Seismic Retrofitting of Reinforced Concrete Structures

إعادة التشكيل الزلزالية للهياكل الخرسانية المسلحة

by

ARCHANA THUNDY KANDY

Dissertation submitted in fulfilment

of the requirements for the degree of

MSc STRUCTURAL ENGINEERING

at

The British University in Dubai

October 2018

DECLARATION

I warrant that the content of this research is the direct result of my own work and that any use made in it of published or unpublished copyright material falls within the limits permitted by international copyright conventions.

I understand that a copy of my research will be deposited in the University Library for permanent retention.

I hereby agree that the material mentioned above for which I am author and copyright holder may be copied and distributed by The British University in Dubai for the purposes of research, private study or education and that The British University in Dubai may recover from purchasers the costs incurred in such copying and distribution, where appropriate.

I understand that The British University in Dubai may make a digital copy available in the institutional repository.

I understand that I may apply to the University to retain the right to withhold or to restrict access to my thesis for a period which shall not normally exceed four calendar years from the congregation at which the degree is conferred, the length of the period to be specified in the application, together with the precise reasons for making that application.

Signature of the student

COPYRIGHT AND INFORMATION TO USERS

ABSTRACT

In the current scenario of ever-increasing incidence of earthquakes around the world, we need to carry out a macro level study and review the causes of earthquakes and consequential effects of it and identification at Source and tracking it to the Society. Natural calamities cause severe and more often than not, indescribable suffering, loss of lives and livelihood, destruction of houses, properties, infrastructural installations and facilities, separation of humans and great many other losses. Thus the implications are multifaceted involving economic hardships, human sufferings and a great deal of severely adverse social effects.

For ascertaining the structure responsiveness and resistance to earthquakes, the source/ path parameters and site features need to be studied and depicted scientifically to assess the geophysical as well as seismological effects. The buildings which were designed and constructed according to the old codes may not satisfy the requirements of the presently used seismic codes and design practices. Therefore, it is crucial to reassess the codes to prevent economic loss and loss to human life and property caused during earthquakes.

Buildings damaged in the past earthquakes or the existing buildings deficient to resist seismic forces need to be retrofitted for better performance in future earthquakes. Seismic evaluation is the first step before retrofitting procedures to determine the most exposed and vulnerable members and deficiencies in a structure in order to protect them during an event of an unexpected earthquake. The seismic retrofit/rehabilitation procedure intents to enhance seismic performance of the structure and rectify the insufficiencies by enhancing stiffness, strength or deformation capacity and also improves connections. The present study deals with the seismic retrofitting of two structures located in diverse seismic zones. Each structure is retrofitted under two different schemes and the results are compared to find out a better method of retrofitting for both structures. After the analysis, the results were compared under time periods, base shear, and modal participating mass ratios, bending moments and shear forces for all beams and moment capacities and axial forces for all columns and were found satisfactory to withstand the design earthquake forces and results are tabulated in the report.

ملخص

في ظل السيناريو الحالي لزيادة حدوث الزلازل في جميع أنحاء العالم ، نحتاج إلى إجراء دراسة على المستوى الكلي ومراجعة (أ) أسباب الزلازل والآثار المترتبة عليها ؛ (ب) تحديد الهوية في المصدر وتتبعه في المجتمع ؛ و (ج) وإيجاد الاستجابة الإنشائية للحركة المتصلة بالزلزال ، والتي تتميز في الغالب بقيم مترابطة بشدة من القوة والصلابة والليونة. وتتسبب الكوارث الطبيعية في حالات خطيرة وأحياناً كثيرة ، تتمثل في خسائر بالأرواح وسبل العيش وتدمير المنازل والممتلكات والبنية التحتية والمرافق. وبالتالي، فإن هذه الآثار متعددة الأوجه تنطوي على صعوبات اقتصادية، وكثير من الآثار الاجتماعية الضارة.

وللتحقق من ردة فعل البنية للزلازل ، يجب دراسة مؤشرات المصدر / المسار وخصائص الموقع وتقييمها علمياً على التأثيرات الجيوفيزيائية والزلزالية. إن المباني المصممة والمشيدة وفق النظم القديمة قد لا تلبي متطلبات النظم الزلزالية وممارسات التصميم المستخدمة حاليًا. لذلك، من الضروري إعادة تقييم القوانين لمنع الخسارة الاقتصادية والخسارة في الأرواح البشرية والممتلكات التي تقع خلال الزلازل.

المباني التي تضررت في الزلازل الماضية أو المباني القائمة التي تعاني من قلة مقاومة للقوى الزلزالية تحتاج إلى التعديل التحديثي لأداء أفضل في الزلازل المستقبلية. يعتبر التقييم الزلزالي الخطوة الأولى قبل إجراءات التعديل التحديثي لتحديد أكثر المكونات عرضةً وضعفاً وأوجه القصور في المبنى أثناء الزلزال المتوقع. تهدف عملية إعادة التأهيل الزلزالي إلى تحسين الأداء الزلزالي وتصحيح أوجه القصور من خلال زيادة قوة أو صلابة أو تشوه القدرة وتحسين الاتصالات. تتناول هذه الدراسة التحديث الزلزالي للمنشأتين الواقعتين في مناطق زلزالية مختلفة. يتم تحديث كل بنية في نظامين مختلفين وتتم مقارنة النتائج بطريقة أفضل في التعديل التحديثي لكلا الهيكلين.

بعد الدراسه والتحليل كانت النتائج في حدود المسموح به حسب المواصفات القياسيه وتم عرضها في هذا التقرير.

AKNOWLDGEMENT

I would like to thank god without whom nothing is possible.

This dissertation was conducted under the supervision of Professor Abid Abu-Tair. I would like to express my sincere appreciation for his guidance, inspiration, valuable comments and continuous support that was provided to me throughout this dissertation and on the course of my entire study at British University at Dubai.

Above all, I would like to acknowledge my sincere gratitude to my father, mother and brother for their continuous support, help and encouraging throughout and guiding me with valuable suggestions throughout my work and motivating me at the crucial stage of this dissertation.

Table of Contents

LIST OF ILLUSTRATIONS

LIST OF TABLES

LIST OF ABBREVIATIONS:

a_g = Design ground acceleration

T = Time period of response spectrum

T_B = Lower limit of the period

T_C = Upper limit of the period

DL = Dead load on the structure

LL = Live Load

Lr = Live loads on the roof

E-X = earthquake loads on X-direction

E-Y = earthquake loads on Y-direction

M_{ux} = Moment of resistance of beams

V_S = Base shear

Δ_s = The storey drift for the structure

S_x = Shear force demands of the beams

P_{ur} = Axial load of the member

M_{urx} =

M_{urz} =

Ux =

Uy =

Uz =

C = Capacity of the member

D = Demand of the member

1. INTRODUCTION:

Earthquakes are the most common natural calamities which are more dangerous than others due to its unpredictability. In simple terms it is an abrupt outburst of energy in the earth's crust, forming seismic waves. Upon its occurrence, saving lives and properties is very difficult. The seismic waves will radiate outwards from the focal source of the earthquake at various speeds, causing intense shaking at the earth's surfaces, leading to the collapse of buildings and roads and causing severe miseries for the people and the environment around (Varam and Kumar, 2017).

For ascertaining the structure responsiveness and resistance to earthquakes, the source and path parameters as well as site features need to be studied and depicted scientifically to assess the geophysical as well as seismological effects. While addressing resistance issues the serviceability of the structure together with structural damage control and failure prevention aspects are to be adequately considered with the ultimate objectives of down time reduction, minimizing repair budgets and defending life so as to lead to performance boosting designs. It is important to address strength, stiffness, ductility, hyper-strength and damping, and present these in a hierarchical structure to decide on any earthquake response strategy.

The maximum seismic resistance of a building is very critical to avoid or minimize the damage happening to a structure during the occurrence of an earthquake. The decision to strengthen it as a precautionary measure before the earthquake depends on the current seismic resistance status of the building. To render a building earthquake resistant, three levels of improvement of the existing RC frame buildings are possible: (1) Repair wherein only visual or cosmetic modifications are effected; (2) Restoration in which case structural modifications are made so as to restore the original performance of the building; (3) Retrofitting where structural modifications are carried out to ensure higher performance of the building than that of the original structure. The aforesaid measures of improvements are decided only after the evaluating the current seismic resistance status of the structure (Agarwal, Chourasia and Parashar, 2002).

Seismic Evaluation is the process of assessment and comparison between additional resistance requirements as demanded by the earthquake scenarios and the current readiness of the structure to meet such challenges. Any retrofitting strategies can be decided only after this exercise which is done by calculating Demand-Capacity ratio (DCR) for each structural member of the building, after which deficient members are further evaluated for retrofitting (Agarwal, 2002).

The critical evaluation and decision to enhance the seismic resistance capability of the building leads to Seismic Retrofitting which is the process of upgrading the structural strength of an existing damaged or undamaged building to enable it to resist probable earth quake generated forces in future.

1.1 Static and Dynamic Analysis:

The process of Seismic Evaluation and strengthening of an existing structure commences with the visual study of the drawings and the structure itself. The aim of the seismic evaluation is to diagnose the system weakness including the lack of secondary load paths, incomplete moment resisting systems, lack of bracing or shear wall to transfer all loads successfully to the foundation for distribution of mass and stiffness. The analysis uses FEMA 273 for structure-01 and IS1893:2002 for structure-02 in the case study section of this document. The analysis uses code specified lateral loads applied to a structure, modelled with an elastic linear stiffness. Response spectrum analysis is used in the dynamic analysis to ensure better distribution of forces that take care of the system irregularities and mass distribution (Marletta, 2005).

1.2 Research Objectives:

The research question for this study is to check the stability of structures to withstand the effect of earthquakes. The main argument raises on the structures which are designed according to the old seismic codes and design practices will be able to withstand the design earthquake forces.

This dissertation executes the following:

- Review Seismic action, seismic resistance and vulnerability. Discussion of previously occurred earthquakes and Risk Assessment.
- Discussion of common problems affecting seismic performance of RC structures and techniques used in seismic retrofitting. Needs and strategies of retrofitting explained.
- Case study comprises of carrying out seismic evaluation in the both the structures selected in different seismic zones.
- If the structures are found deficient to withstand the earthquake forces, then seismic retrofitting is carried out.
- Selection of retrofitting schemes.
- To carry out retrofitting with two different schemes on the structure.
- After the retrofitting procedure, the results are compared for both schemes in the structure under:
    - Time Period

- Modal Participating Mass Ratios
- Base Shear
- Bending moment and shear forces for beams
- Moment of resistance and Axial forces for columns
- Story Drifts.

2. LITERATURE REVIEW-01

2.1 SEISMIC ACTION:

In order to determine the seismic action; importance class of the structure, conditions of the project, geological studies and ground investigation should be carried out.

The earthquake motion at a given point on the surface is represented by an elastic ground acceleration response spectrum, hence forth calling it as “Elastic Response Spectrum”. The horizontal seismic action is described by two orthogonal components assumed as being independent and represented by the same response spectrum. When an earthquake affecting a site are generated by widely differing sources, the possibility of using more than one shape of spectra to enable design seismic action should be considered (Marletta, 2005).

In these cases, different values of a_g (design ground acceleration) is defined for each type of spectrum. Seismic action also depends on the characteristics of the structure such as energy dissipation capacity and fundamental modes of vibration. The elastic spectrum of eurocode-8 (EN-1998) for different values of damping is shown in figure-01.

For very small time periods, the value of pseudo-spectral acceleration is considered as the peak ground acceleration (PGA). At this point, the structure is rigid. When the time period is increased, response spectrum is defined for each region (Marletta, 2005).

In the first region ($0 \leq T \leq T_B$) where T_B is the lower limit of the period, the spectral ordinates escalates linearly with the time period and on the second region where ($T_B \leq T \leq T_C$); T_C is the upper limit of the period. At this region, the spectral ordinates are independent of the time period and when the period is greater than T_C; spectral ordinates decreases rapidly leading to the reciprocal of the time period. When the time period decreases to T_D, spectral ordinates will decrease further leading to the reciprocal of the time period squared.

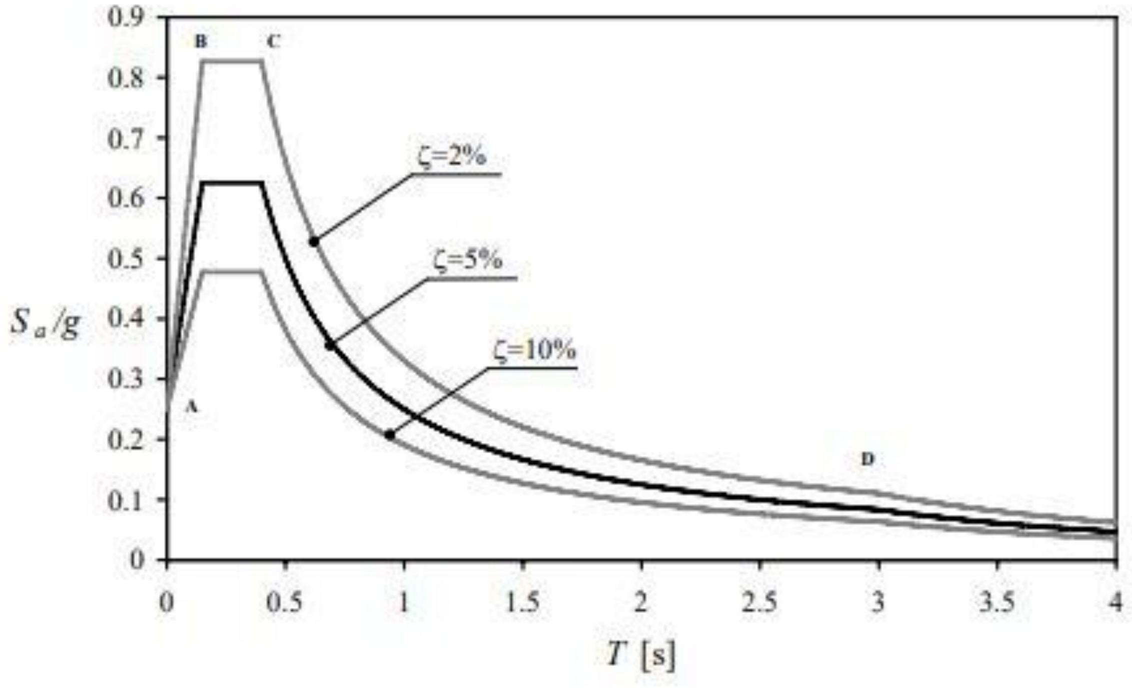


Figure-01: Elastic design spectrum of Eurocode 8, soil type A

(Source: Oliveto, 2002)

2.2 SEISMIC RESISTANCE AND VULNERABILITY:

A vulnerability evaluation is compulsory step before commencing the retrofitting procedures in order to identify the number of deficient members and the quantum and nature of such deficiencies. A probable earthquake is first characterized by means of a Design Spectrum which depends on the energy dissipation capacity through the Structure Behavior Factor. Assuming that the Structure Behavior Factor for the structure being considered can be evaluated, the Design Spectrum can be prepared. An example of such a spectrum is shown in Figure 01.

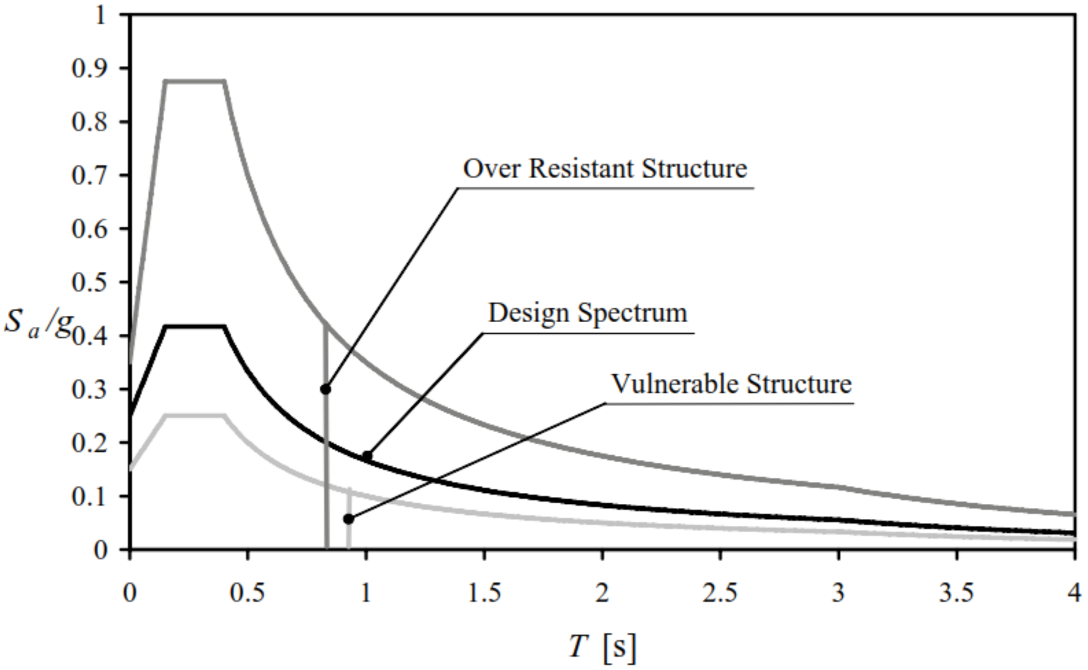


Figure-02: Comparison between Seismic Demand and Seismic Resistance

(Source: Oliveto, 2002)

If the seismic resistance of a structure as depicted in the study is greater compared to the one demanded by the design earthquake, it will be categorized as an over-resistance structure and will therefore is not vulnerable to collapse during a sudden earthquake. This case is showcased as the longer ordinate on Figure 02. A structure with the seismic resistance as illustrated by such an ordinate is well capable in withstanding an earthquake with an anchoring acceleration larger than that is associated with the design earthquake.

Instead, if the seismic resistance of the structure correlates to the shorter ordinate as shown in Figure 02, the resistance capacity is obviously less than the earthquake demand. Thus the structure is vulnerable when compared to the design earthquake. Therefore, the structure can only withstand an earthquake with an anchoring acceleration fewer than the designed one. This necessitates retrofitting of the structure to meet its inadequacy in design inequality to the extend as displayed in the study.

$$\text{Capacity} \geq \text{Demand}$$

The design inequality stated above should be satisfied in terms of stiffness, not only in resistance or strength. The stiffness capacity of the building should not be fewer compared to the stiffness the probable earthquake demands from it. If not, displacements will raise up, especially inter-story drifts, and damage could extend to non-structural components also. The stiffness control is normally carried out indirectly by checking the inter-story drifts [Oliveto and Marletta, 2005].

2.3 RISK ASSESSMENT:

Risk assessment is describing the general process and methods in order to:

- Identify the factors of hazards and risks with potentials to harm (hazard identification).
- Zero in and evaluate all risks related to those hazards (evaluation and analysis of risks).
- Determination of the apt ways for the elimination of such hazards as far as possible, if not, to control and minimize those hazards (risk containment).

The recognition of the concern for human safety and life and the importance of preserving it has ushered civil engineering professionals and experts to the development of minimum codes and standards for buildings so as to resist seismic damages. Fortunately, this is a worldwide recognition. These risks ought to outweigh all other factors, mainly economic, which tend to oversee such hazard potentials. This intense understanding needs to guide the owners, statutory agencies, institutions and government departments as well as law making bodies. Earthquake engineering based on seismic performance and risk identification needs to be thoroughly considered for retrofitting of existing buildings or designing new buildings. These will provide a framework for perceiving, determining and measuring the losses. This mechanism based on earthquake engineering being in its infancy, the designs of buildings continue to be based of the available codes and standards.

Unfortunately, in most communities, old structures which are archaic from seismic resistance perspectives pose heavy and continuous risk. The building laws, codes, specifications, rules and regulations need to recognize this risk for adequate amendments which shall ensure that the owners of such structures carry out such measures as to address and resolve shortages in seismic resistance capabilities of the structures within a reasonable time.

Seismic Evaluation and retrofitting of structures are mainly contempt to provide safety of lives, economic losses and to reduce the casualties during the occurrence of a sudden tremor. In the past decades, interest has been raised to reduce the economic losses caused by the devastating earthquake; this could either be the repairing costs or due to the loss of use of the structure. The cost of retrofit, often comes above 25% of the value of the structure, has also raised interest in researching the costs and benefits of several methods of retrofitting and their various levels of performance expected form the result. This interest in prediction of the damage levels expected in a structure before and after retrofit has resulted in the improvement of formal frameworks for performance based seismic engineering (FEMA, 1977), (Fajfar, 1997).

