A Thesis on

Seismic Analysis of Reinforced Concrete Frame With Bracing Using Pushover Analysis

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By

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DECLARATION

I declare that the research thesis entitled "**Seismic Analysis of Reinforced Concrete Frame With Bracing Using Pushover Analysis**" is the bonafide research work carried out by me, under the guidance of **Mr. Md Tasleem Asst. Professor, Department of Civil Engineering, Integral University, Lucknow**. Further I declare that this has not previously formed the basis of award of any degree, diploma, associate-ship or other similar degrees or diplomas, and has not been submitted anywhere else.

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CERTIFICATE

It is Certified that the thesis entitled **"Seismic Analysis of Reinforced Concrete Frame With Bracing Using Pushover Analysis"** is being submitted by **Mr. Mohd Asad Khan (Roll no-1701432012)** in partial fulfillment of the requirement for the award of degree of Master of Technology (Structures) of Integral University, Lucknow, is a record of candidate's own work carried out by him/her under my supervision and guidance.

The results presented in this thesis have not been submitted to any other university or institute for the award of any other degree or diploma.

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List of Symbols and Abbreviations

Symbol	Description
C_0	Factor for MDOF displacement
C_1	Factor for inelastic displacement
C_2	Factor for strength and stiffness degradation
Sa	Response spectrum acceleration
g	Acceleration of gravity.
Te	Effective fundamental period of the building
Ts	Characteristic period of the response spectrum
$\mu_{strength}$	Ratio of elastic strength
H	Total height of building

Abbreviations

ACI	American Concrete Institute
SBSC	Square Building with Square Column
RBSC	Rectangular Building with Square Column
ASCE	American Society of Civil Engineers
NTC	Norme tecniche per le costruzioni
EC	Euro code

MPMR	Modal Participating Mass Ratios
EMM	Effective Modal Mass
ETABS	Extended Three-Dimensional Analysis of Building System
IO	Immediate Occupancy
IS	Indian Standard
LS	Life Safety
MDOF	Multi Degree of Freedom
MMPA	Modified Modal Pushover Analysis
MPA	Modal Pushover Analysis
NSP	Non-Linear Static Pushover
RC	Reinforcement Concrete
NSP	Non-Linear Static Pushover
RBNB	Rectangular building no bracing
RBDB	Rectangular building diagonal bracing
RBCB	Rectangular building chevron bracing
SBNB	Square building no bracing
SBDB	Square building diagonal bracing
SBCB	Square building chevron bracing

ABSTRACT

The most of the RCC structures were flopped in the past because of horizontal load. Bracing systems are one of the horizontal loads opposing system which has underlying significance uncommonly in reinforced concrete structures. Different bracing systems are proficient enough for seismic reactions. Lateral load opposition in seismic inclined regions, has prompts harms to the construction, to defeat this under seismic excitation steel bracing has demonstrated best systems against earthquake impact. RC frames in seismic regions needs to plan in such a way that they resist considerable amount of lateral loads.

Bracings in RC structure are utilized on the grounds that it can withstand lateral loads due to earthquake, wind, and so forth it's one in everything about best techniques for lateral load resisting system. This strategy gave to lessen the lateral deflection of design. During this postulation 12 storey reinforced concrete frame is analyzed for the square arrangement of 36x36 m and rectangular arrangement 54x24 m by considering Zone - V for soil type-II. The study have been carried out by utilizing the ETABS 2016 programming. In this study the two models are analyzed for three different type of bracing, i.e. Unbraced, Chevron and Diagonal bracing outer edge, for the bracing angle ISA 130x130x8. Results are acquired by considering the parameters like maximum displacement, and base shear by comparing them according to three different seismic codes ASCE 41-13, NTC – 2008, and EC-8.

CHAPTER 1: INTRODUCTION

1.1 GENERAL

One of the most devastating natural disasters to happen in the world are earthquakes that are created by the movement of tectonic plates that lies beneath the crust and also by the volcanic activity. They vary in duration, from few seconds to few minutes, and vary in the intensity. The ground experiences vibrations under the action of earthquakes causing structures to experience high frequency motions caused by the inertia forces of the structure and its parts, that are, the tendency of the structure remains in its original position regardless of the ground move. One of the fundamental challenges when a structure is a seismically dynamic zone is the horizontal solidness of the design. It is because the external force was caused by the earthquake causes large deflection, thereby causing large internal force in the structure. Any structure has its own displacement capacity, i.e. the amount of the lateral displacement caused is limited.

High-rise buildings are closely linked to the city, they are a natural response to dense population, land scarcity and high land prices. Our country is a developing country; therefore, many residential areas and other public facilities are needed for social, ecological, economic and public needs. The structural elements of the building are subjected to gravity, meteorological loads, seismic forces and other man-made forces. Therefore, structural elements must be designed to withstand. In addition, structural engineers must be able to analyze two fundamental aspects of structural behavior.

1) The Stress, strain and deflection characteristics under a static loading condition.
2) The Response and vibration characteristics under the dynamic loading condition.

During seismic movement, deformations occur on the elements of the load-bearing system due to the response of buildings to ground movements. As a result of these deformations, internal forces develop through the elements of the load-bearing system and displacement behavior occurs throughout the building. The resulting displacement requirements vary depending on the rigidity and mass of the building. In general, buildings with higher stiffness and lower mass have a lower need for lateral displacement. On the contrary, the demand for mobility is increasing. On the other hand, each building has specific teleportation ability. In other words, the amount of lateral displacement a building can withstand without collapsing is limited. The purpose of reinforcement methods is to ensure that a building's movement requirements are kept below its movement capacity. This can mainly be achieved by reducing the expected displacement of the structure during large movements or by improving the displacement of the structure. The steel bracing frame is one of the structural systems used to resist seismic loads in multi-storey buildings. Many existing reinforced concrete buildings need to be modernized to overcome defect sand withstand seismic loads. Using steel bracing to reinforce or modernize reinforced concrete frames that are unsuitable for seismic reasons is a feasible solution. The steel bracing system is economical, easy to install, takes up little space and has the flexibility of design to meet the required strength and stiffness. During the last earthquake, some RC buildings were designed for gravity loads only, and buildings with non-woven components suffered moderate to severe damage. The phenomenon of damage to the reinforced concrete frame is due to insufficient transverse reinforcement of beams, columns and joints. Therefore, it is necessary to provide special mechanisms or mechanisms that improve the lateral stability of the structure. One of the main reinforcement approaches is the installation of new structural elements, such as steel bracing, to improve the earthquake resistance of structures using concentric and deflected steel bracing techniques. Although people of tense steel bracing in steel frames and bracing walls in reinforced concrete structures; In recent years, there have been a number of studies on the use of steel bracing in reinforced concrete structures.

Much of India is vulnerable to the extent of the damage caused by earthquake risks. Therefore, it is necessary to take into account seismic loads for the design of the structure. In buildings, lateral loads due to earthquakes are a matter of concern. These lateral forces can create critical stresses in the structure, cause undesirable stresses in the structure, cause undesired vibrations, or cause excessive lateral movement of the structure. Shake or drift is the magnitude of the horizontal displacement at the top of the building relative to its base. Traditionally, seismic design methods have been devised because the structure must be able to sustain mild and frequent shaking intensity without damage, so that the structure is usable after the event. The structure must withstand moderate seismic movement without structural damage, but structural and non-structural damage is possible. This limit state may correspond to seismic intensity equal to the highest experienced or expected seismic intensity on the site. In this study, the results obtained were for further analysis. The main parameters are taken into account in this study to compare the seismic performance of structures with and without steel bracing. Seismic analysis is a subset of structural analysis and is the calculation of a structure's response to earthquakes. A reinforced concrete building must be designed to be able to with stand overall loads at a certain degree of safety and with a certain degree of reliability, so when this design is finally made in the construction process, the expected performance of the structural building will increase. . to the mark. However, this ideal condition does not always materialize. The performance of the structure may be below the expected safety and lifespan criteria for various reasons. Several technologies can be selected for this purpose such as bracing, bracing walls, etc.

1.2 STRENGTHENING OF RC STRUCTURE WITH STEEL BRACING SYSTEM

Steel bracing is an efficient and economical method of resisting lateral forces in structural structures. Bracing systems have been used to stabilize the side part of the majority of the tallest building structures in the world, as well as being one of the main renovation measures. Bracing is effective because the diagonals operate under axial stress and therefore require a minimum member size to provide rigidity and resistance against transverse forces. Several researchers have studied various techniques such as filling walls, adding walls to existing columns, cladding columns and adding steel bracing to improve the strength and / or ductility of existing buildings. The bracing system improves the earthquake resistance performance of the frame by increasing the rigidity and load-bearing capacity of the frame. With the addition of bracing, load scan be transferred out of the frame and into the bracing, bypassing weak columns while increasing strength. Steel bracing is an effective structural system for buildings subject to wind or seismic side loads. Therefore, it is worth using steel bracing to modernize reinforced concrete frames with inadequate lateral strength.

1.3 PURPOSE OF BRACING

The overall purpose of bracing is to provide additional security against external loads in comparable self-built structures. The main function of the bracing system in the steel structure is that the lateral forces caused by wind, earthquakes, etc. is effectively transmitted to the building foundations.

1.4 STEEL BRACINGS

Bracing system is considered as an effective system to improve the rigidity and strength of reinforced concrete frame. It can have high lateral stiffness. The capacity of steel frames can be greatly enhanced under moderate to high magnitude earthquakes by increasing the energy absorption capacity of the structure and reducing the demand caused by seismic loads. Otherwise, connections and foundations that need to be reinforced will be affected by the use of brackets. In addition, it could not be better to change the original architecture of the building by using braces.

Due to its high efficiency and cost saving, RC Bracing Chassis System is widely used. The reinforced concrete brace system is effective if the braces are in the linear phase. The asymmetric response that develops when at the nonlinear stage begins while the lateral stiffness begins is declared. Previous studies have shown a limited redundancy of bracing reinforced concrete frames due to the concentration of seismic loads in a particular soil where phase strength and drift between highways are developed. .Plastic hinges begin to form in this soil and become vulnerable, resulting in the structure collapsing to the side. The frame beams must be sufficiently reinforced to resist the longitudinal shear forces developing from the concentric braces. To withstand seismic loads, steel bracing frames feature several bracing systems, such as concentric bracing, eccentric bracing, knee bracing, and mega bracing.

• External bracing

• Internal bracing

In external bracing, existing buildings are modernized by attaching local or global steel bracing to the exterior frames. Architectural concerns and the difficulty of providing proper connections between the steel bracing and the reinforced concrete frame are two of the shortcomings of this approach.

In the internal bracing method, buildings are modernized by incorporating a bracing system inside the units or individual panels of the reinforced concrete frame. The tie rod can be attached to the RC frame indirectly or directly.

1.5 TYPESOF BRACING

• Concentric Bracing System

• Eccentric Bracing System

Bracing system is concentric when the centerlines of brace elements intersect. The concentric bracing steel frame used in the structure consists of an X, a truss and a knee brace. X-braces are the most common type of bracing. The diagonal elements of the X and Chevron tie rods are subjected to tension and compression. The connections for the X-bracing are located at the beam-column joints. While the Chevron bracing elements are connected to the beam at the top and converge at a common point. The lateral stiffness of the frame is increased, resulting in increased natural frequency and reduced lateral deflection. The larger inertia force in the seismic zone is attracted due to the increased stiffness. Meanwhile, the axial compressive force in bracingly connected columns increases with decreasing bending moment and shear force in the column.

Eccentrically braced frames look similar to frames with Chevron were bracing. The difference between Chevron bracing and eccentric bracing is that the space between the bracing members at the highest gusset connection. In an eccentrically braced frame bracing members connect to separate points on the beam. The energy from the seismic activity through the plastic deformation is absorbed by the beam segment between the bracing members. The lateral stiffness of the system is reduced by eccentric bracing which improves the energy a dissipation capacity. Due to the eccentric connection of the braces to beams, the lateral stiffness of the system depends upon the flexural stiffness of the beams and columns, thus reducing the lateral stiffness of the frame. The vertical component of the bracing forces due to the earthquake causes the lateral concentrated load on the beams at the point of the connection of the eccentric bracing.

1.5.1 Cross Bracing

Cross bracing system is a system used to reinforce building structures in which diagonal supports intersect. Bracing systems can increase a building's resistance to seismic activities. Bracing is important in earthquake-resistant buildings because it helps keep the structure upright. The tie rod is often seen with two diagonal brackets set to an X shape; they support compression and traction. With different forces, one splint will be in tension while the other is in compression. It helps to make structures more stable and resistant to lateral forces. Braces can be applied to any rectangular frame structure, such as chairs and shelves. In metal construction, steel cables can be used due to their very high load-bearing capacity (although they cannot withstand any compressive load and therefore must be pre-bent).Common bracing applications include bridge supports (sides), as well as structural foundations. This method of construction maximizes the weight of loads that the structure can support. It is a common application during the construction of earthquake resistant buildings

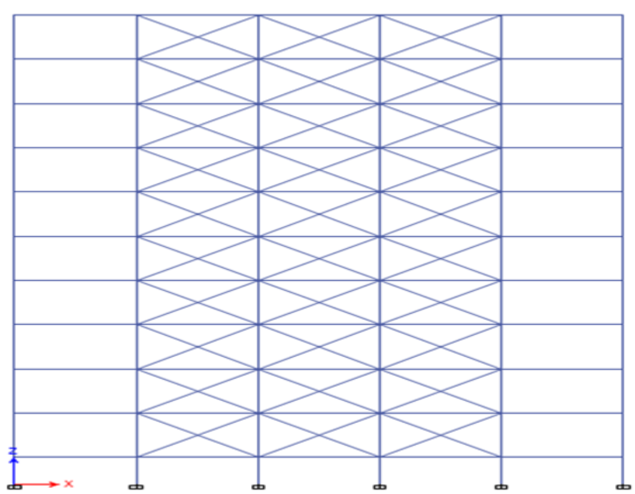

Fig 1.1: Cross Bracing

1.5.2 K Bracing

K bracing connects to the columns at dawn. This frame has more flexibility to provide openings in the facade and helps to minimize bending of the floor beams. Bracing is generally not recommended in seismic are as due to the risk of damage to the column if the compression bracing becomes deformed.

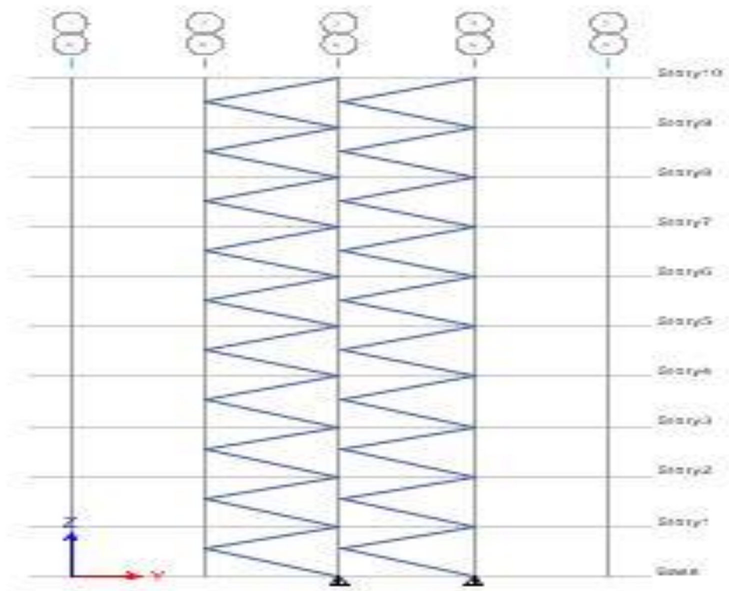

Fig 1.2: K Bracing

1.5.3 Chevron Bracing

Chevron concentric bracings are (CCBs), otherwise called upset V concentric supporting, are normally utilized in the seismic plan of multistory steel/RC structures attributable to both their design usefulness and primary effectiveness. In fact, structures are furnished with Chevron concentric propping (CCBs) are for the most part portrayed by enormous horizontal firmness, which ensures the satisfaction of both arranged float limits and steadiness models (a to be specific P-Delta impacts). Then again, the primary presentation against the solid seismic activity, including the huge plastic commitment, is unequivocally subject to the plan necessities and arrangements gave to guarantee the accomplishment of a bendable worldwide disappointment system.

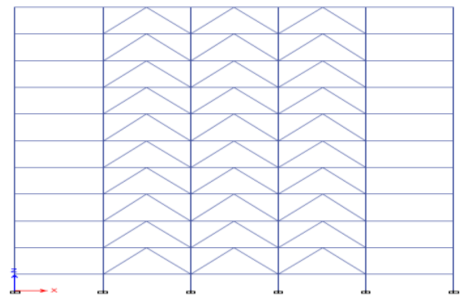

Fig 1.3: Chevron Bracing

1.5.4 Diagonal bracing

This type of bracing is preferred when available clearances in a span are required. The cress-crossing nature is impeding because it impedes the placement of the opening, which ultimately affects the aesthetics of the building's elevation. It also sometimes hinders the transition to use. Without structural impediments, inclining diagonal bracing is considered the best against horizontal wind forces as it frames a totally three-sided vertical bracket. Beams and columns are designed to withstand gravity loads only.

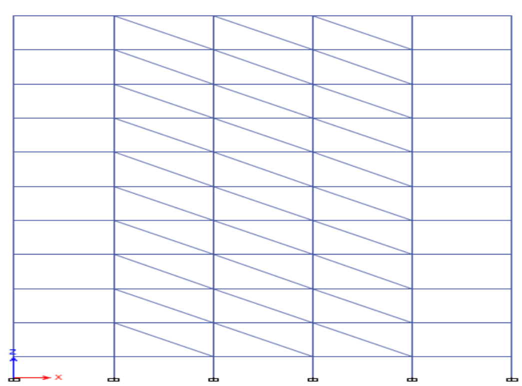

Fig 1.4: Diagonal Bracing

1.5.5 Eccentric bracing

Eccentric bracing is one such bracing arrangement that causes bending of the horizontal members of the folded brace. Typically, these braced bends resist the lateral forces caused by the bending action of beams and columns. This type provides less lateral stiffness and is therefore less effective than diagonal braces.

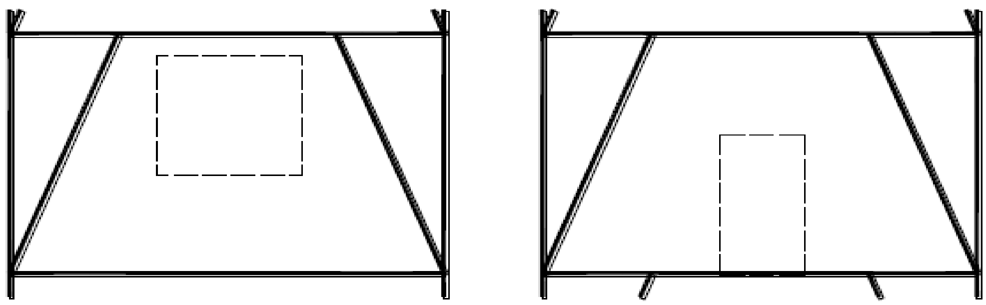

Fig1.5: Eccentric Bracing

1.6 STRUCTURAL MECHANISM OF BRACING SYSTEM

Different bracing systems behave differently under lateral and gravity loads. The mechanism of operation of each type of bracing system against two load cases (lateral and gravity) is explained below:

1.6.1Behavior of bracing under lateral loads

The design of high-rise buildings is governed by lateral forces caused by the wind. Bracing frames are considered to be the most effective against these lateral forces in both directions. The main purpose of bracing is to resist transverse forces caused by lateral forces. The transverse force resistance mechanism can be understood in terms of the path of the shear force along the frame. It can be explained by considering four types of bracing subjected to lateral loads.

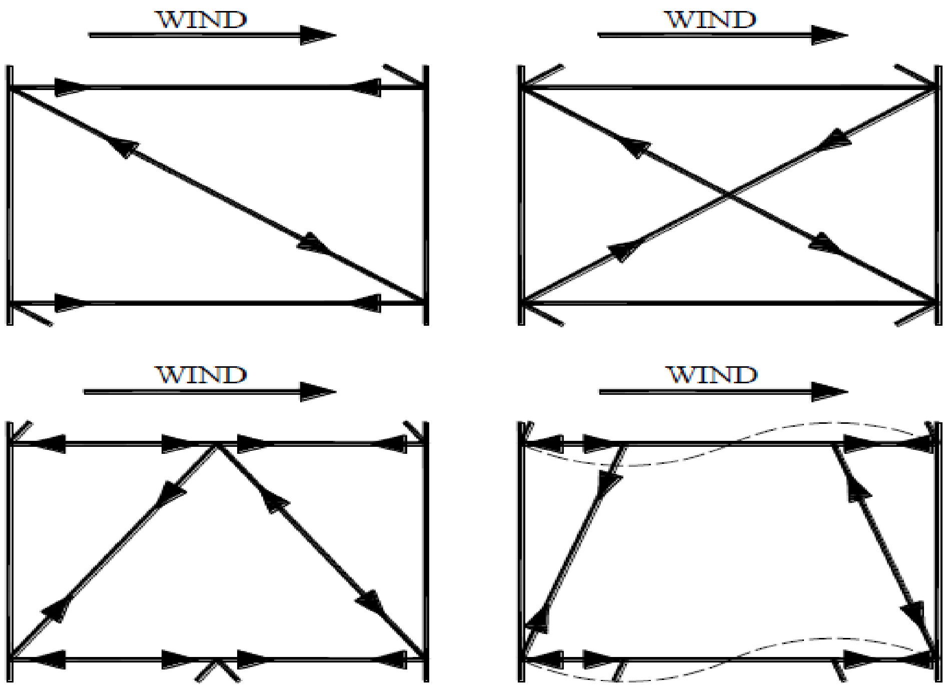

Fig1.6: Path of horizontal force

It is clear from Fig.1.6 that when the diagonals are subjected to compression, the transverse mesh members are subjected to axial tension for balancing in the lateral direction. This will cause shear deformation of the bent spacer. In the case of bracing, half of the web elements and both horizontal and inclined will be subjected to tension and compression simultaneously. The forces and stresses in each part of the bracing will be reversed when the building is loaded in the opposite direction.

1.6.2 Behavior of bracing under gravity loads

Under the action of gravity loads, the columns are axially short due to the compressive load. As a result, the diagonals are subjected to compression and the beam will experience axial tension due to the bonding action. In case the diagonals are not connected to the ends of the beams, the diagonal members will not support any force as there is no resistance provided by the beams to develop the force. Therefore, such bracing will not participate in resisting gravity loads.

