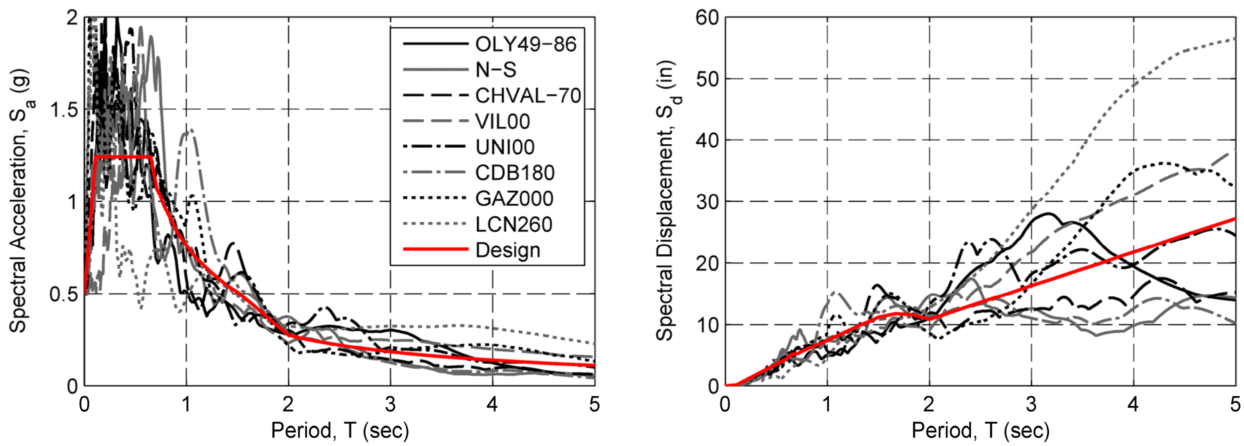
motions were selected. The 1992 Landers – Lucerne ground motion was also included to consider the effects of near fault ground motions that include forward directivity. The Landers motion was left unscaled. The eight ground motions that were selected are listed in Table 4 and the scaled response spectra are shown in
Figure **11**.

Table 4 -- Ground Motions

Event	Station	Mechanism	Distance	Scale
1985 Michoacán, Mexico	La Union (UNI00)	Subduction	52.1 mi (83.9 km)	2.95
1985 Michoacán, Mexico	Villita (VIL00)	Subduction	29.7 mi (47.8 km)	4.03
1985 Valparaiso, Chile	Valparaiso (CHVAL070)	Subduction	80.3 mi (129.2 km)	3.59
2001 Peru	Moquegua (N-S)	Subduction	211.3 mi (340 km)	2.31
1949 Olympia, WA	Olympia (OLY49_086)	Deep Intraplate	46.4 mi (74.7 km)	2.77
2001 El Salvador	Ciudadela Don Bosco (CDB180)	Deep Intraplate	68.5 mi (110.2 km)	2.55
1976 Gazli, USSR	Karakyr (GAZ000)	Crustal - Reverse	8.0 mi (12.8 km)	1.14
1992 Landers, CA	Lucerne (LCN260)	Crustal - Strike Slip	1.3 mi (2 km)	1.0

