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# MODELLING OF 3D ROLLING COLUMNS IN UNCOUPLED BASE DUAL-MECHANISM FOR HIGH-RISE BUILDINGS

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## Abstract

In the context of the accelerating global urbanization, seismic resilience of infrastructure and urban habitat has drawn increasing research interests. To accommodate the soaring urban population given the scarcity of land, high-rise buildings have been constructed in a record-breaking pace all over the world including seismically active areas. While ensuring life-safety under major earthquakes, current seismic design codes do not preclude extensive damage to structures that can lead to high-rise buildings being unsafe for immediate occupancy or even more cost-effective to be demolished.

Aiming at low-damage design, the authors of this paper have proposed an uncoupled base rocking and shear mechanism system for reinforced concrete (RC) core-wall high-rise buildings. While the rocking mechanism limits overturning moments and eliminates flexural hinges at the base of structures, the shear mechanism mitigates higher-mode effects that can otherwise cause unintended inelasticity and brittle failure throughout the height of structures. This proposed system has been validated through extensive numerical analyses.

In this validation process, one of the challenging tasks is to properly account for three-dimensional motions of unconventional mega-columns that are introduced as parts of the rocking mechanism. These mega-columns are specially detailed such that they can roll and wobble while providing supports to the rocking component. This paper proposes a modelling technique in which only frame elements and nonlinear fibre sections are used to simulate this complex three-dimensional action. In a two-step development procedure, a rocking column model is first proposed, numerically validated, and then extended to a rolling column model. The latter is then incorporated into a global model that is built for a high-rise building with the proposed dual-mechanism system at the base. Nonlinear response history analyses (NLRHAs) are conducted using this integrated model. From the response of the mega-columns and the overall mechanism system, it is confirmed that the intended rolling action of the mega-columns is achieved. This facilitates the effective engagement of the dual mechanism, which reduces structural responses that cause damage.

*Keywords: rolling columns; rocking columns; nonlinear modelling; uncoupled dual-mechanism; high-rise buildings*

## 1 Introduction

The rapid growth of the world's population and the accelerating global urbanization lead to a critical challenge to house the soaring number of urban residents. Adopted as an efficient solution, construction of high-rise buildings has been increased all over the world including in urban areas that are earthquake prone. Modern seismic design philosophies prioritize life safety during major earthquakes. Whereas this objective can be reached by preventing overall collapse, structures designed to current codes are still expected to sustain extensive damage for which repair or replacement may be impractical and uneconomical. As a result, buildings with extensive damage may have to be demolished. This can lead to tremendous indirect losses and environmental impacts.

A variety of high-performance systems have been studied for high-rise buildings aiming at resilient structures. Rocking precast wall panels were investigated in the PRESSS program [1, 2]. This concept of base rocking was further studied for RC core-wall buildings [3], and used in practice for retrofitting [4] and new constructions [5]. However, numerical analyses [3, 6] and experimental studies [2] indicated that a base rocking system alone is inefficient in reducing higher-mode responses that are particularly conspicuous in high-rise buildings, leading to distributed damage throughout the structure. Panagiotou et al. [7] and Wiebe et al. [8] respectively suggested to introduce additional plastic hinges and mechanical rocking joints into structures for limiting the higher-mode response. However, both studies [7, 8] proved that multiple flexural mechanisms are more efficient in reducing overturning moments than shear demands. For better limiting higher-mode shears, Wiebe et al. [8] further proposed to incorporate a nonlinear brace in the base storey of controlled rocking steel frames that were designed to carry no gravity load. However, it is more challenging to realize this planar dual-mechanism system in core-wall structures that act in three dimensions and undertake larger displacement demands at the base. For a better mitigation of higher-mode effects, base-isolation was applied for high-rise buildings despite the reduced effectiveness in the period-shifting effect when considering long periods of these structures [9-11]. However, a few studies [12, 13] have raised concerns about overloading that may happen to base isolators that are subjected to tension or overstressed in compression while undergoing large shear deformations. These concerns, as Becker et al. [11] concluded, explain why base isolation is of limited use for high-rise buildings outside of Japan. To alleviate the tension in isolators, strategies were attempted to make isolators tension-resistant [14] or allow for uplifting at the bottom of isolators [15] or at the base of the superstructure [16]. Nevertheless, flexural and shear responses remain in-series and fully coupled, making it challenging to design against compressive overloading.

To this end, the authors of this paper recently proposed an innovative system consisting of rocking and shear mechanisms that are incorporated at the base of high-rise buildings. These two mechanisms are physically separated and behaviourally uncoupled. While the rocking mechanism limits base overturning moments, the shear mechanism is dedicated to reduce lateral forces that can otherwise be significantly amplified due to higher-mode effects. Each of the mechanisms is designed not to carry seismic actions that the other mechanism is intended to resist. To achieve this design intent for the rocking mechanism, mega-columns with a spherical cap at both ends are introduced to minimize the shear resistance through rolling action that marks one of the characteristic novelties of the proposed system.

This paper focuses on the numerical simulation of the spherically capped mega-columns. A new modelling approach is proposed for capturing the three-dimensional rolling motion of these elements. This is one of the critical steps in validating the overall base-mechanism system. This paper firstly provides a brief introduction of the uncoupled dual-mechanism system. This overview is followed by a detailed discussion about the proposed modelling technique which was inspired by an existing method [17] that was developed to model two-dimensional rocking blocks. This paper extends this model into a three-dimensional format to capture the rocking motion of a freestanding cylinder. After numerically validating it, this three-dimensional rocking model is further modified to allow for the intended rolling action. Finally, the rolling mega-columns are incorporated into an advanced nonlinear model that includes a 42-storey RC core-wall building and the base dual-mechanism system. This integrated model is used in NLRHAs that are conducted for verifying the efficiency of the proposed system in achieving low-damage performance.



