16th World Conference on Earthquake, 16WCEE 2017

Santiago Chile, January 9th to 13th 2017

Paper N° 2601

Registration Code: S-L1461955624

EVALUATION OF ASCE-7 REQUERIMENTS FOR ESTIMATING EFFECTIVE SEISMIC MASS

J. Paul Smith-Pardo[1], Esteban Marcillo-Delgado[2], Juan C. Reyes[3], and Oscar A. Ardila-Giraldo[4]

[1]*Assistant Professor, Department of Civil and Environmental Engineering, Seattle University, Seattle, USA. Email: smithjh@seattleu.edu*
[2]*Graduate Student, Department of Civil and Environmental Engineering, Universidad de los Andes, Bogotá, Colombia. Email: e.marcillo1471@uniandes.edu.co*
[3]*Associate Professor, Department of Civil and Environmental Engineering, Universidad de los Andes, Bogotá, Colombia (corresponding author). Email: jureyes@uniandes.edu.co*
[4]*Engineer, Empresas Públicas de Medellín, Medellín, Colombia. Email: OSCAR.ARDILA@epm.com.co*

Abstract

Seismic analysis and design of buildings rely on the proper estimation of the mass that effectively contributes as inertia forces on the structure. For the purpose of estimating seismic loads Standard ASCE7-10 requires calculating the effective seismic weight which includes dead load, partitions and permanent equipment, plus 25% of the floor live load in areas used for storage. This contribution of live load as inertia seems to correlate with the low likelihood that live load objects be present at the time of occurrence of the design earthquake. However, for storage and some commercial facilities live loads may continuously be present and even exceed the dead load. This paper presents the development of a lumped-parameter model of a multi-story shear building supporting rigid blocks that can slide. After successfully comparing with the results of finite element analyses and shake table tests, the model was used to evaluate the adequacy of the standard in relation to the treatment of live load as inertia. Results from a parametric study are presented in terms of the drift response of multi-story shear buildings supporting blocks with the possibility to slide, compared to the drift response of the same multi-story buildings but supporting 0% or 25% of the weight of the blocks as rigidly attached as required by ASCE7-10 for commercial and storage structures, respectively. It was found that the requirements of the ASCE7-10 provisions lead to underestimations of drift demands when live loads are nearly permanent. The results obtained using the ASCE7-10 provisions are highly unconservative for long-period buildings and for structures designed using higher response modification factors.

Keywords: rigid body-structure interaction; live load as inertia; ASCE 7 provision evaluation

1. Introduction

Various provisions related to the treatment of live load as inertia are available to practicing engineers in current codes/standards; ASCE/SEI 7-10 [1] for example requires that at least 25% of the live load in storage areas be included as inertia; however, that percentage becomes 10% in the standard for seismic design of piers and wharves (ASCE 61-14 [2]) while bridge design standards (AASHTO 2012 [3]) neglect the contribution of live load altogether. One could expect that 100% of densely packed live load becomes effective as inertia when it does not experience sufficient acceleration to move relative to the supporting floor/deck; a situation that is possible for long period structures subjected to minor earthquakes. Conversely, for stiff structures under severe ground motions, live load objects could slide, rock, or even topple over, so only small portions of their masses are effective as inertia. A more rational approach to quantify the effects of live load during an earthquake should first assess what portion of the design live load can reasonably be expected during the design earthquake and then based on principles of structural dynamics determine what portion of that expected load is actually effective as inertia. This paper is part of a research study [4] that deals with the latter aspect.

Numerous studies on the dynamics of rigid blocks under base excitation have been published over the last five decades. Housner [5] developed a closed-form solution of the rocking response of rectangular rigid blocks and demonstrated that the problem is scale dependent. Spanos and Koh [6] studied blocks under harmonic shaking and identified domains of amplitude-frequency for which toppling can occur. Experimental programs to study the rocking response of single rigid-block structures have been conducted by Peña et al. [7] and Kirkayak et al. [8]. Relevant to the development of base isolation technologies Crandall et al [9] showed that a structure could be protected from accelerations in excess of the limiting friction force of an isolator at the expense of a residual displacement.

Contrary to the vast amount of research on the behavior of rigid bodies under base excitation, few studies have focused on the dynamic interaction between rigid objects and the structures that support them. Younis and Tadjbakhsh [10] investigated the seismic behavior of a rectangular body with the possibility to slide on a platform restrained by a linear spring and a dashpot. Chandrasekaran and Saini [11] considered an elastic single-degree-of-freedom (SDF) system supporting a rigid block alternatively attached as an elastic spring and viscous dashpot, or Coulomb friction and dashpot, or rigidly mounted. More recently Smith-Pardo et al. [12] presented a lumped-parameter model that describes the seismic behavior of a SDF structure supporting a rigid block with the possibility to slide. This paper presents an extension of that formulation to multi-degree-of-freedom systems.

