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EFFECT OF FRICTION DAMPERS ON SEISMIC PERFORMANCE OF BUILDINGS

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Supervisor
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This is to certify that we have read this thesis and that in our opinion it is fully adequate, in scope of quality as a thesis for the degree of Master of Science.

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Ahmet Hilmi DERİNGÖL

ABSTRACT

EFFECT OF FRICTION DAMPERS ON SEISMIC PERFORMANCE OF BUILDINGS

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Friction damping devices dissipate energy by using the mechanism of solid friction developed at the sliding surface, which is a relatively inexpensive and effective method for stable energy dissipation. It is essential to maintain a consistent and predictable frictional response throughout the life of these devices. In this study, numerical analyses were performed in order to investigate effectiveness of friction dampers (FDs) on the response of a structure under seismic loading. For this, 4 and 8 storey steel buildings with moment-resisting frames were considered. The buildings have the same plan and three bays on each direction. The existing frames were designed according to two different cases. They were designated as flexible moment-resisting frames and rigid moment-resisting frames. The effect of distributing the FDs over the height of the frames on the seismic performance of the framed structures was studied. The structures were modeled using a finite element program and evaluated by both nonlinear static and time history analyses. Capacity curve, interstorey drift index, interstorey index, global damage index, roof displacement, base shear, and hysteretic curves were computed for each frame system. The results indicated a considerable improvement in the earthquake performance of the frames with the FDs.

Keywords: Friction damper; Hysteresis; Moment resisting frame; Nonlinear analysis; Seismic protection.

ÖZ

SÜRTÜNME SÖNÜMLEYİCİLERİN YAPILARIN SİSMİK PERFORMANSLARI ÜZERİNE ETKİSİ

DERİNGÖL, Ahmet Hilmi
İnşaat Mühendisliği, Yüksek Lisans Tezi
Danışman: Yrd. Doç. Dr. Esra METE GÜNEYİSİ
Ocak 2013, 74 sayfa

Enerji sönümlemesinde nispeten daha ucuz ve etkili olan sürtünme sönümleme cihazları, katı sürtünme mekanizması kullanarak kayma yüzeyinde oluşan sürtünmeleri sönümleyerek etkili olmaktadır. Bu cihazların sürtünme tepkilerini servis ömürleri boyunca tutarlı ve öngörülebilir bir şekilde sürdürmeleri gerekmektedir. Bu çalışmada, deprem yüklemesine maruz kalmış bir yapıdaki sürtünme sönümleyicilerin etkilerini araştırmak için sayısal analizler yapılmıştır. Bunun için moment aktaran çerçevelerden oluşan 4 ve 8 katlı çelik yapılar incelenmiştir. Binalar aynı plana ve her yönde 3 eşit açıklığa sahiptir. Mevcut çerçeveler iki farklı duruma göre tasarlanmıştır. Esnek moment aktaran çerçeveler ve rijit moment aktaran çerçeveler olarak adlandırılmışlardır. Sürtünme sönümleyiciler çerçevelerin yüksekliği boyunca yerleştirilerek, yapıların sismik performansı üzerindeki etkileri incelenmiştir. Yapılar sonlu elemanlar programı kullanılarak doğrusal olmayan statik ve dinamik analizler ile irdelenmiştir. Herbir çerçeve sistemi için kapasite eğrisi, göreli kat öteleme indeksi, genel hasar indeksi, taban kesme kuvveti ve histeretik eğriler hesaplanmıştır. Sonuçlar, sürtünme sönümleyici bulunduran çerçevelerin deprem performanslarının önemli düzeyde iyileştiğini göstermiştir.

Keywords: Sürtünme sönümleyici, Histeretik, Moment aktaran çerçeve, Doğrusal olmayan analiz; Sismik koruma.

To My Parents

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LIST OF SYMBOLS / ABBREVIATIONS

ADAS	Added damping and added stiffness
BFs	Bare frames
D	Roof displacement
EBFs	Eccentric braced frames
EDR	The energy dissipating restraint
FDs	Friction dampers
FEMA	Federal emergency management agency
FPS	Friction Pendulum system
H	Total height of the building
Kf	Effective stiffness
MDOF	Multi degree of freedom
MRFs	Moment resisting frames
NEHRP	National earthquake hazards reduction program
PEDS	Passive energy dissipation systems
PFD	Pall friction damper
SBC	Slotted-bolted connections
TADAS	Triangular added damping and added stiffness
UBC	Uniform building code
VE	Viscoelastic dampers
VF	Viscous fluid

CHAPTER 1

INTRODUCTION

1.1.General

Earthquakes are known as the most catastrophic disaster and occur due to sudden seismic excitation. They may lead to casualties, premises and also damage to vital systems such as water communication, power and transportation etc. They not only devastate villages, towns and cities but also lead to horrify the economic and social structure of the nation just as happened on 11 March 2011 in Fukishima nuclear plant. The impact of the situation may be getting more and more terrific as long as releasing elastic energy could not absorbe during ground motion. The damages caused by seismic events have forced engineers and researchers to find out cost-efficient solutions.

In this regard, Povov et al. (1979) improved the eccentric-braced frames and design the diagonal bracing element to cause yielding of the beams during severe the ground motion. However, the designed system has a disadvantage of having to replace the damaged beams, a costly job. Hanson et al. (1986) studied the effects of using viscoelastic dampers on moment resisting frames (MRFs) as additive dissipation devices. They reduced the response of the structures was substantially and improved the seismic performance of the systems. Before the application of this system a critical damping ratio from one to five percent is normally deemed, and then it is possible to obtain damping of more than 5 % of critical damping ratio by means of using additive dampers.

Friction pendulum system (FPS) developed by an engineering firm in California that consists of two steel plates and each one has a concave surface and sliding is allowed on the contact surface. The FPS look likes a modified system for a base isolation technique but using dry friction between two surfaces instead of reinforced rubber pads. It was experimentally performed and found to be efficient in reducing the lateral load effect (Zayas et al., 1987).

After all, Pall et al. (1982) bringing up the idea of removing the excess energy fed into building frames during ground motions via FDs devices. These simple devices have been performed and proved to have a reliable hysteretic loop over many cycles. These devices dissipate energy by at slipping a pre-determined load level. The new damper type has been aroused interest by engineering community. The FDs are made of simple steel elements with heavy duty brake lining pads attached to the slipping surfaces. In order to obtain the design slip load of the FDs, tightening force is provided by a bolt tightened against the braking pads.

Among the common systems for resisting lateral loads in multistorey structures are the MRFs, frames with shear walls, and braced frames. The FDs is a modification of the braced MRFs in that the cross-bracing members are equipped with FDs. The locations of the dampers can be designed according to suitable any architectural requirements.

Baktash et al. (1987) has shown, by computer simulation, the superior performance of the FDs when compared with eccentric braced frames (EBFs). Filiatrault et al. (1982) have carried out testing of a 1/3 scale steel frame with the devices installed at the cross-bracings of the frame. They have described the superior performance of FDs during a severe ground motions when other type of structural systems failed. Pall et al. (1987) has recently compared the FDs was remarkable superior to that of the shear wall system. Kelly and Aiken (1988) have recently performed a nine story steel frame furnished with FDs and they obtained similar results. Filiatrault and Cherry (1988) have also recently compared the seismic performance of FDs and that of a base isolation system, and they attained that the FDs exhibited better than the base isolation systems for earthquakes of various characteristics.

The current provisions of NBCC 2010 do not provide guidelines regarding earthquake resistant structures equipped with FDs devices nor suggestions referring at inserting damping into a structural system in order to reduce the seismic demand. Although FEMA 356 (2000) contains information regarding friction devices and some design recommendations such as all energy dissipation devices shall be capable of sustaining displacements equal to 130% of the maximum calculated displacement in the device when subjected to ground motions defined for 2% in 50 years probability of exceedance, it does not provide the complete design provisions.

1.2.Objective and scope

The main purpose of this study is to compare the seismic performance of different type of moment resisting frame (MRF) buildings, namely flexible and rigid MRF buildings and those frames equipped with friction dampers (FDs). The nonlinear behavior of the structures was inspected. Furthermore, the effect of FDs on the response and performance of the steel structures subjected to earthquake loads were studied and discussed. In all cases, the buildings have the same plan, which consists of three equal bays on each direction. The buildings were deemed to have a uniform mass distribution over their height and a non-uniform lateral stiffness distribution. Santa-Ana and Miranda (2000) first designed the structures which were used as original structures in this study. The 4 and 8 storey structures designated as rigid and flexible frames. Later, FDs were placed into each frame system. Thus, a total of 8 different cases was taken into consideration. Then, series of nonlinear static and dynamic analyses were carried out to specify the seismic performance of aforementioned frames.

1.3.Outline of the Thesis

The major objective of this thesis is to provide a description through nonlinear static and dynamic analysis of the different frame systems with and without FDs, and estimate their effectiveness.

Chapter 1-Introduction: Aim and objectives of the thesis are introduced.

Chapter 2-Literature Review and Background: A literature survey was briefly given. The background on passive energy dissipation devices and more specifically on historical and current studies of FDs, also some practical application of the FDs for the buildings as passive energy dissipation devices was given in this chapter

Chapter 3-Methodology: In this chapter, analytical model of bare frames and those with FDs are explained. The performed analysis cases were elaborately described.

Also, it emphasized the significant parameter in the analytical model.

Chapter 4-Results and Discussions: This chapter presents and compares the results obtained from nonlinear static and dynamic analysis of each frame system in terms of capacity curves, interstorey drift index, global damage index, and etc.

Chapter 5-Conclusion: General conclusions are given in the light of findings from the overall analysis results.

Appendix A: Deflected shapes: The deformed shape of each frame system was illustrated.

CHAPTER 2

LITERATURE REVIEW

2.1 Passive Energy Dissipation Systems

Today conventional structure design criteria is not sufficient to deal with severe earthquakes. However, the serviceability of some buildings must be continued after the earthquakes especially such as airports, hospitals etc. Hence, the structures must be designed with adequate strength to mitigate the inelastic deformations, but that may be expensive or not suitable from architectural point. Since a few decades surprising improvements have been made against preventing the earthquake damages. One of the most simple and reliable method of these improvements is the use of "Passive Energy Dissipation Systems" (PEDS) which are steadily becoming more important as modern structures become rich from architectural point and so they become progressively more flexible (Soong and Dargush, 1997). PEDS are a general approach for reducing the adverse effect of earthquakes by means of mechanical devices which are interconnected with the frame of the structure with various position and dissipate energy through the whole structure. As shown in Figure 2.1, energy is dissipated either yielding of steel, slipping on the friction surface, viscoelastic movement in polymeric materials, movement of a piston with a viscous fluid, or orificing of fluid (Soong and Constantinou, 1994). These dissipation devices were tested in many fields such as buildings, motor vehicles, spacecraft industries.

In this chapter, the principle and application of some passive energy dissipation devices such as viscoelastic dampers, viscous dampers, metallic dampers and tuned dampers were superficially explained. The former and current study of FDs were explained elaborately.

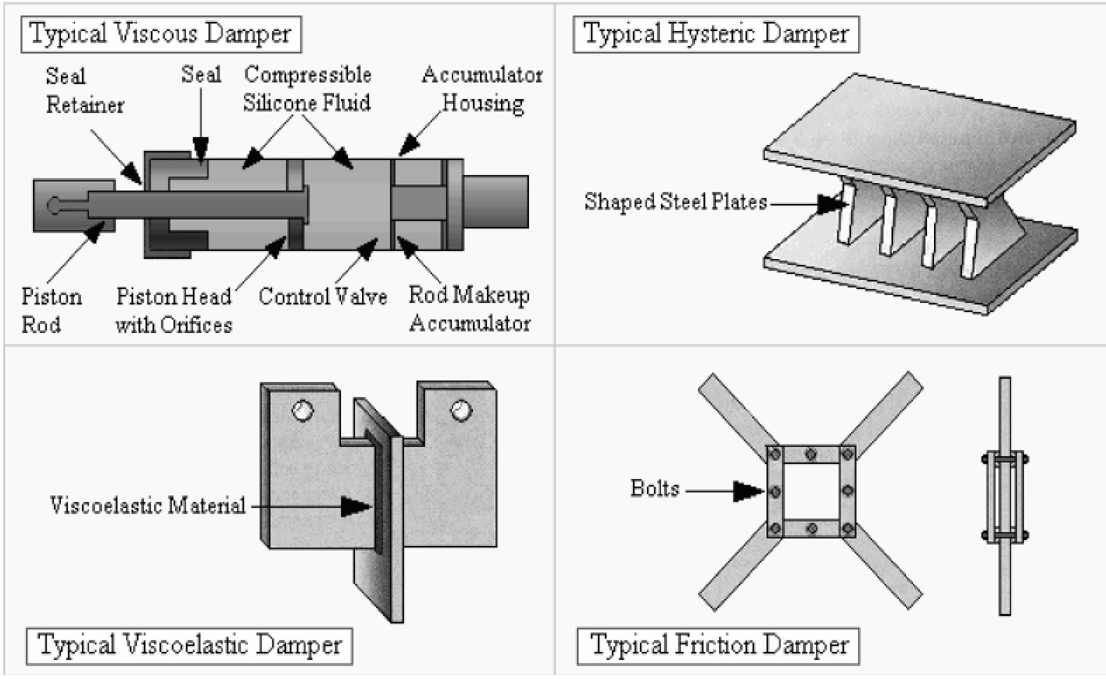


Figure 2.1 Type of passive energy dissipation devices (Soong et al., 1997)

2.1.1 Viscoelastic Dampers

Viscoelastic dampers (VE) are constructed of acrylic polymers sandwiched between, and bonded to steel plates, and placed in braced frames as shown in Figure 2.2. They have been especially developed for controlling the wind vibration in high rise buildings. Examples are the World Trade Center in New York City (110 stories), the Columbia Seafirst Building in Seattle (73 stories) and the Number Two Union Square Building in Seattle (60 stories). The effectiveness of VE dampers for increasing the earthquake resistance of structures has been experimentally studied by many researchers. Several shake table tests of large-scale steel frames and reinforced concrete frames with added VE dampers have been carried out by Ashour et al. (1987); Fujita et al. (1991); Aiken et al. (1990); Chang et al. (1995); Foutch et al. (1993); Min et al. (2004). The test result of Aiken showed that inter-storey drift reductions in comparison to those of the MRFs which were slightly better than those of the friction (Sumitomo damper) damped structure. The ratio of interstorey drift in the viscoelastically damped structure to the inter-story drift in the MRFs differ from 0.5 to 0.9. Base shear forces in the viscoelastically damped structure were about the same as in MRFs (Constantinou et al., 1993).

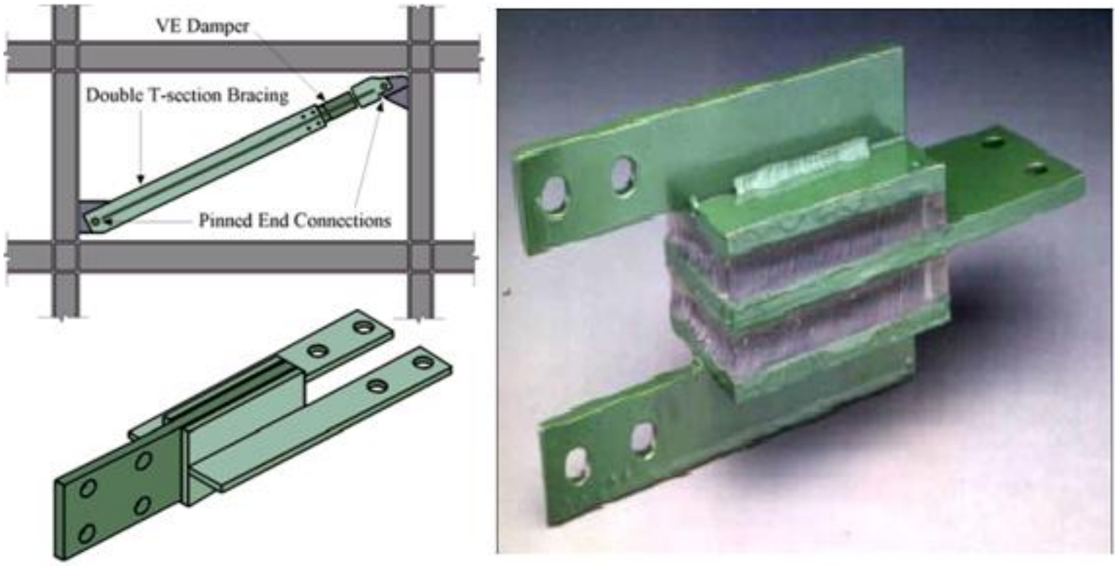


Figure 2.2 Type of passive viscoelastic damper (Changet al., 1995)

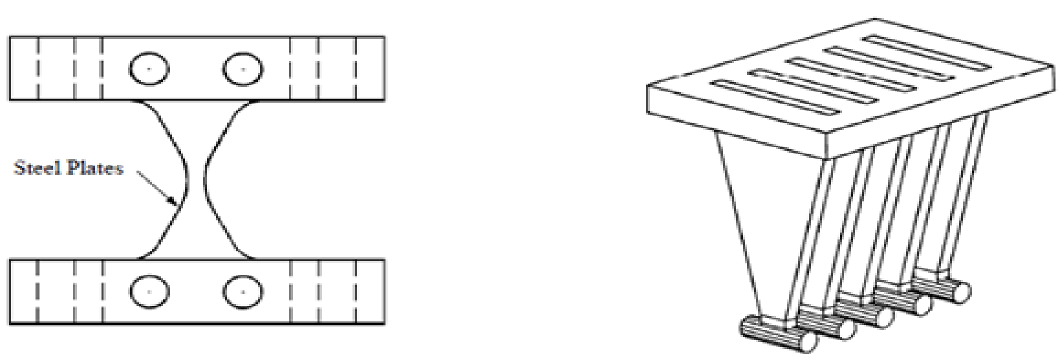
2.1.2 Viscous Dampers

These devices rely on the operating principle of dissipating energy by viscous heating because of the friction between fluid particles and device components. These devices firstly used in aerospace, automotive industries and defense systems against to shock and vibration. In recent years, research and development in this fields triggered to perform new progress on civil engineering buildings. With using these devices researcher carried out viscous fluid (VF) dampers for seismic applications to civil engineering structures. The result of analyses and experiment showed that viscous dampers increase the seismic capacity of structures and also reduce the displacements Soong et al. (1999). Constantinou et al. (1993) used various viscous materials to obtain optimal stiffness and damping properties. Terenzi (1999) revealed different type of viscous dampers those have linear or nonlinear viscous manner and does not change with high temperature. Viscous dampers are efficient due to their operating principle is that structures are not directly subjected to maximum force, so it provides the safety in each member of structure (Constantinou et al., 1993a; Reinhorn et al., 1995).

2.1.3 Metallic Dampers

Metallic dampers dissipate energy with hysteretic behavior of mild steels when going into their inelastic range. A broad sort of devices have been designed and performed that dissipate energy in flexural, shear, or extensional deformation modes. The most attractive behavior of these devices are their stable hysteretic low-cycle and long service life and insensible to temperature change like friction damper. On the one hand, these devices are relatively inexpensive and their behavior will remain stable among the life of structures; on the other hand these devices have limited cycles and nonlinear response. Added damping and added stiffness (ADAS) devices were shown in Figure 2.3 and consist of X-shaped mild steel. Triangular added damping and added stiffness (TADAS) is also show in Figure 2.3 and unlike the ADAS it has triangular steel plates (Niwa et al., 1995).

Phocas and Pocanschi (2003) performed the ADAS device as a retrofit technique. Xia and Hanson (1992) studied the use of the ADAS device in steel MRFs and test result show that X-shaped plate dampers are the most favourable metallic damper type and consist of many x-shaped steel plates. Bergman et al. (1987) and Whittaker et al. (1991) were modified application of X-shaped dampers with metallic dampers. During the design of metallic devices, Xia and Hanson (1992) accepted the ratios of bracing stiffness to device stiffness, brace-device assemblage stiffness to device stiffness, and assemblage stiffness to that of the corresponding story as most important parameters.



(a) (b)

Figure 2.3 Typical metallic devices: a) ADAS device and b) TADAS device (Niwa et al., 1995)

2.1.4 Friction Dampers

Using of FDs in civil engineering structures go back to 1970s, indeed, friction mechanism has been used effectively to regulate the motion of objects for centuries. Friction dampers make use of the mechanism of solid friction that evolves between two solid bodies sliding relative to each other to enable to obtaining the desired energy dissipation. Soong et al. (2002) have been carried out several types of FDs for the purpose of improving seismic response of structure. The device slips at a predetermined load level in order to dissipate energy by friction especially during severe earthquake motions. Many various type of FDs have been studied in structures and also they are commercially available and has been manufactured. X-braced friction damper slotted bolted connection, Sumitomo FDs, energy dissipating restraint and Tekton friction devices. These devices become different with their mechanical property and used materials for the sliding parts from each other. FDs do not affected with thermal fluctuations and also have perfect performance and constant hysteretic behavior under severe seismic excitation (Filiatrault et al., 1987).

FDs are basically considered as listed here:

1. Slotted-bolted connections,
2. Sumitomo passive energy dissipation devices,
3. Piezoelectric FDs,
4. The energy dissipating restraint (EDR),
5. Pall FDs

These dampers are discussed in the next section.

2.1.4.1 Slotted-Bolted Connections

Slotted-bolted Connections (SBC) are one of the basic type of FDs and developed bolted connections in order to dissipate energy by friction surface that is consisted of steel gusset and cover plate, two successive channel and steel bolt with washer. It wasper formed to enable slippage of the connections to prevent any failure such as buckling or yielding members of structure (Fitzgerald et al., 1989).

Grigorian et al. (1992) also tested a characteristic SBC, which dissipates energy by friction mechanism of two steel splice plates tightened against a steel gusset plate by the movement of high strength A325 bolts, as shown in Figure 2.4. In this study, the clamped plates which were made of clean mill-scale A36 steel were performed at the University of California at Berkeley using loading frame. According to test results and comparing with the research of Pall et al. (1979) obtained hysteresis curves have showed not stable cycles due to the changes on the friction coefficient and abrasive wear effects unlike the others. However, they proposed to add 1/8 inches (3.175mm) shims between the gusset and the clamped plates, whereas shims were made of half-hard cartridge brass as described in Figure 2.4.