## 2 Overview of the Uncoupled Base Rocking and Shear Mechanism System

### 2.1 Rocking Mechanism and Rolling Mega-columns

The proposed system is located in the basement of buildings, with the rocking mechanism at the centre and the shear mechanism at the periphery, as shown in Fig. 1. This system is named *MechRV3D* which indicates the rocking (*R*) and shear (*V*) mechanisms (*Mech*) assembled in a three-dimensional (*3D*) configuration. The rocking mechanism consists of a solid concrete block, on top of which RC core walls of the building are anchored. At the rest position, this block is supported by four mega-columns. Once overturning moments at the base of the RC core reach a designated threshold, the block is allowed to uplift atop these mega-columns and rock back and forth. Hereafter, this rocking block is referred to as the *rocker*.

The mega-columns are composite elements with a tube-in-tube built-up, as shown in Fig. 1. They are reinforced using multiple steel sections and confined by steel tubes to achieve high load-bearing capacities. Rather than being cast-in-situ, each of these mega-columns is dry plugged into a *socket* in the soffit of the rocker through a *rolling pipe-pin joint*, as shown in Fig. 1, which was developed for the MechRV3D system. This specially detailed joint includes a series of *concrete-filled steel pipes* that are extended from the perimeter of the mega-columns and inserted into a set of *inverted steel cans* that are embedded in the socket. The pipes and cans carry no load in the vertical direction and are free to move relative to each other, allowing the rocker to uplift atop the mega-columns. The length of the pipes is carefully determined such that the portion in the cans is sufficiently longer than the expected uplifting distance. As such, laterally, the pipe-and-can pairs act as dowels, preventing the mega-columns from sliding off even if the rocker lifts up.

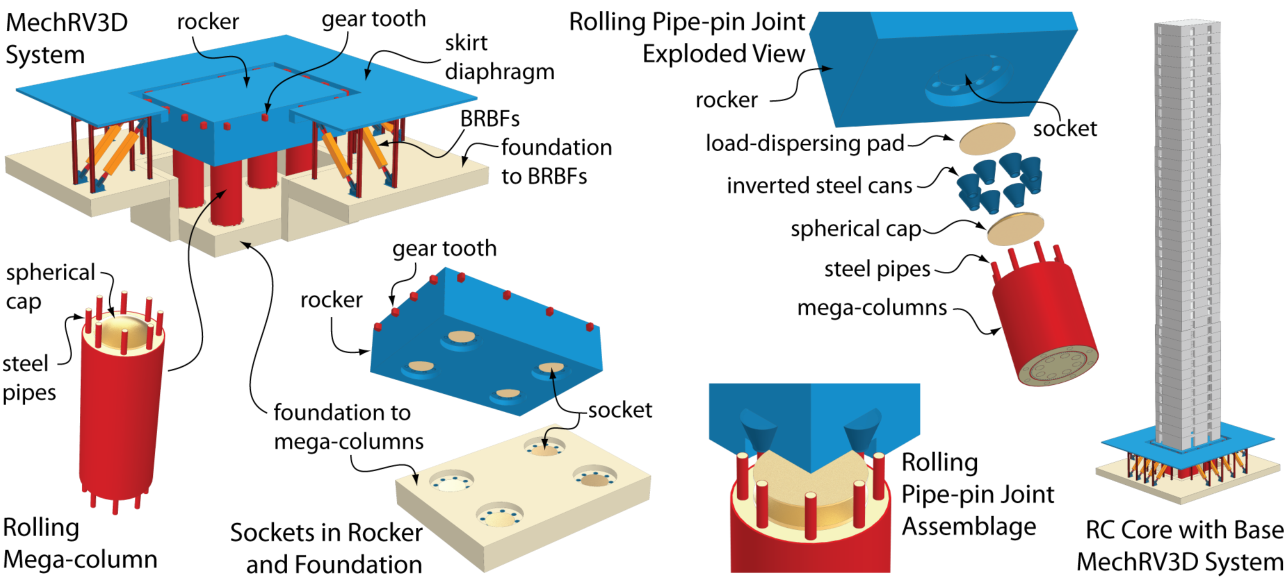



Fig. 1 – Configuration of the proposed MechRV3D System.

Gravity loads from the RC core are transferred to the mega-columns through a *spherical cap*, which can be made of high-strength machined or cast steel and is concentrically anchored to the columns, as shown in Fig. 1. When the whole joint undergoes rotations, this spherical cap rolls against the rocker within the socket unless they are detached due to the uplifting of the rocker. This rolling of a curved surface against a plane creates minimal rotational restraint. No moment resistance is generated by the pipes and cans at the perimeter given their unrestricted vertical mobility. This moment-free design is further enhanced by enlarging the cans at the free end of the pipes to avoid the latter being bent. The proposed rolling pipe-pin joint, in a flipped configuration, is also used at the foundation level where the mega-columns roll in sockets free of moment but never lift off, since they are loaded in compression only. Moment-free at both ends, the mega-columns carry no lateral forces and are referred to as the *rolling mega-columns*.



## 2.2 Shear Mechanism

As shown in Fig. 1, the shear mechanism consists of a *skirt diaphragm*, which, in reality, is the ground floor slab outside of the core area, and a series of *buckling-restrained braced frames* (*BRBFs*) that are arranged between the skirt diaphragm and the foundation. Base shears of the RC core arising at the top face of the rocker are entirely transferred to the skirt diaphragm which then engages the BRBFs to provide lateral strength, stiffness and post-yield ductility in both principal directions. The rocker and the skirt diaphragm are not cast monolithically but linked through gear-tooth-like connections which are arranged in alignment with the BRBFs. These gear teeth work in contact and transmit forces and motions within the horizontal plane. They are however not restrained vertically and thereby allow for rocking to occur at the base of the core. By these means, the BRBFs serve as shear fuses without interfering with the rocking mechanism.

