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Effectiveness of Friction Damper Configurations on the Behaviour of Bundled Tube High-Rise Building System for Lateral Loads

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Abstract Earthquakes and Wind forces pose significant threats to high-rise structures worldwide, often causing extensive damage and loss of life and property. Friction dampers are cost-effective, easy to install and reusable and hence are the best choice among several types of seismic dampers to control the effects of lateral forces on high-rise structures. The arrangement of the dampers plays a prominent role in strengthening high-rise buildings for lateral forces. The present study investigates the efficacy of X-type and diagonal friction damper configurations arranged at the bottom 1/3rd, middle 1/3rd, top 1/3rd and throughout the height in a typical bundled tube high-rise building system in minimizing the structural response to lateral forces. It is observed from the present study that the location of the friction dampers plays a prominent role in controlling structural response to the lateral loads compared to the type of friction dampers adopted. Building system with X-type friction dampers arranged throughout the height of a building is found to be effective in resisting lateral forces in bundled tube high-rise structural systems, especially at higher stories as compared to other configurations considered in the study.

Keywords Bundled Tube System, Friction Dampers, Gust Factor, Response Spectrum Method, Stiffness Modifiers, Time-History Analysis

1. Introduction

The world has been seeing a great deal of destructive earthquakes that have led to major structural damage and in many cases resulted in the collapse of buildings. Heavy winds can equally damage structures because of their unpredictable dynamic characteristics over time. Such damages are proof that lifeline infrastructure in seismically active and high wind areas needs to be carefully engineered to shield them from probable structural damages. With the world's population growing, there is a growing demand for low to high-rise buildings that can withstand the effect of lateral forces. According to Indian standard code IS 16700:2017 [1], a building is considered a tall building if its height varies from 50m to 250m. Analysis and design of any structure for earthquake and wind forces are crucial in the construction industry in minimizing the damaging effect.

The amount of energy delivered to the structural elements due to lateral forces can be significantly reduced by energy dissipation if buildings are installed with dampers. Dampers function similarly to shock absorbers which lessen or "dampen" vibrations. Dampers are efficient in absorbing energy and are simple to install or replace. Dampers are fabricated and carefully engineered to safeguard structural integrity and reduce structural damage by absorbing seismic energy. Several types of dampers are adopted to control lateral forces, such as viscous dampers, visco-elastic dampers, friction dampers, tuned mass dampers, yielding dampers, magnetic dampers etc. The friction damper has been the best choice considering it to be cost-effective, easy installation and reusable properties [2]. Friction dampers can effectively control the seismic response of high-rise buildings in comparison with shear wall systems. The height and aspect ratio of the building affect the ideal number and placement of friction dampers. Friction dampers can help save building costs and enhance the amount of usable floor area [3].

During earthquake or wind, the friction dampers slip at a specific load before the members of a frame yield, dissipating a significant portion of the energy. The friction damper converts kinetic energy into heat through friction, working similarly to a friction brake or coulomb damper as shown in Figure 1(a). The dampers provide elastic movement and energy dissipation within the building. As a result, it is possible to optimize structural components for cost reductions, which results in significant savings. When a frame structure is excited by external forces, the frame moves horizontally and vertically. The middle plate spins around the hinge as the damper follows the motion as shown in Figure 1(b). Due to the tensile pressures in the bracing parts, the horizontal plates rotate in the opposite direction from the central plate. The plates rotate the other way if the applied forces are reversed. The damper minimizes the vibration of the frame structure by converting mechanical energy into thermal energy (heat) through friction between the sliding surfaces throughout this operation.

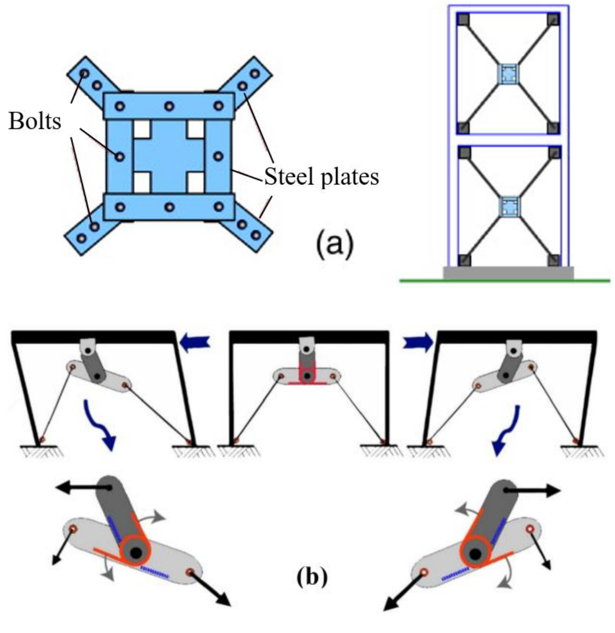


Figure 1. a) Friction damper b) Behaviour of typical (Chevron type) friction damper under seismic loading [4]

However, the bundled tube system is architecturally appealing and is the most efficient structural system with sufficient lateral stiffness in resisting lateral loads as the building acts as a unified system of stiffened tubes. In a bundled tube structural system, there are several interconnected tubes to form a multi-cell tube. This arrangement together resists lateral loads and overturning moments. A bundled tube system is preferred to control the drift and displacement of high-rise buildings vulnerable to lateral forces. The tube-in-tube system is the most economical and versatile building design that helps to create interesting shapes.

High-rise structures vulnerable to seismic and wind forces can be analyzed using a variety of structural methods, including the Equivalent static method, Response spectrum method, Time-history analysis and Gust wind method.

The range of frequency and intensity of seismic waves vary as they travel from the earth’s crust to the surface of the earth. The severe movement, intensity, and frequency of seismic waves have led to the damage or destruction of many buildings. The plan, geometry, and mass irregularities have a major impact on the seismic performance of any high-rise building [5], [6]. Friction dampers can greatly enhance the seismic performance of buildings with reinforced concrete frames [7]. Inter-story drift and acceleration response of the structures during earthquakes can be greatly reduced by the installation of dampers with the best arrangement [6], [8].

The wind load is influenced by the wind intensity, topography, and terrain of the area, as well as the geometry and location of the building. While analyzing a building, it is crucial to consider the gust wind nature. The Gust factor approach is used to investigate the design impacts of wind loads on high-rise buildings [9]. The dynamic reaction of a structure to gust loads is measured by the Gust Response Factor (GRF) and design wind loads can be compared using the GRF and codes of practice [10]. The standard practice of stiffening a building to lessen dynamic responsiveness under wind loads has the unintended consequence of attracting more seismic base shear. It is conceivable to lessen the building's flexural rigidity to lower seismic base shear and simultaneously manage the wind response by adding supplemental damping to the structure [11]. To ensure the safety and stability of high-rise buildings under wind loads, the influence of dynamic factors such as natural frequency, damping ratio, and mode shapes should be predominantly considered [12].

