



ORIGINAL RESEARCH ARTICLE

SEISMIC PERFORMANCE OF HIGH-RISE RC BUILDINGS WITH SQUARE AND CIRCULAR COLUMNS USING FLUID VISCOUS DAMPERS: TIME HISTORY VS. PUSHOVER ANALYSIS

I. A. David^{1*}, N. Muazu¹, K. Mohammed¹, N. D. Usman²

¹Department of Civil and Water Resource Engineering, University of Maiduguri, Borno State, Nigeria

²Department of Building, Modibbo Adama University, Yola, Adamawa State, Nigeria

*Corresponding author's email address: ijanudavid@gmail.com

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ABSTRACT

This study presents a comparative evaluation of the seismic performance of high-rise reinforced concrete (RC) buildings with square and circular columns, analysed with and without fluid viscous dampers (FVDs) using nonlinear time history and pushover methods in ETABS 2022. Four structural configurations were investigated: square columns without FVDs (SC), square columns with FVDs (SCFVD), circular columns without FVDs (CC), and circular columns with FVDs (CCFVD), under identical geometric and material properties. Seismic loading was defined in accordance with EN 1998-1, and Imperial Valley-02 ground motion was used for dynamic simulation. Results indicate that, without dampers, circular columns reduced dynamic base shear demand by up to 20% compared to square columns, although exhibiting slightly longer fundamental periods ($\approx 3\%$). The incorporation of FVDs significantly enhanced seismic performance in both column types; however, circular columns demonstrated superior efficiency. Fundamental time periods were reduced by 24% in SCFVD and 43% in CCFVD, with CCFVD exhibiting a 27% shorter period than SCFVD. Under time history analysis, base shear decreased by 31% in SCFVD and by 44% (X) / 42% (Y) in CCFVD relative to the square-column baseline. However, Pushover analysis showed even greater reductions, reaching 35% in SCFVD and up to 74% in CCFVD. It was clear that Circular column system combined with FVDs provided the most efficient seismic response, achieving the lowest time periods and base shear demands. Thus, the study demonstrate that circular columns enhance the effectiveness of FVDs and offer superior seismic performance compared to square columns in high-rise RC buildings, particularly when evaluated using nonlinear dynamic analysis.

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1.0 Introduction

Nigeria's accelerating urbanisation, particularly in cities such as Lagos, Abuja, and Port Harcourt, has driven the construction of high-rise reinforced concrete (RC) buildings to meet growing demands for commercial, residential, and mixed-use spaces (Ezeugwu, 2023; Ajaero, 2024). To achieve structural efficiency and architectural flexibility, circular RC columns are increasingly favored due to their superior biaxial bending capacity, uniform stiffness distribution, and enhanced confinement under axial and seismic loading (Dong et al., 2021; Eid et al., 2018). While Nigeria is traditionally classified as a low-to-moderate seismicity region, the occurrence of perceptible tremors in Abuja (2018) and earlier events in Ijebu-Ode, Yola, and Saki highlights the presence of active tectonic features, such as the Ifewara–Zungeru fault zone and structural trends within the Benue Trough (Oluwafemi et al., 2018; Umashor and Alutu, 2021). Regional study indicated an increasing frequency of small- to moderate-magnitude events, particularly in the north-central and south-western zones, making seismic resilience a relevant consideration for Nigeria's tall buildings (Oluwafemi et al., 2018).

The National Building Code (NBC, 2018) remains the primary framework for building safety in Nigeria; however, its seismic provisions are relatively limited and not fully calibrated to site-specific hazard levels. Consequently, structural engineers often adopt international guidelines such as ASCE 7-22 or Eurocode 8 to supplement local practice. This gap strengthens the case for performance-based seismic design (PBSD), which allows engineers to explicitly assess demand and capacity using advanced nonlinear methods rather than relying solely on prescriptive code checks (Choudhury, 2024; Padalu and Surana, 2024).

