

# Performance Based Seismic Design for Steel Structures: A Review

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**ABSTRACT:** In the recent earthquake, it is observed that losses are increased due to the seismic design of buildings using codal procedure is not able to achieve best performance during earthquake. So, new design philosophies are required to improve the performance of the building in earthquake. From this design expected performance of the building decided for future earthquake. To overcome the above problems new design philosophy called performance based seismic design used in nowadays. To evaluate the performance of the buildings generally non-linear static analysis (Pushover Analysis) or non-linear dynamic analysis (non-linear time history analysis) are used. It is concluded from performance of the hinges, required performance objective is achieved or not. Generally, the normal steel buildings are not sufficient to resist earthquake forces. Therefore, the need for a bracing system arises that can increase the lateral strength and stiffness of building and reduces lateral displacement and storey drift. In this paper, some literatures studied on performance based seismic design for steel structures and design of the steel structures with bracing system.

**KEYWORDS:** Performance based seismic design, steel structures, bracings, pushover analysis, and non-linear time history analysis.

## I. INTRODUCTION

In the recent major earthquakes, it is noticed that the seismic risk in urban areas is increasing due to which trend in seismic design of building is changed. There is an urgent need to reverse the situation, which believed that one of the most promising ways of doing this is through the performance-based earthquake engineering (PBEE) in which the structural design based on the predicted performance of the structure during an earthquake. The performance based seismic engineering (PBSE) is a rapidly growing idea that is present in all guidelines that were published from past: Vision 2000 (SEAOC, 1995), ATC40 (ATC, 1996), FEMA 273 (FEMA, 1997), SAC/FEMA350 (FEMA, 2000a) and FEMA 356. In coming few years, the performance-based design will become the standard method of delivering Seismic resistant designs. In order to meet this objective it is essential that the structures/constructed facilities respond to the expected earthquake ground motions at the site in a designated manner, which in turn depends on the nature of ground motion exciting the structure. Thus the reliability of achieving, the life safety performance objective of any constructed facility is governed by the most uncertain element in the chain- expected ground motion.

Steel braced frame is one of the structural systems used to resist lateral loads in multi-storied buildings. Steel bracing is economical, easy to erect, occupies less space and has flexibility to design for meeting the required strength and stiffness. Braced frames used to resist lateral loads but braces can interfere with architectural features. A steel frame strengthened in various types to resist lateral forces like moment resisting frame, concentrically braced frame, eccentrically braced frame and buckling resistant brace frame.

### Building performance levels

Seismic performance described by designating the maximum allowable damage state (performance level) for an identified seismic hazard (earthquake ground motion). The target Performance level is split into two levels non-structural damage and structural damage, the combination of the two gives the building a combined performance level.

In increasing order of structural displacement, the various Performance levels are operational, immediate Occupancy, life Safety and collapse Prevention.

**Operational level(O)**In operational performance level, very light damages occur.Strength and stiffness has less reduction factor. Permanent drift in building occurred. Generally in operational level, displacement in building is very less, so more strength is required to sustain in earthquake. Non-structural component has less damages and other utility of the building in good working condition.

**Immediate Occupancy(IO)**In this level, minor cracking allowed. Permanent drifts not allowed. Strength and stiffness has less reduction factor. The displacement in building as compare to operational level is more. Non-structural component has more damages and other utility of the building in working condition.

**Life safety(LS)**Moderate cracking and some permanent drifts allowed in life safety level. Strength and stiffness reduced but no any damage to structure which dangerous to human lives. The displacement in building as compare to immediate occupancy is more.

**Collapse prevention(CP)**Damages and more Strength and stiffness reductions observed in collapse prevention level but collapse not occurred. The displacement in building as compare to life safety level is more.

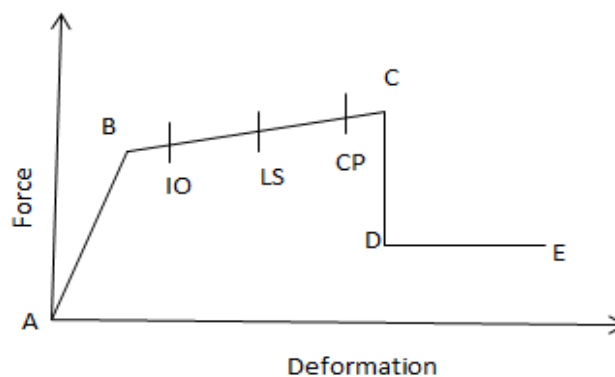


Fig.1: Pushover Curve

### Hazard Level:

The seismic hazard is depends on the ground shaking at that site. According to ground shaking at that site, seismic hazard levels are as follows:

#### The serviceability earthquake

The Serviceability Earthquake (SE) defined probabilistically as the level of ground shaking that has a 50% chance of being exceed in a 50 years period. This level of earthquake ground shaking is typically about 0.5 times the level of ground shaking of the Design Earthquake.The SE represents a frequent level of ground shaking that is likely to be felt during the life of the building. The SE has a mean return period of approximately 75 years.

