SPECIAL CONSIDERATIONS FOR THE SEISMIC ANALYSIS AND DESIGN OF PIERS, WHARVES AND CONTAINER YARDS SUPPORTED ON PRESTRESSED CONCRETE PILES

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Synopsis: The seismic design of pile-supported marine structures such as piers and wharves is largely governed by their unique structural configuration and the special loading conditions associated with the operations that take place on the structure. The operation of heavy equipment and the stacking of heavy loads -usually well in excess of the self-weight of the structure- have significant implications on the seismic analysis and design of this type of structures. This paper reports a series of recommendations for the seismic analysis and design of piers, wharves and platforms supported on prestressed concrete piles, in presence of massive mobile equipment and/or stacked containers. Because of their significance in terms of structural safety and impact on construction costs of container and bulk handling terminals, emphasis is given to the evaluation of the percentage of live load to be considered as a source of seismic mass and a detailed discussion is presented on the need to rationalize the process of combining live loads with dead and earthquake loads as part of the definition of extreme load combinations in the seismic analysis and design of elevated platforms supported on piles. The paper includes a review of the treatment given to these loading aspects by specialized marine infrastructure design codes and offers specific recommendations.

Keywords: Seismic design; Pile-supported piers and wharves; Container yards; Inertial mass.
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**INTRODUCTION**

In addition to stringent durability requirements imposed by their exposure to aggressive (marine/river) environments, the design of piers, wharves and elevated platforms supported on piles is largely affected not only by the unique characteristics of these structures but also by the particular loading conditions associated with the type of operations that take place. For the case of container piers, wharves, or container yards supported on prestressed concrete piles (which is the prevailing piling type used in North America’s west coast ports), the use of heavy equipment, such as ship-to-shore (STS) container cranes, rubber-tire gantry (RTG) cranes, rail-mounted gantry (RMG) cranes, and straddle carriers, to handle containers and cargo and the stacking of containers and heavy loads on the superstructure have significant implications on the analysis and design for both gravity and seismic loads.

Until the late 1990’s, seismic design recommendations for pile-supported piers and wharves were controlled by building/bridge design codes, being the traditional approach that one based on equivalent lateral force methods. In addition to the unique structural features that differentiate pile-supported piers and wharves from buildings and to a lesser extent from bridges, the main drawback of force-based seismic design procedures for pile-supported piers and wharves is that such analysis would pay little emphasis to the inelastic response of the structure during ground shaking.

Except for buildings supported on pile-supported piers or wharves, whose design was covered by local building codes at that time, little consideration was given in the past to the monitoring of progressive structural damage in the seismic design of pile-supported piers and wharves. Damage control, rather than collapse prevention, is the most dominant criteria for the seismic design of this type of structures because of the significant economic cost associated with the interruptions to terminal operations. This is also in part because loss of human life in piers and wharves during an earthquake is less likely compared to that in buildings because occupation density is much less.

Reactive strategies following the devastating effects in port structures in Japan after the 1995 Hyogoken-Nambu (Kobe) earthquake and significant initiatives in the US, like the passing of the Lempert-Keene-Seastrand Oil Spill Prevention and Response Act in 1990 in California -which led to the creation of the Marine Facilities Division (MFD) under the California State Lands Commission (CSLC)- have led the way towards the development of seismic design recommendations explicitly tailored for pile-supported piers and wharves. These include the Marine Oil Terminal Engineering and Maintenance Standards (MOTEMS) guidelines (legally adopted under Section 31F of the California Building Code (CBC) (2010)), the precursor work by Ferritto et al (1999), the PIANC Seismic Design Guidelines for Port Structures (2001), and, more recently, the Port of Los Angeles (POLA) (2010) and Port of Long Beach (POLB) (2009) codes, and the upcoming ASCE/COPRI Standard for the seismic design of pile-supported piers and wharves.

A common feature of these codes and standards is the adoption of a performance-based design rationale which revolves around the evaluation of the displacement demand on the structure using performance-based procedures. Under this approach, the seismic response of the structure is influenced by the ability of the piles and pile-deck connections to undergo large deformations, all encompassed by recognizing that a “weak” column (piles) -“strong” beam (deck) behavior takes place so capacity protection principles are applied to prevent the occurrence of brittle modes of failure. Performance-based approaches usually require flexure-controlled behavior, characterized by the moment-curvature response of relevant members (piles, deck, pile-deck connections) including material overstrength factors, probable -rather than nominal- strengths, and the modeling of non-linear and coupled soil-structure interaction effects.
Current specialized seismic design guidelines for pile-supported piers and wharves in North America often provide specific performance criteria for three earthquake levels—namely Operational, Contingency and Design level—each with a different return period corresponding to distinct hazard levels to be accepted by owners/operators, and with specific performance objectives for concrete and steel reinforcement. The Operational Level Earthquake (OLE) typically corresponds to a design event with a probability of 50% of being exceeded in 50 years, while the Contingency Level Earthquake (CLE) typically has a probability of 10% of being exceeded in 50 years. The third level, the Design Level Earthquake (DE) corresponds to that in ASCE/SEI 7-10 (2010). It is common practice to design for minor damage under the OLE and moderate damage under the CLE. Minor damage conventionally corresponds to that requiring repair that does not compromise the normal operations on the structure whereas moderate damage may permit temporary shutdown but does not compromise the structural integrity of the structure. For more details about the different probabilities of occurrence, seismic performance objectives and design criteria associated with each of the earthquake levels, the reader is referred to the PIANC seismic design guidelines for port structures (2001), POLB (2009), POLA (2010), and MOTEMS (2010) (which is legally enforced by CBC 2010).

**PROBLEM STATEMENT AND SCOPE**

The evaluation of the displacement demand and deformation capacity of structural components in pile-supported piers, wharves or platforms is impacted not only by the designer’s assumptions on the representation of the configuration and mechanical properties of members (including pile stiffness, boundary conditions, connections, soil-structure interaction effects and associated probable material strengths) but also by the evaluation of the dead load and the fraction of the live load that contribute to both the gravity and inertial-type load demands.

Aspects related to the modeling of the structural components and their interaction with the soil have received substantial attention from researchers, practitioners, and code developers through the years. The evaluation of member stiffness is commonly handled through the use of effective flexural stiffness $(EI)_{eff}$. Rules for the definition of the moment-rotation or moment-curvature response at pile locations where plastic hinges are expected to form are also well documented despite variations from standard to standard on the evaluation of plastic hinge lengths and the limiting steel reinforcement and concrete strain values associated with the different seismic performance objectives. Guidance is also available for the modeling of the interaction between the piles and the surrounding soil through the use of nonlinear Winkler-type springs with corresponding P-y curves.

In the authors’ opinion, however, a weak link in the chain seems to be that associated with the treatment of live loads in the seismic analysis and design of pile-supported structures. Design recommendations to determine the level of live load to be considered as seismic mass are scarce. In addition, extreme condition load combinations involving live loads with dead and earthquake loads seldom elaborate on the need to study in detail the type of “live” load being combined in order to define adequate live load factors.

Modern design guidelines list the following contributors as the main sources of seismic mass in pile-supported piers, wharves and elevated platforms: i) the mass of the deck including that of any permanently attached buildings, equipment or fixtures; ii) the tributary mass of the piles; iii) the mass associated with a specified percentage of the applied live load; iv) a fraction of the mass of cranes (if present); and v) the hydrodynamic mass.

The determination of the fraction of the live load contributing to the seismic mass depends largely on the structure being designed. In bridge design, for instance, because the ratio of dead to live load may easily exceed 2 or 3 and the fact bridge live loads are considered to be “sprung” suggests that bridge live loads may not contribute significantly to seismic mass in this type of structures. The opposite would occur in a busy pile-supported container platform, where the heavy loads associated with the stacking of containers together with the operation of heavy container handling equipment such as STS cranes, RTG cranes or RMG cranes could easily dwarf the self-weight of the structure. The latter would lead to a much higher live load contribution to the seismic mass evaluation compared to the traditional bridge case.

Containerized operations, in particular, bring about the need to properly account for the type of live load being handled and the role live loads have on operations when evaluating its contribution to the seismic mass of the structure and the way live loads are to be combined with dead and seismic loads. Containers are often perceived as
live loads but this does not necessarily mean they are mobile. In fact, container boxes could be stacked for a long period of time to the point they will more than likely be present at the time an earthquake hits the terminal.

Figure 1 shows a partial cross-section of a container terminal comprised of a marginal wharf and a large elevated container yard supported on prestressed concrete piles. Container yards of this type are not as common as yards built directly over fill-reclaimed platforms but nonetheless they can be found in several container terminals worldwide. The wharf provides support for STS cranes and the passage of trucks to load/unload the ships whereas the pile-supported platform behind is used for stacking containers. The elevated container yard in this particular example shows seven container “blocks” across the yard for dedicated RTG crane container stacking operations. The number of container blocks is by no means standard. The figure is shown with the objective of highlighting how massive and substantial the loads associated with stacked containers could be on a pile-supported platform of this kind.

Figure 2 shows a zoomed-in detail of the containerized operations on the elevated pile-supported platform. Each container block is comprised of a series of container stacks. In container yards with dedicated RTG operations, a block is typically comprised of six boxes wide and up to six boxes high. The block length (into the page) may vary from block to block as dictated by terminal layout constraints and boundaries including truck corridors perpendicular in plan to the waterfront structure. In container yards with RMG cranes, the blocks could be much wider. The standard container dimensions are 2.44 m (8 ft) wide by 2.44 m (8 ft) tall with a length of either 6.1 m (20 ft) or 12.2 m (40 ft). Container stacks are typically comprised of containers with same length.

The evaluation of what constitutes the “live” load in a pile-supported platform requires rational consideration. The perceived “live” nature of the loads associated with the massive stacking of containers in a pile-supported deck comes associated with specific conditions. First of all, for the case of container terminal piers and wharves, the containers are stacked occasionally whereas in a busy container yard stacking operations involve a quasi-permanent presence of boxes at a given instant of time. What varies is the specific in-terminal dwell time of each box. This depends on the type of container terminal (e.g. imports/exports, trans-shipment of containers, combined operations, handling of empty containers, etc…) together with the particular container stacking/handling arrangement predetermined by the operator to allow expeditious and safe handling of the boxes.

The values reported in Table 1 (taken from the British Ports Association (BPA) guidelines for container yard pavement design (Knapton and Meletiu, 1996)) give the reader an idea of how heavy the container stacking loads on a pile-supported platform could be. Corner loads for stacks higher than one include a reduction coefficient to account for the fact it is unlikely that all containers in a stack will be fully laden. The table also reports equivalent uniformly distributed live loads (UDLL) for container footprints of 2.4 m (8 ft) by 12.2 m (40 ft) for a 40-feet box, and 2.4 m (8 ft) by 6.1 m (20 ft) for a 20-feet box. Traditionally, container yards get designed for UDLL of about 50 kPa (1,000 psf) boxes in addition to other requirements associated with the operations of RTG cranes, RMG cranes, yard tractors and other equipment. It is worth noting that a lower UDLL value is typically specified for the design of the waterfront structure (pier or wharf) because modern containerized operations in piers and wharves dictate the waterfront structures are not meant to be substitutes of the container yard for the stacking of boxes. Designing waterfront structures for higher UDLL values may lead to unnecessarily high construction costs.

Realize that the container load values reported in Table 1 represent only the load quantification defined by BPA for container yard pavement design. More accurate, project-specific container loads and weight distributions can be obtained through the terminal operators themselves based on the historicity of rated container loads. The container loads and weight distribution vary depending on the type of containerized operations that take place in the terminal and also within the container yard itself. The latter observation acknowledges the fact that the space located closer to the waterfront structure usually has higher container stacking densities.

This paper by no means intends to develop specific live load factors to be used when combining loads from stacked containers or mobile equipment as part of the seismic design of pile-supported platforms. Instead, the observations are meant to strengthen the need for the development of a rational methodology to evaluate specific load factors to deal with load combinations that combine “live” loads with other load types including earthquake loads. For simplification purposes, the authors will assume in this study that stacked containers be characterized as a “special” type of live load.
In addition to the complexities associated with the characterization of the live load, the evaluation of the fraction of the live load represented by stacked containers which contributes to the seismic mass of the structure is particularly challenging because the complex motions of stacked containers subjected to ground shaking make it difficult to identify whether the lateral load-displacement response of the structure would be accentuated, reduced or unaffected by the motion of the containers. Intuitively, for a given lateral stiffness of a pile-supported platform, an increase in the magnitude of the seismic mass at the time of occurrence of the ground motion would lead to an increase in the lateral displacement demand on the structure. If the structure depicted in Fig.1 were laterally flexible, then the acceleration of the deck under seismic actions may be small and the containers would tend to move together with the deck but not relative to it. Under this condition, the containers should be considered as rigidly attached to the deck and thus their entire inertia would contribute to the dynamic behavior of the pile-supported platform/container stack system, leading, in turn, to larger lateral displacements. In the event the opposite occurs, and the containers were to move, rock or slide relative to the deck, the question is whether it may be possible for such movement to reduce the displacement demand induced by the earthquake.

The treatment of live loads in conventional liquid bulk and marine oil terminals and their participation as a source of seismic mass differs compared to the container terminal case. Marine oil terminals are typically comprised of a loading platform with independent breasting and mooring dolphins and access trestle(s), interconnected with catwalks, walkways or other deck forms. A pictorial example of a marine oil terminal is given in Fig. 3. In most situations the live load acting on the platforms and dolphins is much lower than that in container wharves or yards, limited to pedestrian access and the use of light maintenance vehicles. Designers should then infer that the configuration and operations in a facility of this type would result in a substantially lower contribution of the live load to the seismic mass compared to that in a pile-supported container pier, wharf or elevated platform.

Another, often disregarded, key aspect in the seismic design of pile-supported piers and wharves refers to the treatment of vertical ground accelerations. Most specialized design standards would, at best, define the ordinates of a vertical acceleration spectrum as a fraction of the horizontal accelerations. The records from the February 22 2011 M6.2 Christchurch, New Zealand, Earthquake showed vertical peak ground accelerations (PGA) as high as 2.0 g, even exceeding the horizontal PGA values. A seismic event with these characteristics could impose severe force and displacement demands on pile-supported elevated decks with stacked containers by amplifying the inertial effect of the stacked loads on the foundation elements. Certainly, the evaluation of the effect vertical ground accelerations could have on the response of pile-supported elevated decks loaded with high stacking loads is worthy of a far more detailed consideration in specialized marine infrastructure design standards.

The most pressing questions stemming from these introductory remarks refer to what percentage of the live load needs to be considered for a safe evaluation of the seismic mass to be used in the seismic analysis of pile-supported piers, wharves and platforms when examining extreme loading conditions. Emphasis will be given to pile-supported structures that form part of container terminals and marine oil terminals. The paper does not intend to address all the variables that define the level of live load to be used in the seismic analysis and design of pile-supported piers, wharves or platforms. Instead, the observations are meant to strengthen the need for the development of a rational methodology to determine specific load factors to deal with load combinations involving seismic loads, dead loads and loads stemming from the stacking of containers or operations in a marine oil terminal or the operations of specialized mobile equipment. Suffices to say these statements are valid regardless of whether the seismic analysis procedure is force- or displacement-based.

OBJECTIVES, SCOPE AND RESEARCH SIGNIFICANCE

The main objective of this paper is to provide an insight on the treatment of live loads coupled with earthquake actions for the seismic analysis and design of piers, wharves and elevated platforms supported on prestressed concrete piles. A large fraction of the work concentrates on the evaluation of the portion of the live load as inertial mass in a pile-supported deck structure supporting a stack of containers, subjected to horizontal ground shaking. The evaluation is performed from a practical viewpoint based on the authors’ experience as marine infrastructure designers rather than formally implementing any statistical treatment and assessment of applied live loads for the seismic design of this type of structures. The authors, however, acknowledge that the latter are future action items which researchers and marine code developers need to investigate.
The seismic mass evaluation is conducted through a series of computer models by examining the effect that container box motions have on the structural performance of an equivalent single degree of freedom (SDOF) structure, representative of a portion of a pile-supported container yard, subjected to different types of ground motions.

Rather than formally developing live load factors as part of extreme loading combinations involving dead, live and seismic loads, the paper concentrates on a rational evaluation of the extreme load combinations adopted by specialized design standards for different types of “live” loads, with emphasis on the stacking of containers and the operations of heavy mobile equipment, and remarks the need to account for the type of operations that take place in a terminal when defining or invoking seismic load criteria.

An important objective of the study is to stir discussion among practitioners about the adequacy of the existing design guidelines so they can exercise educated judgment when evaluating the safe level of live load to be treated as seismic mass, the combination of live loads with earthquake loads, and the treatment of vertical ground accelerations in the seismic analysis and design of a pile-supported piers, wharves and platforms.

**SEISMIC MASS EVALUATION AND LOAD COMBINATIONS IN PILE-SUPPORTED MARINE STRUCTURE DESIGN INVOLVING EARTHQUAKE ACTIONS**

Even though the majority of design standards for marine infrastructure provide precise load factors to combine live and seismic loads under extreme conditions, explicit recommendations for the evaluation of the inertial mass are scarce. Even if available, the recommendations do not make distinction of the widely different types of live loads and equipment that could be present depending on the type of operations that take place in the terminal.

In the absence of a precise seismic mass definition, designers often refer to the available seismic load combinations as an extrapolation vehicle to infer what the live load contribution to seismic mass should be. This is a conceptual mistake. Increasing the live load factor in a seismic load combo will lead to an increase in the resulting load demand, whereas—in the context of container terminals—adding the mass of certain types of STS container cranes may turn out to be beneficial if the decoupling of the mode of vibration of these cranes relative to that of the pier or wharf leads to a reduction of the lateral load demand.

This has never been a problem in the seismic analysis of multi-story buildings because the seismic mass in this type of structures is typically determined by adding the full design dead and live loads applied at each story. For bridge design, the situation is different because the ratio of dead to live load can be substantially larger compared to that in buildings and because of the “sprung” nature of the live load. The 6th edition of the AASHTO LRFD Specifications (2012) establishes the use of a load factor, $\gamma_{EQ}$, up to 0.5 to combine live loads with earthquake loads and also recommend that the live load does not need to be accounted for in the seismic mass evaluation or in pushover analyses. For pile-supported piers, wharves and container platforms, the ratio of dead to live load may vary dramatically depending on the nature and the characteristics of the equipment and the specified live loads linked to operations.

Unfortunately, in absence of seismic mass evaluation guidance, designers are left alone with their own judgment to make the right decision and the extrapolation described above could well be the only way to proceed. Reasonable results can be obtained as long as designers have a thorough understanding of local seismic risk conditions, the type of operations, equipment and associated loads. Otherwise, the extrapolation exercise could lead to a complete misinterpretation of the load combo which, in turn, could lead to either unsafe or onerous structural designs.

The following sections present a summary of the treatment given in modern marine structures design guidelines to the evaluation of live loads as seismic mass source, the combination of live loads with earthquake actions, and the treatment of vertical ground accelerations. Design provisions from traditional building and bridge standards are also invoked for comparison purposes, as applicable.

**Live Load as Seismic Mass Source**

Section 12.7.2 of ASCE/SEI 7-10 defines a minimum of 25% of the live load to be included as part of the effective seismic weight in structures used for storage. The 25% factor seems to be the result of a formal statistical assessment to determine the live load fraction likely to be present in a storage building during a seismic event.
The 6th edition of the AASHTO LRFD Specifications (2012) recommends that the mass of the live load need not be included in dynamic analyses of bridges. Depending on use, it also calls for up to 50% of the live load to be combined with seismic loads. In areas where seismic design is not an issue, some Department of Transportations (DOTs) do not even require combining live loads with seismic loads.

According to POLA (2010) and POLB (2009), only 10% of the design uniform live load (not to exceed 5 kPa (100 psf)) in a pier or wharf is to be consider in the seismic mass evaluation. For the case of STS container cranes operating on marginal wharves, both codes recommend neglecting the STS crane mass effect if

\[ T_{crane} > 2T_{wharf} \]  \[1\]

where \( T_{crane} \) is the translational elastic period of vibration of the STS crane mode with maximum mass participation and \( T_{wharf} \) is the initial elastic period of vibration of the wharf structure based on cracked-section properties.

It is worth noting that the 2004 version of POLA recommended neglecting the STS crane mass effect if

\[ m_{crane} < 0.05m_{wharf} \]  \[2\]

where \( m_{crane} \) is the mass of the portion of the STS crane at or close to the wharf deck level and \( m_{wharf} \) is the wharf mass. Equation 2 has been slightly revised in the 2010 version of the POLA code as Section 1.6.1 now calls for the seismic mass to include the part of the crane not less than \( m_{crane,deck} \) or 0.05 \( m_{crane} \), where \( m_{crane,deck} \) is the part of the crane mass positioned within 3 m (10 feet) above the wharf deck and \( m_{crane} \) is the mass of the crane.

Equation 1 reflects the perceived beneficial effect from the decoupling of modes that typical STS cranes may have on pile-supported marginal wharves. A similar dynamic effect exists when tuning masses are used to damp the response of vibrating systems, whether they are buildings or mechanical components.

Shafieezadeh et al (2012), however, challenge this statement. The results of their studies on STS crane-wharf-soil interaction under seismic loads show that considering the STS crane mass may actually amplify the structural wharf response during an earthquake. One of the reasons behind this conclusion is apparently the fact that the computer models used by Shafieezadeh et al (2012) were not only more complex than those used in the previous analysis but also involved a more accurate description of the inertial properties of the STS crane. These conflicting conclusions warrant further investigation of the wharf-STS crane-soil interaction under seismic loads, especially if one takes into account the different possible variations in the description of the structure, the cranes and the soil that may exist in real life relative to the idealizations performed by Shafieezadeh et al (2012).

For the particular case of elevated pile-supported decks subjected to container stacking loads, it is necessary to examine the interaction of the structure with the modes of vibration (sliding, rocking, and coupling of these two) of the stacked containers to determine whether concepts similar to those established by Eqs. 1 and 2 can be extrapolated to the seismic analysis of pile-supported container piers, wharves or platforms.

**Load Combinations Involving Live and Seismic Loads**

Section 1.5 of POLA (2010) provides the following extreme load combinations (Note: the load factor designation presented herein is consistent with that used in ASCE/SEI 7-10):

\[ (1.0 \pm K)D + 0.1L + 1.0H + 1.0E \]  \[3\]

\[ (1.0 \pm K)D + 1.0H + 1.0E \]  \[4\]

where \( D \) represents the dead load, the self-weight of cranes and the weight of any permanently attached equipment or fixtures, \( L \) is the design uniform live load, \( H \) is the earth lateral pressure load, \( E \) is the earthquake load due to the operational, contingency and design level earthquakes including applicable orthogonality effects, \( K = 0.5 \text{PGA}/g \),
and PGA is the peak ground acceleration. The term $K$ is meant to account for the effects of the vertical ground acceleration.

Several observations can be made about Eq. 3. First of all, for the particular case of container terminals, the 0.1 live load factor seems to be appropriate for pile-supported piers or wharves where stacking of containers is only allowed in extraordinary circumstances. Conversely, a 0.1 live load factor looks inadequate for pile-supported piers, wharves or platforms where containers are stacked on a quasi-permanent basis. Suffices to say modern containerized operations call for no permanent stacking of containers on waterfront piers or wharves but rather on dedicated container yards or platforms. All these observations suggest that the live load factor to be included in a load combo involving seismic loads should depend on the type of live load being considered including the role it has on terminal operations. The need for a formal statistical evaluation of the associated load factor to be used for the different types of live loads that could be present in a pile-supported pier, wharf or platform during an earthquake seems warranted, to say the least.

Equation 4 is nearly the same Eq. 3 except for the live load term. It is worth noting that Eq. 4 is identical to that given by the 2002 version of MOTEMS. The absence of a live load term in Eq. 4 seems sensible for the design of structures in marine oil terminals because, by invoking Turkstra’s rule, the expected level of live load to be present in this type of facilities -which is the structure type scoped under MOTEMS- during an earthquake, can be considered to be very low.

The 2010 version of MOTEMS (CBC 2010) defines the following load combinations for earthquake conditions:

\[
\begin{align*}
(1.2 + K)D &+ 1.0L + 1.2B + 1.2C + 1.6H + 1.0E \\
(0.9 - K)D &+ 0.9B + 0.9C + 1.6H + 1.0E
\end{align*}
\]

In Eqs. 5 and 6, $K = 0.5 \text{ PGA}/g$, $B$ refers to buoyancy loads and $C$ accounts for current loads on the structure. Equation 5 is conceptually similar to Eq. 3 except it includes for the effects of buoyancy and current loads. In terms of the live load factor, the differences between Eq. 5 and Eq. 4 are striking, reflecting what appears to be a completely revised approach by MOTEMS 2010 for the treatment of live loads in seismic design. The use of a live load factor of 1.0 in Eq. 5 may lead to concerns from owners/operators that MOTs designed with MOTEMS 2002 may be under-designed. In the authors’ opinion, the basis for the 1.0 live load factor does not seem to be justified for this type of facilities, especially if it is taken into account that the live load in marine oil terminals is typically limited to small maintenance vehicles and the probability of simultaneous occurrence of a strong earthquake and the full live load is very low.

Finally, UFC 4-152-01 (2005) defines the following load combination for vacant berth conditions:

\[
1.2D + 1.0L + 1.2B + 1.2C + 1.0E
\]

The 1.0 live load factor in Eq. 7 is consistent with the live load factor used by ASCE/SEI 7-10 in its load combination for extreme conditions involving earthquake actions. The similarities between Eq. 5 and 7 may well confirm MOTEMS 2010’s endorsement of the ASCE/SEI 7-10 load factors. This approach is worth a thorough scrutiny though because the effect live loads have on the seismic design of buildings may differ substantially from that of a pile-supported pier, wharf or platform.

**Vertical Ground Acceleration**

As observed in Eqs. 3 and 4, the effect of vertical ground accelerations on the piles and the deck structure is accounted for through the introduction of upper and lower bounds for the dead load, defined as

\[
(1.0 \pm K)D
\]

In ASCE/SEI 7-10, the $K$ factor has been set as $0.2S_{DS}$ where $S_{DS}$ is the short period spectral acceleration. Since $S_{DS}$ is typically set equal to 2.5 (customarily accepted dynamic amplification factor) times the peak ground acceleration (PGA) then the resulting vertical acceleration is $0.2 \times (2.5 \text{ PGA}) = 0.5 \text{ PGA}$, as defined above. The plus or minus sign in Eq. 8 indicates the dead load effect increment due to the vertical acceleration is to be applied downward (positive)
and upward (negative). MOTEMS 2010 presents the equations separately (see Eqs. 5 and 6), with different load factors in parenthesis.

For the particular case of pile-supported container platforms with stacked containers, it is evident that the containers could be particularly sensitive to vertical ground accelerations. This would translate into an additional increase in the axial load applied on the piles which, for the case of prestressed concrete piles, could reduce the pile’s ability to accommodate inelastic deformations and thus affect the lateral response of the elevated platform. This consideration is presently missing in most existing design guidelines for pile-supported marine facilities let alone pile-supported buildings. The situation is particularly aggravated in light of the fact that codes only stipulate a constant value for $K$. The PGA in the vertical direction recorded in the February 22 2011 Christchurch (New Zealand) earthquake exceeded 2.0g, which is much higher than typically adopted values. This high vertical acceleration highlights the fact that the current treatment of vertical ground acceleration effects in the design of pile-supported piers, wharves and platforms -let alone buildings and bridges- needs to be reviewed thoroughly.

**Effect of Live Load as Source of Mass in Lateral response of Pile-supported Decks**

The effect of live load as an additional source of seismic mass on the lateral response of a pile-supported deck can be examined by idealizing the structure as a single degree of freedom (SDOF) oscillator, assuming the mass of the system is concentrated at the deck level. The fundamental period of vibration, $T_{SDOF}$, of the oscillator, without live load, can be calculated as

$$T_{SDOF} = 2\pi \sqrt{\frac{m}{k}} \quad [9]$$

where $k$ and $m$ represent, respectively, the stiffness and mass of the SDOF oscillator. The lateral displacement of the SDOF is proportional to its period. If the mass from the live load (e.g. containers), $m_L$, is added, the period increases, as defined by Eq. 10.

$$T^*_{SDOF} = 2\pi \sqrt{\frac{m + m_L}{k}} \quad [10]$$

The live load effect is conceptually illustrated in more detail in Fig. 4. The figure shows the response of the SDOF oscillator together with the earthquake demand expressed in the form of an acceleration-displacement response spectrum (ADRS). The response with period $T$ is that of the SDOF with its own mass whereas that with period $T^*$ corresponds to the SDOF with the live load added as seismic mass. Both response curves show some inelastic excursion. The period increase from $T$ to $T^*$ leads to an increase in the lateral displacement demand on the structure. The displacement increase depends not only on the stiffness of the structure but also on the amount of seismic mass added through the live load. In pile-supported piers and wharves, such a displacement demand increase can be handled through proper detailing of the piles and the pile-to-deck connections to preclude brittle modes of failure in shear or loss of anchorage.

**METHODOLOGY**

**Modes of Vibration of Rigid Body Supported on the Ground**

The evaluation of the effects that the different modes of vibration of stacked containers have on the response of a pile-supported deck subjected to ground motions can be assumed to be governed by the conditions of stability of free-standing rigid bodies. As a starting point, the fundamental concepts behind rigid body motion of bodies supported directly on the ground when subjected to horizontal ground accelerations will be presented, followed by the modeling of a pile-supported structure providing support for a set of stacked containers, subjected to horizontal ground motion. While the former subject has received considerable attention from many researchers (see for instance, Housner 1963, Shenton 1996, Esfandiari et al 2001, and Nasi 2010), the latter has not been examined in detail before.

Housner (1963) pioneered the efforts to develop a theory for analyzing rocking blocks prone to overturning due to ground shaking. Figure 5 shows a symmetric rigid block stacked on a flat, moving foundation, in 2D space. The
block has a weight $W_I$, width $2B$, and height $2H$, and it is subjected to a normal force, $N$, and friction force, $f_s$, at its base. The static and dynamic friction coefficients at the interface of the block and the foundation are defined as $\mu_s$ and $\mu_d$. Typical values for $\mu_s$ and $\mu_d$ for concrete-steel contact surfaces are, respectively, 0.45 and 0.3.

Shenton (1996) developed a series of criteria for initiation of slide, slide-rock and rock body modes from rest conditions for rigid boxes when subjected to horizontal ground pulses. His analyses show a transition of the mode of initiation of the response of the body from sliding to sliding-rocking and to pure rocking occurs as friction is increased for a given horizontal ground acceleration. The at-rest condition of the rigid block relative to the ground is valid if the following conditions are satisfied: i) the normal force is equal to the weight of the body (i.e. $N = W_I$); ii) the static friction is not overcome (i.e. $f_s \leq \mu_s N$); and iii) the normal force $N$ lies within the base of the block (i.e. $\alpha B < B$ or $|\alpha| < 1$). In the absence of vertical accelerations, $N = W_I$ is always satisfied. The second condition is met if
\[
\frac{\ddot{u}_{g,\text{max}}}{g} < \mu_s,
\]
where $\ddot{u}_{g,\text{max}}$ is the maximum horizontal ground acceleration. The third condition is satisfied if
\[
\frac{\ddot{u}_{g,\text{max}}}{g} < \frac{B}{H}.
\]
The sliding mode is initiated if $\frac{\ddot{u}_{g,\text{max}}}{g} > \mu_s$ and $\frac{\ddot{u}_{g,\text{max}}}{g} < \frac{B}{H}$, whereas the rocking mode is initiated if $\frac{\ddot{u}_{g,\text{max}}}{g} > \mu_s$ and $\frac{\ddot{u}_{g,\text{max}}}{g} > \frac{B}{H}$, as illustrated conceptually in the $\mu_s$ vs. $\frac{\ddot{u}_{g,\text{max}}}{g}$ space plot shown in Fig. 6. Four regions, corresponding to possible modes of initiation of vibration of the box relative to the ground can be identified: i) at rest, ii) sliding, iii) rocking, and iv) coupled sliding and rocking. The regions are bounded by the 45 degree line corresponding to $\mu_s = \frac{\ddot{u}_{g,\text{max}}}{g}$, a horizontal line defined by $\mu_s = \frac{B}{H}$, and a vertical line defined by $\frac{\ddot{u}_{g,\text{max}}}{g} = \frac{B}{H}$. The equation describing the boundary line between sliding/rocking and rocking is reported in Shenton (1996). An interesting observation stemming from the review of the previous relationships is that the criteria are independent of the mass of the rigid body.

To benchmark the occurrence of the rigid body motion initiation criteria, a series of structural models representing the motion of containers stacked on the ground, subjected to a horizontal ground acceleration, were created using the program SAP 2000® version 14.0 (CSI 2011) as depicted in Fig. 7. The rigid body -representing either a single container or a single stack of containers- was modeled with a horizontal and a vertical rigid beam element, connected as shown, with the horizontal element supported on the ground via two friction isolators. The latter have the capability to model the coupled compression-only (gap) and friction behavior of the box-to-ground interface. The length of the horizontal element represents the width of a standard container stack, whereas its height represents half of the height of a container stack assuming the center of gravity is located at mid-height. The model assumes the containers are fully in contact with the ones below. In reality each container has a small casting at each corner (each casting gets embedded in a hole located at the top of the container immediately below). For simplicity, the effect of these castings is ignored and full contact between top and bottom planes of the boxes is assumed.

