Performance Based Seismic Analysis of Steel Frames

Srushti Bagal, Prashant Pawar, Vidyarani Kshirsagar, Avinash Kokare

Abstract*: The Civil Engineering profession has been changing the structural engineering design paradigm from life safety (LS) to Performance Based Seismic Design (PBSD) to tackle catastrophic damage caused by recent earthquakes worldwide. This paper is about the PBSD analysis of steel frame subjected to earthquake loading. Steel is by far the most versatile building material in the world and steel structure has played a major role in construction industry in the last decades. In this a multistoried bare and braced steel frames are analyzed by PBSD procedure in STAAD Pro Advanced following nonlinear static analysis. Frame components (beam, columns, etc.) are progressively adjusted to account for nonlinear elastic–plastic behavior under constant gravity loads and incrementally increasing lateral loads. Capacity curve is obtained for each frame and comparatively studied to decide which type of frame can meet the desired performance level during earthquake. The results of the analysis performed to meet required performance are presented in terms of displacement, shear forces, plastic hinges and capacity curve.

Keywords: Performance Based Seismic Design, Nonlinear Static Analysis, Steel Frames, STAAD Pro advanced

I. INTRODUCTION

Reference [1] says recent earthquake caused catastrophic damage in overall world. Steel structures are considered mostly earthquake resistant structure but some significant failures have occurred. Recent earthquake events demonstrate the necessity of change in structural design guidelines. To protect and maintain the economic activity and prosperity of a region, the performance of structure caused by earthquake became a major factor. That's why Civil Engineering profession is updating structural design paradigm of life safety (LS) to the performance bases seismic design (PBSD). Conventional seismic design approaches have the purpose of ensuring life safety (strength and ductility) and regulation of damage (drift limits for serviceability). The design parameters are specified by the stress limits and the strengths of the members determined from the prescribed lateral shear force.

Reference [2] says that the performance-based design is a more general design philosophy in which the design criteria are expressed in terms of achieving stated performance objectives when the structure is subjected to stated levels of seismic hazard. The performance targets may be a level of stress not to be exceeded, a load, a displacement, a limit state or a target damage state. Reference [3] and [4] says there have been different interpretations of what is meant by performance-based design. The most appropriate definition is that performance-based design refers to the methodology in which structural design criteria are expressed in terms of achieving a set of performance objectives.

Reference [9] using an appropriate structural system is critical to good seismic performance of the buildings. While moment frame is the most commonly used lateral load resisting structural system, other structural system are also commonly used such as braced system. A bracing is a system offered to reduce lateral structural deflection. Braced frame virtually eliminates bending factors for the column and girders and thus improve the efficiency of mere rigid frame behavior. Reference [5] already proved that braced frame decreases the displacement of the structure and absorbs more energy during earthquake. But the study does not comment on the effect of the position of the bracing on the structure. Considering this gap, in this study 3 frames are considered one is moment and remaining 2 are braced frame. In that braced frame one frame is externally braced as reference [10] concluded that external bracings perform well under lateral loads. Second frame is internally braced with optimum position as reference [14] was that adding braces to the core of building reduces the drift much more than adding them to the facades. Comparative study of three frames is presented in the study to demonstrate which structural design shows best performance under earthquake loadings.

II. NON-LINEAR STATIC ANALYSIS

Structural frames considered are analyzed in STAAD Pro advanced by nonlinear static analysis, popularly known as pushover analysis which is one type of PBSD. Reference [6] was that the nonlinear seismic analysis is used in structural Engineering profession to design steel frames for moderate to strong earthquakes. Reference [7] was that the linear procedures maintain the traditional use of a linear stress-strain relationship but incorporate material acceptance criteria to permit better consideration for probable non-linear characteristics of seismic response.

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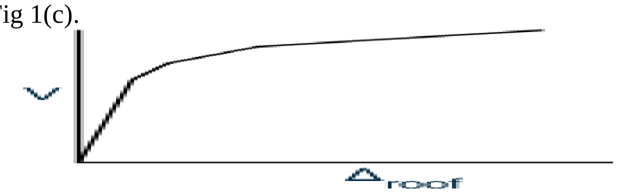
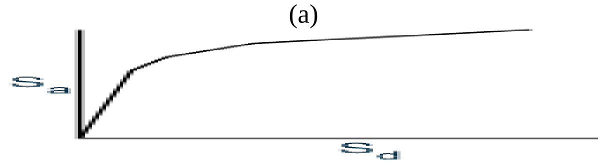
The non-linear static procedure, often called "pushover analysis," uses simplified nonlinear techniques to estimate seismic structural deformations. As per FEMA 356 reference [7], a pushover analysis is a static nonlinear way of estimating seismic structural deformations using a simplified, non-linear technique. Earthquake engineering research is progressing rapidly to consider the nature of buildings that have been exposed to powerful earthquakes. Pushover analysis is done to be able to predict such behavior. The overall capacity of a structure depends on the strength and deformation capacities of the structure's individual components. Reference [11] was to evaluate capacities beyond the elastic limit some form of nonlinear analysis is needed, such as Pushover Analysis. It is a modern performance based seismic design (PBSD) for analytically achieving a structural design that will work reliably under one or more seismic conditions in a specified manner. There are two nonlinear procedures using pushover methods: a. Capacity Spectrum Method b. Displacement Coefficient Method. In this analysis particularly Capacity Spectrum Method is used.

