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SEISMIC RESPONSE OF SELF-CENTERING BASE-ROCKING STEEL STRUCTURES

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ABSTRACT

Recent developments in earthquake engineering have highlighted the desirable features of systems exhibiting a self-centering response. Traditional systems are designed to yield and undergo stable cyclic inelastic action to limit the forces that are applied to all other structural elements and to assure a stable response under severe earthquake ground motions. They do, however, sustain significant damage to the main structural elements, as well as residual drifts. Self-centering systems rely on geometric nonlinearities induced by rocking motion to limit maximum forces applied to the structure and are coupled with a form of energy dissipation to provide damping. Such systems return to their initial position at every loading cycle and do not sustain residual deformations even after being excited beyond their elastic limit. Specialized devices, rocking concrete piers and walls, as well as moment-resisting concrete and steel frames have been proposed as possible systems exhibiting a self-centering response.

The concept is herein extended to steel braced frames and steel wall systems that are designed to rock at their bases. Six configurations for the rocking wall system are examined and the main advantages and disadvantages of each configuration are outlined. A 6-story building is then designed with buckling-restrained braces, and with the proposed rocking system. Yielding elements and viscous dampers are both considered as energy dissipation mechanisms for the rocking steel wall system. The three structures are compared through nonlinear time-history analyses by examining peak interstorey drift demands, peak absolute accelerations, and overall residual deformations after the earthquakes. Based on these analyses, the proposed rocking steel wall system appears to be a viable alternative to traditional steel seismic resisting systems with notable advantages in its overall seismic performance.

Introduction

Seismic design has traditionally been based on the single criterion of preserving life safety in a design-level earthquake. This objective has been found to be satisfied most economically by allowing the structure to respond inelastically to seismic loads, thus limiting the maximum force that any given structural element will experience, and also reducing the peak response exhibited by the structure as a result of hysteretic energy dissipation. The result of this design philosophy, however, is that most structures are expected to have damage following a moderate to large earthquake; this damage may include loss of system strength and stiffness, and residual deformations. This can result in significant

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structural repair costs, as well as undesirable restrictions on building use while those repairs are completed. Thus, building designers are now coming to recognize that life safety is a crucial but not always sufficient performance objective: it would be desirable if a structure could be economically designed not only to protect life safety, but also to be undamaged or nearly undamaged following a significant earthquake.

While many traditional building systems are available to meet the traditional performance objective of life safety, relatively few systems exist that can be designed economically to resist a significant earthquake without sustaining appreciable damage. Such a system, termed "self-centering," would dissipate significant energy in order to limit the structural response, but after the ground motion, the structure would return to the undeformed state without any loss of strength or stiffness. A number of different system configurations that meet these objectives have been developed, such as concrete post-tensioned frames (Priestley and Tao 1993, MacRae and Priestley 1994). These first systems relied only on unbonded post-tensioning to provide moment capacity and self-centering properties and therefore did not dissipate substantial amounts of energy at each loading cycle. This concept was then extended to systems where the self-centering capacity was combined with an energy dissipation system such as longitudinal non-prestressed (mild) steel or additional external dissipation devices designed to yield and to provide the supplemental damping; this was done for both reinforced concrete frames and rocking precast concrete shear walls (Stanton et al. 1993, Priestley et al. 1999, Kurama et al 1999). In steel, a full-scale steel brace element has recently been developed and validated (Christopoulos et al. 2006), and the concepts that were developed first in concrete have been extended to post-tensioned steel frames (Ricles et al. 2001, Christopoulos et al. 2002) and, more recently, to rocking steel concentrically-braced frames (Roke et al. 2006). The research described herein considers six potential configurations of base-rocking steel structures. From these, the rocking zipper-braced frame is selected for more detailed examination and comparison with another structural system that is seeing increased use in seismic regions: a buckling-restrained braced frame (BRBF).

Lateral Load Response of Base-Rocking Systems

Conceptual Description of a Base-Rocking System

A base-rocking system is similar to a traditional structural wall, in that it is a lateral load-resisting system that acts as a vertical cantilever, transmitting wind and earthquake loads to the foundation through shear and bending moment. Unlike a traditional wall, however, a rocking system is permitted to rock on its foundation: it rests on its base rather than being rigidly attached to it by bolts or welds. Thus, tension at the foundation is precluded. A rocking system is able to resist moments at its base because of its self-weight, but once the influence of that weight is overcome, the wall will uplift rather than developing any tension at its base. Unlike a traditional wall, where the attainment of the linear limit is due to plastic deformations near the base, loading beyond the linear limit of a rocking system results only in the opening of a gap at the wall base.