The friction isolators were modeled assuming $k = 3,500$ MN/m (20,000 kip/in). Only the vibrations across the transverse direction of the container stack were examined because results (not reported herein) showed the longitudinal direction is far more stable. Sliding, rocking, and coupled sliding/rocking initiation modes of vibration were simulated, which meant that appropriate values for $\mu_s$ and $B/H$ had to be selected for the model to mimic the corresponding targeted vibration initiation mode. For simplicity purposes, and to confirm the validity of Shenton’s criteria, the horizontal ground acceleration was defined first by a constant amplitude sinusoidal function:

\[
\ddot{u}_g = A_g \sin(\omega t) = A_g \sin\left(\frac{2\pi t}{T}\right)
\]
where $A_g$ and $T$ are the magnitude and period of the sinusoidal horizontal ground acceleration.
Figure 8 shows frozen views of the observed mode of vibration of the rigid body, on the ground, for sliding and rocking conditions. The observed deflected shapes show the model captured the target response hence confirming the validity of the representation of the friction conditions between the body and the ground surface.

This led to the next phase of the analyses which intended to examine the mode of vibration of the rigid body but this time supported on a pile-supported deck structure. Only the results leading to sliding initiation will be reported in detail in this paper. Rocking initiation conditions were not examined as thoroughly because the analysis tools proved to be particularly sensitive to specific assumptions involving the gap elements which led to convergence issues. Their evaluation is part of an ongoing study.

**Modes of Vibration of SDOF Structure with Stacked Containers**

The lateral response of the pile-supported structure with stacked loads was modeled as a SDOF oscillator consisting of a single vertical frame element (representing a pile) connected to a horizontal frame element (representing the deck) providing support for stacked containers. Rather than the whole platform, only a single pile together with its tributary deck area was modeled. The prototype (real-life) pile-supported deck structure was assumed to represent a pile-supported platform with RTG crane beams spaced at 23.5 m in the transverse direction, with RC pile caps spaced every 12.6 m in the longitudinal (into the page) direction, with a deck consisting of precast concrete deck panels and cast-in-place concrete topping. For simplicity and as an initial approach to study the container stack-deck interaction, the pile was modeled as a linear elastic element, with rotations fully restrained at the top.

Figure 9 shows the idealization of the SDOF pile-supported platform with containers on top, subjected to horizontal ground acceleration. Given the appropriate modeling tools and support from experimental evidence to calibrate the finite element model, one could model also the interface between containers as indicated in Fig. 10. However, this is shown for illustration purposes since, for simplicity, the container stack was considered as a single rigid box in this study.

For the SDOF with containers on top, the analyses were conducted for a specific structural configuration with an aspect ratio $B/H = 0.5$ (i.e. stack of two containers) and $m_1/m = 2.67$ where $m_1$ represents the mass of the stacked containers (taken approximately as 250 Tonnes/g), and $m$ is the mass of the SDOF alone including the portion of the deck tributary to one pile. The SDOF structure started at at-rest conditions and was subjected to a series of sinusoidal ground accelerations as defined by Eq. (11). The goal was to ensure that the level of horizontal acceleration in the deck would exceed $\mu_s$ times the acceleration of gravity so that the mode of initiation for the rigid box was sliding. This was achieved by maintaining the product of the dynamic amplification factor and $A_g$ in Eq. (11) the same in all cases and larger than 0.2.

Figure 11 shows the effect of the lateral stiffness of the structure on the deck displacement demand. The values in the ordinates represent the lateral displacement of the deck with a relatively-moving container stack divided by the lateral displacement of the deck with a rigidly attached container stack. The period values in the abscissas represent the period of the oscillator. The variation in lateral stiffness of the structure is represented through $T_{SDOF}^*/T$ which corresponds to the ratio of the period of the oscillator with containers (refer to Eq. 10) to that of the ground excitation.

Figure 11 shows an increase in deck displacements with an increase in the period of the pile-supported deck structure. This is consistent with the fact that as the substructure (i.e. the pile) becomes more flexible, the displacement of the containers relative to that of the deck reduces and thus the container stack approaches the condition of a rigidly attached stack (for which the full live load should be considered as seismic mass). For short periods of the oscillator, some attenuation in the lateral displacement demand on the deck occurs as a result of energy dissipation associated to the relative movement of the container stack (friction and impact). It is also apparent that for $T_{SDOF}^*/T = 4.00$ there is little benefit from energy dissipation associated to the relative movement of the container stack and thus it is appropriate (and slightly conservative) to assume the full container mass as seismic inertia.

Similar analyses of the oscillator with rigid boxes on top were performed using the ground motion records shown in Fig. 12 for the Duzçe (1999) and Imperial Valley (El Centro record, 1979) seismic events. In this case, ground accelerations were scaled based on the peak ground accelerations (0.358g for the Duzçe record and 0.475g for the El
Centro record). The resulting displacement demands are shown in Figs. 13 and 14 as a function of the scaled peak ground acceleration, $A_g$.

It is apparent from Figs. 13 and 14 that the displacement of the deck with relatively moving containers approaches that of the deck with the container stack rigidly attached as $A_g$ decreases. Conversely, the deck displacement with the relatively moving container stack becomes smaller as $A_g$ increases. This is due to the fact that energy dissipation due to the relative movement of the container stack would be more pronounced for higher ground accelerations. The rate of the decrease is higher for stiffer structures (lower $T_{SDOF}$) because more vibration of the container stack, and thus more energy dissipation through friction, could occur. Deck displacement ratios close to unity for $T_{SDOF} = 2.00$ sec indicate there is no benefit due to the relative movement of the container stack for very flexible structures. This is sensible because the high flexibility of the piles in this case tends to isolate the mass of the system relative to the ground shaking.

Figure 15 shows the calculated deck displacements for a structure with $T_{SDOF}^* = 0.5$ sec under the 1979 El Centro record scaled to PGA = 0.5g. One time history corresponds to the case in which the container stack is rigidly attached to the deck whereas the other time history corresponds to the container stack allowed to move relative to the deck. The plot shows slightly reduced peak lateral deck displacements for the case of the deck supporting a container stack that moves relative to the deck. For these particular conditions, the relative movement of the container stack was first observed at approximately 5 seconds, followed by movement in the other direction.

The results shown in Figs. 11 through 15 indicate it is appropriate to assume the full live load likely to be present during the earthquake to participate as a source of seismic mass in the design of pile-supported container platforms. The attenuation of deck lateral displacements due to the contribution of the relative movement of containers (through sliding in this case) is possible but its acceptance requires the implementation of a detailed vibration analysis study similar to that conducted herein using the specific properties of the structure.

Threshold Period - Simplified Approach

A heuristic approach is presented next to determine the conditions for which the inertial contribution from container loads should be considered as part of the mass evaluation of a pile-supported platform. Combining Eqs. 10 and 11, the period of the SDOF with rigidly attached containers can be related to that of the SDOF without containers as follows:

$$T_{SDOF}^* = T_{SDOF} \sqrt{1 + \frac{m_L}{m}}$$

[12]

Figure 16 shows the design spectrum acceleration for a pile-supported platform for which the authors participated as designers. The implications of the relative movement of the container stack set forth are indicated relative to the platform period $T_{SDOF}^*$. For the particular case of a stack of six containers and a static friction coefficient larger than 1/6 between the stack and the deck, the minimum acceleration at which relative movement of the container stack would occur is $A_g/g = B/H = 1/6$. The period corresponding to this threshold acceleration is about 2.1 seconds. For larger periods, the acceleration is too low and the stack of containers behaves as if it were rigidly attached to the deck. For this particular example, the container stack-to-deck mass ratio, $m_L/m$, is about 3.6, so the transition condition occurs when $T_{SDOF}^* = T_{SDOF} \sqrt{1 + \frac{m_L}{m}} = 2.1$ sec. Solving for the period of the platform alone results in $T_{SDOF} = T_{lim} = 0.98$ sec. This can be interpreted as the period of the pile-supported platform above which movement of the stack relative to the deck does not occur. This means that for pile-supported structures with periods greater than $T_{lim}$, container stacks would tend to behave as if they were rigidly attached to the deck and thus it would be appropriate to use the full live load as seismic inertia in the seismic analysis of the structure. Conversely, for structures with periods less than $T_{lim}$, sliding or rocking of the container stack, or a combination of these two modes, would occur. For this particular case, it may be overly conservative to assume all the live load as seismic inertia because this would imply neglecting energy dissipation through the relative movement of the container stack. Any estimate of the extent of such reduction would require the use of extensive nonlinear dynamic analyses which falls outside the scope of this study.
Portion of Live Load Contributing to Seismic Inertia in Response Spectrum Analyses

The following simplified procedure is proposed for a quick evaluation of the portion of the live load contributing to the seismic inertia of a pile-supported platform/container stack system. Assuming the period of a typical pile-supported structure falls in the constant velocity region of the design acceleration spectrum, the lateral displacement can be considered to be proportional to the period of the structure:

\[
\frac{\Delta_l}{\Delta} = \frac{T_l}{T_{SDOF}} \tag{[13]}
\]

where \(\Delta_l\) and \(\Delta\) represent, respectively, the lateral displacement of the structure without and with containers, with corresponding periods \(T_{SDOF}\) and \(T_l\). The period of the structure with relative movement of the container stack, \(T_l\), can be written by analogy with Eq. 12 as follows:

\[
T_l = T_{SDOF} \sqrt{1 + \frac{\eta m_l}{m}} \tag{[14]}
\]

where \(\eta\) is the fraction of the live load that contributes to the inertial forces of the pile-supported platform.

Substituting Eq. (14) into Eq. (13) renders:

\[
\frac{\Delta_l}{\Delta} = \sqrt{1 + \frac{\eta m_l}{m}} \tag{[15]}
\]

Solving for \(\eta\) results in:

\[
\eta = \left[ \frac{(\frac{\Delta_l}{\Delta})^2 - 1}{\frac{m_l}{m}} \right] \leq 1.0 \tag{[16]}
\]

In Eq. (16), \(\Delta_l\) may be estimated from non-linear dynamic analyses including the relative movement (sliding/rocking) of the containers, as discussed in this paper, with the substructure modeled elastically. On the other hand, \(\Delta\) may be estimated directly from the acceleration design spectrum assuming that the substructure is elastic. It is worth noting that the effect of live load appears not only in the \(m_l\) term but also in the \(\Delta_l\) term, therefore if the live load is reduced further and further the ratio \(\Delta_l/\Delta\) would tend to unity and the net result is that \(\eta\) would tend to zero.

NUMERICAL EXAMPLE

The generic views shown in Figs. 1 and 2 describe the configuration of a pile-supported elevated container yard. Assume the container stacks are supported on transverse pilecaps spaced at 12.6 m (41.3 ft). Assume piles are 0.7 m (27.6 in) octagonal prestressed concrete. The weight of the concrete superstructure tributary to a single pile is taken as \(W = mg = 90\) Tonnes (198 kips) and the corresponding design live load is defined as \(W_l = m_l g = 250\) Tonnes (550 kips). Consider two lateral stiffness of a representative pile \(k = 43,000\) kN/m (2,950 kips/ft), which corresponds to a stiff structure, and \(k = 11,000\) kN/m (750 kips/ft), which corresponds to a flexible structure. The value of the lateral stiffness depends on the elastic properties of the supporting soil and the clear height of the piles. Assume a static coefficient of friction \(\mu_s = 0.4\) and a two-high container stack \((B/H=0.5)\).

For these conditions, the simplified SDOF structure shown in Fig. 9 was subject to the Duzce ground motion scaled by the peak ground acceleration (PGA). Calculated displacements of the deck -with and without containers- are shown in Fig. 17 for the two different lateral stiffness values of the structure. Using the ratio of the calculated displacements and Eq. 16, the calculated portion of the live load contributing to inertia of the system is shown in Fig. 18. It is observed that for the flexible structure, most of the live load contributes to an increase in displacement because the container stack behaves as rigidly attached to the deck whereas for the stiff structure only a portion of
the live load contributes because the container stack slides and dissipates energy. Similarly, for higher ground accelerations, the contribution of the live load becomes smaller because the energy dissipation associated with the sliding of the containers is larger.

CONCLUSIONS AND RECOMMENDATIONS

Design standards applicable to pile-supported marine structures seldom address in a rational manner the contribution that live load has on the seismic mass of the structure let alone its participation in load combinations representing extreme loading conditions involving earthquakes.

The results from the numerical analyses presented in this paper indicate it is appropriate to assume the full live load likely to be present in the form of stacked containers during an earthquake when evaluating the seismic mass of a pile-supported pier, wharf or platform. This is because this assumption would neglect the beneficial effect that energy dissipation through friction/sliding of stacked containers would have on the seismic response of the elevated platform.

The evaluation of the amount of energy dissipation in a pile-supported platform with stacked containers requires the implementation of nonlinear dynamic analysis to account for friction coupled with gapping of the container stacks, which may be difficult to represent. A more refined evaluation of the level of live load to be considered as a source of seismic mass is possible as long as the modeling of the platform accounts for the different modes of vibration of the stacked containers and their interaction with the structure.

From a response spectrum perspective, this paper provides guidance on how to estimate a threshold acceleration with a corresponding limiting period as the boundary to determine whether the mass associated with stacked containers is to be included in the seismic mass of the pile-supported structure. From a design viewpoint, if the period of the pile-supported structure exceeds the limiting period then the assumption of fully attached container would be feasible whereas for structures with periods less than the limiting period such an assumption would be considered conservative.

The authors suggest the existing design provisions dealing with the evaluation of live load as a seismic mass source in pile-supported piers and wharves should follow a more rational procedure in which emphasis is given to having a clear understanding of terminal operations in order to determine the role live loads play in the seismic response of the structure. As a minimum, the percentage of live load to be included as inertia should be expressed as a function of the dynamic characteristics of the structure (at least the fundamental period) and not as a fixed value that relies only on the probability of the design motion to take place when a certain portion of the live load is present.

In regards to extreme load combinations involving earthquakes and live loads, the use of fixed live load factors may lead to either unsafe or onerous designs if the role of “live” load is not properly understood. For the case of containers stacked on pile-supported platforms, these are to be considered as a special type of “live” load which, despite its mobility, may remain applied on the structure for a large period of time. Even though codes are not meant to replace a designer’s rationalization process, the language in the majority of design guidelines could be improved to emphasize the need to understand the role of mobile equipment in terminal operations to determine how live loads are to be combined with seismic loads.

Finally, even though these conclusions have been drawn for the case of piers and wharves supported on prestressed concrete piles, they can be extended to other piling types. Future work on the subject of live loads as a source of seismic mass in pile-supported platforms being planned by the authors includes the introduction of inelastic properties in the elevated deck supporting piling system, a more reliable modeling of rocking of stacked containers, and the inclusion of the dynamic interaction among containers within a stack.
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NOTATION

\( A_g \) Amplitude of ground acceleration
\( B \) Half the width of a rigid body, container, or stack of containers
\( D \) Dead Load
\( E \) Earthquake Load
\( E_{I_{eff}} \) Effective flexural stiffness
\( g \) Acceleration of gravity
\( H \) Half the height of a rigid body, container, or stack of containers; It also designates soil pressure load
\( k \) Lateral stiffness
\( K \) Vertical ground acceleration design factor
\( L \) Live load
\( L_{cr} \) Crane load
\( m \) Mass
\( m_L \) Mass associated with live load, containers, or stack of containers
\( N \) Normal force
\( T \) Period of harmonic ground excitation
\( T_{SDOF} \) Period of single degree of freedom system
\( T^{*}_{SDOF} \) Period of single degree of freedom system including the live load as additional mass
\( T_L \) Period of single degree of freedom system including a portion of the live load as additional mass
\( T_{crane} \) Fundamental period of STS crane
\( T_{lim} \) Limiting period
\( T_{wharf} \) Fundamental period of wharf structure
\( W \) Weight of rigid body, container, or stack of containers
\( W_L \) Weight associated with live load
\( \ddot{u} \) Acceleration of SDOF oscillator
\( \ddot{u}_g \) Ground acceleration applied in the horizontal direction
\( \ddot{u}_{g,max} \) Maximum ground acceleration applied in the horizontal direction
\( \dot{U}_g \) Total acceleration in the horizontal direction
\( \alpha \) Factor indicating location of normal force resultant in rigid body at rest
\( \Delta \) Lateral displacement of pile-supported deck or platform
\( \Delta_c \) Lateral displacement of pile-supported deck or platform with stacked containers
\( \eta \) Fraction of live load contributing to additional lateral displacement of pile-supported deck or platform
\( \mu_d \) Dynamic friction coefficient
\( \mu_s \) Static friction coefficient
\( \omega \) Frequency of ground motion or vibration
Table 1 -- Typical Container Loads

<table>
<thead>
<tr>
<th>Stacking Height (No. of boxes)</th>
<th>Reduction in Gross Weight (%)</th>
<th>Container Corner Load* (kN)</th>
<th>Equivalent UDLL (40 ft Box) (kPa)</th>
<th>Equivalent UDLL** (20 ft Box) (kPa)</th>
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<td>76.2</td>
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<td>365.8</td>
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</tr>
</tbody>
</table>

Notes:
1. Reduction in gross weight factor and container corner loads per BPA (1996).
* Based on 40 ft (12.2 m) container.
** Based on 20 ft (6.1 m) container with 245 kN (25 Tonnes) rated load.
1 kN = 4.448 kips; 1 kPa = 20.885 psf
Figure 1 -- Pile-supported Container Wharf and Container Yard

Figure 2 -- Zoomed-in View of Pile-supported Container Yard
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SEISMIC PERFORMANCE OF PILE-SUPPORTED WHARVES

Carlos A. Blandón, José I. Restrepo, Yohsuke Kawamata and Scott Ashford

Synopsis: This paper discusses the results of an experimental program carried out at the Englekirk Structural Engineering Center of the University of California in San Diego (UCSD) to provide data for the performance-based seismic design of vertical pile-supported marginal wharves. Strong earthquake-induced inertial lateral loading may cause significant damage to the wharf in two critical locations (i) at the pile-cap connection, and (ii) at the location of the pile maximum bending moment below the ground. Two pile-cap assemblies, representative of the two most critical piles of a marginal wharf and the surrounding quarry-run fill, were built at full-scale and tested under quasi-static reversed cyclic loading to large lateral displacements. The piles in the test units were precast pretensioned and were connected to the deck through grouted dowels and were also embedded in quarry-run fill, as is often the case in these marine structures. The test units displayed a very stable hysteretic response. This paper describes the test specimens, their hysteretic response together with the predicted response, the progression of damage in the test units, and the distribution of the applied lateral force among the two piles. The paper also highlights the most relevant implications for performance-based design of marginal wharves.

Keywords: Performance-based seismic design, Precast concrete piles, Soil-structure interaction, Testing, Wharves
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INTRODUCTION

Past experience indicates that strong intensity earthquakes can induce unexpectedly large structural damage to port structures.1-5 Extended periods of interruption or partial collapse of these structures, even if have not shown to be life-threatening, have had a large impact in the regional economies.5 Such experience has prompted port authorities the need for developing performance-based seismic design methodologies.6-10(PIANC, Nozu et al. POLA and POLB) Performance-based seismic designs are aimed at narrowing the prediction of the structural response, for a given earthquake intensity.

Ports can incorporate a variety of marine structures among which marginal wharves are common. These wharves are structures oblong in plan with their longitudinal axis running parallel to the shoreline and are typically designed to support a moving container handling gantry crane and heavy cargo to resist berthing, mooring, and seismic loads among others, see Figure 1. A way to build marginal wharves is with vertical piles, which are often precast prestressed that are jetted and/or driven into the ground. In a typical vertical pile supported marginal wharf the clear height of the piles may decrease rapidly from the seaside to the landside. At the landside, the pile clear height ranges between two and three pile diameters only, see pile rows F and G in Figure 1. This geometrical constraint causes these short piles to have the largest lateral stiffness. Consequently, the piles in these two rows transfer a significant percentage of the lateral forces due to berthing and mooring as well as those earthquake-induced lateral inertia forces. Under strong intensity seismic excitations, these short piles will also experience the largest inelastic deformations in the entire structure. Such inelastic deformations will be manifested by large rotations at the pile-cap connection and by large inground pile rotations. Therefore, the seismic performance of the wharf is heavily influenced by the design and detailing of the piles in the landside rows.

The seismic response of the critical portion of a vertical supported marginal wharf was investigated experimentally through two test units. The test units represented an edge segment of a wharf and incorporated the short piles on rows F and G shown in Figure 1. The main variable in the experimental program was the clear height of the piles. The results presented in this paper contribute to efforts carried out by others to define parameters for the performance-based seismic design of these important structures.11-14 The most recent approach is based on setting strain limits to the materials in the critical sections of the piles. Different limits are given for the critical section at the pile-cap connection and for the critical section below ground, with stricter limits for the latter because of the difficulties associated with inspecting and repairing the inground portion of a pile. The main concern associated with the severe damage of the critical sections is the exposure of the longitudinal reinforcement to the aggressive marine environment, which under some circumstances can cause corrosion of the steel reinforcement. At the pile-cap connection, the strain limits have been defined to reduce the exposure of the reinforcement for a commonly occurring earthquake of moderate intensity and to allow for repairs after a rare and strong intensity earthquake. At
the inground critical section, the concrete cover strain limit has been defined such that there will be no spalling after a strong intensity earthquake.

**Figure 1 -- Typical Pile-supported Marginal Wharf Configuration.**

**RESEARCH SIGNIFICANCE**

To the authors’ knowledge no simulated seismic load testing on assemblies of vertical pile-supported wharves comprising the deck, piles, and the ground fill has ever been reported in the literature. The experimental work described herein was carried out on two full-scale test units built with realistic boundary conditions such as the piles, the deck, the pile-cap-connections and the embedment of the most critical length of the piles in quarry-run fill. The test units were subjected to quasi-static reversed cyclic lateral displacements that resulted in significant nonlinear response at the pile-cap connection and at a distance inground in the piles, and eventually resulted in fracture of the steel reinforcement. The damage progression was monitored in the tests with the specific aim to support the development of performance-based seismic design methodologies for these structures, and for the development of computational methods that can account for the nonlinear quarry-run fill-pile interaction.

**STRUCTURAL SEISMIC DESIGN OF VERTICAL PILE-SUPPORTED MARGINAL WHARVES**

The preferred mechanism of inelastic deformation in vertical pile supported marginal wharves subjected to inertia forces during a large intensity and rare earthquake is the strong deck-weak pile, meaning that in such an extreme event flexural plastic hinges will develop below the deck, at the pile-cap connection, and in the piles below ground. This mechanism is preferred because pile repairs after a damaging earthquake can take place while maintaining the operation on the wharf. The best approach to ensure development of a strong deck-weak pile mechanism is capacity design. In capacity design undesirable pile shear, joint and other failures leading to rapid strength degradation are explicitly precluded through this hierarchical design approach. The terms “strong intensity” and “damaging” cited above are highly qualitative. These terms can lead to ambiguity and relate structural performance and seismic intensity only philosophically. Performance-based seismic design intends to remove much of the subjectivity by directly relating structural performance and seismic intensity, the latter which is quantified in terms of the site-specific seismic hazard.

The first reference released that directly relates seismic demand and limit strains for the performance-based seismic design of wharves is a code issued in draft form by the Port of Los Angeles (POLA). This code contains strain limits for piles for the following earthquake levels:

(i) Operating Level Earthquake (OLE) for frequently occurring earthquakes having a probability of exceedance of 50 percent in 50 years of exposure, or a return period of 72 years. For this earthquake hazard level, the wharf shall remain operating.
(ii) Contingency Level Earthquake (CLE) for rare earthquakes with a probability of exceedance of 10 percent in 50 years, or a return period of 475 years. In this case, repairable structural damage requiring repair may occur and a short interruption of business is deemed acceptable.

(iii) Design Earthquake Level (DE) for very rare earthquakes in which case life safeguard must be ensured and major structural failure must be avoided.

The POLA code prescribes strain limits for piles for these intensity earthquake levels. Table 1 shows the strain values for the critical sections for pile-cap connections located at the landside and for pile plastic hinges that develop inground at a depth of less than 10 pile diameters. In this table $\rho_s$ is the effective volumetric confinement ratio and $\varepsilon_{smd}$ is the rupture strain of the pile-deck connection dowels. The strain in the extreme cover concrete, $\varepsilon_c$, the strain in the most stressed strand, $\varepsilon_p$, and the strain in the connection dowel, $\varepsilon_{sd}$, are defined for the critical sections at the pile-cap connection and at the inground hinge.

<table>
<thead>
<tr>
<th></th>
<th>OLE</th>
<th>CLE</th>
<th>DE</th>
</tr>
</thead>
<tbody>
<tr>
<td>Connection concrete</td>
<td>$\varepsilon_c \leq 0.005$</td>
<td>$\varepsilon_c \leq (0.005 + 1.1\rho_s) \leq 0.025$</td>
<td>No Limit</td>
</tr>
<tr>
<td>compressive strain$^1$</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Connection dowel</td>
<td>$\varepsilon_{sd} \leq 0.015$</td>
<td>$\varepsilon_{sd} \leq 0.6\varepsilon_{smd} \leq 0.06$</td>
<td>$\varepsilon_{sd} \leq 0.8\varepsilon_{smd} \leq 0.08$</td>
</tr>
<tr>
<td>tensile strain$^1$</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Inground concrete</td>
<td>$\varepsilon_c \leq 0.005$</td>
<td>$\varepsilon_c \leq (0.005 + 1.1\rho_s) \leq 0.008$</td>
<td>$\varepsilon_c \leq (0.005 + 1.1\rho_s) \leq 0.025$</td>
</tr>
<tr>
<td>compressive strain$^2$</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Inground steel strand</td>
<td>$\varepsilon_p \leq 0.015$</td>
<td>$\varepsilon_p \leq 0.025$</td>
<td>$\varepsilon_p \leq 0.035$</td>
</tr>
<tr>
<td>tensile strain$^2$</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

$^1$ specifications for pile-cap connection located at landside (seismic piles).

$^2$ specifications for plastic hinges located at depth less than 10 pile diameters.

$\varepsilon_{smd}$ is the steel dowels rupture strain.

Previous version of the code included different strain values from those shown in Table 1. However, based on more recent experimental data of the pile-cap connection$^{13}$ and the data reported in this paper, the values were updated to those in Table 1. The tensile steel strains are given for the farthest dowel or the prestressing strand from the neutral axis. The compressive strains are given for the unconfined concrete at the edge of the critical section farthest from the neutral axis. It is clear that the compressive unconfined concrete strain limits defined for some earthquake intensity levels are significantly larger than the crushing strain. However, these values are acceptable as section analyses carried out for typical sections under different axial load conditions revealed that the strains levels at the edge of the confined concrete section remain below the crushing strains for confined concrete$^{18}$.

**EXPERIMENTAL PROGRAM**

**Test Description**

Two test units, termed System Test 1 and System Test 2 were built at full-scale to specifications of the Port of Los Angeles. Figure 2 shows general dimensions. Each system test incorporated two 0.61 m (2 ft) octagonal precast pretensioned concrete piles. One pile was located on level ground and the other pile was located at the crest of an embankment with 1:1.5 slope. Typical wharves have additional pile rows located in the embankment but the two first pile rows, as those tested, have the largest shear force and displacement demands of the system.
The piles were embedded nine pile diameters below the ground surface, passing through 2.1 m (7 ft) gravel fill below 3.3 m (11 ft) quarry-run fill. The two test units had a different clear height to evaluate the effect of different shear force levels. The pile clear heights were 1.1 m (43 in.) and 1.7 m (67 in.) in System Tests 1 and 2, respectively. Nonlinear analyses performed during the design of the experiment indicated that bending moments and shear forces were negligible at depths beyond nine pile diameters below the ground surface. Moreover, the analyses indicated that pile inground plastic hinges would develop within the first 2.4 m (8 ft) below level ground, that is, well within the quarry-run fill. The analyses also indicated the piles with an embedment of nine pile diameters did not have enough capacity to resist the expected pile tensile forces. For this reason a 1.2 m (4 ft) square by 0.9 m (3 ft) deep footing for the level piles and a 1.2 m (4 ft) square by 1.5 m (5 ft) deep footing for the slope piles were cast at the pile ends to preclude a pull out failure during testing, thus providing appropriate boundary conditions for the longer piles that would be used in practice. The quarry-run fill was sourced from a nearby granite quarry and had an average particle size of 152 mm (6 in.). A full description of the experimental work is given elsewhere.17, 18

The piles were manufactured in a precast concrete plant as standard piles, except that they incorporated two 89 mm (3.5 in.) centrally located PVC downhole PVC tubes that were needed for instrumentation, see Figure 3. The piles were pretensioned with 16-15.2 mm (0.6 in.) diameter ASTM A416 strands. The strand stress after losses was estimated at 1055 MPa (150 ksi), resulting in a concrete compressive stress of 8 MPa (1.14 ksi) acting over the pile cross section. The concrete core was confined with ASTM A82 W20, 13 mm (0.5 in.) diameter, steel coil having a nominal yield strength $f_y = 485$ MPa (70 ksi). The concrete cover to the side of the spiral was 76 mm (3 in.), resulting in a confined concrete core of 458 mm (18 in.) measured to the outside of the spiral. The W20 coil was detailed in the piles as a spiral with 63.5 mm (2.5 in.) pitch resulting in a volumetric confinement ratio $\rho_s = 1.79$ percent. The piles were cast using self-compacting concrete and were not steam-cured. The average cylinder concrete compressive strength at 28 days was 63 MPa (9 ksi).
The top of the piles incorporated eight 50 mm (2 in.) diameter by 1400 mm (55 in.) long corrugated steel ducts. These ducts were equally distributed inside the pile concrete core along the circumference of the W20 spiral. The purpose of these ducts was to enable, after erection, the grouting of eight ASTM A706 #10 (1.25 in.) Grade 60 single-headed dowel bars that would form the critical pile-cap connection where plasticity was expected. These bars were grouted 1.0 m (40 in.) into the pile with a cementitious grout of 44 MPa (6.3 ksi) compressive strength at 28 days and protruded 0.71 mm (28 in.) from the top of the pile. The head on these bars was provided to enhance the anchorage conditions of these bars in the deck. The measured yield strength of these dowels was $f_y = 483$ MPa (69 ksi). This type of connection is preferred by the Port of Los Angeles and has displayed excellent hysteretic response and rotation capacity in single pile component tests.\(^\text{12}\)

Instead of casting a reinforced concrete deck between the piles, two 0.9 m (3 ft) high by 1.2 m (4 ft) wide by 1.2 m (4 ft) long reinforced concrete pile-caps were cast. These caps were coupled with a stiff W24x176 steel I-beam. The transverse reinforcement spiral was extended to the cap top horizontal reinforcement. Moreover, the top portion of the prestressed pile had two and a half spiral loops exposed and embedded inside the cap. The upper 50 mm (2 in.) of the pile was embedded into the cap. The average 28-day pile-cap concrete compressive strength was 46 MPa (6.6 ksi).

**Instrumentation and Test Set-up**

The test units, and in particular the piles, were densely instrumented with different types of sensors, totaling about 200 per pile, with the aim of measuring rotation and curvature profiles along the piles, but particularly to record the location and instant at which spalling of the concrete cover would occur inground in the piles. The I-beam was instrumented with an array of strain gauges to decompose the bending moments, axial and shear forces on each of the piles, although these sensors drifted abnormally during testing. Blandon\(^\text{18}\) provides a comprehensive discussion of the instrumentation scheme.

The onset of spalling of the concrete cover in the inground portion of the piles was monitored with two independent sensor types. The first sensor was a displacement potentiometer encased inside a steel cylinder. This cylinder was fixed to the core transversal reinforcement at one side. At the other side, a steel plate was embedded inside the cover concrete so that as the cover spalled off, this plate moved with it, enabling the potentiometer to measure the movement of this plate with respect to the fixed based at the core, see Figure 3. The second sensor was a rubber band placed outside the section and instrumented with strain gages.\(^\text{17,18}\) Inclinometers and potentiometers were also placed at the pile-cap interface to measure rotation and to calculate their relative rotation. Additionally, two dowel bars connecting the pile and the cap were instrumented with strain gages to monitor bar strains to calculate strain limit-states in these bars. Inclinometers were also installed inside a duct embedded along the pile. These sensors were strategically placed to capture the rotation profile along the pile. Finally, the system lateral displacement was measured by a series of string potentiometers fastened at the pile-cap’s mid-height and to a fixed reference point located outside the soil pit.

To protect the sensors placed within the surface of the piles, the construction process differed from actual practice. The piles were first erected into a 12.7 m (42 ft) wide by 13.4 m (44 ft) long by 7 m (23 ft) deep outdoor soil pit. Then the layer of gravel was placed at the bottom of the pit followed by the quarry-run fill. The authors recognize the difference between actual pile driving in marine work, where piles are often jetted and/or driven,\(^\text{19}\) and the construction method used for the tests.

After completion of construction, a 1 MN (220 kip) capacity ±610 mm (24 in.) stroke hydraulic actuator was connected to the level ground pile-cap and to a strong wall outside the soil pit. See Fig 2. The deck’s gravity load was neglected for the tests as numerical analyses show that this axial load had a small influence on the elements compared to the axial load generated due to the coupling of the piles. In addition, including this load in tests set up would be challenging.