2.4 EARTHQUAKE DAMAGE:

Earthquake disaster, more often than not, tops the list of natural calamities unleashed on mankind resulting in serious human injury, loss of life and economic losses. The magnitude of a ground shaking at a particular location will determine the earthquake damage. The tall buildings are mainly affected due to their height, and may fall down or into each other (pounding). Earthquake occurring with liquefaction can be very dangerous. This can lead to sinking of structures into the ground. Liquefaction occurs due to the mixing of sand or soil with the ground water at the moment of occurrence of a strong or moderate earthquake. When the soil gets mixed with ground water, the ground becomes very soft and it acts like quick sand and structures above it can get sinked. Other destructive effects on the structures due to an earthquake are sliding away of foundation and their horizontal and vertical movement that may make the structure unsafe. In many such events of severe and moderate ground motions, civil engineering facilities, especially buildings, which were supposed to be designed and constructed to provide protection against natural hazards utterly failed to live up to the standards. The following are only some of such structures that caved in and were devastated during earthquakes:

2.4.1 Christ Church Earthquake, 2011

An earthquake, considered as the nation's fifth deadliest disaster, with 6.2 magnitudes on the Richter scale occurred in New Zealand's South Island on 22 February 2011. Heritage buildings suffered huge damage, including the Provincial Council Chambers, Lyttelton's Timeball Station, the Anglican Christchurch Cathedral and the Catholic Cathedral of the Blessed Sacrament. More than half of the building in the city has to be demolished since the earthquake happened. This includes the tallest building in the city, the Hotel Grand Chancellor.

The mostly affected areas were Christ Church's central city and the eastern suburbs; with serious damages to structures and infrastructure which were already devastated and weakened by the previous earthquake of 7.1 magnitude that happened in 2010. Several buildings collapsed due to the earthquake, crushing a number of transports, roads and other infrastructures and causing loss of human lives. Damages occurred mostly to old buildings, particularly with unreinforced masonry and those built before stringent earthquake codes were introduced. A six storey television building which incorporated a medical clinic, a TV station and a school collapsed in the earthquake, and fire out broke in the structure, killing 115 people (New Zealand, 2011).

It was later found that the construction of the building was faulty. The spire and part of the tower of the Christ Church cathedrals were destroyed in the earthquake, seriously damaging the structure of the remaining building. The cathedral, constructed of unreinforced masonry, had been retrofitted after the 2010 Canterbury Earthquake, but those retrofits were not sufficient to protect the building from another major earthquake. Christchurch implemented a retrofit code following the

2010 earthquake. When the earthquake occurred, unreinforced masonry buildings without retrofitting only had 1/10 of the code required design-level strength. This further stresses the importance of having adequate retrofitting to bring buildings to the design-level strength as stated in the code. The Catholic Cathedral of the Blessed Sacrament was also seriously damaged, and the towers of the structure fell down. A decision was made to remove the dome because the supporting structure was weakened. Some of the hotels were at the point of collapse, and some other buildings were dislocated by half a meter in the earthquake, and had dropped by one meter on one of the sides. Some of the structures were damaged irreparably and have the potential to bring down all the other structures if it fell; because of which two-block radius around the hotel area had to be evacuated (Bruneau, 2011).

One of the major effects on the earthquake was liquefaction which produced subsequent amount of silt. This cause subsequent loss of population in Christchurch main urban area which fall behind the Wellington equivalent to reduction of population in second to third most populous area in New Zealand.

Historic structures were badly damaged and its stone chambers had completely collapsed. A substantial number of the churches were heavily and adversely affected which were demolished days after the earthquake (Turner, 2012).

Figure-03: Pictures from Christ Church Earthquake, 2011

(Source: Berenstein .E, 2011)

2.4.2 Tōhoku Earthquake, 2011

Another glaring example of the damages caused due to the inability of the structures to challenge the earthquake was the Tohoku earthquake-2011 in Japan.

Referred to as the Great East Japan Earth Quake, it occurred in the pacific coast of Tohoku with a magnitude of 9.0 to 9.1 in the Richter scale. This is rated as the fourth most powerful earthquake in the world and the most powerful in the history of Japan. Ten thousand people were killed and thousands of people were injured by the powerful tsunami waves from the earthquake.

The damage to the infrastructure was very heavy, resulting in extensive and widespread structural damages to dams, railways, roads, buildings etc.

Figure-04: Pictures from Tōhoku Earthquake, 2011

(Source: News Tribune, 2011)

2.4.3 Haiti Earthquake, 2010

One of the most devastating tragedies in human history, a catastrophic earthquake of magnitude 7.0 hit the capital of Haiti on 12 January 2010. As per Haiti government estimation 250,000 residences and 30,000 commercial buildings had collapsed or were severely damaged which were required to be demolished.

Haiti's engineering rules were so lacking that they didn't have even building codes. As a result the buildings which were erected without any reinforcement and safeguarding features against earthquakes crumbled, killing or ensnaring the inhabitants. Many monumental buildings like cathedral, National Palace, UN Head Quarters as well as government and legislative buildings were heavily impaired. While the city's infrastructures were still reeling under the adverse consequences of the previous storms and tornadoes during August–September 2008, the earthquake came as a compounding misfortune. 15 per cent of the buildings in the town were estimated to have perished. Amongst the widespread devastation and damage throughout the city, vital infrastructure necessary to respond to the disaster were also severely damaged or destroyed. This included hospitals, transport facilities and communication systems. The earthquake destroyed the central tower of international airport, damaged the seaport and made it unusable for immediate rescue operations (DesRoches, 2011).

The earthquake also destroyed a nursing school in the capital and severely damaged the country's primary midwifery school. The Haitian art world suffered great losses; artworks were destroyed, and museums and art galleries were extensively damaged. This included among them Port-au-Prince's main art museum, Centre d'Art, College Saint Pierre and Holy Trinity Cathedral. Some buildings performed better than their neighbours because of their low mass. For example, the wood-frame building withstood the earthquake forces while the adjacent structure collapsed after being reinforced concrete structure. Similarly, the one-story church had a light-metal roof supported by masonry walls. Although it appeared to be constructed with materials of poorer quality than those used in a neighbouring concrete bearing-wall house, the masonry church structure suffered less damage (Comerio, 2011).

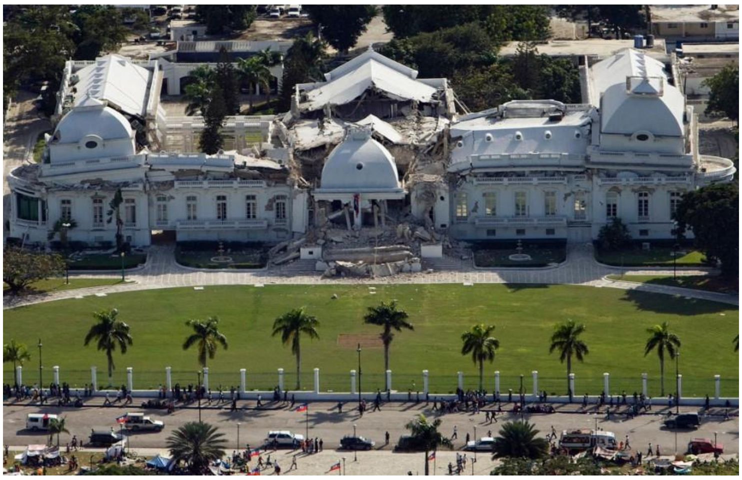


Figure-05: Pictures from Haiti Earthquake, 2010

(Source: Miyamoto International, 2010)

Figure-06: Pictures from Haiti Earthquake, 2010

(Source: Semper.D, 2010)

2.4.4 Bhuj Earthquake,2001

The 2001 Gujarat earthquake, also known Bhuj earthquake occurred on 26th January (52nd Indian Republic day). The intraplate earthquake reached 7.7 on the moment magnitude scale which is categorized as intensity "extreme" according to Mercalli Intensity scale.

The severity of the earthquake was large that the effect was felt in places and far and wide, as distant as about 2,000 kilo metres. But, it was the Kachchh region spread over forty six thousand kilo metres and comprising twenty two percent of the whole area of Gujarat that faced the brunt of the quake. Located along the western area of Indian subcontinent, this region is highly seismic sensitive. The continental southern region of peninsular India is fairly stable. On the north and east it has active plate margins. The Kutchchh region is considered as a transitionary area between these two areas. The seismic code IS: 1893–1984 categorizes India into five seismic zones, the fifth zone being considered as the most serious. However, for design purpose, the total of five zones has been reduced into 4 by combining the first and the second zones into one. The regions around Bhuj are considered as an active seismic zone featured in the most serious seismically sensitive category (Jain, 2012).

A review of the structures in the earthquake areas revealed that the majority of these structures consisted of reinforced concrete framed structure or load-bearing. The masonry units were made of random rubble stones, rough dressed stones, clay bricks, and concrete blocks either solid or hollow. Mortars of mud, lime or cement were used in assembling the units. The structure of the roof was made of Mangalore clay tiles which were laid on timber planks, supported by purlins or rafters made from wooden logs or made of reinforced concrete slab. In the building with more than one storey, the floors and roofs are more often to be non reinforced concrete slabs. The performance of the masonry structures, without any reinforcements, in the Kachchh areas performed very poorly during the earthquake. The walls were neither tied to each other nor to the floors and roofs. The random rubble constructions in mud mortar performed the worst here. Massive destruction and loss of life occured from the collapsing of such large number of buildings. Many reinforced concrete frame buildings, except in the first storey, reserved as parking locations, had infill masonry walls. No wonder, the open first storey either collapsed or suffered serious damages. The review of such failures confirmed the vulnerability of those structural details which are known as causes of distress. In the majority of buildings, including those to 10 and 12 storied, the reinforced concrete columns are supported on isolated spread footings located at some depth below the ground level to go past the fill material on the top. Generally, it is found that the quality of foundation soil is judged on the basis of visual inspection, without any geotechnical investigation having been carried out. Foundation ties are not provided. However, surprisingly, it was found in the review process of the earthquake that masonry infills, even when not tied to the surrounding frame, could save the building from collapse, if such infills are uniformly distributed throughout the height so that sudden changes in stiffness and strength did not occur. (Pathak, 2012).

Figure-07: Pictures from Bhuj Earthquake, 2001

(Source: McGee.C, 2005)

2.5 COMMON PROBLEMS ASSOCIATED WITH SEISMIC PERFORMANCE OF EXISTING RC BUILDINGS:

This section seeks to explain the common deficiencies found in reinforced concrete structures which can result in the failure and collapse of the structure during earthquakes.

2.5.1 INCOMPLETE LATERAL FORCE RESISTING SYSTEM:

One of the major causes of the collapsing of buildings during earthquake is the absence of proper lateral force resisting system in the structure. In order to avoid a collapse, every member in the structure should be connected positively to the whole structure in such a way that the inertial loads generated by element motion in any way can be transferred to the ground in a successful manner.

A complete frame includes moment frame, shear walls or braced frame. At each level of considerable mass, a horizontal diaphragm is required for interconnecting these vertical elements. Together, this assembly will provide adequate rigidity which takes care of lateral structural deformations.

Expansion joints are one of the most common features of large structures, especially in areas where temperatures vary from time to time. The structures with expansion joints designed without a complete lateral force resisting system of the structural segments on each side of the joints will result in a collapse. One other issue commonly related to the buildings with expansion joints is the pounding onto the adjacent structures. The intensity of the issue can be minimized if the diaphragm levels on each side of the expansion joints are aligned (Sinha, 2012).

2.5.2 PLAN CONFIGURATION PROBLEMS:

2.5.2.1 Re-entrant corners:

Re-entrant corners are the most common problems in building configuration (in plan) taking the shape of an H, L, T or combination of these shapes. There are two types of issues caused by re-entrant corners. The first issue is that they have a huge tendency to create variations in rigidity and as a result differential motions are caused which results in local stress concentration at the re-entrant corners at different parts of the structure. The second issue is torsion. This arises because the centre of rigidity and centre of mass cannot geometrically coincide for all possible earthquake directions.

2.5.2.2 Diaphragm Configuration:

Diaphragm is one of the most important elements in the structure which helps in transferring the laterals loads to vertical load resisting elements in the structure. It acts as a horizontal beam on the structure providing enormous stability in lateral direction. Diaphragm penetration and geometrical irregularities lead to torsion and stress concentration in structures.

2.5.3 SOFT STOREY EFFECTS:

Soft storey is one of the vertical irregularities which decrease lateral stiffness of the structure. This condition of the soft storey can develop at any floors in the structure but it becomes critical when it appears on the first storey as the force generated on the structure is concentrated most on this level of the structure. The soft storey at the first level consists of a discontinuity of strength or stiffness, which occurs at the connections in second storey. Any change of vertical and horizontal structures at the second storey also causes discontinuous load paths leading to soft storey.

2.5.4 EXCESSIVE LATERAL FLEXIBILITY:

Occasionally, collapse may occur in buildings equipped with complete lateral force resisting systems, if excessive flexibility exists in the elements of their lateral force resisting. In the case of ground shaking very large lateral displacements may happen to such buildings. Significant gravity loading in structures causes instability when subjected to major lateral deformation. Obviously, flexible structures built on deep soft soils sites may encounter substantial seismic demands.

2.5.5 SHEAR FAILURE OF JOINTS:

The moment resisting frames may experience shear failure in its joints. Transfer of flexural stress between the elements may lead to very large shear at the beam-column joints of the moment resisting frames. The lateral confinement reinforcement in the column not passing continuously through the joint zone causes failure at such joints. Comparatively slender beams as well as frames having eccentric beam-column joints are normally weaker.

2.5.6 INADEQUATE DEVELOPMENT OF REINFORCING STEEL:

Collapsing structures are more often than not seen with reinforcing steel of inadequate development. The flexural reinforcing steel will yield in frames with inadequate strength to minimum elastic at real deformation levels. The bond between the reinforcing steel and concrete is broken by cyclic loadings of repeated nature in the bars. This causes loss of flexural strength and frame instability.

2.5.7 DETERIORATED CONDITION:

Deteriorated condition due to aging of the structure contributes to seismic failure of the structures. Common problems resulting from deteriorated condition include spalling of concrete and rusting of steel on structures constructed offshore and mortar deteriorated due to drastic change in weather in concrete structures.

2.5.8 POOR QUALITY OF CONSTRUCTION:

Poor quality of construction has caused enormously to the earthquake induced failure of structures. Mostly failures occur in the masonry walls (reinforced) because grout is not placed in the reinforced cells. Low quality of mortar is the main reason for this. Welded reinforcing steel splices are often brittle and fail prematurely if proper care is not taken during the construction. Similar issues occur with welded steel connections in steel structures.

2.6 STRATEGIES FOR SEISMIC RETROFITTING:

The primary purpose of retrofitting to counter seismic effects is the enhancement of seismic behaviour structures including buildings in terms of its behaviour towards seismic factors. Different strategies need to be considered for application to achieve this. The future use of the structure needs to be the criterion for the selection of the best retrofitting method and relies on a thorough perception and understanding of the dynamic behaviour of such structures (Holmes, 2000). The maximum coordinated efforts for the optimal combination of stiffness, ultimate resistance and deformation capacity needs to be pillar of any seismic retrofitting strategy. (Holmes, 2000).

The basic purpose of seismic retrofitting is the improvement in the rectification of main weaknesses in the seismic performance of the structure intended to be retrofitted. Most importantly, the connection between the new features and the existing structures should invariably be seamless. Any effort to spread the effect of seismic impact originating at the ground level to the whole building is a crucial aspect (Holmes, 2000).

The first retrofitting strategy of 'Improving Regularity' limits the procedures towards simplicity for the improvement in distinctive features of the building viz. ultimate resistance, mass, stiffness, damping etc. A retrofitting scheme should ensure that the new structural member should fit in a way so as to create a regularity of the structural system [Thomas, 2008]. The second Strategy in retrofitting is strengthening. The proven and most widely used method of retrofitting is enhancing the existing structural systems with new elements of building or by doubling the elements of the existing building. Additionally strengthened concrete walls or steel trusses could be used for this. With this method the resistance and the toughness of the structure is enhanced while retaining the same deformation capacity. As a result of the larger extend of toughness thus achieved; the demand for deformation caused by seismic impacts is reduced to the deformation capacity which is available (Thomas, 2008).

Enhancing structural ductility is another retrofitting strategy. Achieving a plastic deformation capacity which is above the yield limit or the extent of elastic deformation capacity is known as enhancement of ductility. Usage of additional bonded strips converts brittle structural elements like masonry walls more ductile. The whole deformation capacity, elastic as well as plastic, is augmented with this strategy, with only a slight improvement in the ultimate resistance and the stiffness.

An additional strategy is soothing or softening a structure. Soothing causes a reduction in stiffness which reduces the forces by enhancing the displacement from seismic action at the same time. One of the practical applications of this strategy is the transformation of longitudinal bearing system of multi-span girder bridges from rigid to floating on a pier. Applying the softening strategy with seismic isolation by inserting a high damping and horizontally soft seismic bearings crafted from reinforced rubber layers is a wide spread application in seismic retrofitting. Another means of softening is practiced by removing the stiff struts, infills etc. to result in a better horizontal deformation of the structural system. Reduction in seismic impacts with damping process presents another possibility in retrofitting. Damping increases reduction in seismic impacts. This can be carried out by inserting multiple dampers as required. The application of supplementary dampers is discussed further in ensuing sections. Seismic isolation is implemented by inserting horizontally soft and high damping seismic bearing. This increases damping and simultaneously reduces stiffness (Thomas, 2008).

Mass reduction is an additional strategy in seismic retrofitting. The inertial forces and stresses generated by earthquakes are lesser when the mass of a building is reduced. Mass reduction can be implemented by minimizing highest storeys and roof level of the building. Practically, in most cases, this strategy is not applied because of reduction in useable space in such buildings.

The next strategy is to bring about a change in the use of the structure. Along with structural changes, operational changes can also result in the reduction of seismic effect. Declassification by downsizing the activities carried out inside a structure to some lesser intense activity in terms of impact from seismic factors could be possible. A hospital with activities requiring heavy transactions and population could be downgraded to one of less intense activities or to a hostel thereby causing reduction in Seismic action as a result of lower importance factors.

2.7 NEED FOR SEISMIC RETROFITTING:

Seismic Retrofitting is the modification of the existing structures to make them more capable to resist earthquake forces. A higher degree of damage can be expected in a structure during an earthquake if the seismic resistance of the structure is inadequate. The decision to strengthen it ahead an unexpected earthquake depends on the building's current seismic resistance status. In recent years, seismic requirements according to building standards have been getting distinctly more rigorous. Because of the widespread negligence on these regulations, the question of seismic safety is not only for older building but also for newer ones (Sinha, 2002).

Need for seismic retrofitting becomes mandatory to ensure the safety and security of a building, structure functionality, machinery and inventory. One of the main categories of the buildings prone to earthquake induced failures is that the buildings are not designed according to the standard codes or lack of timely revision of codes of practice and standards, and the members are designed only for withstanding gravity loads. Therefore, the need for retrofitting arises for these structures. There are buildings which are designed according to the old seismic codes which are no longer practically relevant. In such cases, seismic evaluation need to be carried out in accordance with updated seismic codes and design practices for the building to be protected from collapse. In the case of up-gradation of the seismic zone of a country, seismic evaluation is necessary and if the demand is greater than the capacity after calculating the C/D ratio, seismic retrofitting is required (Alam, 2015).

Seismic retrofitting is also needed when the structure gets deteriorated due to aging or modification of the existing structure like increasing the number of stories or carrying out an increase in the loading class etc.

Seismic retrofitting is essential to reduce hazard and loss from non-structural elements and for buildings that have quality or safety problems due to design flaws or deficiency in the construction quality. These problems are often met in new construction and for the existing structures (Arora, 2015). When the buildings are located close to the site of deep pit foundation of a new construction, this deep excavation may cause unequal settlement of the surrounding soil and the surrounding buildings may consequently face damages or risks. In this case, the structure must check for seismic evaluation.

In most cases retrofit analysis techniques and design procedures are similar to those applied to new designs, yet seismic evaluation and current status assessment of the existing buildings is unique in the design of retrofitting schemes.