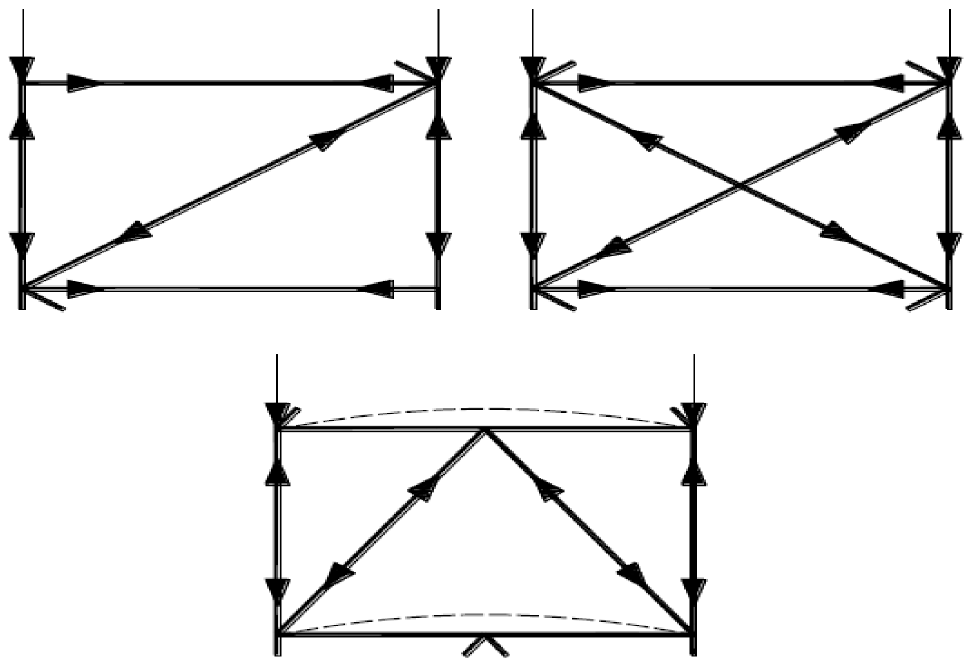

Fig1.7: Path of Gravity load

1.7 BRACED FRAME SYSTEM

The bracing system has a function similar to that of trusses acting as load-bearing and load-bearing elements of the structure. This system reinforces a structure to resist lateral forces from wind or seismic forces, the bracing system can be reinforced concrete bracing or steel bracing.

Diagonal bracing is the most effective bracing system because it fully supports internal loads while columns and beams can only support gravity loads. K-bracing and eccentric bracing are used when opening a window or door is required, these two types of bracing are less effective in supporting some of the side load than diagonal bracing.

1.8 ETABS

ETABS is specially developed for system building and structural analysis. Modeling simple buildings through ETABS can be quick and easy. ETABS can also perform structural analysis of complex building models. In addition, there are many design codes available in ETABS, such as European codes, Chinese codes, American codes, Indian codes, etc. User can specify elements or parameters of desired structural analysis based on desired code, response spectrum analysis, nonlinear static analysis, time history analysis, boost analysis all available in ETABS. Analysis output can be viewed graphically, tabular, and more. Since ETABS is primarily intended for structural design, the analysis options for ETABS are limited (CSI, 2003).

1.9 Statement of the Problem

In the past, most reinforced concrete structures were designed primarily to withstand gravity loads. They are also designed for lateral forces which may be much lower than those specified by current codes. An inadequate washer seal in the longitudinal brace and no limitation in the bend hinge areas can drastically reduce the strength and ductility of the column. Structures with such defects can be prevented from earthquake damage by proper restoration.

1.10 OBJECTIVES

Following are the main objectives of the present study:

a) To investigate the seismic performance of a multi-story RC frame building (square and rectangle)with and without bracing arrangements (Unbraced, Diagonal bracing, Chevron bracing), using Nonlinear Static Pushover analysis method by comparing them according to three different seismic codes ASCE 14-13, NTC – 2008, and EC-8.

b) To evaluate the performance factors for Unbraced frames with diagonal bracing and Chevron bracing.

1.11 METHODOLOGY

a) A thorough literature review to understand the seismic evaluation of both building structures and application of pushover analysis.

b) Seismic behaviour of RC frames with respect to Unbraced, Diagonal bracing and Chevron bracing in both type of buildings.

c) Model the selected in seismic behaviour of RC frames (both building) with and without bracing computer software ETABS (2016).

d) Carry out pushover analysis of seismic behaviour of RC frames with respect to Unbraced, Diagonal bracing and Chevron bracing of both buildings. And arrive at a conclusion.

CHAPTER 2: LITERATURE REVIEW

2.1 GENERAL

Providing a detailed review of the literature involved in the modeling of whole structures would be difficult to cover in this chapter. This section will briefly present previous studies on the application of RC framework pushover analysis. This literature review focuses on recent contributions related to the deep analysis of the RC framework and past efforts most closely related to the needs of the current work.

[1] Abhijeet Baikerikar, Kanchan Kanagali (2014), In this investigation utilized square network of 20m toward every path of 5m sound toward every path, programming utilized is ETABS 9.7.0, we have thought about the consequences of uncovered edge and supported casing and discovered the outcome that propped outline fundamentally bring down the sidelong removals and floats contrasted with exposed casing and subsequently opposing seismic tremor powers proficiently. The investigation has been completed for the Zone V and delicate soil as determined in IS 1893-2002.Concluded Bare Frame produces bigger relocations and floats contrasted with other two cases. Bracings in center have the most reduced time span contrasted with different cases.

[2] Chetan Jaiprakash Chitte (2014), In this study, the effect of various types of concentric braces on the behavior of the structure is tried to be evaluated by using pushover analysis which can be used with due achievement of the economy taking achievement of the economy taking of different types of bracings- Braced, Diagonal Braced, Chevron braced, V- Braced, K- Braced is analyzed using SAP2000.ConcludedThe capacity curves for all types of Braces states that the capacity of the structure is increased. by applying the braces. By using the braces the capacity of structure to withstand an earthquake can being increased. The chevron type of brace gives moderate performance during an earthquake, since the capacity and ductility both are the achieved.

[3] G.Hymavathi, B. Kranthi Kumar, N. Vidya Sagar Lal (2015), 20 storey steel frame structure has been selected to be idealized as the multi storey steel building model. The model is analyzed by using Staad.Pro 2008. At the same time the influence of X-bracing the pattern has been investigated. In this study the node displacements of buildings having with and without bracing of the wind and an earthquake effect of Zone II and Zone V. Concluded Zone II and Zone V earthquake effect that axial force the exterior columns in braced structure are low when compared with unbraced structure, from the top floor both are same. In the interior columns almost same in braced and unbraced structures. The variation of Zone II and Zone V the earthquake effect that Axial Force in the exterior columns in braced and unbraced structure is same.No huge variety is found in the inside column.

[4] Shachindra Kumar Chadhar, Dr. Abhay Sharma (2015), In this study, A G+15 storey reinforced concrete building of 4 bays have been considered for investigating the effect of V type and inverted type bracings and there arrangements in various positions in the building. Concluded Inverted V bracing framework fundamentally decreases the bending moment and shear force than V type bracing framework. Node displacement and story drifts are least for inverted V supported casing when contrasted with V braced outline.

[5] NitinBhojkar, Mahesh Bagade (2015), In this examination, the seismic investigation of reinforced concrete (RC) structures with various kinds of bracing is studied.. A G+9 building is analyzed for seismic zone III as per IS 1893: 2002 using STAAD Pro software. The principles boundaries consider in this paper to think about the seismic examination of structures are lateral displacement, storey drifts, axial force and base shear. It is found that the X type of steel bracing significantly contributes to the structural stiffness and reduces the maximum inter storey drift of the frames. Concluded the lateral displacement of the building is reduced up to 65% by using X type of bracing system. Stiffness of the building is increases. Story drifts are reduces using X type of bracing systems. The axial force is most for X bracing system is up to 22%.

[6] Sachin Dhiman, Mohammed Nauman, Nazrul Islam (2015), In the current examination, the seismic performance of steel structures under braced and unbraced system is researched. A fourteen story building is analyzed for seismic zone IV according to IS 1893:2002 utilizing STAAD V8i software. The performance of various kinds of bracing system has been analyzed.

Concluded the displacement of the structure decreases after the application of bracing system. The greatest decrease in the lateral displacement happens after the use of cross bracing system. Bracing system decreases bending moments and shear forces in the columns. The performance of cross bracing system is superior to the other specified bracing systems.

[7] A Kadid, D. Yahiaoui (2011), they said that new tremors in Turkey (1999), Taiwan (1999) and Algeria (2003) showed the cataclysmic effect of such force upon metropolitan urban areas. A decent number of existing structures in Algeria planned without seismic plan model and enumerating rules for dissipative underlying conduct endured harms which were far more awful than that for fresher structures planned and worked with regards to the more tough seismic code rules. Accordingly, it's of basic significance that the designs that require need seismic retrofitting are accurately distinguished, and an ideal retrofitting is led in an extremely savvy style. Among the retrofitting procedures accessible, steel supports might be thought about commonly of the first productive answer for seismic execution overhauling of RC outline structures. This paper examines the seismic conduct of RC structures reinforced with various types of steel supports, X-propped, rearranged V supported, ZX supported, and Zipper propped. Static nonlinear sucker investigation has been led to assess the limit of 3 story and 6 story structures with various support outline frameworks and distinctive cross areas for the supports. It's discovered that adding supports upgrades the overall limit of the structures as far as strength, disfigurement and flexibility contrasted with the case with no propping, and thusly the X and Zipper propping frameworks performed better looking on the sort and size of the cross segment.

[8] Bharat Patel, Rohan Mali, Prataprao Jadhav (2017), In this, three structure configuration moment resisting frame, X braced frame, V braced frame for G+10building.The frame model analyzed using STADD Pro and ETABS. Parameters considered are bases hear and storey displacement. Concluded The base shear of braced buildings increased as compared to building without bracing which indicates that the stiffness of building increases. The storey displacement of the building is diminished by 55% to 60%by using XBF and VBF. The performance of XBF has maximum margin of safety when compared to VBF.

[9] Dia Eddin Nassani, Ali Khalid Hussein, Abbas Haraj Mohammed (2017), The paper presents an examination of the seismic reaction of steel frames by utilizing various types of bracing systems. The bracing systems are X- bracedframes, V braced frames, inverted V braced frames, Knee braced frames and zipper braced frames. The steel frames are modeled and examined in four different height levels. Concluded the time history investigation was like that of the pushover examination to such an extent that the braced frames show lower displacement and harm in comparison with unbraced frames.IVF, VBF and ZBF show lower storey displacement and between storey drift ratio demonstrating that these systems have strength and stiffness.

[10] Soundarya N. Gandhi, Y.P. Pawar, D. D. Mohite (2017), In this paper presents steel bracings such as X, V, Inverted V. A building is situated at seismic zone V. The building (G+14) models are analyzing using software ETABS, and compare the seismic analysis of buildings for lateral displacement, storey drift, base shear. Concluded The seismic responses in X and Y direction namely bases hear for 15 storied RC structures with X bracing gives maximum result for base shear as compare to without bracing .For structure with bracing X have minimum storey displacement. Storey displacement is uniformly increasing. When structure unbraced and it is maximum at top floor of the structure.

[11] Umesh.R.Biradar, Shivaraj Mangalgi (2017), In this study,7 models with different bracing systems have been modeled. There are various types of bracing systems in common use such as single diagonal bracing, X bracing, V bracing K bracing, inverted V bracing.. Concluded Fundamental time period decreases at the point when the arrangements of various types of bracings are thought of. X bracing appearance great execution in x and y direction henceforth it tends to be suggested. At the point when the impact of various kinds of bracings is given in various models the storey drift and storey displacement get diminished leading to a safe and stiff model.

[12] SachinMetre, Shivanand C ghule, Ravi kiran (2017), In this paper models are compared for different types of bracing such as X, chevron and Single diagonal bracing by placing in different locations like outer Edge, Inner Edge and at centre in X and Y-directions for the bracing angle ISA 130x130x8.ConcludedThe bracing system effectively reduces the lateral displacement and drift for the A bracing of the structure compared to other bracings. Storey

Shear increases for the Bracing models especially for the X bracing .compared to chevron and Single Diagonal bracing.

[13] AnesBabu, Dr. Balaji, K.V.G.D. (2017), In this examination, reinforced concrete framed structure (G+9) was displayed and investigated in three sections 1)Model without steel bracings and shear wall 2)Model with various bracing system 3) Model with shear wall. Bracings and shear walls were set at the center bays and this load of models were examined for seismic forces at various seismic zones utilizing ETABS 2015 software. Concluded The chevron kind of steel bracing is discovered to be more proficient in zones II&III and X type of bracing is discovered to be more productive in Zones IV&V. Steel braced assembling fundamentally lessens the lateral drift when compared with shear wall building. The storey drift of steel braced constructing is less when contrasted with the unbraced structure subsequently the general reaction of the structure diminishes.

[14] MalavikaManilal, Rajeeva S.V. (2017), In the present study, dynamic analysis of regular and plan irregular structures with shear walls at the corners is performed using ETABS . The dynamic behavior of these structures is studied in seismic zone V using the time history method (Bhuj). Storey displacements, storey drifts, time period and base shear of the normal and unpredictable models are compared. Concluded The displacements of regular model are higher when compared to the irregular models. The displacements both regular and irregular models are within the max. Allowable limit prescribed by the code IS1893-2002.The shear walls at the corners of the building are acting like stiffeners thereby reducing the intensity of lateral load acting on the structure. The unpredictable models are exposed to more modest time of vibration because of the presence of shear walls.

[15] AxayThapa, Sajal Sarkar (2017), In this study three models are generated with varying height with and without shear wall. G+5, G+10 and G+15 reinforced concrete frame models with and without shear walls are generated with varying structural member dimensions according to height. The models are analyzed by Static Method and Response Spectrum Method considering seismic zone Vin STAAD. ProV8i. Parameters like lateral displacement, story drift, bases hear and mode shapes are determined. Concluded bare-frame model showed higher displacement, than shear walled-frame model. A lot of expansion in the sidelong solidness has been likewise seen in all models of shear wall outline when contrasted with bare frame. The variety in displacement between the unbraced frame and shear wall frame model increment with the increment of height, the variety in displacement of the two frames for G+5 floors was nearly not exactly that of G+ 15 floors. The displacement values will depend upon frequency of earthquake and natural frequency of the structure and building with short time period will in general experience higher accelerations increases but smaller displacement.

[16] Ayush Kumar, Urja Singh, Vaishali Vishnoi (2017), In this study, two high rise RCC structures have been analyzed along with various bracing systems using Staad.Pro V8i ETABS software. Concluded Out of different arrangements of bracing, V bracing came out to be least proficient and X bracing was best effective bracing system. X bracing are more compelling in expanding lateral load capacity of structure. X bracing are also proficient to reduce lateral displacement and storey drift produced due to seismic vibration sand inertia force. The variety of bracing segment changes the impact of seismic forces. As per this research work, ISMC 300 was found better in resisting lateral loads when compared to ISMC 200 section.

[17] Swetha Sunil, Sujith P.S. (2017), In this study the seismic response of RC building with different type of bracings such as X, Inclined, V and Inverted V by performing response spectrum analysis. Compare various parameters like storey drift, storey displacement, base shear time period for different models considered. Identify suitable bracing system for resisting seismic load effectively. To study effect of aspect ratio by performing response spectrum analysis on most effective bracing system. Concluded shows by the addition of bracing story response like displacement, drift, time period can be reduced. X bracing is more effective than other bracing because it shows less seismic response than other bracing. Because it consist of two elements so it take both compression and tension By the addition of x bracing 45%reduction in displacement, time period. Base shear is increase after the application of brace but after the application of brace building become more stiff to absorb more base shear. After the application of bracing, reduction of displacement increases with increase in aspect ratio.

[18] V. Mani Deep, P. Polu Raju (2017), In this article non-linear static analysis has been done to understand the behavior of G+9 multi-storied residential building located in different seismic zones(II, III, IV, V) of India having similar geometrical properties usingSAP2000. The conduct of multi-storied structure has been explored as far as force-displacement, inelastic behavior of structure and hinges. Concluded As the Seismic Zone changes from II to V total storey drift, maximum base shear and time periods are increased gradually. The harm in building is restricted and columns at the lower storey should be retrofitted dependent on the importance of the structure.

[19] Harshitha M K, Vasudev M V (2018), This study is conducted to know the behavior of the different bracing system for different arrangements.G+10 building situated in zone IV is selected and analyzed with different braces. The performance of the building is studied in terms of lateral displacement base shear and time period. Concluded Maximum reduction in lateral displacement is observed in inverted V braced frame and X braced frames. Chevron braces and X-braces shows better resistance to storey displacement. Hence, it can be recommended.

[20] Dhangar Laxmi Balappa, Venu Malagavelli (2018), In this study, G+10 structure with and without bracings of 4 models has been analyzed using the SAP2000 software. The outcomes as far as Base Shear, Displacements, Time Period, Location of Hinges and Pushover curve are compared. Concluded It has been observed that bracings modeled buildings has less time period than RC Frame(without bracings) buildings, from which we can say that bracings model building is stiffer. The displacement and Drift graphs, it was seen that displacement is radically diminished in bracing models when compared with RC Frame.

[21] Madhusudan G. Kalibhat, Kiran Kamath (2018), In this examination, two different kinds of concentric bracings X and chevron type bracing have been considered for the diverse storey levels. Concluded The arrangement of bracing improves the base shears conveying capacity of frames and decreases roof displacement removal gone through by the constructions. The lateral storey displacements of the building are reduced by the use of chevron bracing in comparison to the X bracing system.

[22] Sripathi Siva Bhanu, Sai Kumar (2019), In this study, seismic performance of stiffness irregular structure was studied by using the response spectrum method, and the performance of structures with shear wall and bracings placed in the least lateral stiffness direction was studied. Concluded Frames with no horizontal force opposing systems have acquired more displacements when contrasted with that of frames with lateral load resisting systems. By placing the shear wall at centre and external bracings in stilt for a vulnerable frame the storey displacements at 1st, 2nd, 3rd, 4th,5th storey levels are reduced by46.1%, 45.65%,44.99%, 44.06%,43% and 41.32% respectively and maximum inter storey drift is reduced by 59.17%.

[23] A. Haamidh, C. Lakshmivarsha (2019), This investigation analyzes the seismic reaction of Reinforced Concrete frames joined with assorted steel bracings system presented to the ground movements of Chamoli earthquake. The structural reaction of Moment Resisting frames (MRs),V-Braced frames (VBs),Inverted V-Braced frames(IVBs) and X-Braced frames (XBs)are dissected for story heights of 4, 8,12 and 16. The reactions of each frame as far as base shear, displacement, plastic hinge development are concentrated by pushover analysis. ETABS 2016 software is utilized for carrying over both on-linear pushover and dynamic time-history analysis. Concluded The fundamental natural periods of braced frames are very well less than that of the unbraced MR frames. In correlation with MR frames the normal rate decrease in displacement if there should be an occurrence of XB is 86.11%,VB is 81.38% and IVB is 73.91%.Whereas the expansion in base shear in case of XB, VB and IVB are 1.34,1.81 and 1.73 occasions that of MRs separately. XBs have the minimum displacement while at the same time contrasting and that of the other braced frames yet considering the most extreme base shear and hinge development perspectives VBs holds the need for the frequent earthquake prone zones of Uttarakhand.

[24] Haamidh A., Sivasubramanian J.(2020), In this study, the structural exhibitions of the distinctive bracing systems are analyzed for storey height of 8, 12 and 16 considering the earthquake information of Bhuj. Different Structural Configurations like X-Braced Frames (XBFs), V-Braced Frames (VBFs), Inverted V-Braced Frames (IVBFs) and Moment Resisting Frames (MRFs) were taken under examination. The structural responses of these frames in terms of base shears. Displacement, hinge development, storey displacements, global harm list, over strength factor and design factor were concentrated through pushover analysis investigation. Concluded The fundamental natural periods for the unbraced MRFs are very much high than that of the braced frames. Thus frames with higher natural periods correspond to lower stiffness. The analyzed results of time history offers the same result as that of pushover analysis. It uncovers that the braced frames show lower displacement and considerably less Global Damage Index in correlation with the unbraced frames.

2.2 CODAL PROVISIONS

IS 1893(Part 1):2016: Criteria for Earthquake Resistant Design of Structures, Part 1: General Provisions and Buildings (Sixth Revision).

IS 875 (Part 1, 2, 3and5): Code of Practice for Design Loads (Other Than Earthquake) For Buildings and Structures.

1. Part 1: Dead Loads--Unit Weights of Building Materials and Stored Materials (Second Revision)
2. Part 2: Imposed Loads (Second Revision) by Bureau of Indian Standards (BIS)
3. Part 3: Wind Loads (Second Revision)
4. Part 5: Special Loads and Load Combinations (Second Revision).

IS 456:2000: Plain and Reinforced Concrete - Code of Practice.

ASCE/SEI4 : American Society of Civil Engineers.

NTC 2008 (2008), Normetecniche per le costruzioni" D.M. 14 Gennaio 2008.

Eurocode 8 (2004), "Design of Structures for Earthquake Resistance, Part-1: General Rules, Seismic Actions and Rules for Buildings", European Committee for Standardization (CEN), Brussels.

2.3 CLOSURE

This literature review shows the published papers till now on the bracing of RC frame with reference to their authors. It is briefly discussed about response of steel bracing on RC structural model, the analysis done using Etabs and the Codal provisions used in this thesis.

Pushover analysis provides insight into the elastic and inelastic response of structures under earthquakes provided that a full structural model is performed, the lateral load model is carefully selected, and the results are interpreted with care. However, the high pushover analysis is more appropriate for low to medium buildings with a dominantfundamental mode response. For special and high-rise buildings, the pushover analysis should be supplemented with other assessment procedures, as higher modes can certainly affect the response.

CHAPTER 3: THEORY AND FORMULATION

3.1 NON-LINEAR STATIC PUSHOVER ANALYSIS

The structural pushover analysis is a static nonlinear analysis under permanent vertical loads and increasing lateral loads. The plot of total fundamental shear versus maximum displacement in the structure obtained by this analysis will show premature failure or weakness. All the beams and columns having reached their capacity or having undergone crushing or even a rupture are identified. A graph of total baseline shear versus interstate drift is also obtained. Pushover analysis is performed by subjecting a structure to a monotonically increasing lateral load model, representing the inertial forces to which the structure is subjected when subjected to ground motion. Under increasing loads, multiple structural elements can fail in succession. Thus, with each event, the structure undergoes a decrease in stiffness. Using nonlinear static pushover analysis, a representative nonlinear force displacement relationship can be obtained.

3.1.1 Background

Nonlinear static analysis, or pushover analysis, has evolved over the past twenty years and has now become the most preferred analytical technique for the design and estimation of seismic performance purposes, because irrelatively simple and taking into account the post-elastic performance. However, this technique involves several approximations and simplifications so that some variation is always likely to exist in the seismic demand prediction from the boost analysis.

