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DYNAMIC EFFECT OF SLIDING RIGID BLOCKS ON THE SEISMIC RESPONSE OF STRUCTURES

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Building design standards provide limited guidance on how to include the inertial effects due to live load on the seismic design of structures. Guidelines suggest different percentages of the design live load to be included as mass but these do not seem to have any physical or mechanical basis, and are rather based on consensus of code writers that solely rely on usage statistics of facilities. The determination of live load that is effective as inertia may be critical to the seismic design of storage structures like waterfront yards used for container stacks that may weigh up to twice as much as the structure itself. In the event of a major earthquake, containers may slide/rock and this dynamic action can affect the way in which the supporting structure responds to the ground motion. Considering the containers as rigidly attached can be too conservative because the energy that is dissipated by friction and impact is neglected. On the other hand, consideration of the containers as fully detached is unsafe because it would generally lead to underestimation of the deformations of the supporting structure during an earthquake. The contribution of live loads to inertial forces depends on the amount of energy dissipated through sliding and rocking of containers. Although much research has been conducted over the last five decades on the dynamic response of rigid blocks excited at their base, block-structure interaction is still rather unknown.

This paper presents analysis results from lumped-parameter and finite element models for a simple structure supporting a rigid block that has the possibility to slide under operational and contingency level ground motions. The paper also discusses some of the details of an ongoing series of shaking table tests on a 1:15 scale specimen used to calibrate the numerical models. An extensive parametric study is presented to show the variation of drift demand as a function of: i) the fundamental period of the structure, ii) the live load (block) to structure mass ratio, iii) the friction coefficient between the structure and the rigid block, and iv) the response modification factor *R* used to design the structure. Preliminary design recommendations are given in regards to the conditions for which the entire live load should be considered as inertia in the seismic analysis/design of storage structures.

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INTRODUCTION

Seismic design/assessment of structures strongly relies on the estimation of inertial forces and thus the proper quantification of the mass that is present at the time of occurrence of an earthquake. While the determination of the self-structural mass is rather straight forward, quantification of the inertial effects due to temporary actions—like the mass of people in an office building, or the mass of cars in a parking garage—can be a daunting task. Such temporary actions are referred to as live loads and building design standards provide limited and somewhat arbitrary guidance on how to include them in seismic analyses of structures. Standard ASCE/SEI 7-10 (2010), for example, establishes that the portion of the design live load mass to be included as inertia is as much as 25% in the case of facilities used for storage. However, this percentage reduces to 10% in guidelines that are applicable to marine structures (Port of Long Beach, 2012) and to zero in bridge design standards (AASHTO LRFD Specifications, 2012). The prescribed percentages are typically not questioned because live loads may not be so significant and because of the perception that only a minor portion of the live load is likely to be present at the time of occurrence of a seismic event. However, for pile-supported container yards such as that depicted in Fig. 1, live loads are nearly permanent and could exceed the dead load by a factor of two or more. In this case, the proper quantification of inertial effects due to live load becomes critical to the seismic design of the structure. Intuitively, the extent of container slide/rock during a ground motion would determine how much these contribute to the inertial forces as some energy is dissipated through friction and impact.