The lateral strength and ductility are the most crucial factors that govern the seismic capacity of the structure. Before adopting the retrofitting technique, its objective should be clearly defined. The purpose of retrofitting is to upgrade the strength and ductility of the existing buildings so that it can withstand future earthquakes with non-occurrence of damage or by incurring minimum damage (Sinha, 2002).

The following concepts are recommended in seismic retrofitting:

1) Upgrade the ultimate strength of the complete structure.
2) Enhancing the ductility or deformation capacity of the structure.
3) Enhancing both of these ultimate strength and deformation capacity.

3. LITERATURE REVIEW-02

3.1 SEISMIC RETROFITTING TECHNIQUES:

Seismic Retrofitting Techniques:

The retrofitting program for a building is decided on the basis of a properly carried out seismic evaluation. Visual inspection of the drawings and the structure itself are critical for the seismic retrofitting schemes and processes. Capacity and demand of each member in the structure is identified for assessment and capacity by demand ratio is calculated in the seismic evaluation (Belali, 2015).

There are two types of retrofitting: Local Retrofitting or Global Retrofitting.

Local Retrofitting:

This is adopted when a few components (such as columns, beams, connections, shear walls, diaphragms, etc) in the existing building do not have substantial strength and stiffness. In this retrofitting procedure, basic configuration of the structure's lateral force resisting system is kept intact. Some of the local deficiencies observed in the structures are inadequate shear capacity in columns, lack of confine of column core, existence of short and stiff columns, Lack of tie reinforcement in beams etc.

Local retrofitting is carried out by enhancing the deformation capacity or ductility of the components, without completely increasing the strength like placement of jackets around the weakened concrete member (reinforced) to enhance its confinement which can improve its ability to deform without causing degradation of reinforcement splices or spalling. Local retrofitting schemes include the local strengthening of columns, beams, slabs, slab to column or beam to column joints, walls and foundations. This strengthening allows one or more vulnerable members or connections to resist the strength demand, without completely affecting the total response of the structures. Local retrofitting procedure is considered as the most economical alternative when only a few elements in the structures are deficient. Local retrofitting techniques include: jacketing of columns, jacketing of beams, jacketing of beam-column joints and strengthening individual foundations.

Global Retrofitting:

Global retrofitting techniques are required when the entire lateral load resisting systems are found deficient. Global (Structural level) Retrofit methods include conventional methods (increase seismic resistance of existing structures) or non-conventional methods (reduction of seismic demand). The common way of global retrofitting is to increase its strength and stiffness. This method concentrates on the structural level and retrofit to obtain a better overall behavior of the entire structure (Arora & Alam, 2015).

Large lateral deformations are included in the structure due to the ground shaking which imposes increased ductility demand on the various members in the structure. Addition of new shear walls or braced frames within an existing structure increases its stiffness to a great extent, while some other existing structures have inadequate strength, which causes inelastic behaviour at very low levels of the earthquake forces and results in large inelastic deformation demands throughout the structure. By strengthening the structure, the threshold of lateral forces at which the damage initiates, can be increased. Moment resisting frames, addition of new structural shear wall or the addition of new bracings can be provided to add more flexibility and strength to the structures (Sinha, 2012). The following are some of the seismic retrofitting techniques:

3.1.1 Base Isolation:

Inducting flexibility at the base of the structure horizontally, and infusing damping elements simultaneously in order for restricting the amplitude of the motion which an earthquake causes is, in principle, Seismic Isolation. The successful development of mechanical-energy dissipaters as well as elastomers possessing high damping properties boosted the concept of base isolation. The Use of Mechanical energy dissipation devices, combined with flexible base isolation devices, controls the seismic response of the structure by limiting displacements and forces. This significantly improves its seismic performance (Sinha, 2012).

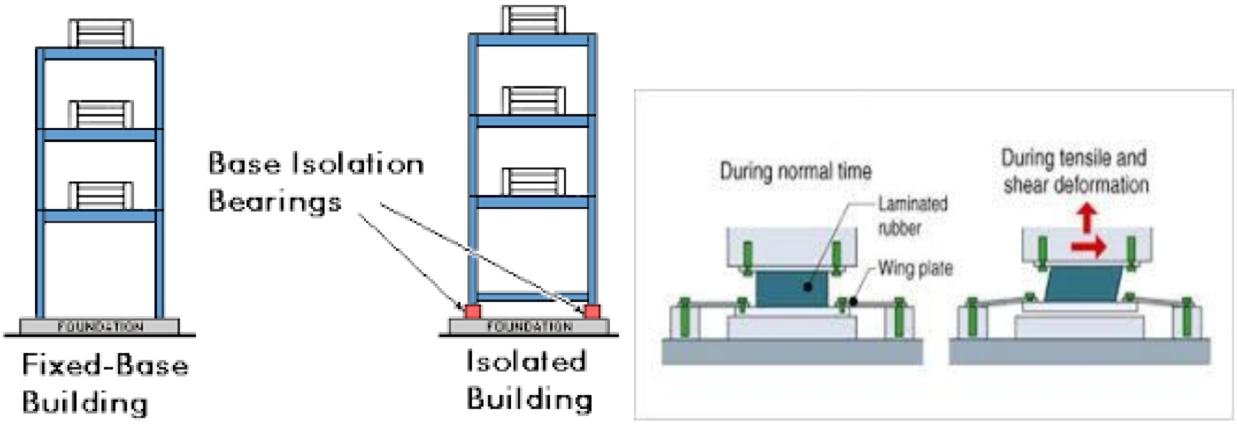


Figure-08: Base Isolation devices

(Sinha, 2012)

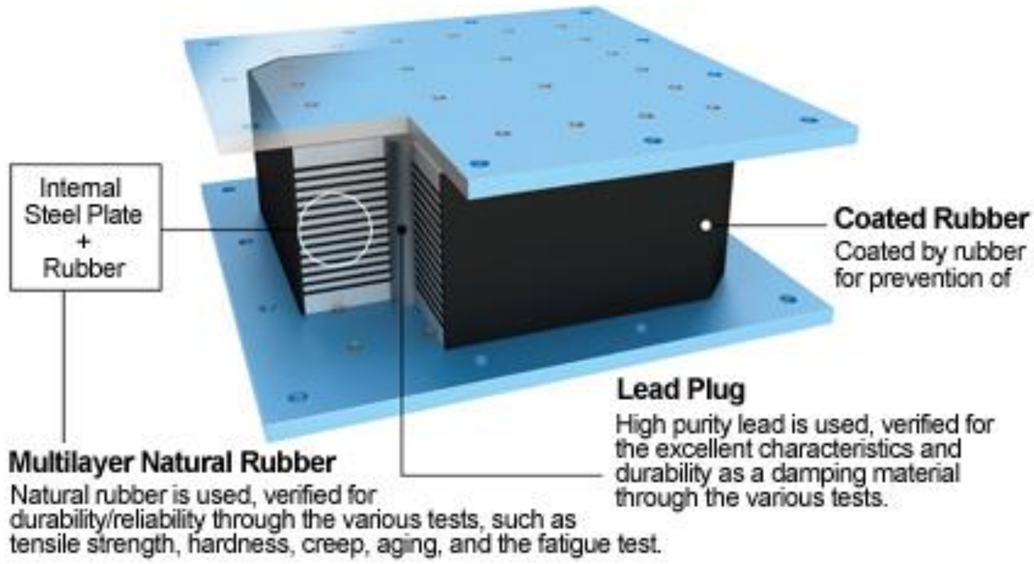


Figure-09: Base Isolation devices

(Source: Kamrava.A , 2015)

The objective of these systems is to decouple the building structure from the damaging components of the earthquake's input motion so as to prevent the superstructure of the building from absorbing the earthquake energy. The basic requirements of these systems are: damping, flexibility and resistance to vertical and other loads.

The main advantages of the base isolations are:

- Better protection against earthquake due to the decreasing of shears,
- Superstructure will not need any reinforcement
- Foundation system will not require any reinforcement to resist overturning moment, which is much smaller than those of the initial design.
- Least interruption in building activities
- Least requirement of temporary works

The commonly used base isolation systems these days are elastomeric bearing or laminated rubber bearings and sliding isolation systems. Elastomeric bearings are designed with a vertical stiffness, which are much higher than the horizontal stiffness (Clemente, 2012).

3.2.2 Addition of new Shear walls:

Adding a new shear walls is one of the most common methods to increase the lateral strength of the reinforced concrete building. It also helps in controlling drift. Therefore, it is the simplest and the best approach for improving the seismic performance which is usually used in retrofitting of non-ductile reinforced concrete frame buildings. The newly added shear walls can either be precast or can be cast-in-place elements.

Addition of shear walls are mostly preferred in the exterior of the buildings, but it may hinder the windows and balcony layouts. New shear walls are not preferred on the interiors so as to avoid interior moldings.

The addition of new shear walls to an existing structure can cause many technical problems. Some of them are: transferring of diaphragm shear into the newly built shear walls with the dowels, adding new collector and drag member to the diaphragm, thereby increasing the weight and concentration of shear by the addition walls which may affect the foundation (Agarwal, 2010).

Location of shear walls also matters a lot. It is desirable to locate the new shear wall adjacent to the beam between columns so that only minimum slab demolition is required with connections made to beams at all sides of the columns.

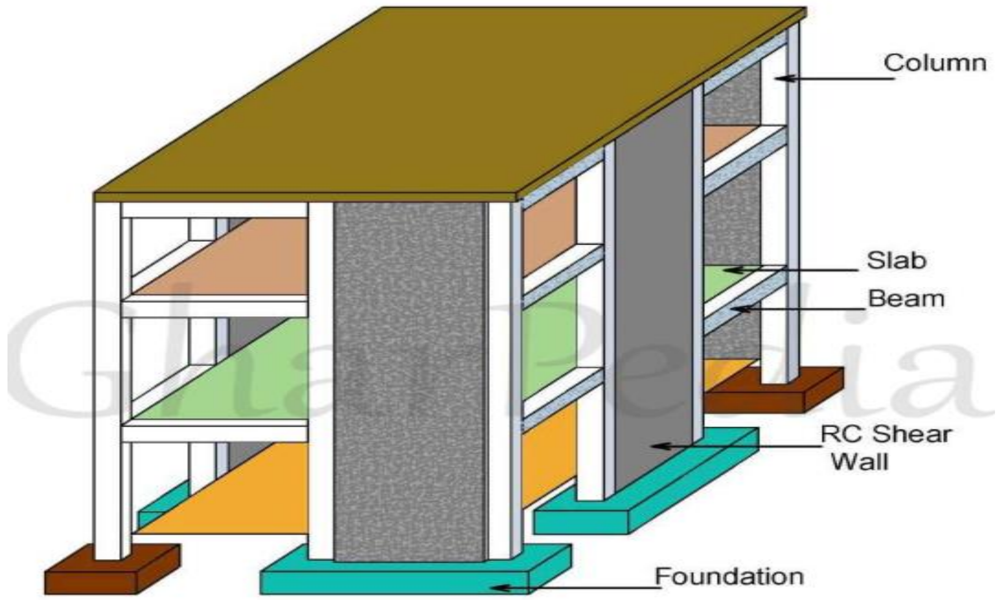
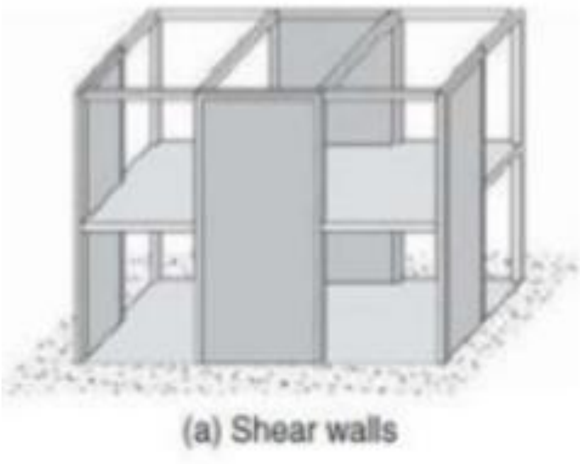


Figure-10: Seismic Retrofitting with Addition of shear walls

(Source: Hueste, M., 2011)



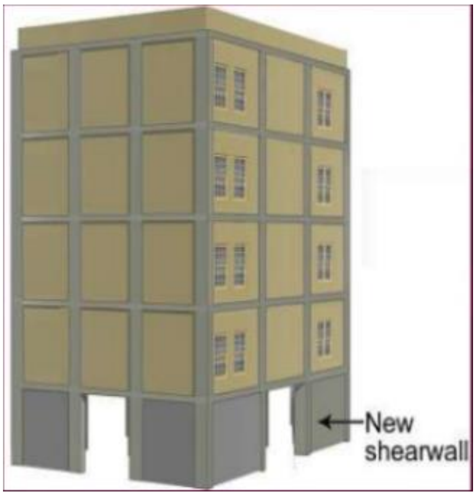


Figure-11: Seismic Retrofitting with Addition of shear walls

(Source: Samor, R., 2013)

3.2.3 Addition of New Steel Bracings:

Another most common method of strengthening a seismically damaged structure is by the addition of steel bracing (cross bracings) on the exterior of the structure. One of the main benefits of steel bracing is that the windows, balconies etc. will not be hindered. Some of the other advantages of using steel bracings compared to other retrofitting schemes are: it provides higher strength and stiffness; bracing will not increase the weight of the structure to a great extent, and the foundation cost can also be minimized. Since some of the retrofitting works can be pre-fabricated, much less disturbance is caused to the occupant. Steel bracing retrofitting techniques can be used with both concrete and steel structures. The installation of steel bracing members can be an effective solution when large openings are required (Agarwal, 2010).

Newly added bracings always require vertical columns at both ends to resist overturning forces to work vertically, as chords of a cantilever truss are arranged horizontally at each floor level.

Several researchers have found that addition of steel bracings has performed well-exhibited linear behaviour even up to twice the design code force.

Bracing should have low slenderness ratio so as to function effectively during compression. Skilled labour is crucial for steel bracing construction. Careful considerations of connections of strengthening elements to the existing structures and to the foundations have to be consciously designed to ensure proper shear transfer. Local reinforcements to the columns may be needed to bear the increased loads generated on them.

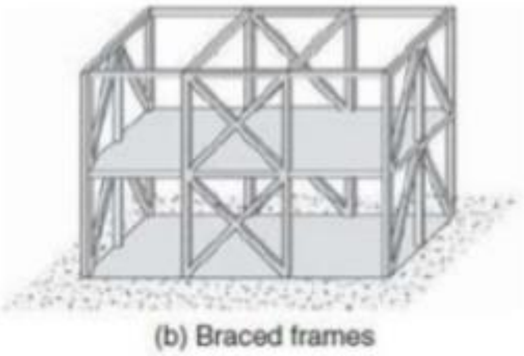


Figure-12: Seismic Retrofitting with Addition of Steel bracings

(Source: Samor, R., 2013)

3.2.4 Jacketing Techniques (Member retrofitting techniques):

Jacketing techniques which are cost effective in comparison to the global (structural level) retrofitting are utilized in upgrading the strength of the seismically deficient members. Addition of concrete, steel, fibre reinforced polymer (FRP) jackets as well as jackets with high tension materials like carbon fibre, glass fibre etc. used in confining reinforced concrete beams, columns, joints and foundation are a few of the Jacketing techniques. Being the most efficacious, the strengthening of columns is the most popular means of jacketing (Sarker, 2010).

Increasing concrete confinement by transverse fibre /reinforcement especially for circular cross sectional columns, Enhancing shear strength by transverse fibre /reinforcement, Escalating flexural strength by longitudinal fibre/reinforcement, all well anchored at critical sections are the paramount purposes of jacketing.

Jacketing is carried out by wrapping the entire circumference of the member with transverse fibre which are either overlapped or welded for enhancing the concrete confinement and shear strength. Members with circular cross-section are jacketed for better confinement like this. Circular, oval or elliptical jackets with spaces between them adequately filled with concrete, are used for jacketing square/ rectangular members.

Even though the flexural capacity of the building frames is not increased to any great extent, the longitudinal fibres which are similar to longitudinal reinforcement could be efficient option in enhancing the flexural strength of the member. This is because the critical moments are concentrated at the ends of beam-column where it is extremely difficult for most of the longitudinal fibres to pierce in through to get anchorage.

Enhancing the seismic capacity of the moment resisting framed structures is predominant purpose of jacketing. As the slab causes hindrance in the jacket, the jacketing of reinforced concrete beams with slab is difficult. The design of the jacket should invariably include probable distribution of loads thorough out the structure. A change in the dynamic properties of the structure might in turn lead to changes in lateral forces induced by an earthquake (Agarwal, 2010).

3.2.4.1 Jacketing of Columns:

Jacketing the damaged column is carried out by placing reinforcements of traverse as well as longitudinal nature around the already existing columns and then filling the spaces with concrete. The axial and shear strength of the columns is enhanced by this. Jacketing of columns does not increase its ductility. A major advantage of jacketing of columns are that they enhances the lateral load capacity of the structure in a reasonably uniform and distributed way and thus avoiding concentration of stiffness as in the case of shear walls.

The jacketing method will not change the original geometry of the building. The foundation loads will not be changed a lot. The jacketing of columns can be done in two ways:

- Reinforced Concrete Jacketing
- Steel Jacketing

Jacketing increases the shear capacity of the columns in order to accomplish strong column-weak beam design, and to improve the column’s flexural strength by the longitudinal steel of the jackets.

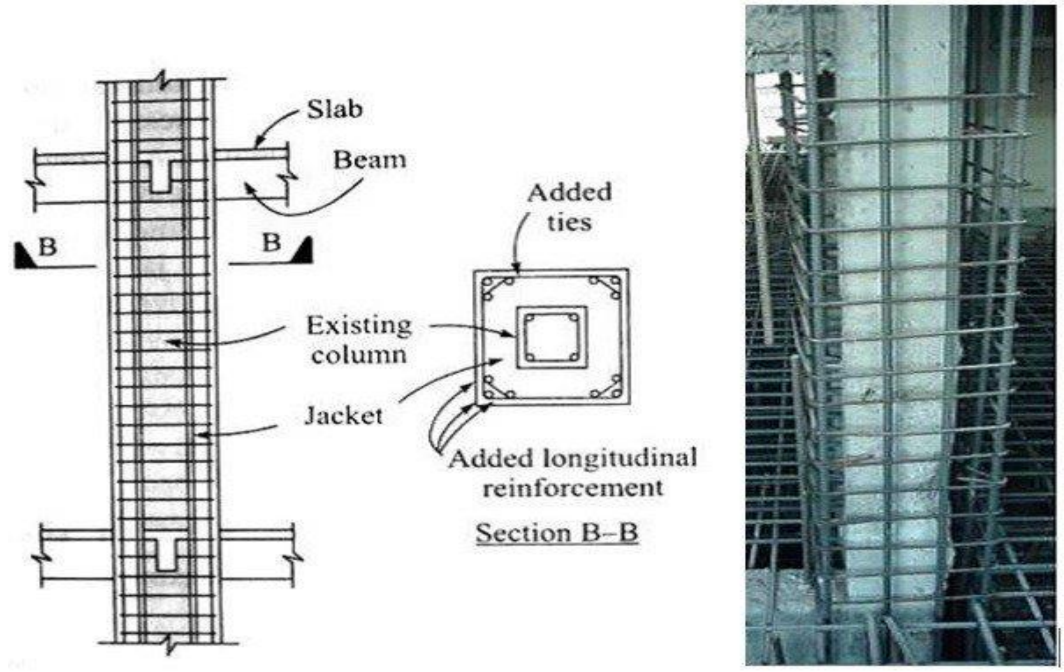


Figure-13: Seismic Retrofitting with Jacketing of Columns

(Source: Nasreen, S., 2016)

3.2.4.2 FRP Jacketing:

During a moderate or strong earthquake, it is very likely that the structure can undergo inelastic deformation and to avoid these structures from collapsing; it depends on the structure’s ductility and energy absorption capacity.

The application of composite materials has been developed in the strengthening and retrofitting of seismically damaged RC structures through recent years, so that many of the concrete structures would be strengthened by these materials.

One of these materials are FRP wrapping (fibre reinforced polymer) of reinforced concrete columns. Jacketing with FRP increases ductility and compressive strength of RC columns. FRP retrofitting enhances the compressive control region and it has no effect on tension control region.

The seismic retrofitting of chimneys, bridge piers and columns use light, high strength and high durability continuous fibre reinforcement such as aramid, glass, carbon etc. FRP wrapping has been developed to increase the flexural capacity of the column, the technique known as Near-Surface Mounted (NSM) FRP rods are proposed. Embedment of the rods is achieved by grooving the surface of the member, to be strengthened along the desired direction.

