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Performance of Steel Moment Resisting Frame with Friction Damper in Different Bracing System

C.S. Patil [a*], V.B. Patil [b], S. B. Kharmale [c]

[a]*Doctoral Candidate, Department of Civil Engineering, B V B College of Engineering & Technology, Hubli, Karnataka, India.*

[b]*Professor, Department of Civil Engineering, B V B College of Engineering & Technology, Hubli, Karnataka, India.*

[c]*Asiistant Professor, Department of Civil Engineering, Government College of Engineering and Research, Awasari (Khurd), Maharashtra State, India.*

Abstract:

Energy dissipation device may play significant role in reducing seismic performance of structure without causing damage to structural and non-structural elements. In this paper, 21-story steel moment resisting frame design with a performance-based plastic design (PBPD) for high ductility demand. The result of nonlinear static pushover analysis (NSPA) shows that PBPD frame achieved preselected performance objectives in terms of yield mechanism and yield drift. Whereas nonlinear time history analysis (NLTA) result shows that the drift ratio exceeds the permissible limit due to P-delta effect is critical in structure for higher ductility demand. To control excessive drift, 21-story SMRF proved supplementary friction damper in different bracing system in exterior and center bay of all story. The results clearly showed that the friction damper can significantly improve the seismic behavior of the SMRF in all bracing configuration. Chevron braced friction damper exhibits excellent seismic performance over single and double diagonal friction bracing arrangements in both exterior and interior bay cases.

Keywords*:* Friction damper**,** Performance based plastic design, Steel moment resisting frame, Nonlinear time history analysis, P-delta, Inter story drift.

1. INTRODUCTION

Performance Based Plastic Design (PBPD) is a rational approach to designing a new building that often produces superior results compared to conventional approaches to forced design. The structural engineer works with the owner to identify specific structural targets for serviceability and strength with PBPD; the structure is then designed to ensure that the stipulated standards are met. PBPD is a rapidly growing technology for new building design and evaluation of existing structure presented in Vision 2000 (SEAOC, 1995), ATC40 (ATC, 1996), FEMA 273 (FEMA, 1997), SAC / FEMA350 (FEMA, 2000a) and FEMA 356. PBPD method presented by Lee and Goel (2001) is based on pre-selected yield mechanism (desired strong column-weak beam yield mechanisms) and target drift as performance target for design of steel moment resisting frame (SMRF). Later on, PBPD successfully applied on SMRF by Chao and Goel (2006), Chao and Goel (2008), Dasgupta et al, (2004) and Kharmale and Ghosh (2011). It is expected that the structural and non-structural components will experience almost no damage in response to the earthquake design, Collapse Prevention (CP), in which the structure should remain standing but is severely damaged. It is based on inelastic demand spectrum formulations derived from the method of capacity spectrum using Newmark– Hall reduction factors (Newmark and Hall, 1982). The design base shear for a specified level of hazard is calculated by equating the work required to monotonically push the structure up to the target drift. Increasing the target ductility ratio also significantly increases structural displacement due to P-delta effect in case of mid-rise to high-rise structure.

In recent decades, significant research has been carried out into the P-Delta effect and the influence it has on structural response. In recent years, displacement-based studies have become more interesting as a result of the shift to performance-based design procedures. In the initial design phases, counting this P-delta effect shows that there is an increase in beam and column section member size. The performance objectives of PBPD is significantly varies due to increasing strength of SMRF. Alternatively, the bracing system is more effective in controlling excessive drift in SMRF. This also changes the structural configuration and the performance targets considered during PBPD are different. The best solution to control excessive drift in mid-rise SMRF due to P-delta effect is provide supplementary damping device.

One of the most cost-effective methods of seismic energy dissipation is the friction damper. Pall Friction dampers reduce the initial construction cost significantly while increasing earthquake resistance to damage dramatically. Friction dampers are made of steel plates that are tightened together by means of high-strength bolts with either axial or rotational deformation mechanisms that transform kinetic energy into thermal energy. The necessary slip load and yield displacement is the two main outputs of the structural design. Pall (1979; 1980; 1981) introduced Pall Friction Dampers that significantly reduced the initial construction costs and dramatically increased earthquake resistance to damage. They can be installed in accordance with single-diagonal or chevron steel braces, at the intersection of the X-bracing system and in parallel with the beam at the top of the chevron bracing system, depending on the type of friction devices. Several researchers have recently been working on optimizing the dampers ' location in structures and their

[*] *Corresponding Author: Email: patil.cs@sginstitute.in*

parameters (Soong and Dargush (1997), Fang et al. (2012), Miguel et al. (2015)).

Section 2 explains the performance-based plastic design (PBPD). Design for 2.0 percent target drift and structural ductility factor = 3.0 in Section 3, 21-story SMRF frames. Section 4 explains 21-story SMRF added with friction damper for different bracing systems. In Section 5, comparison of interstory drift and displacement profile of 21-story SMRF with and without friction damper is explained and at the end; conclusion is explained in Section 6.

2. Performance Based Plastic Design (PBPD)

Lee and Goel (2001) stated that the design base shear for selected levels of ductility is determined by equating the work required to push the structure monotonically to the target drift to the corresponding energy demand. Goel and Chao (2008) further demonstrated that the resulting design base shear obtained from the energy balance could be expressed in Equation (1).

$$\frac{V_{by}}{W} = \frac{-\alpha + \sqrt{\alpha^2 + 4\gamma \left(\frac{S_a}{g}\right)^2}}{2} \tag{1}$$

Where α is a dimensionless parameter, which depends on the stiffness of the structure, the modal properties, and the intended drift level, and it is given by Equation 2, as,

$$\alpha = \left(\sum_{i=1}^{n} \lambda_i \ h_i\right) \frac{\theta_p 8\pi^2}{T_1^2 g} \tag{2}$$

The modification factor is a function of ductility reduction factor (R_μ) and the structural ductility factor (μ_s), according to Equation **3**.

$$\gamma = \frac{2\mu_s - 1}{R_\mu{}^2} \tag{3}$$

Where, $\lambda_i = \frac{V_{by}}{F_i}$ is the shear proportioning factor of the equivalent lateral force at level i. In the current seismic codes, the proportioning factor can be expressed in terms of the weight of the structure at level i and the height of beam level i from the ground. $\frac{S_a}{g}$ is design acceleration spectrum value, V_{by} is the lateral yield base shear and W is the total seismic weight of the structure, the global drift of the structure can be controlled by limiting the amount of plastic rotation θ_p.