In this context, passive energy dissipation systems have gained prominence as an effective strategy to mitigate seismic demands without significant changes to a building's primary load-resisting system (Almajhali, 2023; Priya et al., 2015). Among these, fluid viscous dampers (FVDs) have proven particularly effective for tall RC structures by dissipating energy through velocity-dependent viscous action, reducing interstory drifts, peak accelerations, and residual displacements over a wide range of excitation frequencies (Elhany, 202; David et al., 2025). Their ability to improve seismic performance while maintaining architectural function makes them a viable option for high-rise projects in Nigerian seismic-prone zones.

Analysis methodology is central to quantifying the benefits of such devices. Nonlinear time-history analysis (THA) offers a detailed simulation of building response under actual or spectrum-matched earthquake records, capturing higher-mode effects, period elongation, stiffness degradation, and damper force-velocity characteristics (Lombardi et al., 2019). In contrast, nonlinear static (pushover) analysis (POA) estimates global capacity by incrementally applying lateral load patterns, making it less computationally demanding but potentially less accurate for taller or irregular buildings (Ajmal, 2012; Rooshenas, 2020). Prior comparative studies have shown that while pushover analysis can yield reasonable approximations for low- and mid-rise regular structures, THA is generally superior for high-rise frames with supplemental damping where higher modes significantly influence seismic response.

Modern structural engineering tools, such as ETABS 2022, integrate both THA and POA within a unified modelling environment, supporting nonlinear link elements for FVDs, customizable hinge definitions, and performance-based evaluation (CSI, 2022). Thus,, this study conducts a comparative investigation of time-period variations and seismic performance of high-rise RC buildings with square and circular columns using FVDs, employing ETABS 2022 to perform both time-history and pushover analyses (Hassan and Reyes, 2020; Brunesi et al., 2016).

2. Materials and Method

2.1 Equation of Motion for FVDs

VDs are velocity-dependent damping devices that dissipate energy through fluid resistance. The equation of motion for a system with an FVD is derived using Newton's Second Law of Motion and the dynamic equilibrium of a damped single-degree-of-freedom (SDOF) system.

$$M\ddot{u}(t) + C_D|\dot{u}|^\alpha \text{sgn}(\dot{u}) + Ku(t) = -M\ddot{u}_g(t) \quad 1$$

where: M = Mass of the structure, $\ddot{u}(t)$ = Acceleration of the structure, $u(t)$ = Displacement of the structure relative to the ground, $C_D|\dot{u}|^\alpha \text{sgn}(\dot{u})$ = Damping force, K = Structural Stiffness, $\ddot{u}_g(t)$ = Ground acceleration (earthquake input).

2.2 Time History Analysis Procedure

Step 1: Calculation of Modal Matrix:

$$[m][\ddot{x}] + [C][\dot{x}] + [K][x] = -\ddot{x}_g(t)[m]\{l\} \quad 2$$

Where; $-\ddot{x}_g(t)$ = Ground acceleration and $\{l\}$ = unit vector.

$$\text{Also, } \{x\} = [\Phi]\{q\} \quad 3$$

The solution of the equation of motion for any specified forces is difficult to obtain, mainly due to the coupling of variables $\{x\}$ in the physical coordinates. In modal analysis, a set of normal coordinates, that is, Principal

coordinates, is defined, such that when expressed in those coordinates, the equation of motion becomes uncoupled. That is.

$$\{x\} = [\Phi]\{q\} \quad 4$$

$$\{\dot{x}\} = [\Phi]\{\dot{q}\} \quad 5$$

The physical coordinates $\{x\}$ be related to $\{q\}$ from transformation.