The X-type friction dampers are installed at the intersection of the X-braces. This intersection point is where the braces cross and the damper is placed to absorb and dissipate the energy transferred through the braces during seismic events. During an earthquake, lateral forces cause the building to sway, creating movement in the braces. The friction damper at the intersection slides and generates frictional resistance, absorbing the seismic energy and reducing the forces transmitted to the structure. Diagonal friction dampers are arranged within diagonal braces that connect columns to beams. The friction dampers are integrated into the braces, either at the ends where they connect to the columns/beams or along the length of the brace. During seismic activity, the lateral forces cause the diagonal braces to deform. The friction dampers within the braces slide and generate frictional resistance, dissipating the seismic energy and reducing the forces transmitted to the structure. The choice between X-type and diagonal dampers depends on the specific design and seismic performance requirements of the building. The arrangement of friction dampers in a high-rise building is crucial for a building's seismic performance and overall structural integrity by dissipating seismic energy effectively, reducing structural vibrations, improving lateral stability, controlling lateral deformations, enhancing ductility and enhancing cost-effectiveness.

Even though studies have been carried out with friction dampers and bracings independently on high-rise structures in reducing the lateral response, limited works have been carried out on bundled tube high-rise building systems. In the present study, an attempt has been made to take advantage of both bundled tube systems and friction dampers/bracings to reduce the lateral response more efficiently as compared to the previous studies.

The present study focuses on evaluating the efficacy of X-type and diagonal friction damper configurations on a bundled tube high-rise building system.

2. Methodology

To study the efficacy of friction damper configurations, a typical G+44 multi-storey bundled tube high-rise building as shown in Figure 2 is modelled in ETABS with 8 different friction damper configurations as shown in Figure 3 and analyzed for cracked section property modifiers by considering the P-delta effect. Response spectrum method, time-history and dynamic gust wind analysis are performed using ETABS and building parameters such as story displacement, story drift, story stiffness, joint acceleration, and joint displacement have been investigated. The buildings are analyzed as per IS 875 (Part 1 and 2):1987 [13], [14], IS 1893 (Part 1):2016 [15], IS 16700:2017 [1]and IS 875 (Part 3):2015 [16].

Friction dampers slip at a specific load before the members of a frame yield, dissipating a significant portion of the energy. The concept behind the friction damper slip load design is that before structural elements start to give way, dampers should not slide under intense stress. The force at which the damper starts to distort is known as the slip load. The range of slip load can be predicted from the static analysis of the structure given the horizontal story shear forces. The slip load in a specific story can be calculated by:

$$\text{Slip load} = \frac{1}{3} \text{ x } \frac{Storey\ \ shear\ \ force}{Number\ \ of\ \ dampers\ \ in\ \ a\ \ storey} \quad [3]$$

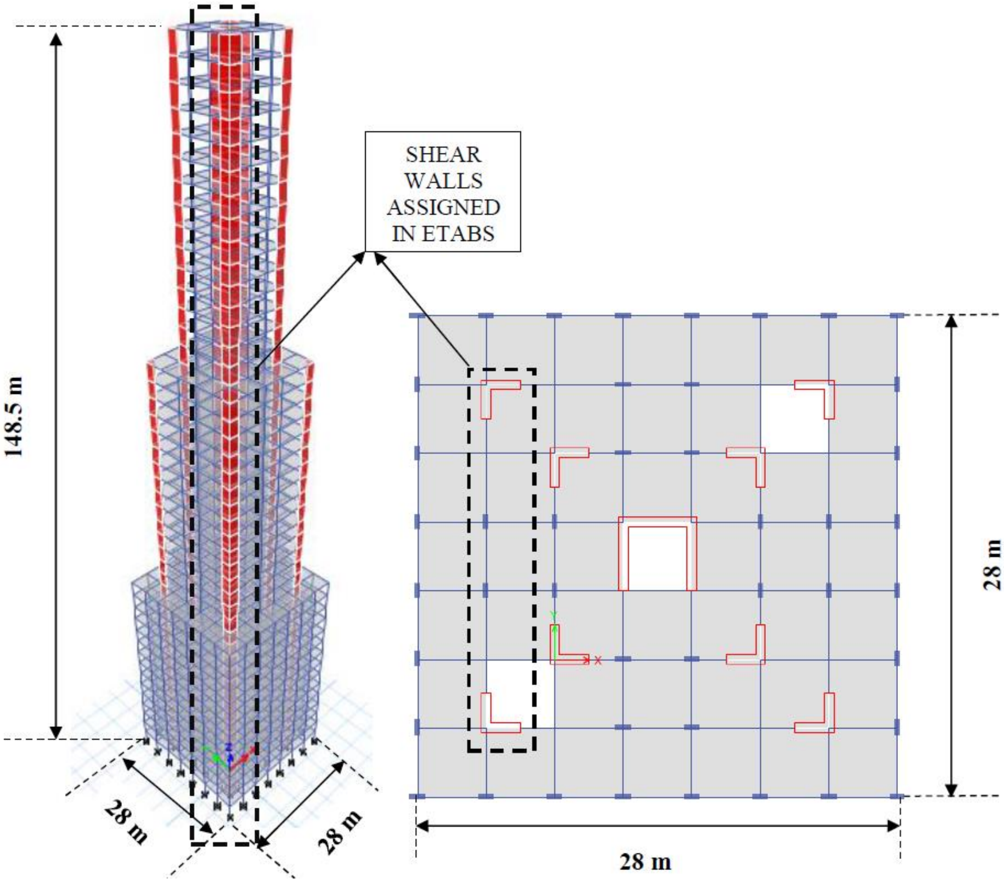


Figure 2. 3D and base plan view of typical G+45 bundled tube high-rise building considered in the study (A1)

