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Higher-mode Mitigation System for High-rise Buildings using Uncoupled Base Rocking and Shear Mechanisms

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ABSTRACT

Soaring world population and unprecedented urbanization have boosted high-rise developments around the world including in cities located in highly seismic areas. Modern seismic design philosophies prioritize life-safety under major earthquakes. High-rise buildings designed to current codes are therefore expected to suffer extensive damage associated with significant losses in major earthquakes. To achieve enhanced seismic resilience, numerous high-performance systems have been developed for high-rise structures. This paper proposes an innovative low-damage design concept consisting of uncoupled rocking and shear mechanisms at the base of high-rise buildings. This proposed dual-mechanism system limits moment and shear demands throughout the building by effectively controlling higher-mode effects. A physical embodiment of the system is developed and then validated through nonlinear response history analyses (NLRHA) for a 42-storey RC core-wall building. The analysis results confirm that the proposed system eliminates damage at the base of the walls and minimizes inelastic demands over the height of the building.

Keywords: high-rise buildings, seismic resilience, higher-mode effects, rocking mechanism, shear mechanism.

INTRODUCTION

The intensifying global urbanization makes it a critical challenge to provide the soaring number of urban migrants with disaster-resilient habitation and has boosted high-rise construction all over the world. CTBUH [1] reported that, by 2018, the number of 200-metre-plus high-rise buildings completed worldwide has reached over fourteen hundred, a large number of which were built in cities that are located in seismically active regions. While modern seismic design aims to achieve life-safety and collapse-prevention under major events, high-rise buildings designed to current codes are expected to sustain extensive damage that jeopardizes their structural soundness for future earthquakes. This alarming consequence has been seen in Christchurch, where, after the M_w6.2 aftershock on February 22, 2011, nearly 50% of the post-1970's concrete wall buildings were assessed as unsafe for immediate occupation and a large number of them had to be demolished even though they had not collapsed [2].

High-rise buildings are characterized by longer fundamental periods and flexure-dominated behaviour. As outlined by Chopra [3], structures with these traits are innately susceptible to pronounced higher-mode effects that can result in significant dynamic amplification to responses that can lead to extensive damage. Initial studies on this phenomenon can be traced back to the 1950's [4-6], but were only focused on the elastic response of structures. Blakeley et al. [7] investigated inelastic higher-mode effects and highlighted a significant dynamic amplification of moments and shears despite the formation of base plastic hinges. Derecho et al. [8] proposed design charts that predict seismic demands considering higher-mode contributions. These early studies were followed by extensive investigations on higher-mode effects as reviewed by Rutenberg [9]. Paulay and Priestley [10] suggested that base hinges only limit the first-mode response but may not affect the higher modes. Wiebe et al. [11] studied the modal differentials in terms of response to varied rotational constraints at the base. Whereas evaluation of higher-mode effects can always be improved, the shear component of RC walls, a critical aspect, is still designed with a level of uncertainty given the difficulty in predicting these effects.

Aiming at damage-resistant design, a few high seismic performance systems have been explored for high-rise buildings. In the PRESSS program [12,13], precast concrete panels were allowed to rock at the base under the clamping force provided by post-tensioned strands. This concept of controlled rocking has been applied in rehabilitations [14] and new constructions [15,16]. Nielson [17] studied a 50-storey rocking core for which gravity loads served as the only re-centering force. In contrast to conventional fixed-base design, these rocking systems reduced seismic demands relying on rocking action that causes minimal material nonlinearity. However, pronounced higher-mode effects were still reported in the analytical [17,18] and experimental [13] validations of these systems.