## 2.3 Three-dimensional Rolling Frame

The rocker and the rolling mega-columns form a *three-dimensional rolling frame* that can sway in any horizontal directions providing minimal lateral resistance to the superstructure, as illustrated in Fig. 2. The rolling frame displays negative lateral stiffness due to P-$\Delta$ effects induced by the rolling mega-columns. This negative stiffness is intended to offset the post-yielding overstrength of the shear mechanism, which allows for a better limit on base shears of the RC core and consequently of the higher-mode effect.

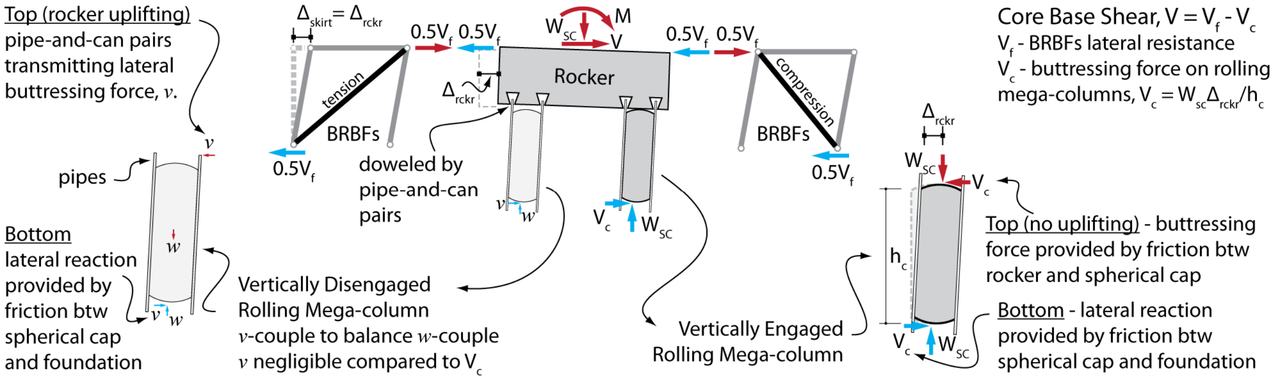



Fig. 2 – Three-dimensional rolling frame with negative lateral stiffness.

# 3 Numerical Modelling of the Rolling Mega-columns

Given the articulated joints, the proposed rolling mega-columns are unconventional compared with cast-in-situ columns or rocking columns that are flat-ended, which makes it challenging to model these elements. Whereas using solid finite elements would be rather accurate, it is computationally expensive for the overall model in which distributed plasticity of the superstructure must be taken into account. To this end, this paper proposes a simplified modelling technique by which only frame elements and fibre sections are needed. The development of this approach starts from a two-dimensional rocking model that Vassiliou et al. [17] proposed and then goes through a two-step procedure as discussed subsequently.

## 3.1 In-plane Rocking Model

Vassiliou et al. [17] proposed a model to capture the response of in-plane rocking systems using OpenSees [18], as shown in Fig. 3. Following this approach, the rocking body is represented using beam-column elements that can be elastic or nonlinear. At the base of the rocking body placed are two nodes that have identical coordinates. One node is connected to the beam-column element, and the other is fixed to the foundation. These two nodes are linked using a zero-length section element that simulates the rocking surface. This zero-length section is built using a row of nonlinear fibres that are assigned no resistance in tension and an elastic response in compression.



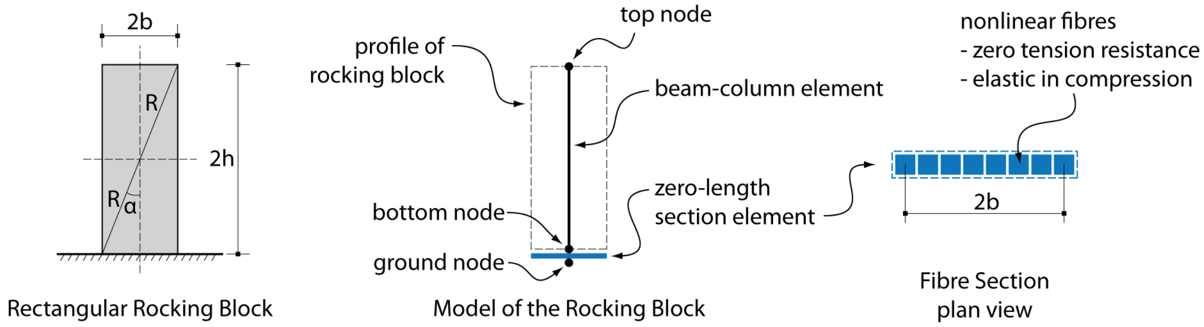



Fig. 3 – OpenSees model for capturing in-plane rocking action (redrawn after [17]).

Vassiliou et al. [17] numerically validated this model through simulating the free rocking motion of a rigid block that is released from an inclined position. The analysis results were compared with the theoretical solution derived with the Housner [19] model, from which a close match was obtained. In these analyses, sensitivity studies were conducted accounting for the influence of the number of the fibres, mesh size of the rocking body, integration time step, and some analysis parameter. It was concluded by Vassiliou et al. [17] that two fibres are adequate to capture the rocking action, and responses are essentially identical when five or more beam-column elements are used for the rocking body. Vassiliou et al. [17] also pointed out that the model is insensitive to the integration time step as long as it is sufficiently shorter than the periods of dominant motion components. Vassiliou et al. [17] recommended to use the Hilber-Hughes-Taylor (HHT) integration algorithm formulated in OpenSees [18] and indicated that the induced numerical damping dissipates the kinetic energy contained in high-frequency impact waves in the rocking body but does not affect the rocking motion whose frequency is relatively low. This is confirmed since the predicted responses matched the Housner model closely as long as the dissipation factor $\alpha_b$ is smaller than 1.