Substituting (3.9), (3.10) and (3.11) in (3.8), and multiplying both sides by $[\Phi]^T$.

$$[\Phi]^T[m][\Phi]\{\ddot{q}\} + [\Phi]^T[c][\Phi]\{\dot{q}\} + [\Phi]^T[k][\Phi]\{q\} = [-\ddot{x}_g(t)][\Phi]^T[m]\{l\}[M]\{\ddot{q}\} + [C]\{\dot{q}\} + [K]\{q\} = \{P_{eff}(t)\} \quad 6$$

where Diagonalised Modal Mass $[M] = [\Phi]^T[m][\Phi]$, Diagonalised Modal Damping $[C] = [\Phi]^T[c][\Phi]$, Diagonalised Modal Stiffness $[K] = [\Phi]^T[k][\Phi]$, Effective Modal Force Vector $\{P_{eff}(t)\} = [-\ddot{x}_g(t)][\Phi]^T[m]\{l\}$

Find $[K]$, $[M]$, $[\omega]$ then $[T]$

$$\text{Find the mode shape } [\Phi] \Rightarrow [-m\omega_i^2 + k]\phi_i = 0$$

$$\text{Then get } [M] = [\Phi]^T[m][\Phi] = \text{Diag}\{1\}$$

$$\text{Damping} = \frac{\text{Actual Damping}}{\text{Critical Damping}} \varepsilon = \frac{c}{c_c} \quad 7$$

$$c_c = 2\sqrt{km} \quad c_c = 2m\omega_n \quad \omega_n = \sqrt{\frac{k}{m}} \quad [C] = \text{Diag}(2M_r\varepsilon_r\omega_r) = \begin{bmatrix} c_1 & 0 & \dots & 0 \\ \vdots & \ddots & & \vdots \\ 0 & \dots & & c_n \end{bmatrix} \quad 8$$

Step 2: Calculation of Effective Force Vector:

$$\{P_{eff}(t)\} = \{[-\ddot{x}_g(t)][\Phi]^T[m]\{l\}\} = [-\ddot{x}_g(t)Y_r] Y_r = \frac{[\Phi]^T_r[m]\{l\}}{[\Phi]^T_r[m][\Phi]_r} = \frac{[\Phi]^T_r[m]\{l\}}{M_r} \quad 9$$

Step 3: Calculation of Displacement Response in Normal Co-ordinates:

The uncoupled equations in normal co-ordinates are

$$[M_1]\ddot{q}_1 + [C_1]\dot{q}_1 + [K_1]q_1 = \gamma_1\ddot{x}_g(t) \quad 10$$

Piece-wise Linear Interpolation.

Solve these equations using New Mark's Method. (At different time histories)

Step 4: Calculation of Displacement Response in Physical Co-ordinates:

$$\{x(t)\} = \sum_{r=1}^n [\Phi]_r q_r(t) \quad n = 1, 2, 3, 4, \dots \text{ modes} \quad 11$$

Step 5: Calculation of Effective Earthquake Forces at Each Storey:

$$F_s(t) = [k]\{x(t)\} \quad 12$$

2.3 Nonlinear Static Equilibrium Equation

For a multi-degree-of-freedom (MDOF) structure, the dynamic equilibrium equation is:

$$[M]\{\ddot{u}\} + [C]\{\dot{u}\} + [K]\{u\} = \{P(t)\} \quad 13$$

Where $[M]$ = Mass matrix, $[C]$ = Damping matrix, $[K]$ = Stiffness matrix, $\{u\}$ = Displacement vector, $\{P(t)\}$ = Time-dependent load vector.

2.3.1 For Static pushover

We neglect inertia and damping:

$$[M]\{\ddot{u}\} = 0 \quad [C]\{\dot{u}\} = 0 \quad 14$$

Thus:

$$[K]\{u\} = \{P\} \quad 15$$

2.3.2 For nonlinear behavior

When yielding occurs, stiffness becomes displacement-dependent:

$$[K_t(u)]\{\Delta u\} = \{\Delta P\} \quad 16$$

Where $K_t(u)$ = Tangent stiffness matrix

This is the fundamental governing equation of pushover analysis.