The groove is filled halfway with epoxy paste. The FRP rod is placed in the groove and lightly pressed, so as to force the paste to flow around the bar and fill completely between the bar and the sides of the groove.

The groove is then filled with more paste and the surface is levelled. The presence of jacket contributes to the stability of the rods and controls epoxy paste cracking. FRP column jacketing systems reduce the maintenance needs of the columns and improve their durability (Agarwal, 2010).

As mentioned above, two mainly used FRP are Glass fibre reinforced polymer (GFRP) and carbon fibre reinforced polymer (CFRP). It was proved in several studies that added confinement with CFRP at critical location can enhance its ductility, strength and energy dissipation capacity.

The principle advantages of FRP jacketing techniques are:

i. Carbon fibre is flexible and can be made to contact the surface tightly for a high degree of confinement.
ii. Confinement is very strong and effective because carbon fibre is highly strong and of high modulus of elasticity
iii. The carbon fibre has light weight and rusting does not take place.

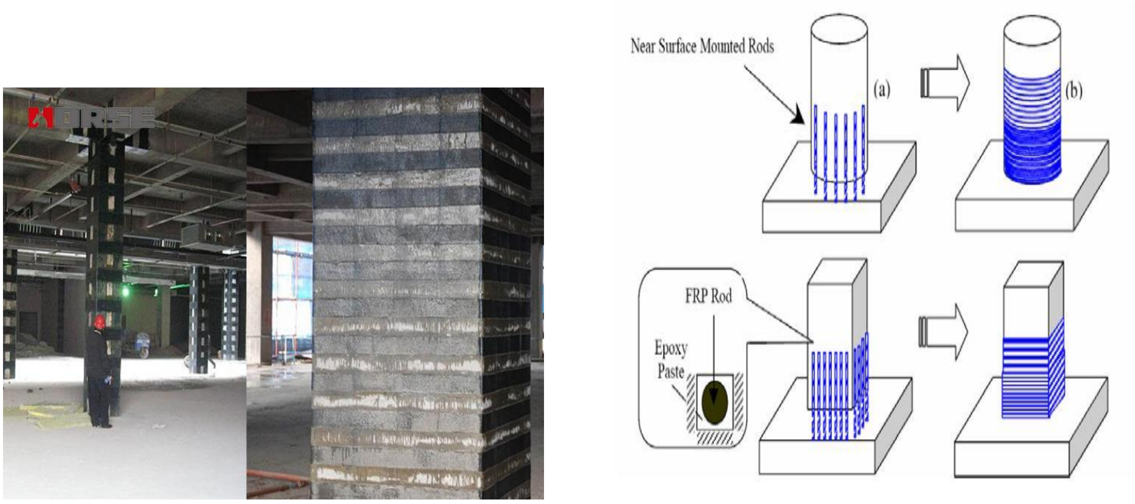


Figure-14: Seismic Retrofitting with Jacketing of Columns using FRP wrapping

(Source: Sarafraz, M., 2008)

3.2.4.3 Jacketing of beams:

Jacketing of beam is endorsed and recommended extensively since it provides continuity to the member and help in increasing the stiffness and strength of the structure. While jacketing a beam, care should be taken while calculating flexural resistance, to avoid the creation of strong beam-weak column system.

The jacketing of beam includes placing an additional layer of concrete around the existing beam, together with the addition of longitudinal bars and stirrups, to increase the shear and flexural capacities of the beam. The positive flexural capacity of the beam at the face of the joint can be increased, based on the generated confinement of concrete and the pattern and strength of the wraps (Sengupta, 2012).

For effective beam jackets, the use of closed stirrups as dowels involves closely spaced drilling of the existing beam. The advantages of the concrete beam jacketing are:

- Jacketing by concrete can increase both flexural and shear capacities of a beam.
- The compatibility of deformation between the existing and new concrete, resistance against delamination, and durability are better as compared to a new material on a different substrate.
- The analysis of retrofitted sections follows the principles of analysis of RC sections.

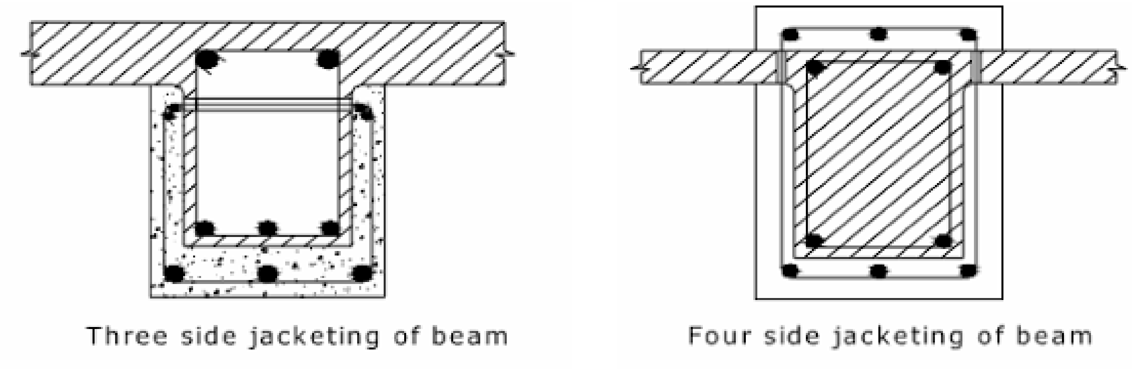


Figure-15: Seismic Retrofitting with Jacketing of Beams

Figure-16: Seismic Retrofitting with Jacketing of Beams

(Source: Khalaf, Q., 2015)

3.2.5 Supplementary Damping Devices:

Mechanical devices capable of being incorporated in the frame structure and disperse energy at discrete locations throughout the structure are known as Supplemental damping systems. Such devices add damping and strengthen structural systems, without causing any changes to the existing components. In retrofitting lightly reinforced frames and other structures these were found extremely useful. Supplemental stiffening and strength to structures that lack such properties, more often than not, without any alternation in the existing components are also supplied by damping devices.

In structures with insufficient stiffening and strength, Damping devices supply supplemental stiffening and strength, usually without alteration in the existing components. Supplementary dampers are provided by incorporating at strategic locations in the building, simple and inexpensive frictional damping devices. Such supplementary damping devices are effective in resisting seismic forces. The incorporation of these devices does not necessitate major disturbances to the structures making it capable of continuous use during retrofitting procedures. However, a thorough understanding of the contribution of dampers towards the enhancement of structures capacities and reduction in the seismic demand is critical to the design process.

To implement this, braces containing yielding metal elements, fluid - viscous or metal or solid visco-elastic or frictional dampers and hysteric dampers can be incorporated into the lightly reinforced frames, similarly with solid metal braces in the conventional retrofitting strategies. Also, visco-elastic or fluid filled walls as infills for frames gained momentum are used in rehabilitation projects. These method are flexible enough to provide greater degrees of either damping or stiffness, or even both. These techniques can also be instrumental in providing better control in interactions with existing components and in reducing the seismic demands without requiring any modification needs in the existing structural components.

For the rehabilitation of lightly reinforced concrete structures, modern and evolving techniques of providing external coating were considered recently. Cementitious coatings, or most modern epoxy layer reinforced with either glass, carbon or steel fibres were introduced and utilized in the restoration of lightly reinforced concrete piers or walls, for the purpose (Reinhorn, 1995).

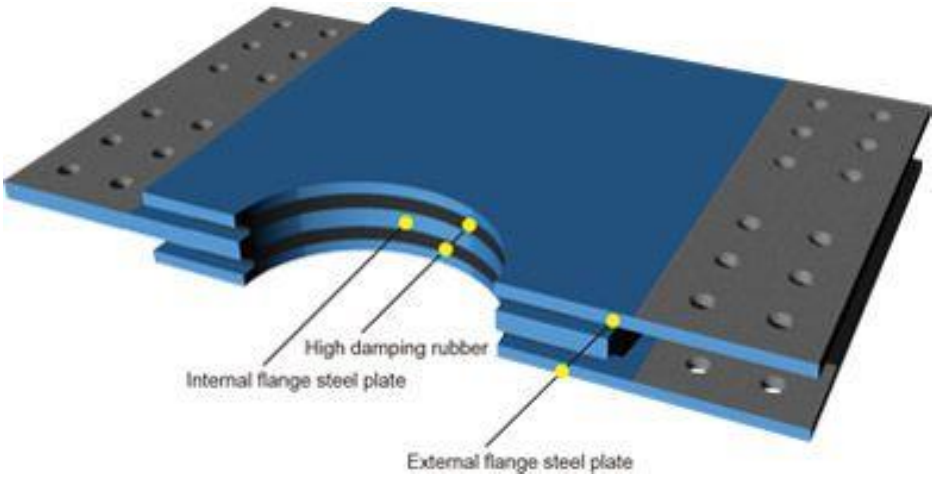


Figure-17: Supplementary Damping Devices

(Source: Kamvara, A., 2015)

Designs using inertial forces lower than expected in an elastic response at the time of earthquake, based on the principle that inelastic action will supply the structure with significant energy dissipation potential so as to survive serious adverse events. Modern seismic design practices invariably utilizes such design possibilities. The inelastic action is thought to be developing in specially detailed critical regions, most commonly, close to connections, possessing ductile properties and capabilities to dissipate energy with minimum deterioration [Kobe, 1995].

A substantially larger increase in damping may lead to a sizeable reduction in accelerations, provided, the structure is retrofitted to respond only elastically.

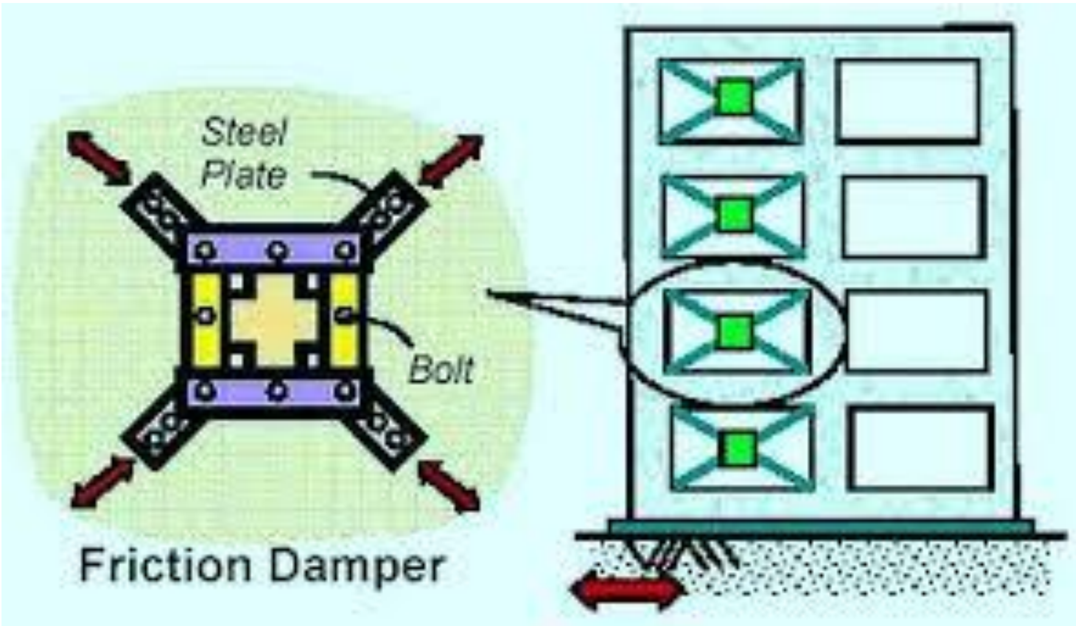


Figure-18: Supplementary Damping Devices

(Source: Kamvara, A., 2015)

3.2 Main Issues in Seismic Retrofitting:

The following are the main challenges encountered in seismic retrofitting of reinforced concrete structures.

3.2.1 Seismic Evaluation

The seismic evaluation has always challenged all stakeholders of buildings including the owners, engineers and architects. The focus of seismic evaluation is on identifying the weak links in the structure having potentials to precipitate a failure in the structure or component. The crux of this process is the juxtaposition of the demands paused by a probable earthquake on one side and the earthquake resisting capabilities of the building to challenge such demands on the other side. This obviously necessitates thorough knowledge of the structures, its components, material strength etc.

Seismic evaluation includes:

- Visual inspection and collection of general information
- Experimental procedures for modified material strength and safety factors.
- Performance objective and modified seismic action.

Seismic evaluation was difficult in olden days but this changed after the development of FEMA 310. Moreover, ATC-14 offered the first technique for adjusting the evaluation for the lack of proper detailing by using three-level acceptance criteria.

Finally, to evaluate the strength of existing or damaged building for which suitable mathematical models are required to be made. A detailed seismic analysis (linear or non-linear) using response spectrum analysis, equivalent lateral force method or pushover analysis need to be carried out with computer software. The deficiencies in the members (beams and columns) are found out by calculating the capacity by demand ratios and if the ratios are less than one, the member needs to be retrofitted [Duggal, 2010]

3.2.2 Design of Retrofitting Scheme:

The retrofitting techniques adopted for each building is unique. This can be found out only after proper seismic evaluation and visual inspection. The main objective should be to retrofit the existing structure but not making any major change to the original geometry and the retrofitting scheme should be functionally and sometimes-if possible, to the maximum extend, aesthetically compatible to the existing building. The retrofitting scheme should be aimed to upgrade both ultimate strength and ductility. One of the main aims in retrofitting is to create a continuous load path to avoid the structural damage due to the discontinuous load path.

4. RESEARCH METHODOLOGY:

4.1 General

The case study is carried out on two already constructed structures located in two different seismic zones. The structure-01 is located in Mexico which is a high seismic area. The building is a 12 storey hotel building constructed in 1927. The lateral load resisting system for the structure was non-ductile reinforced concrete frames. The foundation system was mat foundation on concrete friction piles.

The structure-02 is located in New Delhi, India which falls under zone IV of the seismic map under IS1893:2002. The building is a reinforced concrete framed structure having 14 storeys. The lateral load resisting frame for the structure is ordinary moment reinforced concrete frames. The foundation was considered fixed above the base of the raft foundation.

In this chapter, description of the building features together with its details and the process of seismic evaluation are discussed. Physical verification of the building to verify any variations in the designed and as built could not be carried out. Localized damages to the structure and the general deterioration of the building over a period of time also could not be ascertained for these to be considered in the evaluation procedure.

4.2 General description of the buildings:

The structure-01 is a 12 storey hotel building of reinforced concrete. The building consists of cast-in-place reinforced concrete columns and beams. The typical plan of the existing building is shown in figure-18. The building consists of five frames in X-direction and two frames in the Y direction and 37 metres in height above the ground level. The supports of all frames are considered fixed with respect to translation and rotation.

The floor system for the structure-01 is cast-in-place concrete joint beam construction with 2.5 inch concrete slab. The sizes of all beams are 400mm X400mm and those of all columns are 600mm X 600mm. The structure has no plan irregularity like re-entrant corners, diaphragm discontinuity, out of plane offsets and torsional irregularity.

The structure-02 is 14 storey (G+13) hospital building. The building consists of cast-in-place reinforced concrete columns and beams. The typical plan of the existing building is shown in figure-21. The building consists of five reinforced concrete framed blocks (that is B_1, B_2, B_3, B_4 and central block).The frames of blocks B_2 are considered in the study. The frames have spandrel walls (300mm thick and 1100mm high) in between columns. Concrete ordinary moment resisting frames are expected to resist the lateral forces including the earthquake forces in the existing building. In X-direction frames consists of 4 bays at 6.5 metres each, in Z-direction 3 bays at 6 metres each and having 14 storeys with floor height of 3.1 metres. The supports of all frames are considered fixed with respect to translation and rotation.

The floors of the buildings have 200mm thick, monolithically cast reinforced concrete slabs, with beams and columns. The sizes of beams of the frames are

500mm X 500mm and those of all columns are 700mm X 700mm. The structure is considered to have no plan irregularity like re-entrant corners, diaphragm discontinuity, out of plane offsets and torsional irregularity.

4.3 Seismic Evaluation:

The seismic evaluations of both retrofitted structures were carried out on two schemes. The structure-01 was initially retrofitted with Steel cross bracings (Scheme-01) and in the present study; it was retrofitted with reinforced concrete ductile shear walls (scheme-02) while the structure-02 was initially retrofitted using ductile reinforced concrete shear walls (scheme-03) and in the present study, the same structure was retrofitted using steel cross bracing (scheme-04). The details of reinforcements in members are shown in figure-40 and figure-43 as per available drawings. The process of seismic evaluation is carried out in three stages as explained below.

4.3.1 Estimation of the capacity of the member:

The capacity of the beams and columns are estimated under both schemes for structure-01 and structure-02. For determining the capacity of the existing columns and beams, Microsoft excel sheets are used to calculate the moment of resistance and shear capacity of beams, and the axial load capacity and moment capacity of the columns. The sheets are developed according to the limit state method. The excel sheets for estimating the capacity of the member is attached on Appendix-B. Shear force, bending moment and axial load capacity of the members are calculated on the basis of available reinforcement details in figure-40 and figure-43. In all the beams the bending moment capacities are computed at the start of the section; end of the section and on the middle of the span. Shear capacity is calculated only for the beginning and end of the section.

4.3.2 Estimation of Demand of the member:

The seismic demands of beam and columns are computed from the dynamic analysis of structure-01 and structure-02 under two schemes each using SAP-2000 software. Response spectrum method is adopted using UBC-97 for rock site with 5% damping. CQC combination was used to get peak response for dynamic analysis. Demand of beams and columns are thereby estimated from their capacity by demand ratios.

4.3.3 Identification of deficient members:

After calculating the capacity (C) as explained from section 4.3.1 and the demand (D) from section 4.3.2, the capacity by demand ratios are calculated. If the C/D ratio calculated is less than one, this means member is deficient and retrofitting is a requisition.

4.4 Method of Dynamic Analysis:

For the dynamic analysis of structure-01 and structure-02 under both schemes, response spectrum analysis is used. Dynamic analysis can be carried out using two methods: time-history analysis and response spectrum analysis.

Time history analysis is an important technique for structural seismic analysis especially when the evaluated structural response is non-linear. It determines the response of the structure to a known ground motion at pre-determined time steps. While on the other hand, the response spectrum analysis determines the responses for a few modes of vibration and then combining total response by suitable combination rule.

In the present study, modal combination rules such as square root of sum of squares (SRSS) or complete quadratic combination (CQC) can be used to get peak responses. SRSS gives better results for structures having well-spaced frequencies while CQC provides better results for systems with closely spaced frequencies.

The response spectrum method is a dynamic analysis, because it uses the dynamic characteristics such as natural frequencies, natural modes and modal damping ratios of the structure and dynamic characteristics of ground motion through its response spectrum.

4.5 Load Combinations:

The load combinations considered for the detailed dynamic analysis using SAP-2000 software are listed below:

- 1.4DL
- 1.2DL+1.6LL+0.5Lr
- 1.2DL+1.0LL+0.5Lr
- 1.2DL+1.6Lr+L
- 1.2DL+L+E-X
- 1.2DL+L+E-Y
- 0.9DL+E-X
- 0.9DL+E-Y

Where;

DL = Dead load on the structure

LL = Live Load

Lr = Live loads on the roof

E-X = earthquake loads on X-direction

E-Y = earthquake loads on Y-direction

5. CASE STUDY:

Retrofitting of an Existing Reinforced Concrete Structure:

5.1 General:

This study involves retrofitting of two already retrofitted structures with two different schemes and comparing their results. The first structure (Structure-01) is situated in Mexico and the second structure (Structure-02) is situated in Delhi, India. Structure-01 is a 13-storey multi-storey reinforced concrete structure constructed before 1979 which faced the earthquake in 1979; the structure went through severe spalling and seriously major and minor cracks. On the basis of seismic evaluation carried out, it is found that large numbers of members in the structure are deficient to resist design basis earthquake forces. Thus the need of seismic retrofitting of this structure arises while the structure-02 is a 14-storey hospital building constructed in Delhi. On the basis of seismic evaluation carried out, it is found that the lateral force resisting frames are found very weak to resist the design earthquake force. This leads to the need of seismic retrofitting on this structure.

5.2 Selection of the Retrofitting Scheme:

The seismic retrofitting can be done in various ways e.g. by strengthening the individual members or by adding lateral force resisting systems like shear walls, infill walls, and steel bracing or providing supplementary damping devices/ base isolation at strategic location in the building for the energy dissipation.