#### Design Earthquake (DE)

The design earthquake defined probabilistically as the level of ground shaking that has a 10% chance of being exceed in a 50 years period.The DE represents an infrequent level of ground shaking that can occur during the life of the building. The DE has a mean return period of approximately 500 years.

#### The Maximum Earthquake (ME)

The maximum earthquake defined deterministically as the maximum level of earthquake ground shaking which may ever be expect at the building site within the mown geologic framework. In Seismic Zones III and IV this intensity of ground, shaking may be calculate as the level of earthquake ground motion that has a 5% probability of being exceeded in a 50-yearperiod. This level of ground shaking is typically about 1.25 to 1.5 times the level of ground shaking of the Design Earthquake. This earthquake definition represent an upper bound on the level of ground shaking that could be reasonably expecting to occur at the building site.

**II. METHODS OF ANALYSIS**

According to Indian standards, there is two main analysis approaches for evaluating building response in earthquake, linear and/or non-linear static and dynamic method.

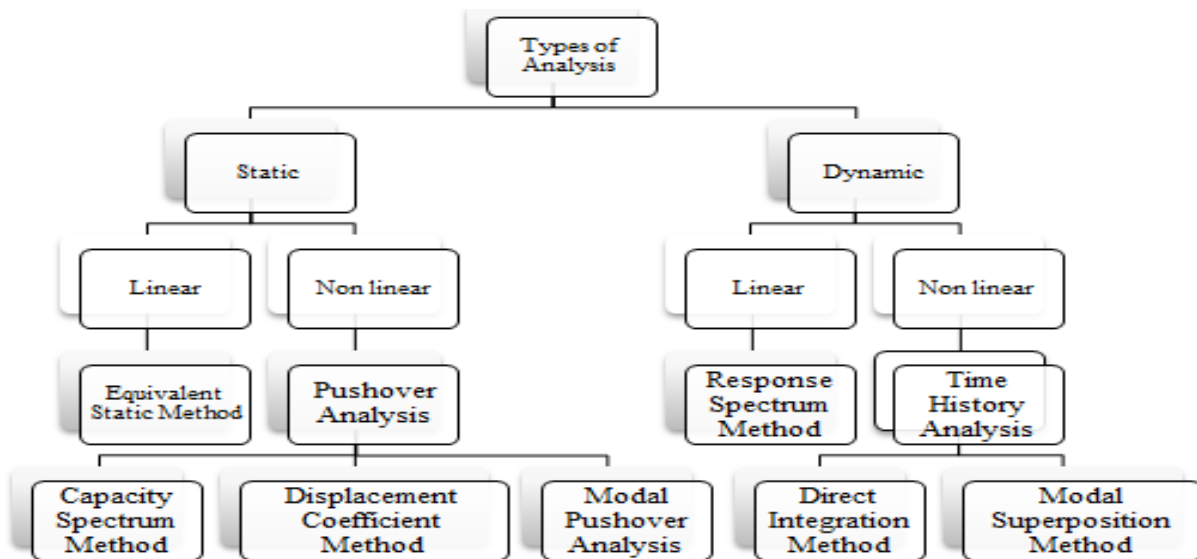


Fig.2: Types of Analysis

**Linear Static Analysis**

Linear static method (seismic coefficient method) explained in IS1893:2002 (part I). In this method, base shear is calculated. This method is formula based, due to which it will give us direct results without taking more efforts. For calculating response of the building 100%, mass participation assumed. In this method, no modal analysis performed for calculating natural timeperiod and frequency of the structures. Lateral load is distributed proportional to an assumed mode shape.

**Non-linear Static Analysis (Pushover analysis)**

Non-linear static analysis(Pushover analysis) explained in FEMA 356 and ATC 40. In this method, lateral loads applied in whole-in one shot at a particular point of the structure. In pushover analysis method, structure responses calculated by applying full force or giving target displacement, which is nothing but the 4%, of the height of the structure. Elastic analysis used to determine the lateral seismic forces, which are the reduced to inelastic design force levels by the response modification factor. The pushover analysis has mainly five methods explained in different codes:

**Capacity Spectrum Method (ATC-40, 1996)**

In this method, performance point calculated comparing capacity curve and demand curve. From Fig.3, red and green lines represent demand and capacity curves respectively. Capacity curvecalculated using the spectral acceleration Vs spectral displacement. Demand curve calculated from ground acceleration and period of the structure. A point where capacity curve and demand curve is meet called performance point of the structure in the expected earthquake. For better visualisation capacity curve needs to convert in ADRS (Acceleration Displacement Response Spectrum) format. As the structures yields in response to a seismic demand, it dissipates energy with hysteresis damping, called effective damping.Pushover curve also gives us information related to base shear, displacement, effective period and effective damping at the performance point.