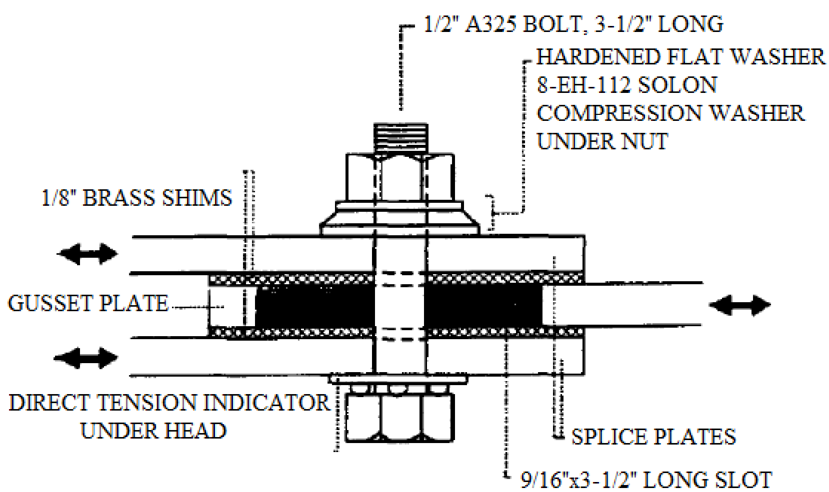
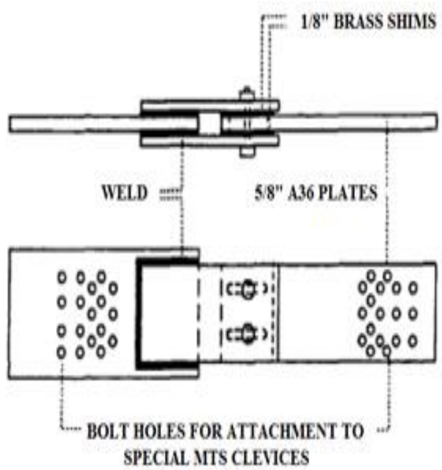
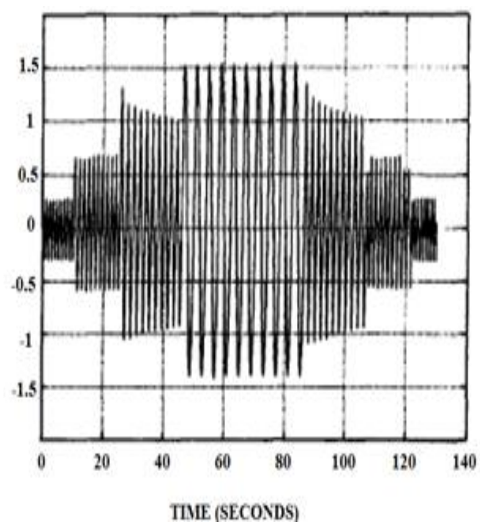


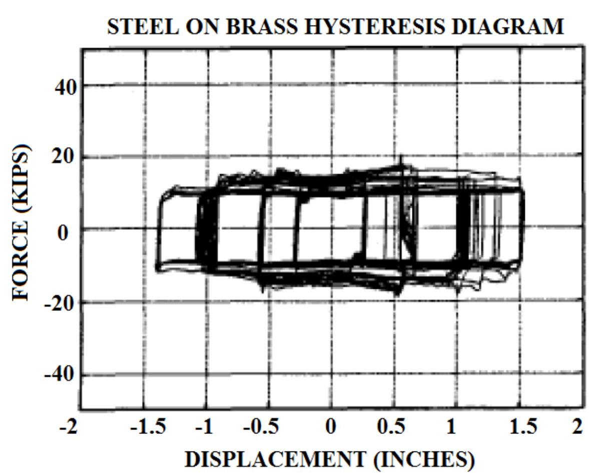
Figure 2.4 Detail of a typical SBC (Grigorian et al., 1992)

The energy dissipated by friction between mill-scale steel and brass surfaces has showed a more stable hysteretic loop than 20 the former one, which was maintained throughout the entire duration of quasi-static displacement loading and is showed in Figure 2.5(c). In spite of the shape of hysteresis cycles correspond to rectangular as per Coulomb friction law, it can be observed some fluctuations. The reason of that roots in the variation of friction coefficient.





(a) (b)



(c)

Figure 2.5 The response of SBC: a) specimen tested b) displacement loading protocol, and c) hysteretic behavior (Grigorian et al., 1992)

Fitzgerald et al. (1989) introduced a study on the SBC. In that study, SBC were inserted one end of the concentrically braced frames as shown Figure 2.6 and a sinusoidal function test was carried out for SBC. Test results indicated that SBC exhibited appropriate and continuous behavior on reducing the effect of ground motions.

Unlike from previous researchers, Grigorian and Popov (1993) also performed different type SBC with the slipping interface composed of brass and steel that exhibited well-adjusted frictional characteristics as shown Figure 2.6. An experimental test of a three-storey steel building equipped with SBC was set up by Grigorian and Popov (1993) who proved the effectiveness of the SBC in dissipating the seismic input energy.

Constatinou et al. (1991) also made use of a frictional surface which consisted of graphite and bronze as shown Figure 2.6. The SBC that was carried out to improve the seismic energy dissipation on bridges.

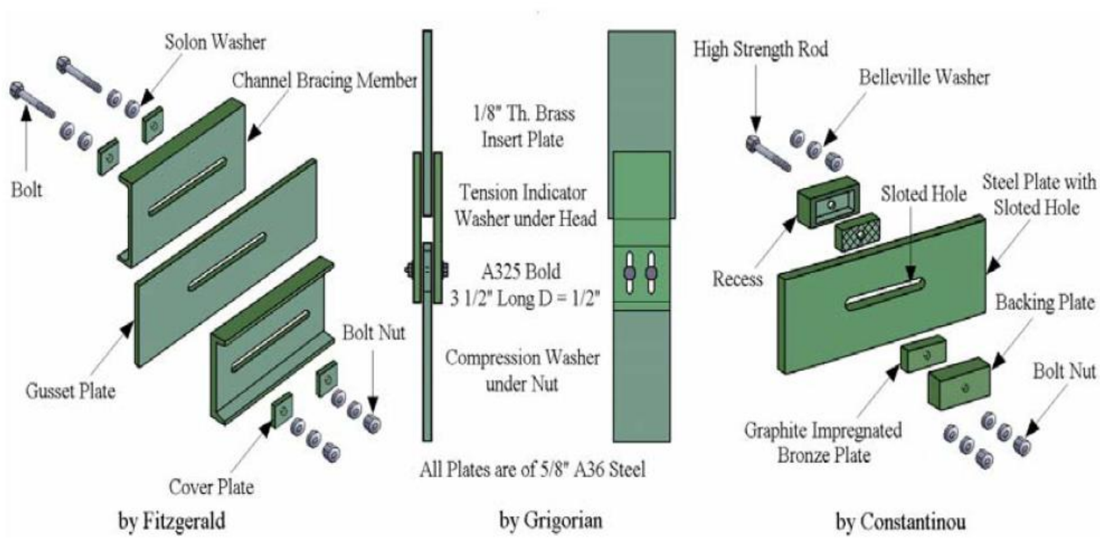
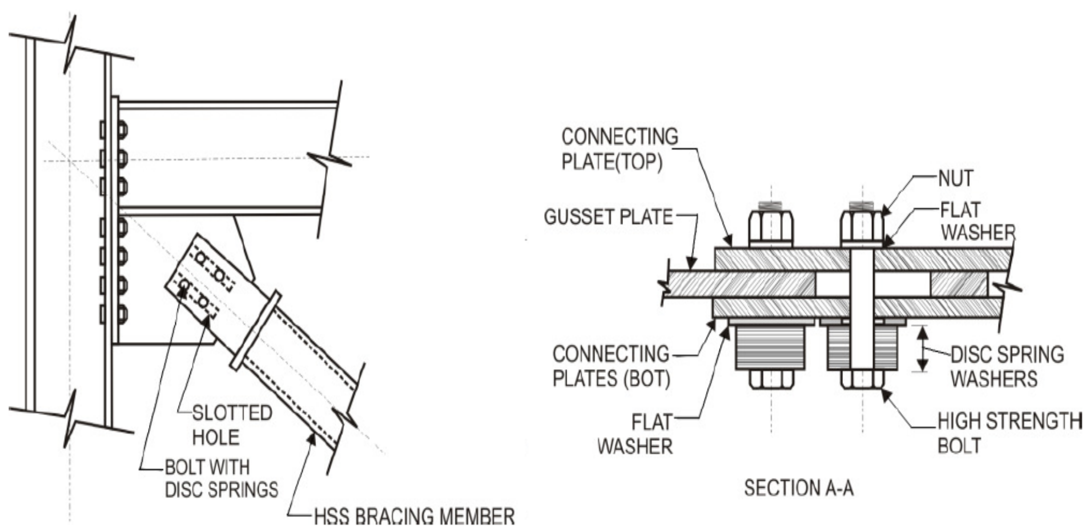


Figure 2.6 Types of slotted bolted connection (Based on Fitzgerald et al., 1989; Grigorian et al., 1993; and Constatinou et al., 1991)

Popov et al. (1995) carried out a experimental test in order to verify the efficiency of SBC in energy dissipation and testing the hysteresis shape. A three storey building was modeled and SBC was inserted end of the both two braces and then performed on a shake table test under the Llolleo earthquake record. The large amount of input energy was absorbed by SBC rather than by the virtue of the inherent damping of the system or inelastic deformation of structural members. Moreover, the obtained hysteretic curves clear up that the ground motion has ended before the presumed slip amplitude was reached.

Tremblay (1993) carried out an extensive experimental program for determining the behavior of concentrically braced frame in seismic zones. He also investigated the effectiveness of SBC with executing many dynamic tests with motion records. It was observed that the mechanism of energy dissipation occur in the SBC by means of the relative movement with the equipped elements. Hence, it was indicated that addition of SBC into structural might be an efficient and inexpensive method because the connections comprise steel plates with a surface clamped against determined filler plates by tensioned bolts as shown in Figure 2.7.



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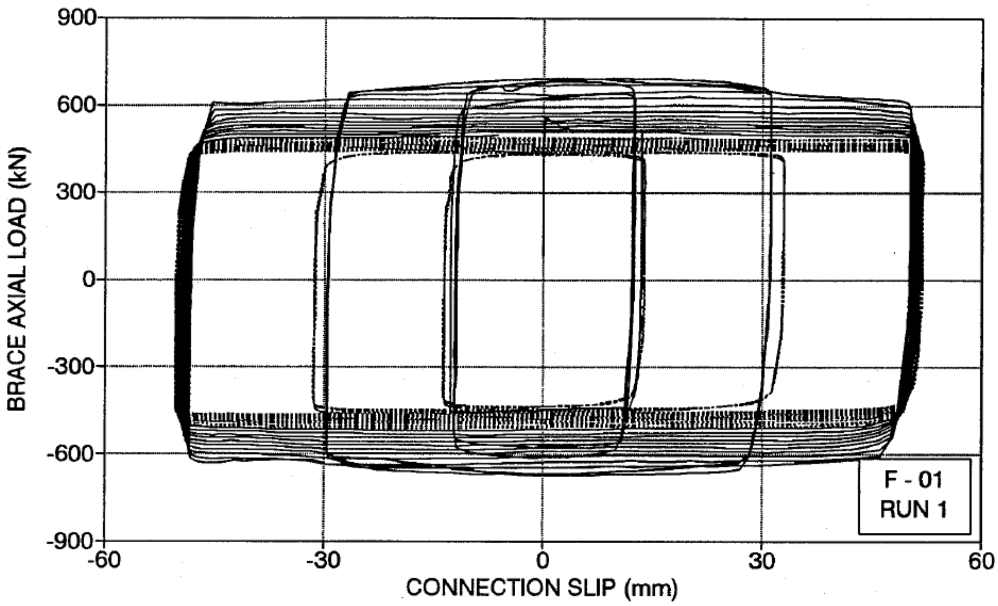
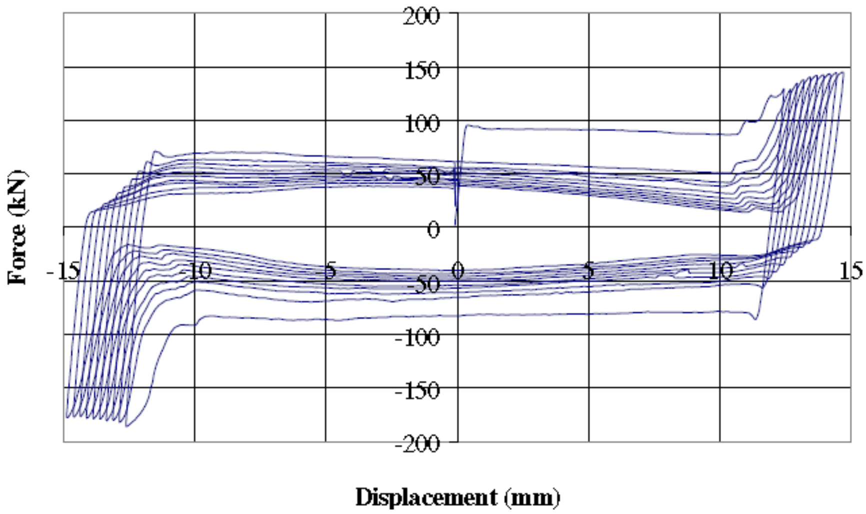


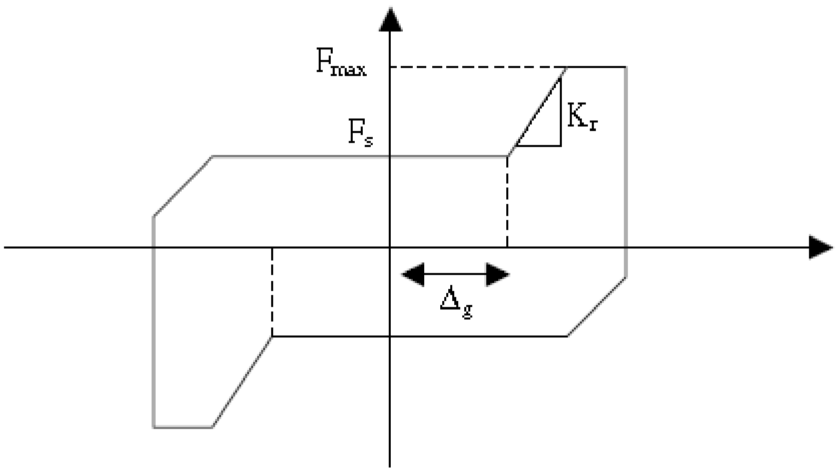
(c)

Figure 2.7 CBF with SBC: a) test set-up; b) section of the SBC, and c) hysteretic behavior under displacement controlled cyclic test (Tremblay, 1993)

Lukkunaprasit et al. (2004) considered the new approach to investigate the behavior of SBC in the event of exceeding the slip travel presumed slot length and also nonlinear dynamic analyses on a designed building equipped with SBC was tested to examine the seismic response of the SBC. The test results indicated that the bolt impact covers a nonlinear supplemental stage added to the rectangular hysteretic characteristic since the reducing of post-tension force in case of the clamping bolts exhibited in bearing. Consequently, as exceeding the slip travel presumed slot length the hysteresis curve was reduced as shown in Figure 2.8. In order to verify this influence, Lukkunaprasit et al. (2004) tested their model with using restrainers and select to employ the concept implemented in SBC devices by Roik et al. (1988).



(a)



(b)

Figure 2.8 Hysteretic behavior of SBC with and without restrainers: a) hysteresis cycles under cyclic loads considering the effect of the bolt impact and b) force-slip relationship of SBC (Lukkunaprasit et al., 2004)

Hence, in the refined model, the bearing force can be obtained when the existing slip distance was reached and on the movement of restrainers after a predetermined force threshold is recorded. This hysteresis model is shown in Figure 2.8 (b) where Δ_g is the slip distance, F_s is the slip load, and F_{max} is the restraining force limited at a threshold value. The maximum force F_{max} is defined in lined with the brace buckling capacity, rather than the capacity of the high strength bolts used in the SBC or the bearing force of adjacent plates. Thus, the restraining stiffness of the device, K_f is equal to the axial stiffness of the attached brace. However, the magnitude of the restraining force might be checked in the course of design (Lukkunaprasit et al., 2004).

2.1.4.2 Sumitomo Passive Energy Dissipation Devices

This devices was firstly generated by Sumitomo Metal Industries as a shock absorber in application of railway, but then, Japanese researchers developed Sumitomo as a FDs energy dissipation devices in order to use in civil engineering structures. They were applied to high rise buildings such as the Sonic City Office Building in Omiya City and the Asahi Beer Azumabashi Building in Tokyo. The development of Sumitomo device was carried out with the experimental test as a passive energy dissipator which was carried out at Earthquake Research Center, Berkeley (Aiken et al., 1990; 1993).These dissipation device placed under the beams as a parallel. One part of it joined to floor beam and the other part of it joined to chevron bracing system. The configuration of these dissipation device in the structures is shown in Figure 2.9.

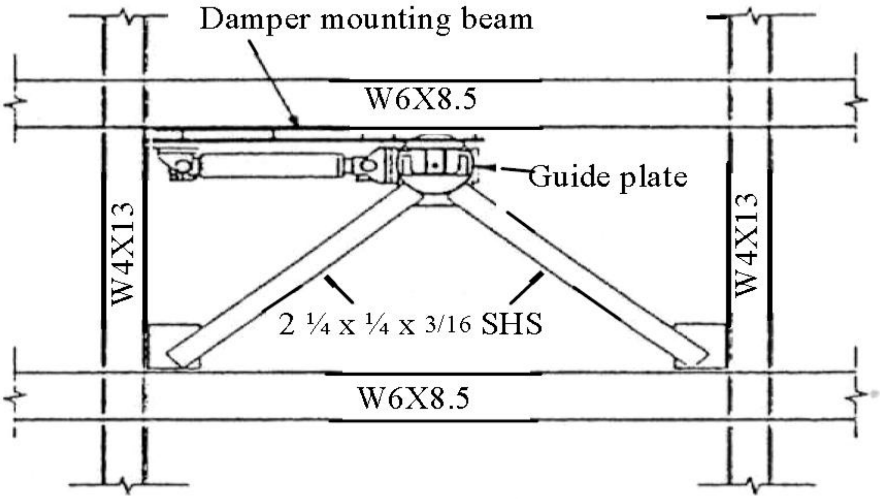


Figure 2.9 Sumitomo friction damper (Aiken et al., 1990)

Their action principle depends on the friction mechanism which occurs between bronze or copper alloy friction pad and between inner part of metal. The motion of friction surface are specified by the position of wedges and conveyed by means of cut springs as shown in Figure 2.10.

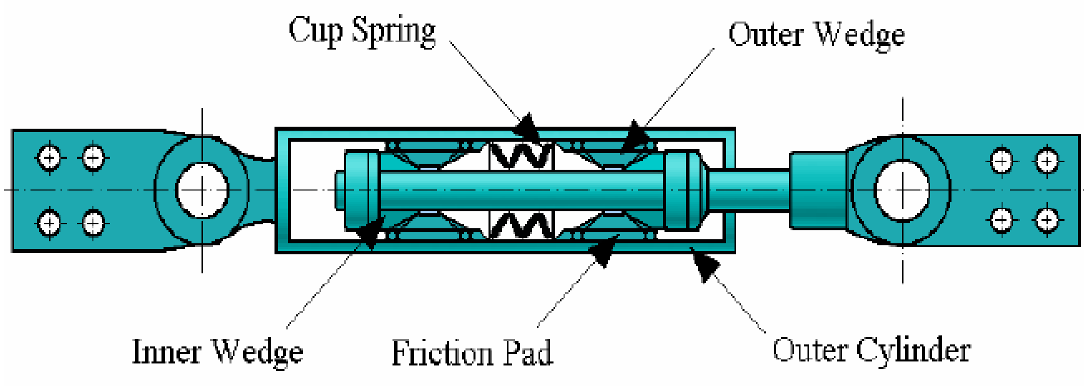


Figure 2.10 Sumitomo friction damper (Based on Sumitomo Metal Industries Ltd. 1992)

One of the first numerical and experimental applications of these dampers performed by Aiken and Kelly (1990). Their model is made of ¼ scale 9-storey steel. It was understood that Sumitomo dampers which expose to seismic excitation exhibited good performance on reducing the effect of seismic excitation and also dissipating a significant amount of input energy. The obtained hysteresis loops from analysis result are similar to other friction dissipator as shown in Figure 2.11.