3 Design of 21-story SMRF as per PBPD

The study frame 21-story SMRF design with spectral response acceleration parameters for a period of 0.2 seconds and 1 seconds calculated as ASCE7-16 (2016), S_{DS}= 1.622 g and S_{D1}= 0.853 g in Loss Angeles, USA (34.052 ° N, 118.243 ° W). The site ground is considered Site Class D (stiff ground) and it is assumed that the building's occupancy category is I based on its use as a building. The structure is designed in order to comply with the PBPD method developed by Lee and Goel (2001) for a displacement ductility ratio of 3.0 at a uniform target drift of 2.0% using an estimated basic period, or steel moment frames are 2.37 sec. The building plan show in Figure 12 is assumed to have uniform seismic weights of 9240 kN per floor from first floor to roof floor. Design base shear coefficient for 21-story SMRF is 0. 053. The members were designed by using plastic design procedure and AISC-LRFD Specification [AISC 2005]. Figure 2 shows the final member sizes of the frames designed by the proposed 2% target drift procedure for ductility ratio 3.0.

In order to study the response of the 21-story SMRF using SAP2000, nonlinear static pushover analysis (NSPA) was performed. Figure 3 showed the base shear versus roof displacement of the 21-story SMRF ductility ratio 3.0 obtained from the pushover analyzes. The base shear coefficient for 21-story SMRF obtained from NSPA are 0.054 (design base shear coefficient 0.053). The yield drifts and the base shear coefficients of the frames evaluated in NSPA are very close to the expected values in the assumed values of the design procedure. Figure 4 shows the location of the formation of plastic hinges at 2 percent target drift for 21-story SMRF are similar to preselected yield mechanism during design. Yielding observed only in the beams and column bases, confirming the reliability of PBPD. The Nonlinear Time History Analysis (NTHA) is performed under the ground motion records of Chi-Chi, Hollister, Kobe and Northridge shows in Figure 5. The scale factor used in NTHA for the selected ground movement records is described in Table 1. For the 21-story frame, the maximum story drift obtained in NTHA is 3.1%, 3.2%, 4.2% and 5.0% respectively for Chi-Chi, Hollister, Kobe and Northridge, Kobe earthquake record which exceed the design target drift (2%) at the lower and middle floors. Because of P-that becomes more significant with increasing stability ratio and intensity values (or demand for ductility). This emphasizes that the addition of supplementary damping device in SMRF can control this excessive drift.