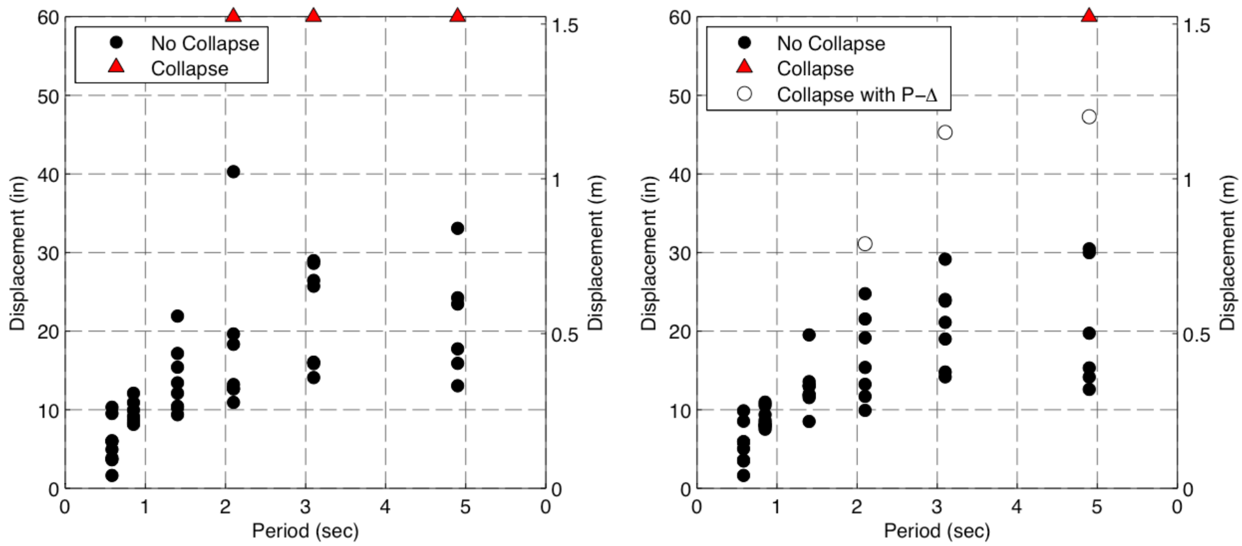


(a) Acceleration Spectra (b) Displacement Spectra

Figure 11 -- Scaled Response Spectra

NONLINEAR DYNAMIC ANALYTICAL RESULTS

With the eight ground motions that are listed above there were a total of 48 analysis cases for both the P-Δ included and excluded analysis types, resulting in a total of 96 response history analyses. The maximum displacement demands for all of the analysis cases are shown in Figure 12 in terms of fundamental period for both P-Δ included and excluded analyses. There does not seem to be a particularly strong correlation between heavily loaded piers (i.e. 100% of live load, fundamental periods equal to 0.9, 2.1, and 4.8 seconds) and an increase in displacement demand, likelihood of collapse, or ground motion record-to-record variability in response. Finally it can also be seen that the cases with high displacement demands for the P-Δ excluded cases result in collapse when P-Δ is considered.



(a) P-Δ Included (b) P-Δ Excluded

Figure 12 -- Displacement Demands

Out of the 48 analysis cases that included P-Δ effects, four indicated instability of the prototype structure and one reached very near collapse (see

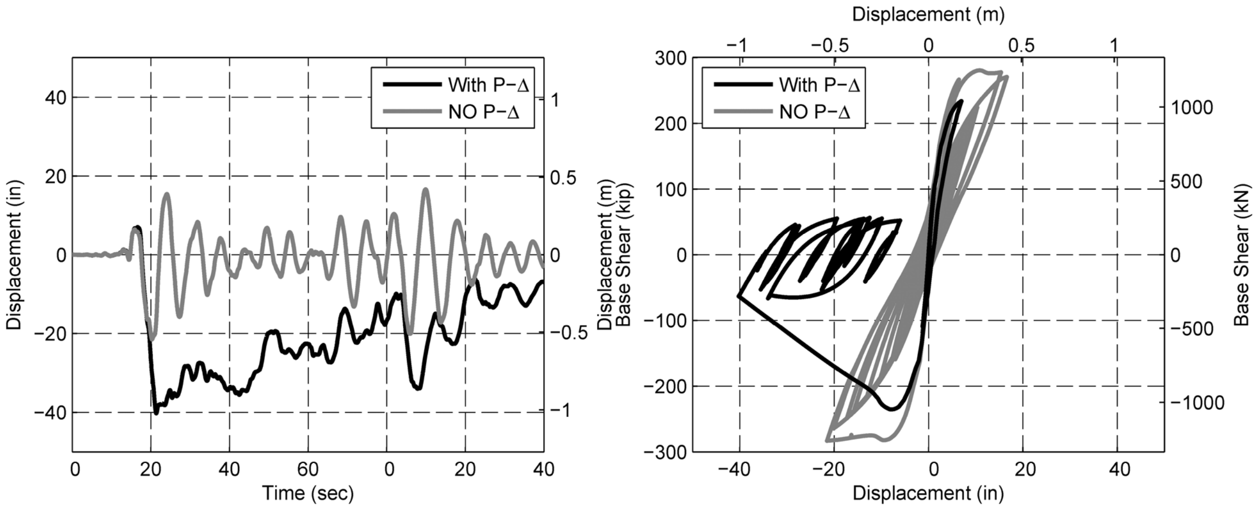
Figure 13), while the remainder of the cases responded with behavior that was symmetric and stable. Two of the more interesting cases will be examined. The first case, a 30ft tall (9.1m) pier with 100% live load, was subjected to the 1985 Michoacán VIL00 ground motion.