The structure-01 was retrofitted and strengthened after the 1979 earthquake using steel bracing in one direction on two frames and new infill reinforced walls in the other direction. The floor plans of the structure-01 are shown in figure-18 while the elevation views of frames 2, 3 and 4 are shown in figure-32 and the frame 1 and 5 in figure-33 in Appendix A.

In the study of structure-01, all efforts are made to retrofit the same building by addition of ductile reinforced concrete shear walls at strategic location and the results compared under various factors. Seismic Evaluation was done on the structure to calculate the capacity by demand ratio of each member and conclude if they are safe to resist the loads or not. Three types of beam reinforcement arrangement are considered and are shown on figure-40.

Scheme-01: Structure initially retrofitted using Steel Bracing

Scheme-02: Structure retrofitted using ductile reinforced shear walls.

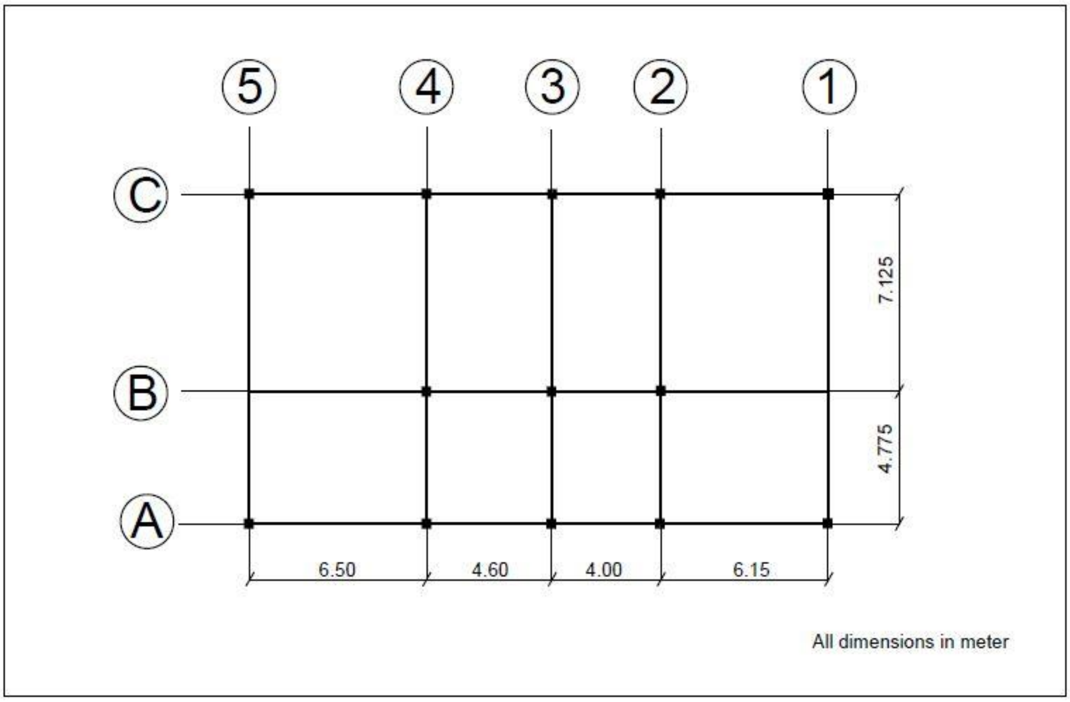


Figure-19: Typical floor plan with column layout

Scheme-01: Structure retrofitted using Steel Bracing.

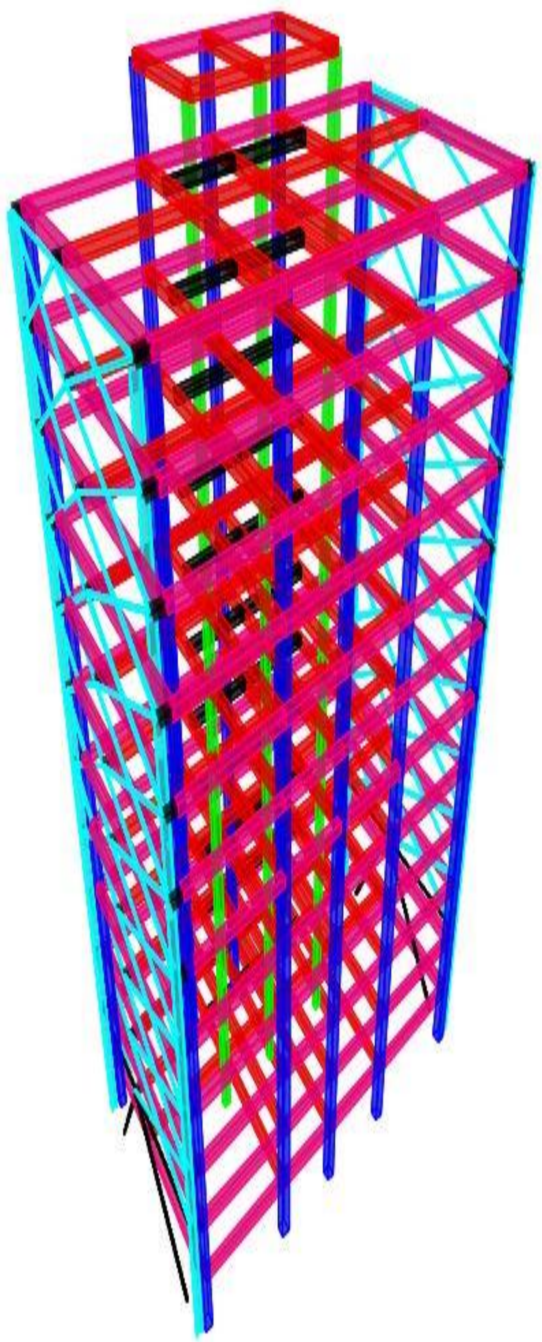


Figure-20: 3D model from the software

Scheme-02: Retrofitted Structure Using Ductile Reinforced Concrete Shear Walls.

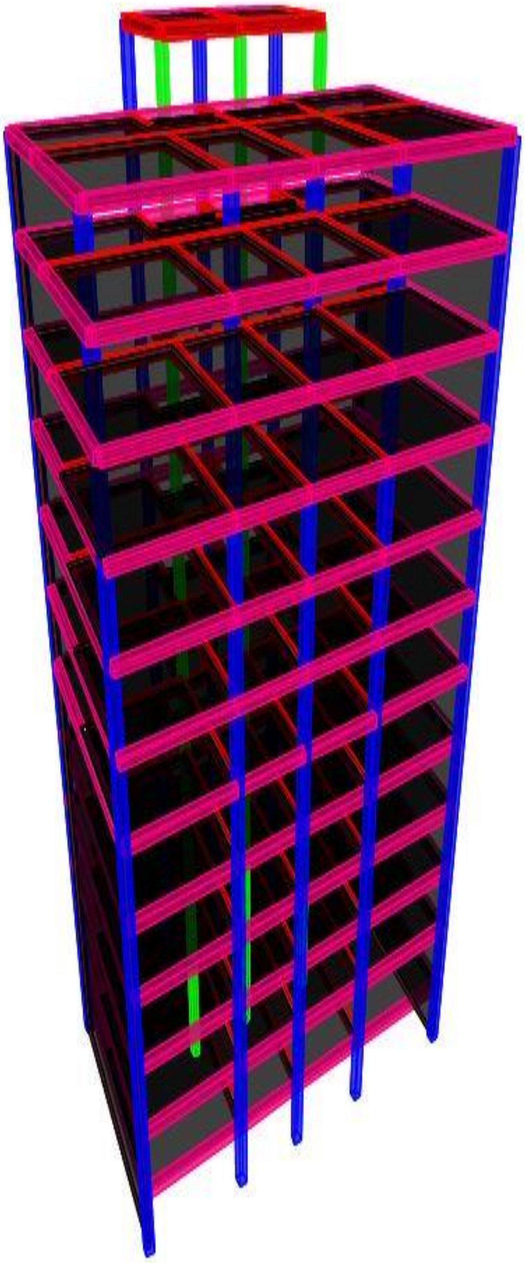


Figure-21: 3D model from the software

5.3 Selection of the Retrofitting Scheme for Structure-02:

In the study of structure-02, almost the same procedures were adopted. The structure-02 was initially retrofitted and strengthened by the addition of ductile reinforced concrete shear walls on the exterior frames along grids 1, 4 between A-B and D-E and along grids A and E between 1-2 and 3-4. The floor plan of the structure-02 is shown on figure-21.

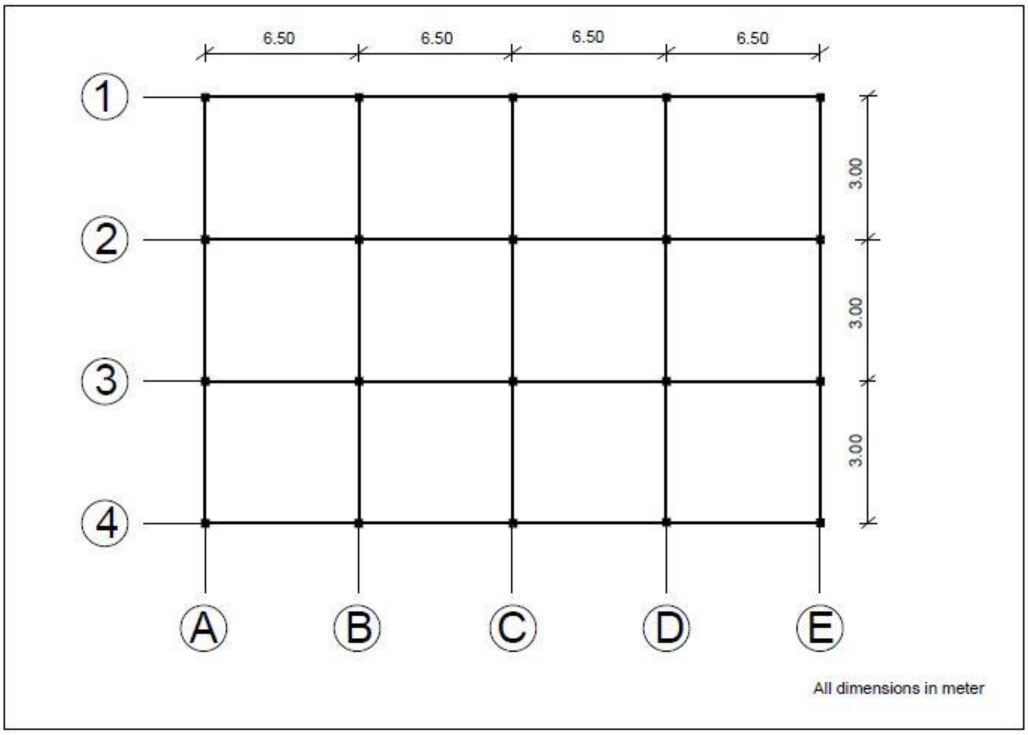


Figure-22: Typical floor plan with column layout

Scheme-03: Retrofitted Structure using ductile reinforced shear walls.

Scheme-04: Structure initially retrofitted using Steel Bracing

Scheme-03: Retrofitted Structure using Ductile Reinforced Shear Walls.

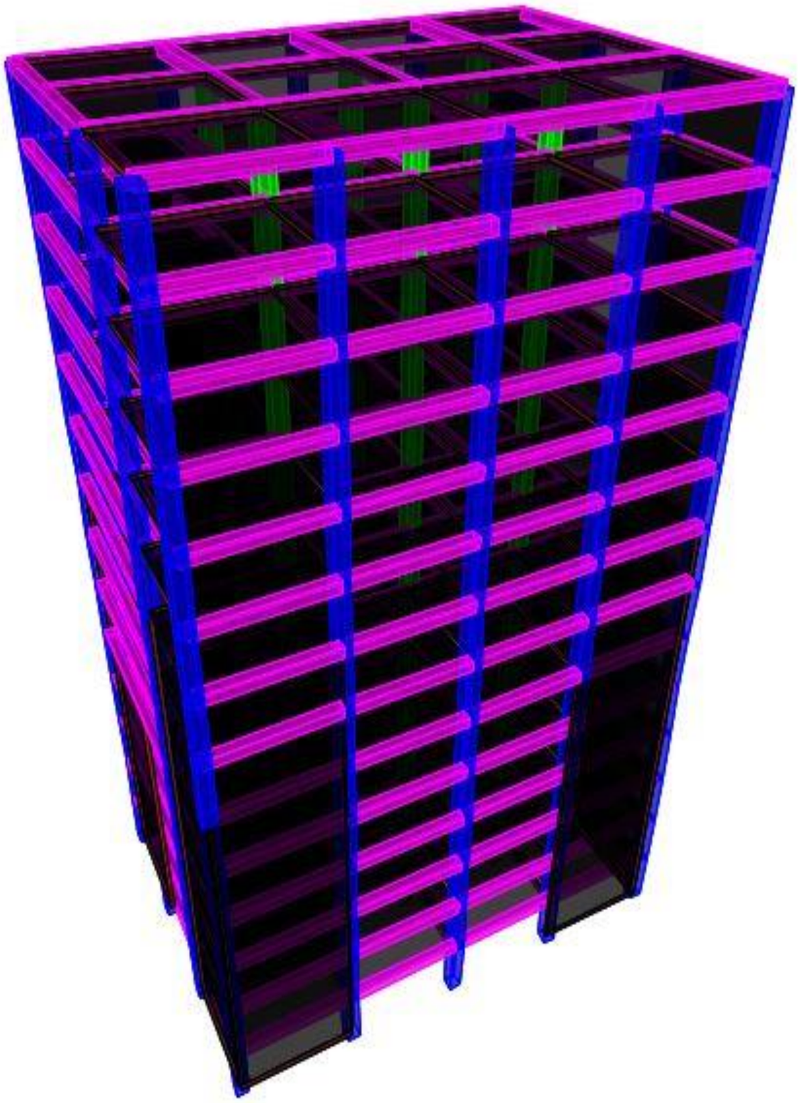


Figure-23: 3D model from the software

In the present study, the efforts are made to retrofit the same building by the addition of steel bracing in one direction on two frames on grids A and E and new infill reinforced walls on the grids 1 and 4.

Scheme-04: Retrofitted Structure using Steel Bracing.

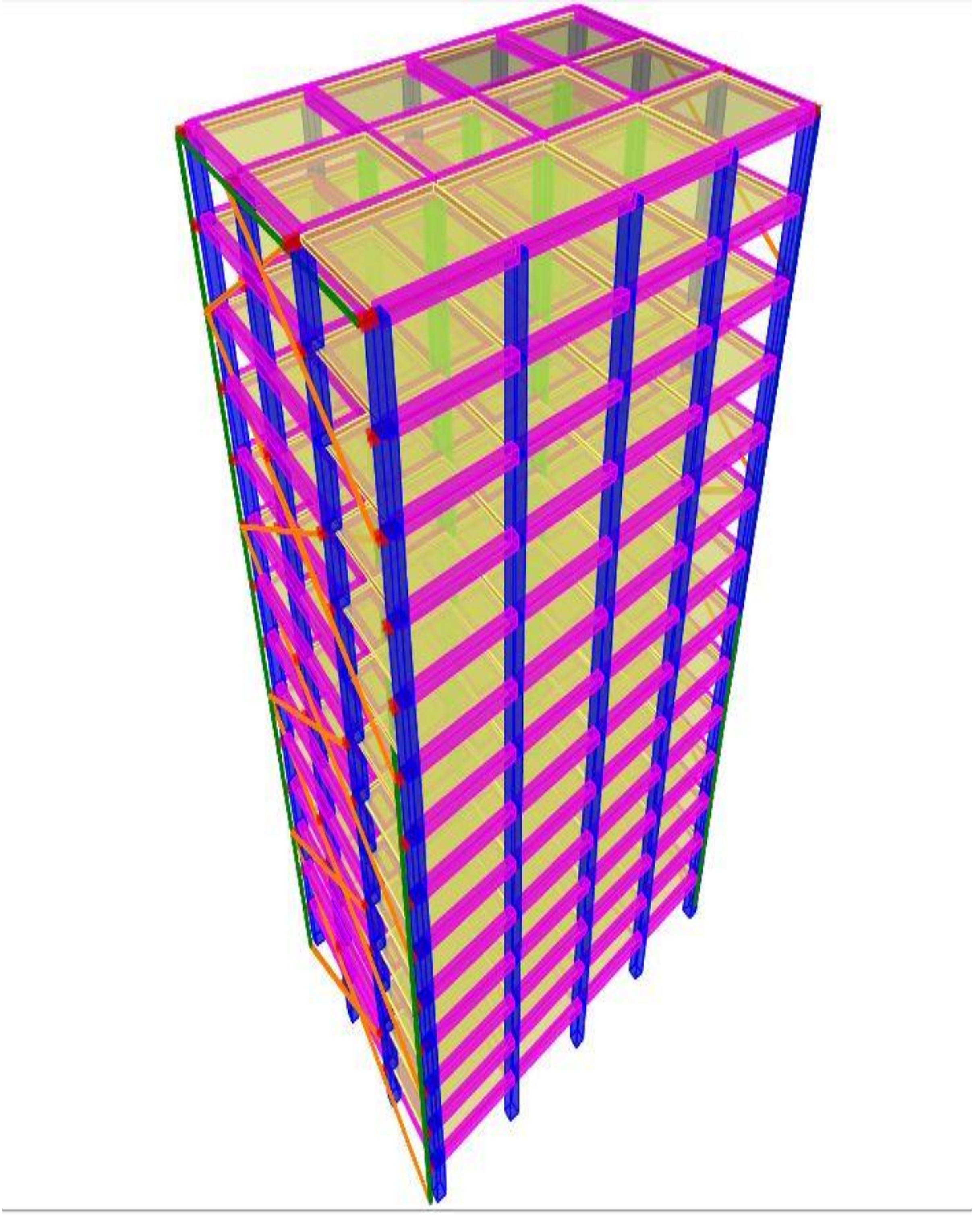


Figure-24: 3D model from the software

Seismic Evaluation was done on both the structures (scheme-03 and scheme-04) to calculate the capacity by demand ratio of each member and concludes if they are safe to resist the loads or not. The results of the above are shown on table-13-18 and table 25-38 in appendix A. Seismic evaluation was carried out considering three types of beam reinforcement arrangement shown in figure-43. After retrofitting both the structures with their respective retrofitting schemes, the results under various factors were compared between their retrofitted results and the new retrofitted results.

5.4 Assumptions in Retrofitting:

The following assumptions are made while retrofitting:

- The foundation of the building is assumed to be safe to withstand the increased load due to the addition of steel bracing (Scheme-01 and Scheme-04) or addition of shear walls (Scheme-02 and Schmeme-03).
- Neglecting the effect of stiffness of the infill, bare frame is analysed.
- Infill walls are considered not to carry loads

5.5 Methods of analysis used for the Retrofitting of the structure:

The Seismic evaluation was initially carried out on the structure-01 before the retrofitting procedure and was found that most of the members have their capacity by demand ratios (C/D ratios) less than 1 which makes them unsafe.

The structure-01 was severely damaged during the 1979 earthquake and was initially retrofitted using steel cross bracings and it performed well during the 1985 earthquake even though this structure was situated near the major earthquake affected areas.

This study involves the retrofitting of the same building with reinforced ductile shear walls and comparing the results of structure under both schemes with regard to their time periods, modal participating mass ratios, base shears, bending moments and shear forces for beams, axial forces and moment of columns and story drifts.

Seismic evaluation was also done on structure-02 and the results shows that most of the structure has their capacity by demand ratios to be less than 1. Initially this structure was retrofitted using ductile reinforced concrete shear walls and they are performing well with this scheme. The present study considers the steel bracing as the retrofitting scheme for the structure and comparing the results to understand which scheme is better in resisting earthquakes.

For the dynamic analysis of structure-01 and structure-02, response spectrum analysis is used which considers the dynamic characteristics such as natural frequencies, natural mode and damping ratio of the structure.

There are no vertical irregularities in both structures like stiffness irregularity (soft storey effects), mass irregularity, vertical geometric irregularity, unsymmetrical bracings and in plane discontinuity and there are no horizontal irregularities like torsion irregularity, re-entrant corners, diaphragm discontinuity, out of plane offsets irregularity and non-parallel system irregularity.

The present study considered Response spectrum analysis for the dynamic analysis of the structures to obtain accurate results.