A. Capacity Spectrum Method

Reference [12] was the Capacity Spectrum Method's goal is to establish suitable demand and capacity spectra for the system and to determine its intersection point. During this process, performance of each structural component is also evaluated. The spectrum of capacity is obtained by converting the base shear versus the spectrum of roof displacement into a spectral acceleration versus the spectral displacement as shown in Fig 1(a). The intersection between a corresponding demand curve and the capacity curve is called the performance point.Capacity curve, in terms of base shear and roof displacement, is converted to capacity spectrum, which is a representation of the capacity curve in Acceleration Displacement Response Spectra (ADRS) format (i.e., Sa versus Sd) as shown in Fig 1(b). This curve is obtained by redrawing the design earthquake response spectra as a curve of spectral acceleration v/s spectral displacement as shown in Fig 1(c).



(a)



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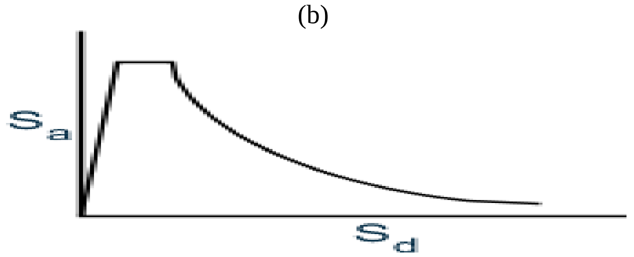


(c)

Fig.1. Curves in capacity spectrum method: (a) Roof deflection, Δroof, plotted versus base shear, V; (b) Spectral displacement, Sd plotted versus spectral acceleration, Sa; (c) Response spectrum

B. Performance Level

It is important to choose performance standards which are ac ceptable to all parties concerned. Reference [7] was that, there are three performance levels now being considered for the seismic risk assessment of steel structures. They are collapse prevention (CP), life safety (LS) and immediate occupancy (IO) of structure. Collapse prevention reflects a level of performance of significant structural damage which can cause collapse. Clearly at this level of damage a building will be unusable. Life safety is a state of significant structural damage; certain component of structure can collapse, and structure must be repaired before reoccupation. The quality of IO efficiency is distinguished by a structure that is essentially undamaged, so the structure can be instantly used. Reference [13] was to know the performance of the building we need to know the performance point (PP). Performance point indicates the damage state for which building is to be designed. The displacement at PP is the target displacement also called design displacement.

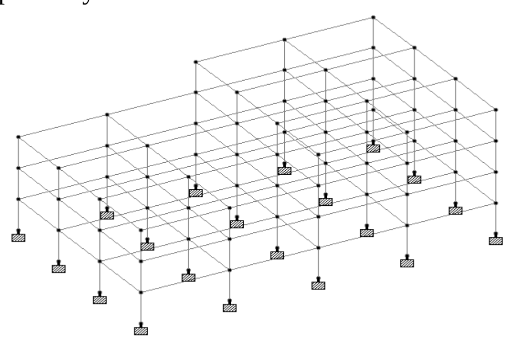
If$\Delta pp < \Delta IO$, it implies IO building.

$\Delta pp > \Delta IO$&<, LS building.

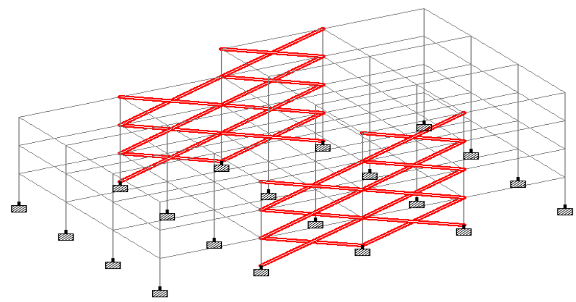
$\Delta pp > \Delta LS$&< ΔCP, CP building.

C. Structural Modeling:

Three structural steel frames of G+3 storey (i) moment frame, (ii) braced frame with external concentric diagonal bracing(bracing section –ISMC100) and (iii) braced frame with internal diagonal bracing (bracing section –ISMC100) at optimal position are considered for the study with same geometry of beam and column as shown in fig 2 (a), (b) and (c) respectively.



(a)



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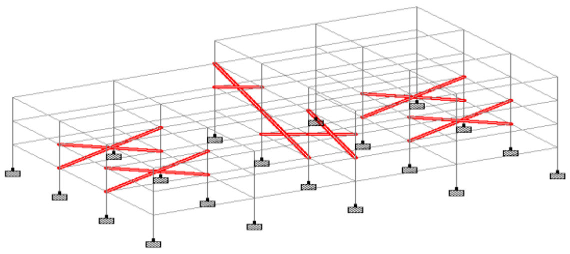

(c)

Fig.2. Structural Modeling of Steel Frames in STAAD Pro. Advanced, (a) moment frame; (b) braced frame with external concentric diagonal bracing; (c) braced frame with internal diagonal bracing.