Incremental Load Application

Load is applied in increments:

$$\{P\} = \lambda\{P_0\} \quad 17$$

Where λ = Load factor, P_0 = Reference lateral load pattern

Substitute into equilibrium:

$$[K_t]\{u\} = \lambda\{P_0\} \quad 18$$

Solving for displacement:

$$\{u\} = \lambda[K_t]^{-1}\{P_0\} \quad 19$$

As hinges form K_t reduces and displacement increases nonlinearly.

2.4 Structural Modelling

The structural elements considered in the analysis included columns, beams, and slabs with varying cross-sections, as outlined below. Different column shapes were evaluated while maintaining a constant total building floor area, as illustrated in Figures 1 and 2

2.4.1 Description of members

Square column size: 450mm x 450mm, Circular column size: 450mm diameter, Beam sizes: 450mm x 450mm, Slab size: (a) Panel Area = 6m x 5m (b) Thickness = 200mm

2.4.2 Material property

Concrete grade = M30

Steel grade = F_e460

2.5 Loads

In applying loads to the structure, only externally imposed loads on the members were considered, while self-weight was excluded, as it is automatically calculated by ETABS 2022.

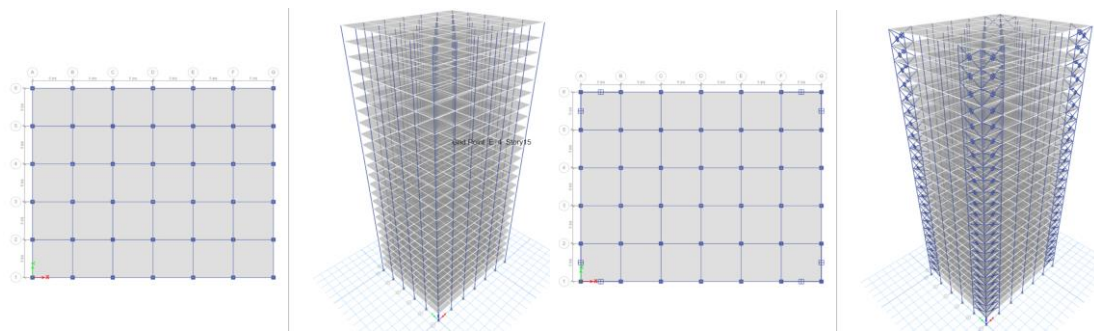


Figure 1: Square column structure with and without FVD 75m height

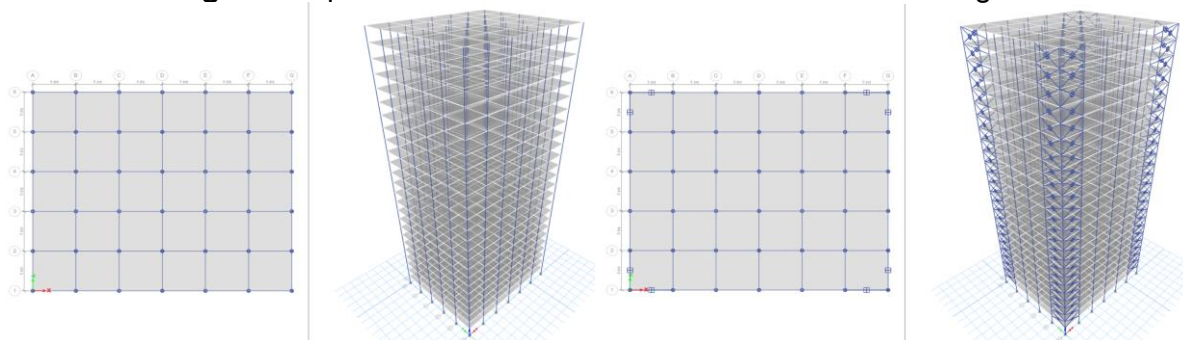


Figure 2: Circular column structure with and without FVD 75m height

2.5.1 Applied loads

For the slab elements, shell loads in the gravity direction were assigned as Dead load = 1.5 kN/m² and Live load = 4.5 kN/m², corresponding to hotel occupancy requirements. Uniformly distributed frame loads were applied to beam elements, with a Dead load = 5.25 kN/m. Seismic loads (EQ-X and EQ-Y) were defined by EN 1998-1, while wind loads (Wind-X and Wind-Y) were specified based on EN 1991-1-4. The load cases generated in ETABS 2022 are summarised in Table 2.