2. Ground motions

Seven ground motions were selected for this study as listed in Table 1. These records are consistent with deaggregation hazard analyses for the Port of Long Beach [13] and correspond to six shallow crustal earthquakes with moment magnitudes varying from 6.5 to 7.1 and fault distances from 1.0 to 13.0 km. The records are representative of a seismic event with a 10% probability of being exceeded in 50 years and are mostly near-fault due to the significant contribution of a local fault (called Palos Verdes) to the seismic hazard of this particular port in California. Reyes and Kalkan [14] studied the effect of rotating pairs of near-fault acceleration records and found that for linear systems near fault effects are critical when an apparent velocity pulse with a period close to the fundamental period of the structure is observed. Following this observation as a criterion to assess near-fault effects, a numerical procedure developed by Baker [15] was implemented in order to identify and characterize possible velocity pulses. Three of the seven records were identified as pulse-like ground motions with periods equal to 2.6 s, 4.2 s and 2.6 s (Table 1). These are significantly higher than the fundamental periods of all the structures considered in this study so near fault effects were not anticipated to influence the results presented in this study.

Table 1 – Selected records.

No.	Record name	Record station	M_w	D, km	Pulse Period, s
1	1999 Hector Mine	Hector	7.1	12.0	-
2	1989 Loma Prieta	Gilroy 03	6.9	13.0	2.6
3	1979 Imperial Valley	Brawley	6.5	10.0	4.2
4	1999 Duzce	Lamont 1059	7.1	4.0	-
5	1992 Erzinkan	Erzinkan	6.7	4.0	-
6	1940 Imperial Valley	El Centro	7.0	6.0	2.6
7	1995 Kobe	Kobe University	6.9	1.0	-

Although both fault-normal (FN) and fault-parallel (FP) components of these records were modified to match the uniform hazard spectrum (UHS) only the FN component is considered in the two-dimensional analyses presented in this paper. Scaling of the records was performed following the ASCE/SEI 7-10 procedure and a method proposed by Reyes and Chopra [16]. Fig. 1 shows the 5%-damping scaled response spectrum for each FN record along with the UHS. As required for near-fault sites, the average response spectrum is always above the design spectrum for the range of periods from $0.2T$ to $1.5T$. In all cases, scaling factors correspond to an oscillator with a fundamental period $T = 1.0$ second.

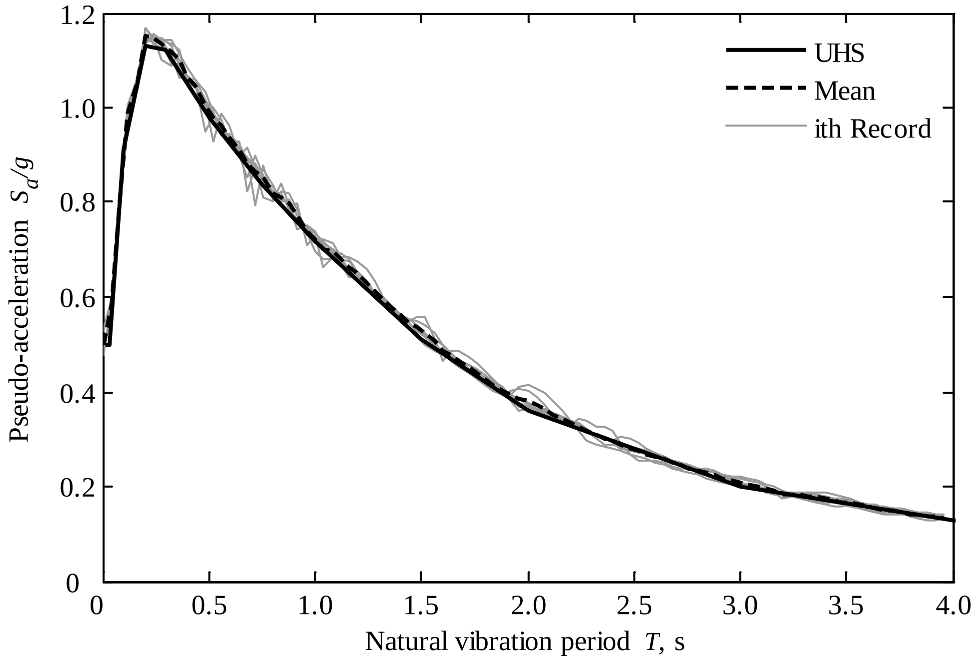

Fig. 1 – Uniform hazard and mean acceleration spectra for 5% damping.

3. Numerical model

Idealized structures considered in this research consist of multi-degree-of-freedom (MDF) shear buildings with translational mass matrix $[m_p]$, lateral stiffness matrix $[k]$, viscous damping matrix $[c]$, and yielding story shear capacity vector $\{V_y\}$. Live load is represented by a rigid block of mass m_{bi} on top of the i^{th} floor as shown in Fig. 2(a). Each block is connected to the structure by Coulomb friction as defined by static and kinematic coefficients μ_s and μ_k at the interface with the supporting deck. The block is squat so it can slide but not rock when the structure is excited by the unidirectional ground acceleration A_g.
Letting $\{u_p\}$, $\{u_b\}$ be the displacement and $\{v_p\}$, $\{v_b\}$ be the velocity vectors of the floors and blocks with

respect to the ground, dynamic equations of equilibrium for the system are given by:

$$[m_b]\left(\{\dot{v_b}\} + \{1\}(A_g)\right) + \{f_x\} = 0$$
$$[m_p]\left(\{\dot{v_p}\} + \{1\}A_g\right) + [c]\{v_p\} + \{f_s\} - \{f_x\} = 0 \quad (1)$$