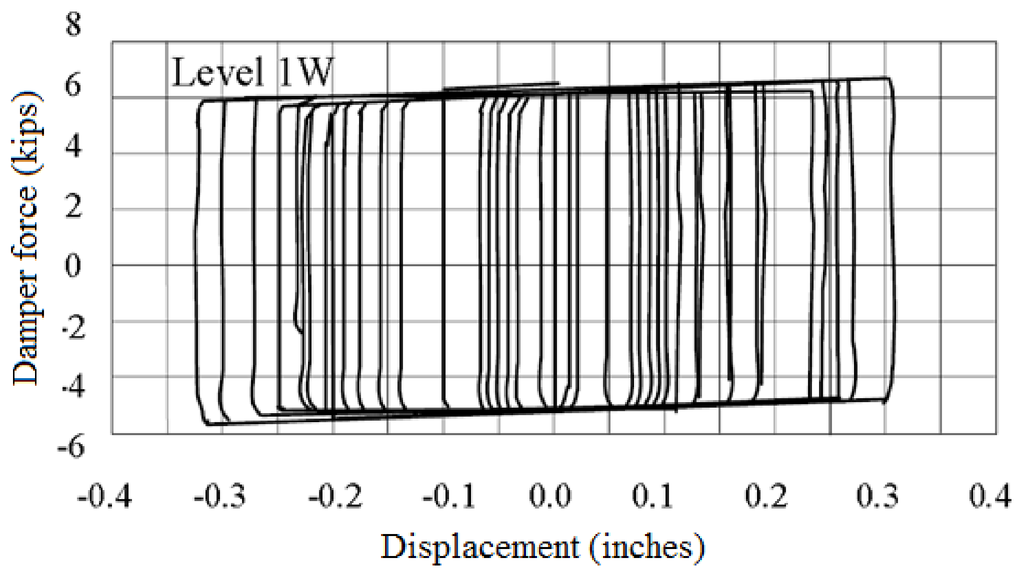


Figure 2.11 Typical hysteretic behavior of Sumitomo friction damper (Aiken et al., 1990)

2.1.4.3 Piezoelectric Friction Dampers

The piezoelectric friction damper made up of different active and passive parts. Figure 2.12 demonstrate the constituents of the piezoelectric friction damper. The shaft is stationary at the bottom part and the mechanical part is joined to the shaft. The active part consists of the outer housing and the air bearing .The exterior elements which are called housing and air bearing are enable the movement of system. The housing is attached to bottom part by means of spring element. When structures meet the seismic excitation, damper start to interconnect the air bearing with surface of friction pads and system dissipate the input energy (Unsal et al., 2002).

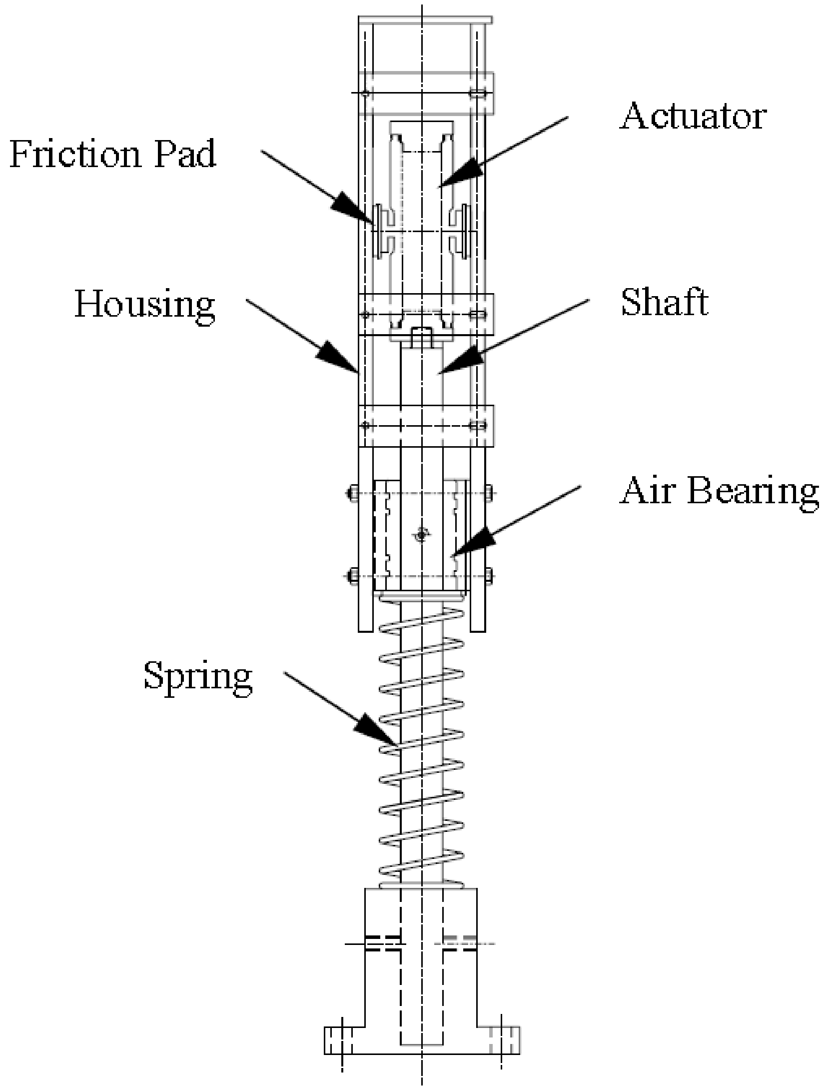


Figure 2.12 Piezoelectric friction damper (Unsal et al., 2002)

2.1.4.4 The Energy Dissipating Restraint (EDR)

The energy dissipating restraint devices (EDR) were manufactured commercially by Fluor Daniel, Inc. and their basic elements are given in Figure 2.13.

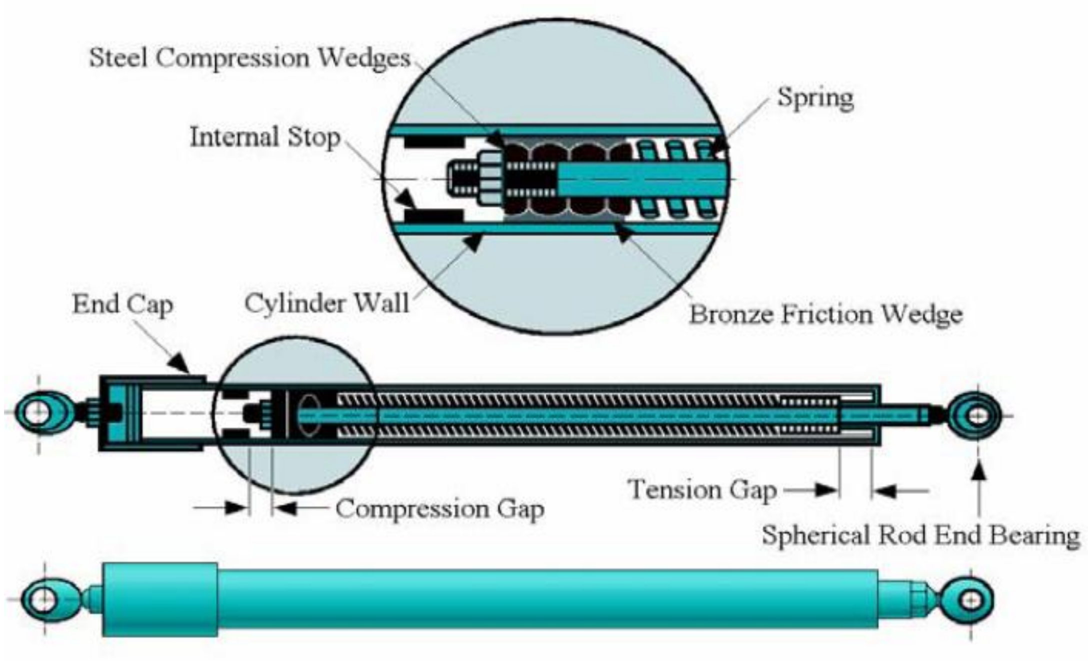
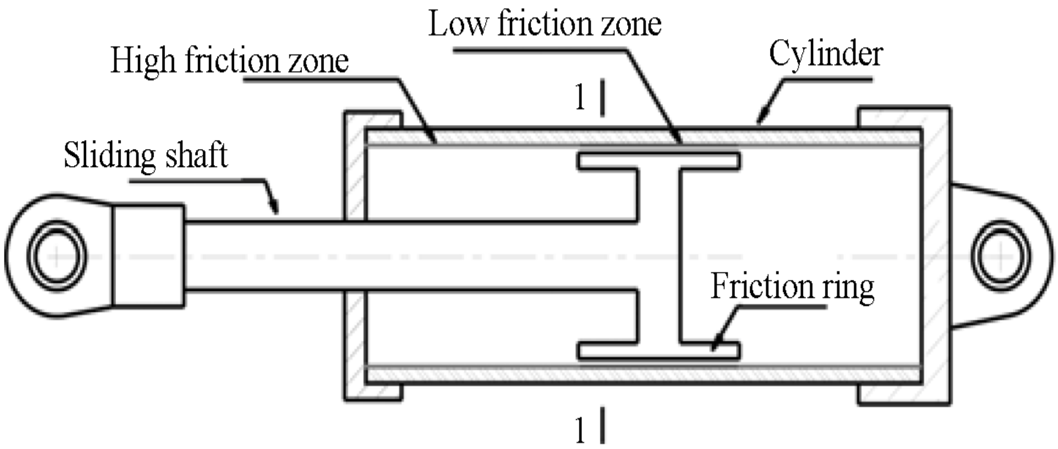


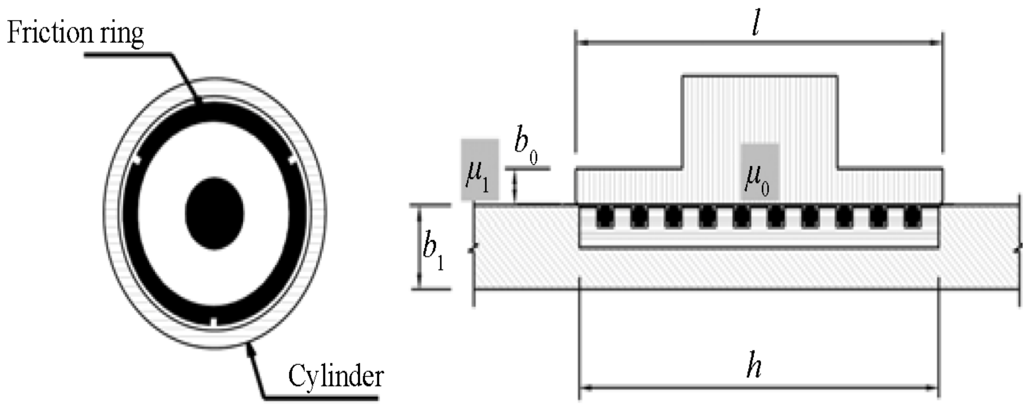
Figure 2.13 Energy dissipating restraint (Flour Daniel, Inc., 1993)

EDR devices were originally designed to prevent the protecting structures of tube system in nuclear plant from ground motion. The EDR device has approximately identical element with the Sumitomo device such as internal spring, wedges, pads in term of components: internal spring, wedges, pads, outer cylinder. Although their mechanism and application may be similar to that in Nims et al. (1993), some features are not same. In this regard, Soong and Dargush (1997) and Zhou and Peng (2009) specified that exerted force through the spring is important parameter for determining the friction force as it turned to a normal force through the cylinder wall within the wedges. However, the elastic behavior of the internal spring and the basic hysteresis cycles are modified the EDR self centering due to the exerting load approached minimum level. Nims et al. (1993) performed the effectiveness of the EDR. Obtained friction force is not sufficient for dealing with formed lateral force during the seismic excitation.

Zhou and Peng (2009) designed an advanced version of EDR device and some component of inner part is replaced with a sliding shaft and a frictional ring as shown in Figure 2.14. Moreover, inner surface of outer cylinder two parts with various friction coefficient were assigned. Unlike from the Sumitomo FDs in this device, the friction force advanced by force of association with the friction ring and internal region of the cylinder.



(a) Configuration



(b) Sectional drawing 1-1 (c) Detail of Friction Surface

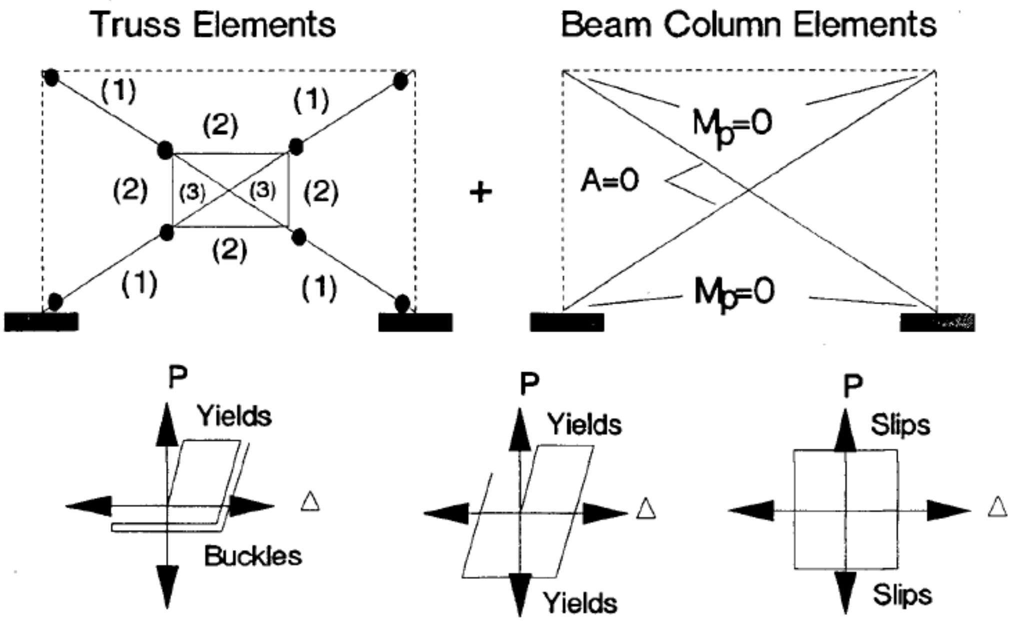
Figure 2.14 A new type of EDR (Zhou and Peng, 2009)

Zhou and Peng (2009) also conducted numerical analyses to determine the performance of EDR. They carried out a 7-story building with this dissipation devices under the simulated earthquake motions and the analyses results indicated that nearly 60% of input energy was dissipated by EDR devices.

2.1.4.5 Pall Friction Damper

The extensively used FDs type is Pall FDs and the main working principle of the passive energy dissipation devices which are mentioned in literature depend on the friction mechanism. In historical background, many researcher and engineer designed mechanical and structural devices with using this mechanism especially in motor vehicle braking system and finally civil engineering buildings. Their purpose was to eliminate the kinetic energy by means of friction mechanism. They also achieved that the researchers broke new ground in their quest to dissipate the input energy at the end of 1970 with emergence of Pall Friction Damper (PFD). From 1970s to present day, PFD has been used extensively to prevent the earthquake destruction effect and these dampers adequate for global success on the seismic design of structures (Pasquin et al., 2002).

PFD may be considered as a developed version of SBC. The basic functioning mechanism is similar to SBC and it refers to the relative sliding within surfaces in contact while the resulted friction force depends on the specific treatment applied to the surfaces in contact and the brake lining pad clamped together by the post-tensioned high strength bolts. PFD made up of steel plates that are bracketed together and treated to enable to slip predetermined design load. Firstly, Pall and Marsh (1980) designed PFD with a basic elastoplastic prototype to exhibit typical friction damper behavior However, Filiatrault et al. (1987) performed that this is merely confirmed if the slip occurrence go on and remain predetermined level. They also carried out a more sophisticated model as shown in Figure 2.15.



(a) Diagonal Braces (b) Links (c) Friction Pads

Figure 2.15 Proposed X-braced type model of PFD (Filiatrault et al., 1987)

The brace of structures are considered that yield under tensional force and buckle in compression loading. However, PFD link elements are assumed to yield under both tensional and compressional loading case. The experiment hysteresis loops of the frame equipped with PFD are shown in Figure 2.16.

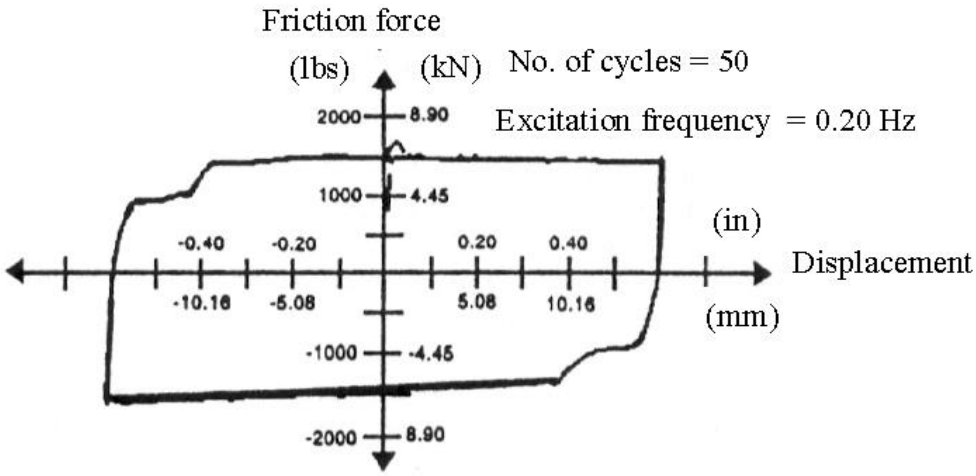


Figure 2.16 Hysteresis loops of PFD (Filiatrault et al., 1987)

PFD has rectangular hysteresis loops and does not change its behavior with temperature or velocity fluctuation. The characteristic damping of BFs is nearly 5%. However, the characteristic damping of frames equipped with PFD can be obtained as 50% and they are generated three type application that is X-braced type, single diagonal type and chevron brace type in order to perform different field cases as shown in Figure 2.17 (Pall and Pall, 2004).

(a) (b)

(c)

Figure 2.17 Type of PFDs a) friction damper in single diagonal brace,
b) friction damper at top of chevron brace, and c) friction damper in X- type
(Pall and Pall, 2004)

The supremacy of PFD to other FDs is that is not active during the minor lateral force such as wind and small seismic excitation. The lateral loads exceed the predetermined slip load level, PFD start to slide and dissipate the input energy. In this regard, this event make the slip load designation the most important parameter in the design of PFD. As shown in Figure 2.18, in the event of the slip load is small, response of PFD also minimum and optimum level. However, PFD does not slide if the slip load excessively small and the amount of dissipated energy will be minimum level (Nishitani et al., 1999). Until the value of slip load reach to ±25% there will not be important change in the dynamic response. In the design of FDs the change of slip load level should not exceed ±15% to provide the design criteria which is implied in ASCE-41/FEMA-356 (ASCE, 2007; FEMA, 2000). They designated the displacement level as 130% under seismic excitation. All bracing and connections are also should resist 130% the slip load of FDs.

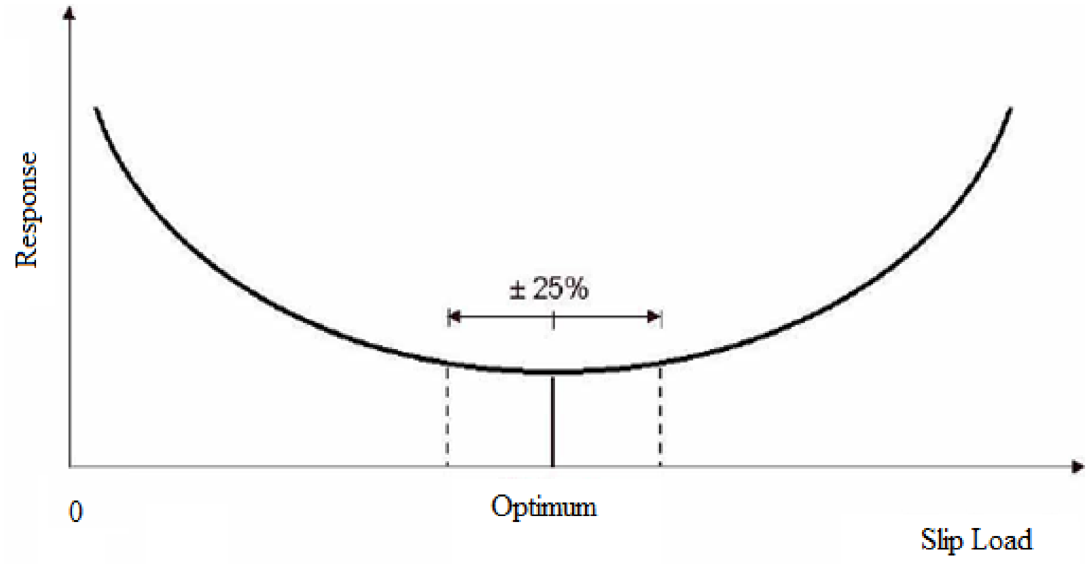


Figure 2.18 Response versus slip load (Pall and Pall, 2004)

Filiatrault et al. (1987) set up a shake table at the University of British Columbia in order to test the performance of frames with FDs under the simulated severe earthquake motions whose peak acceleration designated as 0.9 g. The test results proved that the frames with FDs exhibited good performance and did not cause any failure. However, typical bare frame could not remain in elastic ranges and severely damaged.

Imad et al. (2002) also performed a study on the retrofitting of a single-storey steel frames with FDs in order to improve seismic performance of structures. Similar to the Pall type FDs, they used a new type which is similar to chevron bracing shape as shown in Figure 2.19. This device located below of beams and has a gadget which is also shown in Figure 2.19. The inner part of the FDs was designed with brass or frictional materials. This device joined to frame by means of a hinge which enable to movement of both sides and also dissipate the energy. The bracings which are placed like invert V shape assist to system in case of compression and buckling failure. The both end of bracings are assigned hinge connection to overcome plain bearings.

