

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

Jhon P. Smith-Pardo and Carlos E. Ospina

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.

ACI Member **J. Paul Smith-Pardo**, PhD, PE, is an Assistant Professor of Civil and Environmental Engineering at Seattle University. He is a voting member of the ASCE/COPRI committee on seismic design of piers and wharves, and an associate member of ACI Committee 543, Concrete Piles. He received his MSc and PhD in structural engineering from Purdue University in 2000 and 2004, respectively.

ACI Fellow **Carlos E. Ospina**, PhD, PE, is Senior Project Manager with BergerABAM Inc., Houston, TX, where he works on the planning and design of marine infrastructure. He is a member of ACI/ASCE 445, Shear and Torsion; ACI 543, Concrete Piles; Co-Chair of ACI 445C, Punching Shear; and associate member of the ASCE/COPRI committee on seismic design of pile-supported piers and wharves. He received his MSc and PhD in structural engineering from the University of Alberta, Canada.

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 pre-determined 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 (Knaption 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-foot box, and 2.4 m (8 ft) by 6.1 m (20 ft) for a 20-foot 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 and what load factor is to be used to combine live loads with dead and seismic loads 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, γ_{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 considered 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

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

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

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

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):

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

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

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, $K = 0.5PGA/g$,

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:

$$(1.2 + K)D + 1.0L + 1.2B + 1.2C + 1.6H + 1.0E \quad [5]$$

$$(0.9 - K)D + 0.9B + 0.9C + 1.6H + 1.0E \quad [6]$$

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:

$$1.2D + 1.0L + 1.2B + 1.2C + 1.0E \quad [7]$$

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

$$(1.0 \pm K)D \quad [8]$$

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 $0.2 (2.5 \text{ PGA}) = 0.5 \text{ PGA}$, 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_L , width $2B$, and height $2H$, and it is subjected to a normal force, N , and friction force, f_x , at its base. The static and dynamic friction coefficients at the interface of the block and the foundation are defined as μ_s and μ_d . Typical values for μ_s and μ_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. $N = W_L$); ii) the static friction is not overcome (i.e. $f_x \leq \mu_s N$); 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, $N = W_L$ is always satisfied. The second condition is met if

$$\frac{\ddot{u}_{g,max}}{g} < \mu_s, \text{ where } \ddot{u}_{g,max} \text{ is the maximum horizontal ground acceleration. The third condition is satisfied if } \frac{\ddot{u}_{g,max}}{g} < \frac{B}{H}.$$

The sliding mode is initiated if $\frac{\ddot{u}_{g,max}}{g} > \mu_s$ and $\mu_s < \frac{B}{H}$, whereas the rocking mode is initiated if $\frac{\ddot{u}_{g,max}}{g} > \frac{B}{H}$ and $\mu_s > \frac{B}{H}$, as illustrated conceptually in the μ_s vs. $\frac{\ddot{u}_{g,max}}{g}$ 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 $\mu_s = \frac{\ddot{u}_{g,max}}{g}$, a horizontal line defined by $\mu_s = \frac{B}{H}$, and a vertical line defined by $\frac{\ddot{u}_{g,max}}{g} = \frac{B}{H}$. 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 μ_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:

$$\ddot{u}_g = A_g \sin(\omega t) = A_g \sin\left(\frac{2\pi t}{T}\right) \quad [11]$$

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 $m_L/m = 2.67$ where m_L 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 μ_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:

$$T_{SDOF}^* = T_{SDOF} \sqrt{1 + \frac{m_L}{m}} \quad [12]$$

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 $A_o/g = B/H = 1/6$. 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:

$$\frac{\Delta_L}{\Delta} = \frac{T_L}{T_{SDOF}} \quad [13]$$

where Δ and Δ_L 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:

$$T_L = T_{SDOF} \sqrt{1 + \frac{\eta m_L}{m}} \quad [14]$$

where η 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:

$$\frac{\Delta_L}{\Delta} = \sqrt{1 + \frac{\eta m_L}{m}} \quad [15]$$

Solving for η results in:

$$\eta = \left[\left(\frac{\Delta_L}{\Delta} \right)^2 - 1 \right] \frac{m}{m_L} \leq 1.0 \quad [16]$$

In Eq. (16), Δ_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, Δ 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 Δ_L term, therefore if the live load is reduced further and further the ratio Δ_L/Δ would tend to unity and the net result is that η 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 Duzçe 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 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.

REFERENCES

- ACI Committee 318, 2008, "Building Code Requirements for Structural Concrete (ACI 318-08) and Commentary (ACI 318R-08)," American Concrete Institute, Farmington Hills, MI.
- American Association of State Highway Transportation Officials, 2007, "AASHTO LRFD Bridge Design Specifications and Commentary," 4th Edition, Washington DC.
- ASCE/SEI 7-10, 2010, "Minimum Design Loads for Buildings and Other Structures," American Society of Civil Engineers, Reston, VA.
- California Building Standards Commission, 2010, "California Building Code: California Code of Regulations, Title 24, Part 2, Volume 1 of 2," Chapter 31F, Marine Oil Terminal Engineering and Maintenance Standards (MOTEMS), Sacramento, CA, June.
- Department of Defense (US Navy), 2005, "Design: Piers and Wharves (UFC-4-152-01)," July.
- Esfandiari, S., Kennedy, R., Manrique, M. and Grubb, R., 2001, "Methodology for Prediction of Sliding and Rocking of Rigid Bodies Using Fast Non-Linear Analysis (FNA) Formulation," *Transactions, SMiRT 16*, Paper # 1638, Washington D.C., August, pp. 1-8.
- Ferritto, J., Dickenson, S., Priestley, M.J.N., Werner, S., Taylor, C., Burke, D., Seelig, W. and Kelly, S., 1999, "Seismic Criteria for California Marine Oil Terminals," Vols. 1 and Vol. 2, Technical Report TR-2103-SHR, Naval Facilities Engineering Service Center, Port Hueneme, CA, July.
- Housner, G.W., 1963, "The Behavior of Inverted Pendulum Structures During Earthquakes," *Bulletin of the Seismological Society of America*, Vol. 53, No. 2, pp. 403-417, Feb.
- International Code Council, 2012, "International Building Code", International Code Council, Falls Church, VA.
- Knapton, J., and Meletiou, M. (1996). "The Structural Design of Heavy Duty Pavements for Ports and other Industries." Interpave/British Ports Association, British Precast Concrete Federation. Leicester, U.K.
- Nasi, K., 2010, "The Seismic Response of Rocking Concrete Blocks," *MSc Thesis*, Purdue University, West Lafayette, IN.
- International Navigation Association (PIANC) Working Group No. 34, 2001, "Seismic Design Guidelines for Port Structures," A.A. Balkema Publishers, Lisse, Netherlands, 474 pp.
- City of Los Angeles Harbor Department, 2010, "The Port of Los Angeles Code for Seismic Design, Upgrade and Repair of Container Wharves (POLA Seismic Code 2010)," May, Los Angeles, 22 pp.
- Port of Long Beach Authority, 2009, "Port of Long Beach Wharf Design Criteria, Version 2.0", Jan.
- Shafieezadeh, A., Kosbab, B.D., DesRoches, R., and León, R., 2012, "Dynamic Interaction Behavior of Pile-Supported Wharves and Container Cranes in Liquefiable Soil Embankments," ASCE Structures Congress Proceedings, Chicago.
- Shenton, H.W., 1996, "Criteria for Initiation of Slide, Rock, and Slide-Rock Rigid-Body Modes," ASCE Journal of Engineering Mechanics, July, pp. 690-693.

