Study of Inelastic Behavior of Eccentrically Braced Frames under Non Linear Range

Dr. S. N. Tande
Professor & Head., Department of Applied Mechanics,
Walchand college of Engineering, Sangli, Maharashtra, India

Amol A. Sankpal
Department of Applied Mechanics,
Walchand college of Engineering, Sangli, Maharashtra, India

Abstract:This paper provides an introduction and overview of the design and behavior of seismic-resistant eccentrically braced frames (EBFs). Within last ten years, EBF's have become a widely recognized lateral load resisting system for steel building in areas of high seismicity. In general, braces are the members that resist against lateral forces in a steel structure while the structures are under seismic excitation. Although the height of a structure and the structural system are the two parameters which can affect the inelastic behavior and response of the structure but these parameters have not been taken into consideration in the current design codes for designing of Eccentric Braced Frames (EBFs). In this study six frames were exerted which were braced with three different eccentric braces (V, Inverted-V and Diagonal) in two different heights (4 and 8 story). Then the frames were assessed by nonlinear static (pushover) analysis mainly based on FEMA 440 (2005). As a result of these frame analysis, it can be observed that the plastic hinges firstly occur at the fuse section of braces and then at the compressive members of the eccentric braces. Furthermore, a comparison relevant to the total weight of the frames has been conducted among the above mentioned frames. The primary purpose of this paper is to present the best suitable bracing system up to 8 story level in performance point of view and also economy point of view.

Keywords: nonlinear static analysis, Eccentric diagonal Bracing system, Eccentric Inverted-V bracing system, Eccentric V bracing system, Inelastic Performance

I. INTRODUCTION

Every year, many people die because of earthquakes around the world. Lateral stability has been one of the important problems of steel structures specifically in the regions with high seismic hazard. The Kobe earthquake in Japan and the Northridge earthquake that happened in the USA were two obvious examples where there was lack of lateral stability in steel structures. This issue has been one of the important subjects for researchers during the last three decades. Finally they came up with suggesting concentric, such as X, Diagonal and chevron, eccentric and knee bracing systems and these were used in real life projects by civil engineers for several decades.One of the principal factors affecting the selection of bracing systems is inelastic performance. The bracing system which has a more plastic deformation capacity prior to collapse, has the ability to absorb more energy while it is under seismic excitation.

Eccentrically braced frames (EBF's) are a lateral load-resisting system for steel building that can be considered a hybrid between conventional moment-resisting frames (MRF's) and concentrically braced frames (CBF's). EBFs are in effect an attempt to combine individual advantages of MRFs and CBFs, while minimizing their respective disadvantages. Figure 1 illustrates several common EBF arrangements.

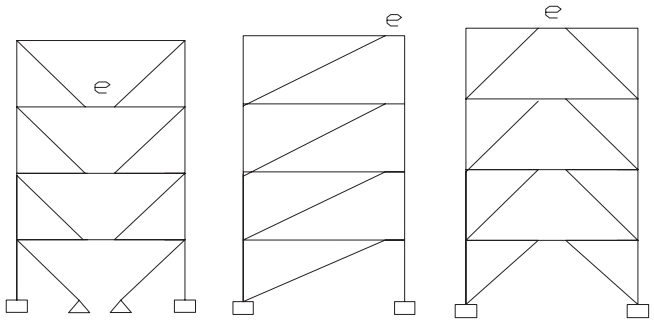


Fig. 1 Typical bracing arrangements for EBFs

The distinguishing characteristic of an EBF is that at least one end of brace is connected to that the brace force is transmitted either to another brace or column through shear and bending in a beam segment called a link. The link length in Fig. 1 are identified by the letter 'e'. The excellent performance of EBFs under severe earthquake loading was demonstrated on one-third-scale model frames at the University of California in 1977. Soon after thisstudy, several major buildings were constructed incorporating EBFs as part of their lateral seismic resisting systems.

The most attractive feature of EBFs for seismic-resistant design is their high stiffness combined with excellent ductility and energy-dissipation capacity. The bracing members in EBFs provide the high elastic stiffness characteristic of CBFs, permitting code drift requirements to be met economically. Yet under very severe earthquake loading, properly designed and detailed EBFs provide ductility and energy dissipation characteristics of MRFs.

During the recent decades, nonlinear response of bracing systems has been studied and consequently parameters such as, seismic behavior factor, R, over strength factor, W, and displacement amplification factor, Cd, were introduced to loading codes of practice like UBC (Uniform Building Code) and IBC (International Building Code). These design codes are widely used in the USA and also throughout the world In the process of the earthquake load calculation of a structure, seismic behavior factor is the parameter illustrating the impact of nonlinear performance of the bracing system that is fundamentally affected by the system ductility. The efficiency of bracing systems is influenced by these key parameters because they directly affect the reduction of the earthquake loads in the structure. In accordance with the loading codes, specific R, W and Cd factors were introduced for various structural systems (illustrating the distinction of their nonlinear behavior), such as concrete moment frame and steel moment frame with high, medium and low ductility, steel frames with concrete shear walls and steel braced frames.

Producing a frame that can remain substantially elastic outside a well-defined linkage is the most important factor that EBF designing is done on the basis of it. While being undergone huge loading, it is foreseen that the link will be distorted inelastically with great ductility and dissipation of energy. The provisions of codes are provided so that they guarantee the beams, braces, columns and their connections to stay and remain in elastic phase and also the links remain stable.