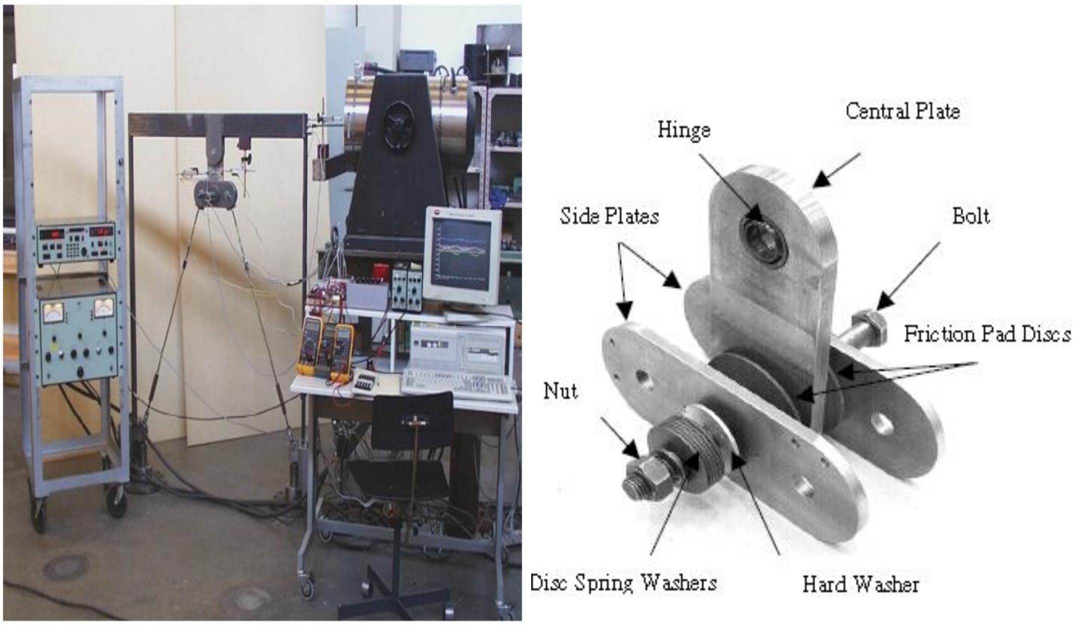


Figure 2.19 The tested new model of FDs in experimental setup (Imad et al., 2002)

Pasquin et al. (2002) carried out a study within ten-storey Eaton’s building which was made of concrete and steel frames retrofitted with types of PFD. FDs are modeled as single diagonal and chevron bracing with using computer program. 3-D nonlinear time-history dynamic analyses performed with using Whittier earthquake of 1987 record in order to specify the seismic response of structure. Result of analysis showed that PFD has a supremacy over braced frame and PFD nearly dissipate the half of total input energy. Maximum drifts are less than 1% that means the structure turned to its original shape without any damage. The numerical analysis results showed that frames with PFD exhibited well performance against to strong ground motion.

Carlos et al. (2003) was aimed to retrofit the Monterey Country Government Center in order to prevent the structural imperfections against severe ground motions. This study was carried out in the light of FEMA 356 criteria. The building has 4 story level and constructed with steel and precast concrete in 1966 and the FDs was placed only ground level and second floor level. According to the result of retrofitting study, story displacement variations were given in Figure 2.20 that shows the reduction of story displacements enormously reach to 45% existing frame and also story shear is reached to 25% of existing frame. Finally, retrofitted structure may withstand to seismic force.

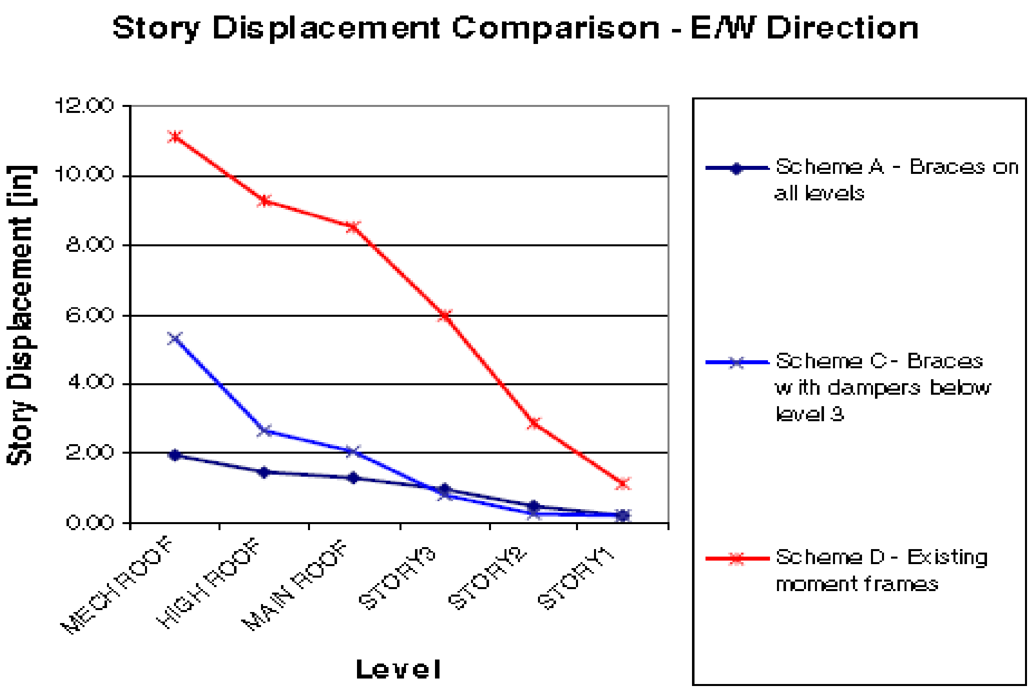


Figure 2.20 Reduction of story displacement (Carlos et al., 2003)

Vassilyet et al. (2004) carried out a seismic design of concrete building whose story level is ten and also two basement levels as shown in Figure 2.21. The buildings are joined at each story level and assigned single diagonal FDs all stories. During the FDs design NEHRP guideline is taken into consideration and FDs are designed for 130% maximum considered earthquake which means a rigorous seismic excitation of probability of 2% in 50 years. The slip load of FDs are assumed 600 kN in order to obtain minimum seismic response and then nonlinear dynamic time history analysis carried out under ground motion with using computer program. The results of analysis showed that after the earthquake FDs turned to its original position without any failure. Hysteretic curve of FDs is just to same as typical FDs shape which is nearly rectangular and important part of total input energy is dissipated with FDs as expected.

Figure 2.21 View of Le Nouvelle Europa (Vassily et al., 2004)

Wu et al. (2005) suggested an improved Pall FDs to diminish the mass-production value and facilitate the design procedure. The developed and original damper were compared in terms of manufacturing cost, operation systems and numbers of connections such as bolts, slip bolts, curve slot etc. It was observed that the damper forces, frictional forces and hysteresis curves were identical for both FDs. Hence, improved FDs exactly duplicated the original damper properties and also have advantage of cost.

Vaseghi et al. (2009) investigated the performance of FDs in 5 and 10-storey steel frames. Both frames have 3 bay layout, 6 m span length and 3 m storey height. A numerical analysis was carried out with using El-Centro (1940), Kobe (1995) and Tabas (1978) earthquake records. By the comparison of analysis results has been understood out that use of FDs reduce the damage effect of earthquake in terms of the base shear, roof displacement and axial column load carrying capacity.

2.1.4.5.1 Application of Pall Friction Dampers

As mentioned previously PFDs are the most impressive, trustworthy and inexpensive dissipation device and do not change its behavior with temperature and velocity fluctuation unlike the other dissipation devices such as VE dampers, viscous dampers etc. After the major ground motion or any lateral force civil engineering buildings do not collapse or not undergo any failure. For that reason many buildings are constructed with PFDs. Hence, there are many applications in new construction and seismic retrofit of existing buildings. Pall and Pall (2004) recorded some of applications those are given below.

Figure 2.22 View of a) Boeing factory and b) Pall friction dampers (Pall and Pall, 2004)

Figure 2.23 a) View of Moscone West convention center, USA and b) Pall friction dampers (Pall and Pall, 2004)

Figure 2.24 a) View of Cafeteria, Auditorium, and fitness building and b) Pall friction dampers (Pall and Pall, 2004)

Figure 2.25 a) View of Ambulatory care center and b) Pall friction dampers (Pall and Pall, 2004)

Figure 2.26 View of a) Million Gallon reservoir water tank and b) Pall friction damper (Pall and Pall, 2004)

Figure 2.27 View of a) Concordia library building and b) Pall friction damper in cross bracing (Pall and Pall, 2004)

Figure 2.28 View of a) Justice headquarters and b) Pall friction damper (Pall and Pall, 2004)

Figure 2.29 View of St. Joseph medical center patient tower with friction damper (Shao et al., 2006)

Figure 2.30 Seismic upgrade with cross-brace friction dampers (Shao et al., 2006)

Figure 2.31 Pall friction damper in cross bracing (Shao et al., 2006)

CHAPTER 3

METHODOLOGY

3.1 Analytical Model of Frames

As frame models, the multi degree of freedom (MDOF) structures were used in this study. These frames were first designed as a typical office building by Santa-Ana and Miranda (2000) and the plan view of the buildings were given in Figure 3.1.

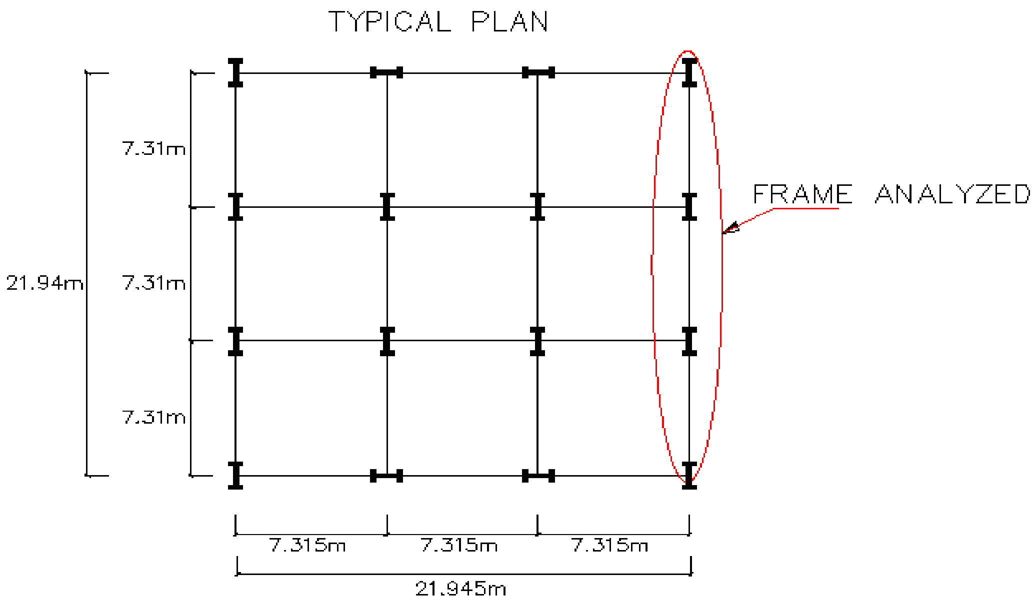
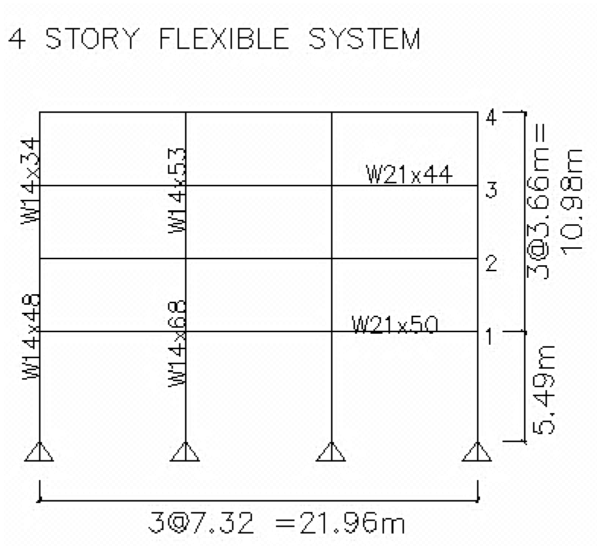


Figure 3.1 Plan view of the multi-story buildings (Santa-Ana and Miranda, 2000)

Four three-bay frame models with two different number of stories (4 and 8) were considered as shown in Figure 3.2 and 3.3. The buildings were assumed to have a uniform mass distribution over the height and non-uniform lateral stiffness distribution. Steel members in the buildings were designed using the lateral load distribution specified in the (UBC, 1994). The member stiffness was tuned to achieve fundamental periods of vibration for each structure representative of those obtained from actual earthquake records. Moreover, with the exception of beam-to-column connections in the top floor, the steel sections of structural members was chosen such that the sum of plastic section modulus of the columns framing into each beam-column joint was greater than the sum of plastic section modulus of the beams framing into the same joint (Santa-Ana and Miranda, 2000).



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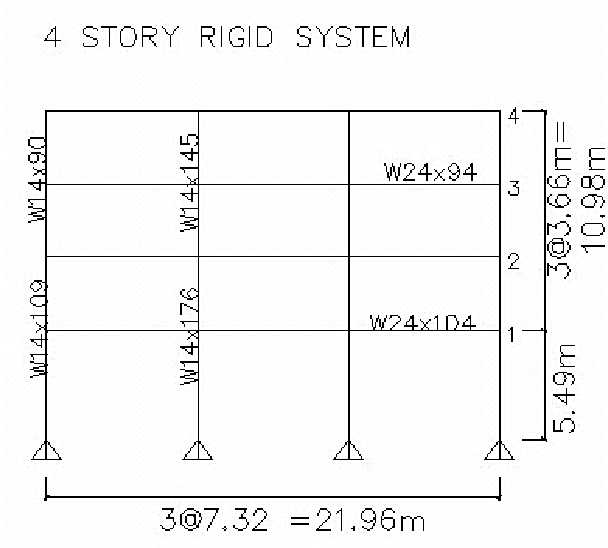

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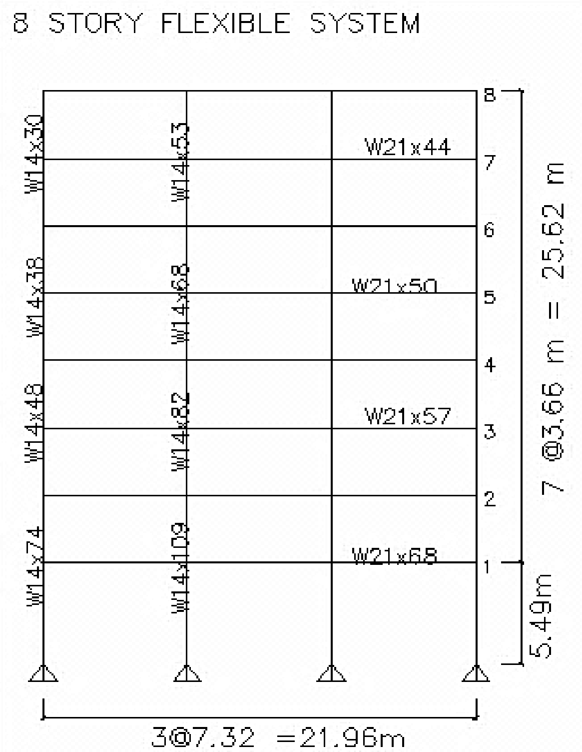

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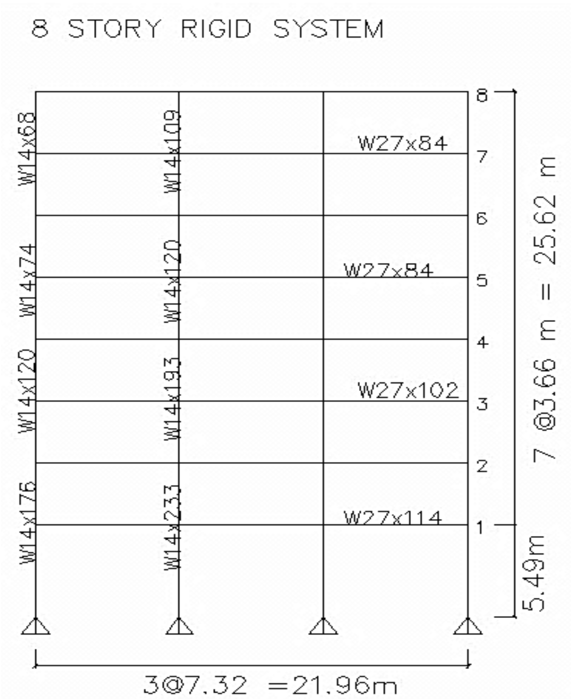


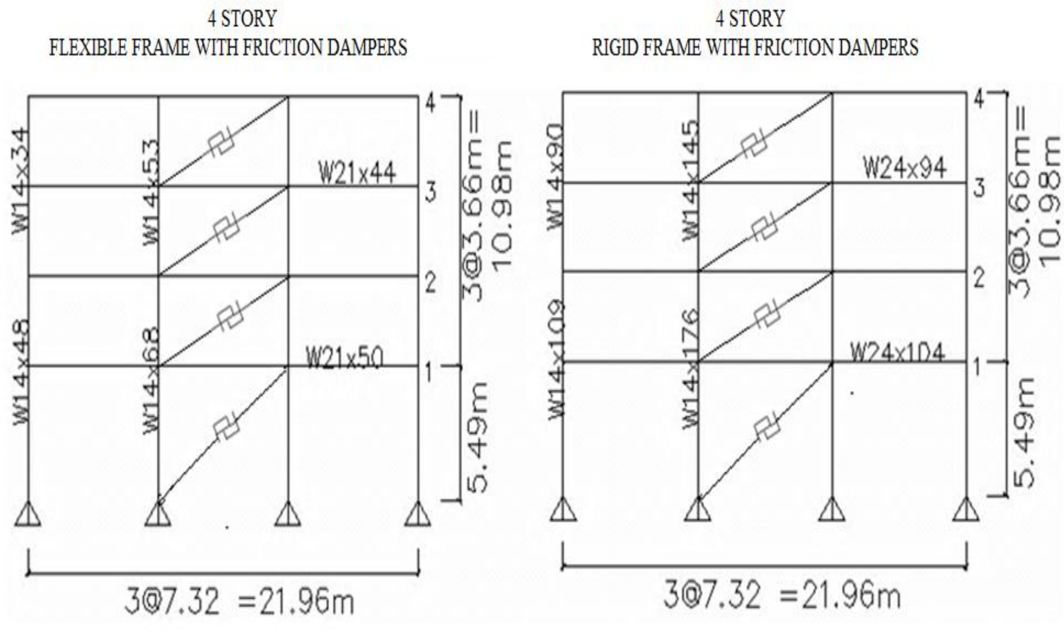
(d)

Figure 3.2 Elevations view of 4 and 8 stories flexible and rigid bare frames (Santa-Ana and Miranda, 2000)

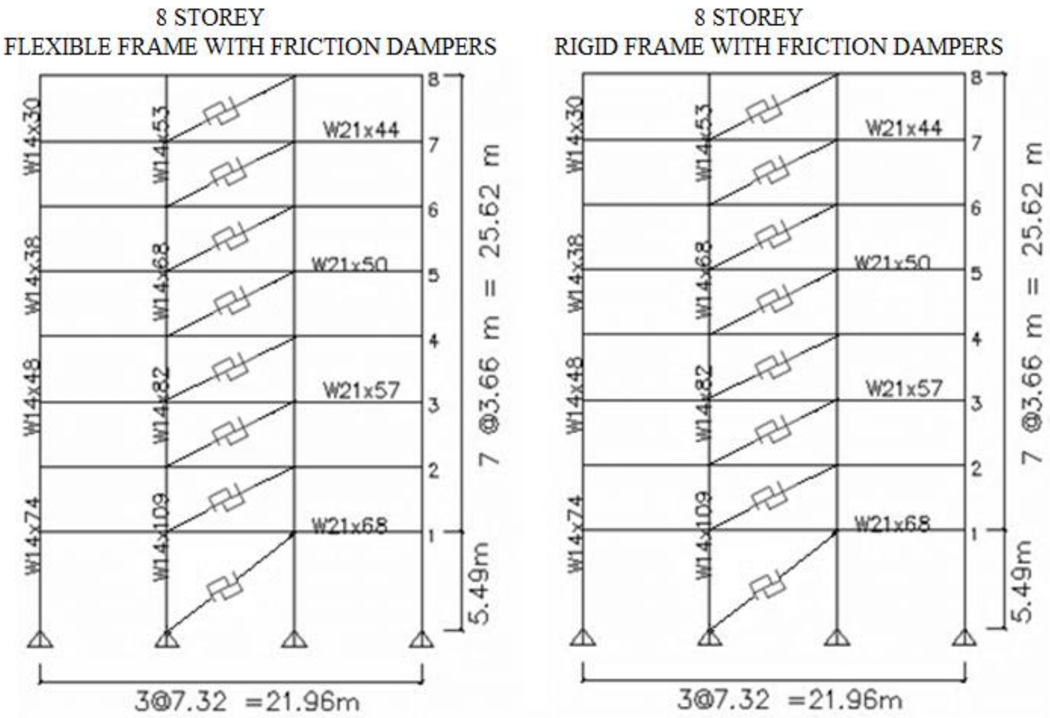
Consequently, two frames (flexible and rigid) with different dynamic properties were considered. It is necessary to note that the flexible frames were designed to supply a realistic lower bound in accordance with lateral stiffness while the rigid frames were adopted with the aim of satisfying upper bound. Furthermore, it was observed that the first two modes captured most of the response of the structure which was about 97%.

The P-delta effect was neglected as it changes over the number of the story and a function of the axial force demand and the interstorey drift. The columns and beams of the frames were designed with different W profiles. For all frame models, the storey height of the frames was 3.66 m for all the floors except in the first floor in which the storey height was 5.49 m.

In order to design the (FDs), Filiatrault and Cherry (1987) approach is taken into account. The system is composed of friction brake lining pads. Figure 3.2 shows three typical sketches of frames with energy dissipaters: a) FDs in single diagonal, b) at top of chevron brace, and c) at the intersection of cross-braces. In this study, the FDs were inserted to middle bays of 4 and 8 storey flexible and rigid frames, as seen in Figure 3.3.



(a)



(b)

Figure 3.3 Elevations view of 4 and 8 stories flexible and rigid frames with friction damper

Most important parameter in the design of FDs is determination of the slip load or slip design load. In order to find the optimum slip-load, a series of analyses is made to specify the optimum slip load of FDs to achieve minimum response and rectangular hysterestic curve. Subsequently, for the purpose of comparison, the slip load of FDs is assigned 400 kN at first story and 40 kN for the next stories for all rigid and flexible frames. Thus, a total of 8 different cases were considered in this study. The dynamic properties of the frames are also given in Table 1. As seen from Table 1, the fundamental periods of the frames with FDs were considerably shorter than the existing frames, which was also an indication that the frames with FDs were stiffer than those without FDs.