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
EI_{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
$\ddot{U}_g$	Total acceleration in the horizontal direction
α	Factor indicating location of normal force resultant in rigid body at rest
Δ	Lateral displacement of pile-supported deck or platform
Δ_c	Lateral displacement of pile-supported deck or platform with stacked containers
η	Fraction of live load contributing to additional lateral displacement of pile-supported deck or platform
μ_d	Dynamic friction coefficient
μ_s	Static friction coefficient
ω	frequency of ground motion or vibration

Table 1 -- Typical Container Loads

Stacking Height (No. of boxes)	Reduction in Gross Weight (%)	Container Corner Load* (kN)	Equivalent UDLL (40 ft Box) (kPa)	Equivalent UDLL** (20 ft Box) (kPa)
1	0	76.2	10.3	16.5
2	10	137.2	18.5	29.7
3	20	182.9	24.6	39.6
4	30	213.4	28.7	46.2
5	40	228.6	30.8	49.5
6	40	274.3	36.9	59.4
7	40	320.0	43.1	69.3
8	40	365.8	49.2	79.2

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

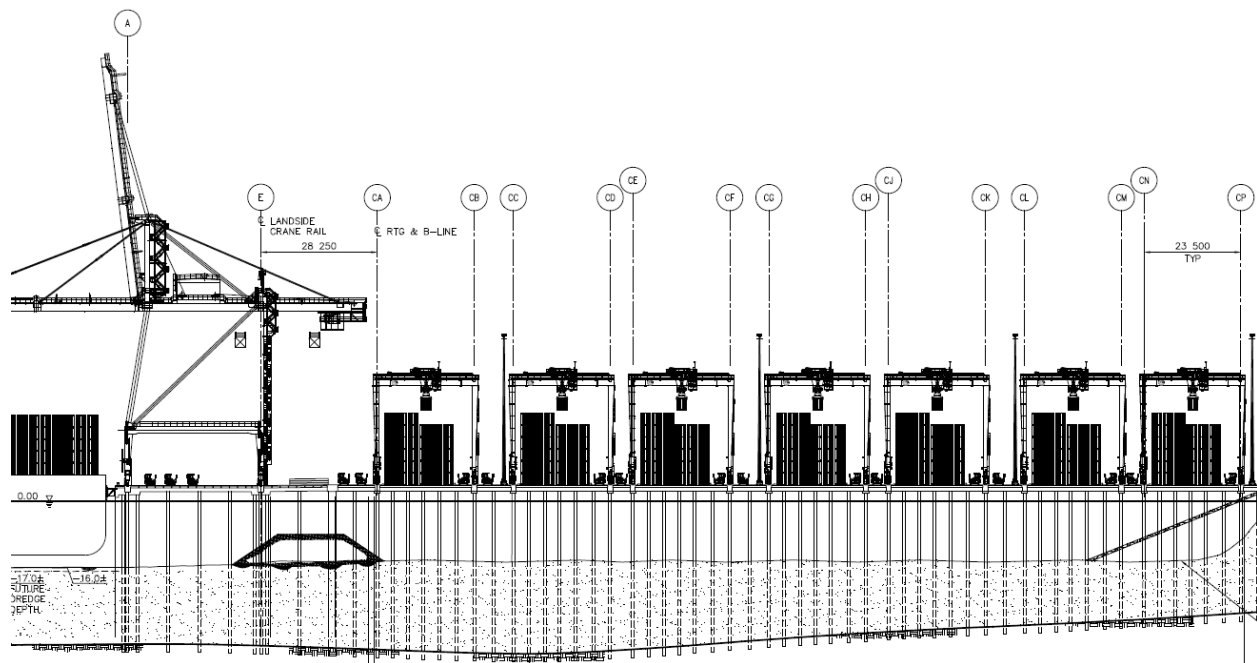


Figure 1 -- Pile-supported Container Wharf and Container Yard

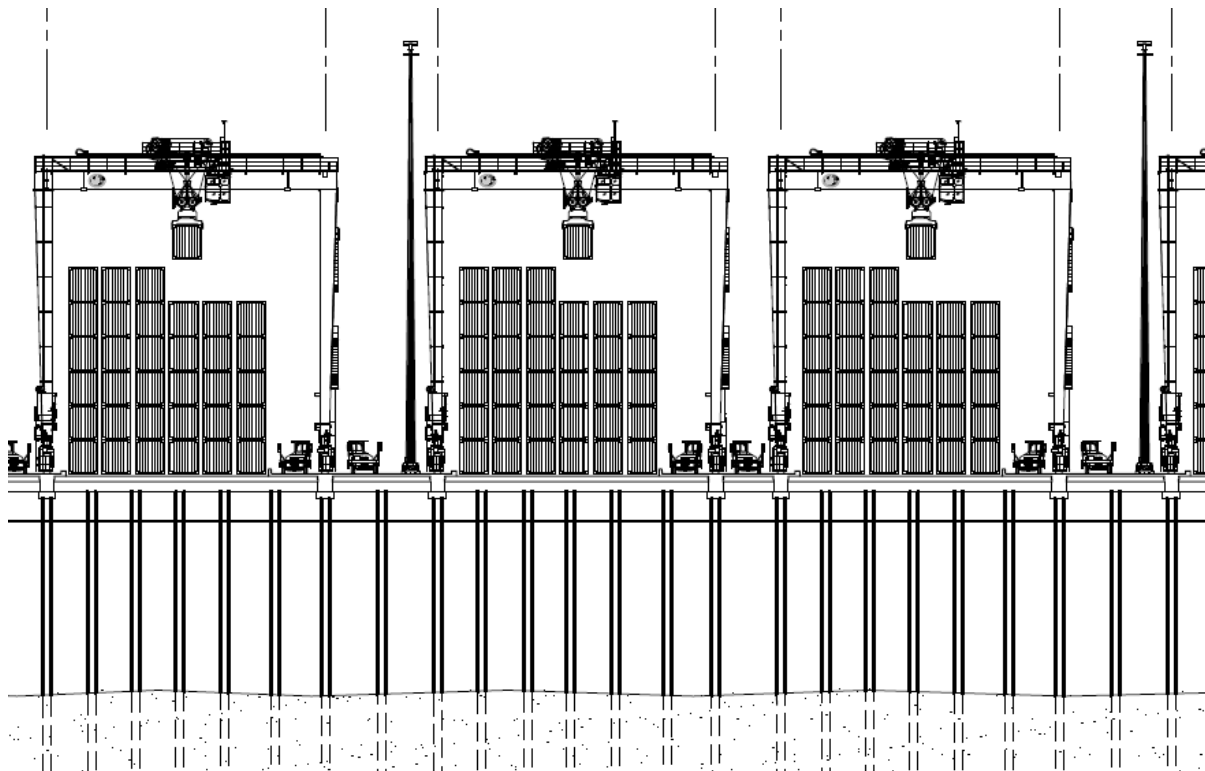


Figure 2 -- Zoomed-in View of Pile-supported Container Yard



Figure 3 -- Marine Oil Terminal (Bunker Fuel Transfer Pier, Melones Island, Panama)