A. Three Important Variables in the Designing of EBF Bracing System,

1) Bracing configuration
2) The link length
3) The link section properties

When these elements are taken into consideration, then the rest of the designing process of the frame can be executed with minimal effect on the link size, configuration or link length. Designating a systematic procedure to assess the effect of the prominent variables is crucial to EBF design. If attention is not paid identify their effect, then the designer may have to iterate through a myriad of probable combinations. The strategy suggested by (Roy Becker & Michael Ishler, 1996) in their guide is as follows: 1) Establish the design criteria. 2) Identify a bracing configuration. 3) Select a link length. 4) Choose an appropriate link section. 5) Design braces columns and other components of the frame.

B. Bracing configuration:

UBC 2211.10.2 the selection of a bracing system configuration is related to various elements. These factors encompass the size and position of required open areas in the framing elevation and the height to width proportions of the bay elevation. These constraints may substitute structural optimization as designing criteria. UBC 2211.10.2 requires at least one end of every brace to frame into a link. There are many frame configurations which meet this criterion.

C. Frame proportions:

Michael D.Engelhardt, and Egorp.popov, p. 504 (1989) in designing EBF systems, the proportions of frames are typically opted to increase the application of the high shear forces in the link. Frame properties of typical eccentric braces are shown in figure 2 below. Shear yielding is very ductile and its capacity for inelastic behavior is very high. This characteristic, as well as the benefits of frames with high stiffness, generally make short lengths desirable.

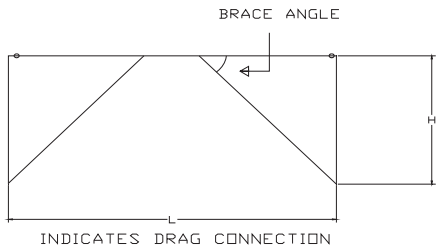


Figure 2: Frame proportions

The desirable angel of the brace as shown in the above picture should be kept between 35° and 60°. If the angle is beyond or below this range, then it will result in awkward details at the brace- to- beam and brace-to-column connections. Meanwhile small angles are also apt to result in a huge axial force member in the link beams (Michael D.Engelhardt, and Egorp.popov, p. 504) (1989).

D. Link length:

The inelastic performance of a link is a great deal affected by its length. If the link length becomes shorter, then the inelastic behavior will become greater as a result the influence of shear forces. Shear yielding has a tendency to occur uniformly alongside the link. Shear yielding has a high ductility and also considerable inelastic performance capacity which is more than that predicted by the web shear area, if the web is braced enough against buckling. (Michael D.Engelhardt, and Egorp.popov, p. 499, 1989). Often the behavior of the links are like short beams which are exposed to equal shear loads applied in opposite directions at the ends of the link. According to this style of loading, the moments produced at both ends are identical and also in the same direction. The shape of the link deformation is like the letter (S), which is distinct at mid span by a point of counter flexure. The measure of moment is equal to 1/2 the shear times the length of the link.

E. Link lengths generally behave as follows:

If E< 1.3 Ms/Vs Guarantees shear performance, and are recommended as upper limit for shear links (Egor p. popov, Kasai, and Michael, p. 46) (1978)

If E< 1.6Ms/Vs Link post - elastic deformation is controlled by shear yielding. UBC2211.10.4 rotation transition. ("Recommended lateral force requirements and commentary", p. 331, C709.4) (1996)

If E=2Ms/Vs theoretically, the behavior of Link is balanced between shear and flexural yielding.

If E<2Ms/Vs Link behavior considered to be controlled by shear for UBC 2211.10.3 (("recommended lateral force requirements and commentary", p. 330, C709.3) (1996)

If E>3 Ms/Vs By flexural yielding, Link post-elastic deformation is controlled. UBC2211.10.4 rotation transition. ("Recommended lateral force requirements and commentary", p. 331, C709.4) (1996)

F. Evaluation of nonlinear static procedures:

Nonlinear static procedures are recommended by FEMA 273 document in assessing the seismic performance of buildings for a given earthquake hazard representation. Three nonlinear static procedures specified in FEMA 273 are evaluated for their ability to predict deformation demands in terms of inter-story drifts and potential failure mechanisms. Two steel and two reinforced concrete buildings were procedures. Strong-motion records during the Northridge earthquake are available for these buildings. The study has shown that nonlinear static procedures are not effective in predicting inter-story drift demands compared to nonlinear dynamic procedures. Nonlinear static procedures were not able to capture yielding of columns in the upper levels of a building. This inability can be a significant source of concern in identifying local upper story failure mechanisms. The American Society of Civil Engineers (ASCE) is in the process of producing an U.S. standard for seismic rehabilitation existing buildings. It is based on Guidelines for Seismic Rehabilitation of Buildings (FEMA 273) which was published in 1997 by the U.S. Federal Emergency Management Agency. FEMA 273 consists of three basic parts:(a) Definition of performance objectives; (b) demand prediction using four alternative analysis procedures; and (c) acceptance criteria using force and/or deformation limits which are meant to satisfy the desired performance objective. FEMA-273 suggests four different analytical methods to estimate seismic demands:

I. Linear Static Procedure (LSP)
II. Linear Dynamic Procedure (LDP)
III. Nonlinear Static Procedure (NSP)

IV. Nonlinear Dynamic Procedure (NDP)

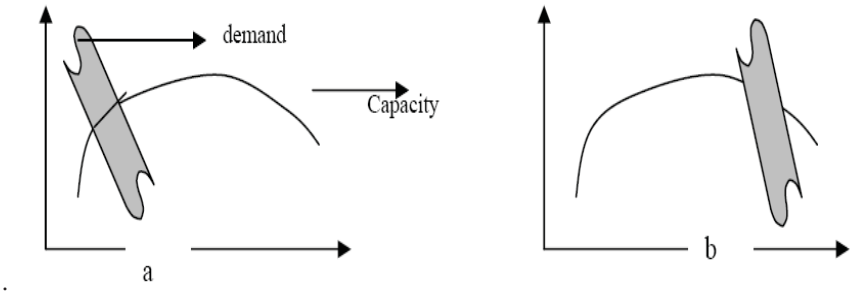
Given the limitations of linear methods and the complexity of nonlinear time-history analyses, engineers favor NSP as the preferred method of analysis. Following the analysis of a building, the safety and integrity of the structural system is assessed using acceptance criteria. For linear procedures acceptance criteria are based on demand-to-capacity ratios and for nonlinear procedures, they are based on deformation demands.