Table 3.1 Dynamic properties of the bare and frames with friction damper

Type of Buildings	T_1 (s)	T_2 (s)	T_3 (s)
4 story flexible bare	1.540	0.344	0.170
4 story flexible friction damper	0.433	0.142	0.091
4 story rigid bare	0.847	0.1926	0.0923
4 story rigid friction damper	0.300	0.103	0.064
8 story flexible bare	1.970	0.616	0.326
8 story flexible friction damper	0.867	0.289	0.170
8 story rigid bare	1.081	0.349	0.170
8 story rigid friction damper	0.566	0.198	0.117

3.2 Nonlinear Analysis Methods

To determine the seismic performance of the existing frames and those with FDs, nonlinear static and dynamic analyses were carried out using the finite element program of SAP 2000 non-linear version 14 (CSI, 2009). Nonlinear static pushover analysis is the most extensively used method to evaluate the nonlinear behavior of the buildings. The pushover analysis of a structure is a static nonlinear analysis under vertical loads as well as gradually increasing lateral loads. The equivalent static lateral loads nearly represent earthquake induced forces. A plot of the total base shear versus displacement in a structure was formed by this analysis that would reveal any premature failure. The analysis was conducted up to failure; hence it was possible to specify yielding point of system. On the frames, the plastic rotation was also monitored, and lateral inelastic forces versus displacement response for the complete structure were analytically computed. According to FEMA 356 (2000), the hinge properties of the structural components were determined considering component type and failure mechanism. After defining the plastic hinge properties in the model, the structures were subjected to monotically increasing lateral forces until a specified displacement was reached. The capacity curves related to base shear versus roof displacement for 4 and 8 story flexible and rigid frames with and without FDs were achieved at the end of the pushover analysis. Subsequently, the target displacements which represent the maximum displacement likely to be experienced during the design earthquake were also computed.

In order to specify the actual nonlinear behavior of buildings, besides carrying out pushover analysis, nonlinear time history analysis was performed. In this method, the buildings were subjected to real ground motion record. Hence, inertial forces were determined from the ground motions and the response of the building either in deformations or in forces were calculated as a function of time. In this study, Duzce earthquake record was used to compare the results of bare frames with and without FDs. The Duzce earthquake of 1999 was an earthquake that occurred on 12 November 1999 with a moment magnitude of 7.2 M_w and the peak ground accelerations were 0.51 g as shown in Figure 3.4. Analyses were carried out using the ground motion occurring 100% along x direction. Critical damping ratio of 5% was also assumed in the analysis.

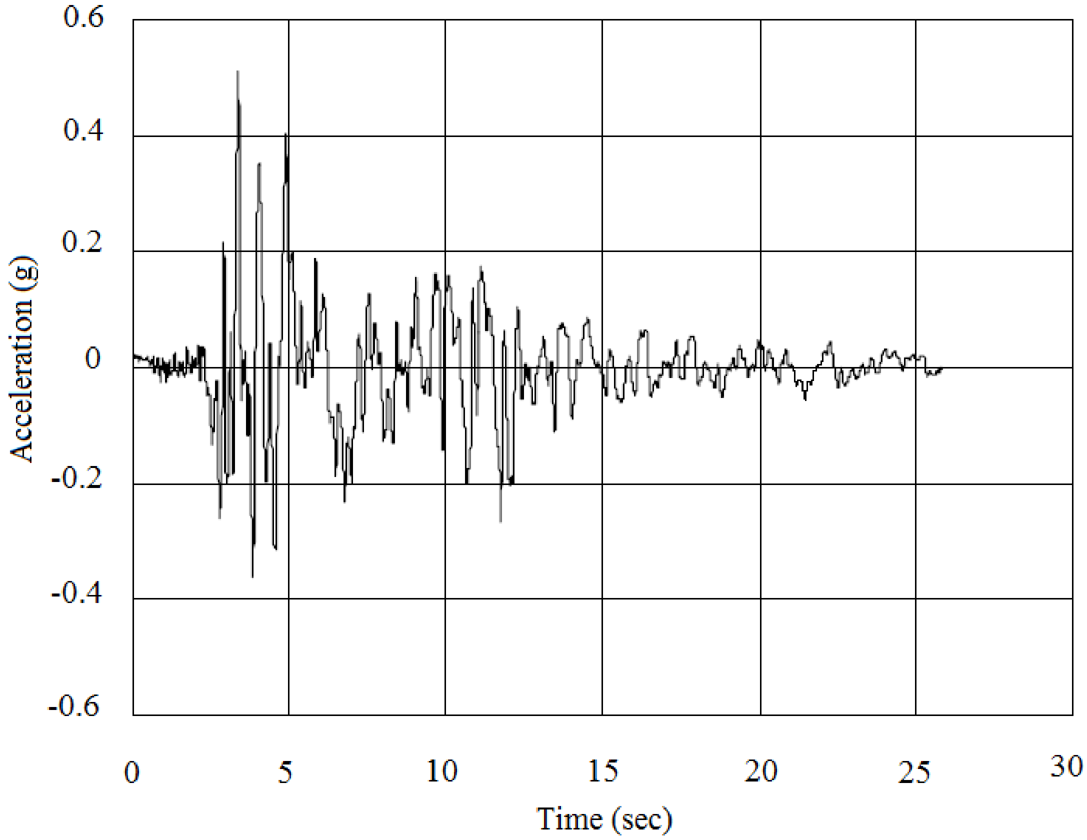


Figure 3.4 Accelerations Recorded in Duzce 270 Component

CHAPTER 4

RESULTS AND DISCUSSIONS

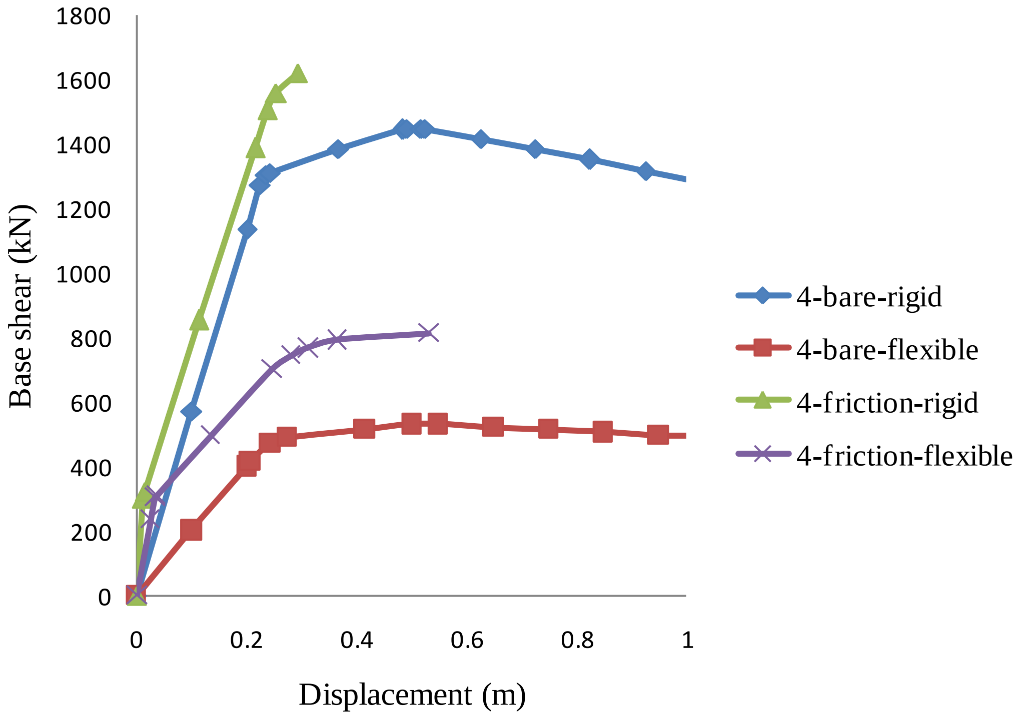
4.1 General

In this section, the results for bare frames (BFs), frames with friction dampers (FDs) obtained nonlinear static and time-history dynamic analysis were showed and discussed comparatively. In this study, a total of 8 different cases were considered and structural performance of BFs and frames with FD systems having different number of stories and different type of frame properties (flexible and rigid) under the effect of earthquake loading were evaluated. Performance characteristics in terms of capacity curves, inter story drift index, global damage index, base shear, hysteretic curves, and roof displacement were given below.

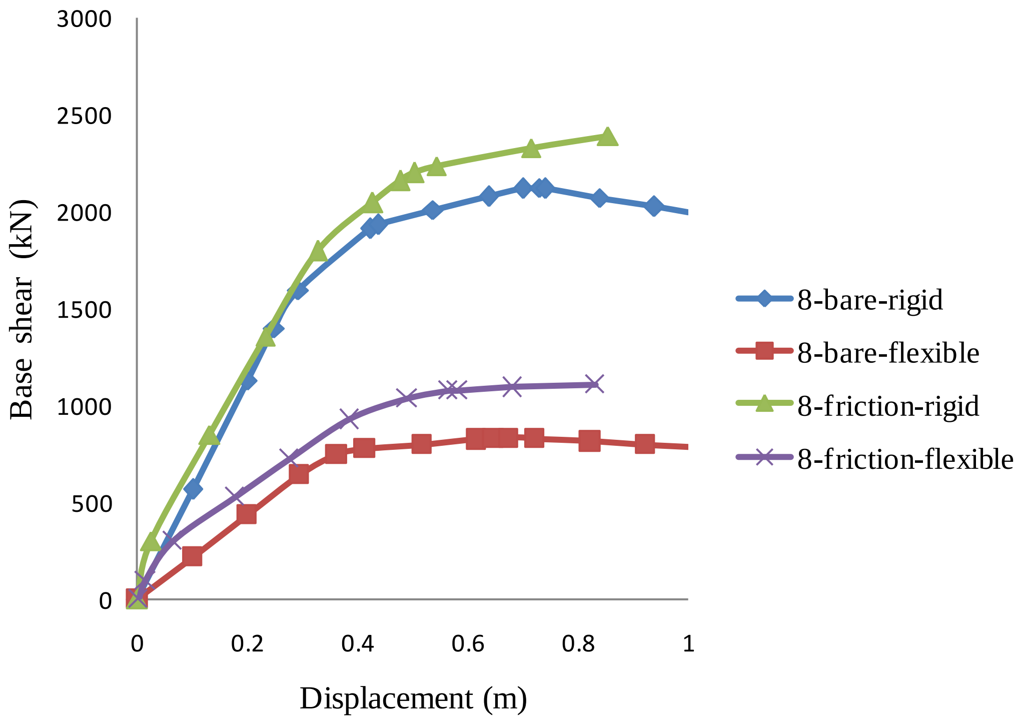
4.1.1 Capacity Curves

The capacity curves (pushover curves) were evaluated for different frame type. Figure 4.1 shows the comparison of the capacity curves of flexible and rigid frames with FDs and BFs. Apparently it pointed out that for both and rigid frame systems, the FDs were much stiffer and performed a better performance compared to the BFs. However, in some cases, there was a remarkable difference in the capacity curves, and that was due to a difference in the number of stories and using a different type of frames.

It was clearly understood that the capacity curves in all circumstances for BFs were bilinear since at the beginning the structure was globally in the elastic stage and provided a linear elastic slope, and then when the base shear was exceeded, some structural members (beams and columns) would yield and trigger to a change in the slope of the capacity curve. However, in the case of the FDs, the first change starts in the elastic slope was owing to the yielding of FDs. As far as explained before, FDs were designed not to slip during wind and any other service loads. During severe seismic excitations, FDs slip at a predetermined optimum load before yielding occurs in other structural members and dissipate a major portion of the seismic energy. After the earthquake, the building returns to its near original alignment without any failure.



(a)



(b)

Figure 4.1 Capacity curves of 4 and 8 stories with different frames

The values of target displacements were obtained from FEMA 356 coefficient method. Generally, as shown in Table 4.1, the rigid frames had smaller values of target displacements. The use of frame with FDs remarkably mitigated the value of target displacement compared to BFs, especially in the case of rigid frames with FDs. Moreover, the number of stories had effect on the this issue. By increasing number of stories, the value of target displacement is also increased.

Table 4.1 Target displacements obtained for the bare and friction damper moment resisting frames

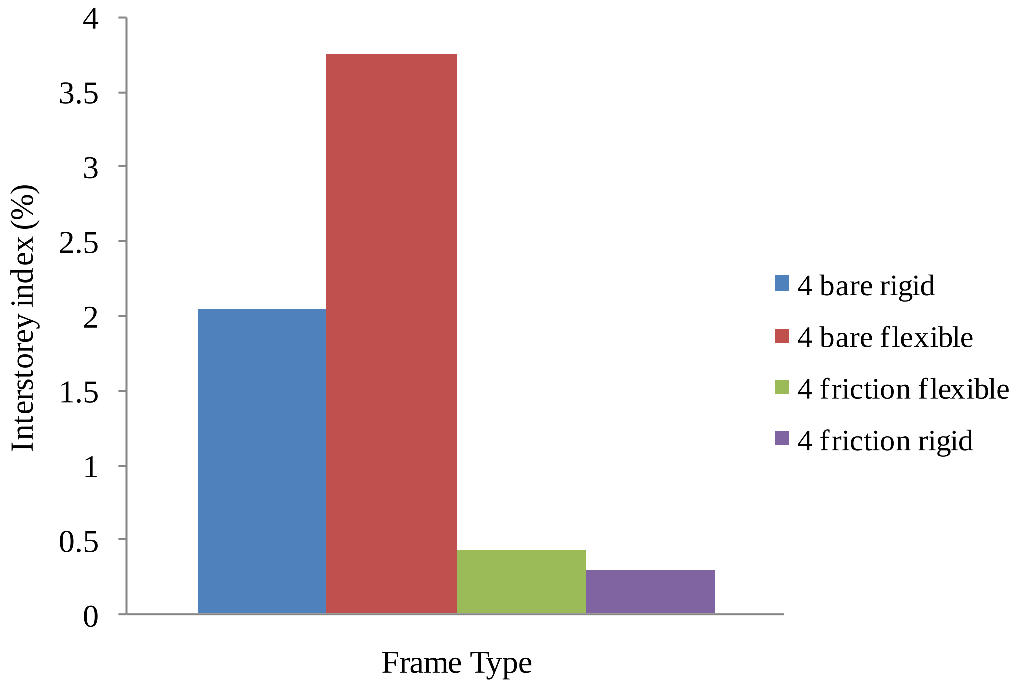
Type of buildings	Target(m)	Base shear (kN)
4 story flexible bare	0.271	487.7
4 story flexible friction damper	0.106	441.5
4 story rigid bare	0.15	854.7
4 story rigid friction damper	0.048	499.3
8 story flexible bare	0.376	825.0
8 story flexible friction damper	0.205	581.8
8 story rigid bare	0.201	1171.0
8 story rigid friction damper	0.128	753.3

4.1.2 Interstorey Index

The maximum inter storey drift (δmax) divided by the storey height (h) is defined as the maximum inter storey index. This index is a good indication of the damages experienced by the structural members.

The maximum inter storey index was assessed for both BFs and frames with FDs subjected to seismic excitation. Figure 4.2 compares maximum inter storey index for BFs and frame with FDs with different frame property. In case of BFs condition and FDs one, for both flexible and rigid systems, frame with FDs showed a better performance compared to BFs.

It was also observed from the figure that there was a difference between the inter storey indexes of the rigid and flexible frames equipped with FDs. However, rigid frames with FDs were performing better than the flexible frames with FDs. As a result, the differences in the interstorey index for the rigid frames with FDs were smaller than the interstorey index for the flexible frames with FDs as shown in Figure 4.2.



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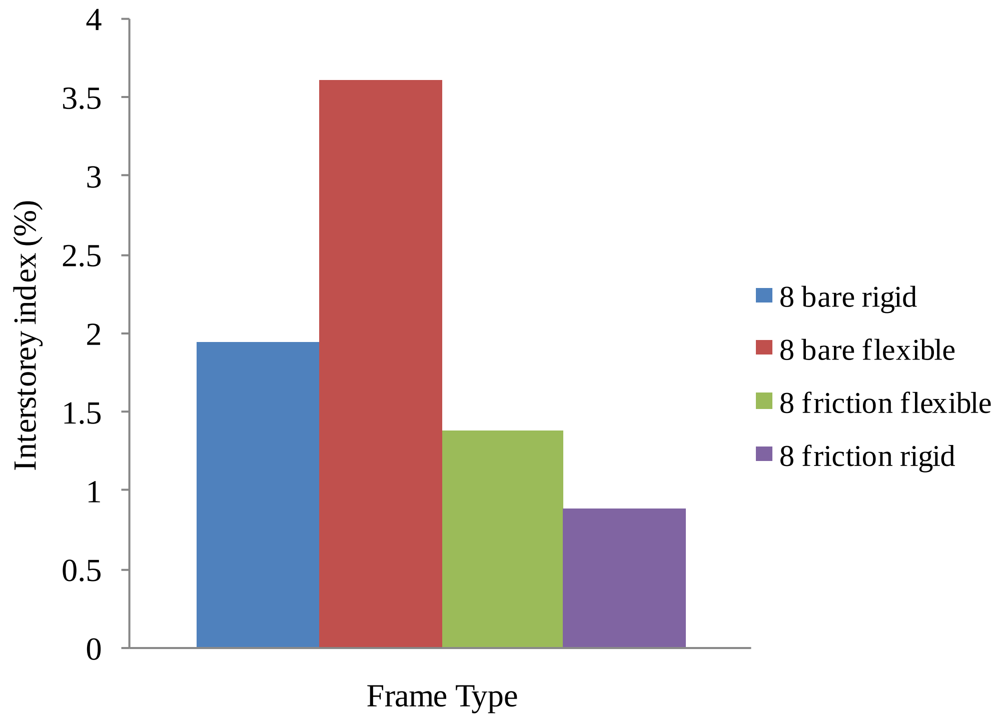
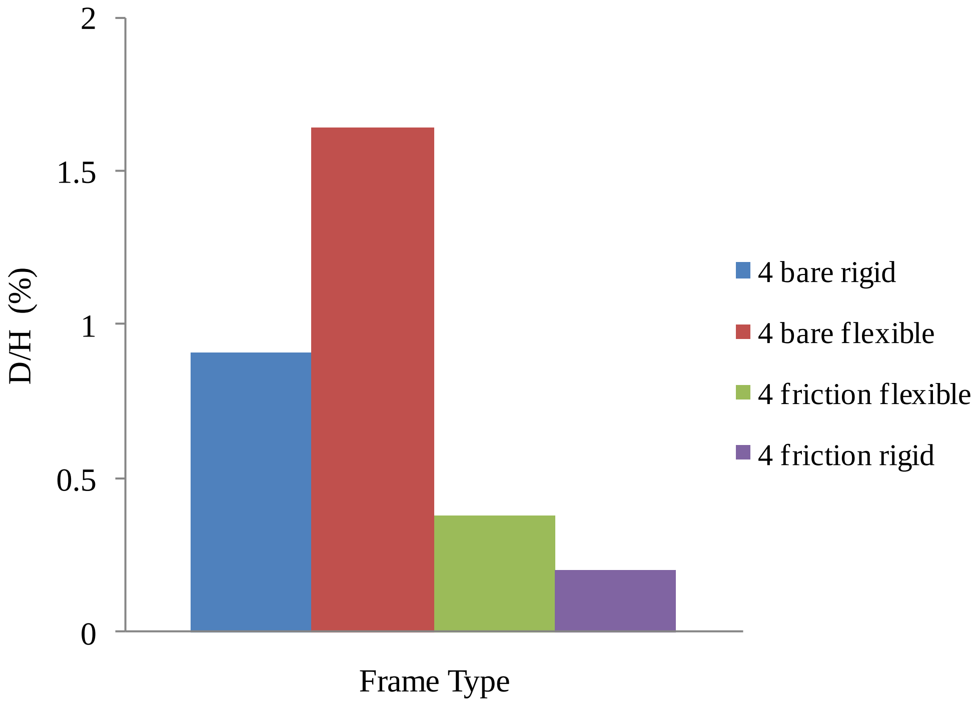


(b)

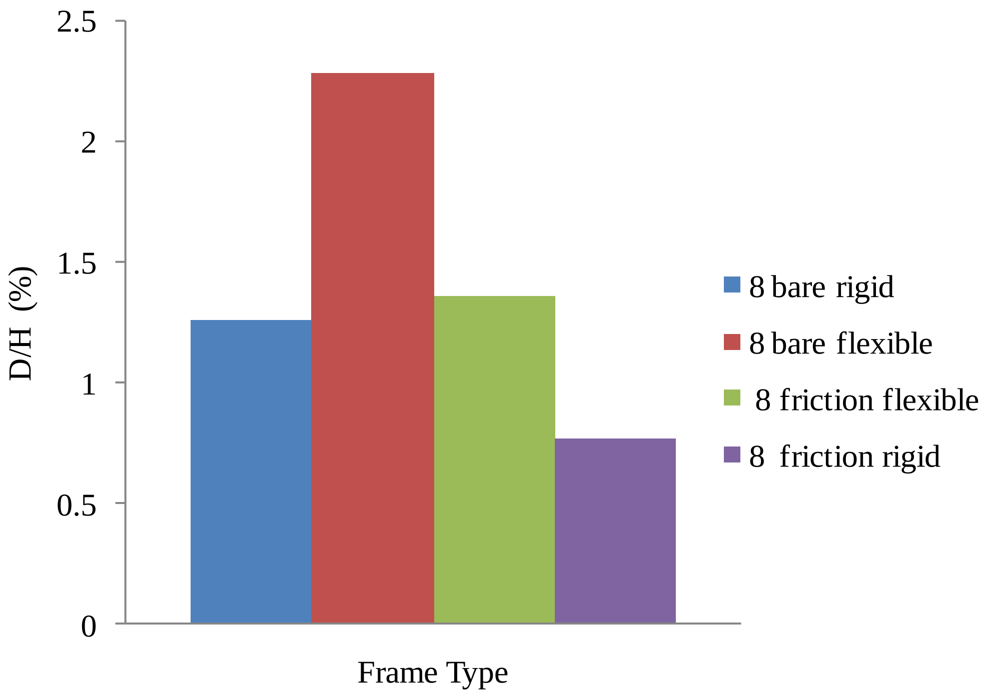
Figure 4.2 Maximum interstorey indexes for different frames

4.1.3 Global Damage Index

The ratio of the roof displacement (D) over the total height of the building (H) is known as the global damage index. Figure 4.3 compares the global damage index for BFs and frames with FDs with different frame property. Comparison of global damage index of the frames showed that the global index for BFs was remarkably greater than that for frames with FDs and showed better performance in comparison to BFs. The use of frames with FDs resulted in minimum reductions of 40%. The magnitude of these global deformations depends mainly upon number of story and especially characteristics of frame (flexible or rigid systems). It was observed that this index had a tendency to diminish with the use of rigid type of frames.