**Test Protocol**

Testing was divided in two stages. The first stage was force-controlled, whereas the second was displacement-controlled. Cycles corresponding to 0.25, 0.5 and 0.75 of the system theoretical yield force, $F_y$, were applied during the first stage. Such force was estimated from a bilinearization of a nonlinear force versus displacement envelope obtained from a pushover analysis of a numerical model developed in OpenSees\(^\text{20}\).
Displacement-based finite elements and fiber sections were used to model the piles, the pile-cap connection was modeled with a rotational spring and the soil was represented with p-y springs provided by a geotechnical team according to current state-of-the-practice. These springs were validated against results obtained from a free head pile tested during the same experimental program. According to the results from this test, the upper bound p-y springs provided the best results. Therefore, upper bound values were used for the p-y springs used in the estimation of \( F_y \). Even if a difference of the applied lateral load for push and pull cycles was expected, due to the considerable uncertainty of the variables associated with the soil stiffness and strength, the same p-y springs and \( F_y \) value were used for both loading directions.

The nonlinear analyses were used to determine the system lateral displacement, \( \Delta_{75} \), at 0.75 \( F_y \) which was later used to define the theoretical reference yield displacement of the system, \( \Delta_{ye} \), as \( \frac{4}{3} \Delta_{75} \). The average value of \( \Delta_{ye} \) of 38 mm (1.5 in.) obtained from the first system test was used to define the lateral displacement protocol for the second stage of testing. It was found later on that the average value of \( \Delta_{ye} \) for the second test was 40 mm (1.6 in.), which was nearly identical to the value obtained from the first test. Such protocol consisted of a set of two cycles of 1.5 \( \Delta_{ye} \) followed by a trailing cycle of 1.0 \( \Delta_{ye} \). The same pattern of two cycles followed by a trailing cycle was repeated for amplitudes corresponding to 2 \( \Delta_{ye} \), 3 \( \Delta_{ye} \), 4 \( \Delta_{ye} \), 6 \( \Delta_{ye} \), 8 \( \Delta_{ye} \) and 10 \( \Delta_{ye} \). The amplitude of the trailing cycle corresponded to the maximum displacement of the previous set of cycles. A final cycle of ± 380 mm (15 in.) was applied for both system tests.

**TEST RESULTS**

**Overall Response**
Both tests showed this pile-supported wharf system has stable hysteretic behavior and large deformation capacity. The piles in this system exhibit damage concentration at the pile-cap connection and below ground in the piles. The latter was observed upon excavation.

Figure 4 and Figure 5 illustrate the extent of damage of the test units above and below ground at the end of testing for System Test 1. By the end of the test, the concrete cover at the cap soffit and at the pile top was near to or had spalled. The damage at the pile-deck connection for System Test 1 extended into the pile whilst for the System Test 2 damage concentrated at the pile deck interface, with very limited damage to the pile itself. Inspection of the underground portion of the piles at the conclusion of the tests revealed that, where plastic hinges developed, the concrete cover was being held firmly by the quarry material. It detached suddenly when the surrounding quarry-run fill was removed. Examination of the pile in this location also revealed that a couple of the visible strands had completely ruptured. The final condition of the piles for System Test 2 was similar to that shown in Figure 5.

*Figure 4 – Extent of Damage for System Test 1. a) Cap Concrete Spalling. b) Damage Spread to Piles.*
Figure 5 -- Extent of Damage After Excavation in the Level Ground Pile of System Test 1.

Figure 6 plots the level ground and slope pile rotation profiles, obtained from the inclinometers placed along the pile, for the two system tests and for the different lateral displacements shown in the figure: ±54 mm (2.1 in.), ±108 (4.2 in.) mm and ±216 mm (8.5 in.). The location of plastic hinges in the piles at the pile-cap connection is evidenced in Figure 6 by the large change in rotation recorded immediately below the pile-cap interface. Plasticity also developed in the piles, in a section located below ground, but concentration of rotation was only evident near the end of testing for large lateral displacement around 216 mm (8.5 in.) for System Test 1 and 305 mm (12 in) for System Test 2. Plastic hinges for both tests developed at approximately 2 m (6.5 ft) or 3.3 pile diameters below ground for the level ground piles. This indicates that the clear height of the piles had little influence in the location at which the inground plastic hinges developed.

Figure 6 -- Rotation Profile for System Test. (Note: Pull is in the Upslope Direction and Push is in the Downslope direction)

For the slope piles, plastic hinges formed at approximately 2 m (6.5 ft) or 3.3 pile diameters below ground when displaced upslope, and about 2.5 m (8.2 ft) or 4.1 pile diameters when displaced downslope. This behavior indicates that the slope of the quarry-run fill run influenced the hinge location on the slope piles. The following sections describe the different relationship observed between rotation or average curvature in the piles and the applied lateral displacements.
**Pile Cap Behavior**

The development of different damage-states was monitored on the piles and on the pile-cap. Figure 7 plots the absolute value of the applied lateral displacement versus the absolute value of the pile-cap rotation for the level ground pile and for the slope pile for both tests. This figure also shows the displacement and corresponding pile-cap rotations at which a specific damage-state or critical strain was recorded in each pile or in the pile-cap adjacent to a particular pile. The lateral displacement versus pile-cap rotation responses of the level ground piles were similar in both tests, irrespectively of the direction of loading. Their responses were within 15 percent of the average response when displaced upslope. Furthermore, the response of the slope pile was similar to the level ground piles when displaced in the upslope (pull) direction but quite different when displaced in the downslope (push) direction. This is clearly the effect that the slope had on the response of these piles and is confirmed by the different location of the inground plastic hinge that developed in the slope pile when displaced in the downslope direction.

![Figure 7 -- Relative Cap Displacement Versus Cap Horizontal Displacement for System Tests.](image)

The key 1 percent dowel tensile strain, which marks the limit-state at the OLE in the pile-cap connection, was attained in all piles at a lateral displacement ranging between 40 mm (1.6 in.) and 50 mm (2 in.) when the pile-cap rotation ranged between 0.0045 and 0.0072 radians. It is clear from Figure 7 that the concrete cover flaking damage limit-states associated with the OLE occurred at lateral displacements larger than those recorded for the 1 percent tensile strain limit in the dowels. This is because of the shallow nature of the neutral axis depth in the piles at the pile-cap connection.

**Pile Behavior**

The behavior of the inground hinges in all piles is shown in Figure 8 in terms of the curvature (Φ) normalized by the diameter (D) and the applied lateral displacement. This figure also shows the points at which different damage limit-states were recorded in the piles during testing. The curvature was calculated at each inclinometer location. The estimation was carried out assuming a linear variation of the rotation along the two segments formed between the point at which the curvature was being estimated and the two consecutive inclinometers: one located above and one located below. The final curvature was obtained as the average from the estimated curvatures in these two segments. For convenience in Figure 8 curvature has been multiplied by the pile diameter, D. The incremental strain of 1 percent, which corresponds to an estimated total strain of 1.5 percent in the strands and is used by the POLA code as the limiting strain for the OLE, was attained in all piles at lateral displacements ranging between 50 mm (2 in.) and 60 mm (2.3 in.) when ΦD ranged between 0.003 and 0.008 radians. This is about the same lateral displacement in which the pile dowels attained the 1 percent tensile strain recommended in the POLA code as the limiting strain in the design for the OLE. A corollary of this finding is that any effort made to increase the pile-cap rotation at which a limiting strain is attained, will not result in improved system performance because the system displacement at OLE will be limited by damage in the inground plastic hinges.

The incremental strand tensile strain of 2 percent, which corresponds to a total tensile strain of 2.5 percent was measured at displacements ranging between 115 mm (4.5 in) and 145 mm (5.7 in.) in both tests. This strain corresponds to the strain limit for the CLE in the POLA code. Due to the total damage of the strain gages in the...
hinge zone it was not possible to identify the displacement corresponding to the strand fracture; however, after the excavation it was possible to observe that only a few strands had fractured during the test.

Figure 8 -- Normalized Pile In-ground Curvature Versus Cap Horizontal Displacement for System Tests.

The onset of spalling in the piles below ground was first detected in the slope pile of System Test 1 at 110 mm (4.3 in.) and also in the slope pile of System Test 2 at 153 mm (6 in.). This indicates that the onset of spalling and the 2.5 percent strands tensile strain occur in the same range of displacements. For the System Test 1, the CLE limiting-state was attained by the onset of spalling of the concrete cover whilst for System Test 2 the 2.5 percent strand tensile strain was reached first.

At the culmination of the tests the quarry-run fill was excavated around the piles to expose the damaged region where plastic hinges had formed. The concrete cover was found with signs of crushing but it was still firmly attached to the pile due to the confinement provided by the quarry-run fill. The cover spalled off when the quarry material was removed, exposing the transverse reinforcement, see Figure 5. Concentrated crushing of the concrete and of the quarry-run fill was also found along the exposed portion of the piles, where the two materials entered in contact.

**Hysteretic response and prediction of the response envelope**

The hysteretic response of the two system tests was characterized by relatively wide and stable loops, see Figure 9. Strength softening was observed in both tests at lateral displacements that ranged narrowly between 203 mm (8 in.) and 228 mm (9 in.) in both directions and in both tests. As it can be observed in Figure 7, at this range of lateral displacement the relative pile-cap rotation ranged between 0.032 radians and 0.058 radians for System Test 1 and between 0.029 radians and 0.047 radians for System Test 2.

A theoretical prediction of the monotonic response of the two system tests was made using the upper and lower bound p-y relationships provided by the geotechnical specialists. Such response is overlayed on the hysteretic response measured for the system tests. The predicted response matches well the initial portion of the experimental response for both tests. However, it underestimates the maximum lateral force, by about 30 percent in the downslope direction and by about 45 percent in the upslope direction in both tests. This large difference is discussed by Kawamata\(^2\) who suggested that p-y relationships for quarry-run fill should not be based on models using friction materials only, as interlocking can cause an apparent cohesion.
Decomposition of Forces
Upper, lower and best estimates of the pile shear forces were calculated by adjusting the point of inflexion in the piles. This because the strain gage data recorded at the coupling steel I beam experienced significant drift. The point of inflexion was determined from the rotations measured by the inclinometers. Figure 10 shows the shear forces obtained for each pile of System Test 1 for the cases of upslope and downslope loading. It is notable that this analysis is conclusive in that the largest shear force was carried by the pile in compression regardless if it was a level ground or a slope pile. This is essentially because of the influence of the axial force in the moment capacity of the plastic hinges developed in the pile and at the pile-cap connections. The analyses also indicate that the maximum axial forces in the piles were in the range of ± 454 kN (100 kip) or 3.1 A_g f'c and ±682 kN (150 kip) or 4.7 A_g f'c for both system tests. These forces have positive and negative signs as, according to the loading direction, the piles experienced cycles of compressive and tensile forces.
CONCLUSIONS

Two critical components of a pile-supported marginal wharf were built at full-scale and tested under quasi-static reversed cyclic loading. The piles in these two system tests were embedded in a sloping embankment built with gravel and quarry-fill run, thus replicating typical conditions in these kind of structures. One of the piles was located at the level ground and the other at the crest of the slope. The piles were coupled via a stiff steel beam that simulated the deck. Because these piles are shear critical, the main variable tested was the clear height of the piles. The tests were conducted to support the development of performance-based seismic design codes for pile supported marginal wharves, which require special considerations for the performance of the structure for several seismic hazard levels including the Operational Level Earthquake (OLE) and the Contingency Level Earthquake (CLE).

Both system tests showed excellent hysteretic response, with stable loops up to a lateral displacement of 203 mm (8.0 in.), when the relative pile-cap rotation in all connections exceeded 0.03 radians. In both tests, plasticity developed at the pile-cap connection and below ground at approximately 3.3 pile diameters for the level ground piles and for the slope piles when being displaced upslope and 4.1 pile diameters when the slope pile was displaced downslope. The presence of the slope had some influence in the response. In System Tests 1 and 2, the ratio of the downslope to upslope lateral force capacities were 0.87 and 0.89, respectively.

The 1 percent tensile strain limits in the pile-cap dowels, which are used in the Port of Los Angeles code to limit the performance of the pile-deck connection at OLE, were reached initially at a lateral displacement of approximately 50 mm (2 in.). The 1.5 percent tensile strain limit in the strands, which is used to define the same performance level but for the inground pile hinges, was reached at a slightly larger displacement. The concrete strain limits for the OLE were reached at lateral displacements greater than the corresponding tensile strain limits in the connecting dowels and in the strands. A corollary of this finding is that any effort made to increase the pile-cap rotation at which a limiting strain is attained, will not result in improved system performance because the system displacement at OLE will be limited by damage in the inground plastic hinges.

The strain limit set by the code of the Port of Los Angeles for the CLE event at the inground hinge, defined by the concrete onset of spalling, was recorded at displacements equal to and larger than 110 mm (4.3 in.) for System Test 1 and at displacements equal to and larger than 150 mm (6 in.) for System Test 2. The 2.5 percent strain on the prestressing strands, which is also a limit strain set by the code of the Port of Los Angeles for CLE events, occurred in a displacement range between 110 mm (4.3 in) and 140 mm (5.5 in.). That is the pile-cap connection and the piles attained their limiting strain within a narrow range of displacements. In System Test 1 the onset of concrete spalling defined the performance limit whilst for the System Test 2 the tensile strain of the strands was the limiting parameter.

The numerical methodology used for the estimations of the lateral force versus lateral displacement response of the system tests provided a good prediction in the initial stage of loading. However, the theoretical response
underpredicted the lateral force capacities at larger displacements for both the upper and lower p-y springs relationships recommended. As the structural elements models have been verified with experimental results having good approximation to the numerical estimations, it seems that additional considerations of the soil model are required to achieve a more reliable numerical representation of the soil-structure interaction problem for structures embedded in quarry-run fill material.

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REFERENCES


DISPLACEMENT-BASED PROCEDURES FOR SEISMIC DESIGN OF PILE-SUPPORTED WHARVES AT THE PORT OF LOS ANGELES AND THE PORT OF LONG BEACH

Omar A. Jaradat and M.J. Nigel Priestley

Synopsis: Over the past several years, the Ports of Los Angeles (POLA) and Long Beach (POLB) have undertaken numerous engineering studies to improve the seismic design of pile-supported wharf structures. It was concluded that the displacement-based seismic design methodology results in more robust and economical wharf structures. The displacement-based design allows plastic hinges to form at predetermined locations, which can be readily identified and repaired after an earthquake.

Both Ports sponsored and funded specialized studies and an experimental program at the University of California at San Diego (UCSD) to confirm seismic design assumptions. Also, port-wide ground motion studies were completed to develop acceleration and displacement response spectra and time-histories for the different levels of earthquakes specific to each Port.

Displacement-based seismic design procedures for pile-supported container wharves are included in two separate documents: “The Port of Los Angeles Code for Seismic Design, Upgrade and Repair of Container Wharves” and the “POLB Wharf Design Criteria”. This paper addresses the seismic, structural, geotechnical and soil-structure interaction aspects of these documents and discusses various studies that were undertaken to support the development of the displacement-based seismic design.

Keywords: Seismic design; Pile-supported piers and wharves; Displacement-based design; Prestressed concrete piles
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BACKGROUND

In the 1990s, it was acknowledged that the continuous increase in container throughput would require establishing consistent standards and guidelines for the design of container wharves to successfully execute necessary capital improvements. The understanding of seismic design requirements of pile-supported wharf structures has considerably increased in recent years. The Ports of Los Angeles (POLA) and Long Beach (POLB) recognized the need for uniform and port-specific guidelines which address performance-based design and soil-structure interaction to reflect the challenges of designing container wharf structures in an active fault region such as Southern California. Over the past several years, the POLA and POLB have undertaken numerous engineering studies to improve the way their pile-supported wharf structures are designed under seismic conditions. These studies, showed that the displacement-based seismic design methodology results in a wharf structure design with predictable behavior as it allows plastic hinges to form at predetermined locations, which can be readily identified and repaired after an earthquake.

The latest seismic design framework uses three levels of design earthquakes, namely the operating level earthquake (OLE), the contingency level earthquake (CLE), and code-level design earthquake (DE) with associated performance requirements. The performance requirements are expressed in terms of material strain limits. Both Ports sponsored and funded specialized studies and an experimental program at the University of California at San Diego to confirm seismic design assumptions. Also, port-wide ground motion studies were completed at both Ports separately to develop acceleration and displacement response spectra and time-histories for all three earthquake levels.

Over the past few decades, the POLA and POLB have evolved into major economic forces for the United States, ranking first in the nation in shipping traffic, and ranking third in the world when combined. The past four decades have seen major development in seismic design of pile-supported structures. The following is a summary of the major milestones in the development of seismic design criteria for pile-supported wharf structures (Yin et al 2007):

- In the 1960s and 1970s, wharf structures were built with battered piles to resist lateral seismic loads. Wharf structures were designed according to building codes, as no seismic design criteria specifically for wharves were available. Equivalent static loads were used for the seismic design using a force-based approach. However, wharf structures are different when compared to building structures; specifically, seismic behavior of wharf structures is primarily controlled by soil-structure interaction especially in the presence of liquefiable soil and sloped embankments. Uniform seismic hazard parameters did not exist, which essentially meant that adjacent wharf structures could have different hazard criteria. Most earthquake research efforts were concentrated on producing design criteria to protect building structures against collapse.

- The damage due to 1971 San Fernando earthquake in California showed that wharf piles needed to withstand not only higher lateral seismic forces, but also simultaneous seismic response in two orthogonal directions (Penzien, 2003). Past earthquakes revealed the vulnerability of the then-prevalent container wharves with battered piles and sloping embankments, and demonstrated the increased risk to port facilities. It was believed that adding more battered piles in two orthogonal directions could improve the wharf seismic lateral capacity. This, however, led to increased construction difficulties.

- The 1995 Kobe earthquake in Japan demonstrated how disruptive a seismic event could be on port operations. Although 75% of the facilities were recovered one year after the event, the initial impact to the business at the Port of Kobe was devastating and some of the Port’s business losses were never recovered.
• In 1990 a Seismic Workshop was sponsored by the POLA to address seismic risk identification, design guidelines, and seismic risk reduction and response plan (Wittkop et al, 1990). In this workshop, the probability of exceedance requirements was established for OLE and CLE. By 1995, the Displacement-Based Design (DBD) methodology gained acceptance in the engineering community. In 1996 the use of the DBD approach with defined strain limits was introduced in a book on seismic design of bridges (Priestley et al, 1996).

• In 1998, the POLA sponsored its first full-scale pile-deck connection test at the UCSD (Sritharan and Priestley, 1998). Based on the test results, the Port developed standard pile-deck connection details used for many construction projects.

• In 2000, the POLA began focusing on the enhancement of wharf seismic design technology. POLA staff with the assistance of outside experts in the structural, geotechnical and seismological disciplines began developing a draft of a code for the seismic design of container wharves. A Technical Advisory Board (TAB) was established to review the Code, which comprised of Professors Nigel Priestley (Structural), Geoffrey Martin (Geotechnical), and Dr. Norm Abrahamson (Seismological).

• In 2003, the POLA organized a workshop to introduce a draft version of its first seismic design code to a panel of consultants. The structural and geotechnical consultants had the opportunity to ask questions to the POLA staff and TAB members. Input from consultants assisted the Port in making the final revisions to the code. Subsequently, a final draft of the code was issued after being reviewed by the TAB. Finally, in May of 2004, the “Code for Seismic Design, Upgrade and Repair of Container Wharves” (POLA, 2004) was adopted and approved by the Port’s Board of Harbor Commissioners.

• In 2005, the POLA and the Coasts, Oceans, Ports, and Rivers Institute (COPRI) of American Society of Civil Engineers (ASCE) co-sponsored a workshop (POLA, 2005) to present the seismic code to the engineering community and solicit their input. Afterwards, the POLA and POLB commissioned port-wide ground motion studies to develop consistent seismic ground motion recommendations for structures within POLA and POLB for OLE, CLE and DE events. A fault map depicting the location of the major fault zone was created and probabilistic seismic hazard analysis of each Port was performed using the latest attenuation models used in California. The ground motion studies produced horizontal and vertical Uniform Hazard Spectra for firm-ground conditions and design response spectra for a range of damping values that can be used for wharf modal response spectrum analysis (EMI, 2006a/b, 2008, 2010, and 2011a/b).

• In 2006, both Ports sponsored and funded specialized studies and an experimental program at UCSD to confirm seismic design assumptions. A series of tests were carried out on different key components of the structural wharf system as part of a program aimed to improve seismic design and construction of pile-supported wharves.

• In 2008, the POLA sponsored experimental program at UCSD (Bell, et al, 2008), (Krier, et al, 2008), and (Kawamata, et al, 2008).

• In 2010, the POLA published the first update of 2004 seismic design code (POLA, 2010). This update was based on the experimental program results and findings of specialized studies.

• In 2009 and 2012, the POLB published versions 2 and 3 of the wharf design criteria based on the experimental program results (POLB, 2009) and (POLB, 2012).

• Currently, ASCE is in the process of balloting the first national standard on the Seismic Design of Pile-Supported Piers and Wharves (ASCE, 2012). This standard includes criteria similar to POLA and POLB seismic design criteria.
DISPLACEMENT-BASED SEISMIC DESIGN REQUIREMENTS

After years of uncertainty about the DBD methodology, port authorities began to implement the new criteria into their seismic design requirements. The objective is to provide a design that safeguards life and protects against major structural failures in addition to limiting damage and minimizing economic losses due to earthquakes.

The displacement-based seismic design approach is based on strain limit criteria and performance objectives associated with three levels of structural damage for typical wharf structures at POLA and POLB. These wharf structures are supported on prestressed concrete piles ((POLA, 2010) and (POLB, 2012)):

a. Operating Level Earthquake (OLE): No significant structural damage. Damage location to be visually observable and accessible for repairs. Minimum or no interruption to wharf operations during repairs may occur. After an OLE event, the intent of the criteria is to limit repairable damages at pile top hinge. Minor plastic hinging in the pile top with occasional light spalling of the cover concrete is expected only directly below the deck, where the damage can be repaired minimizing impact on operations. Although a limited amount of ductile behavior is expected for piles in the ground, at this location no spalling of concrete cover is expected.

b. Contingency Level Earthquake (CLE): Controlled inelastic structural behavior and limited permanent deformations. Damage location to be visually observable and accessible for repairs. Temporary or short-term loss of operations may occur. After a CLE event, the intent of the Code is to have controlled repairable damage at pile top hinge. Plastic hinges in piles with spalling of the cover concrete is expected below the deck and the damage needs to be repaired. Piles designed to CLE strain limits have approximately 50% more reserve displacement capacity at pile top hinge before the moment capacity of the pile deteriorates significantly. The in-ground hinge has more stringent requirements than the top hinge. Although a limited amount of ductile behavior is expected for piles in the ground, at this section no spalling of concrete cover is expected. The temporary interruption of wharf operations may occur for duration of time that depends on several factors such as type of damage, damage locations, site access, availability of qualified repair contractors and engineers.

c. Design Earthquake Level (DE): Safeguard life and against major structural failures. After DE, the intent of the Code is to safeguard life and against major structure failures. It is expected that piles may be severely damaged, but this damage should not cause major structural failure to safeguard life. The loss of wharf operation is expected and the duration of recovery and repair depends on several factors such as type of damage, damage locations, site access, availability of qualified repair contractors and engineers.

Table 1 shows the three ground motion levels included in POLA and POLB seismic design criteria. For OLE and CLE, the probability of exceedance requirements were established based on recommendations of “Proceedings of POLA Seismic Workshop on Seismic Engineering,” (Wittkop et al., 1990). This workshop was sponsored by the POLA with participation by nationally and internationally renowned experts in various fields including earthquake engineering. For DE, the ground motion was developed using approach outline in ASCE 7-05 Section 11.2 (ASCE, 2005).

<table>
<thead>
<tr>
<th>Earthquake</th>
<th>Probability of Exceedance</th>
<th>Return Period (Yrs)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Operating Level Earthquake (OLE)</td>
<td>50% in 50 years</td>
<td>72</td>
</tr>
<tr>
<td>Contingency Level Earthquake (CLE)</td>
<td>10% in 50 years</td>
<td>475</td>
</tr>
<tr>
<td>Design Earthquake Level (DE)</td>
<td>“Design Earthquake” as defined in ASCE 7-05 Section 11.2.</td>
<td></td>
</tr>
</tbody>
</table>

In order to achieve the performance goals, performance criteria were provided in terms of material strain limits for each earthquake level. Compliance with the specified strain limits controls and limits the damage correspondingly to meet the seismic design purpose and intent. In addition, using the appropriate analysis approach, soil-structure interaction and structural model calibration are crucial to achieve the performance goals.

Under lateral seismic movement, plastic hinges form when the moment exceeds the plastic moment capacity of the pile section. Typically, there are three locations for the plastic hinge to form: 1) below deck soffit and top of pile, 2) below ground level within 3 to 5 times pile diameter, and 3) below ground level deeper than 10 times pile diameter, see Figure 1. Also, deep in-ground plastic hinges are expected to form adjacent to sliding soil layers.
Typically, solid prestressed concrete piles are used for the construction of container wharf structures at the POLA and POLB, see Figure 2. Other types of piles such as round hollow concrete piles, steel pipe piles and steel pipe piles filled with concrete are typically not used. However, the strain limits requirements are provided in case the use of these piles is necessary. The solid concrete prestressed precast piles sections have three material properties including concrete, dowels and prestressing strands. Each material has different strain limits at each performance requirement.

The wharf structure is required to be designed as a ductile system. The pile-to-deck interface forms an integral part of the wharf structure, and shall be designed for ductile behavior. The structural system is designed based on the strong beam (deck), weak column (pile) frame concept. The plastic hinges are designed to occur in the piles and not in the deck. This concept is different from the strong column-weak beam structural system concept used for the design of buildings. Furthermore, capacity design is required to ensure that the dependable strengths of the protected members exceed the maximum feasible demand based on upper-bound estimates of the flexural strength of the piles plastic hinges.

At POLA and POLB, battered piles are not used for the construction or repair of wharf structures. When subjected to seismic loading, wharves with battered piles are less ductile and have lower displacement capacities and higher deck shears, attract higher seismic loads, and cause uplift forces in the piles, compared to wharves supported on vertical
Historically, the performance of conventional wharves with battered piles under seismic loading has been poor. Severe damage was observed for battered piles due to the Loma Prieta earthquake in 1989. Wharves with battered piles experienced damage associated with high deck shear, and pile uplift forces (Werner, 1998).

The DBD approach is used to verify that the displacement capacity of the piles exceeds the displacement demand at each of the three performance levels. The POLB Wharf design Criteria shows a flow diagram for the analysis and design steps, see Figure 4. After the design for service static loads is completed, the seismic analysis and design is performed for OLE, CLE and DE. The seismic design may require additional pile rows or a modified pile layout. A model including the effective section properties, seismic mass, and soil springs is prepared. Nonlinear static pushover analysis method provides the displacement capacity based on material strain limits. This method incorporates soil deformation into the total displacement capacity of the pile. The pushover model shall use effective material and section properties while incorporating the soil stiffness with nonlinear upper and lower bound p-y springs. From the analysis, displacement demand should not exceed the displacement capacity. If the demand exceeds the capacity the design must be revised. The global analysis shall account for wharf torsional plan eccentricity, soil structure interaction, multi-directional effects of the ground motion and the interaction between adjacent wharf segments. Displacement demand for regular wharves is determined by the Substitute Structure method, or Modal Response Spectra Analysis. For wharves with irregular geometry, special cases, or when demand/capacity ratios from Modal Response Spectra Analysis are too high, Nonlinear Time-History methods may be employed for the global model to verify the analysis results. If the pile displacement demand is less than the capacity, pile kinematic load, the pile shear, the beam/deck pile joint and P-Δ effects are checked.

The wharf seismic mass is a critical component in determining the seismic displacement demand. Overestimating or underestimating the seismic mass will result in proportionally overestimating or underestimating the seismic displacement demand. The seismic mass includes:

1. Wharf self-weight and added dead load.

2. One third of the pile mass between the wharf deck soffit and 5 times pile diameter below the dike surface because the pile mass close to the deck influences the seismic response. This portion of the pile mass is added to wharf total seismic mass and lumped at the centerline of the deck.

3. Ten percent of the uniformly distributed live load not to exceed 100 psf (50 kPa) provides reserve for unforeseen circumstances in case of more often or occasional storage of containers on the wharf deck. At the POLA and POLB the wharf deck is designed for 1,000 psf (50 kPa) of live load to primarily support occasional short term localized loading by concentrated loads due to containers. Typically, at POLA and POLB the wharf is not used for long term storage of containers. If the wharf structure is used for long term storage of containers, higher percentage of uniform live load should be used as part of seismic mass.

4. Hydrodynamic mass of piles could contribute the wharf seismic mass. At POLA and POLB the wharf structures, typically supported by 24-inch (0.61 m) piles and the hydrodynamic mass is negligible.

5. The crane mass contribution to the wharf seismic is considered by including part of the crane mass positioned within 10 feet (3.28 m) of the wharf deck. Also, at least, 5% of the total crane mass is included in the wharf seismic mass.

Cranes positioned on the wharf may affect the seismic behavior of the wharf structure. Crane-wharf interaction is a key component of the seismic design of wharf structures. Seismic response of the crane may increase or decrease the wharf seismic displacement demand and shear key forces between wharf segments. The POLA sponsored a study to evaluate the crane-wharf interaction (Blandon, 2007). The study results showed that crane-wharf interaction may be ignored if the crane translational period with the maximum participating mass is larger than twice the elastic period of the wharf structure based on cracked section properties.

The pile-to-deck connection is one of the critical components that determine the overall behavior of the wharf structures. The analytical model needs to include the critical elements of the wharf structural system including the deck-to-pile connection as shown in Figure 3. For typical POLA and POLB prestressed concrete piles, the pile has two primary section properties: 1) concrete with dowels only at the top of the pile, and 2) concrete with prestressing
strands mainly at the lower portion of the pile. Piles are connected to the deck using dowels. The first pile element modeled below the deck soffit within 16 inches will have properties of a reinforced concrete section and the remainder of the pile will have properties of a prestressed concrete section. The strain in dowels due to the formation of plastic hinge at the top of pile is expected to continue into the deck since pile-deck connection is integrated with the development of dowels. The interface between the deck and the pile should not be considered rigid because of the strain penetration effects. The effective top of the pile, where the pile is considered fixed to the infinitely rigid deck, should be located at a distance $L_{sp}$ into the deck, see Figure 3, to account for strain penetration. This additional length is for displacement only, and the maximum moment is modeled to form at the deck soffit. In the demand model the additional length will increase displacement demand; however, in the nonlinear static pushover model to determine pile displacement capacity the additional length will not be a factor because plastic hinge will be modeled to occur at the soffit of the deck. The pile elements below mudline should be modeled using cracked section properties and incorporate lateral soil stiffness (p-y springs) to account for soil-structure interaction.

![Figure 3 -- Modeling of Pile-to-Deck Connection (POLA, 2010), (POLB, 2012)](image)

The pile strain limits at the considered performance level, as shown in Table 2, POLB WDC Wharf, are used to determine the displacement capacity for the top plastic hinge and the in-ground plastic hinge; the smaller value is used for the pile displacement capacity. Displacement capacity is calculated for upper bound and lower bound soil conditions to determine the controlling case and compare it to the corresponding displacement demand case.

As stated in POLA (2010), strain limits for OLE, CLE and DE are different because of the different performance goals at each level. For OLE, the intent is to limit repairable damages at the pile top. For CLE, the intent is to have controlled repairable damage at pile top. Plastic hinges in piles with spalling of the cover concrete are expected below the deck. Piles designed to CLE strain limits have approximately 50% more reserved displacement capacity at the pile top hinge before the moment capacity of the pile deteriorates significantly. The in-ground hinge has more stringent requirements than the top hinge. Although limited ductile behavior is expected for piles in the ground, no spalling of concrete cover is expected.

The temporary interruption of wharf operations may occur for a duration of time that depends on several factors such as type of damage, damage locations, site access, availability of qualified repair contractors and engineers. For DE, the intent is to safeguard life and against major structure failures. It is expected that piles may be severely damaged, but this damage should not cause major structural failure and the structure is expected to support gravity loads. The loss of wharf operation is expected and the duration of recovery and repair depends on several factors such as type of damage, damage locations, site access, availability of qualified repair contractors and engineers.
The displacement capacity for a pile is determined from the yield displacement, and the plastic displacement, at the specified strain limit. The plastic displacement depends on the plastic hinge length, as outlined in POLA (2010) and POLB (2012). Plastic hinge length values are based on empirical data obtained from testing sponsored by the POLA. Much of the design procedure is discussed in detail in Priestley et al (2007).

Table 2 -- Strain Limits (POLA 2010, POLB 2012)

<table>
<thead>
<tr>
<th>Component Strain</th>
<th>Design Level</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>OLE</td>
</tr>
<tr>
<td>Top of pile hinge concrete strain</td>
<td>$\varepsilon_c \leq 0.005$</td>
</tr>
<tr>
<td>In-ground hinge concrete strain</td>
<td>$\varepsilon_c \leq 0.005$</td>
</tr>
<tr>
<td>Deep In-ground hinge (&gt;10D_p) concrete strain</td>
<td>$\varepsilon_c \leq 0.008$</td>
</tr>
<tr>
<td>Top of pile hinge reinforcing steel strain</td>
<td>$\varepsilon_s \leq 0.015$</td>
</tr>
<tr>
<td>In-ground hinge prestressing steel strain</td>
<td>$\varepsilon_p \leq 0.015$</td>
</tr>
<tr>
<td>Deep In-ground hinge (&gt;10D_p) prestressing steel strain</td>
<td>$\varepsilon_p \leq 0.015$</td>
</tr>
</tbody>
</table>

For solid round or octagonal piles.

