Efficient Arrangement of Friction Damped Bracing System (FDBS) for Multi-storey Steel Frame

Saikat Bagchi, Avirup Sarkar, and Ashutosh Bagchi

Abstract The Friction Damped Bracing System (FDBS) is able to significantly control the vibration of framed structure without dissipating energy through the inelastic yielding of its structural components. Therefore, it is a useful tool to design the structural system by isolating the energy dissipation components at some specific as well as desired locations. This purposeful isolation of the critical components helps, in turn, to monitor the health of the system efficiently, especially for the large and complicated systems such as process plant structures, offshore structure, etc. Therefore, effective placement of the energy dissipation devices in terms of their numbers as well as locations is essential to meet the optimum requirement of serviceability, safety, and stability. In this article, FDBS is modelled numerically following standard Friction Damper guideline. The 2D building frames with FDBS at various locations are used to study the responses of multi-storey building frames having different vertical bracing configurations. Locations of the energy dissipation devices are altered for each of the structures to study the effect of load flow through the desired load path. It is intended to isolate the FDBS in such a way so that the operational constraints do not interfere with the monitoring and maintenance of the critical dissipating system, which is the lifeline for the structural stability. Nonlinear time history analysis is performed for each of the frames for a scaled ground motion obtained using Conditional Mean Spectra for the city of Vancouver. Energy dissipation behavior of the structures is compared in order to comprehend the effect of damper arrangement. Load versus deflection behaviour of the structures at different levels indicate that structures with regular configurations show better behaviour in comparison to the customized structures with special configurations. Therefore, it is concluded that FDBS enabled structural systems are suitable as well as necessary for the complicated structures where the horizontal load transfer system is expected to be flexible to meet the process requirements.

S. Bagchi (✉) · A. Sarkar · A. Bagchi
Concordia University, Montreal, QC H3G 1M8, Canada
e-mail: er.saikat.ac@gmail.com

1 Introduction

Aseismic design of a multistory steel frame especially in case of an industrial facility
like steel or power plant is very much complicated because of its process-specific
requirement. In those cases, the design engineer needs to take care of not only the
structural constrains but also must accommodate the equipment or facilities for which
the structure is to be constructed. Many a time process requirement is such that the
structural stability system cannot be planned in the desired way. For example, in
case of a silo or water tank supporting Junction House, where in addition to the huge
overburden load belt conveyor enters or exits the building at an angle and an elevation-
difference to each other, usual bracing planning is not possible. Aseismic design of
such structure may call for some amount of energy dissipation using the ductility of
the constituting members. As per the philosophy of capacity design, predetermined
members of the structure are expected to perform in the zone of plastic deformation
while others continue to remain elastic. Energy dissipation through this process
has a serious limitation because in such a situation the weak link of the structural
lattice, i.e., the ones which are designed to operate in the nonlinear range undergo
permanent deformation. In the post-earthquake situation, repair and rehabilitation of
such structures become a difficult task because of those partially damaged members.
Therefore, it is imperative from the perspective of the seismic resilience that the
energy dissipation system be planned and designed in such a way so that it doesn't
impair the major load-bearing system and can perform uninterruptedly even after
the seismic event. Friction damped bracing system (FDBS) is one of such solutions
which has been popular for the last three to four decades. Although the usage of FDBS
is not new, as of now the existing international design standards doesn't provide with
an integrated design approach where the influence of the passive energy dissipation
system, i.e., FDBS in this case, is considered. One of the major reasons behind this
incompleteness is the insufficient study of the structural behaviour with and without
FDBS. The current article reports a numerical study of a five-storey five-bay 2D
frame for different bracing configurations and compares the behaviour of FDBS
with conventional braced frame and moment-resisting frames. The study also tries
to identify the basic scheme for bracing placement which facilitates the maintenance
without hindering the process requirements. Before the reader is introduced with
the detailed working methodology a brief review of the existing state of art friction
damper enabled structural analysis and design practices are presented in the next
section. Subsequently, the models, ground motion input, its selection procedure and
the results are discussed. The article concludes by summarizing the findings of the
study and the scope of further research.

2 Friction Damped Bracing System (FDBS): Brief History and Questions Pertaining to the Design and Application

Passive energy dissipation system works as a response to the external force that is applied to the structure and is intended to be resisted [1]. Friction damper is one of the force-based passive energy dissipation devices which uses friction between two metallic and/or non-metallic surfaces to dissipate energy. The device is widely used in the automobile industry as a component of the automotive brake to dissipate kinetic energy. Avtar Pall introduced the mechanism to the structural requirements with the innovative design of Pall damper applied to the panelled structures [2]. After that, through many studies, the energy dissipation behaviour of this kind of damper including its surface attributes are studied thoroughly [3–5]. Over the time, different types of friction devices are invented and introduced to the structure such as Sumitomo friction damper [6], Energy dissipating restraint [7], the energy dissipator that uses slotted-bolted connection [8, 9], cylindrical friction damper [10], etc. Recently, Quaketek Inc. [11] has developed a friction damper which uses a similar principle that of the Pall damper. In terms of quantifying the behaviour of these devices, two of the important parameters are the initial slip force to storey yield force ratio and the ratio of bracing stiffness to the corresponding storey stiffness [12]. Although several attempts have been made to practice the structural design involving FDBS [13–15], till date no international design standard prescribes designing the structure considering active participation of the passive energy dissipation device [16]. As per the National Earthquake Hazards Reduction Program (NEHRP) recommended provisions (FEMA P-750, 2009) the design load applicable for the seismic force-resisting system without the damper can be reduced up to 75 per cent if the damper is installed [17]. Additionally, in terms of damper arrangement also available guideline is not sufficient. FEMA-356 (2000) stipulates that there should be at least four displacement activated dampers in a storey along any principal direction [18], whereas usually an undamped bracing system may comprise of only one set of vertical bracing in the storey. Therefore, the question is how to use FDBS as a replacement of conventional bracing system (CBS). Moreover, in a horizontally spread building location of the horizontal load transfer system is very important. With the understanding of these requirements in the next section working methodology of this study is described.