Allowing for flexural yielding over the height of RC walls, Rad and Adebar [19] pointed out that dynamic shear amplification due to higher-mode responses can be largely reduced. Panagiotou et al. [20] and Munir et al. [21] respectively developed design strategies that allowed a second or multiple plastic hinges to form at the mid-height or selected levels of RC walls. Following a similar idea, Wiebe et al. [11] investigated the response of segmented base-rocking steel frames using an additional rocking joint over the height of the structure. These systems controlled seismic demands through either plastic hinging or mechanical rocking, but were found more effective in limiting moments than shears [11,20]. Wiebe et al. [11] then achieved more efficient higher-mode mitigation using a shear fuse in the base storey of a rocking frame. This is equivalent to the finding reported by Rad and Adebar [19] who found that shear stiffness reductions in RC walls due to diagonal cracking can significantly reduce the dynamic shear amplification. Whereas it is more straightforward to implement a shear fuse using nonlinear braces in steel frames, it becomes more challenging to incorporate a two-way shear mechanism into a three-dimensional RC core-wall system.

Base isolation has been used for high-rise buildings in Japan for decades. Despite the reduced effectiveness in terms of fundamental period shifting, the decoupling effect provided by isolators is still beneficial, as it largely reduces inter-storey drifts and floor accelerations, both of which are direct causes of damage and highly influenced by higher-mode effects [22,23]. However, significant overturning moments (OTMs) at the base of high-rise structures could lead to isolators being loaded in tension or overstressed in compression beyond their acceptable range. This concern, as Becker [23] concluded, explains why base isolation is of limited use for high-rise buildings outside Japan. To resolve this problem, tension-resistant isolators have been developed, e.g. [24,25]. However, when used for highly slender structures and/or under extreme seismic events, devices with high tensile resistance are still challenging to design and implement. Another strategy to keep bearings from tension is to release the overturning effect by either allowing isolators to uplift, e.g. [26], or the superstructure to rock, e.g. [27]. In these solutions, however, the shear behaviour of base-isolated structures still interacts with the flexural response, which results in considerable design challenges.

This paper proposes a novel concept of uncoupling the flexural and lateral responses at the base of high-rise structures such that the moment and shear demands can be controlled independently using two dedicated nonlinear mechanisms which are physically separated and behaviourally uncoupled. This dual-mechanism system relies on rocking action to limit base OTMs while it mitigates the dynamic higher-mode amplification through a distinct ductile shear mechanism. Among numerous possible ways of implementing the proposed concept, this paper proposes one physical embodiment that is designed and detailed for a RC core-wall building that has been studied by PEER [28]. For validation purposes, advanced NLRHA are conducted on this benchmark building with the proposed system. The enhanced seismic performance demonstrated by these analyses is then highlighted in comparison with the response achieved in the conventional fixed-base design.

UNCOUPLED ROCKING AND SHEAR BASE-MECHANISM SYSTEM

The proposed system, hereafter referred to as the *MechRV3D* which indicates the rocking (*R*) and shear (*V*) mechanisms (*Mech*) in a three-dimensional (*3D*) configuration, is located in the basement under the central core of a building, as shown in Figure 1. The RC core walls are anchored at the ground level to the top face of a solid concrete block, referred to as the *rocker*, which is elevated above the foundation and supported by *mega-columns*. These columns are composite and reinforced using multiple steel sections which enable a single column to carry the entire gravity load of the core in the elastic range. Instead of being cast-in-situ, the mega-columns are dry plugged into *sockets*, which are carved in the soffit of the rocker and the top face of the foundation, through *rolling pipe-pin joints* that are proposed in this paper. In the vertical direction, the proposed joints allow the rocker to uplift at the top level of the mega-columns where the intended rocking action would occur. At the same time, specially detailed *spherical caps*, *steel pipes*, and *truncated-cone-shaped cans* facilitate the rocker and the mega-columns to form a *three-dimensional rolling frames* that can sway in any horizontal direction providing minimal lateral resistance to the superstructure. The pipe-and-can pairs serve as dowels that prevent the mega-columns from sliding off the sockets.