2.5.2 Load Patterns and Cases

Table 1: Load Patterns

| Name | Type | Self-Weight Multiplier | Auto Load |
|--------|---------|------------------------|-------------|
| Dead | Dead | 1 | |
| Live | Live | 0 | |
| EQ-X | Seismic | 0 | EN 1998-1 |
| EQ-Y | Seismic | 0 | EN 1998-1 |
| Wind-X | Wind | 0 | EN 1991-1-4 |
| Wind-Y | Wind | 0 | EN 1991-1-4 |

Table 2: Load Cases

| Name | Type |
|--------|-------------------------------|
| Dead | Linear Static |
| Live | Linear Static |
| EQ-X | Linear Static |
| EQ-Y | Linear Static |
| Wind-X | Linear Static |
| Wind-Y | Linear Static |
| THX | Nonlinear Modal History (FNA) |
| THY | Nonlinear Modal History (FNA) |
| Push-X | Nonlinear Static |
| Push-Y | Nonlinear Static |

2.6 Modelling of Dampers

The fluid viscous dampers were installed in a double-diagonal bracing arrangement, functioning primarily in the axial direction. Accordingly, the UI translational degree of freedom was activated, while all other translational and rotational degrees were restrained, reflecting the non-linear nature of the damper's behavior. During property definition in ETABS 2022, the non-linear option was selected, and parameters for stiffness, damping coefficient, and damping exponent were specified. The rotational properties (R1, R2, and R3) were set to zero, as the damper does not accommodate rotational movement. The damper specifications, obtained from Taylor Devices Inc., USA, are summarized in Table 3 and were applied in ETABS 2022 during the damper modeling process.

Table 3: Properties of FVDs Model

| Parameter | Value | Unit |
|-------------------------------|---------|--------|
| Damper Type | FVD 250 | - |
| Mass | 44 | kg |
| Damping Constant | 460 | kN s/m |
| Damping Exponent (α) | 0.3 | - |

2.7 Time History Function

In this study, the building's dynamic response was evaluated at successive time intervals. The procedure employs recorded ground motion data obtained from historical earthquake databases. When nonlinear behavior is not present, this approach addresses the limitations of the response spectrum method. However, it demands more computational effort since the response must be calculated at discrete time steps. For this work, the Imperial Valley-02 earthquake record was used, with a time step of 0.01 seconds and nine data points per line

2.8 Pushover Analysis

Pushover analysis is a performance-based evaluation method aimed at controlling structural damage. In this study, predefined hinge properties for concrete members, as specified in FEMA 356, were incorporated into the analysis. The procedure was conducted using the nonlinear analysis capabilities of ETABS 2022, which can estimate the displacement level and corresponding base shear at which the structure first yields. Structural plasticity was modeled through force-displacement relationships (axial and shear) and moment-rotation behaviors (torsion and bending). Manual hinge property definitions were applied, while additional performance limit states. Immediate occupancy, life safety, and collapse prevention were included for reference. These limit states, although reported in the results, did not influence the analytical outcomes, as illustrated in Figure 3