(a)



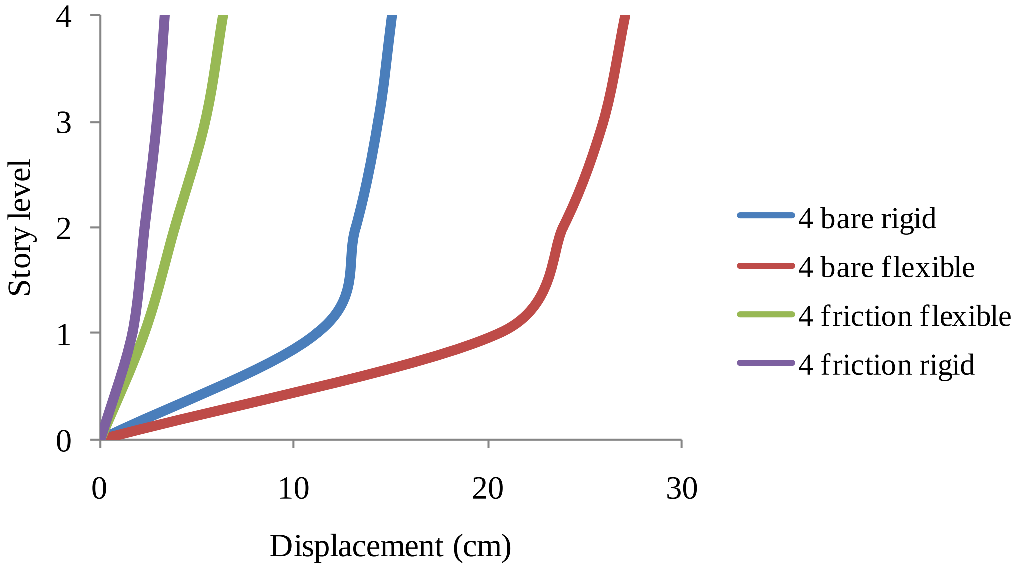
(b)

Figure 4.3 Global damage indexes for different frames

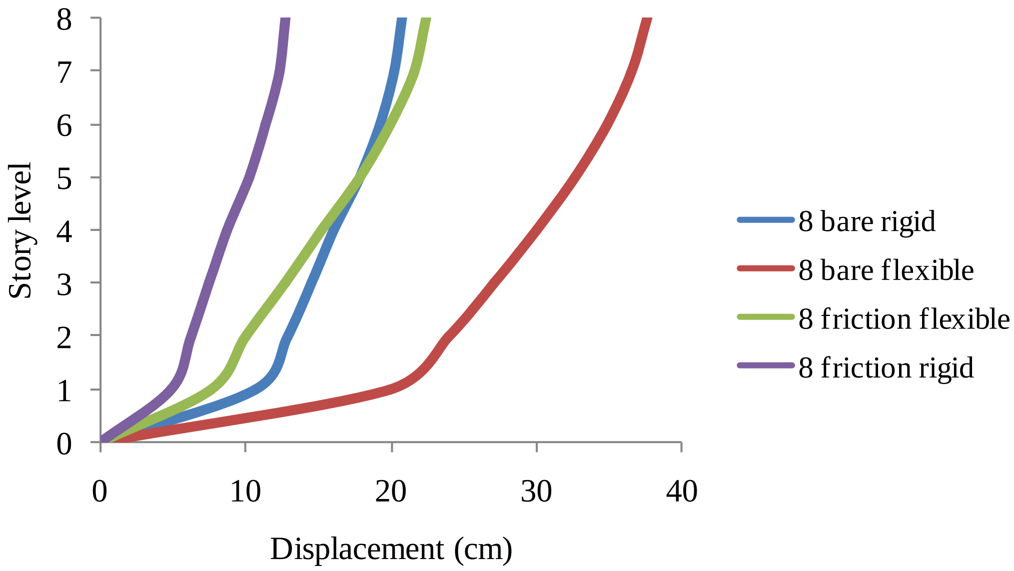
4.1.4 Variation of Storey Displacement

Figure 4.4 shows the deflected shape of BFs and frames with FDs at various cases at the instance corresponding to the target displacement. The use of frames with FDs remarkably decreased the value of maximum storey displacements and compared to BFs, especially in the case of rigid frames with FDs.

The maximum storey displacement was also affected by the number of stories and frame type. For example, in the case of four stories rigid frames with FDs, the maximum storey displacement was smaller than other frames, by increasing number of stories the maximum storey displacements were also increased.



(a)



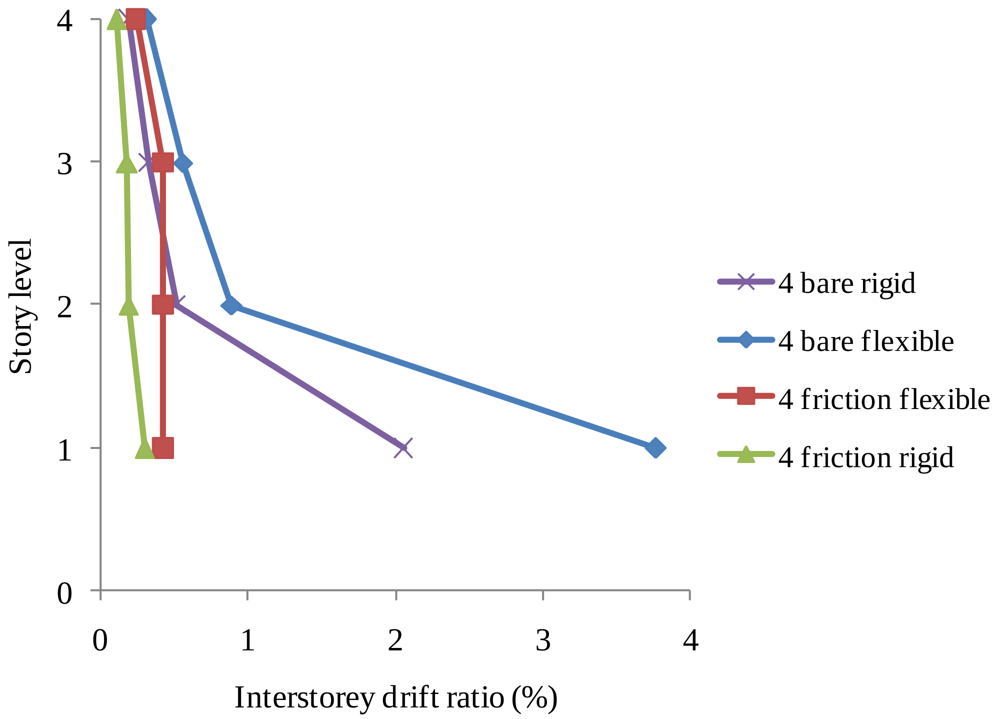
(b)

Figure 4.4 Deflected of 4 and 8 storeys with different frames

4.1.5 Inter-story Drift Ratio

Previous studies have highlighted the fact that steel buildings can perform important lateral deformations after an earthquake ground motion (Pampanin et al., 2003; Ruiz-Garcia et al., 2006). Therefore, inter-story drift demands over height in the BFs and frames with FDs were evaluated as seen in Figure 4.5.

In general, it can be observed that the addition of FDs reduces significantly the drifts in the frames. Figure 4.5 shows that the use of rigid frames is better than flexible frames and in the case of frames with FDs, the rigid frame with FDs, storey drift demands are significantly smaller.



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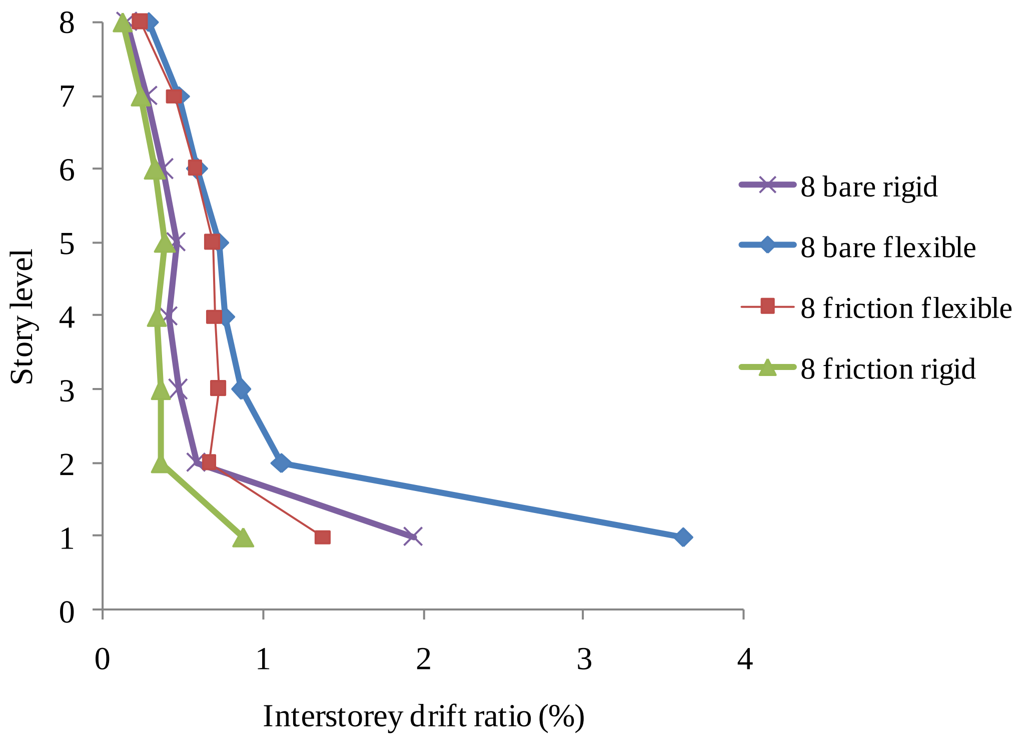
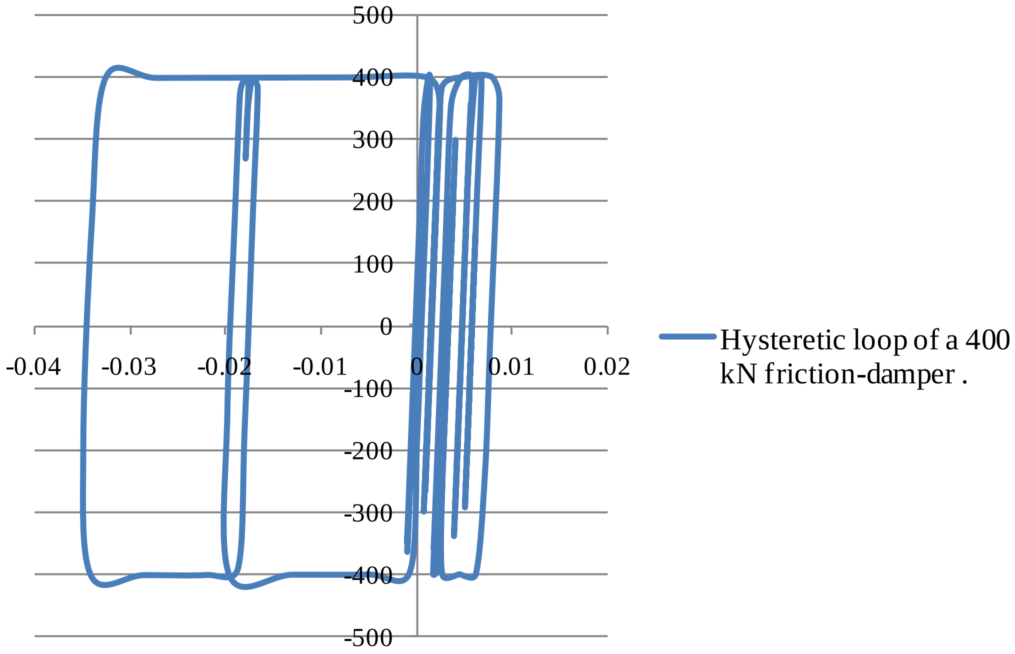


(b)

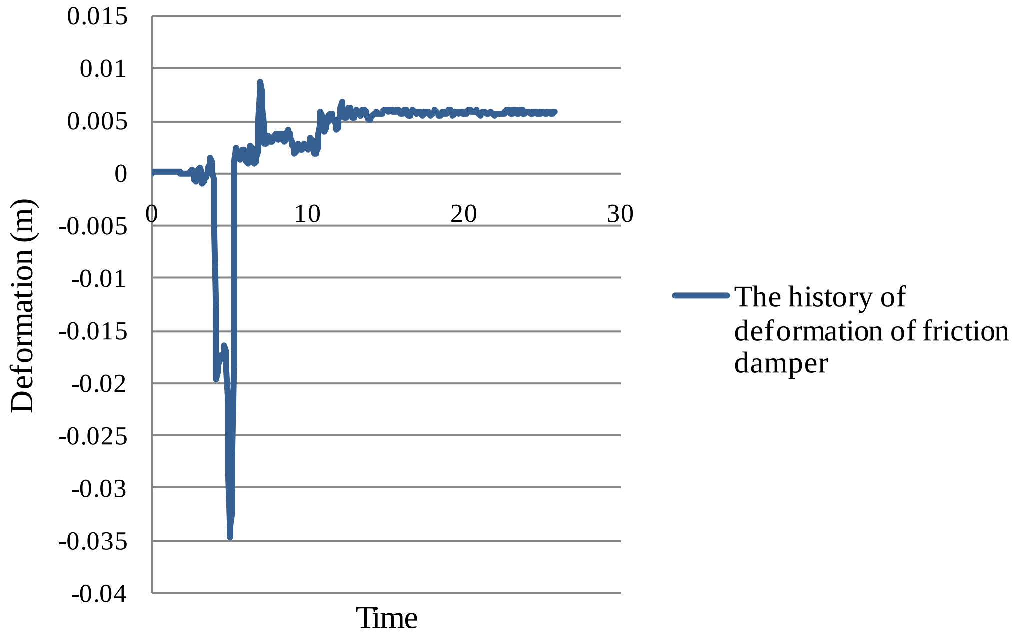
Figure 4.5 Interstorey drift ratio for different frames

4.1.6 Hysteretic Curves

In this study, the hysteretic curves which were obtained from the results of nonlinear dynamic analyses are presented in Figures 4.6-4.9 for rigid and flexible frames equipped with FDs. These figures indicated that after an earthquake, the building could be expected to return to its near original alignment. For example, the maximum amplitude of first story link element on the 4 story flexible frame and 4 story rigid one, the slippage were –35 and 15 mm, respectively. Time history of slippage in these frames is shown in Figure 4.6 and 4.7. The permanent offset in the dampers after the earthquake was about 5 mm on the 4 story flexible frame and 4 story rigid one. The maximum amplitude of first story link element on the 8 story flexible frame and 8 story rigid one, the slippage were 20 and -27 mm, respectively. Time history of slippage in these frames is shown in the figures. The permanent offset in the 8 story rigid and flexible frame was about 2 and 7 mm, respectively. Thus, It was explicit that 8 story rigid frame has supremacy over 8 story flexible frame when considering the permanent offset parameters.

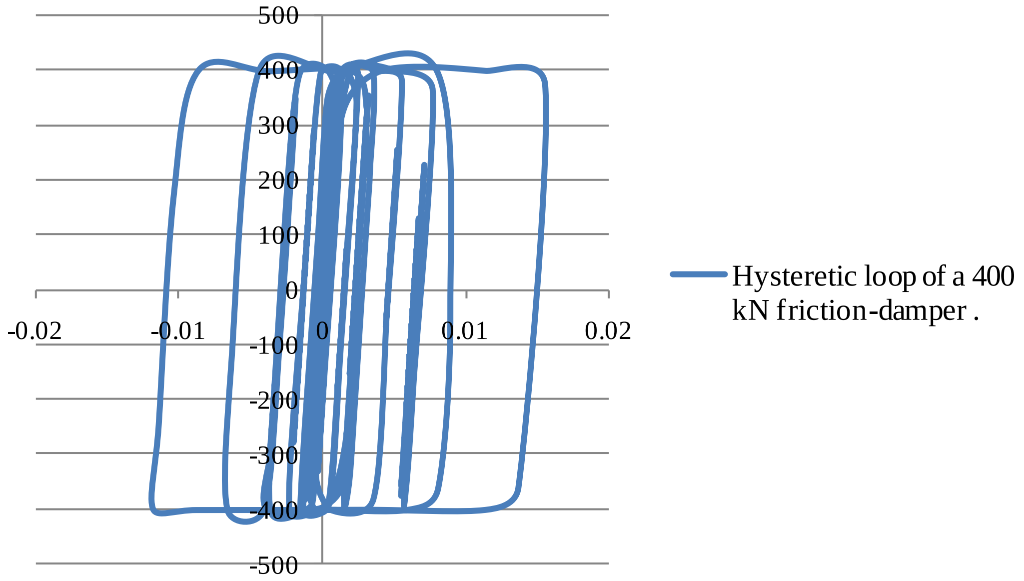


(a)

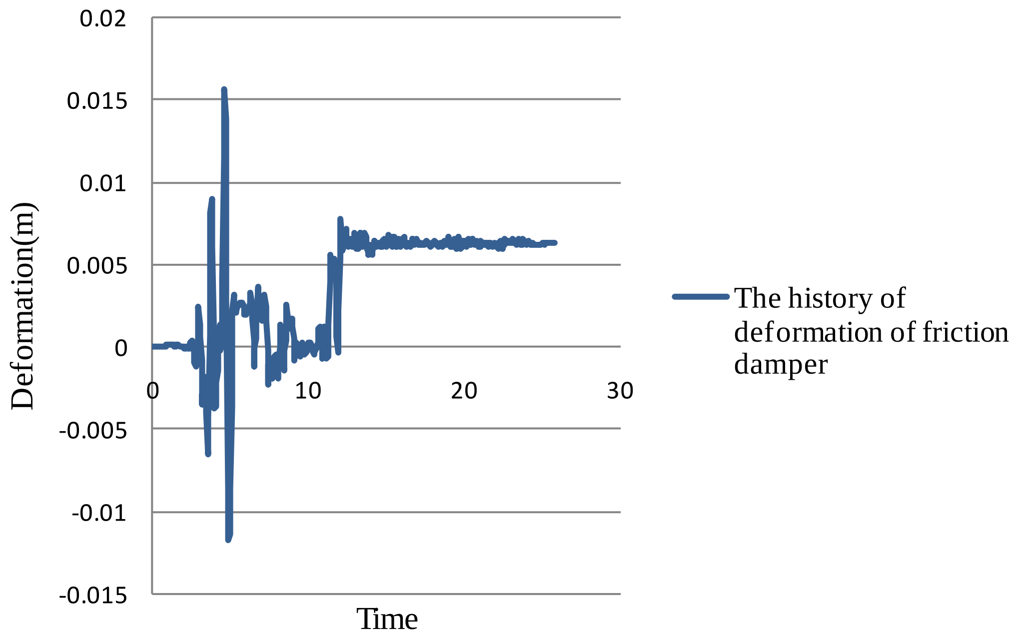


(b)

Figure 4.6 Hysteretic loop and time history of deformation of friction damper of the 1st story link element on the 4 story flexible frame

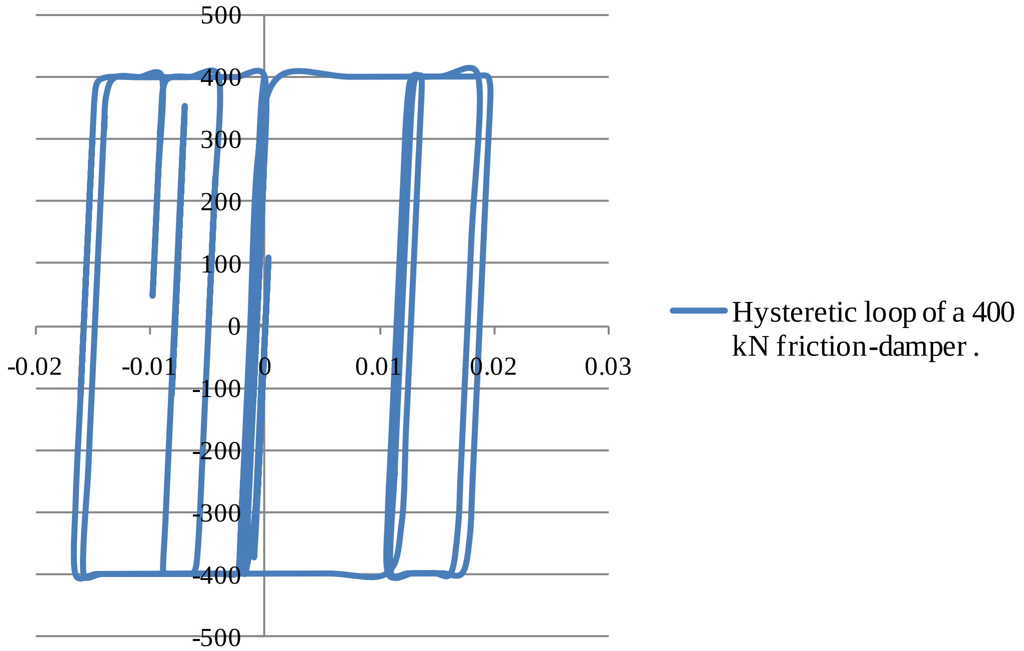


(a)