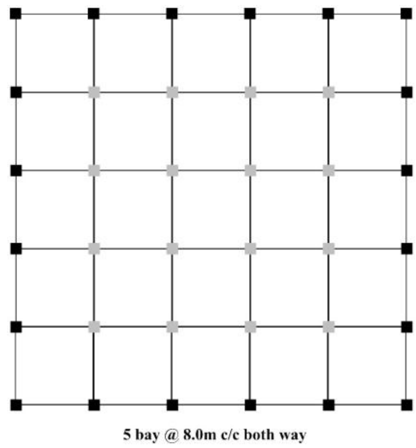


Figure 1. Plan of the 21-Story Frame SMRF

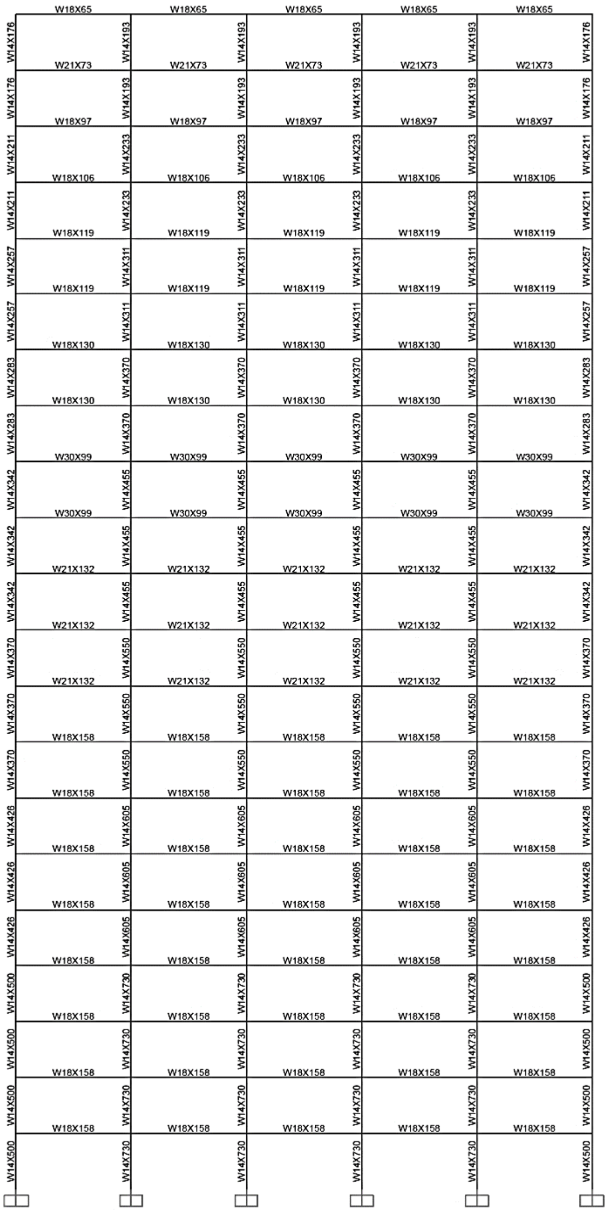


Figure 2. Member Section for Beam and Column of 21-story SMRF

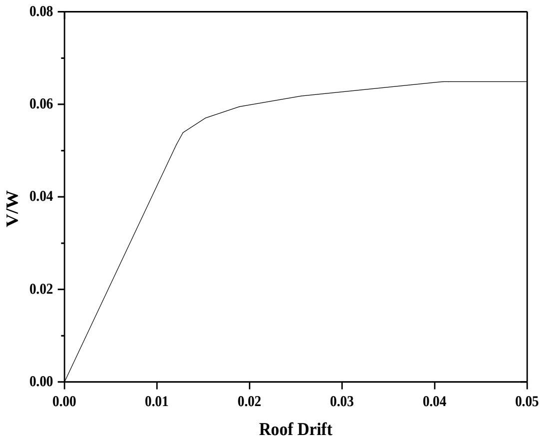


Figure 3. Base Shear Coefficient versus roof drift response of the 21-story SMRF.

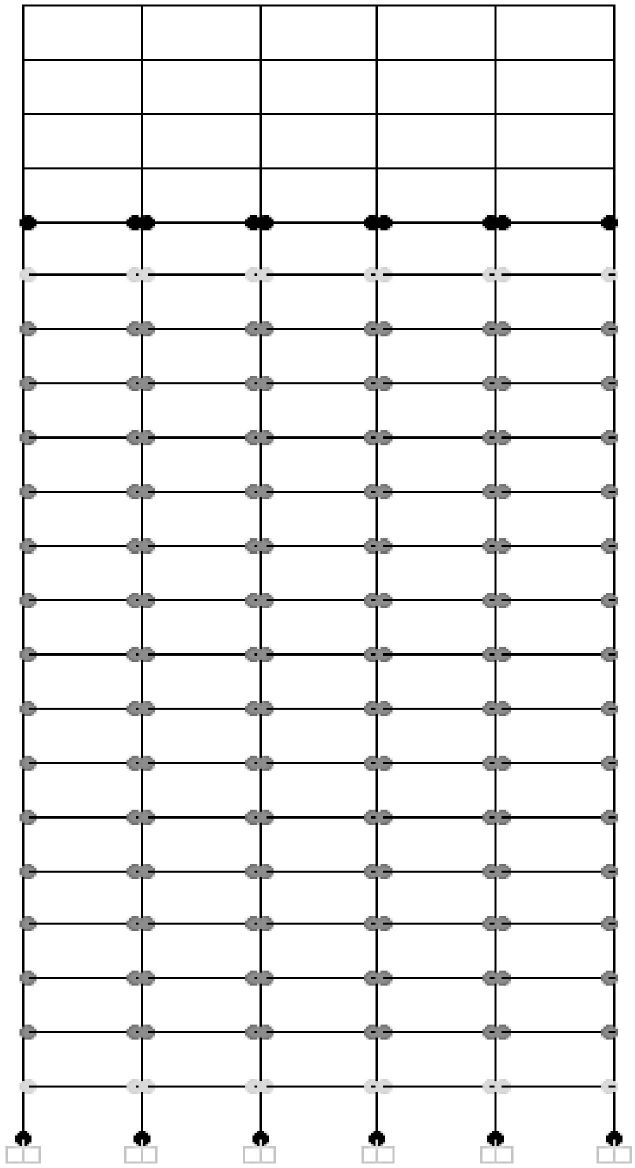


Figure 4. Formation of plastic hinges in NSPA for21-story at 2% target drift.

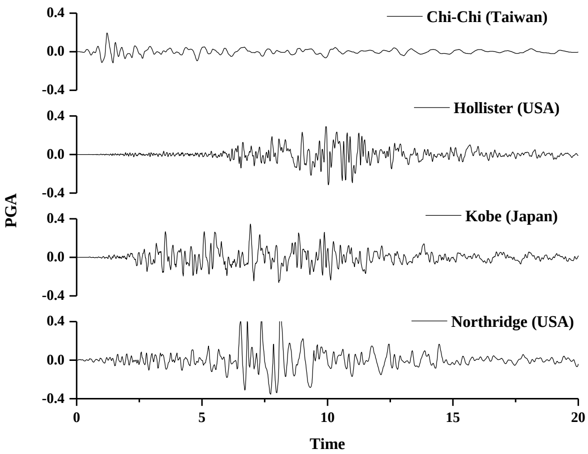


Figure 5. Selected ground motions data for NTHA

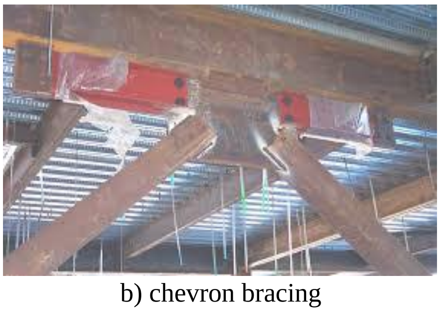
Table 1. Details of ground motion record for NTHA

Earthquake	Location	Scale Factor
Chi Chi	Taiwan: September 20, 1999	6.36
Hollister	USA: April 09, 1961	9.04
Kobe	Japan: January 16, 1995	4.08
Northridge	USA: January 17, 1994	3.98