3 Working Methodology

In order to understand the behaviour of FDBS over the moment-resisting frame and the conventional braced frame, two sets of frames with both types of bracing configuration are studied through nonlinear time history analysis along with additional two moment-resisting frames. Frames of various bracing configurations (both with FDBS and CBS) are studied and compared to achieve a quantitative inference.

## 3.1 The Numerical Model of the Structure

The structure is modelled numerically using ETABS commercial software [19]. In one of the sets, the vertical bracing system (with and without FD) is located at the central bay of the 5 bay 2D frame and in the other sets the same is located at the corner bay. Apart from these, another two structures with moment-resisting frame (MRF) are studied. In these cases, also the location of the MRF is either at the central bay or the corner bay. Frames with the lateral load transfer system located at the central bay and the corner bay are presented in the Figs. 1 and 2, respectively. Figure 3 shows a typical frame with a soft storey; in the presented case it is shown to be in the 2nd storey. Loads are applied as per the recommendations of the National Building Code of Canada, 2015 (NBCC 2015) [20].
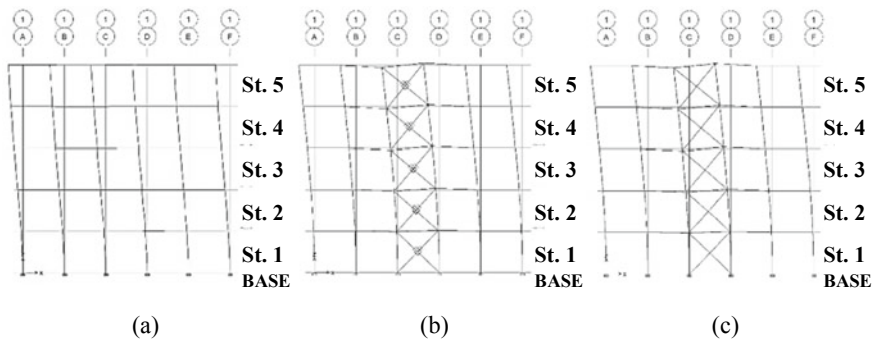
Fig. 1 Frames with the lateral load transfer system located at the central bay; **a** Moment-resisting frame, **b** Frame with FDBS, **c** Frame with CBS
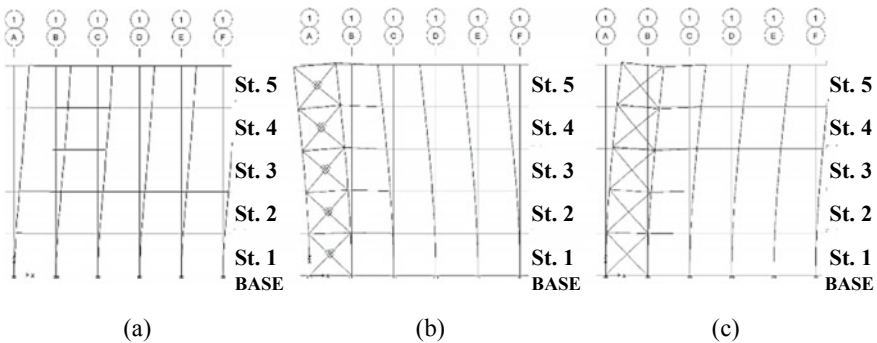
Fig. 2 Frames with the lateral load transfer system located at the corner bay; **a** Moment-resisting frame, **b** Frame with FDBS, **c** Frame with CBS

(a)   (b)   (c)   (d)
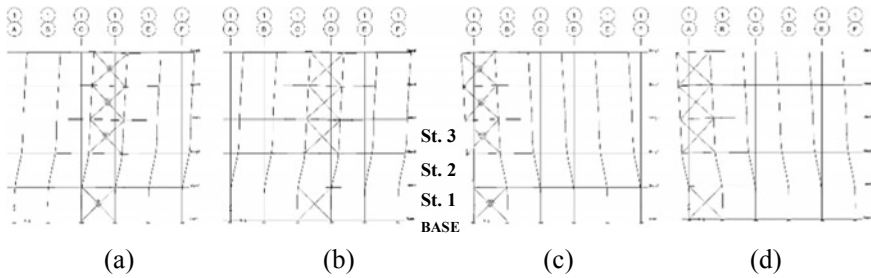
Fig. 3 Typical frames with a soft storey (at the 2nd level); **a** Frame with FDBS at the central bay, **b** Frame with CBS at the central bay, **c** Frame with FDBS at the corner bay, **d** Frame with CBS at the corner bay