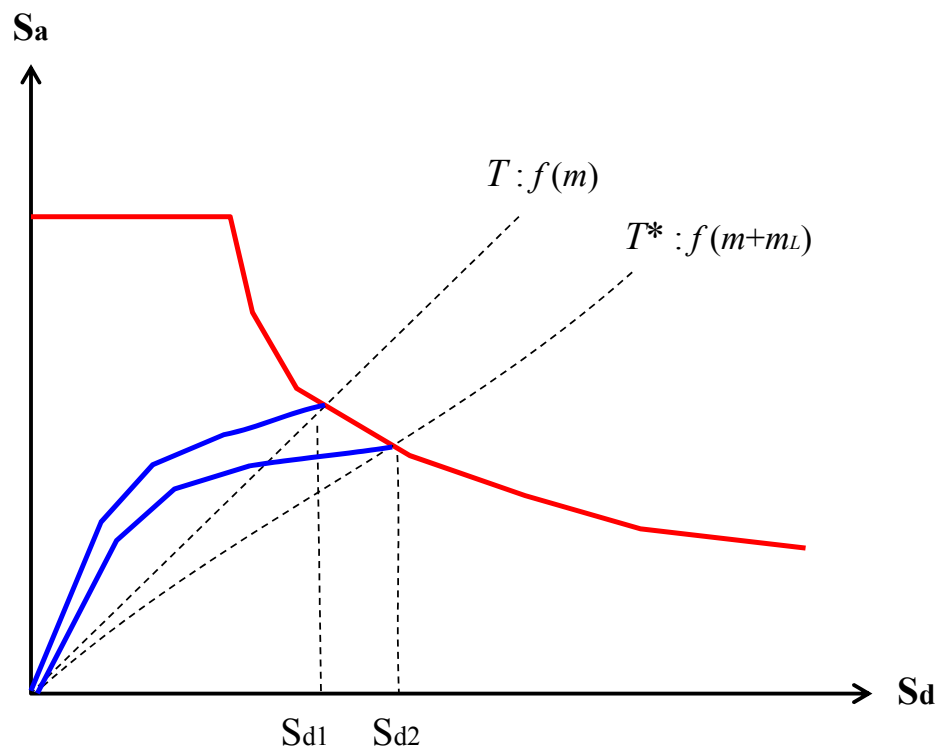


Figure 4 -- Effect of Live Load as a Source of Mass on Lateral Response of Pile-supported Deck

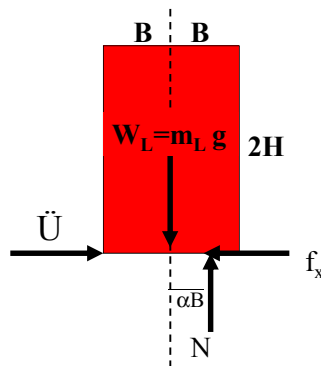


Figure 5 -- Free Body Diagram of Rigid Body on the Ground (At Rest Conditions)

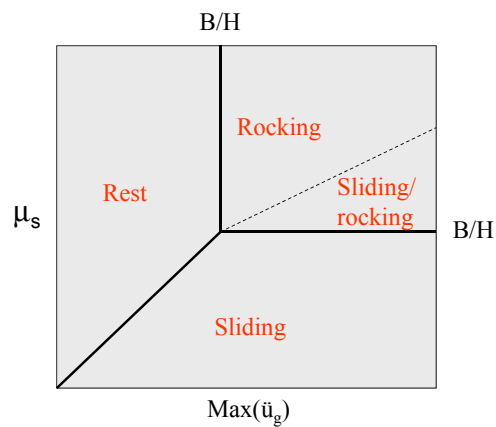


Figure 6 -- Conceptual Description of At-Rest, Sliding, Sliding/Rocking and Rocking of Rigid Body (Shenton, 1996)

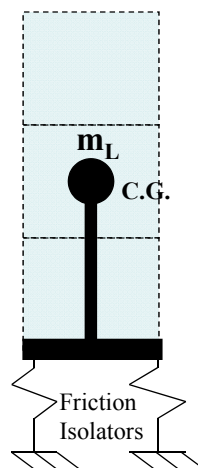


Figure 7 -- Idealization of Single Container Stack as a Rigid Body on the Ground for SAP Analyses

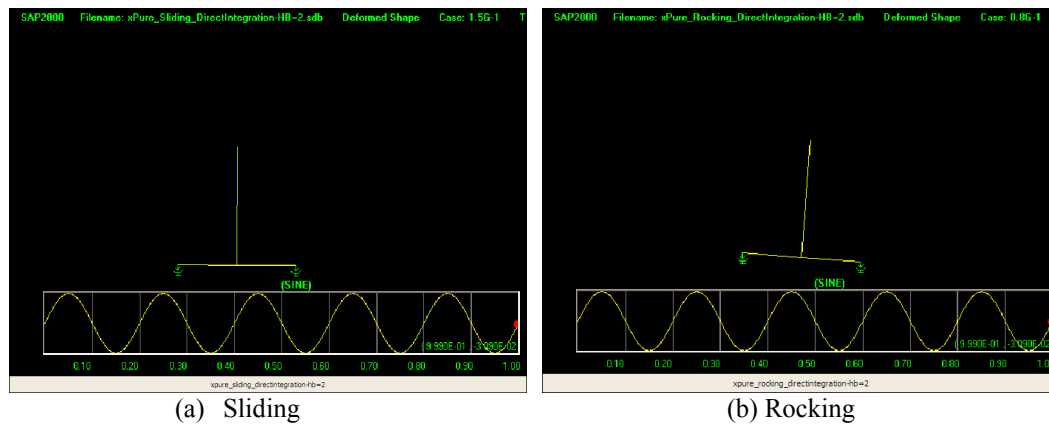


Figure 8 -- Sliding and Rocking Modes of Rigid Body Supported on the Ground, Subjected to Sinusoidal-type Horizontal Ground Acceleration

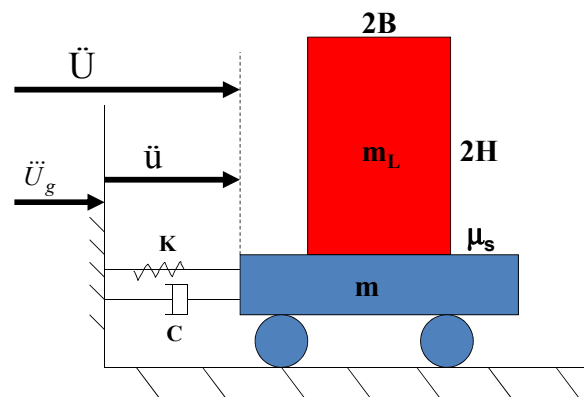


Figure 9 -- SDOF Representation of Pile-supported Deck with Stacked Containers Subjected to Horizontal Ground Acceleration

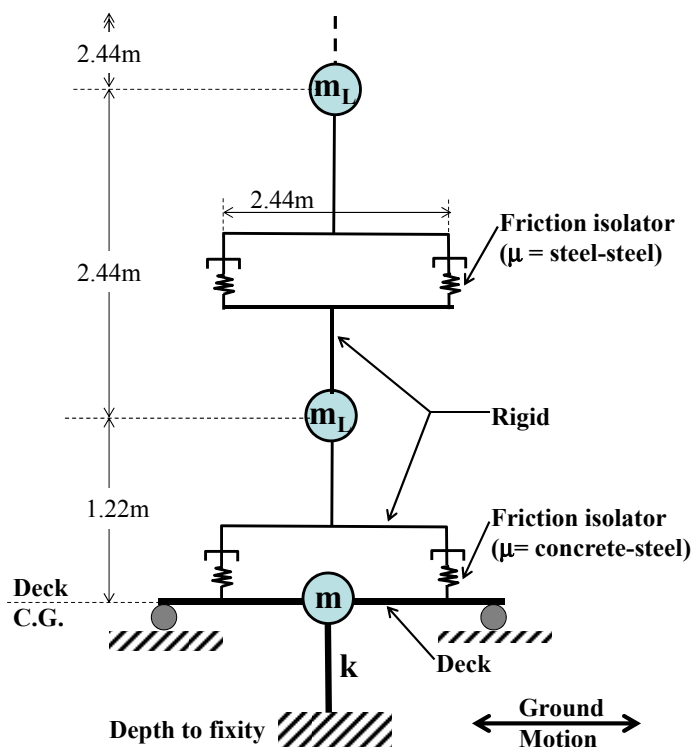


Figure 10 -- Idealization of Pile-supported Deck Structure with Stacked Containers

