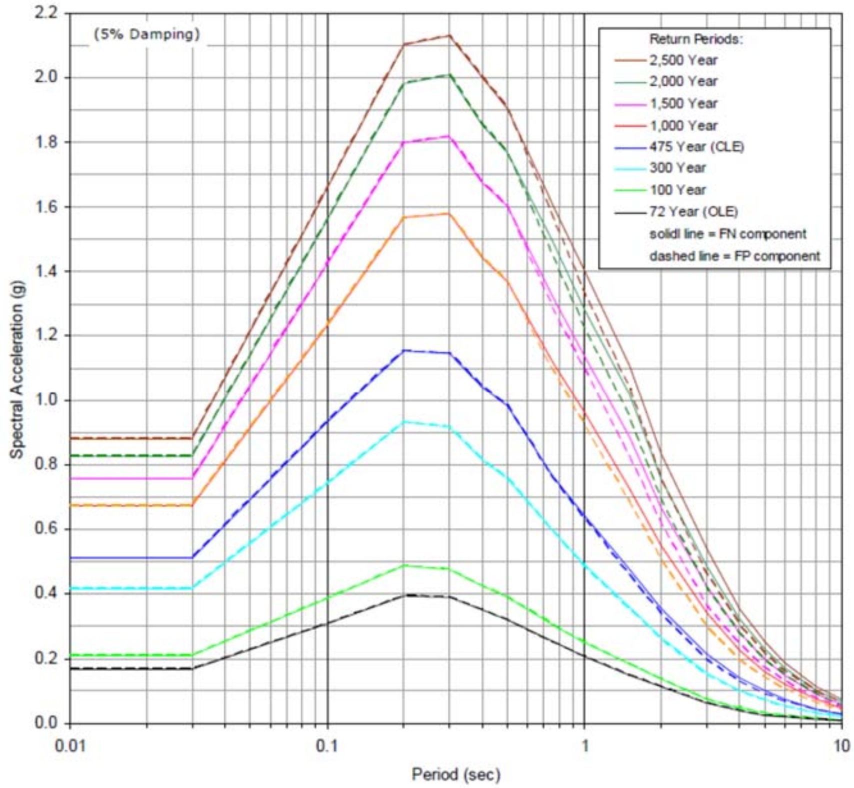

Figure 6: Comparison of firm-ground UHS for various return periods (EMI 2006).

Summary of Modeling Results

The FLAC model geometry is provided in Figure 7. The baseline analyses at lower ground motion levels provided very useful calibration of the model. The OLE analysis resulted a maximum ground displacement of 0.7 ft, with equivalent displacement of the piles and wharf deck. No plastic hinge development in any of the piles was indicated in the analysis.

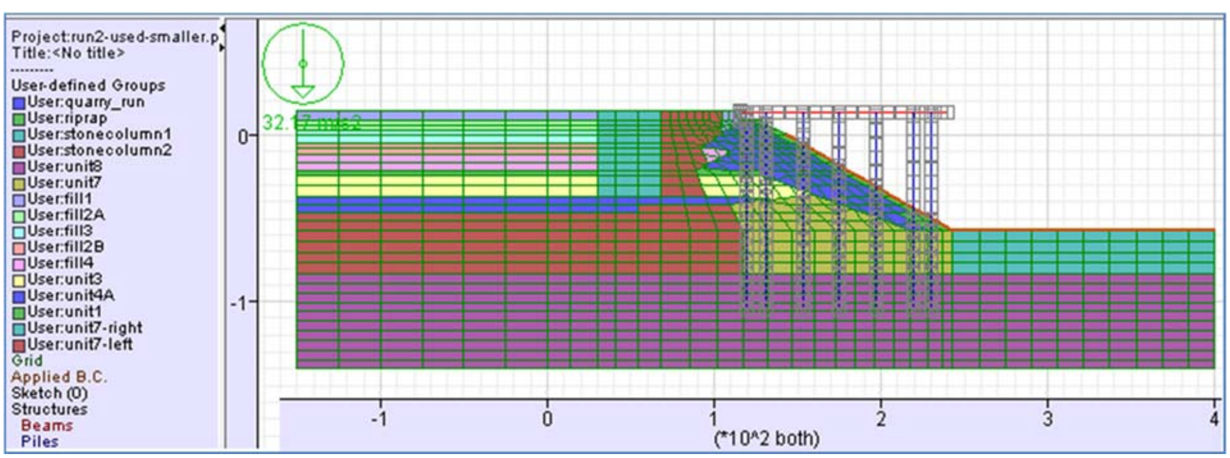

Figure 7: FLAC model geometry for POLA Berth 404 (Pier 400).

