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PERFORMANCE ASSESSMENT OF PINNED ROCKING CORE WALLS IN STEEL MULTI-STORY BUILDINGS SUBJECTED TO STRONG SEISMIC GROUND MOTIONS

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Abstract

It is known that design guidelines within current building codes can ensure adequate safety levels against severe seismic events by promoting energy dissipation through large inelastic deformations and residual drifts. Nevertheless, this ductility-based design philosophy often results in extensive reparations that are neither financially sustainable, nor able to secure the post-earthquake serviceability of the structure. Hence, several strategies have been proposed so far to mitigate potential structural and nonstructural damage in multi-story buildings under strong seismic events. Particularly, the low-damage philosophy is based on the design of high-performing, cost-effective structural systems that can withstand large seismic ground motion intensities with minimal damage and low socio-economic losses. This design methodology leverages on the installation of innovative damage mitigation technologies to improve the seismic performance of structural and nonstructural elements, and it is suitable for the design of new constructions as well as for the retrofitting of existing buildings. Within this framework, the present study investigates the behavior of an inner braced rocking core as a low-damage design solution for mitigating the seismic risk of a multi-story steel frame. The pinned rocking wall system is simulated by including viscous dampers mounted between the column bases and the foundation, whereas its self-centering capabilities basically rely on the self-weight of the structure. Nonlinear models with both fixed- and rocking-base core wall conditions subjected to a large set of seismic ground motion records are analyzed in order to properly appraise the potential benefits of the rocking structure on the seismic performance of the building.

Keywords: Rocking systems, steel rocking core, dampers, low damage structures.

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1 INTRODUCTION

Modern code-compliant structures, designed according to performance-based objectives, are expected to exhibit large energy dissipation capacity upon the occurrence of severe seismic events. In the case of earthquake-resistant Moment Resisting Frames (MRFs), the capacity design promotes the concentration of damage in dedicated ductile regions, at the beam ends and in the column bases, properly selected to sustain significant inelastic deformations. This design philosophy typically ensures satisfactory safety levels by avoiding catastrophic failures under severe events, while accepting considerable cumulative damage in the main structural members and residual deformations. As a result, damaged elements require costly repairs or even replacements, which impose challenges in the post-earthquake rehabilitation with disruption of the building functionality. To alleviate these drawbacks, alternative solutions have been recently proposed for the mitigation of the structural and nonstructural damage in multi-story buildings while ensuring high dissipative behavior.

From the perspective of the low-damage design philosophy, one strategy consists in allowing rocking to develop at the base of structures or portions thereof under severe ground motion excitations. In MRF structures equipped with rocking systems, inelastic deformations are concentrated in replaceable ductile fuses while self-centering capability maintains the building plumb and mitigates residual drifts. Additionally, this might represent a sustainable design strategy as the resulting structural systems would suffer little or no damage during severe excitations with modest post-event repair supplies compared to typical fixed-base structures.

Since the seminal work of Housner [1], the growing interest in rocking systems has encouraged the experimental and analytical studies conducted so far to improve the seismic performance of both new constructions and retrofitted buildings. For instance, Wada et al. [2] developed a retrofitting system of prestressed concrete rocking walls and steel dampers to control the seismic damage mode and increase the strength and energy dissipation capacity of steel reinforced concrete frames.

Successful applications have been also performed by coupling rocking systems with different types of structures, such as bridges [3,4], concrete frames [5,6], steel braced frames [7,8], timber frames [9] or by combining them with embedded devices [10,11,12].

The behavior of an inner braced rocking core as a low-damage design solution for mitigating the seismic risk of a multi-story steel frame is investigated in Mollaioli et al. [13] where the potential of the base-rocking system under a large set of ground motions is assessed. Although an improvement of the seismic performance was observed on average, the results emphasized the need for additional lateral stiffening to prevent damage as well as residual deformations within the structure, especially in the case of near-fault pulse-like records. The recognition of the higher seismic risk to which structural systems equipped with protection devices can be exposed due to the impulsive nature of the ground motion near the causative fault has motivated significant surveys, such as in the case of tuned mass dampers [14,15], viscous or hysteretic dampers [16,17], and isolation bearings [18,19].

