RETROFITTING OF RC FRAME BUILDINGS USING SHEAR LINK BRACED FRAME

A DISSERTATION

Submitted in partial fulfillment of the
requirements for the award of the degree
of
MASTER OF TECHNOLOGY
in
EARTHQUAKE ENGINEERING
(With Specialization in Structural Dynamics)

By

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CANDIDATE'S DECLARATION

I hereby declare that the work which is being presented in this dissertation report entitled, **"RETROFITTING OF RC FRAME BUILDINGS USING SHEAR LINK BRACED FRAME"**, in the partial fulfillment of the requirements for the award of the degree of **Master of Technology** in **Earthquake Engineering**, with specialization in **Structural Dynamics**, submitted in the Department of Earthquake Engineering, Indian Institute of Technology Roorkee, is an authentic record of my own work carried out for a period from July-2010 to June-2011 under the supervision of **Dr. D. K. Paul**, Professor, Department of Earthquake Engineering, IIT Roorkee.

The matter embodied in this report has not been submitted by me for the award of any other degree or diploma of this Institute or any other University/Institute.

Date: 27-06-11
Place: Roorkee

Maniyar Sufyan Usman

CERTIFICATE

This is to certify that the above statement made by the candidate is correct to the best of my knowledge.

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ABSTRACT

The present work attempt to study the behaviour of building designed for gravity loads only under the effect of lateral seismic load. The buildings found deficient are then retrofitted by providing shear link with the help of chevron braces to improve its performance. It also provides the literature study of various types of energy absorbing devices that can be implemented in improving the behaviour of deficient RC frame buildings.

Past earthquakes have shown a great deal of damages to the deficient RC frame buildings design without any considerations to lateral seismic load. Chevron braces with shear link can be implemented as an effective retrofit measure. In the present study the behaviour of buildings designed for gravity load only have been worked out by performing nonlinear static pushover analysis. It also provides a step by step procedure to retrofit deficient RC frame building by implementing chevron braces with shear link. The behaviour of retrofitted buildings is then worked out by performing nonlinear static pushover analysis and series of nonlinear time history analysis.

Parametric study has also been done to study the behaviour of shear link, forces in brace and column after implementing retrofitting scheme. Based on analysis done finally conclusions and useful recommendations are made at the end.

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LIST OF NOTATIONS

A_e	Area under the linear elastic base shear v/s roof displacement
A_p	Area under the elastoplastic base shear v/s roof displacement curve
Q_1	Strength of retrofitting system at the base
V_y	Yield base shear
A_w	Area of web
F_y	Yield force
Φ	Over strength of factor
α	Angle of inclination of brace with horizontal
P_b	Force in brace
L	Length of shear link
d	Height of shear link
t_w	Web thickness of shear link
D	External diameter of brace
t	Thickness of brace
#	Diameter of bar

Chapter - 1

INTRODUCTION

1.1- General

A large number of existing buildings in India has been designed to resist gravity load only without any consideration of lateral seismic load. Most of the important cities of India lies in active seismic zone as per IS:1893-2002 (part-1). It is not economically feasible to retrofit all the buildings but some important buildings need seismic evaluation due to various reasons such as, noncompliance with the codal requirements, updating of codes and design practice and change in the use of building.

The aim of conventional design is to satisfy the two requirements that (i) a structural system under ordinary actions such as lateral loads due to wind and low intensity earthquakes the structure must be stiff in order to minimize structural and non structural damage and (ii) under severe earthquakes the structure must be safe from collapse even if some structural damage may be acceptable. In important buildings, avoidance of structural collapse alone is not enough. The cost of non-structural components and contents is much higher than the cost of the structure itself and must be protected. The buildings of post disaster importance such as hospitals, telecommunications, police stations etc. must remain operational following a major earthquake.

If the amount of energy getting into the structure can be controlled and a major portion of the energy can be dissipated mechanically independent of primary structural component, the seismic response of the structure and damage control potential can be considerably improved. These objectives can be accomplished by adopting new techniques of base isolation and other passive energy dissipation devices. Among the energy dissipation devices there are many alternative devices that can be effectively implemented to alleviate the detrimental effect of earthquake energy. Shear link is one of those alternatives available to reduce the earthquake response of the existing building and can be very effectively implemented in design of new building for the desired performance level. In this thesis a comprehensive study has been made to retrofit buildings originally designed only for gravity loads using chevron braces with aluminium shear link to resist earthquake forces.

1.2- Need for the Study

The recent devastating earthquakes have exposed the vulnerability of the existing reinforced concrete buildings in India. The Bhuj Earthquake (2001) saw a great deal of damages to multi-storey buildings in the urban area of Gujarat. This has posed a serious threat to the many existing RC buildings which were designed mainly for gravity loads.

An earthquake is the most extreme condition that any building may be required to resist during its lifetime. During a major earthquake, a large amount of energy is pumped into the building. The manner in which this energy is consumed in a structure determines the level of damage. The building codes recognize that it is economically not feasible to reconcile this energy within the elastic capacity of materials. Hence damages are bound to occur under maximum considered earthquake from economic consideration. When a building is subjected to such earthquake, energy is absorbed by cracking of concrete and elongation of steel beyond the elastic limits. This damage to the structural members if beyond a threshold level can be dangerous. There exist another way of absorbing earthquake energy i.e. concentrating the damages on a specially design element which are forced to stay ductile through cyclic yielding and dissipates the energy through cyclic yielding. Chevron brace with aluminium shear link provide such an alternative to dissipate the earthquake induced energy and limiting the forces that are transferred to the primary members of the structure. Chevron braces with aluminium shear link can be installed in existing and new buildings with ease. These are ideally suited for retrofit job as it requires minimal civil construction works and the job can be easily undertaken with the building being simultaneously in use. This makes them extremely versatile for retrofit projects.

1.3- Objectives and Scope of the Study

1. To review the available literature covering the various types of energy dissipation devices with particular reference to metallic dampers (Aluminium shear link).
2. To suggest improvement in the performance of buildings designed for gravity load only using aluminium shear link with chevron braces.
3. To evaluate and compare the performance of buildings initially designed for gravity load only with the retrofitted buildings using chevron braces with aluminium shear link.

The present study discusses the response of buildings designed only for gravity load. As these buildings are deficient in resisting the lateral seismic loads, the retrofitting technique using chevron braces with aluminium shear links are suggested to improve the performance of the same building designed only for gravity load. It also make the comparative study of the buildings designed for gravity loads only and the same buildings retrofitted with chevron braces with aluminium shear link after 3D analysis in SAP2000. Finally, useful conclusions and recommendations are provided at the end.

1.5- Outline of the Thesis

Chapter 1 provide the general introduction and the objective of the thesis. It also discusses the need for this study. Chapter 2 deals with the study of the available literature on the various types of energy dissipation devices. It discusses the metallic dampers in general and aluminium shear link with chevron braces in particular. It also provides the principle of using aluminium shear link with chevron braces as an energy dissipation device in the retrofitting of the existing buildings.

Chapter 3 describes the modelling and analysis aspects of the thesis. It also provides the step by step procedure for the performance enhancement of the gravity load design building using aluminium shear link. Nonlinear static and dynamic analyses have been carried out to study the effectiveness of the aluminium shear link in the performance enhancement of buildings designed for gravity loads only.

Chapter 4 presents the results of the nonlinear static and dynamic analyses performed in Chapter 3. The conclusions and suggestions for the future studies are given in Chapter 5.

Chapter -2

LITERATURE REVIEW

2.1- General

Recent earthquakes around the world have shown that non ductile (gravity load-designed) reinforced concrete (RC) buildings are vulnerable to severe damage or complete collapse [27,4]. The primary deficiencies of these buildings are inadequate lateral strength and stiffness, and insufficient energy dissipation potential. Suitable local or global modification techniques can be adopted to enhance the seismic performance of these deficient RC buildings. There are many alternatives that can be adopted to upgrade these deficient buildings. Very often bracings are used to improve the lateral resistance of deficient buildings. However, there are a few disadvantages of bracing systems, such as the complete replacement of the buckled or damaged braces after a major earthquake may be labour intensive and expensive. Shear wall is also another alternative but it requires foundation up gradation. There are many hydraulics devices developed by different companies that can also be effectively used for seismic upgradation of existing buildings [32]. But these methods are generally costly to implement and hence makes them unsuitable for ordinary buildings [4]. Hence we need to have such an alternative which can be easily and economically integrated with the existing RC frame and can be repaired quickly after damage following a major earthquake. Metallic dampers (Aluminium shear link) provide such an alternative and excellent energy dissipation characteristic under the cyclic load and can be integrated easily with the existing RC frame [25, 27]. This chapter provide an over view of different energy absorbing devices that can be implemented in seismic upgradation of existing buildings and design of new buildings. It also provide the basic information required to accomplish the objectives discussed in chapter one.

2.2- Development of Energy Absorbing Devices

Many passive energy dissipation devices have been successfully implemented over the last three decades in different buildings to reduce earthquake induced vibration. Passive energy dissipation devices include Fluid viscous dampers, Viscoelastic dampers, Friction dampers, Yielding metallic dampers etc. Fluid viscous dampers represent a technology which was originally developed for military applications and has now been shown as an effective application for seismic hazard mitigation, either as elements of seismic isolation systems or as elements of seismic energy dissipating systems. Use of fluid viscous dampers for seismic energy dissipation on full size civil engineering structures began in 1993. The first application was on five buildings of the San Bernardino County Medical Centre Replacement Project [32]. A total of 186 dampers were fabricated for the San Bernardino project, each having an output force of 1456 kN, and a maximum energy dissipation rate per damper of 2.17 megawatts. Some recent application of fluid viscous dampers include Solomon R. Guggenheim Museum in New York USA, Mills Peninsula Hospital in Burlingame USA used Taylor Fluid Dampers to retrofit the existing building. To reduce wind induced vibration viscoelastic dampers were first used in World Trade Centre Tower in New York in 1969, but in seismic design these dampers were first used in a reinforced concrete navy office building in San Diego, US in 1995 [26]. These devices are very effective in reducing the detrimental effect of earthquake on building but are limited to important structures only because of cost of implementation [32].

Yielding steel devices were used in majority of application in Mexico and US [26]. The most documented retrofit of an RC frame in US is the Wells Fargo Bank Building in San Francisco, retrofitted in 1993 after Loma Prieta earthquake using seven Added Damping And Stiffness (ADAS) dampers mounted with chevron braces [26]. Pall Friction Dampers was first used in North America in 1987. The mammoth Boeing plant was retrofitted using Pall Friction Dampers in 1996. Later on many other buildings were retrofitted using Pall friction dampers in Canada, the U.S. and China [22].

2.3- Types of Energy Absorbing Devices

Numbers of innovative approaches have been proposed by different researchers to overcome the shortcomings in the conventional method of design. One of these approaches involves adding energy absorbers to a structure. Numerous types of energy absorbers can be implemented in a structure to safeguard the structures against desired level of earthquake. Various types of devices commonly used in practice include Viscous and Viscoelastic Damping Devices, Friction Damping Devices and Metallic or Yielding Damping Devices. A brief introduction of all these types of dampers is presented in this chapter with in depth study on Aluminium Shear Link as a Metallic Damper.

2.3.1- Viscoelastic Dampers

These types of dampers consists of several steel plates and a solid elastomeric (viscoelastic material) pad inserted in between as shown in Fig.2.1 (a) and (b). The main advantage offered by viscoelastic dampers is that they provide a considerable amount of stiffness which may be desired by designers depending on how a structure is intended to function. A brief summary of the work done on viscoelastic dampers by different researchers is presented here under.

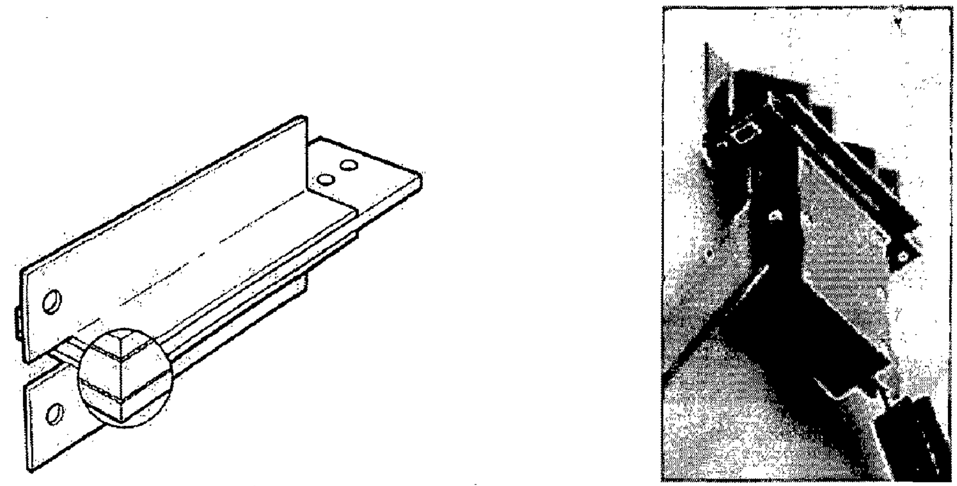


Fig.2.1- (a)Viscoelastic damper [18] (b)Viscoelastic damper installed using chevron braces [5]

Chang et al. (1998) made a comparison on the seismic performance of the viscoelastically damped structure and a conventionally design special moment resisting frame. From their study they proposed that viscoelastic dampers provide an alternative, safe and economic solution for earthquake resistance. From the analysis it is clear that inelastic hysteretic energy of the structure plays only a minor role in dissipating the seismic input energy. Most of the input energy will be dissipated by the viscoelastic dampers and the structure experience only minor yielding in design earthquake [7].

They provided a design methodology for seismic retrofit and for design of new structure with added viscoelastic dampers. As the properties of elastomer used in viscoelastic dampers are much affected by the variation in ambient temperature. Author provided a relation for the temperature rise within the damper during an earthquake excitation. The damping ratio of structure with viscoelastic damper may change from about 36% at 150^0 C to about 15% at 300^0 C [7]. From their study they concluded that viscoelastically damped structure can be designed conservatively as compared to the conventionally designed structure and viscoelastic dampers are effective in reducing the seismic vibration and ductility demand of structures.

Castellano et al. (2001) studied the use of viscoelastic dampers for seismic retrofitting work on the Fermi School building in Fabriano. Original building was constructed in fifty's as a reinforced concrete frame building. Building suffered a great damage following the

Umbrian-Marchein earthquake of 1997. Viscoelastic damper for seismic upgradation was probably used for the first time in this school building. Viscoelastic dampers were provided with the help of chevron braces as shown in the Fig.2.1 (b). The conventional braces were used at the first floor level and the braces with viscoelastic dampers at the second and third floor levels. In their study they provided a comparison of retrofitting scheme of conventional braces with viscoelastic dissipative braces. They used an energy oriented approach for the design of viscoelastic dampers.

A total of 33 viscoelastic dampers were applied in the retrofit. There are three types of dampers characterised by their stiffness values determine at 100% shear strain i.e. 19.8kN/mm, 14.8kN/mm and 7.4kN/mm respectively. The dynamic structural behaviour has been analysed through finite element model. From the time history analysis for the input ground motion it has been proved that the energy dissipated by the viscoelastic dampers is almost 50% of the total input energy and the remainder of it will be dissipated in the yielding of structural elements, in masonry infill damage etc. As great amount of energy is dissipated by the viscoelastic dampers so the demand exposed to building structural elements is less as compared to that of original structural configuration. Thus the safety of the structure is considerably increased for a particular level of earthquake.

Application of Viscoelastic Dampers

Viscoelastic dampers were first utilized in the Twin World Trade Centre Towers in New York in 1969 to reduce the wind induced vibrations [21]. In the 1980s, the Columbia Seafirst and Two Union Square Buildings in Seattle used dampers to reduce wind induce vibrations. In 1994 the Chien-Tan railroad station roof in Taipei, Taiwan utilized viscoelastic dampers to reduce wind induced vibrations [21]. A 13 story steel moment frame building in Santa Clara County was retrofitted with viscoelastic dampers in 1994 to reduce seismic vibrations [21]. A non-ductile concrete building in San Diego was retrofitted with viscoelastic dampers in 1996. Fermi School building in Fabriano was retrofitted for the seismic damage in 2001 by utilising viscoelastic dampers [5].

Advantages

- Viscoelastic dampers are activated at low displacement as that of fluid viscous dampers.

- Viscoelastic dampers do add some stiffness to the structure being retrofitted or newly designed.

Disadvantages

- Properties of elastomer provided are temperature dependent. This has the potential to be problematic if used in a region with variable climate.
- Possible debonding and tearing of viscoelastic material may take place.

2.3.2- Fluid Viscous Dampers

Viscous fluid dampers are one of the most commonly used passive energy dissipation devices currently being applied to structures to provide protection during seismic events [29]. As shown in Fig.2.2, these dampers consist of a hollow cylinder, fluid (often silicon based), and a piston [18].

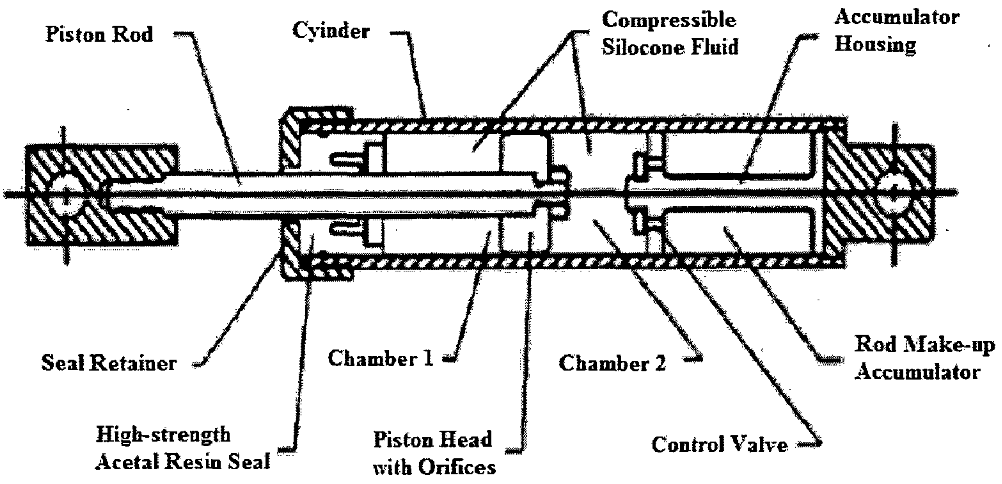


Fig.2.2- Viscous fluid damper [18]

During an earthquake, the piston moves through the chamber which contains the silicon-based fluid. The piston head either contains orifices (small cavities) or a small gap between it and the chamber through which the fluid can pass. When this occurs, a difference of pressure results along the sides of the piston head (greater pressure lies on the side from which the fluid flows). This pressure differential produces strong forces that resist the movement of the damper. Because of this, the fluid begins to flow at a high velocity through the piston head, resulting in an exorbitant amount of collisions between the particles that make up the fluid. So that the particles flow with high velocities, the fluid used inside the chamber actually has a low viscosity, though its name may imply otherwise [29]. These collisions produce a substantial amount of molecular friction that dissipates energy by generating heat. In this way, the damaging energy of an earthquake is absorbed by the dampers.

Tarassoly and Anderson (2004) carried out analytical investigations and compared retrofit schemes using Fluid Viscous Dampers (FVD) and conventional chevron braces for a nine story steel moment resisting frame building. This building was designed for the lateral load requirement of Universal Building code (UBC 1994). In their investigation this structure was subjected to a series of near fault earthquake records with and without retrofit schemes. It was concluded that the fluid viscous dampers are most effective when they are placed in the area of the building with large inter story drift. The dampers in this case provided supplemental damping of 20% of critical damping and 5% is the inherent damping of the structure i.e. a total of 25% of critical damping

Application of FVDs results in substantial reduction of the buildings inter-story drifts. However, the inter-story drifts are not reduced sufficiently for the structure to meet the life safety performance criteria of FEMA 356. The supplementally damped building requires upgrade of structural member sizes to meet the life safety performance criteria [31]. The columns and supporting foundations of the supplementally damped buildings receive lower axial loads and base shears which results in lower foundation costs. In retrofitting of existing structures and their foundations, this saving may prove to be quite substantial.

Chang et al. (2008) proposed displacement based design procedure to retrofit building using the fluid viscous dampers and then provided the experimental and analytical comparisons. They carried out shake table study on 2/3 scaled three story steel structure retrofitted with fluid viscous dampers. The experimental results were verified by performing the nonlinear time history analysis using SAP2000 v8.3 CSI2003. The damper properties inputted in the programme were those obtained from the damper properties test.

From their experimentation they proposed that viscous dampers have no influence on the stiffness of structure, it only contribute to damping of the structure. The maximum responses of the retrofitted structure are only 20-30% of those of the structure without dampers. The advantages of displacement based design methodology can also be accomplished by utilising fluid viscous dampers in the design of structures.

Application of FVD

Use of fluid inertial dampers for seismic energy dissipation on full size civil engineering structures began in 1993. The first application was on five buildings of the San Bernardino County Medical Centre Replacement Project, totalling 84,000 m^2 floor area. Located in a high seismic zone east of Los Angeles, California[32]. The Money Store National Headquarter located in Sacramento, California is probably the second application of fluid viscous dampers[30]. In this structure dampers are provided in newly constructed pyramid shaped 11-story building. A third significant application was the Woodland Hotel, located in the city of Woodland, California. This historic 4-story structure dates to 1927, and is of non-ductile concrete construction with a so called "soft" first story [32]. Similarly there are many other examples in which fluid viscous dampers have been utilised successfully [32, 30].

Advantages

- Compare to other types of dampers fluid viscous dampers enhance only damping properties with negligible increase in stiffness, thus base shear remains same.
- Fluid viscous dampers are activated at low displacement levels.

Disadvantages

- Properties of fluid viscous dampers are frequency and temperature dependent.
- Viscosity of fluid may vary with temperature.
- Possible fluid seal may leak due to intense shaking.

2.3.3- Friction Damping Devices

As the name implies friction dampers dissipates the energy through the friction i.e. rubbing of two surfaces via sliding. As compared to other types of energy absorbing devices there is a variety of energy absorbing devices that can be utilise for structural purpose. The simplest type of friction damper is the Slotted Bolted Connection [11]. PALL's devices also form an important type of friction damper, there are again different devices manufactured by different industries which dissipate the energy through the action of friction, one such device available in the market is Sumitomo Friction Damper.

Grigorian et al. (1993) proposed the Slotted Bolted Connections (SBC's) to dissipate the energy through the action of friction under the effect of cyclic loading. The earliest investigation of the slotted bolted connections as energy dissipater dates back to 1976 when a series of experiments were carried out at San Jose State University (SJSU) [11].

They proposed two types of arrangement in their study. One is with brass insert plate and one without. From their experimental and analytical results they concluded that the slip force in SBC'c where friction occurs between the like steel plates has been shown to vary significantly. On the other hand in SBC'c where because of brass insert plate friction occurs between brass and steel, slip force has been shown to remain relatively constant over the range of interest. In view of these results SBC's with steel on the brass frictional surfaces posse's significant advantages in terms of efficient energy dissipation and can be used as an alternative choice for energy dissipation in seismic design and retrofit of the structures. Following figure shows details of the SBC'c proposed by the authors.

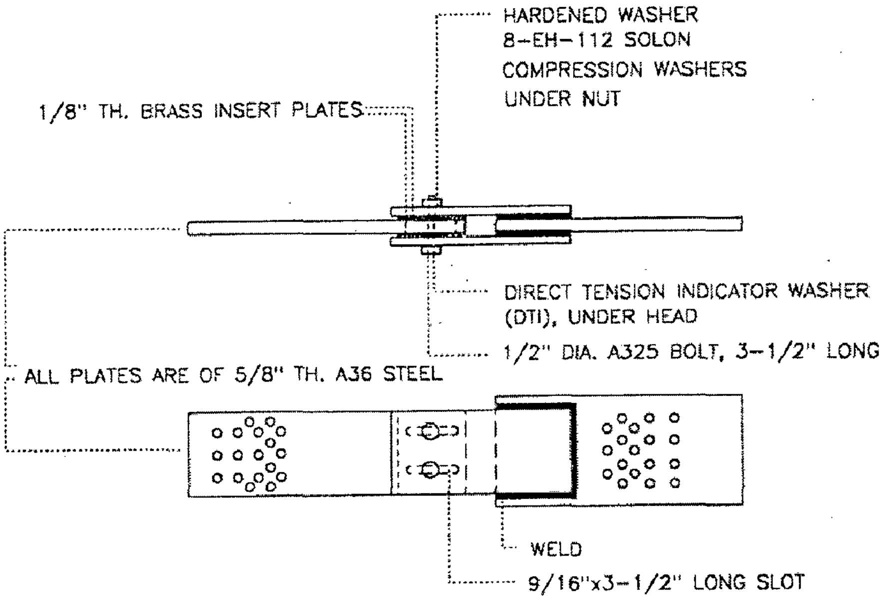


Fig.2.3- Details of slotted bolted energy dissipater [11]

Pall and Pall (2004) devices consists of series of steel plates, which are specially treated to develop very reliable friction. These plates are clamped together and allowed to slip at a predetermined load (Slip load). Their performance is reliable, repeatable and they possess large rectangular hysteresis loops. Their performance is independent of velocity and hence exerts constant force for all future earthquakes, design-based earthquake (DBE) or maximum credible earthquake (MCE). In a typical undamped structure, the inherent damping is merely 1-5% of critical. With the introduction of Pall Friction Dampers, structural damping of 20-50% of critical can be easily achieved [22]. Schematic view of Pall friction damper used in cross bracing is shown below.

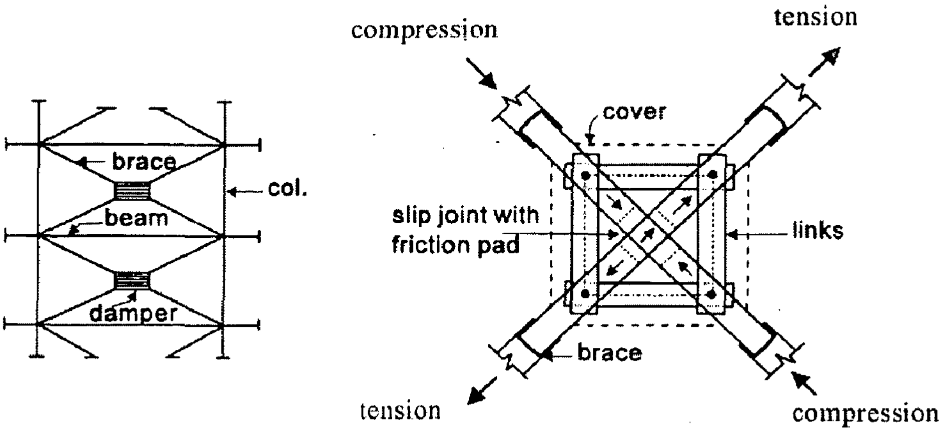


Fig.2.4- Installation of Pall friction damper in conjunction with cross bracing [18]

These union elements ensure that when the axial load acting along the braces is high enough to initiate the sliding on the tension diagonal then the compression diagonal will slide an equal quantity in the opposite direction. Thus dissipating the energy through friction under repetitive sliding over the friction hinges.

Pall's friction damper was first used in North America in 1987 [22]. Then after this the Pall's damper found an increasing application worldwide. This damper has been used in seismic protection of more than 80 buildings in Canada and the U.S. The Mammoth Boeing Plant is the world largest building in volume[22] has used Pall's Friction Dampers for its seismic upgradation. Other buildings which include the application of friction dampers are Moscone West Convention Centre, San Francisco, USA; Seismic retrofit of Boeing Development Centre, Cafeteria and Auditorium Buildings, Boeing Field, Seattle, WA, USA; Concordia University Library Building, Montreal, Canada and many other important building have been retrofitted by using Pall's friction damping devices. Authors concluded that the Pall Friction Dampers provide a practical, economical and effective approach for the performance-based design of new and retrofit of existing structures to resist major earthquakes.

Advantages

- Compared to the devices based on yielding of metals, friction dissipaters posses a great capability of absorbing energy. However, this feature fails when the sliding surfaces wear.
- Friction dissipaters are not affected by fatigue effects.
- They are not active during service loads and wind. Hence, no possibility of failure due to fatigue before an earthquake

Disadvantages

- Durability of the frictional dissipaters is a controversial issue, mostly due to the high sensitivity of the coefficient of friction to the conditions in the sliding surfaces.
- Friction dampers fail to offer long-term reliability as it is likely that the coefficient of friction of the plates will diminish after several use.

2.3.4- Metallic Dampers

The reliable yielding properties of mild steel and aluminium have been explored in a variety of ways for improving the seismic performance of structures. These include ADAS (Added Damping And Stiffness devices), TADAS (Triangular Added Damping and Stiffness devices), Aluminium shear link etc. One of most effective mechanisms for dissipating input energy in the structure during an earthquake is plastic deformation of metals. During earthquakes, the inter-story drifts cause movement of the upper end of ADAS & TADAS devices or shear links relative to the lower end. This causes yielding of metallic plates of the damper devices and as a result, the energy is dissipated. Energy dissipation is primarily concentrated at these especially design elements which are likely to suffer only localized damage in severe earthquakes. Once these devices get damaged following the major earthquake it can be easily replaced without disturbing the functionality of building. Summary of the works contributed by different authors on metallic dampers are presented here.

Alehashem et al. (2008) studied the behaviour of metallic dampers which include Added Damping and Stiffness devices (ADAS) and Triangular Added Damping and Stiffness devices (TADAS). They studied behaviour and performance of a ten story steel structure equipped with ADAS and TADAS metallic dampers and compared with conventional earthquake resisting steel structures such as Concentrically Braced Frame (CBF), Chevron Braced Frame and Eccentrically Braced Frame (EBF) systems. To assess the performance of the structures, nonlinear time history analysis was carried out. In their study they compared induced base shear force in the structures equipped with metallic dampers and the conventional structure. Induced base shear force in Elcentro earthquake equipped with ADAS and TADAS dampers is less compared to EBF, CBF and Chevron systems (about 50%, 66% and 76% respectively). Drift recorded in all the stories was also small due to which non structural damages are considerably reduced.

Vargas and Bruneau (2009) have carried out analytical and experimental works for the design of earthquake fuses using Buckling Restrained Bracings (BRBs). These BRBs also serves as metallic dampers. They have developed a systematic procedure for design of multi-storey building using BRBs. The procedure can be used for retrofitting of existing building with slight modification. In their study, one of the Structural Engineering Association of California (SAC) model building was selected as the prototype for the experiment. The model building was provided with BRBs in the diagonal member with the help of removable gusset plate connection to facilitate easy replacement of the yielded bracings. Seismic response of the system is then evaluated by nonlinear time history analysis using SAP2000 as well as the experimental setup to verify the performance of the BRBs. It was observed that the beams and column performed elastically while most of the seismically induced energy was dissipated through yielding of the BRBs.

2.3.4.1- Shear Link with Chevron Braces

Rai and Wallace (1998) have also worked on the same principle of dissipating the earthquake energy by utilising metallic dampers, but the element what they have used in dissipating the earthquake energy was different. They have used an aluminium shear link which can be viewed as a damping device which dissipates the earthquake induced energy through inelastic deformations (metallic yielding).

An I-shaped aluminium beam is sandwiched between the tops of the diagonal braces and a beam from the floor above, as shown in Fig.2.5. The aluminium beam is designed to yield in shear mode at a lateral force less than that required to buckle the compression brace, eliminating the severe loss of storey strength and stiffness due to compression brace buckling. An obvious advantage of this bracing system is that the floor beam continues to carry gravity loads even after link collapse.