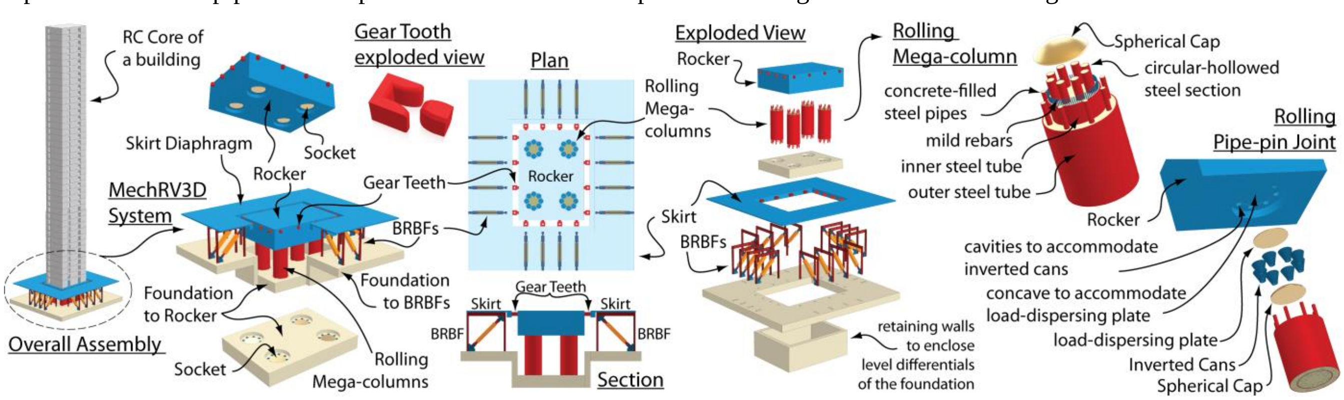

Figure 1. Physical embodiment of the MechRV3D system.

Lateral forces of the RC core are entirely transferred to distinct peripheral elements that form a dedicated mechanism for limiting shear forces. The shear mechanism includes a *skirt diaphragm*, or *skirt* in short, which is made of the ground floor slab outside the core area, and a series of *buckling-restrained brace frames* (*BRBFs*) that connect the skirt to the foundation. These frames are braced diagonally with all the BRBs inclining toward the rocker. Collecting base shears of the core, the skirt engages the BRBFs that provide shear strength, stiffness and ductility in the both principal directions. Given the diaphragm action of the skirt, all the BRBFs lying in the same direction will be engaged together, regardless on which side of the rocker they are. The rocker and the skirt are not cast monolithically but linked through *gear teeth* that work in contact and transmit forces and motions within the horizontal plane. Being unrestrained vertically, these gear teeth allow for the intended rocking to occur creating minimal moment resistance at the base of the core.

DESIGN OF THE MECHRV3D SYSTEM FOR THE BENCHMARK BUILDING

The MechRV3D system is applied to the benchmark building that was studied in a Tall Buildings Initiative (TBI) launched by PEER [28]. This building is located in Los Angeles, California. Its lateral-force-resisting system consists of RC core walls coupled by deep beams, as shown in Figure 2. It was designed by Magnusson Klemencic Associates (MKA), following the performance-based seismic design procedure recommended by Los Angeles Tall Buildings Structural Design Council. The benchmark building has minimum strengths of 802MN-m and 1249MN-m about the north-south (NS) and east-west (EW) directions respectively [28]. These values, denoted as OTM_y, mark the onset of yielding in the fixed-based structure. To emulate the conventional design, M_{rock}^{rNS} =800MN-m and M_{rock}^{rEW} =1200MN-m were adopted as the basic design of the rocking mechanism.