In order to provide the rocking system with post-uplift lateral stiffness once it begins to rock, unbonded post-tensioning is provided between the top of the wall and the foundation. The post-tensioning also increases the lateral load at which the wall will begin to uplift because it increases the precompression acting at the base of the wall; the designer can select the load at which the wall will begin to rock by adjusting the post-tensioning force applied to the wall.

Push-Pull Response of a Base-Rocking System

As shown in Step 1 of Fig. 1, the post-tensioning and self-weight cause a precompression at the base of the wall. As the lateral load increases, it causes a change in the force distribution at the base of the wall, as shown in Step 2 of Fig. 1, until the vertical stress reaches zero at one edge of the wall, as shown in Step 3 of Fig. 1. The wall responds with its elastic stiffness up to this point; its behaviour is the same as that of a traditional wall.

Beyond this point, further lateral load applied to a traditional wall would produce tension at the base, but because this tension cannot be resisted, the wall instead begins to lift off of its foundation, as shown in Step 4 of Fig. 1. In doing so, the post-tensioning is lengthened, resulting in a larger post-tensioning force. While this increase in post-tensioning force as the wall rocks ensures that the wall has some lateral stiffness, that stiffness is substantially reduced relative to the initial elastic stiffness of the wall. The wall continues to deform elastically along its height as the load is increased, but it is the opening of the gap at the interface of the wall and its foundation that dominates the post-elastic deformations.

The wall continues to rock in this manner until it reaches a limit state, such as yielding of the post-tensioning or of an element within the wall. Assuming that no such limit state is reached, unloading of the wall follows the same path as loading of the wall, and for a symmetric wall, the response is identical in the other direction (see Step 5 of Fig. 1). Upon removal of all lateral load, the roof drift returns to zero, as shown in Step 6 of Fig. 1.

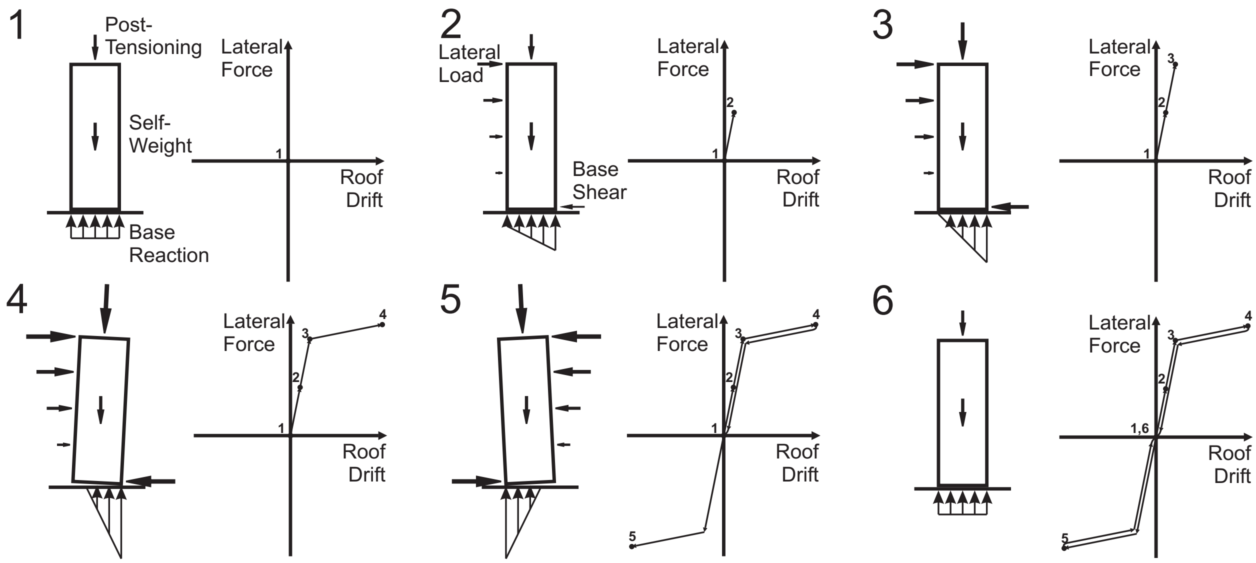


Figure 1. Schematic response of a base-rocking structure.

