SEISMIC PERFORMANCE OF ROCKING BRACED FRAME EQUIPPED WITH RESILIENT SLIP FRICTION JOINT AS A SHEAR KEY

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Abstract

Since ancient time, when structures were engineered, the rocking mechanism, including rocking columns or rocking walls was one of the applicable structural systems. However, this mechanism at first place was considered to be a gravity system, but surviving over the years during various natural hazards (hurricane, tornado, earthquake) has revealed their acceptable effectiveness as a main lateral system as well. In recent years, inspired by this traditional concept and in an effort to achieve a low damage lateral system, the rocking motion has been combined with one of the dissipation energy mechanisms and also hold down systems to reach a stable and reliable seismic performance. The two of the most common approaches are employing pre-tension cables as a system retainer along with yielding or friction mechanism or also mounting a self-centring system like RSFJ as a hold down in the rocking toes. Resilient Slip Friction Joint (RSFJ) is a self-centring friction damper which, contrary to the other passive controlled systems not only dissipates the input energy but also is able to restore structure to its original position.

In this paper, to achieve a self-centring damage avoidance rocking framed system, RSFJ has been employed as a shear link between braced frame and its boundary columns. Such a system could be used for a single or coupled braced frame. This system could be adopted as an alternative to conventional rocking systems to reduce their design's complexity and implementation challenges. Covering taller rocking systems is another advantage of this configuration. In this study first, this mechanism is introduced, and its performance has been assessed, then to evaluate the seismic performance of the introduced system the result has been compared with a braced frame with LRB isolation system placed the ground level. For the case study, a seven-story prototype building equipped with a conventional special braced frame and both isolation concepts have been analysed. As the results demonstrated, the proposed rocking system could provide structures with a fully self-centring low damage lateral system, which is a crucial factor to evaluate the required time and cost for building rehabilitation and also the vulnerability of structures against severe aftershocks. Also, as both base isolating and rocking systems rely on shifting the effective mode to decline the transmitted forces, compare to the braced frame, the induced overturning moment placed a lower rate. However, as the rocking systems are under the influence of higher mode effect, to improve its efficiency the impact of having multiple rocking levels also have been studied.

Keywords: RSFJ, Resilient Slip Friction Joint, Self-centring, Rocking braced frame, Energy dissipation, Low damage design

1. Introduction

Some of the survived ancient structure in Greek, Iran and other seismic prone areas have been tolerated the lateral force including earthquake and wind during their life relying on the rocking mechanism. Might be claimed that the mechanism was not designed intentionally but what is undeniable is that they could resist and also return to their position after subjecting seismic forces by this kind of mechanism. Temples in Greece or some slender structure in Chile are a convincible example of this kind of structures [1]. Muto et al. [2] had done the first analysis of rigid rocking body explained the rocking mechanism. He conducted a small-scale experimental test subjected to ground excitation and theoretically derived the relation of rocking of a rigid body. He concluded a rocking body resists the lateral static load till the center of mass reach to the rocking toe.

However, the concept of rocking structures as a lateral resisting system which mitigate the seismic demand was initially introduced by Housner [3]. He reported his observation regarding the less amount of damage in flexible slender structures compares to stiffer structures as a result of ductility of such systems. Cough et al. [4] tested a half-scale three-story steel rocking moment frame. This result was compared with the fix base situation, and the results showed a reduction in members force and also stories acceleration. Displacement reduction was a concern of such a system, so Kelly et al. [5] consider a yielding type damper in the base of the same structure. The designed passive system activated during the uplift phase and dissipated energy by material nonlinearity. As expected, results represented less amount of deflection demand while base shear was at the same level to the rocking system without those fuse elements. Midorikawa et al. [6] designed a yieldable base plate to dissipate energy in the rocking frame for a half-scale three-story building. They tested a range of thickness for base plates, and as expected the thicker base plate, the more force reduction and more displacement demand reported. Roke et al. [7] conducted a controlled rocking steel frame (CRSBFs) with post-tensioning tendons and supplementary energy dissipation in both sided of the wall attached to boundary columns. In fact, by such an approach, the rocking system was separated from gravity columns. Roke based on the result provided a procedure for designing CRSBF system using quasi-static method while considering the effect of higher modes. For a case study of six-story frames designed base on just firs mode, time history analysis showed 3.6 times more axial force demand in the bracing system. The simplified method proposed by Roke et al. [7] estimated the force design more than this demand.