## 3.2 Three-dimensional Rocking Model

The two-fibre rocking section only allows a rectangular block to pivot in a single vertical plane about edge toes at the base. However, it cannot account for rocking modes of a freestanding cylinder whose motion can be viewed as a composition of three elemental rotations including *nutation*, *precession*, and *spinning* that can occur simultaneously. To capture these complex motions, this paper extends the planar rocking model into a three-dimensional formation, as shown in Fig. 4.

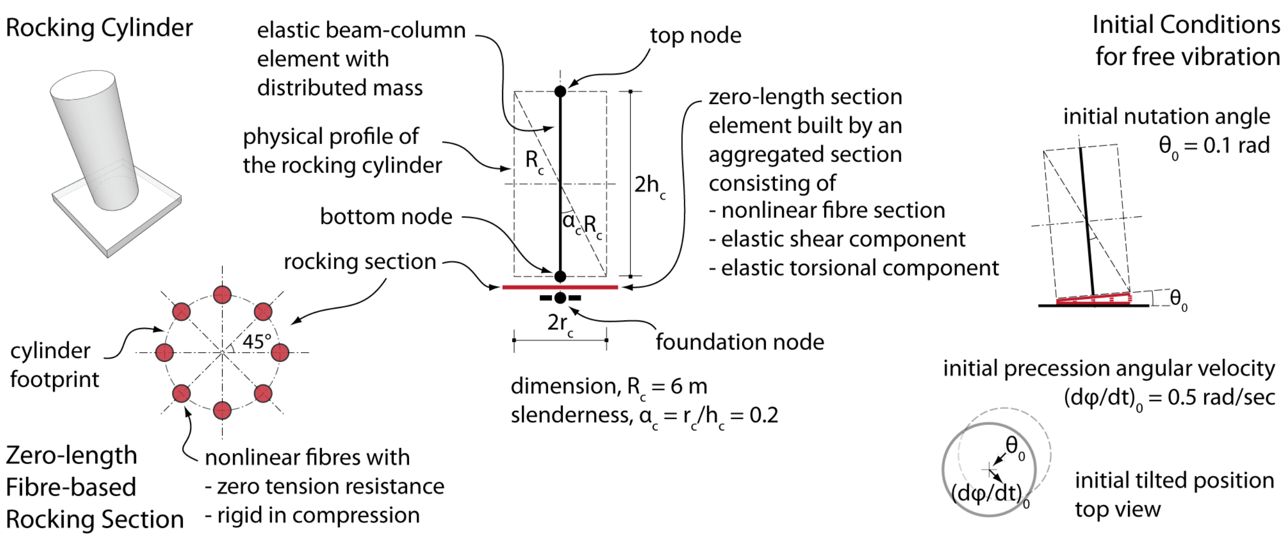



Fig. 4 – The proposed model for a three-dimensional rocking cylinder.



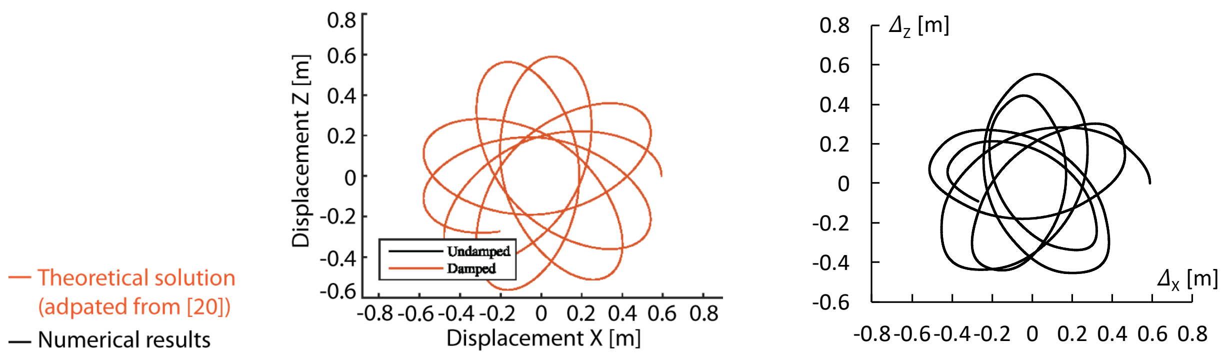
 

In this extended model, the cylinder is represented using one single elastic beam-column element. This is less than five as recommended in [17], but does not significantly affect the accuracy as can be seen in later numerical validation. Using a single element is the only choice in order to impose the initial conditions, $\theta_0$ and $(d\varphi/dt)_0$ as denoted in Fig. 4, on a free rocking cylinder as will be explained later. Uniformly distributed mass is applied along the height of this beam-column element. However, the moment of inertia of this element about its base point is different from that of the physical cylinder pivoting about edge toes. This is compensated by assigning a moment of inertia at the top and bottom nodes of the beam-column element.

The rocking surface at the base is still modelled using a zero-length section element. This section takes a circular shape that is identical to the cross-section of the cylinder. Eight fibres are evenly distributed on the perimeter. Each fibre is assigned an area equal to 1/8 of the cross-sectional area of the cylinder. All the fibres are tension free and elastic in compression. The compressive stiffness of each fibre is set to be ten times the axial stiffness of the cylinder. Rigid-elastic shear and torsional components are aggregated with the fibre section such that sliding and spinning are restrained at the rocking surface. No Rayleigh damping is included in the nonlinear fibres that are expected to undertake abrupt stiffness changes.

For validation purposes, this three-dimensional rocking model was used to predict the free vibration of a cylinder. As shown in Fig. 4, this cylinder has a dimension $R_c$ = 6 m and a slenderness $\alpha_c$ = 0.2. It was released from an initial nutation angle $\theta_0$ = 0.1 rad and an initial precession angular velocity $(d\varphi/dt)_0$ = 0.5 rad/sec. The loading pattern of multiple support excitations formulated in OpenSees [18] was used to apply these initial conditions as imposed motions on the beam-column element that represents the cylinder. If more than one element was used, these imposed motions could not be reflected accordingly over the height of the cylinder. This explains why a single element was used here.