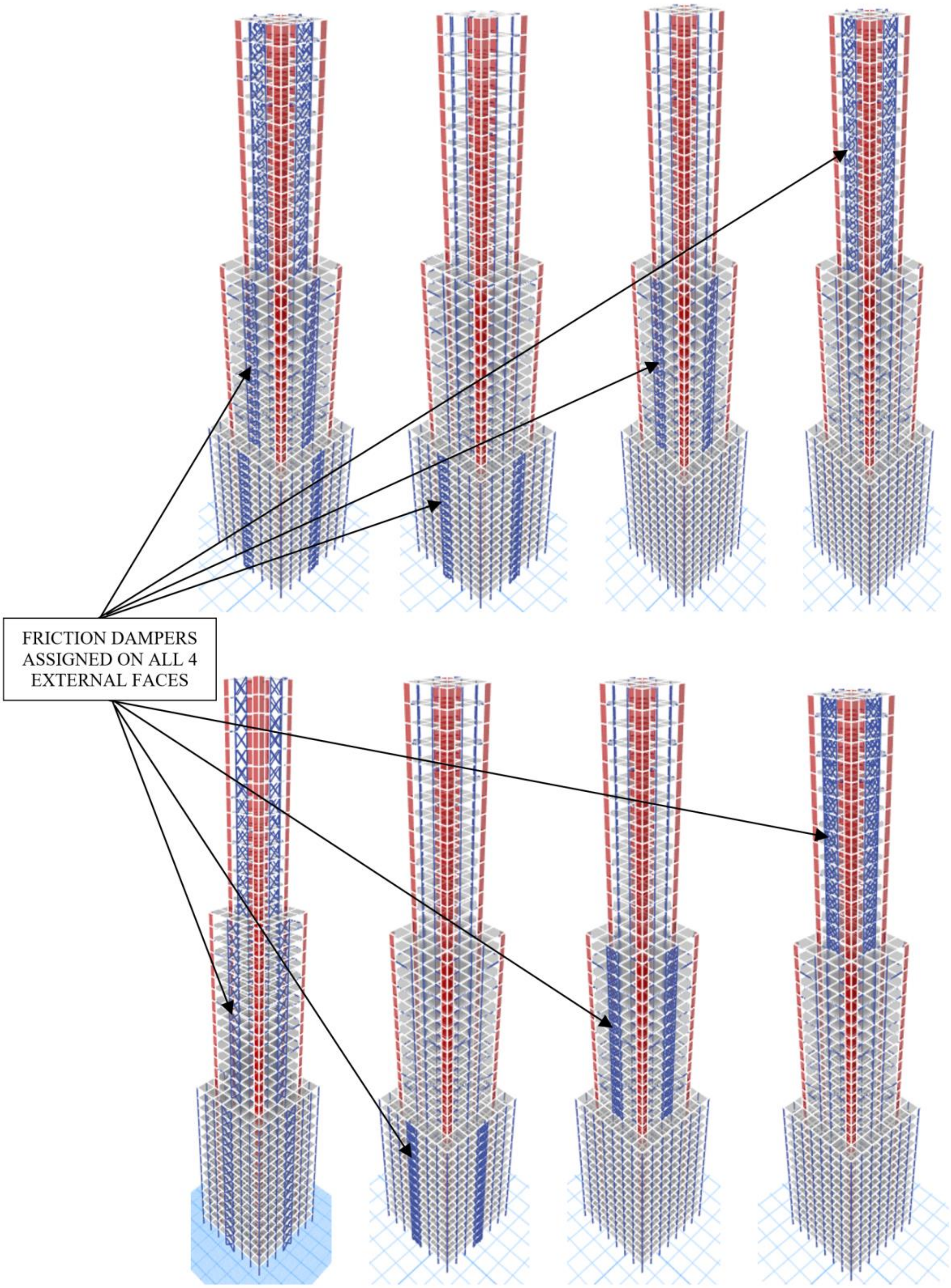


Figure 3. Friction damper configurations considered in the study (A2 – A9)

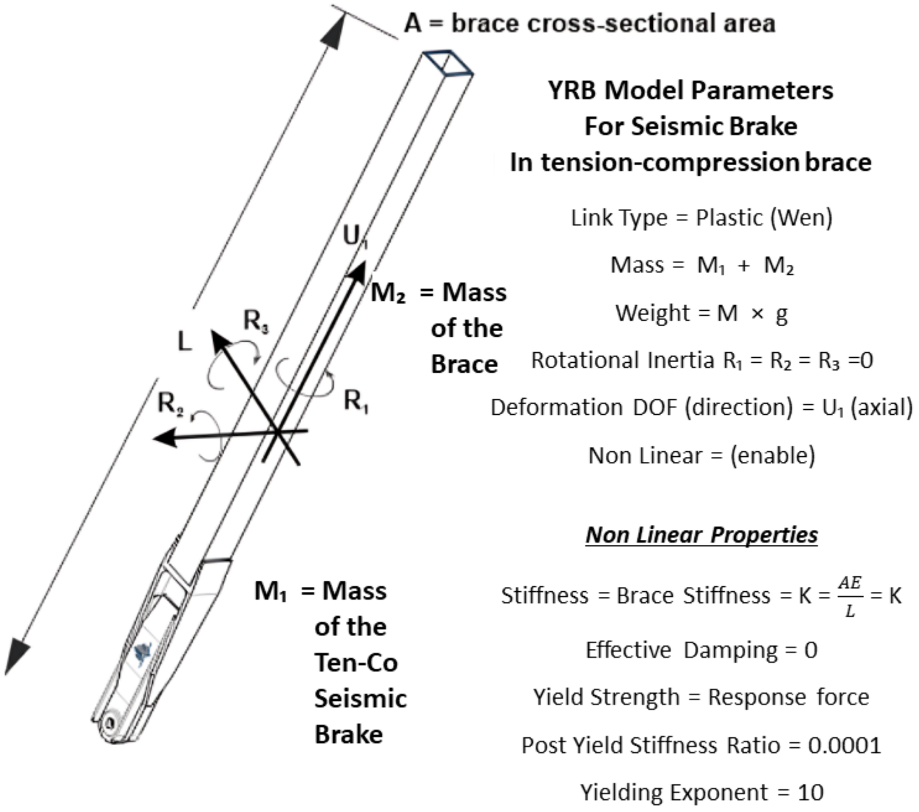


Figure 4. Wen model friction damper used in the study and its properties [19]

Friction dampers are modelled as link elements in ETABS. Wen Model friction damper shown in Figure 4 has been used in the present study and the link properties adopted while modelling are tabulated in Table 1. Gravity loads and seismic parameters assigned for building models are tabulated in Tables 2 and 3 respectively. Friction damper configurations are tabulated in Table 4 and these are arranged on the frames of all four exterior faces symmetrically.

Geometric and material properties of G+44 bundled tube high-rise buildings considered in the study are tabulated in Table 5.