The University of Illinois at Urbana-Champaign (Eatherton, Hajjar et al. [8]) had investigated the performance of CRSBFs with post-tensioned cables to have the self-centring force and yielding damper to dissipate energy. The system was designed to have fuse just in damper at roof drift of 3% while the test pushed to the 4%. However, the PT system experienced failure at strain as low as 0.85%, even though component tests of strains showed elongation greater than 4.5%. For the next phase, the strain capacity of strands improved by 1.3%. The structure proved could remain self-centred up to 2500-year seismic event while the post-tensioning yielded remarkably at this amount (3.9% roof drift) (Eatherton and Hajjar [9]). Wiebe [10] conducted a test setup of a 30% scale of an eight-story post-tensioned frame. In this research, he proposed two approaches to mitigate higher mode effect: 1- multiple rocking joints; 2- shear control brace, which he considered nonlinear brace at the first story. No significant structural damage was observed in more than 300 dynamic tests, many of them beyond the 2500-year level. He reported a higher rate of force in member to compare to push over analysis due to higher mode effect. The outcome of this research was used in the New Zealand design guide for CRSBF [11] systems which proposed the ductility factor of four for the load-bearing system and six non-load bearing systems. Zarnani and Quenneville [12] introduced a new generation of friction damper which provides restoring force and energy dissipation combined in one compact joint. Such resilient slip friction joint (RSFJ) was initially studied in a rocking timber wall application as a hold-down (Hashemi et al. [13]), which later was employed in the different application including tension only brace (H. Bagheri et al [14]) and also in a practical project (Nelson airport terminal, NZ). Darani et al. [15] extended this concept to rocking concrete shear walls as a with RSFJ hold-downs. Sahami et al. [16] introduce the idea of rocking wall with self-centring shear keys (rotational-RSFJs). Less possibility of the creating high-stress zone in single and also couple shear walls and reducing the higher effect were advantages of the introduced system which make it a more facilitate to be employed in taller rocking buildings. In this study this idea has been extended to braced frame rocking system and the results has been compared with the other concept of isolating structure (LRB isolation).

2. Resilient Slip Friction Joint (RSFJ)

In RSFJ restoring force comes from a specific steel grooved plates which are tied through high strength bolts and disk springs. By slipping of grooved plates, the input energy is dissipated through frictional resistance. Based on the free-body diagrams presented in Fig. 1, the design procedure is developed for the prediction of the performance of the RSF joint [13]. The slip force (F_{slip}) and residual force (F_{res}) are determined by the following equations:

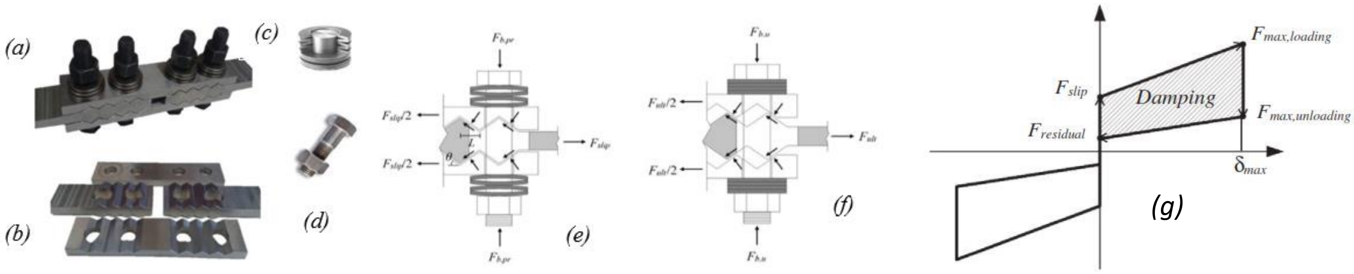


Fig. 1 – a) Assembly of the RSF Joint; b) Cap plates and slotted centre plates; c) Disk springs; d) High strength bolts; e) Free body diagrams RSF join on the brink of slippage; f) at ultimate defection; g) The general hysteresis behaviour of RSFD [13]

$$F_{RSFJ,slip} = 2n_b F_{b,pr}\left(\frac{\sin\theta + \mu_s \cos\theta}{\cos\theta - \mu_s \sin\theta}\right) \quad (1)$$

$$F_{RSFJ,res} = 2n_b F_{b,pr}\left(\frac{\sin\theta - \mu_k \cos\theta}{\cos\theta + \mu_k \sin\theta}\right) \quad (2)$$

Where n_b is the number of bolts on each splice, θ is groove angle, $F_{b,pr}$ is clamping force of prestressing and the μ_s and μ_k are the static and kinetic coefficient of friction respectively, while considered $\mu_k = .85\mu_s$ [13].

The general hysteresis behaviour of RSFJ is illustrated in Fig. 1(g). $F_{ult,loading}$ and $F_{ult,unloading}$ are the system forces at the maximum disk springs displacement and bolts force.

$$F_{b,u} = F_{b,pr} + K_s\Delta_s \quad (3)$$

$F_{ult,loading}$ and $F_{ult,unloading}$ is derived by replacing the bolt forces in Eq. 1 and Eq. 2 by Eq. 3, and μ_s and μ_k with μ_k and μ_s.