The free vibration responses are shown in Fig. 5, in comparison with the results that Vassiliou et al. [20] obtained using a theoretical model. In the latter study, closed-form equations of motions and analytical solutions were derived for a rigid cylinder that is allowed to rock and wobble in three dimensions on a rigid surface without any damping mechanism involved. While this analytical model can be viewed as a three-dimensional extension of the planar Housner rocking model [19], it differs from the latter in the sense that the energy dissipation mechanism accounted in the Housner model [19] through a coefficient of restitution is not available in the former. As can be seen from Fig. 5, the numerical results reasonably match the theoretical solutions in terms of movement trajectories, angles and angular velocities of both nutation and precession. However, some discrepancy is seen in time histories of the nutation angle. While peak amplitudes are constant in the theoretical solution, the numerical result displays some decay. This is expected since the analytical model accounts for no energy dissipation, but, as pointed out by Vassiliou et al. [17], in the numerical analysis, kinetic energy can be transformed from the rocking motion to the form of impact waves in the rocking body. The impact waves are high-frequency and dissipated by the numerical damping induced by the HTT algorithm. As a result of the decay in nutation angle, the orbit recorded at the mid-height of the cylinder is not as polar symmetrical in the numerical analysis as in the theoretical solution.

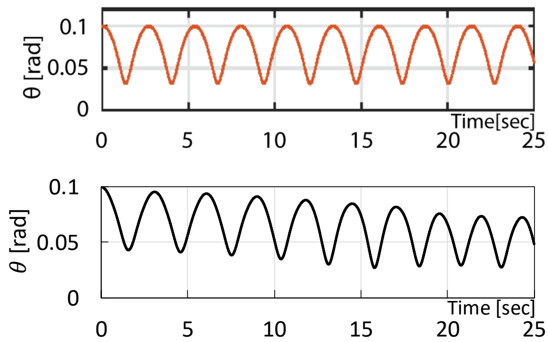



(a) Orbits recorded at the mid-height of the cylinder.

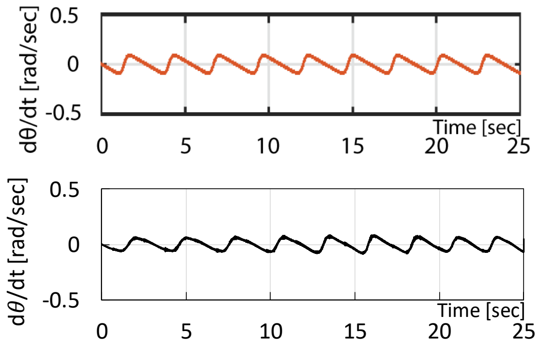
Fig. 5 – Validation of the three-dimensional rocking model against theoretical solutions.



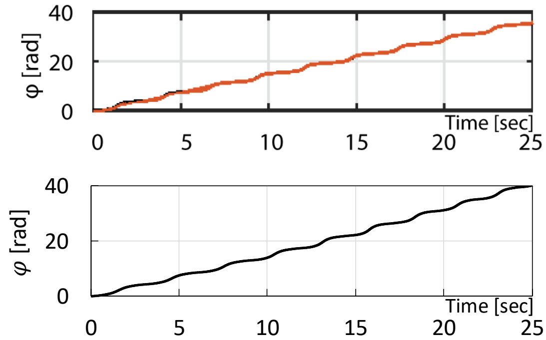
 



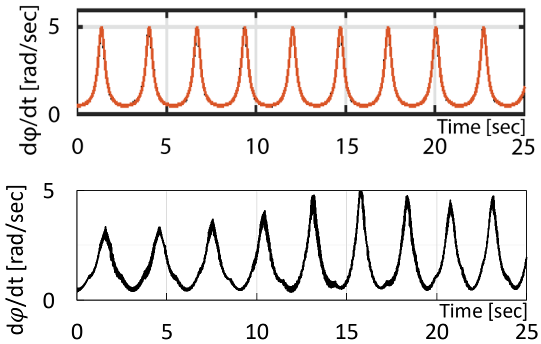

(b) Time histories of the nutation angle.




(c) Time histories of the nutation angular velocity.




(d) Time histories of the precession angle.




(e) Time histories of the precession angular velocity.

Fig. 5 (continuous) – Validation of the three-dimensional rocking model against theoretical solutions.

## 3.3 Rolling Section

The rocking model developed in the previous section is further modified to capture three-dimensional rolling motions. In the rocking surface, multiple layers of fibres are introduced and arranged on a polar grid, as shown in Fig. 6. All the fibres remain compression-only but have to undergo varied gap distances before they can be engaged to carry loads. These gap distances, denoted as $g$, are determined based on the geometry of the spherical cap, as illustrated in Fig. 6. By these means, the modified fibre section simulates a smooth transition of contact point when the spherical cap rolls against the rocker or the foundation, and is referred to as the *rolling section*. Sliding and spinning are also restricted by aggregating rigid elastic shear and torsional components to the fibre section. These assumptions are reasonable given the physical dowel action provided by the pipe-and-can pairs.