After the retrofitting procedures for all the schemes, capacity demand ratios of the members are again calculated to verify the safety of the members. The results of the capacity by demand ratios are tabulated in table-5-15. The results of time periods, Modal participating mass ratios, Base shears, bending moments and shear forces for beams (at beginning, midspan and end of the beams), axial forces and moment of columns (at the start and end) are calculated for the structures under two schemes are compared. The story drifts for both the structures are calculated under both the schemes and compared to check which scheme for a particular structure has lower story drift compared to the other scheme. The results of the story drifts are shown graphically in figure-26 and 27 for structure-01 and figure-30 and 31 for structure-02 in X and Y direction. The results for beam type-01 in figure-40 and 43 for all floors are shown below while rest of the tabulated results are shown in Appendix-A and graphical results are shown on figure-44-63.

6. DISCUSSION AND ANALYSIS RESULTS OF STRUCTURE-01:

The following results are obtained from the dynamic analysis of the retrofitted models under both schemes for structure-01.

Scheme-01: Structure initially retrofitted using Steel Bracing

Scheme-02: Structure retrofitted using ductile reinforced shear walls

6.1 Time Period:

The time periods of the fundamental modes of structure-01 after the seismic retrofitting under both schemes are tabulated below.

Modes	Time Period (sec)
1	1.9011
2	1.338
3	0.9832
4	0.6154
5	0.4445
6	0.3490
7	0.3265

8	0.2557
9	0.247
10	0.2442
11	0.2352
12	0.1837

Table-01: Time period of structure under scheme-01

Modes	Time Period (sec)
1	2.099
2	1.0933
3	0.9794
4	0.677
5	0.5362
6	0.4762
7	0.3841
8	0.363
9	0.3302
10	0.281
11	0.2664

12	0.2631

Table-02: Time period of structure under scheme-02

The time period of structure-01 under scheme-01 is less compared to the time periods under scheme-02. The value of time periods largely depends on the flexibility and mass of the structure. Larger time periods in the structure gives more flexibility to the structure. Flexible structure can undergo large relative horizontal displacements which may result in the damage of the structure. Thus seismic retrofitting under scheme-01 is considered more effective as compared to retrofitting under scheme-02.

6.2 Mass Participation Ratios:

The result obtained for the Modal mass participation ratios are shown below:

TABLE: Modal Participating Mass Ratios												
Period	UX	UY	UZ	SumUX	SumUY	SumUZ	RX	RY	RZ	SumRX	SumRY	SumRZ
Sec	Unitless	Unitless	Unitless	Unitless	Unitless	Unitless	Unitless	Unitless	Unitless	Unitless	Unitless	Unitless
1.901	0.802	0.000	0.000	0.802	0.000	0.000	0.000	0.147	0.000	0.000	0.147	0.000

TABLE: Modal Participating Mass Ratios												
Period	UX	UY	UZ	SumUX	SumUY	SumUZ	RX	RY	RZ	SumRX	SumRY	SumRZ
Sec	Unitless	Unitless	Unitless	Unitless	Unitless	Unitless	Unitless	Unitless	Unitless	Unitless	Unitless	Unitless
2.0949	0.8100	0.0000	0.0000	0.8100	0.0000	0.0000	0.0000	0.2400	0.0000	0.0000	0.2400	0.0000
1.0933	0.0000	0.7400	0.0000	0.8100	0.7400	0.0000	0.0001	0.0000	0.0037	0.0001	0.2400	0.0037
0.9794	0.0000	0.0043	0.0000	0.8100	0.7500	0.0000	0.0000	0.0001	0.6800	0.0002	0.2400	0.6800
0.6778	0.0984	0.0000	0.0000	0.9100	0.7500	0.0000	0.0000	0.3400	0.0000	0.0002	0.5700	0.6800
0.5363	0.0000	0.0085	0.0000	0.9100	0.7600	0.0000	0.5600	0.0000	0.0001	0.5600	0.5700	0.6800
0.4763	0.0000	0.0001	0.0000	0.9100	0.7600	0.0000	0.0028	0.0000	0.0126	0.5600	0.5700	0.7000
0.3842	0.0358	0.0000	0.0000	0.9400	0.7600	0.0000	0.0000	0.0403	0.0000	0.5600	0.6100	0.7000
0.3631	0.0000	0.1800	0.0000	0.9400	0.9400	0.0000	0.0709	0.0000	0.0009	0.6400	0.6100	0.7000
0.3302	0.0000	0.0006	0.0000	0.9400	0.9400	0.0000	0.0003	0.0000	0.2300	0.6400	0.6100	0.9300
0.2807	0.0000	0.0011	0.0000	0.9400	0.9400	0.0000	0.2000	0.0000	0.0003	0.8300	0.6100	0.9300
0.2662	0.0066	0.0000	0.0000	0.9500	0.9400	0.0000	0.0036	0.0174	0.0018	0.8300	0.6300	0.9300
0.2631	0.0000	0.0000	0.0000	0.9500	0.9400	0.0001	0.0000	0.0000	0.0000	0.8300	0.6300	0.9300

Figure-25: Modal mass participation ratios of Structure-01 under Scheme-01

Figure-26: Modal mass participation ratios of Structure-01 under Scheme-02

Modal Participating mass ratios for structure-01 under scheme-01 and scheme-02 are shown in figure-24 and figure-25. Modal Participating mass ratios of Structure-01 under scheme-01 is 94% while under scheme-02, the ratio is 95%. In both the cases, the modal participating mass ratio is above 90% which shows that sufficient numbers of modes are combined to obtain modal mass participation of at least 90 percent of the actual mass in each of the orthogonal horizontal directions of response considered by the model

6.3 Base Shear:

Base shear (V_S) calculated from the dynamic analysis carried out using SAP-2000 software on the structure-01 under scheme-01 and scheme-02 are tabulated in table-03

Base Shear After Retrofitting in kN	
Retrofitted using Steel Bracing (Scheme-01)	**Retrofitted using Ductile Reinforced Shear Walls (Scheme-02)**
1407.527	1525.209

Table-03: Comparison of the base shear calculated from dynamic analysis of the structure-01 under scheme-01 and scheme-02

Base shear (V_S) calculated from the above table is 1407.527 kN for Scheme-01 while the base shear is 1525.209 kN for scheme-02. Base shear for scheme-02 is higher compared to scheme-01. Therefore, scheme-02 is more effective as compared to scheme-01. The base shear calculated from the response spectrum load case is 1211.95 kN for scheme-01 while it is 1307.521 kN for scheme-02. Therefore, the combined response for the modal base shear is less than 85 percent of the calculated base shear.

6.4 Bending Moments:

The bending moment demands and hence their capacity-demand ratios for the structure-01 under scheme-01 and scheme-02 are tabulated in table-04, 05 for beam type-01 on figure-40. The bending moments for beam type-02 and 03 are tabulated on Appendix-A on table-19, 20, 25, 26, 30, 31.

Before retrofitting, most of the beams and columns were deficient to withstand the bending moment demands as their capacities by demand ratios were less than one. But after retrofitting under both schemes, all the beams and column sections became safe for the bending moment demands as they have obtained capacity by demand ratios to be greater than one.

The variation of the capacity by demand ratios of the beams and the columns under scheme-01 and scheme-02 at supports and mid span are shown in Figure-44, 45, 46, and 47 on Appendix-A.

6.5 Shear Force:

Shear force demands (S_x) of the beams with their capacity by demand ratios of structure-01 after retrofitting under scheme-01 and scheme-02 are tabulated in table-06 (scheme-01) and table-07 (scheme-02) for beam type-01 in figure-40. The results of beam type-02 and beam type-03 in figure-40 are tabulated in appendix-A on table 21, 22, 27, 28, 33, 34. Before retrofitting, most of the beams were deficient to withstand shear forces and obtained capacity by demand ratios less than one.

After retrofitting under both schemes, capacity by demand ratios comes out to be greater than one which makes them safe to withstand shear forces.

Comparative variations of storey shear at different storey after retrofitting under both schemes are shown in figure-48 and figure-49 in Appendix-A.

6.6 Axial Force:

Axial forces capacity by demand of the columns of the structure-01 after retrofitting under scheme-01 is tabulated in table-08 and under scheme-02 is tabulated in table-09 for column type-01 shown on figure-40 and the results for column type-02 are shown on appendix-A.

All the columns have become safe after the retrofitting procedure under both schemes since their capacity demand ratios are greater than one. During seismic evaluation two types of column reinforcement arrangement are considered for interior and exterior columns as are shown in figure-40

The variations of the capacity by demand ratios for the axial force and moment (M_{ux}) of the columns after retrofitting under both are shown in figure-60, 61 (Column type-01) and figure-62, 63(Column type-02).

6.7 Storey Drifts (Δ_s):

The storey drift (Δ_s) for the structure-01 under scheme-01 and scheme-02 in X and Y direction are plotted in figure-26 and 27. Storey drifts calculated under scheme-01 is less compared to scheme-02.

BENDING MOMENT CAPACITY DEMAND RATIO FOR THE BEAMS AFTER RETROFITTING WITH SCHEME-01

Beam Designation	Capacity (C) (kN-m)			Demand (D) (kN-m)			C/D Ratio		
	Start	Midspan	End	Start	Midspan	End	Start	Midspan	End
B_{1-1-1}	178.52	148.73	178.52	108.54	86.6	97.36	1.645	1.717	1.834
B_{1-2-1}	178.52	148.73	178.52	101.23	84.9	94.85	1.764	1.752	1.882
B_{1-3-1}	178.52	148.73	178.52	115.32	90.54	110.95	1.548	1.643	1.609
B_{1-4-1}	178.52	148.73	178.52	112.35	89.54	99.59	1.589	1.661	1.793
B_{1-5-1}	178.52	148.73	178.52	95.64	83.74	92.84	1.867	1.776	1.923
B_{1-6-1}	178.52	148.73	178.52	112.35	89.54	99.59	1.589	1.661	1.793
B_{1-7-1}	178.52	148.73	178.52	101.23	84.9	94.85	1.764	1.752	1.882
B_{1-8-1}	178.52	148.73	178.52	94.23	84.9	91.35	1.895	1.752	1.954
B_{1-9-1}	178.52	148.73	178.52	112.35	89.54	99.59	1.589	1.661	1.793
B_{1-10-1}	178.52	148.73	178.52	108.54	86.6	97.36	1.645	1.717	1.834
B_{1-11-1}	178.52	148.73	178.52	94.23	84.9	91.35	1.895	1.752	1.954
B_{1-12-1}	178.52	148.73	178.52	115.32	90.54	110.95	1.548	1.643	1.609
B_{1-13-1}	178.52	148.73	178.52	101.23	84.9	94.85	1.764	1.752	1.882

Table-04: Bending Moment C/D ratio after retrofitting under Scheme-01 for Beam type-01 on figure-40

BENDING MOMENT CAPACITY DEMAND RATIO FOR THE BEAMS AFTER RETROFITTING WITH SCHEME-02

Beam Designation	Capacity			Demand			C/D Ratio		
	Start	Midspan	End	Start	Midspan	End	Start	Midspan	End
B_{1-1-1}	175.69	146.46	175.69	110.6388	106.752	122.26	1.588	1.372	1.437
B_{1-2-1}	175.69	146.46	175.69	107.68	100.367	104.822	1.632	1.459	1.676
B_{1-3-1}	175.69	146.46	175.69	103.537	98.452	100.885	1.697	1.488	1.741
B_{1-4-1}	175.69	146.46	175.69	98.594	94.822	96.732	1.782	1.545	1.816
B_{1-5-1}	175.69	146.46	175.69	99.816	90.534	93.468	1.760	1.618	1.880
B_{1-6-1}	175.69	146.46	175.69	93.844	85.361	89.482	1.872	1.716	1.963
B_{1-7-1}	175.69	146.46	175.69	107.68	100.367	104.822	1.632	1.459	1.676
B_{1-8-1}	175.69	146.46	175.69	101.833	95.771	99.573	1.725	1.529	1.764
B_{1-9-1}	175.69	146.46	175.69	110.6388	106.752	122.26	1.588	1.372	1.437
B_{1-10-1}	175.69	146.46	175.69	98.594	94.822	96.732	1.782	1.545	1.816
B_{1-11-1}	175.69	146.46	175.69	107.68	100.367	104.822	1.632	1.459	1.676
B_{1-12-1}	175.69	146.46	175.69	103.537	98.452	100.885	1.697	1.488	1.741
B_{1-13-1}	175.69	146.46	175.69	99.816	90.534	93.468	1.760	1.618	1.880

Table-05: Bending Moment C/D ratio after retrofitting under Scheme-02 for Beam type-01 on figure-40

SHEAR FORCE CAPACITY DEMAND RATIO FOR THE BEAMS AFTER RETROFITTING WITH SCHEME-01(Structure-01)

Beam Designation	Capacity (C) in kN		Demand (D) in kN		C/D Ratio	
	Start	End	Start	End	Start	End
B_{1-1-1}	161.00	161.00	102.26	108.52	1.574	1.484
B_{1-2-1}	161.00	161.00	99.57	102.84	1.617	1.566
B_{1-3-1}	161.00	161.00	125.60	110.59	1.282	1.456
B_{1-4-1}	161.00	161.00	93.56	99.67	1.721	1.615
B_{1-5-1}	161.00	161.00	113.52	99.67	1.418	1.615
B_{1-6-1}	161.00	161.00	92.57	100.36	1.739	1.604
B_{1-7-1}	161.00	161.00	98.56	101.61	1.634	1.584
B_{1-8-1}	161.00	161.00	99.57	102.84	1.617	1.566
B_{1-9-1}	161.00	161.00	93.56	99.67	1.721	1.615
B_{1-10-1}	161.00	161.00	102.26	108.52	1.574	1.484
B_{1-11-1}	161.00	161.00	125.60	110.59	1.282	1.456
B_{1-12-1}	161.00	161.00	113.52	99.67	1.418	1.615
B_{1-13-1}	161.00	161.00	99.57	102.84	1.617	1.566

Table-06: Shear Force C/D ratio after retrofitting under Scheme-01 for Beam type-01 on figure-40

SHEAR FORCE CAPACITY DEMAND RATIO FOR THE BEAMS AFTER RETROFITTING WITH SCHEME-02 (Structure-01)

Beam Designation	Capacity		Demand		C/D Ratio	
	Start	End	Start	End	Start	End
B_{1-1-1}	160.000	160.000	94.870	101.8300	1.687	1.571
B_{1-2-1}	160.000	160.000	84.540	96.6600	1.893	1.655
B_{1-3-1}	160.000	160.000	93.572	91.5880	1.710	1.747
B_{1-4-1}	160.000	160.000	90.334	94.5830	1.771	1.692
B_{1-5-1}	160.000	160.000	90.567	92.8430	1.767	1.723
B_{1-6-1}	160.000	160.000	83.482	85.1990	1.917	1.878
B_{1-7-1}	160.000	160.000	88.367	92.4280	1.811	1.731
B_{1-8-1}	160.000	160.000	94.870	101.8300	1.687	1.571
B_{1-9-1}	160.000	160.000	87.295	90.4340	1.833	1.769
B_{1-10-1}	160.000	160.000	90.334	94.5830	1.771	1.692
B_{1-11-1}	160.000	160.000	90.567	92.8430	1.767	1.723
B_{1-12-1}	160.000	160.000	90.334	94.5830	1.771	1.692
B_{1-13-1}	160.000	160.000	93.572	91.5880	1.710	1.747

Table-07: Shear Force C/D ratio after retrofitting under Scheme-02 for Beam type-01 on figure-40

C/D RATIO ESTIMATION OF THE COLUMNS FOR AXIAL FORCE AND BENDING MOMENT AFTER RETROFITTING WITH SCHEME-01

Column Designation	Capacity (C) (kN)			Demand (D) (kN-m)			C/D Ratio		
	Axial Load P_{ur}(kN)	Moment M_{urx} (kN-m)	Moment Murz (kN-m)	Axial Load Pur (kN)	Moment Murx (kN-m)	Moment Murz (kN-m)	For P_u	For M_{ux}	For M_{uz}
$C_{1\text{-}0\text{-}1}$	2192.000	219.000	219.000	1624.021	205.384	209.676	1.350	1.066	1.044
$C_{1\text{-}1\text{-}1}$	2192.000	219.000	219.000	1499.104	211.650	214.872	1.462	1.035	1.019
$C_{1\text{-}2\text{-}1}$	2192.000	219.000	219.000	1254.726	213.146	215.410	1.747	1.027	1.017
$C_{1\text{-}3\text{-}1}$	2192.000	219.000	219.000	1099.646	213.965	215.149	1.993	1.024	1.018
$C_{1\text{-}4\text{-}1}$	2192.000	219.000	219.000	1764.278	200.264	208.462	1.242	1.094	1.051
$C_{1\text{-}5\text{-}1}$	2192.000	219.000	219.000	1389.684	165.970	173.284	1.577	1.320	1.264
$C_{1\text{-}6\text{-}1}$	2192.000	219.000	219.000	1340.930	185.689	192.864	1.635	1.179	1.136
$C_{1\text{-}7\text{-}1}$	2192.000	219.000	219.000	1280.492	189.469	195.762	1.712	1.156	1.119
$C_{1\text{-}8\text{-}1}$	2192.000	219.000	219.000	1344.520	169.862	176.592	1.630	1.289	1.240
$C_{1\text{-}9\text{-}1}$	2192.000	219.000	219.000	1326.462	165.490	169.712	1.653	1.323	1.290
$C_{1\text{-}10\text{-}1}$	2192.000	219.000	219.000	1306.750	151.427	156.836	1.677	1.446	1.396
$C_{1\text{-}11\text{-}1}$	2192.000	219.000	219.000	1241.350	163.794	172.513	1.766	1.337	1.269
$C_{1\text{-}12\text{-}1}$	2192.000	219.000	219.000	1224.350	159.840	166.699	1.790	1.370	1.314
$C_{1\text{-}13\text{-}1}$	2192.000	219.000	219.000	1119.760	168.954	176.591	1.958	1.296	1.240

Table-08: C/D ratio estimation of columns for axial force and Bending Moment after retrofitting under Scheme-01 for Column type-01 on figure-40

C/D RATIO ESTIMATION OF THE COLUMNS FOR AXIAL FORCE AND BENDING MOMENT AFTER RETROFITTING WITH SCHEME-02

Column Designation	Capacity (C) (kN)			Demand (D) (kN-m)			C/D Ratio		
	Axial Load P_{ur} (kN)	Moment M_{urx} (kN-m)	Moment M_{urz} (kN-m)	Axial Load P_{ur} (kN)	Moment M_{ux} (kN-m)	Moment M_{uz} (kN-m)	For P_u	For M_{ux}	For M_{uz}
$C_{1\text{-}0\text{-}1}$	2192.00	219.00	219.00	1524.32	210.23	212.67	1.438	1.042	1.030
$C_{1\text{-}1\text{-}1}$	2192.00	219.00	219.00	1411.89	215.83	217.05	1.553	1.015	1.009
$C_{1\text{-}2\text{-}1}$	2192.00	219.00	219.00	1387.29	204.42	206.72	1.580	1.071	1.059
$C_{1\text{-}3\text{-}1}$	2192.00	219.00	219.00	1252.19	211.12	214.86	1.751	1.037	1.019
$C_{1\text{-}4\text{-}1}$	2192.00	219.00	219.00	1149.83	204.59	206.34	1.906	1.070	1.061
$C_{1\text{-}5\text{-}1}$	2192.00	219.00	219.00	982.43	212.51	215.39	2.231	1.031	1.017
$C_{1\text{-}6\text{-}1}$	2192.00	219.00	219.00	1055.97	210.27	215.48	2.076	1.042	1.016
$C_{1\text{-}7\text{-}1}$	2192.00	219.00	219.00	1326.12	206.47	209.59	1.653	1.061	1.045
$C_{1\text{-}8\text{-}1}$	2192.00	219.00	219.00	1251.86	201.01	203.56	1.751	1.089	1.076
$C_{1\text{-}9\text{-}1}$	2192.00	219.00	219.00	1167.43	215.83	217.69	1.878	1.015	1.006
$C_{1\text{-}10\text{-}1}$	2192.00	219.00	219.00	1041.42	206.12	208.78	2.105	1.062	1.049
$C_{1\text{-}11\text{-}1}$	2192.00	219.00	219.00	980.82	202.16	205.35	2.235	1.083	1.066
$C_{1\text{-}12\text{-}1}$	2192.00	219.00	219.00	990.88	211.12	213.73	2.212	1.037	1.025
$C_{1\text{-}13\text{-}1}$	2192.00	219.00	219.00	1015.68	211.97	219.22	2.158	1.033	0.999