where $\{f_x\}$ is the friction force vector at the block-floor interface and it is given by the following expression at the i^{th} floor:

$$f_{xi} = \begin{cases} -m_{bi}\left(\dot{v_{pi}} + A_g\right) & if\ \left(\dot{v_{pi}} + A_g\right) < \mu_s g \ \ and\ \ |v_{bi} - v_{pi}| = 0 \\ \mu_k m_{bi} g \times \mathrm{sign}(v_{bi} - v_{pi}) & if\ \left(\dot{v_{pi}} + A_g\right) \geq \mu_s g \ \ and\ \ |v_{bi} - v_{pi}| \neq 0 \end{cases} \quad (2)$$

where sign is the mathematical signum function = -1 if the relative velocity of the block with respect to the floor $(v_{bi} - v_{Pi})$ is negative, +1 if the relative velocity is positive, or zero if $(v_{bi} - v_{Pi}) = 0$.

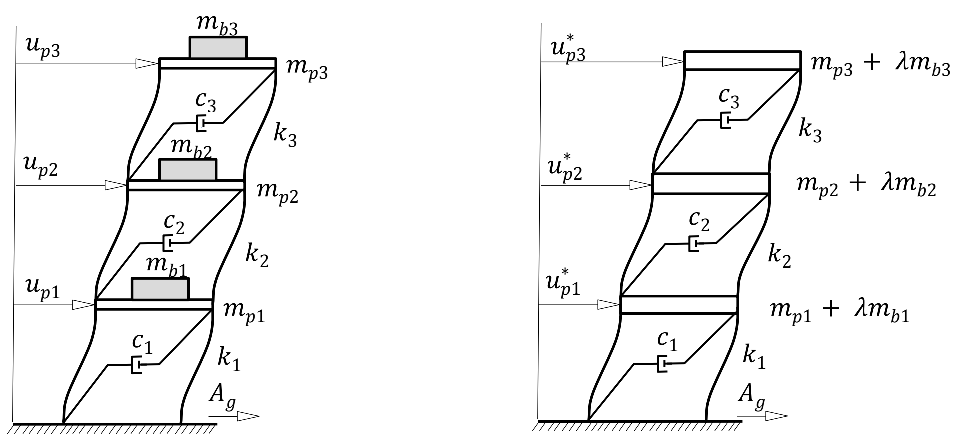

Fig. 2 – (a) MDF model supporting rigid blocks, (b) ASCE 7 MDF model with larger story mass but with no rigid block ($\lambda = 0$ for commercial, $\lambda = 0.25$ for storage facilities).

In Eq. (1), $\{f_s\}$ is the vector of story resisting forces and it depends on the lateral stiffness k_i and the shear capacity ΩV_{yi} of the i^{th} story (Fig. 3), where Ω is an over-strength factor. In the definition of the buildings included in this study, the story yielding shear V_{yi} is calculated as V_{ei}/R, where R is the response modification factor and V_{ei} is the elastic story shear, obtained from using the mean pseudo-acceleration in Fig. 1 and conducting a linear response spectrum analysis (RSA).

The two equations above were first re-written as a system of first-order ordinary differential equations and then solved using an implicit 4th order Runge-Kutta numerical algorithm. In order to facilitate subsequent discussions, a parameter called the block-to-floor mass ratio for the i^{th} floor was defined:

$$\alpha_i = {}^{m_{bi}}/_{m_{pi}} \quad (3)$$

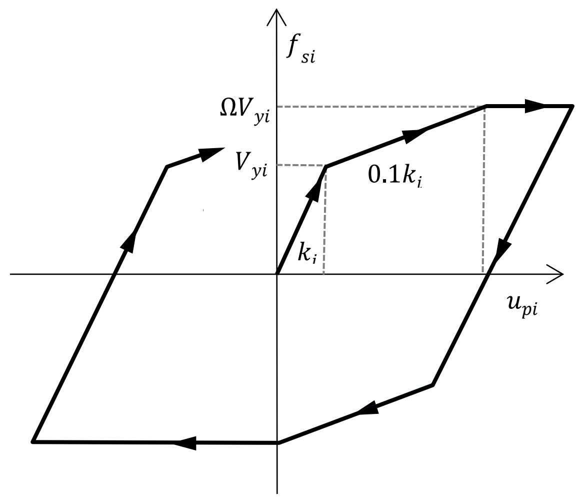

Fig.3 – Story trilinear restoring force model.

4. Comparison with shake table tests and FE analyses

The numerical model presented above was first evaluated against available shake table test results [17] from a 1:15 scale single story structure with a squat block placed atop. Base excitation corresponded to record No. 5 listed in Table 1 but with time scale reduced by a factor of 1/√15 to satisfy similitude requirements. The level of shaking was sufficient to cause sliding of the block although the structure remained elastic. Fig. 4 shows measured and calculated drift response histories of the test platform structure when it separately supported rigid blocks corresponding to α = 1.51 and 1.77. It is apparent that the calculated drift response is similar to the measured history thus providing confidence in the numerical model for the particular case of one-story structures.