3. Structural Isolation, Effective Approach to Decrease Seismic Transmitted Force

Contrary to gravity load, structural responses to seismic actions depend on the dynamic specifications of a structure. Strengthening structures somehow is along with stiffening them and therefor lead to absorbing a higher level of earthquake energy which create a loop to finally structural strength overcome the induced forces. The more consistent the structures are with seismic movement; the less internal interactions and reactions create in structures. This is the root idea of developing the isolation system which nowadays accounts one of the most effective approaches especially for acceleration sensitive structures including hospitals, telecommunication center and so on. Base isolation systems by sliding motion shift the fundamental period to a higher level to decrease the force level. The other similar approach which follows this concept is rocking systems. Rocking motion let the structure to get the harmonic of the seismic excitation by a rotational movement which leads to a lower level of energy absorption. This mechanism pointed out as one the key factor to save many ancient structures which have no rigid base and are free to slide and rock on the ground.

All rocking systems need to have a hold-down together with dissipating mechanism to reach the desired level of seismic performance. In conventional rocking systems, to satisfy mentioned conditions, PT tendons with a kind of sacrificial element for dissipating input energy are employed. However, apart from unbonded post-tensioning implementation complexity, especially for tall structures, loss of tension in strands, always has been a concern for engineers. Also, systems with yielding mechanism are vulnerable to severe aftershocks. So, such systems require to have particular inspection and maintenance after an event. RSFJ make it possible that this joint being used with boundary column to provide structures with sufficient restoring force as well as damping mechanism simultaneously, which eliminates the need for regular inspections and post-event maintenance.

3.1 Single rocking braced frame

The proposed configuration for a single rocking frame with RSFJs is shown in Fig. 2. Braced frame and columns in this configuration are attached to the floor and bracket beams are used to connect shear links to frame and end columns. The rocking moment can be found by taking the moment about the rocking base:

$$M_{rock} = M_{weight} + M_{damper} = W\frac{l}{2} + n_d\left[F_{DL_i}(l+d) + F_{DR_i}(d)\right] \quad (4)$$

where n_d is the number of dampers in each side of the columns, l and d are frame width and the gap between the frame and column and F_{DL_i}, F_{DR_i} are the dampers force in the left and right sides of the rocking toe. Assuming that the bracket beams, columns are all rigid compared to RSFJs, the deflection in dampers in the right side (δ_{DR}) and left side (δ_{LR}) of the frame is determined.

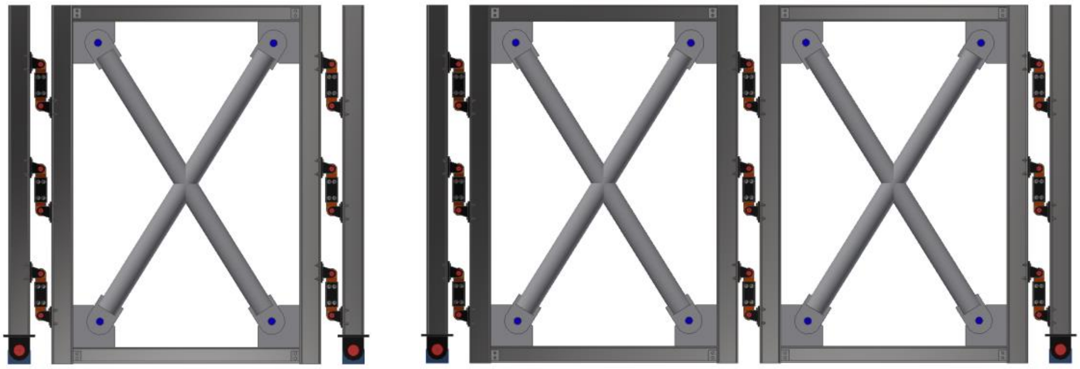


Fig. 2 – Schematic of single and coupled braced frame

$$\delta_{DR} = (l+d)\sin(\theta) \quad (5)$$

$$\delta_{DL} = d\sin(\theta) \quad (6)$$

While θ is rotating angle. The general push-pull response of the system is shown in Fig. 3. Before the slipping point ($M \leq M_{slip}$), stiffness of each link connected to other elements can be determined by:

$$k_{ini} = \left(\frac{1}{k_{col,axial}} + \frac{1}{k_{bb,bending}} + \frac{1}{k_{frame,axail}} + \frac{1}{k_{RSFJ,initial}}\right)^{-1} \quad (7)$$

$k_{col,axial}$: axial stiffness of column

$k_{bb,bending}$: bending stiffness of bracket beam ($\frac{b_{bb}h_{bb}^3}{12}$)

$k_{frame,axial}$: axial stiffness of column in the frame

While b_{bb}, h_{bb} are thickness and height of bracket beam. After slipping point, as the stiffness of RSFJs decreases