Figure 13a shows the displacement history of the deck for the analysis runs with and without P-Δ, where it can be seen that when P-Δ is neglected the response is stable and symmetric with moderate total deflections. However once P-Δ is included in the analysis the structure undergoes a very large displacement pulse early in the record, reaching near collapse. During subsequent ground shaking the structure slowly re-centers due to the restoring force provided by the prestressing within the piles (see

Figure 6).

Of particular note is that during the large displacement pulse of the analysis including P-Δ, the maximum connection strain reached 0.057. This strain is of some concern because experimental testing (Raynor 2000) of reinforcing bars grouted into metal ducts has shown that the ducts greatly inhibit strain penetration along the length of the bar due to the confinement provided by the duct. It therefore takes repeated cycles of high strain demand on the bar to break down the bond between the dowel and the grout and thereby develop the plastic hinge length given in [4. This then means that the plastic hinge length is dependent on the loading history of the connection. Considering the pier in this case experiences an essentially monotonic push with very few previous cycles it is likely that the connection will have a very short effective plastic hinge length. This could lead to premature bar fracture as dowel bar strains are rapidly accumulated at the pile-to-deck interface.

This is an area that certainly warrants further experimental testing as there have only been standard cyclic load protocol tests on pile-to-deck connections and therefore there is uncertainty regarding the relationship between the connection strains, the plastic hinge length, and the loading history of the connection. Because of this, until experimental research can be conducted, it is recommended that the dowel bars be intentionally debonded to mitigate the effects of pulse type loading whereby the debonding will provide a minimum effective plastic hinge length helping ensure adequate rotational capacity of the connection.