Table 1 shows frame geometry for all three structural steel frames and Table 2 shows the cross sectional details of beam and columns used in all three frames. While assigning steel sections to column and beam, strong column weak beam concept is taken into consideration. The properties of steel used for the construction of 4-Storey braced frame are; modulus of elasticity is 199947.9611 N/mm2; Poison's ratio is 300E-3; Density is 283E6 kip/in3; ὰ/ºF is 6.5E6

Table- I: Geometry of Braced Steel Frame

Floor	Height (m)	Length (m)	Width (m)
Ground Floor	3.048	28.48	18.28
1st Floor	3.048	28.48	18.28
2nd Floor	3.048	28.48	18.28
3rd Floor	3.048	14.124	18.28

Table- II: Size of Steel Cross Section Details for Existing 4-Storey Braced Steel Frame

Model	Storey/ Floor	Column		Beam	
		Exterior	Interior	Exterior	Interior
4-Storey	4	ISHB350H	LSLB450	ISHB200	ISHB200
	3	ISHB350H	LSLB450	ISHB200	ISHB200
	2	ISHB350H	LSLB450	ISHB200	ISHB200
	1	ISHB350H	LSLB450	ISHB200	ISLB300

On each frame respective self weight and live load of 3kN/m assigned as shown in fig 3. Self weight and live loads assigned under the gravity load conditions to perform the pushover analysis in STAAD Pro. advanced. In fig 3 red color of entire structure shows the self weight of structure and green colored arrows in downward direction shows the gravity load of 3Kn/m acting in global Y direction.

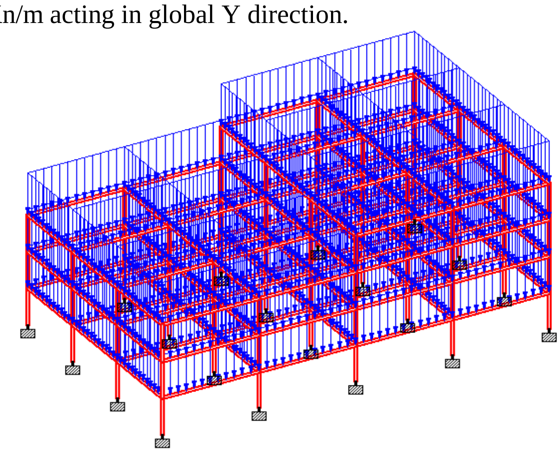

Fig.3. Gravity Loading on Structural Steel Frame for Pushover Analysis

III. PERFORMING PUSHOVER ANALYSIS

Pushover analysis on each structural steel frame of G+3 storey is performed in STAAD Pro. advanced. Following steps were done while performing non-linear static analysis.

1. **Defining Type of Frame:** While performing pushover analysis in STAAD Pro. Advanced firstly type of the frame should defined. For the first frame, frame type is defined as moment frame and for second and third frame, frame type is braced frame.

Geometric Non-linearity: Some structural damage is allowed during strong earthquake shaking in normal buildings, even though no collapse must be ensured. This implies that nonlinearity will arise in the overall response of building. Hence the geometric non linearity is considered while analyzing the all three steel frames**.** Convergence of geometric non linearity is taken as 0.254mm and the numbers of iterations performed for geometric nonlinearity are 50.

2. **Defining Loads:** Loads are defined under gravity loading case. Gravity loads include dead loads and (typically) most live loads. Live load of 3kN/m is given to each steel frame as shown is earlier fig 3.

Defining Loading Pattern: In this step base shear is defined up till which pushover analysis will be performed. Defined base shear is more than the designed base shear. Here design base shear is 933.33KN and it is calculated by using dynamic response spectrum analysis in STAAD Pro and the defined base shear is more than this. Because design base shear excludes non linear effect. When the structure undergoes a strong earthquake, the actual base shear may be very high compared to the base shear design. To distribute base shear vertically method 3 section 3.3.3.2.3 of FEMA 356 reference [8] is used. Incremental value of base shear is taken as 5kN for multiple steps output result. Number of push loads defined are 250.

3. **Defining Spectrum Details:** Critical damping of 5.00% is assigned to all three frames. Site class category considered is D of FEMA 356 section 1.6.1.4.1 i.e. hard rock with average shear wave velocity, v_s > 5,000 ft/sec is considered as per the location of structure to generate demand spectrum.

Defining acceptance criteria: Reference [8] used to define performance parameters in which all elements are considered as primary elements. Hence performance points are as shown on curves of figure 4. IO is the deformation at which permanent, visible damage occurred in the experiment but not greater than 0.67 times the deformation limit for LS. LS is 0.75 the deformation at point 2 on the curves. CP is the deformation at point 2 on the curves but not greater than 0.75 times the deformation at point 3.