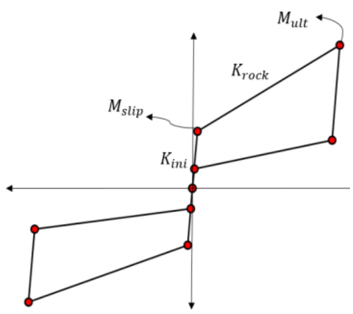


Fig. 3 – General push-pull performance of rocking system equipped with RSFJs

considerably ($k_{rock} \ll k_{ini}$), the stiffness of the links is significantly smaller in comparison to other elements so that rocking stiffness could be directly derived by:

$$k_{rock} = n_d k_{d,ini}\left[(L+d)^2 + (d)^2\right] \tag{8}$$

3.2 Coupled rocking braced frame

In the coupled braced frames system, the boundary element in the middle could be removed, and the frames are connected directly by the shear links. Similar to the single braced frame, all columns and frames have the same horizontal displacement as they are connected to the floor. Coupled braced frames by such configuration follow the same pattern of single braced frame. Therefore, the rocking base moment and rocking stiffness are given by:

$$M_{rock} = M_{weight} + M_{damper} = Wl + n_d\left[n_w F_{DL_i}(l+d) + F_{DR_i}(d)\right] \tag{9}$$

$$k_{rock} = n_d k_{d,ini}\left[n_w (L+d)^2 + (d)^2\right] \tag{10}$$

n_w is the number of coupled frames. Comparing coupled braced frames capacity with two single frames all identical in length - if architectural requirements provide enough space for coupled frames - the same capacity could be achieved while the middle column and a row of the shear link according to the number of coupled frames have been eliminated.

4. Case Study of a Seven-Story Office Building

The aim of this study in addition to introducing a new mechanism for the rocking system is having an estimation of the performance of a rocking brace frame compare to a conventional braced frame as well as base isolation system which is a well-proven efficient structural system. So, in the first step, a seven-story special braced frame building has been designed according to ASCE/SEI 7-16 and then the building equipped with LRB isolation system to reach a desire fundamental period. Then based on the outcome of the LRB system, the rocking system and the shear keys have been designed and adjusted.

4.1 Braced Frame System

A seven-story office building with overall height 24.5 m (3.5-meter height for each story) with plan dimension of 42 m by 42 m and six bays of braced frame in external perimeter in each direction have been considered (Fig 4).

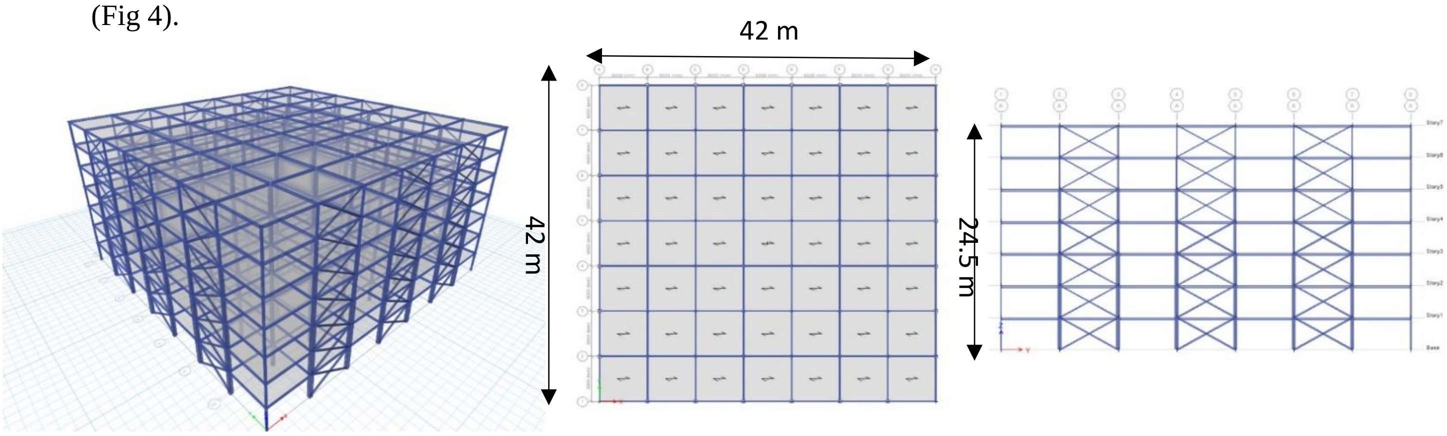


Fig. 4 – prototype office building

The assumed seismic factors for steel special concentrically braced frames are summarized in Fig 5.