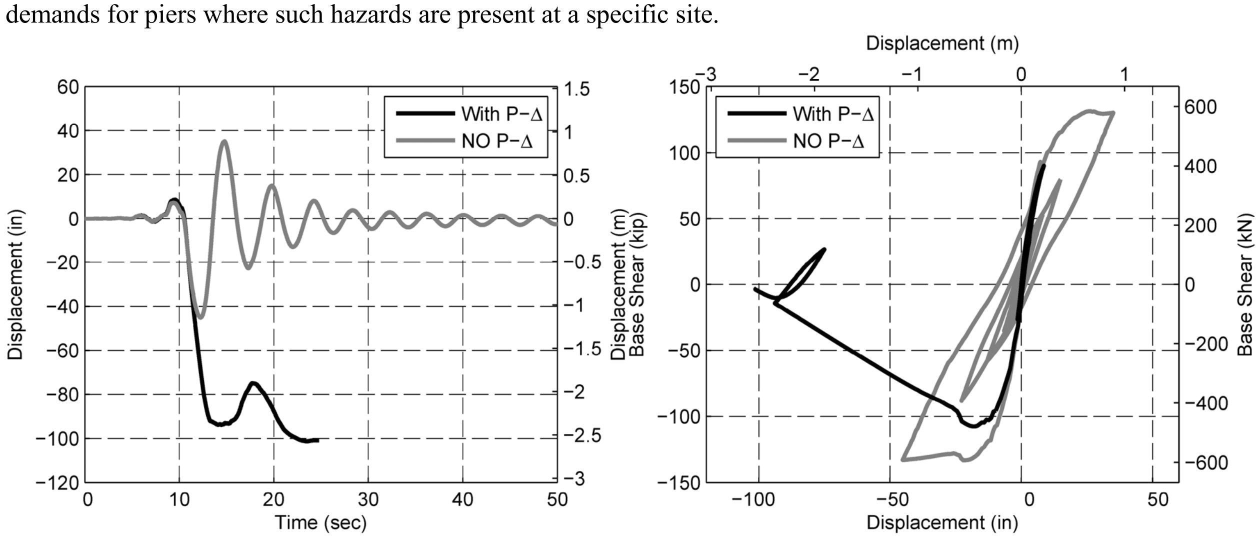


(a) Deck Displacement History (b) Base Shear – Deflection Response

Figure 13 -- Dynamic Analysis Results - VIL00, 30ft (9.1m) Pier, 100% Live Load, T = 2.1 seconds

The second analysis case of interest is a 60ft (18.3m) pier, with 10% of the live load, subject to the 1992 Landers LCN260 ground motion (see

Figure 14). Again when the analysis is conducted neglecting P-Δ effects the response is stable and symmetric. When P-Δ was included in the analysis the structure undergoes a large initial pulse at 10 seconds that eventually resulted in collapse 25 seconds into the record. This indicates the importance of considering near fault forward directivity demands for piers where such hazards are present at a specific site.



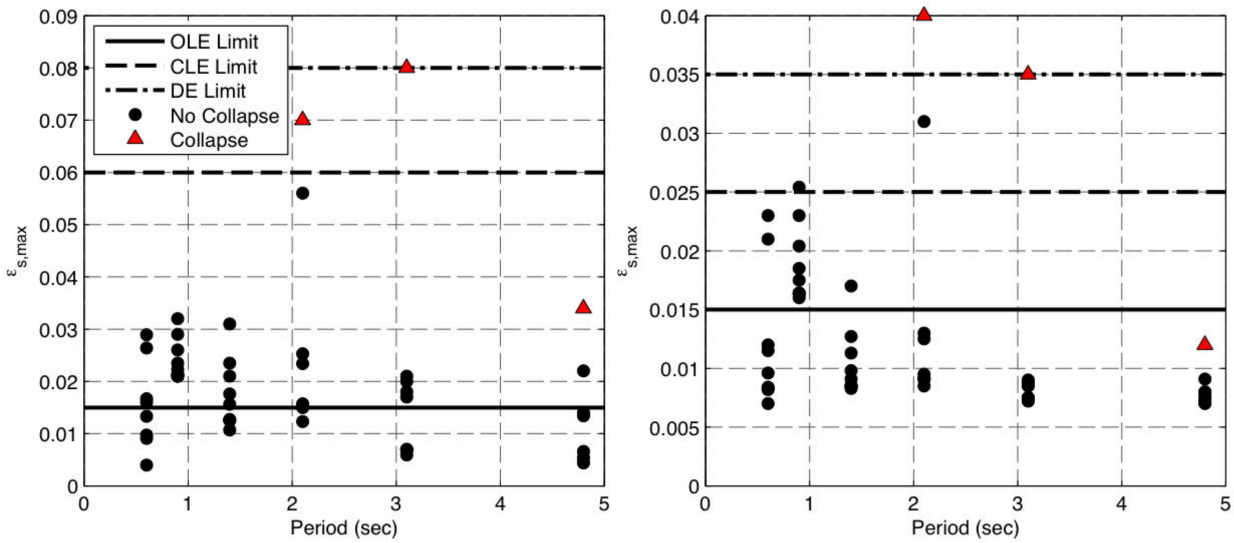
(a) Deck Displacement History (b) Base Shear – Deflection Response

Figure 14 -- Dynamic Analysis Results – LCN260, 60ft (18.2m) Pier, 10% Live Load, T = 3.1 seconds