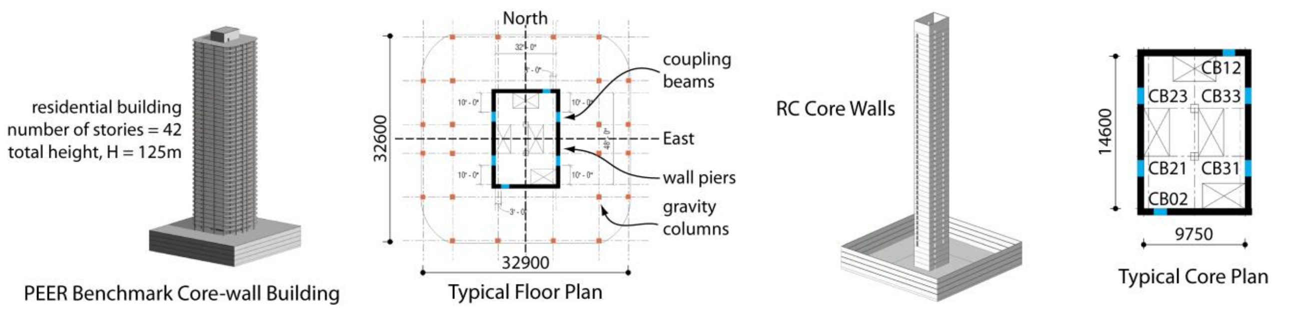

Figure 2. PEER TBI core-wall building. (adapted from Moehle et al. [28])

To ensure an effective rocking response, the MechRV3D system was assigned a lateral resistance that is greater than the base shear expected at the onset of rocking. These lower-bound shears were evaluated as 9.6MN (EW) and 14.4MN (NS), given M_{rock}^{rNS}=800MN-m and M_{rock}^{rEW}=1200MN-m and the assumed effective height of 2H/3, where H is the height of the building above the ground. The upper-bound lateral resistance was obtained from a special *rocking-only* scenario, labelled as *1M0V*, where only the rocking action (*1M*) of the MechRV3D system is activated, while the shear mechanism is assumed elastic (*0V*) allowing for base shears of the core to reach maximum possible values including higher-mode contributions that can be excited at the MCE level. These upper-bound shears, denoted as V_{1M0V}, were evaluated as 67.0MN (EW) and 64.7MN (NS). In selecting a proper lateral resistance within the bounded range, there is a trade-off between controlling base displacements of the core and limiting higher-mode contributions. To find a balanced design, ultimate lateral resistance, V_u, ranging from 0.5× to 0.8×V_{1M0V}, were considered, as listed in Table 1. According to the designated V_u, BRBs were proportioned with the areas of the yielding segments that are listed in Table 1. In this design, eight 7.5m-tall BRBFs were used to carry the entire lateral force in each of the principal directions. The BRBs were inclined at an angle of α=45° and made of LYP100 steel for which F_y=100MPa and R_y= 1.2 were assumed.

Table 1. Design Parameters for the Shear Mechanism

V_{1M0V}	$\kappa_V=V_u/V_{1M0V}$	Direction	V_u	$A_{y,BRB}$
67.0 MN (EW) 64.7 MN (NS)	0.8	EW	53.6 MN	585 cm^2
		NS	51.8 MN	566 cm^2
	0.7	EW	46.9 MN	434 cm^2
		NS	45.3 MN	420 cm^2
	0.6	EW	40.2 MN	347 cm^2
		NS	38.8 MN	336 cm^2
	0.5	EW	33.5 MN	281 cm^2
		NS	32.4 MN	274 cm^2

ADVANCED MODELLING OF THE BENCHMARK BUILDING AND VALIDATION

The benchmark building was represented using a three-dimensional nonlinear model that only included the core for simplicity. This model was built in OpenSees [29] following the wide-column frame analogy (WCFA) method, as schematically shown in Figure 3. Core walls were modelled using vertical force-based beam-column elements made of nonlinear fibre sections. The shear component of wall sections was modelled as elastic with the flexure-shear-interaction not considered. At floor levels, rigid horizontal links extended out from the vertical elements, simulating the wide-column effect. The free end of these links was connected to the adjacent coupling beam which was modelled using a compound element consisting of an elastic beam and a shear hinge at the mid-span. In modelling the RC core, references were made to the recommendations that were proposed in the PEER TBI guidelines [30] and by Naish [31].