Seismic Coefficient	Value
I_e (Importance Factor)	1.25
R (Response Modification Factor)	6
C_d (Deflection Amplification Factor)	5
Ω_0 (Overstrength Factor)	2

S_s	1.6 g	S_{MS}	1.92	S_{DS}	1.28	F_v	1.5
S_1	0.5 g	S_{M1}	0.75	S_{D1}	0.5	F_a	1.2

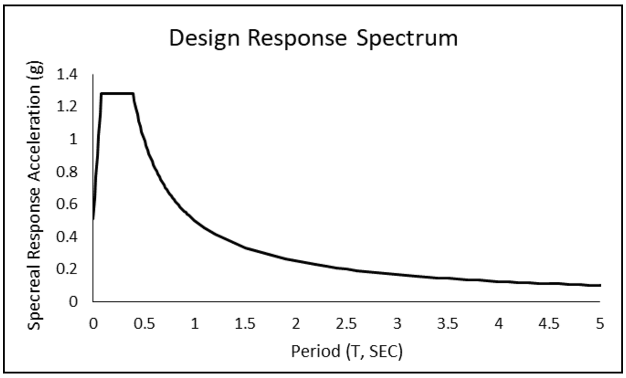


Fig. 5 – Seismic coefficient and response spectrum curve

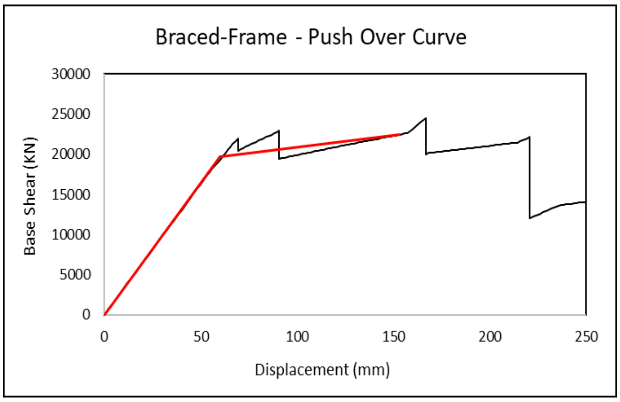
Considering the superimposed dead load and live load of 500 kg/m^2 and 400 kg/m^2 the base shear calculated by the following equations:

$$V = C_s.m.g \rightarrow C_s = \frac{S_{DS}}{\left(\frac{R}{I_e}\right)} \tag{11}$$

$$S_{M1} = F_v\, S_1 \ , \ S_{MS} = F_a\, S_s \tag{12}$$

$$S_{D1} = {}^2/_3\, S_{M1} \ , \ S_{DS} = {}^2/_3\, S_{MS} \tag{13}$$

The first mode period determined by numerical model for both directions equal is 0.9 sec. To meet the code strength and drift limitation for the first four story, the pipe 200X12 and the next three story pipe 150X12 have been chosen. Based on the coefficient method, target displacement of push-over analysis is derived:



Target Displacement (mm)	153
Base Shear (KN)	22440
C0	1.21
C1	1
C2	1
Sa, g	0.9
Ti	0.74
Dy(mm)	60
Vy(KN)	1974

Fig. 6 – Push-over result of special braced frame

Pushing the structure to target displacement the performance of the braces under compression and tension are depicted in Fig 7 and as can be seen almost braces in compression in the first three story reach to the ultimate capacity in 150 mm displacement.

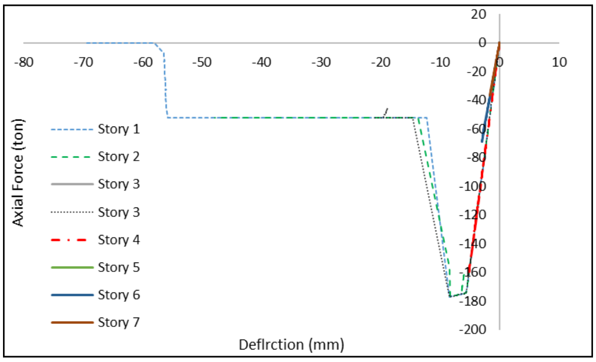


Fig. 7 – Hinge responses of braces (axial force)

4.2 LRB Isolation System

The design of the LRB joints for the prototype building has been done based on chapter 17 of ASCE/SEI 7-16. For the first step, a target period and desire damping are considered to be 2.75 and 15% respectively. The total number of columns is 64, and the LRB specifications should be designed to reach those values. First, the effective stiffness is determined as below:

$$K_{Dmin} = \frac{4\pi^2}{g} \times \frac{W}{{T_D}^2} = 41507 \text{ KN} \quad (14)$$

$$K_{Dmax} = 1.3\ K_{Dmin} = 53960 \text{ KN} \quad (15)$$

Then maximum and design displacement calculated as:

$\beta = 15\% \rightarrow B = 1.35$

$$D_D = \frac{g}{4\pi^2} \times \frac{S_{D1}T_D}{B_D} = 253 \text{ mm} \quad (16)$$

$$D_M = \frac{g}{4\pi^2} \times \frac{S_{M1}T_M}{B_M} = 379 \text{ mm} \quad (17)$$

Then the shear force for isolation unites, isolated structure and structure below are achieved:

$$V_b = K_{Dmax}D_D = 13650 \text{ KN} \quad (18)$$

$$V_{MCE} = K_{Dmax}D_M = 20490 \text{ KN} \quad (19)$$

$$V_s = \frac{V_b}{R_I}, 1 < \frac{3}{8R_I} < 2 \rightarrow V_s = 6830 \text{ KN} \quad (20)$$

Then LRB details including K_e, K_d and F_y calculated as 4918 KN.m, 492 8 KN.m and 44.1 KN respectively.