4. 21-story SMRF with friction damper

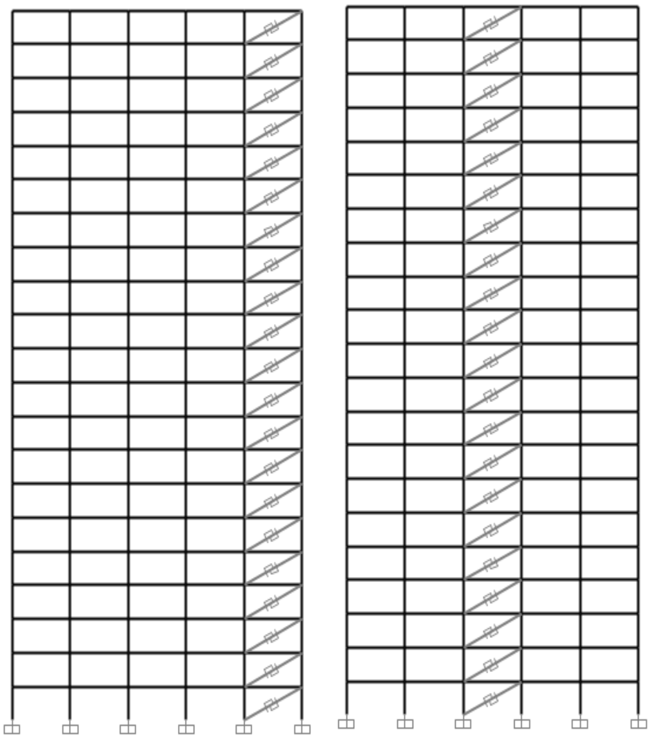
The main objective is to emphasize the advantages of using friction dampers to reduce the displacement. Single and double braced consist a diagonal friction device with slotted bolted connections as shown Figure 6 (a). The chevron friction damper (CFD) modeled as friction damper was added to connect the top part of chevron braces to the mid-span of a moment of frame beam resistance as shown in Figure 6 (b). The friction damper is equipped with a pin connected to the frame at the center of each frame. When placed in the bracing system, a friction damper behavior is a non-linear. Their slip force strongly influences the reaction of friction damping structures. The selected slip force must be high enough to prevent damper slippage at low lateral load values, whereas it must be low enough to allow slippage before the main structural elements are produced. 21-story SMRF is modified by adding supplementary friction device in six model as single braced friction damper at center span (S21.SBC) and at end span (S21.SBE), double braced friction damper at center span (S21.DBC) and at end span (S21.DBE) and chevron braced friction damper at center span (S21.CBC) and at end span (S21.CBE) shown if Figure 7, Figure 8 and Figure 9.

a) single bracing



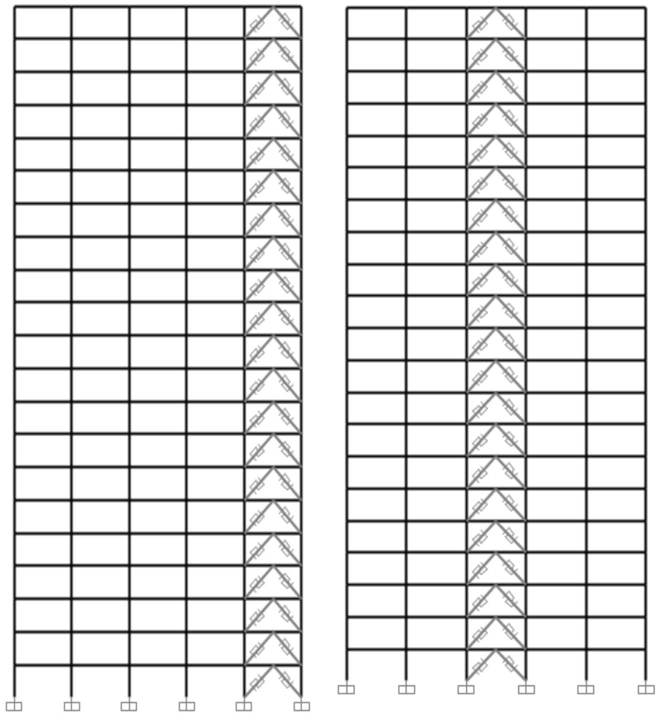
b) chevron bracing

Figure 6. Friction Damper



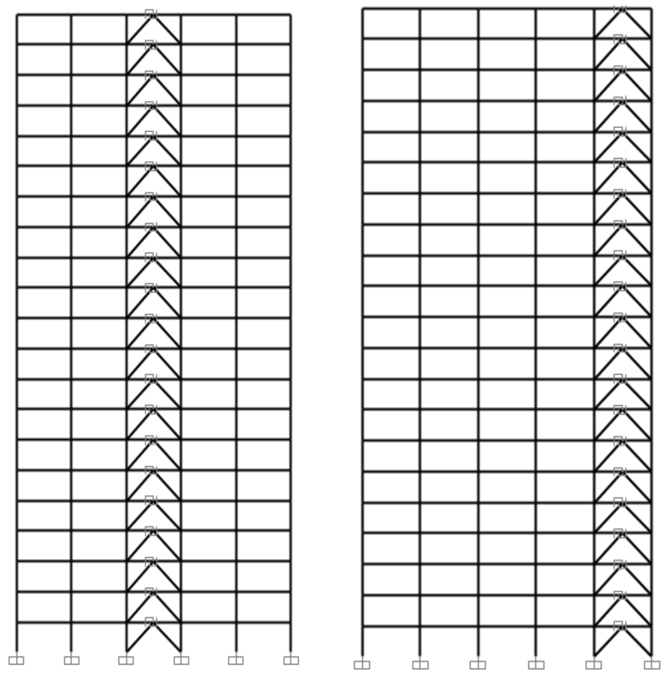
(a)Exterior Bay (S21.SBE) (b)Center Bay (S21.SBC)