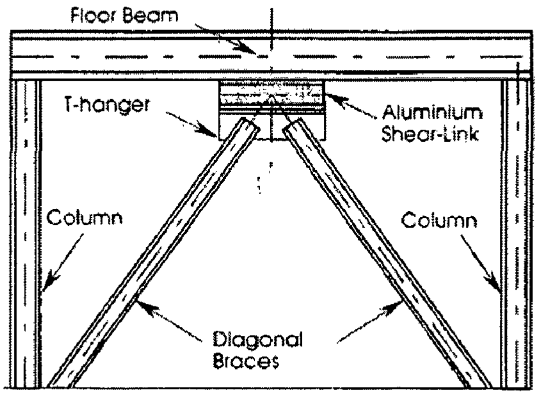


Fig.2.5- Schematic diagram of frame with aluminium shear link and chevron braces [25]

Author proposed a design methodology for a Shear Link Braced Frame (SLBF) system utilising aluminium shear link as a metallic damper. They made a comparison of SLBF with conventional brace frame system. A chevron type Ordinary Chevron Braced Frame (OCBF) system for an office building is designed in accordance with Uniform Building Code (UBC) 1994 and, for the same building a SLBF is designed with suitable assumptions. Their performance under a static pushover and dynamic nonlinear analysis are then investigated.

The failure mechanism and the location of plastic hinges at collapse under static pushover analysis are shown in Fig. 2.6 (a)-(b). The Fig. 2.6 (a) shows that SLBF is almost as stiff as the OCBF before yielding of the first-storey shear-link at about 60 per cent of the design base shear. More energy is dissipated in yielding of shear link whereas in chevron brace frame system primary members dissipates the energy through formation of hinges as shown. Author concluded from their study that aluminium shear-link demonstrated excellent stiffness and energy dissipative capacity over a wide range of strains. Stable hysteretic loops are observed for aluminium shear link whereas for chevron braced frame pinched and degraded hysteretic loops are observed. The aluminium shear-links proved to be very effective and reliable in dissipating large amounts of seismic energy thus avoid the primary members of the structure to go in inelastic range.

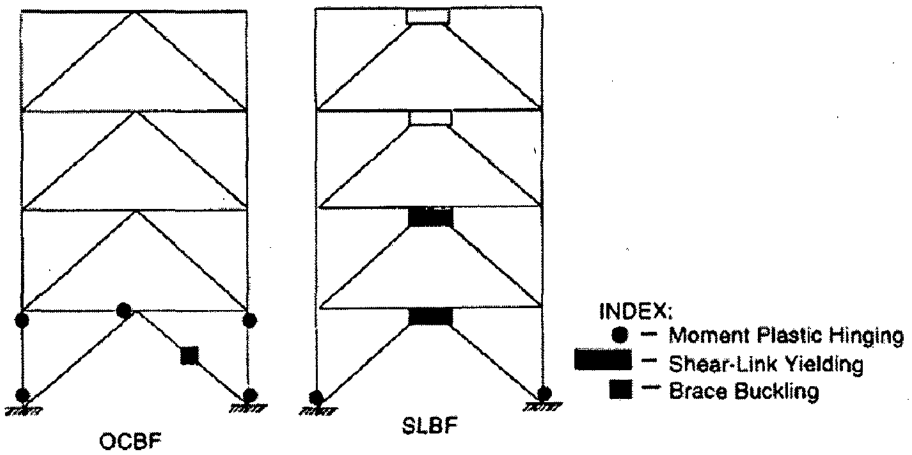


Fig.2.6- (a) Location of inelastic activities in the OCBF and SLBF at the collapse

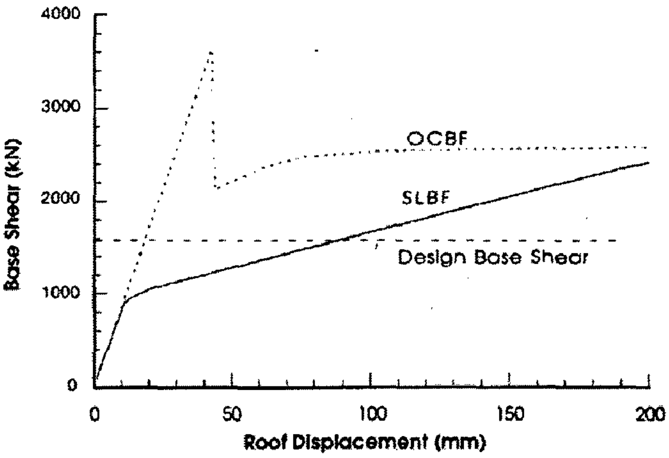


Fig.2.6- (b) Plot of base-shear v/s roof displacement for the pushover analysis [25]

Rai and Sahoo (2008) carried out analytical work for strengthening of existing building using aluminium shear link as an energy dissipaters. They suggested a performance based design methodology based on energy balanced concept for seismic strengthening of existing buildings. In their study they made a comparison between the existing non ductile Reinforced Concrete (RC) building and the retrofitted buildings. RC frames with ground-story columns strengthened by diagonal brace only are termed as partially-strengthened (PS) frames, whereas frames strengthened with aluminium panels i.e. shear links as energy dissipation devices in addition to diagonal brace of ground-story columns are termed as fully-strengthened (FS) frames. Seismic performance of both existing and strengthened frames is evaluated by nonlinear static and time-history analyses using a computer program SAP2000 (CSI 2006) to validate the proposed design methodology.

Both the RC and PS frames did not show any notable hysteretic response due to their premature failure at very low drift levels. They concluded that aluminium panels contribute significantly in sharing the load, thus the seismic demand on existing RC members reduces considerably. The energy dissipation capacity of the RC frame strengthened with aluminium shear panels increases by about 8 times that of the RC frame with strengthened columns only. Hence, energy dissipation potential of non-ductile open ground-story RC frame was significantly improved using aluminium shear panels as energy dissipation devices.

Sahoo and Rai (2010) proposed seismic strengthening technique using aluminium shear links as energy dissipating devices to enhance the lateral strength, stiffness, and energy dissipation potential of the existing reinforced concrete frames. They carried out an experimental study on a reduced-scale non ductile RC frame and the same frame strengthened with aluminium shear link to investigate the effectiveness of the strengthening system under constant gravity loading and gradually increasing reversed cyclic lateral displacements. The snap shot of their experimental setup is as shown in Fig.2.7 below.

The performance of the test specimens was evaluated in two phases. First, slow cyclic testing was carried out on the strengthened specimen and later, the performance of RC (bare) specimen was evaluated under the same loading conditions. They observed that the initial fundamental period reduced significantly as compared to RC (bare) frame. The initial equivalent damping at small displacements for the strengthened structure increased to about 2.0 times as compared to RC (bare) frame. The total lateral load shared by the shear link is found to be significantly. The lateral load shared by the shear link was nearly constant (about 95%) up to 1.0% story drift level, which gradually reduced to a value of 45% at a story drift of 3.5%.

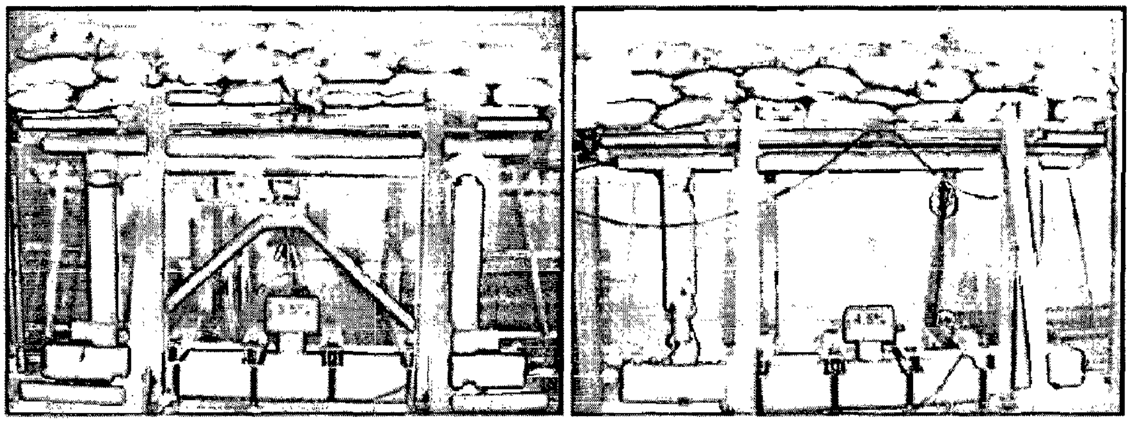


Fig.2.7- Test arrangement a) Strengthen frame b) RC (bare) frame [27]

From their study they concluded that the Strengthening of a non-ductile RC frame with an aluminium shear link as an energy-dissipation device significantly enhanced the lateral strength, stiffness, damping and energy dissipation potential. The load–displacement response for the strengthened frame consisted of full and stable hysteresis loops with significant post-yield strain-hardening behaviour without any degradation in strength and stiffness. Nearly two-third of the total energy dissipation of the strengthened frame was contributed by the aluminium shear link thus reducing the demand on other primary members of the structure and avoiding the damages to those members.

Peter Dusicka et al. (2010) carried out experimental investigations on the cyclic performance of shear links of various grades of plate steel. Steel plate offers great advantage as the shear links required for a particular design may not be available in rolled shapes. However, rolled sections are not always suitable and additional design freedom of geometry and material selection can be realized through the use of steel plates [9]. In their experimentation they studied the cyclic behaviour of five different types of shear links made up of steel grade ranging from 100 to 485 MPa yield strength. The links were categorized in two distinct groups, first one is conventionally designed links using higher grade steel with web stiffeners and second one is links using low yield point steel and without any web stiffeners.

The cyclic response was assessed by plotting the shear force versus the plastic shear deformation. Figure2.8 below shows the force-deformation response for shear link made up of 100 MPa grade steel plate. In all the cases, the cyclic response exhibited stable hysteretic behaviour without resulting in any pinching or degradation.

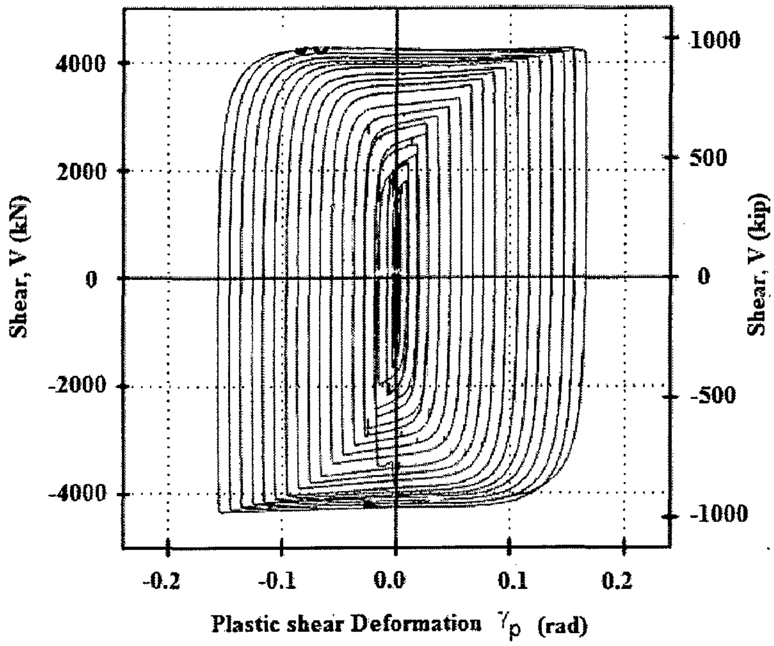


Fig.2.8- Link shear force versus plastic shear deformation [9]

The effectiveness of the shear links as hysteretic energy dissipating device was evaluated by calculating the efficiency factor as the ratio of E_{hyst} / E_{max} as shown in Fig.2.9 below. The theoretical energy E_{max} represented the maximum energy that could be achieved as bound by a rectangular area between the maximum and minimum values in each cycle. Hence, a hysteretic efficiency factor of 0% would represent an elastic behaviour, while a factor of 100% would represent an infinitely stiff elastoplastic response. They further concluded that the links with lower grade steel were somewhat more effective in dissipating energy. The performance of shear links tested with and without web stiffeners was different in terms of mode of failure.

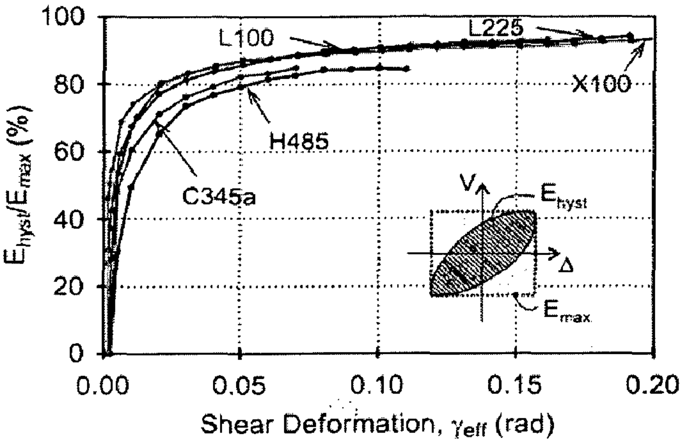


Fig.2.9- Hysteretic efficiency comparison of different links.[24]

Durucan and Dicleli (2010) have carried out an elaborate work on retrofitting of building using aluminium shear link. He worked on the same line as that of Rai and Wallace [25] and developed a performance based design methodology for retrofitting of building using aluminium shear link. In addition to design methodology he also made a comparative study for different configuration of the shear links that can be incorporated in retrofitting of RC non ductile buildings to select the best possible alternative as shown in Fig.2.10 below. He made a comparison of building retrofitted with aluminium shear link, building retrofitted by addition of shear wall and an original RC frame building for different performance levels i.e. immediate occupancy, life safety and collapse prevention. The proposed retrofitted scheme has been validated by performing a series of non linear pushover analysis and nonlinear time history analysis. To study the effectiveness of their proposed retrofitting scheme two existing buildings were selected. The first building is a two-storey school building built in 1987 in compliance with the 1975 Turkish Seismic Design Code. The second building is a six-storey RC office building built in 1954.

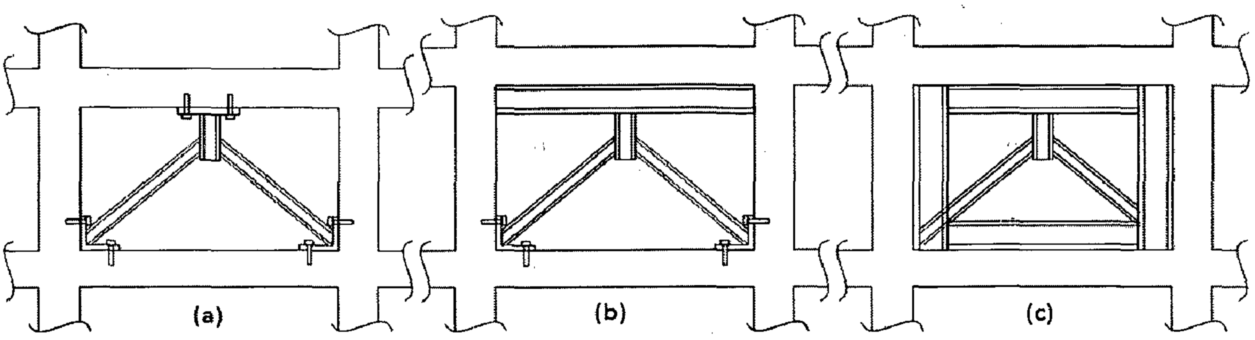


Fig.2.10- Different configuration of shear link with RC frame[8]

Based on the study the authors proposed that the interstorey drift demands severely exceed the capacities for all performance levels. For the building retrofitted with the shear links however, the interstorey drifts demands are smaller than the corresponding capacities for all the performance levels considered. Nevertheless, this is not the case for the building retrofitted with shear wall. The nonlinear static pushover and nonlinear time history analysis results revealed that proposed retrofitting scheme can efficiently alleviate the detrimental effects of earthquake on the buildings. The building retrofitted with the shear links have more stable lateral force-deformation behaviour with enhanced energy dissipation capability than that of one retrofitted with shear wall as shown in Fig.2.11. For immediate occupancy performance level (small intensity ground motions), the maximum interstorey drift of the building retrofitted with the proposed retrofitting scheme is comparable to that of the one retrofitted with shear wall. But, for life safety and collapse prevention performance levels (moderate to high intensity ground shaking), the maximum interstorey drift of the building retrofitted with the shear links is considerably smaller than that of the one retrofitted with shear wall.

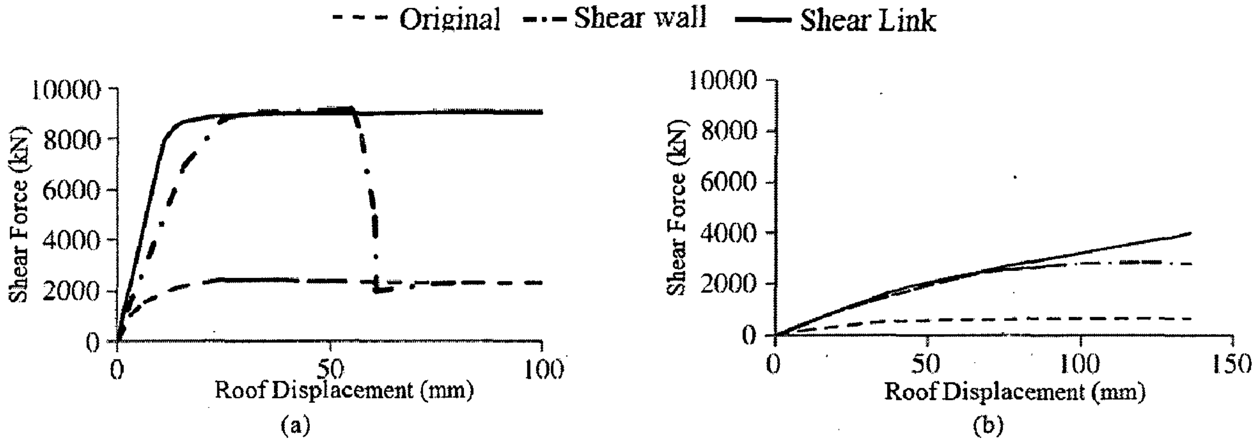


Fig.2.11- (a) Base shear as a function of the drift at the top-storey level for the school building in the x-direction. (b) Base shear as a function of the drift at the top-storey level for the office building in the x-direction. [8]

2.4- Why to use Metallic Dampers

As discussed above there are various alternatives that can be implemented easily in design of new structure or retrofitting of existing structure to dissipate earthquake induces energy. Each system has its own merits and demerits. Among passive energy dissipation systems, metallic dampers have some advantages over other types of energy dissipation devices: no complicated technology is needed to manufacture them, they can easily be integrated in structures, and they show stable behaviour in earthquakes and no environmental (temperature, humidity, etc.) factors affect their performance. These dampers, increase damping and stiffness of structures and increase energy dissipation capacity in them. Adding metallic dampers to the structures can cause concentration of energy dissipation in the dampers. After earthquakes, dampers can easily be replaced for strengthening structure for future earthquakes.

2.5- Principle of Metallic Dampers

One of the most effective mechanisms available for the dissipation of energy input to a structure during an earthquake is through the inelastic deformation of metallic substances [25]. Seismic design, for conventional structures, relies on the post yield ductility of the structural frame for energy dissipation, indicating permanent deformation or damage to the structures. However, implementing completely separate metallic dampers into the structural frame with the intent of dissipating large portions of seismic energy can protect the structures from earthquake damage. During the earthquake story drift causes the relative movement of the both ends of the metallic dampers (shear link). This causes the yielding of metallic plates of the dampers and as a result energy is dissipated. Energy dissipation demand is concentrated over these specially design link elements to safeguard the primary members of the structure. These metallic dampers (shear link) are often termed as metallic fuses. These shear link plays an important role in limiting the forces that are transferred to primary member of the structure and the damages are concentrated only on these dampers which can be easily replaced and can be repaired quickly following a major earthquake.

2.-6 Concluding Remarks

Different types of energy absorbing devices have been studied in this chapter. These energy dissipation devices are broadly classified as viscoelastic dampers, viscous fluid dampers, friction dampers and metallic dampers. A brief summary of all types of energy dissipation devices have been provided with reference to their behaviour in building performance and their past applications. Metallic dampers have been studied in details with particular reference to aluminium shear link with chevron braces. Aluminium shear link with chevron braces is an important device in retrofitting the buildings designed for gravity load only to improve their strength, stiffness and energy dissipation characteristics. A comparative study has also been provided on the basis of merits and demerits of different energy absorbing devices.

MODELLING AND ANALYSIS

3.1- Building Description

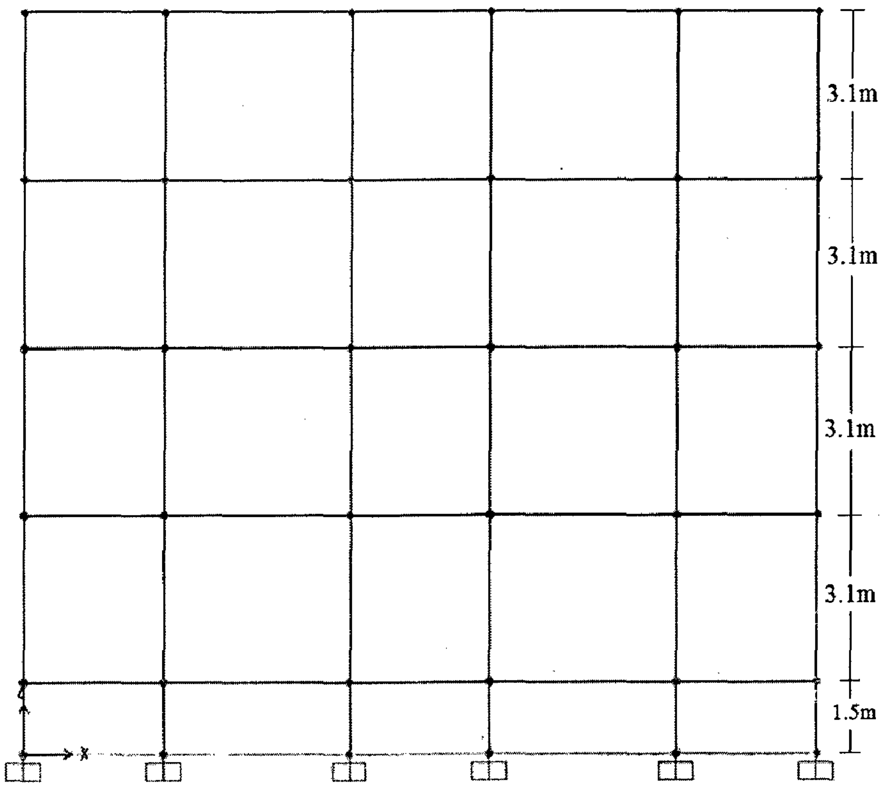
The building configuration selected is a representative office building that is common in Indian seismic zones IV as per IS:1893-2002 (part-1) located in medium soil region. A symmetric floor plan and floor levels of equal height were used to avoid any irregular behaviour that might lead to complexities in the interpretation of the response. It was found deficient for the lateral seismic load of that particular earthquake zone. Hence it needs to be strengthened by providing shear links. Two buildings are selected for the performance evaluation and strengthening by using aluminium shear links. These two building are designated as Bldg-1 and Bldg-2 here after. Bldg-1 is a four storey and Bldg-2 is a six storey office building. Details of these buildings are provided here.

Details of Bldg-1 and 2 selected and load assumed are as follows:

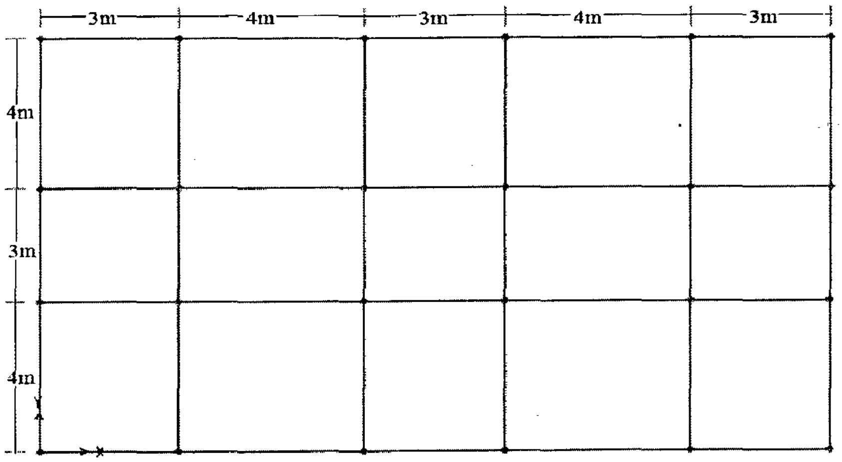
Seismic Zone	IV
Soil type	Medium soil
Importance factor	1.0
Response reduction factor	3
Number of storey	4
Grade of concrete	M-20
Grade of steel	Fe-415
Slab thickness	150 mm
Exterior wall thickness	230 mm
Interior wall thickness	120 mm
Dead load of floor finish	1 kN/m2
Dead load of roof treatment	2 kN/m2
Live load of floor	3 kN/m^2
Live load of roof	1.5 kN/m^2
Height of storey	3.5 m
Depth of foundation	1.5 m
Height of plinth	1.0 m
Height of parapet wall	1.0 m

Unit weight Masonry 18 kN/m^2

Isolated type of foundation is provided for all the columns.



b) Elevation



a) Plan

Fig.3.1-Details of Bldg-1

Table 3.1- Schedule of reinforcement for columns (Bldg-1)

Designation	Size	Longitudinal Reinforcement	Transverse Reinforcement	Remark
Column-C1	350 X 350	4 # 16 mm + 4 #12 mm	#8 @ 190 mm c/c throughout	All columns are of same size

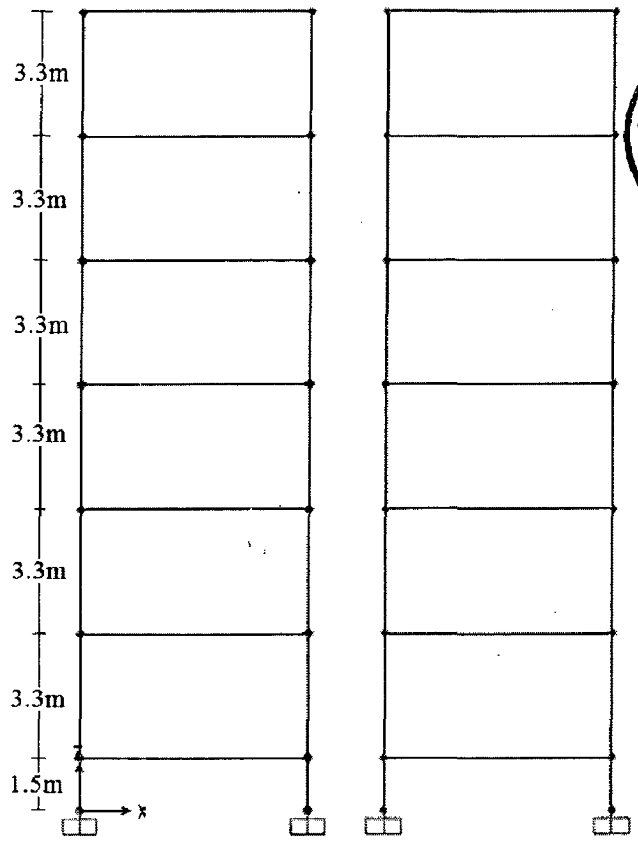
Table 3.2- Schedule of reinforcement for beams (Bldg-1)

Designation	**Size**	**Longitudinal Reinforcement**		**Transverse Reinforcement**		**Remark**
		At Support	**At mid span**	**At Support**	**At mid span**	
B1-(3m)	350 X 230	2 # 16mm	2#16mm	# 8 mm @250 mm c/c throughout		Ground floor and roof level beams
B2- (4m)	350 X 230	2 #16mm + 1 #12mm	2#16mm	# 8 mm @250 mm c/c throughout		
B3- (3m)	350 X 230	2 #16mm	2#16mm	# 8 mm @250 mm c/c throughout		All intermediate floor beams
B4-(4m)	350 X 230	2 #16mm + 2 #16mm	2#16mm + 1 #12mm	# 8 mm @250 mm c/c throughout		

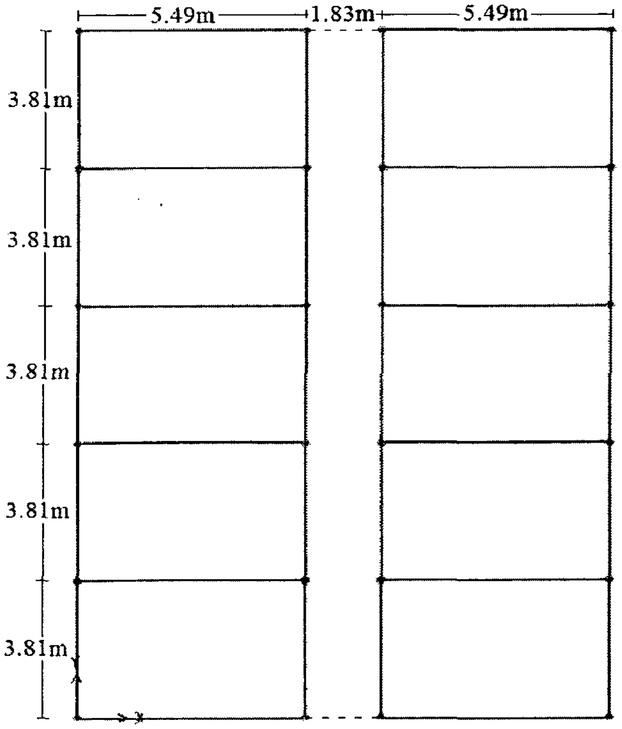
Loading data for Bldg-2 is same as that of Bldg-1. Only difference is that Bldg-2 is a six storey office building. Plan and elevation of the building is as given in Fig.3.2.

Table 3.3- Schedule of reinforcement for columns (Bldg-2)

Designation	**Size**	**Longitudinal Reinforcement**	**Transverse Reinforcement**	**Remark**
Column-C1	400 X 400	4 # 20 mm + 4 #12 mm	#8 @ 200 mm c/c throughout	External column in ground and first floor
Column-C2	400 X 400	4 # 20 mm + 4 #16 mm	#8 @ 200 mm c/c throughout	Internal column in ground and first floor
Column-C3	400 X 400	4 # 16 mm + 4 #12 mm	#8 @ 200 mm c/c throughout	External column in above storey
Column-C4	400 X 400	4 # 20 mm + 4 #12 mm	#8 @ 200 mm c/c throughout	Internal column in above storey



b) Elevation



b) Plan

Fig.3.2- Details of Bldg-2

Table 3.4- Schedule of reinforcement for beams (Bldg-2)

Designation	**Size**	**Longitudinal Reinforcement**		**Transverse Reinforcement**		**Remark**
		At Support	**At mid span**	**At Support**	**At mid span**	
B1-(3.81m)	350X230	3 # 12mm	2 #12mm	# 8 mm @230 mm c/c throughout		Ground floor beams
B2-(5.49m)	400X230	3 # 16mm	2 #12mm	# 8 mm @230 mm c/c throughout		Ground floor beams
B2-(3.81m)	400X230	2 # 16mm	2 #12mm	# 8 mm @200 mm c/c throughout	# 8 mm @230 mm c/c throughout	Intermediate floor beams
B4-(5.49m)	450X230	2 # 20mm + 2 # 16mm	2 # 20mm + 1 # 16mm	# 8 mm @110 mm c/c throughout	# 8 mm @180 mm c/c throughout	Intermediate floor beams
B5-(3.81m)	350X230	2 # 16mm	2 #12mm	# 8 mm @230 mm c/c throughout		Roof level beams
B6-(5.49m)	400X230	2 # 16mm + 1 # 16mm	2 # 16mm + 1 # 12mm	# 8 mm @130 mm c/c throughout		Roof level beams

3.2- Modelling

The building has been modelled as a three dimensional frames using the commercially available software SAP2000 v14.2.4. The modelled building involves modelling of beam, column, braces and modelling of shear link. Shear link has been modelled as a Plastic Wen link element.

3.2.1- Modelling of Frame Elements

Frame elements like beams and column have been modelled as a line frame elements with six degrees of freedom at each node. Slab has not been modelled but its effect of inplane rigidity has been simulated by assigning floor diaphragm constraint at each floor level. Yield line pattern for slab has been assumed as given in IS:456-2000. Live loads and dead loads over the slab have been distributed to the adjacent supporting beams as per this yield line pattern in the form of trapezoidal and triangular loads. Dead load of masonry has been assigned to the respective beams as a uniformly distributed load. Base of the building is assumed as fully restrained neglecting the effect of soil structure interaction.

