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PARAMETRIC STUDY OF STEEL MOMENT FRAMES CONSIDERING FOUNDATION ROCKING

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Abstract

Allowing critical lateral force resisting components in a building structure, such as shear walls or frames, to rock and re-center under ground motion excitation have been proven through numerous experimental and analytical studies to improve the building's overall seismic performance. This is achieved due to the protection of critical components and the enhanced energy dissipation characteristics developed through rocking, which can help reduce the building's peak drifts, residual drifts, and peak floor accelerations, if properly designed. Additionally, the rocking behavior can reduce construction costs when compared to conventional designs incorporating deep foundation systems due to simplified construction details and smaller foundation sizes in the rocking system. This paper presents a parametric study of several conventionally-designed steel moment frame buildings analyzed considering rocking foundations. Steel Special Moment Resisting Frames consisting of 1, 2, and 3 bays in buildings of 3, 6, and 9 stories were designed per ASCE/SEI 7 for three levels of seismicity corresponding to three major cities in California. The resulting 27 building configurations were analyzed using 2-dimensional frame models in SAP2000. A suite of 7 ground motion records with representative seismic characteristics for the three building sites was used to carry out nonlinear response history analysis of the frames. Peak roof displacements, peak roof accelerations, and base shear demands, among other response parameters, were compared for all building configurations. The study assesses the effects of key design parameters that can influence the performance of rocking systems, including the building height, lateral stiffness, and ground motion intensity. Important structural analysis results are presented in detail, showing how each variable affects the seismic behavior of the rocking system.

Keywords: Rocking foundation; seismic performance; nonlinear analysis; steel moment frame

1. Introduction

The use of rocking behavior to improve structural performance dates back to Greek antiquity, if not earlier. Many Greek temples were built with segmented masonry columns containing a lead core. These columns were significantly ductile, and withstood limited seismic displacements by allowing uplift at the base [1]. Our modern understanding of rocking behavior dates back to George Housner's analytical work in 1963 [2]. Following the 1960 Valdivia earthquake in Chile, Housner conducted field reconnaissance and observed that some elevated water tanks were relatively undamaged, even when more apparently stable structures were severely damaged. Housner concluded that tall, slender blocks, similar to water tanks, have a counterintuitively large resistance to overturning when subjected to dynamic excitation.

Several studies have shown enhanced seismic performance by allowing rocking behavior in structures. Clough and Huckelbridge [3] conducted experimental work on two identical frames; one specimen was fixed at the base while the second frame was allowed to uplift promoting rocking behavior. A reduction of 33% in the seismic forces was observed in the frame that was allowed to uplift. Midorikawa et al. [4] conducted shake table tests on a three dimenional half scale model of a 3-story building. The columns were allowed to uplift at the base and test results showed a 52% base shear reduction compared to the fixed base model. However, these past research studies have not been translated into design guidelines for building codes. For typical linear-static analysis, shallow foundations and deep foundations are both assumed to be rigid supports. Furthermore, prescriptive building codes assume lateral displacements based upon a first-mode response with a fixed base. This can result in a significant overestimation of overturning force demands, a predicament which is mentioned only briefly by the building code. For example, ASCE/SEI 7[5], Section 12.8.5 requires overturning stability to be assessed using essentially the same forces employed for strength and stiffness checks, but does not address the different building characteristics that contribute to overturning behavior.

Academic research, as well as observations from past earthquakes, indicate that some structural configurations with shallow footings resulting in foundation uplift and rocking exhibit enhanced seismic performance than other fixed-base systems. In order to put this knowledge into practice, design engineers need to understand which building characteristics influence rocking behavior. In this study, the authors designed a range of buildings consisting of steel moment frames following typical code requirements in California, and analyzed their seismic performance using nonlinear response history analysis, assuming both rocking and fixed-base support conditions. Further research is needed to understand the effects of rocking behavior on other structural systems and building configurations.

2. Project description

In this study a range of archetype buildings were designed with the following characteristics: 12 ft story heights, 30 ft bay lengths, 6 ft wide continuous concrete grade beams, Site Class D, and rigid floor diaphragms. All designs incorporated Special Steel Moment-Resisting Frames with Risk Category II per ASCE/SEI 7 [5]. All moment frame connections were assumed to be welded unreinforced flange - welded web (WUF-W). All column bases were assumed to be restrained against rotation.