Within this framework, the present study primarily focuses on the behavior of a multi-story steel frame with braced rocking core subjected to a selected set of impulsive ground motions to provide a better recognition of the seismic risk on rocking systems close to active faults. The seismic response of the integrated system is compared to that of the fixed-base building to deliver a comprehensive understanding of the extent to which pulse-like seismic records affect the structural performance.

2 NUMERICAL MODEL

The case study is a 10-story structure with a total height of 41m (Figure 1). A constant interstorey of 4m is assumed, except for the first floor which is 5m. The plan dimensions of the building are 21mx28m. The structure consists of a steel frame with an inner braced core. Bracing members are placed every two floors and designed as inverted-V types (i.e. chevron frames). Applied vertical loads are 4.5 kN/m^2 for dead load, 3.0 kN/m^2 for live load and 3.0 kN/m^2 for the accidental load considering a residential building. In the last floor, the dead load is reduced to 3.5 kN/m^2. The materials used are steel S275 for bracings and S355 for beams and columns. For a detailed description of the cross-sections designed for the structural members, the reader is referred to Mollaioli et al. [13].

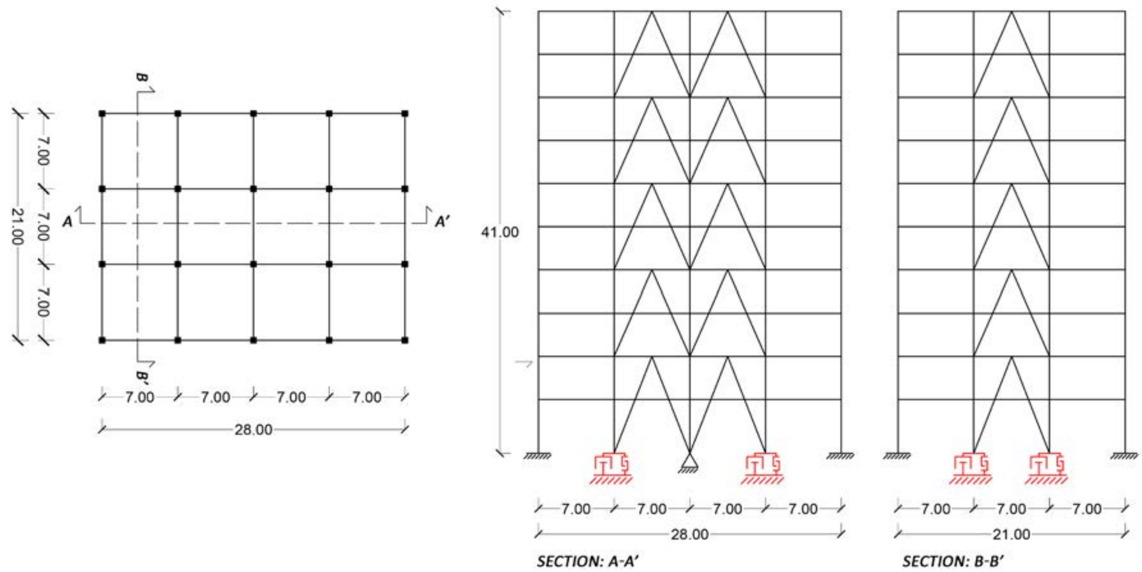

Figure 1: Rocking model dimensions: plan and sections.

Based on this reference building, two numerical models are developed using SAP2000 program [20]. The rocking behavior is properly simulated by combining viscous dampers, gap elements and base hinges as shown in Figure 1 and Figure 2. Specifically, the nonlinear viscous dampers are equipped with gap links and are placed between the foundations and the base of the four corner columns of the core. In agreement with the Maxwell's model of viscoelasticity, the nonlinear viscous damper component has three main nonlinear properties: stiffness, damping coefficient and damping exponent; here respectively set to 3x10^6 kNm^2; 130 kNm and 0.22. Tension-only Hook links are not needed as the recentering of the structure is guaranteed by the building self-weight.

On the other hand, rocking is entrusted by placing pinned connections at the base of the two central columns of the core. Because the rocking behavior could cause damage to the joints connecting the perimeter frame with the inner core, these nodes are modeled as internal hinges in order to exhibit elastic behavior at least for design earthquakes.