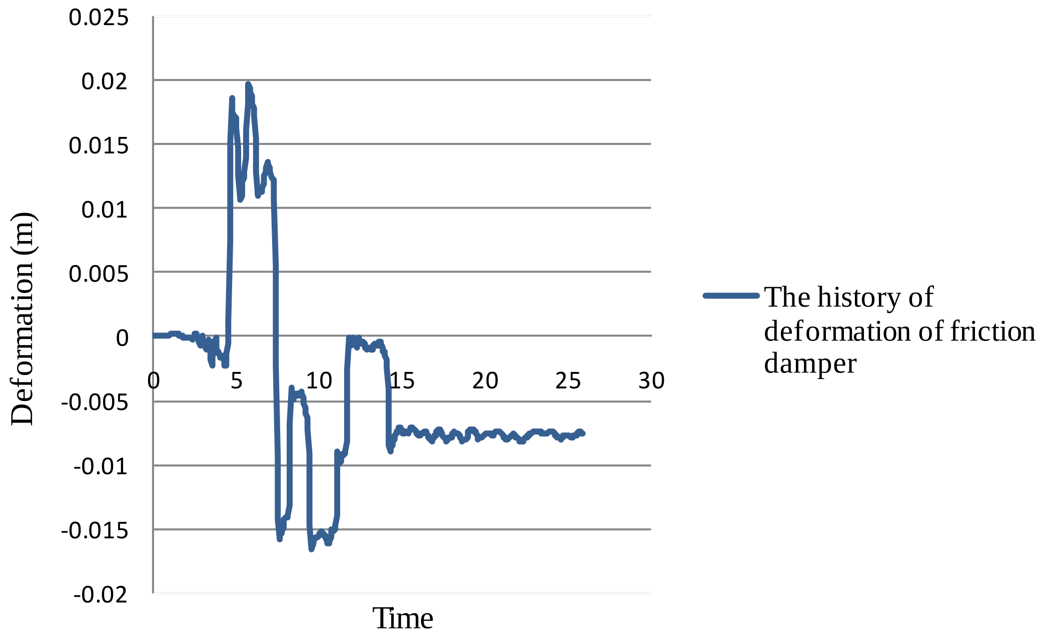


(b)

Figure 4.7 Hysteretic loop and time history of deformation of friction damper of the 1st story link element on the 4 story rigid frame

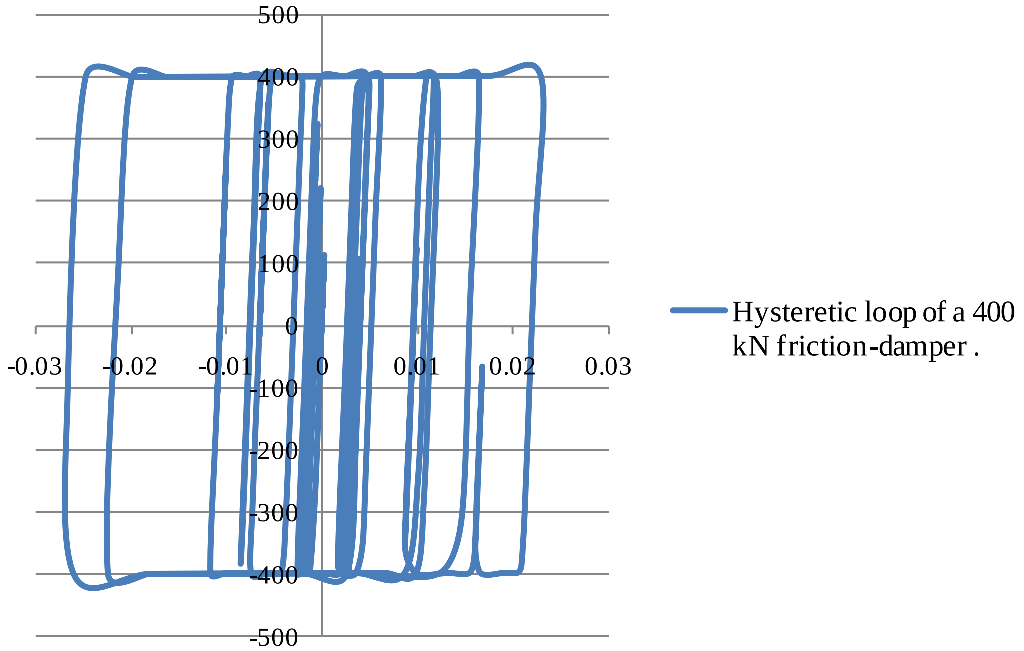


(a)

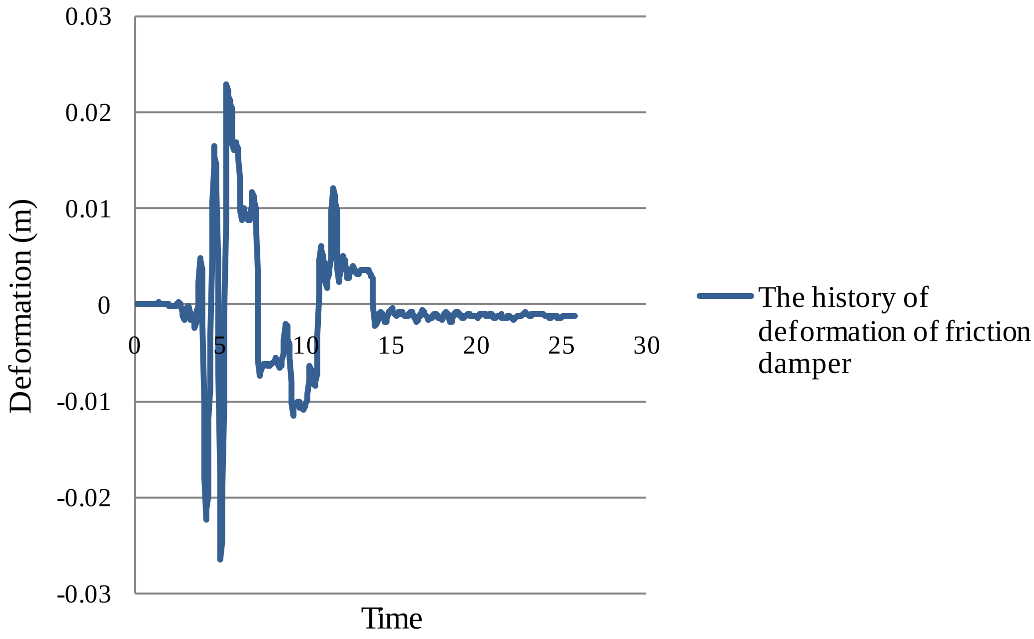


(b)

Figure 4.8 Hysteretic loop and time history of deformation of friction damper of the 1st story link element on the 8 story flexible frame



(a)



(b)

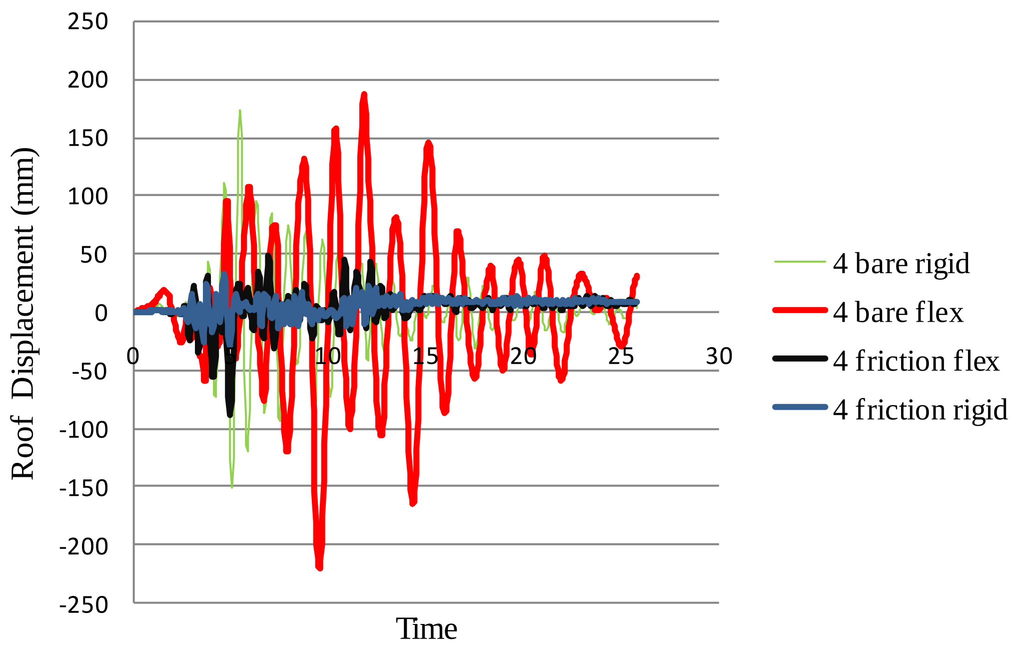
Figure 4.9 Hysteretic loop and time history of deformation of friction damper of the 1st story link element on the 8 story rigid friction frame

4.1.7 Roof Displacements

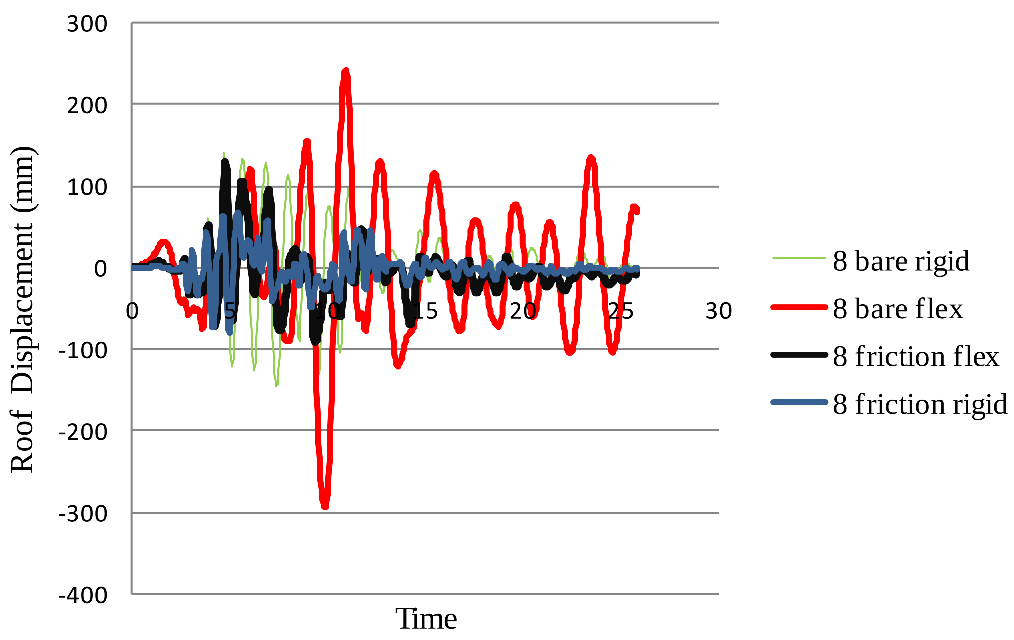
Figure 4.10 shows the roof displacement of the frames with and without FDs. The inclusion of FDs remarkably decreased the value of roof displacement compared to bare frames, especially in the case of rigid frames with FDs. The maximum roof displacement was influenced by the number of stories and frame type. For example, increasing the story level and using the FDs led to a downward trend for the maximum story displacement of the roof level for all cases.

In the case of 4 storey rigid frames with and without FDs, the maximum roof displacement was 32.0 mm and 172.8 mm, respectively while in the case of 8 storey rigid frames with and without FDs, that was -145.0 and -79.6, respectively. Moreover, in the cases of 4 and 8 storey flexible frames with and without FDs, the values of maximum roof displacement was observed to be -88.6 mm and -221.1 mm, 129.40 and -293.5 mm, respectively.

It was also evident from Figure 4.10 that the inclusion of FDs into 4 and 8 storey rigid frames resulted in reduction of roof displacement as 81% and 45%, respectively. Using FDs in 4 and 8 storey level flexible frames reduced roof displacement as 60% and 55%, respectively. Moreover, the peak amplitude in x direction was -293.5 mm for flexible frames while minimum amplitude in x direction was occurred as 32.1 mm for rigid frames with FDs. At this low level of drift, no damage was expected during this kind of a major earthquake. Moreover, all deformed shapes were given in Appendix A in order to compare the all cases.



(a)



(b)

Figure 4.10 Roof displacement for different frames

CHAPTER 5

CONCLUSIONS

This study investigated the structural performance of different types of MRFs buildings and those equipped with FDs. From the results of this study, the following conclusions can be drawn:

- From the capacity curves, it was observed that the base shear, which is the capacity of the frame to resist lateral loads, was considerably increased in the presence of FDs.

- Depending on the design properties of the BFs, frames with FDs provided smaller interstorey drift index compared to BFs. The results of the performed nonlinear static analysis and nonlinear time-history dynamic analysis indicated that frames with FDs were effective in diminishing drifts since the reduction of interstorey drifts with respect to the BFs was on average equal to 50%. Similarly, the use of frames with FDs significantly reduced the global damage index of both flexible and especially rigid frames.

- The use of frames with FDs remarkably decreased the value of roof displacement and compared to BFs, especially in the case of rigid frames with FDs.

- The maximum storey displacement is also affected by the number of stories and frame type. For example, in the case of four stories rigid frames with FDs the roof displacement is smaller than other frames, by increasing number of stories roof displacement are also increased.

- The target displacement was reduced significantly by using frame with FDs. Moreover, frame with FDs were performed better performance compared to BFs. Thus, the results of analysis showed that as the rigidity of the frame increased, smaller target displacement values were obtained.

- The nonlinear dynamic analysis and static analysis results showed that the BFs added with FDs generally satisfied the capacity curves, inter story drift index, global damage index, base shear, hysteretic curves, and roof displacement. The supremacy of rigid frame with FDs systems were explicit in all cases.

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APPENDIX

Appendix A: Deflected shapes

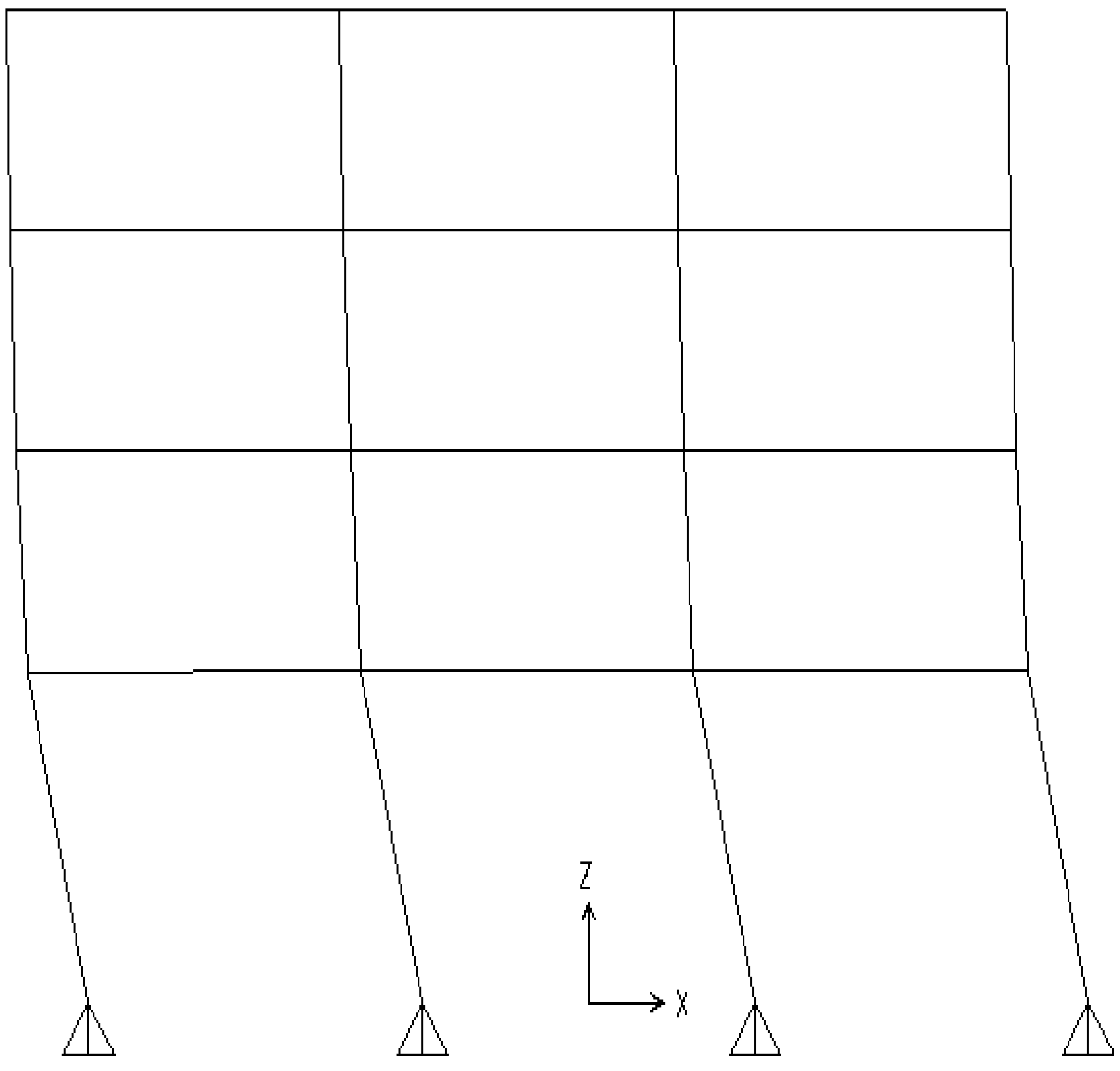

Figure A1. Deformed shape of four stories flexible bare frame at T_1=1.540 s

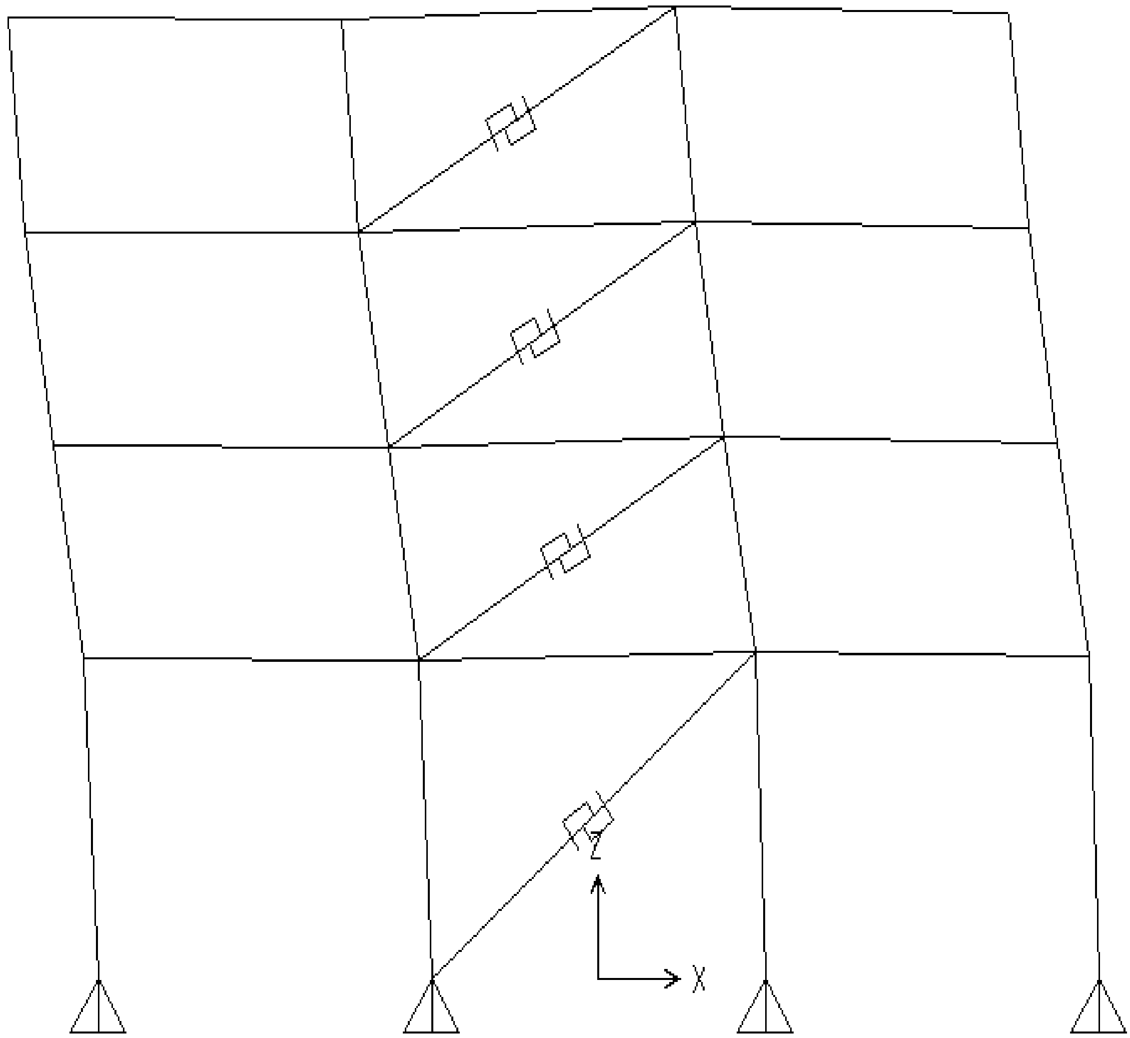

Figure A2. Deformed shape of four stories flexible frame with friction damper at T_1=0.433 s