3.2.2- Modelling of Chevron Braces with Shear Link

SAP2000 facilitate the modelling of shear link by using Plastic Wen link element which has an elastoplastic shear force displacement hysteresis. Rai and Wallace [25] have carried out extensive experimental work on aluminium shear links to assess its performance under cyclic loading. Initial stiffness, effective stiffness, yield shear force, and post yield ratio is required to define the Plastic Wen link element in SAP2000. These properties have been provided by the Rai and Wallace through their experimentation. An idealised bilinear representation of the aluminium shear link is as shown in Fig.3.3[25]. The first stiffness is taken as the secant stiffness corresponding to a shear strain of 0.002 at which the general yielding of the shear link specimens was observed. At this stage the average shear stress was 0.722 times the tensile yield strength i.e. 35.2 MPa and can be taken as the yield shear stress[25]. This value is then multiplied by the horizontal web area A_g of the shear link to give R_y i.e. yield shear force. The second stiffness can be easily computed from the fact that the maximum shear force allowed at the shear strain of 0.2 marks the end point of the second linear branch of the primary curve as shown in Fig.3.3. Steel braces have been modelled using steel beam element with releases for major and minor moment at each node. Brace elements are connected at the intersection of beams and column elements so as to form concentric joints.

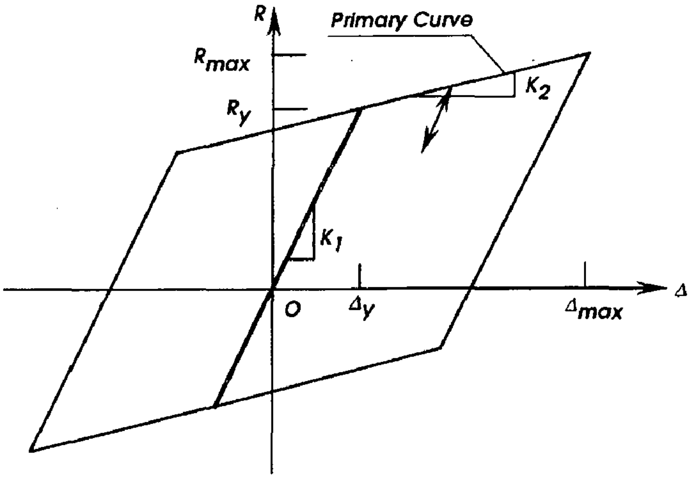


Fig.3.3- Bi-linear hysteretic model for shear-link [25]

3.3- Assigning Non-linearity to Frame Element

To carry out nonlinear analysis we have to define the force deformation relationship for all the members beyond its linear range. SAP2000 facilitate the assignment of nonlinearity in the form of auto hinges and user defined hinges based on FEMA-356[10]. Auto hinges assign the nonlinearity based on the capacity of that particular member. Auto hinges have been used to assign non linearity to the frame elements. Nonlinearity in beams have been considered by assigning flexure hinges (M3) at both ends where as in case of column P-M2-M3 hinges have been assigned at both the ends. In braces P-axial hinges have been assigned at both the ends.

3.4-Analysis

Procedure employed in dissertation for seismic retrofitting of the gravity load design building involves response spectrum analysis and nonlinear static pushover analysis. Nonlinear dynamic time history analysis also performed to study the effectiveness of the shear links in the retrofitted system.

3.4.1- Response Spectrum Analysis

Response spectrum analyses have been performed on the original building as well as the retrofitted building to study the linear elastic behaviour of the structure as per the IS:1893-2002(part-1). In general, linear procedures are applicable when the structure is expected to remain nearly elastic for the level of ground motion that can be expected at that particular site. Due to the strong ground motion most of the structural member goes into inelastic range to dissipate the earthquake induced energy, hence for the desired performance objective only response spectrum analysis results are not reliable. In order to know the behaviour of the structure nonlinear static (pushover) analysis has been performed on both original as well as retrofitted structure.

3.4.2- Nonlinear Static Analysis

A nonlinear static analysis also known as pushover analysis is performed by subjecting the above structure to a monotonically increasing lateral loads, representing the inertial forces which would be experienced by the structure when subjected to ground shaking. Parabolic distribution as suggested by IS:1893-2000 (part-1) is used to distribute the lateral load along the height of building. Under incrementally increasing loads various structural elements may yield sequentially. Consequently, at each event, the structure experiences a loss in stiffness and strength. Using a pushover analysis, a characteristic non linear force-displacement relationship can be determined.

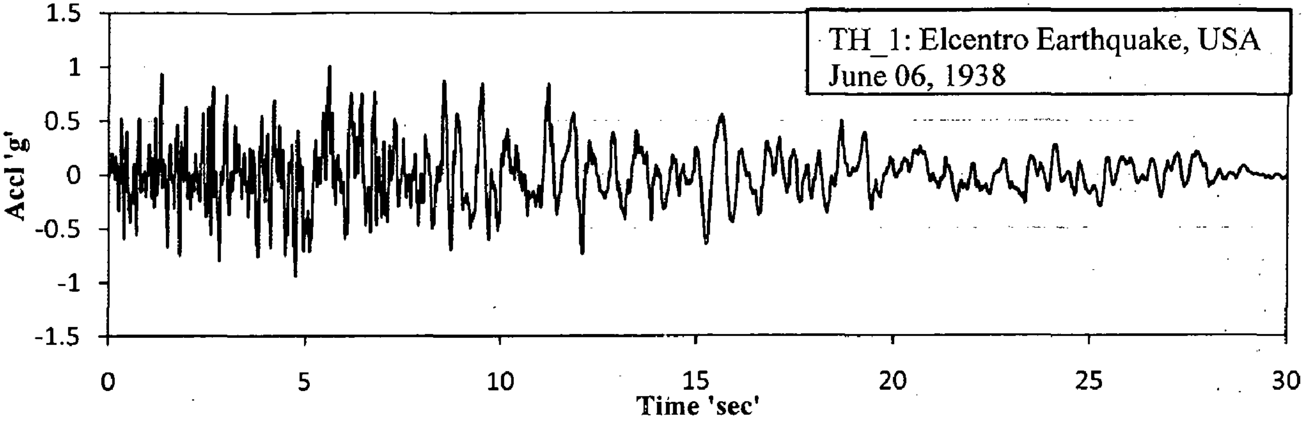
3.4.3- Nonlinear Time History Analysis

Nonlinear time history analysis is the most reliable and realistic and considers the structural behaviour as a whole in the analysis. This method is realistic, sophisticated, time consuming and also highly sensitive to small changes in assumptions. Time history records of the several earthquakes which have caused severe damage to the buildings are considered in the present study. Time histories are selected based on their predominant frequency. One record each for low frequency, medium frequency and high frequency earthquakes are selected for time history analysis. Motions are made compatible to the IS-1893:2002 response spectra corresponding to 5% damping for medium soil using algorithm developed by Mukherjee and Gupta [20]. A wavelet based iterative procedure has been used by the author to modify a recorded accelerogram so that it becomes compatible with a given design response spectrum. The objective behind this iterative procedure is to modify a recorded accelerogram such that the temporal variations in its frequency content are retained in the synthesized accelerogram. The proposed procedure is computationally efficient as decomposing of the recorded accelerogram is carried out only once and since an acceptable matching with the target spectrum is achieved after about 6-8 iterations.

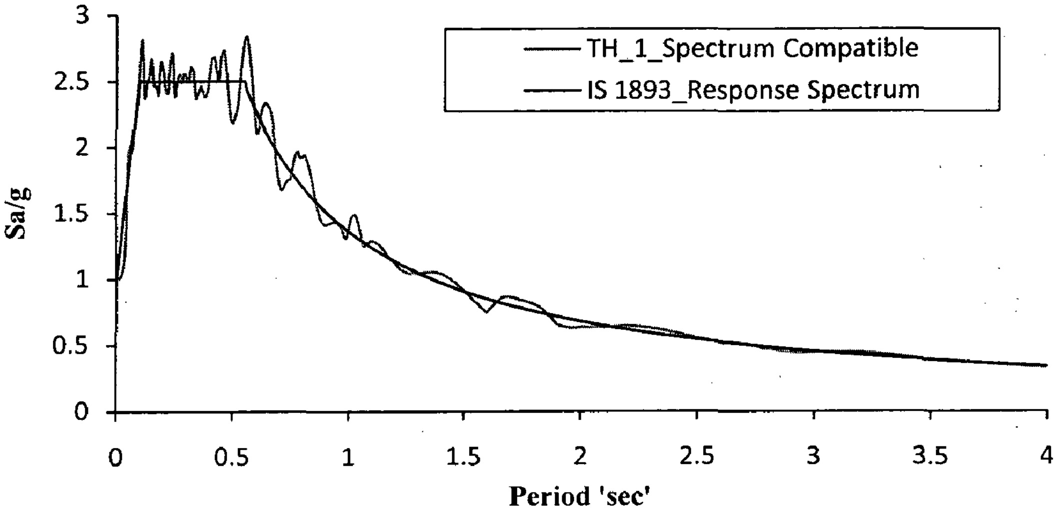
In the present study, ground motions are applied in both the principal directions independently. Figure 3.4-3.5 below show the compatible time history records and their response spectra for 5% damping level.

Table3.5- Details of earthquakes considered

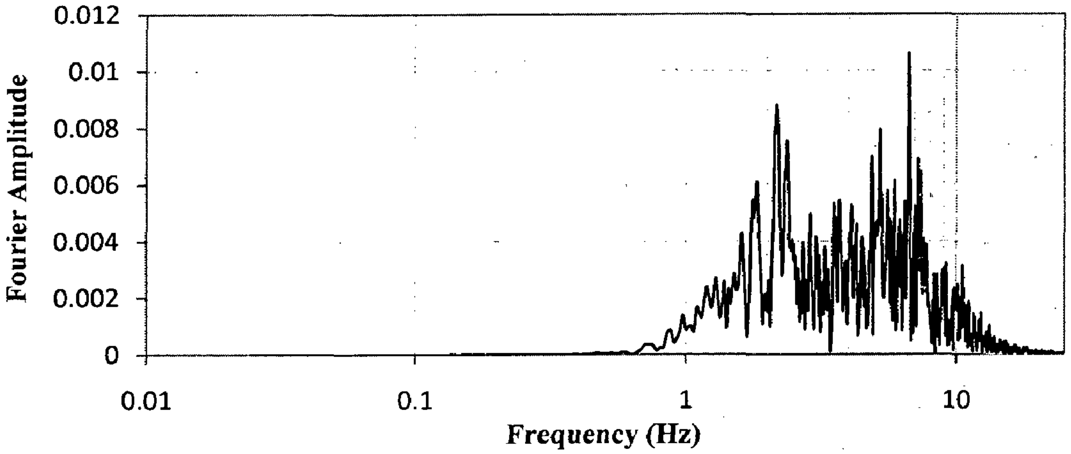
Sr. No.	Earthquake	Magnitude	Predominant Frequency (Hz)	PGA (g) (Spectrum Compatible)	Remarks (Frequency)
1	1938 Imperial Valley (Elcentro, USA)	5.00	6.67	0.24	High
2	1994 Northridge (California)	6.69	2.26	0.30	Medium
3	1995 Kobe (Japan)	6.90	0.59	0.26	Low



(c) Time history

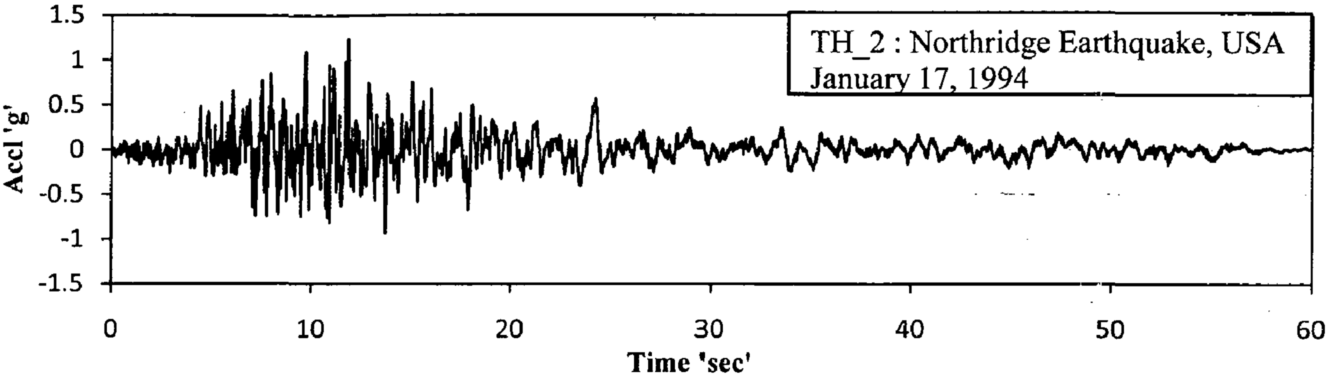


(b) Response spectra

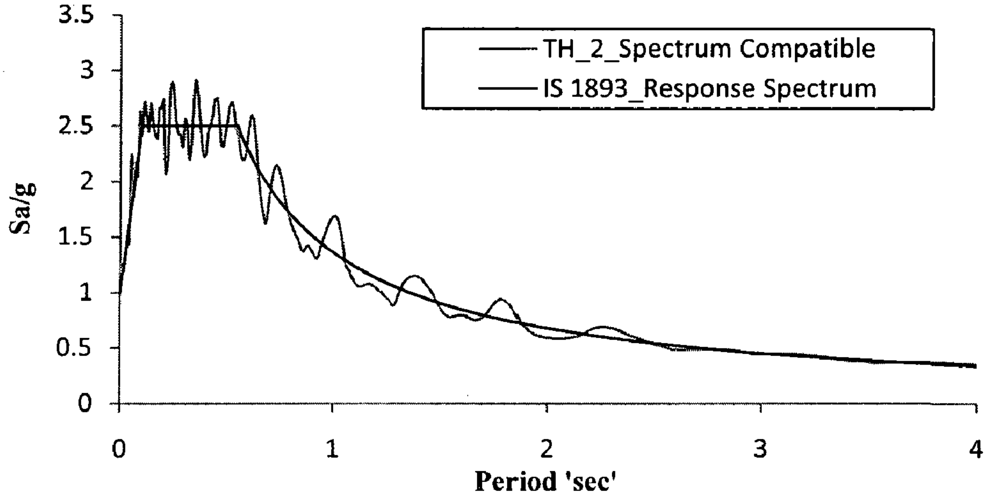


(a) Fourier spectra

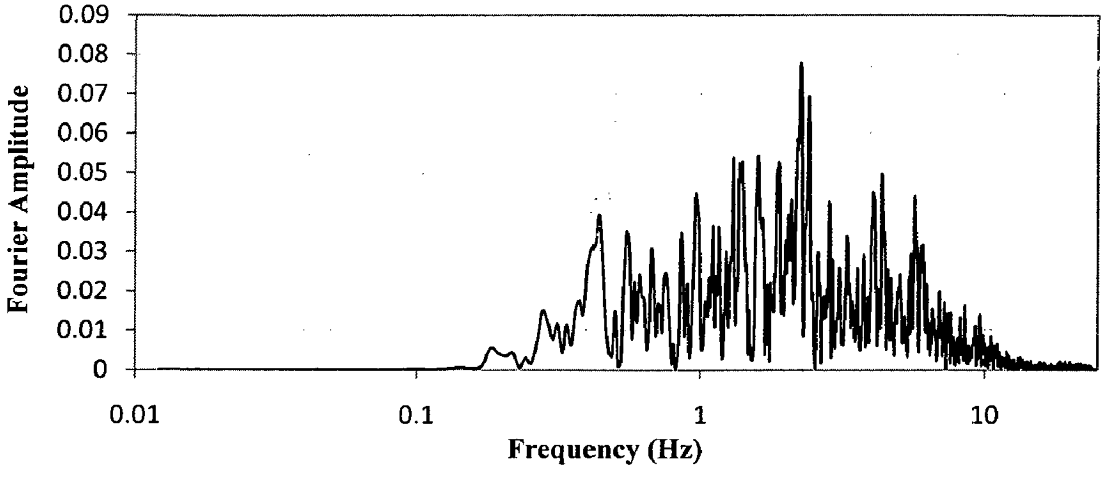
Fig.3.4- Time history and response spectra for 1938 Elcentro earthquake



(c) Time history

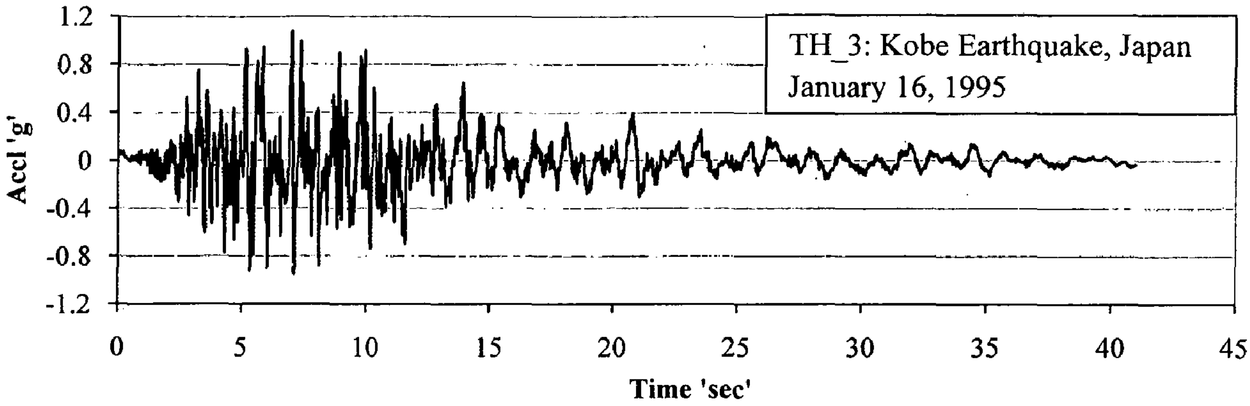


(b) Response spectra

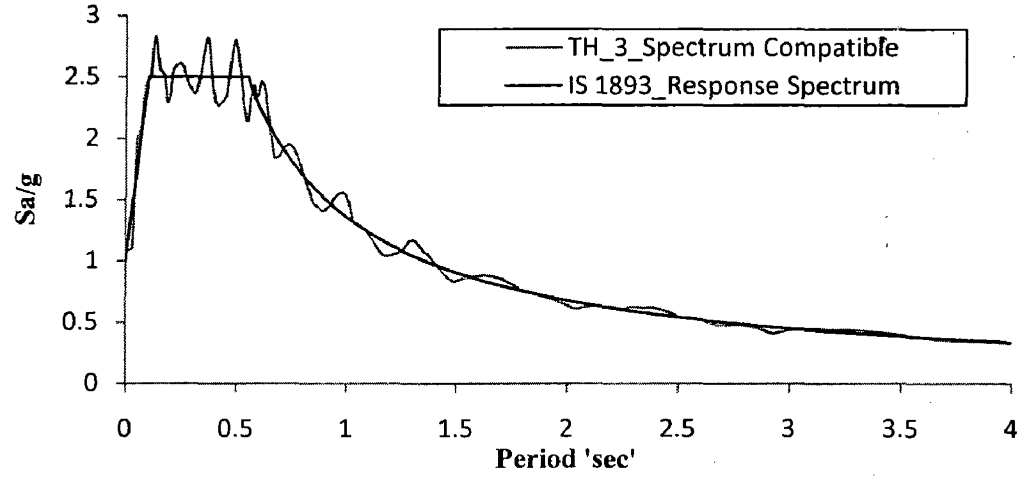


(a) Fourier spectra

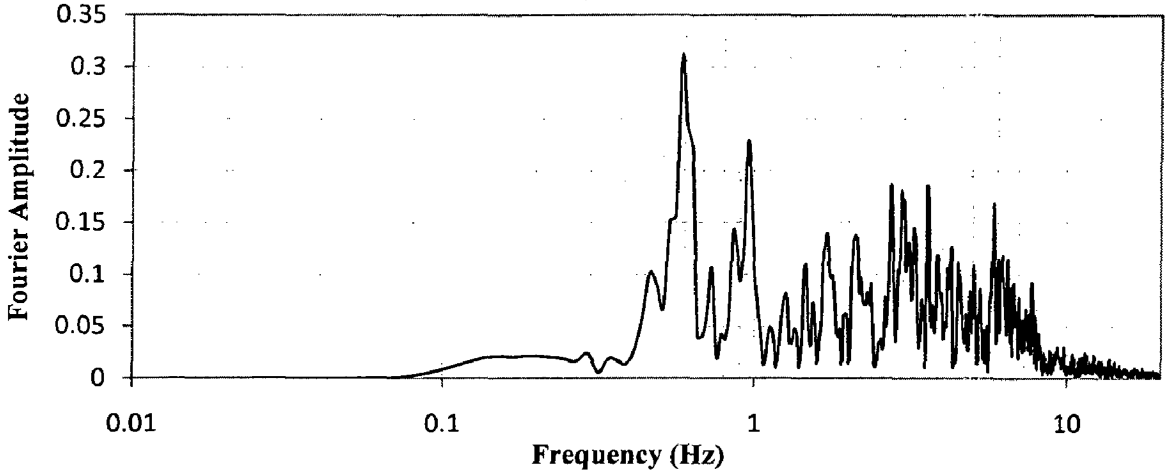
Fig.3.5- Time history and response spectra for 1994 Northridge earthquake



(c) Time history



(b) Response spectra



(a) Fourier spectra

Fig.3.6- Time history and response spectra for 1995 Kobe earthquake

3.5- Retrofitting Design Procedure

A performance based approach is used for the seismic retrofitting design of the buildings considered in this dissertation. The performance based design approach is based on matching the performance of the building designed for gravity load only to the immediate occupancy (IO) level under the MCE level of the earthquake.

The iterative procedure is outlined in this section as suggested by Durucan and Dicleli[8] which is employed in the seismic retrofitting design of the buildings. The procedure involves both response spectrum (RS) and nonlinear static pushover (NLSP) analyses. The permissible stresses and bilinear model for aluminium shear link as proposed by Rai and Wallace[25] is used to fix up the sizes of shear links for the designed forces. The seismic retrofitting design methodology used in this study is mainly based on equal energy dissipation principle.

In this methodology, the monotonic energy dissipation capacities of the buildings in the linear elastic range based on the roof displacement obtained from response spectrum analyses and nonlinear static pushover analysis are calculated and compared as shown in Fig.3.7. The difference between the areas under the elastic and inelastic base shear v/s roof displacement curves is the required additional energy that needs to be absorbed by the chevron braces with shear link system. The step by step seismic retrofitting design procedure is given below.

1. In the first step response spectrum analysis of the building in the retrofitted stage are conducted to obtain the linear elastic base shear force v/s roof displacement relationship as shown in Fig.3.7 (a). Since the retrofitting scheme and sizes of shear link and braces are not known at the initial stage of the design procedure, the unretrofitted building is used in the analysis. However, since seismic retrofitting results in an increase in the lateral stiffness of the building, the lateral stiffness of the original deficient building needs to be increased by a certain amount (e.g. initially by 50%) in the response spectrum analysis by adjusting the modulus of elasticity of the reinforced concrete members of the structure.

2. In the second step, the nonlinear static pushover analysis of the original building is conducted to obtain the base shear v/s roof displacement relationship as shown in Fig. 3.7(b). This relationship is plotted up to the displacement level corresponding to the

displacement capacity of the building for the performance level under consideration i.e. immediate occupancy level. The plotted curve is then idealized to have an elastoplastic shape as described in FEMA 356[10]. The base shear R_y at the yielding as obtained from the elastoplastic curve and the elastic stiffness of the structure in the retrofitted stage is used for subsequent calculations.

3. In the third step, first the area A_e under the linear elastic base shear v/s roof displacement curve is calculated as shown in Fig.3.7 (a). Then the area A_p under the elastoplastic base shear v/s roof displacement curve is calculated as shown in Fig.3.7(b). The monotonic energy A_d that needs to be dissipated by the retrofitting system is then calculated as;

$$A_d = A_e - A_p \tag{3.1}$$

4. In this step, the required total strength, Q_1 of the retrofitting system at the base of the building is obtained by using following equation. One of the roots from the equation will give the required strength Q_1 of the retrofitted system at the base of the building.

$$Q_{1,2} = K_e d_p \pm \sqrt{(V_y - K_e d_p)^2 - 2K_e A_d} \tag{3.2}$$

5. In this step of the design procedure, the seismic shear force capacity R_{Fi} of each storey of the original unretrofitted building is obtained as per FEMA356 [10].

6. In this step of the retrofitting design procedure, the retrofitting system is designed for the whole building. The design is based on the uniform energy dissipation throughout the height of the building. For this purpose, the elastic shear V_i at each storey level i is obtained from the response spectrum analysis result as in step 1. Then the total strength R_1 at the base of the retrofitted building is obtained by summing up its base shear capacity R_{F1} and the required strength of the retrofitting system i.e.

$$R_1 = R_{F1} + Q_1 \tag{3.3}$$

To ensure uniform energy dissipation along the height of the building, the ratio of the total strength of the retrofitted building at each storey level i (i.e. $R_i = R_{Fi} + Q_i$) to the elastic shear V_i, at the corresponding storey level must be nearby equal. That is;

$$\frac{R_1}{V_1} = \frac{R_2}{V_2} = \cdots\cdots = \frac{R_i}{V_i} = \cdots\cdots = \frac{R_n}{V_n} \tag{3.4}$$

Where the subscript n in the above equation denotes the number of the storey. This will ensure that yielding is more likely to occur at all the storey levels. The ratio of the total strength R_1 of the retrofitted building to the elastic shear V_1 at the base of the building is already known. To calculate the required retrofitting system at any storey level i the following relationship is used;

$$\frac{R_1}{V_1} = \frac{R_{Fi} + Q_i}{V_i} \tag{3.5}$$

Then solving for Q_i the following equation is obtained;

$$Q_i = \frac{R_1}{V_1} V_i - R_{Fi} \tag{3.6}$$

7. Following the procedure outlined above the design force Q_i for retrofitted system at each storey level is already determined. To proportion the size of shear link, yield strength V_y of the shear link is assumed as;

$$V_y = F_y A_w \tag{3.7}$$

Where F_y is the average shear stress of the shear link as provided by Rai and Wallace [25] and A_w is the cross section area of the web of the shear link. Setting $V_y = Q_i$ the cross sectional area A_w can be calculated as;

$$A_w = Q_i / F_y \tag{3.8}$$

8. At the verge of buckling instability of the compression brace, the axial tensile and compressive forces in the tension and compression brace both are equal to the buckling load P_b. Consequently to prevent the buckling instability of the compression brace, the sum of the horizontal components of the buckling loads of the two braces must be larger than the yield strength of the shear link times an over strength factor Φ. Thus;

$$2 P_b \cos\alpha \geq \Phi Q_i \tag{3.9}$$

In the above equation α is the angle that brace make with the horizontal. Solving for P_b from the above equation the required buckling strength of the brace is obtained as;

$$P_b = \frac{\Phi\, Q_i}{2\, Cos\, \alpha} \tag{3.10}$$

The braces are selected to have minimum buckling capacity calculated from the above equation.

9. The elastic stiffness of the designed building is recalculated and compared with the stiffness assumed in the Step 1 of the procedure. If the difference is negligible the design procedure is complete. Otherwise the stiffness is updated and Step 1, 3, 4, 6, 7 and 8 are repeated.

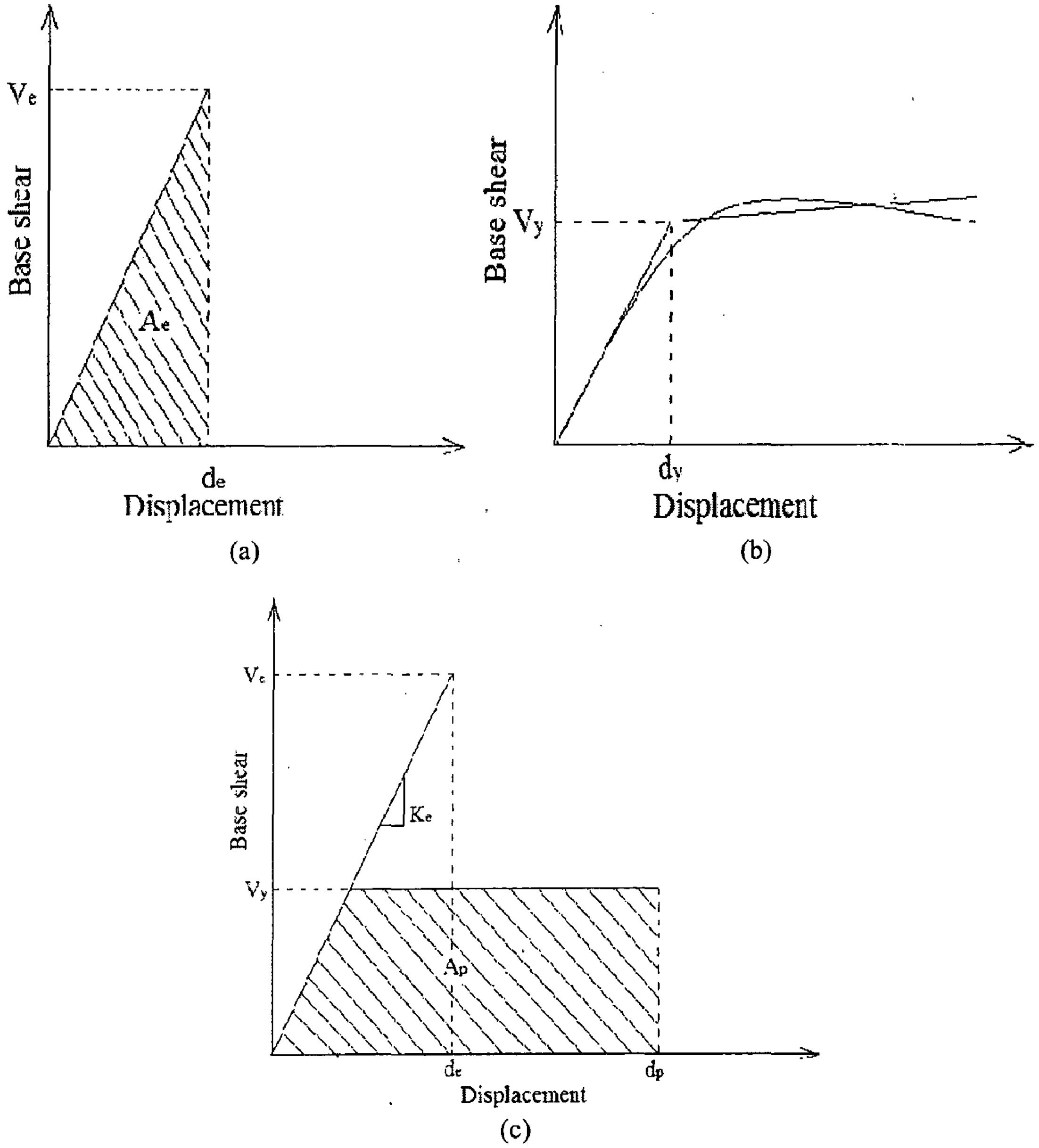


Fig.3.7- (a) Linear elastic base shear v/s roof displacement relationship (b) Elastoplastic base shear v/s roof displacement (c) Elastic v/s plastic base shear

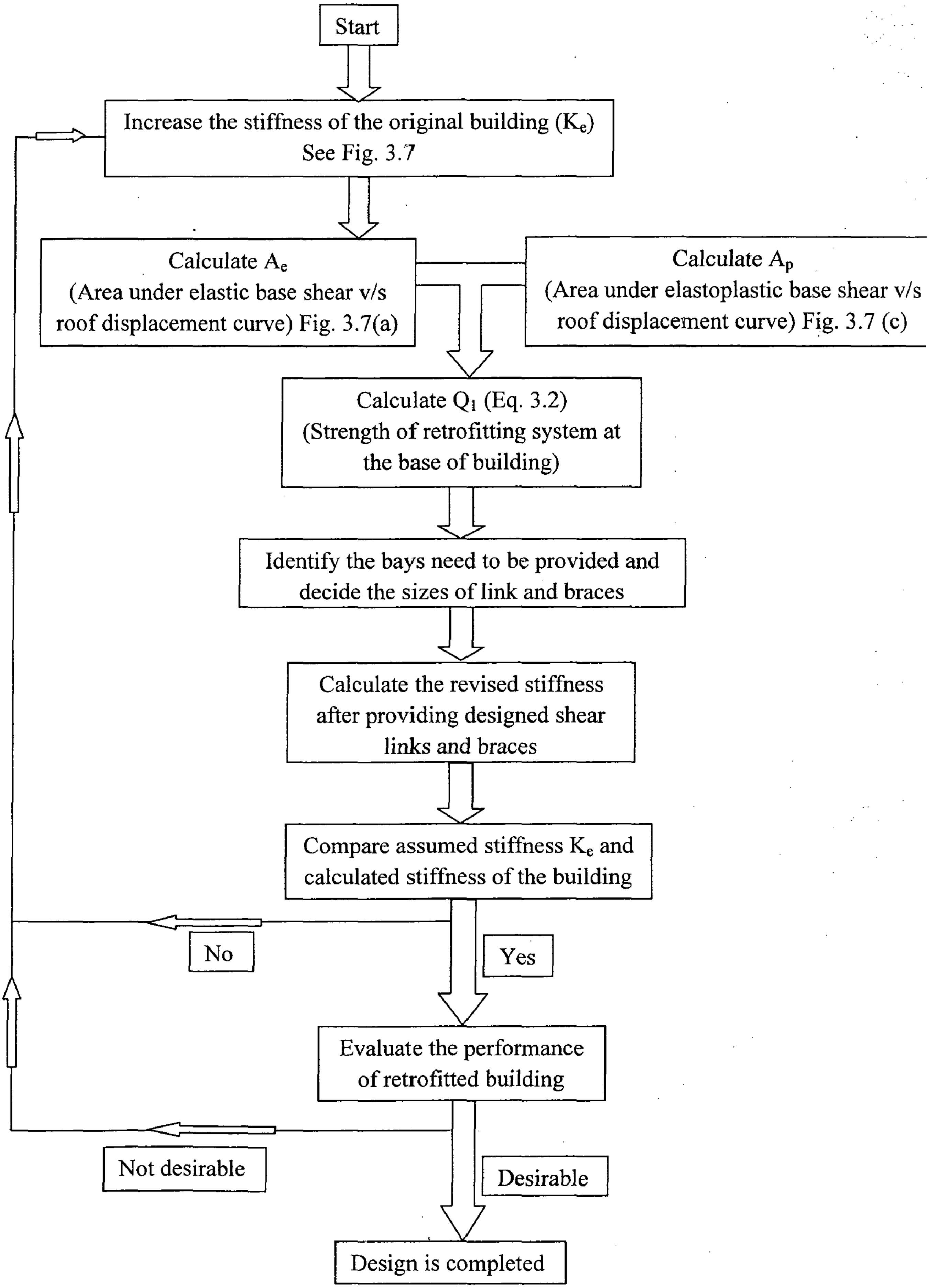


Fig. 3.8- Flow chart for design procedure

Since the main function of the chevron braces with aluminium shear link is to dissipate the input energy through metallic hysteresis, a successful attachment of the retrofitting system to the existing RC frame is necessary for effective transfer of the frame lateral load to the shear link and braces to cause shear yielding of web plate of the shear link. Shear link is incorporated in the structure as shown in Fig.3.9 below. The top flange of the shear link is attached to the beam soffit, whereas the bottom flange of the shear link was properly held in position by the chevron braces and T-hanger. Both ends of the RC columns need to be provided with steel collars formed by welding batten plates to four steel angle sections at the corners to facilitate the attachment of the braces and the collector beam to the existing RC frame. Due to the confinement action of steel collars, the lateral strength and plastic rotational capacities of RC columns would be enhanced to some extent.

