

Seismic Performance Evaluation of Steel Moment Resisting Frames

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ABSTRACT: It is a well-known fact that multistoried buildings attract large lateral forces from wind and earthquake. Forces that are caused due to earthquake may lead to severe damage on structural elements resulting in collapse particularly in seismically active areas. In steel structures, the lateral resistance is achieved by means of providing special moment resisting frames or braced frames. The assessment of the seismic vulnerability of structures is a very complex issue due to the non-deterministic characteristics of the seismic action. Hence, there is a need for an accurate evaluation of the seismic responses beyond the level of linear behavior. Pushover analysis is one of the method to evaluate the seismic response of buildings. In this paper, a pushover analysis is carried out to study the performance of steel moment resisting frame designed as per IS 800:2007. For this, a two bay ten storied moment resisting frame is designed for forces/loads according to IS 875 and IS 1893: 2002 (seismic zone IV). In push over analysis, the plastic hinge formation is considered according to FEMA 356. The results of the present investigation are reported in terms of capacity curve, sequence of formation of plastic hinges, demand curve and performance point.

KEYWORDS: Pushover analysis, Capacity curve, Demand curve, steel moment resisting frame, performance point

I. INTRODUCTION

In present scenario steel structure plays an important role in the construction industry. It is necessary to design a structure to perform well under seismic loads. The Northridge earthquake (1994) has exposed significant problems in steel moment resisting frame structures with welded connections. Fractures were detected in and around full penetration welds connecting beam flanges to column flanges. Even though these fractures did not lead to collapses of buildings, they raise concerns about the seismic safety of steel frame structures and may necessitate a performance evaluation of the structures. Several earthquakes that caused in India leads to major damage to structure and loss of life. Various studies has been made in major earthquake caused in India. Experience gained from Bhuj earthquake 2001 demonstrates that the most of buildings collapsed were found deficient to meet out the requirements of the present day codes. Indian codes of practice for earthquake resistant design (IS: 1893) are revised periodically. Assessing the capacity of building as per the requirement of current codes of practice is an important task. In order to check the performance of the building as against the demand of present seismic code IS: 1893 – 2002, nonlinear static analyses are carried out to generate the capacity curve using finite element software, SAP2000. Various analysis methods, both elastic (linear) and inelastic (non-linear), are available for the analysis of steel buildings. Elastic analysis method include code static lateral force procedures, code dynamic lateral force procedures using demand capacity ratios.

The most basic inelastic analysis method is the non-linear time history analysis, which at present is considered overly complex and impractical for general use. The use of the nonlinear static analysis (pushover analysis) came in to practice in 1970's but the potential of the pushover analysis has been recognized for last two decades. The use of inelastic procedures for design and evaluation is an attempt for better understanding of how structures will behave when subjected to major earthquakes where it is assumed that the elastic capacity of the structure will be exceeded. Pushover analysis is an inelastic static analysis method of analyzing a structure using a specified (constant or variable) force pattern from zero load to a prescribed ultimate displacement. This procedure is mainly used to estimate the strength and drift capacity of existing structure and the seismic demand for the structure subjected to selected

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earthquake. The non-linear static analysis is carried out using the details provided in the present Codal provisions. This paper highlights the performance evaluation of a two bay ten storied moment resisting frame subjected to seismic loads which is designed as per IS 1893:2002 for zone IV by determining the capacity curve, demand curve and performance point. The effect of hinges is used to study the deformation controlled action. In this present study, a model is designed by incorporating all the conditions that are given in Indian Codal Provisions by using SAP 2000. The process is continued until a control displacement at the top of building reaches a certain level of deformation or structure becomes unstable. The roof displacement is plotted with base shear to get the global capacity curve.