Figure 7. 21-story SMRF with Single Braced Friction Damper



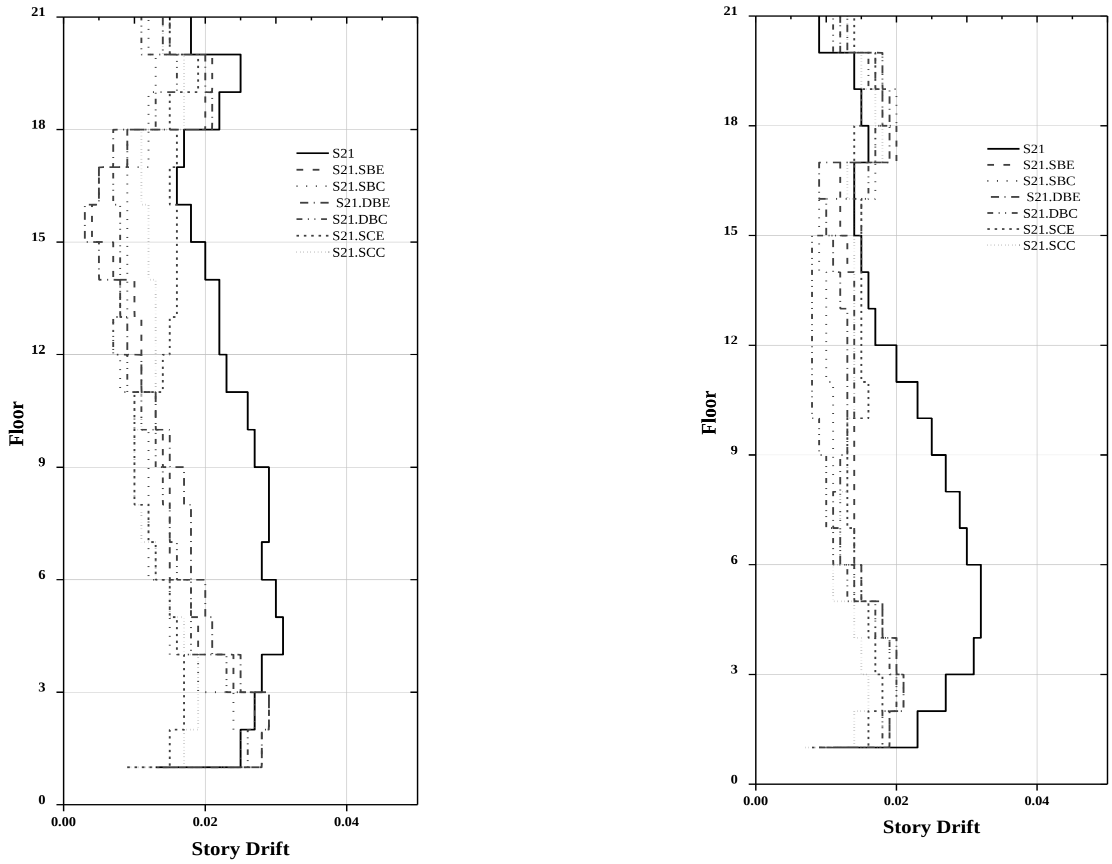
(a)Exterior Bay (S21.SBE) (b)Center Bay (S21.SBC)

Figure 8. 21-story SMRF with Single Braced Friction Damper



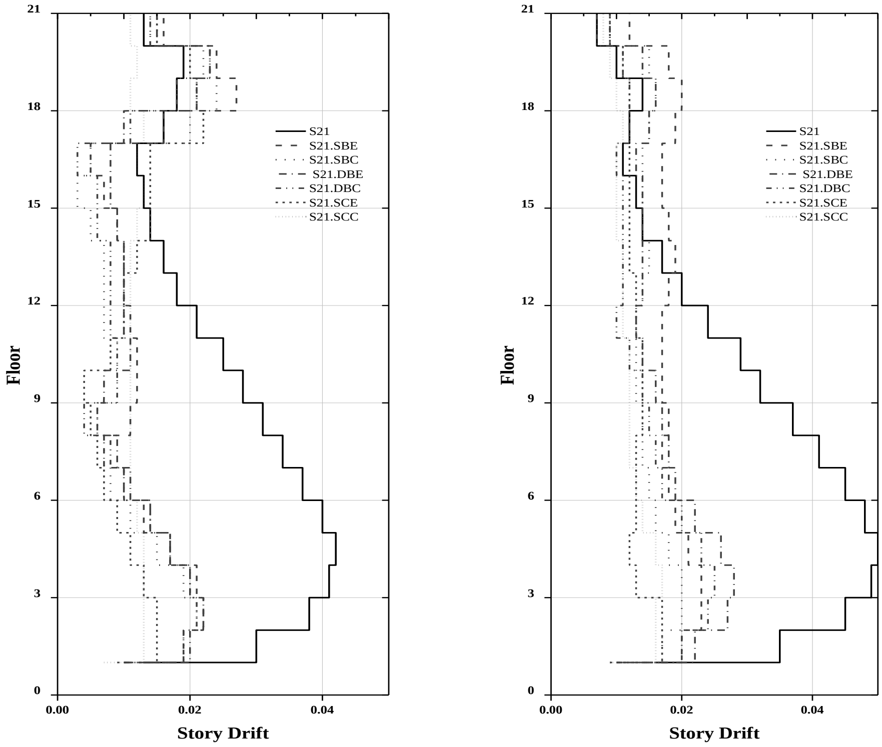
(a)Exterior Bay (S21.CBE)   (b)Center Bay (S21.CBC)