The ability of a member to resist deformation because of applied loads is known as the member's rigidity or stiffness. When structures are subjected to these loads, internal forces are generated, which cause compression and tension in the concrete fibres that lead to cracks. To reduce the stiffness of RCC sections to represent the behaviour of cracked concrete under load, wind and earthquake stiffness modifiers are considered as per IS 16700:2017 [1] in the present study. It is necessary to consider second-order and P-Delta effects for high-rise buildings with significant heights. The combined effect of axial forces with lateral load-induced deflections results in the additional effects known as second-order effects. First-order deflections create extra moments brought on by the axial stresses, which in turn cause the deflections to increase to a higher order. In the present study, P- Delta properties are assigned to building models by considering scale factors as per IS 16700:2017 [1]corresponding to load patterns.

The ratio of peak wind gust for a given period of time to the mean wind speed is known as the gust factor (GF). It depends on a variety of factors, including the roughness length (exposure), separation from a change in the terrain upstream, stability, height, and, maybe, the presence of convection. Variations in the wind speed lead to fatigue loading on different structural elements. Gust factor calculation is obtained as per IS 875 (Part 3): 2015. Factors considered for building models are tabulated in Table 6. The limiting value of drift = 1/500 = 0.002 and the limiting value of displacement = Height of building/500 = 297mm are due to lateral loads.

Table 1. Link properties used in ETABS models

Mass	395 kg
Effective stiffness	602545 kN/m
Yield strength	1500 kN
Yielding exponent	10

Table 2. Live and dead loads considered in the study as per IS 875 (Part 1 and 2):1987 [13], [14]

Live load	3 kN/m^2
Dead load	Calculated by ETABS as per 25 kN/m^3 density of RCC
Floor Finish Load	1.5 kN/m^2
Dead load of wall	5 kN/m
Dead load of parapet wall	2 kN/m

Table 3. Seismic parameters considered in the study as per IS 1893 (Part 1): 2016 [15]

Seismic Zone	IV
Zone Factor	0.24
Importance Factor	1.2
Response Reduction Factor	5
Damping	5% of Critical
Location	Noida, India
Soil Type	II (medium)
Time period along X and Y directions	2.525 s

Table 4. Friction damper configurations considered in the study

Designation	Friction damper configurations considered in the study
A1	Building without friction dampers
A2	Building with diagonal friction dampers throughout the building height
A3	Building with diagonal friction dampers in the bottom 1/3rd of the building height
A4	Building with diagonal friction dampers in the middle 1/3rd of building height
A5	Building with diagonal friction dampers in the top 1/3rd of building height
A6	Building with X-type friction dampers throughout the building height
A7	Building with X-type friction dampers in the bottom 1/3rd of the building height
A8	Building with X-type friction dampers in the middle 1/3rd of building height
A9	Building with X-type friction dampers in the top 1/3rd of building height

Table 5. Geometric and material properties of building models considered in the study

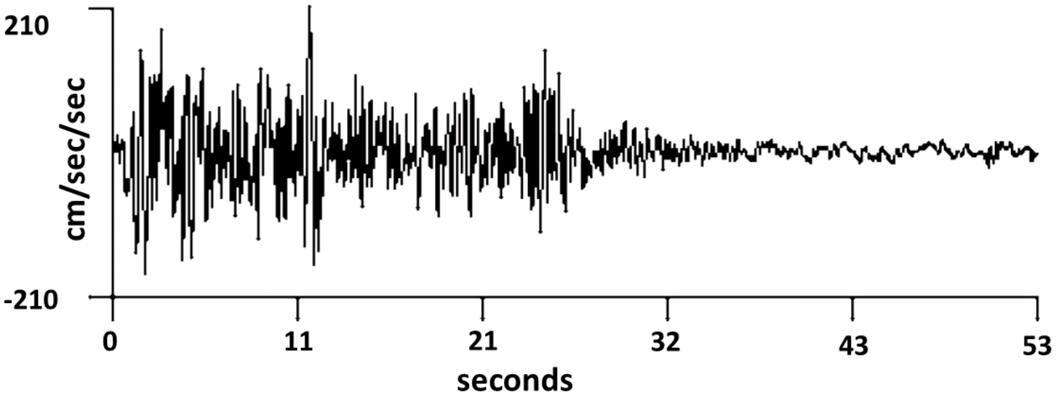
Structure type	Bundled tube RCC building
Type of the Structure	Symmetric
Number of Storeys	G+44
Floor to Floor Height	3.3 m
Length of each bay in plan	4m
No of bays in the bottom 1/3rd height of a building	7
No of bays in the middle 1/3rd height of a building	5
No of bays in the top 1/3rd height of a building	3
Grade of concrete	M40 (for Beams) M60 (for Columns and Shear Walls)
Grade of steel	Fe500
Size of columns	300x900mm
Size of beams	300X750mm (Primary) 300X600mm (Secondary)
Slab thickness	200mm

Table 6. Gust factor parameters assigned to building models in the study as per IS 875 (Part 3): 2015

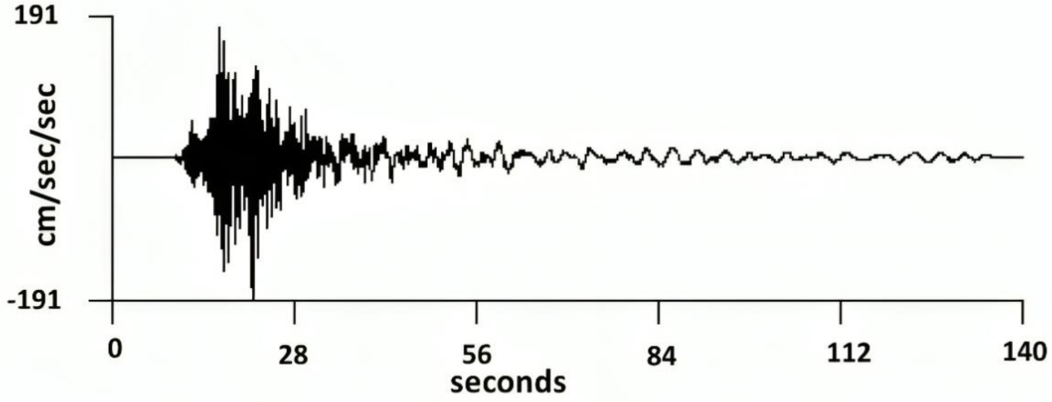
Basic wind speed, V_b	50 m/s
Risk coefficient, K_1	1
Terrain Category, K_2	4
Topography Factor, K_3	1
Importance Factor, K_4	1
Roughness Factor	0.4239
Soil Type	II (medium)
Equivalent aerodynamic roughness Height	2m
Peak factor for upward wind velocity fluctuation, g_v	4
Measure of effective turbulence length scale at height h, L_h	137.41
Damping Coefficient, β	0.02
Peak factor for resonant response, g_r	3.71
Width to Depth of building, a/b	1.0
Height to depth of building, h/b	5.3
Drag coefficient, C_f	1.4
Time Period in X direction	3.643s
Frequency in X direction	0.2744Hz
Time Period in Y direction	3.893s
Frequency in Y direction	0.2568Hz
Peak factor, g_h	3.695
Ratio $V_{hd}/f_c{*}b$	4.47
Cross wind force coefficient, C_{fs}	0.002
Turbulence Intensity at Height 2*(h/3)	0.259

However, from response spectrum analysis, it is possible to estimate the structural response to brief, transitory dynamic events. Because of uncertainty, examination of the load's precise temporal history is challenging. Due to its short duration, the event cannot be regarded as a stationary process. The response spectrum approach makes use of a certain kind of mode superposition. To set a restriction on how much an event of this kind can excite a mode with a particular natural frequency, one must give an input. This approach can be determined manually. However, doing so is difficult because the base shear is calculated manually, and the base shear predicted by the program seldom ever matches.