Placement of these designed chevron braces with shear link is also an important issue. The locations of these braces have been selected based on various available options and considering the functional aspects of building. The arrangement giving the maximum performance of the structure has been selected by different trials.

The sizes of the shear links and braces as derived from the calculation made following the procedure outlined in the above section are given in Tables 3.6 and 3.7.

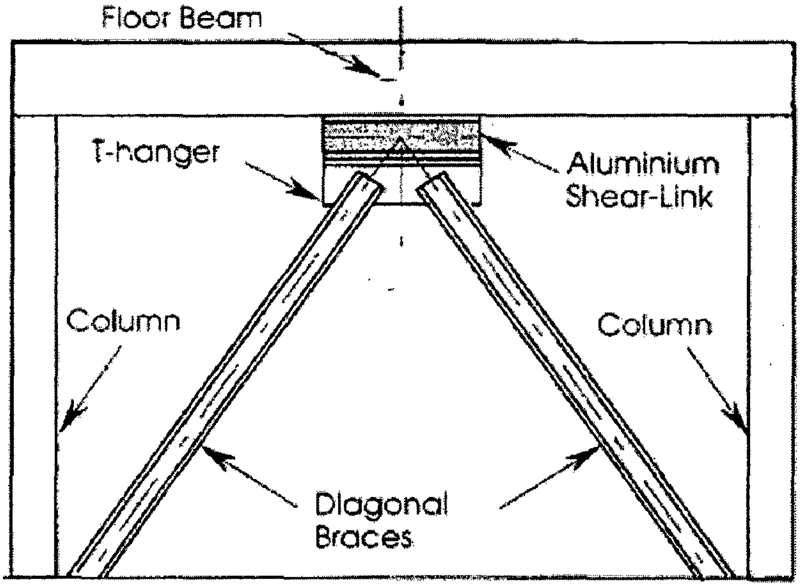


Fig.3.9- Shear link braced frame

Table 3.6- Sizes of shear links and braces in Bldg-1

Direction	Storey	Size of Shear link			Size of Brace	
		L (mm)	d (mm)	t_w (mm)	D (mm)	t (mm)
X	Ground floor	320	250	16	110	5
	First floor	320	250	16	110	5
	Second floor	280	250	15	100	5
	Third floor	270	250	10	90	4
Y	Ground floor	300	250	16	110	5
	First floor	300	250	16	110	5
	Second floor	280	250	16	100	5
	Third floor	250	250	10	90	4

Table 3.7- Sizes of shear links and braces in Bldg-2

Direction	Storey	Size of Shear link			Size of Brace	
		L (mm)	d (mm)	t_w (mm)	D (mm)	t (mm)
X	Ground floor	300	250	15	110	5
	First floor	280	250	15	110	4
	Second floor	280	250	15	110	4
	Third floor	250	250	14	100	5
	Fourth floor	250	250	10	90	5
	Fifth floor	200	250	8	90	4
Y	Ground floor	330	250	15	120	4
	First floor	330	250	15	120	4
	Second floor	330	250	15	120	4
	Third floor	260	250	15	100	5
	Fourth floor	250	250	10	90	5
	Fifth floor	150	250	10	80	4

The locations of shear link braced frames in deficient buildings are shown in Fig.3.10 and 3.11 below.

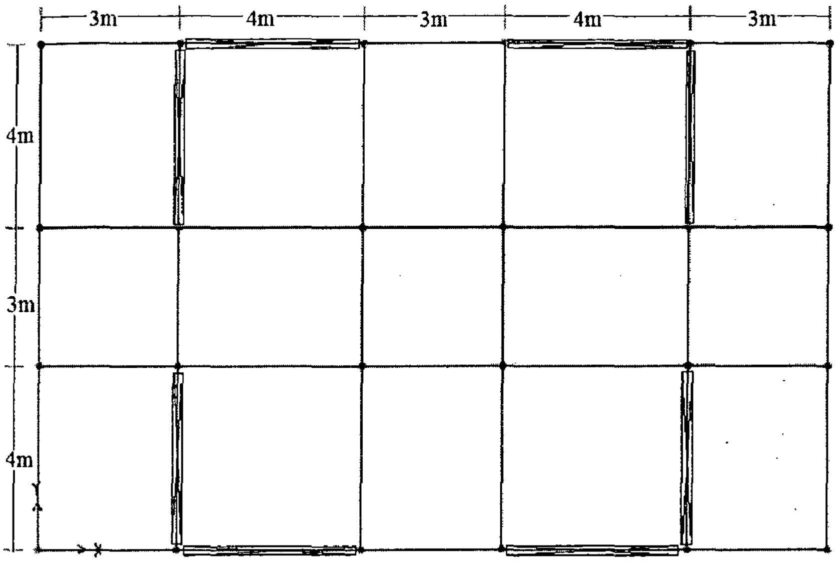


Fig.3.10- Shear link braced frame arrangement in Bldg-1

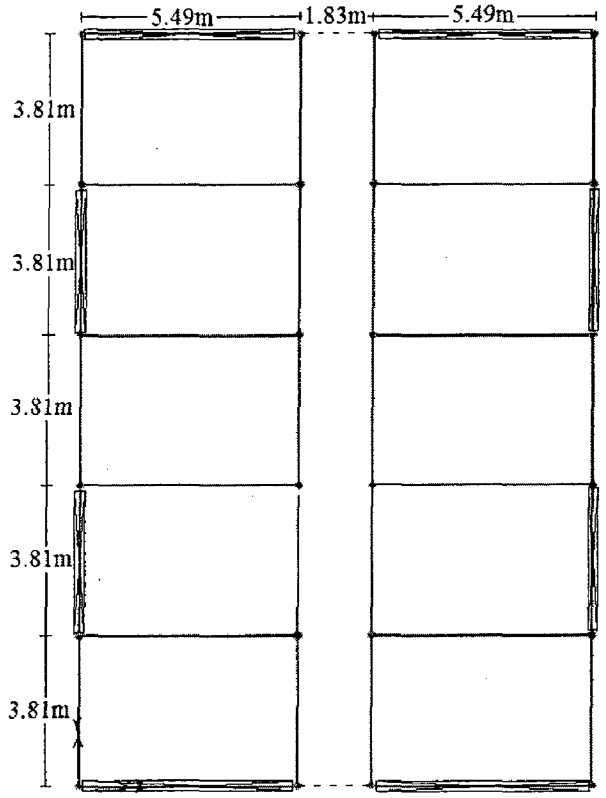


Fig.3.11- Shear link braced frame arrangement in Bldg-2

3.6- Concluding Remarks

Modelling and analysis aspects have been discussed in this chapter. The buildings have been modelled in SAP2000 as a 3D frame. The performance of the original deficient buildings has been evaluated by performing the nonlinear static pushover analysis. Aluminium shear link with chevron braces has been suggested to improve the seismic performance of these deficient buildings. A step by step procedure has been provided to retrofit deficient RC frame building by making use of aluminium shear link with chevron braces. The performance of retrofitted buildings are then worked out by performing a series of nonlinear static pushover analysis and nonlinear time history analysis. The location of shear link braced frame in deficient buildings have been provided after making different trial and selecting the best possible location considering the available option and functionality of building. The locations giving the highest performance are then selected.

Chapter -4

RESULTS AND DISCUSSIONS

4.1- General

An elaborative nonlinear analysis has been carried out to understand the behaviour of the original deficient and retrofitted structures as discussed in previous chapter. The study mainly focussed towards the performance enhancement of the building which was designed only to resist gravity load without any consideration to seismic lateral loads. The performance based approach has been used to verify the effectiveness of the aluminium shear link system in resisting the lateral load.

4.2- Performance of the Original and Retrofitted Buildings

As the original building was designed only to resist gravity load, behaviour under the lateral loads has been ascertained by performing nonlinear static pushover analysis.

4.2.1- Performance Level

Performance levels describe the level of the damage to the structural as well as non-structural member of the structure. It is generally measured in terms of member rotation as per FEMA-356. Performance of Bldg-1 and 2 designed for gravity load only lies in life safety and collapse prevention range under DBE and MCE levels of earthquakes respectively, as shown in Fig. 4.2-4.5. The performance of the building is almost same in both orthogonal directions X and Y. The building is then strengthened by providing aluminium shear link with the help of chevron braces to improve its lateral load resistance. The performance of the building after providing aluminium shear link with chevron braces lies in immediate occupancy range for both the buildings in both the directions, except in X-direction for Bldg-2 the performance lies in life safety range as the beams having larger span are yielding to that level of hinges. To bring the performance of the Bldg-2 in immediate occupancy range strengthening of beams need to be carried out. This strength can be achieved in the form of jacketing of beams. Figures 4.2-4.5 below show the performance levels of the original and retrofitted buildings for DBE and MCE level of earthquakes in both X and Y directions. From nonlinear static push over analysis it is observed that the ductility of retrofitted building is reducing after providing chevron braces with aluminium shear link. This is on account of increase in column forces due to the addition of chevron braces.

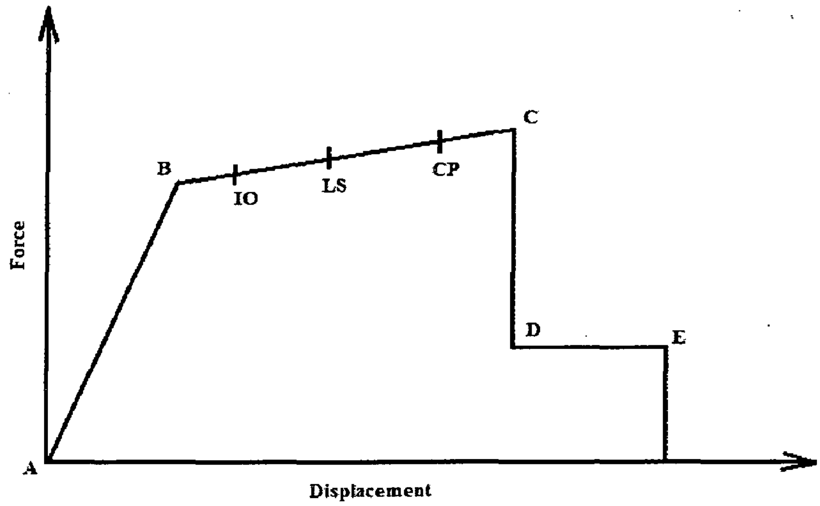


Fig.4.1- Force displacement behaviour for a typical RC member [10]

In the above figure, IO, LS and CP represent the immediate occupancy, life safety and collapse prevention structural performance level respectively. Point B in the above figure mark the end of the elastic range.

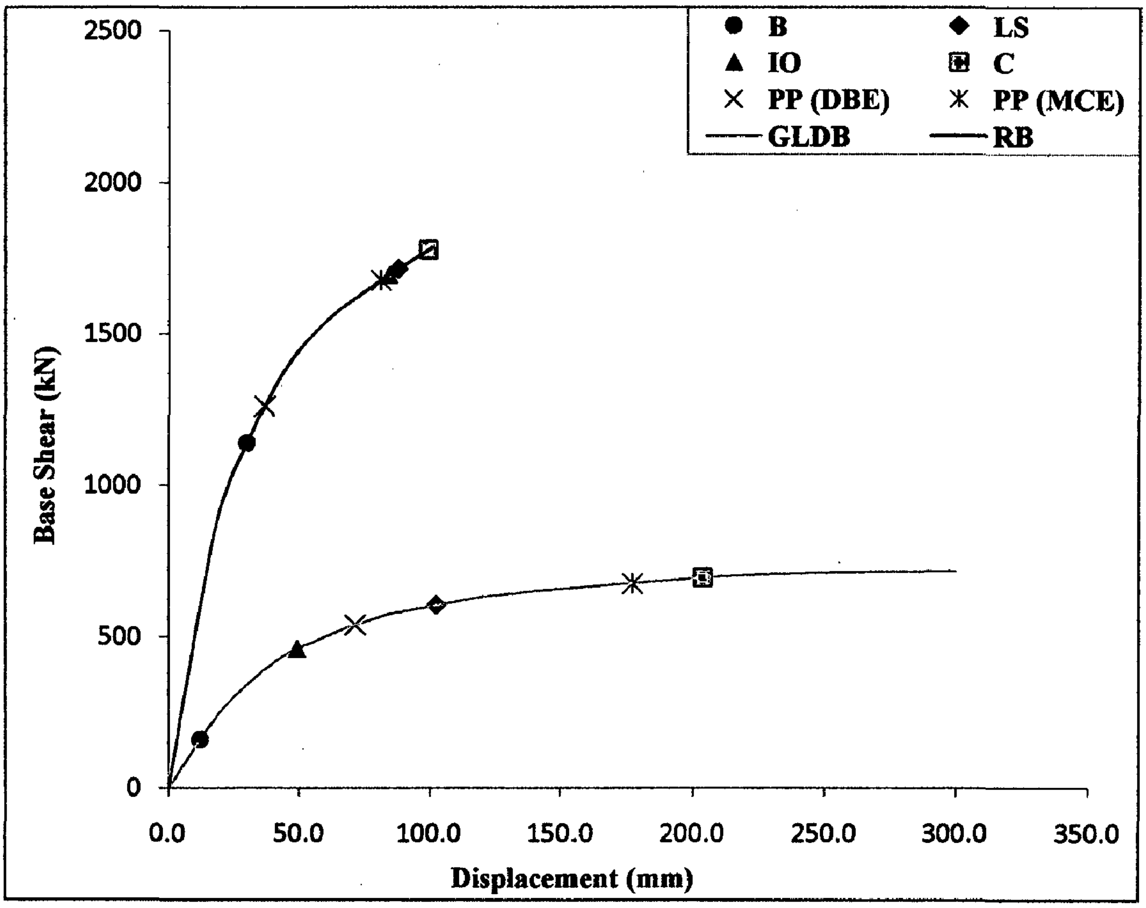


Fig.4.2- Pushover curve in X- direction (Bldg-1)

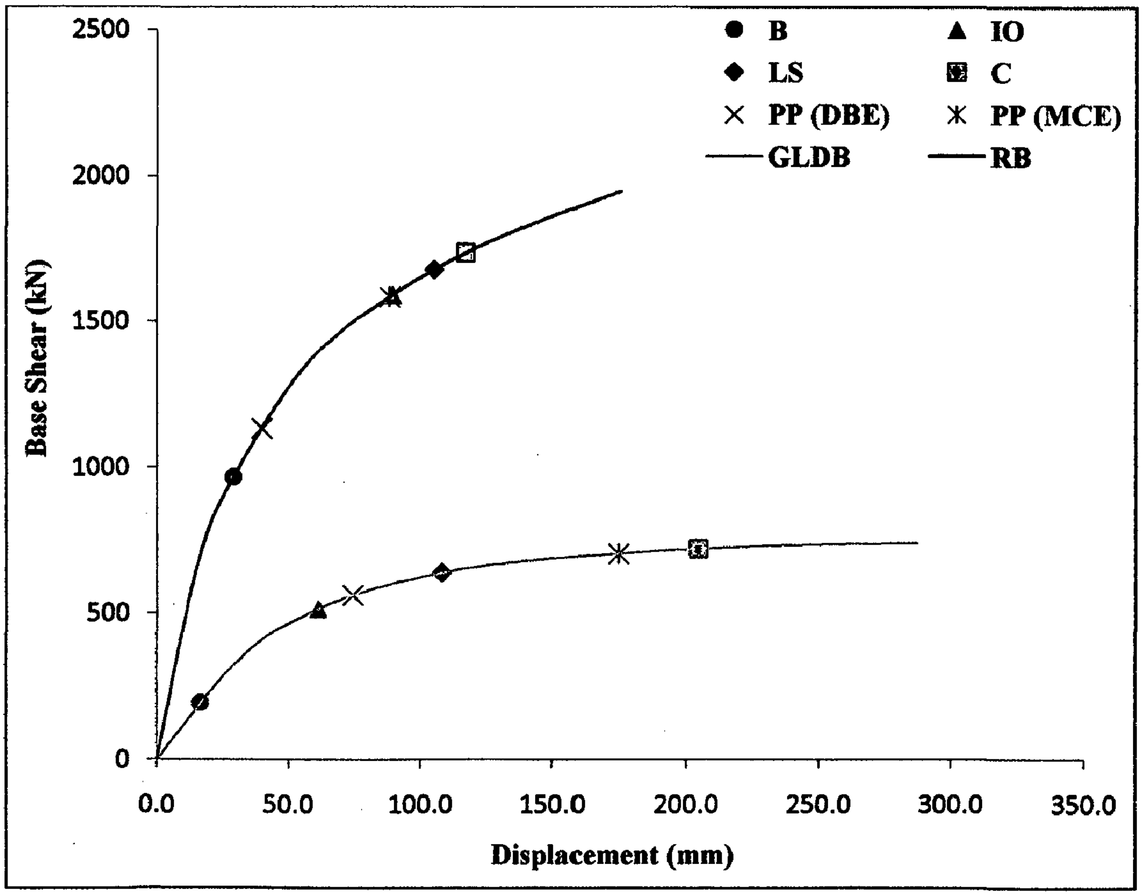


Fig.4.3- Pushover curve in Y- direction (Bldg-1)

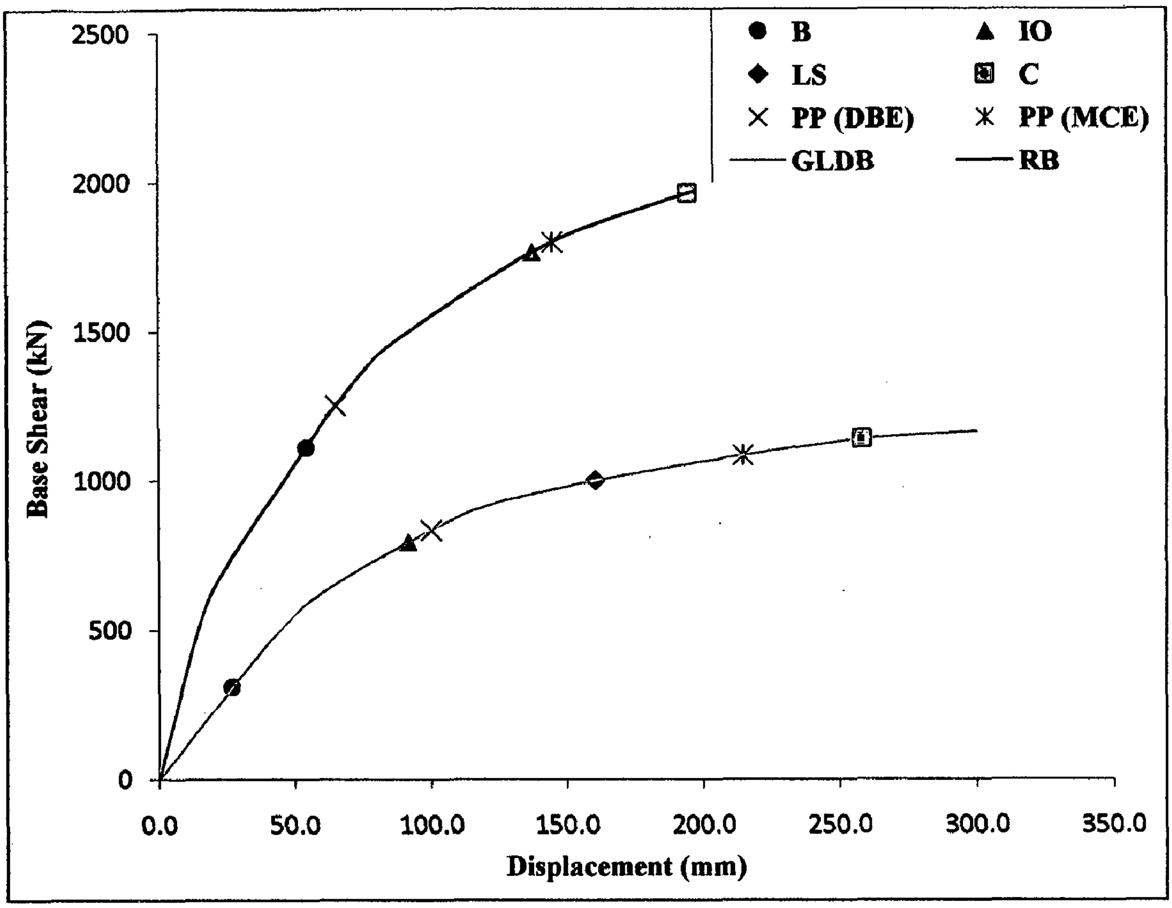


Fig.4.4- Pushover curve in X- direction (Bldg-2)

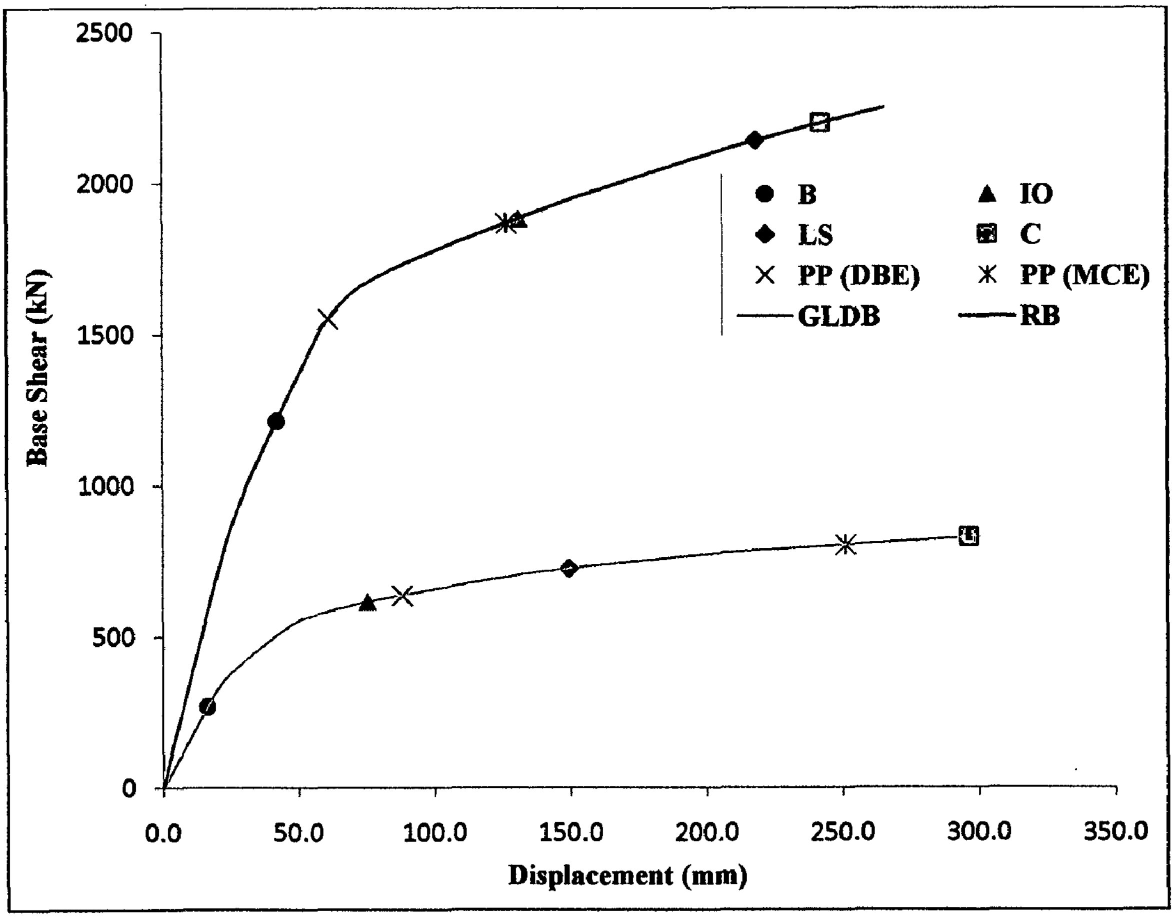
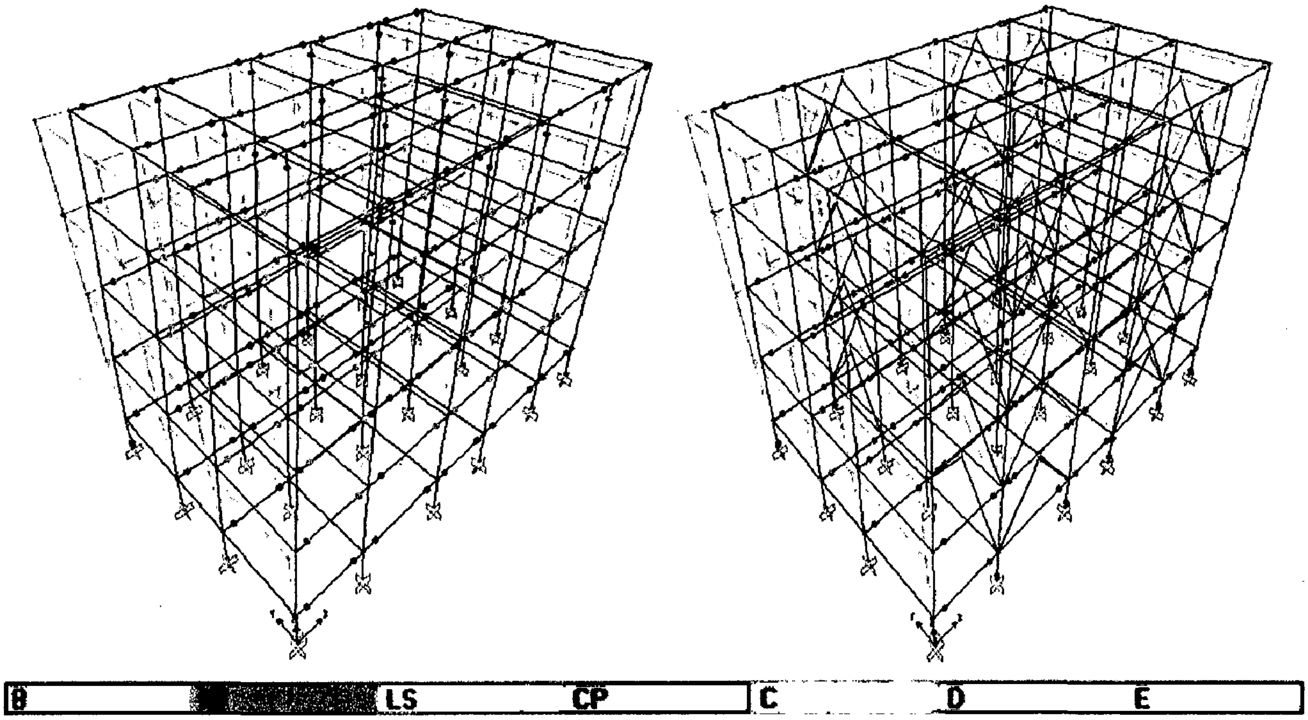


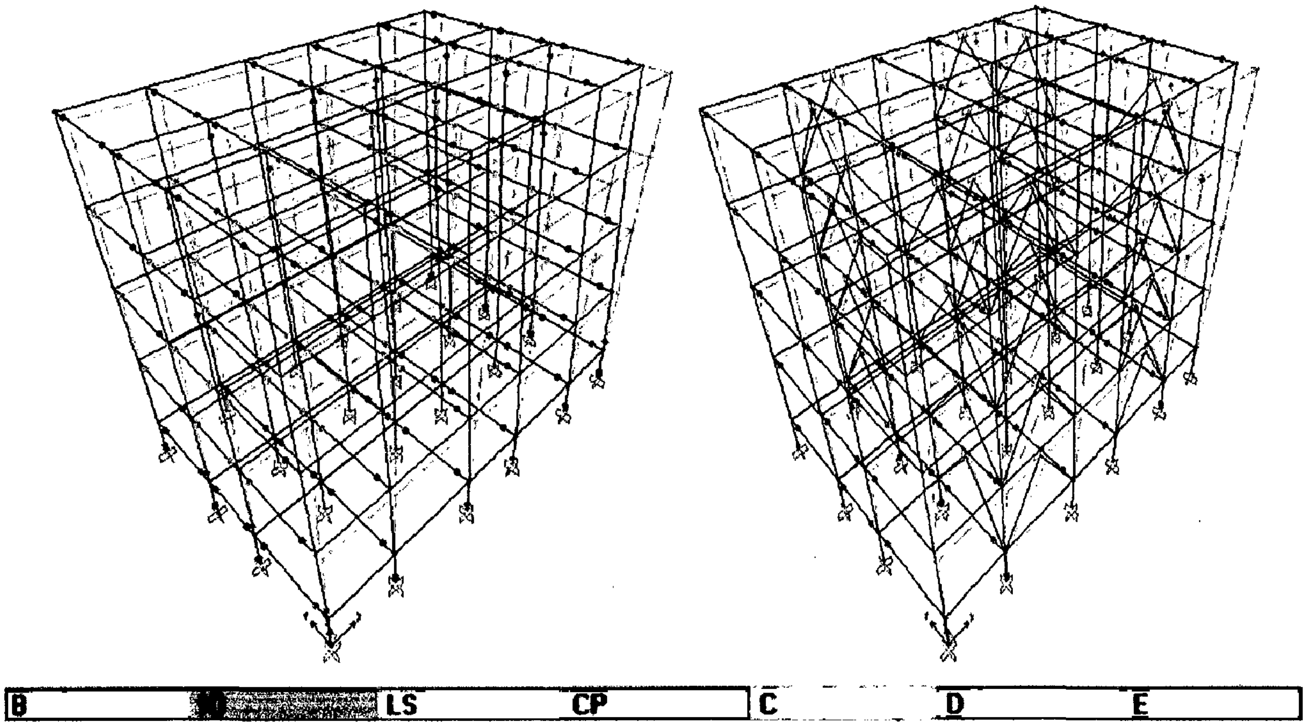
Fig.4.5- Pushover curve in Y- direction (Bldg-2)

Figure 4.6 below shows the deformed shapes and hinge pattern of the original and retrofitted buildings at the performance point for MCE level of earthquake. Almost all members respond elastically except few beams in Bldg-2.