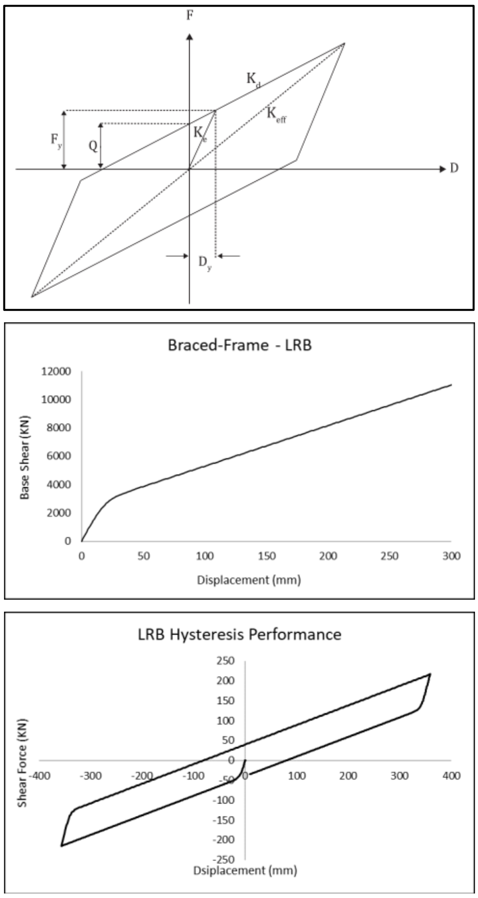


Fig. 8 – (a) LRB general hysteresis performance [17];

(b) Push over curve of LRB system;

(c) Push-pull hysteresis curve of defined LRB

4.3 Rocking Braced Frame Equipped with RSFJs

The intention of this study was to investigate the seismic behaviour of rocking braced frame with RSFJs and also compare the two isolation concepts. Base isolation is known as one of the most efficient structural systems for low and mid-rise building so could be a proper benchmark to be compared. Therefore, for designing the rocking braced frame, the force demand of push-over analysis for LRB has been used to design the RSF joints and then the code limitations, strength and drift have been controlled. Based on the overturning moment achieved from the push-over of structure with LRB joints and Eq (4) the RSF joints details are determined as below:

Table 1 – RSFD calculation details and specifications

L (m)	6
D (m)	0.4
F_{DL}	$0.3F_{DR}$
n_w	1
n_d	21
$M_{rocking}$ (KN.m)	227
F_{total} (KN)	3482
F_{damper} (KN)	165
Deflection (mm)	90

Slipping Force (KN)	45
Ultimate Force (KN)	180
Residual Force (KN)	48
Maximum Displacement (mm)	100

As can be seen, the hysteresis performances of the two isolation approaches have been tune to be almost the same. However, it is expected the performance of the rocking system is affected by the higher mode effect, which is not a concern in base isolation system, so the realistic seismic performances must be investigated by NTH analysis.

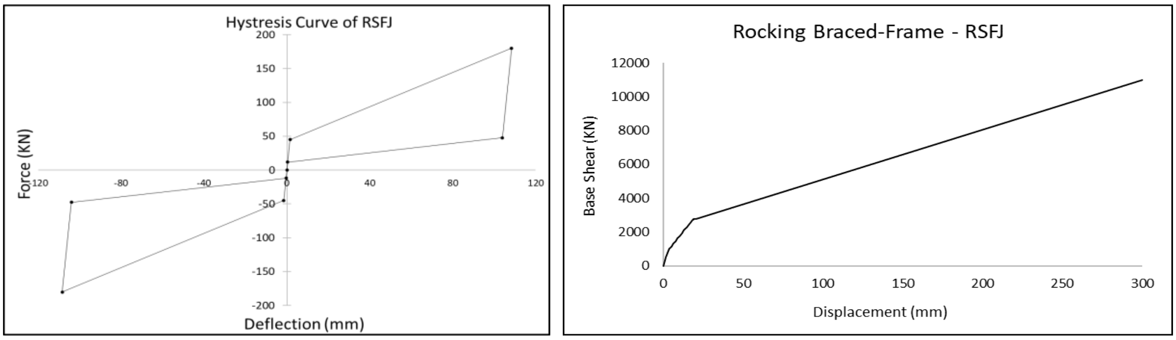


Fig. 9 – RSFJ hysteresis performance and push over the curve of rocking braced frame

Regarding the brace performance, as mentioned before the diagonal braces of the first three story experienced the buckling while in the two isolation concepts, none of the braces reaches to the buckling or yielding phase.

5. Seismic Performance of the Proposed System

For non-linear dynamic time-history analyses, a suite of seven ground motions have been scaled to match ASCE/SEI 7-16 spectrum considering $S_1 = 0.5$, $S_s = 1.6$, Soil type C and $I_e = 1.25$.