Although high pushover analysis is known to capture important structural response characteristics when the structure is subjected to seismic action, the reliability and accuracy of the pushover analysis in estimating the calculation of overall seismic requirements and premises for all structures has been a topic of discussion and is improved in Proposed Acceleration Procedures to overcome some limitations of traditional pushing techniques. However, improvement techniques are mostly computationally difficult and theoretically complex, so their use is impractical in the engineering profession and in code. Since traditional pushover analysis is widely used for seismic performance estimation and design purposes, its weaknesses, limitations, and predictive accuracy in a normal application should be identified by studying all the factors that prediction favors. That is, the applicability of pushover analysis to predict seismic demand should be investigated for low, medium and high rise structures recognizing some issues such as efficient modeling. Lateral load samples to demonstrate higher mode effects, variations in the estimates of different lateral load samples are used in traditional pushover analysis and precise estimation of target displacement or prediction of seismic demand propulsion is carried out.

3.1.2 Necessity of Non-linear static Pushover Analysis:

Since the main building of the Institute (the structure under study) was built more than 50 years ago, it can be easily stimulated by earthquakes. Therefore, to estimate the performance of the structure, a pushover analysis of the structure was performed. If the structure shows signs of damage, appropriate retrofit measures may also be suggested.

3.2 Purpose of doing pushover analysis

The purpose of the pushover analysis is to evaluate the expected performance of the structural system by estimating the performance of the structural system by estimating its resistance and deformation requirements during dynamic events. Design soils using static inelastic analysis and compare these needs with existing capabilities at performance levels of interest. Assessment is based on the assessment of important performance parameters including over all deflection, interstate drift, inelastic strain (absolute or normalized for yield strength), inter-member strain, and connection forces elements (for elements and connections that cannot withstand inelastic deformations), Static pushover analysis can be considered as a method to predict the demand and deformation of the seismic force, which roughly explains the redistribution of internal forces that no longer resist in the elastic range of structural behavior.

Pushovers are required to give data on many response characteristics that can't be acquired from elastic static or dynamic investigation. The following is an example of such response characteristics.

i) Realistic force requirements for potentially fragile elements, such as column axial force requirements, brace connection force requirements, moment requirements for bow column connections, and concrete beam shear force requirements.

ii) Estimate the deformation request of an element that must be formed elastically in order to dissipate the energy given to the structure.

iii) Consequences of the strength deterioration of individual elements on behaviour of the structural system.

iv) Identification of the critical regions in which the deformation demands are expected to be high and that have to become the focus through detailing.

v) Identification of the strength irregular in arrangement rise that will lead to changes in the dynamic qualities in elastic range.

vi) Estimated inter-story drift that can be used to account for strength and stiffness discontinuities and to control damage and assess the P Delta effect.

vii) Verification of the completeness and adequacy of load path, considering all the elements of the structural systems, all the connections, and stiff non-structural elements of significant strength, and the foundation system.

The last item is the most relevant item because the analytical model integrates all the elements that contribute significantly to the lateral load distribution, both structural and unstructured. Load transfer through connections through ductile elements can be seen in real force. You can evaluate the effect of the filled wall at the height of the hard part of the shear force of the column. And in many cases it is possible to estimate the maximum overturning moment of the wall, which is limited by the uplift capacity of the underlying element.

These advantages are associated with integrating all the important elements, modeling the inelastic load deformation properties of those elements, and preferably using a 3D analytical model to perform incremental inelastic analysis. It occurs at the cost of interpretation efforts.

3.3 Limitations of Pushover Analysis:

Although pushover analysis has certain advantages over elastic analysis methods, the accuracy of pushover prediction underlying various assumptions and limitations of current pushover procedures must be recognized. The target displacement estimation is an important issue affecting the selection of lateral load patterns, and the accuracy of specific pushover results of higher vibration modes-induced damage mechanisms. The target displacement is the Earth-scale displacement that a design earthquake could cause.

In pushover analysis, the target displacement of a multi-degree-of-freedom (MDOF) system is usually estimated in the same way as the displacement demand of the corresponding single-degree-of-freedom (SDOF) system. The basic properties of an equivalent SDOF system are taken from the shape vector that represents the biased shape of the MDOF system. Most researchers recommend using a displacement profile normalized by the target displacement level as the shape vector, but this displacement is not known in advance and requires repetition. Therefore, most approaches do not consider higher modes and use the elastic first mode, which is a fixed shape vector, for simplicity. The target displacement is determined by the displacement of the roof at the center of gravity of the structure.

Exact assessment of target relocation for a particular performance target affects the accuracy of seismic expectation in pushover analysis. In addition, the hysteresis characteristics of MDOF should be included in the equivalent SDOF model in case the displacement demand is affected by a decrease in stiffness or a decrease in clamping strength, PΔ effect. Rising of the foundation, twisting effects, and semi-rigid diaphragms can also affect the displacement of the object.

However, in the pushover analysis, an invariant lateral load pattern, which is generally considered to not change the distribution of inertial force during earthquakes, is used. This is what I experienced with design earthquakes Therefore, as a response of the structure, the capacity curve is very sensitive to the horizontal load distribution, and the choice of the horizontal load pattern is more important compared to the accurate estimation of the displacement of the object.

The immutable load pattern cannot explain the redistribution of inertial forces duet other gradual yield and changes in the dynamic properties of the structure. Fixed load patterns also lack the ability to predict the effects of higher modes in the post-elastic range. Due to these limitations, many researchers have proposed adaptive loading patterns that take into account changes in inertial forces that correspond to inelastic levels. The basic approach of this technique is to reconstruct the lateral load type using the degree of inelastic deformation. Better predictions have been found with adaptive load patterns, but pushover analysis is computationally difficult and theoretically complicated. Metrics of improvement at the expense of accuracy, simple invariant load patterns have been a widely favored topic of discussion. Here we used an invariant triangular load pattern.

3.4 PLASTIC HINGE

Area of inelastic activity of the structural member is called as plastic hinge.

3.4.1 Formation of Plastic Hinge

Maximum seismic moments are likely to occur near the ends of beams and columns to form plastic hinges, and most ductility requirements apply to sections near joints.

3.5 DESCRIPTION TO PUSHOVER ANALYSIS

The Federal Emergency Management Agency (FEMA) and the Applied Technical Commission (ATC) are two agencies that develop and propose nonlinear static analysis or pushover analysis under seismic rehabilitation programs and guidelines. This included documents FEMA356, FEMA273 and ATC40.

3.5.1 Introduction to FEMA-356

The main role of FEMA-356 archive is to supply actually technically sound and nationally acceptable rules for the seismic restoration of structures. The principles for the seismic restoration of the structures are proposed to fill in as a prepared instrument for plan proficient for finishing the arranging and investigation of the structures, regulatory officials and a foundation for the long run development and implementation of the code provisions and standards.

3.5.2 Introduction to ATC-40

Seismic evaluation and retrofit of concrete buildings commonly named as ATC-40 was developed by the Applied Technology Council (ATC) with funding from California Safety Commission. Although the procedures recommended during this document are for concrete buildings, they're applicable to most building types.

3.6 TYPES OF PUSHOVER ANALYSIS

There are two nonlinear static analysis procedures currently available. One is the Coefficient of Variation Method (DCM), the documented FEMA356 and the other is the Capacitive Spectrum Method (CSM) described in ATC40. Both methods rely on the ideal lateral load due to the variation in lateral load deformation and seismic effects obtained using nonlinear static analysis under gravitational loads. This analysis is called pushover analysis.

3.6.1 Displacement Coefficient Method:

Displacement coefficient method is a nonlinear static analysis procedure that provides a numerical process for estimating the displacement demand of a structure by calculating the target displacement using a bilinear representation of a capacity curve and a set of correction factors or coefficients. The points on the capacity curve of the target displacement correspond to the performance points of the capacity spectrum method.

3.6.2 Capacity Spectrum Method

The capacity spectrum method is a non-linear static analysis procedure that graphically represents the expected seismic performance of a structure by crossing the capacitance spectrum of the structure with the seismic response spectrum (demand spectrum). The intersection is called the performance point, and the displacement coordinate dp of the performance point is the expected displacement requirement of the structure for the specified seismic risk.

3.7 Pushover analysis procedure

Pushover analysis is a static non-linear procedure in which the magnitude of the lateral load increases monotonically along the height of the building while maintaining a predefined distribution pattern (Figure 3a).The building will be displaced until the "control node" reaches the "target displacement" or the building collapses. Procedure Observe the order of cracks, plastic hinges and breaks in the components of the overall structure. The relationship with the base shear control node displacement is displayed for all pushovers analyze (Figure 3b).

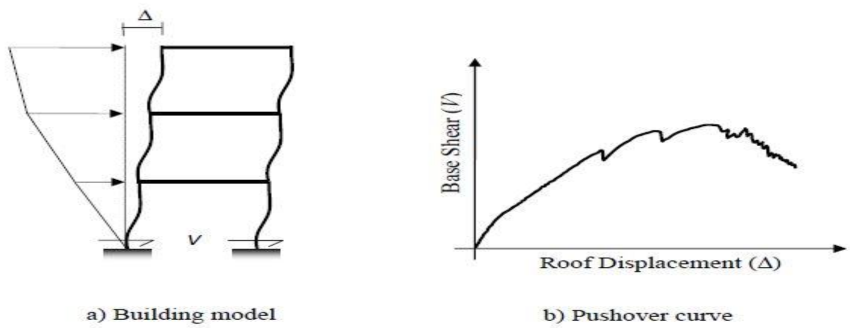

Fig 3.1: Schematic representation of pushover analysis procedure

Base Shear Generation-Control node displacement curves are the most important part of pushover analysis. This curve is commonly referred to as the pushover curve or capacitance curve. The capacitance curve is the basis for estimating the "target displacement". Therefore, the pushover analysis may be performed twice. Estimate the target displacement first until the building at (a) collapses, and (b) next time estimate the earthquake demand to the target displacement. The seismic requirements of the selected seismic(layer drift, floor force, component deformation and force) are calculated at the target displacement level.

Next, understand how the structure exhibits performance by comparing the seismic demand to the capacity of the structure or the state of the predefined performance limits. If you do not need to evaluate the effects in both direction sat the same time, you can analyze independently along each of the two orthogonal principal axes of the building.

The investigation result is sensitive to the determination of the control node and the choice of the lateral load pattern. Typically, a central location on the roof of a building is considered a control node. A set of guidelines according to FEMA356 is described for selecting a lateral load pattern in a push-over analysis. Transverse loads, usually applied in positive and negative directions, along with gravitational loads (part of dead and live loads) to study real-world behavior.

3.7.1 Lateral Load Patterns

In pushover analysis, buildings are pushed in a specific load distribution pattern based on the height of the building. The total force magnitude increases, but the load pattern remains the same until the end of the process. The results of the push-over analysis (i.e. the capacity seismic demand of the yield-order building of the push-over curved member) are very sensitive to the load pattern. The horizontal load pattern should approximate the inertial forces expected for the building during an earthquake. The distribution of horizontal inertia forces determines the relative magnitude of shear moments and deformations in a structure.

The distribution of these forces continuously changes during the seismic response as the yield and stiffness properties of the members change. It also depends on the type and magnitude of the earthquake. Although the distribution of inertial forces depends on the severity and time of the earthquake, the FEMA 356 mainly recommends an invariant load pattern for push-over analysis of framed buildings. Some investigations (Mwafy and Elnashai 2000; Gupta and Kunnath 2000) have found that the lateral load triangular or trapezoidal shape fits the dynamic analysis leads to the elastic range well, but with large deformations, the dynamic envelope approaches uniformly the dispersion force pattern. Since the uniform distribution method cannot capture these changes within the structural behavioral properties of seismic loads, FEMA 356 suggests using a minimum of two different patterns for all pushover analyses. The employment of two lateral load patterns is geared toward limiting the range that may occur in real dynamic response. FEMA 356 recommends selecting one load pattern from each of the subsequent two groups:
Group – I:

i) A vertical distribution based on the lateral force code used for equivalent static analysis (only allowed if at least 75% of the total mass participates in the primary mode of the orientation under consideration).

ii) A vertical distribution proportional to the shape of the primary mode in the direction under consideration (only allowed if at least 75% of the total mass participates in the mode).

iii) A vertical distribution proportional to the story shear distribution calculated by combining modal responses from a response spectrum analysis of the building (sufficient number of modes to capture a minimum of 90% of the entire building mass required to be considered). This distribution shall be used when the amount of the basic mode exceeds 1.0 second.

Group – II:

i) A consistent distribution consisting of lateral forces at each level proportional to the overall mass at each level.

ii) An adaptive load distribution that changes because the structure is displaced. The adaptive load distribution shall be modified from the initial load distribution employing a procedure that considers the properties of the yielded structure.

Instead of using the uniform distribution to bind the answer, FEMA 356 also allows adaptive lateral load patterns to be used but it doesn't elaborate the procedure. Although adaptive procedure may yield results that are more in line with the characteristics of the building into consideration it requires considerably more analysis effort. Fig. 2.2 shows the common lateral load pattern utilized in pushover analysis.

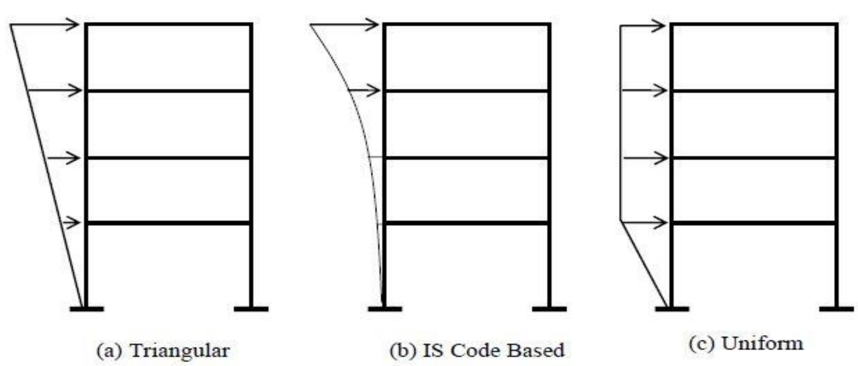

Fig 3.2: Lateral load pattern for pushover analysis as per FEMA 356(considering uniform mass distribution)

3.8 Performance point

A point on the pushover curve corresponding to the likely peak inelastic control-node displacement (x_{in}^{r}) during the ground shaking. In other words, the future earthquake is expected to push the building up to this point.

x_{in}^{r} is also sometimes referred to as the “Target Displacement”.

Fig 3.3: Pushover Analysis for Performance Evaluation of Building

3.9 Target Displacement

The displacement coefficient method of ASCE/SEI 4113 mainly assumes valid linear properties and damping of the seismic excitation under consideration, and estimates the elastic displacement of an equivalent single degree of freedom (SDOF) system. Multiplying such a series of displacement factors estimates the sum of the maximum inelastic displacement responses of the building on the roof. The expected maximum roof displacement (target displacement) of a building in a seismic motion selected at a specific performance level according to the displacement coefficient method (ASCE/SEI 4113) is given as

$$\delta_t = C_0 C_1 C_2 S_a \frac{T_e^2}{4\pi^2}$$

Sa = Response spectrum acceleration at the effective fundamental period and damping ratio of the building in the direction under consideration.

g = acceleration of gravity.

C_0 = Modification factor to relate spectral displacement of an equivalent single-degree-of-freedom (SDOF)system to the roof displacement of the building multi degree-of-freedom (MDOF) system calculated using one of the following procedures:

The first mode mass participation factor multiplied by the ordinate of the first mode shape at the control node.

The mass participation factor calculated using a shape vector corresponding to the deflected shape of the building at the target displacement multiplied by ordinate of the shape vector at the control node.

C_1 = Modification factor to relate expected maximum inelastic displacements to displacements calculated for linear elastic response. For periods less than0.2 s, C_1 need not be taken greater than the value at T = 0.2 s. For periods greater than 1.0 s, C_1= 1.0.

$$C_1 = 1 + \frac{\mu_{strength} - 1}{aT_e^2}$$

Where a = Site class factor:

= 130 Site Class A or B;

= 90 Site Class C;

= 60 Site Class D, E, or F;

Te= Effective fundamental period of the building in the direction under consideration, in seconds.

Ts= Characteristic period of the response spectrum, defined as the period associated with the transition from the constant acceleration segment of the spectrum to the constant velocity segment of the spectrum.

$\mu_{strength}$ = Ratio of elastic strength demand to yield strength coefficient calculated in accordance.

Use of the NSP is not permitted where $\mu_{strength}$ exceeds μ_{max}.

Number of Stories	Shear Buildings[a] Triangular Load Pattern (1.1, 1.2, 1.3)	Uniform Load Pattern (2.1)	Other Buildings Any Load Pattern
1	1.0	1.0	1.0
2	1.2	1.15	1.2
3	1.2	1.2	1.3
5	1.3	1.2	1.4
10+	1.3	1.2	1.5

Table 3.1 Values for Modification Factor C_0

C_2 = Modification factor to represent the effect of pinched hysteresis shape, cyclic stiffness degradation, and strength deterioration on the maximum displacement response. For periods greater than 0.7 s, C_2= 1.0.

$$C_2 = 1+ \frac{1}{800}\left(\frac{\mu_{strength}-1}{T_e}\right)^2$$

3.10 The analysis of ETABS

1. Modelling

2. Design

3. Pushover analysis

3.11Steps for Pushover Analysis in ETABS

1. ETABS contains basic ACI318 material ratios ATC40 and FEMA273 hinge properties. It also has the ability to enter any material or hinge properties. ETABS only deals with buildings with binding moments M2 and M3torsion T axial forces p and V2 and V3.The axial load of the column changes under the lateral load, which can define the force displacement relationship. The combined PM2M3 (PMM) hinge also yields under the interaction of axial forces and bending moments. It can be assigned to the same position on the hinge frame element. The next step is to

perform a 3D frame-building pushover analysis that creates a base model without push-over data in the usual way.

2. Definition of Pushover Hinge Properties and Tolerance Criteria The program includes some basic hinge properties based on the average ATC 40 for concrete parts and the average FEMA 273 for steel parts. I will. While these basic attributes can be useful for preliminary analysis, the characteristics are recommended for final user-defined analysis.

3. Find pushover hinges in the model by choosing at least one frame individuals and assigning them at least one hinge properties and positions.

4. Push-over analysis Load Definition Internal tab multiple push-overloads can be analyzed in the same way. Additionally, push-overloading conditions that were run prior to analysis, such as starting from a file, are typically gravity loading pushovers. Force control and transverse pushover displacement are controlled.

5. Run the basic static investigation and if desired dynamic analysis then run the static nonlinear pushover analysis.

6. Display the pushover curve.

7. Review a pushover displaced shape and arrangement of hinge development on a step-by-step basis.

CHAPTER 4: MODELLING

4.1 GENERAL

The study in this thesis is predicated on nonlinear analysis of two RCC frame (rectangular and square plans) with diagonal bracing, chevron bracing and without bracing. This chapter presents a summary of various parameters defining the computational models, the fundamental assumptions and also the RCC frames geometry considered for this study. Accurate modeling of the nonlinear properties of varied structural elements is extremely important in nonlinear analysis. Within the present study column, beam and bracings are modeled with inelastic flexural deformations using nonlinear Hinges.

4.2 COMPUTATIONAL MODAL

Modelling a structure invoice, the modelling and assemblage of its different load conveying a component the model must ideal address the imprints dissemination strength stiffness and deformability modelling of the material properties and structural elements employed in this study is discussed below.

4.3 DESIGN DATA

4.3.1 Material Properties:

Grade M25 concrete and grade Fe415steel are used for all columns and beams in buildings, but M25 is also used for steel columns of the same grade. The elastic material properties of these materials are obtained according to IS 456:2000. The short-term modulus of elasticity (Ec) of concrete is retrieved as:

$$E_C = \sqrt[5000]{fck}$$

Where fck=characteristic compressive strength of concrete cube

For the Steel rebar with stress and modulus of elasticity is taken according to IS 456-2000.

Material Properties

Sl.no	Material Properties	
1	Grade of concrete	M25
2	Grade of reinforcing steel	Fe 415
3	Density of concrete	25 kN/m3

Table 4.1 Material Properties

4.4 Model Description

Sl.no	Model Description		
1	Type of building	Square	Rectangular
2	No. of stories	G+12	G+12
3	Height of each Storey	3m	3m
4	Plan of building	36mx36m	54mx24m
5	Column size	600mmx600mm	600mmx600mm
6	Bracing	ISA 130x130x8	ISA 130x130x8
7	Beam size	300mmx650mm	300mmx650mm
8	Thickness of slab	150mm	150mm
9	Dead load	4.75KN/m2	4.75KN/m2
10	Live load	4KN/m2	4KN/m2
11	External wall thickness	230mm	230mm
12	Internal wall thickness	115mm	115mm
13	Seismic zone	V	V
14	Zone factor	0.36	0.36
15	Importance factor	1.5	1.5
16	Distance in X-Direction (Length)	36m	54m
17	Distance in Y-Direction (Width)	36m	24m
18	Distance in Z-Direction (Height)	36m	36m
19	Soil Type	II	II
20	outer Wall Load	12.5 kN/m	12.5 kN/m
21	inner Wall Load	6.25 kN/m	6.25 kN/m
23	Software used	ETAB 2016	ETAB 2016

Table 4.2 Model Description

Storey Data:-

Name	Height mm	Elevation mm	Master story	Similar To	Splice Story
Storey 12	3000	36000	Yes	None	No
Storey 11	3000	33000	No	Story 12	No
Storey 10	3000	30000	No	Story 12	No
Storey 9	3000	27000	No	Story 12	No
Storey 8	3000	24000	No	Story 12	No
Storey 7	3000	21000	No	Story 12	No
Storey 6	3000	18000	No	Story 12	No
Storey 5	3000	15000	No	Story 12	No
Storey 4	3000	12000	No	Story 12	No
Storey 3	3000	9000	No	Story 12	No
Storey 2	3000	6000	No	Story 12	No
Storey 1	3000	3000	No	Story 12	No
Base	0	0	No	None	No

Table 4.3 Storey Data

4.5 Loads

When applying loads to a structure, ETABS 2016 automatically takes the weight of the member, so it ignores the weight and considers only the external load that actually acts on the member.

Load Cases:-

Name	**Type**
Dead	Linear Static
Live	Linear Static
EQ-x	Linear Static
EQ-y	Linear Static
Wind-x	Linear Static
Wind-y	Linear Static
RS-x	Response Spectrum
RS-y	Response Spectrum
Thx	Nonlinear Modal History (FNA)
Thy	Nonlinear Modal History (FNA)
Push-x	Nonlinear Static
Push-y	Nonlinear Static

Table 4.4 Load Cases – Summary

4.6 Square buildings with square columns (SBSC) for Unbraced, Diagonal bracing and Chevron bracing

This is square shape building with 6 rows of Slab panels in both X; Y directions and 49 square shaped Columns with 6m spanning connected beams as shown in fig. It's elevated for twelve floors with height of 3m between adjacent floors as shown in figure.