Hasan et al. [1] conducted a simple computer-based push-over analysis technique for performance-based design of building frameworks subject to earthquake loading. And found that rigidity-factor for elastic analysis of semi-rigid frames, and the stiffness properties for semi-rigid analysis are directly adopted for push-over analysis. Ghobarah A. et al.[2] mentioned that the control of inter story drift can also be considered as a means to provide uniform ductility over the stories of the building. A story drift may result in the occurrence of a weak story that may cause catastrophic building collapse in a seismic event. Uniform story ductility over all stories for a building is usually desired in seismic design. R. B. AKBAS et al. [3] conducted a push over analysis on steel frames to estimate the seismic demands at different performance levels, which requires the consideration of inelastic behavior of the structure. X.-K. Zou et al. [4] presented an effective technique that incorporates Pushover Analysis together with numerical optimization procedures to automate the Pushover drift performance design of reinforced concrete buildings. PBD using nonlinear pushover analysis, which generally involves tedious computational effort, is highly iterative process needed to meet code requirements. Mehmet et al. [5] explained that due to its simplicity of Push over analysis, the structural engineering profession has been using the nonlinear static procedure or pushover analysis. Pushover analysis is carried out for different nonlinear hinge properties available in some programs based on the FEMA-356 and ATC-40 guidelines and he pointed out that Plastic hinge length (L_p) has considerable effects on the displacement capacity of the frames. The orientation and the axial load level of the columns cannot be taken into account properly by the default-hinge properties. Girgin et al.[6], has preferred the method for seismic performance evaluation of structures by the major rehabilitation guidelines and codes because it is computationally and conceptually simple. Pushover analysis allows tracing the sequence of yielding and failure on member and structural level as well as the progress of overall capacity curve of the structure. From the literature, it is found that the behaviour of the frame subjected to seismic loads is well influenced by the plastic hinge formation, capacity curve and demand curve which is determined by using SAP 2000.

In the present paper, an attempt has been made to evaluate the performance of two bay ten storied moment resisting frame depending on the capacity curve, demand curve and the hinges obtained from the pushover analysis. The plastic hinge formation is considered according to FEMA 356.

II. PUSHOVER ANALYSIS

The recent advent of performance based design has brought the nonlinear static pushover analysis procedure to the forefront. 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 found. The loading is monotonic with the effects of the cyclic behaviour and load reversals being estimated by using a modified monotonic force-deformation criteria and with damping approximations. Pushover analysis may be classified as displacement controlled pushover analysis when lateral displacement is imposed on the structure and its equilibrium determines the forces. Similarly, when lateral forces are imposed, the analysis is termed as force-controlled pushover analysis. Response of structure beyond maximum strength can be determined only by displacement controlled pushover analysis. Hence, in the present study, displacement-controlled pushover method is used for analysis of structural steel moment resisting frames. The ATC-40 and FEMA-356 documents have developed modelling procedures, acceptance criteria and analysis procedures for pushover analysis. These documents define force-deformation criteria for hinges used in pushover analysis. As shown in Figure 1, five points labelled A, B, C, D, and E are used to define the force deflection behaviour of the hinge and three points labelled IO, LS and CP are used to define the acceptance criteria for the hinge. (IO, LS and CP stand for Immediate Occupancy, Life Safety and Collapse Prevention respectively.) The values assigned to each of these points

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vary depending on the type of member as well as many other parameters defined in the ATC-40 and FEMA-356 documents. The description for structural performance level definition is given in Table 1.