This study focused on the following variables: 1) Building heights of 3, 6, and 9 stories; 2) Frames consisting of 1, 2 and 3 bays; and 3) Level of seismicity corresponding to three major cities in California with different seismic hazard: Sacramento, San Francisco, and Los Angeles (see Fig. 1). This resulted in 27 different building configurations, where the designs were drift-controlled rather than governed by strength requirements, as is typically the case with moment-resisting frame structures. The Equivalent Lateral Force (ELF) procedure per the ASCE/SEI 7 [5] was performed utilizing a maximum drift limit of 2% to determine the steel member sizes.

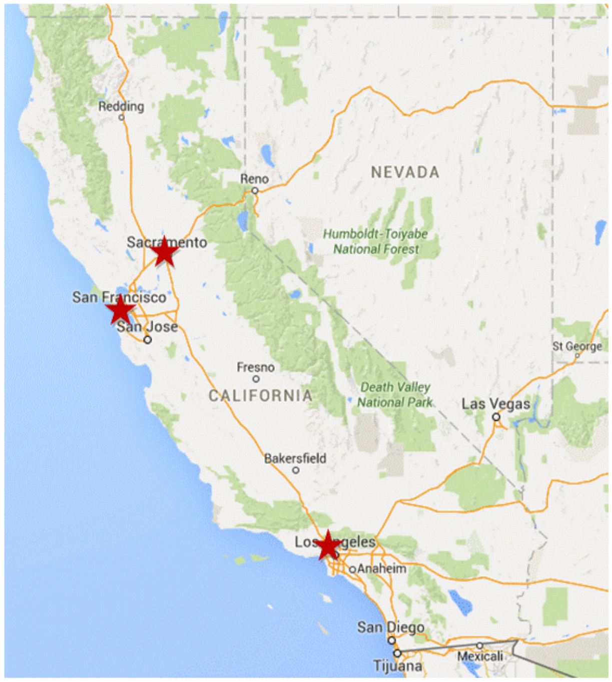
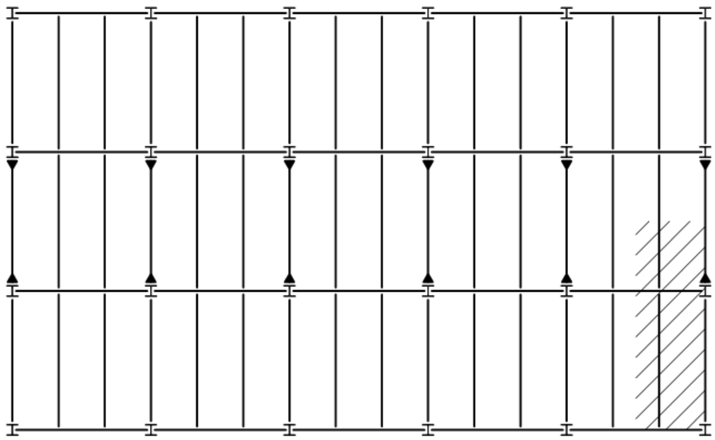
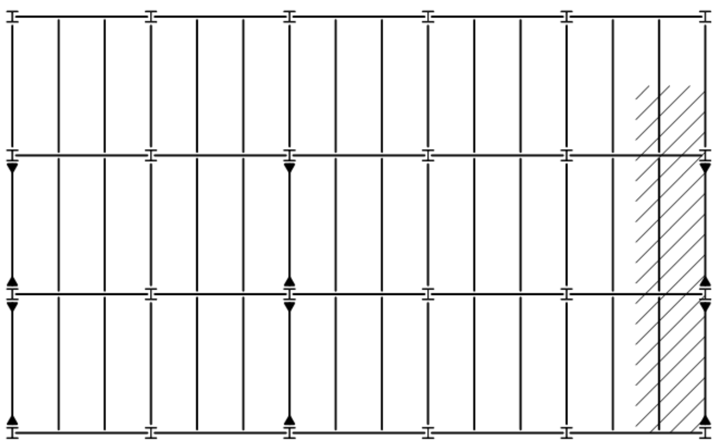