3.2 Modelling the Friction Damped Bracing System (FDBS)

Passive energy dissipation using friction damper (FD) involves nonlinear hysteresis energy dissipation technique where nearly rectangular hysteresis loop of the FD characterizes the damper behaviour. In this case, the slip load of a specific damper is considered as the yield load of the member which connects the device. The current numerical study considers Pall type friction damper in the model [21]. Here the FD modelling guidelines provided by Pall Dynamics Inc. and Quaketek Inc. [11] are considered. As stipulated in by the Pall Dynamics, for tension only cross bracing when the damper in the tension brace slips, the compression brace is shortened suitably to avoid the buckling. This feature enables the compression brace to remain perfectly elastic and therefore, during the next cycle of loading when this brace takes the load in tension, it remains ready to take tension in the similar way. The friction damper is idealized as an ideal elasto-plastic link (Wen) as per the procedure provided by Quaketek Inc. The mass of the damper enabled bracing is taken as the sum of the bracing mass and the mass of the damper provided in the catalogue for 450 kN friction damper. The rotational inertia of the damper is not considered because of its strictly axial behaviour.

3.3 Ground Motion Selection

The Selection and scaling of time history records is an important part of the procedure of performance evaluation of structures by time history analysis. There are several methods used in the scaling of earthquake time histories. Scaling makes the selected time histories compatible with a target design spectrum at the fundamental period of the structure so that the results of time history analysis are comparable to the results of response spectrum analysis. In general, the target design spectra for scaling is the Uniform Hazard Spectra (UHS) provided in the design codes for a particular country. However, research has suggested that the UHS is an envelope curve and not

suitable as a target spectrum for scaling procedures. Baker developed the concept of Conditional Mean Spectra (CMS) to be used as a suitable response spectrum for scaling procedures, which is not an envelope curve and matches the spectral amplitude of the UHS at the period of interest, i.e., the fundamental period of the structure but has much lower spectral ordinates at other time period values, using the concept of conditional probability [22]. In this article, scaling has been done using the CMS method provided by Baker. Selection of time history is also important; the chosen time history records should satisfy the seismic parameters of the site for which the structure is designed. In this case, as the building is designed for the city of Vancouver, tentime history records are selected which has a ratio of peak ground acceleration and peak ground velocity as 1, which matches the seismic parameter of Vancouver [23]. For all these ten records, the corresponding response spectrum is developed by MATLAB [24] coding. For the CMS method, the sum of squared errors, over the entire period range, is calculated between the developed CMS for Vancouver and the spectra developed by MATLAB codes corresponding to the time histories selected. Only those time histories which give the minimum values of the sum of squared errors are chosen for scaling by CMS method. Finally, the ratio of spectral ordinates of the CMS and the response spectra of the chosen time histories are calculated to be the scale factor and the time histories are multiplied by that factor to obtain the scaled records, used as the input motions for time history analysis. The input acceleration time histories are shown in Fig. 4.

4   Results and Discussion

As elaborated in the previous section, 2D numerical models of four sets of braced frame steel structures and two moment-resisting frame structures are analyzed against three scaled realistic ground motion acceleration time histories for the site at Vancouver city to comprehend the influence of FDBS and its arrangement in the structural response. In this section, the hysteresis plot of the FDs and different response parameters of the frames such as Base shear, Horizontal displacement, Pseudo spectral acceleration (PSA), etc. are presented.

4.1   Hysteresis Behavior

The Hysteresis behaviour shown in Fig. 5 represents the nonlinear behaviour of the friction dampers for the five storeys of the central FDBS against the GMR—1. It is evident that the damper in the top storey doesn't operate under enough load to be predominantly in the nonlinear range. Thus, the hysteresis loop for the fifth storey is not that prominent.
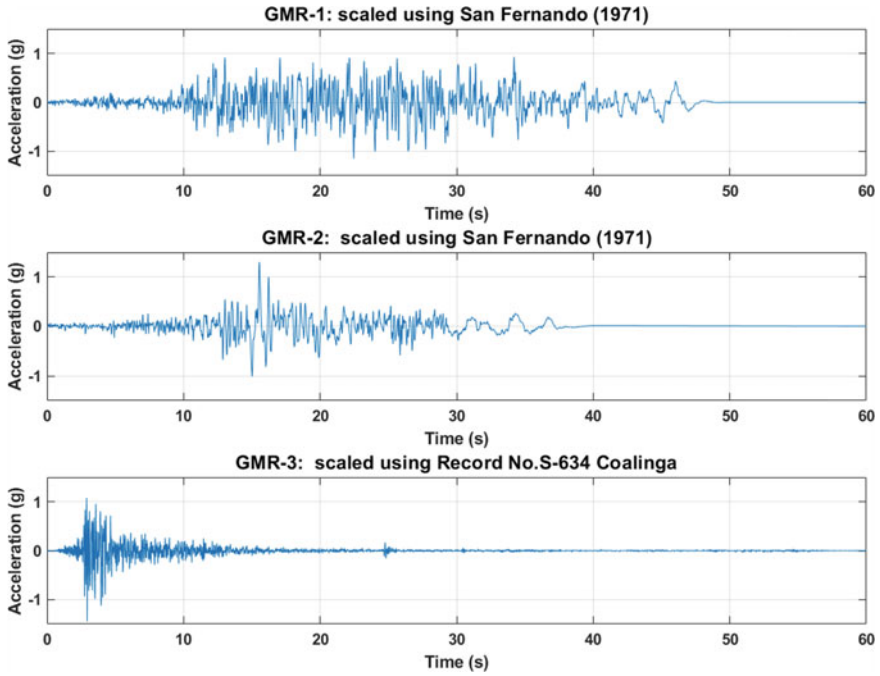
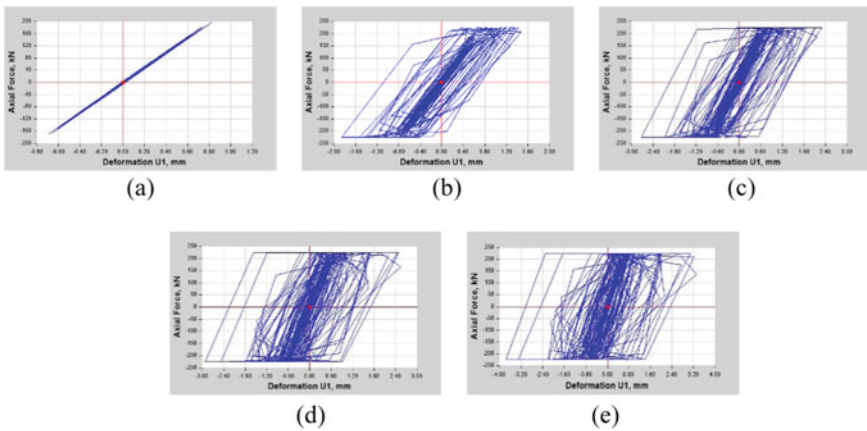**Fig. 4** Scaled ground motion records for Vancouver obtained using conditional mean spectra [25, 26]