Figure 15 shows the maximum tensile strain demands for the pile-to-deck connection and in-round plastic hinges plotted against the fundamental period of the pier for the analysis cases including P-Δ effects. For the cases where collapse occurred the recorded strains are those associated with the collapse displacement, Δ_C (defined in Figure 2). Results indicate that for non-collapse cases the steel strains demands are low, even under the MCE ground motion, typically between 0.01 and 0.03 for both the deck connection and in-ground plastic hinges, with longer period piers generally experiencing lower demands. The OLE, CLE, and DE steel strain limits from POLB are shown for reference for mild steel in the connection and prestressing steel in the in-ground plastic hinges. The one non-collapse case with high strain demands is the case discussed in relation to

Figure 13 where connection and in-ground steel strains reached 0.057 and 0.031 respectively. Even collapse cases did not have excessive steel strain demands, with strains at the collapse displacement around the DE strain limits. Therefore, for the conditions considered in this study the connection and in-ground demands are low and material failures such as dowel bar or strand fracture may not be a primary concern as collapse is typically caused by sway instability due to P-Δ effects and not material failure, this validates the expected performance based on data presented in

Figure 10.



(a) Pile-to-Deck Connection Plastic Hinge (b) In-ground Plastic Hinge

Figure 15 -- Maximum Steel Strain Demands

Finally it is of interest to determine the relationship between the stability index (θ^o) and displacement amplification factor (*DAF*). The results are plotted in Figure 16, where considerable scatter, can be seen. This shows that as θ^o is increased, the variability between the analytical results generated including and neglecting P-Δ effects increases. In common design practice P-Δ effects are ignored as it simplifies the analysis, therefore a justifiable limit is needed to define the θ^o range where P-Δ can safely be ignored. MOTEMS (2011) and the Port of Long Beach (2009) have defined this limit to be $\theta^o = 0.25$, based on reinforced concrete bridge column research. Above this limit, P-Δ must be explicitly incorporated into the analysis, which can only be done using NLRHA. The *DAF* values of 2 in Figure 16 for cases that collapsed were assigned arbitrarily to show the stability index values where collapse did occur. The *DAF* is actually unbounded for the collapse case.

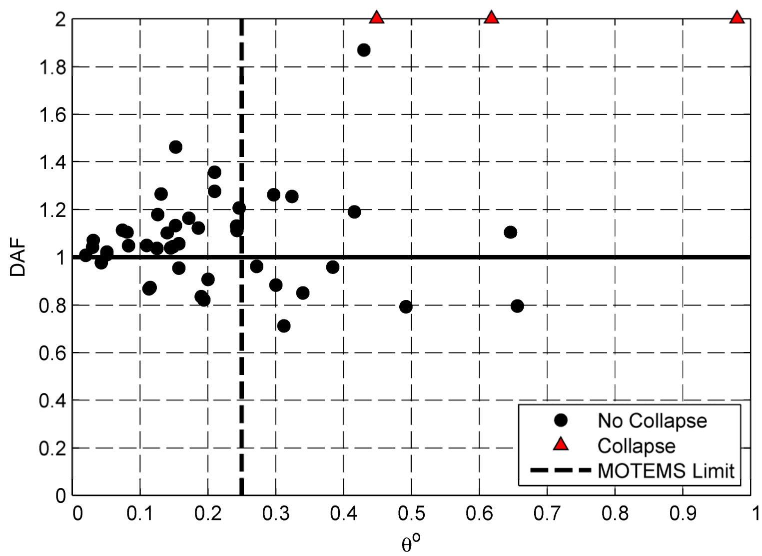


Figure 16 -- Displacement Amplification Factor

The statistics for the displacement amplification factor are presented in Table 5 for all analysis cases excluding those that resulted in collapse. It can be seen that the mean *DAF* value for all analysis cases, excluding collapse cases, is 1.07 with a coefficient of variation (COV) of 0.19. This shows that on average ignoring P-Δ effects will under-predict the displacement demand by seven percent. It is also evident that for cases above the θ^o limit of 0.25 the COV increases substantially.