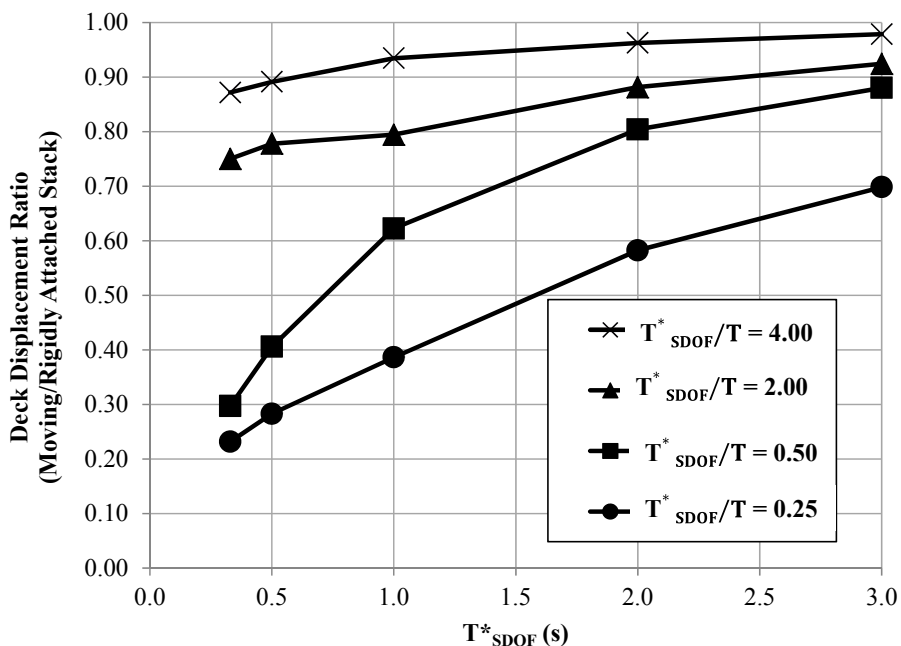


Figure 11 -- Effect of Lateral Stiffness of Structure on Deck Displacement Demand (Sinusoidal Horizontal Ground Motion)

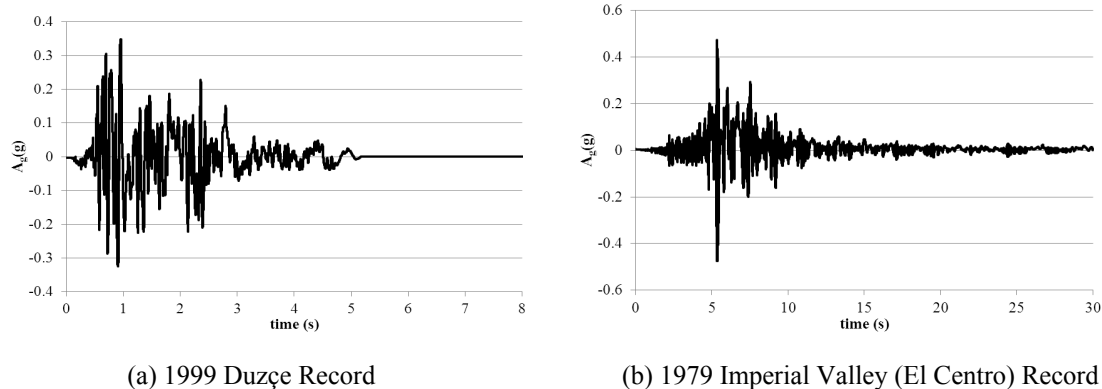


Figure 12 -- 1999 Duzçe and 1979 Imperial Valley (El Centro) Records

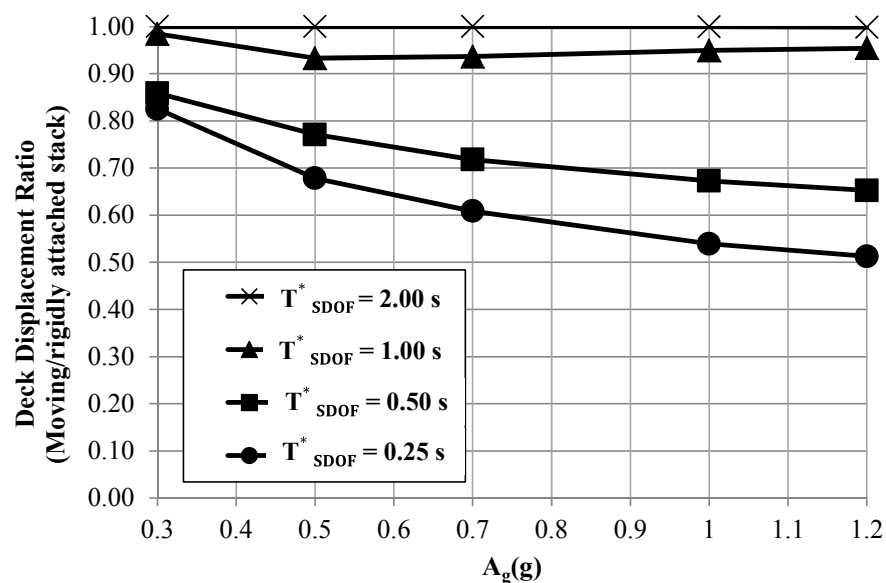


Figure 13 -- Lateral Deck Displacement Ratio under Scaled 1999 Duzçe Record

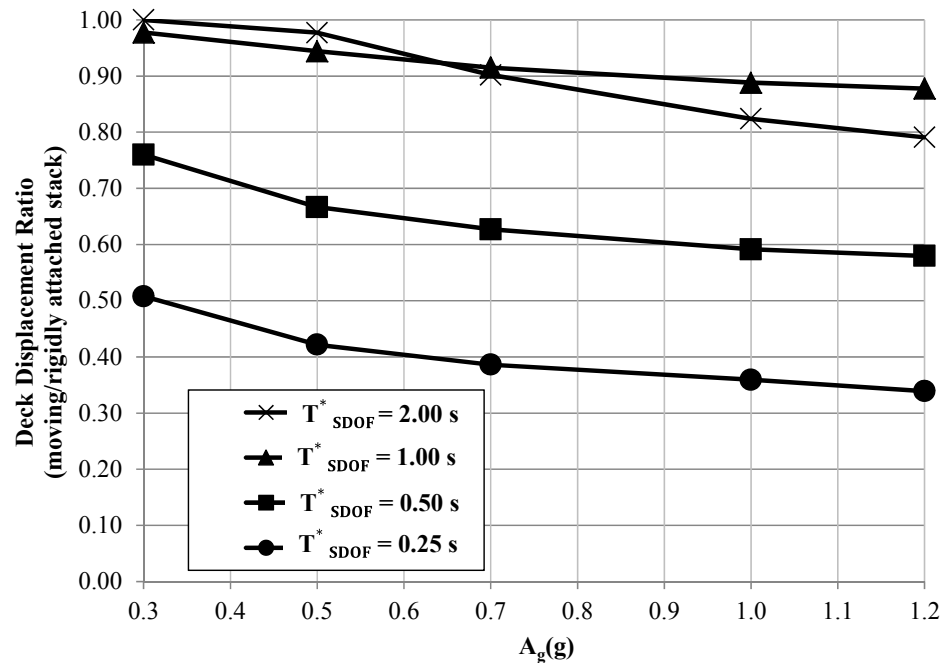


Figure 14 -- Lateral Deck Displacement Ratio under Scaled 1979 Imperial Valley (El Centro) Record