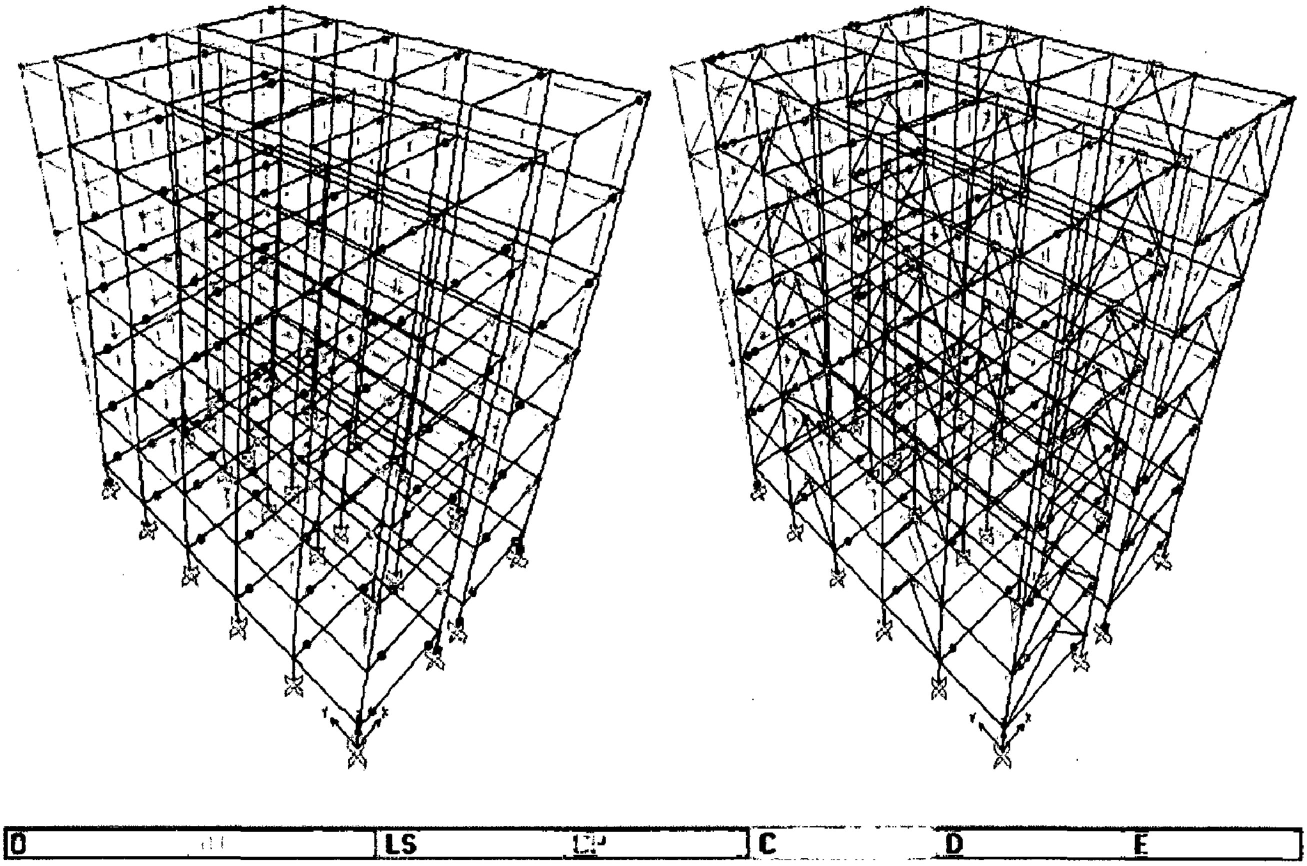
a) Original deficient building b) Retrofitted building

Fig.4.6- Hinge pattern at the MCE performance point in X-direction (Bldg-1)



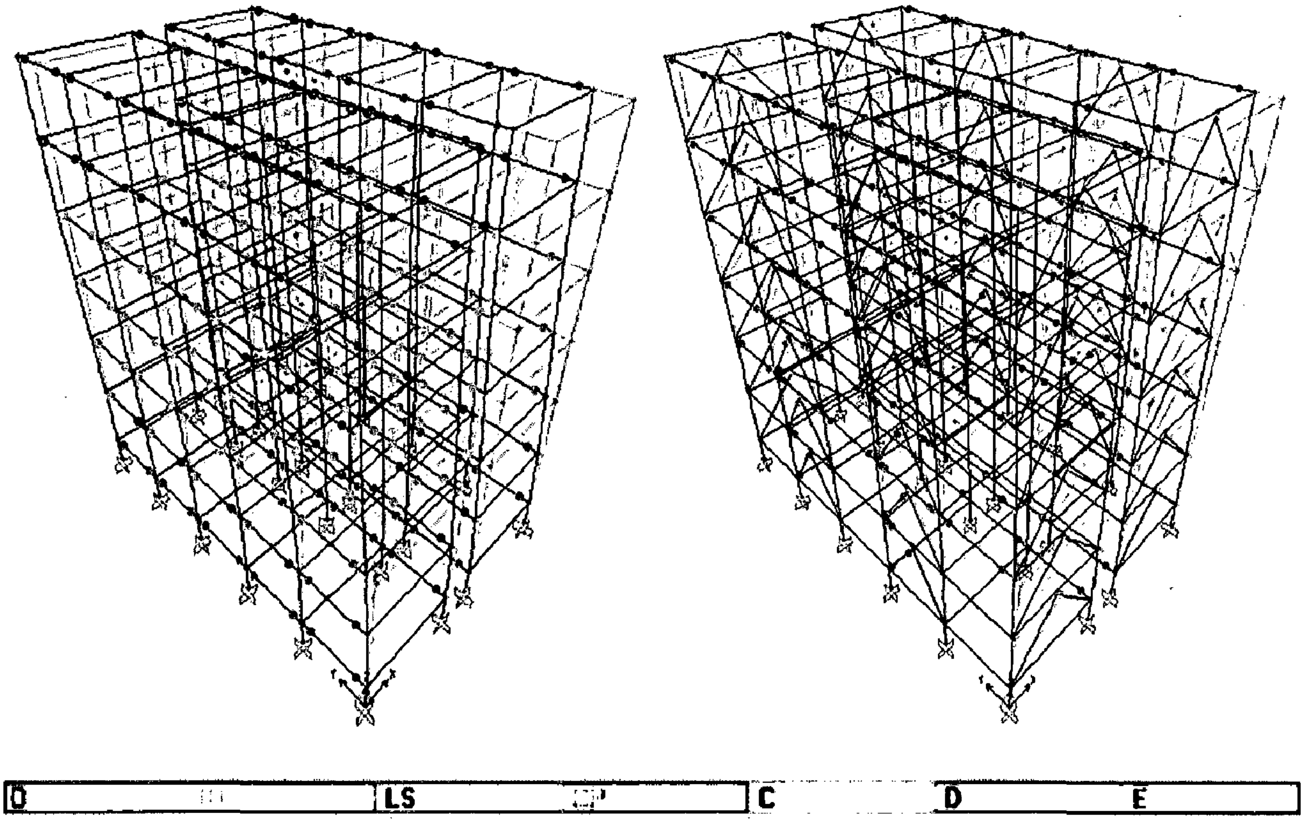
a) Original deficient building b) Retrofitted building

Fig.4.7- Hinge pattern at the MCE performance point in Y-direction (Bldg-1)



a) Original deficient building b) Retrofitted building

Fig.4.8- Hinge pattern at the MCE performance point in X-direction (Bldg-2)



a) Original deficient building b) Retrofitted building

Fig.4.9- Hinge pattern at the MCE performance point in Y-direction (Bldg-2)

Table 4.1- Details of hinges at DBE performance point for original buildings

Building	Direction	Hinge Details							
		A to B	B to IO	IO to LS	LS to CP	CP to C	C to D	D to E	Beyond E
1	X	466	136	18	0	0	0	0	0
	Y	492	106	22	0	0	0	0	0
2	X	708	68	8	0	0	0	0	0
	Y	564	184	36	0	0	0	0	0

Table 4.2- Details of hinges at DBE performance point for retrofitted buildings

Building	Direction	Hinge Details							
		A to B	B to IO	IO to LS	LS to CP	CP to C	C to D	D to E	Beyond E
1	X	670	14	0	0	0	0	0	0
	Y	666	18	0	0	0	0	0	0
2	X	861	19	0	0	0	0	0	0
	Y	794	86	0	0	0	0	0	0

Note: Above tables show the number of hinges formed for DBE level of earthquake for original deficient building and retrofitted building. The provision of shear link with chevron braces enhances the performance of structure. It can be seen from above Tables 4.1-4.2.

Table 4.3- Details of hinges at MCE performance point for original buildings

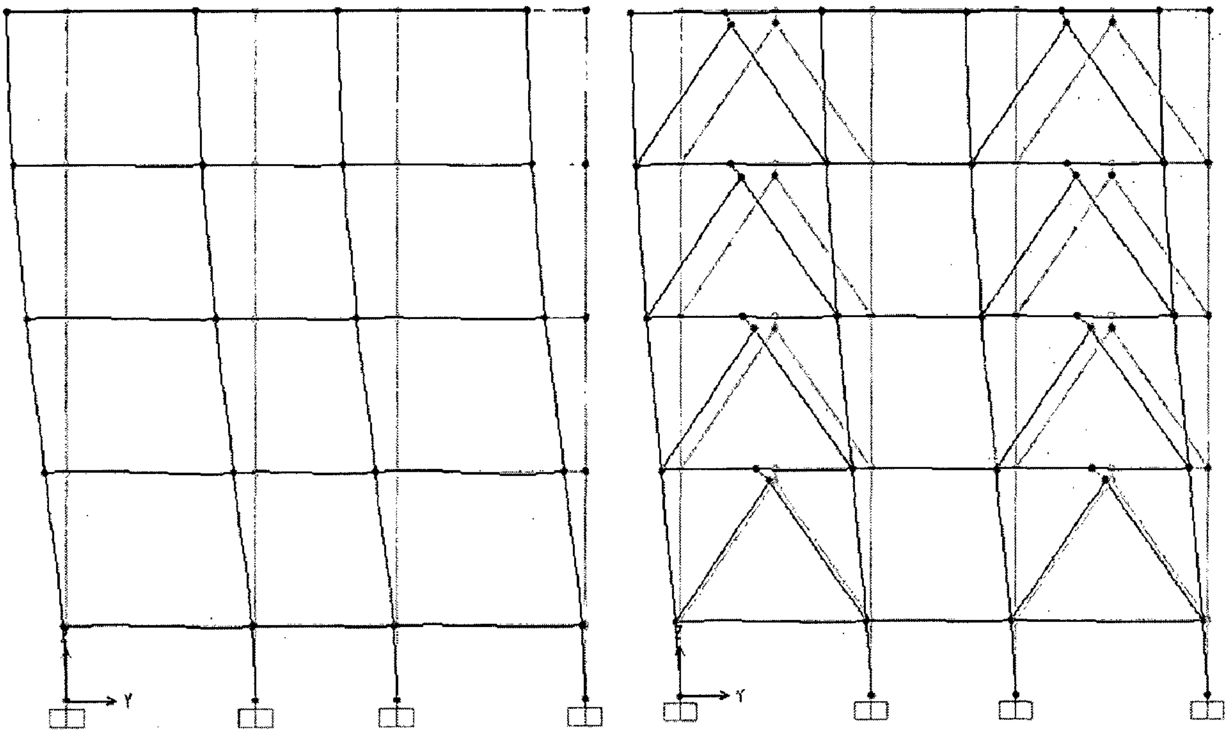
Building	**Direction**	**Hinge Details** **A to B**	**B to IO**	**IO to LS**	**LS to CP**	**CP to C**	**C to D**	**D to E**	**Beyond E**
1	X	394	50	88	88	0	0	0	0
	Y	410	60	86	64	0	0	0	0
2	X	632	60	56	36	0	0	0	0
	Y	504	58	82	140	0	0	0	0

Table 4.4- Details of hinges at MCE performance point for retrofitted buildings

Building	**Direction**	**Hinge Details** **A to B**	**B to IO**	**IO to LS**	**LS to CP**	**CP to C**	**C to D**	**D to E**	**Beyond E**
1	X	496	188	0	0	0	0	0	0
	Y	528	156	0	0	0	0	0	0
2	X	752	104	24	0	0	0	0	0
	Y	610	270	0	0	0	0	0	0

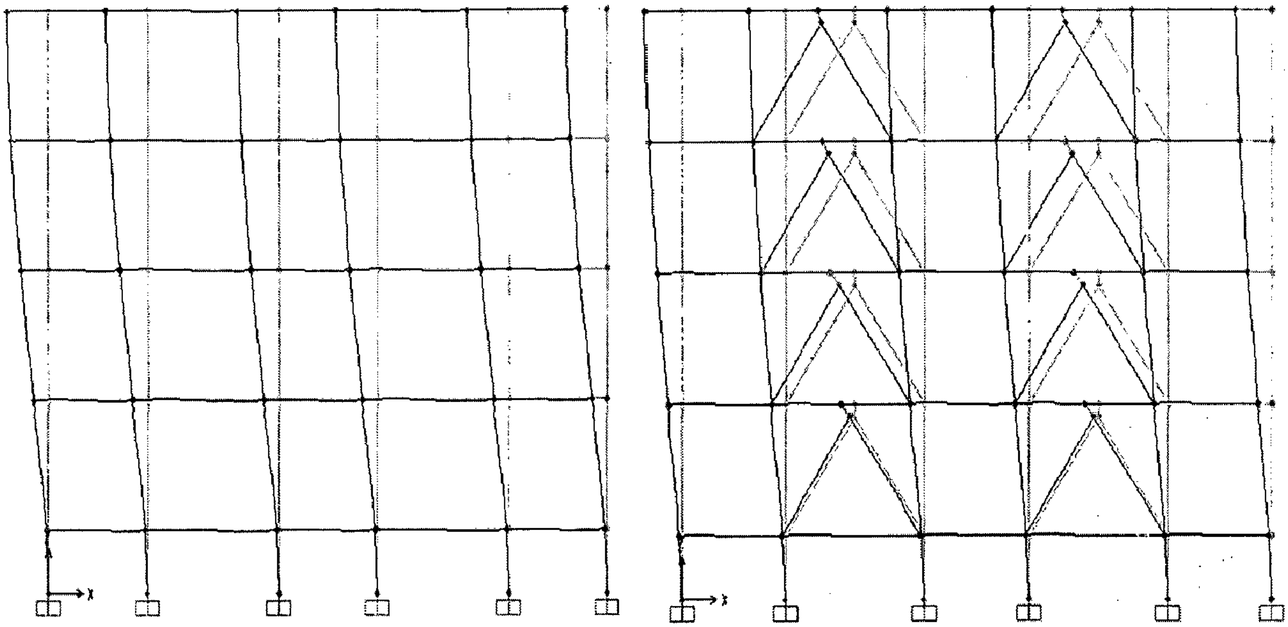
Note: The number of hinges formed after retrofitting the building with aluminium shear link and chevron braces have reduced significantly. The original deficient buildings when subjected to MCE level of earthquake performed to collapse prevention structural performance level. In retrofitted building all the member respond to immediate occupancy performance level. Only few beams those are having larger span have formed life safety level of hinges.

Figure 4.10-4.13 below shows the mode shapes of the original deficient building and the retrofitted buildings. From the mode shapes it has been observed that the relative pattern of deflected shape for both the building remains same even after retrofitting. Thus, the addition of chevron braces and shear link are not going to change the deflected shape of building in different modes of vibration.



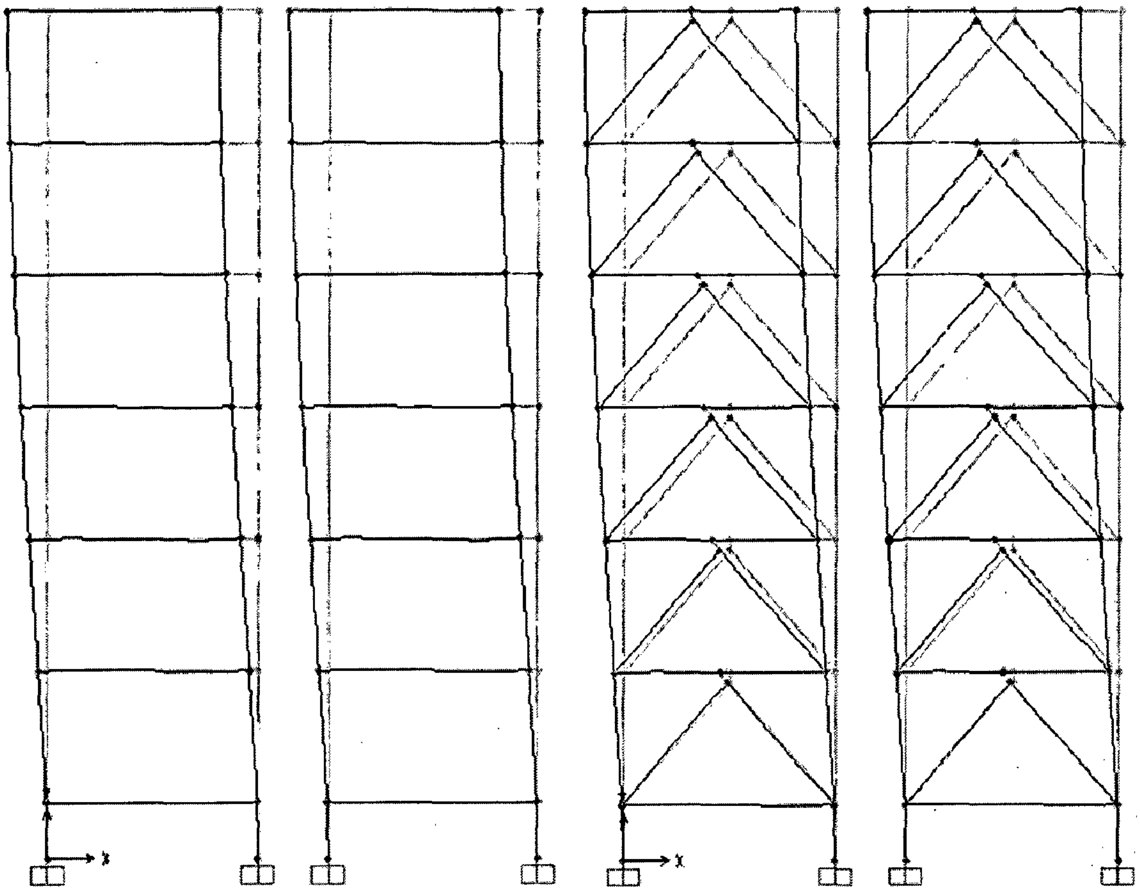
a) Original deficient building (Freq.=0.91 Hz) b) Retrofitted building (Freq.=1.09 Hz)

Fig.4.10- First mode for Bldg-1

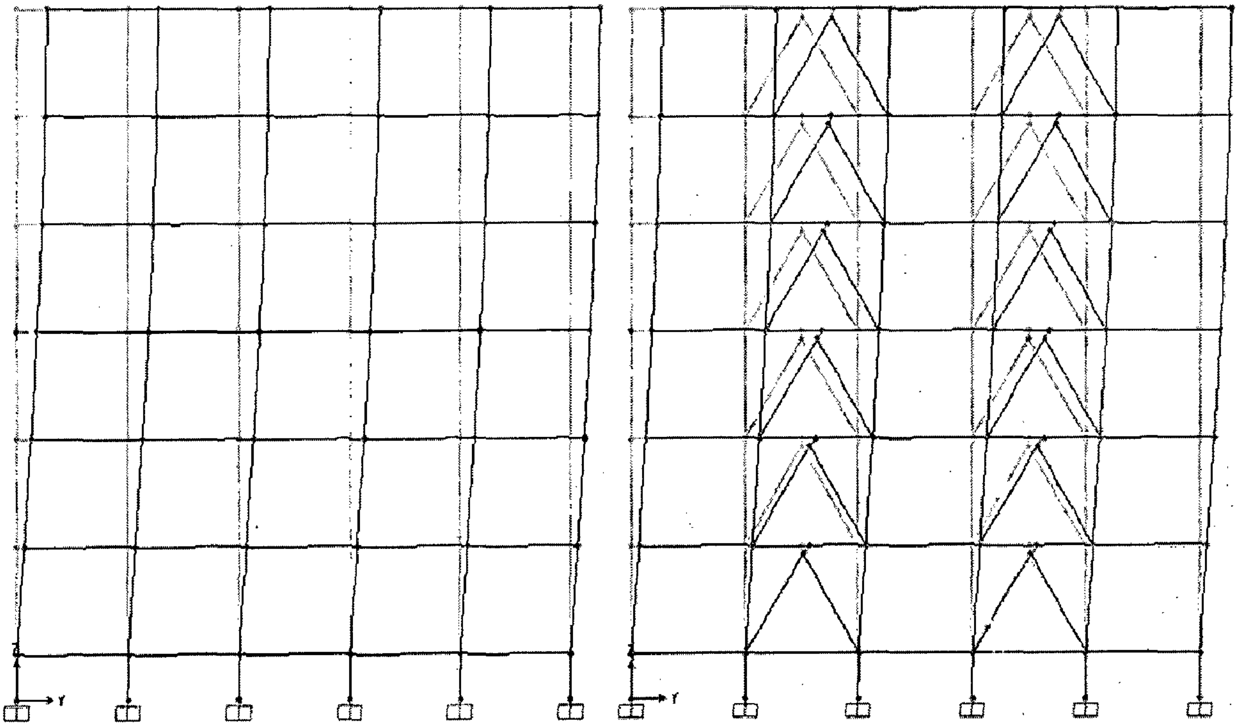


a) Original deficient building (Freq.=0.9 Hz) b) Retrofitted building (Freq.=1.14 Hz)

Fig.4.11- Second mode for Bldg-1



a) Original deficient building (Freq.=0.67 Hz) b) Retrofitted building (Freq.=0.77 Hz)
Fig.4.12- First mode for Bldg-2



a) Original deficient building (Freq.=0.81 Hz) b) Retrofitted building (Freq.=0.86 Hz)
Fig.4.13- Second mode for Bldg-2

4.2.2- Interstorey Drift

Inter storey drift is an important parameter in limiting the damages to the structural and non-structural members of the structure. The inter storey drift in longitudinal and transverse directions of both the buildings are shown in Fig. 4.14-4.21 for DBE and MCE level of earthquakes respectively. As expected the increase in stiffness of the structure results in reduced inter storey drift and can be verified from the figure below.

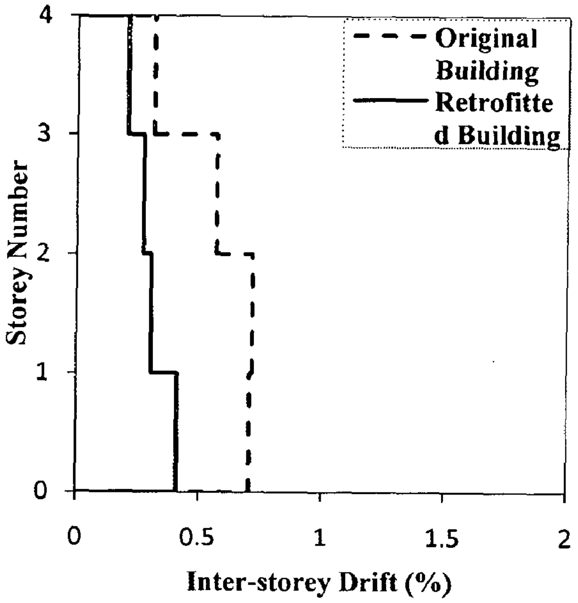


Fig.4.14- Inter storey drift at DBE performance point in X-direction (Bldg-1)

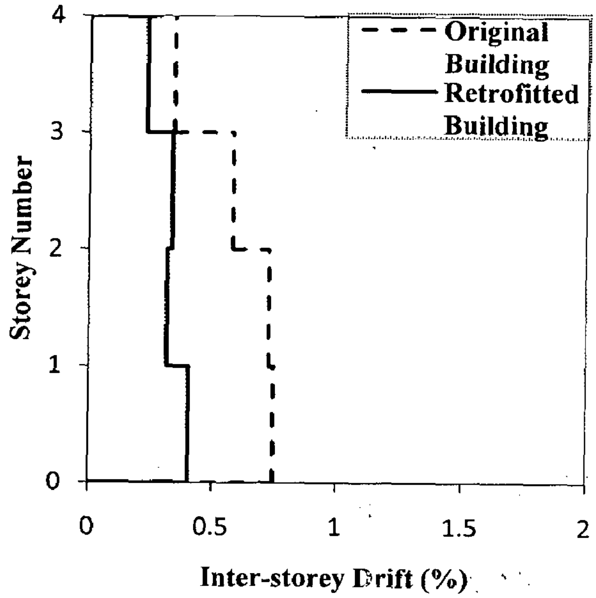


Fig.4.15- Inter storey drift at DBE performance point in Y-direction (Bldg-1)

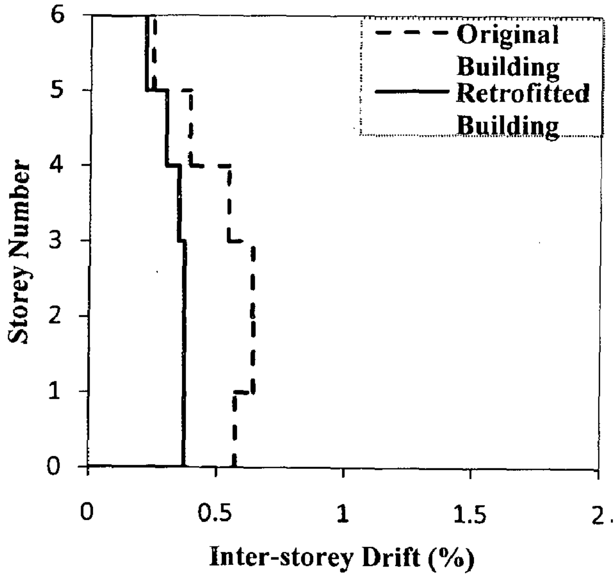


Fig.4.16- Inter storey drift at DBE performance point in X-direction (Bldg-2)

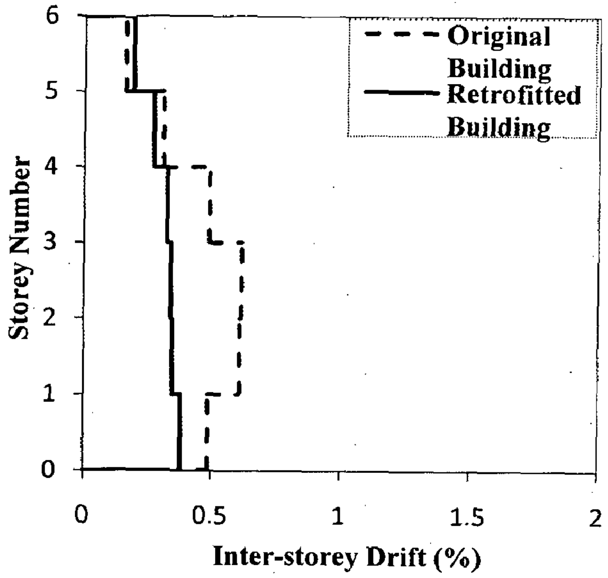


Fig.4.17- Inter storey drift at DBE performance point in Y-direction (Bldg -2)

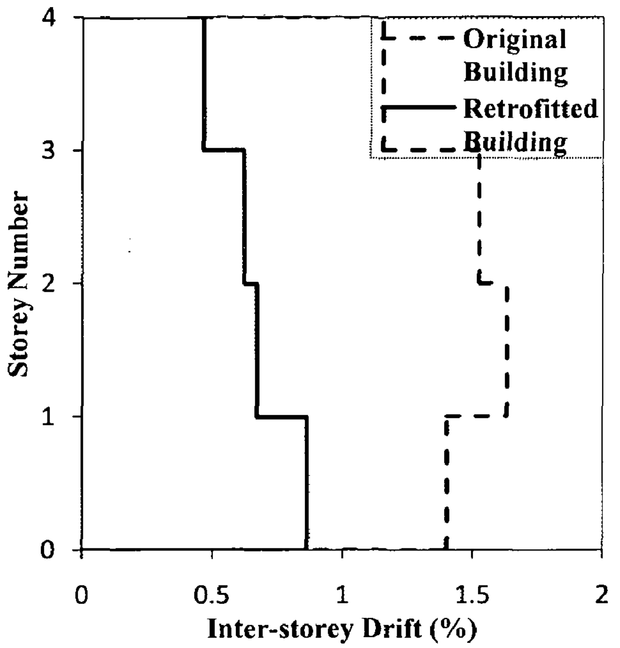


Fig.4.18- Inter storey drift at MCE performance point in X-direction (Bldg -1)

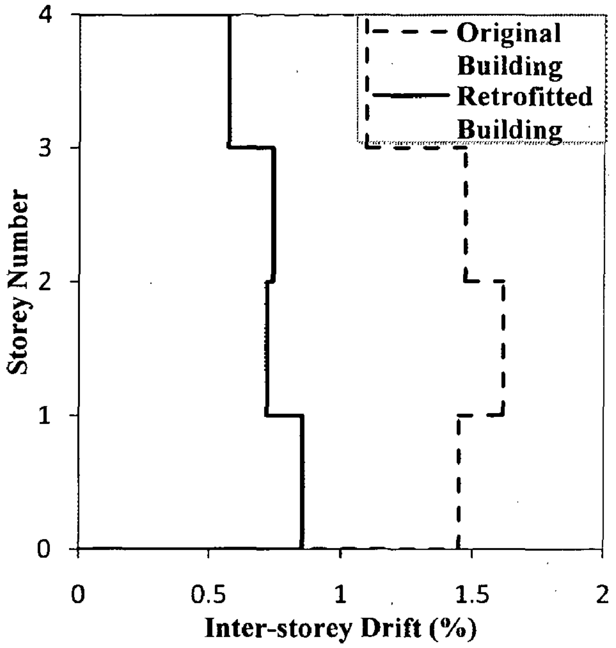


Fig.4.19- Inter storey drift at MCE performance point in Y-direction (Bldg -1)

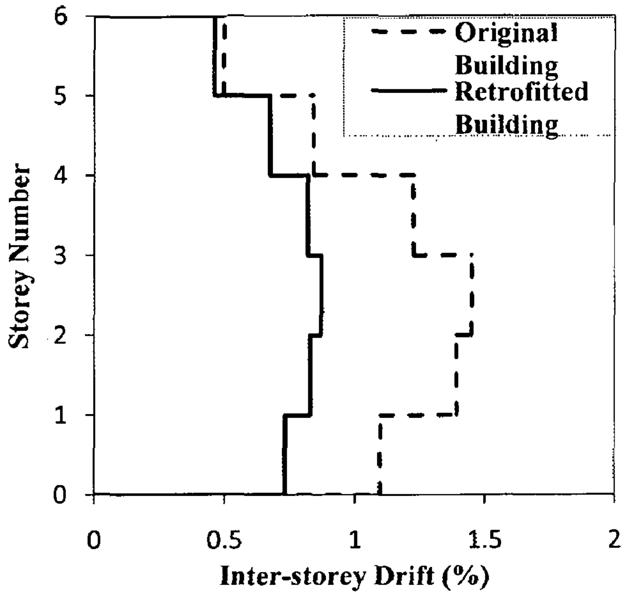


Fig.4.20- Inter storey drift at MCE performance point in X-direction (Bldg -2)

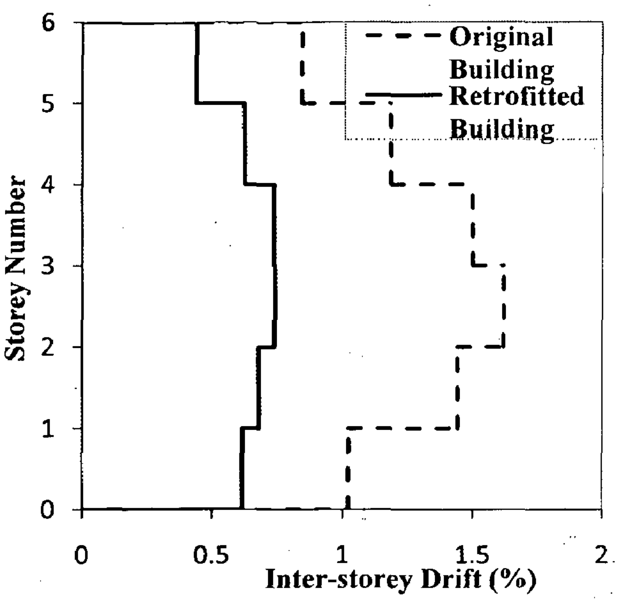


Fig.4.21- Inter storey drift at MCE performance point in Y-direction (Bldg -2)

4.2.3- Stiffness and Base Shear

Provision of shear link with chevron braces to the structure not only going to enhance the energy dissipation capacity of the structure, but it stiffened the structure as well. The initial elastic stiffness of both original deficient and retrofitted structures as calculated by idealising the capacity curve or pushover curve are summarised in Table 4.5 for both the buildings in X and Y directions. For Bldg-1 the increase in initial elastic stiffness are 4.26 and 3.89 times in X and Y directions respectively, whereas for Bldg-2 the increase in the stiffness are 2.52 and 2.18 times of original stiffness of structure as shown in Table 4.5 below.