The analyses performed for CLE motions resulted in permanent deformations of the rock dike and wharf ranging from 2.3 ft to 3.3 ft. The computed ground deformation pattern is illustrated in Figure 8. The slope movement is largely associated with shear band development in the undifferentiated fine-grained deposits (Unit 7) underlying the rock fill. The localization of deformation at the top and bottom of Unit 7 has resulted in greater displacement demand within several pile diameters of the interfaces and the formation of plastic hinge development at these elevations, as illustrated in Figure 9. The location and extent of the pile hinge development was similar for both CLE analyses. The timing of the pile hinge development can be most directly linked to the concurrent, progressive increase in permanent horizontal displacement. These trends are plotted in Figure 10. The displacement time histories (Figure 10b) of two soil nodes correspond to; (i) a node at ground surface and approximately 30 ft behind the stone column improved zone, and (ii) a soil node located approximately 25 ft below the ground surface and directly behind the rock dike.

It is important to note that the pile moment development adjacent to the pile-deck connection is related to both the inertial loading and the displacement demand (pile head rotation), although this two components of loading do not act in phase with each other. The computational results demonstrate that the progressive increase in moment is closely related to the cumulative increase in the horizontal soil movement. The relative influence of the transient inertial loading and cumulative soil displacement for pile moments in the Row G "seismic pile" is illustrated in Figure 11.

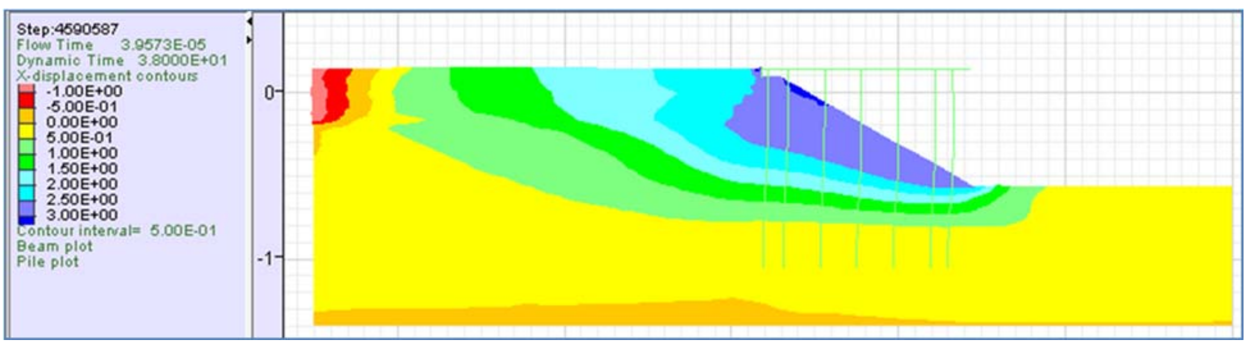

Figure 8: Horizontal displacement contour at the end of shaking using the CLE Set 1 motion (Units of ft).