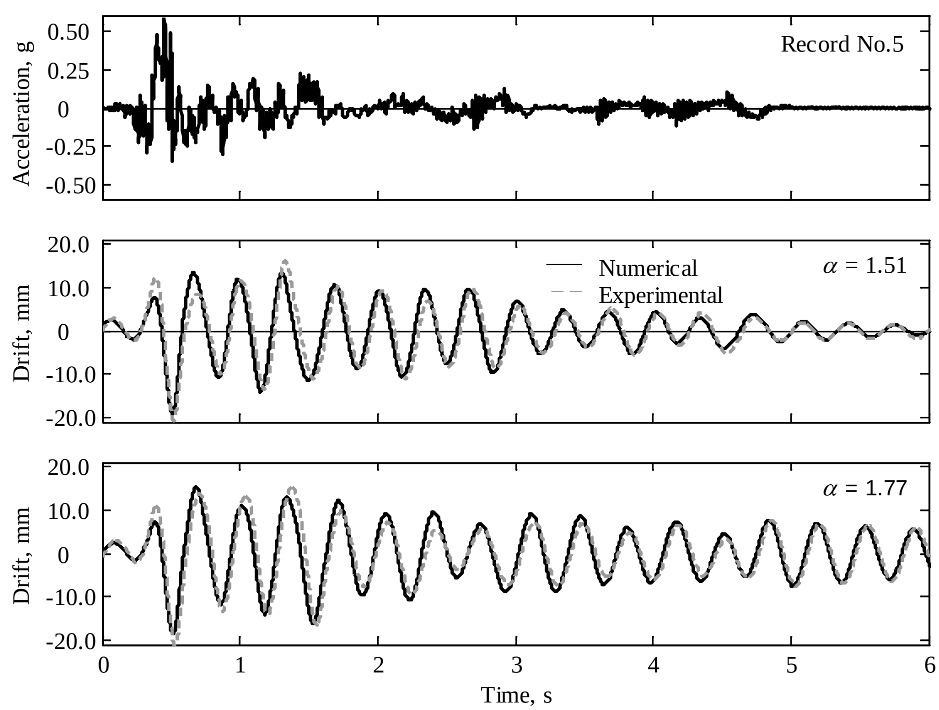

Fig. 4 – Platform drift time series for block-to-platform mass ratio α =1.51 and 1.77: numerical versus experimental.

The numerical solution of a MDF shear building supporting sliding blocks on all floors was also compared to the results from commercial Finite Elements (FE) programs ANSYS [18] and PERFORM [19]. An idealized 3-story shear building with floor lumped mass m_p and story lateral stiffness k was selected. Periods of vibration for three modes were 0.60, 0.21 and 0.14 s while damping ratios were taken as 5% in each mode. Story yield shear capacities were selected as $V_{y1} = 0.24W$, $V_{y2} = 0.20W$, and $V_{y3} = 0.12W$ (W is the seismic weight of the structure alone) and such that both story yielding and block sliding could simultaneously occur. The over-strength factor was taken as $\Omega = 2$ for all stories. Each floor supports a rigid block with the possibility to slide as determined by friction coefficients of 0.3 (static and kinematic). Blocks had the same mass as that of the supporting floors so $\alpha_1 = \alpha_2 = \alpha_3 = 1$.

Fig. 5 shows the story drift and block sliding time series for Imperial Valley (record No. 3), ground motion (Table 1). Restoring forces in all three stories were confirmed to reach yielding so coupling of the nonlinear response of the structure with the sliding of the blocks (mostly occurring at the first floor) was represented in the analyses. It is observed that the three modeling alternatives essentially produce the same time series thus giving further confidence in the developed numerical model.

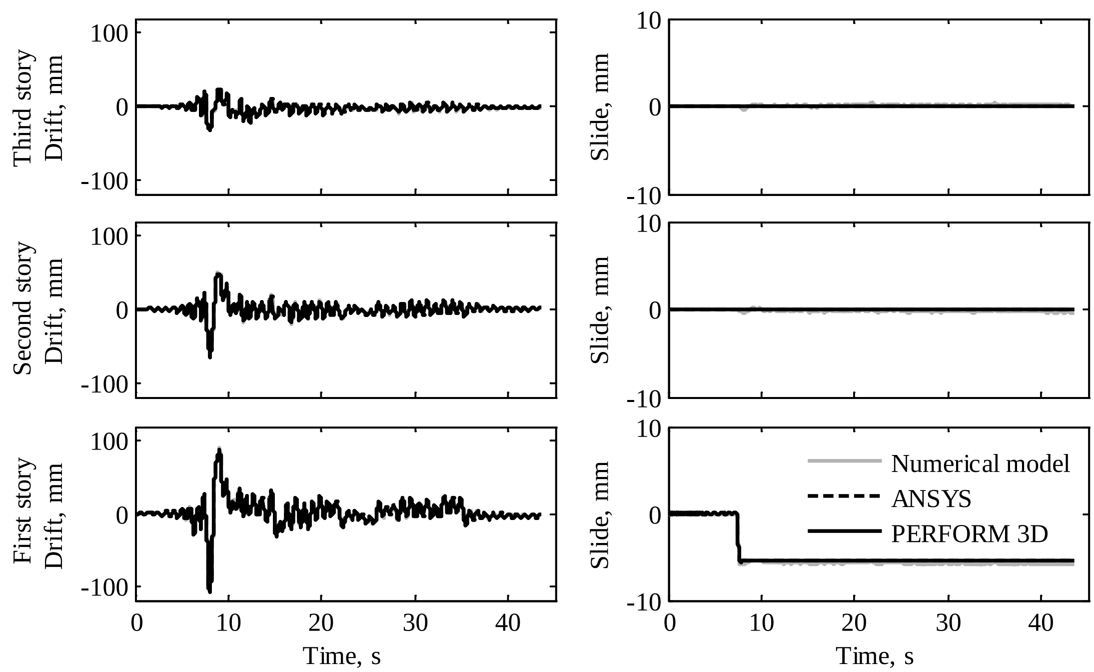

Fig.5 – Comparison of results from numerical model and commercial FE software for a 3-story shear building.