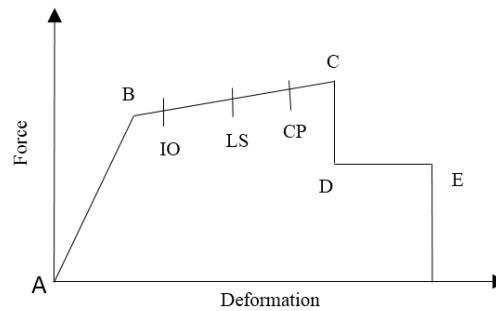


Fig.1 Load –Deformation curve

Table 1. Structural Performance Levels Definition

Performance level		Description
Immediate Occupancy (IO)	Operational	No significant damage has occurred to structure, which retains nearly all its pre-earthquake strength and stiffness. Non-structural elements are secure and most would function, if utilities were available. Building may be used for intended purpose, albeit in an impaired mode.
Life Safety	Life safe	Significant damage to structural elements with substantial reduction in stiffness, however margin remains against collapse. Non-structural elements are secured but may not function. Occupancy may be prevented until repairs can be instituted.
Collapse Prevention	near collapse	Substantial structural and non-structural damage. Structural strength and stiffness substantially degraded. Little margin against collapse. Some falling debris hazards may have occurred.

2.1 Important features of performance evaluation of building

2.1.1 Capacity

The overall capacity of a structure depends on the strength and deformation capacities of the individual components of the structure. The mathematical model of the structure is modified to account for reduced resistance of yielding components. A lateral force distribution is again applied until predetermined limit is reached. Pushover capacity curves approximate how structure behaves after exceeding the elastic limits. The capacity curve is shown in Fig 2.

2.1.2 Demand

Ground motions during an earthquake produce complex horizontal displacement patterns in structure that may vary with time. For nonlinear method it is easier and more direct to use a set of lateral displacement as a design condition for a given structure and ground motion, the displacement is an estimate of the maximum expected response of the building during ground motion. The demand curve is shown in Fig 3.

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2.1.3 Performance point

Performance point can be obtained by superimposing capacity spectrum and demand spectrum and the intersection point of these two curve is performance point. Fig.4. shows superimposing demand spectrum and capacity spectrum. Check performance level of the structure and plastic hinge formation at performance point. A performance check verifies that structural and non-structural components are not damaged beyond the acceptable limits of the performance objective for the force and displacement implied by the displacement demand. The diagrammatic representation of performance point is shown in Fig 4.

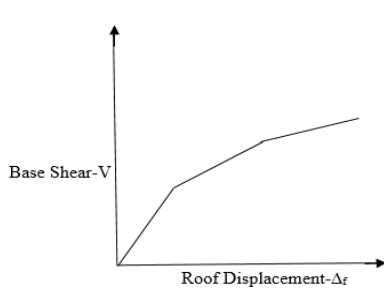


Fig 2. Capacity curve

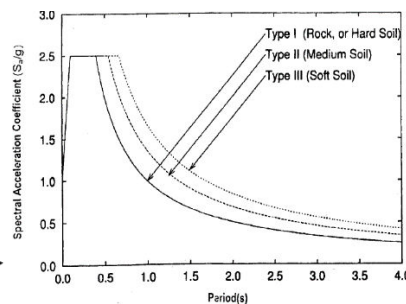


Fig 3. Demand Curve

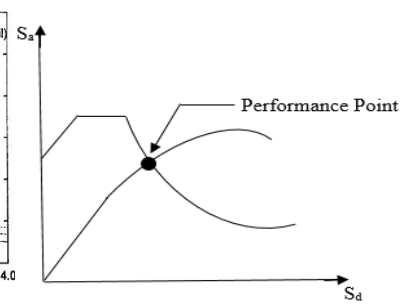


Fig 4. Performance point

III. DESIGN APPROACH

A building frame of G+10 floors is considered to verify its performance under seismic conditions. It consists of two bays in both the directions. The bay width along X and Y direction is 5m and the story height is taken as 3.5m. The frame is located in seismic zone IV. The dead load (12.5 kN/m²) and live load (10 kN/m²) are obtained from IS 875 (Part I and II). The seismic load is obtained in accordance with IS 1893:2002. The frame is designed according to IS 800:2007 in which the provision specified for earthquake resistant is also taken care. The major observation/details during the design of frame is discussed below

3.1 Calculation of design base shear

The design base shear V_B is calculated as per IS: 1893:2002 [5] based on the following Geometric details:

Table 2. Geometric Details

SNo.	Building Description	
1	Zone	IV
2	Zone Factor	0.24
3	Response Reduction Factor	SMF
4	Importance Factor	1.0
5	Height of building	35 m