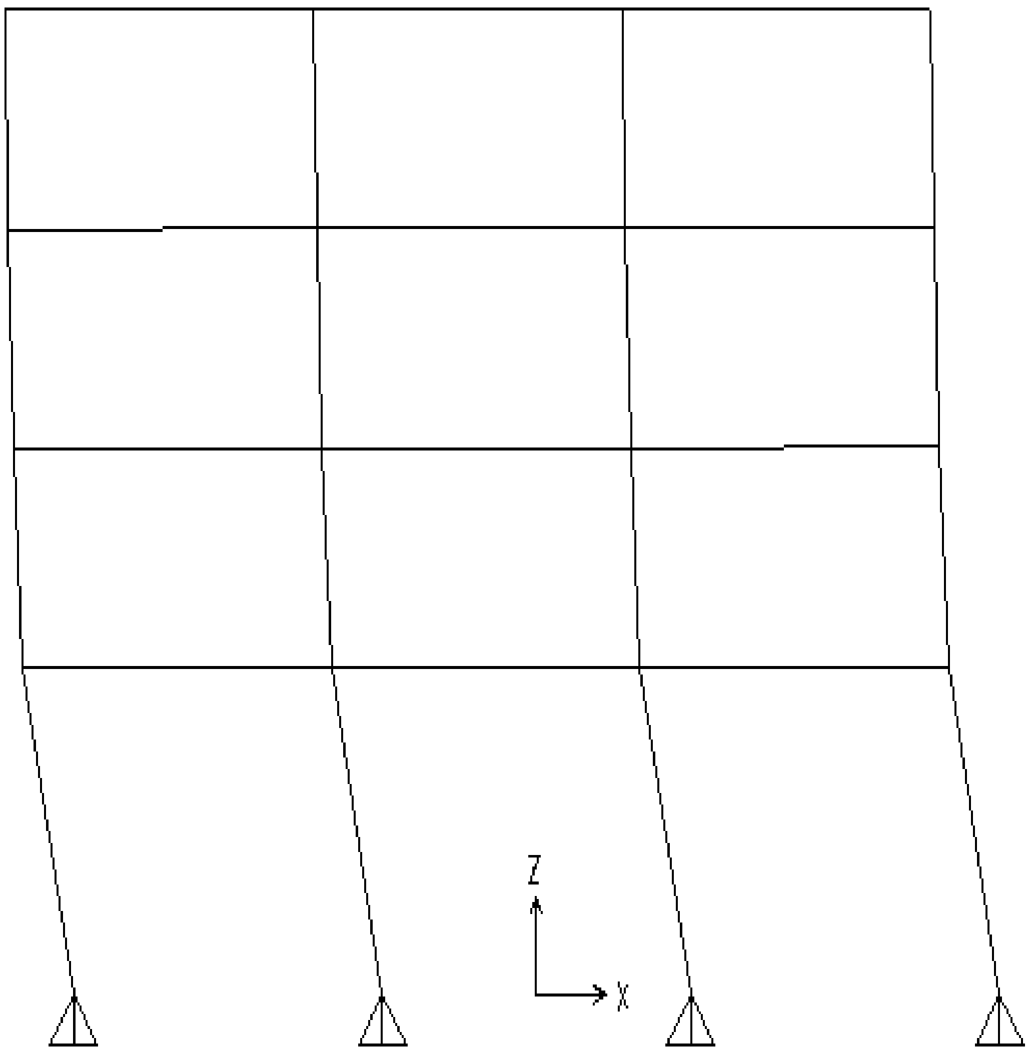

Figure A3. Deformed shape of four stories rigid bare frame at T_1=0.847 s

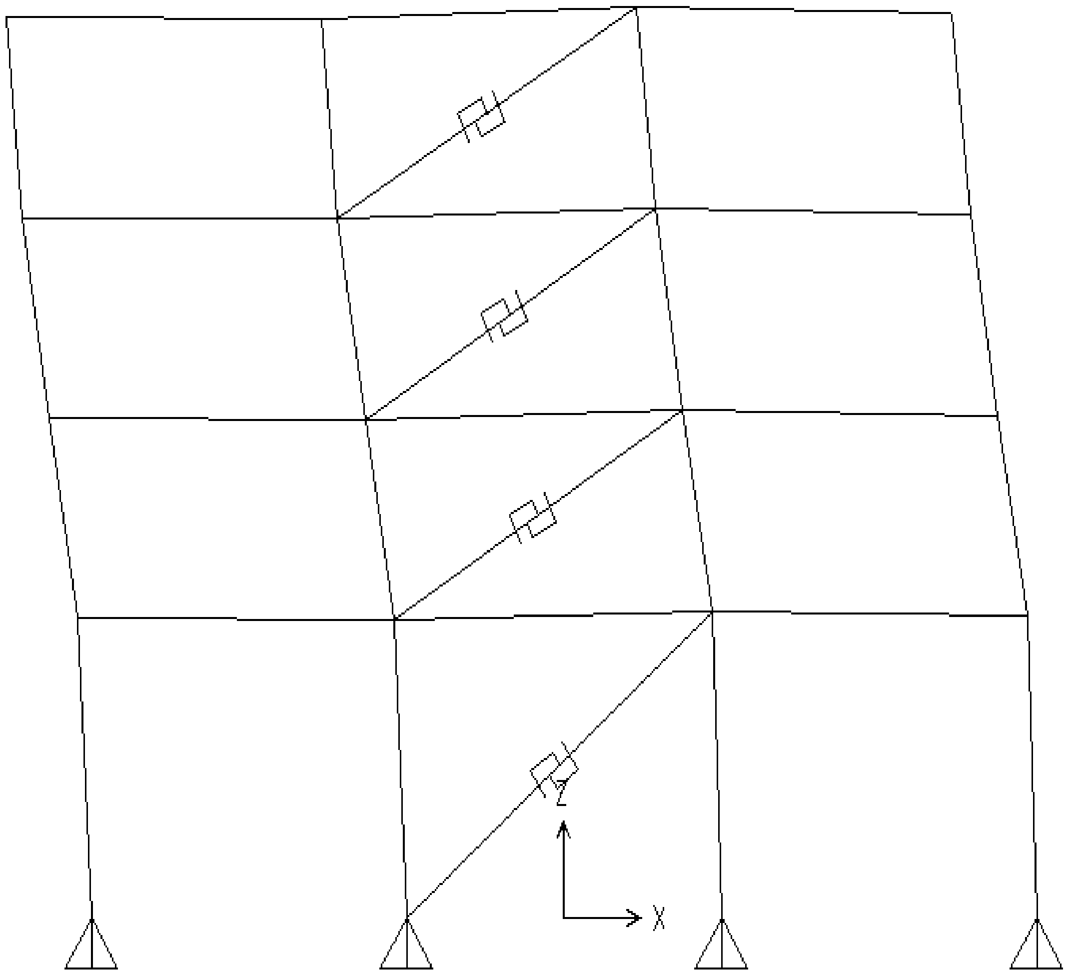

Figure A4. Deformed shape of four stories rigid frame with friction damper at T_1=0.300 s

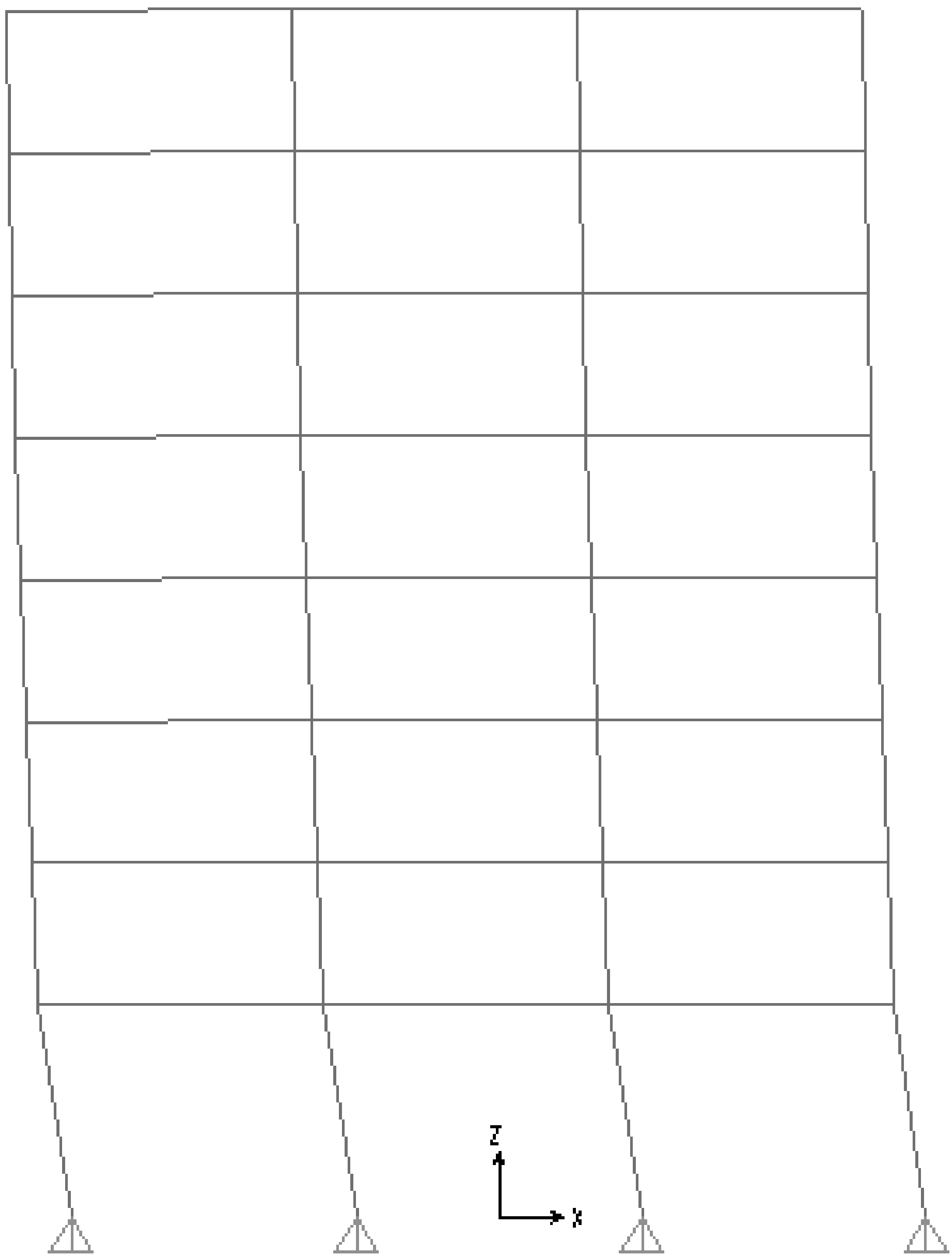

Figure A5. Deformed shape of eight stories flexible bare frame at T_1=1.970 s

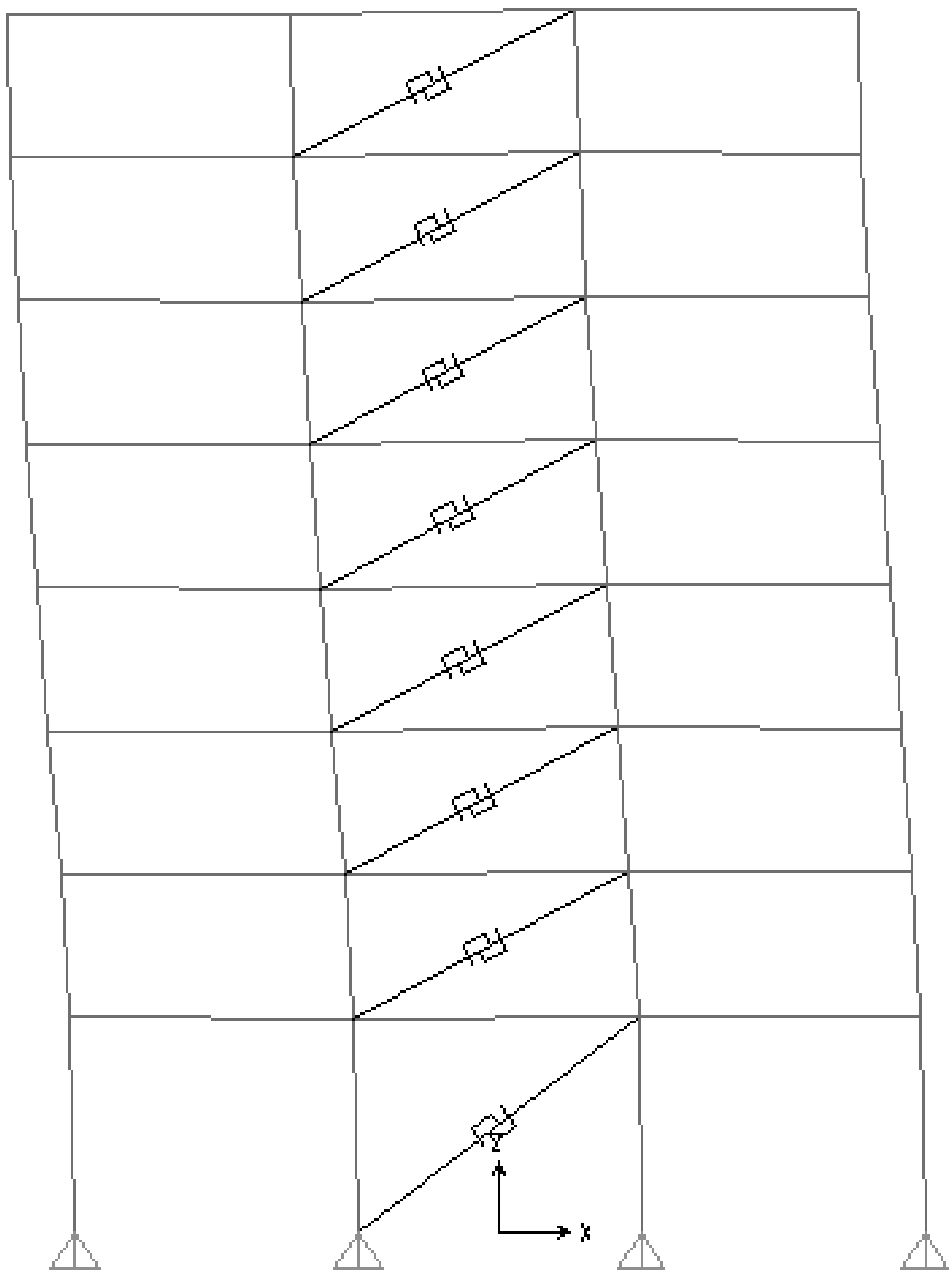

Figure A6. Deformed shape of eight stories flexible frame with friction damper at T_1=0.867 s

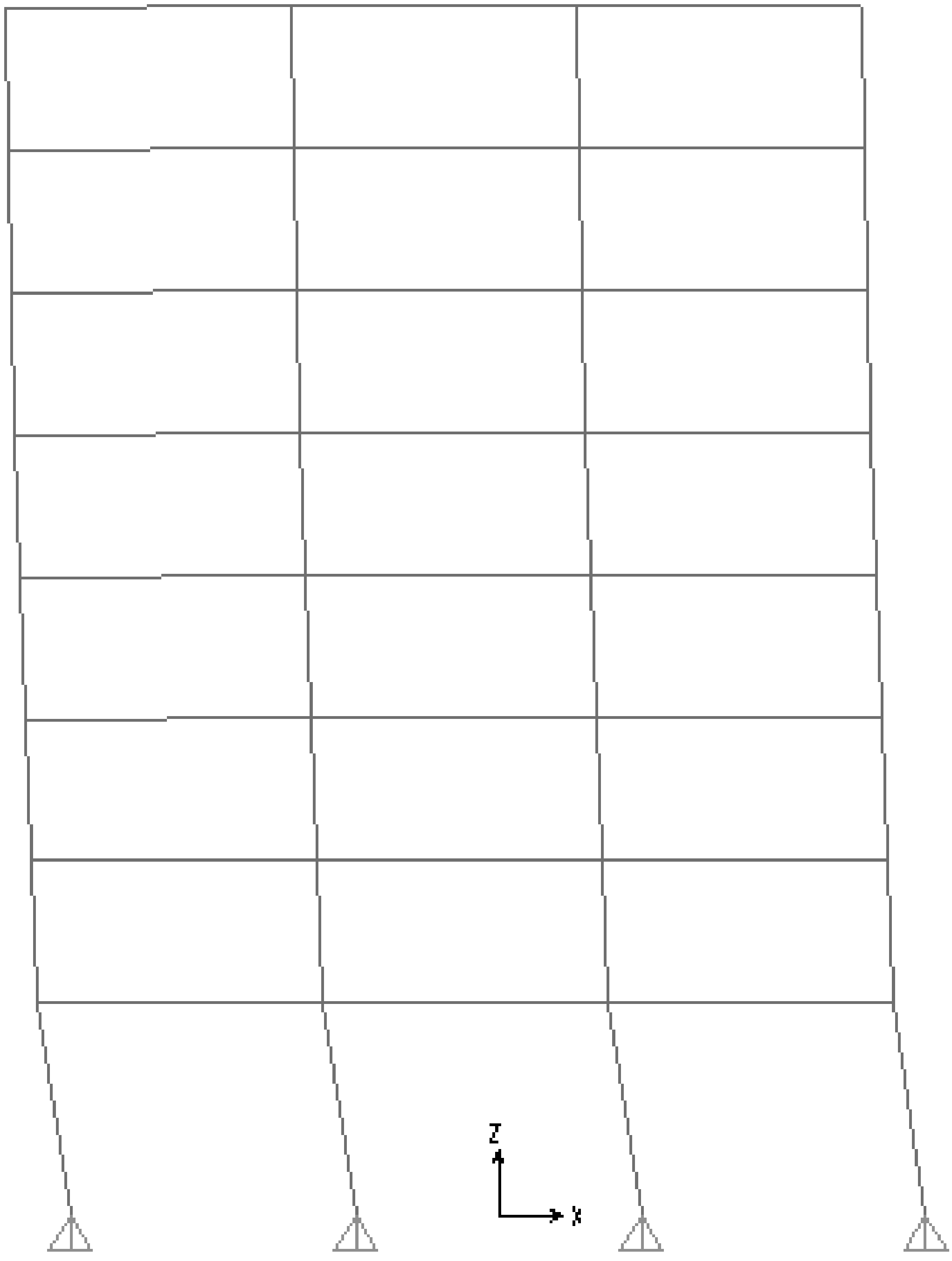

Figure A7. Deformed shape of eight stories rigid bare frame at T_1=1.081 s

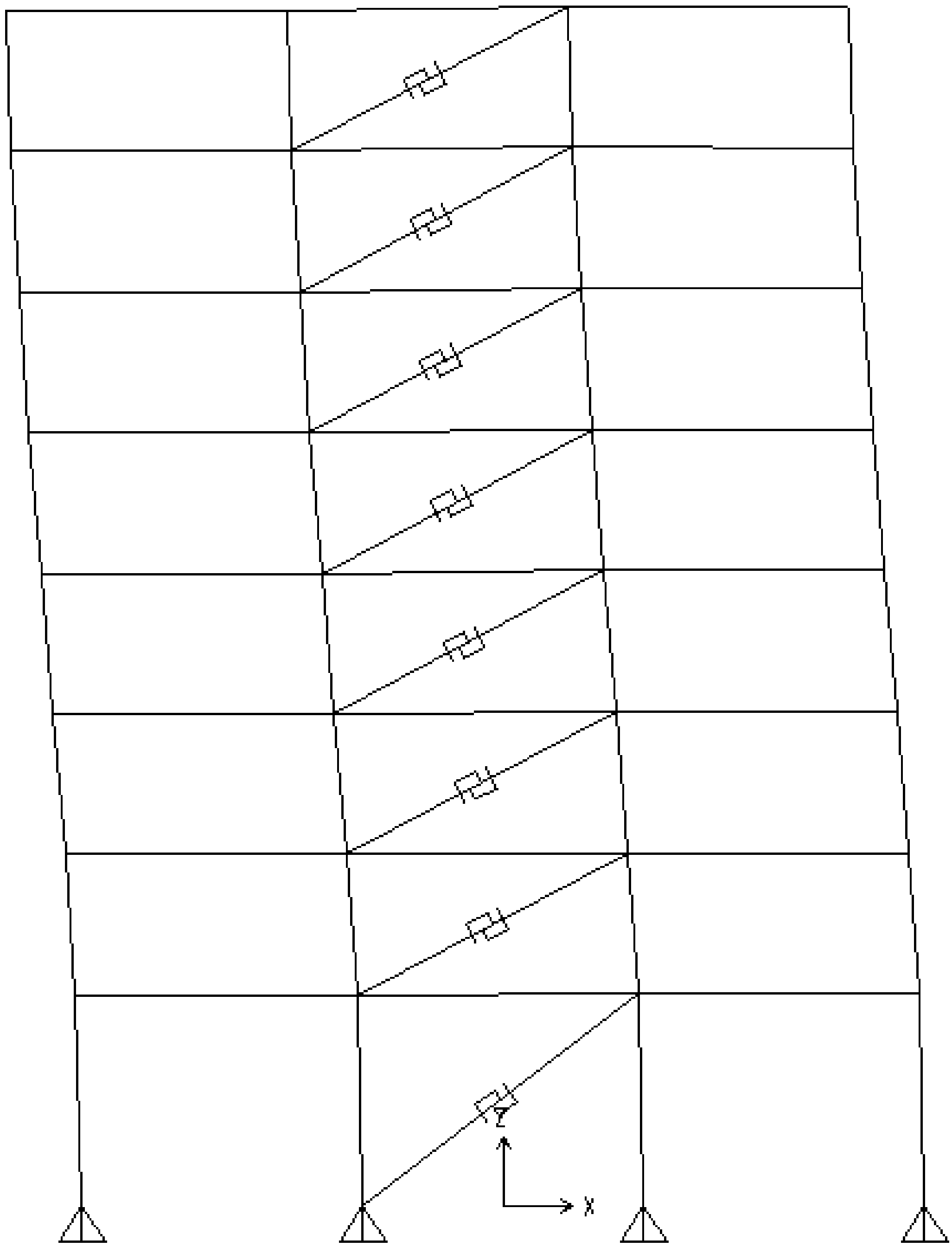

Figure A8. Deformed shape of eight stories rigid frame with friction damper at T_1=0.566 s