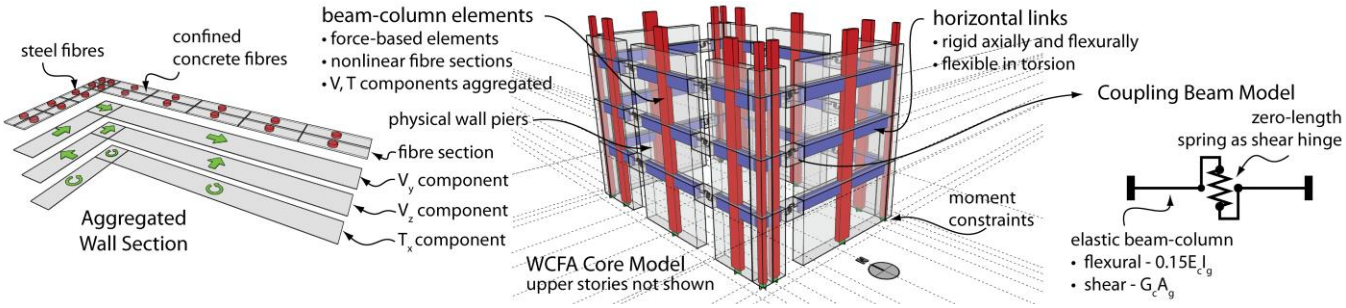

Figure 3. WCFA model of the PEER benchmark building.

P-Δ effects were accounted for using leaning columns. Rigid diaphragms were defined at elevated floors. Seismic masses and rotational inertia were assigned to the master node of the floor diaphragms. Rayleigh damping proportional to mass and stiffness was used with a damping ratio of 2.5% assigned at periods of 1 sec and 5 sec, which is consistent with the two reference analyses [28,32] used subsequently for validating the WCFA model. A suite of seven ground motions was selected by MKA for design assessments. MacKay-Lyons [32] used the same set of records to verify an independent model of the same building. This paper adopted the records used in [32] for the NLRHA as listed in Table 2.

Table 2. Ground Motion Records and Scaling Factors (adapted from [32]).

No.	Event	Year	Magnitude	Station	MCE Scale Factor
1	Superstition Hills, California	1987	6.54	Parachute Site Test	1.24
2	Northridge, California	1994	6.69	Sylmar - Converter Station	1.24
3	Loma Prieta, California	1989	6.93	Saratoga - Aloha Ave	2.99
4	Duzce, Turkey	1999	7.14	Duzce	0.89
5	Landers, California	1992	7.28	Yermo Fire Station	1.68
6	Kocaeli, Turkey	1999	7.51	Izmit	2.41
7	Denali, Alaska	2002	7.90	TAPS Pump Station #9	3.09

The WCFA model was validated against the NLRHA that MKA [28] and MacKay-Lyons [32] conducted using Perform3D separately. Fundamental periods and peak responses, as listed in Table 3, and height-wise envelopes of peak responses (not shown) are found reasonably close to those obtained in the reference analyses. Hence, this WCFA model is considered as a good representation of the benchmark building and used consistently throughout all analyses that are subsequently presented.

Table 3. Mean Peak Responses of the Benchmark Building at the MCE Level.

Responses	Variables	OpenSees Analysis	MKA [28]	MacKay-Lyons [32]
Periods	T_{EW}	4.1 sec	4.2 sec	4.4 sec
	T_{NS}	3.2 sec	3.4 sec	3.3 sec
	T_{tor}	2.6 sec	2.3 sec	2.2 sec
Base Shears	$V_{b,EW}$	58.3 MN	61.0 MN	66.1 MN
	$V_{b,NS}$	54.7 MN	61.8 MN	62.0 MN
Base OTMs	$M_{b,rNS}$	1716 MN-m	1664 MN-m	n.a.
	$M_{b,rEW}$	2323 MN-m	2209 MN-m	n.a.
Inter-storey Drift Ratios (IDRs)	$\delta_{s,EW}$	2.5%	2.0%	2.5%
	$\delta_{s,NS}$	1.6%	1.3%	1.8%
Peak Floor Accelerations (PFAs)	a_{EW}	0.74 g	0.75 g	1.04 g
	a_{NS}	0.70 g	0.70 g	0.81 g