The design base shear ($V_B = 52.16$ kN) computed is distributed along the height of the building as per the following expression:

$$Q_i = \frac{V_B}{\sum_{j=1}^n W_j h_j^2} W_i h_i^2 \quad (1)$$

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The member selection is carried out in accordance with IS 800:2007 which satisfies the strength and serviceability criteria. Also, chosen section is verified for various conditions listed in Section 12 (Design and Detailing for earthquake loads of IS 800:2007 as follows

i) Column strength

$$P_r/P_d > 0.4 \quad (2)$$

Where,

P_r – Required Compressive strength of the member

P_d – Design Stress in axial compression

Based on the column strength suitable load combination is chosen for the design of member section.

ii) Beam and column limitation

Beam and column section must be plastic sections where the hinge formation expected in the frame. Further, the following condition shall satisfy to arrive strong column weak beam approach. However, such equation doesn't differentiate the interior and exterior joints and also no specification is given in the consideration of effect of axial force in the plastic moment capacity of column section.

$$\Sigma M_{pc} / \Sigma M_{pb} \geq 1.2 \quad (3)$$

Where,

ΣM_{pc} – Sum of moment capacity in the column above and below the beam center line.

ΣM_{pb} – Sum of the moment capacity in the beams at the intersection of the beam and column center lines.

iii) Storey drift

According to IS 1893: 2002, the storey drift in any storey due to the minimum specified design lateral force, with partial load factor of 1, shall not exceed 0.004 times the storey height. Based on this condition, the inter storey drift obtained is shown in Table 3.

Table 3. Storey Drift

Storey	Interstorey Drift (mm)	Limitation (mm)
1	4.34	14
2	9.55	14
3	11.876	14
4	12.856	14
5	13.248	14
6	13.982	14
7	13.753	14
8	12.098	14
9	9.648	14
10	7.366	14

Based upon the codal provisions, the design is carried out and the section chosen are represented in the following table 4.

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Table 4. Section Details

Member	Storey	Section
Beam	1-5	ISMB 250
Beam	6-10	ISMB 225
Column	1-5	ISMB 500
Column	6-10	ISMB 450

3.2 Structural modelling

The two bay moment resisting frame (G+10 floors) is modelled using Sap-2000. Sap model showing the member details of the frame is shown in Fig 6a. Dead load and live load of the sections. Initially, non-linear analysis is performed using only dead load of the member is assigned with a reduction factor of 1 and live load with reduction factor of 0.25. Lateral load applied to frame is shown in the Fig 6b. Hinges were defined at the ends of beam and column. According to FEMA 356, M3 hinges are assigned to beams and P-M3 hinges are assigned to the column. M3 hinges defined in FEMA 356 is represented in Fig 5.