The behavioural examination of a structure under a past tremor or wind speeding up information is called time-history analysis. A plot of amplitude (the maximum extent of a vibration or oscillation) or acceleration v/s time of an event is called a time history. The seismic response of a structure under dynamic loading from a typical earthquake can be accessed via time-history analysis. In basic terms, time-history investigations look at how a structure responds to a load that changes over time as an acceleration. A similar approach has been adopted in previous studies [17], [18]. Every time the structure is subjected to a particular ground acceleration, the dynamic reaction of a building is examined. This technique can be carried out using recognized concepts of earthquake structural dynamics and is based on adequate ground acceleration. The most thorough type of analysis is provided by non-linear dynamic analysis. The term "performance-based method" refers to a methodology in which criteria for preventing structural damage and collapse are expressed in terms of induced levels of ductility demand and the building's inelastic drift index. While designing either a significant structure or a taller structure, where the modal mass contributes to the seismic force and higher modes are present, the nonlinear time-history analysis is typically assumed. Detailed dynamic nonlinear analysis is necessary to meet this, as the structural elements/members tend to respond in elastic behaviour when the structure is subjected to ground vibrations. In the present study, ground acceleration data from El Centro (near fault) and Kobe earthquakes (far fault) as shown in Figure 5 are used.



(a) El Centro earthquake



(b) Kobe earthquake

Figure 5. Accelerograms of earthquakes considered in the study.

3. Results and Discussions

To evaluate the efficacy of the various configurations of X-type and diagonal friction dampers, ETABS models with 8 different configurations of friction dampers are analyzed by response spectrum method, time-history and dynamic gust wind analysis and building parameters such as story drift, story displacement, story stiffness, joint acceleration and joint displacements have been investigated. The buildings are analyzed as per IS 1893 (Part 1):2016, IS 16700:2017, IS 875 (Part 3):2015. To explain the effectiveness of dampers, each building model is divided into three stages. Stage 1 – 0 to 15th storey, Stage 2 – 16th to 30th storey and Stage 3 – 30th to 45th storey. The stages considered here are w.r.t vertical geometric irregularity.

3.1. Storey Drift, Storey Displacement and Storey Stiffness by Response Spectrum and Dynamic Gust Wind Analysis

Figure 6 and Figure 7 show the variations of storey drift and Figure 8 and Figure 9 show the variations of storey displacements by response spectrum method and dynamic gust wind analysis respectively for all the friction damper configurations considered in the study. The sudden split of drift and displacement values at the 15th and 30th storey levels is due to the changes in the aspect ratio at these levels. The storey stiffness values are observed to peak at a 1st-floor level for all the configurations due to the maximum story shear.

In stage 1, storey drifts of A6, A2, A7 and A3 seem to be closer upto the 10th storey as seen from Figure 6 whereas, storey displacements of A6, A2, A7 and A3 seem to overlap in Figure 8. Both drift and displacement values of these configurations are lesser than in any other configurations considered in the study. Both in stages 2 and 3, storey drifts and storey displacements of A6 are the least as compared to other configurations at any storey and displacements of A2 are very close to A6. The topmost storey drift and the topmost storey displacement in A6 are respectively 26.96% and 19.19% lesser than that of A1. From Figure 7 and Figure 9, in Stage 1, storey drifts of all configurations are observed to follow the same pattern and storey displacements of all configurations are closer with A2 being the least followed by A6. In stage 2, drift values of A6 and A2 are very close up to the 25th storey and displacement values of A6 and A2 are very close up to the 30th storey. Both in stages 2 and 3, storey drifts and storey displacements of A6 are the least as compared to other configurations at any storey. Topmost storey drift and topmost storey displacement in A6 are respectively 20% and 16% lesser than that of A1. In both response spectrum and dynamic gust wind analysis, the storey drifts of A6 and A2; A8 and A4; A7 and A3; and A9 and A5 are seen to follow similar patterns in stages 2 and 3. Storey drifts of all configurations from dynamic gust analysis are found to exceed the limiting value apart from A6. In general, the location of the friction dampers played a more prominent role in controlling storey drifts than the type of friction dampers.

Figure 10 and Figure 11 show the variations of storey stiffnesses by response spectrum method and dynamic gust wind analysis respectively. A6 has higher storey stiffness at all the storeys. 45th storey stiffness of A6 is 25.75% and 50.84% higher by dynamic gust wind and response spectrum analyses respectively than that of A1. Hence, A6 can be concluded as a better friction damper configuration in controlling storey drifts and storey displacements in bundled tube high-rise structural systems especially at higher storeys and this is justified by higher storey stiffness.

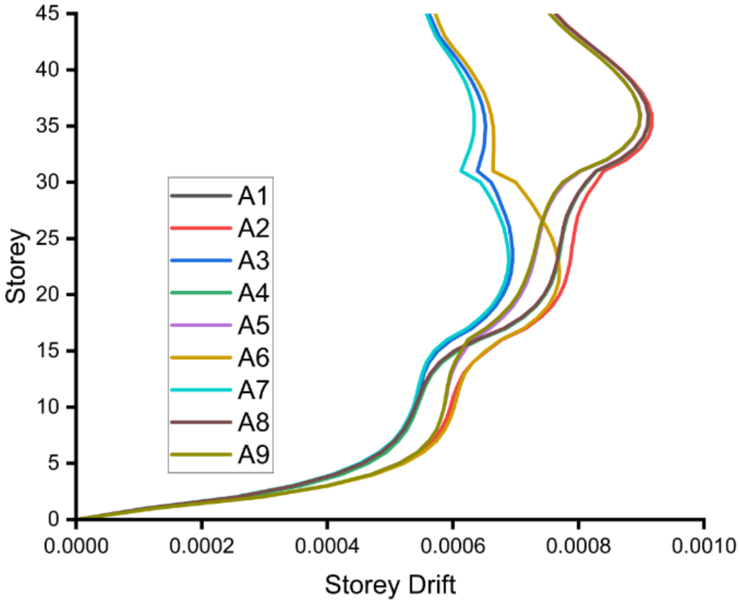


Figure 6. Variations of storey drifts of all friction damper configurations by response spectrum method