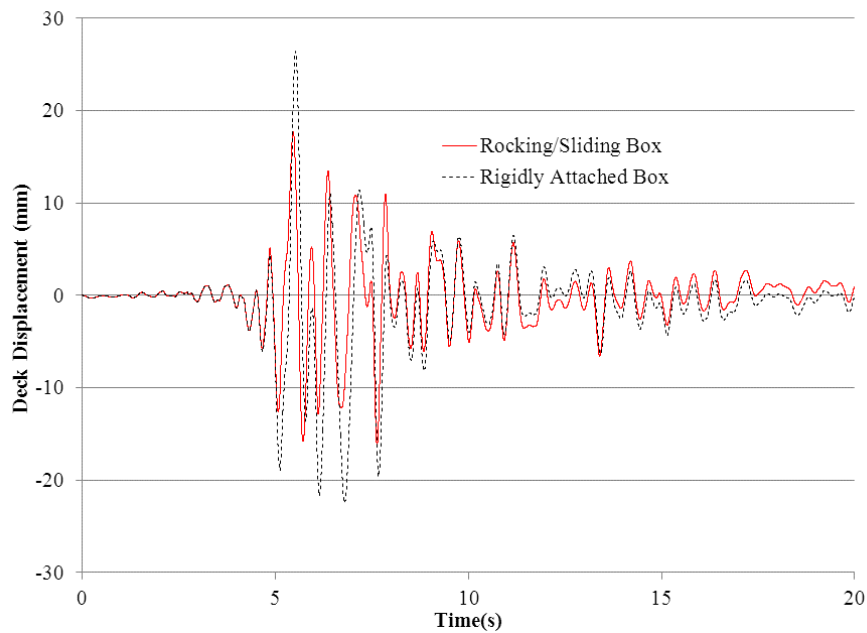


Figure 15 -- Lateral Deck Displacement Response to El Centro Record Scaled to PGA = 0.5g

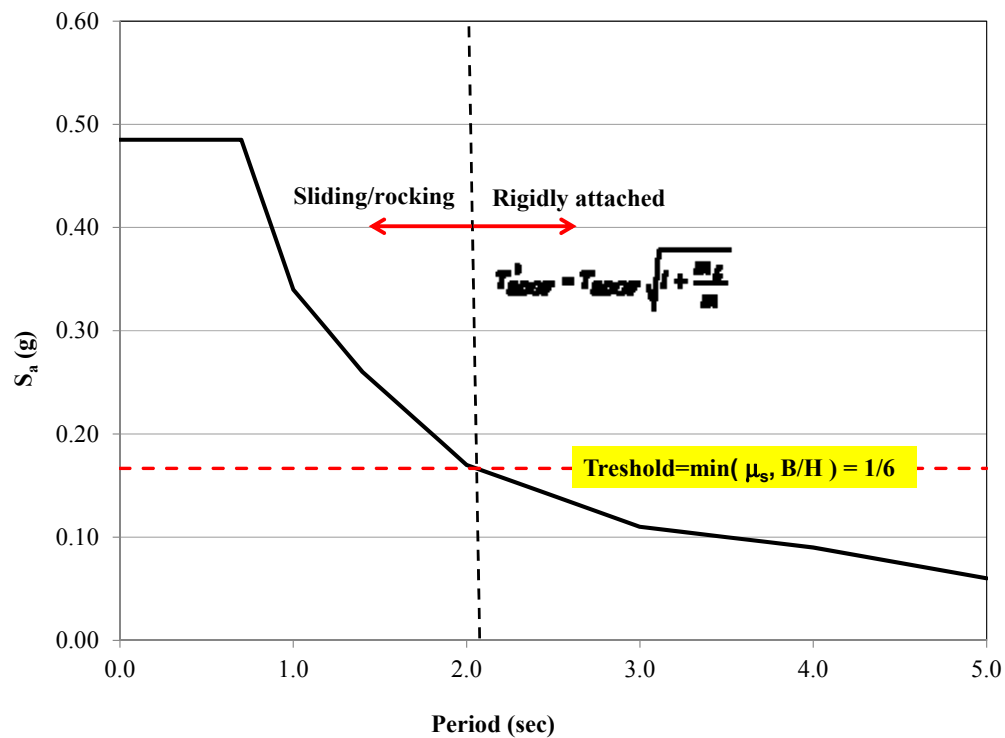


Figure 16 -- Threshold Acceleration and Limiting Period Concept

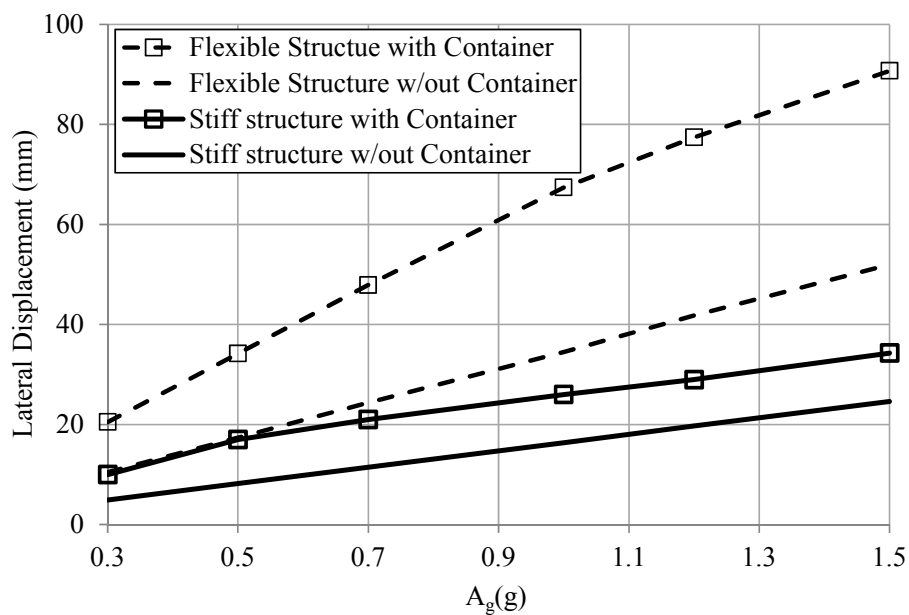


Figure 17 -- Deck Displacement with and without Containers - Numerical Example

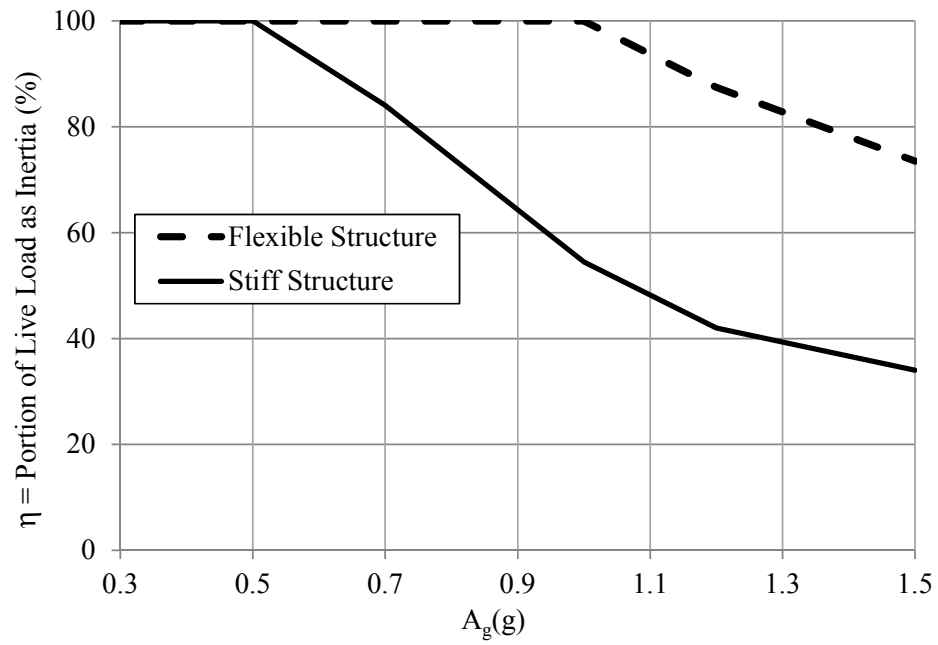


Figure 18 -- Effective Live Load as Inertia - Numerical Example

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

Carlos Blandón is professor of structural engineering at the School of Engineering of Antioquia (EIA) in Colombia. He received his MS and PhD in earthquake engineering from the ROSE School, University of Pavia, Italy. His research interests are seismic design of reinforced concrete structures, ports and soil-structure interaction.