II. PUSHOVER METHODOLOGY

Pushover analysis is a static, nonlinear procedure in which the magnitude of the structural loading is incrementally increased in accordance with a certain predefined pattern with the increase in the magnitude of the loading, weak links and failure modes of the structure are identified. The loading is monotonic with the effects of the cyclic behavior and load reversals, being estimated by using a modified monotonic force deformation criteria and with damping approximations. Static pushover analysis is an attempt by structural engineering profession to evaluate the real strength of the structure and it promises to be a useful and effective tool for performance based design.

Local non-linear effects, such as flexural hinges and shear hinges, assumed to occur at the ends of the members, should be appropriately modelled. Redistribution of internal forces occurs with progressive plastic hinge formation, and at some hinge locations, local collapse may occur due to the plastic deformation limit being exceeded. The structure is pushed until global collapse, which is triggered when adequate number of plastic hinges forms to developed collapse mechanism or when the system cannot mobilize load paths to sustain force equilibrium (after local hinge break-downs). The main output of the pushover analysis is in the form of a force-displacement curve, called pushover curve. It is a plot of the base shear (total lateral load) versus the lateral displacement (drift) at some point at the roof level, including all the stages of lateral load/ displacement increments.

Pushover analysis results are obtained in terms of response demand versus capacity. If the demand curve intersects the capacity envelope near the elastic range, Figure 3a, then the structure has a good resistance. If the demand curve intersects the capacity curve with little reserve of strength and deformation capacity, Figure 3b, then it can be concluded that the structure will behave poorly during the imposed seismic excitation and need to be retrofitted to avoid future major damage or collapse



(a) Safe design (b) unsafe design

Figure3: Typical seismic demand versus capacity

III. DESCRIPTION OF FRAME STRUCTURE

A G+3 storied steel frame building model is developed using ETABS 2013

Steel -	:	Fe 250
No. of bays -	:	3
Size of Bay -	:	6*6m
Plan Area -	:	18m*18m.
Floor to Floor Height -	:	3.2m
Base story Height -	:	2.8m
Earthquake Zone -	:	II
Soil Type -	:	II medium soil

Importance factor -	:	1
Response reduction factor-	:	5

IS CODE 1893:2002 For Earthquake analysis

IS CODE 800:1998 for design.

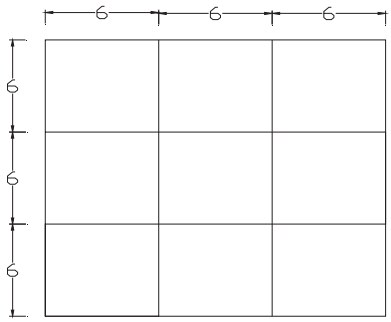


Figure4: Typical Plan

A. 2-d versus 3-d models:

Mwafy and Elnashai (2001), who were the first researchers exerting these approaches, acclaim that as a result of regularity of the frames, 2-D frames are apt to show clearly The behavior of the structure and exerting 3-D models are not needed as they decrease the analysis speed and also are time consuming. Also Maheri and Akbari (2003) have introduced their frame models in two dimensional forms, authenticating their style by quoting and referring to the previous reference. Also in this research two dimensional modelling has been considered in order to imitate the two above mentioned studies while extra verification and veracity has been brought by using and the implementation of FEMA 356 rules. This method has been considered since it has been widely used by researchers. According to the above mentioned rules of FEMA 2 dimensional models are allowed for structures, if the structure has the following 2 provisions:

1) If the diaphragms of the structure are rigid
2) If the effects of horizontal torsion have been taken into consideration in the model

Since these two conditions exist in the frames of this study, then the model 2-D of 2-D is selected rather than 3-D models and this is also in line with FEMA 356.

B. Design criteria:

IS 800:1998 is used as steel design code. IS 875 (part I-V) were used as loading codes. IS 1893:2002 part (II) used for earthquake loadings and load combinations.

C. Design software:

ETABS 9.7.2 and SAP 2000 V16.0.2 is used for this study since this software is widely used amongst Civil Engineers and also it is powerful enough to perform nonlinear static analysis as well as linear, static and dynamic analysis

D. Design sections:

ISMB sections are used for beams and Boxed Column sections are used for the columns and bracing members. This is in line with most of the practical works in India. It should be noted that I-sections (ISMB) for beams and rectangular hollow sections for braces. For a structure in which I sections are used for the columns, the horizontal loads (earthquake load) become more critical, if the columns are set in the direction where weak axis bending occurs. In order to consider this matter, while drawing the structure; rotate the columns 90 degrees so that the weaker axes are parallel to the lateral load direction. But in this study the cross-sections are square and therefore it is needless to rotate the columns as the results are identical.