The rocking system described above has all elements remaining elastic. While this has the advantage of ensuring a self-centering response, the hysteresis encloses no area, so energy dissipation is limited to the assumed inherent damping, in addition to energy radiation into the foundation as the wall pounds during rocking. Therefore, supplemental energy dissipation that is activated by rocking of the wall is usually provided in locations such as at the wall base (e.g. Kurama 2002) or between two adjacent rocking walls (e.g. Nakaki et al. 1999). With appropriate properties, this results in a flag-shaped hysteresis, such as that shown schematically in Fig. 2.

Comparison of Possible Base-Rocking Steel Structures

Desired Attributes of Base-Rocking Systems

One of the advantages of a rocking system is that its nonlinear response to large lateral loads can be designed almost separately from its initial response to service lateral loads, since the rocking response of the base governs the nonlinear behaviour while the elastic response of the wall governs the initial behaviour. Thus, a wall that is to be made to rock would ideally be very stiff, so as to satisfy serviceability requirements under service loading and to simplify the design process by allowing the system to be accurately modeled as a rocking rigid body. The wall should also make economical use of materials and labour, and it should be readily adapted to accommodate the changed force distribution that results from the rocking motion and the applied post-tensioning force.

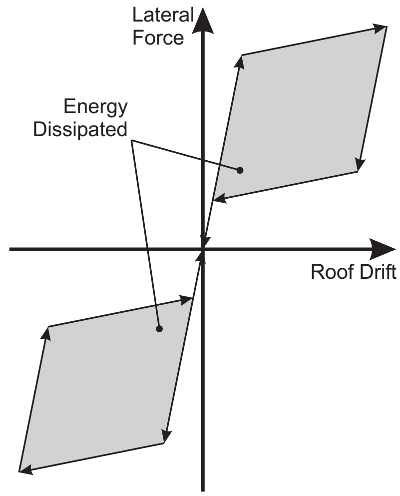


Figure 2. Schematic response of a base-rocking structure with hysteretic damping.

The rocking system will be post-tensioned to control its resistance to overturning and to provide post-uplift stiffness during rocking. One possible location for the post-tensioning is at the edges of the wall, so as to act directly on the boundary columns and to maximize the lever arm provided by the post-tensioning. If the post-tensioning steel is moved from the two edges to the centre of the wall, however, the maximum roof drift that the wall can withstand prior to yielding of the steel is doubled, since the strain in the post-tensioning at a given base rotation is halved. The lever arm by which the post-tensioning generates a moment that resists overturning is also halved, but the area of post-tensioning that is activated by the rocking of the wall in each direction is doubled. Thus, the maximum lateral load that can be applied prior to yielding the post-tensioning is unchanged, and the post-uplift stiffness of the wall is reduced by a factor of two. This observation is in keeping with the results of previous numerical studies (Kurama et al. 1999; Roke et al. 2006).

Because it is preferable to avoid yielding of post-tensioning steel under large lateral deformations, and because a smaller positive post-uplift stiffness generally reduces the peak accelerations of the system, all rocking system designs considered in this paper have the post-tensioning located at the centre of the wall. Since the force that will be generated in this post-tensioning is potentially large, consideration must be given to the ability of each configuration to be economically adapted to accommodate a large point force at the top centre of the wall where the post-tensioning is anchored.

Six possible systems that were considered with regard to the above-mentioned attributes are shown schematically in Fig. 3 and briefly discussed below. First, a steel plate shear wall was considered, but preliminary analyses showed that, while the inherent ductility of the system and the stiffness that results from the minimum thickness requirements of the plates can be advantageous for a traditional system, they are of little benefit for a rocking system. Five configurations of braced frames were then considered: a vertical truss with tension-only X-braces, one with tension-compression braces, a chevron-braced frame, a two-storey X-braced frame, and a zipper-braced frame. It was noted that configurations (b), (c), and (e) would not easily accommodate the significant forces that are generated at the bottom corners when the wall rocks. Preliminary analyses of the remaining two configurations showed them to be similarly attractive, so the zipper-braced frame was selected for the analyses described here.