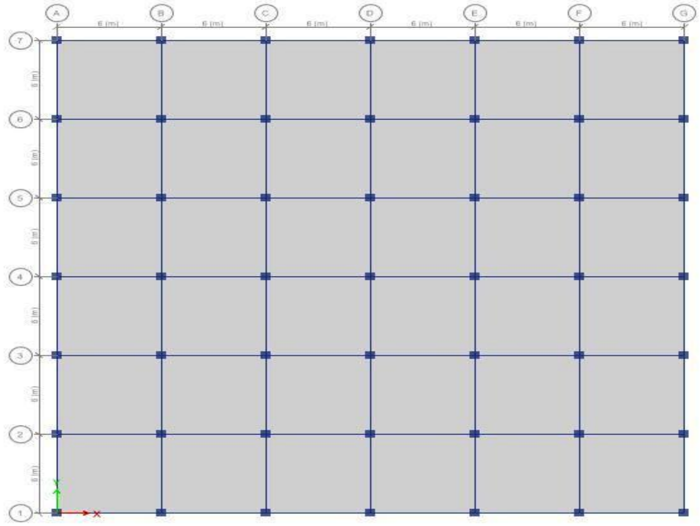

Fig 4.1: Unbraced Plan

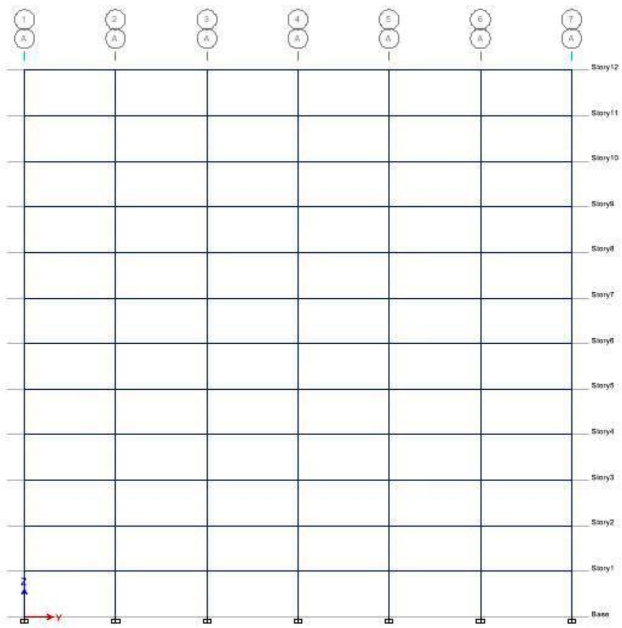

Fig 4.2: Unbraced Elevation

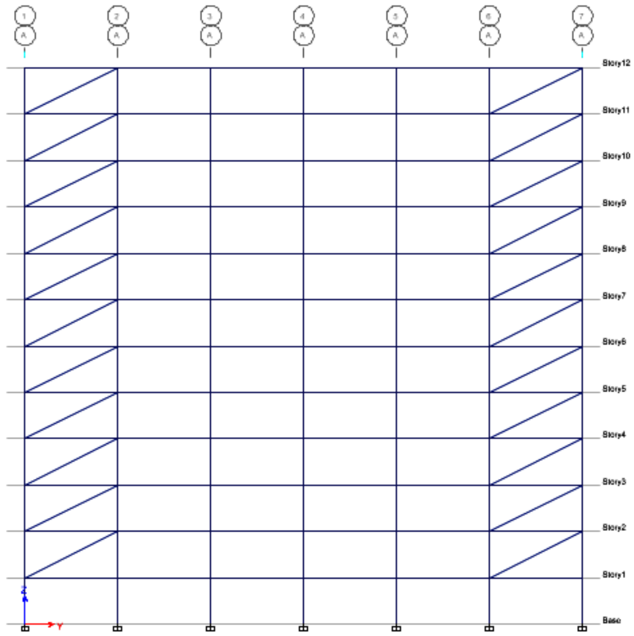

Fig 4.3: Diagonal Bracing Elevation

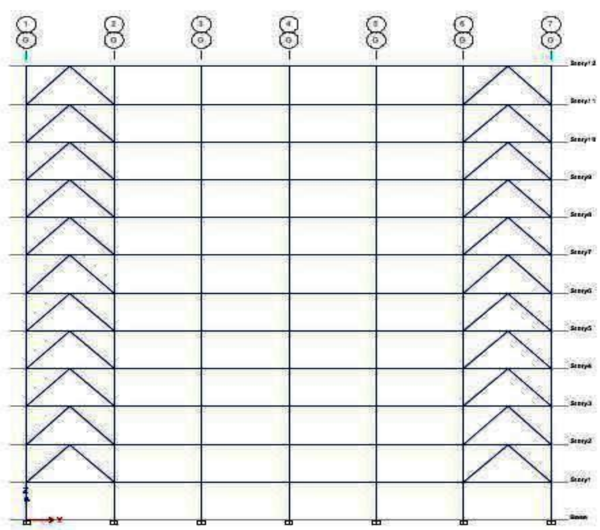

Fig 4.4: Chevron Bracing Elevation

4.7 Rectangular building with square columns (RBSC) for Unbraced, Diagonal bracing and Chevron bracing

This is rectangular shape building with 9 rows of Slab panels in X and 4 in Y directions; and 50 square shaped Columns with 6m spanning connected beams as shown in figure. It's elevated for twelve floors with height of 3m between adjacent floors as shown in figure.

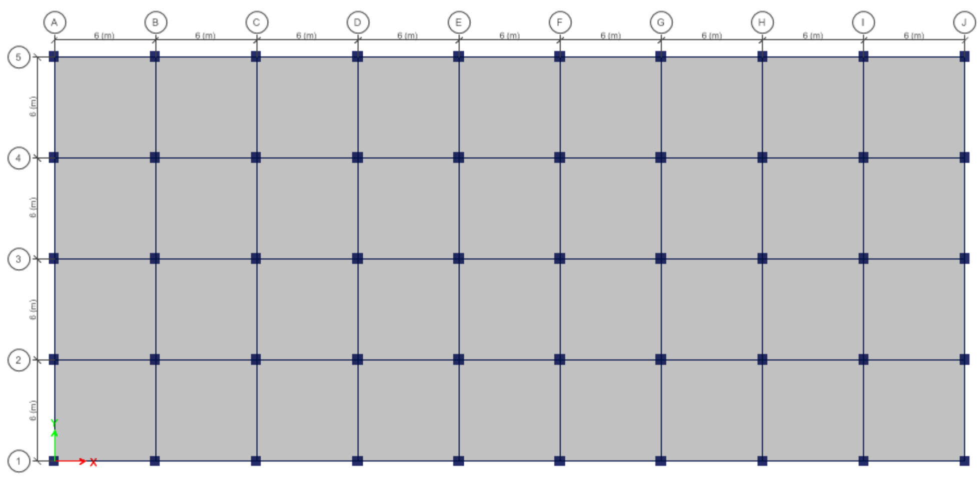

Fig 4.5: Unbraced Plan

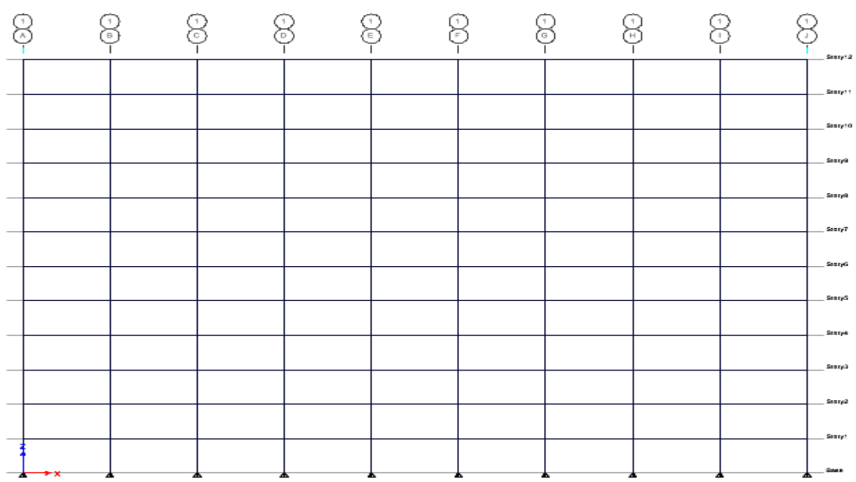

Fig 4.6: Unbraced Elevation

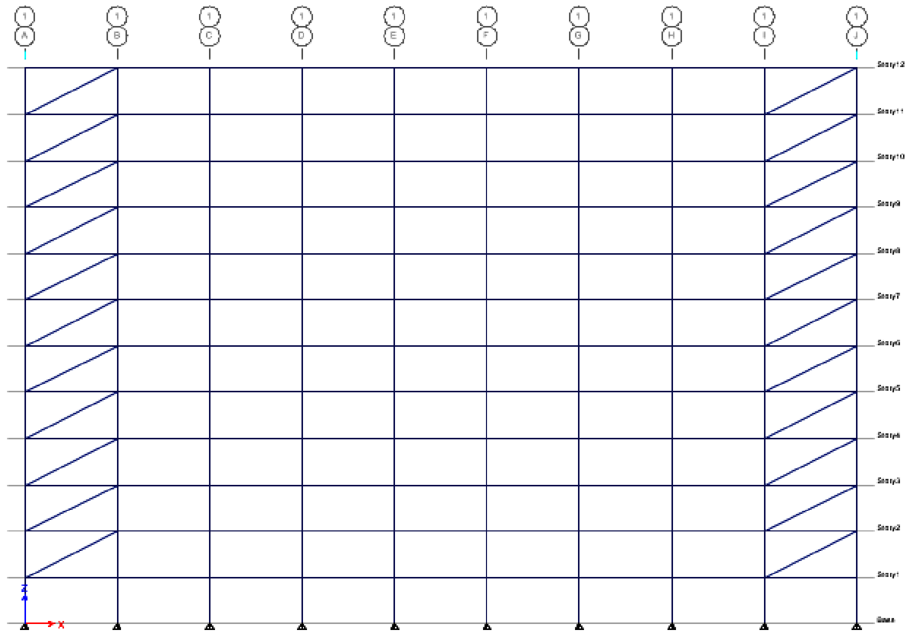

Fig 4.7: Diagonal bracing Elevation

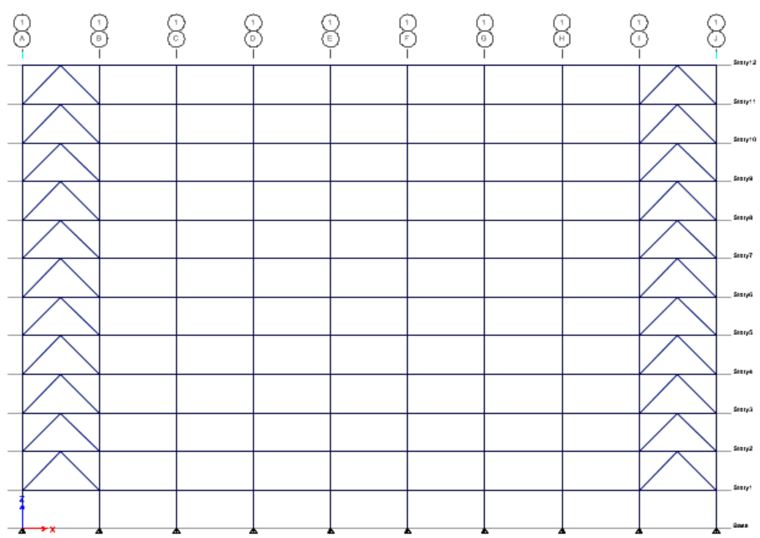

Fig 4.8: Chevron bracing Elevation

4.8 ASSIGNMENT OF HINGES FOR PUSHOVER ANALYSIS

For nonlinear static and nonlinear direct integral time history analysis, the user can assign focused plastic hinges to frame and tendon objects to simulate their post-yield behavior. Elastic behavior occurs over the entire length of the member and deformations beyond the elastic limit occur completely at the hinge. The hinge is modeled in a separate place.

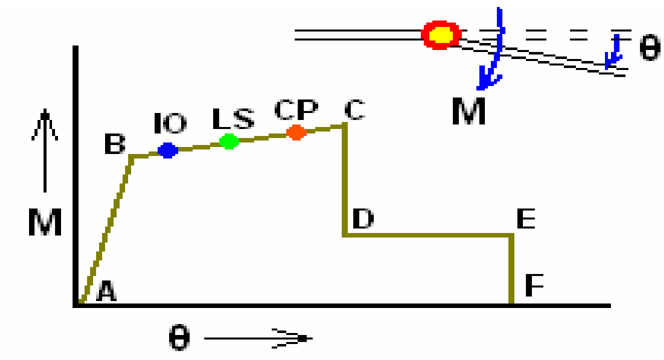

Fig.4.9: A Typical Flexural Hinge Property, showing IO (Immediate Occupancy), LS (Life Safety) and CP (Collapse Prevention)

Inelastic behavior is achieved through the integration of plastic curvature and plastic deformation that occurs within a user-defined hinge length (typically ordered by depth of part (FEMA356)).A series of hinges can be modeled to catch plasticity distributed along the length of the member. Sometimes multiple hinges match in the same place.

Plasticity can be recognized to force-displacement behavior (axial and shear) or moment rotation (torsion and bending). Hinges can be assigned (not combined) to one of the six DOFs. The behavior after surrender is explained by the general backbone relationship shown below. Strength loss modeling is not recommended to reduce redistribution of loads (possibly leading to decay of progression) and to ensure numerical convergence.

You can use the overwrite option, but the CSI software automatically limits the tilt of the sound to 10% of its elastic stiffness. You can specify additional restricted states (IO, LS, CP) that are reported by the analysis for informational purposes and do not affect the results. At the point of plastic deformation, the title follows the slope of the initial stiffness.

Both PM2M3 hinges and fiber hinges can be used to capture axial and biaxial bending motion. PM2M3 hinges are sensible for non-linear static pushovers, while fiber hinges are reasonable for hysteresis the dynamics.

4.8.1 Frame Non-linear Hinge

Hinge properties are used to define nonlinear force-displacement or moment-rotational behavior that can be assigned to discrete locations or mid-heights of wall objects along the length of the frame (line) object. These nonlinear hinges are used during static nonlinear analysis, fast nonlinear analysis (FNA) modal time history analysis, and nonlinear direct integral time history analysis. All other types of analysis show that the hinge stiffness does not affect the operation of the member. The number of hinges affects not only the computation time, but also the behavior of the model and the ease of interpretation of the results. Therefore, it is recommended to assign hinges only to places where nonlinear behavior is likely to occur.

Note: it is important to design the objects of the frame and walls before performing a non-linear analysis using hinges, the reinforcement of the concrete frame and walls must be defined. ETABS can use the following3types of hinge properties.

4.8.2 Auto Hinge Properties.

The auto-hinge attribute is programmatically defined. The program cannot fully define the automatic properties until the applicable section is identified. Therefore, the automatic properties are assigned to the frame or wall object and you can see the properties of the resulting hinge.

4.8.3 User-Defined Hinge Properties.

User-defined hinge properties can be founded on auto properties or they can be completely user defined.

4.8.4 Program Generated Hinge Properties.

The generated hinge properties are employed in the analysis. They'll be viewed, but they can't be modified. Generated hinge properties have an programmed naming show of Label H#, where

Label is that the frame or wall object label, H represents for hinge, and # addresses the hinge number. The program starts with hinge no 1 and increments the hinge number by one for every consecutive hinge applied to the frame or wall object as an example, if a frame object label is the C4, the produced the hinge property name for the subsequent hinge applied to the frame object is the C4H2.

The main reason for distinguishing between defined attributes (defined during this context means both automatic and custom) and generated attributes is that hinge attributes generally depend upon this section. Therefore, you wish to define a special set of hinge attributes for every frame or wall section style of your model. This may potentially mean that a awfully sizable amount of hinge attributes must be defined. ETABS uses the concept of attributes generated to simplify this process. When the generated attributes are used, the program combines the built-in criteria with the section properties defined for every object to get the ultimate hinge attribute. The web effect of this can be that defining hinge properties are going to be much less work as you do not should define every hinge.The user appoints auto hinge properties and user-defined hinge properties to a frame or wall object. The program then automatically creates a replacement generated hinge property for each assigned hinge.

4.13 CLOSURE

The displaying, load and conditions are applied to the structure before investigation. Providing the planning data to the modeling of structural elements, bracing and hinges details. The subsequent chapter deals with the analysis of those structures generated here. The results obtained are shown in tabulated forms.

CHAPTER 5: RESULTS & DISCUSSION

5.1 GENERAL

An analysis performed on both RC frame structures (square and rectangular) with three types of bracing (unbraced, diagonal and chevron) is shown. The results obtained from the analysis are considered according to the purpose of the study. After getting the results, compare them and draw conclusions from them. In Chapter 4, a pushover analysis is run and the effect of these modals results is shown.

5.2 PUSHOVER ANALYSIS:

The procedure for pushover analysis is given below:

5.2.1 Create the computational model

- Create the computational model, without pushover data, using conventional modelling techniques.
- Define properties for pushover hinges using Define > Section Properties > Hinge Properties. Hinges could also be defined manually or by using one amongst several default specifications which are available.
- Assign the pushover hinges to selected frame objects using Assign > Frame > Hinges.
- Select Define > Load Patterns to define load patterns which is able to contain the loads applied during pushover analysis.

5.2.2 Define a nonlinear static load case

- Select Define > Load Cases > Add New Load Case to define a nonlinear static load case which is able to apply the previously-defined load pattern. This load case could also be force-controlled (pushed to a specified force level) or the displacement-controlled (pushed to a specified the displacement).
- Select Other Parameters > Results Saved to Multiple States specified various parameters could also be plotted for every increment of applied loading.

5.2.3 Run the analysis

Select Analyse > Run Analysis to run the static-pushover analysis.

5.2.4 Review results

i) To plot base shear vs. monitored the displacement, selects Display > Show Static Pushover Curve. Additional variables are also available for plotting.

ii) To plot hinge deformation vs. applied loading, select Display > Show Hinge Results. Moment as a component of plastic rotation is one such choice.

iii) To review displacement and the step-by-step sequence of hinge arrangement, select Display > Show Deformed Shape.

iv) To review member forces on a step-by-step basis, select Display > Show Forces/Stresses > Frames/Cables.

v) Select Display > Show Plot Functions to plot reaction at each progression of the pushover analysis, including joint displacement, frame member forces, etc.

5.3 RESULTS

5.4 Modal Participating Mass Ratios

In the sense of interpretation: The Modal Participating Mass Ratio (MPMR) is oscillating the mass participation; and Effective Modal Mass is an effective amount of a type of the oscillation.

Oscillating mass participation –The Participating Modal Mass Ratio (MPMR)

According to documents Reference Analysis of CSI, **Modal Participating Mass Ratio** is defined as follows:

$$\mathrm{MPMR} = \frac{\left(\sum_{j=1}^{n} m_{j} * \emptyset_{j,i}\right)^{2}}{\left(\sum_{j=1}^{n} m_{j}\right)}$$

Effective volume - Effective Modal Mass (EMM)

Effective mass of the vibration pattern is determined by the formula:

$$M_{td,i} = \frac{\left(\sum_{j=1}^{n} m_j * \emptyset_{j,i}\right)^2}{\sum_{j=1}^{n} m_j * \emptyset_{j,i}^2}$$

Expressed as a percentage of the total volume of works as follows:

$$\%\left(M_{td,i}\right) = \frac{\left(\sum_{j=1}^{n} m_j * \emptyset_{j,i}\right)^2}{\left(\sum_{j=1}^{n} m_j * \emptyset_{j,i}^2\right) * \left(\sum_{j=1}^{n} m_j\right)}$$

Compare

Oscillating mass participation (MPMR) was wont to assess the amount of importance of a fluctuating form and therefore the kind of such oscillations.

Effective volume (EMM) or **Percentage effective volume** % (EMM) is employed to calculate the masses exerted on the seismic structural system multiple degrees of freedom (MDOF) on the idea of calculation of load effects major earthquake to have a degree of freedom system (SDOF) have the identical mass and oscillation cycle.

Mathematically, we are able to easily realize that **MPMR** and zippers **(EMM)** is decided different amounts (Σm.Φ2) is termed the mass of fluctuating form.

In special cases, the flat problem, so (Σm.Φ2) = 1; Meanwhile MPMR =% (EMM)

In the space problem, the volume of oscillator types including volume fluctuation in X, Y and Z axis torsion oscillator; the sum of the means = 1; and also the value of every is a smaller amount than 1; so, the matter is in each respective space have MPMR <% (EMM).