E. Connections:

The connections of beam-column have been assumed to be pinned. Also the connections of braces were assumed to have pinned joint. The column base plate connections are fixed in order to reduce the column sections.

F. Loading:

Type of Load	Load in psf
Roofing and insulation	7
Metal Deck	3
Concrete fill	44
Ceiling & Mechanical	5
Steel Framing & fire proofing	8
TOTAL	67

Table No.1: Loading on roof

Total SDL on roof = 67*0.0478803 = 3.21 KN/m^2

Live Load = 1.5 KN/m^2

Type of Load	Load In psf
Metal Deck	3
Concrete fill	44
Ceiling & Mechanical	5
Steel Framing & fire proofing	13
Partitions UBC 1604.4	20
TOTAL	85

Table No. 2: Loading on floor

Total SDL on roof = 85*0.0478803 = 4.07 KN/m^2

Live Load = 3.2 KN/m^2

Load	For 3d frame	For 2d frame
SDL	On Roof 3.21	3.21*3 = 9.63
	On Floor 4.07	40.7*3 = 12.21
LIVE	On Roof 1.5	1.5*3 = 4.5
	On floor 3	3*3 = 9
WALL	On Roof 0.9 (UDL)	0.9 KN/m
	On floor 1.7 (UDL)	1.7 KN/m
SECONDARY BEAM WT.		On Roof 1.7 KN @ 1.5m C-C
		On floor 2 KN @ 1.5m C-C

Table No. 3: Loading on 2D frame

IV. PUSHOVER ANALYSIS

After designing steel frame frame structure, a nonlinear pushover analysis is carried out for evaluating the structural seismic response. The pushover analysis consists of the application of gravity loads and representative lateral load pattern. The lateral loads were applied monotonically in a step-by-step nonlinear static analysis. The applied lateral loads were accelerations in the x direction representing the forces that would be experienced by the structures when subjected to ground shaking. Under incrementally increasing loads some elements may yield sequentially. Consequently, at each event, the structure experiences a stiffness change as shown in figure 8, IO, LS and CP stand for immediate occupancy, life safety and collapse prevention respectively.

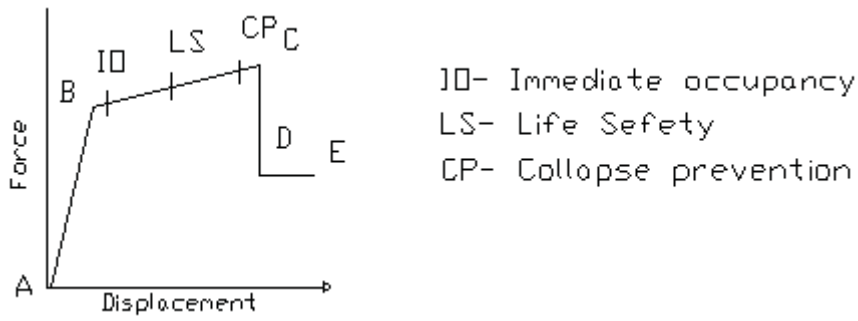


Figure 5: Force Deformation for pushover hinge

A. PLASTIC HINGE PROPERTIES

Comprehensive and complete information about plastic hinge properties of all of the structural segments are rendered by Federal Emergency Management agency in their Table that are fulfilled by engineers throughout the world. All the information relevant to this table is at disposal as default hinge properties in ETABS software.

B. COLUMN HINGE PROPERTIES

In accordance with FEMA 356, occurrence of a plastic hinge in a column is as a result of the interaction amongst axial force (P), moment in the stronger (M2) and weaker (M3) direction of the section. Therefore, interaction of P-M2-M3 is exerted to illustrate plastic hinges at the two ends of the columns (beginning and ending positions) that are in fact considered as the junction points with the other structural elements (Table 5-6 of FEMA 356). Here for 2D frame I have taken P-M_3 interaction.

C. BRACE HINGE PROPERTIES

Nonlinear behavior of brace elements can be best modelled by assuming a hinge (being made under pure axial load) in the middle of the element. An axial load plastic hinge is modelled in the 0.5 relative distances of all bracing elements as per Table 5-6 of FEMA 356 [Appendix] in this study.

D. BEAM HINGE PROPERTIES

Considering the fact that the beam to column connections is rigid, two plastic hinges (one at the beginning and the other one at the end) will be obtained. But for the beams that are braced with eccentric braces, the plastic hinges will occur at the place of fuses. For these kinds of beams the M3 and V2 are taken into consideration.

E. FAILURE CRITERIA

According to FEMA 356, decreasing of more than 20% or more decrease in the lateral force of the idealized pushover curve of the frame can be considered as a failure mode. This failure mode had also been considered by

other researchers, such as: Inel and Ozmen(2006) and ArashFarzam (2009) in their study. In this study, the same failure mode has been exerted.

V. RESULTS AND DISCUSSION

A. DESIGN RESULTS:

The process of frame modelling and their analysis were finished, and then they were designed according to IS 800:1998 and therefore the outcomes are rendered below for 4 and 8 story structures.