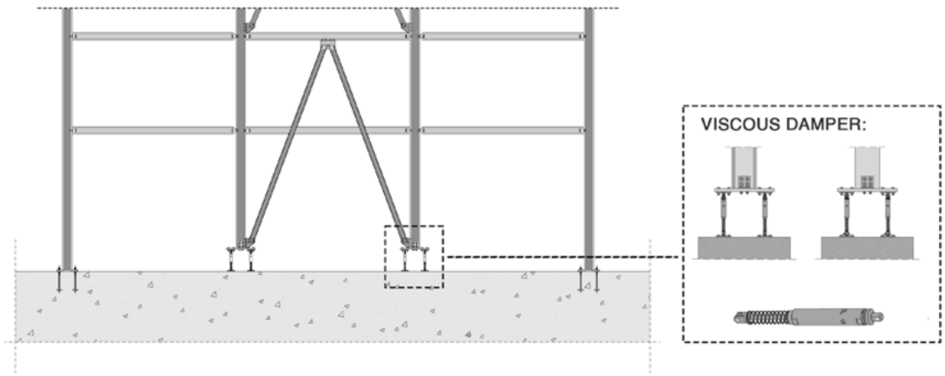

Figure 2: Schematics of the rocking core model.

The modal properties of the fixed base and rocking models are provided in Table 1. The introduction of the rocking system induces an elongation of the first natural vibration period of the structure by 2s, i.e. from 2.04s in the fixed base model to 4.14s in the rocking model. The change of the dynamic behavior with resulting shift of the natural period has a major effect in reducing the seismic base shear forces experienced by the structure due to the abrupt drop of the spectral accelerations.

PERIOD	FIXED BASE MODEL [FM]			ROCKING MODEL [RM]		
Mode	T [s]	Modal direction	Modal mass ratio [%]	T [s]	Modal direction	Modal mass ratio [%]
1	2.045	T	80	4.141	X	77
2	1.721	Y	78	2.146	Y	78
3	1.474	X	79	1.898	T	80

Table 1: Modal properties of the fixed base and rocking models.

3 NEAR FAULT PULSE-LIKE GROUND MOTIONS

It is widely recognized that structures situated near causative seismic faults can be exposed to strong ground motions characterized by a distinctive pulse-like component in the fault-normal velocity waveform. In these cases, most of the damage experienced by the structural systems is attributable to the high energy levels imparted by the dominant high-amplitude component. Motivated by this scenario, past studies have been devoted to characterizing the main features of pulse-like seismic ground motions [22] as well as to estimate the corresponding pulse periods [23,24].

Moreover, previous findings on MRF structures coupled to oscillating cores have revealed that the impulsive characteristics of the ground motions can increase the displacement demands, potentially anticipating structural damage [13]. With the aim of better understanding the difference between the base-rocking model and the fixed base structure under earthquakes, nonlinear time history analyses are performed considering a suitable collection of twenty-one worldwide near-fault records. The selected ground motions are listed in Table 3 and are grouped based on their magnitude, depending on whether it is greater than 7.0 or comprised between 5.0 and 7.0.

Earthquake	Year	Station	Record ID	M	EPID (Km)	PGA [m/s2]	Duration [s]	T_p [s]	T_{FM}/T_p	T_{RM}/T_p
Wenchuan, China	2008	Deyangbaima	FN134	7.90	111.42	7.37	119.36	7.22	0.28	0.57
Chi-Chi, Taiwan	1999	CHY101	FN069	7.62	31.96	4. 43	90.00	4.59	0.44	0.90
Chi-Chi, Taiwan	1999	TCU052	FN081	7.62	39.58	3. 83	90.00	7.91	0.26	0.52
Chi-Chi, Taiwan	1999	TCU065	FN090	7.62	26.67	8.06	90.00	4.84	0.42	0.86
Kocaeli, Turkey	1999	Duzce	FN062	7.51	98.22	6. 97	48.13	4.77	0.43	0.87
Landers	1992	Lucerne	FN042	7.28	44.02	2. 78	27.19	6.71	0.30	0.62
Duzce, Turkey	1999	Bolu	FN111	7.14	41.27	7.67	55.90	5.87	0.35	0.71
Cape Mendocino	1992	Cape Mendocino	FN040	7.01	10.36	12.44	30.00	2.44	0.84	1.70
Darfield, NZ	2010	DSLC	FN152	7.00	13.41	2.72	160.02	6.70	0.30	0.62

Table 1: Database of Near Fault records with M>7.0.