5.4.1 UNBRACED MODAL FOR SQUARE BUILDING

Mode	Period sec	UX	UY	UZ	Sum UX	Sum UY	Sum UZ
1	1.946	0	0.8061	0	0	0.8061	0
2	1.946	0.8061	0	0	0.8061	0.8061	0
3	1.788	0	0	0	0.8061	0.8061	0
4	0.635	0	0.0971	0	0.8061	0.9032	0
5	0.635	0.0971	0	0	0.9032	0.9032	0
6	0.584	0	0	0	0.9032	0.9032	0
7	0.365	0	0.0365	0	0.9032	0.9397	0
8	0.365	0.0365	0	0	0.9397	0.9397	0
9	0.337	0	0	0	0.9397	0.9397	0
10	0.248	0	0.0201	0	0.9397	0.9597	0
11	0.248	0.0201	0	0	0.9597	0.9597	0
12	0.23	0	0	0	0.9597	0.9597	0

Table 5.1SBSC - MPMR values

5.4.2 DIAGONAL BRACING MODAL FOR SQUARE BUILDING

Mode	Period sec	UX	UY	UZ	Sum UX	Sum UY	Sum UZ
1	2.906	0.4019	0.4019	0	0.4019	0.4019	0
2	2.903	0.4018	0.4017	0	0.8036	0.8036	0
3	2.012	0.0001	0.0001	0	0.8038	0.8038	0
4	0.938	0.062	0.062	0	0.8658	0.8658	0
5	0.937	0.0621	0.0621	0	0.9278	0.9278	0
6	0.665	0.07425	0.07425	0	0.9278	0.9278	0
7	0.516	0.0169	0.0169	0	0.9448	0.9448	0
8	0.516	0.0169	0.0169	0	0.9617	0.9617	0
9	0.368	0.0211	0.0211	0	0.9618	0.9618	0
10	0.356	0.0077	0.0077	0	0.9695	0.9695	0
11	0.356	0.0077	0.0077	0	0.9771	0.9771	0
12	0.268	0.0042	0.0042	0	0.9813	0.9813	0

Table 5.2 SBSC - MPMR values

5.4.3 CHEVRON BRACING MODAL FOR SQUARE BUILDING

Mode	Period sec	UX	UY	UZ	Sum UX	Sum UY	Sum UZ
1	2.529	0.1682	0.6298	0	0.1682	0.6298	0
2	2.529	0.6298	0.1682	0	0.798	0.798	0
3	1.691	0	0	0	0.798	0.798	0
4	0.821	0.0728	0.0721	0	0.8708	0.8701	0
5	0.821	0.0721	0.0728	0	0.9429	0.9429	0
6	0.57	0	0	0	0.9429	0.9429	0
7	0.434	0.017	0.0147	0	0.96	0.9577	0
8	0.434	0.0147	0.017	0	0.9747	0.9747	0
9	0.296	0.0071	0.0045	0	0.9818	0.9793	0
10	0.296	0.0045	0.0071	0	0.9864	0.9864	0
11	0.296	0	0	0	0.9864	0.9864	0
12	0.224	0.0036	0.002	0	0.99	0.9884	0

Table 5.3 SBSC - MPMR values

5.4.4 UNBRACED MODAL FOR RECTANGULAR BUILDING

Mode	Period sec	UX	UY	UZ	Sum UX	Sum UY	Sum UZ
1	2.278	0	0.8803	0	0	0.8803	0
2	2.157	0.885	0	0	0.885	0.8803	0
3	2.1	0	0	0	0.885	0.8803	0
4	0.735	0	0.0814	0	0.885	0.9617	0
5	0.697	0.0783	0	0	0.9633	0.9617	0
6	0.677	0	0	0	0.9633	0.9617	0
7	0.412	0	0.0207	0	0.9633	0.9824	0
8	0.394	0.0201	0	0	0.9834	0.9824	0
9	0.382	0	0	0	0.9834	0.9824	0
10	0.275	0	0.0082	0	0.9834	0.9906	0
11	0.265	0.0078	0	0	0.9912	0.9906	0
12	0.255	0	0	0	0.9912	0.9906	0

Table 5.4 RBSC - MPMR values

5.4.5 DIAGONAL BRACING MODAL FOR RECTANGULAR BUILDING

Mode	Period	UX	UY	UZ	Sum UX	Sum UY	Sum UZ
	sec						
1	3.501	0.0004	0.9067	0	0.0004	0.9067	0
2	3.425	0.9080	0.0004	0	0.9084	0.9071	0
3	2.622	0.0002	0.0001	0	0.9086	0.9072	0
4	1.132	0.0268	0.0764	0	0.9086	0.9836	0
5	1.108	0.0752	0.0000	0	0.9839	0.9837	0
6	0.812	0.0698	0.0000	0	0.9839	0.9837	0
7	0.572	0.0022	0.0098	0	0.9839	0.9935	0
8	0.563	0.0098	0.0000	0	0.9937	0.9935	0
9	0.396	0.0000	0.0000	0	0.9937	0.9935	0
10	0.379	0.0000	0.0030	0	0.9937	0.9965	0
11	0.374	0.0029	0.0000	0	0.9966	0.9965	0
12	0.279	0.0000	0.0014	0	0.9966	0.9979	0

Table 5.5 RBSC - MPMR values

5.4.6CHEVRON BRACING MODAL FOR SQUARE BUILDING

Mode	Period	UX	UY	UZ	Sum UX	Sum UY	Sum UZ
	sec						
1	1.923	0	0.888	0	0	0.888	0
2	1.852	0.8924	0	0	0.8924	0.888	0
3	1.522	0	0	0	0.8924	0.888	0
4	0.63	0	0.0881	0	0.8924	0.9761	0
5	0.606	0.0843	0	0	0.9767	0.9761	0
6	0.497	0	0	0	0.9767	0.9761	0
7	0.331	0	0.0146	0	0.9767	0.9906	0
8	0.322	0.0143	0	0	0.991	0.9906	0
9	0.252	0	0	0	0.991	0.9906	0
10	0.22	0	0.0045	0	0.991	0.9952	0
11	0.214	0.0044	0	0	0.9954	0.9952	0
12	0.167	0	0	0	0.9954	0.9952	0

Table 5.6 RBSC - MPMR values

5.5 Modal Periods and Frequencies

One analytical method for calculating the linear response of a structure to dynamic loading is modal analysis. Modal analysis decomposes the response of a structure into several modes of vibration. Modes are defined by frequency and shape. Structural engineers call the mode with the shortest frequency (longest period) the basic mode.

Not all modes are equally exciting between dynamic loads, i.e. seismic, wind, or explosion loads. The extent to which a dynamic load excites a specific vibration mode depends on the spatial distribution and frequency component of the load.

Hence the Eigen values, frequencies and periods for 12 modes of each Modal are shown below.

5.5.1 UNBRACED MODAL FOR SQUARE BUILDING

Mode	Period	Frequency	Circular Frequency	Eigen value
	sec	cyc/sec	rad/sec	rad²/sec²
1	1.946	0.514	3.2286	10.4237
2	1.946	0.514	3.2286	10.4237
3	1.788	0.559	3.5141	12.3487
4	0.635	1.576	9.9009	98.0271
5	0.635	1.576	9.9009	98.0271
6	0.584	1.712	10.7548	115.6655
7	0.365	2.743	17.2319	296.9374
8	0.365	2.743	17.2319	296.9374
9	0.337	2.963	18.6174	346.6059
10	0.248	4.031	25.3246	641.3334
11	0.248	4.031	25.3246	641.3334
12	0.23	4.349	27.3278	746.811

Table 5.7 SBSC modal periods and frequencies

5.5.2 DIAGONAL BRACING MODAL FOR SQUARE BUILDING

Mode	Period	Frequency	Circular Frequency	Eigen value
	sec	cyc/sec	rad/sec	rad^2/sec^2
1	2.906	0.344	2.1625	4.6764
2	2.903	0.344	2.1645	4.685
3	2.012	0.497	3.1222	9.7483
4	0.938	1.066	6.6988	44.8739
5	0.937	1.067	6.7059	44.9697
6	0.665	1.504	9.4473	89.2521
7	0.516	1.937	12.1684	148.0693
8	0.516	1.939	12.1849	148.4719
9	0.368	2.719	17.0819	291.7897
10	0.356	2.805	17.6249	310.6385
11	0.356	2.809	17.6499	311.5186
12	0.268	3.729	23.4297	548.9501

Table 5.8 SBSC modal periods and frequencies

5.5.3 CHEVRON BRACING MODAL FOR SQUARE BUILDING

Mode	Period	Frequency	Circular Frequency	Eigen value
	sec	cyc/sec	rad/sec	rad^2/sec^2
1	2.529	0.395	2.4841	6.1708
2	2.529	0.395	2.4841	6.1708
3	1.691	0.591	3.7149	13.8006
4	0.821	1.218	7.6541	58.5852
5	0.821	1.218	7.6541	58.5852
6	0.57	1.754	11.0217	121.4775
7	0.434	2.306	14.4869	209.8696
8	0.434	2.306	14.4869	209.8696
9	0.296	3.374	21.197	449.3107
10	0.296	3.374	21.197	449.3107
11	0.296	3.38	21.2368	451.0019
12	0.224	4.473	28.107	790.0028

Table 5.9 SBSC modal periods and frequencies

5.5.4 UNBRACED MODAL FOR RECTANGULAR BUILDING

Mode	Period	Frequency	Circular Frequency	Eigen value
	sec	cyc/sec	rad/sec	rad²/sec²
1	2.278	0.439	2.7579	7.6062
2	2.157	0.463	2.9123	8.4812
3	2.1	0.476	2.9917	8.9502
4	0.735	1.36	8.5481	73.0696
5	0.697	1.435	9.0191	81.3439
6	0.677	1.477	9.2779	86.0804
7	0.412	2.429	15.263	232.9598
8	0.394	2.536	15.9331	253.8634
9	0.382	2.618	16.4489	270.5658
10	0.275	3.64	22.8718	523.119
11	0.265	3.779	23.7442	563.7863
12	0.255	3.915	24.5963	604.9798

Table 5.10 RBSC modal periods and frequencies

5.5.5 DIAGONAL BRACING MODAL FOR RECTANGULAR BUILDING

Mode	Period	Frequency	Circular Frequency	Eigen value
	sec	cyc/sec	rad/sec	rad²/sec²
1	3.501	0.286	1.7946	3.2205
2	3.425	0.292	1.8343	3.3648
3	2.622	0.381	2.3965	5.7432
4	1.132	0.883	5.5484	30.7851
5	1.108	0.903	5.6712	32.162
6	0.812	1.231	7.7332	59.803
7	0.572	1.748	10.9802	120.5656
8	0.563	1.776	11.1573	124.4851
9	0.396	2.523	15.8541	251.3513
10	0.379	2.639	16.5787	274.8527
11	0.374	2.676	16.8151	282.7466
12	0.279	3.583	22.512	506.7923

Table 5.11 RBSC modal periods and frequencies

5.5.6 CHEVRON BRACING MODAL FOR RECTANGULAR BUILDING

Mode	Period	Frequency	Circular Frequency	Eigen value
	sec	cyc/sec	rad/sec	rad²/sec²
1	1.923	0.52	3.2675	10.6766
2	1.852	0.54	3.3932	11.5141
3	1.522	0.657	4.1292	17.0502
4	0.63	1.587	9.9717	99.4347
5	0.606	1.649	10.3623	107.3781
6	0.497	2.013	12.6475	159.9601
7	0.331	3.018	18.9597	359.4697
8	0.322	3.101	19.4857	379.693
9	0.252	3.975	24.9757	623.7859
10	0.22	4.555	28.6178	818.9764
11	0.214	4.664	29.3044	858.7497
12	0.167	5.996	37.6757	1419.4555

Table 5.12 RBSC modal periods and frequencies

5.6 Storey Maximum and Average Lateral Displacements

ETABS provides a simple table within the summary output with "Story Maximum and Average Lateral Displacements". This provides indication of maximum to average ratio to test torsion irregularity. The most Displacements because of Push-X in X-direction are:

5.6.1 For Square building (X-direction)

Storey	ASCE 41-13			NTC-2008			EC-8		
	Unbraced	Diagonal	Chevron	Unbraced	Diagonal	Chevron	Unbraced	Diagonal	Chevron
Storey 12	346	289	274	345	208	203	341	314	291
Storey 11	344	284	272	342	205	202	338	312	288
Storey 10	341	283	266	341	204	200	323	309	285
Storey 9	336	274	261	338	202	196	316	305	280
Storey 8	330	255	254	336	201	192	314	294	274
Storey 7	317	236	244	334	196	186	308	284	267
Storey 6	294	227	214	320	184	179	303	277	257
Storey 5	257	212	168	300	162	159	291	238	231
Storey 4	211	190	111	277	130	123	271	190	181
Storey 3	154	143	75	254	93	85	237	133	122
Storey 2	95	83	68	229	53	43	193	74	67
Storey 1	54	41	37	205	14	11	119	24	22
Base	0	0	0	0	0	0	0	0	0

Table 5.13 Max. Disp. of Modals at different stories due to Push X

5.6.2 For Rectangular building (X-direction)

Storey	ASCE 41-13			NTC-2008			EC-8		
	Unbraced	Diagonal	Chevron	Unbraced	Diagonal	Chevron	Unbraced	Diagonal	Chevron
Storey12	384	357	334	265	199	146	263	252	244
Storey11	382	351	331	262	196	145	260	248	241
Storey10	378	348	323	258	194	142	255	247	238
Storey 9	372	339	316	255	192	138	247	241	233
Storey 8	365	320	308	249	187	134	236	226	221
Storey 7	348	301	298	240	182	128	222	204	203
Storey 6	320	292	268	226	170	121	181	173	168
Storey 5	283	276	221	206	148	106	149	134	126
Storey 4	237	252	164	183	116	88	128	108	104
Storey 3	180	198	125	160	79	63	94	86	73
Storey 2	119	138	115	134	39	38	63	58	44
Storey 1	78	96	84	110	28	13	53	34	19
Base	0	0	0	0	0	0	0	0	0

Table 5.14 Max. Disp. of Modals at different stories due to Push X

ETABS provides a simple table within the summary output with "Story Maximum and Average Lateral Displacements". This provides indication of maximum to average ratio to test torsion irregularity. The most Displacements because of Push-Y in Y-direction are:

5.6.3 For Square building (Y-direction)

Storey	ASCE 41-13			NTC-2008			EC-8		
	Unbraced	Diagonal	Chevron	Unbraced	Diagonal	Chevron	Unbraced	Diagonal	Chevron
Storey 12	346	289	274	345	208	203	341	314	291
Storey 11	344	284	272	342	205	202	338	312	288
Storey 10	341	283	266	341	204	200	323	309	285
Storey 9	336	274	261	338	202	196	316	305	280
Storey 8	330	255	254	336	201	192	314	294	274
Storey 7	317	236	244	334	196	186	308	284	267
Storey 6	294	227	214	320	184	179	303	277	257
Storey 5	257	212	168	300	162	159	291	238	231
Storey 4	211	190	111	277	130	123	271	190	181
Storey 3	154	143	75	254	93	85	237	133	122
Storey 2	95	83	68	229	53	43	193	74	67
Storey 1	54	41	37	205	14	11	119	24	22
Base	0	0	0	0	0	0	0	0	0

Table 5.15 Max. Disp. of Modals at different stories due to Push Y

5.6.4 For Rectangular building (Y-direction)

Storey	ASCE 41-13			NTC-2008			EC-8		
	Unbraced	Diagonal	Chevron	Unbraced	Diagonal	Chevron	Unbraced	Diagonal	Chevron
Storey12	409	382	362	293	252	156	292	283	272
Storey11	394	362	342	282	246	153	281	272	263
Storey10	383	358	338	264	238	152	265	261	247
Storey 9	378	342	326	262	212	149	253	254	242
Storey 8	371	331	312	255	208	142	242	231	236
Storey 7	356	312	302	242	193	136	231	218	223
Storey 6	338	298	279	238	177	127	197	187	175
Storey 5	293	284	237	211	158	112	156	144	134
Storey 4	242	262	175	186	126	96	134	118	112
Storey 3	196	207	136	172	84	73	105	92	86
Storey 2	128	147	122	144	42	44	76	63	57
Storey 1	84	98	89	113	36	26	68	38	28
Base	0	0	0	0	0	0	0	0	0

Table 5.16 Max. Disp. of Modals at different stories due to Push Y

5.7 Capacity curve SBSC for Unbraced

5.7.1 For (ASCE 41-13 code)

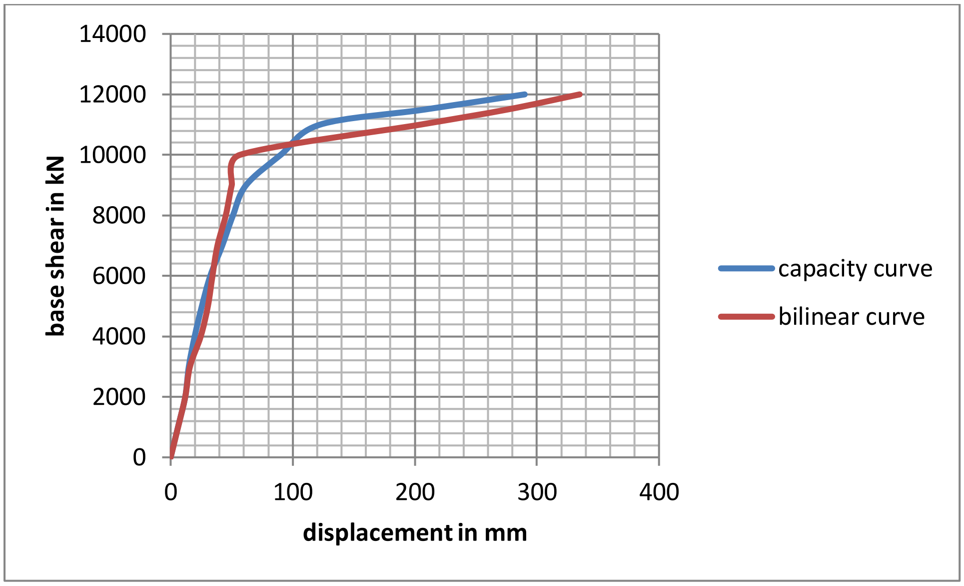

Fig 5.1: Capacity curve (unbraced)

5.7.2 For (NTC-2008 code)

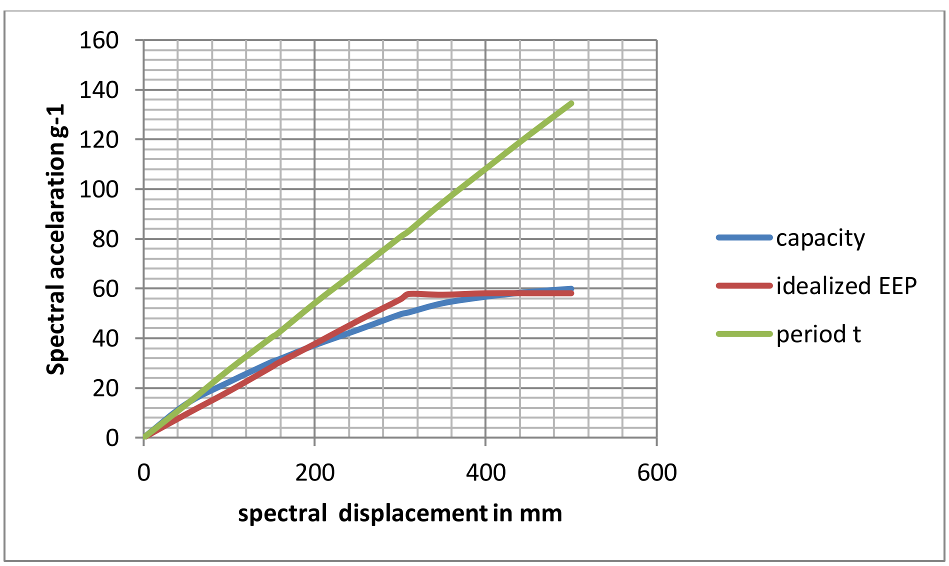

Fig 5.2: Capacity curve (unbraced)

5.7.3 For (EC-8 code)

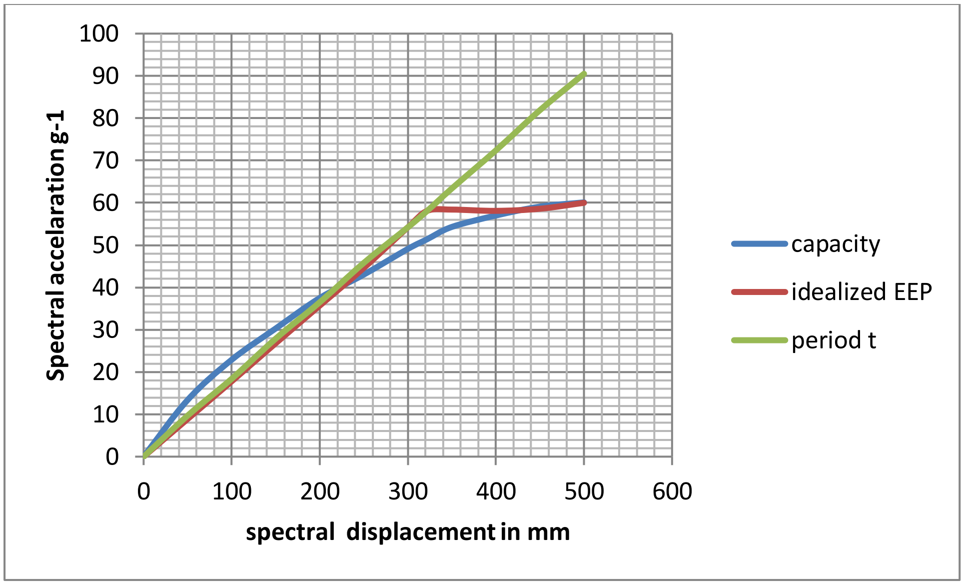

Fig 5.3: Capacity curve (unbraced)

5.8 Capacity curve SBSC for Diagonal bracing

5.8.1 For (ASCE 41-13 code)

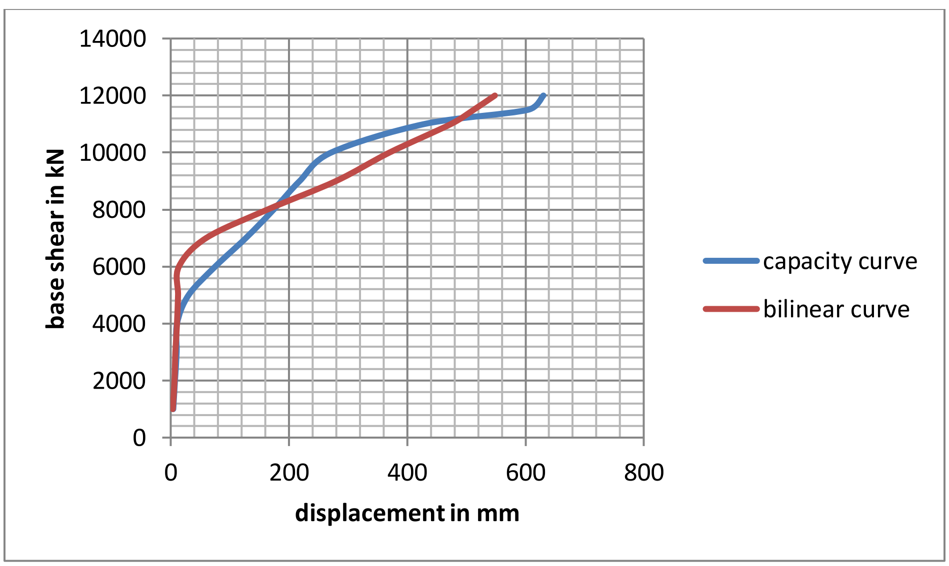

Fig 5.4: Capacity curve (Diagonal bracing)

5.8.2 For (NTC-2008 code)

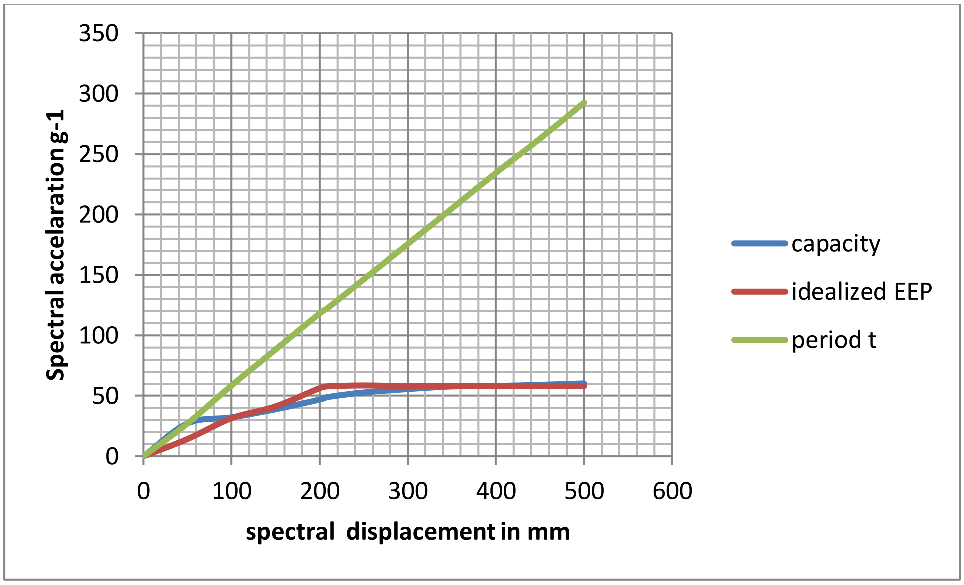

Fig 5.5: Capacity curve (Diagonal bracing)