Table 5 -- Displacement Amplification Statistics

θ^o **Range**	**Average Value**	**COV**
0 – 1.0	1.07	0.19
0 – 0.1	1.04	0.05
0 – 0.25	1.07	0.13
0.25 – 1.0	1.05	0.30

These results indicate that below the 0.25 limit there is still relatively high variability, +45% and -20%, with most displacements being amplified by the incorporation of P-Δ effects, however despite the variability in displacement amplification the 0.25 limit on θ^o appears to provide reasonable protection against instability. The lowest stability index value associated with collapse was 0.45, which provides a factor of safety 1.8 against collapse in terms of θ^o. It is therefore judged to be reasonable to use the current stability index limit of 0.25 for piers built on prestressed concrete piles with grouted dowel bar connections to protect against instability. However this stability index limit does not fully protect against the variability within the displacement amplification associated with P-Δ effects. It is recommended then that when increased analytical accuracy is desired P-Δ effects should be included in the analysis if the stability index is greater than 0.10 as the *DAF* variability below this limit is of very little consequence.

PROPOSED STABILITY CHECK

Due to the uncertainty in the plastic hinge length resulting from load history effects as discussed above, a stability check incorporating dowel bar fracture is proposed in the forthcoming ASCE Standard. This stability check is intended to be conservative and easily applied to the analysis and design of a pier. This procedure can be implemented in typical pushover analysis software packages.

(1) The connection moment-rotation response (typically modeled as elastic-perfectly plastic) should be modified to account for the reduction in moment resistance associated with dowel bar rupture.

This can be done by reducing the connection to a pin (i.e. zero moment resistance) once the rotation associated with a steel strain of 0.06 is reached.

(2) The structures pushover response should be calculated using normal procedures ignoring P-Δ effects. Note: there will be drop in lateral capacity once the connection hinge reaches the rotation associated with dowel bar rupture.

(3) The demand analysis (typically using the substitute structure approach) with pin connections should be run normally.

(4) Once the displacement demand has been determined, the stability index should be calculated using the following equation:

$$\theta^o = \frac{P\Delta_d}{V_d H}$$

Where:

$P =$ The weight of the pier including dead load and live load

$\Delta_d =$ The design displacement demand determined in step 3

$V_d =$ The base shear associated with Δ_d

$H =$ The distance from the point of maximum in ground moment to the soffit of the deck

(5) Stability considering dowel bar fracture is determined using the following two rules:

a. If θ^o is less than 0.25 the structure is stable

b. If θ^o is greater than 0.25 the structure is potentially unstable and the P-Δ effects must be considered explicitly. If the pushover response with P-Δ has a continual positive stiffness up to the displacement demand the structure is stable. If the pushover response with P-Δ has a negative post yield stiffness before reaching the displacement demand then nonlinear response history analysis or strengthening is required. Piles may be added to the system until θ^o is less than 0.25.

Based on

Figure 10 it seems unlikely that piers with long piles under typical gravity loading of 0.02 to 0.05$f'_c A_g$ will experience connection dowel tensile demands significant enough to cause dowel bar fracture as the displacement limit will likely be controlled by the standard stability index check, and the global displacements will be very high.. However, for piers with short to moderate length piles, less than around 20ft (6.1m) to 30ft (9.1m), the aforementioned stability check should provide a conservative estimate of when dowel bar fracture may precipitate global collapse.

FURTHER STUDY

While the current study considers the behavior of prestressed concrete piers subject to strong ground shaking, there are areas that warrant further study. First as described previously there are uncertainties in regard to the effect loading history has on the pile-to-deck connection plastic hinge length and the associated strain capacity of the dowel bars. Experimental testing should be conducted to determine if grouted dowel bar connections are susceptible to premature dowel bar fracture during earthquakes imposing large displacement pulses early in the record.

Secondly it would also be prudent to examine the 3-dimensional response of piers, as there can be significant torsional response due to changes in pile length and/or mass distribution along the pier. Finally as this study was conducted on piers founded in dense sand, the effects varying site conditions (loose to dense sands and clays) should be investigated.