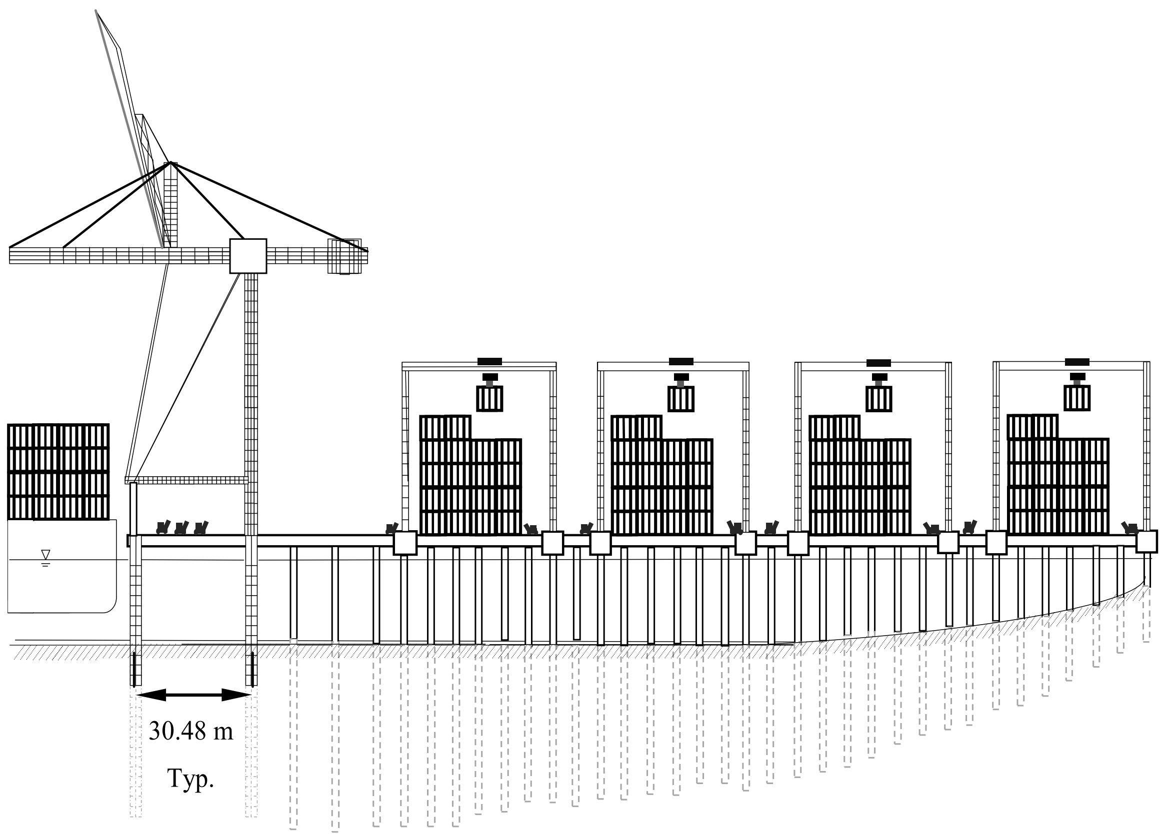

Figure 1. Pile-supported container terminal

The behavior of rigid bodies—like the containers in a terminal—is a complex problem of dynamics that has been studied extensively for many decades. Literature on the subject ranges from closed-form and numerical solutions describing the rocking response of blocks under base excitation (e.g., Housner, 1963; Spanos and Koh, 1984; Makris and Roussos, 1998) to shake table tests with blocks and scaled ISO container models (e.g., Peña et al., 2007; Kirkayak et al., 2011). Understanding the dynamic response of a sliding rigid mass driven by Coulomb friction was not only pivotal to the development of seismic isolation principles (Crandall et al., 1974) but it was also useful to studying the seismic behavior of gravity dams (Chopra and Zhang, 1991). The study of rigid block dynamics has also been relevant to the development of anchorage mechanisms for sensitive equipment and containers with hazardous materials.

Conditions for initiation of rocking-only, sliding-only, or coupled rocking-sliding of rigid blocks under one-dimensional base excitation were identified by Shenton (1996) in terms of the friction coefficient between the block and the ground, the aspect ratio of the block, and the magnitude of the ground acceleration. Realizing a block can be considered at-rest relative to the support when its acceleration (in g units) is less than the friction coefficient μ (to prevent sliding), and also less than the width-to-height aspect ratio B/H of the block (to prevent tipping), Smith-Pardo and Ospina (2013) proposed the definition of a threshold acceleration value for the linear-response spectral analysis of pile-supported container terminals. As depicted in Figure 2, the authors heuristically suggested that when the acceleration demand of the structure-block system is below such limiting value, container stacks may not move relative to the supporting platform and thus behave as rigidly attached to the structure. Conversely, for acceleration demands larger than the limiting value, the linear response spectrum is no longer valid as sliding or rocking of container stacks would take place. The latter condition is precisely the main purpose of this paper.