ACI Fellow **José I. Restrepo** is professor of structural engineering at UCSD. He received his PhD from the University of Canterbury, New Zealand. His research interests include structural dynamics and earthquake engineering. Dr. Restrepo is a past recipient of the ACI Chester Paul Siess Award. He is currently a member of the ACI Committee 550, Precast Concrete Structures, and is also a corresponding member of ACI Committee 543, Concrete Piles.

Yohsuke Kawamata is a research fellow at Hyogo Earthquake Engineering Center, National Institute for Earth Science and Disaster Prevention (NIED), Japan. He got a MS degree at Tohoku University in Japan and a Ph.D. degree from Oregon State University. His interests in research are soil-structure interaction, liquefaction, and ground improvement.

Scott Ashford is professor in geotechnical engineering and head of the civil engineering and construction department at the Oregon State University. He received his PhD from the University of California, Berkeley. His main interest is to enhance public safety and reduce potential economic loss worldwide from earthquake and coastal events, focusing on full-scale modeling of soil-foundation-structure interaction, seismic site response, coastal erosion, and slope stability.

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.

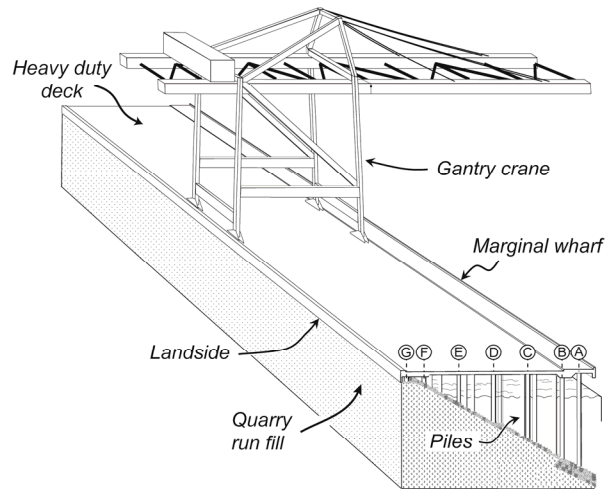


Figure 1 -- Typical Pile-supported Marginal Wharf Configuration.

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.¹⁵ 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 design¹⁶ 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).^{8,9} 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, reparable 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 ρ_s is the effective volumetric confinement ratio and ϵ_{smd} is the rupture strain of the pile-deck connection dowels. The strain in the extreme cover concrete, ϵ_c , the strain in the most stressed strand, ϵ_p , and the strain in the connection dowel, ϵ_{sd} , are defined for the critical sections at the pile-cap connection and at the inground hinge.

Table 1 -- Material Strain Limits for Solid Octagonal Concrete Piles.⁹

	OLE	CLE	DE
Connection concrete compressive strain ¹	$\epsilon_c \leq 0.005$	$\epsilon_c \leq (0.005 + 1.1\rho_s) \leq 0.025$	No Limit
Connection dowel tensile strain ¹	$\epsilon_{sd} \leq 0.015$	$\epsilon_{sd} \leq 0.6\epsilon_{smd} \leq 0.06$	$\epsilon_{sd} \leq 0.8\epsilon_{smd} \leq 0.08$
Inground concrete compressive strain ²	$\epsilon_c \leq 0.005$	$\epsilon_c \leq (0.005 + 1.1\rho_s) \leq 0.008$	$\epsilon_c \leq (0.005 + 1.1\rho_s) \leq 0.025$
Inground steel strand tensile strain ²	$\epsilon_p \leq 0.015$	$\epsilon_p \leq 0.025$	$\epsilon_p \leq 0.035$

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

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

ϵ_{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 connection¹³ 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 concrete¹⁸.

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.

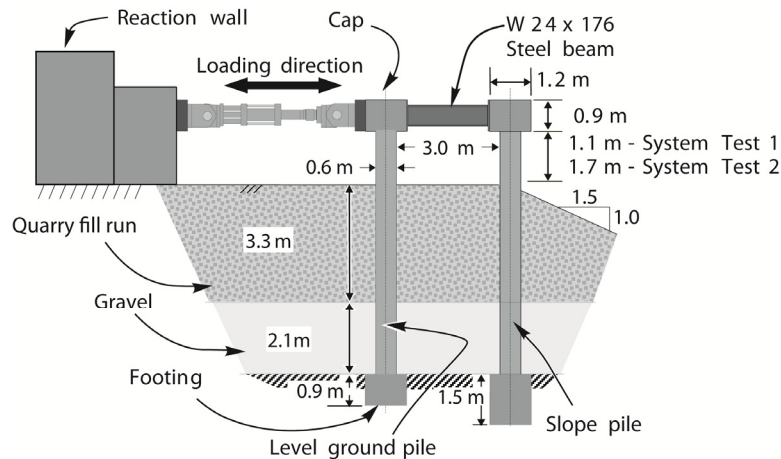


Figure 2 -- System Pile Test Set-up.

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).

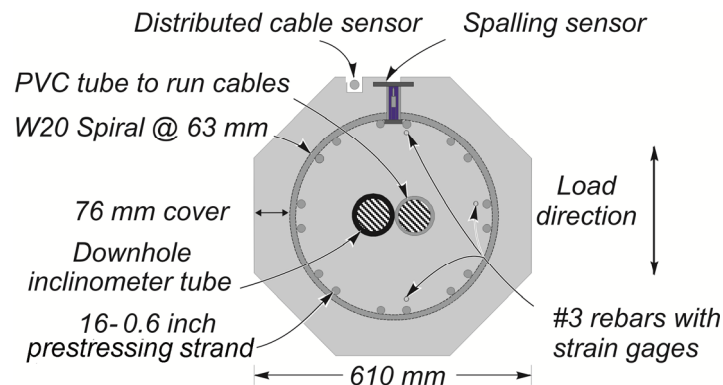


Figure 3 -- Pile Cross-section and Instrumentation.

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 m (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.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. Blandon¹⁸ 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 base at the core, see Figure 3. The second sensor was a rubber band placed outside the section and instrumented with strain gages.^{17,18} Inclometers 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. Inclometers 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,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 OpenSees²⁰.

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.¹⁸ 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, Δ_{75} , at $0.75 F_y$ which was later used to define the theoretical reference yield displacement of the system, Δ_{ye} , as $\frac{4}{3} \Delta_{75}$. The average value of Δ_{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 Δ_{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.