Fig. 1 – Building site locations

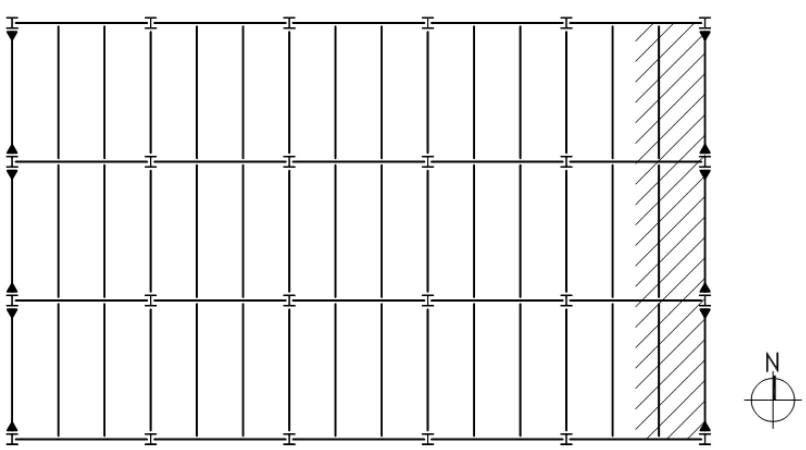
Each building analysis was carried out using a 2-dimensional frame only in the North-South direction as shown in Fig. 2; we did not consider the buildings' response in the East-West direction for simplicity. In all cases, an exterior frame of the building was selected for the analysis. Exterior frames typically experience greater rocking behavior than interior frames since they carry reduced tributary gravity loads compared to interior frames. Fig. 2 shows the three different building frame configurations, as well as the tributary gravity load area for the frame under consideration. These frame layouts consisted of six 1-bay frames, three 2-bay frames, and two 3-bay frames in the North-South direction. Fig. 3 shows a matrix of the different frame elevations used in the study; the frame sizes shown are for the building design corresponding to the San Francisco site.



1-bay frames



2-bay frames



3-bay frames

Fig. 2 – Building frame configurations showing tributary gravity load area (cross-hatched areas)

Fig. 3 – Building frame elevations – San Francisco building site

2.1 Seismic hazard

The seismic hazard for the three building sites in this study was determined using the USGS Seismic Design Maps tool [6] and the new 2015 NEHRP Recommended Seismic Provisions [7]. The design values provided by the USGS tool are the lesser of the probabilistic hazard values computed by the USGS on a national scale and the deterministic ground motion values based on site-specific procedures for seismic design stipulated in Chapter 21 of the 2015 NEHRP Provisions [7] and the ASCE/SEI 7 Standard [5]. For the 2015 NEHRP Provisions [7] (as well as the ASCE/SEI 7 Standard [5] and 2012 International Building Code [8]) the probabilistic values are risk-targeted rather than uniform-hazard ground motions, and both the probabilistic and deterministic values are defined in terms of maximum-direction rather than geometric-mean, horizontal spectral acceleration. The USGS derived national maps of design values for Site Class B were adjusted to Site Class D (determined), assumed for the current study, using the 2015 NEHRP site coefficients, which differ from the ASCE/SEI 7 values. Table 1 summarizes the resulting seismic design parameters used in the analysis.

Table 1 – Seismic design parameters

Site	Approximate Location	Latitude	Longitude	S_S	S_1	S_{DS}	S_{D1}	$T_s=S_{D1}/S_{DS}$ (sec)	$T_0=0.2T_s$ (sec)	T_L (sec)
San Francisco	Civic Center	37.782	-122.432	1.500	0.600	1.000	0.680	0.68	0.14	12
Los Angeles	Downtown	34.041	-118.247	1.942	0.691	1.295	0.783	0.60	0.12	8
Sacramento	Downtown	38.578	-121.494	0.568	0.253	0.510	0.353	0.69	0.14	12

For the nonlinear response history analysis, a single suite of seven ground motion records was selected and scaled to all three target design spectrum corresponding to the different building sites. These sites corresponded to relatively high seismicity areas with Strike-slip, Reverse, or Reverse-Oblique faulting mechanisms, maximum expected earthquake moment magnitudes, M_w of 6-8, and a distance to fault, R_{jb} greater than 10 km, therefore not affected by near-fault directivity effects. In addition, a shear wave velocity, $v_{s,30}$ of approximately 750 m/sec corresponding to a stiff soil or Site Class D was assumed for these sites. The selected records therefore excluded motions that exhibit a pulse-like shape in the velocity time history resulting from near-fault directivity. To ensure that the single set of motions adequately fitted all three target spectrum while still upholding the original earthquake characteristics, the scaling factors were limited to an approximate range of 1/3-3. The period range used in the scaling was $0.2T_1$=0.1 sec to $1.5T_1$=0.75 sec, where T_1 was an average first-mode period assumed for the set of the mid-rise buildings ranging from 3 to 9 stories.

The record selection process was also heavily based on the overall shape and maximum spectral amplitude of the response spectrum corresponding to the geometric mean of the two horizontal components of each record. Given the significantly different seismic hazards at the three sites and the limiting scale factors, the resulting mean spectrum of the record suite required an additional amplification factor of 1.2 to ensure all spectral values were equal or greater than the design spectrum in the target period range, as required by ASCE/SEI 7 [5]. Table 2 summarizes the selected records and resulting scale factors for the three building sites.