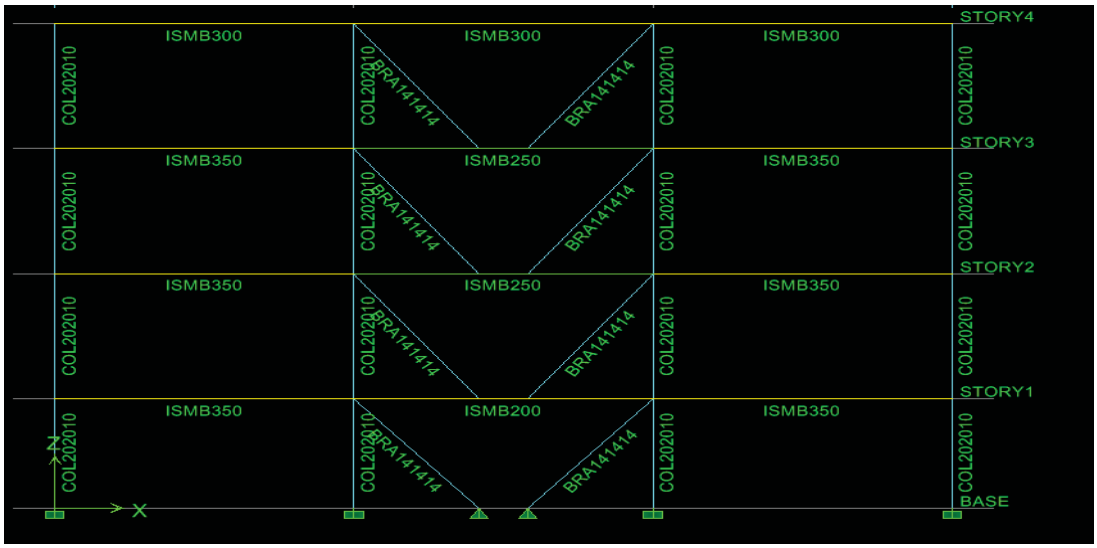


Figure 6: 4 story structure with Eccentric V bracing system

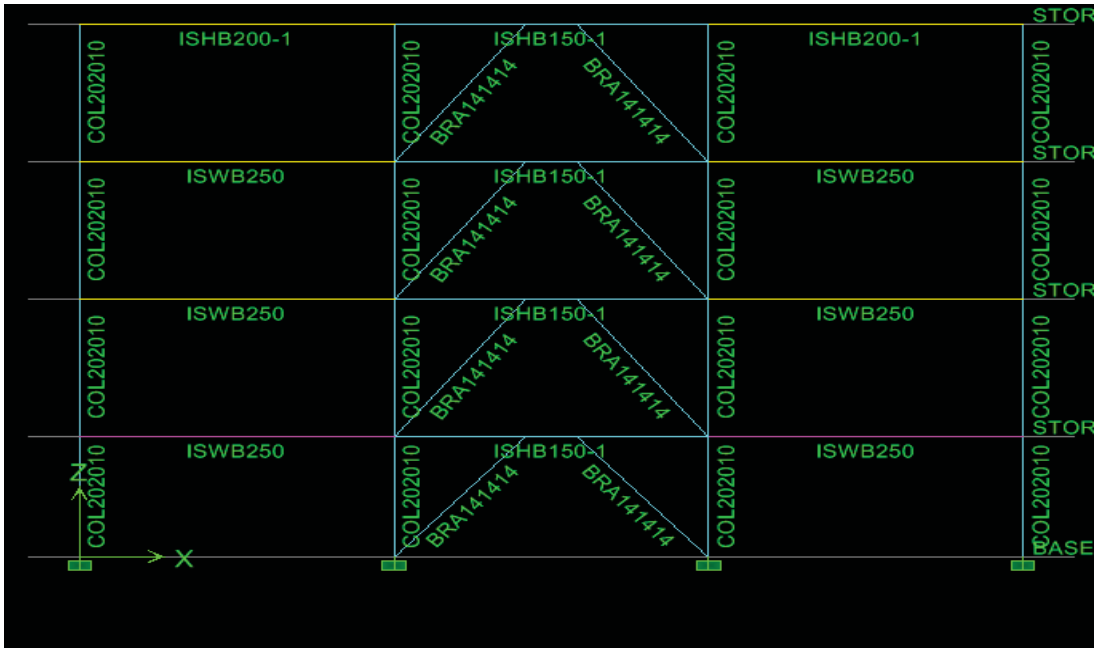


Figure 7: 4 story structures with Eccentric inverted _V bracing system

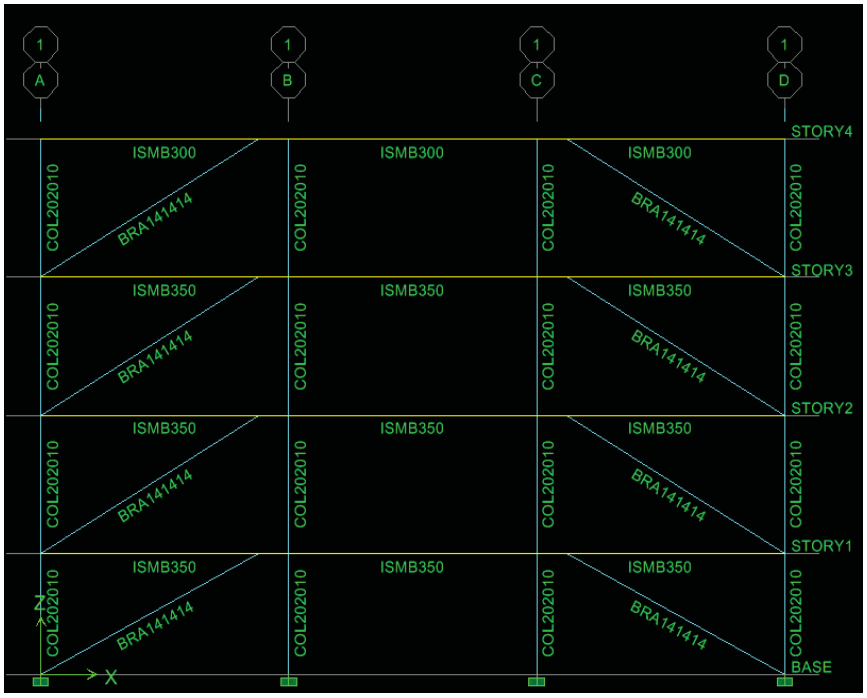


Figure 8: 4 story structure with Eccentric Diagonal bracing system

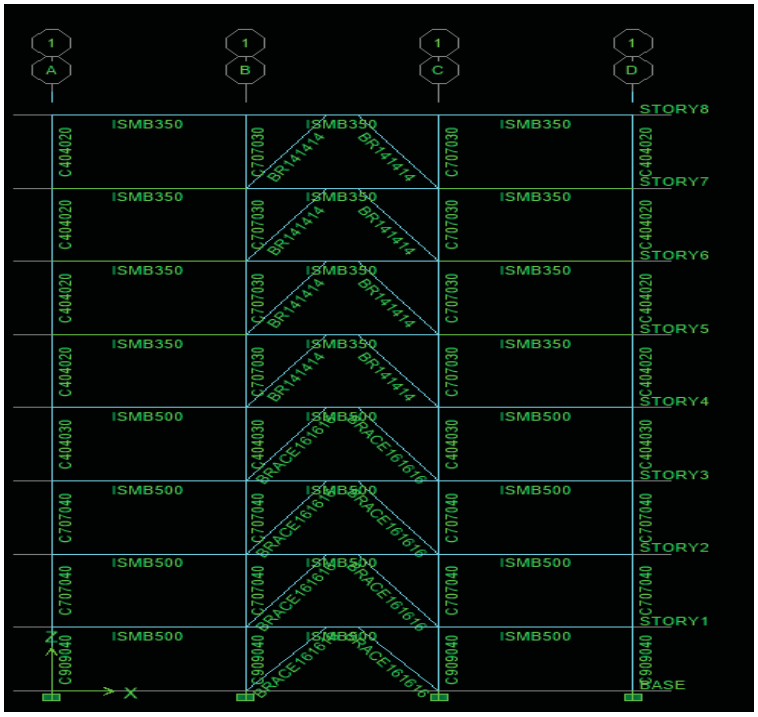


Figure 9: 8 story structures with Eccentric inverted _V bracing system

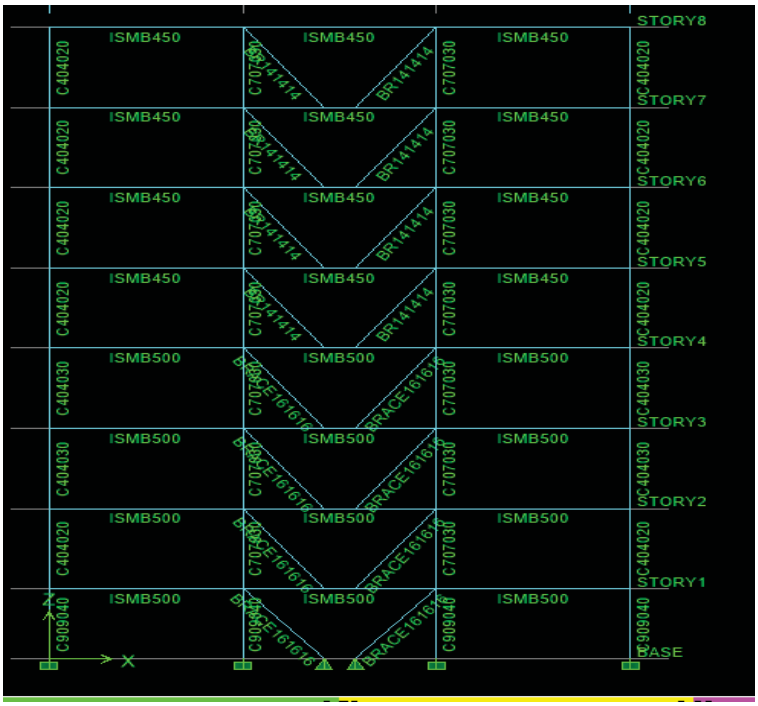


Figure 10: 8 story structures with Eccentric V bracing system

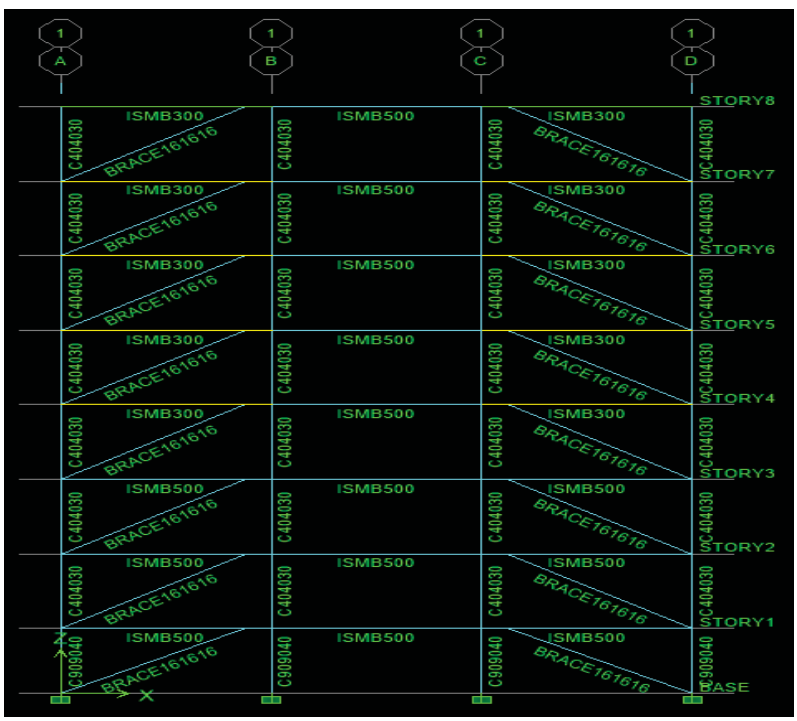