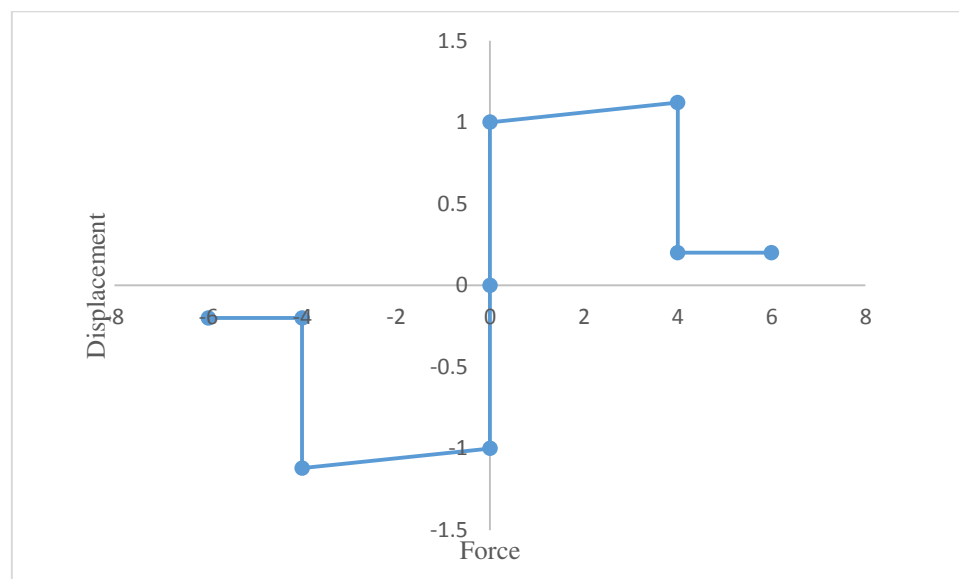


Fig 5. M3 Hinges (FEMA 356)

Displacement control analysis is carried out for the push load case, considering the P-Δ effects. For hinge loading local redistribution is considered. In Pushover analysis the iteration is done using Newton-Raphson method.

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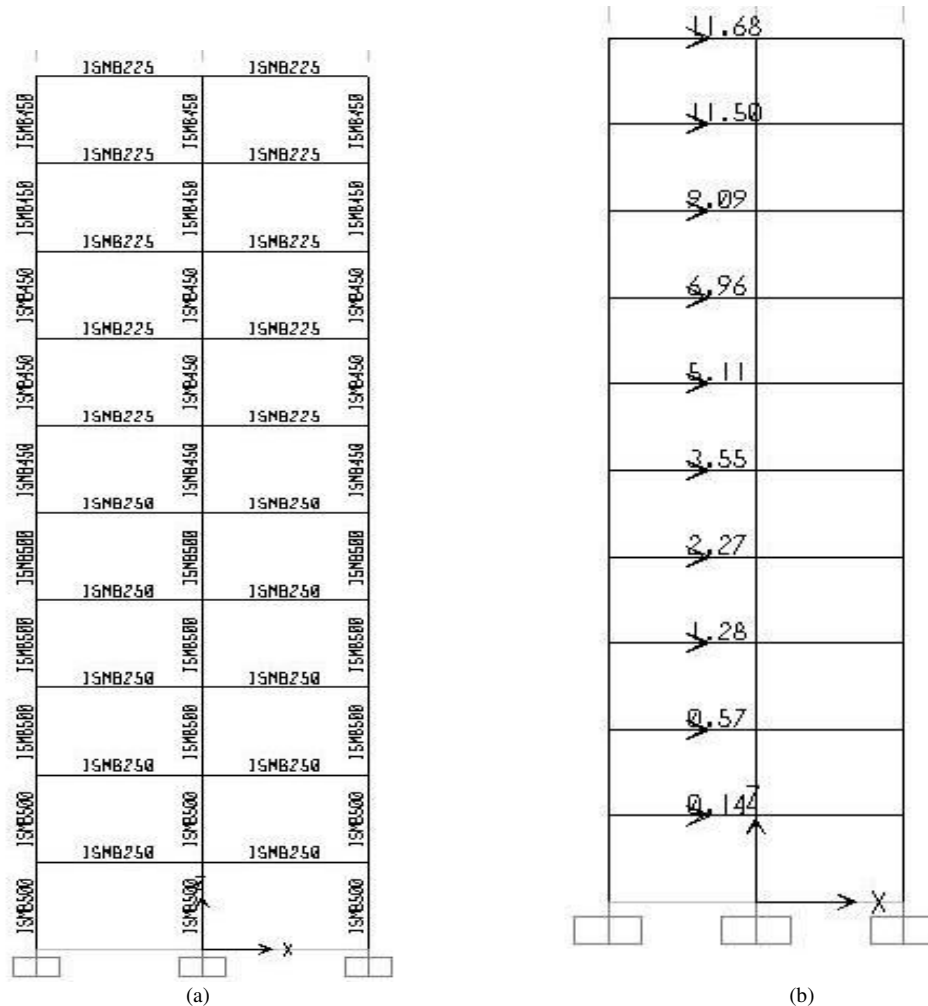