Table 2 – Characteristics of selected ground motion records

Earthquake	Year	RSN #*	Station Name	M_w	Mechanism	Scale Factor		
						San Francisco	Los Angeles	Sacramento
Imperial Valley	1940	6	El Centro Array #9	6.95	Strike slip	1.467	1.893	0.761
Kern County	1952	15	Taft Lincoln School	7.36	Reverse	2.299	2.966	1.193
Northern Calif.	1954	20	Ferndale City Hall	6.50	Strike slip	2.499	3.224	1.297
San Fernando	1971	57	Castaic- Old Rdg Rte	6.61	Reverse	1.462	1.886	0.759
Friuli, Italy	1976	125	Tolmezzo	6.50	Reverse	1.116	1.440	0.579
Imperial Valley	1979	164	Cerro Prieto	6.53	Strike slip	1.876	2.421	0.974
Coalinga	1983	340	Parkfield-Flt Zone 16	6.36	Reverse	2.484	3.205	1.289

*RSN=Record sequence number used in the NGA-West2 On-Line Ground-Motion Database Tool

Fig. 4 shows the response spectra of individual ground motion records (specifically, the geometric mean of horizontal components), scaled to the San Francisco design spectrum. The mean response spectrum of the record suite is also shown, as well as the Mean ± Sigma (one standard deviation). As discussed above, to ensure the target spectrum is satisfied for the full period range of interest, a 1.2 amplification factor is applied to the individual records. The scale factors for the records ranged from 0.6 to 3.2, as shown in Table 2 above, while the resulting spectra for all three sites are shown in Fig. 5 (denoted as Mean x SF). The corresponding design spectrum for the three sites are also shown for comparison. It is important to note that the resulting input spectrum (i.e., Mean x SF) fitted well the target spectrum well beyond the target period range from approximately 0.06 to 1.5 sec, thus also satisfying the 3- and 9-story building periods used in the scaling process.