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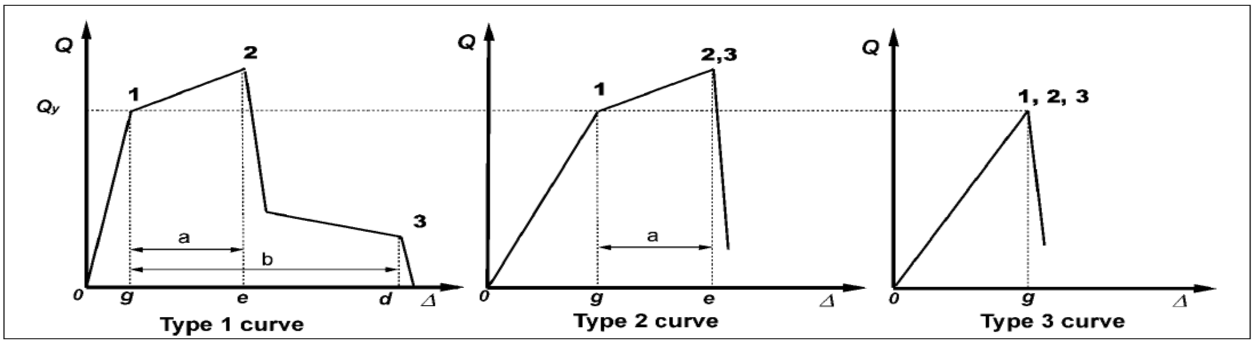

Fig.4. Fig 4 Component force v/s deformation curves from FEMA 356

Defining Solution Control: Analysis can be done either by defined base shear or by defined displacement at controlled joint. Here push up to defined base shear approach is used as earlier discussed in step 4 of performing pushover analysis.

Performance Check: Performance of all three G+3 structural steel frame obtained by performing pushover analysis and comparative results of base shear, displacement, capacity curve and plastic hinges are computed to find out the which structure meets the required performance under earthquake loading.

IV. RESULT AND DISCUSSION

Performance of Moment Frame: After performing pushover analysis on the G+3 storey steel frame, frame performed linearly up to the base shear of 135.09KN then after it started performing nonlinearly as base shear increased. First plastic hinge shown by green is developed in column 9 and column 12 as shown in figure 5. Performance level at that point is IO. Further push load steps carried out, when base shear is increase up to 3758.82 KN blue colored plastic hinges developed at a column 5, 8, 13 and 16 as shown in fig 6 which shows the structure lies in between IO – LS performance level. When base shear reached the value of 4218KN pink colored plastic hinges started developing into the column 5, 8, 13 and 16 as shown in fig 7 which implies that the those structural components lies in LS-CP performance level. At the base shear 4392.60 KN red colored plastic hinges started developing into the column 5, 8, 13 and 16 as shown in fig 8 which implies that the those structural components are in CP level. As the member of structure comes into the CP performance level base shear started redistributing to check the performance of other structural elements. Till that point no member were collapsed. First member column 5 and 8 failed at the base shear 4260.87KN and they are indicted by red color. But entire structure was not failed at those points as the maximum columns lies into the IO performance level as shown in fig. After distributing and redistributing base shear up to the push load step 173 maximum number of beams and columns are into the CP level and LS level while some of them are collapsed as shown in fig when the redistributed base shear was 2380.60 KN. After that point entire structure will fail as maximum number of columns from base storey was failed as shown in fig 9. Capacity curve for moment frame is as shown in fig 10 in which X-axis indicates the displacement at roof due to base shear indicated on Y- axis.

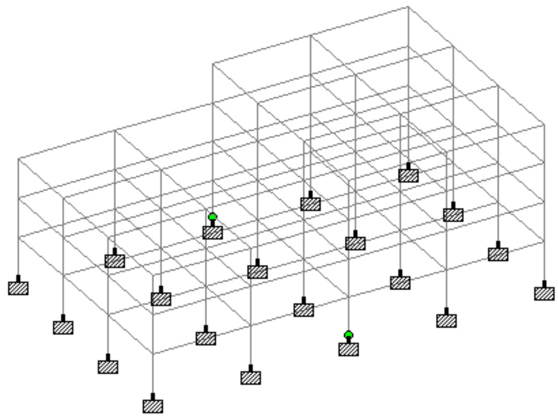

Fig.5. Members of Moment Steel Frame in IO Performance Level

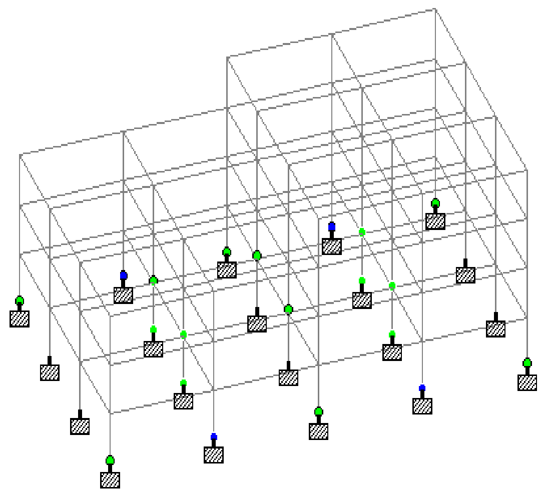

Fig.6. Members of Moment Steel Frame in IO-LS Performance Level

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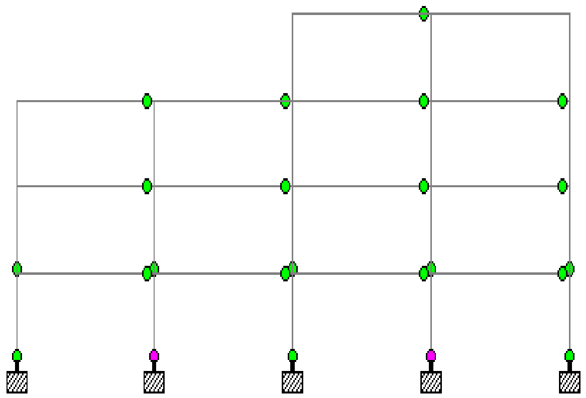