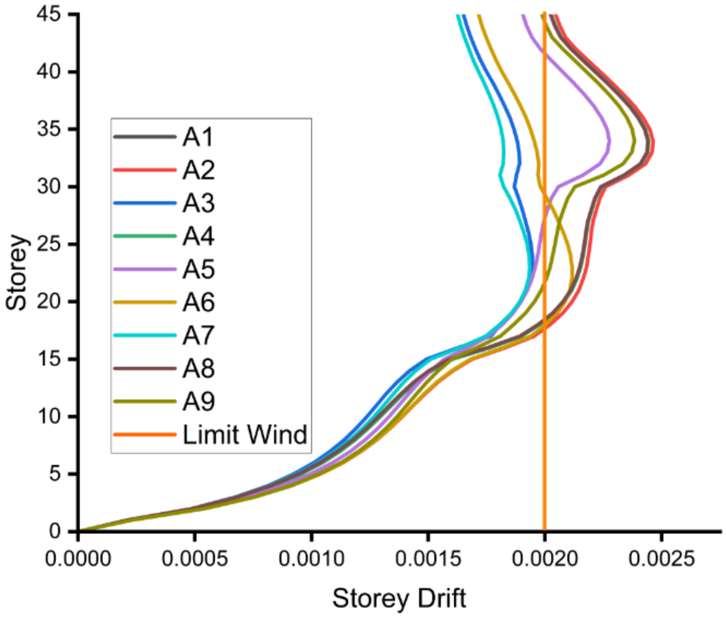


Figure 7. Variations of storey drifts of all friction damper configurations by dynamic gust wind analysis

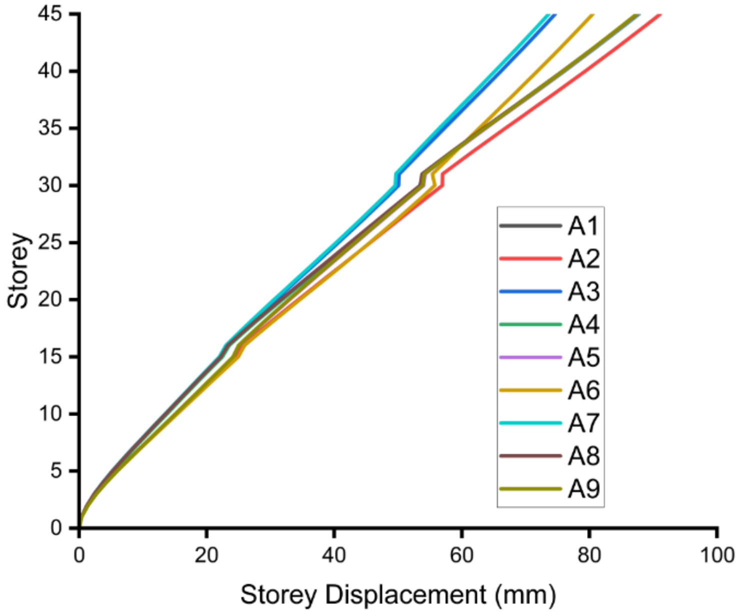


Figure 8. Variations of storey displacements of all friction damper configurations by response spectrum method

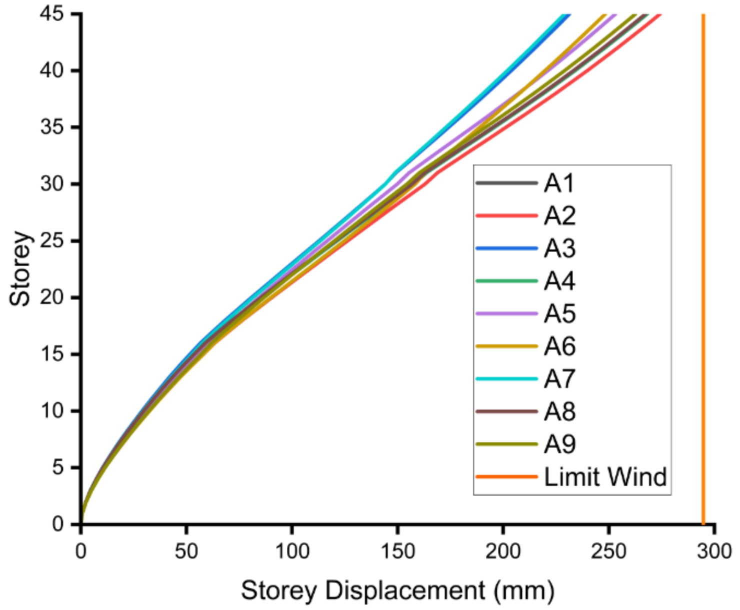


Figure 9. Variations of storey displacements of all friction damper configurations by dynamic gust wind analysis

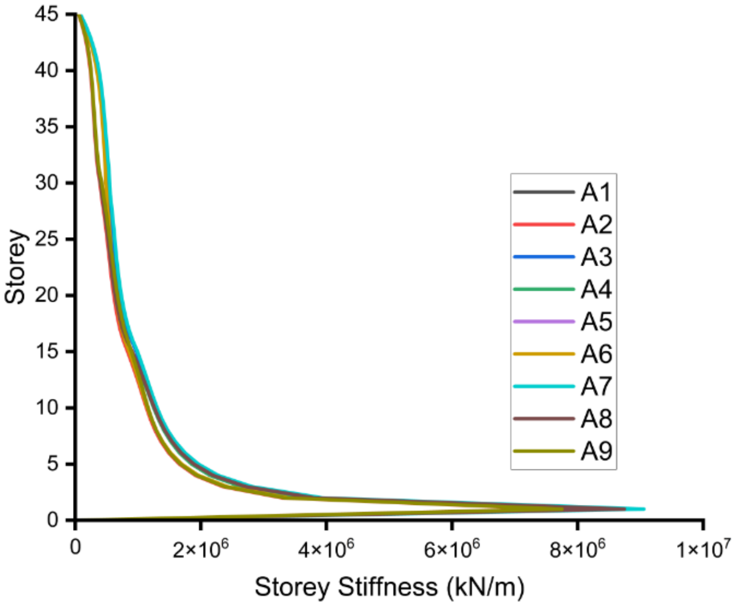


Figure 10. Variations of storey stiffnesses of all friction damper configurations by response spectrum method