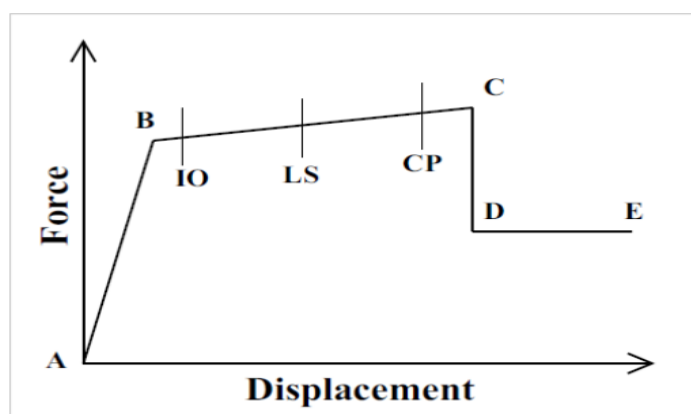


Figure 3: Force-deformation relationship of a typical plastic hinge (Priyanka et al., 2019).

3. Results and Discussion

This section presents the results and discussion for this study.

3.1 Variations in Time Period for High-rise Buildings With and Without FVDs

The study of variations in the time period of high-rise buildings, both with and without Fluid Viscous Dampers (FVDs), focuses on natural time period is a key parameter that reflects a building’s flexibility and response to external dynamic forces such as wind and earthquakes

3.1.1 Time periods due to earthquake with and without FVDs

The time period of high-rise buildings under earthquake loading is a critical factor in assessing their dynamic response and seismic performance. It represents the time a structure takes to complete one cycle of vibration when subjected to ground motion. Figure 4 presents time periods for different modes compares the modal time periods of four structural systems; Square Column with Fluid Viscous Dampers (SCFVD), Square Column without FVD (SC), Circular Column with Fluid Viscous Dampers, (CCFVD), and Circular Column without FVD (CC), across the first 12 vibration modes. It was observed that CC exhibits the highest fundamental time period at approximately 3.7 s, followed closely by SC (3.6 s), while SCFVD and CCFVD demonstrate reduced fundamental periods of about 2.8 s and 2.05 s, respectively.