MODELLING OF THE MECHRV3D SYSTEM

As shown in Figure 4, the rocker of the MechRV3D system was modelled using rigid frame elements that established a cage-like enclosure simulating the idealized rigidity of the block and the encasement it provides to the core walls. The mega-columns were modelled using elastic beam-column elements using the transformed properties of the composite section. The rolling joints at each end of these columns was simulated using a zero-length section consisting of multiple fibres that were arranged on a circular grid, as shown in Figure 4. These fibres were set as tension-free and rigid in compression. They must undergo a preset gap distance before they can be engaged to carry loads in compression. These gap distances were carefully determined, as illustrated in Figure 4, to simulate the geometry of the spherical cap of the mega-columns. Sliding and spinning were restrained in this fibre section using aggregated elastic components of shear and torsion.

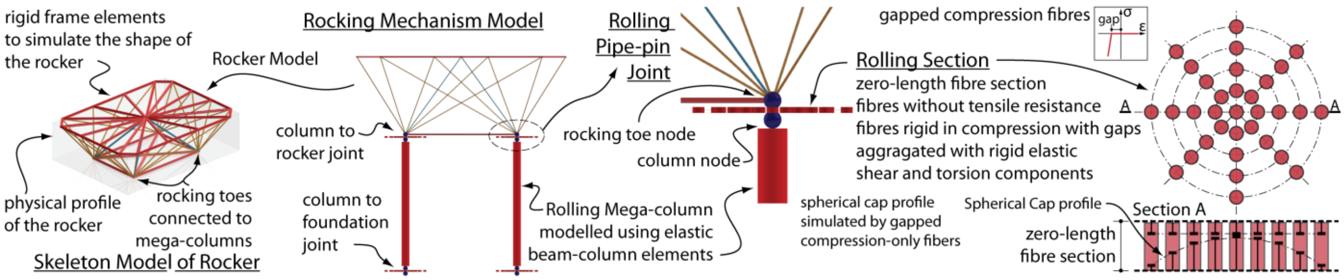

Figure 4. Schematic model of the rocking mechanism.

The BRBFs were modelled as pin-connected at the column bases and beam-to-column joints. Their out-of-plane stiffness was neglected such that lateral forces in one direction would be entirely carried by the frames in the same direction. The BRBFs in the both principal directions were interconnected at the beam-to-column joints using rigid truss elements, which simulated the diaphragm action of the skirt. The gear teeth linking the BRBFs and the rocker were modelled using zero-length springs that were assigned no tensile resistance but a large rigidity in compression. The BRBs were modelled using fiber-based truss elements. The fibres were defined using the uniaxial material model, *Steel4*, that was formulated by Zsarnóczay [33] in OpenSees [29]. Material parameters were selected as recommended by Zsarnóczay [33] accounting for the asymmetrical hardening in tension and compression. The BRB model was calibrated against the BRB tests carried out by Zsarnóczay [33].

ENCHANCED SEISMIC PERFORMANCE OF THE 1M1V-BASED BENCHMARK BUILDING

The physical embodiment of the MechRV3D system is referred to as *1M1V*, since the both mechanisms were designed to be activated. Seismic responses of the 1M1V-based benchmark building were compared with those of the fixed-based reference structure through NLRHA that were conducted at the MCE level using the same suite of ground motions as listed in Table 2. In these analyses, the rocking moment was set equal to OTM_y, while all the design variants of the shear mechanism, as listed in Table 1, were considered. Most of these numerical analyses converged except in the case for the Northridge Earthquake in the design scenario V_u=0.5V_{1M0V}. The blow-out was caused by excessive deformations of the BRBFs. Figure 5 shows V-Δ_{skirt} relations that represent the overall lateral response of the MechRV3D system. At all the lateral strength levels, inelastic excursions of the BRBFs were achieved in both principal directions as intended. Stockier hystereses were seen for the lower strengths while thinner ones for the higher strengths.