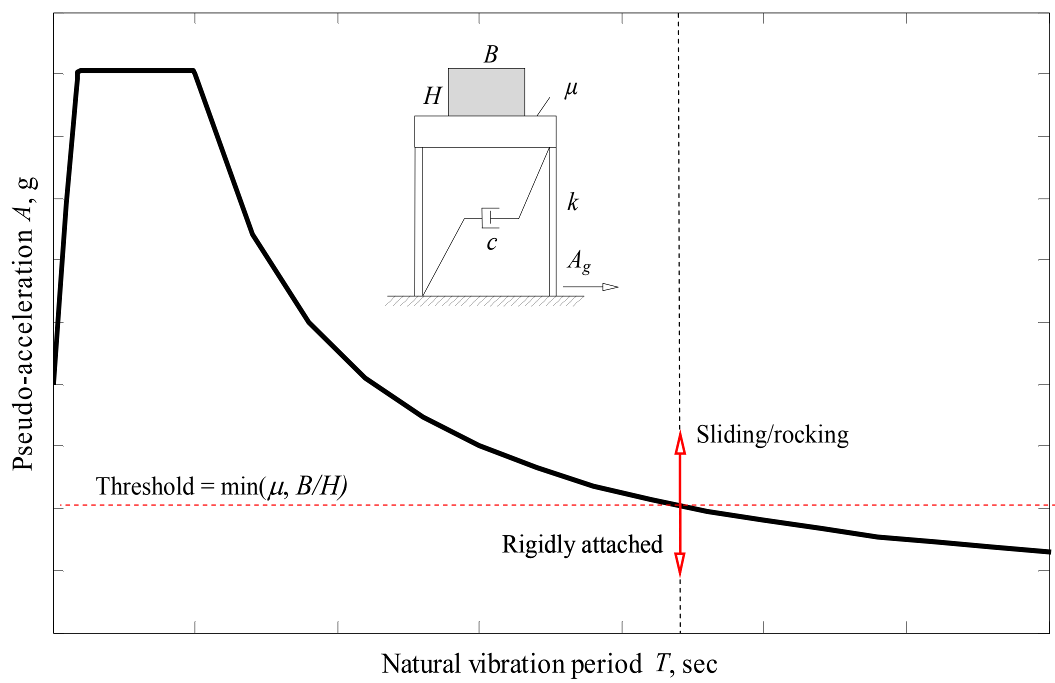

Figure 2. Threshold acceleration for rigid attachment of a block (after Smith-Pardo and Ospina, 2013)

The interest to investigate the seismic response of the structures supporting rigid blocks with the possibility to slide/rock is surprisingly scarce and out of date. Chandrasekaran and Saini (1969) presented a numerical solution of the equations of motion for a single-degree-of-freedom system supporting a rigid block with alternative types of attachment such as spring and viscous dashpot, Coulomb friction and dashpot, and rigidly mounted. Results from their analyses were limited to only two particular ground motions with no consideration to seismic risk or in reference to any design acceleration spectrum. Another shortcoming of the study is that the structure was considered to be elastic, which is generally applicable only to service-level ground motions. In response to the limited research on the dynamic effect of live loads on the seismic behavior of structures, this paper presents the development of a numerical model to characterize the dynamic interaction between a rigid block and its supporting structure including coupled nonlinearity from sliding of former and yielding of the latter. Operational and contingency levels of seismic risk were considered in the analyses consistent with conventional design provisions for waterfront structures.

SELECTED GROUND MOTIONS

Although applicable to storage facilities in which live loads are nearly permanent, this study was motivated by an actual consulting project on a pile-supported container yard. Specialized seismic design guidelines for piers and wharves in North America often provide specific performance criteria for operational and contingency design levels. The Operational Level Earthquake (OLE) is intended to be representative of an event with a probability of 50% of being exceeded in 50 years. The Contingency Level Earthquake (CLE) corresponds to an event with a 10% probability of being exceeded in 50 years. It is common practice to design for minor damage under OLE and repairable damage under CLE excitation. In such context, the occurrence of plastic hinges is desirable only at the pile-to-platform connection but not at the portion of the pile that is embedded into the ground.

Seven records were selected as representative of the operating level (OLE) seismic hazard. These corresponded to six shallow crustal earthquakes compatible with the following scenario:

- Moment magnitude: 6.0 to 6.9
- Fault distance: 2.0 to 39.8 km

Seven additional records were selected for the contingency level (CLE) seismic hazard which corresponded to six shallow crustal earthquakes compatible with the following scenario:

- Moment magnitude: 6.5 to 7.1
- Fault distance: 1.0 to 13.0 km

These ground motions were taken from the recommendations of Earth Mechanics (2006) in a port-wide seismic study for the port of Long Beach, California. In this study, the fault normal FN and fault parallel FP components of these records were modified to match the uniform hazard spectrum UHS for the site using the program RspMatch. Herein, only the FN component is considered. In addition, scaling of the records was performed according to the ASCE/SEI 7-10 procedure using a method proposed by Reyes and Chopra (2012). Fig. 3 shows the 5%-damping scaled response spectrum for each record of the OLE and CLE events. In all cases scale factors correspond to an oscillator with a fundamental period of $T = 1.0$ seconds. As required by ASCE/SEI 7-10 for near-fault sites, the average response spectrum is always above the design spectrum for the range of periods from $0.2T$ to $1.5T$.

NUMERICAL MODEL

An idealization of a platform structure consists of an elastoplastic single-degree-of-freedom (SDOF) system with mass m_p, lateral stiffness k, velocity-proportional (viscous) damping c, and base shear capacity $V_y = C_y\, m_p\, g$, where g is the acceleration of gravity and C_y is the base shear coefficient. As shown in Fig. 4, a rigid block of mass m_b lies on top of the platform and is assumed to be connected to the structure by Coulomb friction only. The static and kinematic coefficients of friction at the block-platform interface are μ_s and μ_k, respectively. The block is sufficiently squat so it can slide but it does not rock when the platform is excited at its base by the ground acceleration A_g.