Earthquake	Year	Station	Record ID	M	EPID [Km]	PGA [m/s^2]	Duration [s]	T_p [s]	T_{FM}/T_p	T_{RM}/T_p
Loma Prieta	1989	LGPC	FN036	6.93	18.46	9.26	25.005	3.85	0.53	1.08
Irpinia, Italy-01	1980	Sturno	FN017	6.90	30.35	2.27	39.34	3.46	0.59	1.20
Kobe, Japan	1995	Port Island (0 m)	FN057	6.90	19.25	3.76	42.00	4.37	0.47	0.95
Kobe, Japan	1995	Takatori	FN059	6.90	13.12	6.69	40.96	2.13	0.96	1.94
Chuetsu-oki	2007	Joetsu Otemachi	FN143	6.80	58.14	0.95	120.00	6.85	0.30	0.60
0Northridge-01	1994	Rinaldi Receiving	FN051	6.69	10.91	8.53	19.91	1.61	1.27	2.57
Bam, Iran	2003	Bam	FN114	6.60	12.59	8.33	66.55	1.82	1.12	2.27
Imperial Valley-06	1979	EC Meloland Overpass FF	FN006	6.53	19.44	3.70	39.98	2.80	0.73	1.48
L'Aquila, Italy	2009	L'Aquila castello	FN133	6.30	2.20	1.627	90.01	1.35	1.51	3.07
Christchurch, NZ	2011	CHHC	FN165	6.20	7.35	4.43	69.74	3.61	0.57	1.15
Coalinga-05	1983	Transmitter Hill	FN021	5.77	5.99	8.43	21.76	1.77	1.15	2.34
Yountville	2000	Napa Fire Station	FN113	5.00	9.89	5.89	72.00	0.75	2.72	5.52

Table 2: Database of Near Fault records with 5.0<M<7.0.

Figure 3 shows the acceleration spectra of the considered records with indications of the first natural periods of the fixed-base model (T_{FM} = 2.045s, red dotted line) and that of the rocking structure (T_{RM} = 4.141s, blue dotted line). Specifically, a significant reduction is shown at the fundamental period of the structure with rocking core, except for sporadic cases such as the Landers (FN042) and Loma Prieta (FN036) seismic records that reveal a second peak. Another noteworthy case is the Cape Mendocino seismic record (FN040) where high values of the spectral acceleration for the fixed-base structure are accompanied by an abrupt drop for the rocking structure.

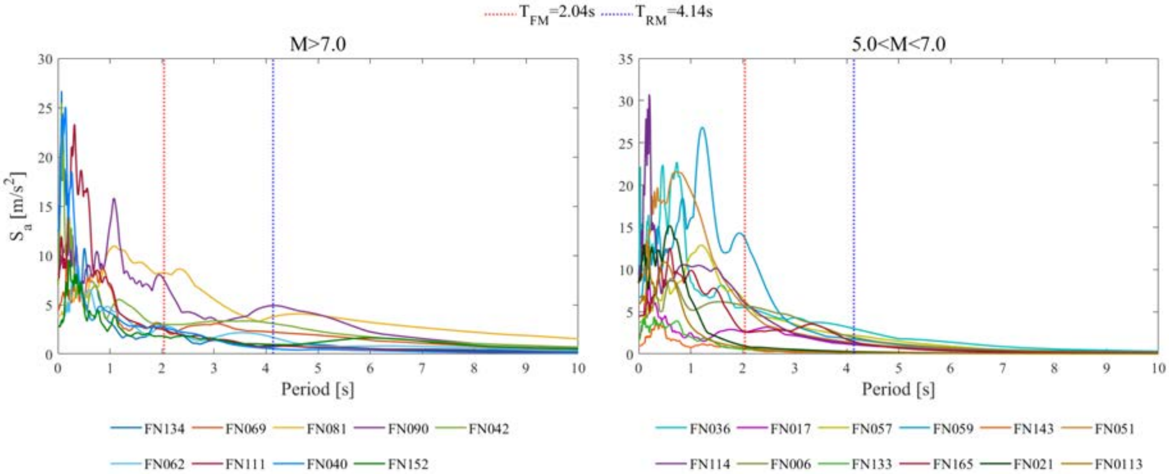

Figure 3: Elastic acceleration spectra: M>7.0 (on the left) and 5.0<M<7.0 (on the right).