5. Evaluation of live load as inertia in ASCE/SEI 7-10

Upon validation with shake table tests and FE analyses, the numerical model was used to evaluate the provisions of ASCE/SEI 7-10 in relation to the treatment of live loads as inertia. As illustrated in Fig. 2(b), when using the ASCE/SEI 7-10 provisions live load objects are represented simply as additional floor masses corresponding to a portion λ of the live load itself and equal to 0.25 for storage facilities or zero, otherwise.

A parametric study was conducted for idealized commercial and storage facilities consisting of: (a) 2, 3, and 4-story shear building structures with floor lumped mass, m_p, story lateral stiffness, k, and corresponding fundamental periods of $T =$ 0.38, 0.47, and 0.57 s respectively; (b) response modification factors $R =$ 1 (elastic), 2, 4, and 6; and (c) block-to-floor mass ratios $\alpha = 0.5, 1.0,$ and 1.5 - equal at all floors except at the roof where $\alpha = 0$. Damping ratio was taken as 5% for all modes, while static and kinematic friction coefficient was taken as $\mu = 0.4$ in all floors of the buildings. The combination of these variables leads to 72 different structure models each subjected to the seven ground motions listed in Table 1. It should be recognized that the parametric study has less practical relevance to commercial facilities because of the variability/uncertainty in live loads during the earthquake as compared to storage structures.

Figs. 6 through 9 show the average maximum drift demands for the structures supporting rigid blocks with the possibility to slide (solid line) as calculated from the numerical model. In order to facilitate the discussion these are referred to as *Real* drift demands. The first three columns of these figures also show the calculated drifts of the structures when the live load blocks are not modeled but $\lambda = 0.25$ of their mass is added to the corresponding floor mass as permitted by ASCE/SEI 7-10 for storage structures. Similarly, the remaining three columns of these figures also show the calculated drift demand corresponding to ignoring the rigid blocks altogether as implied in the standard for non-storage facilities.

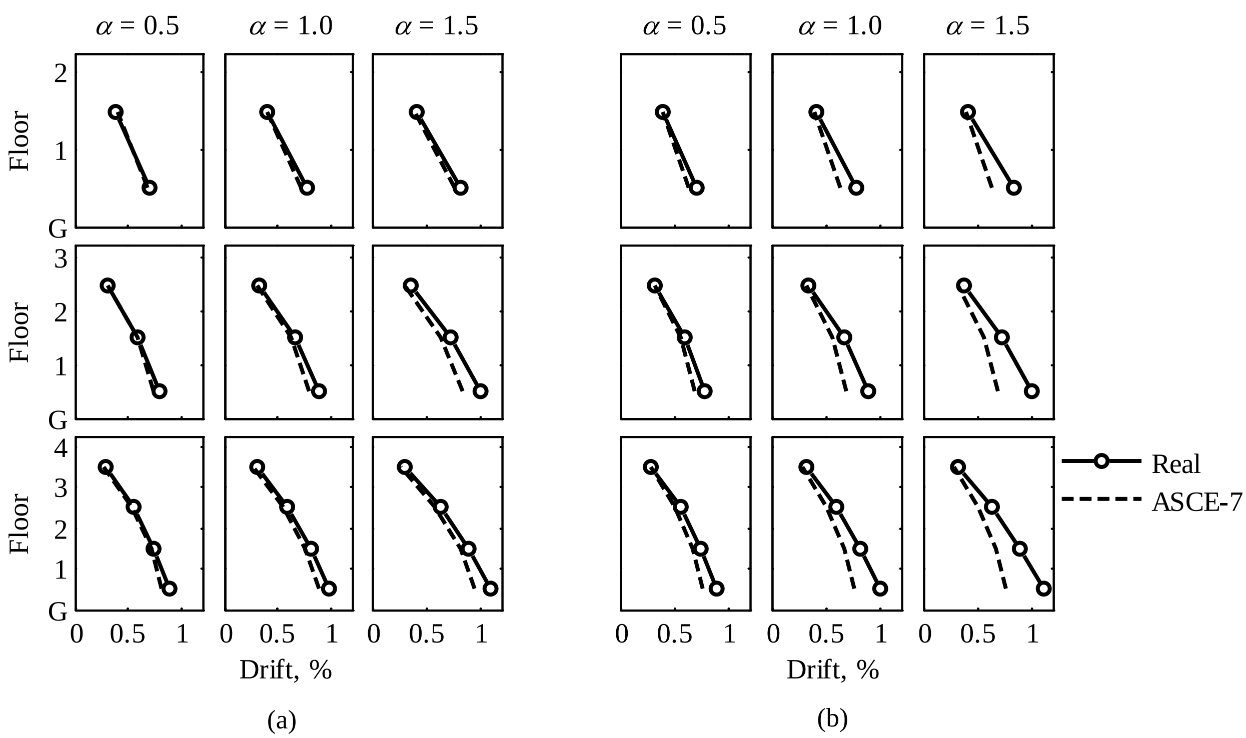

Fig. 6 – Comparison of calculated drifts of 2, 3 and 4-story elastic (R = 1) shear buildings supporting squat live load objects with results from using ASCE/SEI 7-10 provisions for: (a) storage facilities; (b) commercial facilities.