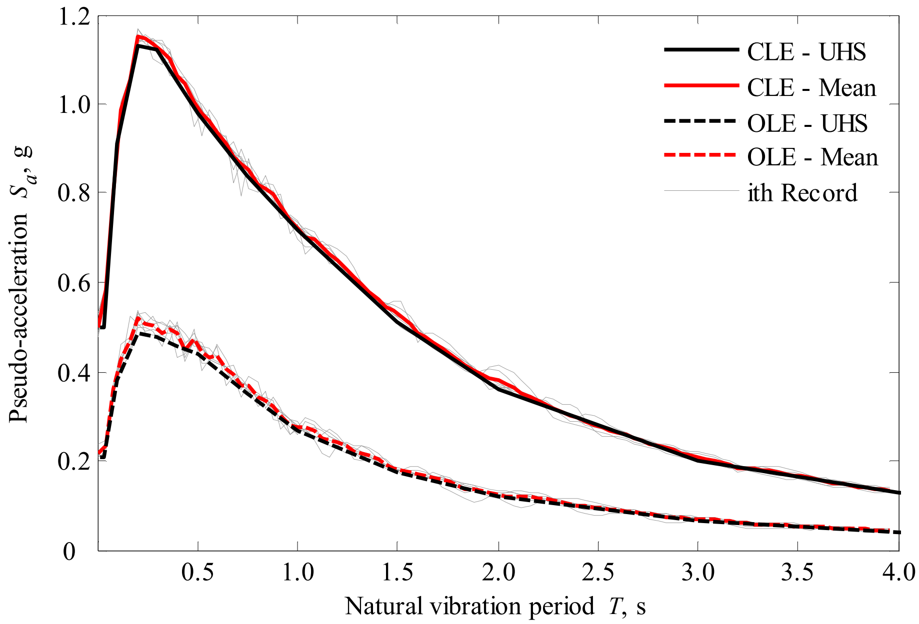

Figure 3. Uniform hazard and mean acceleration spectra for OLE and CLE levels for 5% damping.

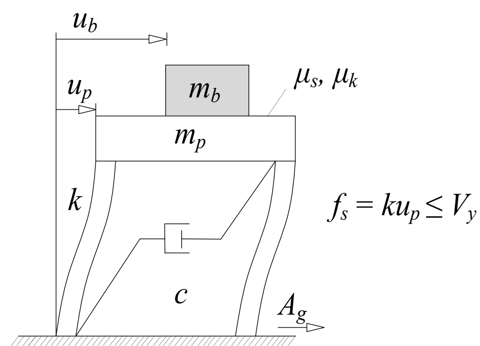

Figure 4. Platform structure supporting a squat rigid block

Let u_p, u_b and v_p, v_b be the displacements and velocities of the platform and the block with respect to the ground. The equations of equilibrium for the resulting two-degree-of-freedom system are:

$$\begin{aligned} m_b\left(\dot{v}_b + A_g\right) + f_x &= 0 \\ m_P\left(\dot{v}_p + A_g\right) + c\,v_P + f_s - f_x &= 0 \end{aligned} \tag{1}$$

where f_s is the resisting force of the elastoplastic SDOF structure with kinematic hysteresis and f_x is the friction force at the contact interface given by:

$$f_x = \begin{cases} -m_b\left(\dot{v}_P + A_g\right) & \left|v_b - v_p\right| = 0 \;\;\&\;\; \left|\dot{v}_P + A_g\right| < \mu_s\, g \\ \mu_k\, m_b g \times \mathrm{sign}\left(v_b - v_P\right) & \left|\dot{v}_P + A_g\right| > \mu_s\, g \end{cases} \tag{2}$$

Eqs. 1-2 were written as a system of first-order ordinary differential equations and then solved using an implicit 4th order Runge-Kutta algorithm terms of the period T, and damping ratio ξ, of the SDOF platform, as well as the block-to-structure mass ratio α, given by:

$$T = 2\pi\sqrt{\frac{m_p}{k}} \tag{3.a}$$

$$\xi = \frac{c}{2\sqrt{km_p}} \tag{3.b}$$

$$\alpha = \frac{m_b}{m_p} \tag{3.c}$$

The numerical solution of the seismic response of the platform structure supporting a sliding block was compared to the results from a finite element (FE) model developed in ANSYS (ANSYS 2009, 2010a&b). The static and kinematic coefficients of friction at the platform-block interface were taken as 0.4 and 0.3 (representative values for steel on concrete) while damping of the SDOF system was taken as 5% of the critical. Excitation at the base of the structure was chosen to be one of the operating level ground motions (OLE) included in the Earth Mechanics study but scaled up by a factor of four in order to ensure significant sliding of the rigid block. Fig. 5 shows the calculated structure drift (u_p) and block sliding (u_b–u_p) time series for arbitrary values of the block-to-platform mass ratio α, base shear coefficient C_y, and period of the SDOF system T. The calculated response modification factor R (ratio of the spectral acceleration over g to base shear coefficient C_y) is also included to identify cases in which yielding of the SDOF takes place in combination with block sliding.