Since the impulsive component in near fault records is usually detected into the velocity time history, the energy imparted to a structural system likely represents the most relevant indicator in determining the corresponding damage potential associated with seismic loads in near-fault areas. Therefore, an energy-based standpoint is generally deemed to be more rational for assessing the seismic demand associated with the pulse-like shaking. The energy spectra in Figure 4 show that the relative and absolute energy contributions have similar trends and values, or even overlap in some cases (e.g., FN090). In other cases, the relative energy can have lower values but the same trend (e.g., FN081 seismic record).

On the other hand, ground motions with magnitude ranging between 5.0 and 7.0 exhibit lower energy values absolute and relative, with the only exception of the Kobe earthquake (FN059). The accelerogram concentrates the higher energy values especially between the period 1 and 2s, moreover the fig.4 right demonstrates how the second peak is precisely at the first period of vibration of the fixed base structure.

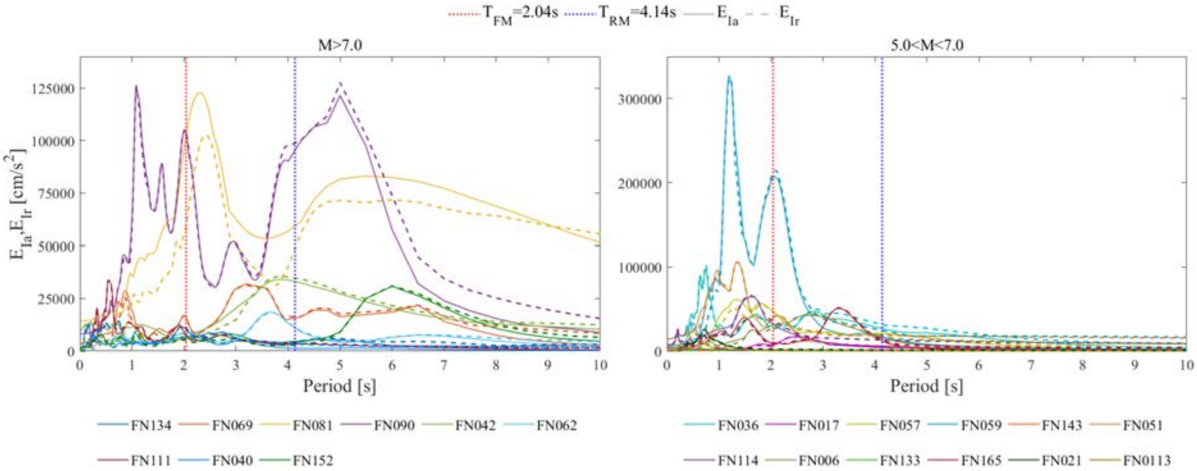

Figure 4: Input Energy Spectra: M>7.0 (on the left) and 5.0<M<7.0 (on the right).

4 RESULTS

Nonlinear dynamic response analyses are performed considering the selection of ground motion records described so far to properly appraise the seismic performance of the fixed-base and the rocking structures under different impulsive characteristics of the earthquake excitation.

Figure 5 shows the results of the Peak Relative Acceleration (PRA) and Peak Roof Displacement (PRD) for a roof control node of the structure. It is clearly noted that the duration and frequency content of the ground motions have a great influence on the structural response. The results obtained for seismic records with magnitude greater than 7 show an increase of the relative acceleration values, especially in x-direction. The relative accelerations for ground motions with magnitude between 5 and 7, show instead higher values both in x and y directions.

The roof drift decreases in x direction while grows in y-direction for most of the seismic records. This may be due to the asymmetry of the structure: the structure is more flexible in the y direction with an increase of the roof displacements in most records regardless of the magnitude considered.