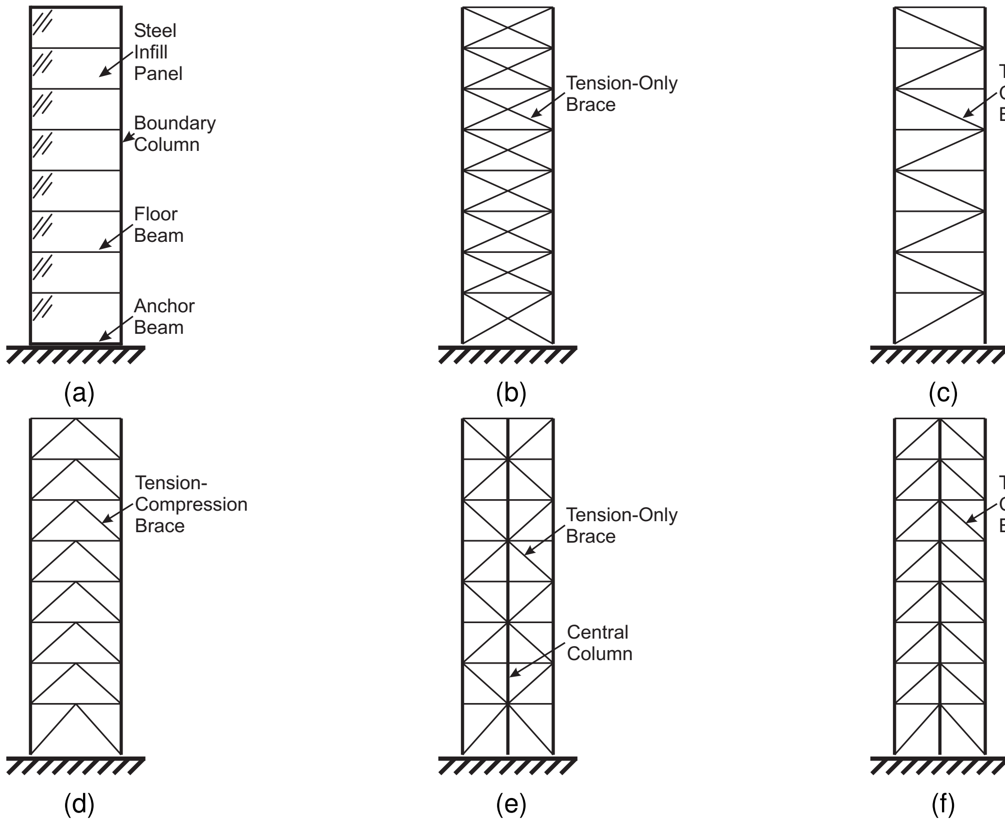


Figure 3. Schematic drawings of considered base-rocking systems: (a) steel plate shear wall; (b) vertical truss with tension-only braces; (c) vertical truss with tension-compression braces; (d) chevron-braced frame; (e) two-storey X-braced frame; (f) zipper-braced frame

Numerical Analysis of Selected Systems

Summary of Designs

In order to perform a preliminary assessment of the behaviour of rocking steel systems, and so to determine whether they are meritorious of further research, the rocking zipper-braced frame (RZBF) configuration that was selected previously was designed for the east-west lateral load-resisting system of a six-storey structure located in Vancouver. For comparison, a buckling-restrained braced frame (BRBF) was also considered. The plan of the building that was considered is shown in Fig. 4. The dead load was taken as 1.5 kPa at the roof and 4.5 kPa at all other levels, and the live load was taken as 1.0 kPa at the roof, 4.8 kPa at the first floor, and 2.4 kPa at all other levels. Precast concrete cladding was considered as a dead load of 3.0 kPa. For this building and the assumed loads, the seismic weight acting at the roof was calculated as 5770 kN, and the seismic weight at all other levels was calculated as 14025 kN. The two designs are summarized in Fig. 5.

The rocking wall was modeled using SAP2000 (CSI 2005). All members of the rocking wall were modeled using traditional frame elements, and the base connections were modeled using nonlinear links with gap properties, which have a very high compressive stiffness and no tensile stiffness. Two rocking systems were modeled, one with a yielding energy dissipation element and the other with a viscous energy dissipator; the properties of the energy dissipating elements are summarized in Table 1, and their design is discussed below. The period of the first mode of both RZBFs was found to be 0.86s. The buckling-restrained braced frame (BRBF) was modeled using elastoplastic nonlinear elements with equal tensile and compression yield loads to model the braces and elastic frame elements for all other members. The first mode period of the BRBF was 1.06s.