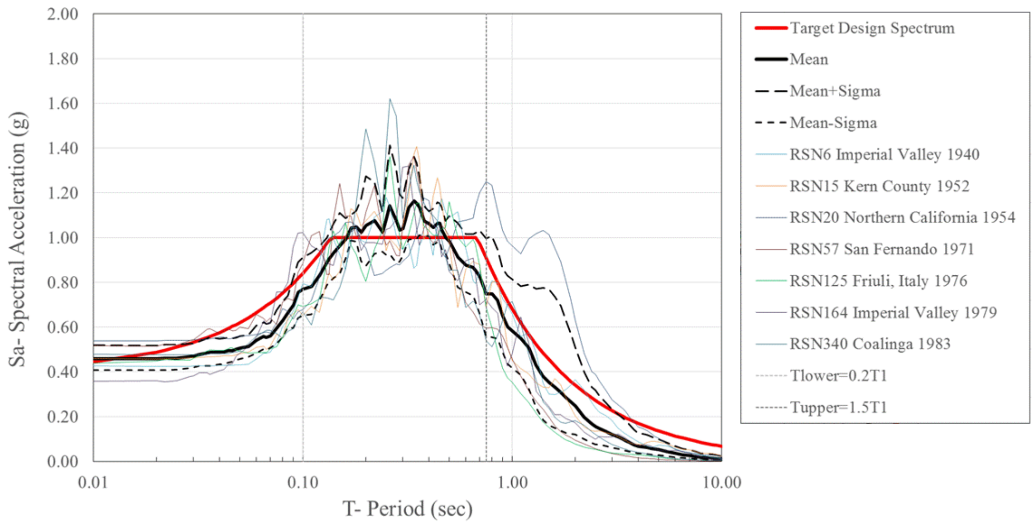


Fig. 4 – Ground motions spectral accelerations for San Francisco

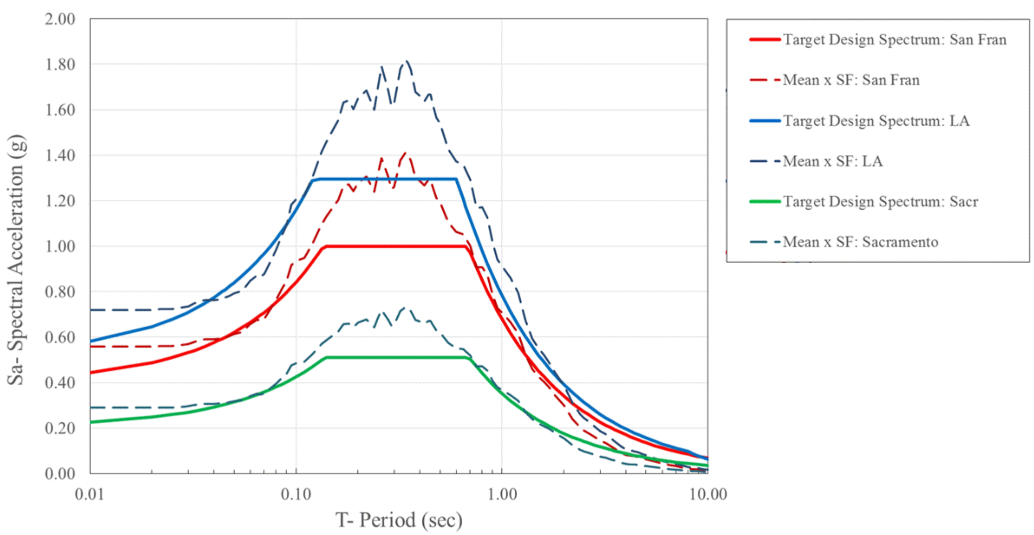


Fig. 5 – Scaled ground motion response spectra for each of the three locations

3. Nonlinear model description

For each of the 27 building designs, two nonlinear models were created: one with fixed supports (i.e., restraints on all 6 degrees of freedom), the other with horizontal restraints and gap-spring vertical supports, allowing rocking behavior to occur. The rocking model included an explicitly-modeled concrete grade beam. In compression, the gap-spring supports below the grade beam represented an elastic soil subgrade modulus of 800 kcf. The fixed-base and rocking models were subjected to identical ground motion time histories, described in the previous section. SAP2000 V.18 structural analysis program was used for all analyses. Nonlinear response history analyses were carried out following ASCE/SEI 41 [9] using a 1.1 x Dead Load plus 0.275 x Live Load combination. The models included a Rayleigh damping value of 1-2% in the period range of interest. Fig. 6 shows a schematics for the expected behavior and plastic mechanism of the fixed-base and rocking models.

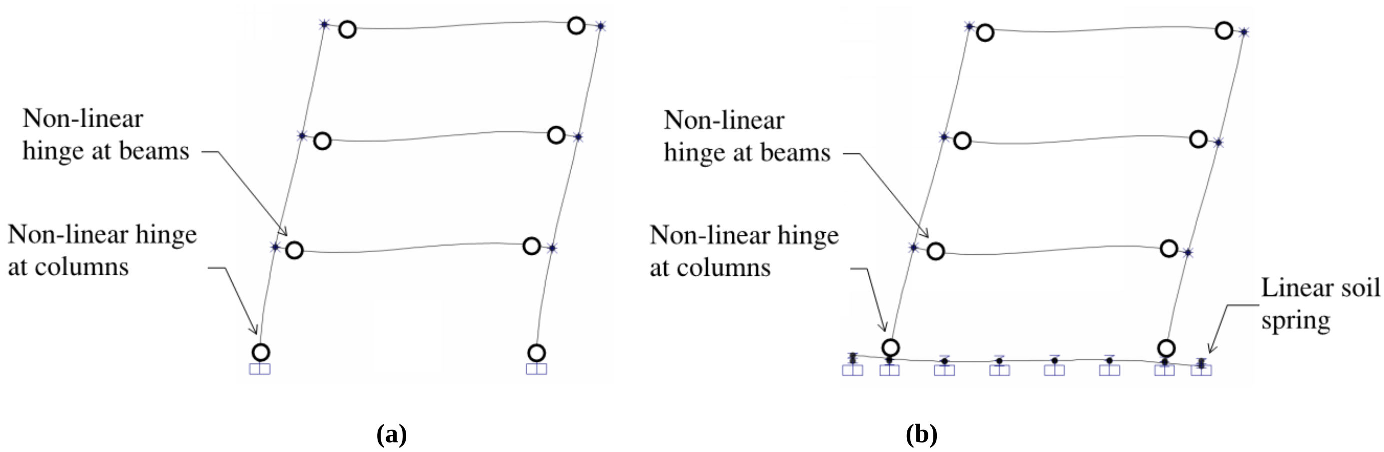


Fig. 6 – Expected plastic mechanism for a) conventional fixed-base frame and b) rocking frame

The nonlinear hinges at beam ends and column bases were modeled following the performance criteria defined in ASCE/SEI 41 [9]. Fig. 7 shows the schematic moment-rotation diagram for this behavior. Outside the discrete plastic hinge regions, offset a distance of d/2 from the beam-column joints, linear-elastic frame elements were used. As previously mentioned, flexural restraint and a plastic hinge were defined at the base of each column.