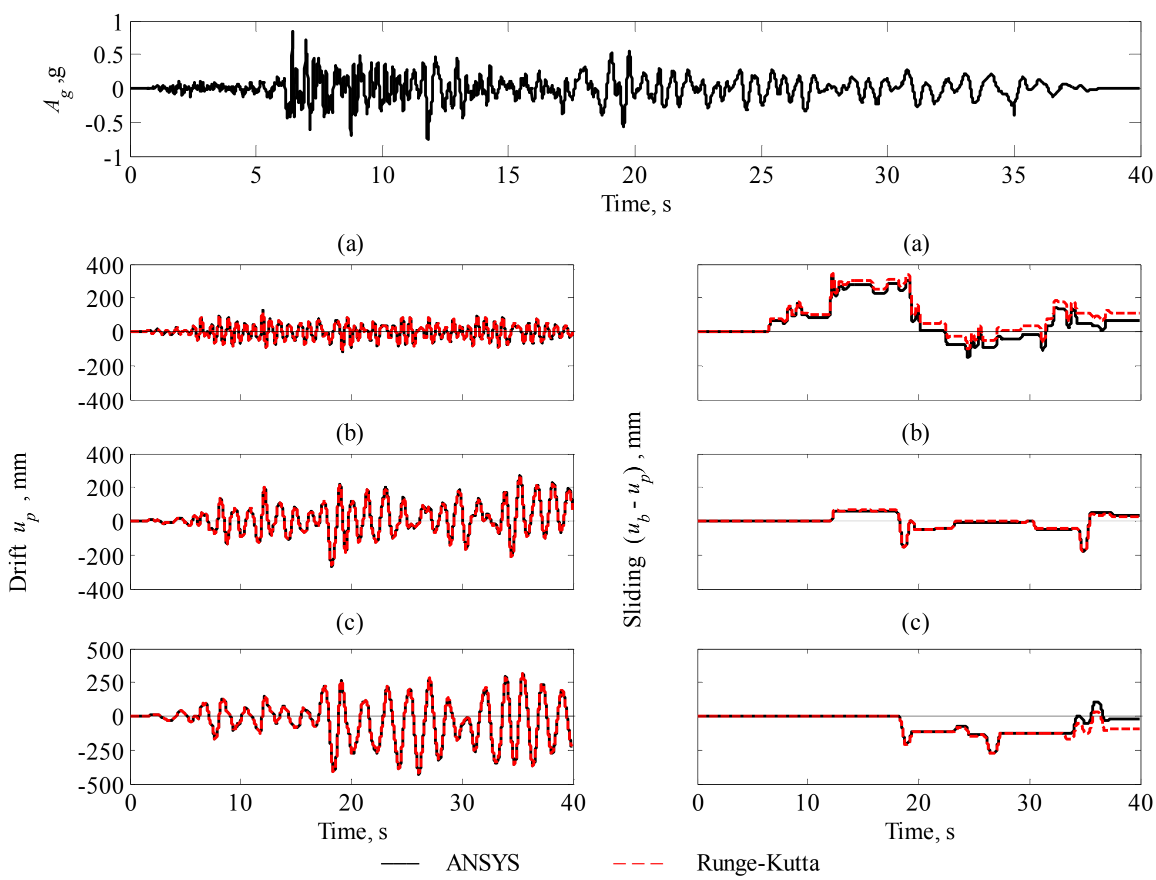

Figure 5. Comparison of results from Runge-Kutta algorithm and ANSYS model for: (a) $\alpha = 2.0$, $C_y = 3.0$, $R = 1.0$, $T = 0.5$s; (b) $\alpha = 1.0$, $C_y = 0.8$, $R = 1.44$, $T = 1.0$ s; (c) $\alpha = 0.5$, $C_y = 0.6$, $R = 1.18$, $T = 1.5$ s

As expected, for short-period linear elastic structures (R=1), the extent of block sliding is significantly more because the absolute acceleration of the platform is higher. It is also observed that calculated structure drift responses obtained from the FE and lumped-parameter models are almost identical, but the estimated block sliding histories are somewhat different. It should be noted that for structural design purposes, platform drift is much more important than sliding of the rigid block.

PARAMETRIC STUDY

The lumped-parameter numerical model was used to evaluate the influence of live load (as represented by a rigid block) on the seismic response of single-story platform structures. Variables of interest in the parametric study included: (1) the fundamental period of the structure T (Eq. 3.a); (2) the block-to-platform mass ratio α (Eq. 3.c) (3) the coefficient of friction at the interface of the block and platform μ (assumed to be the same at impending motion and during sliding); and (4) the response modification factor R. Viscous damping ξ (Eq. 3.b) was taken as 5% of the critical damping of the platform structure in all cases.