Fig.7. Members of Moment Steel Frame in LS-CP

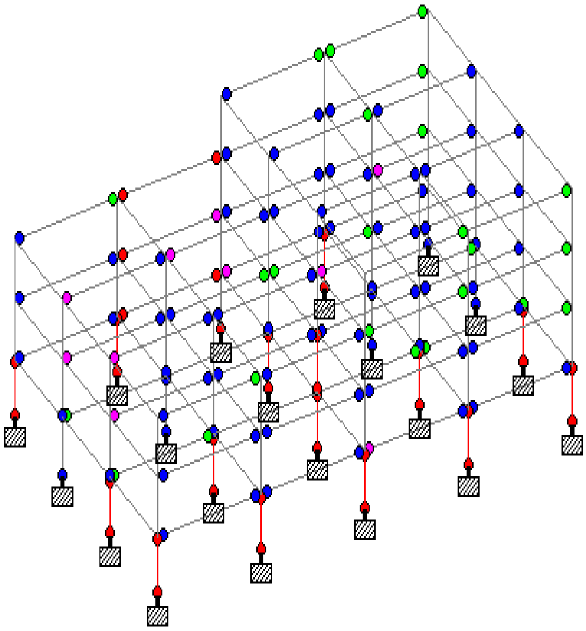

Fig.9. Members of Moment Steel Frame Failed in Pushover Analysis

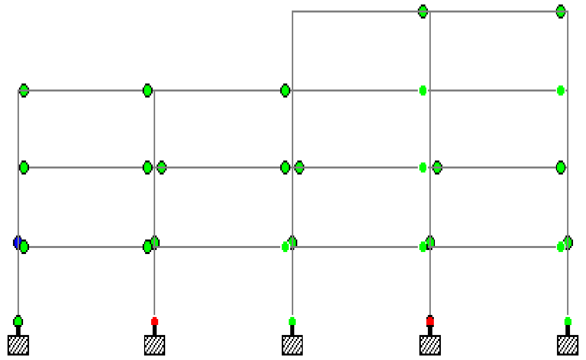

Fig.8. Members of Moment Steel Frame in CP

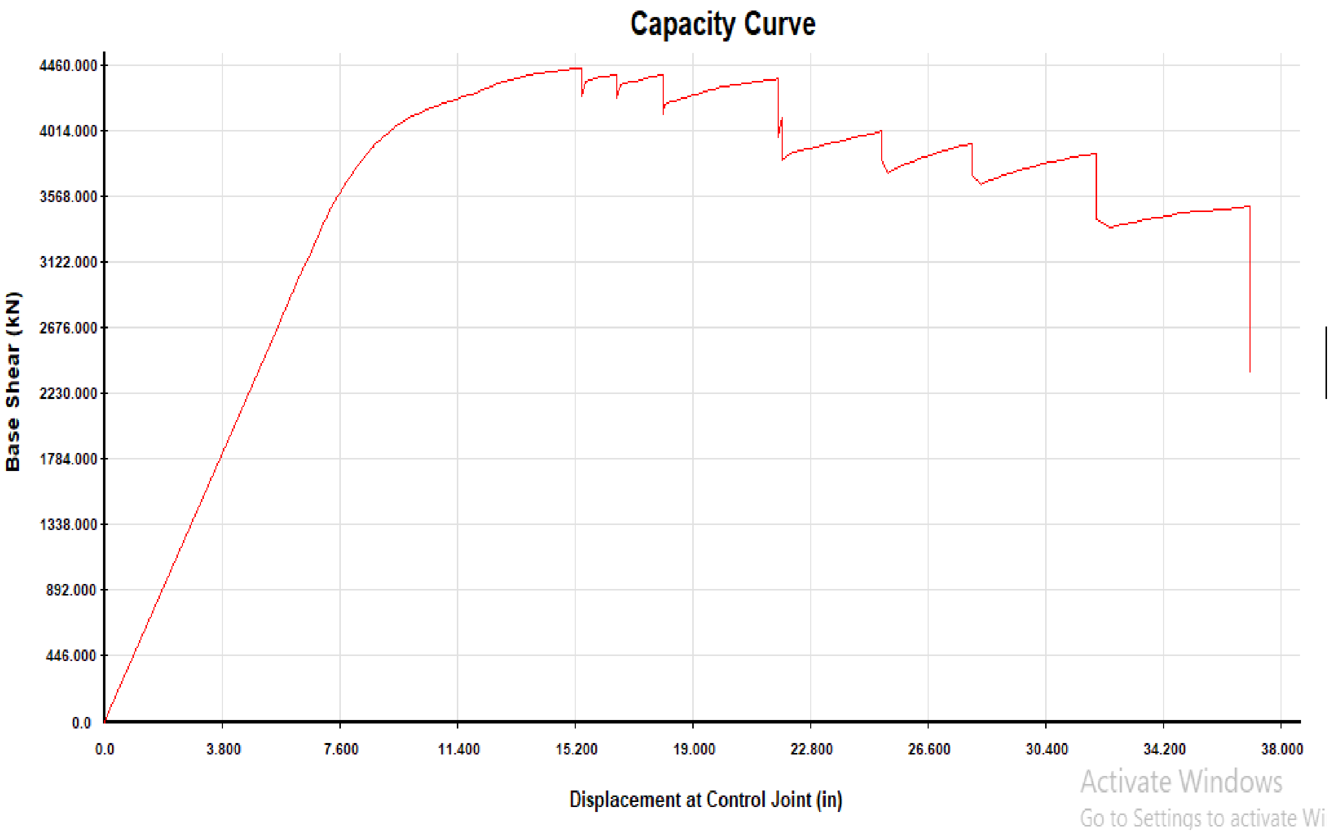

Fig.10. Capacity Curve of Moment Frame

1. **Performance of Externally Braced Frame**: G+3 storey frame with analyzed by static nonlinear process, frame performed linearly up to the base shear 137.70KN. When base shear is 3879.86KN column 6,7,14 and 15 are in IO performance level as green colored plastic hinges developed in it as shown in fig11. When base shear reached the value 5161.37KN column10 and 11 is in IO – LS performance level as shown in fig 12 column 10 and 11 reached LS-CP performance level at base shear 5406.2794KN and in complete CP level when base shear 5545.04KN as shown in fig 13. Bracing provided started failing when base shear 5636.119KN as shown in fig 14.It is observed that due to external bracings lateral load carrying capacity of structure is increased but displacement is also more which laid to failure of structure. After that base shear redistributed up to the push load stem 231 and the Maximum columns of basement were failed at base shear 6039.11KN. After which entire structure will collapses. Capacity curve obtained for this frame is as shown in fig 15.

