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

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

This paper by no means intends to develop specific live load factors to be used when combining loads from stacked containers or mobile equipment as part of the seismic design of pile-supported platforms. Instead, the observations are meant to strengthen the need for the development of a rational methodology to evaluate specific load factors to deal with load combinations that combine "live" loads with other load types including earthquake loads. For simplification purposes, the authors will assume in this study that stacked containers be characterized as a "special" type of live load.

In addition to the complexities associated with the characterization of the live load, the evaluation of the fraction of the live load represented by stacked containers which contributes to the seismic mass of the structure is particularly challenging because the complex motions of stacked containers subjected to ground shaking make it difficult to identify whether the lateral load-displacement response of the structure would be accentuated, reduced or unaffected by the motion of the containers. Intuitively, for a given lateral stiffness of a pile-supported platform, an increase in the magnitude of the seismic mass at the time of occurrence of the ground motion would lead to an increase in the lateral displacement demand on the structure. If the structure depicted in Fig.1 were laterally flexible, then the acceleration of the deck under seismic actions may be small and the containers would tend to move together with the deck but not relative to it. Under this condition, the containers should be considered as rigidly attached to the deck and thus their entire inertia would contribute to the dynamic behavior of the pile-supported platform/container stack system, leading, in turn, to larger lateral displacements. In the event the opposite occurs, and the containers were to move, rock or slide relative to the deck, the question is whether it may be possible for such movement to reduce the displacement demand induced by the earthquake.

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

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

The most pressing questions stemming from these introductory remarks refer to what percentage of the live load needs to be considered for a safe evaluation of the seismic mass to be used in the seismic analysis of pile-supported piers, wharves and platforms 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 consider in the seismic mass evaluation. For the case of STS container cranes operating on marginal wharves, both codes recommend neglecting the STS crane mass effect if

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

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

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

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, $K = 0.5 PGA/g$, and PGA is the peak ground acceleration. The term K is meant to account for the effects of the vertical ground acceleration.

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

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

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

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

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

In Eqs. 5 and 6, K = 0.5 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 PGA) = 0.5 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

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

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.

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

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. $f_x \le \mu_s N$); 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 $\frac{\ddot{u}_{g,max}}{g} < \mu_s$, where $\ddot{u}_{g,max}$ 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.