Fig 6. SAP Model (a) Member details (b) Lateral load distribution

IV.RESULTS AND DISCUSSIONS

4.1 Plastic Hinges

Plastic hinges formation for the building mechanisms have been obtained at different displacement levels. The hinge patterns are plotted at different levels in Fig 10. The first hinge (i.e. inelastic state) is formed in 6th storey and in the next step itself several plastic hinges is formed in various floor beam and columns. Subsequently, the column hinges in 6th and 7th storey reached the collapse stage. The initial formation plastic hinge in storey may be due sudden change in storey stiffness and also it can be seen in Table.1 the storey drift is higher than the other floor levels. The hinges reached the life safety level in step 4 and a sudden collapse level occurred in step 5 in columns. From the formation of collapse hinge in the column, it can be observed that the soft-storey mechanism developed. Such, storey mechanism should not be happened due to which sudden collapse occur.

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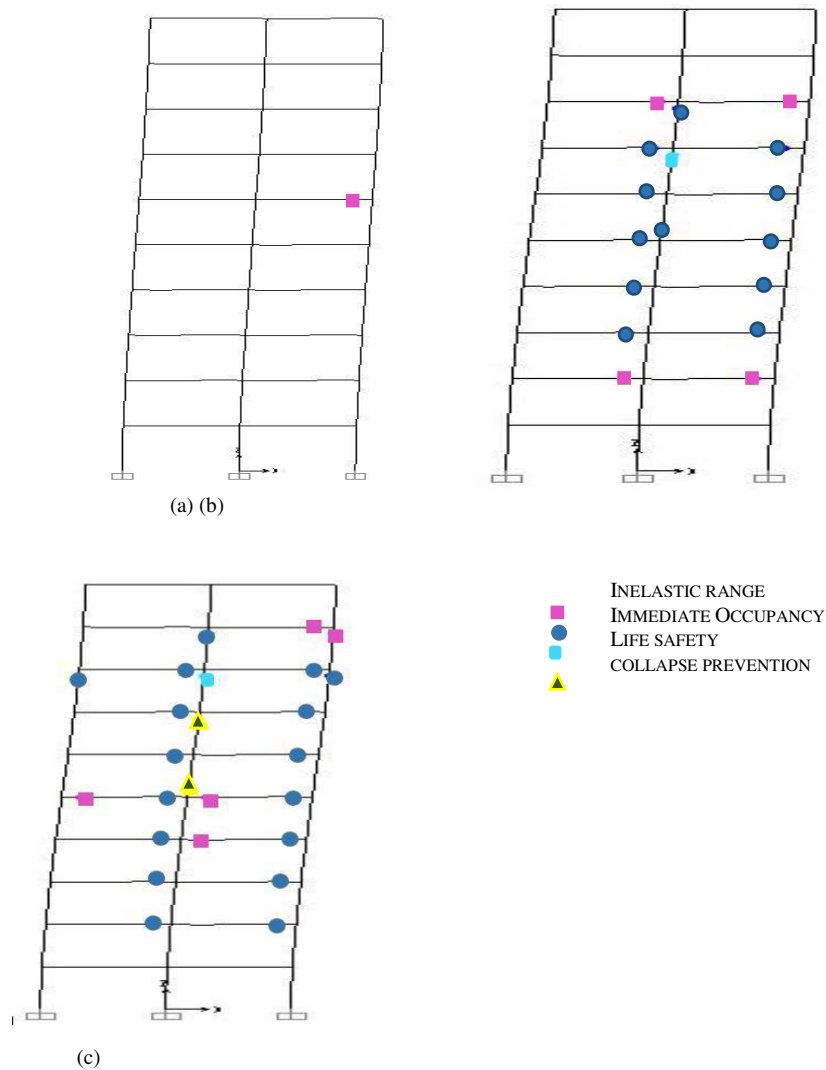


Fig. 7 Plastic hinge formation (a) Step 3 (b) Step 4 (c) Step 5

4.2 Pushover Curve

The resulting pushover curve for G+10 building is shown in the Fig.8. Pushover curve is linear up to immediate occupancy level beyond that the curve is non-linear to collapse prevention level. When the building is pushed well into the inelastic range, the curve become linear again but with a smaller slope. The target displacement is $760 \times 10^{-3} \text{m}$ and the base shear is 230kN.