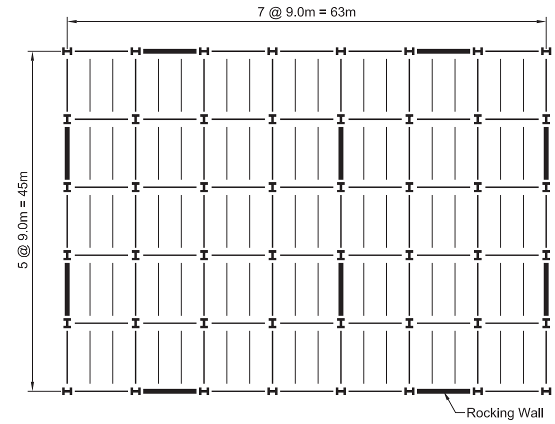
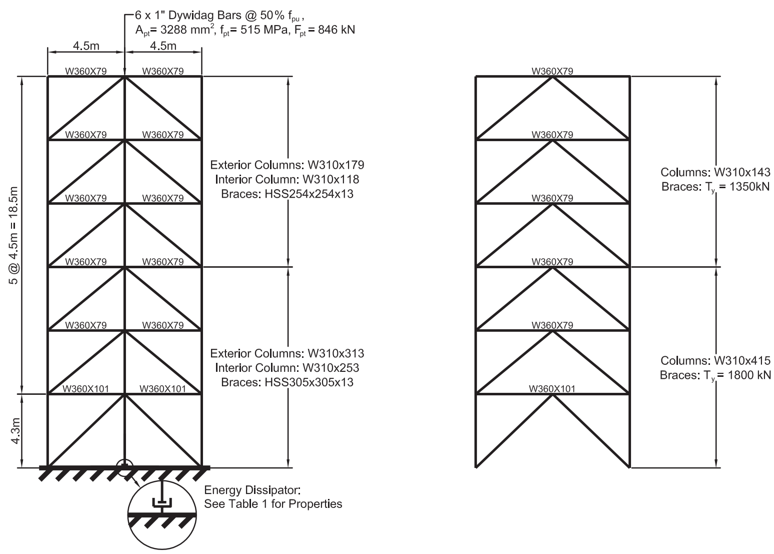


Figure 4. Plan view of design building.



(a) (b)

Figure 5. Designs of (a) rocking zipper-braced frame; (b) buckling-restrained braced frame.

Table 1. Properties of energy-dissipating elements.

Energy Dissipator Type	Properties
Hysteretic	F_y = 3800 kN k = 19000 kN/mm
Viscous	c = 5000 kN-s/m

Push-Pull Response

The push-pull response of the BRBF is shown in Fig. 6, where the gradual yielding of the system that is due to the yielding of the BRB elements at different levels results in an elastoplastic hysteresis.

Fig. 6 also gives the push-pull response of the RZBF with hysteretic damping (RZBFh). The rocking system has been designed to rock at a lower load than the load causing the first yield of the BRBF. This was done so that all elements in the rocking system remain linear without requiring significantly more material than is used by a more traditional wall. The wall was designed to have approximately the same base shear at a roof drift of 2% as the BRBF. The base shear at which the wall should begin to rock and the optimal post-rocking stiffness, both of which can be selected by the designer by adjusting the area and initial prestress of the post-tensioning steel, will be topics of future research.

Finally, Fig. 6 gives the push-pull response of the RZBF with viscous damping (RZBFv). The same wall was used in this case as for the RZBFh, but the hysteretic damper was replaced by a linear viscous damper. The backbone curve of the system is shown, where the load is applied sufficiently slowly that the viscous damper has negligible effect. The damper properties were chosen by considering an effective single degree-of-freedom system with stiffness equal to the secant stiffness of the undamped RZBF at 2.0% drift. The damping coefficient of the damper was then selected to give this single degree-of-freedom system a damping ratio of approximately 20%.