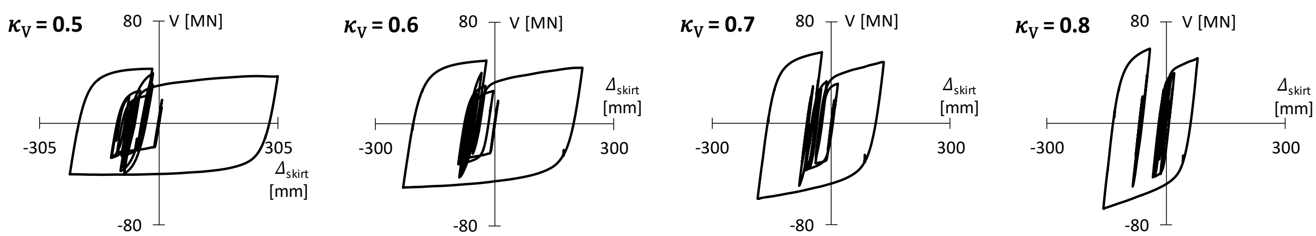

Figure 5. The overall lateral response of the MechRV3D system (Superstition, MCE, east-west).

Table 4 summarizes actual lateral responses of the MechRV3D system that were obtained from the analyses. Mean values of the ultimate lateral resisting force closely matched the designated resistance at each of the lateral strength levels. And these ultimate resisting forces were achieved when the BRBFs underwent the peak drift ratios, as shown in Table 4, that were assumed in the design of the frames. Hence, these design assumptions are verified. In the case of V_u=0.6V_{1M0V}, the BRBFs drifted by 2.9% (EW) and 3.3% (NS, not shown), which resulted in mean strains of 2.8% (EW) and 3.2% (NS) in the BRBs. The mean-plus-one-standard-deviation stains reached 5.1% (EW) and 6.2% (NS), which, according to Tremblay et al. [34] is realistically achievable using BRBs made of high-ductility Japanese steel grades.

Table 4. Ultimate Lateral Responses of the MechRV3D system in the EW Direction.

Shear Reduction Factor, κ_V	0.8	0.7	0.6	0.5
Design Lateral Resistance, $V_u = \kappa_V V_{1M0V}$	53.6 MN	46.9 MN	40.2 MN	33.5 MN
Mean Lateral Resistance obtained from the NLRHA	52.2 MN	46.8 MN	40.6 MN	34.6 MN (with divergence)
Assumed BRBF Drift Ratios	1.5%	2.0%	3.0%	4.0%
Mean BRBF Drift Ratios obtained from the NLRHA	1.5%	2.1%	2.9%	2.5% (with divergence)

Figure 6(a) shows critical strain demands at the MCE level. At the NW corner of the core, while the yielding of rebars was totally avoided at the lower half of the building, the activation of the shear mechanism led to reductions in steel strains around the mid-height and in the stories above. Smaller strains were obtained as V_u was reduced. In the case of V_u=0.6V_{1M0V}, rebars around the mid-height remained totally in the elastic range. This was essentially achieved in upper stories as well despite a slight exceedance at the 31st floor.