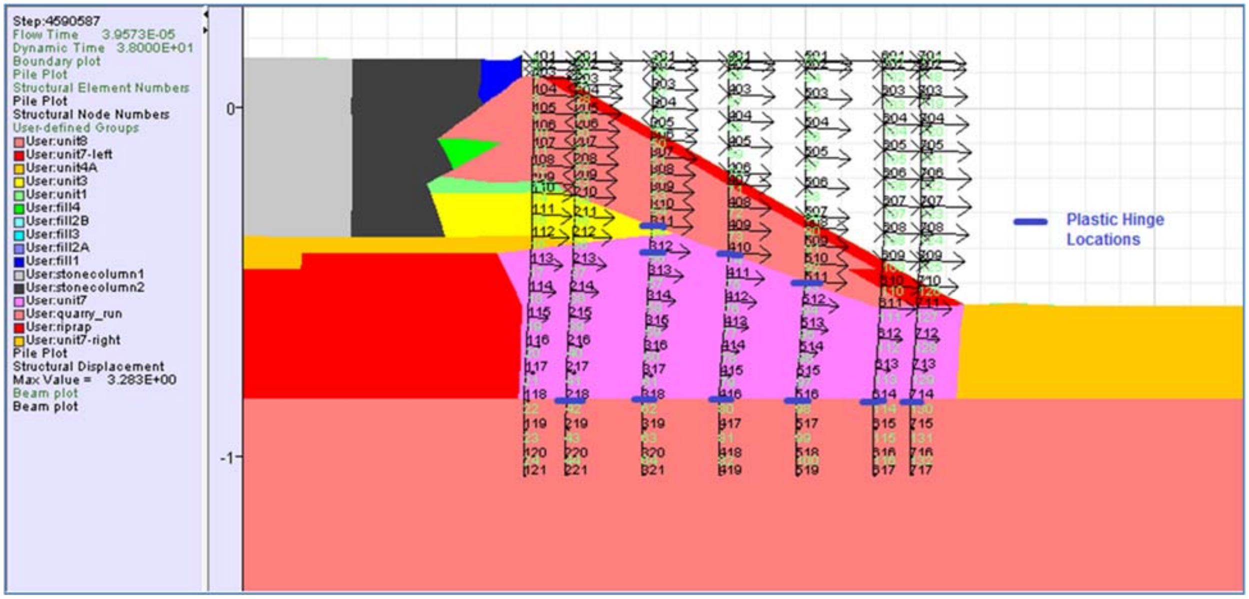

Figure 9: Location of plastic hinges in piles due to CLE motions (note: the black numbers represent the pile node numbers, and light green numbers represent the pile element numbers).

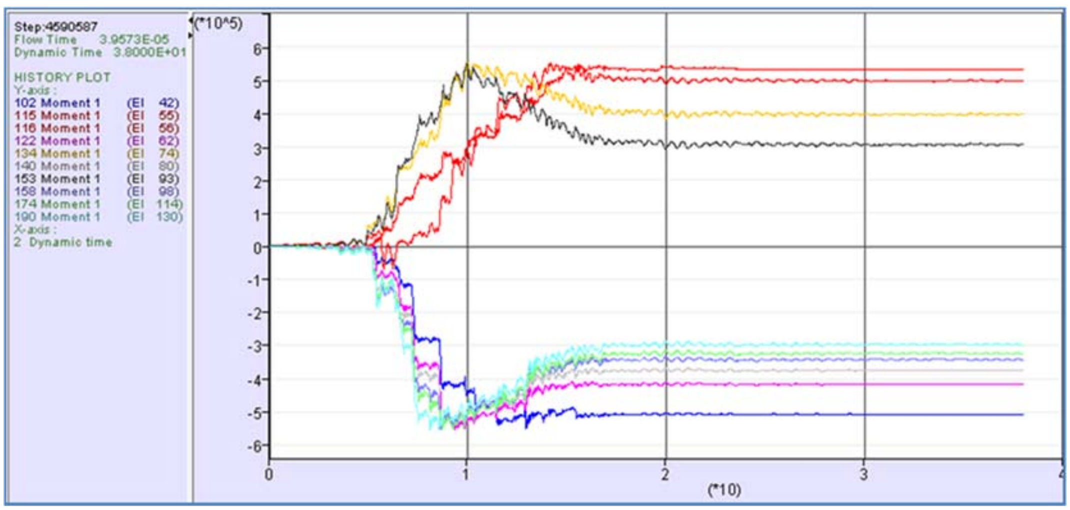

Figure 10a: Moment time history illustrating the occurrence of the plastic hinges in piles during a CLE motion. (X-coordinate: Second; Y-coordinate: Moment in lb-ft)