Earthquake ground motion	PGA
El Centro (1951)	0.3
Landers (1992)	0.74
Northridge (1994)	0.61
Kobe (1995)	0.35
Chi-Chi (1999)	0.36
Bam (2003)	0.16
Christchurch (2011)	0.1

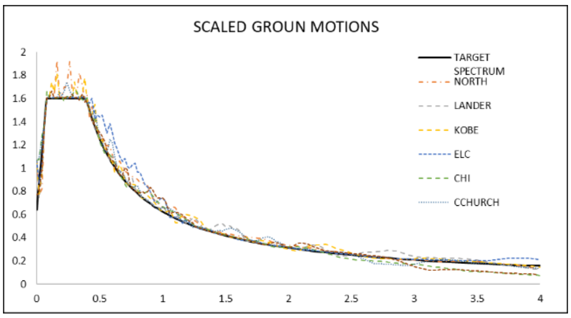
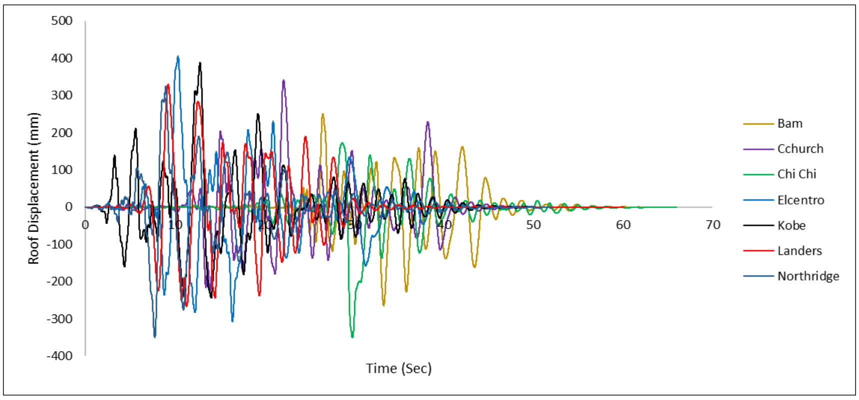


Fig. 10 – The selected ground motions and scale factor

As results illustrate (Fig 11) there is not residual displacement in structure as the RSFJs have a self-centring feature and capable of pulling back structure to its initial position and therefore normally there would not be permanent structural damages as long as the RSF joints are appropriately designed in seismic force level. In this structure, the drift limit considered being 2%, which all records are placed lower this limit.



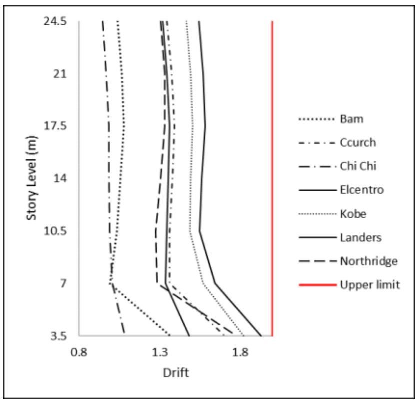
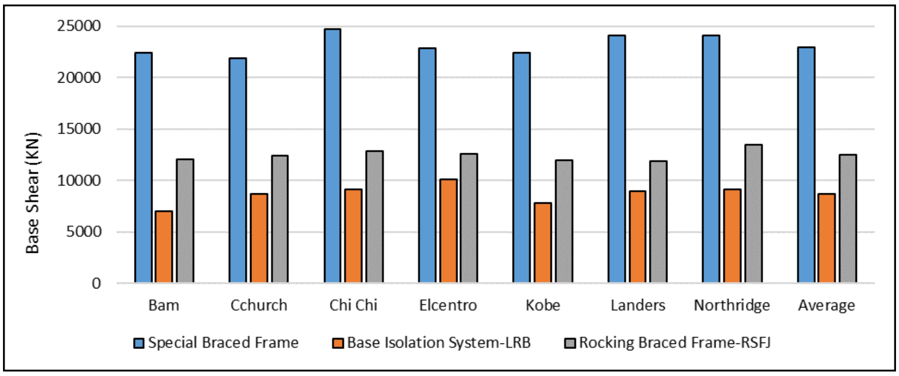


Fig. 11 – Roof time history displacement and inter-story drift subjected to selected ground motion

Comparing the results for the three designed systems, the amount of base shears in LRB isolation system on average is 33% of base shear in the special braced frame. This amount for the rocking frame is around 55% (Fig 12). The differences in base isolation systems and rocking frames are related to the nature of the movement. In the base isolation method, the structure is allowed to slide while rocking is about to rotate, and the parameter which uses to designing rocking shear is overturning moment. In the introduced rocking mechanism as the resisting force is distributed along with the height of the structure, so while the base shear is at a higher level but its effective height is lower so the overturning moment even compares to base isolation system placed at a lower rate. This amount averagely for base isolation and rocking frame are 23% and 18% of the braced frame respectively (Fig 12).