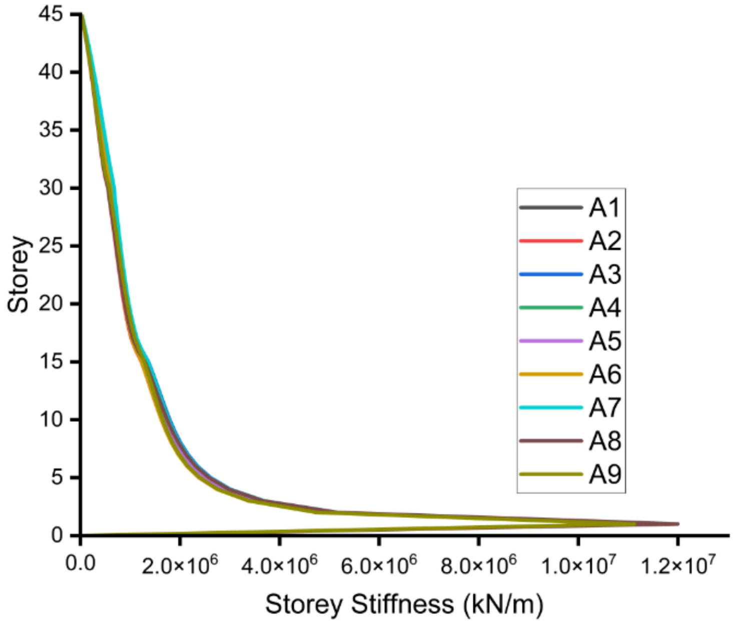


Figure 11. Variations of storey stiffnesses of all friction damper configurations by dynamic gust wind analysis

3.2. Joint Displacements and Joint Accelerations by Time-history Analysis

Figure 12 and Figure 13 show the variations of maximum joint displacement and maximum joint acceleration values for all the friction damper configurations considered in the study for ground accelerations of El Centro (near fault) and Kobe earthquakes (far fault) by time-history analysis. The least of maximum values of all configurations has been obtained for A6 followed by A2.

From Figure 12, less joint displacement is obtained for A6 followed by A2. For all other configurations, higher joint displacements are obtained with a maximum value for A1. Maximum joint displacements obtained for A6 for El Centro and Kobe earthquakes are 58.94mm and 224.91mm respectively which are 33.9% and 26% less than that of A1. Similarly, for A2, joint displacements obtained for El Centro and Kobe earthquakes are 62.73mm and 232.26mm respectively which are 25.8% and 21.9% lesser than that of A1. From Figure. 13, lesser joint acceleration is obtained for A6 followed by A2 similar to joint displacement values. For all other configurations, higher joint accelerations are obtained with a maximum value for A1. Maximum joint accelerations obtained for A6 for El Centro and Kobe earthquakes are 1.11m/s^2 and 3.7m/s^2 respectively which are 15.3% and 12.7% less than that of A1. Similarly, for A2, joint accelerations obtained for El Centro and Kobe earthquakes are 1.15m/s^2 and 3.81m/s^2 respectively which are 11.3% and 9.5% less than that of A1.

From time-history analysis, A6 can be concluded as best suited to control joint displacement and joint accelerations in bundled tube high-rise structural systems for near and far fault data as compared to other friction damper configurations considered in the study due to its considerably enhanced stiffness.

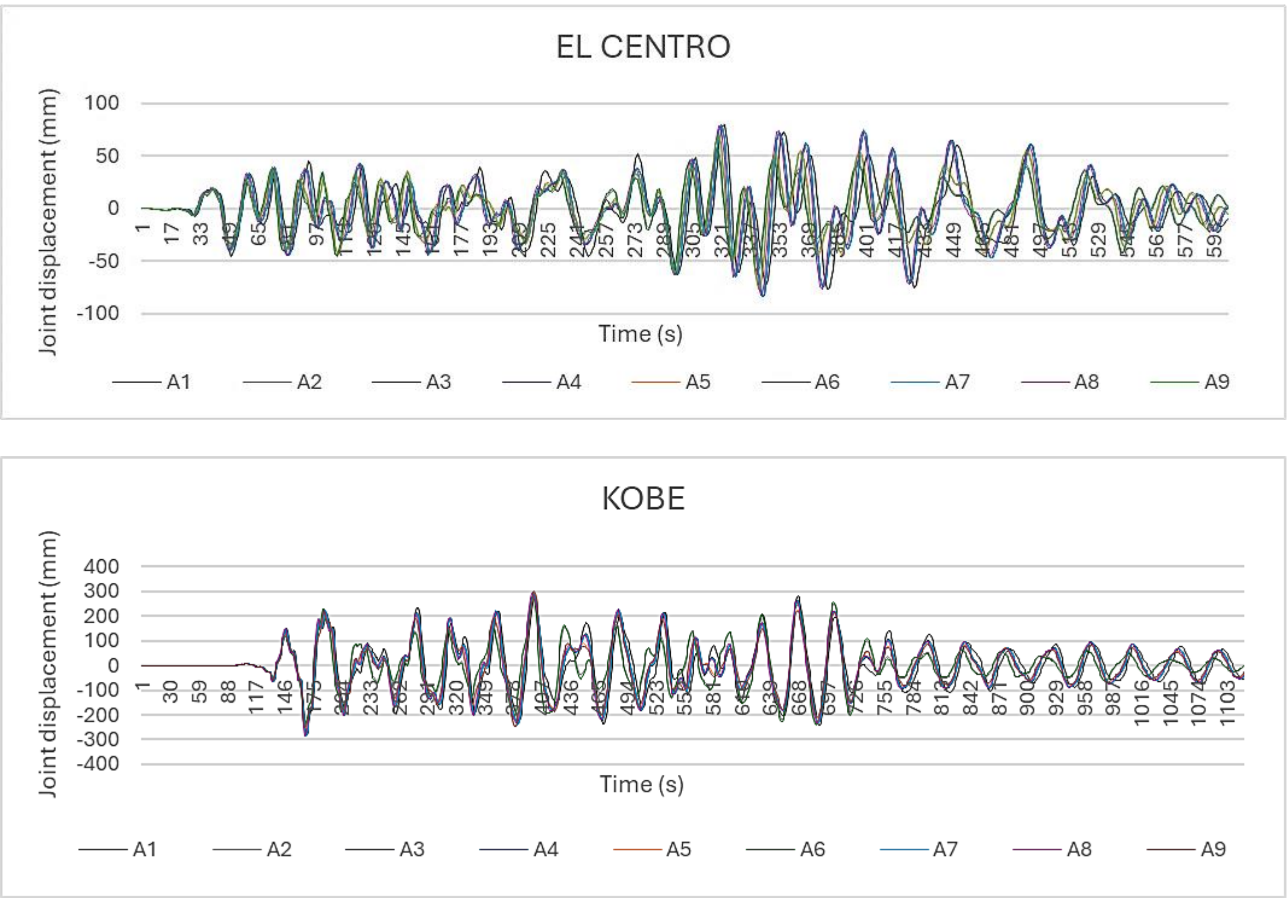
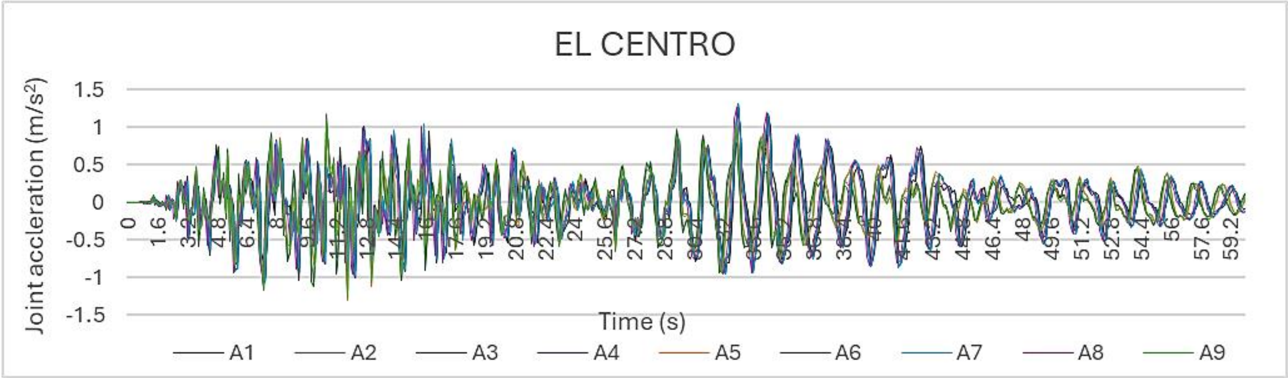