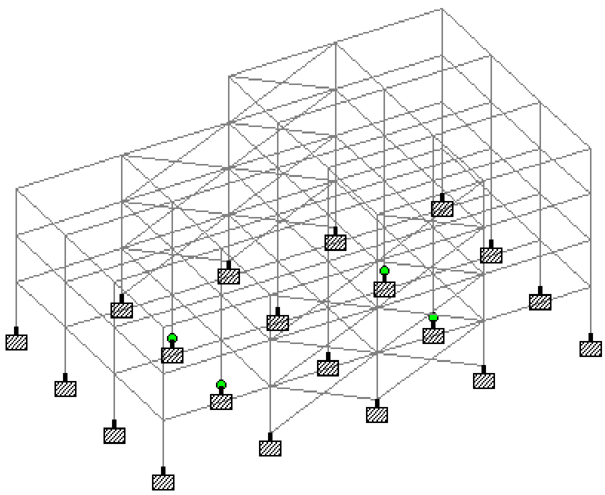

Fig.11. Members of Externally Braced Steel Frame in IO

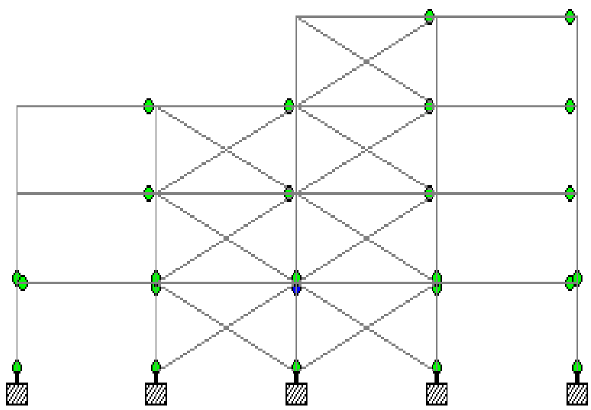

Fig.12. Members of Externally Braced Steel Frame in IO-LS

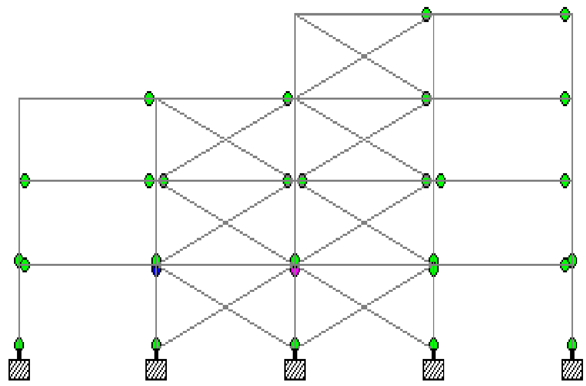

Fig.13. Members of Externally Braced Steel Frame in LS-CP

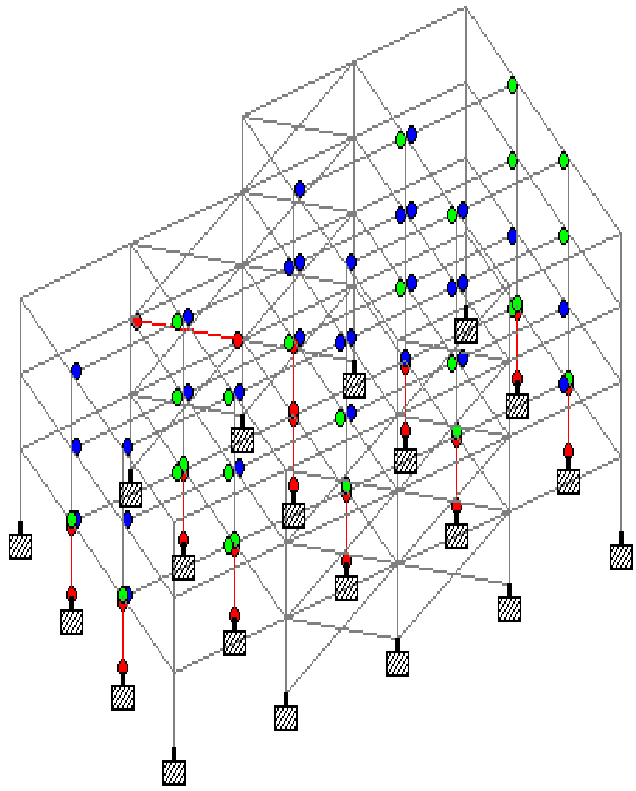

Fig.14. Members of Externally Braced Steel Frame Failed in Pushover Analysis

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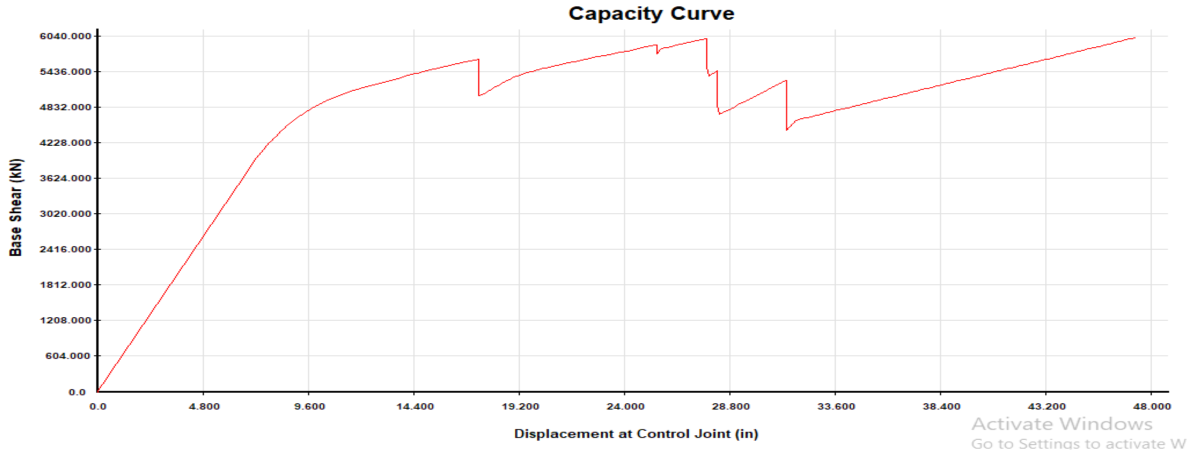

Fig.15. Capacity Curve for Externally Braced Steel Frame

2. **Performance of Internally Braced Frame at Optimal Position**: After the results of above two frames third frame is designed in such way that internal bracings were provided at position to avoid the failure of members observed in first and second frame case. For that purpose different position of the bracing tried and the frame represented in this paper with the optimal position of bracing is with the most accurate collapse prevention results. Column 5,8,912,13 and 16 in G+3 internally braced steel frame is in IO performance level ate base shear 3536KN as shown in fig 16. Column 27 and 31reched the IO-LS performance level when base shear is 4517.66 KN as shown in fig 17. Column 10 and 11 as shown in fig 18 are I LS-CP level when base shear 5128.46KN. Same columns reached CP level first in entire structure at base shear 5222.86KN. In this design, it is observed that base shear carrying capacity of structure is increased but displacement of structure is less as compare to externally braced frame. Hence structure prevented from collapse. After completing all push load step it is observed that no member failed only column 10 and 11 are in CP level but structure is safe on remaining columns. Capacity curve is as shown in fig 19.