Fig. 5 Hysteresis behaviour of the plastic links at different storey levels; **a** Fifth (Top) storey, **b** Fourth storey, **c** Third storey, **d** Second storey, **e** First (Ground) storey

4.2 Base Shear (V_b)

Base shear time histories for the three types of frames with no soft storey are presented in Figs. 6, 7, 8, 9, 10, and 11. It is evident that for all the cases the base shear is lesser with FDBS in comparison to the conventional frame and in some cases, it is lesser than the rigid frame system. Moreover, FDBS slightly reduces the shake duration of the frame.
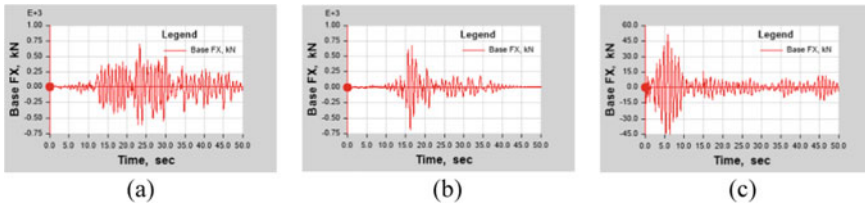

Fig. 6 Base Shear time histories of the steel frame with the central rigid frame system, **a** GMR—1, **b** GMR—2, **c** GMR—3
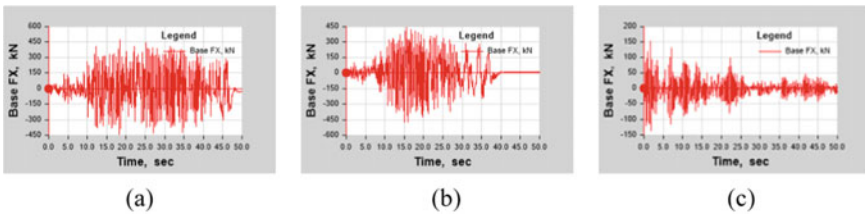
Fig. 7 Base Shear time histories of the steel frame with the central FDBS, **a** GMR—1, **b** GMR—2, **c** GMR—3
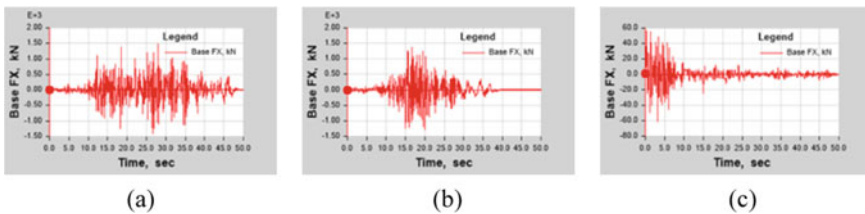

Fig. 8 Base Shear time histories of the steel frame with the central CBS, **a** GMR—1, **b** GMR—2, **c** GMR—3
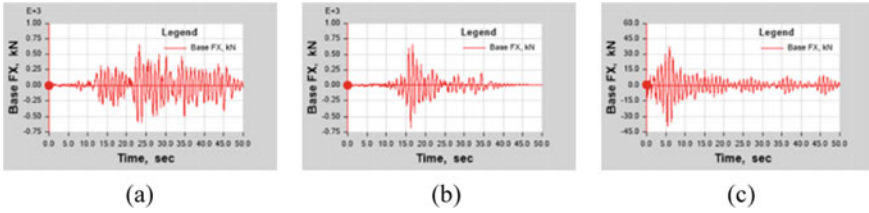
Fig. 9 Base Shear time histories of the steel frame with corner rigid frame system, **a** GMR—1, **b** GMR—2, **c** GMR—3
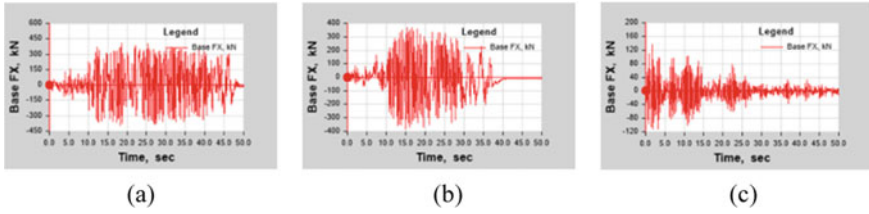