Steel pipe deck connection shall be accomplished by concrete plug with dowel reinforcement.

Definitions:

- $D_p$ = Pile diameter
- $\varepsilon_c$ = Concrete compression strain
- $\varepsilon_s$ = Steel tensile strain
- $\varepsilon_{smd}$ = Strain at maximum stress of dowel reinforcement
- $\rho_s$ = Effective volumetric ratio of confining steel
Design for Service Static Load

Determine Performance Criteria (OLE, CLE, DE)

- Calculate effective Section & Material Properties ($A_e$, $I_e$, $E_s$)
- Soil Springs (Upper Bound, Lower Bound)
- Seismic Mass
- Nonlinear Properties ($M$, $\phi$, $\theta_p, L_p$)

Nonlinear Static Pushover Analysis

Pile Displacement Capacity $\Delta_c$

Displacement Demand, $\Delta_d$

$\Delta_d = \Delta x DMF$

Irregular Structure or Special Case

No

Yes

Yes

Revise Design

Check (Optional)

Nonlinear Time-History Analysis

Preliminary Design: Equivalent Lateral Stiffness Method

Elastic Stiffness Method

Substitute Structure Method

Modal Response Spectra Analysis

Component Capacities

Deck Expansion Joint

Kinematic Load

Seismic Detailing Requirements

Figure 4 -- Flow Chart for Seismic Design Analysis and Steps (POLB, 2012)
EXPERIMENTAL PROGRAM

The POLA sponsored an experimental program at UCSD. A series of tests were carried out on different key components of the structural wharf system as part of a program aimed to improve the seismic design and construction of pile-supported wharves. Four tests were carried out on pile deck connections, and three tests were carried out on prestressed piles embedded in quarry material representative of dikes at POLA. The first set of tests, A.1 and A.2, examined two different existing connection details from older wharf designs, see Figure 5 and Figure 6. The second set of tests, B.1 and B.2, used octagonal prestressed pile sections with the recommended reinforced pile-deck connection for secondary and primary seismic piles in order to verify design performance under lateral load, see Figure 7 and Figure 8.
The full-scale tests on prestressed piles embedded in quarry material were called the Cantilever Pile Test, System Test 1 and System Test 2 (called soil pit tests), see Figure 9. The Cantilever Pile Test consisted of a free head prestressed pile subjected to lateral load that was tested to verify the actual procedure for evaluating the inertial soil-pile interaction and the design strains recommended in the Code at in-ground hinge locations. System Test 1 consisted of two piles connected at the head by a stiff beam to model fixed head conditions, which are the most
Design of Prestressed Concrete Piles in Marine Structures

realistic conditions for wharf primary seismic piles. The third test (System Test 2) was similar to System Test 1, but had increased ground clearance.

Detailed description of the tests setup and test results for existing pile deck connections tests A.1 and A.2, new pile deck connection tests B.1 and B.2 and Cantilever Pile, System Tests 1 and 2 are presented in UCSD test reports by Bell, et al. (2008), Krier, et al. (2008), and Kawamata, et al. (2008) respectively. Interpretations of all test results are summarized in Priestley, (2008).

Figure 9 -- Cantilever Pile Test Set-Up

Design Recommendations for New Piles

The test data provided useful confirmation of expected response, but indicated that some changes should be made to the preliminary recommendations for performance determination used in previous versions of POLA and POLB ((POLA, 2004), (POLB, 2009)). Revised recommendations are listed below (Priestley, 2008).

1. Strain Penetration and Plastic Hinge Length

Data from the tests indicated that the preliminary estimate for the strain-penetration length overestimated the experimental response. The revised estimate of the strain penetration length for the top of pile is:

$$ L_{sp} = 0.1 f_{ye} d_b $$

where the yield stress $f_{ye}$ is expressed in ksi, and $d_b$ is expressed in in$^2$.

As a consequence, the plastic hinge equation applicable for the pile top hinge is

$$ L_p = 0.08 L + 0.1 f_{ye} d_b \geq 0.2 f_{ye} d_b $$

The test data from the Cantilever and System Tests in quarry-run material indicated that the plastic hinge length at the in-ground hinge was not a constant value, but decreased as the peak curvature increased. In the initial stages of response the reduction of plastic hinge length was comparatively gradual, and the previously recommended value of $2D_p$ was a conservative estimate. At curvatures corresponding to compression fiber surface strains exceeding 0.009 the plastic hinge length appeared to condense rapidly. On the basis of these findings, it is recommended that the in-ground plastic hinge length be retained at the previously recommended level:

$$ L_p = 2D_p $$
2. Strain Limits

The structural limit states are defined by the material strains and many of these strain limits were determined based on testing, as discussed below:

$\varepsilon_{sd}$ for all piles:

The reduction in plastic hinge length means that crack widths are less for a given dowel strain. As a consequence the dowel strain limits can be increased, particularly for the OLE. The following recommendations will not produce larger residual crack widths at the performance levels than with the previous equations defined in the 2004 POLA Code.

- **OLE:** $\varepsilon_{sd} = 0.015$
- **CLE:** $\varepsilon_{sd} = 0.06$ but not greater than $0.6\varepsilon_{umd}$

The component tests and the in-situ field tests at UCSD’s Englekirk center invariably failed by fracture of the dowel reinforcement at high strains. This occurred despite moment-curvature analyses results, which indicated that in all cases concrete compression strain limits should govern based on standard confinement theory. It appears that, as a consequence of the confinement provided at the top of the pile by the deck and the rapid pick-up of prestress in the pile, cover spalling is significantly reduced. Because of this, the expansion of the spiral reinforcement in the plastic hinge region is inhibited, and a failure based on concrete compression strain limits could not occur.

$\varepsilon_c$ for solid concrete piles at OLE:

Although many sources in the literature limit the extreme concrete compression strain to 0.004, the value of 0.005 is adopted in the Code. This value is based on full scale tests of the pile-deck connection and takes into account the high moment gradient in the critical landward piles, which results in additional confinement of cover concrete at the critical section. This defers the onset of cover spalling. Some cosmetic repairs may be required after an OLE, but they can be performed without disruption of the wharf operations.

$\varepsilon_c$ for solid concrete piles at CLE:

Assuming that dowel fracture occurred at an average strain of 0.08, the moment-curvature analyses based on zero axial load indicated that the concrete compression strain would be about 0.046. Requiring a factor of safety of 1.5, this would imply a safe compression strain limit of about 0.03, though it would appear that higher strains could be sustained.

It does not appear that the equation originally incorporated in the 2004 POLA Code that was based on the energy balance approach in Mander, et. al. (1988) is valid for the prestressed piles used by POLA. On the other hand, there should be some limit on the compression strain to cope with unlikely situations where seismic piles with much greater clearance from dike to deck are relied on for primary seismic resistance.

As a consequence of these arguments, the following equation is suggested for compression strain limits for the top of pile under CLE:

$$\varepsilon_c = 0.005 + 1.1\rho_s \leq 0.025$$

where $\rho_s$ is the volumetric ratio of transverse confinement in the pile.

For standard POLA piles with W20 spirals at 2.5 inches on-center, the upper limit will apply. In combination with the reduced plastic hinge length of Equation (2), the plastic rotation will still be less than with the 2004 POLA code. Although spalling of the cover concrete at the in-ground hinge would violate the performance criterion for CLE, an extreme fiber strain of 0.008, which is significantly higher than the unconfined spalling strain of 0.005, is permitted. This is because confinement of the cover concrete by the surrounding soil at the critical section enhances the spalling strain of the in-ground hinge. Peak confinement pressure exerted by the soil on the cover concrete on the compression side of the pile are of similar magnitude to the confining pressure provided to the core concrete by spiral reinforcement. The Cantilever Pile Test and the System Tests have confirmed this behavior, and indicated that cover concrete was able to fully participate in resisting flexural and axial compression stress to strains higher than the recommended limit of 0.008. Therefore, the following equation is recommended for concrete compression strain limits for the in-ground hinge under CLE:
The Cantilever Pile Test and the System Tests indicated that the strand limit strains for the OLE and CLE performance levels were excessively conservative, as inspection of crack widths after testing the cantilever pile to the CLE limit, and then exposing the hinge region indicated only extremely fine crack widths. As a consequence the following strain limits are recommended:

OLE: $\varepsilon_p = 0.015$
CLE: $\varepsilon_p = 0.025$

Strand limit strains for the in-ground hinge at OLE of 0.015 are compatible with serviceability criteria. The strand limit strain of 0.025 for CLE is conservative and should not result in any residual crack width causing corrosion potential, particularly when it is realized that strand strain predicted by a “plane-section” analyses will yield conservative results.

Strains in Secondary Seismic Piles:
The strain limits for secondary seismic piles for OLE and CLE are typically limited to the OLE strain limits for seismic piles. The pile-deck connection for secondary seismic piles can be considerably simplified when the rotational demand on the pile-top plastic hinge is small. When the deck displacement reaches the CLE limits for the seismic piles, the secondary seismic piles will usually be subjected to strains that are lower than the OLE strain limits. Because of this, the pile-deck connection for the secondary seismic piles can be simplified. Testing has shown that the simplified connection has very little deterioration at the OLE limit strain and a very large reserve of displacement capacity beyond OLE strain limits. However, if the strains in the secondary seismic pile top hinge are close to or exceed the OLE strain limits the pile-deck connection may be detailed with adequate confinement similar to seismic piles, allowing the CLE strain limits for seismic piles to be used.

3. Cover Concrete at the In-Ground Hinge

Previously the moment capacity of the in-ground hinge was estimated based on the assumption that the cover concrete was completely unconfined. The test results for both the Cantilever and System Tests in quarry run material indicated that the cover concrete was effectively confined up to compression strains exceeding the CLE limit. The soil provided confining stresses of similar magnitude to levels expected from a W20 spiral at 2.5 inches on center. Including the compression resistance of the cover concrete results in flexural strength increase in the vicinity of 30%, and helps to explain the increased lateral strength of the embedded piles compared to estimates based on spalling cover. It is recommended that confined cover concrete be included in moment-curvature analyses of in-ground hinge characteristics.

4. Equivalent Viscous Damping

System Test 1 enabled estimates of equivalent viscous damping to be made for incorporation in substitute structure analyses. These estimates indicated that at the early stages of response, damping attributable mainly to soil hysteresis was equivalent to 10%, based on secant stiffness. Making adjustments based on tangent-stiffness elastic damping (expected to be conservative, since radiation damping is ignored) the following damping equation is recommended for substitute-structure design or analysis:

$$\xi = 0.10 + 0.565 \left( \frac{\mu - 1}{\mu} \right)$$ [6]

In carrying out inelastic time-history response analyses, it is recommended that 10% tangent stiffness elastic damping be used, and a Takeda hysteresis rule with hysteretic parameters $\alpha = 0.3$, $\beta = 0.5$ be used.

5. Pile Shear Capacity

The tests showed very good behavior in shear, with little evidence of shear failure, even at displacements beyond the OLE and CLE limits. However, based on the UCSD shear equations, which use ductility to reduce the shear capacity of concrete, the demand-capacity ratios of the seismic piles were approaching and sometimes exceeding 1.0. The shear equations were overly conservative, particularly in predicting concrete strength, so it is recommended
that the ductility factor for concrete be revised to be less conservative. The new curvature ductility factor is defined by the following:

\[ \mu \phi \]

**Figure 10 -- Curvature Ductility Factor**

For the DE, the shear demand will essentially remain the same as CLE, but the capacity will diminish due to increased ductility and a larger distance to the neutral axis, which will decrease the contributions of the concrete and the spiral reinforcement to the total shear capacity. However, test results indicate that at excessive displacements and ductility, shear failure does not occur even though the calculated shear strength has been exceeded. Furthermore, a conservative design-based estimate of pile shear strength is used in this approach, incorporating a shear strength reduction factor of 0.85, and conservative estimates of concrete, axial force, and transverse reinforcement components for shear strength.

Determining the actual shear strength would result in an increased value for predicted shear strength of about 38% higher than the design-based value, which more than compensates for possible reduction in shear strength resulting from increased ductility demand at the DE. Typically, the strength reduction factor of \( \phi = 0.85 \) for shear capacity provides adequate design conservatism. For OLE and CLE, shear damage should be prevented, so the strength reduction factor provides a suitable level of conservatism. For the extreme case of DE, damage is expected, so the pile shear capacity may be calculated without the strength reduction factor in order to evaluate the actual shear capacity of the pile. Conservatism will still exist since the design equations for shear strength are conservative, even if strength reduction factors are not applied. Therefore, for DE the strength reduction factor may be ignored.

6. Existing Pile-Deck Connection

In Test A.1, the existing piles achieved ductility greater than 8 in both directions. Based on the results from Test A.2, the existing pile connections have sufficient capacity to support crane loading after being subject to OLE displacement.

**CONCLUSIONS**

The POLA and POLB have dedicated considerable resources to develop seismic engineering criteria to protect its investment and allow continuous business growth. Displacement-based procedures for seismic design of pile-supported wharves are implemented in the design and construction of many recent projects at the POLA and POLB. Loss of business is expected to be minimized by following these criteria. It is anticipated that this Criteria will provide more uniformity, focus on design, quality assurance, and minimize construction costs.

Tests A.1 and A.2 showed that the existing piles could support the specified crane load at OLE displacement. Tests B.1 and B.2 and the soil pit tests provided information that assisted in developing design recommendations for future wharf design. From these tests, new equations for strain penetration length and plastic hinge length were developed for the top hinge. Also, the strain limits proposed in the (POLA, 2010) and (POLB, 2012) were revised based on test results. A new equation for equivalent viscous damping for substitute structure analysis is
recommended, as well as new damping coefficients for the Takeda model used in nonlinear time-history analysis. The tests also showed very good behavior in shear, although the calculated shear capacity based on the UCSD equations was less than the shear in the section. Based on this, the ductility factor for concrete shear capacity was modified to provide more shear capacity.
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SEISMIC STABILITY OF MARINE PIERS BUILT WITH PRESTRESSED CONCRETE PILES

Stuart Stringer and Robert Harn

Synopsis: 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) 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. (2) 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. (3) 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, while a stability index limit of 0.1 will protect against significant P-Δ displacement amplification variability when increased analytical accuracy is desired. (4) For typical pile lengths and axial loading the P-Δ sensitive behavior is expected and the stability index limit will likely control the displacement capacities over material strain limits. Finally a simple procedure was proposed to help identify when a pier is potentially at risk from instability due to dowel bar fracture.

Keywords: Prestressed Concrete Piles, Seismic Stability, Marine Piers
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**INTRODUCTION**

Piers and wharves are pile-supported platforms, where piers are constructed perpendicular or nearly perpendicular to the shore and wharves are constructed parallel or nearly parallel to the shore. There are three seismic design codes currently used in the United States that are applicable to piers and wharves. Two of the codes, the Port of Los Angeles, 2010, Code for Seismic Design Repair and Upgrade of Container Wharves, (POLA), and The Port of Long Beach Wharf Design Criteria, Version 3.0, 2012 (POLB), focus on marginal wharves. While the third code, Marine Oil Terminals Engineering and Maintenance Standards (MOTEMS) which is Chapter 31F of the California Building Code, which was developed by the California State Lands Commission, addresses marine oil terminal piers and marginal wharves. A fourth code, being developed by the American Society of Civil Engineers (ASCE), “Seismic Design of Pile Supported Piers and Wharves” (ASCE Standard), expected to be published in 2013, will incorporate and expand on the provisions of POLA, POLB and MOTEMS.

For marginal wharves seismic $P-Δ$ sway instability is typically not an issue, as the very short landslide piles effectively brace the structure, greatly reducing the global deck displacements. For piers however, large sway displacements are expected under seismic attack because of the preponderance of long piles and the flexibility of the plumb pile moment frame system. Consequently the seismic stability of a pier may be more dependent upon the integrity of the pile-to-deck connections and the in-ground plastic hinges than for a marginal wharf, and the loss of capacity at either the pile-to-deck connection or the in-ground hinge could precipitate into global collapse of the system.

It is therefore important to evaluate the conditions that can cause collapse of typical piers, so that once identified, analysis and design recommendations can be made to help ensure collapse prevention performance under strong ground shaking. The focus of this study is then to identify conditions such as ground motion characteristics, gravity loading, pier geometry, and inelastic demands that increase the risk of global side-sway collapse, and propose design considerations to help mitigate the threat of collapse.

**CASE STUDIES**

Six case studies were performed to evaluate a common type of pier construction used in the west coast of the United States using prototypes with three different pile lengths under two levels of loading.

The prototype piers consist of seven-pile bents, spaced at 20 feet (6.1 m) on center with a 100 feet (30.5 m) wide, 30 inch (750 mm) thick reinforced concrete flat-slab deck supported on 24 inch (610 mm) piles as shown in Figure 1. The typical piles, driven into dense sand, are spaced at 15 feet (4.6 m). The properties of the piles and pile-to-deck connection, which consists of mild steel dowel bars grouted into metal ducts cast into the end of the pile are provided in Table 1. All properties of the prototype piers were held constant except for the pile length, measured as the depth to mudline, which was taken as 10ft (3.0 m), 30ft (9.1 m), and 60ft (18.3 m). The live load on the deck was 800 psf (38 kPa) which is typical for a pier of this type.
Table 2 summarizes the properties, and loading of the prototype structures. It can be seen that a wide range of fundamental periods are included in this study, from 0.6 to 4.8 seconds.

The six case studies evaluated the sensitivity of the prototype piers to seismically induced P-Δ instability using 2-dimensional nonlinear static (pushover) analyses and nonlinear response-history analysis (NLRHA) of a single pier bent. While the single bent model represents a simplification of the actual pier structure it nonetheless provides insight into the nonlinear and dynamic behavior that may lead to collapse. For each of the prototype piers in Table 2 two NLRH analyses were conducted for each ground motion where P-Δ effects were either included or excluded. The exclusion of P-Δ effects results from determining the equations of equilibrium in the initial undeformed configuration, whereas the analyses that account for P-Δ effects calculate equilibrium in the deformed configuration.

Table 1 -- Pile and Connection Properties

<table>
<thead>
<tr>
<th>Pile</th>
<th>Connection</th>
</tr>
</thead>
<tbody>
<tr>
<td>Diameter</td>
<td>24in (610mm) octagonal</td>
</tr>
<tr>
<td>Cover</td>
<td>3in (76mm)</td>
</tr>
<tr>
<td>Concrete Strength</td>
<td>8ksi (55MPa)</td>
</tr>
<tr>
<td>Reinforcement</td>
<td>(22) ½ in - 270ksi (1,860 MPa) prestressing strand (8) #10 – gr.60 (414MPa) mild steel</td>
</tr>
<tr>
<td>Effective Prestress</td>
<td>1,200psi (8.2MPa)</td>
</tr>
<tr>
<td>Spiral</td>
<td>W20 @ 2in (51mm)</td>
</tr>
</tbody>
</table>

In order to determine the sensitivity to gravity loading two load cases were considered. These consisted of 10% and 100% of the design uniform live load (live load coefficients of 0.1 and 1.0 respectively) in addition to the full dead load of the structure acting as both weight and mass. While it is unlikely that 100% of the uniform live load will be present on the entire pier during an earthquake, it is valuable to see how a heavily loaded pier, or segment of a pier, responds to strong ground shaking.
Table 2 -- Prototype Model Properties

<table>
<thead>
<tr>
<th>Model Number</th>
<th>1</th>
<th>2</th>
<th>3</th>
<th>4</th>
<th>5</th>
<th>6</th>
</tr>
</thead>
<tbody>
<tr>
<td>Depth to Mudline</td>
<td>60 ft  (18.3 m)</td>
<td>60 ft  (18.3 m)</td>
<td>30 ft  (9.1 m)</td>
<td>30 ft  (9.1 m)</td>
<td>10 ft  (3.0 m)</td>
<td>10 ft  (3.0 m)</td>
</tr>
<tr>
<td>Live Load</td>
<td>800 psf (38 kN/m²)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Live Load Coefficient</td>
<td>0.1</td>
<td>1.0</td>
<td>0.1</td>
<td>1.0</td>
<td>0.1</td>
<td>1.0</td>
</tr>
<tr>
<td>Superstructure Weight</td>
<td>1,020 kip (4.6 MN)</td>
<td>2,460 kip (10.9 MN)</td>
<td>1,020 kip (4.6 MN)</td>
<td>2,460 kip (10.9 MN)</td>
<td>1,020 kip (4.6 MN)</td>
<td>2,460 kip (10.9 MN)</td>
</tr>
<tr>
<td>Initial Stiffness</td>
<td>11 k/in (1.9 MN/m)</td>
<td>11 k/in (1.9 MN/m)</td>
<td>53 k/in (9.3 MN/m)</td>
<td>57 k/in (10.0 MN/m)</td>
<td>316 k/in (55.3 MN/m)</td>
<td>350 k/in (61.3 MN/m)</td>
</tr>
<tr>
<td>Fundamental Period</td>
<td>3.1 sec</td>
<td>4.8 sec</td>
<td>1.4 sec</td>
<td>2.1 sec</td>
<td>0.6 sec</td>
<td>0.9 sec</td>
</tr>
</tbody>
</table>

OVERVIEW OF SEISMIC P-Δ EFFECTS

Collapse of structural systems can be divided into three categories; sway instability due to secondary (P-Δ) forces, material failure instability, and instability under gravity loads. Of primary concern in this study is collapse due to lateral loads combined with P-Δ effects, where the structure displaces laterally to the point where its ability to resist the P-Δ induced secondary forces is overcome. This collapse mode is distinct from material compressive or shear failures, which can lead to collapse due to the inability to support gravity loading, although material softening due to damage worsens P-Δ sensitivity. Collapse in this study will refer to the inability of the structure to resist lateral forces due to the loss of strength associated with P-Δ effects and flexural damage.

In order to understand the impact secondary P-Δ effects have on a structural system Figure 2 shows idealized base shear – displacement curves for an entire structure with and without the influence of P-Δ. In the figure $V_y^o$ represents the strength of the system at the idealized yield point ($\Delta_y^o$), and $V_d$ is the base shear resistance at the design displacement demand ($\Delta_d$). The superscripts ‘o’ and ‘p’ indicate whether P-Δ was neglected or incorporated in the analysis respectively. It can be seen that in general P-Δ effects cause a reduction in stiffness and base shear capacity, and there is a possibility that the post-yield tangent stiffness will become negative. Collapse is then defined when the ordinate of the base shear – displacement response reaches zero during a loading excursion, this is denoted in Figure 2 as $\Delta_c$. Collapse due to P-Δ effects will only occur if a negative post-yield stiffness is achieved, as $\Delta_c$ will never be reached if a positive post-yield stiffness is present.
There have been a number of expressions developed to capture the sensitivity of a structure to P-Δ instability. Several are presented here that will be used in this study to quantify the behavior of marine piers. The first expression, shown in [1, is the ratio of second order (P-Δ) overturning moments to the base overturning capacity for the structure analyzed without P-Δ, and is known as the stability index.

\[
\theta = \frac{P\Delta_o}{V_{d_o}H}
\]  

[1]

The stability index is a function of the inverse of the structures secant stiffness \( \frac{V_{d_o}}{\Delta_o} \) at the design displacement demand \( \Delta_o \), the weight of the superstructure (P) and the distance from the location of the maximum in-ground moment to the soffit of the pier (H). According to POLB and MOTEMS P-Δ effects may be ignored in analysis if the stability index is less than 0.25, which falls into the regime when the secondary overturning moment demand equals less than 25% of the base overturning moment capacity. Beyond this, the probability of dynamic instability increases and P-Δ must be explicitly incorporated into the analysis using a nonlinear response history analysis. This limit has been determined from experimental testing of reinforced concrete bridge columns and SDOF inelastic response history analyses (MacRae et al. 1993), (Priestley et al. 1996). As a part of this study the suitability of the \( \theta = 0.25 \) limit for piers built with prestressed concrete piles will be evaluated.

The next metric to evaluate P-Δ sensitive behavior is the displacement amplification factor (DAF), which is the ratio of maximum displacement determined through nonlinear response history analysis of a structure with and without P-Δ effects.

\[
DAF = \frac{\Delta_{d_p}}{\Delta_{d_o}}
\]  

[2]

If the DAF is greater than unity then there is an amplification in displacement demand due to P-Δ, conversely if the DAF is less than unity there is a deamplification. The amount of amplification is dependent on not only the structure, but on the spectral shape of input ground motion as yielding of the system lengthens the period of vibration. The farther that DAF is from unity, the higher the sensitivity of the structure is to P-Δ for a given ground motion.

\[
\mu_\Delta = \frac{\Delta_{y_p}}{\Delta_{y_o}}
\]  

[3]

Finally, the displacement ductility of the system can be quantified using [3, which relates the effective yield displacement of the bi-linear approximation of the pushover curve to the maximum displacement (either capacity or demand) of the entire system. The displacement ductility is therefore a normalized measure of the amount of plastic deformation within the structural system. To generate the bilinear approximation of the pushover, first the initial stiffness is determined by a line from the origin through the point of first yield of either the mild steel in the pile-to-deck connection or the prestressing steel within the pile body. The post-yield segment is then determined by a line running through the OLE and CLE displacement capacities, which are defined by strain limits in the following section. The effective yield of the system was then taken as the intersection of the initial stiffness and the post-yield segment. Note that this formulation of the system displacement ductility is not equal to the member displacement ductility due to the flexibility of the soil, flexibility of the deck, and overall system configuration.

**STRAIN LIMITS**

Currently in POLA and POLB there are three seismic hazard levels; the Operating Level Earthquake (OLE), the Contingency Level Earthquake (CLE), and the Design Earthquake (DE). The OLE and the CLE seismic hazards have 72-year and 475-year return periods respectively, while the DE is the hazard associated with 2/3 of the 2,475-year earthquake (the Maximum Credible Earthquake (MCE) stipulated in ASCE 7). Acceptable structural performance in each hazard level is determined by associated performance levels; for the OLE minimal damage (i.e.
cosmetic damage that does not disrupt facility operation) is allowed, for the CLE controlled and repairable damage is allowed, and for the DE life-safety and collapse protection are enforced. Each performance level has associated material strain limits, which are used to prevent and limit damage during small and moderate sized earthquakes (OLE and CLE) and collapse in large, rare earthquakes (DE).

This study adopts the POLB strain limits defined in Table 3 to define the expected levels of damage and the deformation capacity of the system. The material strain limits are applied to plastic hinges within the pile body and the pile-to-deck connection to limit the plastic rotation based off of the sectional response. The strain limits have typically been defined using lower-bound conservative estimates of strains at the onset of damage states. For example the OLE concrete strain limit for the pile connection of 0.005 should prevent spalling of the cover concrete, and the DE reinforcing steel strain of 0.08 should prevent bar fracture.

However, dowel bar fracture has been observed at strains as low as 0.06 in pile-to-deck connection tests at the University of Washington (Jellin 2008), (Stringer 2010) due to low-cycle fatigue and local effects of the dowel bar. Due to the importance of the pile-to-deck connection moment resistance to the stability of piers the DE level strain limits for the connection reinforcing steel may be unconservative and bar fracture may occur before a 0.08 strain is achieved. It is therefore important to determine if connection demands for a typical pier will produce reinforcing steel strains larger than the CLE strain limit of 0.06.

<table>
<thead>
<tr>
<th>Component Strain in Plastic Hinge</th>
<th>OLE</th>
<th>CLE</th>
<th>DE</th>
</tr>
</thead>
<tbody>
<tr>
<td>Pile-to-deck connection concrete strain</td>
<td>$\varepsilon_c \leq 0.005$</td>
<td>$\varepsilon_{cc} = 0.005 + 1.1 \rho_s \leq 0.025$</td>
<td>No limit</td>
</tr>
<tr>
<td>In-ground concrete strain</td>
<td>$\varepsilon_c \leq 0.005$</td>
<td>$\varepsilon_{cc} = 0.005 + 1.1 \rho_s \leq 0.008$</td>
<td>$\varepsilon_{cc} = 0.005 + 1.1 \rho_s \leq 0.025$</td>
</tr>
<tr>
<td>Pile-to-deck connection reinforcing steel strain</td>
<td>$\varepsilon_s \leq 0.015$</td>
<td>$\varepsilon_s \leq 0.6 \varepsilon_{mdl} \leq 0.06$</td>
<td>$\varepsilon_s \leq 0.8 \varepsilon_{mdl} \leq 0.08$</td>
</tr>
<tr>
<td>In-ground prestressing tendon strain</td>
<td>$\varepsilon_p \leq 0.015$</td>
<td>$\varepsilon_p \leq 0.025$</td>
<td>$\varepsilon_p \leq 0.035$</td>
</tr>
</tbody>
</table>

$\varepsilon_c$ = Maximum unconfined (cover) concrete compression strain.
$\varepsilon_{cc}$ = Maximum confined (core) concrete compression strain.
$\varepsilon_s$ = Maximum steel tensile strain.
$\varepsilon_{mdl}$ = Maximum uniaxial tensile strain capacity of mild steel.
$\varepsilon_p$ = Maximum total prestressing steel tensile strain.
$\rho_s$ = Volumetric ratio of spiral reinforcement.

**ANALYTICAL MODEL DEFINITION**

In order to evaluate the vulnerability of piers to instability during seismic attack analytical models that capture all key aspects of hysteretic behavior, including modes of strength and stiffness degradation must be developed. In this study the finite element analysis package OpenSees was employed.

Each pile element within the pier bent is modeled according to the schematic shown in Figure 3. The model consists of centerline beam-column elements (see Figure 3b) with fiber-discretized cross-sections (see Figure 3c). The use of fiber sections adds some computational cost, but allows full P-M interaction throughout the analysis and the ability to monitor material strains at any point within the element and cross-section. At the cross-sectional level the concrete was divided into confined and unconfined fibers, which were both modeled using a Kent-Scott-Park (Kent and Park 1971) backbone (the Concrete01 model in OpenSees) with the enhanced strength and ductility of the confined concrete determined according to the Mander model (Mander et al 1988). The mild steel (dowel bar) and
prestressing tendons were modeled using the Giuffré-Menegotto-Pinto model (Menegotto and Pinto 1973) as implemented in OpenSees by the Steel02 material model. Prestressing was modeled by utilizing the initial strain feature of the Steel02 model. Dowel bar fracture was accounted for using the Fatigue material implemented in OpenSees, which accounts for low-cycle fatigue using a linear strain accumulation model based off of the Coffin-Manson log-log relationship (Coffin 1954), (Manson 1953). A modified rainflow cycle counter was used to track strain amplitude.

The pile-to-deck connection was modeled using a fiber based concentrated plasticity hinge located at the soffit of the deck. A concentrated plasticity model is attractive here for two reasons. First the spread of plasticity in the pile-to-deck connection is limited because a lap splice region exists directly adjacent to deck soffit, where the pile prestressing strand overlaps with the mild steel in the connection. Secondly because (as will be seen in the following section) the pile-to-deck connection plastic hinge exhibits a softening response, a concentrated plasticity hinge prevents numerical concentration issues from arising through the use of a predetermined fixed plastic hinge length. It is well documented (Coleman and Spacone 2001), (Almeida et al. 2010) and (FIB 2008) that distributed plasticity elements become non-objective when a softening local (moment-curvature) response exists. This creates a situation where the curvature demand becomes dependent on the discretization of the element (i.e. the maximum predicted curvature increases without bound with increased member discretization).

\[
L_p = 0.25 f_{y} d_b \quad (f_{y} \text{ in ksi, } d_b \text{ in inches})
\]
\[
L_p = 0.0367 f_{y} d_b \quad (f_{y} \text{ in MPa, } d_b \text{ in meters})
\]

The remainder of the pile length was modeled using distributed-plasticity force-based beam-column elements (Neuenhofer and Filippou 1997). This formulation captured the spread of plasticity along the embedded length of the pile allowing a variable-length in-ground plastic hinge to form. Each element was 0.5D (12 in or 300 mm) long.
and had three Gauss-Lobatto integration points\(^1\). A sensitivity analysis was conducted to ensure that element length provided accurate results with minimal computational cost, whereby the element length was incrementally reduced from 1.0D to 0.1D. It was found that for a given displacement demand the maximum curvature within the in-ground plastic hinge converged with element lengths shorter than 0.5D.

Local soil structure interaction was captured using p-y springs (see Figure 3d) spread over the embedded length of the pile at 0.5D (12in or 300mm spacing). The p-y relationship is determined from a macroelement, which combines elastic, plastic, and gap springs in series. The gap component of the spring consists of a nonlinear closure component and a nonlinear drag component. Viscous damping is included in the elastic far field component of the spring to approximate radiation damping (Boulanger et al. 1999). This p-y formulation allows the spring to approximate the API sand model (American Petroleum Institute 2007) under dynamic loading conditions. Parameters for the p-y curve generation were taken to represent submerged sand with a friction angle of 37 degrees under cyclic loading. Sample p-y curves at various depths are shown in Figure 4.

![Sample p-y Curves](image)

The deck was modeled with a fiber based centerline beam-column element defined by the tributary width of the deck between pile bents. This resulted in a 20ft wide (6.1 m) by 30-inch (750 mm) deep cross section, which effectively acts as a rigid link between the piles. The seismic mass was applied as a discrete point mass at the intersection of the deck and pile centerlines. The mass was taken to only act in the horizontal direction. Axial loads on the piles were applied at the same location by converting the mass to a weight applied as a point load. The ground motion was applied uniformly to all fixed boundary conditions on the pile; this included the pin support at the pile tip and the restrained end of the p-y macroelements. While this does not account for the variation of the seismic waves with depth within the soil column adjacent to the pile, it does provide a reasonable estimate for the homogeneous stiff soil assumed within this study.