Figure 11: 8 story structure with Eccentric Diagonal bracing system

B. Base shear and displacement @ collapse level (4 story frame):

No. Of story	Frame type	Base shear	Displacement	B.s./displacement

4 STORY	INVERTED V BRACED	610.7139	0.2508	2435.0633
4 STORY	V BRACED	927.8524	0.2842	3264.7867
4 STORY	DIGONALY BRACED	839.6998	0.2325	3611.6120

Table No. 4

C. Base shear and displacement @ collapse level (8 story frame):

No. Of story	Frame type	Base shear	Displac -ement	B.s./displace-ment
8 STORY	INVERTED V BRACED	6831.5059	0.5694	11997.7272
8 STORY	V BRACED	4859.9907	0.1290	10077.3604
8 STORY	DIGONALY BRACED	5071.3325	0.4255	11918.5252

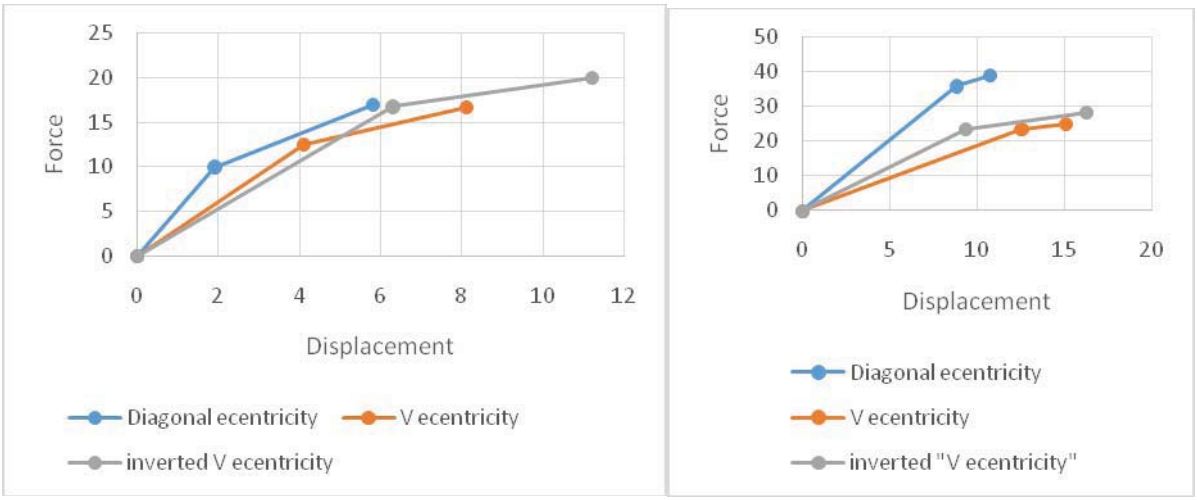
Table No. 5

D. Frame weight comparison:

Number of frames	Explanation	Total weight (ton)
1	4 story, V-braced	12.70
2	4 story, I-V-braced	15.43
3	4 story, Diagonal-braced	17.36
4	8 story, V-braced	58.55
5	8 story, I-V-braced	78.37
6	8 story, Diagonal-braced	78.75

Table No. 6

E. Comparison amongst idealized curvatures:



(a) Four story (b) 8 story

Figure 12: Comparison amongst 3 different kinds of bracing system

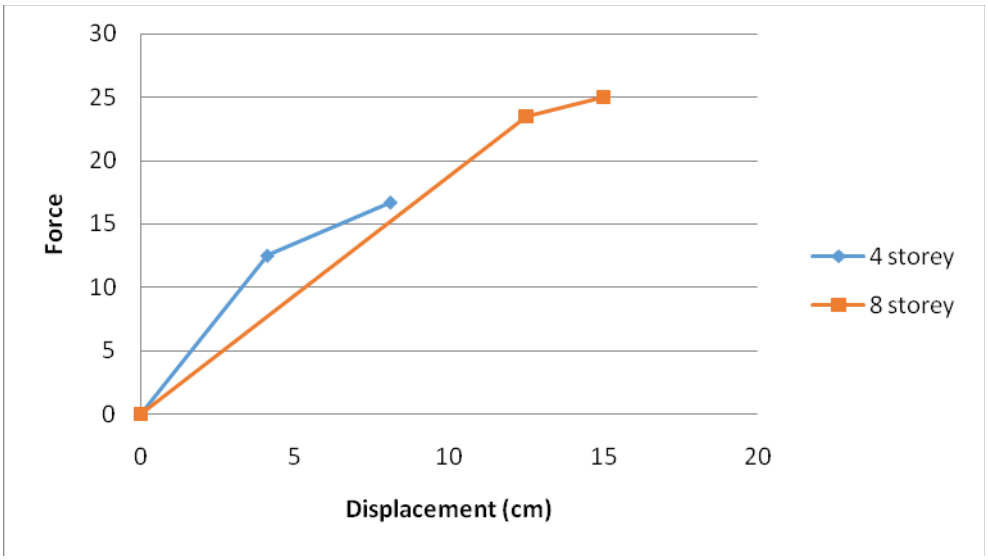


Figure 13: Comparison of eccentric -v-bracing system amongst the 2 differentheights of buildings (4 and 8 story)