Figure 9. 21-story SMRF with Chevron Braced Friction Damper



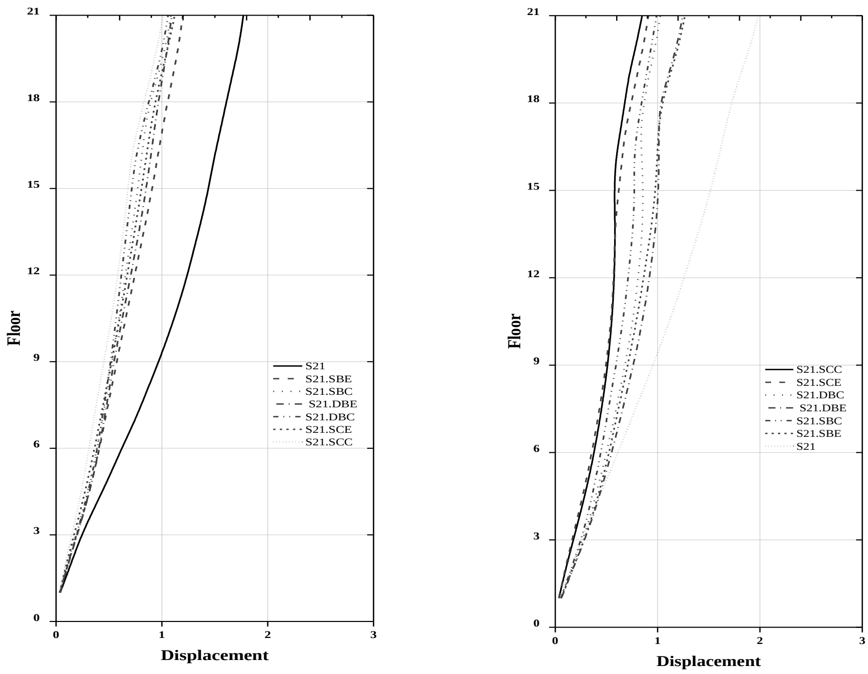
a) Chi-Chi   **b) Hollister**

Figure 10. Maximum IDR of 21-story SMRF under selected earthquakes.



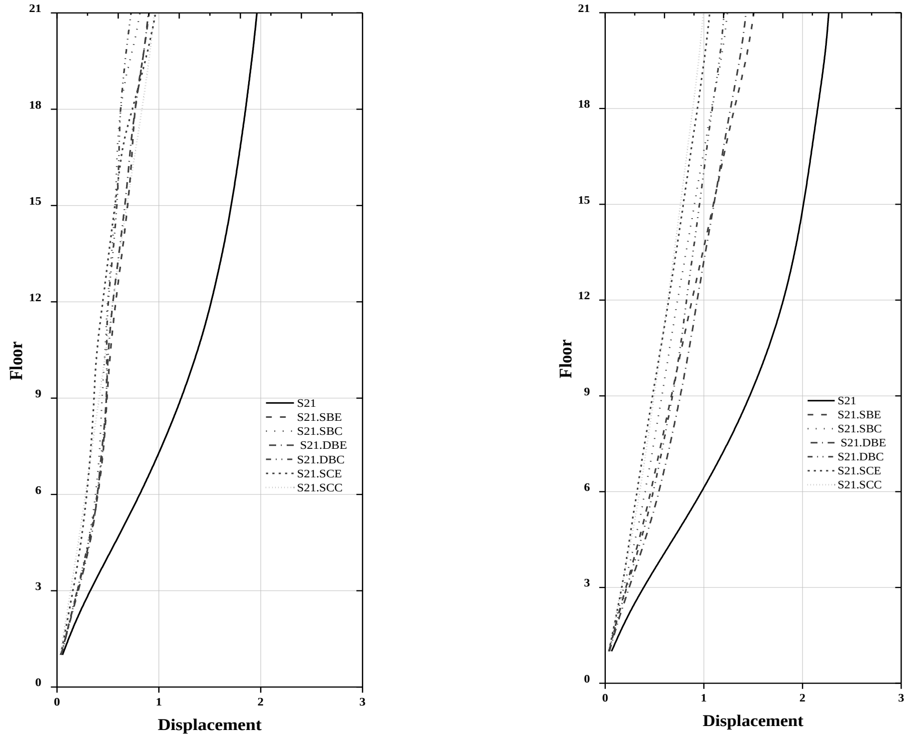
c) Kobe **d)** Northridge

Figure 10. Maximum IDR of 21-story SMRF under selected earthquakes.



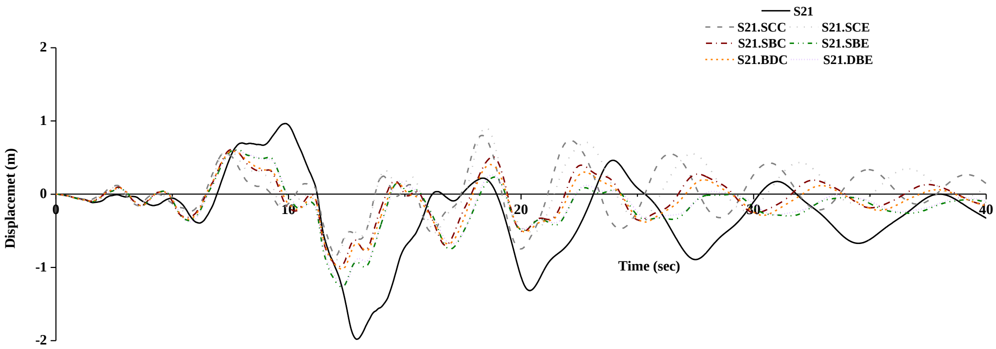
a) Chi Chi **b) Hollister**

Figure 11. Maximum Story Displacement of 21-story SMRF under selected earthquakes.