5.8.3 For (EC-8 code)

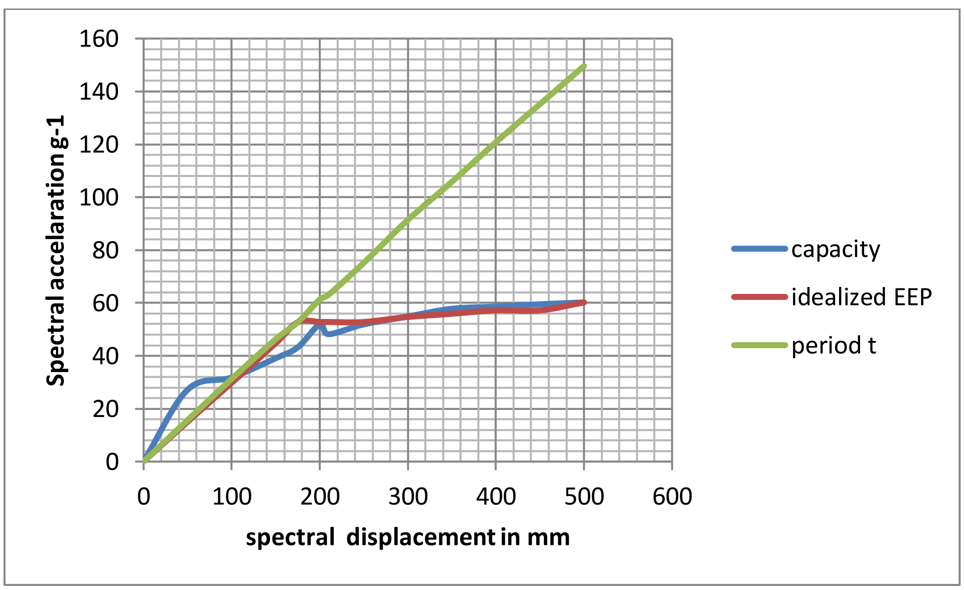

Fig 5.6: Capacity curve (Diagonal bracing)

5.9 Capacity curve SBSC for Chevron bracing

5.9.1 For (ASCE 41-13 code)

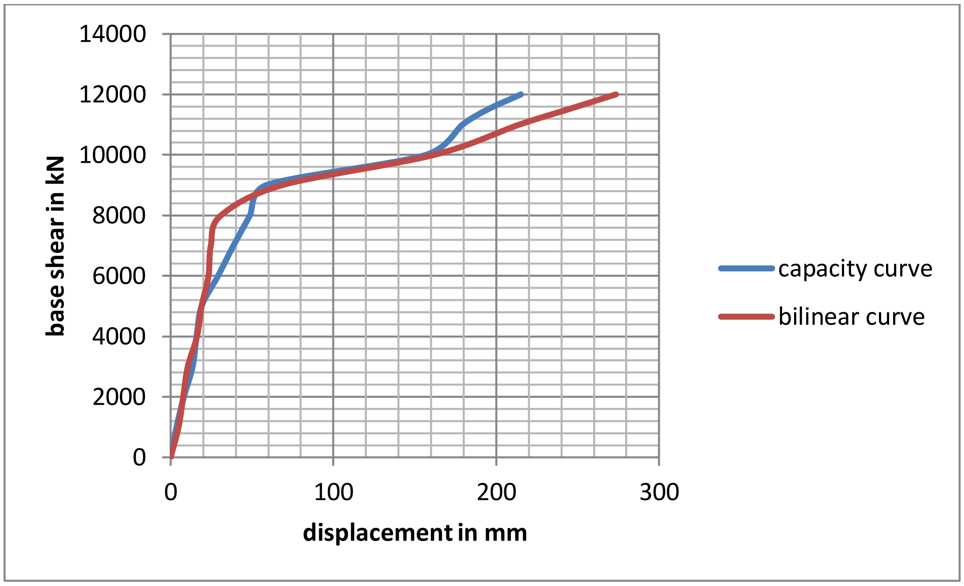

Fig 5.7: Capacity curve (Chevron bracing)

5.9.2 For (NTC-2008 code)

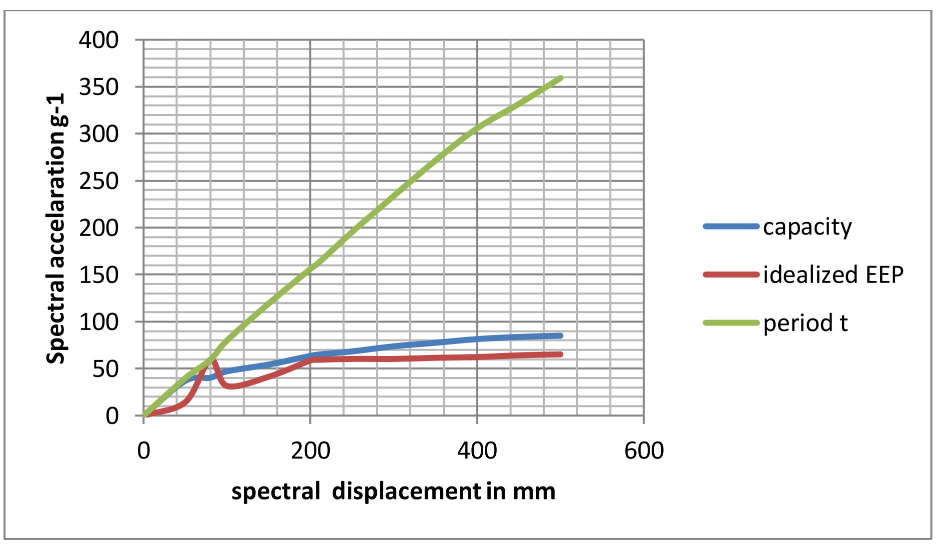

Fig 5.8: Capacity curve (Chevron bracing)

5.9.3 For (EC-8 code)

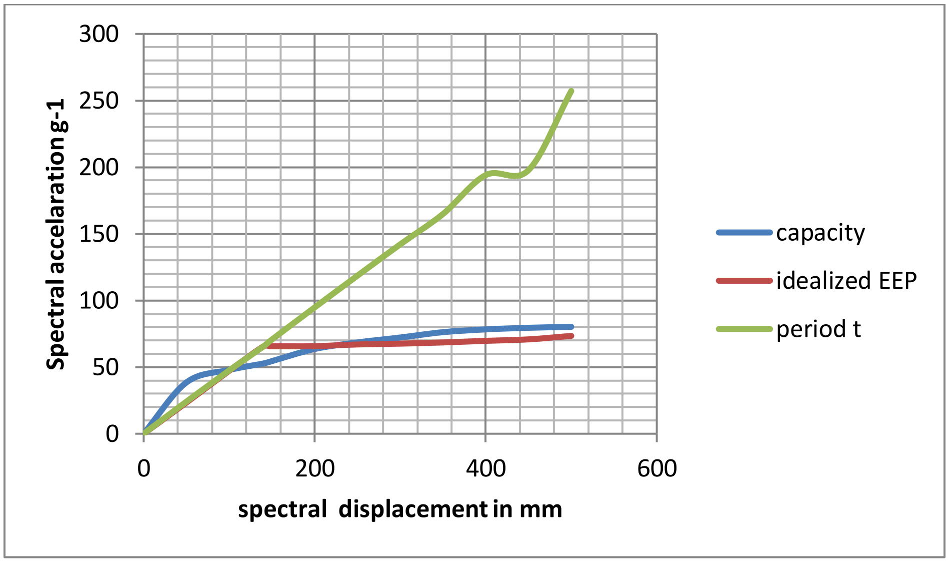

Fig 5.9: Capacity curve (Chevron bracing)

5.10 Capacity curve RBSC for Unbraced

5.10.1 For (ASCE 41-13 code)

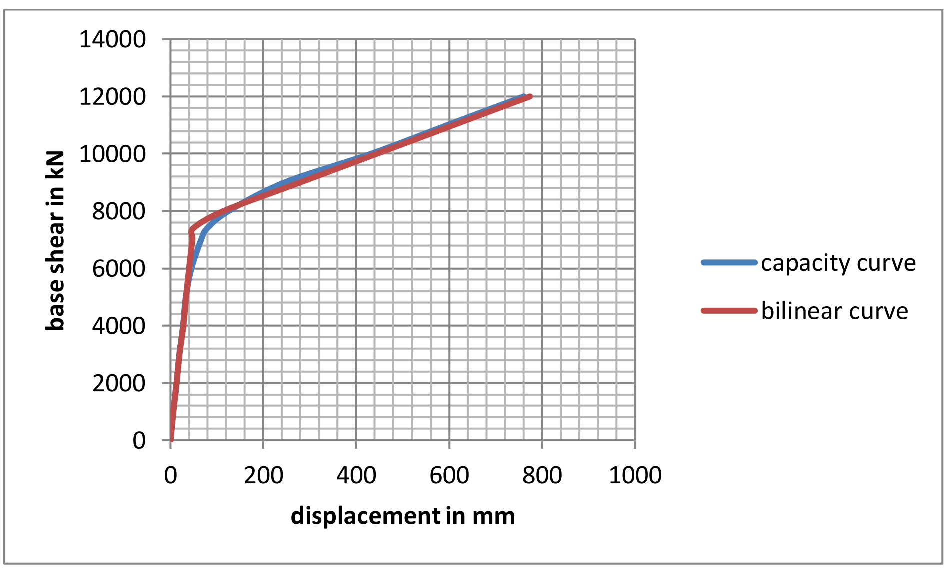

Fig 5.10: Capacity curve (unbraced)

5.10.2 For (NTC-2008 code)

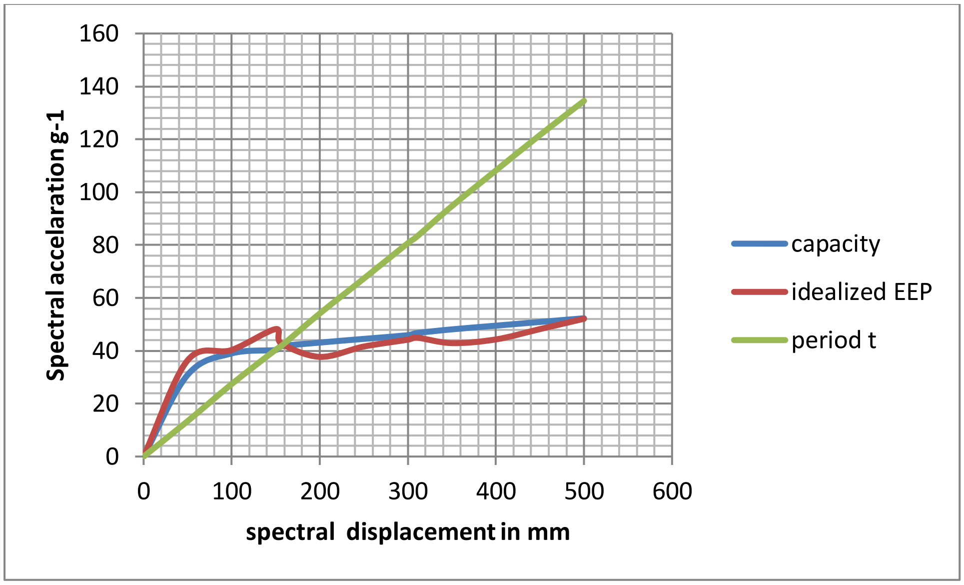

Fig 5.11: Capacity curve (unbraced)

5.10.3 For (EC-8 code)

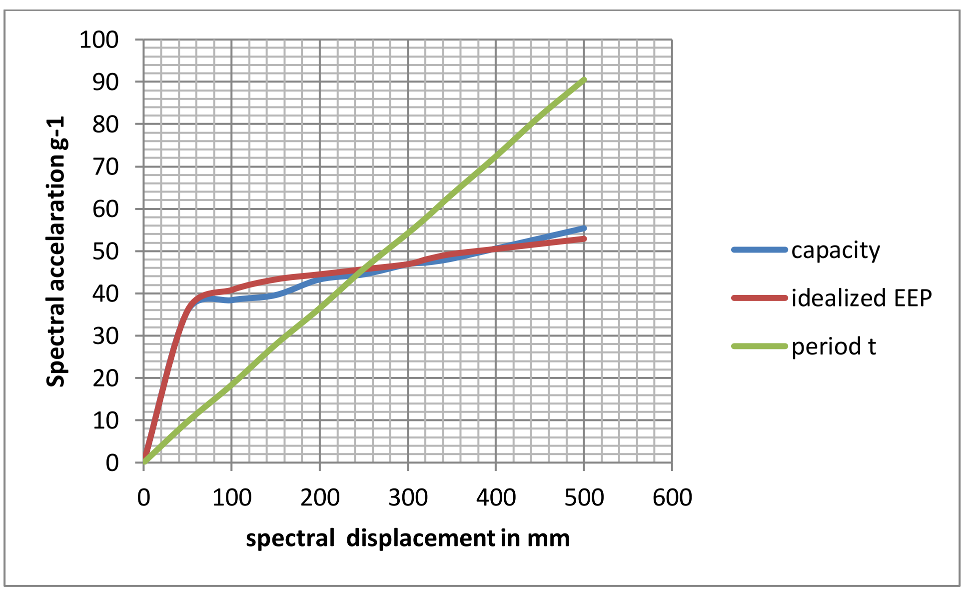

Fig 5.12: Capacity curve (unbraced)

5.11 Capacity curve RBSC for Diagonal bracing

5.11.1 For (ASCE 41-13 code)

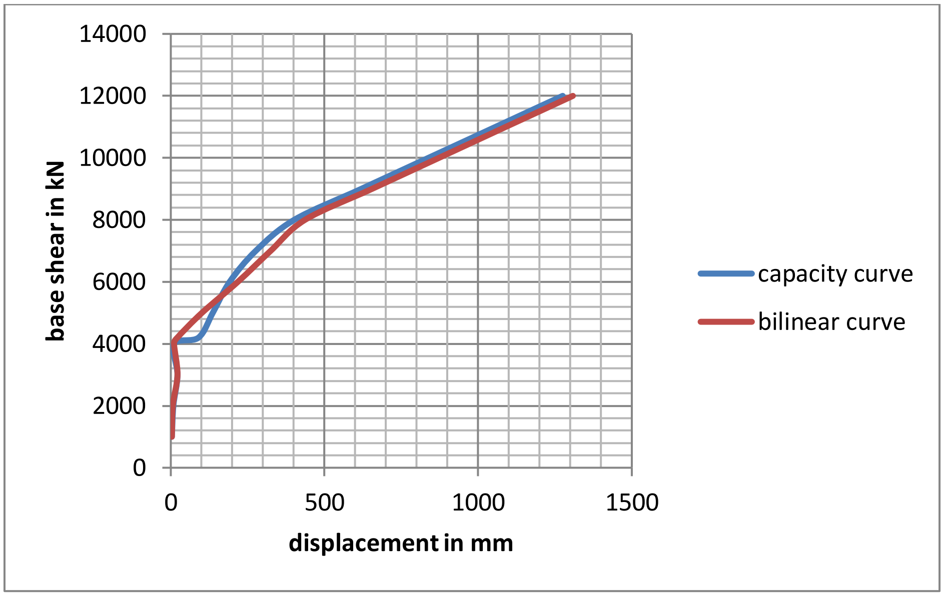

Fig 5.13: Capacity curve (Diagonal bracing)

5.11.2 For (NTC-2008 code)

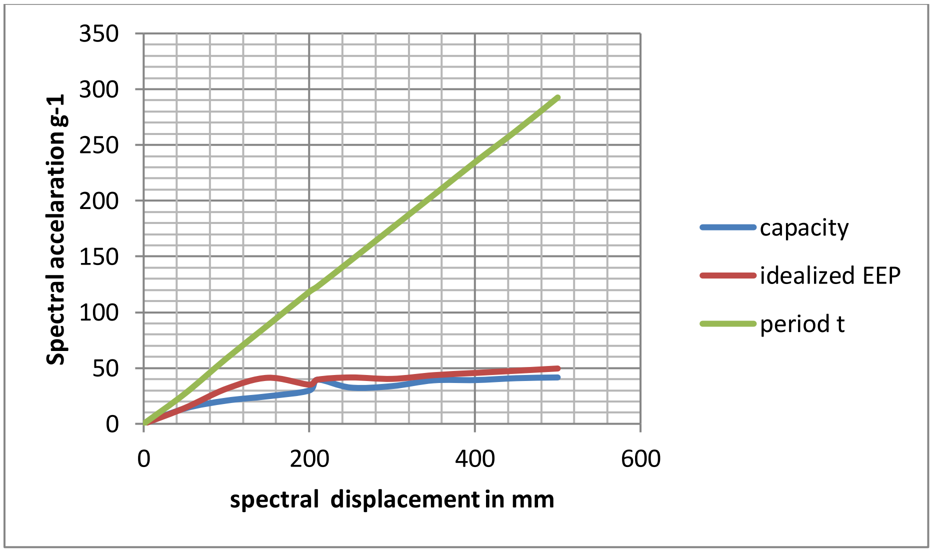

Fig 5.14: Capacity curve (Diagonal bracing)

5.11.3 For (EC-8 code)

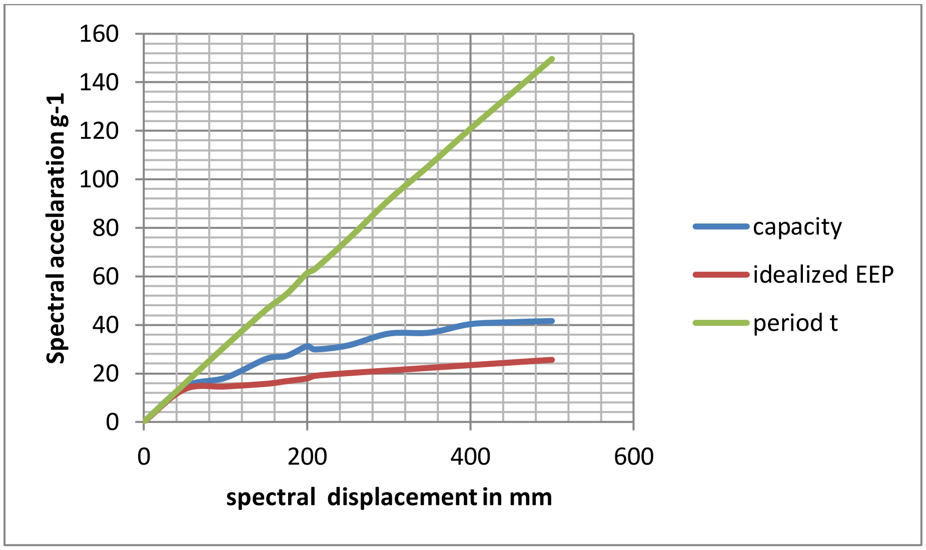

Fig 5.15: Capacity curve (Diagonal bracing)

5.12 Capacity curve RBSC for Chevron bracing

5.13.1 For (ASCE 41-13 code)

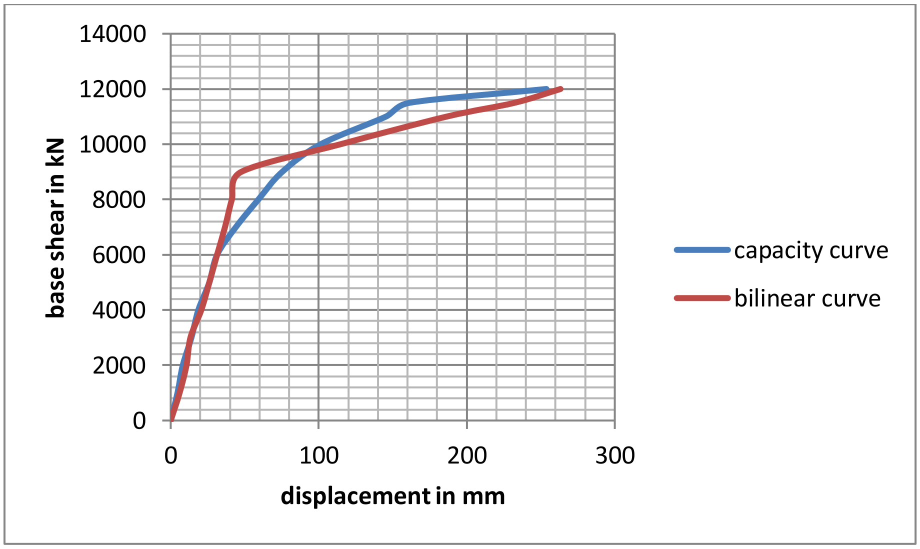

Fig 5.16: Capacity curve (Chevron bracing)

5.12.2 For (NTC-2008 code)

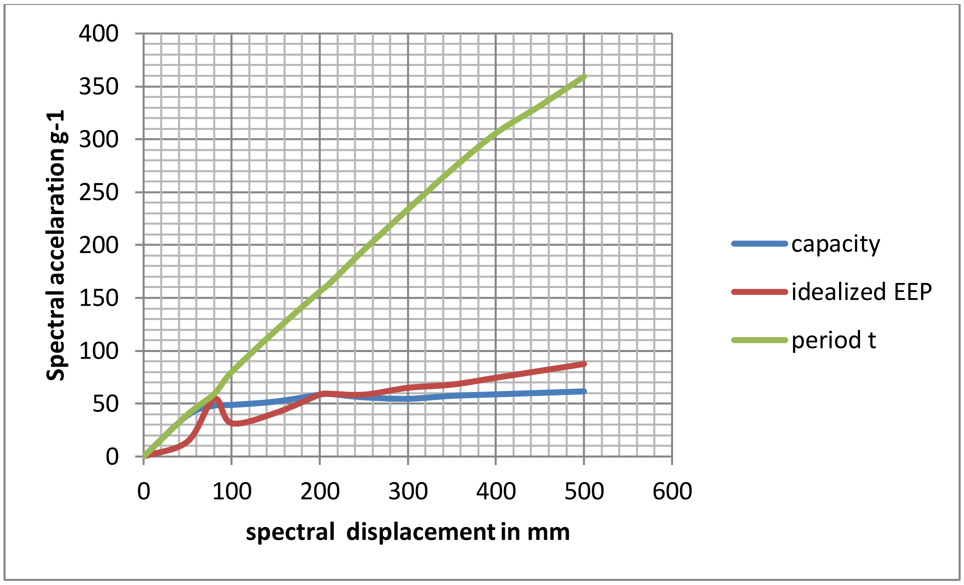

Fig 5.17: Capacity curve (Chevron bracing)