Fig. 10 Base Shear time histories of the steel frame with corner FD brace system, **a** GMR—1, **b** GMR—2, **c** GMR—3
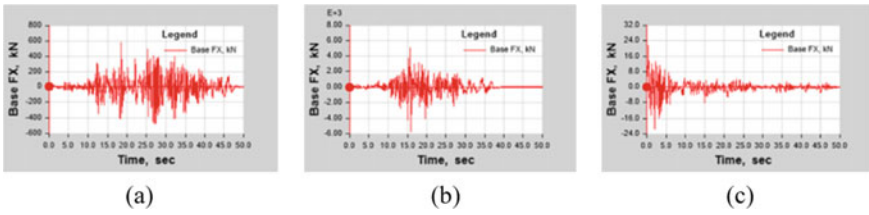

Fig. 11 Base Shear time histories of the steel frame with the corner brace system, **a** GMR—1, **b** GMR—2, **c** GMR—3
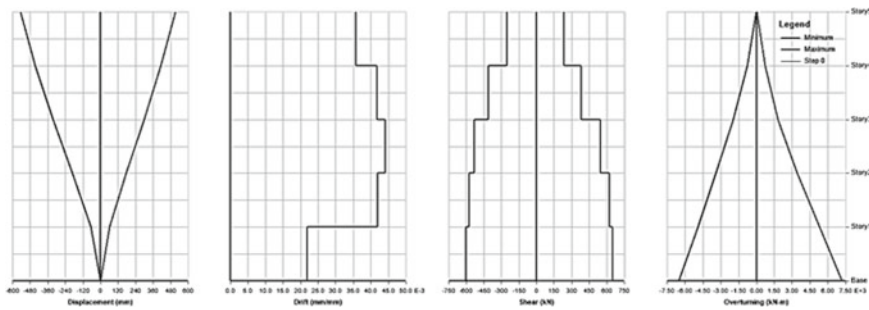

Fig. 12 Envelope of the storey responses of the steel frame with the rigid frame system located at the corner bay of the frame

4.3 Storey Response

Figure 12 shows the storey responses for the moment-resisting rigid frame system located at one corner bay of the frame. Displacement mode shape for both the types of bracing systems are found to be similar but the magnitude of the displacement usually increases significantly if FDBS is used. However, corresponding storey shear decreases. Figures 13 and 14 show storey responses for different soft storey locations in the frames with FDBS and CBS, respectively. As expected, with the upward shift of the soft storey deflection increases and base shear decreases but the trend is not true for the ground storey and the 4th storey.

It is observed that if the bays adjacent to the soft storey are braced using FDBS an optimized combination of the deflection and base hear can be achieved. Figures 15 and 16 present the storey responses for such an arrangement where both base shear and horizontal deflection of the top storey (5th storey) for FDBS are lesser than the corresponding values for CBS. The horizontal deflection at the top storey, i.e., cumulative elastic maximum drift at the top of the building is 65 mm which is well within the prescribed limit of 2.5 per cent of the height. Corresponding base shear is ~457.34 kN which is less than the corresponding base hear value for the CBS system (~777.83 kN).

4.4 Pseudo Spectral Acceleration (PSA)

Pseudo spectral acceleration (PSA) noted at the top floor for different locations of the soft storey in the FDBS are presented in Fig. 17. It shows that the presence of the soft storey increases the value of PSA at the top floor. As the location of the soft storey moves upward, there is a slight reduction in the spectral quantity.

5 Conclusions

The prime objective of the current study is to identify the suitable scheme and the corresponding parameters to quantify the influence of the FDBS in a steel building frame. It is well known that for the same ground motion force in a particular structure is dependent on its fundamental natural period of vibration (T_f), which in turn is a signature of its flexibility. Hence, in order to capture the actual frame response depending on the structure-specific T_f, period-specific conditional mean spectra is used in this study, i.e., eventually, each of the structures is analyzed against a unique input ground motion tailor-made for that specific frame.