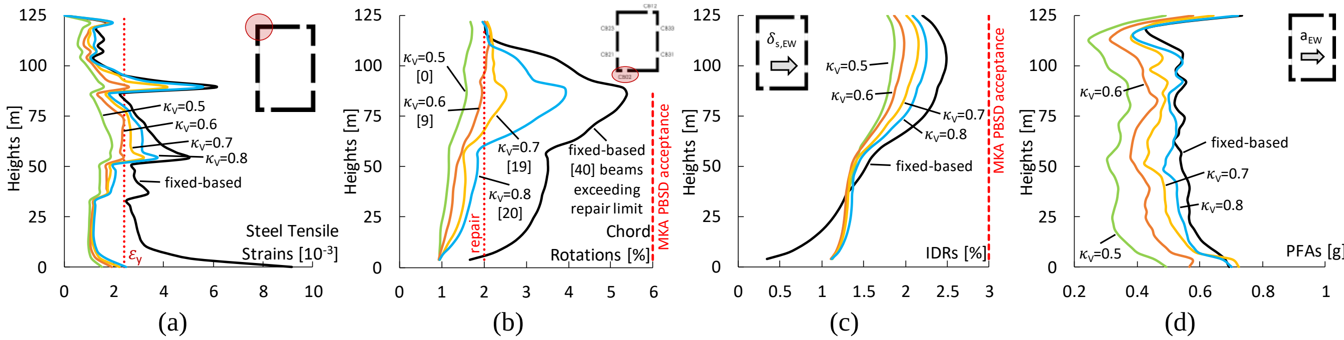

Figure 6. Mean responses associated with damage: (a) steel strains, (b) coupling beam rotations, (c) IDRs, (d) PFAs.

As can be seen from Figure 6(b), no coupling beams in the fixed-based building exceeded the maximum acceptable rotation of 6% as adopted in [28]. However, a large number of them suffered rotations over 2% which, as suggested by Naish [31], is the limit beyond which substantial repair would be required. In the 1M1V-based building, the number of beams that require substantial repair was largely reduced, e.g. the beam on the south elevation as shown in Figure 6(b). In the case of V_u=0.6V_{1M0V}, the repair limit was surpassed only at 9 floors compared to 40 floors in the fixed-based building.

IDRs are checked to ensure minimal cracking in partitions and claddings, as plotted in Figure 6(c). In the 1M1V-based building, IDRs kept declining as the design lateral resistance decreased. When V_u=0.6V_{1M0V}, the maximum IDR dropped from 2.5% to 2.0%. Highly influenced by the base rocking, the IDRs were higher at lower stories in the 1M1V-buildings than in the fixed-based structure. However, the increased IDRs were less than 1.5%. As for the acceleration-dependent damage to non-structural elements, PFAs were significantly reduced in the 1M1V-based building, as shown in Figure 6(d). Again, 0.6V_{1M0V} was found to be the optimal design choice as it achieved nearly 50% reduction in PFAs.

CONCLUSIONS

Aiming for low-damage seismic design, this paper proposed the MechRV3D system that consists of independent rocking and shear mechanisms that cap off flexural and shear seismic demands at the base of high-rise buildings. The proposed system is realistically implemented with the detailed design provided in this paper. In this physical embodiment, components of the dual-mechanism were built using materials and elements that have been used in practice.

As an example, a 42-storey RC core-wall building was used to validate the MechRV3D system through extensive NLRHA from which the enhanced seismic performance was verified. In these analyses, varied design options were examined parametrically. While a rocking moment equal to OTM_y was chosen to emulate the conventional design, V_u of 0.6V_{1M0V} was found to be the balanced design for the shear activation in terms of the efficiency in both higher-mode mitigation and base movement control.

In a more general sense, the proposed system can be applied for any type of slender structure that can be capacity designed to the rocking moment and the lateral resistance assigned to the dual-mechanism. Further theoretical studies are being conducted to determine optimal design parameters for the MechRV3D system with respect to buildings of different heights and varied seismicities. Advanced finite element analyses are also being carried out to further verify local and global responses of the MechRV3D system even though the skeleton models currently used in this paper reliably captured the intended mechanics of the system. The feasibility of the proposed system will also be validated through large-scale shaking-table tests for which hybrid simulation techniques can be used considering the size of high-rise structures of interests.

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