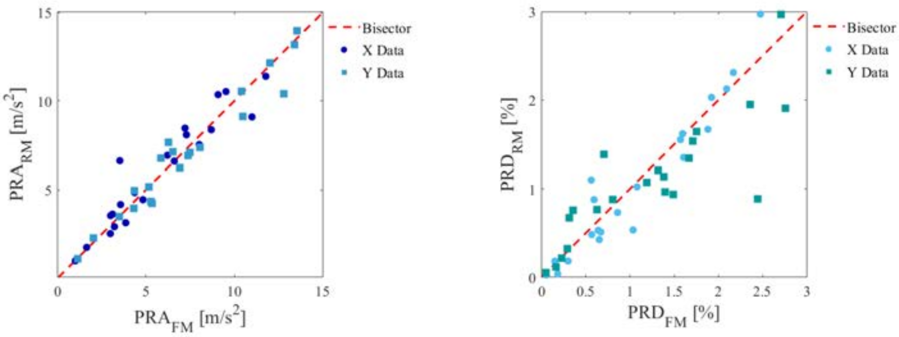

Figure 5: (a) Peak relative acceleration (on the left) and (b) Peak roof displacement (on the right).

The distribution of the peak inter-story drifts over the height of the building under the individual records is plotted in Figure 6 for both the fixed-base (FM) and the rocking (RM) models and it is compared against a limit ratio of 2.0% (red dotted lines).

As illustrated, most pulse-like ground motions with M<7 trigger uniform peak drift ratios comprised within the threshold. Few exceptions are especially apparent in y-direction where a maximum drift of 4.22% is attained for the Kobe earthquake (FN059). At the same time, it's shown how the rocking structure allows to decrease the IDR values which is 3,49%.

On the other hand, the outcomes reveal that the maximum inter-story drift demands tend to migrate toward the bottom stories of the structure (i.e. 2nd and the 3rd floors) and exceed the threshold of 2.0% as the severity of the ground motion increases (M>7). In this latter case, the maximum drifts are attained for the FN090 and FN081 records in x- and y-direction, respectively, both corresponding to the Chi-Chi earthquake.

Although the introduction of a rocking system does not always reduce both roof and inter-story displacement values rather than a fixed base structure, since it is a more flexible structure; it helps to decrease the dispersion of deformation between the stories ensuring a more uniform behavior and minimizing the probability of collapse of a soft story [25]. This statement is also in line with the outcomes provided in Mollaioli et al. [13] where peak drift ratios under near-field records were generally larger than those under far-field ground motions.