It is observed that the frames with corner bracing system are flexible in comparison to the frames with the central bracing system. However, as expected, the deference is comparatively less in the case of FDBS. Another important observation is
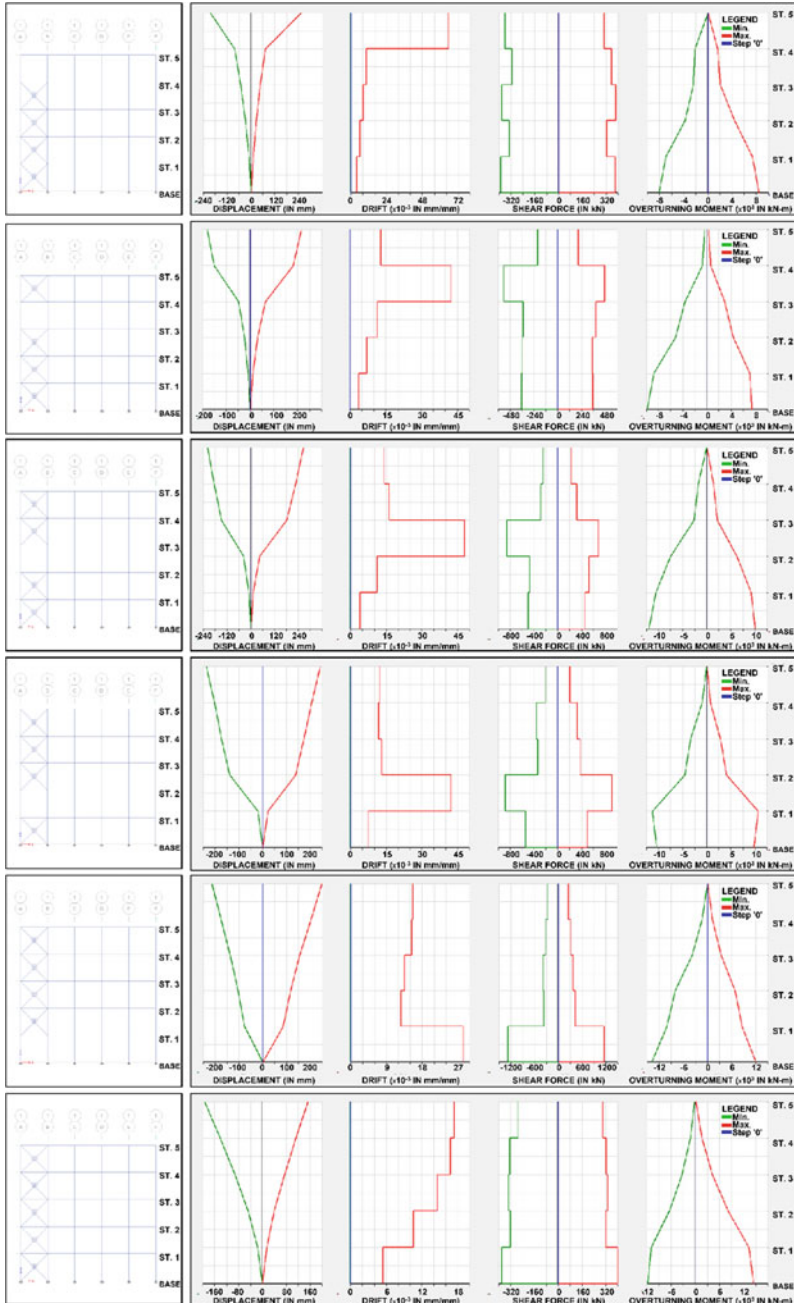
Fig. 13 Envelope of the storey responses of the steel frame with FDBS located at the corner bay of the frame for different soft storey locations
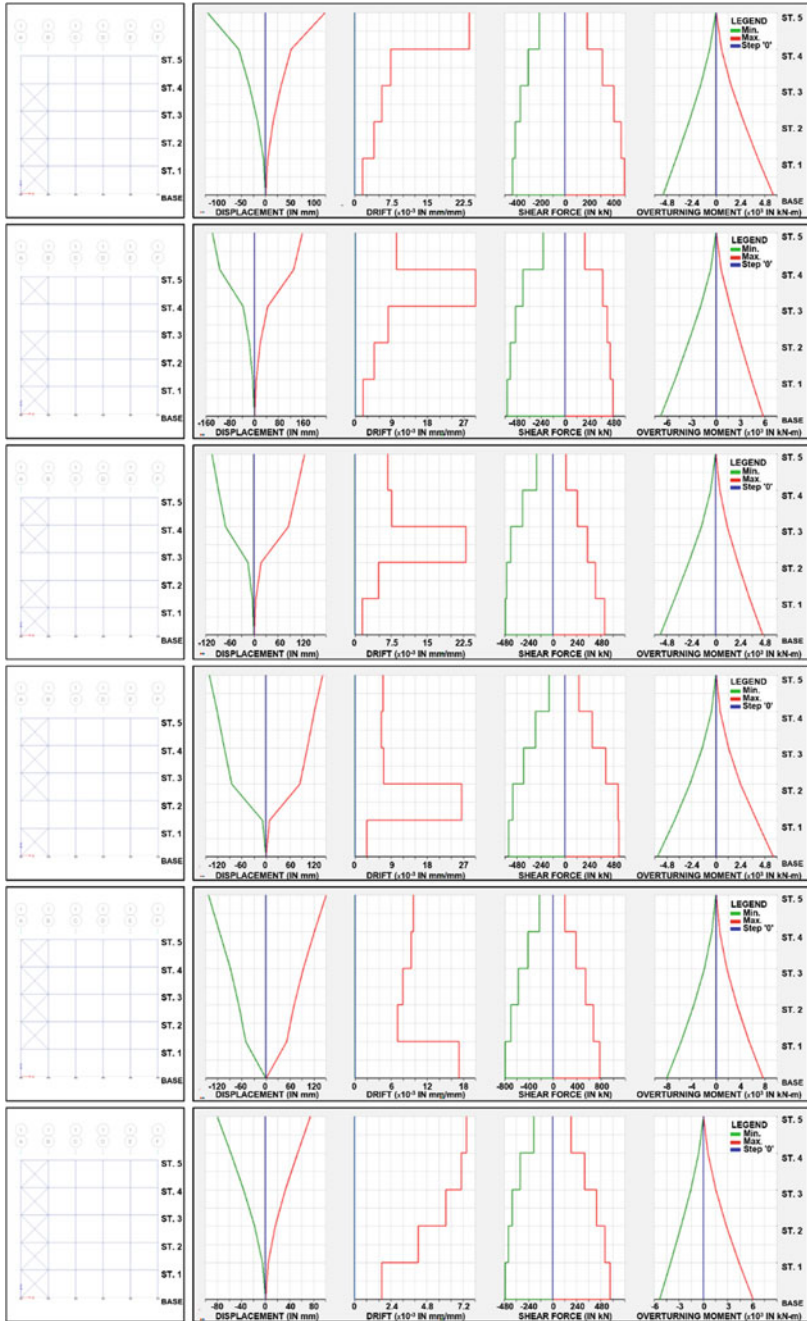
Fig. 14 Envelope of the storey responses of the steel frame with CBS located at the corner bay of the frame for different soft storey locations
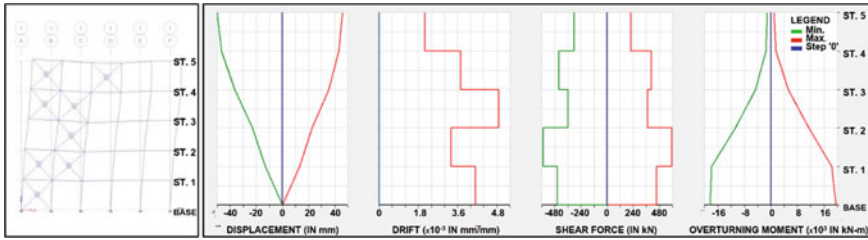
Fig. 15 Storey response of the frame with FDBS and a soft storey at the 3rd level
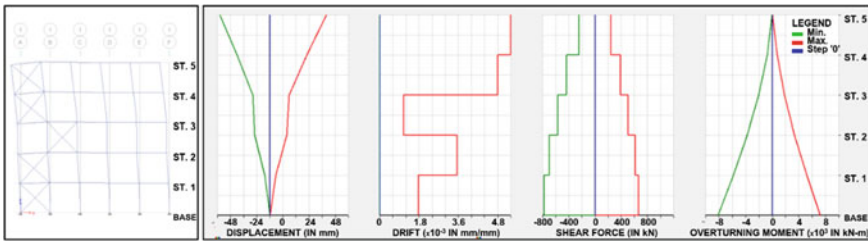