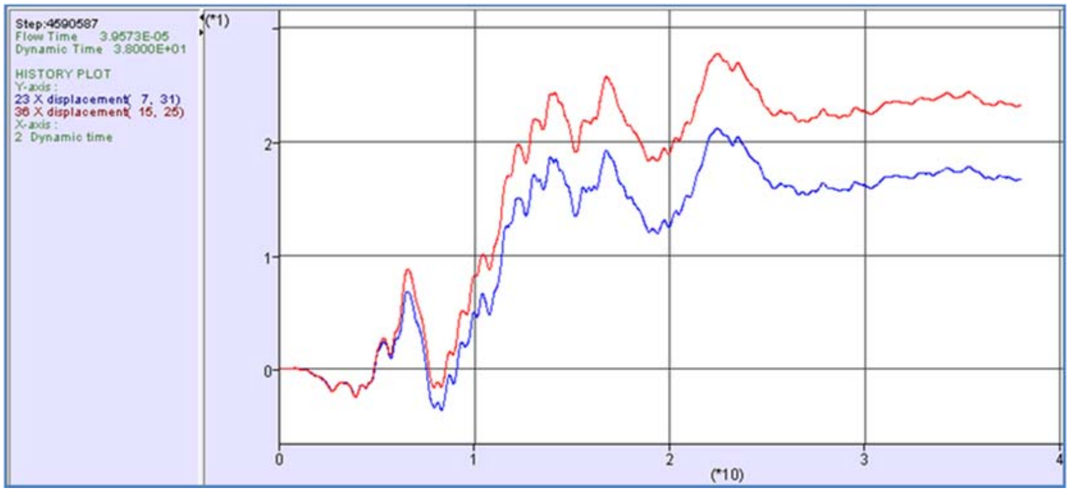

Figure 10b: Time history of horizontal soil displacement due to a CLE motion. (X-coordinate: Second; Y-coordinate: ft)

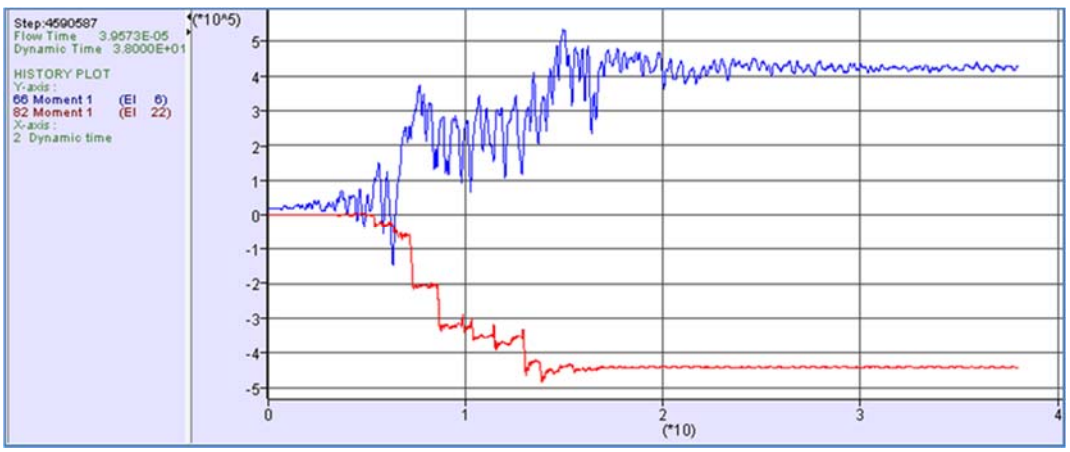

Figure 11: Time history of pile moment in Row G at the pile-deck connection and at the bottom elevation of Unit 7. (Units: ft-lb, sec; blue line is moment at pile-deck connection; red line is moment at bottom elevation of Unit 7)

The analyses performed for long-duration motions (spectrally-matched to the CLE UHS) resulted in permanent deformations of the rock dike ranging from 3.3 ft to 7.0 ft, for the La Union and TCG005 motions, respectively. At 7 ft of displacement the analysis terminated due to excessive deformation and numerical instability. In both models the permanent pile and deck displacement was roughly 85% to 90% that of the global slope movement. The moment development, extent and location of plastic hinges, and permanent deformations computed using the La Union time history was quite similar to the results of the CLE Set 1 analysis. In this case the relatively small increase in Significant Duration and Arias Intensity was not sufficient to induce additional ground deformations and pile damage. Conversely, the TCG005 motion (which is considered an extreme case) resulted in considerably more damage to the piles. It is interesting to note that much of the damage was experienced in the "non-seismic" piles due to ground deformation at layer interfaces, and that the unsupported lengths of the piles remain undamaged.