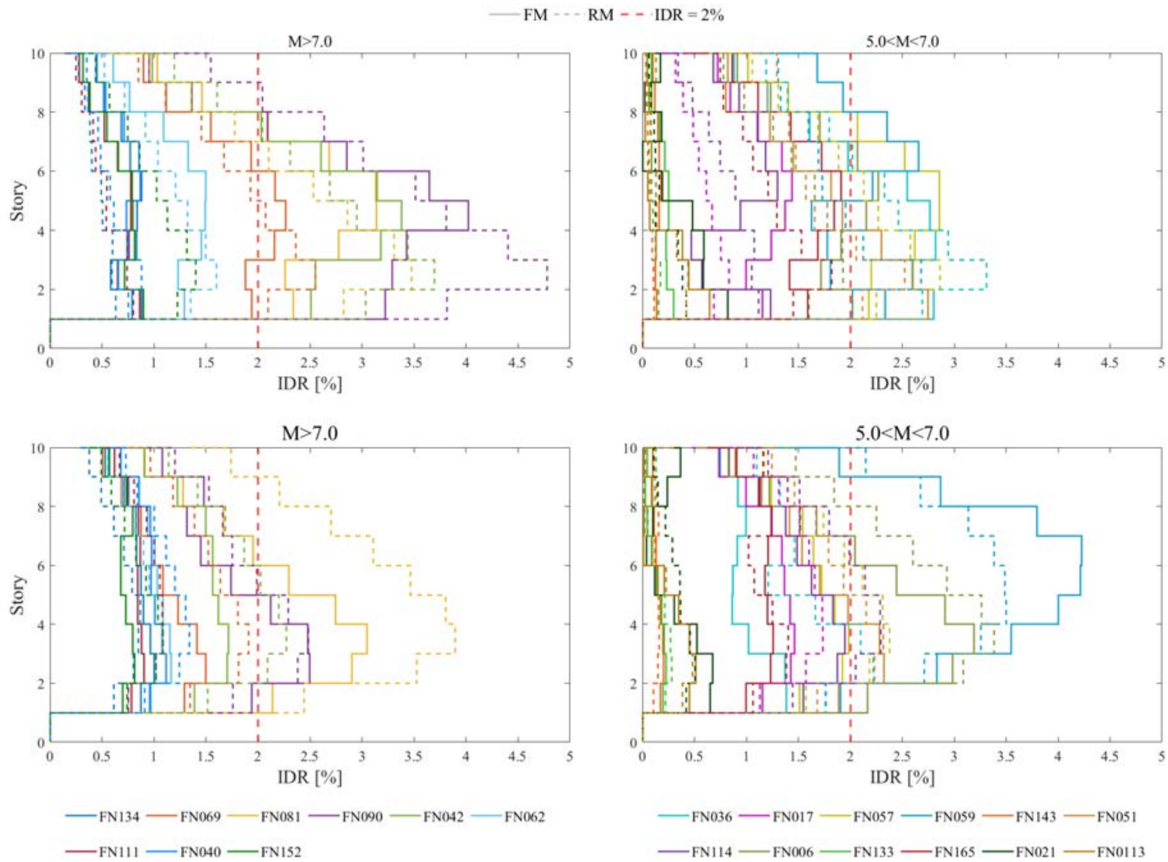

Figure 6: Interstory drift ratio: M>7.0 (on the left) and 5.0<M<7.0 (on the right); x-direction (top) and y-direction (bottom).

Even if higher displacement demands are observed in some events, the pinned core is still effective in reducing the seismic loads experienced by the MRF structure.

As shown in Figure 7 and Figure 8, in fact, the introduction of a rocking system provides a significant drop of the base shear and the overturning moment with maximum reduction ranging between 40% and 70% for x-direction and 16% and 55% for y-direction. It is also observed that the rocking structure is especially beneficial under near fault pulse-like records, which could lead to very large local ductility demands at the base and pose a collapse risk in the case of fixed-base structures. Even in this case in fact, the rocking structure continues to exhibit elastic recentering behavior.

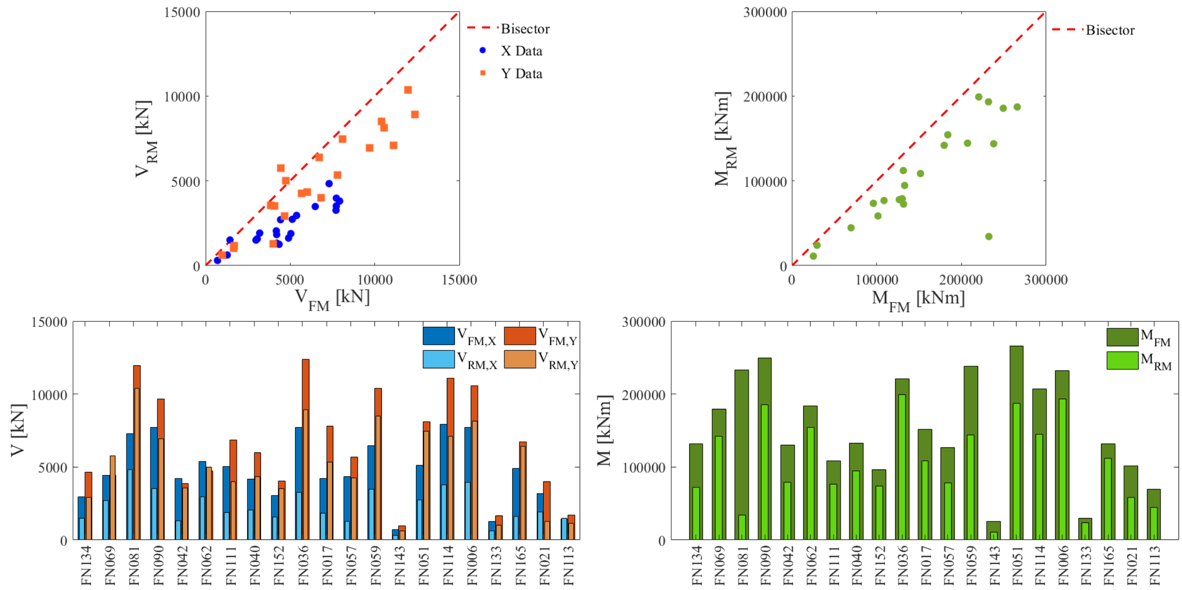

Figure 7: (a) Base shear forces (on the left) and (b) base overturning moment (on the right).