Table-09: C/D ratio estimation of columns for axial force and Bending Moment after retrofitting under Scheme-02 for Column type-01 on figure-40

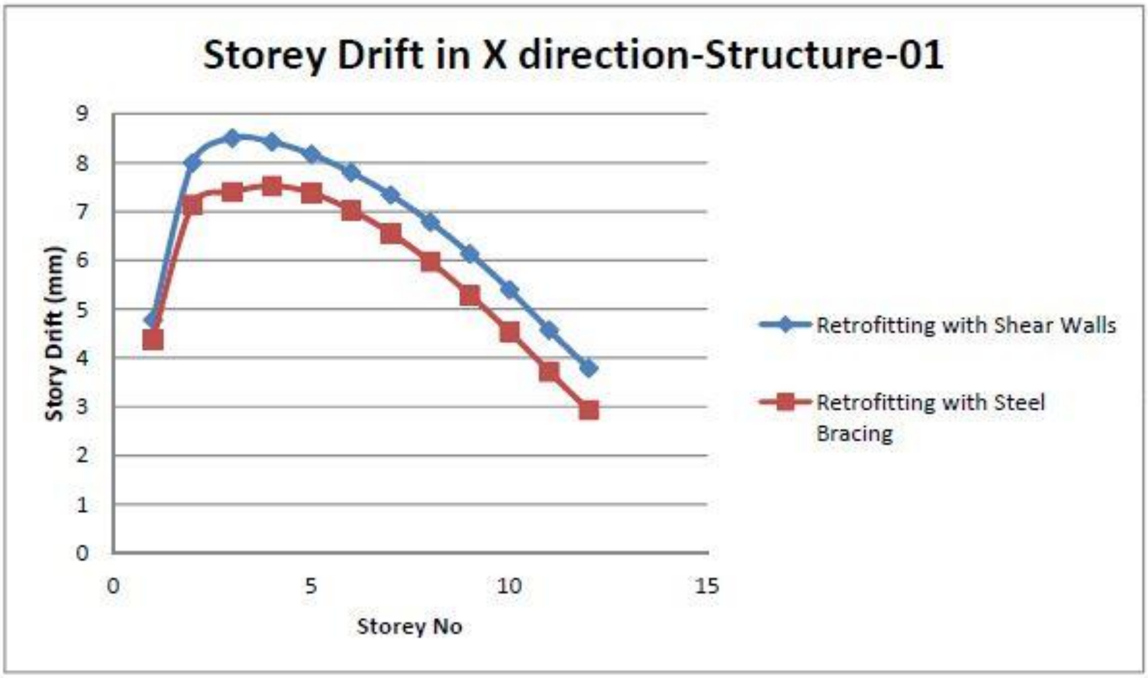


Figure-27: Story Drift of structure-01 in X-direction

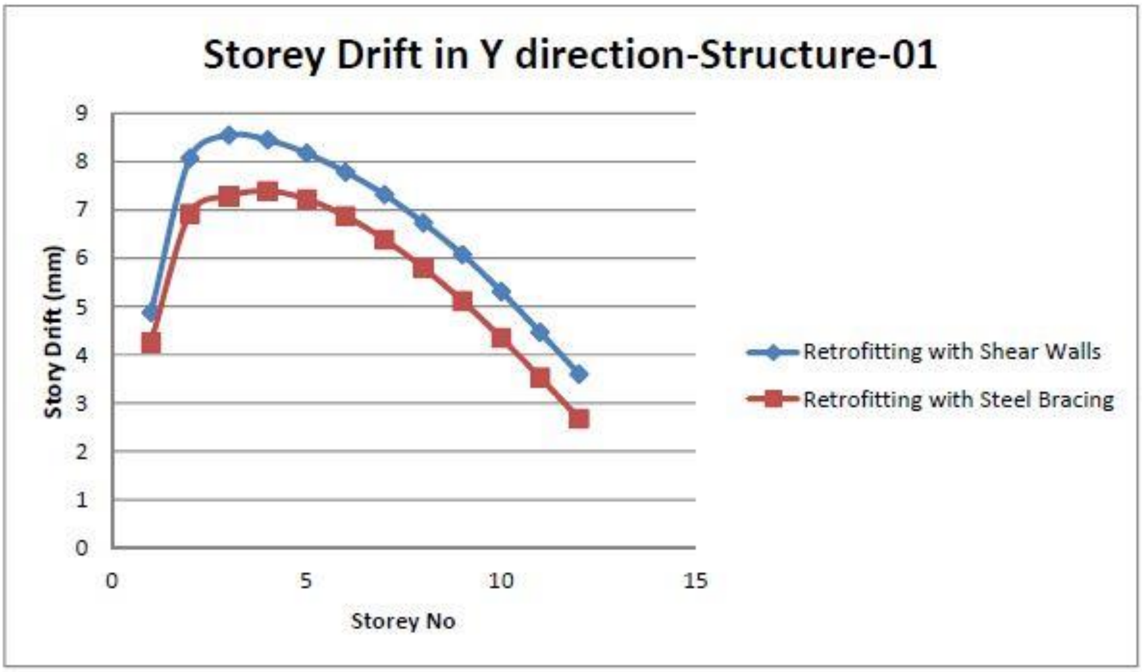


Figure-28: Story Drift of structure-01 in Y-direction

7. DISCUSSION AND THE RESULTS OF ANALYSIS - STRUCTURE-02:

The following results are obtained from the dynamic analysis of the retrofitted models under both schemes for structure-02.

Scheme-03: Structure initially retrofitted using ductile reinforced shear walls.

Scheme-04: Structure retrofitted using Steel Bracing

The dynamic analysis is carried out on structure-02 (constructed in Delhi) under both schemes and the results like time period, mass participation ratios, base shears, bending moments, shear forces and axial forces are compared. The results are shown on the tables below and are graphically represented in the appendix-A.

Initially seismic evaluation is carried out by estimating capacity and demand and thereby computing the capacity by demand ratios for the structure before and after retrofitting procedures. Before retrofitting, most of the members in the structure were unable to withstand the shear forces and bending moments. Bending moments of the section are calculated at the beginning, end of the section and at the midspan of the section while shear force is calculated for the start and end of the sections. But after the retrofitting procedure under scheme-01 and scheme-02, all the beams and column sections have their capacity by demand ratios greater than one.

7.2 Time Period:

The time periods of the fundamental modes of structure-02 after the seismic retrofitting under both schemes:

Modes	**Time Period (sec)**
1	1.3462
2	1.3402
3	1.2288
4	0.4318
5	0.43037
6	0.3968
7	0.3672
8	0.2984
9	0.2807
10	0.2411
11	0.2284
12	0.1972

Table-10: Time period of structure-02 under scheme-03

Modes	Time Period (sec)
1	1.6827
2	1.5501
3	1.3904
4	0.5468
5	0.4977
6	0.4456
7	0.3104
8	0.2833
9	0.2573
10	0.2088
11	0.1969
12	0.1811

Table-11: Time period of structure-02 under scheme-04

The time period of structure-02 under scheme-03 is less compared to the time periods under scheme-04. The value of time periods largely depends on the flexibility and mass of the structure. Larger time periods in the structure gives more flexibility to the structure. Flexible structure can undergo large relative horizontal displacements which may result in the damage of the structure. Thus seismic retrofitting under scheme-03 is considered more effective as compared to retrofitting under scheme-04.

7.2 Mass Participation Ratios:

The result obtained for the Modal Mass Participation Ratios are shown below:

TABLE: Modal Participating Mass Ratios												
Period	UX	UY	UZ	SumUX	SumUY	SumUZ	RX	RY	RZ	SumRX	SumRY	SumRZ
Sec	Unitless	Unitless	Unitless	Unitless	Unitless	Unitless	Unitless	Unitless	Unitless	Unitless	Unitless	Unitless
0.9881	0.0000	0.8732	0.0000	0.0000	0.8732	0.0000	0.1461	0.0000	0.0000	0.1461	0.0000	0.0000
0.9792	0.8747	0.0000	0.0000	0.8747	0.8732	0.0000	0.0000	0.1070	0.0000	0.1461	0.1070	0.0000
0.7593	0.0000	0.0000	0.0000	0.8747	0.8732	0.0000	0.0000	0.0000	0.8147	0.1461	0.1070	0.8147
0.5265	0.0000	0.0092	0.0000	0.8747	0.8825	0.0000	0.5305	0.0000	0.0000	0.6766	0.1070	0.8147
0.5254	0.0041	0.0000	0.0000	0.8788	0.8825	0.0000	0.0000	0.4690	0.0000	0.6766	0.5760	0.8147
0.4303	0.0000	0.0000	0.0000	0.8788	0.8825	0.0000	0.0000	0.0000	0.0336	0.6766	0.5760	0.8483
0.3225	0.0534	0.0000	0.0000	0.9322	0.8825	0.0000	0.0000	0.0148	0.0000	0.6766	0.5908	0.8483
0.3176	0.0000	0.0504	0.0000	0.9322	0.9328	0.0000	0.0213	0.0000	0.0000	0.6979	0.5908	0.8483
0.2962	0.0000	0.0000	0.0000	0.9322	0.9328	0.0000	0.0000	0.0000	0.0849	0.6979	0.5908	0.9332
0.2162	0.0176	0.0000	0.0000	0.9498	0.9328	0.0000	0.0000	0.0500	0.0000	0.6979	0.6409	0.9332
0.2126	0.0000	0.0183	0.0000	0.9498	0.9511	0.0000	0.0578	0.0000	0.0000	0.7557	0.6409	0.9332
0.1995	0.0000	0.0000	0.0000	0.9498	0.9511	0.0000	0.0000	0.0000	0.0136	0.7557	0.6409	0.9468

Figure-29: Modal Mass Participation Ratios of Structure-02 under Scheme-03

TABLE: Modal Participating Mass Ratios												
Period	UX	UY	UZ	SumUX	SumUY	SumUZ	RX	RY	RZ	SumRX	SumRY	SumRZ
Sec	Unitless	Unitless	Unitless	Unitless	Unitless	Unitless	Unitless	Unitless	Unitless	Unitless	Unitless	Unitless
1.6827	0.7925	0.0000	0.0000	0.7925	0.0000	0.0000	0.0000	0.1494	0.0000	0.0000	0.1494	0.0000
1.5502	0.0000	0.7794	0.0000	0.7925	0.7794	0.0000	0.1889	0.0000	0.0000	0.1889	0.1494	0.0000
1.3905	0.0000	0.0000	0.0000	0.7925	0.7794	0.0000	0.0000	0.0000	0.7797	0.1889	0.1494	0.7797
0.5468	0.0996	0.0000	0.0000	0.8920	0.7794	0.0000	0.0000	0.3677	0.0000	0.1889	0.5171	0.7797
0.4977	0.0000	0.1132	0.0000	0.8920	0.8926	0.0000	0.4195	0.0000	0.0000	0.6084	0.5171	0.7797
0.4457	0.0000	0.0000	0.0000	0.8920	0.8926	0.0000	0.0000	0.0000	0.1120	0.6084	0.5171	0.8918
0.3105	0.0374	0.0000	0.0000	0.9295	0.8926	0.0000	0.0000	0.0391	0.0000	0.6084	0.5562	0.8918
0.2833	0.0000	0.0365	0.0000	0.9295	0.9291	0.0000	0.0467	0.0000	0.0000	0.6551	0.5562	0.8918
0.2573	0.0000	0.0000	0.0000	0.9295	0.9291	0.0000	0.0000	0.0000	0.0357	0.6551	0.5562	0.9274
0.2089	0.0210	0.0000	0.0000	0.9504	0.9291	0.0000	0.0000	0.0622	0.0000	0.6551	0.6183	0.9274
0.1969	0.0000	0.0198	0.0000	0.9504	0.9488	0.0000	0.0684	0.0000	0.0000	0.7236	0.6183	0.9274
0.1811	0.0000	0.0000	0.0000	0.9504	0.9488	0.0000	0.0000	0.0000	0.0192	0.7236	0.6183	0.9466

Figure-30: Modal Mass Participation Ratios of Structure-01 under Scheme-04

Modal Participating mass ratios for structure-02 under scheme-03 and scheme-04 are shown in figure-00 and figure-00. Modal Participating mass ratios of Structure-02 under scheme-03 is 94% while under scheme-04, the ratio is 95%. In both the cases, the modal participating mass ratio is above 90% which shows that sufficient numbers of modes are combined to obtain modal mass participation of at least 90 percent of the actual mass in each of the orthogonal horizontal directions of response considered by the model

7.3 Base Shear:

Base shear (V_S) calculated from the dynamic analysis carried out using SAP-2000 software on the structure-01 under scheme-03 and scheme-04 are tabulated in table-12.

Base Shear After Retrofitting in kN	
Retrofitted using Ductile Reinforced Shear Walls (Scheme-03)	**Retrofitted using Steel Bracing (Scheme-04)**
4464.997	3184.592

Table-12: Comparison of the base shear calculated from dynamic analysis of the structure-01 under scheme-03 and scheme-04

Base shear calculated from the above table is 4464.997 kN for Scheme-03 while the base shear is 3184.592 kN for scheme-04. Base shear for scheme-03 is higher compared to scheme-04. Therefore, scheme-03 is more effective as compared to scheme-04. The base shear calculated from the response spectrum load case is

3746.193 kN for scheme-01 while it is 2667.476 kN for scheme-02. Therefore, the combined response for the modal base shear is less than 85 percent of the calculated base shear.

7.4 Bending Moments:

The bending moment demands and hence their capacity-demand ratios are calculated for the structure-02 under scheme-03 and scheme-04. Results of bending moments of three beams types (beams with different reinforcement arrangement shown in figure-43) on each floor are tabulated in table-13, 14 and tables 35-38 on appendix-A.

Before retrofitting most of the beams and columns were deficient to withstand the bending moment demands as their capacity by demand ratios was less than one. But after retrofitting under both schemes, all the beams and column sections became safe for the bending moment demands as they have obtained capacity by demand ratios to be greater than one.

The variation of the capacity by demand ratios of the beams and the columns under scheme-03 and scheme-04 at supports and mid span are shown in Figure-54-59.

7.5 Shear Force:

Shear force demands (S_x) of the beams with their capacity by demand ratios after retrofitting under scheme-03 and scheme-04 are tabulated in table-15(scheme-03) and table-16 (scheme-04) for beam type-01 as shown in figure-43. Results of bending moments of beams types-02 and beam type-03 on each floor are tabulated in table-27, 28, 33, 34, 3, 7, 38 on appendix-A.

Before retrofitting, most of the beams were deficient to withstand shear forces and obtained capacity by demand ratios less than one. After retrofitting under both schemes, capacity by demand ratios comes out to be greater than one which makes them safe to withstand shear forces.

Comparative variations of story shear at different storeys after retrofitting under both schemes are shown in figure-58, 59.

7.6 Axial Force:

Capacity by demand calculated for the axial force and uniaxial and biaxial bending moments of the columns after retrofitting under scheme-03 is tabulated in table-17 and under scheme-04 is tabulated in table-18 for column type-01 as shown in figure-43.

All the columns have become safe after the retrofitting procedure under both schemes since their capacity demand ratios are greater than one.

The variations of the capacity by demand ratios for the axial force of the columns after retrofitting under both are shown in figure-29 (column type-01) and figure-30 (column type-02).

7.7 Storey Drifts:

The storey drift (Δ_s) for the structure-02 under scheme-03 and scheme-04 in X and Y direction are plotted in figure-30, 31. Storey drifts calculated under scheme-04 is less compared to scheme-03.

BENDING MOMENT CAPACITY DEMAND RATIO FOR THE BEAMS AFTER RETROFITTING WITH SCHEME-03

Beam Designation	Capacity			Demand			C/D Ratio		
	Start	Midspan	End	Start	Midspan	End	Start	Midspan	End
B_{1-1-1}	231.25	186.60	231.25	186.69	150.95	170.55	1.239	1.236	1.356
B_{1-2-1}	231.25	186.60	231.25	183.53	147.60	163.82	1.260	1.264	1.412
B_{1-3-1}	231.25	186.60	231.25	180.87	146.15	161.73	1.279	1.277	1.430
B_{1-4-1}	231.25	186.60	231.25	120.59	143.60	116.48	1.918	1.299	1.985
B_{1-5-1}	231.25	186.60	231.25	125.49	138.49	107.11	1.843	1.347	2.159
B_{1-6-1}	231.25	186.60	231.25	225.68	137.78	103.21	1.025	1.354	2.241
B_{1-7-1}	231.25	186.60	231.25	113.83	91.85	181.05	2.032	2.032	1.277
B_{1-8-1}	231.25	186.60	231.25	228.43	120.87	230.88	1.012	1.544	1.002
B_{1-9-1}	231.25	186.60	231.25	228.09	212.48	225.36	1.014	0.878	1.026
B_{1-10-1}	231.25	186.60	231.25	227.19	140.60	224.39	1.018	1.327	1.031
B_{1-11-1}	231.25	186.60	231.25	222.34	210.87	228.47	1.040	0.885	1.012
B_{1-12-1}	231.25	186.60	231.25	222.43	183.48	228.80	1.040	1.017	1.011

Table-13: Bending Moment C/D ratio after retrofitting under Scheme-03 for Beam type-01 on figure-43

BENDING MOMENT CAPACITY DEMAND RATIO FOR THE BEAMS AFTER RETROFITTING WITH SCHEME-04

Beam Designation	Capacity			Demand			C/D Ratio		
	Start	Midspan	End	Start	Midspan	End	Start	Midspan	End
B_{1-1-1}	231.25	186.60	231.25	189.44	155.99	172.16	1.221	1.196	1.343
B_{1-2-1}	231.25	186.60	231.25	185.73	152.93	168.78	1.245	1.220	1.370
B_{1-3-1}	231.25	186.60	231.25	183.89	151.41	167.11	1.258	1.232	1.384
B_{1-4-1}	231.25	186.60	231.25	180.28	148.44	163.84	1.283	1.257	1.411
B_{1-5-1}	231.25	186.60	231.25	178.50	146.97	162.21	1.296	1.270	1.426
B_{1-6-1}	231.25	186.60	231.25	175.00	144.09	159.03	1.321	1.295	1.454
B_{1-7-1}	231.25	186.60	231.25	172.90	143.11	157.94	1.337	1.304	1.464
B_{1-8-1}	231.25	186.60	231.25	169.50	140.30	154.84	1.364	1.330	1.493
B_{1-9-1}	231.25	186.60	231.25	167.86	138.91	153.31	1.378	1.343	1.508
B_{1-10-1}	231.25	186.60	231.25	164.67	136.19	150.31	1.404	1.370	1.539
B_{1-11-1}	231.25	186.60	231.25	160.09	134.84	148.82	1.444	1.384	1.554
B_{1-12-1}	231.25	186.60	231.25	158.62	132.19	145.91	1.458	1.412	1.585

Table-14: Bending Moment C/D ratio after retrofitting under Scheme-04 for Beam type-01 on figure-43

SHEAR FORCE CAPACITY DEMAND RATIO FOR THE BEAMS AFTER RETROFITTING WITH SCHEME-03

Beam Designation	Capacity		Demand		C/D Ratio	
	Start	End	Start	End	Start	End
B_{1-1-1}	271.160	271.160	223.150	200.7360	1.215147	1.350829
B_{1-2-1}	271.160	271.160	212.486	194.8310	1.276130	1.391770
B_{1-3-1}	271.160	271.160	211.508	190.7460	1.282032	1.421576
B_{1-4-1}	271.160	271.160	211.248	187.2215	1.283610	1.448338
B_{1-5-1}	271.160	271.160	208.558	180.6744	1.300166	1.500821
B_{1-6-1}	271.160	271.160	204.867	181.7340	1.323590	1.492071
B_{1-7-1}	271.160	271.160	164.583	200.5956	1.647556	1.351774
B_{1-8-1}	271.160	271.160	164.960	191.6397	1.643792	1.414947
B_{1-9-1}	271.160	271.160	157.864	183.7556	1.717681	1.475656
B_{1-10-1}	271.160	271.160	174.835	184.6338	1.550948	1.468637
B_{1-11-1}	271.160	271.160	167.119	174.1189	1.622556	1.557327
B_{1-12-1}	271.160	271.160	148.811	171.5927	1.822177	1.580254

Table-15: Shear Force C/D ratio after retrofitting under Scheme-03 for Beam type-01 on figure-43