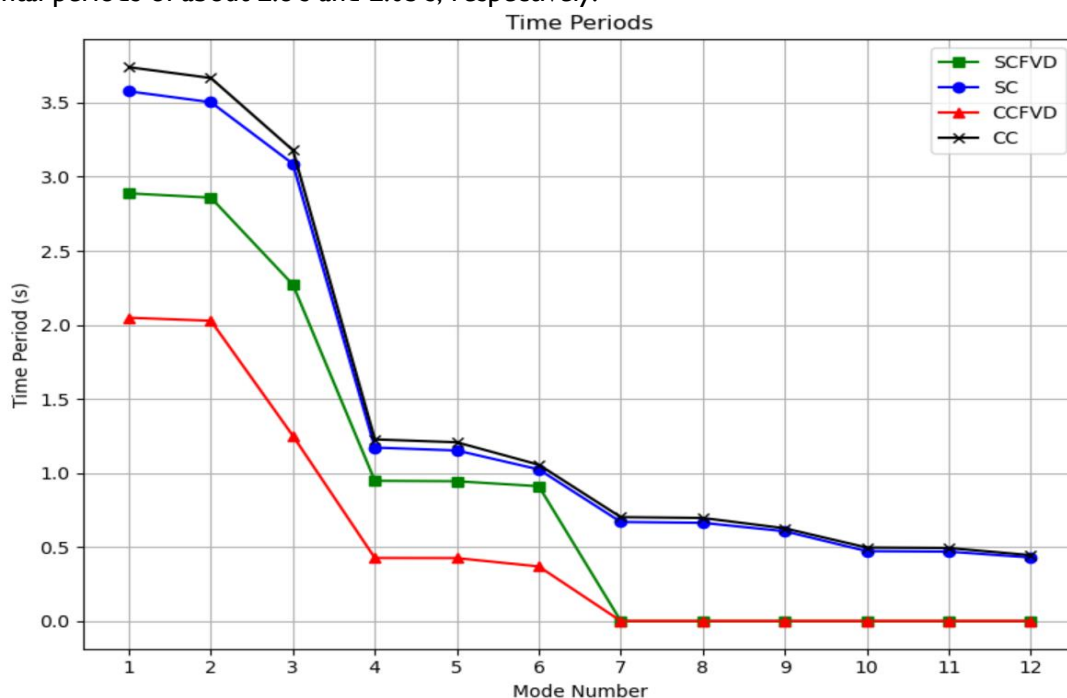


Figure 4: Time period variations due to earthquake with and without FVDs

3.2 Comparison of Time History to Pushover Analysis with and without FVDs

The comparison between Time History Analysis and Pushover Analysis for high-rise buildings, both with and without Fluid Viscous Dampers (FVDs), provides valuable insight into structural performance under seismic loading.

3.2.1 Base shear comparison due to time history analysis with and without FVDs

Base shear represents the total lateral force acting at the base of a structure during an earthquake and is a key indicator of structural response. Figure 5 reveals that the Square Column without FVD (SC) configuration experiences the highest seismic demand (~22,000 kN), serving as the baseline for comparison. Incorporating FVDs in the SCFVD model leads to a significant base shear reduction of approximately 31% in both directions. The Circular Column with FVDs (CCFVD) model exhibits the most substantial improvement, reducing base shear by around 44% under TH-X and 42% under TH-Y. Meanwhile, the Circular Column without FVDs (CC) system achieves a moderate reduction of about 13% and 20% in TH-X and TH-Y, respectively. These findings demonstrate that applying FVDs, particularly in circular column systems, considerably enhances seismic performance by effectively reducing base shear demands under time history loading.

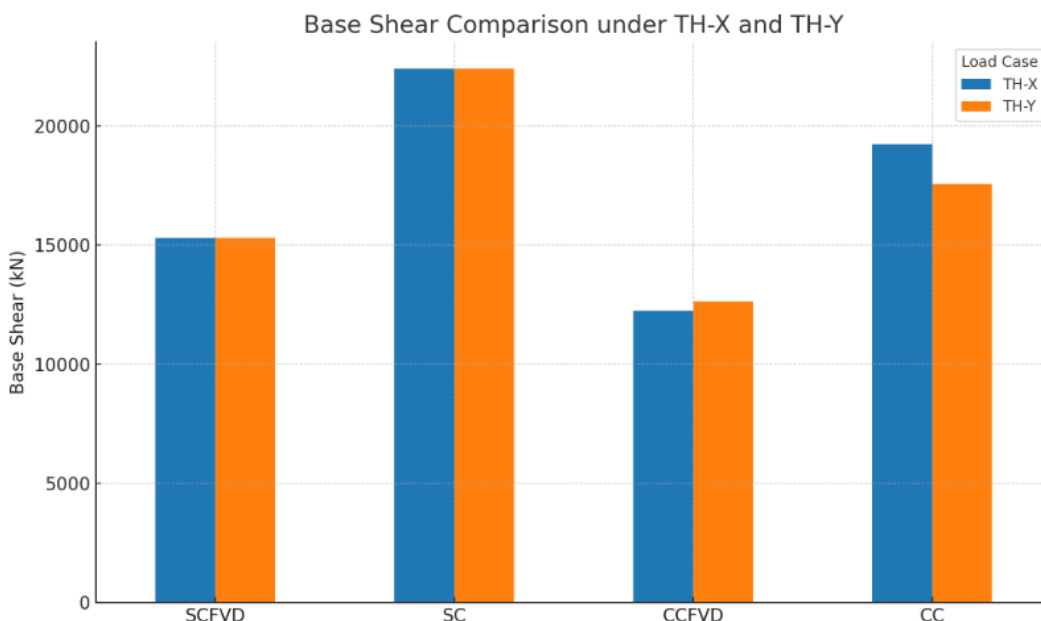


Figure 5: Base shear comparison under time history analysis in X and Y directions