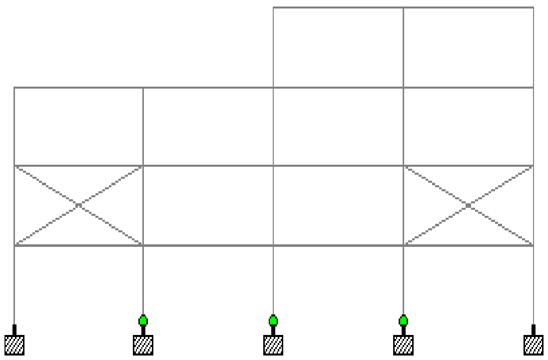

Fig.16. Members of Internally Braced Steel Frame in IO

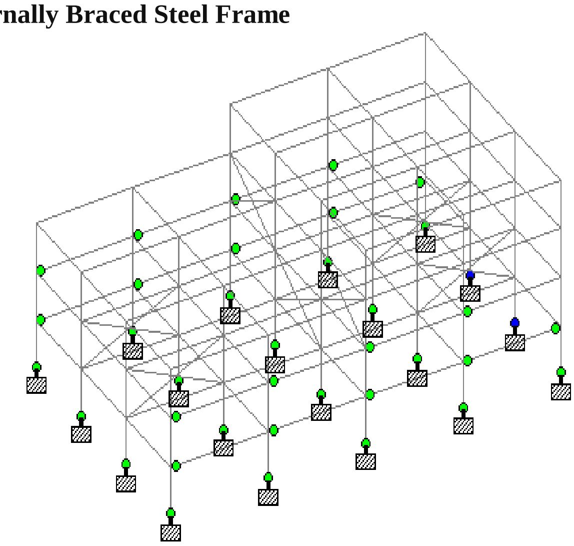

Fig.17. Members of Externally Braced Steel Frame in IO-LS

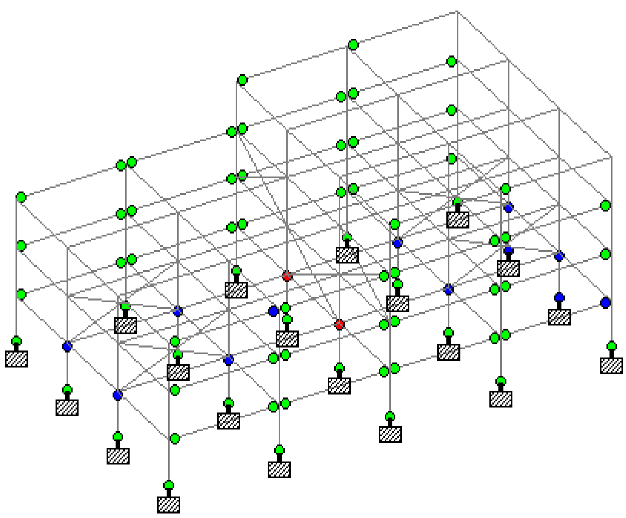

Fig.18. **Members of Externally Braced Steel Frame in LS-CP Performance Level**

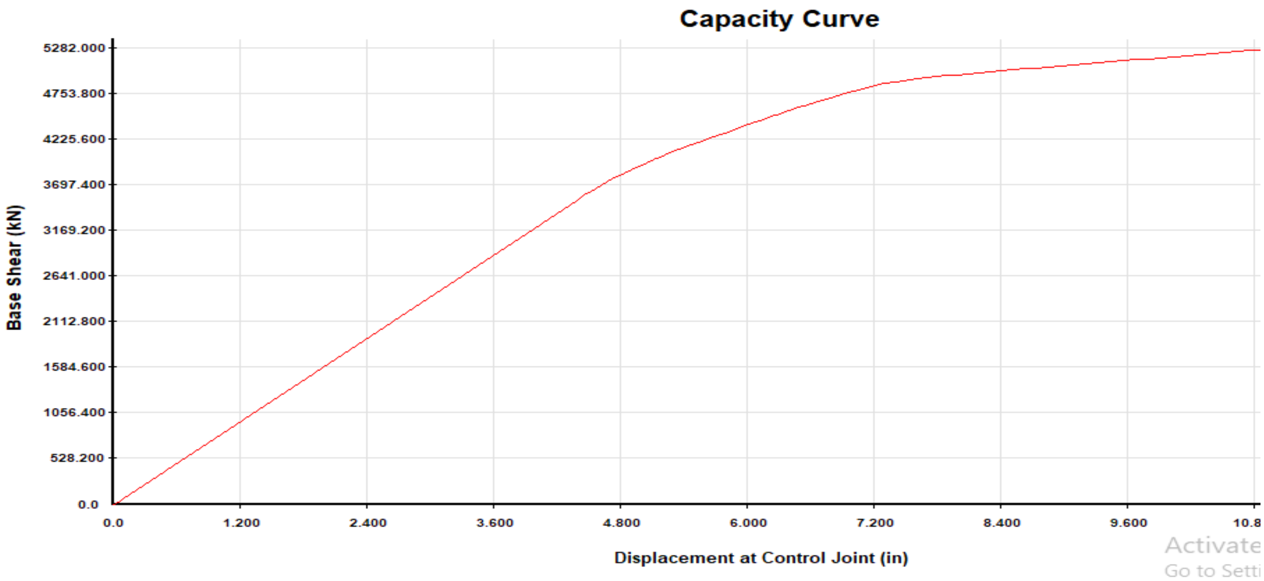

Fig.19. Capacity Curve for Externally Braced Steel FramE

V. CONCLUSION

This paper presented and documented performance based seismic analysis for steel frames. The concept of performance based seismic design was successfully implemented by nonlinear static analysis by applying incremental lateral loads on braced and non braced steel frames. The performance criteria suggested by FEMA 356 can be successfully implemented in PBSD pushover analysis method by using STAAD Pro. Advanced. Maximum members of moment frame reaches to Collapse prevention level and ultimately fails under the incremental push loads. This leads the collapse of entire steel frame during the earthquake. The Shear capacity of the structure can be increased by introducing external steel bracings in the structural system. But under the incremental lateral loads bracing also fail. This leads to the maximum members to be in CP level and causes failure of structural members during earthquake. To avoid this position of the bracing can be optimized by using pushover analysis by identifying which members are failing after incremental lateral load and identifying the position of bracing which prevents the failure of these members. Such optimal position of bracing saves the structure during earthquake. It is concluded in this paper that such braced steel frame at optimal position increases the shear capacity of structure and performs well, maximum in LS level. No collapse of member is observed in this frame after incremental lateral loads. Pushover analysis is successfully implemented to study non linear behavior of structure under earthquake loading.

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Publications: 04 Conferences/workshops: 10