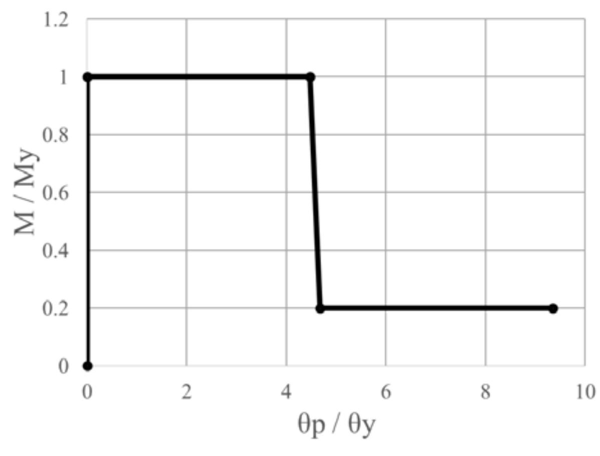


Fig. 7 – Nonlinear hinge model for beam ends and column bases

3.1 Analysis results

Three main response quantities were investigated in this study for each building: peak roof displacement, peak roof acceleration, and peak base shear demand. A ratio of the rocking response divided by the fixed-base response are presented for each response estimate at each building site. Discussion of analysis results for each response quantity and all 27 building configurations is summarized in the following sections.

3.1.1 Peak roof displacement

For the San Francisco (S_{DS}=1.0) and Los Angeles (S_{DS}=1.3) sites, the peak roof displacement was generally higher for the rocking frames, particularly the tall, single-bay frames, in comparison to the corresponding fixed-base buildings. For these 18 buildings, the ratio of rocking to fixed-base displacements ranged from 93% to 176%. This is primarily due to the contribution of rocking frames rigid body rotation to the peak roof drift, and not necessarily due to internal flexural deformations of the frames resulting in structural damage. Conversely, for the shorter 3- and 6-story building frames, rocking behavior often resulted in a decrease of maximum roof drifts. Fig. 8 provides a graphical representation of the rocking-to-fixed-base roof displacement for all 27 building configurations.

For the Sacramento site (S_{DS}=0.5) very small differences were observed between the rocking response and the fixed-base response. This result was expected since ground motion excitation was insufficient to cause foundation uplift. For all nine Sacramento building configurations, the ratio of the rocking response to the fixed-base response ranged from 98% to 108%.