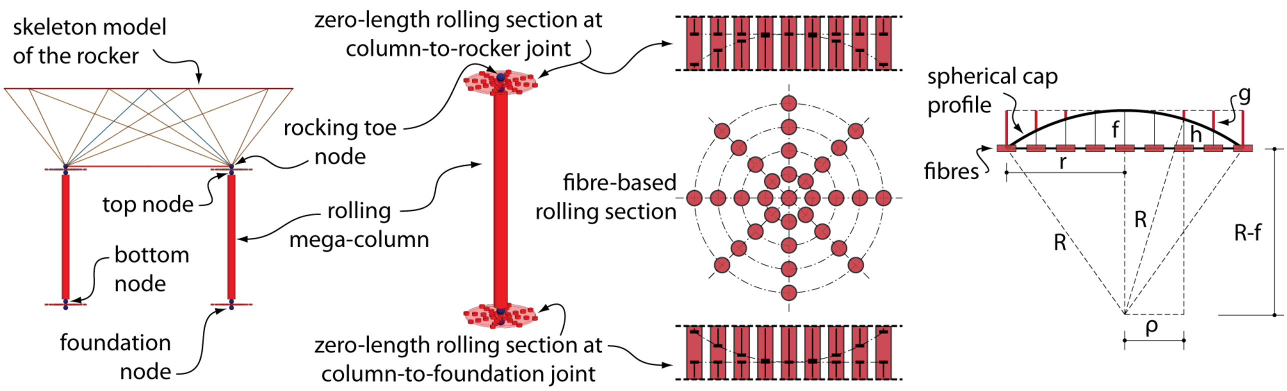



Fig. 6 - Schematic model of the fibre-based rolling section.

## 4 Design of the MechRV3D System for a Benchmark Building

The proposed model for rolling action was applied to the mega-columns in the MechRV3D system when this system was numerically validated. For this validation purpose, the MechRV3D system was designed for a benchmark building that has been studied by Magnusson Klemencic Associates (MKA) as part of the Tall Buildings Initiative project [21] launched by the Pacific Earthquake Engineering Research Centre (PEER). This benchmark building is a 42-storey residential development located in Los Angles, California, where high seismicity is expected. The lateral-force-resisting system of this building consists of RC walls that are coupled using lintels, forming a closed core at the centre. This central core is about 10 m-by-15 m on plan and 125 m tall above the ground, as shown in Fig. 7. Given the high aspect ratio and the fundamental period of about 4 sec, the benchmark building is susceptible to pronounced high-mode effects [21].

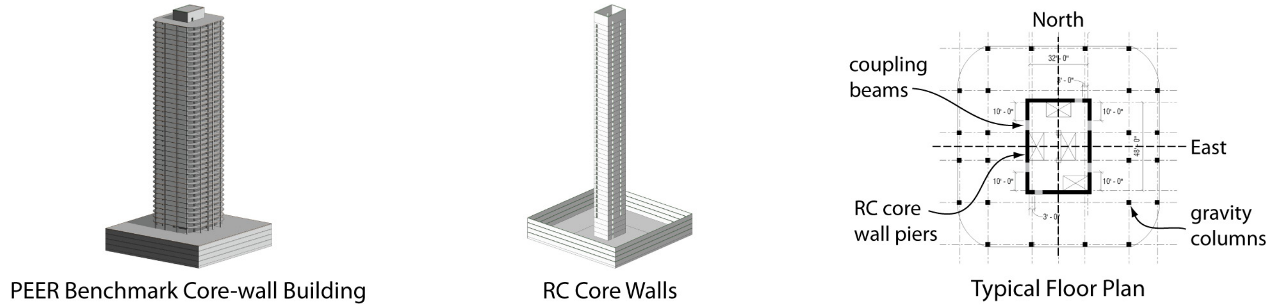



Fig. 7 – PEER benchmark RC core-wall building (adapted from [21]).

Activation moments of the rocking mechanism were set to be 800 MN-m about the north-south (NS) direction and 1200 MN-m about the east-west (EW) direction, both of which are equivalent to the minimum flexural strengths of the benchmark building. These design rocking moments were achieved by setting the centre-to-centre distance between the mega-columns to be 7.2 m (EW) and 10.7 m (NS), given the gravity load of 206 MN from the core plus 10% of this load taken as an allowance for the weight of the rocker. The rolling mega-columns are 8 m tall with an overall section size of 3.5 m diameter. Given the rolling pipe-pin joints, these columns carry compression forces only without being subjected to tension or bending moments. The mega-columns are reinforced using multiple steel sections, such that each of them is adequate to carry the entire gravity load.

The shear mechanism consists of eight BRBFs in each principal direction. These frames are 7.5 m tall and 7.5 wide, with diagonal BRBs inclined 45° towards the rocker. The BRBs were sized to provide ultimate lateral strengths of 40.2 MN (EW) and 38.8 MN (NS). These design strengths are 60% ($\kappa_V = 0.6$) of the base shear demands that were obtained from a reference scenario in which only the rocking action was activated, while the shear mechanism was intentionally set to be elastic such that higher modes were barely affected.

## 5 Nonlinear Modelling of the PEER Benchmark Building and the MechRV3D System

An inelastic model was built for the PEER benchmark building using OpenSees [18]. For simplicity, only the RC core was included and assumed to be fixed at the ground level. Rigid diaphragms were defined at floor levels. Leaning columns were used to account for P-$\Delta$ effects. Seismic masses and moment of inertial were assigned at the centre of the core. The RC core was modelled using the wide-column frame analogy that was experimentally validated by Beyer et al. [22]. Distributed plasticity of RC walls was accounted for using nonlinear beam-column elements composed of fibre sections. The shear component was modelled as elastic with an effective stiffness without considering shear-flexure interaction. Horizontally, rigid arms were extended from the wall elements, simulating actual width of the walls. The free ends of the rigid arms were connected to adjacent coupling beams that were modelled using a compound element consisting of an elastic beam and a shear hinge lumped at the midspan. Modelling parameters were adopted as recommended in the PEER guidelines [23] and by Naish [24]. This model was validated against reference analyses conducted by MKA [21] and MacKay-Lyons [25] and close matches were obtained.



Despite a solid body, the rocker of the MechRV3D system was modelled using frame elements with large rigidities for the sake of computational efficiency. Rigid elements were arranged at the top face of the rocker connecting wall piers and perimeter nodes where the BRBFs are framed in, and at the bottom face linking four rocking toe nodes. The space between these two faces was then enclosed using interweaving elements on side faces, establishing a cage-like skeleton. This near-rigid skeleton carries lateral and vertical forces to the BRBFs and the mega-columns respectively.