Geometric nonlinearity was accounted for in the model so that structural (P-\(\Delta\)) and element (P-\(\delta\)) geometric nonlinearity effects could be captured. Structural (P-\(\Delta\)) effects were captured using the OpenSees PDelta linear geometric transformation which accounts for P-\(\Delta\) effects due to axial load on an element. Member (P-\(\delta\)) effects were captured through the use of multiple beam-column elements along the unsupported length of the pile.

One of the advantages to the fiber-discretized cross section is the automatic consideration of elastic damping due to energy dissipation from the elastic hysteretic response. This circumvents the need to apply a fictitious viscous damping.

---

\(^1\) Three point Gauss-Lobatto quadrature places integration points at each end and the middle of each element. The integration weights are 4/3 at the end points and 1/3 at the middle point.
damping to the linear-elastic portion of the hysteresis by treating inherent damping as a hysteretic response (Priestley et al. 2007), (Charney 2008). During a free-vibration of the system it was found that piers in this study had elastic damping ratios of approximately 4% of critical. This corresponds well to expected values for prestressed concrete systems without nonstructural components. Additional damping due hydrodynamic drag on the piles was neglected.

ANALYTICAL MODEL CALIBRATION

To ensure accuracy in the modeling technique employed components that are expected to sustain considerable inelastic action were calibrated against experimental test results. The pile-to-deck connection was calibrated against the University of Washington Test Specimen 9 (Jellin 2008). The specimen consisted of a 103 in (2.6m) long cantilever 24-inch octagonal prestressed pile with a grouted dowel bar connection. This test represents a pile with roughly a 17ft (5.2m) length between the soffit of the deck and the maximum in ground moment. As this is the longest pile length that has been used in a pile-to-deck connection test it best represents the moment gradient and shear demand of the piles typically found in piers. The experimental rotation was measured using inclinometers distributed along the length of the pile. The estimated response was calculated using a moment curvature analysis and an experimentally calibrated plastic hinge length. The moment rotation hysteretic responses are compared in Figure 5 where it can be seen there is good agreement in initial stiffness, location of peak resistance, strength degradation, unloading stiffness, and ultimate deformation capacity. It should be noted that in the positive loading direction the analytical model has very nearly the same moment capacity as the test specimen, but in the negative direction under predicts the strength as UW-9 has an asymmetric response. Dowel bar rupture was reasonably well estimated using the previous described low-cycle fatigue model.

For the embedded length of the pile it was assumed that the soil provided confinement to the cover concrete, which is justifiable based on full-scale pile tests (Budek et al. 1997). The confined plastic hinge specimen PS10 tested by (Budek et al. 1997) was used to calibrate the model, as shown in Figure 6. Specimen PS10 was a 20ft (6.1m) long, 24in (610mm) diameter prestressed concrete pile tested under three-point bending. The pile was reinforced with 24 1/2in (13mm) 270ksi (1860MPa) prestressing strands. The geometry of the test set-up roughly approximates the moment and shear distribution of a pile fully embedded in soil with a subgrade modulus of 163kip/ft³ (25600kN/m³). The plastic hinge region was confined by 70 Duro ‘A’ rubber to simulate soil confinement. The experimental curvature was measured by an array of displacement transducers spread along the plastic hinge region. It can be seen under positive curvature loading the predicted and experimental responses agree reasonably well, with near perfectly plastic post yield behavior and minimal residual curvature upon
unloading. In negative loading the experimental response shows a drop in strength due to cover spalling, while the OpenSees model shows a symmetric response. The OpenSees model was considered acceptable. It should be noted that only dense soils are likely to increase the confinement of the pile cover concrete.

![Figure 6 -- Pile In-Ground Plastic Hinge Calibration](image)

After the analytical model was calibrated to experimental tests, the material properties and reinforcement layout were adjusted to reflect the properties and configuration of the prototype structures (see Table 1). Once this was completed moment-curvature analyses were performed on the pile body and pile-to-deck connection under the axial loading associated with 10% and 100% of the live load. The results from these analyses can be found in Figure 7, where it can be seen that the pile body retains its near perfectly plastic post-yield response. The pile-to-deck connection shows a sharp drop in flexural capacity due to cover spalling. It is also apparent that the pile body is stronger than the pile-to-deck connection.

![Figure 7 -- Pile and Connection Moment – Curvature Responses](image)

**NONLINEAR STATIC ANALYSIS RESULTS**

The first step in collapse assessment was to conducted nonlinear static (pushover) response of the prototype piers to examine the base shear – displacement response. A displacement was imposed at the center of gravity of the
superstructure. The base shear resistance for the two gravity load cases were recorded and plotted against the drift ratio to generate the pushover curves which are shown in Figure 8 for each prototype pier. The drift is the defined as the ratio between the superstructure displacement and the distance from the soffit of the pier to maximum in-ground moment. This distance is 18ft (5.5m), 36ft (11m), and 64ft (19.5m) for the 10ft (3m), 30ft (9.1m), and 60ft (18.3m) to mudline piers respectively. P-Δ effects were included in all of the pushover analyses. It can be seen that all of the prototype piers have a negative post-yield tangent stiffness and the additional gravity load increases the strength loss significantly; this effect is more pronounced as pile length increases. For piers with long piles Δ_C will be reached before material strain limits control the displacement capacity.

![Figure 8 -- Prototype Structure Nonlinear Static Response](image)

Once the pushover response is calculated the kinematic relationship between curvature ductility and displacement ductility can be determined for both the pile-to-deck connection and the in-ground plastic hinges. The curvature ductility is calculated as the curvature demand divided by the yield curvature defined by the bilinear moment curvature approximation specified in POLB. The kinematic relationship is shown in Figure 9 where the black and grey lines represent the deck connection and the in-ground responses respectively. The kinematic relationship for 10% and 100% live load are virtually identical so only the results from the 10% live load case are provided.
Kinematic relationships are shown for the three pile lengths included in this study. It can be seen that as the pile length is increased a larger curvature demand is imposed upon deck connection plastic hinge, while the in-ground plastic hinge has a reduced curvature demand. This is because the deck connection plastic hinge length does not increase with pile length beyond that predicted by [4 due to the increased strength region adjacent to the deck soffit caused by the overlap of the dowel bars and the prestressing strand, while the in-ground plastic hinge length increases due to the shallower moment gradient. The in-ground plastic hinge of the soil-pile system was determined to be between 1.5 and 2.3 pile diameters depending on the length of the pile above the mud line.

It can be seen that plastic hinges form first at the pile-to-deck connection, while the in-ground plastic hinges begin to form at a displacement ductility of approximately 2.5. It should be noted that the connection kinematic relationships begin at a displacement ductility of roughly 0.75, this is because of the bilinear definition of displacement described earlier in this paper, where the effective yield displacement of the system is always greater than the displacement at the true first yield.

Superimposed onto the kinematic relationships shown in Figure 9 are the curvature ductility limits for the 10% live load case as calculated using the strain limits of Table 3. For the conditions considered in this study the CLE limits are controlled by either the connection or the in-ground hinge depending on the length of the pile, and the DE limit is always controlled by the in-ground plastic hinge. This is contrary to what is typically expected in reinforced concrete construction where the connection at the top of the pile/drilled shaft controls the displacement limits. However if the pile-to-deck connection mild steel fractures at 0.06 strain the displacement ductility capacity will be significantly reduced. This strain level is indicated by the CLE curvature capacity for the connection.

If the displacement ductility limits for the CLE and DE (per POLB strain limits) are calculated for multiple pile lengths and axial loadings trends in the controlling components and materials can be determined. To do this single fixed-head pile pushovers were conducted under three axial loads, 2%, 5%, and 10% of the gross axial pile capacity ($\Gamma_c A_g$), with pile lengths from zero to 60ft (18.3m) to mudline. It should be noted that the 10% and 100% live load cases for the prototype piers discussed previously result in axial loads of roughly $0.04\Gamma_c A_g$ and $0.09\Gamma_c A_g$. 

![Figure 9 -- Kinematic Relationship Between Displacement Ductility and Curvature Ductility](image)
The results of these analyses are shown in Figure 10 where the solid lines are the displacement ductility capacities according to the material strain limits, while the dashed lines are the displacement ductility capacity values associated with a stability index value of 0.25. In general two observations regarding the ductility capacity based off of material limits; first, the ductility capacity increases with axial load as steel tension strain limits control the displacement capacity and second the CLE and DE displacement ductility capacities increase with pile length as was discussed in relation with Figure 9. It is also of interest to see when the stability index limit controls the displacement capacity, as this indicates the conditions that may induce P-Δ sensitive behavior. For typical pile loading values of 0.02 to 0.05\(f'_c A_g\) the P-Δ limit controls CLE displacement ductility limits for pile lengths greater than 20ft (6.1m) and 40ft (12.2m) respectively. The DE displacement limit is controlled by the P-Δ limit at slightly shorter pile lengths. Heavily loaded piles (0.1\(f'_c A_g\)), as expected, are controlled by the P-Δ limit for almost all pile lengths at both the CLE and DE level. This shows that for typical pile lengths and axial loading the P-Δ sensitive behavior is expected and the stability index limit will likely control the displacement capacity over material strain limits.

SEISMIC DEMANDS

To provide a realistic estimate of the seismic demand on the prototype pier a site-specific probabilistic hazard analysis was conducted for a firm soil/soft rock site located in the Puget Sound region of Washington state. The seismic hazard was defined as an event with a 2,475-year return period, which corresponds to the Maximum Considered Earthquake (MCE) event, as this is the strongest ground motion that the structure is expected to endure without collapse based on the NEHRP Provisions (and therefore the International Building Code). The Puget Sound region has three distinct types of fault mechanisms, which are interplate subduction, deep intraplate, and shallow crustal earthquakes.

Because a nonlinear response history analysis will be used in this study the selection and scaling of the input ground motions is critical. The site hazard analysis and ground motion selection were conducted by a geotechnical engineer for a project that the authors were recently involved with. The selection of ground motions considered the earthquake source mechanism, the likely magnitude of the events, the source-to-site distance, and the spectral match to the target (design) acceleration response spectrum. Once the ground motions were selected they were linearly scaled so that the average of the scaled spectra was within ± 10% of the design acceleration spectrum for the structural period range of 0.6 seconds to 5.0 seconds. As part of the site-specific hazard analysis seven ground
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**

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

**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.
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.
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.

Figure 15 shows the maximum tensile strain demands for the pile-to-deck connection and in-ground 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.

Finally it is of interest to determine the relationship between the stability index ($\theta^*$) and displacement amplification factor ($DAF$). The results are plotted in Figure 16, where considerable scatter, can be seen. This shows that as $\theta^*$ 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 $\theta^*$ range where P-Δ can safely be ignored. MOTEMS (2011) and the Port of Long Beach (2009) have defined this limit to be $\theta^* = 0.25$, 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.
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-$\Delta$ effects will underpredict the displacement demand by seven percent. It is also evident that for cases above the $\theta^o$ limit of 0.25 the COV increases substantially.

<table>
<thead>
<tr>
<th>$\theta^o$ Range</th>
<th>Average Value</th>
<th>COV</th>
</tr>
</thead>
<tbody>
<tr>
<td>0 – 1.0</td>
<td>1.07</td>
<td>0.19</td>
</tr>
<tr>
<td>0 – 0.1</td>
<td>1.04</td>
<td>0.05</td>
</tr>
<tr>
<td>0 – 0.25</td>
<td>1.07</td>
<td>0.13</td>
</tr>
<tr>
<td>0.25 – 1.0</td>
<td>1.05</td>
<td>0.30</td>
</tr>
</tbody>
</table>

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-$\Delta$ effects, however despite the variability in displacement amplification the 0.25 limit on $\theta^o$ 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 $\theta^o$. 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-$\Delta$ effects. It is recommended then that when increased analytical accuracy is desired P-$\Delta$ 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:

\[ \theta^* = \frac{P \Delta_d}{V_d H} \]

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 \( \Delta_d \)
- \( H \) = 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 \( \theta^* \) is less than 0.25 the structure is stable
   b. If \( \theta^* \) 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 \( \theta^* \) 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 0.05f_cA_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)

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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
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**INTRODUCTION**

Ports are a vital link in the transportation of goods worldwide, with more than a trillion dollars of goods flowing through US shipping terminals every year (Port of Seattle, 2009). Economies worldwide rely heavily on ports, and it is estimated that $1,000 is brought into the local economy from each shipping container. Therefore continuous operation of port facilities in high seismic regions is vital to the US economy.

Many ports are located in high seismic regions and are susceptible to strong ground motions, resulting in the potential for devastating physical and economic damage. In addition, most ports are built on poor soils that are susceptible to liquefaction, lateral spreading, and differential settlements, which compound the likelihood of earthquake damage.

Ports damaged by earthquakes require repair. Equally or even more important, ports suffer loss of income and reduced activity due to repair downtime. The loss in income can be long-term or permanent, since tenants move to other regional ports while the damaged port is repaired, and may not return after repairs are completed. An example of the impact of port damage on the economy of the region was demonstrated after the 1995 Hanshin earthquake. Prior to the earthquake, the Port of Kobe was ranked as the 6th busiest port for container shipping in the world. In 1997, shortly after the earthquake, the port had slipped to 17th, and by 2005 it had dropped to 39th (Chang 2000). Reducing port damage is a central goal of improving their sustainability.

Most marginal wharves in the US are partially or fully supported by plumb precast, prestressed concrete piling, as depicted in Fig. 1a. Vertical pile-supported wharf structures are designed to act as a ductile moment frame with plastic hinge formation at the connection to the wharf deck; a second hinge may form below the surface of the soil. As a result of the framing action, these connections experience significant moment and rotation demands, and they are vital to the overall performance of the system. Although there are a variety of connections used, in modern design most of the pile-to-wharf connections are variations of the embedded dowel connection, as shown in Fig. 1b, or the extended pile connection, as shown in Fig. 1c. (Extended pile connections are only required when the pile is driven below the deck level to achieve the required bearing capacity and are not preferred.) Prior research suggests that these connections are vulnerable to seismic damage (Roeder et al. 2005).

As a result of their prevalence in practice and their susceptibility to seismic damage, a research program was undertaken to improve the seismic performance of pile-to-wharf dowel connections. Other connections are no longer in favor for most west-coast ports and therefore variations of those connections were not a subject of this study. This program has developed a new connection to improve the seismic performance of pile-to-wharf connections, with an eye towards delaying structural damage. The connection borrows some technologies from moment-resisting connections for precast building design, including debonded dowels. Addition of a bearing pad between the pile and the wharf deck minimizes concrete damage at moderately large levels of drift. The performance of the new connection is compared with prior connections using fragility curves. A method to achieve the new connection design is presented for implementation in the design office.

**RESEARCH SIGNIFICANCE**

Marginal wharves are critical structures to the economy. Typical design of US marginal wharves uses prestressed piles as the vertical elements. Dowels are used to connect the precast piles to a cast-in-place concrete deck, and these connections are intended to provide moment and shear transfer. However, recent experimentation indicates that most of these connection do not sustain their peak resistance at low to moderate levels of drift, a result of concrete damage that is sustained at these drift levels. The conventional connection sustains concrete damage including cover spalling and shearing of the deck. To mitigate damage and improve their seismic drift capacity, a new connection was developed. The new connection utilizes debonded...
reinforcement to maximize deformability and a bearing pad between the pile and deck that sustains large compressive strains under combined loading while mitigating damage. The result is a damage-resistant connection with similar stiffness and strength values as the conventional dowel connection design. The paper demonstrates the robustness of this new connection through large-scale testing. A comparison with a range of connections shows a substantial improvement in the seismic performance.

PERFORMANCE OF CONVENTIONAL DOWEL CONNECTIONS

A number of past studies have examined the seismic performance of embedded dowel connections and extended pile connections (Joen and Park 1994, Silva et al. 1997, Silva 1998, Sritharan and Priestley 1998, Graff 2001, Soderstrom 2001, and Roeder et al. 2005). The details of the test specimens vary widely, but consistent observations can be made from those test results. Figure 2 provides the lateral resistance-deflection and moment-rotation behavior of three test specimens. Figure 2a shows the behavior of an extended pile connection without an axial load, while Figs. 2b and 2c show, respectively, the behavior of embedded dowel connections with precast concrete piles without and with axial, respectively. The axial load for the connection of Fig. 2c was approximately 10% of the gross compressive load capacity ($0.1f'_cA_p$) of the prestressed pile, which is within the range of the level of axial load expected in practice.

These figures not only demonstrate the response of these connection types, but also provide a basis for comparison. Figs. 2a and 2b show the behavior of the extended-pile connection and the embedded-dowel connection, respectively. There are apparent differences. The behavior of the extended-pile connection is similar to a cast-in-place reinforced concrete connection with fuller hysteresis loops, and limits deterioration and loss of resistance to larger drifts. In comparison, the precast-pile connection without an axial load loses strength at a lower deformation although the maximum resistance is similar to an extended pile connection with the same cross section and material properties.

The seismic response of the marginal wharf system depends on both the pile and connection properties. In contrast to reinforced concrete piers in bridge construction, precast piles deteriorate at lower deformations because the flexural deformation capacity is limited in this component, which largely act as a rigid body. In this case, it is the connection that must sustain the cyclic deformations demands.

For a dowel connection, rotation demands results in large edge stresses on the pile and which can result in large compressive strains and stresses at the end of the pile that is embedded into the deck. Early spalling of the pile and deck can result (Fig. 3c). Typically, the cover on the pile is large (more than 76 mm) to prevent corrosion of the prestressing strands. However, loss of this cover has structural implications since the gross-to-core area ratio is larger in a precast pile than a typical reinforced concrete component. This results in a more significant decrease in the bending resistance than a typical reinforced concrete column. This loss of concrete cover results in loss of resistance with increasing deformation, as shown in Figs. 2b and 2c. Note that extended pile connections are used only in special cases where the pile is driven below the bottom of the soffit of the wharf deck. Taking this exception out of the evaluation, the conclusion is that dowel connections do not meet the performance expectations.

In addition to the type of connection, the axial stress demand is important. In a marginal wharf application, the piles support compressive load due to the heavy deck system. The addition of compressive load to the pile significantly changes the connection response, as seen by comparing Figs. 2b and 2c. It is of note that the pile is prestressed, which results in large compressive stresses. The addition of an axial load of only 10% of the gross axial capacity, as was used for the specimen that resulted in the measured behavior in Fig. 2c, significantly changes the response. The axial load significantly increases the moment capacity and maximum lateral resistance of the connection, but it also dramatically increases the rate of post-peak strength deterioration. Part of the lost lateral resistance can be attributed to P-Δ effects at large deflections (compare the deformation in the force-deflection and moment-rotation plots of Fig. 2c). These P-Δ moments also reduce the effective lateral resistance because a portion of the moment capacity resists the P-Δ effects.

The post-peak response noted with the moment-rotation curves is a result of and therefore a true measure of damage to the pile and its connection. At large rotations, the compressive force at the edge of the pile, associated with the bending moment and further increased by the axial load, results in large compressive stresses (Roeder et al. 2005). Hence, with an increase in axial force, spalling occurs at smaller deformations (Figs. 3a and 3b) and is more severe, both of which result in larger deterioration (Figs. 2b and 2c).
Figure 3 shows damage of the three connections at the same connection rotation level of 0.03 radians. The actual displacement of the pile depends on its effective length. For a short pile at the end of the wharf, the displacement associated with this level of rotation is relatively small since the effective pile length is small. The extended pile connection (Fig. 3a), which was subjected to an axial load, is largely intact with flexural cracking. The precast pile with a dowel connection and no axial load (shown in Fig. 3b) is still intact, but the crack pattern suggests that spalling will initiate at slightly larger deformations.

The precast pile with a dowel connection and an axial load ratio of 0.1, shown in Figs. 2c and 3c, sustains significant spalling damage. Other specimens exhibited this level of spalling, but at significantly higher wharf displacements and connection deformations. Pile and deck spalling have structural implications beyond seismic resistance, since they cause loss of prestressing force and expose the reinforcing steel to the harsh marine environment. Hence, repairs are required for these connections even after modest earthquakes. However, the spalling and its correlation to the deterioration in resistance suggest that the performance of the connections may be significantly improved at all performance levels if spalling is delayed.

INVESTIGATION OF DAMAGE-RESISTANT DOWEL CONNECTION

Characteristics of Connection
The subject research program was undertaken to improve the seismic performance of pile-to-wharf connections. The study examined the results of prior research, evaluated recent connection designs, and, in consultation with practicing engineers in this field, developed improved connection designs. Experiments were conducted to investigate the connection designs. Several options were proposed for experimental evaluation, including:

- Partial debonding of the dowel bars into the wharf deck and the pile itself. Debonding the longitudinal bar reduces the maximum strain demand in the steel. For a given level of strain, a debonded bar has a larger axial deformation than a bonded bar, a direct result of the strain gradient (uniform vs. non-uniform). This delays dowel bar fracture and increases the rotation for a given level of strain demand (Stanton et al. 1997). This technique has been used successfully in precast, prestressed moment frames for buildings. A second feature of debonding the dowel bars is that it alleviates bond stress transfer in that region, which can damage the deck concrete.

- Addition of a flexible material between the pile and the deck to sustain the rotation demands and reduce damage. This feature mitigates the damage resulting from rocking of the pile under cyclic loading, which results in edge loading of the pile and the premature concrete damage (Fig. 3). To reduce the damage, the edge loading needs to be distributed to a larger area of the pile cross-section to reduce the compressive stress demands. Here, a bearing pad manufactured of a material strong enough to sustain the compressive stress demands but flexible enough to sustain the rotation demands was used.

- Flexible joint sealant wrap around the embedded perimeter of the pile. There was discussion with practicing engineers to modify the 75 to 100 mm (3 to 4 in.) embedment of the pile into the wharf deck, by either increasing the embedment depth or eliminating it. This short embedment is approximately equal to the cover depth, and results in shear stress demands on cover concrete which in turn results in significant concrete damage. It was postulated that either a much larger or elimination of the embedment depth would reduce the spalling of the deck at larger rotations. However, the practicing engineers deemed neither option acceptable. The concerns were that eliminating the embedment depth would (1) result in intrusion of salt water into the connection and (2) decrease the shear resistance. As an alternative, a gap was placed between the pile and deck and filled with a flexible material, to reduce the shear stress demands. This material seals the joint to prevent intrusion of salt water into the connection.

Experimental Program
An experimental program was initiated at the University of Washington to investigate the characteristics of pile-wharf connections that result in these damage-mitigating characteristics. Eight (8) full-scale precast concrete pile-wharf deck connections were tested (Jellin 2008, Stringer 2010) and included one or more of these aspects of the connections, as well as the applied axial load and the bearing pad configuration and material. The eight test specimens had the same piles and deck reinforcing and configuration, which are illustrated in Figs. 4 and 5. These aspects of the specimen design were determined from an infrastructure review of 14 marginal wharf structures from the Port of Los Angeles, the Port of Oakland, and the Port of Seattle (Roeder et al. 2005).
The specimens had a 610 mm (24 inches) prestressed concrete pile and utilized an embedded T-headed dowel bar connection. The pile had twenty-two 12.7 mm (0.5 inch) diameter low relaxation strands with tensile strength of 1,860 MPa (270 ksi). Each strand was pretensioned to 138 kN (31 kips), resulting in a service level prestress of approximately 9.7 MPa (1.4 ksi) after accounting for stress losses. The spiral reinforcement was W11 (9.5 mm diameter) smooth continuous wire, and the pitch varied along the pile length from 25 mm (1 inch) at the ends to 76 mm (3 inches) in the middle, as shown in Fig. 4.

Compressive strength data up to the 28-day strength were provided but test cylinders were not available to determine the day-of-test strength. The pile concrete was a 9.5 mm (5/8 in.) max aggregate 55.1 MPa (8,000 psi) mix with a 76 to 229 mm (3 to 9 in.) slump (measured by the pile fabricator which is located in Seattle). The piles used for the test were cast from one of three different batches, which had 28-day strengths of 66.7 MPa (9,680 psi), 73.4 MPa (10,650 psi) and 58.0 MPa (8,410 psi). These high concrete strengths are typical for precast construction in the Pacific Northwest. Due to incomplete documentation, it was not possible to determine which pile came from which batch.

The piles were embedded 76 mm (3 inches) into a cast-in-place reinforced concrete section representing a segment of the deck. The connection was made using eight ASTM A706 (482 MPa yield stress) No. 10 T-headed dowel bars, which were embedded 508 mm (20 inches) into the deck. The dowels were 1.93 m (6.3 ft) long and grouted 1.5 m (5 ft) into corrugated ducts in the pile using high strength, non-shrink grout. The grout achieved 34.5 MPa (5,000 psi) and 69 MPa (10,000 psi) compressive strengths in 24 hours and 28 days, respectively.

The pile length was 2.62 m (8.6 ft) from the soffit of the deck to the horizontal loading point. This length was chosen to be representative of short pile length in typical wharf structures based on prior analyses (Yoo 2001). The deck dimensions (see Fig. 5) were maximized to ensure simulation of force transfer without interference from the test setup, but constrained to fit into the test rig. The deck reinforcement is shown in Fig. 5 and simulates reinforcement layouts typically used in the prototype marginal wharves (Roeder et al. 2005).

The specimens are identified as Specimens 9 through 16 (numbered to be consecutive following the specimens studied by Graff 2001 and Soderstrom 2001) and are presented in Table 1 and Fig. 6. The series of specimens was designed to study the connection modifications, axial load, and bearing pad characteristics. The primary study parameter for each specimen is shown in the table. A brief description of each specimen follows.

- Specimen 9 was designed using the embedded dowel bar connection commonly used in current design and serves as the reference specimen.
- The dowel bars in Specimen 10 were intentionally debonded into the connection and into the deck, 190 mm for each. All other aspects of the connection were the same as Specimen 9.
- The connection of Specimen 11 included the debonded region and the addition of a cotton-duck bearing pad (CDP) between the pile and deck interface. CDP have high compressive strength and can accommodate large rotations (Lehman et al. 2005). All bearing pads had an octagonal shape with 610 mm outside dimensions to cover the entire head of the pile.
- Specimen 12 was identical to Specimen 11 (with a debonded region and a CDP) with the addition of a soft foam wrap that was added around the perimeter of the length of the pile embedded into the deck.
- Specimen 13 was nominally identical to Specimen 12 to investigate use of different materials in the gap between the pile and the deck. Specimen 13 used a CERAMAR flexible expansion material (CFEM) for this gap (Meadows 2001).
- Specimen 14 was nominally identical to Specimen 13 but subjected to twice the axial load.
- Specimens 15 and 16 explored the effects of using different bearing pads. Specimen 15 had an annular 19mm CDP to improve constructability. Specimen 16 had a 13mm random oriented fiber bearing pad (ROFP). These annular rings pads had 457mm diameter center hole that cleared the dowels and ducts of the pile.
**Test Setup and Procedure**

The specimens were constructed to simulate the field orientation (deck cast above the pile), however were tested in an inverted position to fit within the constraints of the test setup and laboratory. A self-equilibrating test rig, shown in Fig. 7, was used. The deck portion of the specimen was post-tensioned to a reaction block at the base of the test frame using four 32mm, 1,034 MPa high strength threaded rods, post-tensioned to 556 kN.

The vertical axial load was applied by a 10.6 MN Baldwin universal testing machine (UTM). An assembly was constructed to ensure the axial load imparted minimal horizontal force on the loading head of the UTM. A recessed, dimpled, and lubricated polytetrafluoroethylene (PTFE) sliding surface with a mirror finished stainless steel plate mating was set inside a C15x15 section, which was attached to the UTM loading head. A spherical bearing was placed below the PTFE sliding surface to accommodate end rotation of the pile tip.

The horizontal cyclic load was applied using a 977 kN (220 kips), 508 mm (20 inches) stroke MTS actuator, which was attached to and reacted against the test frame. The actuator was connected to the pile using, four 22 mm (0.87 inches) diameter B7 steel rods and two 25.4 mm (1 inch) thick steel plates placed on either side of the pile. A 3.2 mm (0.15 inches) elastomeric pad was placed between the steel plates at the end of the actuator and the face of the pile to evenly distribute the bearing stresses. The specimens were subjected to a constant vertical load. The cyclic lateral loads were applied under displacement control with increasing amplitude deformation cycles with a test protocol based upon the ATC 24 procedure (ATC 1992).

Figure 8 shows the external instruments used to monitor the specimen. String pots were used to monitor the horizontal movement at several locations along the pile length (P9-P11). Vertical potentiometers were used to monitor segment rotations, from which average curvatures could be calculated (P14-P21). The bottommost vertical potentiometers monitored connection rotation (P5-P6). Other potentiometers monitored specimen slip, overturning, and movement of the setup. Finally a series of inclinometers were placed on either face of the specimen to monitor rotation; the inclinometers are shown at the lower end of the pile in the figure. Additional details of the test specimens, setup and instrumentation can be found in Stringer (2010).

**Experimental Results**

Assessment of the specimen performance was made through consideration of the damage sustained through loading and the measured global response. The moment-rotation response for six specimens is provided in Figure 9, where moment-rotation response is shown because the force-drift response curves include degradation resulting from P-Δ effects. The moment-rotation curves show the actual specimen response.) Key characteristics that are important to understanding the specimen response and performance are provided in Figure 10.

The response is quantified in term of the connection rotation of the specimens. For the specimens, connection rotation is approximately equal to drift ratio since the majority of the deformation is derived from rotation of the connection. This is a more meaningful quantity for system response and performance prediction since drift depends on the effective length of the specimen. Under earthquake ground motions, this quantity varies with the dynamic response of the system. In addition, the effective length depends on the soil properties, which have a more significant degree of uncertainty than the structural properties. These two aspects of the response result in an effective length that is variable and uncertain, therefore use of drift is not meaningful. Here, the term connection rotation is used for a more meaningful comparison with system response.

Specimen 9 was the reference specimen and simulated current practice; it provides a baseline comparison for the other specimens. As shown in Fig. 9, the connection reached its peak moment resistance at a connection rotation of 1.4% and deterioration of resistance initiated shortly thereafter. At the connection rotation of 8%, four dowel bars had fractured and the moment resistance dropped to 47% of the maximum. At this deformation, the lateral resistance was essentially zero because of the deterioration and P-Δ moments. Figure 10 shows the observed response at approximately 2.5%, 4% and 8.4% rotation angle, which are approximately the damage states corresponding to initial spalling, substantial spalling (includes complete spalling of the cover and up damage to the core), and bar fracture.

Specimens 10-13 investigated fundamental improvements to the connection. Specimen 10 had debonded dowel bars and exhibited similar moment and shear capacities but with reduced deterioration in resistance relative to Specimen 9. The debonded dowel bar steel caused initial pile cover spalling to occur at a slightly lower rotation, but delayed substantial spalling and dowel bar fracture. Specimen 10 showed significantly less degradation in post peak strength than Specimen 9 between 8% and 9% connection rotation despite experiencing dowel bar fracture.
The response of Specimen 13 is depicted and quantified in Figs. 9 and 10 and Table 2, respectively. Specimen 13 included debonded dowel bars as well, however it was the addition of a 19 mm (0.75 inch) thick 610 mm (24 inches) diameter octagonal CDP and a 19 mm (0.75 inch) thick CFEM wrap around the perimeter of the embedded pile segment that greatly reduced the damage at moderate to large levels of deformation demand. The behavior of Specimen 13 is much improved in comparison with Specimens 9 and 10. Specimen 13 exhibited considerable delay in initial and substantial spalling, which occurred at 3.8% and 8.4% connection rotation, respectively. None of the dowel bars fractured during this test. By 8% connection rotation the moment resistance had only reduced by 19% of the peak resistance, which is considerably less than either Specimens 9 or 10. However, despite these significant improvements in performance the addition of the full bearing pad reduced the elastic stiffness of the connection by nearly 50% compared to Specimens 9 and 10.

It is of note that the stiffness of Specimen 13 is significantly lower than Specimens 9 or 10 (Table 2). The impact of this difference depends on the system characteristics and ground motion. A recent study by Chiaramonte (2011) studied the response of a port in Oakland California. Two simulations models were developed with connections represented by Specimen 9 and Specimen 13. One of the study objectives was to determine the impact of the connection on the seismic performance. A two-dimensional nonlinear model of the wharf and supporting soil was developed and extensive response-history analyses were performed. The results showed an improved performance of the system at serviceability levels when the proposed connection was used; there was no discernible difference at higher demands levels.

The remaining specimens were detailed with some variation of the connection used in Specimen 13; this connection controlled damage through rocking and is termed the controlled rocking or “CR” connection. Specimen 14 was identical to Specimen 13, but it was subjected to a 4,000 kN (900 kips) axial load as compared with 2,000 kN (450 kips) for Specimen 13. Specimen 14 exhibited a higher elastic stiffness and peak flexural resistance but has larger and more rapid degradation of resistance (55% loss) compared with Specimen 13. This is consistent with observations in prior research illustrated in Fig. 2. The hysteresis loops remain quite full with less pinching than any other specimen tested, and energy dissipation is quite large. However, the P-Δ moments reduce the lateral resistance of the connection, and these P-Δ moments exceed the connection resistant at approximately 7% connection rotation. The test was terminated shortly after this deformation to protect the testing equipment, because the actuator was supporting rather than loading the specimen. Dowel bar fracture did not occur for this test.