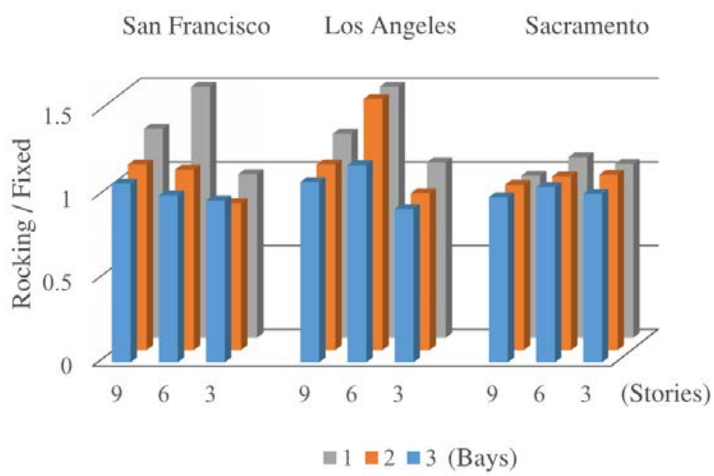


Fig. 8 – Roof displacement ratio for rocking vs. fixed-base response

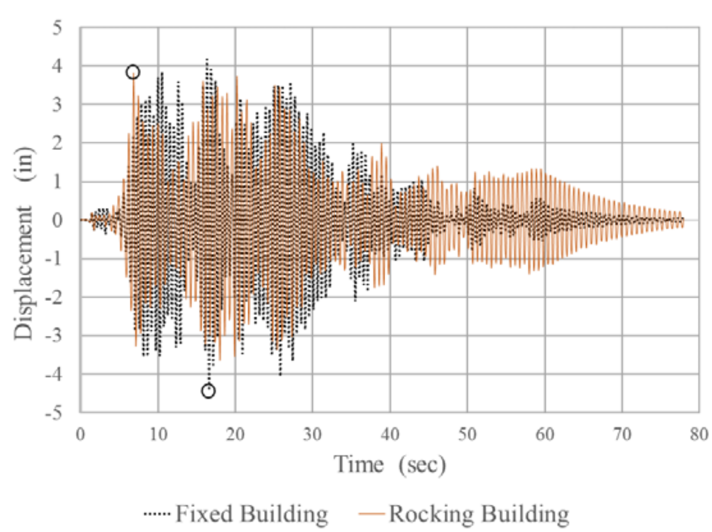


Fig. 9 – Roof displacement response history for the 3-story, 2-bay frame at the San Francisco site (RSN#6)

Fig. 9 above demonstrates the resulting reduction in peak roof displacements due to rocking motion through a comparison of nonlinear response history analysis results. Clearly, since foundation uplift changes the dynamic characteristics of the moment-resisting frames, the peak response of the two systems compared (i.e., rocking and fixed-based) occur at different times of the ground motion, as shown in Fig. 9, and may actually result in amplified displacement demands for an individual record. However, if properly designed, rocking motion can ultimately help reduce peak and residual drifts, as well as internal deformation demands on the lateral force-resisting system.

3.1.2 Peak roof acceleration

For the San Francisco and Los Angeles sites, the roof acceleration ratios varied based on the building height. For these 18 buildings, the rocking-to-fixed-base ratio ranged from 66% to 115%. In general, rocking reduced the peak roof acceleration for short buildings, but not for tall buildings. Again, for the Sacramento site, there was very little difference between the rocking and the fixed-base response. For these nine buildings, the rocking-to-fixed-base ratio ranged from 89% to 100%. Fig. 10 shows a graphical representation of the Rocking/Fixed ratio for roof acceleration for all 27 buildings.

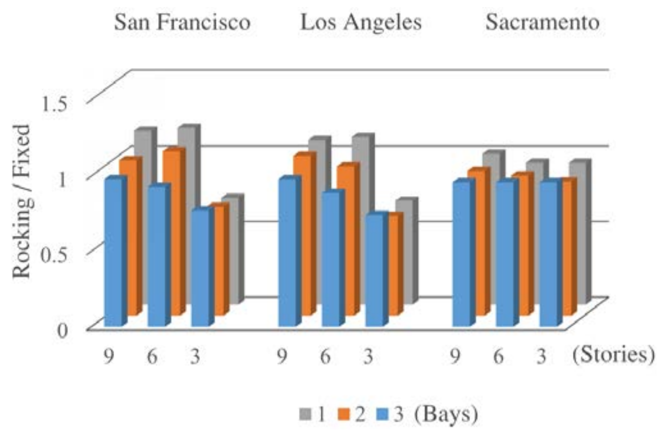


Fig. 10 – Roof acceleration ratio for rocking vs. fixed-base response

A higher acceleration for the rocking buildings at the first level was observed for the buildings in the relatively higher seismic areas, in comparison to those at the Sacramento site. For the buildings in San Francisco and Los Angeles an average of 30% and 40% higher accelerations were observed, respectively, compared to the buildings at the Sacramento site. This observation is especially important for anchorage design of nonstructural components. Typical practice in building design consists of using either pinned or fixed base supports, leading to smaller accelerations and force demands than what the building will experience when rocking is considered. Fig. 11 below demonstrates the resulting reduction in peak roof acceleration due to rocking motion through a comparison of nonlinear response history analysis results.

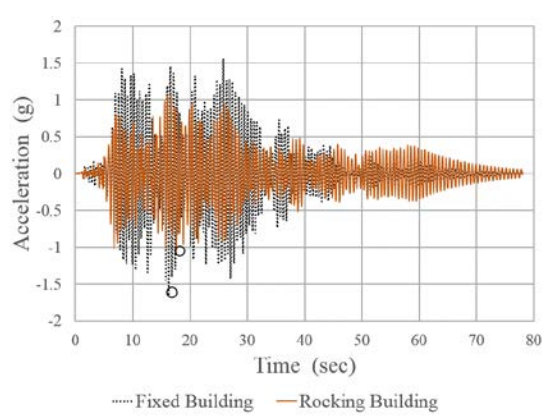


Fig. 11 – Roof acceleration response history for the 3-story, 2-bay frame at the San Francisco site (RSN#6)

3.1.3 Peak base shear

For the fixed-base structures, the peak base shear demand was much greater than the design base shear. This is because the frame design was controlled by drift requirements, and the resulting frames are stronger and stiffer than that required through the use of an inelastic response modification factor, R of 8 for Special Moment Resisting Frames per ASCE/SEI 7 [5].