3.2.2 Base shear comparison due to push over analysis with and without FVDs

Base shear in pushover analysis represents the maximum lateral force a structure can resist before reaching a target displacement or collapse condition. Figure 6 highlights that the SC model experiences the highest base shear demand (6515 kN in Push-X and 5279 kN in Push-Y), making it the stiffest and most force-attracting configuration. Introducing FVDs in the SCFVD model significantly reduces the base shear by approximately 35% and 32% in Push-X and Push-Y, respectively. The CC configuration exhibits a more flexible response with reduced base shear values, and its damped counterpart, CCFVD, achieves the greatest reduction, by approximately 71% in Push-X and 74% in Push-Y compared to SC. This indicates that the incorporation of FVDs substantially enhances energy dissipation and reduces seismic demand, particularly in circular column systems, leading to more efficient seismic performance under nonlinear pushover loading.

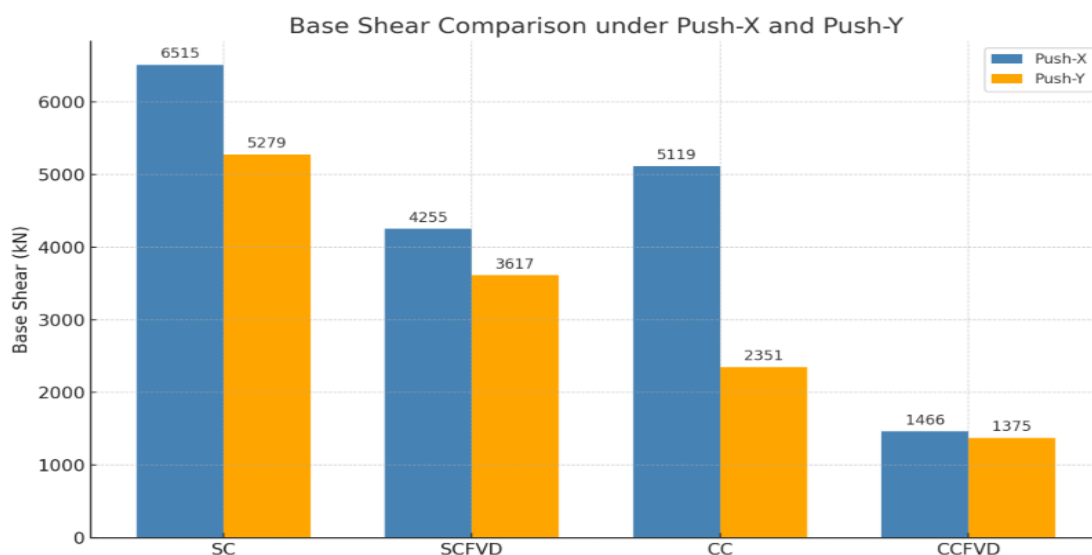


Figure 6: Base shear comparison under Push over analysis in X and Y direction

4. Conclusion

Based on the results of this study the following are the conclusion;

1. Installation of FVDs reduced the fundamental time period by 24% in square columns (3.6 s, 2.8 s) and 43% in circular columns (3.7 s, 2.05 s). The circular column with FVD (CCFVD) achieved a 27% shorter period than the square column with FVD (SCFVD), indicating superior stiffness enhancement.

2. Relative to the square column without FVD (SC baseline $\approx 22,000$ kN), base shear reduced by 31% in SCFVD and by 44% (X) / 42% (Y) in CCFVD. Circular columns with FVDs therefore provided about 13–15% greater reduction than square columns with FVDs.
3. Compared to SC (6515 kN in X; 5279 kN in Y), SCFVD reduced base shear by 35% (X) and 32% (Y), while CCFVD achieved the highest reduction of 71% (X) and 74% (Y). Circular columns enhanced damper efficiency by approximately 36–42% over square columns under nonlinear static loading.
4. Pushover analysis predicted larger reductions (up to 74%) compared to time history analysis (maximum 44%), indicating that pushover overestimates seismic performance improvement by approximately 30% for high-rise buildings with FVDs.

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