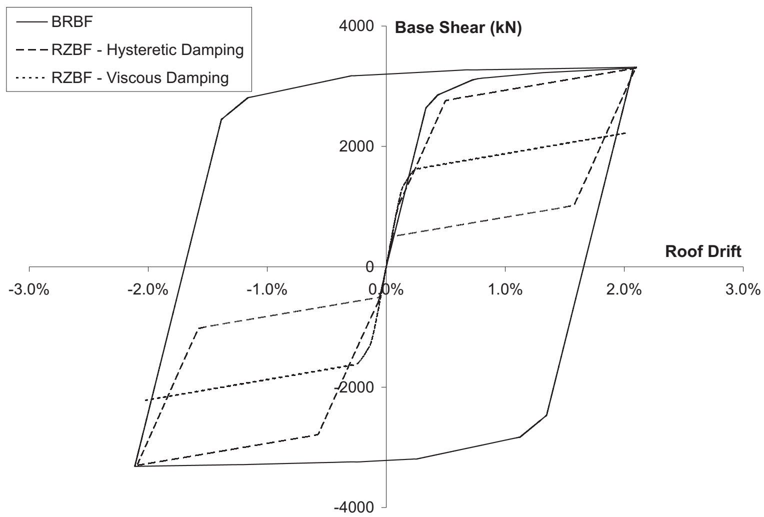


Figure 6. Push-pull response of walls.

Seismic Response

The system models previously described were subjected to three magnifications of ground accelerations recorded for three earthquakes in California: Cape Mendocino 1992 (CM2), Landers 1992 (LAN2) and Northridge 1994 (NOR9). Their respective acceleration spectra are given in Fig. 7. The time-traces of the roof drifts and accelerations of the two rocking systems and of the BRBF are shown in Fig. 8 for the CM2 ground motion scaled by a factor of 2. It is noted that, while the peak roof drifts are similar for the three systems, the rocking systems tend to behave reasonably symmetrically about the undeformed state, with similar peak drifts being reached in both directions before returning to the undeformed configuration after the ground motion. In contrast, the buckling-restrained braced frame responds mostly in one direction, with a similar peak roof drift but a substantial residual drift of 0.68%. The roof acceleration time-history is very similar for the two rocking systems, with a peak acceleration that is larger in both cases than that of the buckling-restrained braced frame.

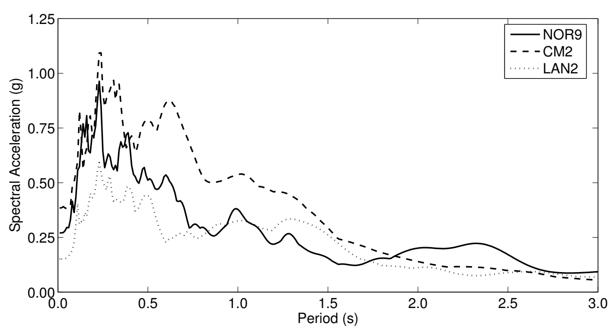


Figure 7. Acceleration spectra of considered earthquakes

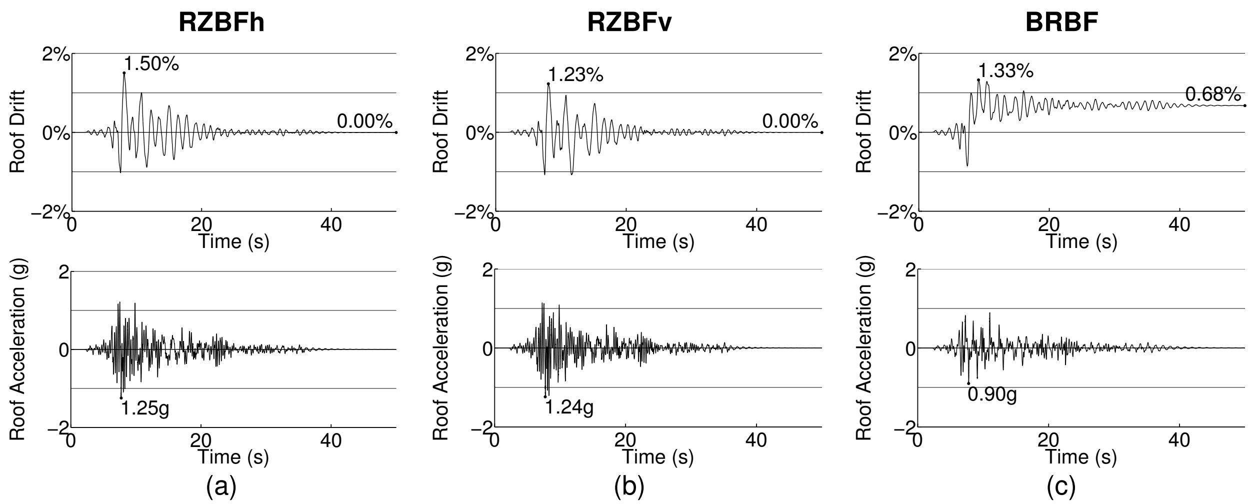


Figure 8. Time-history roof drift and acceleration response to record CM2 with magnification of 2 for: (a) rocking zipper-braced frame with hysteretic damping; (b) rocking zipper-braced frame with viscous damping; (c) buckling-restrained braced frame