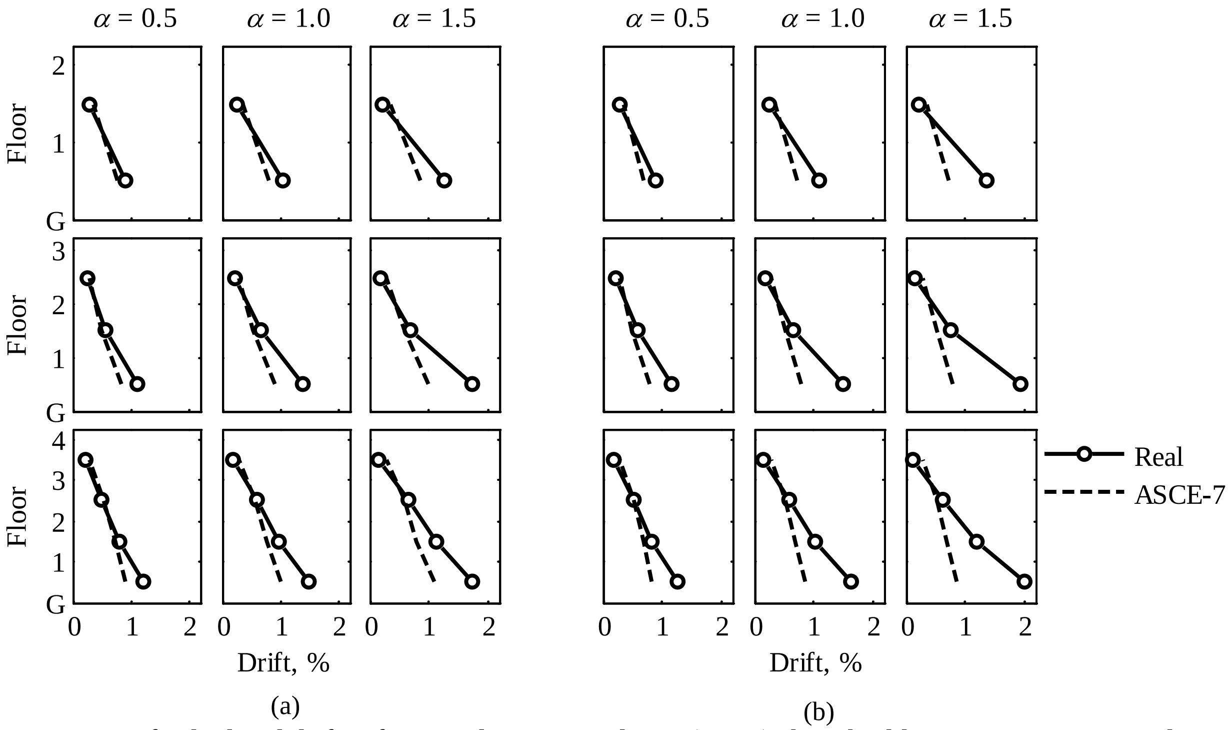

Fig. 7 – Comparison of calculated drifts of 2, 3 and 4-story inelastic (R = 2) shear buildings supporting squat live load objects with results from using ASCE/SEI 7-10 provisions for (a) storage facilities; (b) commercial facilities.