Fig. 16 Storey response of the frame with CBS and a soft storey at the 3rd level

the arrangement of the FDBS has very minimum influence on the variation of T_f values.

Usage of FDBS reduces the base shear for all the cases in comparison to CBS. Usually, as the flexibility of the frame increases, base shear reduces. On the other hand, as the location of the soft storey elevates, base shear usually increases but for a rigid base frame with a soft storey at the 2nd level also increases the base shear significantly. The trend is true for both FDBS and CBS. Additionally, FDBS slightly reduces the shake duration also. The study concludes that it is possible to optimize the base shear value and the corresponding elastic cumulative maximum top storey drift using FDBS.

PSA values are found to be more if the soft storey is introduced in the structure. The important inference is that the spectral location of the amplification has a weak but direct relation with the T_f value of the structure. For the frames with a soft storey at the 4th or 5th storey, reduction in the spectral quantity may be attributed to the reduction in the overburden load, not to any structural attribute.

5.1 Contribution, Limitation, and Further Scope

Summarizing the inferences drawn in this article it is possible to say that FDBS can emerge as a reliable solution to handle the dynamic response of the framed structure. The major take-aways from the current study are as follows.
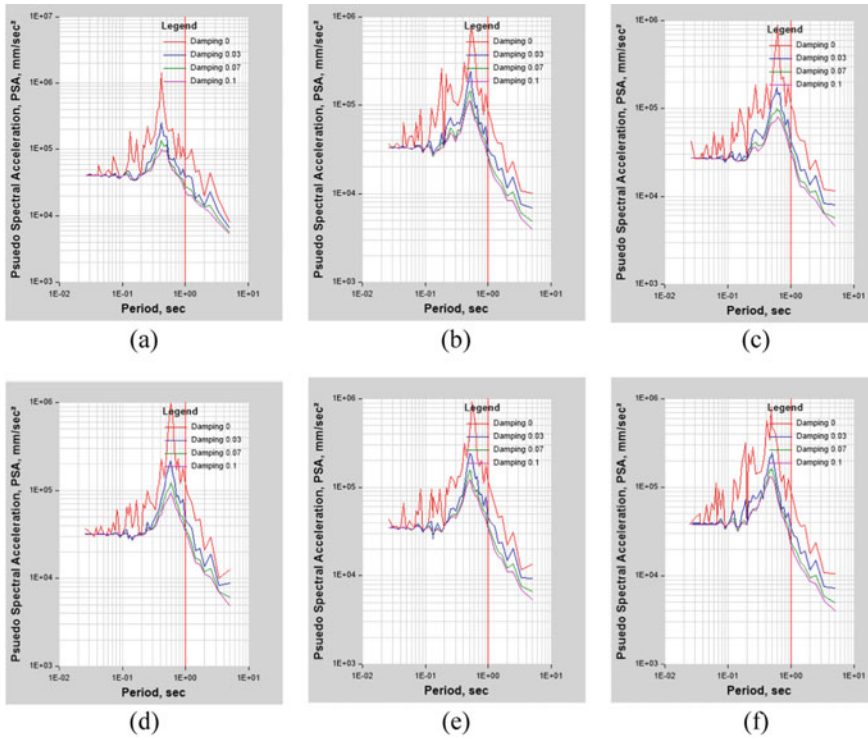
Fig. 17 Pseudo Spectral Acceleration (PSA) at the 5th storey (St.) of the steel frame with the corner FDBS, **a** No soft storey, **b** soft storey at St. 1, **c** soft storey at St. 2, **d** soft storey at St. 3, **e** soft storey at St. 4, **f** soft storey at St. 5