Table 4.5- Comparison of initial elastic stiffness for original deficient and retrofitted buildings

Building	**Direction**	**Original Structure Stiffness (kN/mm)**	**Retrofitted Structure Stiffness (kN/mm)**	**% Increase in Stiffness**
1	X	11.48	48.98	4.26
	Y	11.42	44.44	3.89
2	X	11.35	28.60	2.52
	Y	15.79	34.44	2.18

The base shear has been calculated by performing the response spectrum analysis on original and retrofitted structures. The addition of shear link with chevron braces tends to increase the base shear on account of stiffening of the original structure. The response spectrum analysis result shows that the increase in the base shear of the original building is of the order of 1.25 and 1.26 times in X and Y directions respectively for Bldg-1 and for Bldg-2 it is of the order of 1.14 and 1.11 times of the original structure base shear in X and Y directions respectively. Table 4.6 shows the increase in base shear values of the original and retrofitted structures in both X and Y direction.

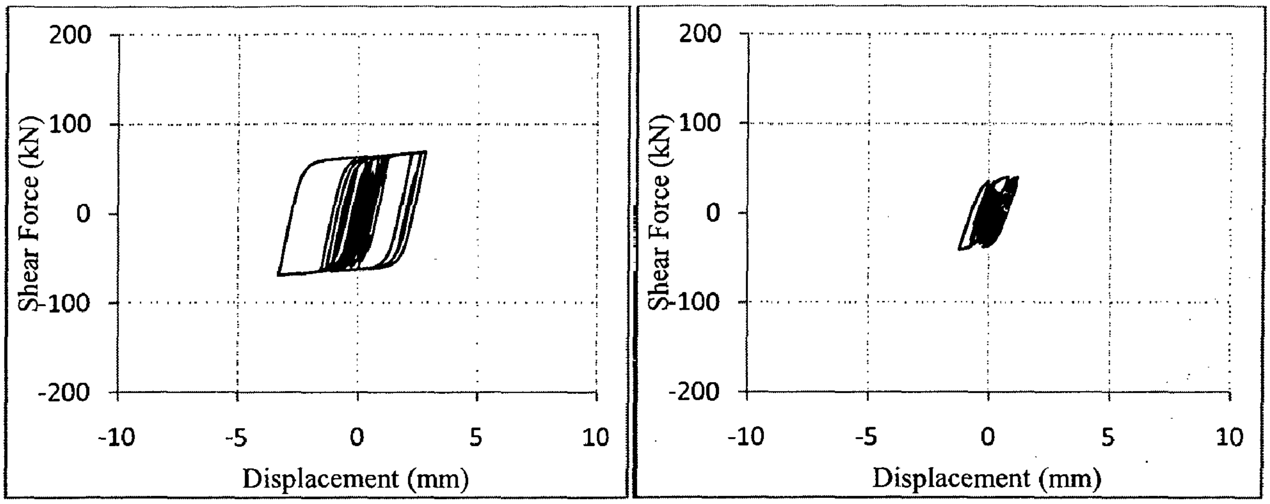
Table 4.6- Comparison of Base Shear for original deficient and retrofitted buildings

Building	Direction	Original Structure Base Shear (kN)	Retrofitted Structure Base Shear (kN)	% Increase in Base shear
1	X	796.92	1000.00	1.25
	Y	753.58	947.92	1.26
2	X	1035.81	1179.23	1.14
	Y	1215.93	1343.08	1.11

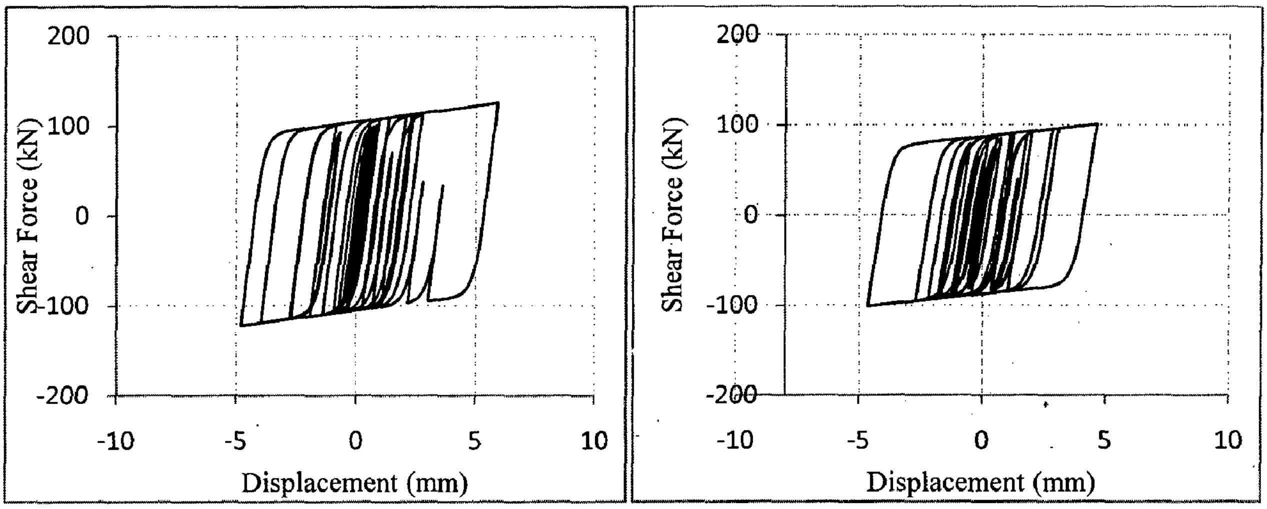
4.3- Behaviour of the Shear Link and Braces

Behaviour of the structure after providing chevron brace with aluminium shear link has already been discussed in the previous section by performing nonlinear static push over analysis. The effectiveness of the aluminium shear link in limiting the forces that are transferred to the primary member of the structure has been assessed by performing the nonlinear time history analysis. Aluminium shear link sizes are proportioned in such a way that yielding is designated to occur first in aluminium shear link that can be replaced easily following a major earthquake. Nonlinear time history analysis result shows that aluminium shear link dissipates earthquake induced energy in the form of stable hysteretic loops. Figure 4.22-4.25 below show the typical hysteretic loop for the shear links in X and Y directions respectively in all the storey of building when subjected to DBE and MCE level of ground shaking in nonlinear time history analysis.

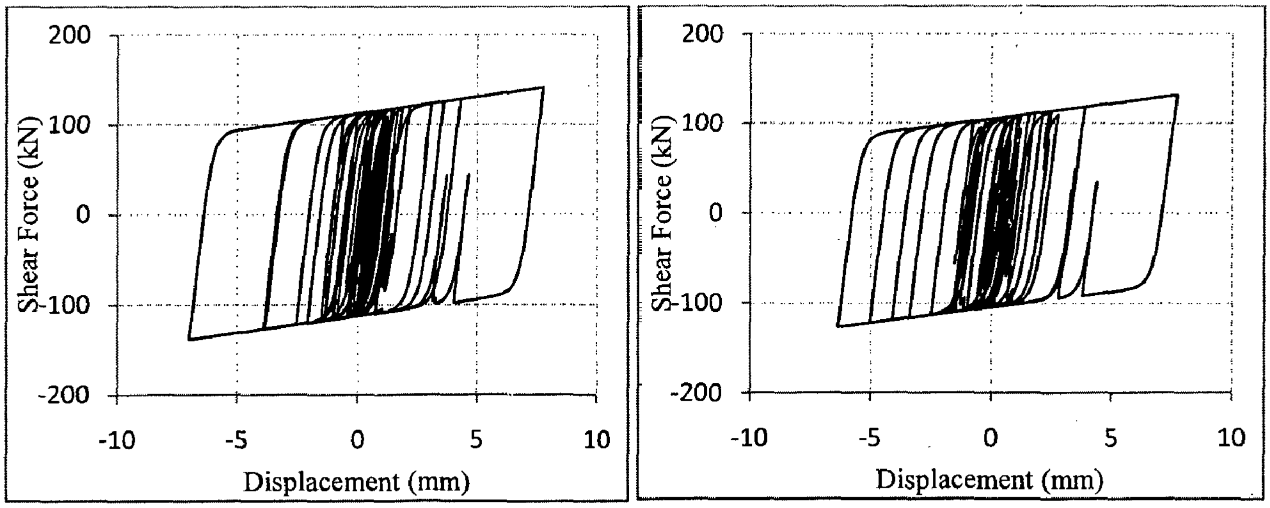
As the braces have been designed for a buckling load equal to yield load for the shear link times an over strength factor in all the storey, all the brace respond elastically and yielding is limited to shear link only for MCE level of earthquake under nonlinear static pushover analysis. Figure 4.26-4.27 below shows the variation of axial force in brace with respect to drift ratio. Force in both tension and compression brace have been plotted with respect to drift ratio.



e) Fourth floor shear link f) Fifth floor shear link

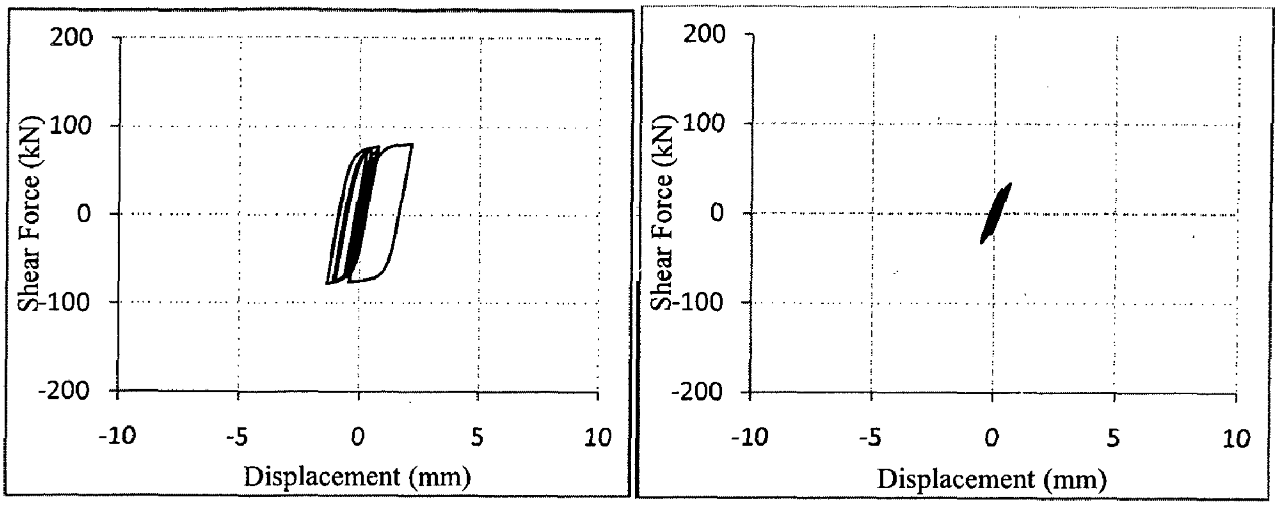


c) Second floor shear link d) Third floor shear link



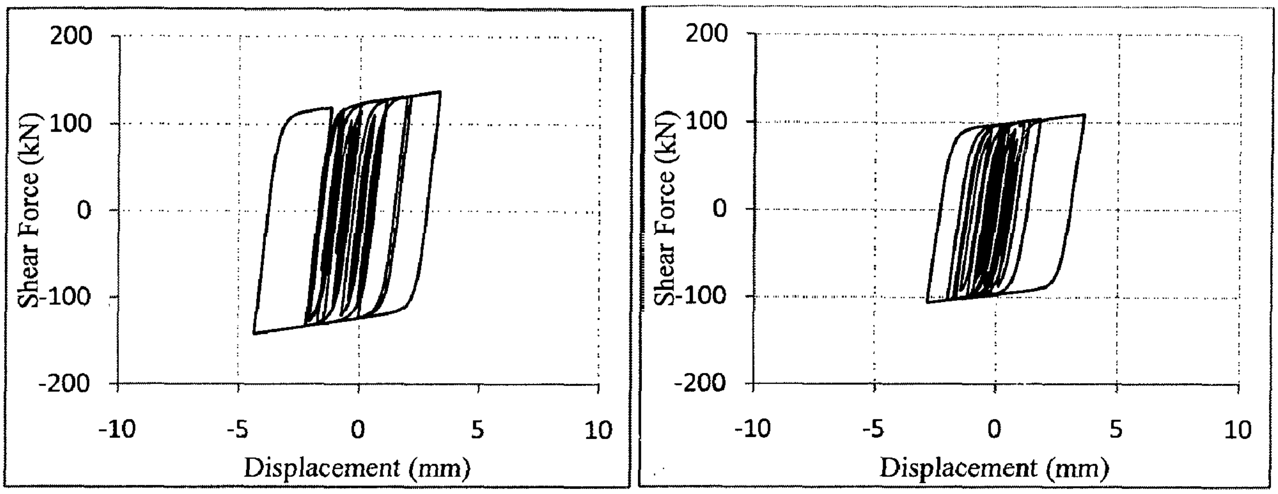
a) Ground floor shear link b) First floor shear link