Figs. 9, 10, and 11 summarize the peak interstorey drifts and residual interstorey drifts obtained for the three records that were considered in this study for three scaling factors (100%, 200% and 400%). In these figures, RZBFh refers to the RZBF with a yielding energy-dissipating element, while RZBFv refers to the RZBF with a viscous damper. The figures also summarize the variation of peak roof accelerations for each of the different scaled records. From these, it is observed that the peak drifts exhibited by the rocking systems are, on average, similar to those exhibited by the BRBF. The peak interstorey drifts are more uniform for the rocking systems, however, because the interstorey drifts result primarily from the base rotation, which affects all stories in the same way, whereas the interstorey drifts of the BRBF are dependent on the amount of inelasticity at each level. The maximum interstorey drift considering all stories is smaller for the rocking systems than for the BRBF in all cases considered, and while the two rocking systems do not have the same interstorey drifts as each other, there is no clear trend for one to have smaller peak interstorey drifts than the other.

The residual drifts of the rocking systems are always essentially zero because of the self-centering nature of the systems, whereas the residual drifts of the BRBF are non-zero because of yielding of the brace elements. Because residual drift is associated with structural damage, this aspect of the performance of the rocking systems is preferable to that of the BRBF.

The accelerations experienced by the rocking system are generally larger than those of the BRBF, and the difference tends to increase with increasing ground motion intensity. The peak roof accelerations of the two rocking systems are generally similar.

Summary and Conclusions

The concept of a base-rocking system was introduced and its advantages discussed. Six potential configurations of steel walls were considered with regard to the characteristics that are desired of a base-rocking steel system, and of those configurations, the zipper-braced frame was selected for further study. This rocking system was designed for a six-storey building, and both yielding energy-dissipating elements and viscous dampers were examined. Comparison with a buckling-restrained braced frame (BRBF) using non-linear time-history analyses demonstrated that the rocking systems sustained similar average peak interstorey drifts to the BRBF in all cases, but with more consistent interstorey drifts over the building height. The rocking system sustained no residual drifts in any case, whereas the BRBF often had significant residual drifts. The peak roof accelerations of the rocking systems were generally higher than

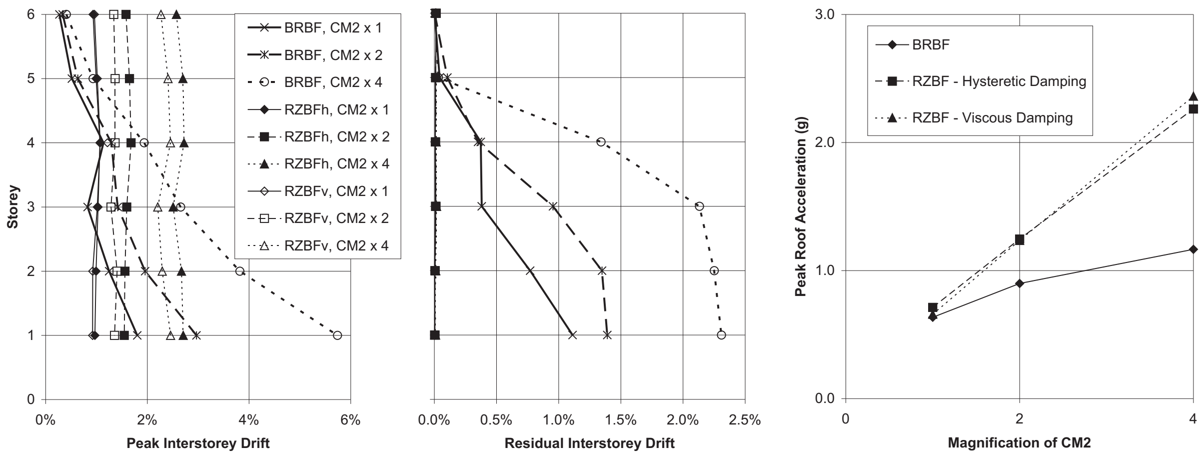


Figure 9. Peak interstorey-drifts, residual interstorey-drifts, and peak roof accelerations of test structures subjected to three magnifications of CM2 ground acceleration record.