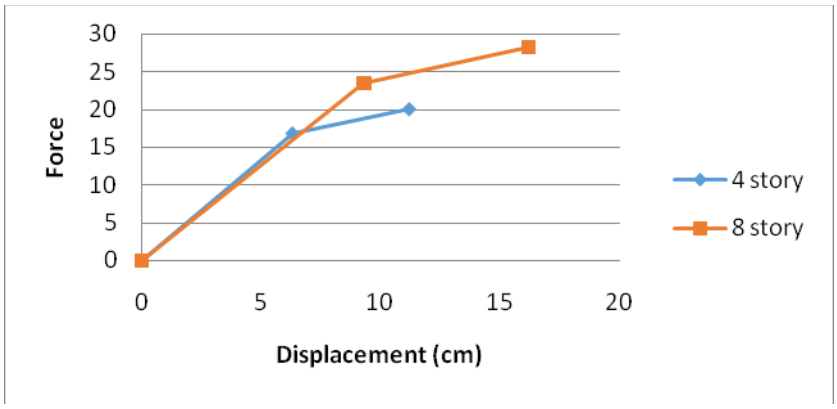


Figure 15: Comparison of eccentric inverted-v-bracing system amongst the 2 differentheights of buildings (4 and 8 story)

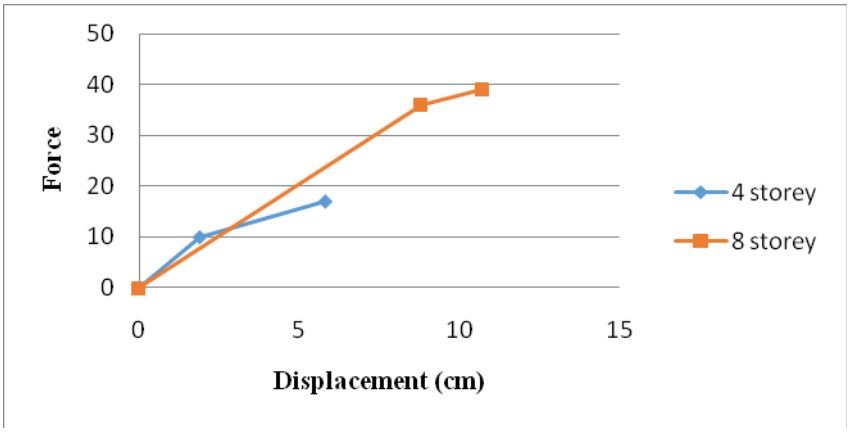


Figure 16: Comparison of diagonal bracing system amongst the 2 differentheights of buildings (4 and 8 story)

VI. CONCLUSIONS

The performance of eccentrically braced steel frames was investigated using the pushover Analysis. Based on the above results and observations the following conclusions are drawn.

1. Comparing the idealization curvature of frames with the same height but braced with different type of eccentric bracing system: In the 4 story models on the basis of initial slopes (in the idealization curvature) it can be observed that the eccentric diagonal bracing system has more stiffness in comparison with eccentric Inverted-V bracing system and meanwhile the stiffness of eccentric Inverted-V bracing system is more than eccentric V bracing system. Furthermore, considering the deduction which can be derived from the second part of the idealization curvature we can understand that the stiffness of the model structures still increases even after the occurrence of some plastic hinges, then again it can be noticed that the slope of the second part in idealization curvature relevant to the frame which is braced with eccentric Diagonal bracing system is more than that of eccentric Inverted-V bracing system and the slope of eccentric Inverted-V bracing system is more than that of eccentric V bracing system. According to the surface measurement of the lower part of Force-Displacement diagram that bodes and illustrates the energy absorption and dissipation, it can be inferred that the eccentric Diagonal bracing system has more capacity than Inverted-V bracing system and the also the eccentric Inverted-V bracing system has more capacity than eccentric V-bracing system.
2. Comparing amongst the performance of the frames with identical bracing system but at different heights (4-story, 8-story): With regard to the fact that by increasing the height of the structure then the degree of indetermination becomes more, and also as a result of this more plastic hinges will occur therefore it can be concluded that the model structure with 8-storey has more capacity than that of 4-storey

3. Comparison amongst braces with the same height but with different type of eccentric bracing system from the economical point of view:From the table it can be inferred that amongstamongst 4-storey frames, The V-braced structure is lightest and diagonal braced frame is the heaviest while the inverted V-braced is in between. So the V-braced frame is better from economical point of view.

   For 8_storey frames there is a slight difference between Inverted-V and Diagonal braced frames that are considered as heavy frames in this case, but the V braced frame is about 20 (ton) lighter than the others which could be more economical in this case.

   On the basis of the above mentioned reasons and discussions about the different frames in this study it can be concluded that amongst 4 story frames in accordance with performance and also from the economical point of view the Eccentric Diagonal bracing system is better because of its good performance. Although the Eccentric Diagonal braced frame in 4 story is the heaviest but the weight differences can be ignored for low stories (4 stories), as their weights are to some extent in the same range

   In 8 story frames the Eccentric V-braced frame has the lightest frame than the other two eccentric bracing systems. Although the weight of V-bracing system is low but because of its bad performance it is not selected as the best option here. Therefore in accordance with their performance, the Eccentric Inverted-V is the best.

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