Fig.4.22- Typical link hysteresis in X-direction for DBE level of earthquake



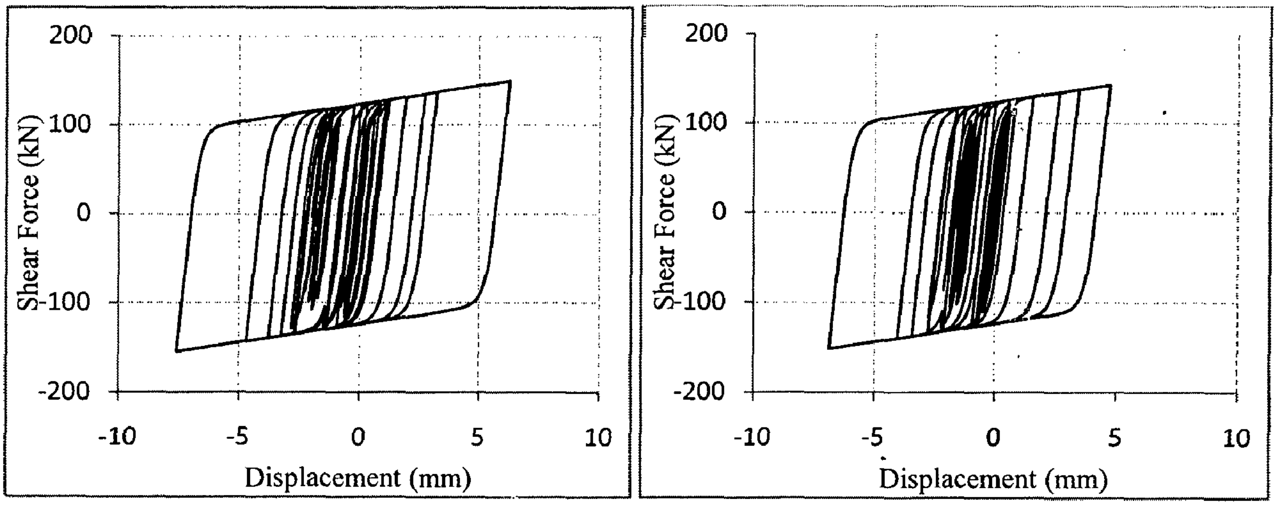
e) Fourth floor shear link

f) Fifth floor shear link



c) Second floor shear link

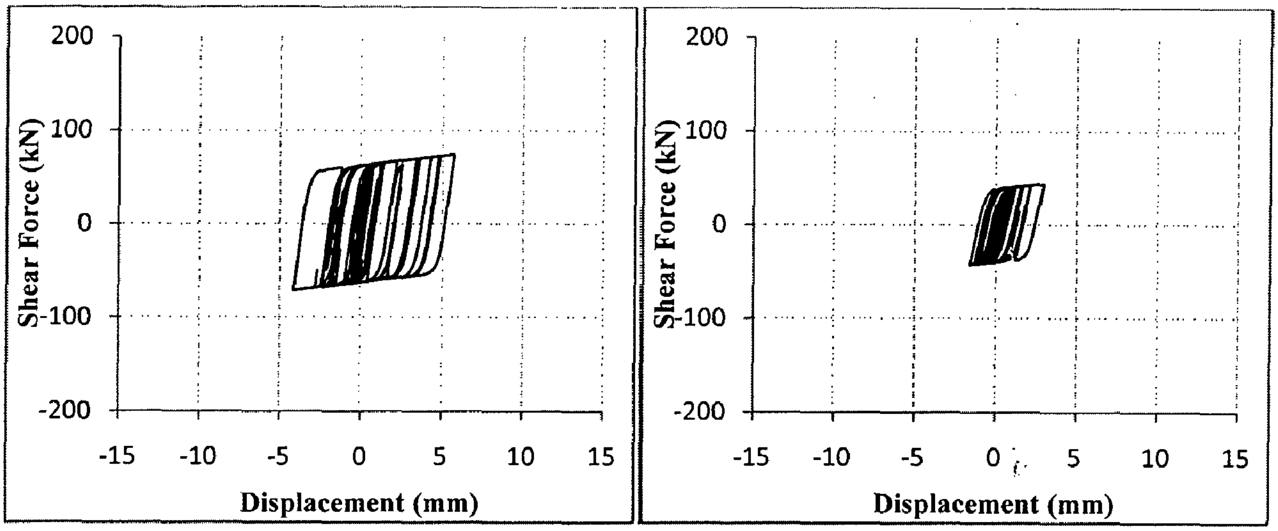
d) Third floor shear link



a) Ground floor shear link

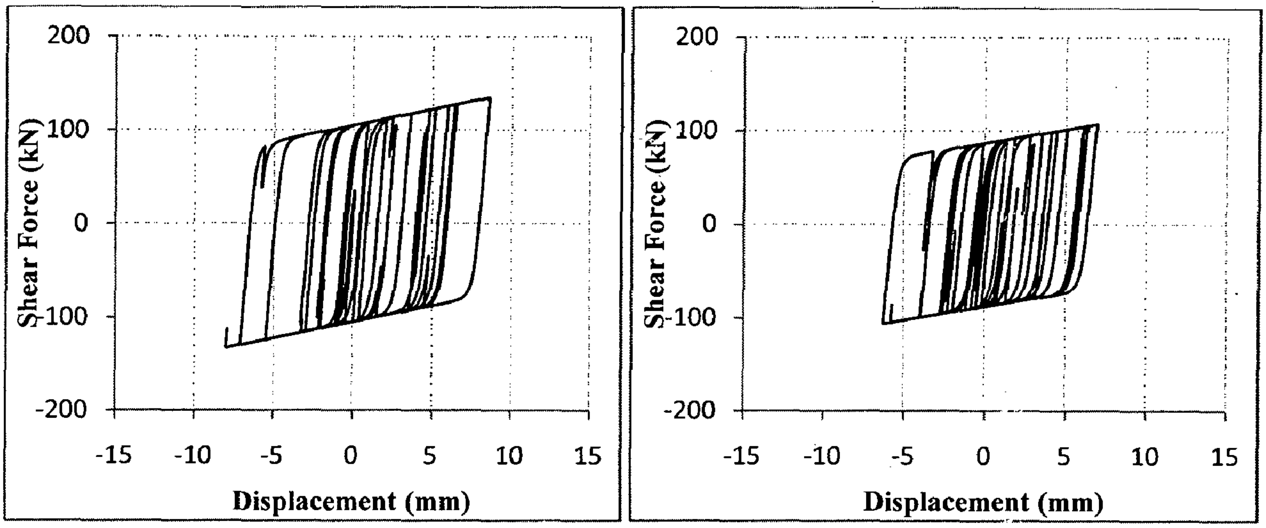
b) First floor shear link

Fig.4.23- Typical link hysteresis in Y-direction for DBE level of earthquake



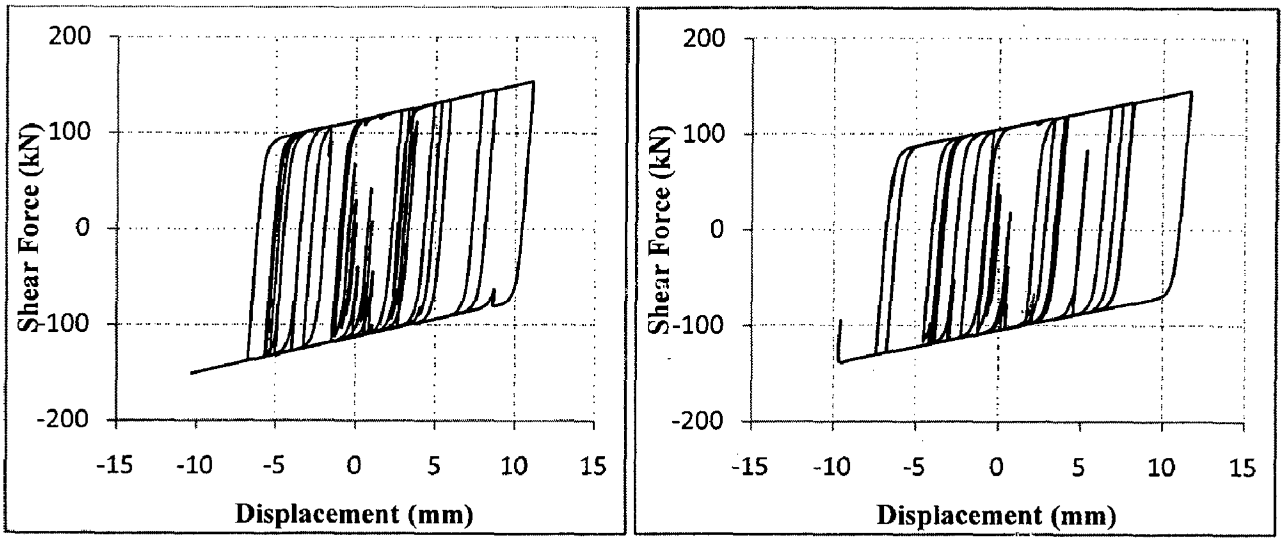
e) Fourth floor shear link

f) Fifth floor shear link



c) Second floor shear link

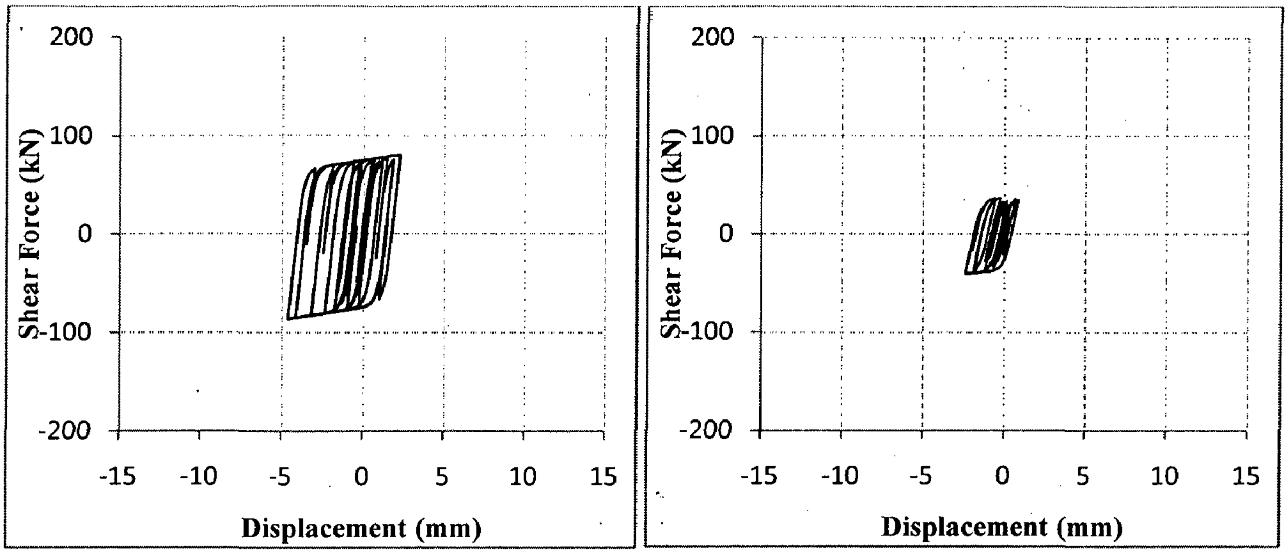
d) Third floor shear link



a) Ground floor shear link

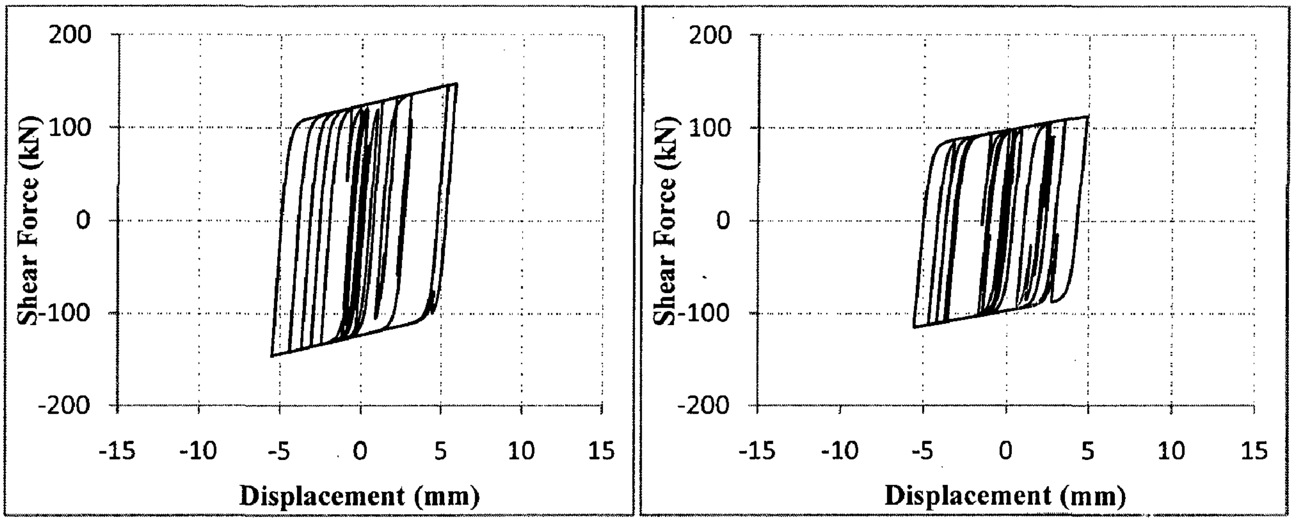
b) First floor shear link

Fig.4.24- Typical link hysteresis in X-direction for MCE level of earthquake



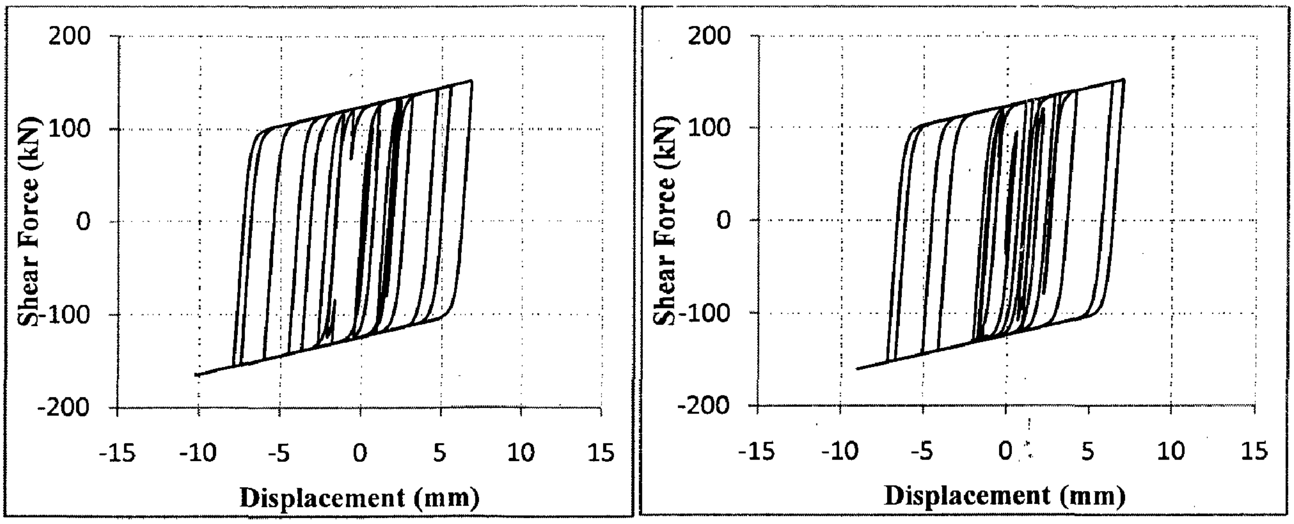
e) Fourth floor shear link

f) Fifth floor shear link



c) Second floor shear link

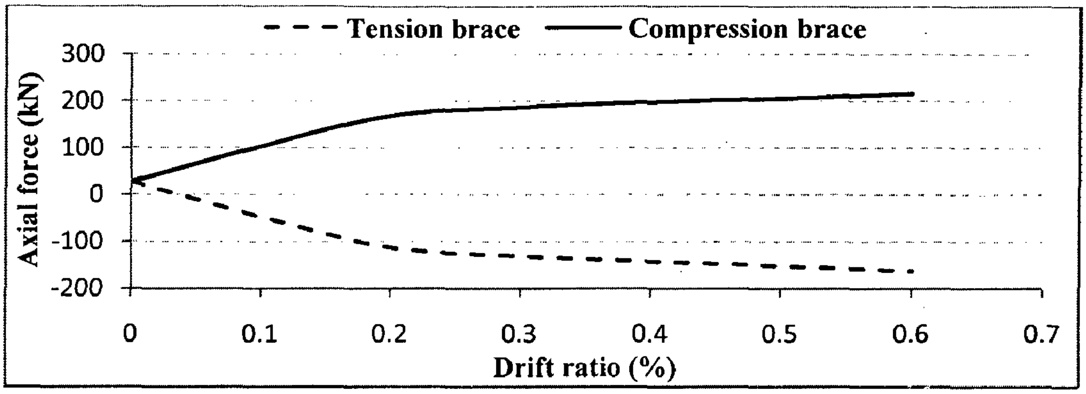
d) Third floor shear link



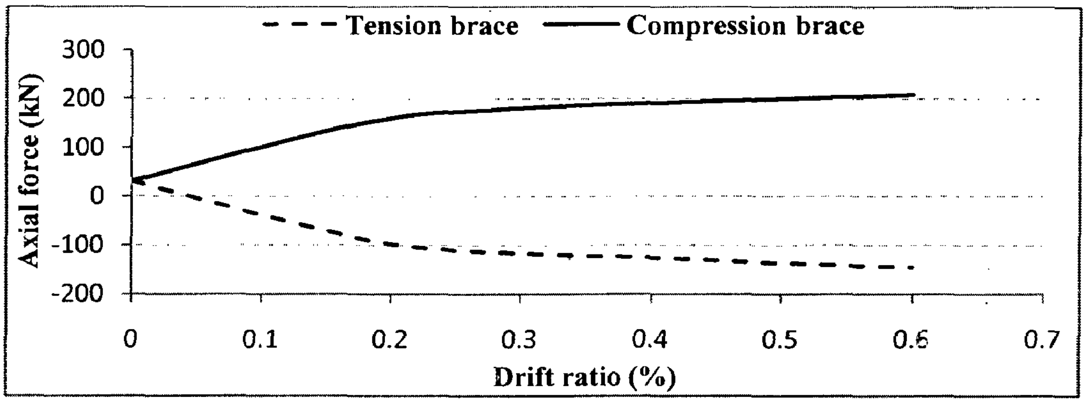
a) Ground floor shear link

b) First floor shear link

Fig.4.25- Typical link hysteresis in Y-direction for MCE level of earthquake

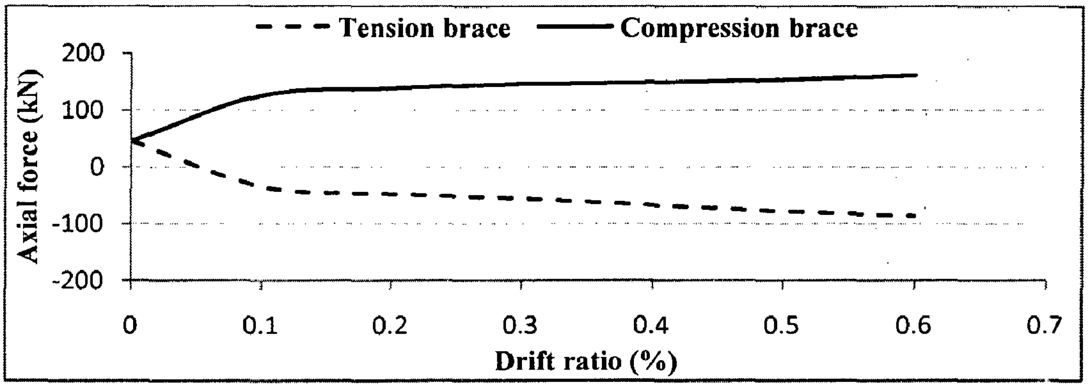


a) Axial force in brace in X-direction

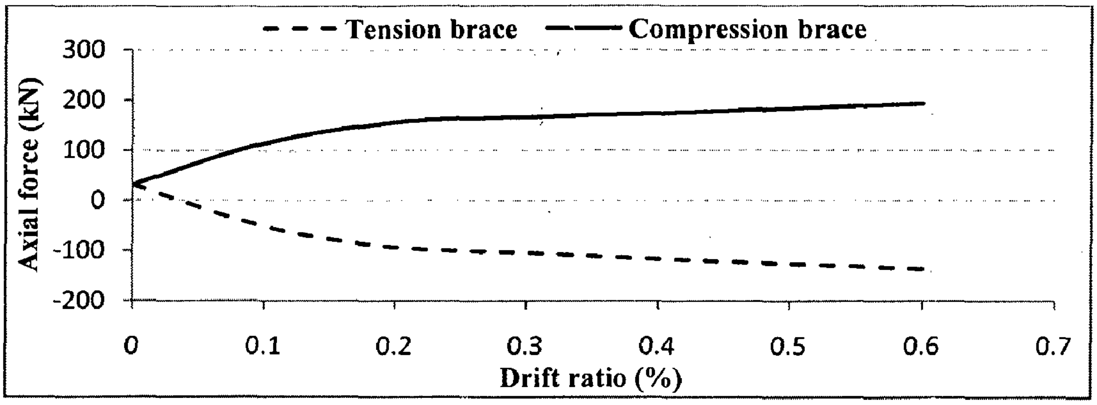


b) Axial force in brace in Y-direction

Fig.4.26- Variation of axial force in brace with respect to drift ratio for Bldg-1



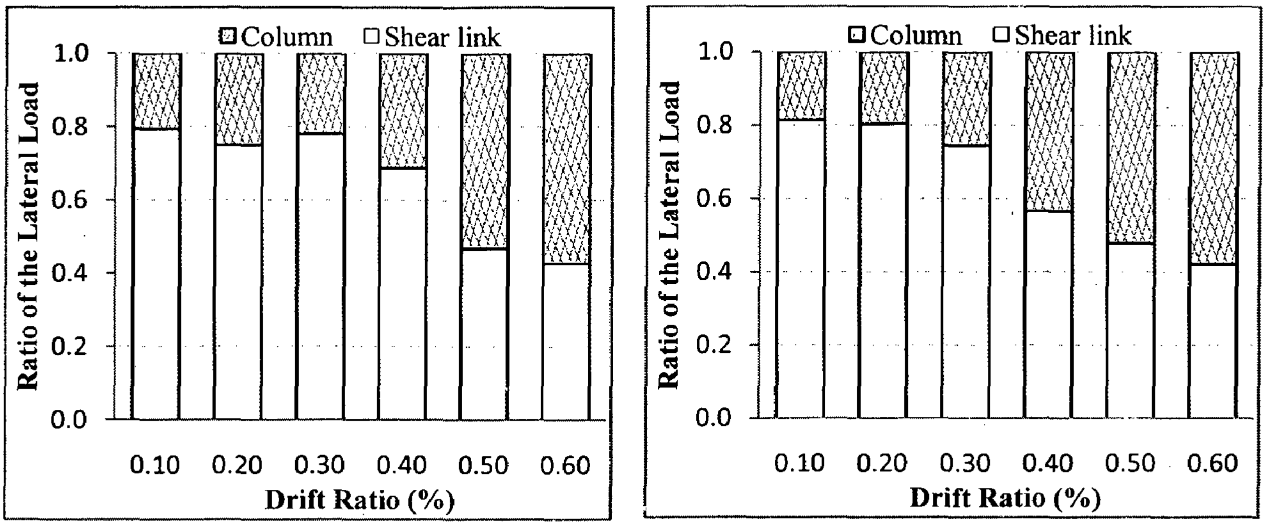
a) Axial force in brace in X-direction



b) Axial force in brace in Y-direction

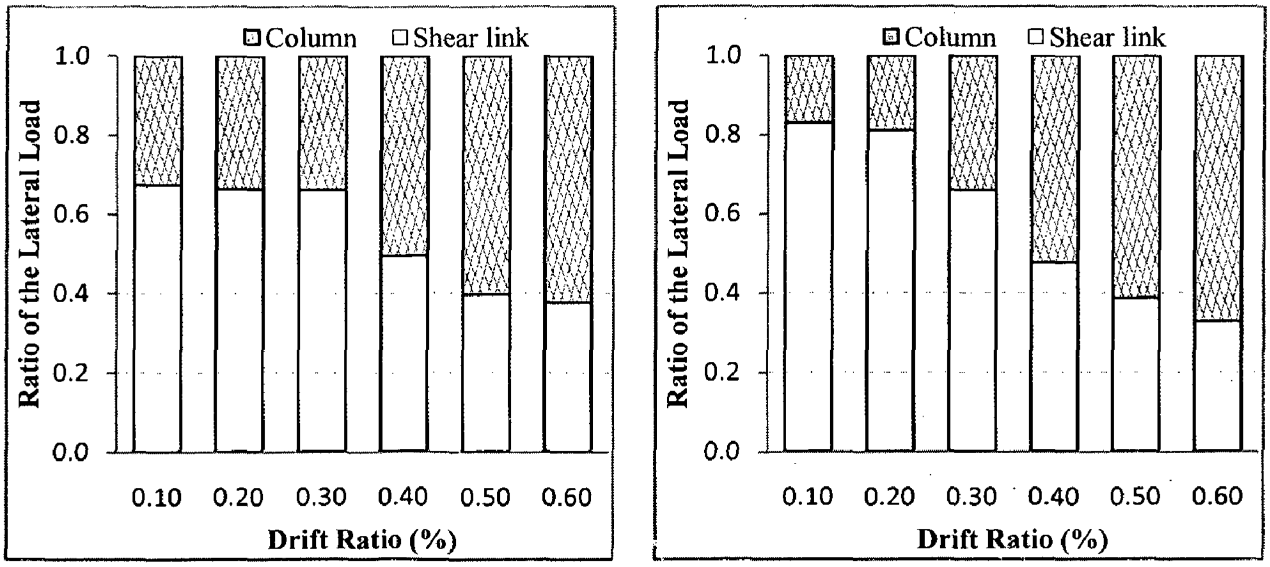
Fig.4.27- Variation of axial force in brace with respect to drift ratio for Bldg-2

Figure 4.28-4.29 below show the plot of drift ratio v/s ratio of the lateral load. This ratio of the lateral load is the ratio of shear force in all the shear links provided in the ground floor to the total base shear at different drift ratios. Because of significant contribution of the aluminium shear link in sharing the lateral load seismic demand on the existing reinforced concrete member is reduced considerably. For a small drift upto 0.3 % aluminium shear link share about 75 to 80% of the total lateral load as shown. As the drift ratio goes on increasing the capacity of shear link in sharing the lateral load goes on reducing. This can be seen from the graph below.



a) X-direction b) Y-direction

Fig.4.28- Comparison of the lateral load of the shear links and reinforced concrete columns (Bldg-1)



a) X-direction b) Y-direction

Fig.4.29- Comparison of the lateral load of the shear links and reinforced concrete columns (Bldg-2)

4.4- Behaviour of the Columns

The column subjecting to extreme forces after providing shear link brace frame has been studied and reported in this section. The behaviour of the forces in column after adding braces are going to change. Figure 4.32-4.39 below shows the variation of axial force and moment with respect to drift ratio for original deficient and retrofitted buildings. From the axial force plot it has been observed that the axial compression in column is increasing on one side whereas on the other side of brace the axial force is decreasing. But again from the nonlinear static pushover analysis it has been observed that instead of changing the pattern of load in columns the response of columns for MCE level of earthquake remains in immediate occupancy level.

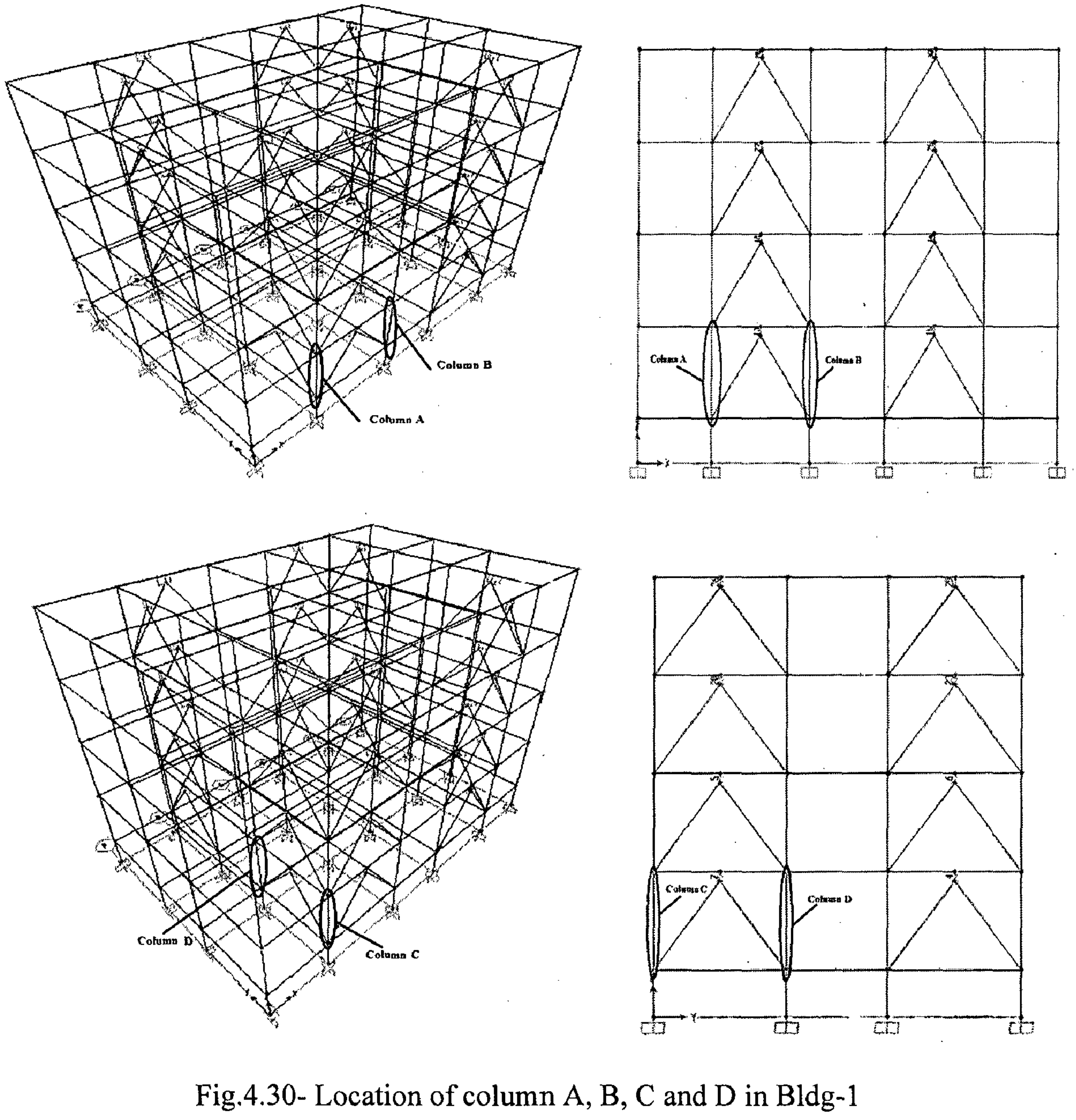


Fig.4.30- Location of column A, B, C and D in Bldg-1

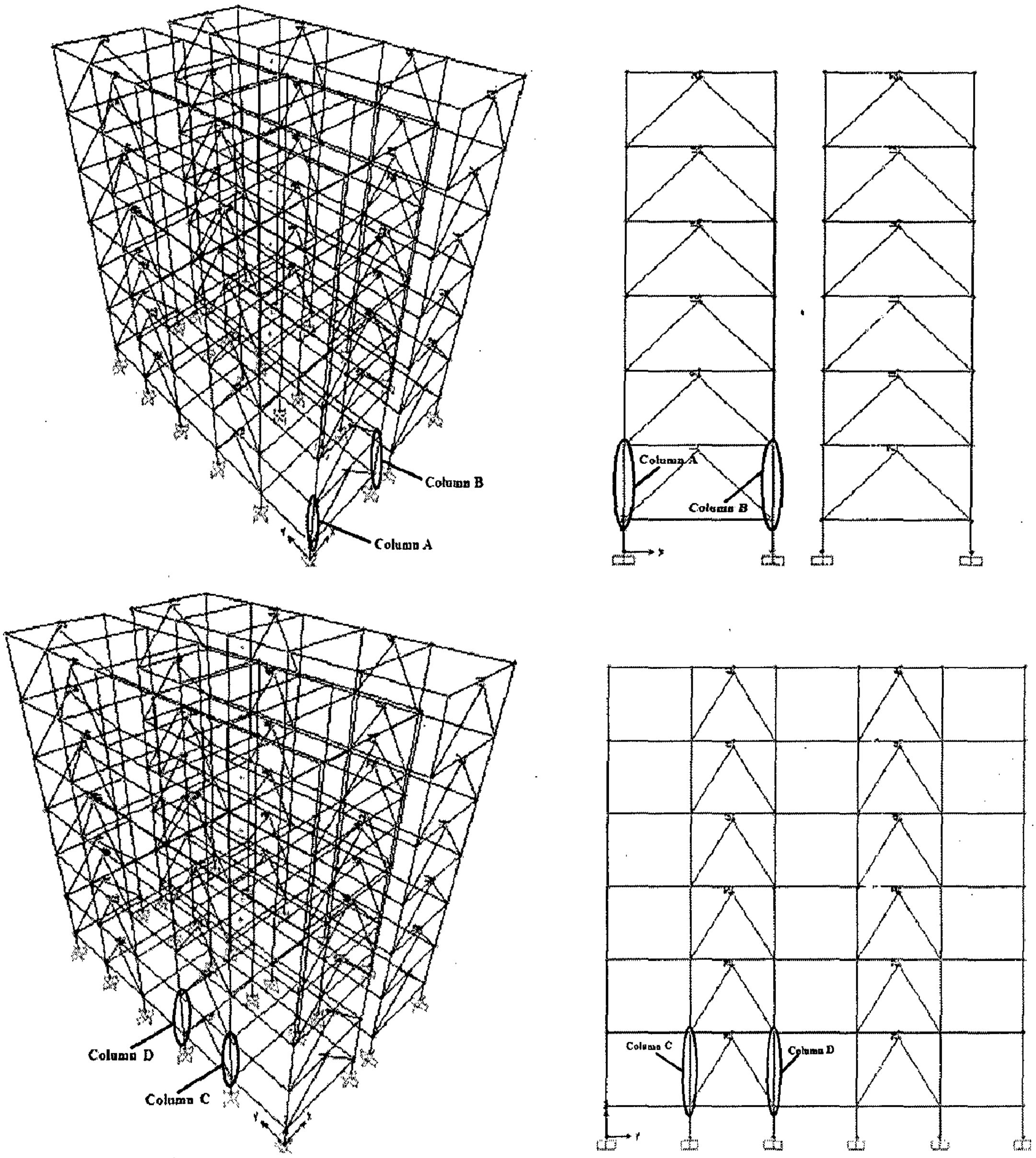
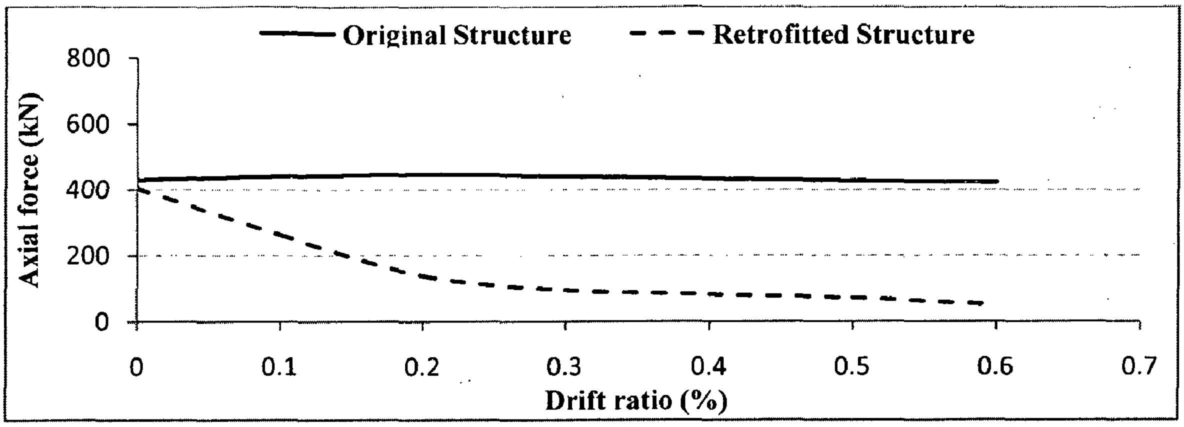
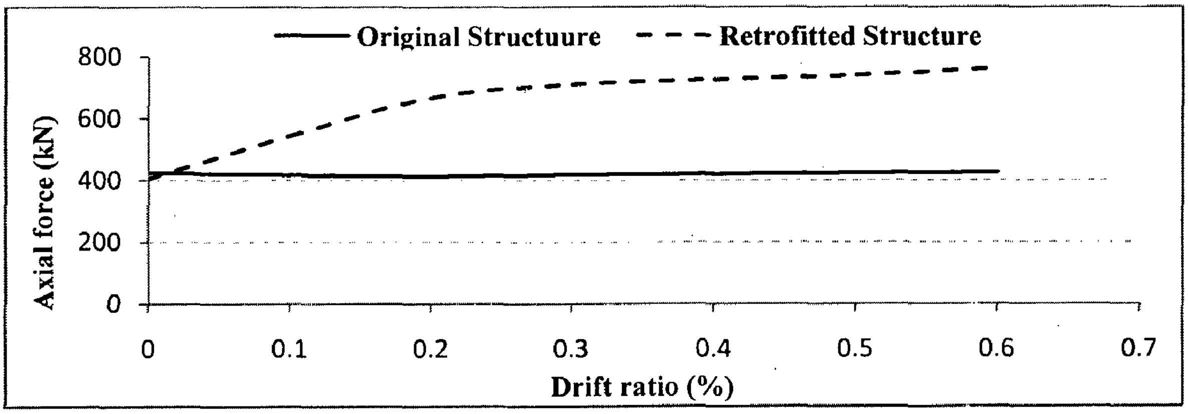


Fig.4.31- Location of column A, B, C and D in Bldg-2

Moment has been plotted with respect to drift ratio. Figure 4.33, 4.35, 4.37 and 4.39 below shows the plot for moment v/s drift ratio for typical columns as shown in Fig. 4.30 and 4.31 above. The addition of braces in the building results in reduction in moment of the columns. For the whole range of drift ratio the moment in the columns have been reduced.

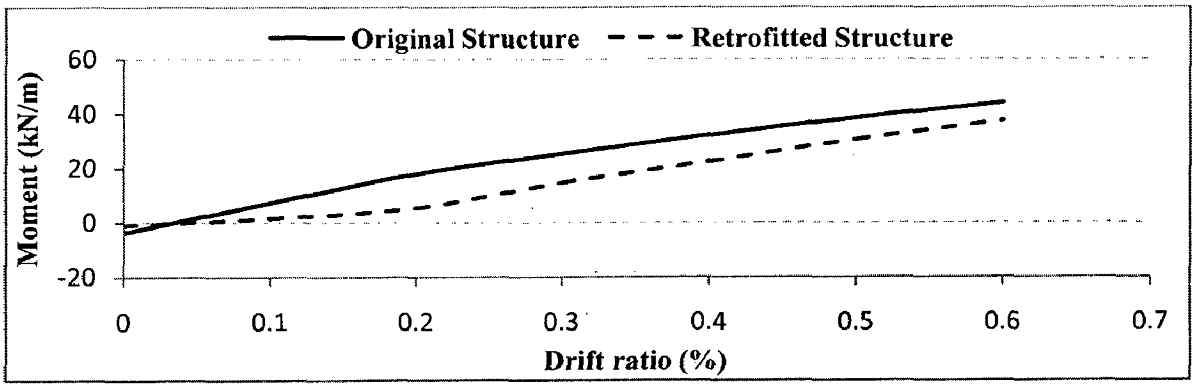


(a) Variation of axial force in column A

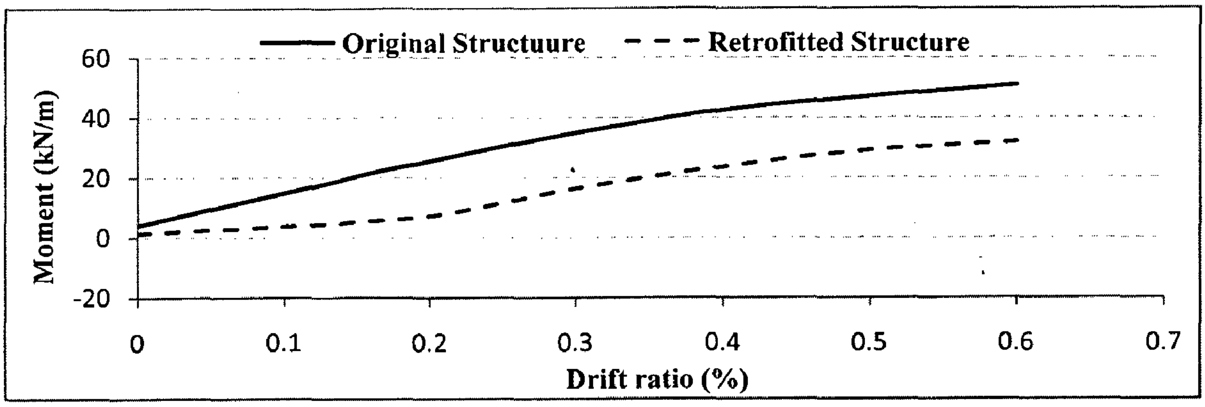


(b) Variation of axial force in column B

Fig.4.32- Variation of axial force in typical column in X-direction (Bldg-1)

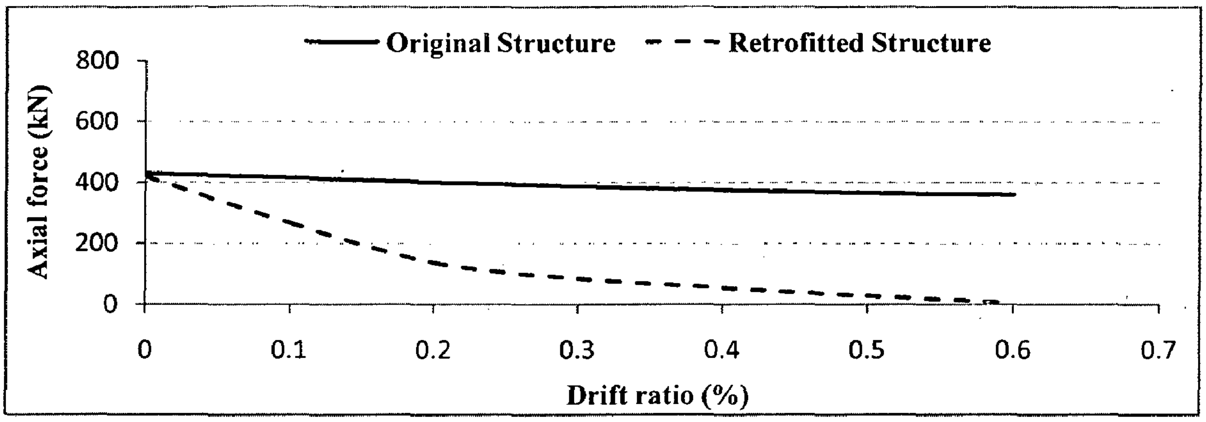


(a) Variation of moment in column A

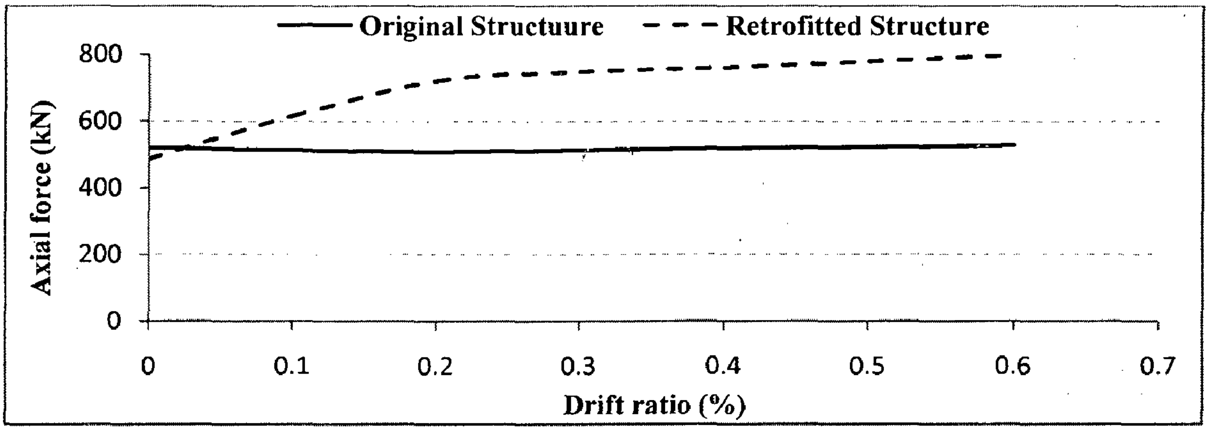


(b) Variation of moment in column B

Fig.4.33- Variation of moment in typical column in X-direction (Bldg-1)

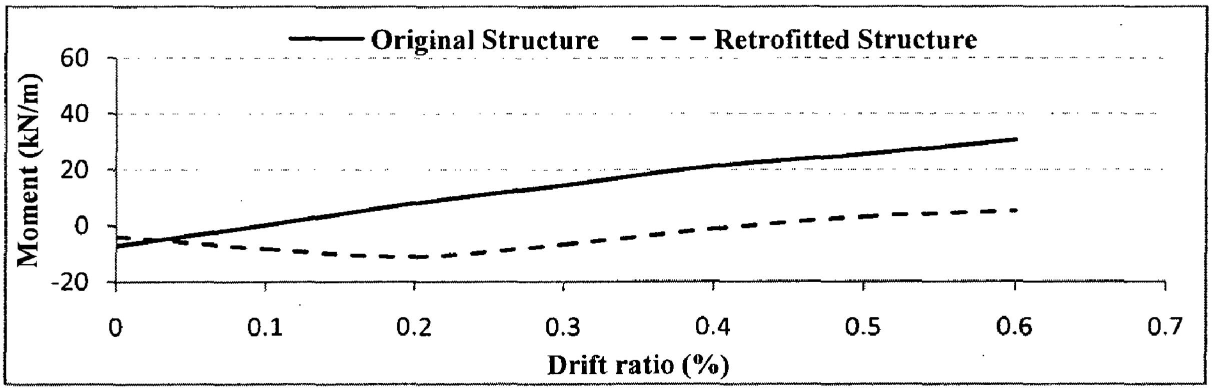


(a) Variation of axial force in column C

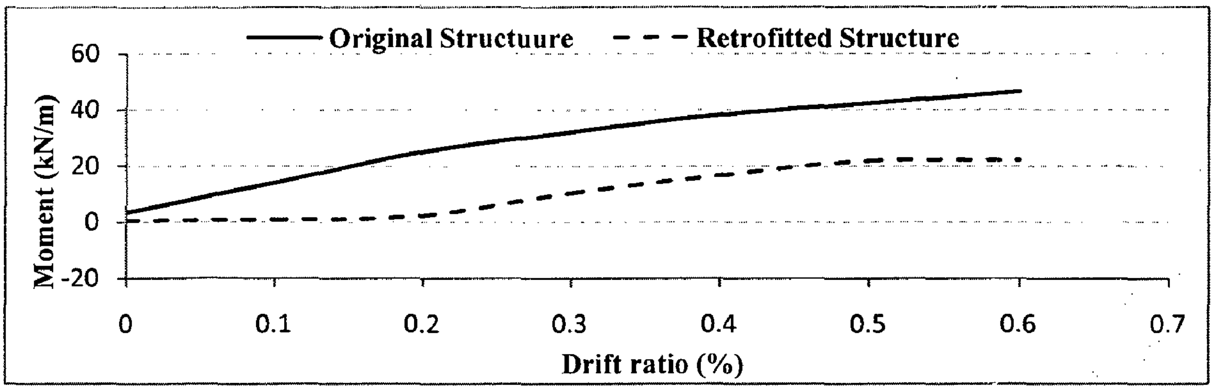


(b) Variation of axial force in column D

Fig.4.34- Variation of axial force in typical column in Y-direction (Bldg-1)