Specimens 15 and 16 used annular 19-mm thick CDP and a 13mm thick ROFP, respectively, to improve constructability and to increase the stiffness of the connection. These two tests evaluated different pad thickness and material, since ROFP is a recycled material and less expensive than CDP. Comparing both tests with other specimens evaluates the effectiveness of the annular bearing pad concept. Both specimens experienced higher elastic and axial stiffness values than Specimen 13, and a slightly higher peak moment resistance. Spalling of the connection initiated at 5.5% connection rotation, a delay of 4.1% rotation and 1.7% rotation from Specimens 9 and 13 respectively. While the connection performance was dramatically improved prior to initial spalling, Specimens 15 and 16 showed rapid loss of resistance after initiation of spalling. Nevertheless, the total deterioration was significantly smaller than that noted with Specimens 9, 10, and 14. Most spalling occurred as a single action for Specimens 15 and 16 rather than a continuous progression noted for other specimens. The spalling and deterioration of resistance at ultimate deformation were less severe for Specimens 15 and 16 than for other specimens, and there was not a dramatic difference in the CDP and ROFP performance. Specimens 15 and 16 also exhibited a delay in the initiation of dowel bar fracture.

The maximum moment resistance of all specimens was essentially identical for specimens with the bearing pads (Specimens 13, 14, 15 and 16) and for the current connection design (Specimen 9). At first glance, Table 2 suggests that the current design is 16% stronger than the modified bearing pad designs, but Table 1 shows that the yield stress of the dowel bars for Specimens 9 and 10 is 11% larger than that for Specimens 13 through 16. Since the flexural resistance of reinforced concrete is dominated by the yield stress, area, and placement of the reinforcing steel, the effective moment resistance for all specimens is nearly identical, as seen by comparing the normalized moment resistance as provided in Table 2.

There are distinct differences in performance for specimens with no bearing pad over the head of the pile (Specimens 9,10), specimens with a full CDP over the head of the pile (Specimens 13,14) and specimens with an annular CDP or ROFP over the head of the pile (Specimens 15,16). Figure 11 shows the visible connection damage for Specimens 9, 13, and 15 at a rotation of 0.05 radians. Specimen 9 has substantial spalling to the deck and the pile exposing the spiral reinforcing of the pile with some spalling beginning to penetrate the core of
the pile. This spalling is comparable to that illustrated in prior tests of similar specimens in Fig. 3c. Specimen 13 experienced only moderate spalling to the pile cover, with no exposure of any reinforcing steel. Additionally the deck concrete is completely intact. Finally Specimen 15 has only pile cracking with no spalling to either the pile or the deck concrete. The advantages of the bearing pad connections, and in particular the annular pad configuration, is clear in comparison to other connections.

Specimens 9, 10, 13, 14, 15, and 16 were evaluated in greater detail and their moment-rotation hysteretic response is shown in Fig. 9. The total connection moment includes the P-Δ effects caused by the axial loading on the pile. Therefore any observed strength loss in these plots can only be attributed to damage to the pile and connection. Specimens 11 and 12 are omitted from this detailed evaluation, because they were intermediary tests used to develop the damage resistant connection details. Table 2 details the performance of each specimen including peak shear and moment resistances, the normalized moment capacity (which compares the measured and predicted flexural strengths), the elastic rotational stiffness, the connection rotations at initial spalling, substantial spalling (exposure of reinforcing steel), and dowel bar fracture, as well as the deterioration in moment resistance at a connection rotation of 0.08 radians.

FRAGILITY CURVES OF PILE-TO-WHARF CONNECTIONS

Fragility functions, which are probabilistic distribution functions, have become one of the more common tools used to estimate the level of damage a structure may have sustained due to various levels of seismic excitation. A set of fragility functions was developed for piles and pile-to-wharf connections to aid in the performance evaluation and potential seismic vulnerability of these important structures. The curves are based on performance data gathered from experimental studies of the pile connections using appropriate damage states.

Different engineering demand parameters (EDPs) were evaluated to ensure the most accurate representation of the component performance. For each test specimen, the damage states were quantified using the selected EDPs. Fragility curves were developed for specific categories of piles and connections. Comparison of the curves illustrates differences in performance with different design strategies. Finally, past response calculations on wharf systems are compared to these fragility curves to demonstrate the application of these curves.

Damage States for Pile-to-Wharf Connections

Three damage states were chosen for the moment-resisting pile-to-wharf connections, as shown in Table 3. The first damage state, DS1, is reached at initiation of pile or deck spalling. Minor repairs such as injecting the cracks with epoxy or applying shotcrete are necessary at this damage state, because of the corrosion potential of the steel reinforcement with port structures, and these repairs can be performed while the port is in operation. The second damage state, DS2, is reached when spalling becomes substantial and reinforcement becomes exposed to weathering agents. Although the concrete cover may be fully spalled the integrity of the core of both the pile and the deck is still intact. Therefore, the drop in resistance of the connection will be small, and the structural reliability of the connection can be considered sound. However, more extensive repair will be required to assure the durability of the connection. The exposed steel may be painted with zinc silicate, the spalled concrete can be removed and replaced with shotcrete, and cracks can be injected with epoxy and then coated. The final damage state, DS3, is reached when permanent damage requires significant replacement or repair. This damage state would be caused by either spalling of the core concrete or fracture of the dowels, since these cause significant and permanent loss of resistance. At this damage state, significant repair and disruption of service is required.

Several parameters including displacement or deformation, drift ratio, connection rotation, deformation ductility, strain in steel and concrete were investigated as possible EDPs for the pile-to-wharf connections. Connection rotation was determined to be the best candidate, since it was measured for each damage state observed for all connection categories.

Statistical Development of Fragility Curves

Fragility functions are cumulative distribution functions (CDFs) that relate the EDP value associated with a given damage state for all test specimens in a pile or connection group or category (Porter 2007). The data are first arranged with the EDP in increasing order and each occurrence is counted. The resulting data can be expressed as an empirical CDF. The logarithmic mean, \( \theta \), and the logarithmic standard deviation, \( \beta \), are then determined from this data. The standard deviation, \( \beta \), is increased to account for uncertainties in the data and number of tests. This is accomplished by adding an uncertainty component, \( \beta_u \), which is based upon the number of tests and reliability of the experimental database. Then the logarithmic standard deviation is established from this modified value.
The theoretical CDF, $F_i$, is based on a lognormal Gaussian distribution function. The theoretical fragility curves sometimes require additional adjustment to avoid crossing of fragility curves for higher and lower damage states. The accuracy and reliability of the theoretical prediction is then evaluated by a goodness of fit test. The test applies a statistical test to compare the fit of the empirical and theoretical CDFs. Alternate statistical tests are available for this evaluation, but the Liilliefors Test was found to be most suitable for these pile connections. This test compares the maximum distance between the theoretical and empirical CDFs, and the distance is compared to the 0.05 and 0.025 significance levels to establish the goodness of fit. Large significance values imply poor fit, and small values imply better fit. Additional details on development of these curves can be found in (Porter 2007).

Fragility curves for the dowel, extended pile, embedded pile and extended strand connection are provided in Figs. 12a, b, c, and d, respectively. Table 4 shows the goodness fit information for these fragility functions. There was insufficient data to develop fragility functions for other more narrowly defined connection categories and design parameters. However, examination of the data showed that increased axial stress within the core of the pile increases the influence of the P-Delta effect and decreases the ductility of the connection. The various connection categories achieved the DS1 damage state at similar connection rotations, since the initial spalling and cracking occurred in the concrete cover and appeared to be independent of the design characteristics and connection category.

Comparison of Fig. 12b with the other figures shows that the extended pile connection provided the best performance with less severe damage at lower connection rotations. This occurs because it is effectively a reinforced concrete connection. Comparison of Fig. 12d to the other figures shows that extended strand connection had the worse response, since it consistently had more severe damage at smaller deformations and connection rotations.

Experimental results for this connection showed that the connection had less damage and smaller deterioration in resistance at small and moderate deformation levels, and comparable resistance, stiffness and ultimate deformation capacity to those achieved with the dowel connection. The EDPs at the three damages states for CR connection were plotted on the dowel connection fragility curve in Fig. 13. The CR connection performance is plotted as large solid dots on the figure. Figure 13 shows that the CR connection provided better DS1 performance than 95% of dowel connections, better DS2 performance than 88% of the dowel connections, and better DS3 performance than 75% of the dowel connections. This comparison suggests that the CR connection is a promising alternative for good performance of future port systems.

CONCLUSIONS

This research has evaluated embedded dowel moment resisting prestressed concrete pile-to-wharf connections that are commonly used in current seismic design of marginal wharves and developed an improved connection detail that significantly delays the damage and deterioration of moment resistance of the pile, wharf deck and connection until much larger seismic deformations. When combined with prior research, this study shows that current connections sustain significant damage and deterioration of resistance starting at relatively small seismic deformations. In particular:

- They have early onset of pile and deck spalling at relatively small inelastic deformations. Significant spalling to the deck soffit cover concrete, and severe spalling into the core of the pile was noted with increasing deformation. This spalling causes dramatic loss in resistance even at moderate deformation levels. The spalling damage and resulting deterioration of resistance that results from this damage increases dramatically with increasing compressive axial load.

- The spalling damage requires significant repair even after moderate seismic deformation levels.

- Extended pile connections (build-ups) -used when the pile is driven below the wharf deck soffit level, exhibit different behavior, which is closer to that of a cast-in-place reinforced concrete moment connection.

As a consequence of this behavior, research was completed to develop an improved connection for the prestressed concrete piles. The research showed that:

- The addition of a bearing pad over the head of the pile reduces the initial elastic stiffness of the connection compared to current connection designs. It significantly delays (by approximately 0.04 radian connection
The addition of a CFEM flexible joint sealant wrapped around the perimeter of the embedded pile segment effectively eliminates spalling of the wharf deck.

Connections employing a full CDP over the head of the pile displaced in near rigid body rotation with an average 90% of the pile tip displacement resulting from end rotation of the pile at the interface. They also experienced axial deformations due to the low compressive stiffness of the pad material.

Annular bearing pad connections also experienced mostly rigid body rotation but to a slightly lesser degree than the full pad connections, with approximately 80% of the measured displacement resulting from end rotation of the pile. These specimens have a higher elastic lateral stiffness than full bearing pad connections, and they maintained similar axial stiffness noted with current connection design due to the concrete plug in the center of the annular bearing pad.

The normalized moment resistance of the connections subjected the same axial load were similar and symmetric.

Higher axial load resulted in larger moment resistance, greater deterioration of resistance and greater loss of effective resistance due to P-Δ effects.

Spalling of the pile cover concrete coincides with bearing pad strains of 0.4 and 0.6 mm/mm for CDP and ROFP materials, respectively.

An improved moment resisting connection is proposed and significant improvements in performance are achieved. This connection:

Employs a CDP or ROFP bearing pad over the head of the pile. The pad is an octagonal shaped annular ring, which covers all the pile concrete outside the perimeter of dowel bars. The annular ring permits easier placement of the grouted dowel bars, and it improves the stiffness and deformation of the connection. The improved connection also includes a flexible CFEM wrap around the perimeter of the embedded section of the pile, and the dowel bars are deliberately debonded over a significant length.

These changes result in 1) significant delays and reductions in spalling of the pile and wharf deck, 2) identical maximum moment resistance of the connection, 3) large reductions in the deterioration of resistance of the connection, 4) large reductions in repairs required after seismic deformation with no repair required for connection rotations in the order of 0.05 radians, and 5) slightly larger total rotational capacity than the current connection design.

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REFERENCES

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Axial Load (kN)</th>
<th>Debonded Length (mm)</th>
<th>Interface Bearing Pad</th>
<th>Pile Isolation</th>
<th>Yield Stress of Dowel (MPa)</th>
</tr>
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<tbody>
<tr>
<td>9</td>
<td>2,000</td>
<td>None</td>
<td>None</td>
<td>None</td>
<td>495</td>
</tr>
<tr>
<td>10</td>
<td>2,000</td>
<td>381</td>
<td>None</td>
<td>None</td>
<td>495</td>
</tr>
<tr>
<td>11</td>
<td>2,000</td>
<td>381</td>
<td>19mm Full CDP</td>
<td>None</td>
<td>480</td>
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<tr>
<td>12</td>
<td>2,000</td>
<td>381</td>
<td>19mm Full CDP</td>
<td>19mm foam</td>
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<td>13</td>
<td>2,000</td>
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<td>19mm CFEM</td>
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<td>14</td>
<td>4,000</td>
<td>381</td>
<td>19mm Full CDP</td>
<td>19mm CFEM</td>
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<td>381</td>
<td>19mm Annular CDP</td>
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<tr>
<td>16</td>
<td>2,000</td>
<td>381</td>
<td>13mm Annular RFOP</td>
<td>19mm CFEM</td>
<td>445</td>
</tr>
</tbody>
</table>

Notes: 1 kN = 0.224 kips; 1 mm = 0.039 inches; 1 MPa = 0.145 ksi
Table 2 -- Performance Characteristics of Six Key Specimens

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Peak Shear</th>
<th>Moment Capacity</th>
<th>Normalized Moment Capacity</th>
<th>Elastic Rotational Stiffness @ 1% Rot.</th>
<th>Initial Spalling</th>
<th>Substantial Spalling</th>
<th>Bar Fracture</th>
<th>Decrease in Moment 8% Rot.</th>
</tr>
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<tbody>
<tr>
<td>9</td>
<td>436 kN</td>
<td>1,216 kN-m</td>
<td>1.07</td>
<td>1,330 kN-m/% rad</td>
<td>1.42%</td>
<td>3.98%</td>
<td>8.21%</td>
<td>53%</td>
</tr>
<tr>
<td>10</td>
<td>440 kN</td>
<td>1,228 kN-m</td>
<td>1.08</td>
<td>1,330 kN-m/% rad</td>
<td>1.19%</td>
<td>5.64%</td>
<td>8.52%</td>
<td>40%</td>
</tr>
<tr>
<td>13</td>
<td>334 kN</td>
<td>1,029 kN-m</td>
<td>0.99</td>
<td>530 kN-m/% rad</td>
<td>3.81%</td>
<td>8.40%</td>
<td>&gt; 8.85%</td>
<td>19%</td>
</tr>
<tr>
<td>14</td>
<td>366 kN</td>
<td>1,184 kN-m</td>
<td>0.96</td>
<td>800 kN-m/% rad</td>
<td>2.47%</td>
<td>4.10%</td>
<td>None</td>
<td>55%</td>
</tr>
<tr>
<td>15</td>
<td>314 kN</td>
<td>1,072 kN-m</td>
<td>1.03</td>
<td>800 kN-m/% rad</td>
<td>5.45%</td>
<td>7.43%</td>
<td>8.94%</td>
<td>28%</td>
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<tr>
<td>16</td>
<td>316 kN</td>
<td>1,048 kN-m</td>
<td>1.03</td>
<td>750 kN-m/% rad</td>
<td>5.40%</td>
<td>6.36%</td>
<td>9.23%</td>
<td>24%</td>
</tr>
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</table>

Note: 1 kN = 0.224 kips; 1 m = 3.28 ft
### Table 3 -- Damage States and Repair Methods

<table>
<thead>
<tr>
<th>Damage States</th>
<th>DS1</th>
<th>DS2</th>
<th>DS3</th>
</tr>
</thead>
<tbody>
<tr>
<td>Initial cracking and spalling of the pile or deck</td>
<td>Remove spalled concrete, epoxy inject cracks, apply shotcrete, and coat repair</td>
<td>Substantial spalling of the pile exposing the spiral or in the deck to the depth of the embedded pile or spalling that exposes the deck reinforcing</td>
<td>Spalling into the core of the pile, or fractured bars or strands.</td>
</tr>
<tr>
<td>Repair</td>
<td>Replace with new pile adjacent to broken pile or significant repair to connection.</td>
<td>Replace with new pile adjacent to broken pile or significant repair to connection.</td>
<td>Replace with new pile adjacent to broken pile or significant repair to connection.</td>
</tr>
</tbody>
</table>
### Table 4 -- Summary of Data for Finalized Connection Fragility Functions

<table>
<thead>
<tr>
<th>Damage State</th>
<th># of data points</th>
<th>θ</th>
<th>βu</th>
<th>β</th>
<th>Quality</th>
</tr>
</thead>
<tbody>
<tr>
<td>Embedded Dowel</td>
<td>DS1</td>
<td>1.47</td>
<td>0.10</td>
<td>0.27</td>
<td>Moderate</td>
</tr>
<tr>
<td></td>
<td>DS2</td>
<td>4.85</td>
<td>0.10</td>
<td>0.46</td>
<td>Good</td>
</tr>
<tr>
<td></td>
<td>DS3</td>
<td>6.88</td>
<td>0.10</td>
<td>0.29</td>
<td>Good</td>
</tr>
<tr>
<td>Extended Pile</td>
<td>DS1</td>
<td>1.53</td>
<td>0.25</td>
<td>0.45</td>
<td>Moderate</td>
</tr>
<tr>
<td></td>
<td>DS2</td>
<td>6.68</td>
<td>0.25</td>
<td>0.32</td>
<td>Moderate</td>
</tr>
<tr>
<td></td>
<td>DS3</td>
<td>8.16</td>
<td>0.25</td>
<td>0.30</td>
<td>Moderate</td>
</tr>
<tr>
<td>Embedded Pile</td>
<td>DS1</td>
<td>1.57</td>
<td>0.25</td>
<td>0.37</td>
<td>Moderate</td>
</tr>
<tr>
<td></td>
<td>DS2</td>
<td>4.00</td>
<td>0.25</td>
<td>0.25</td>
<td>Poor</td>
</tr>
<tr>
<td></td>
<td>DS3</td>
<td>5.65</td>
<td>0.25</td>
<td>0.25</td>
<td>Poor</td>
</tr>
<tr>
<td>Embedded Strand</td>
<td>DS1</td>
<td>1.88</td>
<td>0.25</td>
<td>0.31</td>
<td>Poor</td>
</tr>
<tr>
<td></td>
<td>DS2</td>
<td>3.17</td>
<td>0.25</td>
<td>0.30</td>
<td>Moderate</td>
</tr>
<tr>
<td></td>
<td>DS3</td>
<td>5.94</td>
<td>0.25</td>
<td>0.32</td>
<td>Moderate</td>
</tr>
</tbody>
</table>
Figure 1 -- Typical Connections Used in Current Marginal Wharf Practice:

a) Wharf Cross Section, b) Embedded Dowel Connection, and c) Extended Pile Connection
Figure 2 -- Typical Connection Force-Deflection and Moment-Rotation Results:

a) Extended Pile with No Axial Load, b) Precast pile with no axial load, and c) Precast Pile with 10% Axial Load

Figure 3 -- Pile-Cap Connections: a) Extended pile with no axial load, b) Precast pile with no axial load, and c) Precast pile with 10% axial load.
Figure 4 -- Overall Specimen Dimensions and Reinforcing Steel

Figure 5 -- Deck Reinforcement (All Specimens)

Figure 6 -- Test Specimen Details (Deck and Pile Spiral Reinforcement Not Shown)
Figure 7 -- Test Rig

Figure 8 -- Overview of Instrumentation Layout
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Figure 10 -- Comparisons of Connection Performance

Figure 11 -- Visual Damage at Connection Rotation of 0.05 Radians
Figure 12 -- Fragility Curves for Pile Connections; a) Dowel Connection, b) Extended Pile Connection, c) Embedded Pile Connection, and d) Extended Strand Connection.

Figure 13 -- Performance of the Proposed (CR) Connection ($P/P_n = 0.1$) Relative to All Dowel Connections.
Synopsis: This paper summarizes the seismic aspects of the recently updated ACI 543 Committee document on design, manufacture, and installation of concrete piles. Although re-approved in 2005, the original Committee document was last updated in 2000. As part of the latest update, an entire Chapter on seismic design and detailing was prepared. The current state of practice regarding seismic ground motion determination and seismic soil–structure interaction was reviewed so as to be incorporated into the Committee document. In addition to summarizing the key seismic aspects of the Committee document, the paper will highlight the changes from previous versions.

Keywords: Seismic design, seismic forces, horizontal loading, batter pile, seismic design category
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INTRODUCTION

The ACI 543 Committee document on design, manufacture, and installation of concrete piles contains extensive guidance on a variety of concrete pile types. Prior to the latest edition, seismic aspects were addressed to a limited extent. The current document dedicates an entire chapter to seismic design and detailing.

SEISMIC ASPECTS OF PRE-2012 COMMITTEE DOCUMENT

ACI 543 “Design, Manufacture, and Installation of Concrete Piles” was last updated in 2000. As such, the Committee document captured the state of practice prior to that date. In re-approving the document in 2005, it was the Committee’s consensus that the document, as re-approved, captured current key seismic aspects of concrete pile design, manufacture and installation up to that time. The re-approved Committee document addressed seismic aspects in Chapter 2 – Design (ACI 543, 2005).

General design considerations were presented in Section 2.1. At the time, the horizontal component of seismic loading was the primary loading component considered. As batter (inclined) piles were often used to resist horizontal loading, unique considerations for batter piles resisting seismic forces were captured as follows: “The use of batter piles to resist seismic forces requires extreme care because these piles restrain lateral displacement and may require unattainable axial deformation ductility. When batter piles are used, a complete structural analysis that includes the piles, pile caps, structure, and the soil is necessary if the forces are to be properly accounted for, including the possibility of tension developing in some piles.” The Committee document gives references for conducting such analysis procedures.

Seismic loads and stresses to be resisted were presented in Section 2.2.1.4, with both horizontal and vertical seismic loading components to be addressed. The horizontal seismic loading, or base shear, is a function of the earthquake magnitude, the geographical area’s seismicity, and the structure’s fundamental period. Vertical seismic loading (both uplift and compression) result as the structure tends to overturn. The report cautioned that “battered piles supporting bulkheads or wharves have suffered great distress because they tend to resist all of the horizontal force in the structure, leading to failure of either the pile or the pile cap supported by the pile. Longer, more flexible batter piles have performed better. Other pile failures have occurred because of poor connection details between the piles and the cap, lack of adequate strength and rotational ductility in the pile section, and because of faulty analyses.”

Seismic design of piles was presented in Section 2.3.6. A critical component to seismic design is the pile’s ductility. Ductility is presented in the Committee document as “the capacity to undergo measurable amounts of inelastic deformation with little change in the forces causing deformation before reaching a failure state. Curvature or rotational ductility is important to seismic response. Ductility is a measure of toughness.” Key components for increasing ductility are identified, including: increasing compressive reinforcement, increasing concrete strength, reducing axial force, and providing confining reinforcement. Providing adequate ductility especially under fully reversed moments is emphasized, with 1) pile to cap connection and 2) along the pile (such as interfaces between soils of significantly differing stiffness) identified as key focus areas.

The Committee document presents the “ACI Spiral” (ACI 318, 1995) with slightly modified notation as a common process for detailing confining reinforcing. However, challenges regarding adapting the ACI Spiral to concrete piles are presented, including:

- The ACI Spiral has been widely adopted for use in the design of building columns and bridge piers to resist major seismic forces and deformations where the goal is to provide flexural ductility. For example, the ACI Spiral is used in Chapter 21 of ACI 318-95 with a lower limit to ACI 318-95 Eq. (10-6) of: minimum $\rho_s = \ldots$
0.12 \( f'_c / f_{sh} \) (Eq. 2.8) The minimum \( \rho_s \) requirement of Eq (2.8) governs when the ratio of \( A_g / A_{core} \) becomes less than approximately 1.27, which occurs only in large columns.

- Although an ACI Spiral provides excellent ductility, it is extremely difficult to provide the resulting amount of spiral reinforcement in many practical cases, such as square piles with longitudinal reinforcement arranged in a circular pattern. This difficulty arises because the area ratio \( A_g / A_{core} \) is unfavorable for square members containing round spirals and becomes especially unfavorable for small members. High concrete strengths also lead to large steel \( \rho_s \) requirements.

- It is not desirable to have the pitch too small because it makes concrete placement very difficult during manufacturing. Also, as the pitch becomes smaller, there is an increased tendency for the concrete cover outside of the closely spaced spiral to spall off during pile-driving operations.

Both the Uniform Building Code (UBC 1994) and Prestressed Concrete Institute (PCI 1993) recognized such challenges. Uniform Building Code (1994) adopts the ACI Spiral but limits the spiral steel ratio so that it need not be larger than \( \rho_s = 0.12 \frac{f'_c}{f_{sh}} \) for nonprestressed concrete piling in zones of high seismic risk. The PCI Committee on Prestressed Concrete Piling (1993) recommended minimum spiral steel ratios for members with round steel patterns and minimum steel areas for members with square steel arrangements for regions of high seismic risk.

The PCI recommendations were endorsed by ACI Committee 543 for application to both prestressed and reinforced concrete piling in regions where seismic resistance is required.

- In low to moderate seismic risk regions, the lateral steel reinforcement detailing ratio was presented as follows:

\[
\rho_s = 0.12 \frac{f'_c}{f_{sh}} \geq 0.007 \tag{2.9}
\]

with two limits on materials

\[
f'_c \leq 6000 \text{ lb/ft}^2 \text{ (40 MPa); and}
\]

\[
f_{sh} \leq 85,000 \text{ lb/ft}^2 \text{ (585 MPa).}
\]

- In high seismic risk regions, the lateral steel reinforcement detailing ratio was presented as follows:
2.3.6.2 Regions of high seismic risk—In regions of high seismic risk, the following minimum amounts of confinement reinforcement are recommended:

- Reinforcement of circular ties or spiral

\[ \rho_s = 0.25 \frac{f'_c}{f_{yh}} \left( \frac{A_g}{A_{core}} - 1 \right) \left( 0.5 + 1.4 \frac{P_u}{A_g f'_c} \right) \]  \hspace{1cm} (2.10)

but not less than

\[ \rho_s = 0.12 \frac{f'_c}{f_{yh}} \left( 0.5 + 1.4 \frac{P_u}{A_g f'_c} \right) \]  \hspace{1cm} (2.11)

where

\[ P_u = \text{factored axial load on pile;} \]

and with two limits on materials

\[ f'_c \leq 6000 \text{ lb/in.}^2 (40 \text{ MPa}); \text{ and} \]

\[ f_{yh} \leq 85,000 \text{ lb/in.}^2 (585 \text{ MPa}). \]

- Reinforcement of square spiral or ties

\[ A_{sp} = 0.3 s_{sp} h_c \left( \frac{f'_c}{f_{yh}} \right) \left( \frac{A_g}{A_{core}} - 1 \right) \left( 0.5 + 1.4 \frac{P_u}{A_g f'_c} \right) \]  \hspace{1cm} (2.12)

but not less than

\[ A_{sp} = 0.12 s_{sp} h_c \left( \frac{f'_c}{f_{yh}} \right) \left( 0.5 + 1.4 \frac{P_u}{A_g f'_c} \right) \]  \hspace{1cm} (2.13)

where

\[ h_c = \text{cross-sectional dimension of pile core measured center-to-center of spiral or tie reinforcement and with the limit that} \]

\[ f_{yh} \leq 70,000 \text{ lb/in.}^2 (480 \text{ MPa}). \]

The formats of the equations for high seismic risk regions, follow research conducted in New Zealand, and the New Zealand Standard Code of Practice for the Design of Concrete Structures.

The Committee document identified future needed research, since: 1) most reversed bending tests of piles had been conducted on octagonal pretensioned members, 2) tests (and supporting analytical work) of square members with round and square reinforcement patterns and round members of both reinforced and prestressed concrete (both solid and hollow) were lacking, and 3) rotation capacity and demand studies that are possible from various members with various soil profiles are needed. In addition, the importance of incorporating vertical accelerations was presented: “Experience from the 1994 Northridge earthquake in California reveals that at and near the epicenter, vertical accelerations
approached the magnitude of horizontal accelerations. This is significant because accelerations on the order of 1.0 g were recorded. The ramifications of high vertical accelerations should be considered by the structural engineer relative to piling because severe axial overloading of piles can occur under earthquake conditions. In geographic areas where high vertical accelerations are possible, it may be advisable to consider another case of loading that codes do not now consider, namely, normal service axial load plus that produced by an earthquake.” Lastly, seismic liquefaction of granular soils and its impact on pile axial and lateral design were presented.

2012 COMMITTEE DOCUMENT

As part of the 2012 Committee document update, an entire Chapter 5 – Seismic design and detailing considerations was prepared. The current state of practice regarding seismic ground motion determination and seismic soil–structure interaction was reviewed so as to be incorporated into the document. This effort was lead by the late Roy Armstrong. The Committee and the industry have been well served by Roy’s efforts, and the Chapter stands as a lasting legacy to Roy and his work.

General Seismic Impacts

General seismic impacts on pile behavior are presented in Section 5.2, recognizing that piles tend to move with the earth’s crust during an earthquake, and pile distortions result due to permanent ground displacements (PGDs) or transient ground shaking (TGS).

- Fault ruptures and general ground subsidence or upheaval can result in large PGD. Large PGD can also result from earthquake-induced ground settlement, lateral ground-spread movement or flow slides associated with liquefaction, and from earthquake-induced landslides. Piles experience permanent distortions (locked-in pile loads or strains) when pile materials can accommodate the induced PGD, or structural damage when induced displacements are larger than the pile materials can sustain. Thus, an analysis approach is to identify PGD type(s) likely to develop, potential ground displacement magnitudes, and pile strains and loads that would develop when estimated ground displacements are imposed on the piles. PGD effects are superimposed on the pile loads for the supported structure and the earthquake-induced structural loadings.

- TGS motions occur as earthquake-induced ground stress waves propagate though the earth’s crust. Transient waves induce strains in piles as they distort to conform with free-field strains. Thus, an analysis approach is to estimate free-field strains (depend on: wave type, velocity, ground strength, and ground motion magnitude). Loads are estimated by imposing free-field motions on pile, or by using dynamic soil-pile interaction (account for pile flexibility and soil-pile interaction with ground motions). Another aspect is to account for pile and ground motions excite the superstructure. Superstructure inertial forces produce dynamic forces (axial, shear, and moments) that are transmitted to the substructure. Substructure and piling undergo additional dynamic movements as these inertial forces are transferred to the supporting soils. The Committee document gives references for conducting such analysis procedures. The analyses are complex, and the kinematic and inertial forces are not generally in phase.

- Piles, caps, and other elements connecting the piles and structure designed to resist kinematic and inertial forces plus PGD effects. Piles designed to resist TGS kinematic and inertial forces, and transferring them to the supporting soil without developing intolerable structural displacements.

- Observed pile behavior during earthquakes: 1) highest damage rates are associated with areas where PGD develop, 2) damage associated with TGS can be less intense, 3) TGS can occur over large areas away from fault zones, and 4) absent large PGD, the structure-induced inertia forces will generally control seismic pile design rather than the TGS-induced pile strains.

Seismic Pile Behavior

Seismic pile behavior is presented in Section 5.3. Observed behavior from the previous Committee document on the topics of batter piles, and liquefaction and soil spread, are reiterated. In addition, cracking to crushing or hinge formation at pile to cap interface, and cracking at other below grade levels (maximum moment points, large soil stiffness boundaries, changes in reinforcement) from recent earthquakes observations are presented.
Geotechnical and Structural Seismic Design Considerations

Geotechnical and structural seismic design considerations are presented in Section 5.4. Geotechnical design considerations include: 1) axial and transverse drag loads or displacements induced on the pile by settling and spreading ground, plus earthquake-induced inertial loads and the normal structural loads that could act concurrently, and 2) capacity developed below any liquefiable zones. Structural design considerations include: 1) resist forces under applicable seismic service or factored load combinations of ACI 318 or other controlling codes, and to sustain imposed PGD where present, 2) pile sections without lateral soil support should be designed as columns (consider seismic-induced P-Δ effects), 3) design and detail piles to accommodate thrust, moment, and shear loads imposed by load combinations, to transmit these forces between the pile and pile-cap or other structure connections, and accommodate the installation and handling forces associated with developing the required geotechnical capacities, and 4) designing on the basis of strength alone may not be adequate - need to address ductility. Piles properly structurally detailed will often satisfy most model code reinforcing-detail requirements and perform adequately structurally.

Seismic Detailing

Seismic detailing of concrete piles is presented in Section 5.5. This section reiterates: 1) the importance of providing ductility, 2) the ACI Spiral and its challenges applying to concrete piles, and 3) the continued endorsement of UBC and PCI guidance regarding concrete pile reinforcement detailing.

- In low to moderate seismic risk regions, the lateral steel reinforcement detailing ratio was presented as follows:

Regions of low to moderate seismic risk (SDC C)—In regions of low to moderate seismic risk, PCI (1993) recommends the lateral reinforcement for prestressed concrete piles meet the following steel ratio

$$\rho_s = 0.12 \frac{f'_c}{f_{sh}} \geq 0.007$$

(5.5.2.2a)

with two limits on materials:
- $f'_c \leq 6000 \text{ psi (40 MPa)}$
- $f_{sh} \leq 85,000 \text{ psi (585 MPa)}$.

- In high seismic risk regions, the lateral steel reinforcement detailing ratio was presented as follows:
Regions of high seismic risk (SDC D, E, or F)—In regions of high seismic risk, PCI (1993) recommends the following minimum amounts of confinement reinforcement.