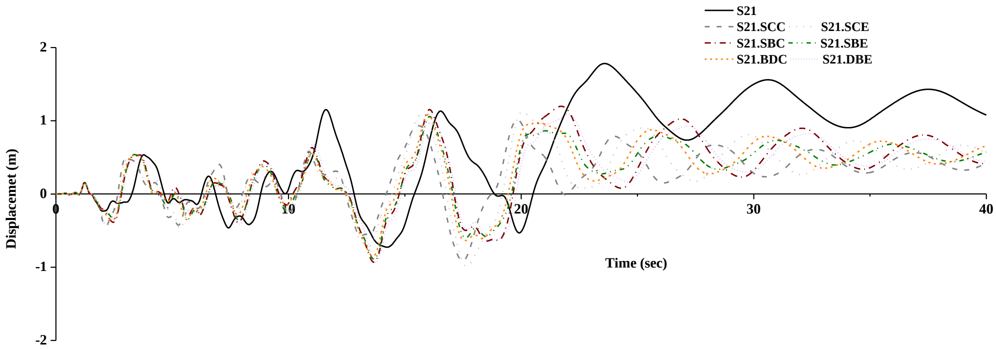


c) Kobe **d)** Northridge

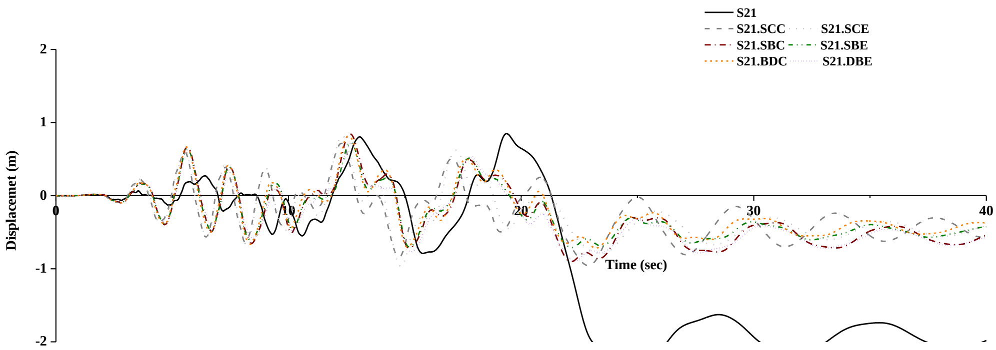
Figure 11. Maximum Story Displacement of 21-story SMRF under selected earthquakes.



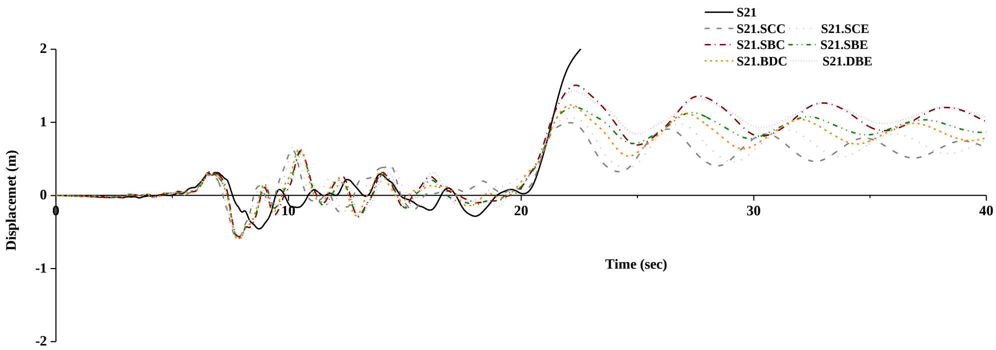
a) Chi-Chi



b) Hollister



c) Kobe



d) Northridge

Figure 12. Roof Story Displacement of 21-story SMRF under selected earthquakes.

5. RESULTS & DISCUSSION

For selected ground motion data, Figure 10 shows the maximum inter-story drift of 21-story SMRF with and without friction damper. As it is easy to recognize the structure equipped with friction devices, the drift of the story was significantly reduced. The 21-story SMRF with single braced friction damper reduced maximum interstory drift from 5.0%, 4.2%, 3.2% and 3.1% to 2.3%, 2.7%, 2.1% and 2.9% for Northridge, Kobe, Hollister and Chi-Chi earthquake records respectively when added at exterior bay in all story level. Similarly, maximum interstory dirt reduced up to 2.5%, 2.0%, 2.4% and 2.0% respectively when single braced friction damper added at center bay in all story level of 21-story SMRF (Table 2). In fact, the maximum interstory drift for selected ground motion data is estimated 2.8%, 2.3%, 2.1% and 2.9% respectively for 21-story SMRF with double braced friction damped added in exterior bay, while estimated 2.5%, 2.3%, 2.1% and 2.7% respectively for 21-story SMRF with double braced friction damped added in center bay. When friction damper provided exterior bay with chevron bracing for 21-story SMRF, maximum interstory drift reduced 1.7%, 2.2%, 1.8% and 1.9% respectively 1.82% and 1.72% respectively for Northridge, Kobe, Hollister and Chi-Chi earthquake records. Whereas, reduction in maximum interstory drift was estimated 1.7%, 1.3%, 1.8% and 1.9% respectively when chevron bracing friction damper added in center bay of 21-story SMRF. Result showing in Table 2 confirming the fulfillment of the first design objective of installing friction dampers in all arraignment of 21-story SMRF.