The parameter to quantify the effect of the live load on the seismic response of the platform was the ratio between the drift demand in a structure that supports a sliding block $(u_p)_{sliding}$ and the drift demand in the same structure but supporting a block that is rigidly attached $(u_p)_{attached}$. This parameter is referred to as Drift Demand Ratio DDR in the foregoing discussion and is calculated as:

$$DDR = \frac{(u_p)_{sliding}}{(u_p)_{attached}} \tag{4}$$

In each case, drift demand was defined as the mean value of the maximum drifts obtained for an array of seven scaled ground motions. A DDR approaching one indicates that most of the rigid block mass is effective as inertia in the seismic design of the structure; whereas a DDR approaching zero shows that the block does not significantly contribute to the inertia forces on the structure. Because spectral displacement tends to increase with period, the drift demand ratio is always between zero and one.

Fig. 6 summarizes the results of the parametric study for the scaled records of the OLE records. Because the level of excitation corresponds to a service condition, R was taken as 1.0 in all cases to reflect the fact that minor to no damage as it is typically expected for this hazard level.

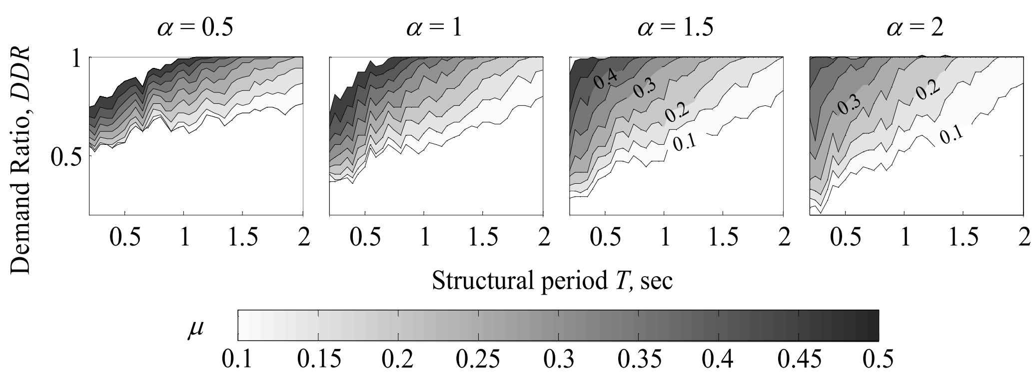

Figure 6. Drift demand ratio DDR (drift for sliding block to drift for rigidly attached block) for platform structures under Operational Level Earthquakes (OLE)

As expected, the Drift Demand Ratio DDR significantly increases with the period of the structure and the friction coefficient, especially for larger values of the block-to-structure mass ratio. This is because the platform of a flexible structure experiences small total accelerations, which tend to be insufficient to overcome the static friction at the interface with the containers. A practical implication that can be drawn from Fig. 6 is that because the friction coefficient between steel and concrete may be around 0.4, squat container stacks on pile-supported container yards with T>1.0 seconds could behave as rigidly attached regardless the live load to structure mass ratio. In such case, it may be appropriate to design the structure for the entire mass of the containers that is expected to be present at the time of occurrence of the seismic event. It should be noted that because the level of excitation corresponds to service conditions, the expected live load may be similar to the actual design live load and not just a small fraction of it as indicated in provisions of current design codes and standards. It is also inferred from Fig. 6 that for platform structures with periods of 2.0 seconds or longer, squat rigid blocks would behave as rigidly attached unless an external mechanism was implemented to significantly reduce friction at the block-platform interface. The drift demand ratio decreases with the block-to-structure mass ratio for T<1.0 seconds but the influence of this parameter is less significant for longer periods.

Fig. 7 presents the results of the parametric study for the scaled records of the CLE seismic hazard level. The columns of the array of plots correspond to various block-to-platform mass ratios α and the rows show various response modification factors R. Although pile-supported structures can be detailed to achieve high displacement ductility, only response modification factors R≤4 were included. Fig. 7 confirms that also for extreme seismic events the Drift Demand Ratio *DDR* increases significantly with the period of the structure T and the friction coefficient μ. The contribution of live load also increases rapidly with the response modification factor R. This is to be expected because structures that are designed to be ductile also have smaller acceleration demands and, therefore, it is less likely that the block overcome the static friction at the interface. A significant design implication that emerges is that one-story platform structures that are detailed for R≥4 would also need to be designed for the entire mass of the live load that can be expected during the extreme event, unless external mechanisms were used to significantly reduce friction with the live load objects.

Drift Demand Ratio *DDR* tends to decrease noticeably with the block-to-structure mass ratio for extreme events, especially in the lower period range (T<1.0 sec). This observation coincides with one of the conclusions drawn by Chandrasekaram and Saini (1969) for linear elastic structures (R=1.0).