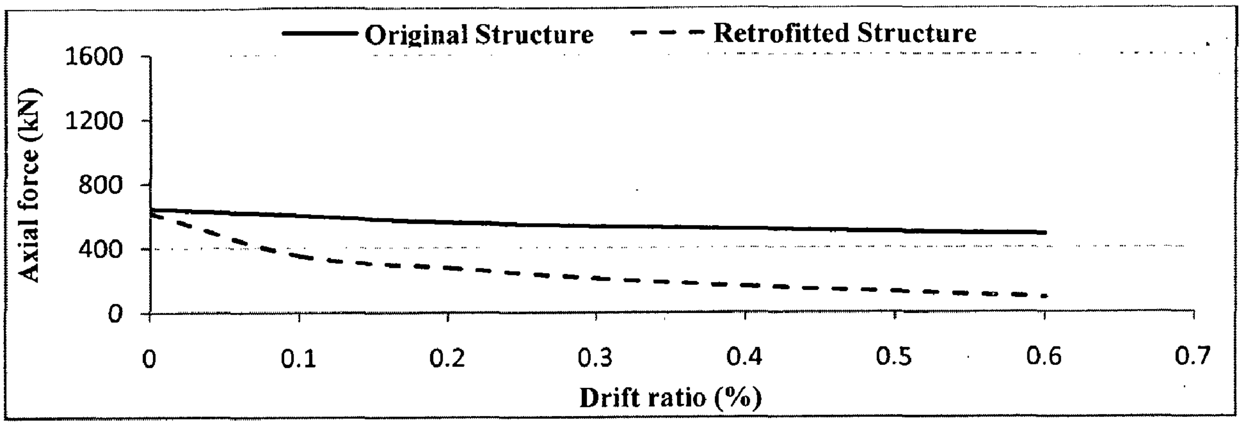


(a) Variation of moment in column C

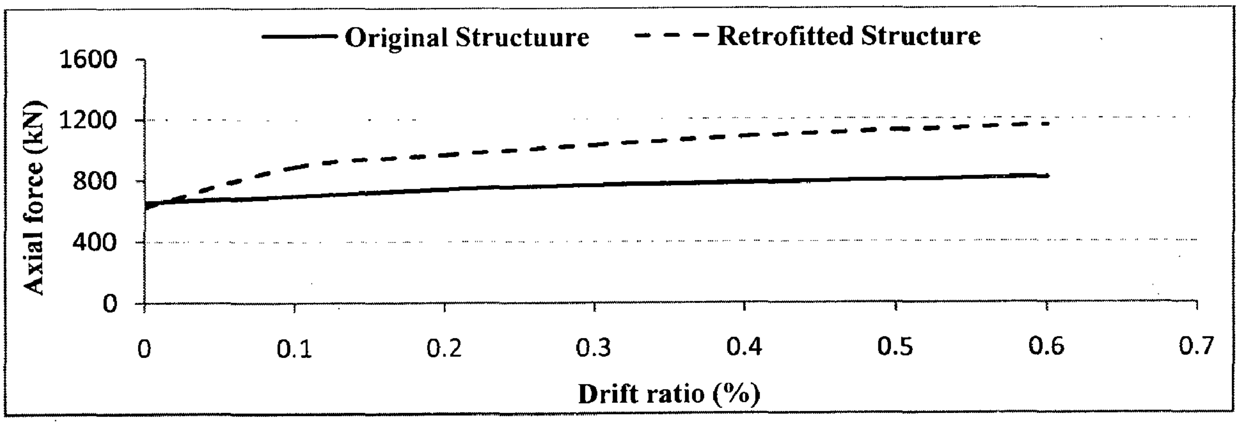


(b) Variation of moment in column D

Fig.4.35- Variation of moment in typical column in Y-direction (Bldg-1)

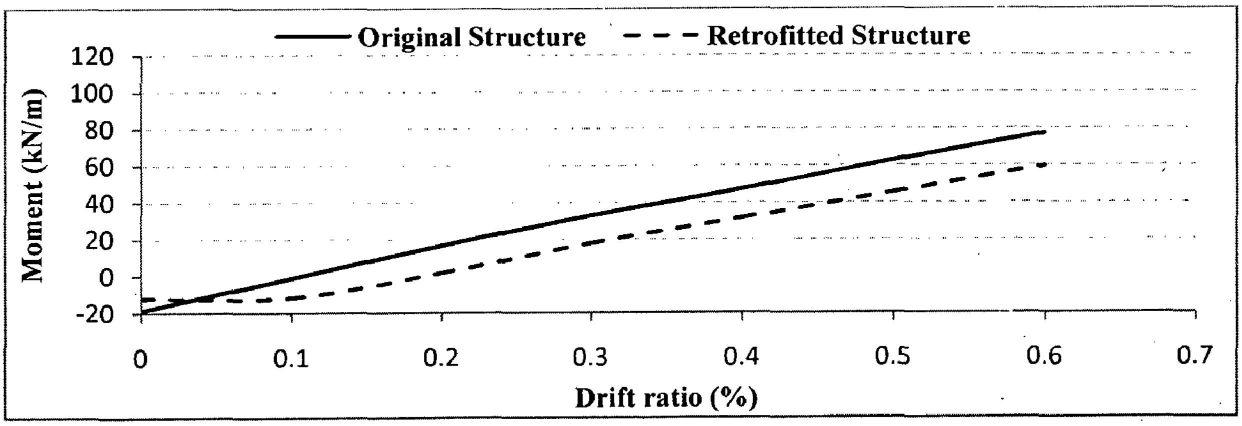


(a) Variation of axial force in column A

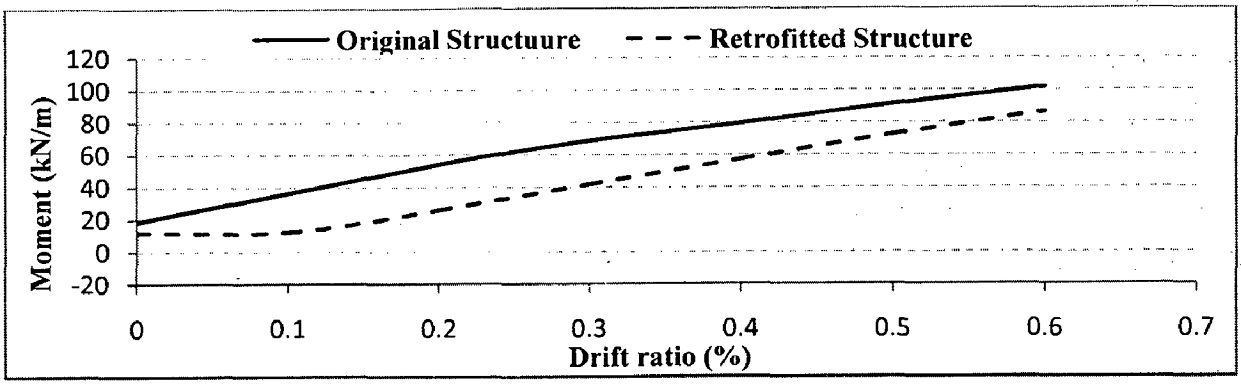


(b) Variation of axial force in column B

Fig.4.36- Variation of axial force in typical column in X-direction (Bldg-2)

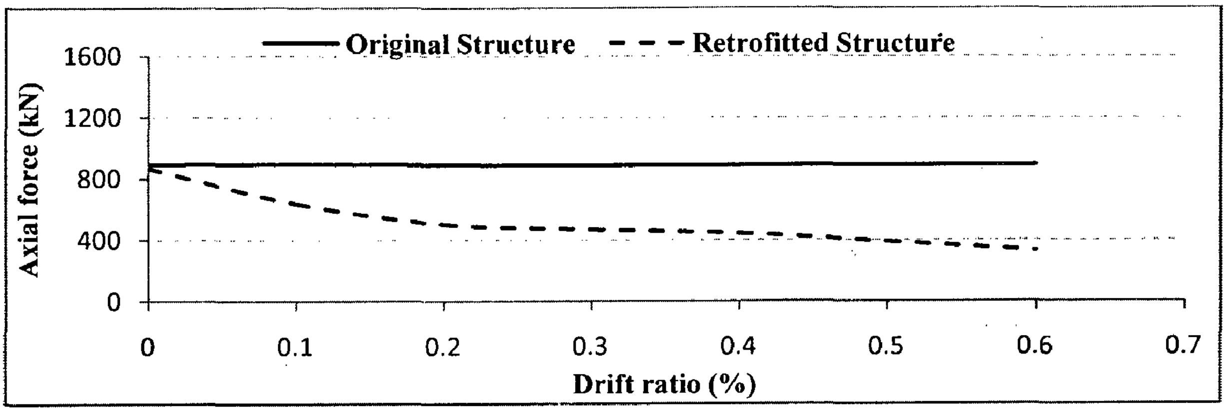


(a) Variation of moment in column A

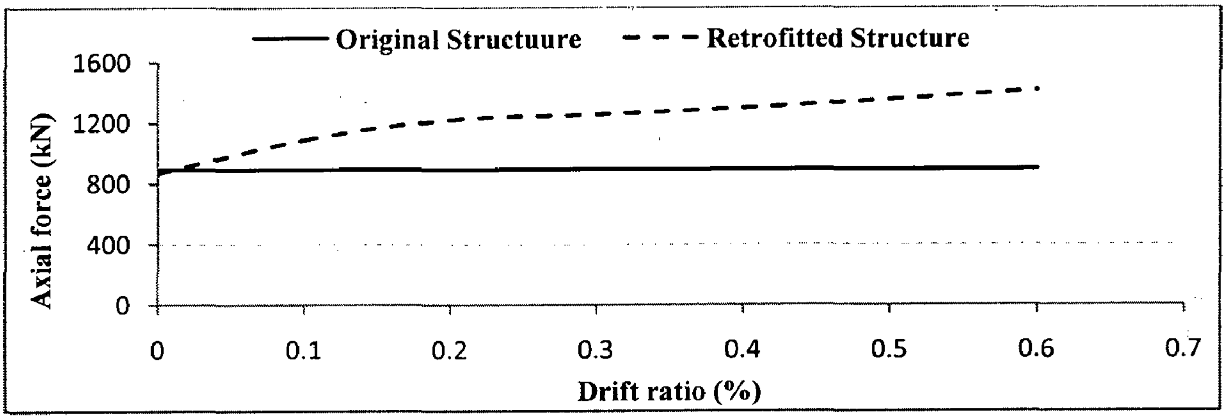


(b) Variation of moment in column B

Fig.4.37- Variation of moment in typical column in X-direction (Bldg-2)

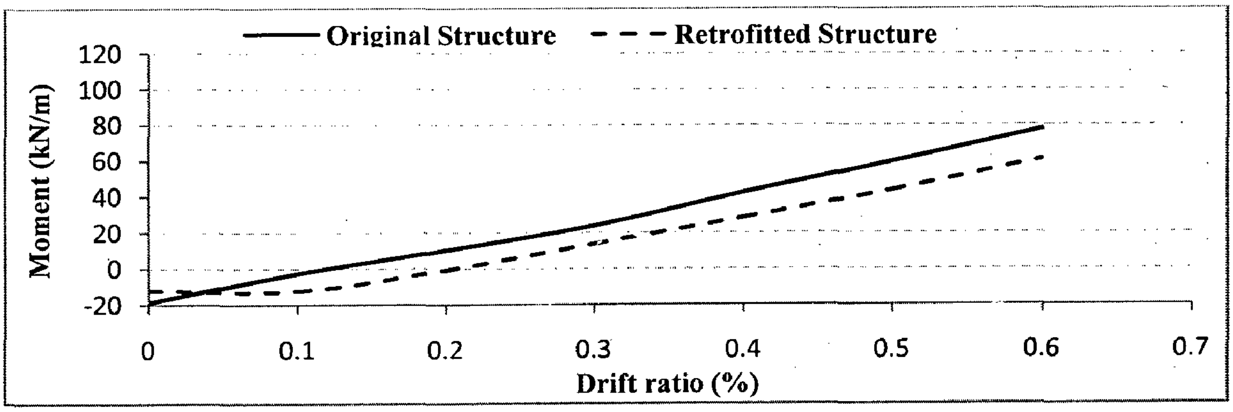


(a) Variation of axial force in column C

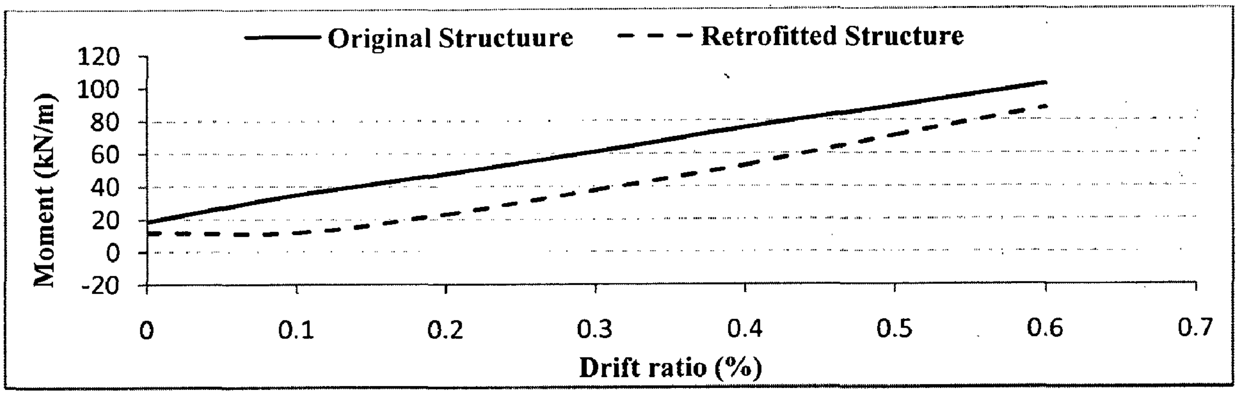


(b) Variation of axial force in column D

Fig.4.38- Variation of axial force in typical column in Y-direction (Bldg-2)



(a) Variation of moment in column C

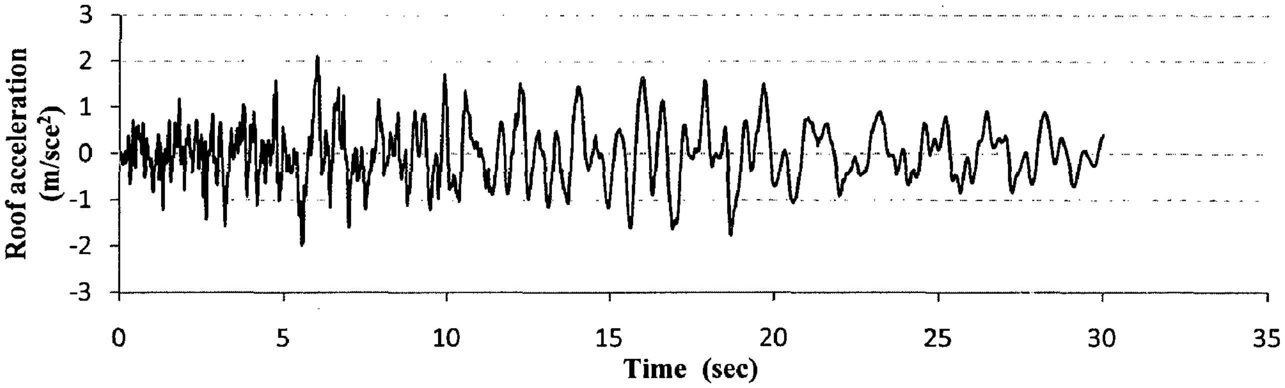


(b) Variation of moment in column D

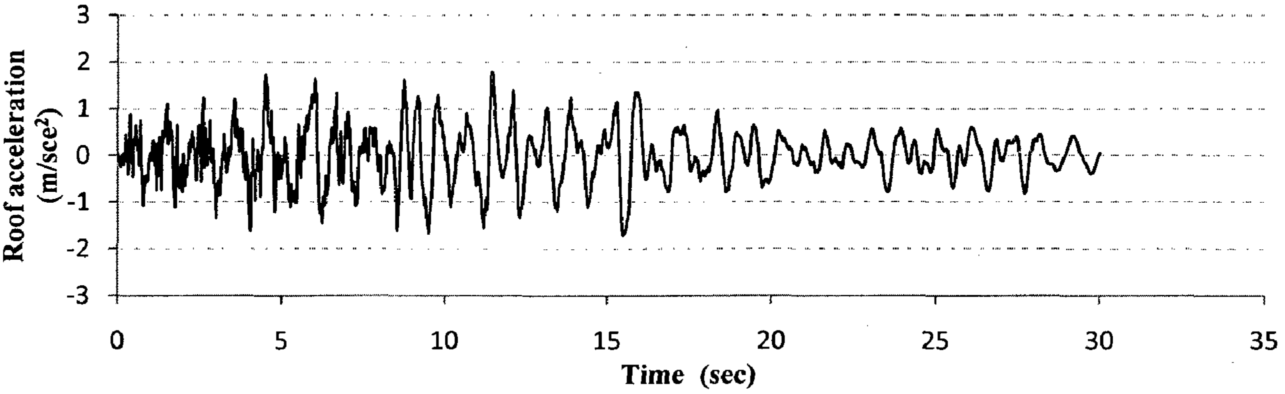
Fig.4.39- Variation of moment in typical column in Y-direction (Bldg -2)

4.5- Acceleration and Displacement History

Figure 4.40-4.43 below shows the acceleration and displacement history at the roof level respectively. It has been observed form the plot that the acceleration at the roof has not been reduced considerably, but the displacement at the roof level has reduced significantly after retrofitting. Thus, the non-structural damages in buildings can be reduced because of reduced displacement at the roof.

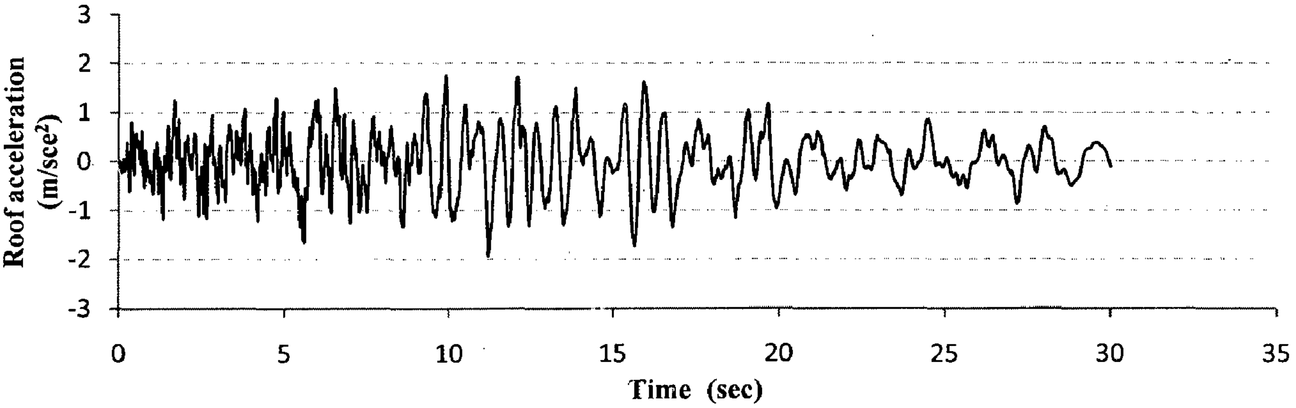


(a) Original deficient building

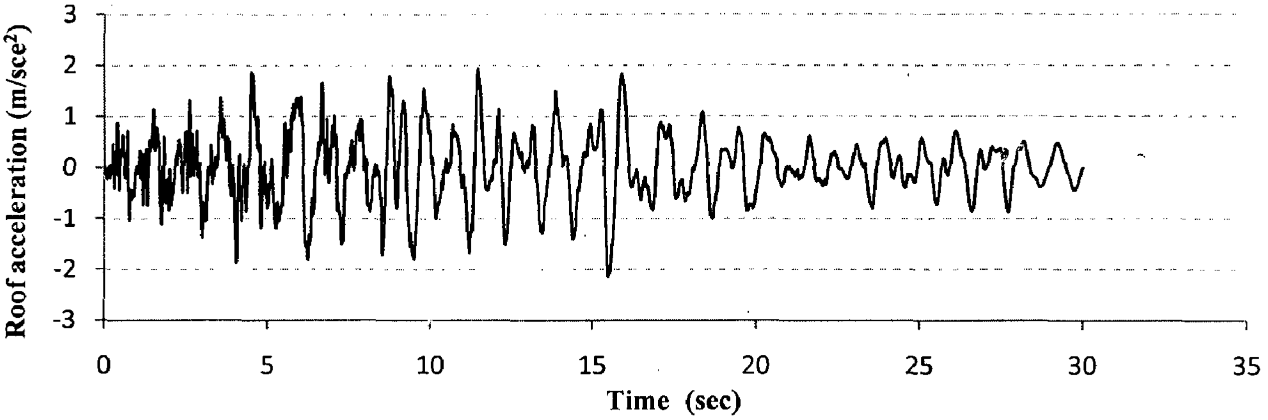


(b) Retrofitted building

Fig. 4.40- Roof acceleration in X-direction

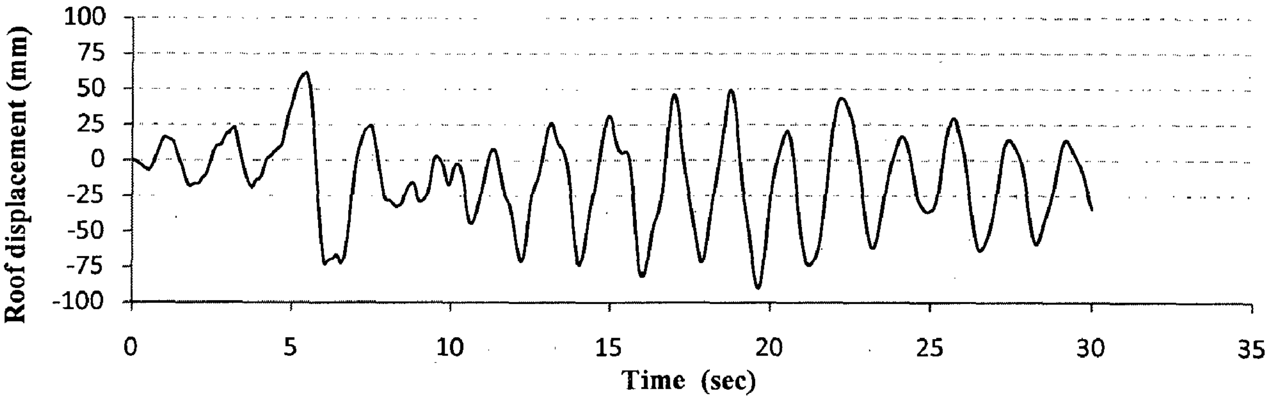


(a) Original deficient building

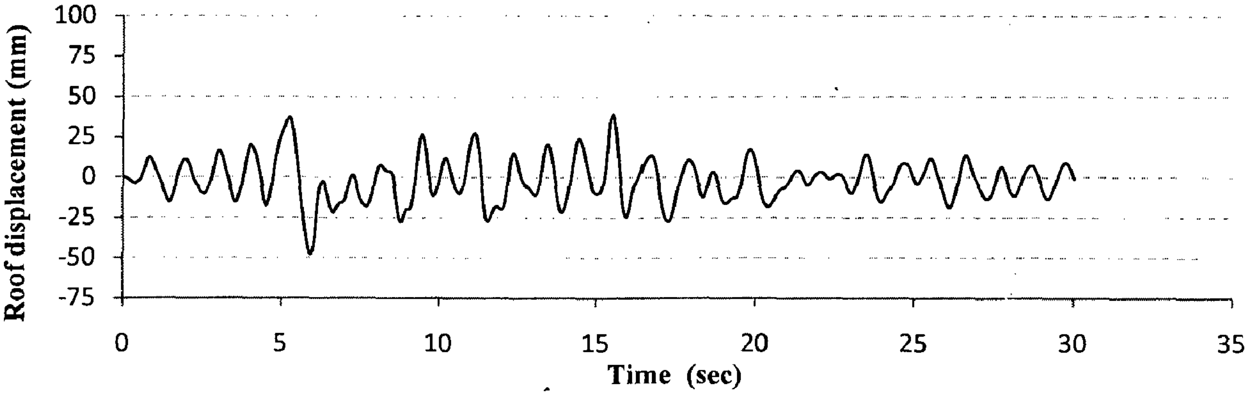


(a) Retrofitted building

Fig. 4.41- Roof acceleration in Y-direction

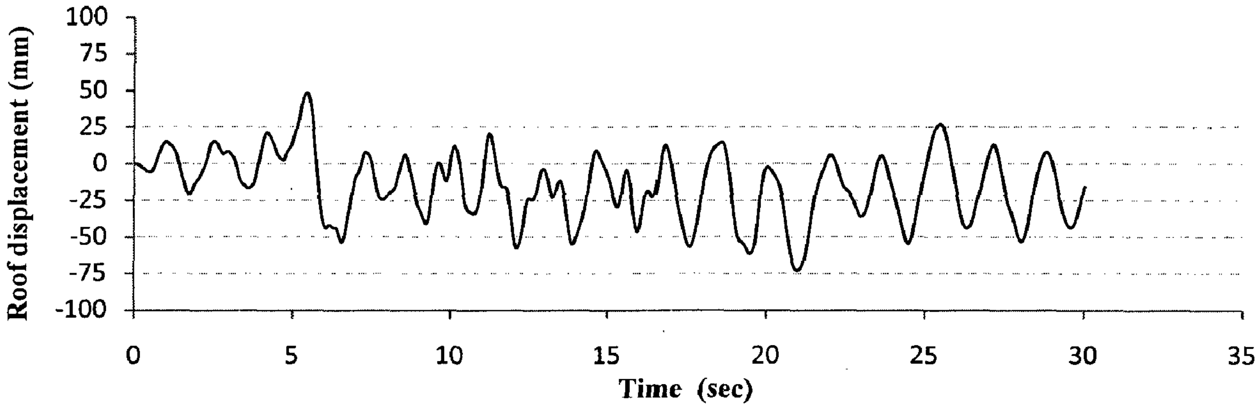


(a) Original deficient building

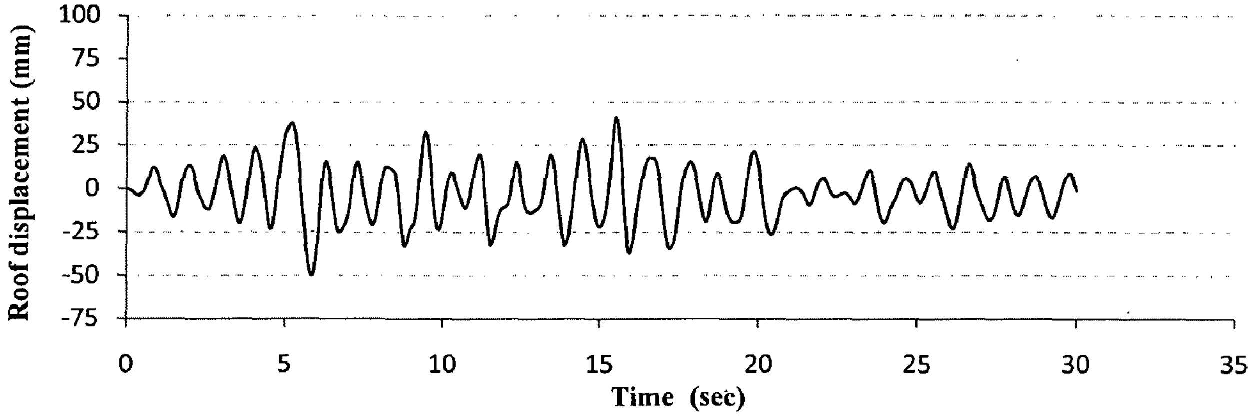


(a) Retrofitted building

Fig. 4.42 -Roof displacement in X-direction



(a) Original deficient building



(a) Retrofitted building

Fig. 4.43- Roof displacement in Y-direction

Chapter -5

CONCLUSIONS AND RECOMENDATIONS

The present work has been undertaken to suggest improvement in the performance of buildings designed for gravity load only using aluminium shear link with chevron braces. A large number of existing buildings have been designed to resist gravity loads only without any seismic codal compliance. This is a matter of great concern. To avoid major disaster in future these building have to be upgraded so as to resist major earthquake.

A systematic review of the literature has been undertaken to improve the performance of buildings designed for gravity load only. The state of the art in this area has brought out the following, (i) different energy absorbing devices are available in the market, (ii) application of different energy absorbing devices in improving the seismic performance of deficient buildings in the past, (iii) behaviour of various types of metallic dampers and with particular reference to aluminium shear link with chevron braces in improving the seismic performance of deficient buildings.

Based on available literature a designed methodology has been presented to upgrade the seismic performance of these deficient buildings. Step by step procedure has been presented by using combination of aluminium shear link and chevron braces. The provision of such device in deficient building increases the stiffness and strength of deficient building. It is also important to study the placement of these devices at suitable bays in either direction.

Placement of Shear Link with Chevron Braces

Placement of these shear link with chevron braces in building is an important factor. Location of these braces in structural frame has been decided based on various available options and considering the functionality of building. The best possible locations have been selected by trials that gives the higher performance.

Performance of Overall Building

From the present study it has been observed that the RC frame buildings designed for gravity loads suffer significant level of damages due to earthquake. The performance of RC buildings designed for gravity loads only as evaluated from the nonlinear static pushover analysis lies in life safety and collapse prevention range for DBE and MCE level of earthquakes respectively. These buildings are than retrofitted by using chevron braces with aluminium shear link. From the analysis it has been observed that the performance of retrofitted buildings have improved considerably. For DBE level of earthquake the members of Bldg-1 and 2 respond elastically in both orthogonal directions. Similarly for MCE level of earthquake Bldg-1 performance lies in immediate occupancy range, but the retrofitted Bldg-2 performed to life safety level for MCE level of earthquake. Some beams suffer life safety level of damage even after retrofitting as these beams are having larger span. The addition of shear link with braces has resulted in increase of base shear. The increase in base shear for Bldg-1 is 25% and 26% in X and Y directions respectively. Similarly for Bldg-2 increase in base shear is 14% and 11% in X and Y directions respectively. Drift ratio in retrofitted buildings have also been reduced to almost 50% as that of in original deficient RC buildings. Additions of shear link with the help of chevron braces are going to change the pattern of loading in some frame members e.g. the columns to which braces are connected, but the response of these members are also in the acceptable range.

Behaviour of Shear link and Chevron Braces

Though the addition of shear link with chevron braces are going to stiffen the structure and will increase earthquake forces but shear link with chevron braces plays an important role in limiting the forces that are transferred to primary members of the structure, thus damages in primary member of the building are reduced considerably. After following a major earthquake it is most likely that shear links will be the only component of braced frame that will need replacement. From nonlinear time history it is proved that the shear link dissipates the earthquake induced energy in the form of stable hysteresis. From the analysis it is observed that for small drift ratio upto 0.3% chevron braces with shear link share about 75-80% of the total lateral load in Bldg-1 in both the directions, similarly for Bldg-2 it share about 65% of the total lateral load in X-direction and 75-80% of the total lateral in Y-direction. Beyond 0.3% drift ratio the yielding of link starts and columns take more share of lateral loads.

Ductility of Building

Addition of shear link with braces has improved the performance of building as concluded above but the ductility of building has reduced to some extent. This is on account of increase in column forces due to the addition of braces.

The device consisting of chevron braces with aluminium shear link is very simple and can be put in building very easily. The device is found very effective in terms of economy and overall seismic performance. It is also suitable as it does not interfere with the normal operations of the building.

Scope for the Future Work

Chevron braces with aluminium shear link have proved to be an effective retrofitting measure in regular buildings. The study can be further extended in retrofitting of irregular buildings by making use of shear link with chevron braces. Different materials available in market, which have excellent energy dissipation capacity, can be experimentally tested to use as a shear link.

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