Table 2. Maximum Story Drift obtained in NTHA

Earthquake	Maximum Interstory Drift						
	S21	S21.SBE	S21.SBC	S21.DBE	S21.DBC	S21.CBE	S21.CBC
Chi Chi	3.10%	2.90%	2.50%	2.90%	2.68%	1.90%	1.90%
Hollister	3.23%	2.10%	2.00%	2.10%	2.05%	1.80%	1.80%
Kobe	4.20%	2.70%	2.40%	2.30%	2.26%	2.20%	1.30%
Northridge	5.00%	2.30%	2.00%	2.80%	2.48%	1.70%	1.70%

It is observed that, interstory drift demand can be significantly reduced by adding friction dampers in center bay as compare to friction damper added exterior bay. Also, friction dampers added with chevron bracing was more effective than friction damper added single bracing and double bracing system in both exterior and interior bay. Figure 10 clearly show that for four earthquake record, interstory drift uniformly distributed over all floor of 21-story SMRF. P-delta effect significantly reduced in Kobe and Northridge earthquake record when 21-story SMRF added with supplementary friction device in all different arraignment. Friction damper with chevron bracing shown nearly equal distribution of interstory drift at all floor of 21-story SMRF when provided both exterior and center bay.

Story displacement is an important parameter for analyzing structure response under dynamic loading and provides a better understanding into structure performance as a whole as shown in Figure 11. Average roof displacement (mean + standard deviation) 21-story SMRF without supplementary friction damper was 2.48 m. As 21-story SMRF with single braced friction damper added to exterior bay is 1.49 m and added to center bay 1.27 m. If friction damper with double braced, average displacement of 21-story SMRF is 1.45 m and 1.26 m respectively for dampers added exterior bay and center bay. 21-story SMRF added with friction dampers by using chevron bracing system in exterior bat and center bay is 1.19 m and 1.15 m respectively. Displacement profile of 21-story SMRF with supplementary friction damper shows linear as without damper shows nonlinear nature.

Plots of roof displacement verses time period were plotted under seismic loading for two building models with and without additional damping shown in Figure 12. Using dampers has successfully reduced displacement values of 21-story SMRF at all times compared to building without dampers and has managed to keep the building's overall displacement within a limited range with smooth transitions preventing sudden reversal of displacement load.

6. CONCLUSIONS

From the present study is it concluded that:

1. Adding friction device has been shown to be effective in reducing displacement amplification to within permissible design limits as compared to the original response.
2. Optimum selection and installation of friction dampers at various critical locations can further reduce the damping demand for the structure.
3. Chevron braced friction damper shows excellent seismic performance in 21-story SMRF over single and double diagonal friction bracing arrangement in both exterior and interior bay cases.

References

[1] AISC (2005). ANSI/AISC 360-05. Specifications for structural steel buildings. Chicago (IL): American Institute of Steel Construction.

[2] ASCE 7-16 (2016). Minimum design load for buildings and other structures. Reston (VA): American Society of Civil Engineers.

[3] Fang, J., Wang, Q., Wang, S., Wang, Q. (2012). Min-max criterion to the optimal design of vibration absorber in a system with Coulomb friction and viscous damping. Nonlinear Dynamics, v. 70, p. 393-400.

[4] Goel SC, Chao SH (2008). Performance-based plastic design-earthquake resistant steel structures. International Code Council, Paper No. 266.

[5] Housner G (1956). Limit design of structures to resist earthquakes. In Proceedings of the first world conference on earthquake engineering.

[6] Kharmale S.B. Ghosh Siddhartha (2011). Seismic Lateral Force Distribution for Ductility-Based Design of Steel Plate Shear Walls. Journal of Earthquake and Tsunami: 1250004-1-24.

[7] Lee SS, Goel SC (2001). Performance-based design of steel moment frames using target drift and yield mechanism. Report no. UMCEE 01-17; MI: University of Michigan at Ann Arbor.

[8] Leelataviwat S, Goel S C and Stojadinovic B (1999). Toward Performance Based Seismic Design of Structures. Earthquake Spectra, EERI. 15: 435-461.

[9] Leelataviwat S. (1998). Drift and Yield Mechanism based Seismic Design and Upgrading of Steel Moment Frames, Ph.D. Thesis, Dept. of Civil & Env. Engrg., University of Michigan, Ann Arbor, Michigan.

[10] Miguel, L. F. F., Fadel Miguel, L. F., and Lopez, R. H. (2012). Simultaneous optimization of force and placement of friction dampers under seismic loading. *Engineering Optimization*, v.48, n.4, p. 582-602.

[11] Newmark NM, Hall WJ (1982). Earthquake spectra and design. El Cerrito (CA): Earthquake Engineering Research Institute, Berkeley, USA

[12] Pall AS and Marsh C (1981). Friction-Devices to Control Seismic Response. ASCE/EMD Specialty Conference on Dynamic Response of Structures, Atlanta, USA. 809-818.

[13] Pall AS, Marsh C, Fazio P (1980). Friction Joints for Seismic Control of Large Panel Structures. Journal of Prestressed Concrete Institute. 25:38-61.

[14] Pall AS, Pall RT (2004). Performance-Based Design Using Pall Friction Dampers - An Economical Design Solution. 13th World Conference on Earthquake Engineering. Vancouver, B.C., Canada.

[15] Pall AS., Marsh C (1979). Energy Dissipation in Large Panel Structures Using Limited Slip Bolted Joints. AICAP/CEB Seismic Conference, Rome, Italy. 3:27-34.

[16] Priestley MJ and Kowalsky MJ (2000). Direct Displacement-Based Design of concrete buildings. Bulletin of earthquake engineering. 33:421-444.

[17] Sahoo DR, Chao SH (2012). Seismic Behavior of Steel Buildings with Hybrid Braced Frames. Journal of Structural Engineering:139(6):1019-1032.

[18] SAP2000. Integrated Software for Structural Analysis and Design. Computers and Structures. Inc. Berkeley, California, USA.

[19] Soong, T.T., Dargush, G.F. (1997). Passive energy dissipation system in structural engineering. Chichester, U.K., Wiley.

[20] Uniform Building Code (1994). International Conference of Building Officials, Whittier, California.

[21] Whittle JK, Williams MS, Blake Borough A (2012). Performance of structural members in seismic retrofitted frames with viscous dampers. In Proceedings of 15WCEE.