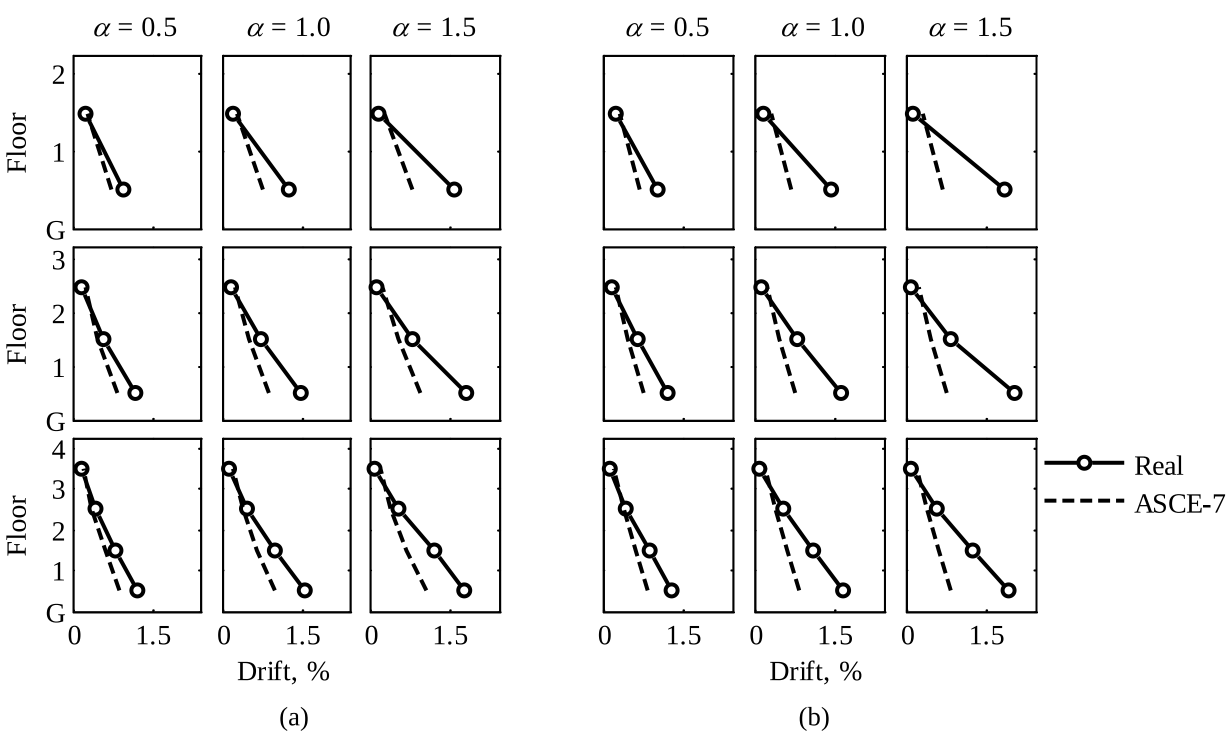

Fig. 8 – Comparison of calculated drifts of 2, 3 and 4-story inelastic (R = 4) shear buildings supporting squat live load objects with results from using ASCE/SEI 7-10 provisions for (a) storage facilities; (b) commercial facilities.

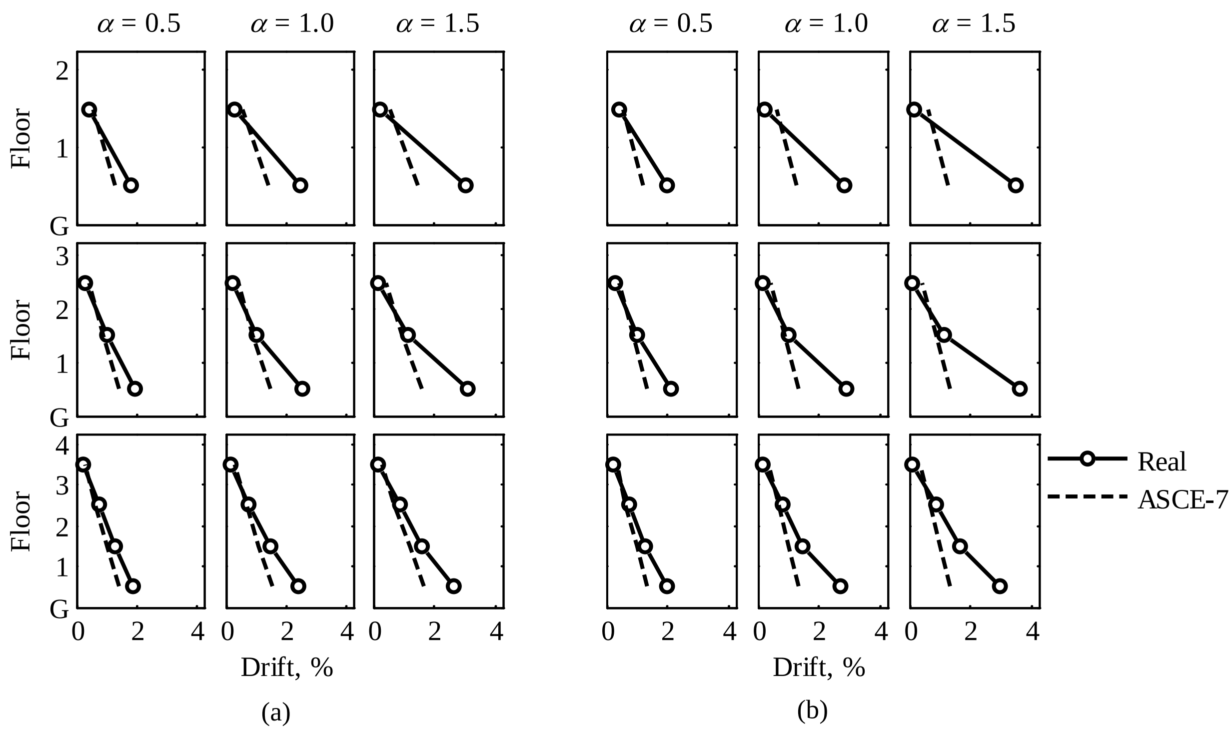

Fig. 9 – Comparison of calculated drifts of 2, 3 and 4-story inelastic (R = 6) shear buildings supporting squat live load objects with results from using ASCE/SEI 7-10 provisions for (a) storage facilities; (b) commercial facilities.