A meaningful subset of data that can be extracted from Fig. 7 consists of the Drift Demand Ratio *DDR* for a fixed friction coefficient $\mu = 0.4$. Because such coefficient is representative of steel-on-concrete interfaces, the filtered results may be relevant to pile-supported yards storing squat container stacks. Fig. 8 shows the variation of *DDR* with the period of the structure for different response modification factors and constant live-load-to-structure mass ratio $\alpha = 1.0$. It is evident that the portion of the live load that is effective as inertia is rather significant as the *DDR* is at least 0.67. For pile-supported container yards that are designed for R≥2, it is also appropriate to design for the entire mass as inertia of the live load that can be expected at the time of occurrence of the seismic event. These observations hold for other block-to-structure mass ratios, in addition to $\alpha = 1.0$.

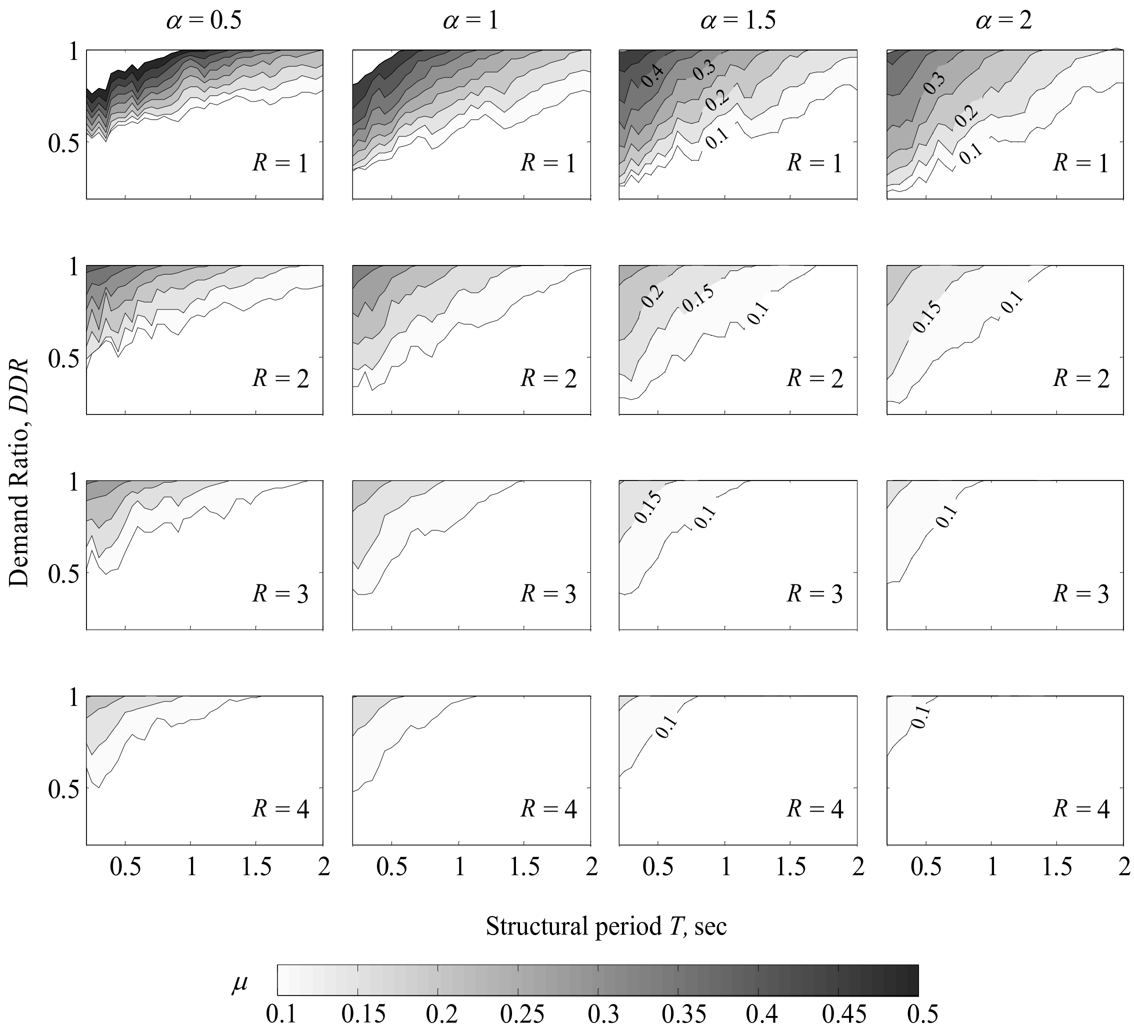

Figure 7. Drift demand ratio DDR (drift for sliding block to drift for rigidly attached block) for platform structures under Contingency Level Earthquakes (CLE)