SHEAR FORCE CAPACITY DEMAND RATIO FOR THE BEAMS AFTER RETROFITTING WITH SCHEME-04

Beam Designation	Capacity		Demand		C/D Ratio	
	Start	End	Start	End	Start	End
B_{1-1-1}	271.16	271.16	226.83	240.51	1.195	1.127
B_{1-2-1}	271.16	271.16	222.52	235.65	1.219	1.151
B_{1-3-1}	271.16	271.16	216.25	229.23	1.254	1.183
B_{1-4-1}	271.16	271.16	214.16	226.75	1.266	1.196
B_{1-5-1}	271.16	271.16	210.10	222.67	1.291	1.218
B_{1-6-1}	271.16	271.16	206.40	218.50	1.314	1.241
B_{1-7-1}	271.16	271.16	171.95	204.65	1.577	1.325
B_{1-8-1}	271.16	271.16	168.58	200.64	1.608	1.351
B_{1-9-1}	271.16	271.16	163.94	195.11	1.654	1.390
B_{1-10-1}	271.16	271.16	162.22	193.07	1.672	1.404
B_{1-11-1}	271.16	271.16	159.22	189.49	1.703	1.431
B_{1-12-1}	271.16	271.16	156.32	186.05	1.735	1.457

Table-16: Shear Force C/D ratio after retrofitting under Scheme-04 for Beam type-01 on figure-43

C/D RATIO ESTIMATION OF THE COLUMNS FOR AXIAL FORCE AND BENDING MOMENT AFTER RETROFITTING WITH SCHEME-03

Col Designation	Capacity (C) (kN)			Demand (D) (kN-m)			C/D Ratio		
	Axial Load	Moment	Moment	Axial Load	Moment	Moment	For P_u	For M_{ux}	For M_{uz}
	P_{ur} (kN)	M_{urx} (kN-m)	M_{urz} (kN-m)	P_{ur} (kN)	M_{urx} (kN-m)	M_{urz} (kN-m)			
$C_{1\text{-}0\text{-}1}$	2991.17	348.97	348.97	2808.15	338.66	342.85	1.065	1.030	1.018
$C_{1\text{-}1\text{-}1}$	2991.17	348.97	348.97	2859.10	327.22	331.61	1.046	1.066	1.052
$C_{1\text{-}2\text{-}1}$	2991.17	348.97	348.97	2796.79	314.28	317.50	1.069	1.110	1.099
$C_{1\text{-}3\text{-}1}$	2991.17	348.97	348.97	2658.74	311.33	314.67	1.125	1.121	1.109
$C_{1\text{-}4\text{-}1}$	2991.17	348.97	348.97	2637.99	290.05	293.86	1.134	1.203	1.188
$C_{1\text{-}5\text{-}1}$	2991.17	348.97	348.97	2532.75	285.74	288.14	1.181	1.221	1.211
$C_{1\text{-}6\text{-}1}$	2991.17	348.97	348.97	2483.84	262.09	265.24	1.204	1.331	1.316
$C_{1\text{-}7\text{-}1}$	2991.17	348.97	348.97	2515.10	266.22	270.40	1.189	1.311	1.291
$C_{1\text{-}8\text{-}1}$	2991.17	348.97	348.97	2499.15	269.14	273.49	1.197	1.297	1.276
$C_{1\text{-}9\text{-}1}$	2991.17	348.97	348.97	2649.55	272.20	278.97	1.129	1.282	1.251
$C_{1\text{-}10\text{-}1}$	2991.17	348.97	348.97	2815.68	258.77	263.11	1.062	1.349	1.326
$C_{1\text{-}11\text{-}1}$	2991.17	348.97	348.97	2532.87	253.51	256.34	1.181	1.377	1.361
$C_{1\text{-}12\text{-}1}$	2991.17	348.97	348.97	2683.52	260.73	264.45	1.115	1.338	1.320

Table-17: C/D ratio estimation of columns for axial force and Bending Moment after retrofitting under Scheme-03 for Column type-01 on figure-43

C/D RATIO ESTIMATION OF THE COLUMNS FOR AXIAL FORCE AND BENDING MOMENT AFTER RETROFITTING WITH SCHEME-04

Column Designation	Capacity (C) (kN)			Demand (D) (kN-m)			C/D Ratio		
	Axial Load P_{ur} (kN)	Moment M_{urx} (kN-m)	Moment M_{urz} (kN-m)	Axial Load P_{ur} (kN)	Moment M_{urx} (kN-m)	Moment M_{urz} (kN-m)	For P_u	For M_{ux}	For M_{uz}
C_{1-0-1}	2991.17	348.97	348.97	2671.68	286.14	288.65	1.120	1.220	1.209
C_{1-1-1}	2991.17	348.97	348.97	2828.99	283.33	285.79	1.057	1.232	1.221
C_{1-2-1}	2991.17	348.97	348.97	2502.37	270.67	272.92	1.195	1.289	1.279
C_{1-3-1}	2991.17	348.97	348.97	2367.35	262.05	264.46	1.264	1.332	1.320
C_{1-4-1}	2991.17	348.97	348.97	2333.77	256.22	258.52	1.282	1.362	1.350
C_{1-5-1}	2991.17	348.97	348.97	2073.92	231.73	233.61	1.442	1.506	1.494
C_{1-6-1}	2991.17	348.97	348.97	1550.64	179.25	180.86	1.929	1.947	1.929
C_{1-7-1}	2991.17	348.97	348.97	1549.09	163.79	165.14	1.931	2.131	2.113
C_{1-8-1}	2991.17	348.97	348.97	1545.99	156.14	157.51	1.935	2.235	2.216
C_{1-9-1}	2991.17	348.97	348.97	1499.55	151.43	152.71	1.995	2.305	2.285
C_{1-10-1}	2991.17	348.97	348.97	2499.75	243.17	245.15	1.197	1.435	1.424
C_{1-11-1}	2991.17	348.97	348.97	2239.93	243.42	245.40	1.335	1.434	1.422
C_{1-12-1}	2991.17	348.97	348.97	1769.18	188.56	190.27	1.691	1.851	1.834

Table-18: C/D ratio estimation of columns for axial force and Bending Moment after retrofitting under Scheme-04 for Column type-01 on figure-43

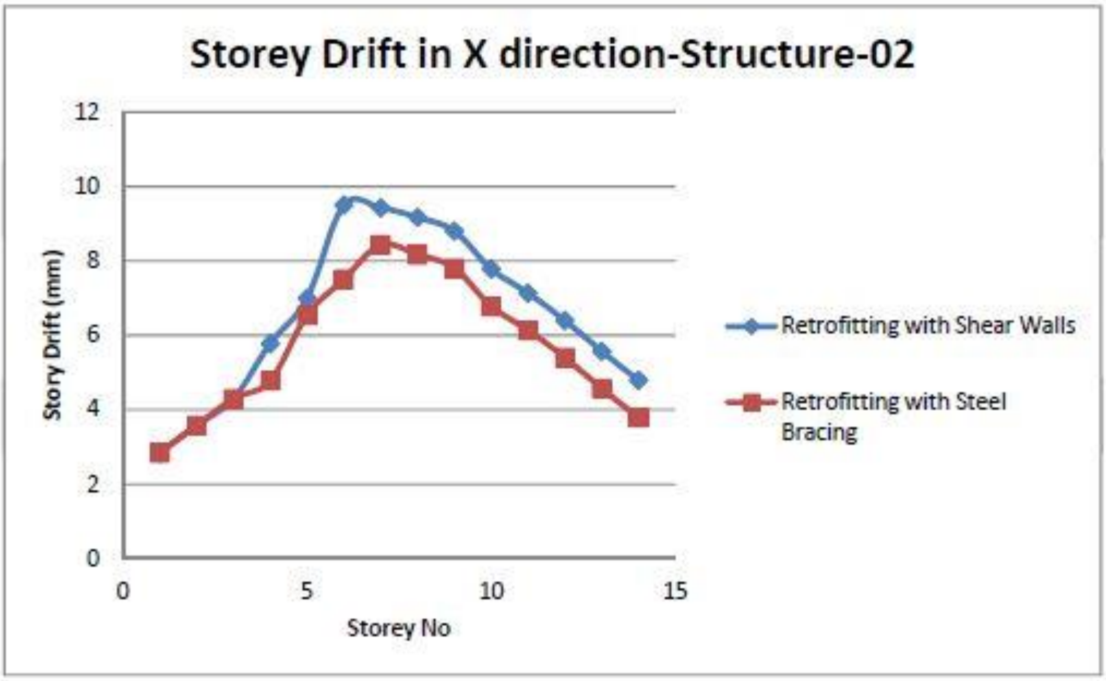


Figure-31: Story Drift of structure-02 in X-direction

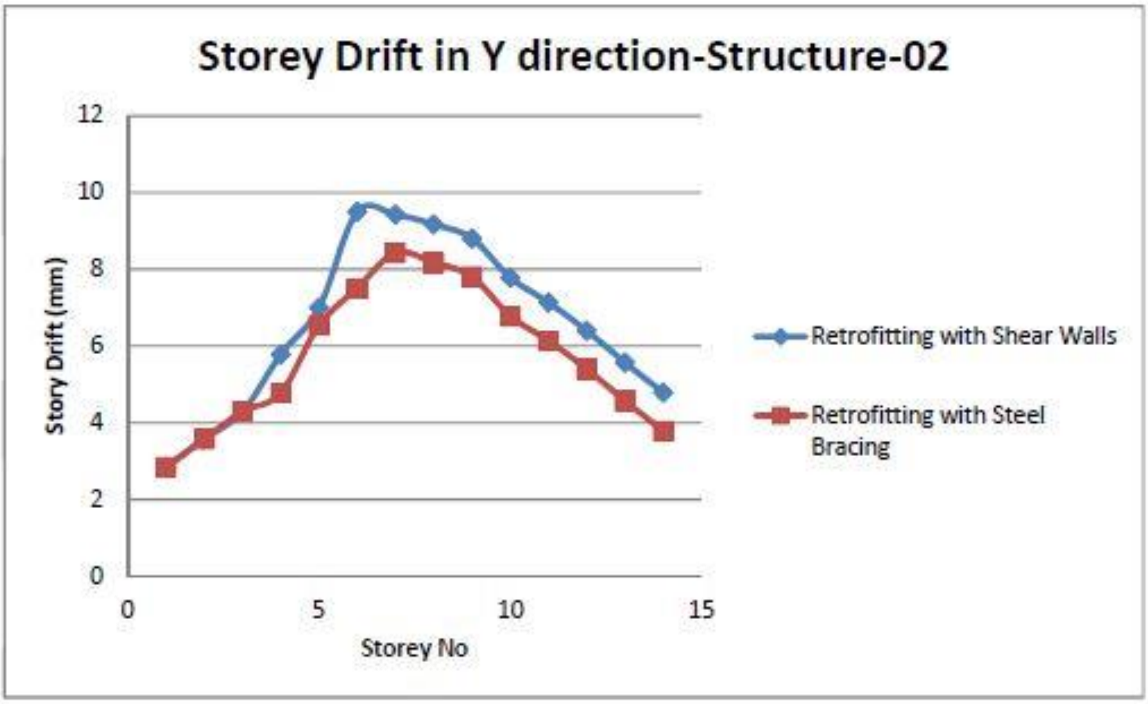


Figure-32: Story Drift of structure-02 in Y-direction

8. CONCLUSION:

The case study conducted on Structure-01 and Structure-02 invariably leads to the following conclusions:

- The seismic evaluation of both structures depicts most of the beams and columns in both the Structures to be deficient to resist design earthquake forces and in serious need of seismic retrofitting.
- The initial retrofitting scheme having been found inadequate in resisting the design earthquake forces, augmentation of retrofitting was envisaged in both structures, with new retrofitting schemes, and a comparison of the results.
- The comparison of time periods of Structure-01 under Scheme-01 (addition of steel bracings) and Scheme-02 (addition of ductile reinforced shear walls) substantiated the following:
    - The time period under Scheme-01 is less than that for Scheme-02.
    - Larger the time period in the Structure, higher the flexibility to the Structure and, flexible Structure can undergo large horizontal displacements.
    - Whereas similar comparison of Structure-02 under Scheme-03 (addition of ductile reinforced shear walls) and Scheme-04 (addition of steel bracings) indicated the time periods of Scheme-03 to be less than that in Scheme-04.
- The comparison of the results of base shear for Structure-01 showed the base shear to have increased in Scheme-02 when compared to Scheme-01. While for Structure-02, the base shear for Scheme-03 is much higher when compared to Scheme-04.
- The computation of the Mass Participation Ratios for Structure-01 and Structure-02 under both the Schemes indicated excellent ratios exceeding 90% in all the four cases.
- The calculation of the Shear Forces and Bending Moment for all beams and axial forces as well as Moment Capacity of all columns in all the four Schemes proved most of the members to be resistant to withstand the design earthquake forces.
- The result of Story drift has shown that for Structure-01, Scheme-02 has larger story drift as compared to Scheme-01 while for Structure-02, Scheme-03 has larger story drift as compared to Scheme-04.

9. RECOMMENDATIONS:

I recommend the following for further studies on the seismic retrofitting of both multi-storied Structures:

- The intensity of the shock being directly related to the soil type and the soil stratification, further studies are warranted to ensure the adequacy of the foundation based on the soil Structure interaction.

- In addition to the two seismic retrofitting Schemes (addition of ductile reinforced shear walls and addition of steel bracing) adopted for the present study, further researches could be carried out towards the application of addition of infill masonry walls, or base isolation techniques.

- All the analyses and designs for the current study were done in single computer aided software (SAP-2000), a widely accepted software in engineering for structural analysis and design. For further studies, it is desirable to use another computer application for checking, comparison and confirmation of the results, different structural analysis software is recommended rather than depending on a single software source.

- This study applied linear analyses on three-dimensional model with basic structural framing which excluded certain structural elements like lifts, stairs and mechanical equipment. Non-linear time-history analyses with response spectrum analysis were not carried out. Therefore, three-dimensional non-linear analyses in terms of time-history analysis and pushover analysis are strongly recommended in further studies.

- The detailing of framing elements and design of connections are of paramount importance. However, time constraints pre-empted these elements in the scope of this study. Consideration and application of connection designs in the seismic retrofits are, therefore, recommended in future research efforts.

10. REFERENCES:

- Agarwal, P. & Shrikande, M. (2007). *Earthquake Resistant Design of Structures*, Prentice Hall of India Private LTD, New Delhi, India.
- Agarwal,P & Navya, G. (2015). *Seismic Retrofitting of structures by steel bracings*, Department of Earthquake Engineering, Indian Institute of Technology, Roorkee.
- Agarwal, S.K., Chourasia, A. and Parashar, J. (2002).*Seismic evaluation and retrofitting of existing building-A case study*, 12th SEE-2002.
- Ashour, S.A., Hanson, R.D. (1987). *Elastic Seismic Response of Buildings with Supplemental Damping*, Report UMCE 87-1, Dept. of Civil Engineering, The University of Michigan, Ann Arbor, Michigan.
- ATC-40 (1996). *Seismic Evaluation and Retrofit of Concrete Buildings- Vol. I & II*, Applied Technology Council, California Seismic Safety Commission, Redwood City, California.
- ATC-40. (1996). *Seismic Evaluation and Retrofit of Concrete Building*, Applied Technology Council, Redwood City. CA.
- Calio, I. & Oliveto, G. (2004). *Retrofitting of Reinforced Concrete Buildings Not Designed To Withstand Seismic Action: A Case Study Using Base Isolation*, Department of Civil and Environmental Engineering, University of Catania, Italy.
- Clemente, P. and Stefano, A. (2012). *Seismic isolation of existing Complex Structures*, Casaccia Research Centre, Italy.
- Chopra, A.K. (1998). *Dynamics of Structure Theory and Application to Earthquake Engineering*, Prentice Hall Publication, New Delhi.

- Duggal, S.K. (2010). *Earthquake Resistant Design of Structures*, Oxford University press YMCA library building, Jai Singh road, New Delhi, India.
- Fiorato, A., Oesterle, R. and Corley, W. (1983). *Behavior of Earthquake Resistant Structural Walls before and After Repair*, ACI Journal.
- FEMA 273. (1997). *NEHRP Guidelines for the Seismic Rehabilitation of Buildings*, the Federal Emergency Management Agency, Washington, DC.
- FEMA 356. (2000). *Prestandard and Commentary for the Seismic Rehabilitation of Buildings*, the Federal Emergency Management Agency, 1801 Alexander Bell Drive, Reston, Virginia.
- Hanson, R.D. (1997). *Evaluation of Reinforced Concrete Members Damaged by Earthquakes*, Earthquake Spectra, Vol. 13, 1997.
- Gupta, K., Kumar, A. and Khan, A. (2015). *Review paper on seismic retrofitting of structured*, International Research Journal of Engineering and Technology (IRJET)
- IS: 1893-1984, *Criteria for Earthquake Resistant Design of Structures*, Bureau of Indian Standard, New Delhi.
- IS: 456-2000, *Plain and Reinforced Concrete Code of Practice (Fourth Revision)*, Bureau of Indian Standard, New Delhi.
- Leblouba, M. (2012). *Response spectrum analysis for regular base isolated buildings subjected to near fault ground motions*, Structural Engineering and Mechanics.
- Marletta, M. and Oliveto, G. (2005). *Seismic Retrofitting of reinforced concrete buildings using traditional and innovative techniques*, Department of Civil and Environmental Engineering, University of Catania, Italy.

- Marletta, M. and Calio, I. (2004). *Seismic Resistance of a Reinforced Concrete Building before and after retrofitting*, Department of Civil and Environmental Engineering, University of Catania, Italy.
- Marletta, M. and Calio, I. (2005). *Seismic Performance of s Reinforced Concrete Building not designed to withstand Earthquake loading*, Department of Civil and Environmental Engineering, University of Catania, Italy.
- Mahin, S., Mosalam, K. and Rojansky, M. (2003). *Evaluation of Seismic Performance and Retrofit of Lightweight Reinforced Concrete Shear walls*, ACI Structural Journal.
- Nanni, A., Lorenzis, L. and La Tegola, A. (2006). *Strengthening of Reinforced Concrete Structures with Near Surface Mounted FRP Rods*, Journal of Composites, Part B: Engineering.
- Newmark, N.M. & Hall, W.J. (1982). *Earthquake spectra and design*, EERI Monograph Series, EERI, Oakland, California, U.S.
- Oliveto,G. , Granata, M. & Ferraro, G. (2001). *Seismic design of new strategic buildings and seismic retrofitting of existing vulnerable buildings*, Department of Civil and Environmental Engineering, University of Catania, Italy.
- Oliveto, G. and Decanini, L.D. (1998). *Repair and Retrofit of a Six Storey Reinforced Concrete Building Damaged by the Earthquake in South-East Sicily on 13th December 1990*. Soil Dynamics and Earthquake Engineering.

- Patil, A.S & Kumbhar, P.D. (2013). *Time History Analysis of Multi-storied RCC buildings for different seismic intensities*, ISSN 2319 –6009, Vol. 2, No. 3.
- Rabganadhan, A. & Paul, P. (2005). *Seismic Retrofitting of an Existing Structure*, Dept. of Civil Engineering, TIST Arakkunnam.
- Rodriguez, M., EERI, M. and Part, R. (1991). *Repair and Strengthening of Reinforced Concrete Buildings for Seismic Resistance*, Department of Civil Engineering, University of Canterbury, New Zealand.
- Sarker,P. , Begum, M. and Nasrin, S. (2010). *Fiber Reinforced Polymers for Structural Retrofitting*, Department of Civil Engineering, Bangladesh.
- Sahin,C. (2014). *Seismic retrofitting of Existing Structures*, Portland State University.
- Sinha, D. (2002). *Seismic Retrofitting of Existing Reinforced Concrete Buildings,* Department of Earthquake Engineering, I.I.T., Roorkee.
- Structural Engineers Association of California (SEAOC). (1995). *Performance-based seismic engineering of buildings*, Proceedings, Vision 2000 Committee, SEAOC, Sacramento, California.
- Thakkar, S.K. & Agarwal, P. (2002*). Effect of shear wall location on response of retrofitted multi-storeyed building*, Department of Earthquake Engineering, I.I.T., Roorkee.
- Varam, V., Kumar, V. & Kumarraju, K.V. (2017).*Effect of shear wall position in multi-storied building*, Department of Civil Engineering, MVGR College of Engineering.

APPENDIX-A

(Attached in PDF file)-tables

APPENDIX-B

(Attached in PDF file)-tables