For the Los Angeles and San Francisco sites, rocking behavior reduced the base shear demand, particularly for shorter buildings. For these 18 buildings, the Rocking/Fixed ratio ranged from 57% to 98%. For the nine buildings in Sacramento, there was a small difference between the rocking response and the fixed base response. For these nine buildings, the Rocking/Fixed ratio ranged from 90% to 103%. Fig. 12 shows a graphical representation of the Rocking/Fixed ratio for base shear for all 27 buildings.

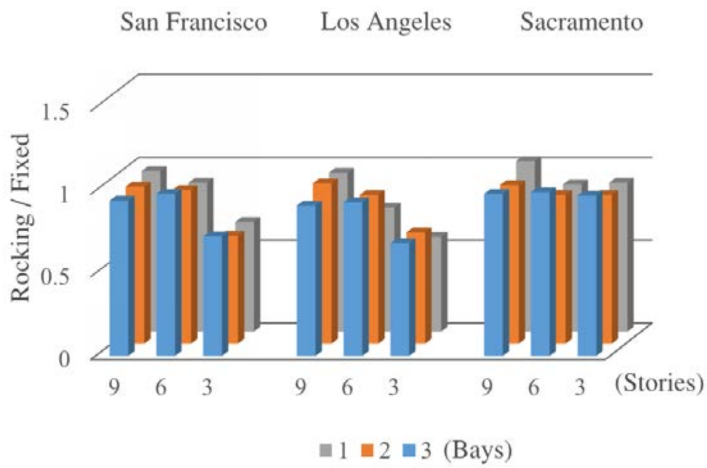


Fig. 12 – Base shear ratio for rocking vs. fixed-base response

Fig. 13 shows the base shear response history for the fixed-base and rocking frames. For this particular record, the rocking base shear demand reached approximately 600 kips at multiple time steps, but never exceeded this value. This is because the base shear demand cannot exceed the gravity resisting moment divided by the effective height of the applied lateral load. Although the effective height can vary depending on which vibrational modes are excited, this represents an upper bound on the base shear.

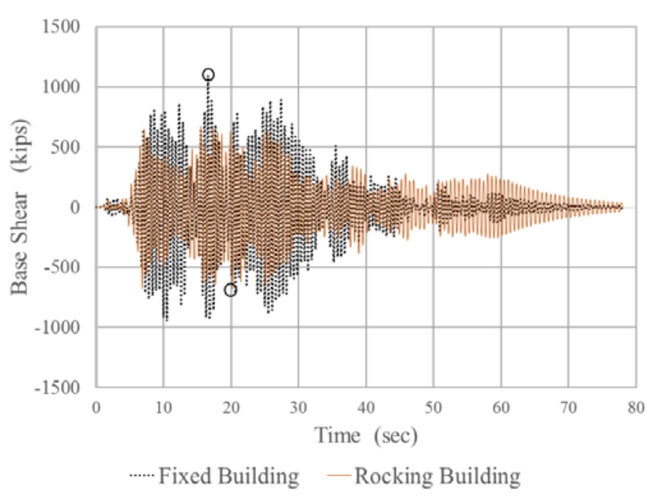


Fig. 13 – Base shear response history for the 3-story, 2-bay frame at the San Francisco site (RSN#6)

4. Conclusions

Conventional structural design practice involves treating spread footings and deep foundations similarly when performing seismic analysis of a building structure. Our study indicates that this approach is inaccurate for regions of higher seismicity. Rocking behavior can have an adverse effect on building displacement and floor accelerations for taller buildings. However, for shorter buildings, the roof displacement and floor accelerations are reduced. Rocking can provide a significant benefit by reducing maximum base shear demand. This can have significant implications for foundation design and seismic retrofit design. For buildings in high seismic zones, an average of 30% to 40% higher accelerations were observed for the rocking buildings in the first level, in comparison to fixed base buildings. This indicates that buildings designed using conventional fixed-base procedures might experience higher accelerations at the first level in actual earthquakes than building codes would imply. Rocking behavior is likely more significant for stiffer lateral systems, such as braced frames and concrete shear walls. Further investigation is required to assess these two structural systems. The results of this study, which employed elastic springs for the foundation, indicate that rocking behavior is mostly beneficial for shorter structures. Nonlinear soil behavior supporting the foundation system would tend to decrease further the response in rocking buildings, and may prove it is favorable for a wider range of building structures.

5. Acknowledgements

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