The rolling mega-columns were modelled as described in Section 3. Through the proposed rolling section, these columns were connected to the rocker and the foundation as shown in Fig. 6. Each column was represented using five elastic beam-column elements with the axial stiffness amplified to account for the actual rigidity increased by the steel sections. Totally 33 nonlinear fibres were arranged across the rolling section following the pattern as shown in Fig. 6. Each compression-only fibre was assigned an axial stiffness that is $10^2$ times that of a single column, and set to contribute no Rayleigh damping.

The BRBFs were pin connected at the base and beam-to-column joints. The out-of-plane rotation was also released at the column bases. As such, lateral forces developed in one direction is entirely carried by the BRBFs oriented in the same direction, and the load tributary to each frame goes to the BRB only. BRBs were modelled using truss elements composed of nonlinear fibre sections. The BRBFs in both principal directions were interconnected at beam-to-column joints using rigid truss elements to achieve the skirt diaphragm action. The inner beam-to-column joints were connected to the rocker through zero-length gap elements that were assigned no tension resistance but a large rigidity in compression, simulating the gear teeth.

Using these models, NLRHAs were conducted at the MCE level. A suite of seven ground motions that match the response spectrum, as shown in Fig. 8, were used. These records were amplitude-scaled and used by MacKay-Lyons [25] to verify an independent model of the same benchmark building.

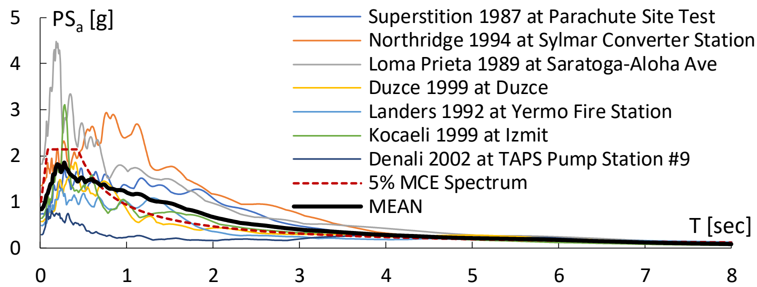



Fig. 8 – Ground motions scaled to match the response spectrum at the MCE level.

## 6 Seismic Response of the Benchmark Building and the MechRV3D System

The seismic response of the benchmark building with the MechRV3D system at the base are compared with that of the fixed-based design, as shown in Fig. 9. It can be seen that storey overturning moments and shears were effectively limited at the base and considerably reduced throughout the height of the structure as results of the engagement of the dual-mechanism.

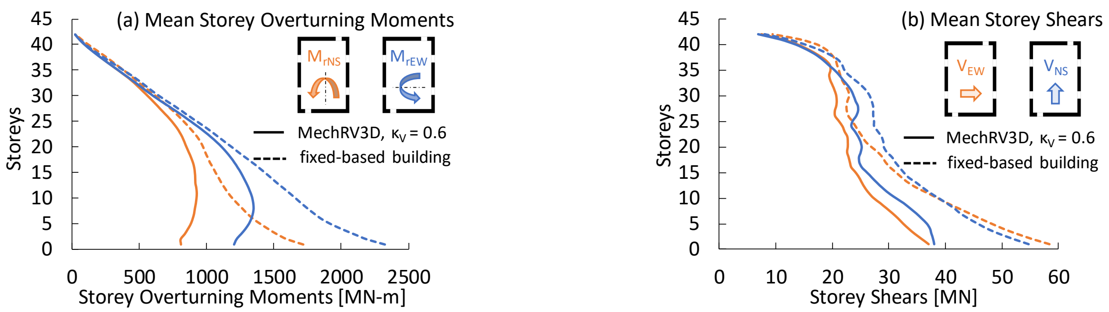



Fig. 9 – Seismic responses of the PEER benchmark building with the MechRV3D system at the base.



Results of the NLRHAs are also investigated to verify if the rolling mega-columns that were modelled using the proposed method act as they are intended. Fig. 10 shows horizontal displacements that are recorded at the top of the four rolling mega-columns under the ground motion Loma Prieta at the MCE level. These columns, respectively indicated in colors red, yellow, green and blue in Fig. 10, share similar trajectories in response to the lateral movement of the rocker. This is ensured by the dowel action of the pipe-and-can pairs which was properly captured in the model. These translational orbits are not expected to be identical due to the torsional motion of the rocker within the horizontal plane.