Figure 12. Variations of maximum joint displacement values of all building configurations by time-history analysis of El Centro and Kobe earthquakes



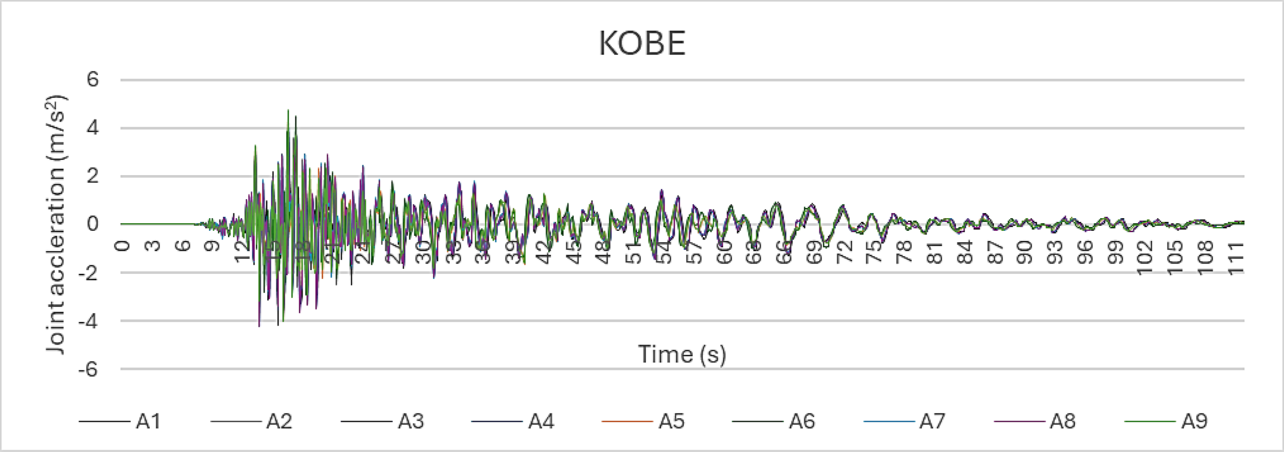


Figure 13. Variations of maximum joint acceleration values of all building configurations by time-history analysis of El Centro and Kobe earthquakes

4. Conclusions

In the present study, an attempt has been made to minimize the response to the significant threats caused by earthquakes and wind forces to high-rise structures causing extensive damage and loss of life and property. Effects due to lateral forces have been controlled in this study using friction damper arrangements, which act as a reusable fuse to control the lateral forces in a bundled tube high-rise structural system. The adoption of friction dampers in the system significantly reduced the drift, displacement, joint acceleration, and joint displacements due to the increase in story stiffness. The findings from the investigation of structural systems with various friction damper configurations highlight the significance of the arrangement of friction dampers in improving the resistance of structural systems to lateral loads. Hence suitable arrangement of friction dampers in a bundled tube system is effective in controlling the effect of lateral forces with the additional advantage of reusability and cost-effectiveness.

- At lower stories, storey drift and storey displacement values of various configurations are very close and at higher stories least values are obtained for A6. By the response spectrum method, the topmost storey drift and displacement values in A6 are respectively 26.96% and 19.19% less than that of A1.
- By dynamic gust wind analysis, topmost storey drift and topmost storey displacement in A6 are respectively 20% and 16% less than that of A1. Storey drift values of dynamic gust analysis of all configurations are found to exceed the limiting value apart from A6.
- The topmost storey stiffness of A6 has been obtained as 25.75% and 50.84% higher than A1 by dynamic gust wind and response spectrum analyses respectively.
- From time-history analysis considering near and far fault seismic data, minimum joint displacement and accelerations have been obtained for A6. 33.9% and 26% lesser joint displacement values were obtained for A6 as compared to A1 for near-fault and far fault respectively. Similarly, 15.3% and 12.7% lesser joint acceleration values were obtained for A6 as compared to A1 for near-fault and far-fault respectively.
- Building system with X-type friction dampers arranged throughout the height of the building (A6) can be concluded as a better friction damper configuration in resisting lateral forces in bundled tube high-rise structural systems, especially at higher stories.

Scope for future work:

- Studies can be considered for different degrees of vertical irregularities.
- Studies can be carried out for different heights of bundled tube high-rise building system.
- Time-histories of different intensities can be considered in obtaining refined lateral response.

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