5.12.3 For (EC-8 code)

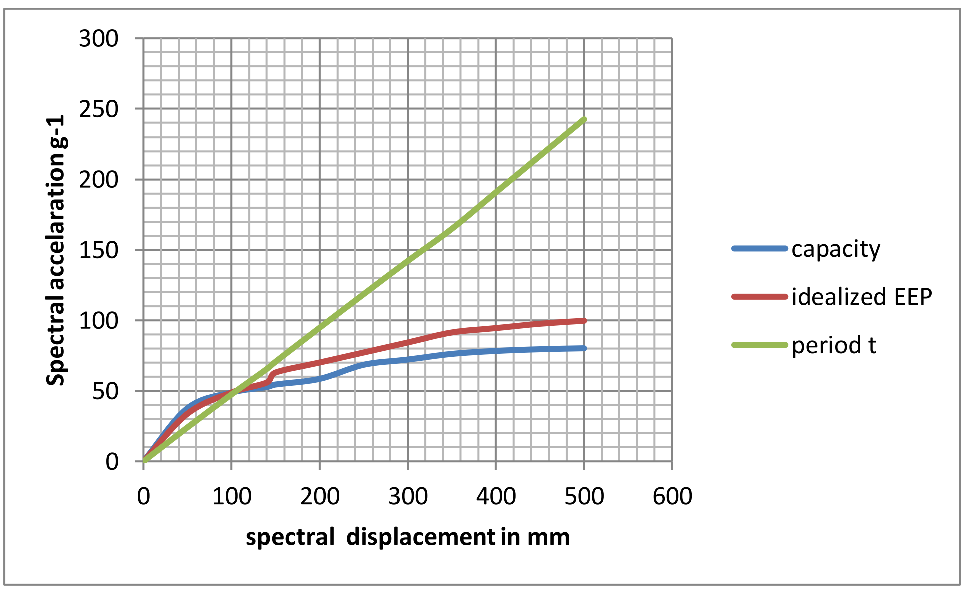

Fig 5.18: Capacity curve (Chevron bracing)

5.13 Target displacement and maximum base shear for rectangular building due to push X

Bracing	Target Displacement mm			Base Shear KN		
	ASCE 41-13	NTC-2008	EC-8	ASCE 41-13	NTC-2008	EC-8
Unbraced	384	265	263	9762	8445	9094
Diagonal bracing	357	199	252	10592	9028	9557
Chevron bracing	334	146	244	12979	11066	11852

Table 5.17 Maximum displacement and maximum base shear due to push X

5.14 Target displacement and maximum base shear for square building due to push X

Bracing	Target Displacement mm			Base Shear KN		
	ASCE 41-13	NTC-2008	EC-8	ASCE 41-13	NTC-2008	EC-8
Unbraced	346	345	341	10421	8103	8972
Diagonal bracing	289	208	314	11251	8686	9434
Chevron bracing	274	203	291	11992	11064	11411

Table 5.18 Maximum displacement and maximum base shear due to push X

5.15 Target displacement and maximum base shear for rectangular building due to push Y

Bracing	Target Displacement mm			Base Shear KN		
	ASCE 41-13	NTC-2008	EC-8	ASCE 41-13	NTC-2008	EC-8
Unbraced	409	293	292	10752	9430	10067
Diagonal bracing	382	252	283	11572	9993	10540
Chevron bracing	362	156	272	13954	12041	12845

Table 5.19 Maximum displacement and maximum base shear due to push Y

5.16 Target displacement and maximum base shear for square building due to push Y

Bracing	Target Displacement mm			Base Shear KN		
	ASCE 41-13	NTC-2008	EC-8	ASCE 41-13	NTC-2008	EC-8
Unbraced	346	345	341	10421	8103	8972
Diagonal bracing	289	208	314	11251	8686	9434
Chevron bracing	274	203	291	11992	11064	11411

Table 5.20 Maximum displacement and maximum base shear due to push Y

5.17 Maximum displacement due to push X and push Y

The maximum displacement obtained from rectangular and square building due to push X as per ASCE 41-13 code as mentioned in table5.17 and table 5.18. Below are the compared bar-charts:

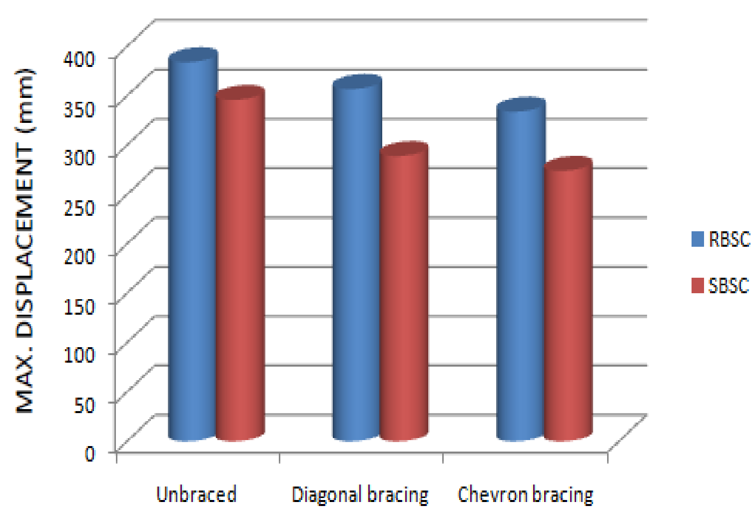

Fig 5.19: Comparison on displacement for ASCE 41-13 due to push X

The maximum displacement obtained from rectangular and square building due to push X as per NTC-2008 code as mentioned in table5.17 and table 5.18. Below are the compared bar-charts:

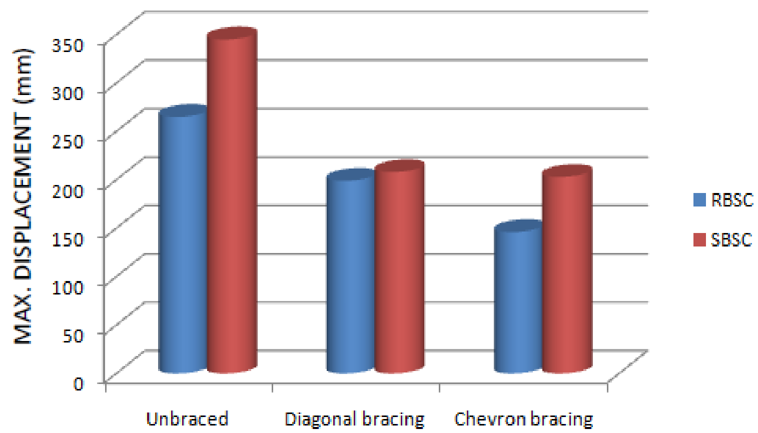

Fig 5.20: Comparison on displacement for NTC-2008 due to push X

The maximum displacement obtained from rectangular and square building due to push X as per EC-8 code as mentioned in table5.17 and table 5.18. Below are the compared bar-charts:

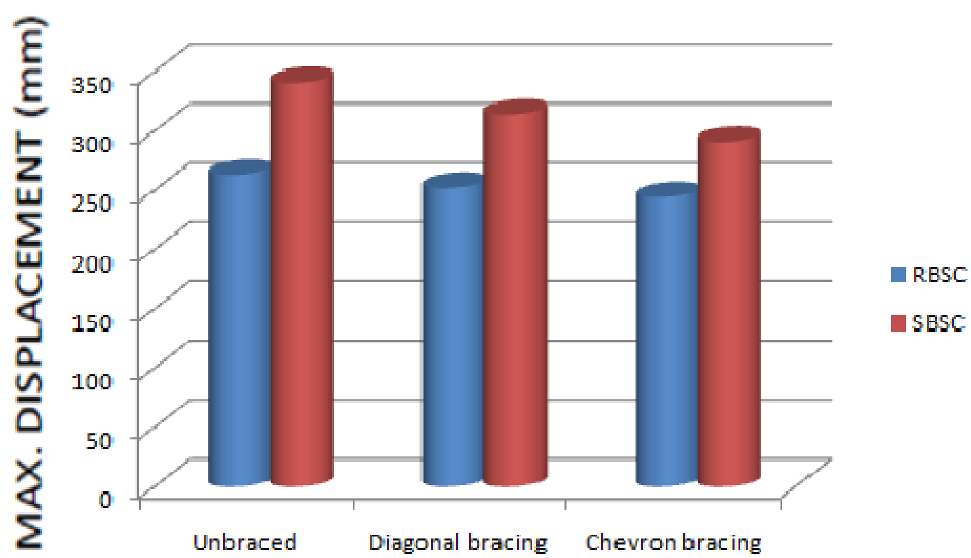

Fig5.21: Comparison displacement for EC-8 due to push X

From the comparison values in fig 5.19, fig 5.20, fig 5.21, it can be clearly found that due to introduction of bracing in both (rectangular, square) structures in X-direction the displacement have been reduced by 10% in SBSC using ASCE 41-13 code for unbraced plan, 19% reduce SBSC as compared to RBSC using ASCE 41-13 code for diagonal bracing, 18% reduce SBSC as compared to RBSC using ASCE 41-13 code for chevron bracing, 30% reduce RBSC as compared to SBSC using NTC-2008 code for unbraced plan, 5% reduced RBSC as compared to SBSC using NTC-2008 code for diagonal bracing, 39% reduced RBSC as compared to SBSC using NTC-2008 code for chevron bracing, 29% reduce RBSC as compared to SBSC using EC-8 code for unbraced plan, 24% reduce RBSC as compared to SBSC using EC-8 code for diagonal bracing, 19% reduce RBSC as compared to SBSC using EC-8 code for chevron bracing.

The maximum displacement obtained from rectangular and square building due to push Y as per ASCE 41-13 code as mentioned in table5.19 and table 5.20. Below are the compared bar-charts:

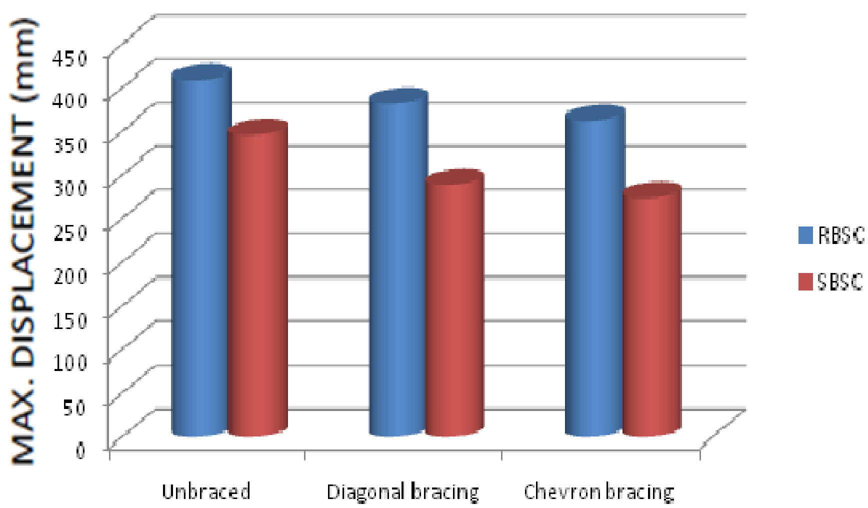

Fig 5.22: Comparison on displacement for ASCE 41-13 due to push Y

The maximum displacement obtained from rectangular and square building due to push Y as per NTC-2008 code as mentioned in table5.19 and table 5.20. Below are the compared bar-charts:

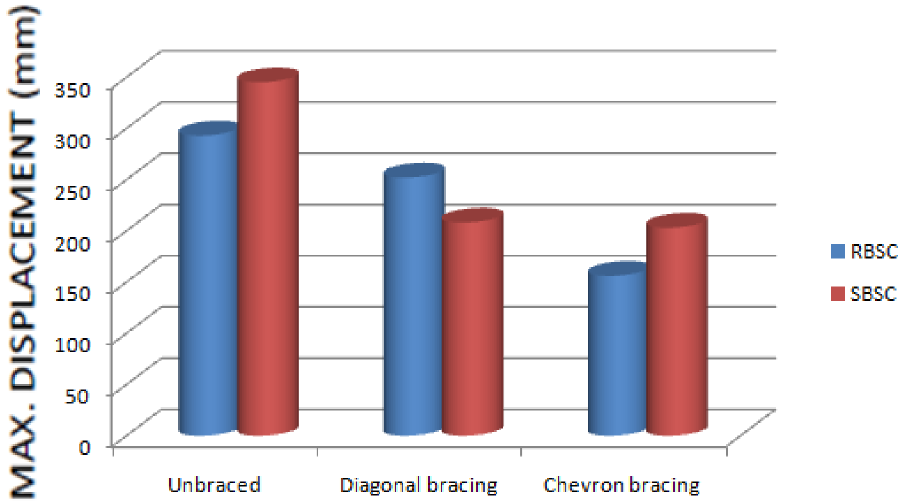

Fig 5.23: Comparison on displacement for NTC-2008 due to push Y

The maximum displacement obtained from rectangular and square building due to push Y as per EC-8 code as mentioned in table5.19 and table 5.20. Below are the compared bar-charts:

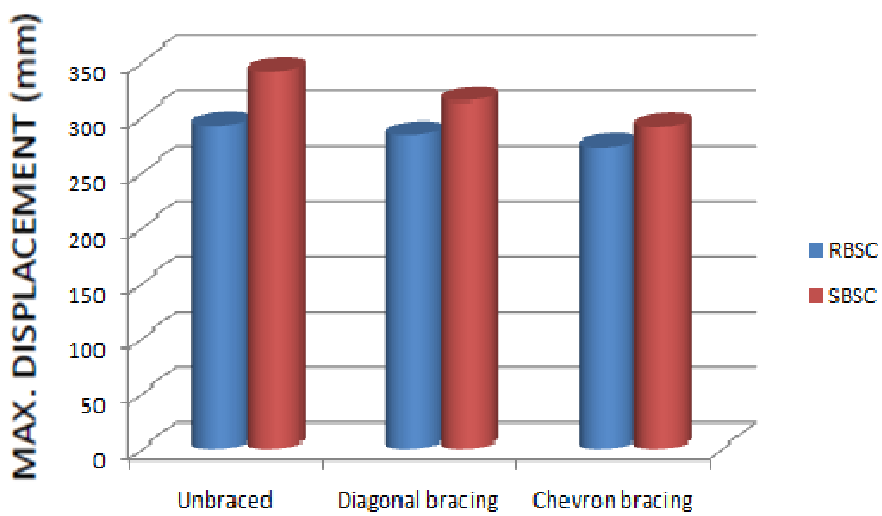

Fig5.24: Comparison displacement for EC-8 due to push Y

From the comparison values in fig 5.22, fig 5.23, fig 5.24, it can be clearly found that due to introduction of bracing in both (rectangular, square) structures in Y-direction the displacement have been reduced by 15% in SBSC using ASCE 41-13 code for unbraced plan, 24% reduce SBSC as compared to RBSC using ASCE 41-13 code for diagonal bracing, 24% reduce SBSC as compared to RBSC using ASCE 41-13 code for chevron bracing, 18% reduce RBSC as compared to SBSC using NTC-2008 code for unbraced plan, 17% reduced SBSC as compared to RBSC using NTC-2008 code for diagonal bracing, 30% reduced RBSC as compared to SBSC using NTC-2008 code for chevron bracing, 17% reduce RBSC as compared to SBSC using EC-8 code for unbraced plan, 20% reduce RBSC as compared to SBSC using EC-8 code for diagonal bracing, 7% reduce RBSC as compared to SBSC using EC-8 code for chevron bracing.

5.18 Base shear due to push X and push Y

The base shear obtained from rectangular and square building due to push X as per ASCE 41-13 code as mentioned in table5.17 and table 5.18. Below are the compared bar-charts:

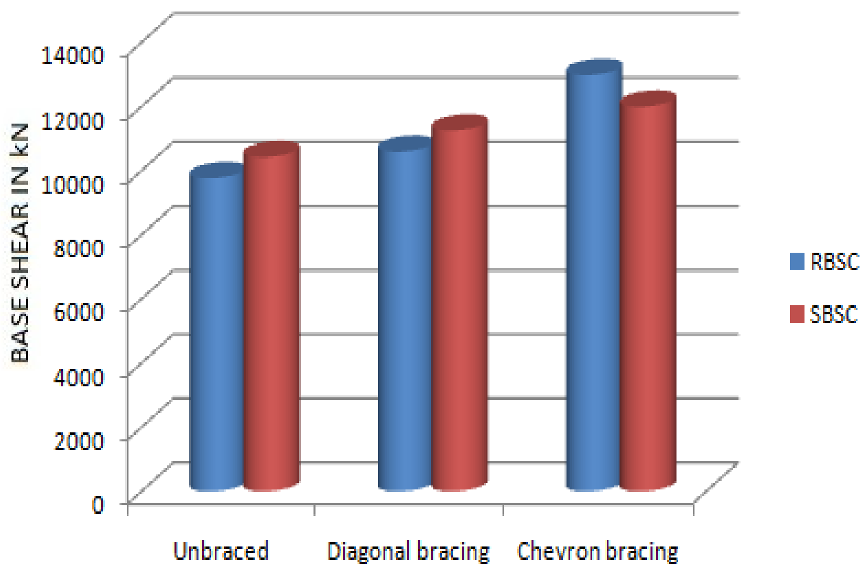

Fig5.25: Comparison base shear for ASCE-41-13 due to push X

The base shear obtained from rectangular and square building due to push X as per NTC-2008 code as mentioned in table5.17 and table 5.18. Below are the compared bar-charts:

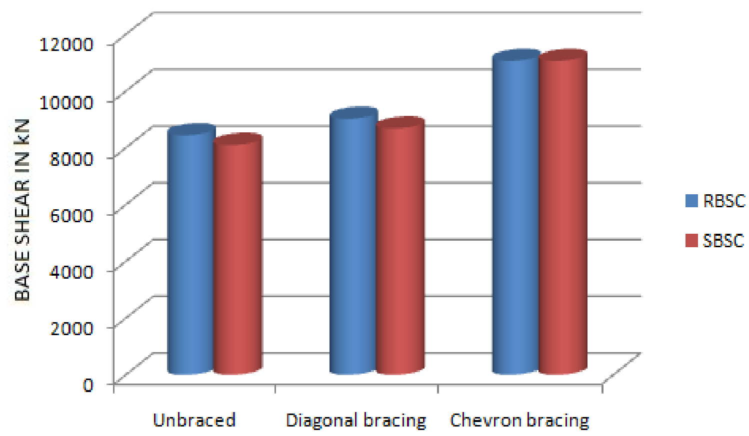

Fig 5.26: Comparison base shear for NTC-2008 due to push X

The base shear obtained from rectangular and square building due to push X as per EC-8 code as mentioned in table5.17 and table 5.18. Below are the compared bar-charts:

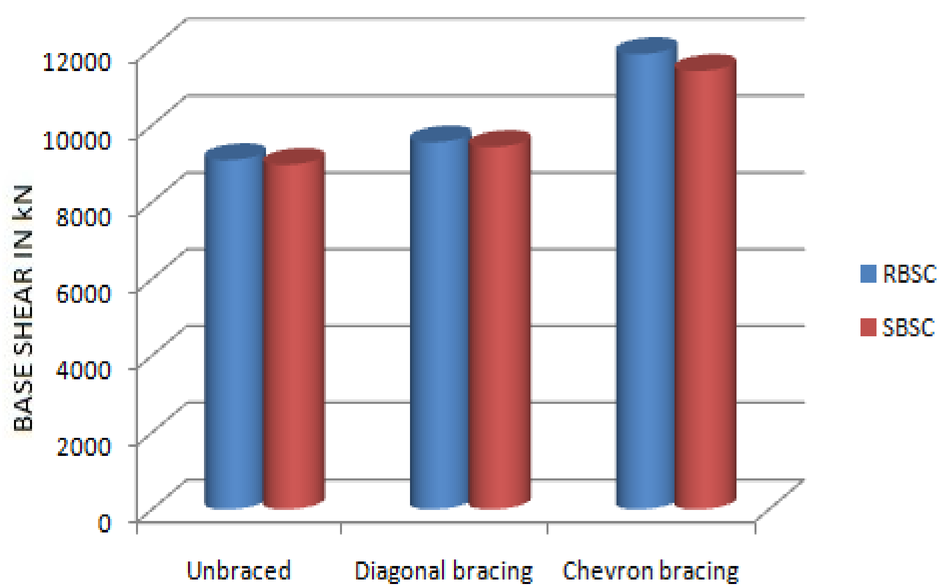

Fig 5.27: Comparison base shear for EC-8 due to push X

From the comparison values in fig 5.25, fig 5.26, fig 5.27, it can be clearly found that due to introduction of bracing in both (rectangular, square) structures in X-direction the base shears have been reduced by 7% in RBSC using ASCE 41-13 code for unbraced plan, 6% reduce RBSC as compared to SBSC using ASCE 41-13 code for diagonal bracing, 8% increased RBSC as compared to SBSC using ASCE 41-13 code for chevron bracing, 4% increased RBSC as compared to SBSC using NTC-2008 code for unbraced plan, 4% increased RBSC as compared to SBSC using NTC-2008 code for diagonal bracing, 2% increased RBSC as compared to SBSC using NTC-2008 code for chevron bracing, 2% increased RBSC as compared to SBSC using EC-8 code for unbraced plan,2% increased RBSC as compared to SBSC using EC-8 code for diagonal bracing, 4% increased RBSC as compared to SBSC using EC-8 code for chevron bracing.