CONCLUSIONS

This study was conducted to examine the seismic behavior of piers built on prestressed concrete piles founded in dense sand with grouted dowel bar connections. The following key observations were made.

1. For the conditions considered in this study the in-ground plastic hinge will control the ultimate displacement capacity of the system if POLB strain limits are used, however if the pile-to-deck connection mild steel fractures at a 0.06 strain then the displacement capacity of the system will be significantly reduced.
2. The ground motions that caused collapse typically had a displacement pulse or fling in the record. These characteristics were particularly harmful to longer period, more flexible piers.
3. In general connection and in-ground steel demands were low; with few cases experiencing steel strains larger than 0.03. This indicates that sway instability due to P-Δ effects is the most common cause of collapse for piers.
4. A stability index limit of 0.25 provides sufficient protection against dynamic collapse when P-Δ effects are ignored in the analysis for piers supported on prestressed concrete pile. A simple procedure was proposed to help identify when a pier is potentially at risk from instability due to dowel bar fracture.
5. A stability index limit of 0.1 will protect against significant P-Δ displacement amplification variability when increased analytical accuracy is desired.
6. For typical pile lengths and axial loading the P-Δ sensitive behavior is likely and the stability index limit will probably control the displacement capacities over material strain limits.
7. The proposed P-Δ stability check will be useful in preventing collapse of piers with short to moderate length piles, less than around 20ft (6.1m) to 30ft (9.1m)

ACKNOWLEDGEMENTS

The authors would like to thank Dr. M. Lee Marsh and Dr. Stephen P. Schneider for their guidance and consultation, and BergerABAM for graciously supporting this research study.

BIBLIOGRAPHY

Almeida, J., Das, S., & Piho, R. (2010). Guidelines for Fiber-based Inelasticity Modelling of Reinforced Concrete Members. *14th European Conference on Earthquake Engineering.* Ohrid, Macedonia.

American Petroleum Institute. (2007). *Recommended Practice for Planning, Designing and Constructing Fixed Offshore Platforms.* Washington DC: API.

ASCE. (2012). *Seismic Design of Pile-Supported Piers and Wharves - DRAFT.* ASCE.

Blandon, C. (2007). *Seismic Analysis and Design of Pile Supported Wharves.* Pavia, Italy: ROSE School.

Boulanger, R., Curras, C., Kutter, B., Wilson, D., & Abghari, A. (1999). Seismic Soil-Pile-Structure Interaction Experiments and Analysis. *Joournal of Geotechnical and Geoenvironmental Engineering*, 750-759.

Budek, A., Benzoni, G., & Priestley, M. (1997). *Analytical Studies on the Inelastic Seismic Response of Solid and Hollow Prestressed Piles.* La Jolla, CA: University of California San Diego.

Budek, A., Benzoni, G., & Priestley, M. (1997). *Experiemntal Investigation of Ductility of In-Ground Hinges in Solid and Hollow Prestressed Piles.* La Jolla, CA: University of California San Diego.

Charney, F. (2008). Unintended Consequenced of Modeling Damping in Structures. *Journal of Structural Engineering*, 581-592.

Coffin, L. (1954). A Study of the Effects of Cyclic Thermal Stresses on a Ductile Metal. *American Society of Mechanical Engineers*, 931-950.

Coleman, J., & Spacone, E. (2001). Localization Issues in Force-Based Frame Elements. *Journal of Structural Engineering*, 1257-1265.

FIB. (2008). *Bulletin 45: Practitioners' Guidle to Finite Element Modelling of Reinforced Concrete Structures.* Lausanne, Switzerland: International Federation of Structural Concrete.

Hutchinson, T., Boulanger, R., Chai, Y., & Idriss, I. (2002). *Inelastic Seismic Response of Extended Pile Shaft Supported Bridge Structures.* Berkeley, CA: PEER.

Jellin, A. (2008). *Improved Seismic Performance for Pile-Wharf Construction.* Seattle, WA: Univeristy of Washington.