Reinforcement of circular ties or spiral

\[ \rho_s = 0.25 \frac{f_c'}{f_{sh}} \left( \frac{A_s}{A_{core}} - 1 \right) \left( 0.5 + 1.4 \frac{P_u}{A_s f_c'} \right) \]  

but not less than

\[ \rho_s = 0.12 \frac{f_c'}{f_{sh}} \left( 0.5 + 1.4 \frac{P_u}{A_s f_c'} \right) \]

where \( P_u \) is the maximum factored axial compressive load on pile with two limits on materials:

- \( f_c' \leq 6000 \) psi (40 MPa); and
- \( f_{sh} \leq 85,000 \) psi (585 MPa).

Reinforcement of square spiral or ties

\[ A_{sp} = 0.3 s_{wp} h_t \frac{f_c'}{f_{sh}} \left( \frac{A_s}{A_{core}} - 1 \right) \left( 0.5 + 1.4 \frac{P_u}{A_s f_c'} \right) \]

but not less than

\[ A_{sp} = 0.12 s_{wp} h_t \frac{f_c'}{f_{sh}} \left( 0.5 + 1.4 \frac{P_u}{A_s f_c'} \right) \]

where \( A_{sp} \) is the total area of area of transverse reinforcement in the direction considered, and \( h_t \) is the cross-sectional dimension of pile core measured center-to-center of spiral or tie reinforcement; and with the limit that

- \( f_{sh} \leq 70,000 \) psi (480 MPa)

Center-to-center spacing of transverse reinforcement in the ductile region should not exceed the lesser of: one-fifth of the pile diameter, six times the longitudinal strand diameter, or 8 inches (203 mm).

At the time of the Committee document update, ACI 318 provisions: dealt with unsupported piling lengths, the short zone below the pile cap where a plastic hinge might form, and the pile to cap connection. In abbreviated form, ACI 318-08 Section 21.12.4, requires: 1) continuous longitudinal reinforcement in the pile region resisting the design tension forces, and detailing of the reinforcement into the pile and pile cap, 2) tests demonstrating that grouted bars or dowels, when used as connections, develop at least 125% of the specified bar yield strength, 3) transverse reinforcement in accordance with ACI 318-08 Section 21.6.4 for a distance equal to the greater of five pile widths or 6 feet (1.83 m) below the pile top, or below the unsupported length when the pile penetrates air, water, or soil incapable of providing lateral pile support, 4) the length of transverse reinforcement provided in precast concrete piles should be sufficient to
account for potential pile tip variations, and 5) pile caps containing batter piles should be designed to resist the full compressive strength of the batter piles acting as short columns.

Also at the time of the Committee Document update, International Building Code (IBC) provisions closely, but not exactly follow NEHRP provisions. The Committee noted this variation, and cautioned that a rational technical definition of the earthquake-induced flexural demand based on different pile types is needed.

The document gives additional guidance for different concrete pile types.

- **Prestressed piles:**
  - ACI 318-08 Chapter 21 does not apply
  - SDC C: use Eq. 5.5.2.2a
  - SDC D through F: use Eq 5.5.2.2b through 5.5.2.2e
  - Upper limit of $\rho_s = 0.021$
  - Minimum concrete strength $f'c$ is 5000 psi (34.5 MPa)
  - For the portion of the pile below the flexural length, minimum transverse reinforcement is one-half of that recommended in the flexural length.
  - SDC C, IBC does not provide special seismic limits on spiral or tie sizes and spacing requirements, so static code provisions for prestressed piles apply.
  - SDC D through F structures, IBC provisions limit the maximum spacing of spiral and hoop reinforcement in the ductile zone to the lesser of one-fifth the pile width, six times the longitudinal-strand diameter, or 8 inches (203 mm).

- **Reinforced precast (nonprestressed) piles:**
  - SDC C, transverse-confinement reinforcement:
    - within 3D below the cap, closed ties or spirals 0.375 inch (10 mm) or larger diameter spaced the lesser of eight longitudinal bar diameters or 6 inches (150 mm).
    - In the remainder of the pile, tie or spiral spacing the lesser of 16 bar diameters or 8 inches (203 mm).
  - SDC D through F structures, transverse-confinement reinforcement:
    - within 3D below the cap, in accordance with ACI 318-08 (ie. the ACI Spiral). For Site Classes A through D that are not subject to liquefaction, the spiral steel ratio can be limited to one-half of that required by Eq. (5.5.1a) and (5.5.1b).
    - Maximum tie or spiral spacing lesser of one-fourth the pile width, six longitudinal bar diameters, or 4 to 6 inches (100 to 150 mm), depending on the pile width and transverse bar spacing.
    - At depths below 3D, same as SDC C structures

- **Uncased, auger grout, concrete-filled shell piles**
  - SDC C, transverse-confinement reinforcement:
    - 3/8 inch (10 mm) or larger closed ties or equivalent spirals spaced the lesser of eight longitudinal bar diameters or 6 inches (150 mm) in the upper 3D.
    - Spaced no more than 16 longitudinal bar diameters in the remainder of the flexural length.
  - SDC D through F, transverse-confinement reinforcement:
    - Within 3D below the cap, in accordance with ACI 318-08 (ie. ACI Spiral), except that for Site Classes A through D, which are not subject to liquefaction, spiral steel ratio reduced to one-half required by Eq. (5.5.1a) and (5.5.1b). Transverse reinforcement also required to be a minimum No. 3 (No. 10) bars for 20 inch (500 mm) or less piles, and No. 4 (No. 13) bars for piles greater than 20 inches (500 mm).
    - In the flexural length below 3D, minimum spacing the lesser of 12 longitudinal bar diameters, one-half of the pile width, or 12 inches (300 mm).
    - IBC waives transverse reinforcement requirements for concrete-filled shells meeting shell confinement requirements of Chapter 4, Table 4.3.2.1.
Concrete-filled steel pipe and tube piles
  - IBC does not mandate transverse reinforcement, but requires minimum wall thickness of 3/16 inch (5 mm) as opposed to the minimum 0.1 inch (2.5 mm) thickness for nonseismic loadings.

Seismic axial reinforcing is addressed by providing continuous longitudinal reinforcing in section resisting tension forces per ACI 318-08, Section 21.12.4.2. In addition, IBC additional minimum longitudinal steel ratios are identified for some piles:
  - Reinforced precast piles, minimum longitudinal reinforcement equal to 1% of concrete section for the full pile length in seismic SDC C through F. (less than the 1.5% recommended in the Report to cover handling and installation conditions).
  - Uncased, auger grout, and concrete-filled shell piles, longitudinal steel ratios of 0.25 percent for SDC C structures and 0.5 percent for SDC D through F structures. This longitudinal steel, which is to consist of a minimum four bars, is to extend throughout the flexural demand zone.
  - Concrete-filled pipe and tube piles, minimum wall thickness of 3/16 in. (5 mm).

Pile-to-cap connections are discussed along with case history and laboratory testing results. Detailing per IBC requirements are presented, including:
  - SDC C: confinement reinforcement surrounding the anchored steel, with at least half the reinforcement required for a column.
  - SDC D, E, or F:
    - Tension: anchored to develop the smaller of the tensile strength of the pile reinforcement, the pile-soil uplift capacity multiplied by 1.3, or the maximum force from the appropriate factored loads.
    - Bending: anchored to develop the smaller of the nominal strength of the pile or the factored loads from the appropriate load combinations

Lastly, potential vertical acceleration impacts are presented. It is noted that near the earthquake epicenter, vertical accelerations can approach horizontal accelerations. In addition, in geographic areas where high vertical accelerations are possible, the designer should consider the loading case of normal service axial load plus that produced by vertical seismic accelerations, which is not considered in current Codes.

**NEEDED RESEARCH**

The document identifies areas of needed research. Tests (and supporting analytical work) of square members with round and square reinforcement patterns, and round members of both reinforced and prestressed concrete are needed. Such tests should include a full range of confinement reinforcement, ratios or areas, and should include tests with and without axial loads. As of 2010, to the Committee’s knowledge no tests have included tension thrusts. Both solid and hollow members should be considered. Also, studies of the rotation capacities that are possible from various members, and studies of the rotational demands or requirements that can be imposed by the supported structure with various soil profiles are needed. Lastly, the IBC and NEHRP differences presented in the document require resolution.
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STRUCTURAL ANALYSIS OF PRETENSIONED CONCRETE PILES

William L. Gamble

Synopsis: The structural analysis of prestressed concrete piles is similar to the analysis of reinforced concrete columns in many respects but there are important detail differences. The construction of the $M - P$ (moment-thrust interaction) curve requires consideration of the stress-strain curve for strand and of the substantial initial strains in the steel and concrete. When considering length effects, it will be found that much of the length of a prestressed pile will remain uncracked, which contributes significantly to its stability against buckling.

Keywords: pretensioned concrete; pretensioned piles; reinforced concrete; moment-thrust interaction; $\varphi$-factors; length effects; $P - \Delta$ effects.
ACI Fellow William L. Gamble is Professor Emeritus of Civil and Environmental Engineering at the University of Illinois at Urbana-Champaign. He has been engaged in teaching, research, and consulting on reinforced and prestressed concrete topics for five decades. He is the immediate past chair of ACI Committee 543, on Concrete Piles. He is a Licensed Structural Engineer in Illinois.

**INTRODUCTION**

Pretensioned concrete piles have been in use for a long time, so they are familiar objects in foundation construction. Precast reinforced concrete piles also exist and are widely used in some areas, but prestressing gives some advantages. If the precompression is 1,000 lb/in.$^2$ (7 MPa), a common value, the cracking moment and the direct tension cracking force are roughly three times that of a reinforced concrete member with the same cross section. This has two important benefits. Much longer members can be handled without cracking, and the resistance to reflected tension waves that exist in some pile driving situations is greatly increased.

Figure 1 shows three typical cross sections. The 14-in. (360 mm) square pile will often have the strands in a square pattern, the larger square pile may have a central circular void, and 0.6 in. (15 mm) strands will be common. For non-seismic cases, the spirals will be light. The spirals will be much heavier for short sections near the ends to improve driving resistance, and much heavier spirals will be required for seismic cases. Spirals will satisfy ACI 318, Sec. 10.9.3 only for the most severe seismic cases.

![Figure 1 -- Typical Pretensioned Pile Sections (1 in. = 25.4 mm)](image)

The analysis of these sections for thrust plus moment is best approached through moment-thrust ($M-P$) interaction diagrams. This similar to the case for reinforced concrete columns, but with important detail differences that will be considered in the next section.
Before considering the $M$-$P$ relationship, it must be noted that many piles in non-seismic situations have only very small moments, or very small eccentricities of the applied loads relative to the pile centroids. The ACI 543 Guide (Ref. 1) gives several equations for the allowable axial pile loads at service load conditions.

It also must be noted that these allowable load calculations, whether by the simple equations or a complete $M$-$P$ analysis, give the structural capacity of the pile member. The capacity of the soil to resist the applied forces must also be demonstrated, and in many practical cases will govern. The ACI 543 Guide has an extensive discussion of this important issue.

The calculation of the effects of length requires two steps. First, the Moment-Curvature ($M$-$\Phi$) relationship is required. Again, this is similar to the case of reinforced concrete columns but with important detail differences. Then, an analysis is required to find the deflected shape of the member, considered as a beam-column, taking into account the effects of deflection on the moment diagram, that is considering $P$-$\Delta$ effects or second-order effects.

**MOMENT-THRUST INTERACTION ($M$-$P$) CURVES**

The first detail difference from reinforced concrete is the stress-strain curve for the steel, as shown in Fig. 2. The steel is very strong, compared to reinforcing bars, and it has no well-defined yield point.

![Typical Stress-Strain Curve for Grade 270 Strand (1 ksi = 6.9 MPa)](image)

**Fig. 2 -- Typical Stress-Strain Curve for Grade 270 Strand (1 ksi = 6.9 MPa)**

This curve is for low-relaxation strand, and stress-relieved strand will have slightly lower stresses when the strain is around 1%. The type of strand will make very little difference to the nominal strength of a member, but the low-relaxation strand will lead to somewhat higher concrete precompression and thus to higher cracking moments and to higher cracking forces in direct tension, sometimes a major consideration in pile driving.

Since this is not an elastic-plastic stress-strain curve, the determination of the stress at a particular strain is somewhat more complex than for reinforced concrete. Equations for this curve have been derived, as curve-fitting exercises, or the curve can be described by a series of straight-line segments, as in Fig. 2.
The second detail difference from reinforced concrete is the presence of a large initial tensile strain in the steel, and a much smaller initial compressive strain in the concrete. Fig 3 (b) illustrates this.

![Fig. 3 -- Strains and Stresses in Pile Section (1 in. = 25.4 mm)](image)

This particular pile section is used in all of the following examples. Notice the strand arrangement is slightly different relative to the two axes, but the capacities about the $x$- and $y$-axes are not much different.

The pile was used in a large job that illustrated the need to conduct static load tests to confirm the results of wave-equation and other analyses. No load test was done before beginning production pile driving, contrary to ACI 543 (Ref. 1) and many other recommendations, and that decision cost the owner something in the range of $25$ million in added costs, not to mention a huge delay in getting the project completed, since the first load test failed miserably.

Figure 3 (c) shows strain conditions for a single neutral axis position, leading to a single point on a moment-thrust ($M-P$) diagram. Each steel strain value leads to a stress, and then a force, as shown in Fig. 3 (d). The concrete force is exactly as in a reinforced concrete case. Forces and moments of forces are then summed:

$$P_a = 0.85 \cdot f'c \cdot (a) \cdot b - \Sigma T_s - Cc - \Sigma Ts$$

The $\beta_i$ term in Fig 3(d) is a stress-block factor from ACI 318-11 (Ref. 2).
One difference from reinforced concrete is seen here. All layers of steel are in tension, for all plausible strain distributions. Moments are then summed about the mid-depth axis:

\[ M_n = C_c \times (h/2 - a/2) + \sum (T_s \times (h/2 - d_s)) \] where \( h \) = total depth of the section.

In this summation, some of the steel forces have moments of the same sign as the concrete force, and some are opposite. This process is repeated for different neutral axis depths until the \( M-P \) curve has been defined to sufficient precision.

The axial compression case, in absence of moment, is a separate calculation, as it is for reinforced concrete. A major difference from reinforced concrete exists, however, as all strands will have significant tension stresses when the member fails in compression. The concrete failure strain merely represents a reduction in stress from the pretension level, and the steel strain is in the elastic range. This leads to:

\[ P_{0n} = 0.85 \times f'c \times (A_g - A_s) - \left(\frac{f_{se}}{E_s} - 0.002\right) \times A_{st} \times E_s \]

where \( A_g \) is the total area of prestressed steel. The steel stress is large, and will normally exceed 100 ksi. There is no moment, as long as the steel arrangement is symmetrical. One consequence of this large steel stress is that direct compression failures will be even more explosive than in the case of reinforced concrete.

The pure tension capacity is sometimes needed, and is simply the steel area times steel strength:

\[ T_{0n} = A_{st} \times f_{pu} \]

The resultant \( M-P \) curve for the member in Fig. 3 (a) is shown in Fig. 4. This specifically shows the nominal strength curve for a member with 14 - 0.6-in. (15 mm) low-relaxation strands, and with \( f'c = 6,000 \text{ lb/in.}^2 \) (41 MPa)

The arithmetic was done with a spreadsheet. An obvious difference from the reinforced concrete column case is the absence of a well-defined balance point, because of the lack of a well-defined yield point for the reinforcement. In the cases of large tension combined with a moderate moment, fracture strain of the strand may be reached before the concrete fails in compression, so caution is needed in this situation. The material to this point is well known. See, for example, Refs. 3 or 5.

ACI design procedures require that the nominal capacity be reduced by a \( \phi \)-factor that is dependent on the reinforcement strain. The \( \phi \)-factor depends on whether the section is tension-controlled or compression-controlled, as defined in ACI 318, Sec. 10.3.

The tension- or compression-controlled ranges are governed by a steel strain called \( \varepsilon_t \), which is the same as the \( \Delta \varepsilon_{syn} \) of Fig. 3 (c). The ACI definition is “\( \varepsilon_t \) = net tensile strain in extreme layer of longitudinal tension steel at nominal strength, excluding strains due to effective prestress, creep, shrinkage, and temperature.” The critical points are \( \phi = 0.65 \) for \( \varepsilon_t \leq 0.002 \) tension and \( \phi = 0.9 \) for \( \varepsilon_t \geq 0.005 \). A linear interpolation is used for points between the two critical strains. Fig. 5 is a plot of the nominal thrust capacity versus \( \varepsilon_t \) for the example cross section.

For this member, the tension-controlled region occurs only with a tension force of about 130 kips (580 kN) or more, and it is compression-controlled for all compression forces over about 235 kips (1050 kN).

Fig. 6 shows the almost-linear variation between \( P_n \) and \( \phi \).
Fig. 4 -- *M-P Curve for 20-in Square Pile, Compression Positive* (1 kip = 4.45 kN, 1 ft-kip = 1.356 kN-m)

Fig. 5 -- *Pₙ versus εₜ for Example Pile* (1 kip = 4.45 kN)
The appropriate value of $\phi$ is then multiplied by the corresponding values of $M_n$ and $P_n$ to give a point on the $M_n - P_n$ diagram. Fig. 7 shows the $M-P$ diagram including the $\phi$-factor for the same example member, and this is the resistance that is compared with the effects of the factored loads.
The $M - P$ relationship is not exactly what one would expect from a completely rational system, but it is the result of the ACI process. This problem has been observed in the past (Ref. 4), although the rectangular reinforced concrete case is odd in a different way, and a circular reinforced concrete column is again different. Since $\phi$-factors have changed over time and differ from organization to organization, one must be certain which set was used in preparing various existing design aids.

In many cases, this is as far into the analysis as one needs to go. A fully-embedded pile in a non-seismic application will have neither length effects nor ductility concerns. The maximum moment is ordinarily at the bottom of the pile cap, and the moment rapidly reduces with distance below the cap. Even very weak soils will provide enough lateral support to prevent buckling. The ACI 543 Guide addresses these concerns.

Seismic situations may require large amounts of confinement reinforcement, in the form of spirals or square hoops. The ACI 543 Guide has considerable information on this important aspect, extracted from various previous studies and current codes and recommendations.

The next section deals with some aspects of ductility, but not in the seismic context. It is primarily intended to give information needed for the rational analysis of the effects of length.

**MOMENT-CURVATURE RELATIONSHIPS**

Figure 8 contains moment-curvature ($M - \Phi$) curves for the example pile from Fig. 3 (a), for two different values of thrust.

![Moment-Curvature Curves](image)

**Fig. 8 -- Moment-Curvature Relationships for Example Pile Section**

(1 ft-kip = 1.356 kN-m, 1 rad/in. = 39.4 rad/m)

The initial straight line is the elastic response of the uncracked section, ending at the cracking moment. This can be expressed as:

$$\Phi = \frac{M}{EI}$$
where $M$ is the applied moment, $E$ is the elastic modulus, and $I$ is the moment of inertia. Gross section properties will be adequate.

The cracking moment is then computed as $M_{cr} = \Delta f_c * S$, where $\Delta f_c$ is the sum of the precompression, $f_{ce}$, the tensile strength of the concrete in flexure, $f_t$, and the effects of the external load, $P/A_s$. The tensile strength is usually taken as $6 \sqrt{f_c'}$ in lb/in.$^2$ where $f_c'$ is also in lb/in.$^2$ units ($0.17 \sqrt{f_c}$ MPa where $f_c'$ is also in MPa). This assumes that the compression side stresses are low enough to ensure elastic behavior, and this will usually be the case.

The end points of the curves come from the same calculations leading to the $M_n-P_n$ diagram. Fig. 3 (c) is an example case, where $\Phi = \epsilon_{cu}/c$, where $\epsilon_{cu} = 0.003$ in most instances. The neutral axis depth is adjusted to produce the desired value of thrust, $P_n$.

Intermediate points are found in a similar manner. A concrete strain is selected, $\epsilon_c = 0.0025$ for instance, and the neutral axis, $kd$, is adjusted until the desired thrust is reached. Moments of the forces are summed, and curvature is found as $\epsilon_c/kd$. However, a stress-strain curve is required for the concrete since the rectangular stress distribution shown in Fig. 3 (d) is satisfactory only at ultimate. The simple elastic-plastic curve shown in Fig. 9 will lead to reasonable results with fairly minimal arithmetic.

In this curve, $\epsilon_p = 2*(1 - \beta_1)*\epsilon_{cu}$. With this definition of the plastic strain, the area under the curve at ultimate is identical to that of the “ACI Stress-Block” illustrated in Fig. 3 (d) but the centroid of the area is slightly farther from the compression face than that in Fig 3 (d).

Intermediate points are found, with different concrete strain values, until the curve is adequately defined. However, if one traces the curve point-by-point back from the ultimate value, it is seen that it does not reach the previously calculated cracking moment and curvature. This because the process based on Fig. 3 ignores the tensile strength of the concrete, but rather assumes that if the concrete strain is tension, the resulting stress is zero. One can either greatly complicate the analysis with consideration of “tension-stiffening,” or simply draw a reasonable curve, using a bit of engineering judgment. Fig. 8 shows one version of this, for the 500 kip compression case. The straight line connecting the cracking and ultimate points is a reasonable lower-bound to the post-cracking stiffness.

It is fair to observe that the loss of stiffness accompanying cracking is very large, and is likely to be important in any length-effect calculations. A reinforced concrete member with a similar moment capacity would also have a stiffness reduction at cracking, but the reduction will be considerably less, primarily because the reinforced concrete section will have much more steel. This pile has a gross steel ratio of 0.76%, much lower than the comparable reinforced concrete member.

The $M-P$ calculations were done with a spreadsheet. The curvature calculations were done with a now-ancient program written in Turbo Pascal.
LENGTH EFFECTS

Length effects have been computed using various approaches. One of the earlier papers on prestressed pile length effects (Ref. 5) used the Secant equation, stated as:

\[ M_{\text{max}} = M_{\text{end}} \times \text{Sec} \left( \frac{P l^2}{8 EI} \right) \]

where \( l \) is the length of the member.

This solution is correct, given three specific conditions: (1) Elastic, (2) Prismatic, and (3) Equal moments of the same sense at both ends of the member.

An uncracked pretensioned pile satisfies the first two, but the third describes a moment situation seldom encountered in the real world of design and construction. Compared to a more realistic moment diagram, for instance maximum first-order moment at one end and zero at the other, this equation leads to conservative or very conservative results.

Reference 5 has additional problems leading to more conservatism. The section stiffness, \( EI \), used in the equation is very low, and is equivalent to taking a straight line from zero to ultimate moment, that is:

\[ EI = \frac{M_u}{\Phi_u} \]

The analyses in effect apply this stiffness to the full length of a member which is actually uncracked over much of its length, adding more conservatism. In addition, the many curves presented in the paper include the \( \phi \)-factor from the ACI Code of that era, reducing their potential usefulness since the current approach to \( \phi \)-factors is quite different in ACI Codes.

One consequence of the very low value of \( EI \) used in Ref. 4 can be shown with the Secant equation. Consider the cross section shown in Fig. 3 (a), with \( E = 4400 \text{ k/in.}^2 \) (30.3 GPa), \( I_g = 13,333 \text{ in.}^4 \) (5.55 * 10^9 mm^4) (\( EI = 58.7 * 10^6 \text{ k-in}^2 \)) (\( EI = 1.68 * 10^9 \text{ kN-m}^2 \)), \( l = 47.5 \text{ ft} = 570 \text{ in.} \) (14.48 m) and \( P = 500 \text{ kips} \) (2225 kN). The Secant of this combination is 1.485. This is a fairly slender member, with \( l/r = 95 \), if \( r = 0.3*\text{member thickness.} \)

For this member, \( M_u/\Phi_u = 456.77*12 \text{ k-in/0.000249} \text{ Rad/in.} = 22 * 10^6 \text{ k-in}^2 \) (619.4 kN-m/0.00980 Rad/m = 63 * 10^3 kN/m^2), much lower than the uncracked stiffness. For the same length and axial force, the Secant = 4.74, three times greater than for the uncracked pile.

This set of analyses has been adopted in at least two PCI Reports (Refs. 6 and 7), without discussion of how conservative the results may be.

A logical alternative is a second-order analysis, as is required by the ACI Code for columns over some critical length. This can be done in various manners, but the use of the Newmark Numerical Analysis Method (Ref. 8) is an easily implemented way.

The original paper considered elastic members, but the method is quite general and will work with any case where the \( M - \Phi \) curve can be defined. The method was derived for hand calculations, and gave one fairly simple methods to consider non-prismatic members, for example, but it was in some respects a method waiting on the invention of the spread-sheet. The analysis is an efficient method for integrating curvatures to compute slopes and deflections.

For a second-order analysis, this becomes an iterative method where a moment diagram, including the effects of the deflections on moments, is assumed. A reasonable starting point is to use the deflected shape caused by the end moments only, ignoring the axial thrust. This is a first-order analysis. The next step is to add product of the local deflection times the axial thrust to the first-order moments at each node point. With this new moment diagram, curvatures are again computed and then the new deflected shape is found. The computed deflected shape is then compared with the previously assumed deflected shape. If they are the same, within some small tolerance, the solution is the answer, giving a deflected shape that is in equilibrium with the applied loads. If not, the new deflected shape is used as the starting point for another round of calculations. Stable members lead to convergence.
in only a few cycles, marginally stable members may take many cycles, and unstable cases diverge. The macro functions in a spread-sheet give an easy way of doing the required iterations.

In the implementation used, the pile length was divided into 20 equal segments, that is, there were 19 internal nodes where moments and curvatures were computed, plus two end nodes. As a simple test, the two cases using the Secant equation noted above were repeated in the elastic ranges, and the results agreed exactly.

The effects of length can be considered in several different ways. In this case the axial force was held constant and the end moments, or end moment, were increased until instability occurred. The same 47.5 ft (14.48 m) long member considered previously, subjected to 500 kips (2,225 kN) compression, was used.

The bilinear $M - \Phi$ curve shown in Fig. 8 was used, with the straight line extending from cracking to ultimate taken for the post-cracking part of the analysis. This represents a lower bound to the cracked section stiffness. Two cases were analyzed, one with equal moments of the same sense at both ends, as in the Secant equation case, and one with moment applied to one end and the other end pinned. This can also be thought of in terms of eccentricities of end forces, with the eccentricity increased until instability is reached. The case with equal end moments followed the Secant equation perfectly until the mid-length section cracked, when the end moment was 225.0 k-ft (305.1 kN-m). At $M = 226$ k-ft (306.5 kN-m), the solution barely converged, and $M = 226.08$ k-ft (306.6 kN-m) was the largest end moment where convergence could be achieved. At this force, the center three node points had cracked, that is cracking extended over just 0.1 of the pile length. The pile in effect went from being quite stable to quite unstable at the instant it cracked. If one plots the mid-length moment versus the end moment, the graph barely deviates from the initial straight line.

The case with one end pinned and the moment applied at the other was quite different. The maximum moment was only very slightly larger than the end moment until the moment was near the instability point. The maximum moment occurred at the first node point from the end until well after first cracking. At first cracking, the maximum moment was 0.4% larger than the end moment. The end moment at first cracking was 333.33 k-ft (452.0 kN-m), and the maximum moment at which convergence occurred was 355 k-ft (481 kN-m). As the end moment increased beyond first cracking, cracking gradually spread, reaching the 7th node, or 35% of the pile length. The point of maximum moment moved to the 3rd node from the end, and the final point the maximum moment was 4.5% larger than the end moment. The effects of length and of cracking were obviously much less than in the case of equal end moments of the same sense.

Fig. 10 shows the 1st-order and 2nd-order moment diagrams at the maximum moment where convergence could be achieved.
Fig. 10 -- First-Order and Magnified Moment Diagrams for Example Pile
(1 ft-kip = 1.356 kN·m, 1 ft = 304.8 mm)

Fig. 11 shows the deflected shapes corresponding to the first order moments and the final converged shape.

Fig. 11 -- First-Order and Magnified Deflected Shapes for Example Pile (1 in. = 25.4 mm, 1 ft = 304.8 mm)
A more traditional way of looking at length effects is to assume an eccentricity and gradually increase the axial force. An eccentricity of 5.0 in. (12.7 mm) was assumed, for the two cases of equal moments at both ends and moment at one end and a pin support at the other. Some of the results of these analyses are shown in Fig. 12.

Figure 12 shows the $M_n-P_n$ interaction diagram, the end moment curve, and the mid-length moment curve for the equal end moment case. This curve stops at 500 kips (2,225 kN), when the pile is on the verge of instability. The numerical solution converged, but the convergence was slow compared to slightly lower thrusts. At this point the pile is uncracked and the results agree with the Secant equation.

With moment applied to one end only, the maximum moment remained at the end of the member until the thrust exceeded 470 kips (2,090 kN). At 500 kips (2,225 kN), the maximum moment was 0.4% larger than the end moment, and was located at the first node from the end.

CONCLUSIONS

There are two important conclusions concerning the length effect analyses. First, the published data on length effects may be very conservative in many common cases. Second, unconditional stability is highly dependent on the state of cracking. In the worst case considered here, with equal end moments of the same sense, instability occurred almost immediately after first cracking.
REFERENCES

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NOTATION

\( a = \) Depth of equivalent rectangular stress block, in. (mm)

\( A_g = \) Gross area of pile cross section, \( \text{in.}^2 \) (\( \text{mm}^2 \))

\( A_{sn} = \) Area of prestressing steel in layer \( n \), \( \text{in.}^2 \) (\( \text{mm}^2 \))

\( A_{st} = \) Total area of prestressing strands, \( \text{in.}^2 \) (\( \text{mm}^2 \))

\( b = \) Width of member section, in. (mm)

\( c = k_d d = \) Depth to neutral axis from compression face at failure, in. (mm)

\( C_c = \) Force in compression zone, kips (kN)

\( d_s = \) Depth to layer of strands from compression face, in. (mm)

\( E_c = \) Elastic modulus of concrete, \( \text{k/in.}^2 \) (GPa)

\( E_s = \) Elastic modulus of steel, \( \text{k/in.}^2 \) (GPa)

\( f'_c = \) Compressive strength of concrete, \( \text{k/in.}^2 \) (MPa)

\( f_{ce} = \) Precompression in concrete, after losses, \( \text{k/in.}^2 \) (MPa)

\( f_{se} = \) Pretension in steel, after losses, \( \text{k/in.}^2 \) (MPa)

\( f_{pu} = \) Stress in steel at flexural failure, \( \text{k/in.}^2 \) (MPa)

\( I_g = \) Moment of inertia of gross section, \( \text{in.}^4 \) (\( \text{mm}^4 \))

\( k_d = \) Depth to neutral axis from compression face, in. (mm)

\( k_d d = c = \) Depth to neutral axis from compression face at failure, in. (mm)

\( h = \) Height of member cross-section, in. (mm)

\( l = \) Length of compression member, in. (m)

\( M = \) Bending moment, k-ft (kN-m)

\( M_{cr} = \) Bending moment at first cracking

\( M_{end} = \) Moment at end of long column

\( M_{max} = \) Maximum bending moment in long column, considering length effects

\( M_n = \) Bending moment at nominal capacity

\( M_u = \) Bending moment capacity reduce by \( \varphi \)

\( P = \) Axial force in member, kips (kN)

\( P_n = \) Axial force at failure of member

\( P_u = \) Axial force capacity reduce by \( \varphi \)

\( T_s = \) Force in layer of prestressed reinforcement, kips (kN)
\( \beta_1 \) = Stress block factor from ACI 318

\( \Delta \) = Deflection of long member, in. (mm)

\( \Delta \varepsilon_{\text{sen}} \) = Change in strain in lowest layer of prestressing strand = \( \varepsilon_n \)

\( \varepsilon_{\text{ce}} \) = Compressive strain in concrete due to stress \( f_{ce} \)

\( \varepsilon_{\text{cu}} \) = Compression strain at failure of concrete

\( \varepsilon_{\text{sun}} \) = Total strain in layer \( n \) of prestressing strand at failure

\( \varepsilon_{\text{se}} \) = Tensile strain in steel due to stress \( f_{se} \)

\( \varepsilon_t \) = Net tensile strain in extreme layer of longitudinal tension steel at nominal strength, excluding strains due to effective prestress, creep, shrinkage, and temperature (ACI 318)

\( \varphi \) = Capacity reduction factor from ACI 318

\( \Phi \) = Curvature, rad/in. (Rad.m)
DISPLACEMENT DEMANDS AND SHEAR KEY FORCES IN PILE-SUPPORTED MARGINAL WHARVES WITH STRONG LATERAL-TORSIONAL COUPLING

Carlos Blandón, José I. Restrepo, and Omar Jaradat

Synopsis: Pile-supported marginal wharves have geometrical characteristics that make them prone to torsional response when subjected to earthquake induced inertial forces. Because of expected early system non-linear response due to the soil-structure interaction, lateral displacement demands on the piles cannot readily be estimated from conventional elastic modal response spectrum analyses and modal combination techniques. These displacement demands may be obtained using non-linear time-history analysis. Nevertheless, modeling the non-linear response of the wharf is still impractical in many design offices. For this reason, simple approximate methods that can estimate the critical pile displacement demand as the spectral displacement corresponding to a predominant translational (transverse) mode natural period of the wharf multiplied by a Displacement Magnification Factor (DMF) is adequate for design purposes. This paper revisits the earlier work of Benzoni and Priestley (2003) and computes, through non-linear time-history analysis, DMFs of short, long and linked segment wharves. Furthermore, the paper also reports shear key forces observed in the non-linear analyses of linked segment wharves. Finally, equations are proposed for calculating the DMFs and to estimate the forces for the design of shear keys.