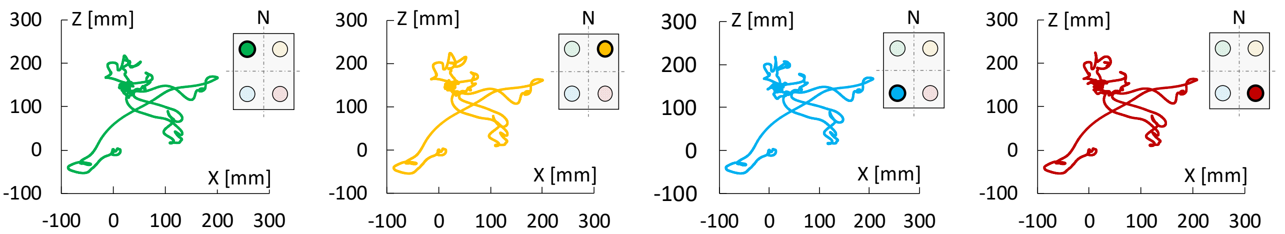
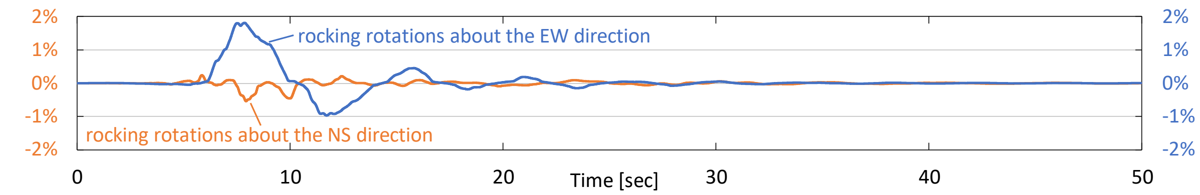
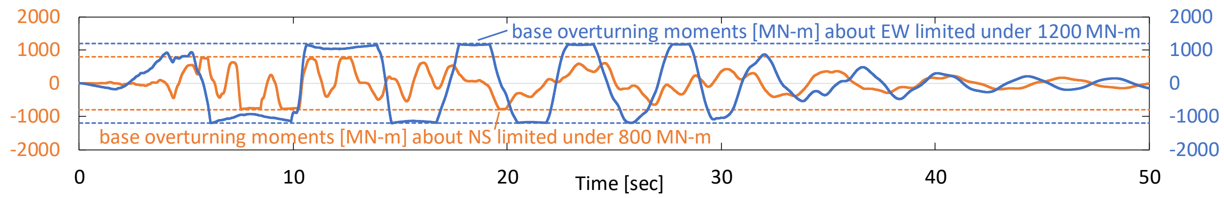



Fig. 10 – Orbits recorded at the top of the rolling mega-columns (MCE, Loma Prieta, Orientation 1).

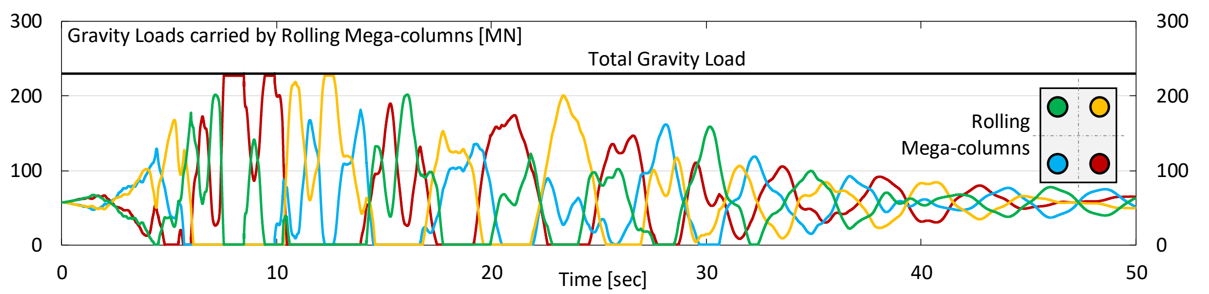
While swaying laterally, the rolling mega-columns allowed the rocker to uplift. This is confirmed in Fig. 11 (a) where angles that opened at the base of the rocker are plotted with respect to time steps. It can be seen that the intended rocking action was activated about both principal directions. As a result, overturning moments at the base of the RC core were limited under the designated rocking moments as indicated in Fig. 11 (b). During the bi-directional rocking motion, gravity loads that were carried by the rolling mega-columns varied with the time, as shown in Fig. 11 (c). In the most critical case, the entire gravity load from the core was carried by one single mega-column which has been designed to be adequate to meet this load demand.




(a) Base rotations of the rocker.




(b) Base overturning moments in [MN-m].




(c) Gravity loads carried by the rolling mega-columns.

Fig. 11 – Response histories of the rocking mechanism under MCE Loma Prieta (Orientation 1).



While the rolling mega-columns allowed for the rocking action, they induced negative lateral stiffness as discussed in Section 2.3 and demonstrated in Fig. 12. These two diagrams show hystereses between the total lateral resistance, $V$, and the horizontal displacement of the skirt diaphragm, $\Delta_{skirt}$. The total resistance (plotted in black) resulted from the lateral resistance provided by the BRBFs, $V_f$ (in blue), and the softening effect, $V_c$ (in red), provided by the spherically capped mega-columns. Due to the induced negative stiffness, the overall hystereses take a shape that is less skewed. This helped impose a more rigorous limit on shear forces that were developed at the base of the core, which is desirable for controlling higher-mode responses.


Fig. 12 – Lateral hysteretic response of the MechRV3D system.

Given the reduced higher-mode dominance and the engaged rocking action, low-damage performance was achieved for the superstructure. This includes the elimination of flexural hinges at base of the walls and the prevention of unintended plastic hinging over the height of the structure. In addition, inelastic rotations of the coupling beams were largely reduced such that no significant repair would be required. At the same time, considerable reductions were also obtained for inter-storey drift ratios and peak floor accelerations, leading to reduced damage to non-structural elements.

## 7 Conclusions

This paper proposed a modelling technique that is capable of capturing the three-dimensional wobbling motion in a computationally cost-effective way that can be implemented in an entire building model. The feasibility of this approach is verified through numerical validation against a theoretical model that was derived for predicting the response of free rocking bodies. In these comparisons, the numerical results well matched the analytical predictions, confirming that the proposed method provides an efficient way of simulating rolling action with reasonable accuracy in lieu of a sophisticated finite element model.

This method was then applied in the numerical validation of the MechRV3D system, an uncoupled dual-mechanism system that the authors developed for the resilient design of high-rise buildings. In addition to the advanced nonlinear models that were built for the RC core-wall building and the other mechanism components, the proposed wobbling model was used to represent the spherically capped mega-columns. Based on the NLRHAs, it is verified that these rolling mega-columns allowed for the uplifting of the rocker and, meanwhile, displayed the intended negative lateral stiffness while swaying laterally with the rocker. At the system level, the expected low-damage design was achieved for the RC core-wall structure, relying on the engagement of the rocking and shear mechanisms.

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