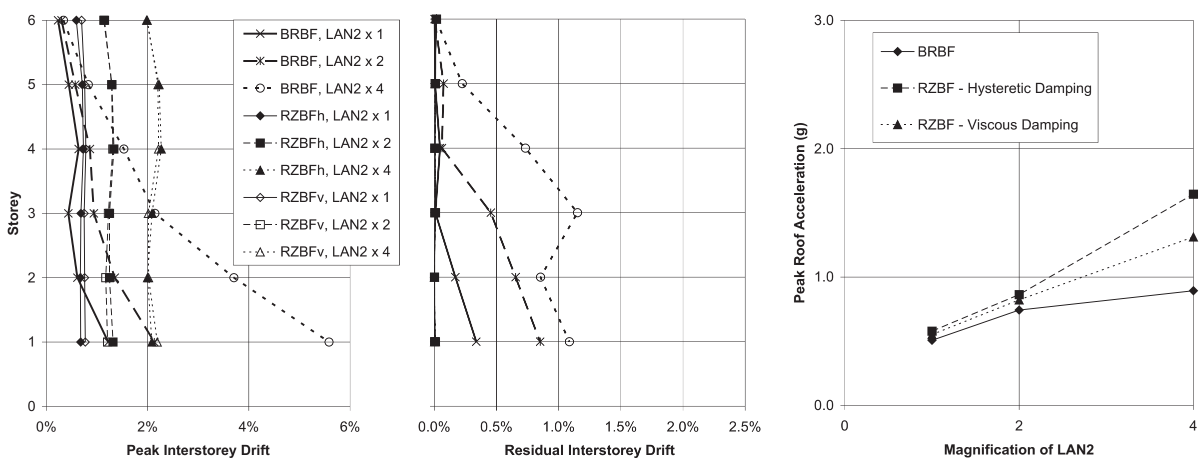


Figure 10. Peak interstorey-drifts, residual interstorey-drifts, and peak roof accelerations of test structures subjected to three magnifications of LAN2 ground acceleration record.

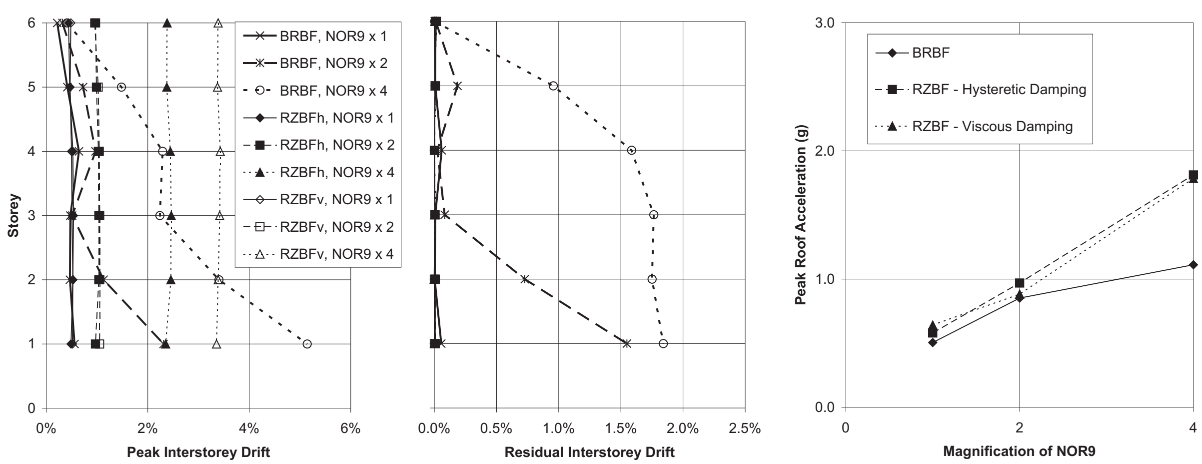


Figure 11. Peak interstorey-drifts, residual interstorey-drifts, and peak roof accelerations of test structures subjected to three magnifications of NOR9 ground acceleration record.

those of the BRBF systems, and the base-rocking systems with yielding and with viscous dampers were seen to have similar behaviour.

The proposed base-rocking steel system successfully eliminated residual drifts without having larger peak drifts than a more traditional system, although it did experience larger peak accelerations. Future research will use both numerical and experimental studies to further verify the positive performance attributes seen in this research, and to develop rational design recommendations for the implementation of such a system in practice.

Acknowledgments

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