The base shear obtained from rectangular and square building due to push Y as per ASCE 41-13 code as mentioned in table5.19 and table 5.20. Below are the compared bar-charts:

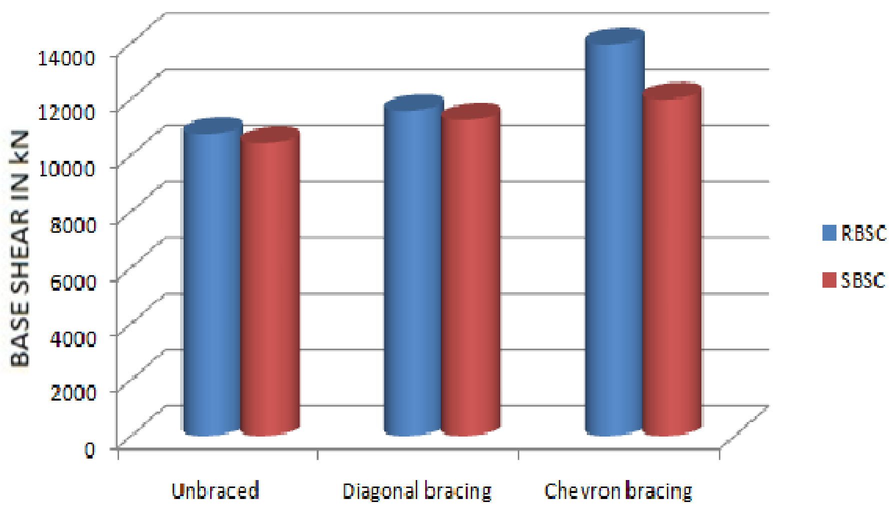

Fig5.28: Comparison base shear for ASCE-41-13 due to push Y

The base shear obtained from rectangular and square building due to push Y as per NTC-2008 code as mentioned in table5.19 and table 5.20. Below are the compared bar-charts:

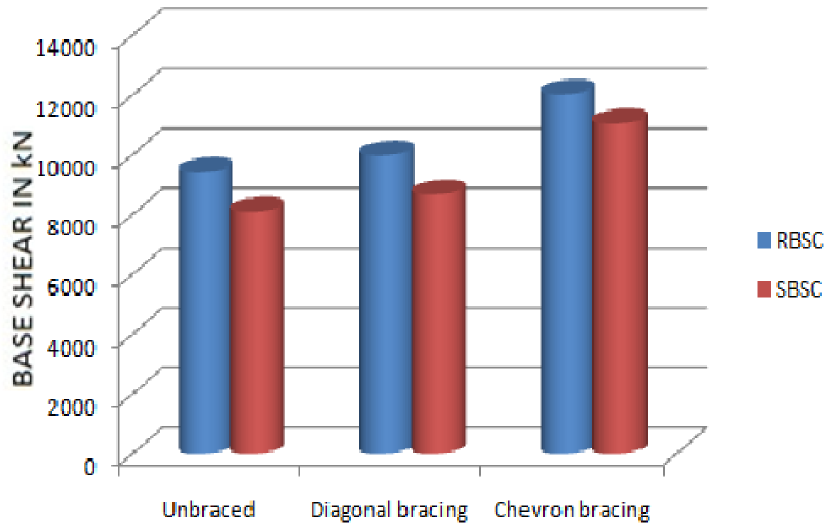

Fig 5.29: Comparison base shear for NTC-2008 due to push Y

The base shear obtained from rectangular and square building due to push Y as per EC-8 code as mentioned in table5.19 and table 5.20. Below are the compared bar-charts:

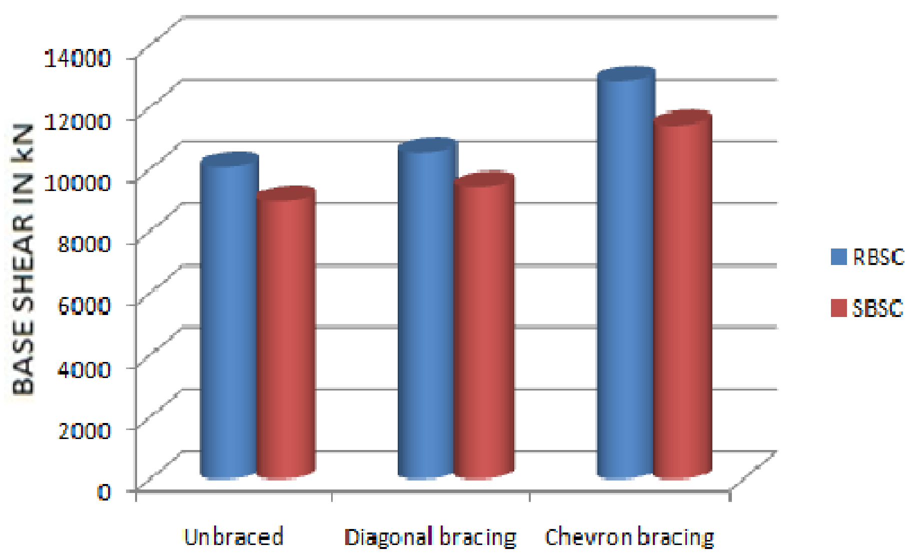

Fig 5.30: Comparison base shear for EC-8 due to push Y

From the comparison values in fig 5.28, fig 5.29, fig 5.30, it can be clearly found that due to introduction of bracing in both (rectangular, square) structures in Y-direction the base shears have been increased by 3% in RBSC using ASCE 41-13 code for unbraced plan, 3% increased RBSC as compared to SBSC using ASCE 41-13 code for diagonal bracing, 14% increased RBSC as compared to SBSC using ASCE 41-13 code for chevron bracing, 14% increased RBSC as compared to SBSC using NTC-2008 code for unbraced plan, 13% increased RBSC as compared to SBSC using NTC-2008 code for diagonal bracing, 8% increased RBSC as compared to SBSC using NTC-2008 code for chevron bracing, 11% increased RBSC as compared to SBSC using EC-8 code for unbraced plan,10% increased RBSC as compared to SBSC using EC-8 code for diagonal bracing, 11% increased RBSC as compared to SBSC using EC-8 code for chevron bracing.

5.19 Storey Drift

Total storey drift is that the absolute displacement of any storey relative to the base. According to IS.1893-2002 provision 7.11.1 the storey drift in any storey because of the minimum specified design lateral force with partial ratio 1.00 shall not be exceeding 0.004 times the storey height.

5.20 Storey Max/Avg. Displacements

Suggested maximum drift at the highest point of buildings vary between H/50 and H/2000 Where H is that the height of the building. A limiting value for the most displacement within the elastic limits was obtained as a function of the peak of a story, the stiffness of a story, number of stories, effective depth d of a shear wall, the yield strain of steel εy and so the most allowable concrete strain εc. However, the worth H/50 suggested by UBC97 and IBC 2006 generates large strains at the underside of a shear wall.

Hence for story 12, H=36m.

Limiting displacement =$H/$50= 36000/50 = 720mm=0.72m

Therefore, obtained values are within limits. Below figures are curves Max. Displacements vs. Story levels for Push X

Maximum storey displacement for rectangular and square building due to PUSH X.

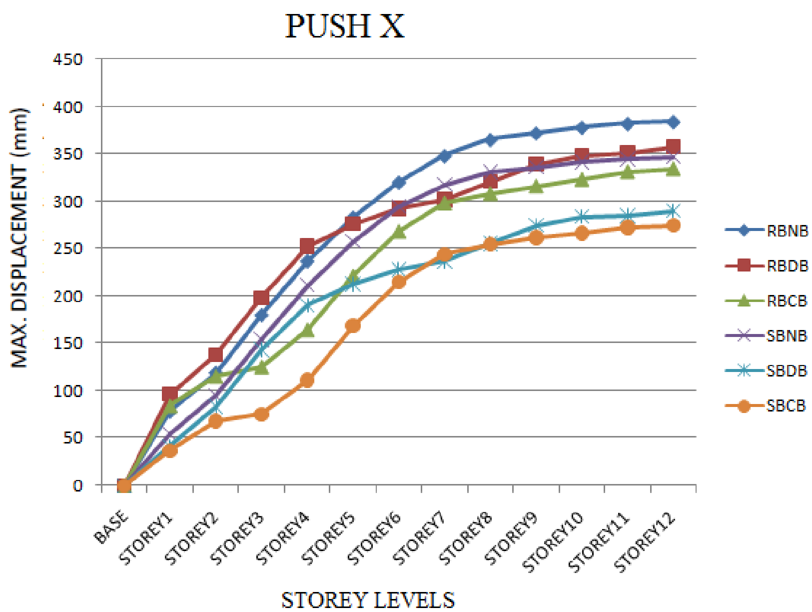

Fig 5.31: Comparison Maximum storey displacements ASCE 41-13 code due push X

Maximum storey displacement for rectangular and square building due to PUSH X

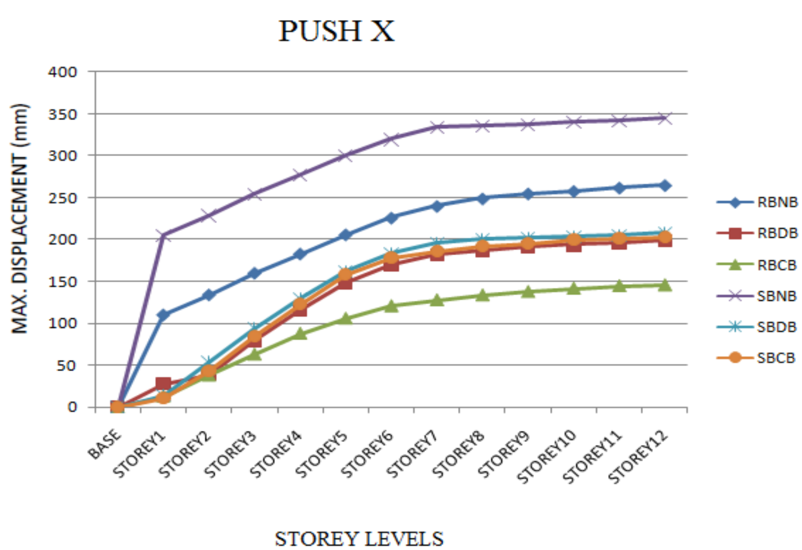

Fig 5.32: Comparison Maximum storey displacements for NTC-2008 code due to push X

Maximum storey displacement for rectangular and square building due to PUSH X

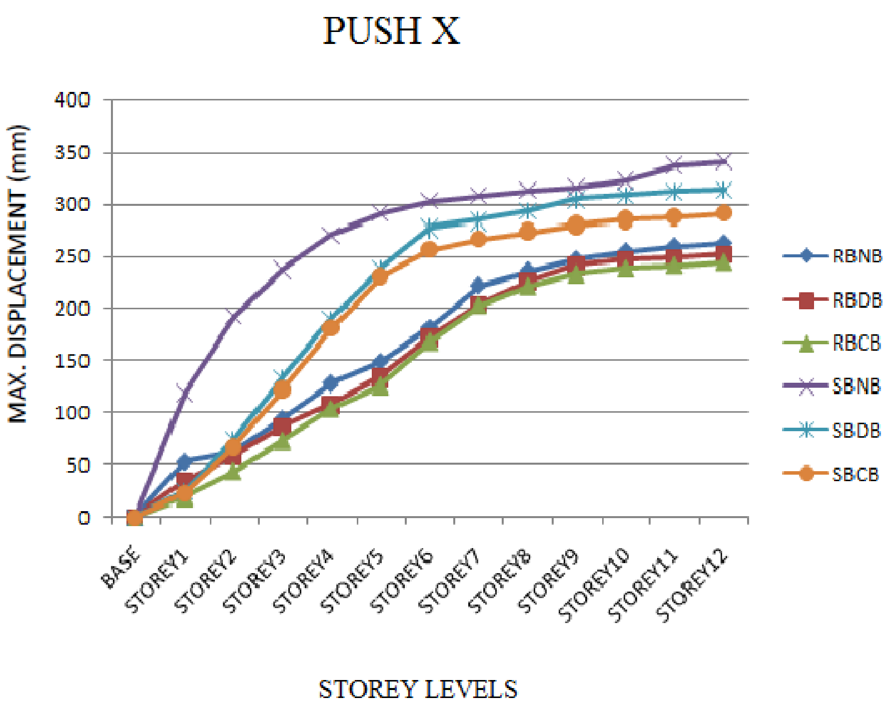

Fig 5.33: Comparison Maximum storey displacements for EC-8 code due to push X

From X-direction the comparison curves in fig5.31, fig5.32, fig 5.33, maximum displacement 39% achieved after applying chevron bracing in RBSC using NTC-2008 code.

Maximum storey displacement for rectangular and square building due to PUSH Y.

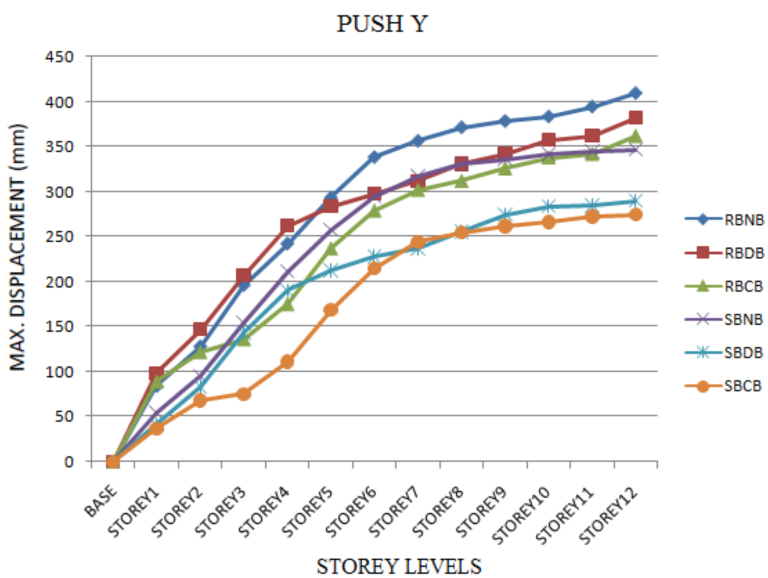

Fig 5.34: Comparison Maximum storey displacements ASCE 41-13 code due to push Y

Maximum storey displacement for rectangular and square building due to PUSH Y.

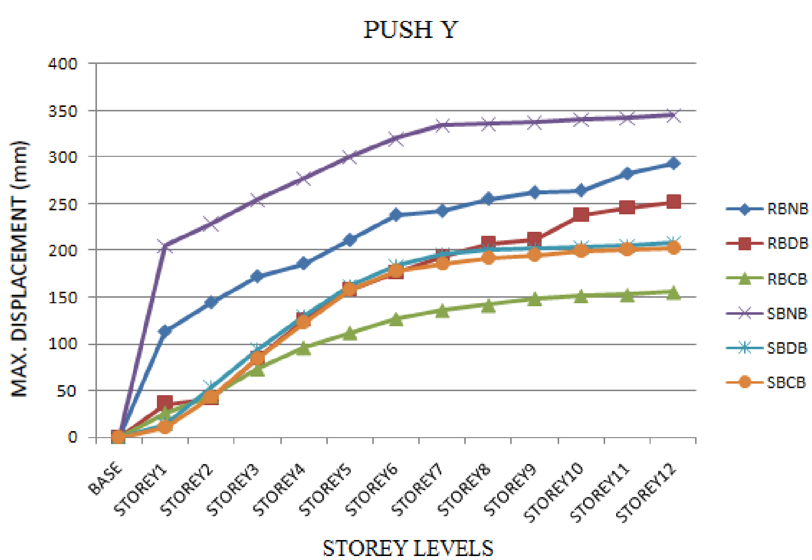

Fig 5.35: Comparison Maximum storey displacements for NTC-2008 code due to push Y

Maximum storey displacement for rectangular and square building due to PUSH Y.

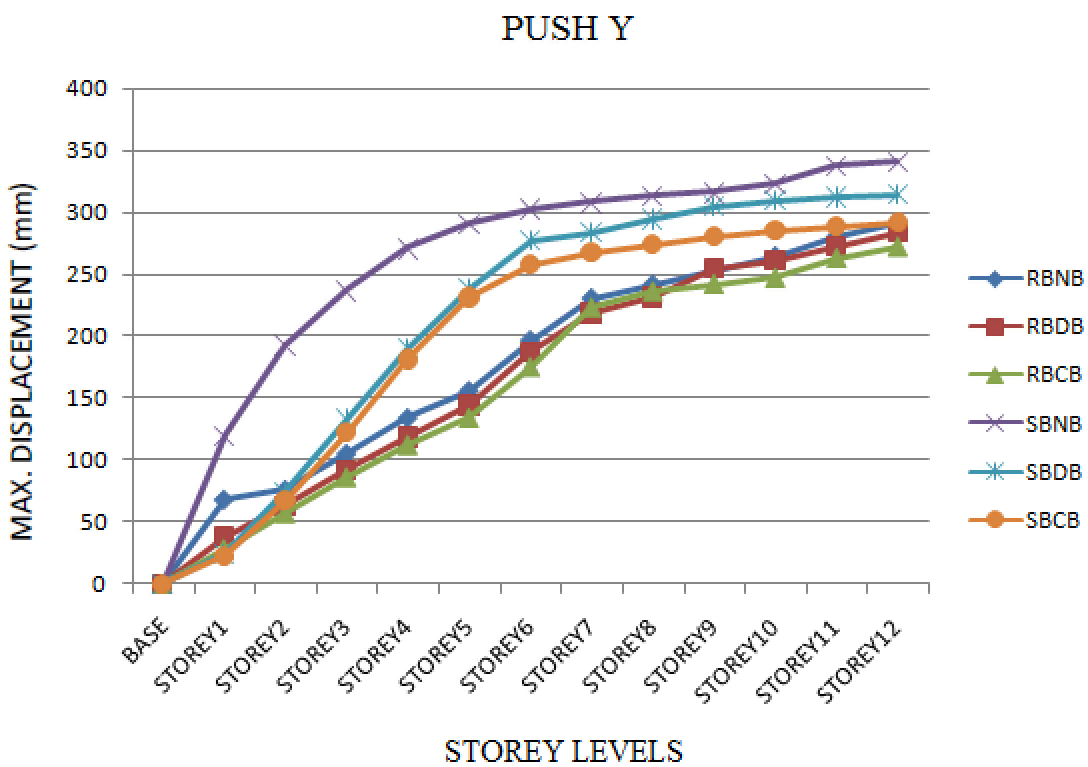

Fig 5.36: Comparison Maximum storey displacements for EC-8 code due to push Y

From Y-direction the comparison curves in fig5.31, fig5.32, fig 5.33, maximum displacement 30% achieved after applying chevron bracing in RBSC using NTC-2008 code.

CHAPTER 6: SUMMARY AND CONCLUSION

6.1 SUMMARY

The procedure of analysis using Etabs 2016 software obtains certain results from the analysis. Two Buildings Square and Rectangular are compared (X and Y direction) with three different type of bracing i.e. Unbraced, Diagonal bracing, Chevron bracings analyzed and comparative results were developed through pushover analysis.

It is found that due to X-direction the displacement have been reduced by 10% in SBSC using ASCE 41-13 code for unbraced plan, 19% reduce SBSC as compared to RBSC using ASCE 41-13 code for diagonal bracing, 18% reduce SBSC as compared to RBSC using ASCE 41-13 code for chevron bracing, 30% reduce RBSC as compared to SBSC using NTC-2008 code for unbraced plan, 5% reduced RBSC as compared to SBSC using NTC-2008 code for diagonal bracing, 39% reduced RBSC as compared to SBSC using NTC-2008 code for chevron bracing, 29% reduce RBSC as compared to SBSC using EC-8 code for unbraced plan, 24% reduce RBSC as compared to SBSC using EC-8 code for diagonal bracing, 19% reduce RBSC as compared to SBSC using EC-8 code for chevron bracing.

It is found that due to Y-direction the displacement have been reduced by 15% in SBSC using ASCE 41-13 code for unbraced plan, 24% reduce SBSC as compared to RBSC using ASCE 41-13 code for diagonal bracing, 24% reduce SBSC as compared to RBSC using ASCE 41-13 code for chevron bracing, 18% reduce RBSC as compared to SBSC using NTC-2008 code for unbraced plan, 17% reduced SBSC as compared to RBSC using NTC-2008 code for diagonal bracing, 30% reduced RBSC as compared to SBSC using NTC-2008 code for chevron bracing, 17% reduce RBSC as compared to SBSC using EC-8 code for unbraced plan, 20% reduce RBSC as compared to SBSC using EC-8 code for diagonal bracing, 7% reduce RBSC as compared to SBSC using EC-8 code for chevron bracing.

For base shear, it can be clearly found that due to introduction of bracing in both (rectangular, square) structures in X-direction the base shears have been reduced by 7% in RBSC using ASCE 41-13 code for unbraced plan, 6% reduce RBSC as compared to SBSC using ASCE 41-13 code for diagonal bracing, 8% increased RBSC as compared to SBSC using ASCE 41-13 code for chevron bracing, 4% increased RBSC as compared to SBSC using NTC-2008 code for unbraced plan, 4% increased RBSC as compared to SBSC using NTC-2008 code for diagonal bracing, 2% increased RBSC as compared to SBSC using NTC-2008 code for chevron bracing, 2% increased RBSC as compared to SBSC using EC-8 code for unbraced plan,2% increased RBSC as compared to SBSC using EC-8 code for diagonal bracing, 4% increased RBSC as compared to SBSC using EC-8 code for chevron bracing.

For base shear, it can be clearly found that due to introduction of bracing in both (rectangular, square) structures in Y-direction the base shears have been increased by 3% in RBSC using ASCE 41-13 code for unbraced plan, 3% increased RBSC as compared to SBSC using ASCE 41-13 code for diagonal bracing, 14% increased RBSC as compared to SBSC using ASCE 41-13 code for chevron bracing, 14% increased RBSC as compared to SBSC using NTC-2008 code for unbraced plan, 13% increased RBSC as compared to SBSC using NTC-2008 code for diagonal bracing, 8% increased RBSC as compared to SBSC using NTC-2008 code for chevron bracing, 11% increased RBSC as compared to SBSC using EC-8 code for unbraced plan,10% increased RBSC as compared to SBSC using EC-8 code for diagonal bracing, 11% increased RBSC as compared to SBSC using EC-8 code for chevron bracing.

6.2 CONCLUSION

From the results discussed with respect to the building models considered, leads to the following conclusions:

1. After the analysis of both structures with different types of Bracing, it has been concluded that the Storey Displacement of the structure decreases after the application of bracing system.
2. The maximum reduction in the storey displacement occurs after the application of Chevron bracing system in Rectangular building.
3. The maximum displacement of the rectangular building is reduced by 39% with the use of Chevron bracing for X-direction when compared with square building for NTC-2008 code.

4. The maximum displacement of the rectangular building is reduced by 30% with the use of Chevron bracing for Y-direction when compared with square building for NTC-2008 code.
5. In both buildings the maximum displacement decreases for diagonal and chevron bracing system (in both directions) used compared to without bracings and the base shear increases for diagonal and chevron bracing as compared to unbraced frame structures.
6. The base shear of the rectangular building is increased by 8% with the use of Chevron bracing in X-direction when compared with square building for ASCE 41-13.
7. The base shear of the rectangular building is increased by 14% with the use of Chevron bracing in Y-direction when compared with square building for ASCE 41-13.
8. It is observed that rectangular buildings are performing well as compared to square building.

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2. Mohd Asad Khan, MD Tasleem and Shariq Masood Khan, (2021) 'Pushover Analysis of Reinforced Concrete High Rise Building with and without Bracing', International Research Journal of Engineering and Technology, Volume 8,, Issue 7, July 2021.