Kent, D., & Park, R. (1971). Flexural Members with Confined Concrete. *Journal of Structural Engineering*, 1969-1990.

Krier, C., Restrepo, J., & Blandon, C. (2008). *Seismic Testing of Full-Scale Precast Prestressed Pile to Deck Connections.* La Jolla, CA: University of California San Diego.

MacRae, G. A., Priestley, M., & Tao, J. (1993). *P-Delta Design in Seismic Regions.* La Jolla, California: University of California San Diego.

Mander, J., Priestley, M., & Park, R. (1988). Theoretical Stress-Strain Model for Confined Concrete. *Journal of Structural Engineering*, 1804-1826.

Manson, S. (1953). Behavior of Metals Under Conditions of Thermal Stress. Ann Arbor, Michigan: University of Michigan Research Institute.

Menegotto, M., & Pinto, P. (1973). Method of Analysis for Cyclically Loaded RC Plane Frames Including Changes in Geometry and Nonelastic Behavior of Elements Under Combined Normal Force and Bending. *Symposium on the Resistance and Ultimate Deformability of Structures Acted on by Well Defined Reprated Loads* (pp. 15-22). Zurich, Switzerland: International Association for Bridge and Structural Engineering.

MOTEMS. (2011). *Marine Oil Terminal Engineering and Maintenance Standard, Title 24 California Code of Regulations, Part 2, California Building Code, Chapter 31F (Marine Oil Terminals).* Washington DC: International Code Council.

Neuenhofer, A., & Filippou, F. (1997). Evaluation of Nonlinear Frame Finite Element Models. *Journal of Structural Engineering*, 958-966.

Port of Long Beach. (2009). *Port of Long Beach Wharf Design Criteria V2.0.* Long Beach, CA.

Port of Los Angeles. (2010). *Code for Seismic Design, Upgrade nad Repair of Container Wharves.* Los Angeles, CA: City of Los Angeles Harbor Department.

Priestley, M., Calvi, G., & Kowalsky, M. (2007). *Displacement - Based Seismic Design of Structures.* Pavia, Italy: IUSS Press.

Priestley, M., Seible, F., & Calvi, G. (1996). *Seismic Design and Retrofit of Bridges.* New York: Wiley-Interscience.

Raynor, D. (2000). *Bond of Reinforcing Bars in Grouted Ducts.* Seattle, WA: University of Washington.

Scott, M., & Fenves, G. (2006). Plastic Hinge Integration Methods for Force-Based Beam-Column Elements. *Journal of Structural Engineering*, 244-252.

Stringer, S. J. (2010). *Sesimic Performance of Improved Damage Resistant Pile to Wharf Deck Connections.* Seattle, WA: University of Washington.

A NEW PILE-DECK CONNECTION FOR SEISMIC PERFORMANCE ENHANCEMENT OF MARGINAL WHARVES

Dawn Lehman and Charles Roeder

Synopsis: Pile-supported marginal wharves are a critical component of port infrastructure. A primary region of post-earthquake structural damage is the connection between the pile and the wharf deck. Review of prior experimental studies into state-of-the-practice connections indicates these can sustain cyclic deformation demand but at the cost of deterioration in resistance and significant damage. Damage within the connection is difficult to access and its repair is costly. Therefore, there is an interest in reducing the damage under moderate levels of seismic demand while sustaining the capacity under large cyclic drifts. An experimental study was undertaken to investigate mechanisms to limit damage while maximizing strength and deformation capacities of precast piles and their connections. Several structural concepts were investigated including (1) intentional debonding of the headed reinforcing bars, (2) supplemental rotation capacity through the addition of a cotton duck bearing pad above the head of the precast pile and (3) supplemental material to sustain the lateral deformations while minimizing deck damage. The final design incorporated all of these concepts. The results show significantly reduced damage. A design method is proposed to facilitate adoption of the proposed connection design in structural engineering practice. A comparison with other connection designs is made via fragility functions to assess their seismic performance.

Keywords: Connections; precast piles; marginal wharves; seismic evaluation; performance-based seismic design; seismic design; debonded dowels