Figs. 6-9 show that implementing the provisions of ASCE/SEI 7-10 regarding the fraction of the design live load to be considered as seismic mass leads to underestimation of the drift demands of a multi-story building, especially for commercial facilities supporting nearly permanent live load objects. The difference between the actual and the ASCE/SEI 7-10 drift demands increases with the live-load-to-floor mass ratio α and the response modification factor R, and can be as large as 100%. Such trend can be explained by considering that the presence of heavier blocks makes the system more flexible while the use of larger values of R decreases the story shear capacity. Both of these effects result in smaller accelerations of the floors and less sliding of the live load objects. This leads to less energy dissipation through friction and the blocks approaching the condition of fully attached, for which 100% of their mass becomes effective as inertia.

The results above show that the provisions of ASCE/SEI 7-10 may be significantly unconservative for calculation of interstory drift demands and, consequently, for seismic design of multi-story commercial and storage facilities supporting nearly permanent live loads. Therefore, a methodology to quantify the inertial effects of live load objects, accounting for the dynamic properties of the structures and level of ground excitation, should be developed. This is the main goal of a research study currently being conducted by the authors [4].

6. Summary and conclusions

A numerical model that describes the nonlinear drift response of a multi-degree-of-freedom (MDF) shear building supporting rigid blocks with the possibility to slide was developed. Upon successful validation against the results of one-story shake table tests and finite element analyses of multi-story shear buildings, the model was used to evaluate the provisions of ASCE/SEI 7-10 in regards to the treatment of live loads as inertia. It was found that for multi-story buildings supporting nearly permanent live load objects, using the recommended 0% (commercial facility) or 25% (storage facility) of the design live load as additional mass on the structure may result in significant underestimation of the drift demand and unconservatism in the seismic design of the structure. This unconservatism is larger for greater live load object-to-floor mass ratios and greater response modification factors because in these cases floors may not experience enough acceleration to overcome friction and thus live load objects could behave as rigidly attached to the structure. For this reason, it is necessary to develop a methodology to rationally account for the fraction of live load that should be included as inertia in the seismic design of multi-story buildings.

7. References

[1] ASCE/SEI7-10 (2010): *Minimum design loads for buildings and other structures*. American Society of Civil Engineers, Reston Virginia.

[2] ASCE/COPRI 61-14 (2014): *Seismic design of piers and wharves*. American Society of Civil Engineers, Reston, Virginia.

[3] AASHTO (2012): AASHTO LRFD Bridge design specifications and commentary. 6th Edition. American Association of State Highway Transportation Officials, Washington, DC.

[4] Reyes JC, Marcillo-Delgado E., Smith-Pardo JP., Ardila-Giraldo OA (2016): Effects of sliding live load objects on the seismic response of commercial and storage Structures. (Under review).

[5] Housner GW (1963): The behavior of inverted pendulum structures during earthquakes. *Bulletin of the Seismological Society of America*, **53**(2), 403-417.

[6] Spanos PD, Koh AS (1984): Rocking of rigid blocks due to harmonic shaking. *Journal of Engineering Mechanics*, **110**(11), 1627-1642.

[7] Peña F, Prieto F, Lourenço PB, Campos-Costa A, Lemos JV (2007): On the dynamics of rocking motion of single rigid-block structures. *Earthquake Engineering & Structural Dynamics*, **36**(15), 2383–2399.

[8] Kirkayak L, De Souza VA, Suzuki K, Ando H, Sueoka H (2011): On the Vibrational characteristics of a two-tier scaled container stack. *Journal of Marine Science and Technology*, **16**(3): 354-365.

[9] Crandall SH, Lee SS, Williams JH (1974): Accumulated slip of a friction-controlled mass excited by earthquake motions. *Transactions of the ASME Winter annual meeting*, New York.

[10] Younis CJ. Tadjbakhsh IG (1984): Response of sliding rigid structure to base excitation. *Journal of Engineering Mechanics*, **110**(3), 417-432.

[11] Chandrasekaran AR, Saini SS (1969): Live load effect on dynamic response of structures. *Journal of Structural Division*, **5**(ST4), 649-659.

[12] Smith-Pardo JP, Reyes JC, Ardila-Bothia L, Villamizar-Gonzalez JN, Ardila-Giraldo OA (2015): Effect of live load on the seismic design of single-story storage structures under unidirectional horizontal ground motions. *Engineering Structures*, **93**: 50-60.

[13] Earth Mechanics (2006): *Port-wide ground motion study - Port of Long Beach*, Long Beach, California.

[14] Reyes JC, Kalkan E (2015): Significance of rotating ground motions on behavior of symmetric-and asymmetric-plan structures: Part 1. single-story structures. *Earthquake Spectra*, **31**(3), 1591–1612.

[15] Baker JW (2007): Quantitative classification of near-fault ground motions using wavelet analysis. *Bulletin of the Seismological Society of America*, **97**, 1486–1501.

[16] Reyes JC, Chopra AK (2012): Modal pushover-based scaling of two components of ground motion records for nonlinear RHA of buildings. *Earthquake Spectra*, **28**, 1243-1267.

[17] Reyes JC, Ardila-Bothia L, Smith-Pardo JP, Villamizar-Gonzalez JN, Ardila-Giraldo OA (2016): *Evaluation of the effect of live load on the seismic response of pile-supported storage structures*. *Engineering Structures*, under review.

[18] ANSYS (2010): *Mechanical APDL and mechanical applications theory reference*, ANSYS release 13.0.

[19] PERFORM 3D (2006): *PERFORM 3D user guide v4*, Computers and Structures (CSI), Inc., Berkeley, California.