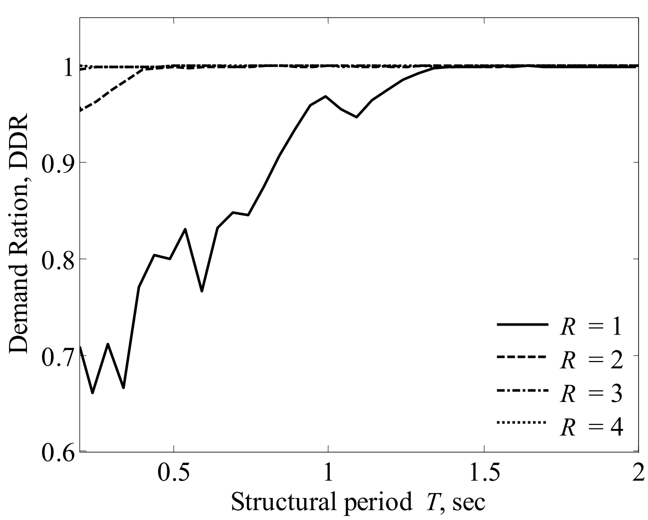

Figure 8. Drift demand ratio for $\mu = 0.4$ and $\alpha = 1$

EXPERIMENTAL PROGRAM

Laboratory experiments are being conducted using a 1:15 scale specimen made of a concrete platform on four steel tubes with a rigid block (representing live load) placed atop (Fig. 9). The specimen is anchoraged to a shake table that imposes base excitation corresponding to the OLE and CLE records described in this paper (upon reducing the time scale to satisfy similitude requirements). Variables of interest in the experimental program include the block-to-structure mass ratio α and the aspect ratio of the rigid block B/H. Some of the blocks used in the experimental program are slender and tend to rock during excitation, while other blocks are squat and tend to slide. The alternative aspect ratios will permit drawing conclusions about the influence of friction (sliding) versus impact (rocking) on the seismic response of the structure.

The rigid blocks are made of a series of exchangeable plates of steel and wood such that various weights can be achieved for the same aspect ratio and thus permit evaluating the effect of the block-to-structure mass ratio. The combination of two aspect ratios (one for rocking-dominated and one for sliding-dominated block response) with six block-to-structure mass ratios and seven ground motions leads to a total of 91 shake table tests for each seismic risk level (OLE and CLE), which includes tests of the model and no block . During the current phase of this research project the test specimen is to remain elastic under the two levels of excitation but future tests would also be devised to represent yielding at the connections for the CLE events. Further details about the experimental program are provided in a manuscript that is currently under preparation.

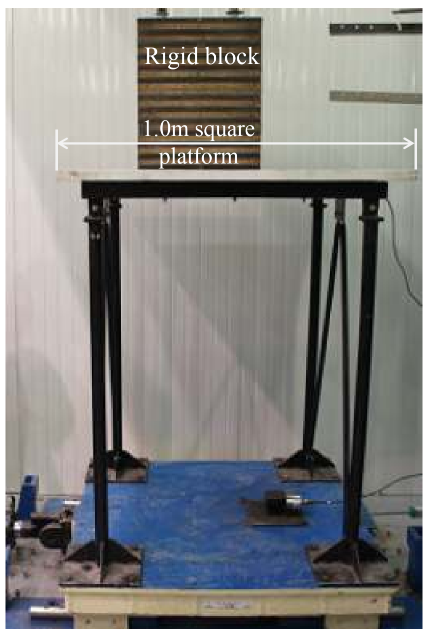

Figure 9. Scaled model of platform structure supporting a rigid block

CONCLUSIONS

The portion of live load that should be included as inertia in the seismic design/analysis of storage structures largely depends on: 1) the dynamic properties of the structure, 2) the selected response modification factor (force-based approach) or acceleration capacity of the structure (performance-based approach), 3) the seismic hazard level (operational versus extreme events), and 4) the attachment conditions of the live load to the structure (represented by the friction coefficient μ). In particular, it was found that it is appropriate to design for the entire live load that can be expected at the time of occurrence of a seismic event in the following cases: a) structures with $T \geq 2$ seconds under operating level ground motions; b) structures with $T \geq 1$ seconds under operating level ground excitation when the friction coefficient at the interface of the live load object with the structure is larger or equal than 0.4; c) structures that are detailed for R>4 under an extreme event, unless external mechanisms were used to significantly reduce friction with the live load objects; d) pile-supported container yards where $\mu \cong 0.4$ and $R \geq 2$ under an extreme event. In all these cases, the amount of acceleration that the block experiences tends to be insufficient to overcome the static friction with the live load objects so they behave as rigidly attached to the structure.

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