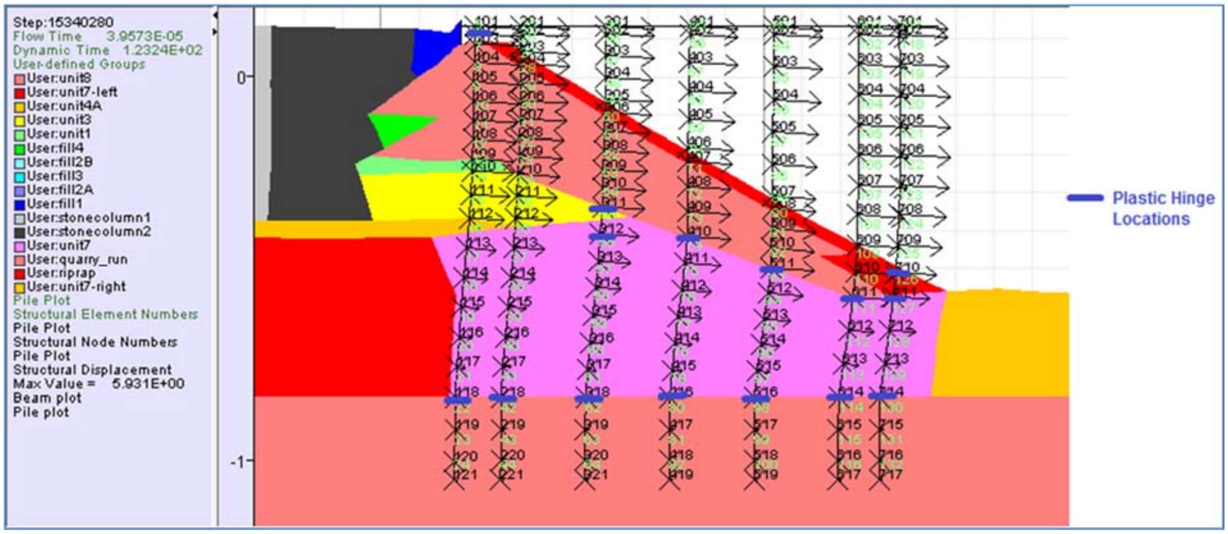

Figure 12: Location of plastic hinges in piles due to a long-duration, CLE motion.

Conclusions

This project is examining the effectiveness of current seismic design codes and performance-based provisions (ASCE/COPRI Standard 61-14, 2014; CSLC MOTEMS, 2010; POLA, 2010; POLB, 2012) for achieving the defined performance requirements for large magnitude earthquakes that generate long-duration ground motions. The topic is important because: (a) recent experience demonstrates that loss of serviceability at port terminals is strongly correlated with permanent ground deformations, and (b) long-duration ground motions have much greater potential for generating damaging wharf and embankment deformations at lower force levels relative to stronger, but brief, seismic loading. Several practical observations and considerations have been made regarding 2D numerical modeling of wharf structures, including:

1. The 2D geomechanical model, using practice-oriented procedures and approximations, have been demonstrated to provide representative seismically-induced permanent deformations, accelerations, and excess pore pressure generation for the low- to moderate-levels of shaking experienced in the validations performed in this investigation (Dickenson et al. 2013).
2. Pertinent aspects of the cyclic lateral behavior of piles in sloping rock fill can be well modeled using a 2D continuum model provided that following considerations are made;
   a. The interlocking nature of the rock fill is accounted for in the model.
   b. The influence of rock fill size on lateral pile behavior (scale effects) is modeled.
   c. The difference between upslope and downslope SSI spring stiffness (p-y behavior) is accounted for in the model.
3. Specific aspects of analysis that warrant consideration for long-duration motions include; (a) fatigue, plastic hinge development, and hinge softening models for the post-yield hysteretic behavior in both the “seismic” and “non-seismic” piles, (b) stress concentrations at pile-wharf deck connection, and (c) patterns of deformation in the rockfill embankment and foundations soils.

4. It is clear that large pile moments develop at depth due to permanent deformation even at moderate soil displacement (≈ 1.0 ft). These pile moments are only predicted through the use of analysis methods that have the capability to model the global wharf-embankment-foundation system.
5. As has been demonstrated in numerous applications involving pile foundations the relative contributions of the inertial loading and kinematic loading to the total demand on structural elements is a complex function of; the characteristics of the time history, timing and pattern of ground deformations, and structural response characteristics. These two primary modes of loading are not in phase and attempts to assign weighting factors to determine the total load should be used with great caution.

The following limitations in the application of 2D continuum models and avenues for continuing investigation have been identified;

1. Limitations in structural characterization due to the pile segment length in the model and plastic hinge length required for assessing curvature and loads at the pile-deck conection.
2. Accurate characterization of pile-deck connection in the model. The model segment should optimally be a fraction of the pile diameter, which is computationally difficult for the continuum model.
3. Hysteretic models should incorporate both; (i) strength and stiffness deterioration of the piles, and (i) pile-soil interaction at large deflections.
4. The data from the surface wave investigation is being evaluated to determine the possible influence of geophysical modeling assumptions (plane waves) and 3D nature of the stone column improvement on the “composite” V_s values provide. This is a valuable data set that is also being used to assess strain-compatibility concepts as applied for the dynamic behavior of stone column treated soils. Modeling the mass behavior in 2D plane-strain of a zone of sand treated with vibrocompaction and or stone columns requires gross approximation that warrants additional refinement.

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