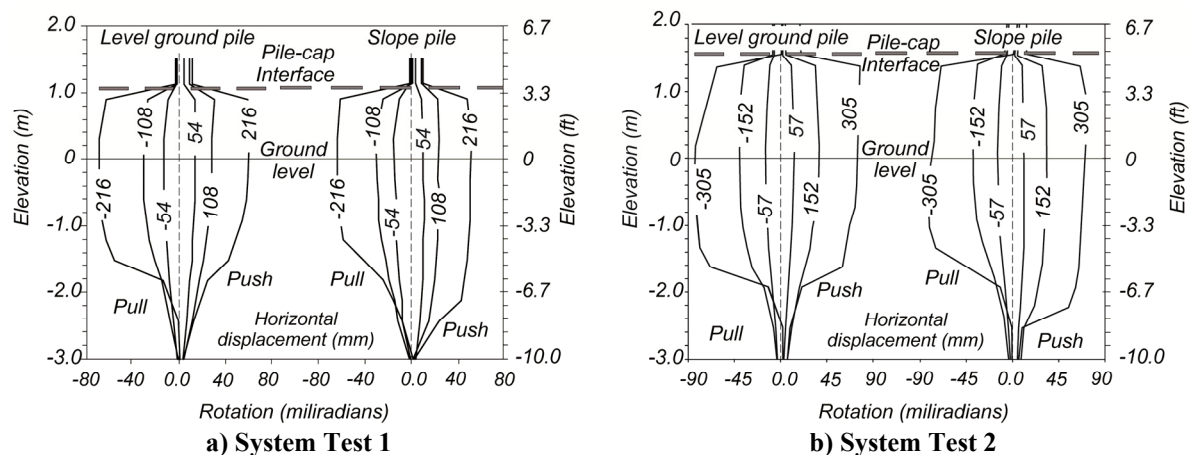


Figure 6 -- Rotation Profile for System Test. (Note: Pull is in the Upslope Direction and Push is in the Downslope direction)

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.

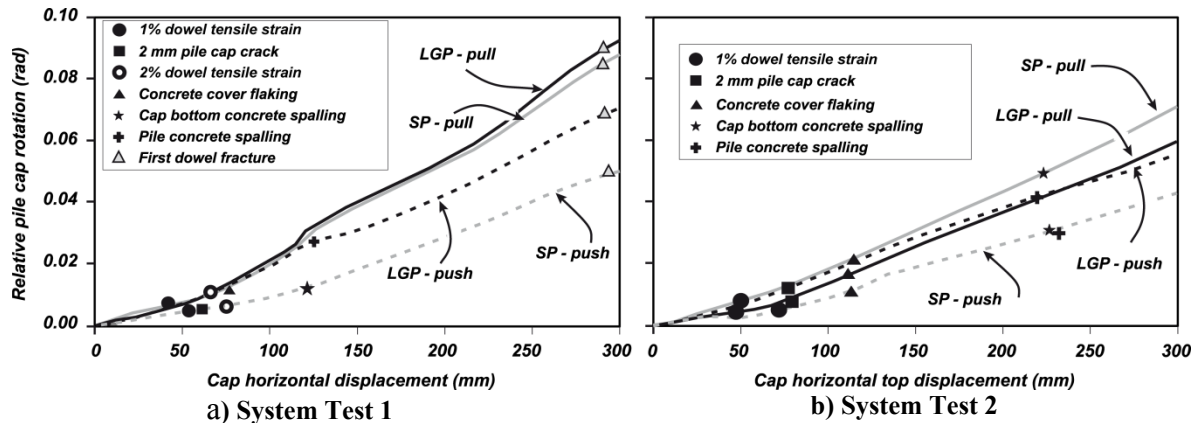


Figure 7 -- Relative Cap Displacement Versus Cap Horizontal Displacement for System Tests.

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.

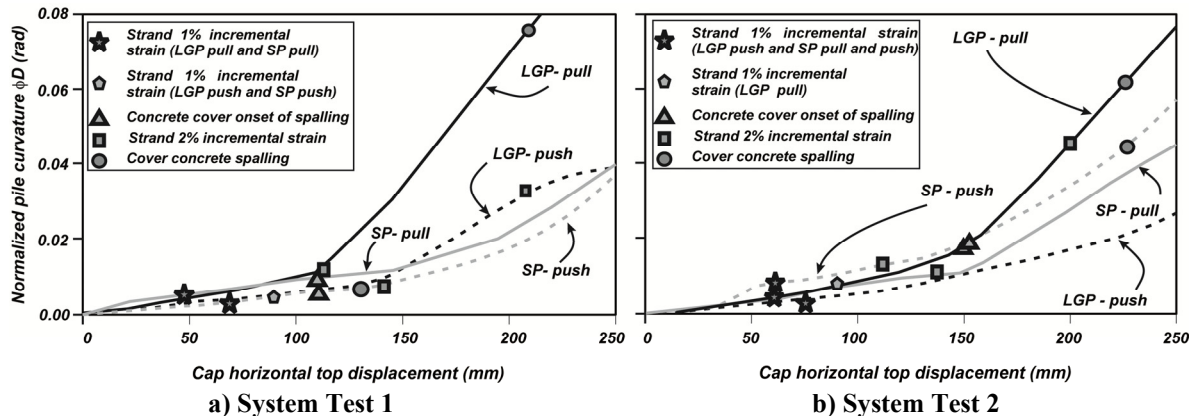


Figure 8 -- Normalized Pile In-ground Curvature Versus Cap Horizontal Displacement for System Tests.

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 Kawamata²¹ 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.

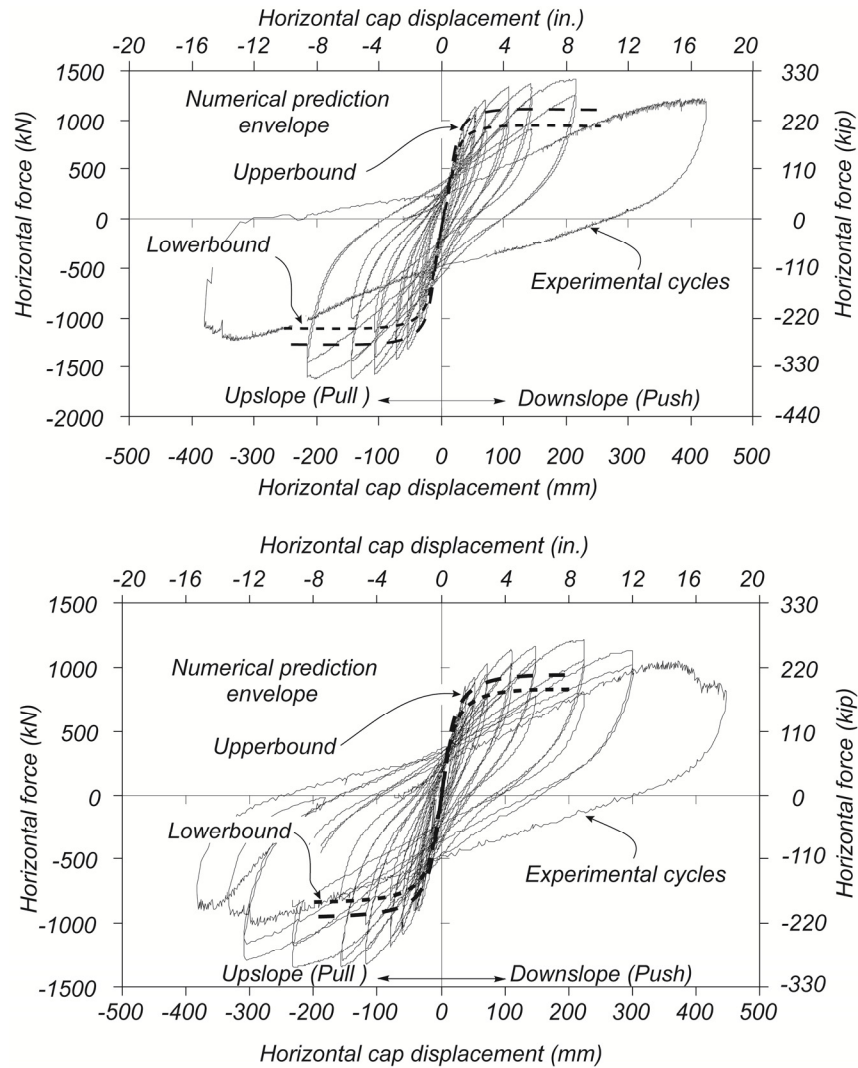


Figure 9 -- Hysteresis Loops and Prediction Envelopes for System Test 1 and System Test 2

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.