Contribution: The article reports the first-ever quantitative study of steel structural frame with FDBS against T_f specific acceleration time history. It tries to identify the possible parameters, necessary to formulate the basic member arrangement scheme, which may be important for a process plant structure where bracing arrangement may be random to meet the process requirements. Once identified, the exercise is helpful for structural maintenance also. The learnings can as well be applied for the regular frames. Nevertheless, the study confirms that if possible, the regular arrangement of bracing is beneficial.

Limitations: Two-dimensional model of the study is its major limitation. A 3D model will be able to capture the real behaviour of the structure including the influence of the torsional modes. Additionally, the current study doesn't report the velocity response.

Further scope: The preliminary results obtained in this study opens the scope for further detail study of the frames with regular as well as irregular configurations in the temporal and spectral domains. Storey wise transfer functions for displacement, velocity, and acceleration need to be studied for 3D frames with various configurations.

References

1. Housner GW, Bergman LA, Caughey TK, Chassiakos AG, Claus RO, Masri SF, Skelton RE, Soong TT, Spencer BF, Yao JTP (1997) Structural control: past, present, and future. J Eng Mech 123(9):897–971
2. Pall AS (1979) Limited slip bolted joints: a device to control the seismic response of large panel structures. Doctoral dissertation, Concordia University
3. Filiatrault A, Cherry S (1987) Performance evaluation of friction damped braced steel frames under simulated earthquake loads. Earthq Spectra 3(1):57–78
4. Filiatrault A, Cherry S (1990) Seismic design spectra for friction-damped structures. J Struct Eng 116(5):1334–1355
5. Cherry S, Filiatrault A (1993) Seismic response control of buildings using friction dampers. Earthq Spectra 9(3):447–466
6. Aiken IA, Kelly JM (1990) Earthquake simulator testing and analytical studies of two energy-absorbing systems for multistory buildings. Report No.: CB/EERC, 90(03)
7. Nims DK, Richter PJ, Bachman RE (1993) The use of the energy dissipating restraint for seismic hazard mitigation. Earthq Spectra 9(3):467–490
8. Fitzgerald TF, Anagnos T, Goodson M, Zsutty T (1989) Slotted bolted connections in aseismic design for concentrically braced connections. Earthq Spectra 5(2):383–391
9. Grigorian CE, Yang TS, Popov EP (1993) Slotted bolted connection energy dissipators. Earthq Spectra 9(3):491–504
10. Mirtaheri M, Zandi AP, Samadi SS, Samani HR (2011) Numerical and experimental study of hysteretic behavior of cylindrical friction dampers. Eng Struct 33(12):3647–3656
11. https://www.quaketek.com/
12. Scholl RE (1993) Design criteria for yielding and friction energy dissipators. In: Proceedings of ATC 17-1 on seismic isolation, energy dissipation and active control, vol. 2, pp 485–495
13. Moreschi LM (2000) Seismic design of energy dissipation systems for optimal structural performance. Doctoral dissertation, Virginia Tech
14. Ciampi V, De Angelis M, Paolacci F (1995) Design of yielding or friction-based dissipative bracings for seismic protection of buildings. Eng Struct 17(5):381–391
15. Moreschi LM, Singh MP (2003) Design of yielding metallic and friction dampers for optimal seismic performance. Earthq Eng Struct Dyn 32(8):1291–1311
16. Christopoulos C, Filiatrault A (2006) Principles of passive supplemental damping and seismic isolation, 1st edn. IUSS Press, Pavia, Italy
17. NEHRP Recommended Seismic Provisions for New Buildings and Other Structures (FEMA P-750) (2009) Report Prepared for the Federal Emergency Management Agency (FEMA), Building Seismic Safety Council (BSSC), Washington D. C., USA
18. FEMA (2000) Prestandard and commentary for the seismic rehabilitation of buildings (FEMA-356) Federal Emergency Management Agency, American Society of Civil Engineers (ASCE), Reston, USA
19. ETABS®, Computers & Structures, Inc., 1646 N. California Blvd., Suite 600 Walnut Creek, CA 94596 USA. Retrieved from https://www.csiamerica.com/products/etabs)
20. National Building Code of Canada (2015) NRCC, Ottawa, Canada (2015)
21. http://www.palldynamics.com/
22. Baker JW (2011) Conditional mean spectrum: tool for ground motion selection. J Struct Eng 137(3):322–331

23. Naumoski N, Saatcioglu M, Amiri-Hormozaki K (2004) Effects of scaling of earthquake excitations on the dynamic response of reinforced concrete frame buildings. In: Proceedings of 13th World conference on earthquake engineering, Vancouver BC, Canada, Paper No. 2917
24. The MathWorks Inc., Natick, A. H. Campus, 1 Apple Hill Drive, Natick, MA 01760-2098
25. Ram SRKMM, Bagchi A (2015) Seismic performance evaluation of steel moment resisting frames. In: 11th Canadian conference on earthquake engineering
26. PEER Ground Motion Database (Web source: https://ngawest2.berkeley.edu/).