The input energy values in Figure 8(a) demonstrates that there is an increment for the rocking model along the x direction when M>7.0 while there is a reduction along the y direction in almost all the accelerograms for 5.0<M<7.0.

Figure 8(b) shows the comparison between the hysteretic energy $E_{H,FM}$ of the fixed base model together and the dissipated energy $E_{HD,RM}$, i.e. the sum of the hysteretic energy (E_H) and the energy dissipated by the viscous dampers of the rocking structure (E_D). The results emphasize that in most cases the energy dissipated by the structure with inner rocking core is higher.

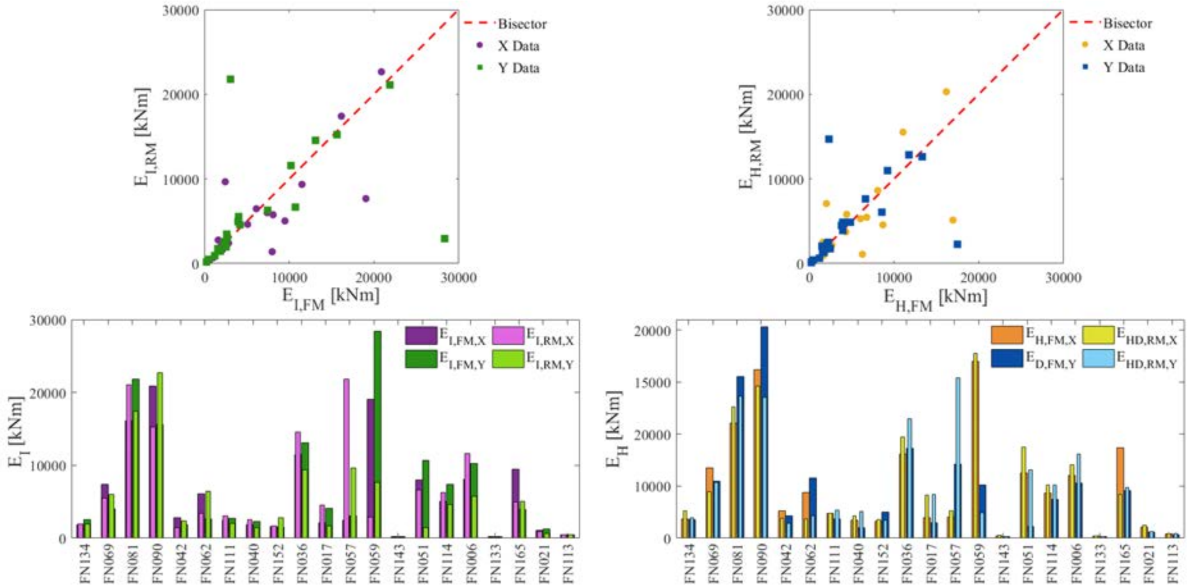

Figure 8: (a) Input energy (on the left) and (b) hysteretic energy (on the right).

5 CONCLUSIONS

Finding low-damage solutions for both new and existing buildings is currently a key issue. In this context, the objective of this work aims at demonstrating the efficacy of the rocking base structures in mitigating the seismic risk of new steel buildings subjected to near-fault pulse-like ground motions. The increase of the natural period of the rocking structure with respect to the fixed base model conveniently determines a shift in the elastic response spectra towards smaller values with consequent reduction of the seismic forces acting on the structure. In addition, by reducing the stiffness at the base of the structure, the rocking system also contributes in limiting the base shear forces as well as the overturning moments. No residual deformations are observed due to the re-centering capabilities of the system in recovering the original position. Finally, it is observed that the introduction of the rocking core positively concurs in uniforming the inter-story drifts along the height of the building, greatly reducing potential story deformations as well as the formation of weak stories, which can cause severe damage or even collapse in traditional structures.

The findings presented here should be considered as a first attempt toward the assessment of the structural performance of structural systems equipped with pinned-base cores. Further extensions will be included in future works to completely describe the behavior of rocking systems located in near-fault areas, for which objective a larger dataset of pulse-like ground motions will be ultimately considered.

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