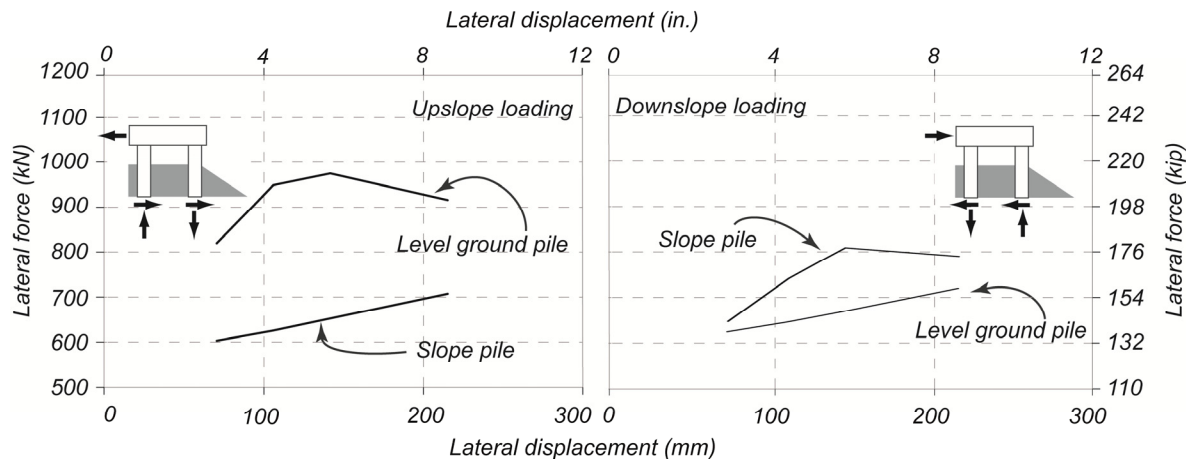


Figure 10 -- Lateral Force Distribution for Level Ground Pile and Slope Pile for System Test 1 for Upslope and Downslope Loading.

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.

ACKNOWLEDGMENTS

The experimental research presented in this paper was conducted at the Englekirk Structural Research Center at the University of California, San Diego. Testing was funded by the Port of Los Angeles, the Port of Long Beach and PEER under NSF Cooperative Agreement Award No EEC-9701568. Prof. M.J.N. Priestley, Dr. O. Jaradat and M. Weismair from Mofatt and Nichol, Prof. G. Martin from Earth Mechanics Inc. provided generous input in the design of the test program. The guidance provided by P. Yin of the Port of Los Angeles and C. Lai of the Port of Long Beach are greatly appreciated. Prof. G. Chen from the Missouri University of Science and Technology provided funding and support for the installation and monitoring of the continuous sensors in the piles.

REFERENCES

1. Tanaka, M., 1998, "Reconstruction and Revival of the Port of Kobe," *Port Technology International*, Iss. 8, Sep., 1998.
2. EQE, 1989, "The October 17, 1989 Loma Prieta Earthquake," EQE Report, October 1989, <<http://www.absconsulting.com/catastropheReports.html>>. Accessed 13 Jun. 2009.
3. EQE, 1994, "The January 17, 1994 Northridge, CA Earthquake," EQE Summary Report, March 1994, <<http://www.absconsulting.com/catastropheReports.html>>. Accessed 13 Jun. 2009.
4. EERI Reconnaissance Team, 2003, "Learning from earthquakes: Preliminary Observations on the Tecomán, Colima, Mexico, Earthquake of January 21, 2003," *EERI Special Earthquake Report*, http://www.eeri.org/lfe/pdf/mexico_colima_EERI_preliminary.pdf. Accessed 13 Jun. 2009.
5. Chang, S.E., 2000, "Disasters and transport systems: loss, recovery and competition at the Port of Kobe after the 1995 earthquake," *Journal of Transport Geography*, Vol. 8, Iss. 1., Jan. 2000, Pages 53-65.
6. PIANC, 2001, *Seismic Design Guidelines for Port Structures*, International Navigation Association Working Group No. 34, A.A. Balkema.
7. Nozu, A., Ichii, K., Sugano, T., 2004, "Seismic Design of Port Structures," *Journal of Japan Association for Earthquake Engineering*, Vol. 4, No. 3 (Special Issue).
8. Port of Los Angeles, 2010, "Commentary to the Code for Seismic Design, Upgrade and Repair of Container Wharves," 2010.
9. Port of Los Angeles, 2010, "Code for Seismic Design, Upgrade and Repair of Container Wharves," 2010.
10. Port of Long Beach, 2010, "Port of Long Beach Wharf Design Criteria, Version 2.0," <http://www.polb.com/civica/filebank/blobdload.asp?BlobID=3704>. Accessed 10 February 2010.
11. Roeder, C.W., Graff, R., Soderstrom, J.L., and Yoo, J.H., 2002, "Seismic Performance of Pile-Wharf Connections," *Technical Report* PEER 2002/07, University of Washington.
12. Krier, C.J., Restrepo, J.I., and Blandon, C.A., 2006, "Seismic Testing of Full-Scale Precast Prestressed Pile to Deck Connections," *Technical Report* No SSRP-06/26, University of California, San Diego, Department of Structural Engineering.
13. Bell, J.K., Blandon, C.A., and Restrepo, J.I., 2007, "Seismic Testing of Existing Full-Scale Pile to Deck Connections: Precast Prestressed and Steel Piles," *Technical Report* No SSRP-07/22, University of California, San Diego, Department of Structural Engineering.
14. Budek, M., 1997, "The Inelastic Behavior of Reinforced Concrete Piles and Pile Shafts", PhD Dissertation, University of California, San Diego.
15. Priestley, M.J.N., Seible, F., and Calvi, G.M., 1996, *Seismic Design and Retrofit of Bridges*, John Wiley and Sons, Inc., 1996.
16. SEAOC, 1995, "Vision 2000 - A Framework for Performance Based Earthquake Engineering," *Structural Engineers Association of California*, Vol. 1.
17. Kawamata, Y., Blandon, C.A., Ashford, S., and Restrepo, J.I., 2008, "Seismic Performance of Container Wharf Piles for the Port of Los Angeles," *Research Report*, TR 2007/06, University of California, San Diego.
18. Blandon, C., 2007, "Seismic Analysis and Design of Pile Supported Wharves". PhD thesis, University of Pavia, Italy.
19. Tsinker, G.P., 1988, "Pile Jetting," *ASCE Journal of Geotechnical Engineering*, Vol. 114, No. 3, March 1988, pp. 326-334.

20. McKenna, F., Fenves, G. L., Scott, M. H., and Jeremic, B., 2002, *Open System for Earthquake Engineering Simulation (OpenSees)*. Pacific Earthquake Engineering Research Center, University of California, Berkeley, CA.
21. Kawamata, Y., 2009, “Seismic Performance of Pile-supported Container Wharf Structures in Rockfill,” PhD dissertation, Oregon State University.

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

Omar Jaradat is a Senior Project Manager in Moffatt & Nichol's Structures group. Dr. Jaradat has more than 20 years of management and technical project experience. He is a registered civil engineer in California and Washington. He received his PhD degree in earthquake engineering from Washington State University. He specializes in earthquake engineering, soil-structure interaction and non-linear structural analysis. He is a member of several technical committees in ACI, ASCE, TCLEE and ATC.

ACI Fellow **M.J. Nigel Priestley** is Professor Emeritus of Structural Engineering, University of California at San Diego, La Jolla, California, and Emeritus Co-Director, Rose School, Pavia, Italy.

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 POLB developed its “Wharf Design Criteria” (POLB, 2008). This document included geotechnical, structural, seismic and electrical design criteria.
- 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.