Keywords: Dynamic Magnification Factor, Non-linear response, Marginal wharves, Performance-based seismic design, Shear keys, Torsion
INTRODUCTION

Ports are essential facilities for the economic development of a region, and even for an entire country. Damage resulting in loss of operation of a port following an earthquake, may result in significant economic hardship and long-term loss of competitiveness. Unexpected performance of port structures to earthquake demands has prompted port authorities to develop their own performance-based seismic design codes. Examples include the Port of Los Angeles (POLA) and the Port of Long Beach (POLB) (POLA 2010; POLB 2012). This is because performance-based seismic design aims at specific structural performance objectives for a given earthquake intensity. Hence, designs with such methodology will result in behavior that is more predictable than those designed with traditional methods, given the demands imposed onto them. Performance-based seismic design is, therefore, seen as a more suited design method for risk management of port infrastructure.

Pile supported marginal wharves have oblong deck supported on several rows of piles installed in a sloping embankment. See Figure 1. Inclined piles may also be used to support the deck and to provide stiffness for lateral load, however in this paper only vertical piles are considered. Decks in long wharves are often segmented and connected through shear keys. The distribution of the individual pile lateral stiffness and strengths in a single segment of linked segment marginal wharves results in mass-stiffness and mass-lateral strength eccentricities that make these wharves prone to torsional response in plan when subjected to earthquake input ground excitation. See Figure 2.

Figure 1 — Prototype marginal wharf section
Due to the soil-structure interaction and the intrinsic non-linear soil properties, it is expected to have an early non-linear structural response of the wharf subjected to earthquake loading. Such behavior indicates that the lateral displacement demands on critical piles (the corner piles in row F shown in Fig. 1 and forces in shear keys cannot readily be estimated from conventional elastic modal response spectrum analyses and modal combination techniques. In particular, the use of these commonly available analysis techniques will result in underestimation of the displacement demands.

**RESEARCH SIGNIFICANCE**

In support of the development of performance-based seismic design methods for marginal wharves, this paper revisits the early work of Benzoni and Priestley (2003) and summarizes the results obtained from a number of non-linear time-history analyses of short, long and linked wharf segments.

These wharves were subjected to simultaneous two-component horizontal input ground motions. Analyzes were carried out for two seismic intensity levels: Operative Level Earthquake (OLE) and Contingency Level Earthquake (CLE), and for upper and lower bound soil models.

The results from the analyses are used to calibrate equations for calculating the lateral displacement demands on the critical piles of a wharf as well as design shear key forces in linked segments wharves.

![Figure 2 — Coupled torsional response of marginal wharf](image)

**REVIEW OF PREVIOUS WORK**

Priestley (2000), and Benzoni and Priestley (2003) derived, through static equilibrium, the lateral displacement in the critical pile of a marginal wharf when subjected to an orthogonal lateral force combination of 100 to 30 percent, as commonly prescribed in seismic codes.

They proposed that the lateral displacement in the critical pile ($\Delta_{WA}$), located at the landside corner, could be calculated as the lateral displacement for 100 percent lateral loading in the transverse direction of the wharf alone, $\Delta_W$, multiplied by a dynamic magnification factor, DMF,

$$\Delta_{WA} = \Delta_W \cdot DMF$$

(1)

Where, for single segment wharves and for the end segments in linked segment wharves, Benzoni and Priestley (2003) proposed equations to estimate the DMF. Among these, equations 2a and 2b govern the common range of $e/L$ ratios in wharves:
DMF = \sqrt{1 + \left[0.3 \left(1 + \frac{20e}{L}\right)\right]^2} \tag{2a}

Whereas for intermediate segments in linked segment wharves:

DMF = 1.1 \tag{2b}

In Eq. 2(a), $e$ is the effective eccentricity between the center of mass and the center of stiffness, where the latter is based on the secant stiffness to maximum response and $L$ is the length of the wharf segment. For multiple wharf segments, $L$ corresponds to the length of the end segments, see Figure 2.

To validate Eq. 1, Priestley (2000) and Benzoni and Priestley (2003) used the superpile concept proposed as “Method D” by Priestley (2000). The results of a number of time-history analyses of single and multi-segment wharves for two earthquake intensities are presented by these authors.

In these works, pushover analyses were carried out for 6.1 m (20 ft) wharf strip, as shown in Fig. 2, to the define the superpile properties. Each of the four superpiles was modeled with two orthogonal non-linear springs connected to the deck as shown in Fig. 3. The wharf masses were added at each superpile node.

The initial stiffness of these springs was made equal to the effective stiffness determined from the bilinear approximation of the response of a composite group of piles. The pushover analyses of the 6.1 m wharf strip, used to obtain the stiffness and strength vs displacement envelope of the superpile springs, included the pile-soil-structure interaction (Ferritto et al. 1999, Priestley 2000).

Figure 3 briefly described the procedure to obtain the parameters for the superpiles based on the pushover analyses of the wharf strip and on the pile distribution of the wharf piles. The location of the superpiles was defined so that the eccentricity, translational and torsional stiffness and torsional mass of the wharf were correctly represented. The superpile concept, the location of these elements in a wharf segment and the procedure to obtain the spring properties is further discussed by Priestley (2000).
Benzoni and Priestley (2003) also extracted the maximum shear force resisted by shear keys during the time-history analyses of multiple-segment marginal wharves. They suggested that the shear key force, $V_{sk}$, is directly proportional to the lateral strength of the wharf in the transverse direction, $V_\Delta$, and to the eccentricity, $e$, and inversely proportional to the wharf segment length, $L$,.

$$V_{sk} = \beta \left( \frac{V_\Delta e}{L} \right)$$  \hspace{1cm} (3)

$V_\Delta$ in this equation corresponds to the yield lateral force which is obtained from a bilinear approximation of the lateral force vs lateral displacement envelope of a wharf segment subjected to lateral loading in the transverse direction (Priestley 2000).  

In Eq. 3 it is important to define the value of $V_\Delta$ for a segment with length $L$. For the analyses carried out in this study $V_\Delta$ was estimated considering only one 6 m (20 ft) long wharf strip, hence, a value of $L=6$ m was used in Eq. 3. If $V_\Delta$ is estimated for an entire wharf segment, the value of $L$ for Eq. 3 should be the total segment length. Coefficient $\beta$ was found to vary between 1.4 and 1.7.

The work of Benzoni and Priestley (2003) had the following limitations: (i) superpiles were modeled with two orthogonal springs, and (ii) input ground motions were spectrally matched and one of the components was reduced to 30 percent. The first limitation causes a bias in the system response because the response of the superpile is not axis-symmetric as intended. A suitable model will be discussed in the following section. The second limitation comes from the decision of scaling one of the components of the input ground motion. The 100/30 percent combination of responses was proposed by Rosenblueth and Contreras (1977) by making a number of assumptions, none requiring scaling of one of the components of the input ground motions, but requiring that the orthogonal components be largely statistically uncorrelated.

**PROCEDURE**

The procedure used herein to estimate the DMFs for a number of single and multi-segment marginal wharves was that described by Priestley (2000):

1. Using the concept of superpiles characterized by a bilinear response with an initial effective stiffness, develop a two dimensional non-linear model of a single or multiple-segment linked-deck wharf.
2. Establish the transverse lateral stiffness of the wharf, and with the mass of the wharf, calculate the predominant fundamental period in the transverse direction.

3. Obtain $\Delta_r$ from the displacement response spectrum for the earthquake intensity in consideration for the predominant fundamental period in the transverse direction of the wharf.

4. Perform a non-linear time-history analysis of the wharf using a pair of orthogonal spectrum matching ground motions. The properties of the records are described in the next section. Repeat the analysis for six other input ground motion pairs.

5. Obtain the average maximum displacement, $\Delta_{\text{max}}$, of the corner piles located at the landside of the wharf for the seven input ground motion pairs.

6. Obtain $\text{DMF} = \Delta_{\text{max}} / \Delta_r$.

The shear key forces and the factor $\beta$ in Eq. 3 were also evaluated according to the following procedure:

1. Carry out pushover analysis of a 6 m (20 ft) long wharf strip in the transverse direction.

2. Define $V_\Delta$ for the maximum displacement in the Contingency Level Earthquake (CLE) displacement response spectrum.

3. Calculate the eccentricity at the lateral displacement found in step 2.

4. Obtain the shear key force from the time-history analysis, $V_{sk}$.

5. Solve Eq. (3) for $\beta$

In the development of the model, the piles were assumed to be embedded in soil and quarry run having uniform properties along the entire wharf. Furthermore, no accidental mass eccentricities were considered and the analyses were carried out with synchronous ground motion pairs. The soil-pile interaction was assumed to be appropriately represented by massless and uncoupled p-y springs that had near axisymmetric behavior, that is, the up and downslope soil p-y response characteristics were assumed identical.

Finally, the eccentricities ratios $e/L$ investigated ranged between 0.06 and 0.12, which limits the applications of this study to typical wharf typologies. Further details about the procedure, the two dimensional model, soil properties, seismic hazard and ground motions are described in the following sections.

**INPUT GROUND MOTIONS**

The estimation of DMF factors was carried out for the Operative Level Earthquake (OLE) and Contingency Level Earthquake (CLE) intensities (POLA 2010, Earth Mechanics 2001), which included forward directivity effects in the evaluation of the seismic hazard. The OLE corresponds to a probability of exceedance of 50% in 50 years, whilst the CLE corresponds to a probability of exceedance of 10% in 50 years. The pseudo-acceleration response spectrum (PSA) for each of these two hazard levels is shown in Figure 5. Seven input ground motion pairs matching the OLE 5% damped response spectrum and seven pairs matching the CLE response spectrum were used to perform the non-linear time-history analyses. The records used were largely statistically uncorrelated.
Table 1 presents the cases studied. The response of single and linked segment wharves was investigated, each under OLE and CLE, and for the upper (UB) and lower bound (LB) soil conditions.

The geometry of the wharves studied was based on typical vessel and constructions at the Port of Los Angeles. Single and multiple wharf segments were 33.5 m (110 ft.) wide by either 121.5 m (400 ft), 182.5 m (600 ft) or 243 m (800 ft) long.

The original model proposed by Benzoni and Priestley (2003) characterized the superpiles with two orthogonal springs (L-array), see Figure 3. Although this approach gives correct results when loading acts in the direction of any of the two springs, it results in a bias when loading is at a different angle as shown in Figure 6. For example, the yield strength of the superpiles represented by the L-array (bold line) overestimates the actual pile strength (dashed line) by a factor of $\sqrt{2}$ when loading acts at an angle of 45 degrees.

Table 1—Case Studies

<table>
<thead>
<tr>
<th>Segment length (m)</th>
<th>Ground Motion</th>
<th>Spring Properties</th>
</tr>
</thead>
<tbody>
<tr>
<td>121.5</td>
<td>OLE,CLE</td>
<td>LB,UB</td>
</tr>
<tr>
<td>182.5</td>
<td>OLE,CLE</td>
<td>LB,UB</td>
</tr>
<tr>
<td>243</td>
<td>OLE,CLE</td>
<td>LB,UB</td>
</tr>
<tr>
<td>121.5 + 121.5</td>
<td>OLE,CLE</td>
<td>LB,UB</td>
</tr>
<tr>
<td>182.5 + 182.5</td>
<td>OLE,CLE</td>
<td>LB,UB</td>
</tr>
<tr>
<td>243 + 243</td>
<td>OLE,CLE</td>
<td>LB,UB</td>
</tr>
<tr>
<td>243 + 121.5</td>
<td>OLE,CLE</td>
<td>LB,UB</td>
</tr>
<tr>
<td>243 + 182.5</td>
<td>OLE,CLE</td>
<td>LB,UB</td>
</tr>
<tr>
<td>182.5 + 243 + 243</td>
<td>OLE,CLE</td>
<td>LB,UB</td>
</tr>
<tr>
<td>243 + 182.5 + 243</td>
<td>OLE,CLE</td>
<td>LB,UB</td>
</tr>
<tr>
<td>243 + 243 + 243</td>
<td>OLE,CLE</td>
<td>LB,UB</td>
</tr>
</tbody>
</table>
This problem was mitigated in this study by modeling the piles with a prime number of equal evenly-spaced angle springs radiating from the pile center, see Figure 7. As Figure 8 illustrates, an increase in the number of springs reduces the strength biases. The use of a prime number of springs requires the use of strength and stiffness correction factors. Such factors are discussed by Blandon (2007).

The deck cross-section of the prototype wharf is shown in Fig. 1. For normal weight concrete, this section results in a mass per unit length \( m = 110 \text{ kN/m/g} \) (2.2 kip/ft/g). The deck of the typical wharf analyzed was supported on six rows of 0.61 m (2 ft) prestressed concrete hexagonal piles. Piles at row A (waterside) were located every 2.4 m (8 ft), rows B,C and D had the piles located every 6 m (20 ft), in row E the piles were located every 3 m (10 ft) and finally, in row F, the piles were placed every 2 m (6.5 ft).

The deck consisted on a 0.61 m (2 ft) thick concrete slab. The deck thickness increases to 1.8 m (6 ft) in row A and to 0.9 m (3 ft) in rows E and F. The center of mass was estimated considering the thickness enlargement at both wharf ends and the pile length variation. The center of mass was located 17 m (56 ft) from row F.
The piles in the prototype structure were assumed precast and pretensioned with 16–15 mm (0.6 in) diameter A416 strands with a total area of 140 mm$^2$ (0.2 in.$^2$). The strands had an ultimate tensile strength of 1,860 MPa (270 ksi) at a tensile strain of 0.06, a yield strength of 1,490 MPa (216 ksi) and a tensile stress after losses of 1,055 MPa (153 ksi). The transverse reinforcement in the piles consisted on 13 mm (0.5 in. W20) spiral at 63.5 mm (2.5 in.) pitch and had a yield strength of 480 MPa (70 ksi). The pile-deck connection consisted of 8-32 mm (8#10) grouted dowels with a yield strength of 475 MPa (68 ksi), an ultimate tensile strength of 680 MPa (97.5 ksi) and an ultimate tensile strain of 0.12. The concrete strength was 48 MPa (7.0 ksi). The concrete cover to the hoops was 76 mm (3 in.).

The properties of the soil p-y springs were obtained from recommendations made to the Port of Los Angeles (Earth Mechanics 1998). These recommendations define the best estimate of p-y springs that could be used to model the static lateral soil structure interaction of the piles and the embankment. Upper and lower bound p-y spring non-linear responses were used in this study. These bounds were obtained by multiplying the best estimate soil strength by a factor of 2.0, for the upper bound, and 0.5, for the lower bound.

The superpiles and the superpile-soil interaction were modeled with three springs in a delta array. The hysteretic response of each spring was represented with the Takeda rule (Otani 1974). The factors that characterize the unloading and reloading stiffness in this rule were made equal to 0.5. These two factors result in a hysteresis loop with intermediate energy dissipation. The selection of these factors was based on the consideration that the total energy dissipation will be the result from the non-linear response of the piles, the pile-deck connection and the soil. Large energy dissipation could be expected from all these mechanisms but as there is not enough experimental information to calibrate a hysteresis model, it is expected that the parameters used for the analyses represent a conservative case. Non-linear analyses were carried out using the software Ruauomo (Carr 2005) with a time step of 0.005 s and a tangent stiffness proportional Rayleigh damping of 5% in the first two modes. The output data was obtained for each time step of the analyses.

Shear keys in linked segment wharves were included in the model with simplified approach using a spring at the location of the shear key. One spring element, with stiffness in the transverse direction and no stiffness in the axial direction, was introduced between two consecutive segments. These elements were considered to evaluate the forces in the shear keys due to the seismic demands imposed in the analyses. Since rotation between consecutive wharves can result in impact, hook-gap elements with 100 mm (4 in) opening were placed at the corners of the individual wharves, see Figure 9. The stiffness of the springs was made equal to 18,000 kN/mm (100,000 kip/in.). Following the earlier work of Priestley (2000, 2007) the value of the shear key spring stiffness was defined from preliminary analyses so for CLE intensity the relative transverse displacement between two adjacent segments was limited to 0.5 mm to 2 mm (0.02 in – 0.08 in).
Several non-linear time-history and modal analyses were carried out as a previous step before carrying out the analyses to obtain the DMF values. Such models were analyzed to verify modeling assumptions and effects of secondary variables. The parameters evaluated were: equivalent pile assumption, landside soil-structure interaction, shear key stiffness. The results from these analyses indicate that the superpile model provided conservative results compared with models that included all the piles, see Figure 10. These parametric studies also showed that it was possible to neglect the effect of the interaction with the landside soil, and that significant large variation of the shear key spring stiffness did not affect the results. Blandon (2007) discusses further the validation of the model.

RESULTS AND DISCUSSION

Averaging of the lateral displacement of the critical piles located at the wharf landside corners was carried out considering the direction of the displacement, as it will be explained below. This approach is based on the fact that the structural performance is related to the strain levels of the materials of the structural elements. It is expected that the maximum strains would be aligned in the same direction of the maximum displacement (Priestley et al. 2007). However, for each record it is not likely that this would occur in the same direction for all the records analyzed.

Two possible averaging approaches are depicted in Figure 11. The first approach, termed the Vectorial Averaging Approach (VAA), defines the maximum average displacement as the average from the maximum displacements obtained from each strong motion regardless of the direction. For the example, in Figure 11a, each arrow represents the displacement of the corner pile for each one of the seven time histories ($\Delta_i$) analyzed. The average in this case is obtained as the simple average from these displacements without considering the direction.
The second approach, termed the Directional Averaging Approach (DAA), shown in Figure 11 b, defines the maximum displacement of the corner pile in all directions for each strong motion. Then a polar plot of the displacement envelope is computed for each input ground motion pair ($\Delta$). Finally, a polar plot is computed with the average of the displacements at a given angle ($\Delta_{\text{average}}$). The maximum displacement ($\Delta_{\text{directional}}$) is now defined from this plot, considering the displacement. The DAA was used in this paper to obtain the DMFs.

\[ \Delta_{\text{directional}} \]

The eccentricity variation was estimated for lower and upper bound p-y spring properties as shown in Fig. 12. DMFs were controlled by lower bound soil conditions in all cases. The eccentricities at OLE and CLE for this soil condition were $e = 14 \text{ m} (46 \text{ ft})$ and $e = 13.1 \text{ m} (43 \text{ ft})$, respectively. On the other hand, the maximum shear key forces were found for CLE for the upper bound soil condition. The eccentricity for this case was $e = 14.1 \text{ m} (46 \text{ ft})$.

![Figure 11 — Critical Pile Lateral Displacement Averaging Approach. a) VAA b) DAA.](image)

The eccentricity variation for lower and upper bound p-y spring properties as shown in Fig. 12. DMFs were controlled by lower bound soil conditions in all cases. The eccentricities at OLE and CLE for this soil condition were $e = 14 \text{ m} (46 \text{ ft})$ and $e = 13.1 \text{ m} (43 \text{ ft})$, respectively. On the other hand, the maximum shear key forces were found for CLE for the upper bound soil condition. The eccentricity for this case was $e = 14.1 \text{ m} (46 \text{ ft})$.

![Figure 12 — Eccentricity Variation for LB and UB Soil Strength](image)

Single segment results, listed in Table 2, show that DMFs decrease as ratio $e/L$ decreases. For lower bound soil properties and for the OLE, the DMF = 1.25 computed for the wharf with the smallest $e/L$ ratio ($e/L = 0.058$ for OLE) is 87 percent of the DMF = 1.44 computed for the wharf with the largest $e/L$ ratio ($e/L = 0.115$ for OLE). For CLE and for the same soil model, a DMF = 1.32 was computed for the wharf with the smallest $e/L$ ratio, whereas a DMF = 1.58 was computed for the wharf with the largest $e/L$ ratio, that is, the DMF of the wharf with the smallest $e/L$ ratio is 84 percent of that observed for the wharf with the largest $e/L$ ratio. Note that for OLE and CLE, for lower bound soil properties and for a given $e/L$ ratio, the DMFs are not very dissimilar as they are within 6 percent of their average in all three cases.

DMFs computed for two linked segment wharves are listed in Table 3. In wharves with unequal length segments, the critical piles in the shortest segment attained the maximum DMFs. DMFs in these wharves are lightly correlated to the sum of the $e/L$ ratios of the individual segments. For the most part, the trends observed for these wharves are in line, to some extent, with those observed for single deck wharves.
described above. That is, DMFs tend to become smaller as \( \Sigma e/L \) decrease. An exception is the case study of the segment with the smallest \( e/L \) (\( e/L = 0.058 \) for OLE) linked to the segment with the largest \( e/L \) ratio (\( e/L = 0.115 \) for OLE). This case study gives a DMF = 1.02 at OLE for the lower bound soil condition and looks like an outlier.

Table 2 – DMFs for Single-Segment Wharves

<table>
<thead>
<tr>
<th>Length (m)</th>
<th>( e/L ) OLE</th>
<th>( e/L ) CLE</th>
<th>OLE LB</th>
<th>CLE LB</th>
</tr>
</thead>
<tbody>
<tr>
<td>121.5</td>
<td>0.115</td>
<td>0.107</td>
<td>1.44</td>
<td>1.58</td>
</tr>
<tr>
<td>182.5</td>
<td>0.077</td>
<td>0.072</td>
<td>1.34</td>
<td>1.44</td>
</tr>
<tr>
<td>243</td>
<td>0.058</td>
<td>0.054</td>
<td>1.25</td>
<td>1.32</td>
</tr>
</tbody>
</table>

Table 3 – DMFs for Two-Segment Wharves

<table>
<thead>
<tr>
<th>Length (m)</th>
<th>( \Sigma e/L ) OLE</th>
<th>( \Sigma e/L ) CLE</th>
<th>OLE LB</th>
<th>CLE LB</th>
</tr>
</thead>
<tbody>
<tr>
<td>121.5+121.5</td>
<td>0.230</td>
<td>0.215</td>
<td>1.23</td>
<td>1.33</td>
</tr>
<tr>
<td>243+121.5</td>
<td>0.173</td>
<td>0.161</td>
<td>1.02</td>
<td>1.24</td>
</tr>
<tr>
<td>182.5+182.5</td>
<td>0.153</td>
<td>0.143</td>
<td>1.10</td>
<td>1.21</td>
</tr>
<tr>
<td>243+182.5</td>
<td>0.134</td>
<td>0.125</td>
<td>1.01</td>
<td>1.19</td>
</tr>
<tr>
<td>243+243</td>
<td>0.115</td>
<td>0.107</td>
<td>1.01</td>
<td>1.18</td>
</tr>
</tbody>
</table>

The end segments in three linked segment wharves also behave in similar manner to segments in for two linked segment wharves. The largest DMFs also occur in the segment with the largest \( \Sigma e/L \) ratio and, generally, DMFs decrease when the sum of the end segments \( e/L \) ratios decrease, see Table 4. The analysis shows that DMFs in these segments are practically independent from the \( e/L \) ratio as Benzoni and Priestley (2003) had noted, see Table 5. This table shows that DMFs in these segments are slightly dependent on the earthquake intensity.

Table 4 – DMFs for End Segments in Three-Segment Wharves

<table>
<thead>
<tr>
<th>Length (m)</th>
<th>( \Sigma e/L ) OLE</th>
<th>( \Sigma e/L ) CLE</th>
<th>OLE LB</th>
<th>CLE LB</th>
</tr>
</thead>
<tbody>
<tr>
<td>182.5+243+243</td>
<td>0.203</td>
<td>0.179</td>
<td>1.14</td>
<td>1.24</td>
</tr>
<tr>
<td>243+182.5+243</td>
<td>0.203</td>
<td>0.179</td>
<td>1.06</td>
<td>1.20</td>
</tr>
<tr>
<td>243+243+243</td>
<td>0.183</td>
<td>0.161</td>
<td>1.06</td>
<td>1.20</td>
</tr>
</tbody>
</table>

Table 5 – DMFs for Intermediate Segments in Three-Segment Wharves

<table>
<thead>
<tr>
<th>Length (m)</th>
<th>OLE LB</th>
<th>CLE LB</th>
</tr>
</thead>
<tbody>
<tr>
<td>182.5+243+243</td>
<td>0.95</td>
<td>1.14</td>
</tr>
<tr>
<td>243+182.5+243</td>
<td>0.95</td>
<td>1.14</td>
</tr>
<tr>
<td>243+243+243</td>
<td>0.94</td>
<td>1.13</td>
</tr>
</tbody>
</table>

As described previously, the largest shear key force values were obtained for the cases with upper bound soil conditions for the CLE. Shear key forces for two linked segments, \( V_{sk} \), shown in Table 6 present a small increase as the segment length increases. However, for design purposes such variation may be considered negligible. For the case of three linked segments, shown in Table 7, the forces on the shear keys are smaller than for the two linked segment wharves.
Table 6 – Linked Segments Shear Key Results (2 Segments)

<table>
<thead>
<tr>
<th>Length (m)</th>
<th>e/ΣL</th>
<th>W (kN)</th>
<th>V_d/W</th>
<th>V_δd (kN)</th>
<th>V_d/V_δd</th>
<th>β</th>
</tr>
</thead>
<tbody>
<tr>
<td>121.5+ 121.5</td>
<td>0.06</td>
<td>261954</td>
<td>0.47</td>
<td>9279</td>
<td>0.08</td>
<td>1.29</td>
</tr>
<tr>
<td>182.5+ 182.5</td>
<td>0.04</td>
<td>393470</td>
<td>0.47</td>
<td>9492</td>
<td>0.05</td>
<td>1.32</td>
</tr>
<tr>
<td>243 + 243</td>
<td>0.03</td>
<td>523908</td>
<td>0.47</td>
<td>9727</td>
<td>0.04</td>
<td>1.36</td>
</tr>
<tr>
<td>243 + 121.5</td>
<td>0.04</td>
<td>392931</td>
<td>0.47</td>
<td>9313</td>
<td>0.05</td>
<td>1.30</td>
</tr>
<tr>
<td>243 + 182.5</td>
<td>0.03</td>
<td>458689</td>
<td>0.47</td>
<td>9488</td>
<td>0.04</td>
<td>1.32</td>
</tr>
</tbody>
</table>

Table 7 – Linked Segments Shear Key Force (3 Segments)

<table>
<thead>
<tr>
<th>Length (m)</th>
<th>e/ΣL</th>
<th>W (kN)</th>
<th>V_d/W</th>
<th>V_δd (kN)</th>
<th>V_d/V_δd</th>
<th>β</th>
</tr>
</thead>
<tbody>
<tr>
<td>182.5 + 243 + 243</td>
<td>0.02</td>
<td>720643</td>
<td>0.47</td>
<td>8414</td>
<td>0.02</td>
<td>1.17</td>
</tr>
<tr>
<td>182.5 + 243 + 243</td>
<td>0.02</td>
<td>720643</td>
<td>0.47</td>
<td>7988</td>
<td>0.02</td>
<td>1.11</td>
</tr>
<tr>
<td>243 + 182.5 + 243</td>
<td>0.02</td>
<td>720643</td>
<td>0.47</td>
<td>7830</td>
<td>0.02</td>
<td>1.09</td>
</tr>
<tr>
<td>243 + 182.5 + 243</td>
<td>0.02</td>
<td>720643</td>
<td>0.47</td>
<td>7777</td>
<td>0.02</td>
<td>1.08</td>
</tr>
<tr>
<td>243 + 243 + 243</td>
<td>0.02</td>
<td>785862</td>
<td>0.47</td>
<td>8082</td>
<td>0.02</td>
<td>1.13</td>
</tr>
<tr>
<td>243 + 243 + 243</td>
<td>0.02</td>
<td>785862</td>
<td>0.47</td>
<td>7996</td>
<td>0.02</td>
<td>1.11</td>
</tr>
</tbody>
</table>

RECOMMENDATIONS

Figure 13 plots the DMFs compiled for single segment wharves for OLE and CLE for the governing lower bound soil condition. The Figure also plots the prediction given by Eq. 2 (Benzoni & Priestley, 2003). Recall that Eq. 2 was derived from fundamental principles of static equilibrium using as basis a 100/30 percent static lateral force combination. Thus, this equation does not account for dynamic effects, including the rotational inertia. For single segments for OLE and CLE, DMFs can be predicted by the following empirically fitted straight line equation also shown Figure 13,

\[
DMF = 1.1 + 4.1e/L
\]  

(4)

where all the DMFs predicted by Eq. 4 are within 7% of those computed in the analyses. Given this small variation and also considering the several sources of data uncertainty and in the methodology it was considered that Eq. 4 is adequate to estimate the DMF values for both OLE and CLE intensities.

Figure 13 – DMFs Computed for Single Segment Wharves and Predictive Equations
For two and for the end segments in three linked segment wharves, Eq. 2 overpredicts the DMFs for OLE and CLE, particularly in shorter decks when the largest of the $e/L$ ratios of the two segments or of the two-end segments exceeds 0.08 as shown in Figure 14.

![Figure 14 – DMFs Computed for Two-Segment and End Segments of Three-Segment Wharves and Comparison with Benzoni and Priestley (2003)](image1)

In this paper, the authors propose a two-tier approach, shown in Figure 15, where the DMF is computed by either Eq. 5(a) or Eq. 5(b) given by,

$$DMF = 1.25$$  \hspace{1cm} (5a)

But the DMF does not need to be greater than,

$$DMF = 1.0 + 1.2 \Sigma \frac{e}{L}$$  \hspace{1cm} (5b)

![Figure 15 – DMFs Computed for Two-Segment and End Segments of Three-Segment Wharves and Comparison with Proposed Equations.](image2)
Equation 5(a) is very simple since it makes the DMF independent from the sum of the eccentricity ratios in two segment wharves, $\Sigma e/L$. In three segment wharves $\Sigma e/L$ is the sum of the eccentricity ratios of the end segments. The more refined Eq. 5(b) predicts all DMFs for OLE within 13 percent and those for CLE within 6 percent, except the case of the two segment wharf with $\Sigma e/L = 0.173$ at OLE, which is overestimated by 17 percent.

The analyses show that the DMFs in intermediate segments of in three linked segment wharves are practically uncorrelated with ratio $e/L$ and that these factors have a slight correlation with the earthquake intensity. These observations are in line with those made earlier by Benzoni and Priestley (2003) who recommended Eq. 2b for intermediate segments. Eq. 2b overpredicts the OLE DMFs by 16% and underpredicts the CLE DMFs by only 3.6%. For these intermediate segments the DMF could be adjusted to

$$DMF = 1.05$$

which predicts all DMFs within 11 percent.

Coefficients $\beta$ in two-segment wharves were found to be slightly smaller, around 4 percent, than the smallest value proposed by Priestley and Benzoni (2003) for Eq. 3. However, for three-segment wharves, coefficients $\beta$ were found to be around 20 percent smaller, see Figure 16. This paper recommends that coefficient $\beta$ be taken as:

$$\beta = 1.3$$

for two-segment wharves and

$$\beta = 1.1$$

for three-segment wharves.

![Figure 16 — $\beta$ Variation for Two-Segment and Three-Segment Linked-Deck Wharves](image)

CONCLUSIONS

Non-linear time-history analyses were carried out in different single and linked wharf configurations to verify existing equations used for the computation of Dynamic Magnification Factors (DMF) to estimate
design displacements in critical piles in marginal wharves for a range of typical wharf geometries.

Based on the results obtained, empirically fitted equations are proposed to estimate the DMF for the critical corner piles of single and multi-segment deck-linked wharves. For single and multi-segment linked-deck segments the equations proposed are a function of \( e/L \) and in any case the DMF is defined to be smaller than 1.25.

Most of the values obtained from the time-history analyses were well predicted (within 7%) by the proposed equations with a few exceptions. For intermediate segments the DMF was defined as 1.05, which also resulted in a good estimation of the numerical values (within 11%). For practical design purposes and considering the minor variation of DMF for different seismic intensities, the same equations are proposed for the estimation of DMF for OLE and CLE.

The existing equation used for the estimation of shear key forces proposed in the literature was also verified in this study. The forces estimated using a \( \beta \) coefficient of 1.3 for two-segment wharves were found to have good match with the numerical results. For three-segment wharves the \( \beta \) coefficient was estimated equal to 1.1.

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REFERENCES


LIST OF SYMBOLS AND ABBREVIATIONS

OLE: Operative Level Earthquake.
CLE: Contingency Level Earthquake.
UB: upper bound soil properties.
LB: lower bound soil properties.
DMF: Displacement Magnification Factor.
POLA: Port of Los Angeles.
POLB: Port of Long Beach.

$\Delta_{ea}$: horizontal displacement in the critical pile including amplification due to torsional response.

$\Delta_{h}$: horizontal displacement in the critical pile due to earthquake loading in the transverse direction of the wharf.

$e$: wharf eccentricity.

$L$: wharf segment length.

$V_{sk}$: shear key force.

$V_{\Delta}$: lateral strength of the wharf in the transverse direction.

$\beta$: shear key force coefficient.

$\Delta_{max}$: critical pile average maximum displacement from time history analyses.

VAA: Vectorial averaging approach.

DAA: Directional averaging approach.

$\Delta_{e}$: critical pile displacement envelope from time history analyses for one ground motion record.

$\Delta_{average}$: critical pile displacement envelope from time history analyses for all the set of ground motions.

$\Delta_{directional}$: maximum critical pile displacement from $\Delta_{average}$.

$\Sigma L_i$: Total length of the linked segment wharf.