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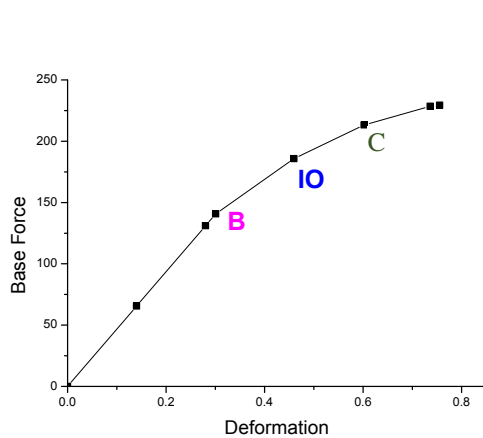


Fig.8 Pushover Curve

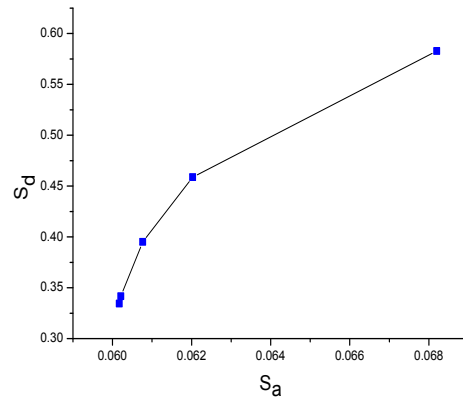


Fig 8 Demand Curve

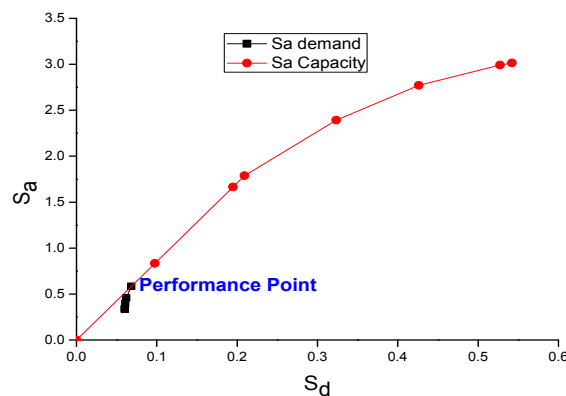


Fig. 9 Performance point

Fig 9 shows that capacity and demand curves are intersected in life safety zone. This point of interaction is called performance point. Based on this performance point location the damage intensity is calculated.

V. CONCLUSION

A two bay 10 storeyed frame has been analyzed as per IS 1893:2002 and designed as per IS 800:2007. The frame has been assessed for its performance through pushover analysis by using SAP 2000. From the study it is demonstrated that the frame has a better performance. The design base shear of the building frame is found to be 52.16 kN as per calculation. After performing the analysis the base shear is found to be 230 kN which is greater than design base shear. Based on the code provision, the condition satisfies to arrive strong column weak beam approach doesn't differentiate the interior and exterior joints and also no specification is given for the consideration of effect of axial force in the plastic moment capacity of column section. Further, because of change in storey stiffness, it might lead to formation of storey mechanism in 6th storey. Such failure mechanism is generally not recommended due to lesser ductility of structure. Present code provision doesn't provide any condition to avoid such failure mechanism. From this study, it is concluded that the provisions available in IS 1893:2002 for the earthquake resistant design need further developments towards improving the performance.

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