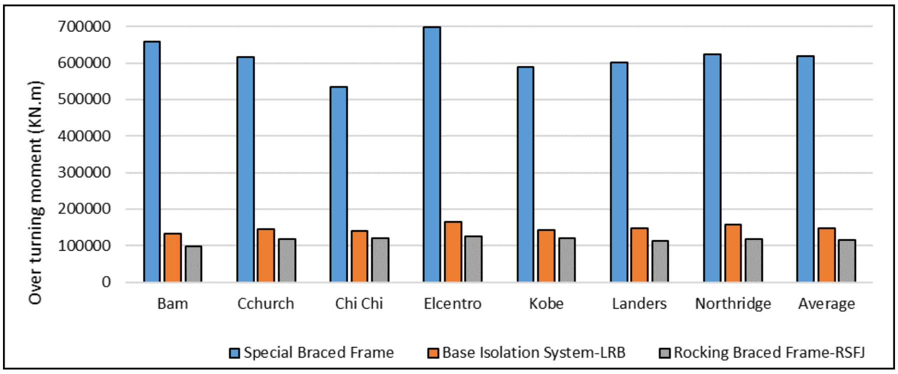


Fig. 12 – Base shear and an overturning moment of the three structural system

6. The Effect of Multiple Rocking Spots

On the effective approach to improving the performance of rocking systems is considering the different level for rocking motion. Contrary to base isolation system the higher modes contribution in the rocking system is essential so in this study, various levels of rocking defined to compare the results. By releasing rotation in all story, the base shear almost 24% drops which is the lower rate. Base on the first three modes, where the structure tends to bend could be a place to let structure to rotate and decrease the bending demand. Considering the two spots of second and four floors follow the two and third mode shapes and stand at the closest distance to the minimum amount in all level.

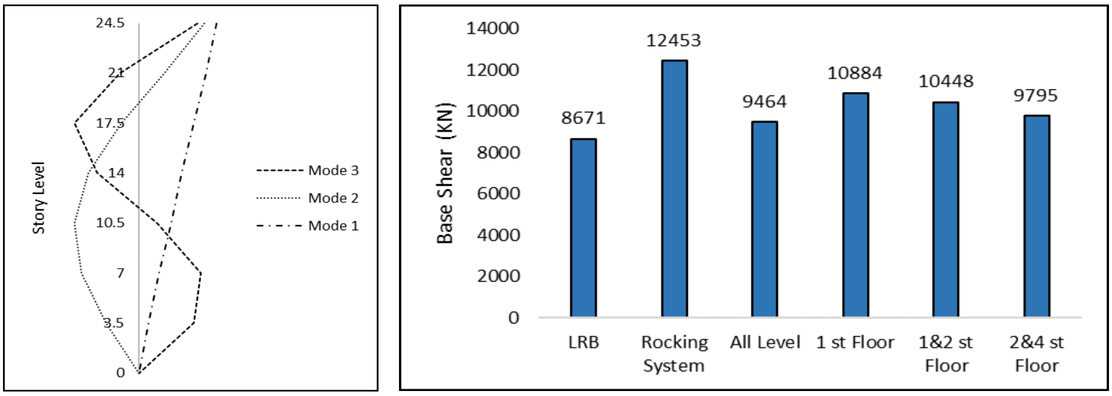


Fig. 13 – Modal shaped and base shear results for multiple isolation levels

7. Summary and Conclusion

In this paper, a new approach has been introduced for rocking braced frame. This proposed configuration relies on RSFJs as shear keys, which connect the braced frame to the boundary columns. The most conventional rocking system provides the required restoring force by the post-tension technology and adds sufficient damping through the yielding dampers. In this study, all these features brought by RSFJs which have been distributed along the frame. This system could be adopted as an alternative to conventional rocking systems to reduce their design's complexity and implementation challenges while covering taller rocking systems. Such a system with boundary columns could be used for a single, coupled or core braced frames, concrete shear or timer walls. In this system, as the resisting force are supplied in all stories makes it possible to have multiple rocking levels in different stories which improve the attenuate the effect of the higher modes.

To investigate the performance of the proposed system and also compare it with the braced frame and also LRB base isolation system, as an efficient lateral resisting system, the seismic behaviour of a seven-story office building has been investigated. Based on the achieved results, the performance of a rocking system equipped with RSFJs technology reasonably improve the dynamic performance of the structure. As results of NTHA of seven ground motions illustrate, compare to the braced frame, the LRB isolation and rocking system lead to 67% and 45% base shear reduction in turn. These amounts, considering the overturning moment, are about 18% and 23% respectively which is a crucial factor of designing the rocking braced frames. The proposed mechanism is fully self-centred and capable of satisfying the inter-story drift limitation. The proposed system would have much less constructional complexity compared to the conventional approach of PT tendons and also because of its configuration could have multiple rocking level which especially for the taller rocking braced frame would be a vital advantage. Experimental testing is also planned to validate the proposed concept further.

Acknowledgements

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