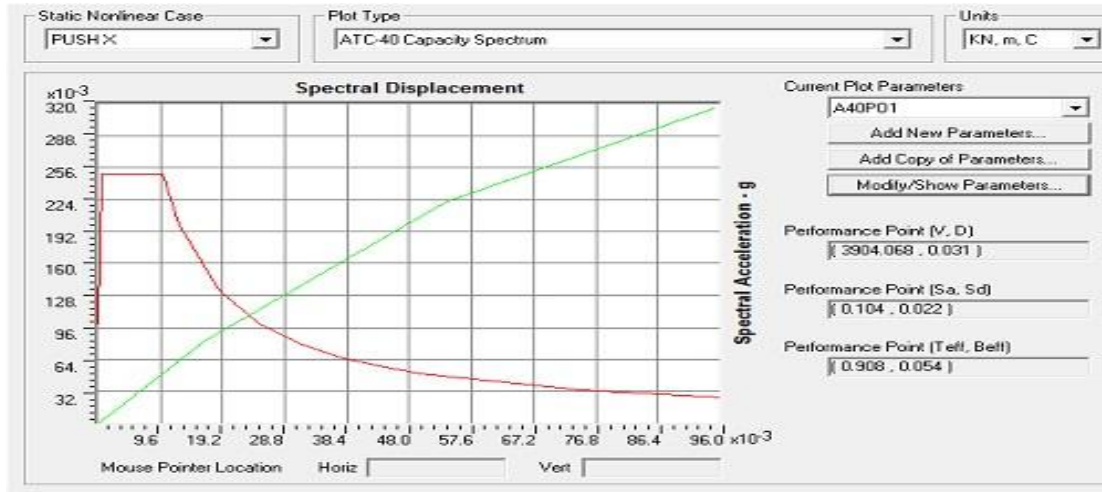


Fig.3: Pushover Curve (ATC 40)

### Displacement Coefficient method (FEMA 356, 2000)

In this method, the displacement modification factors applied to the maximum deformation of an equivalent elastic single-degree-of-freedom (SDOF) system, to estimate the maximum inelastic displacement demand of the multi-degree-of-freedom (MDOF) system. In the FEMA-356 document, the DCM used to characterize the displacement demand as shown in fig. 4. This method primarily estimates the elastic displacement of an equivalent SDOF system assuming initial linear properties and damping for the ground motion. In this method, the demand represented by reducing the elastic demand spectra by the correction factor to the inelastic demand spectra (constant-ductility demand spectrum) which are more accurate than the elastic spectra, with equivalent viscous damping. Pushover curve generated from FEMA 440 shown in fig. 5. The FEMA 356 uses the coefficient method, whereby displacement demand calculated by modifying elastic predictions of displacement demand. The FEMA 440 is improvement over the FEMA 356 in which modified coefficients used to calculate displacement demand.

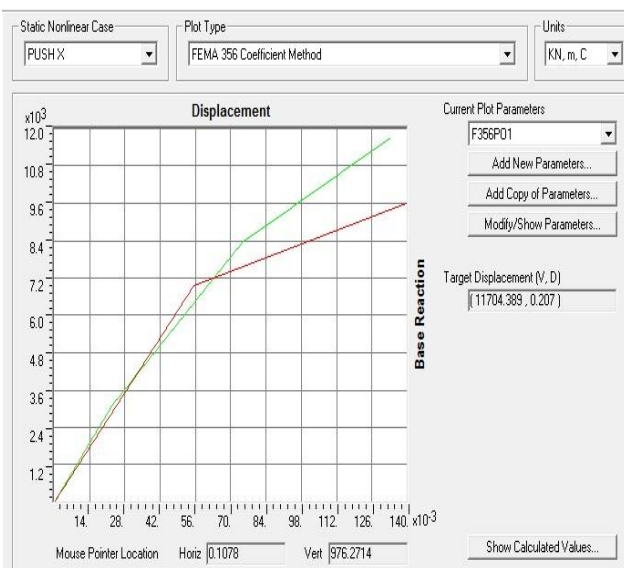


Fig.4: Pushover Curve (FEMA 356)

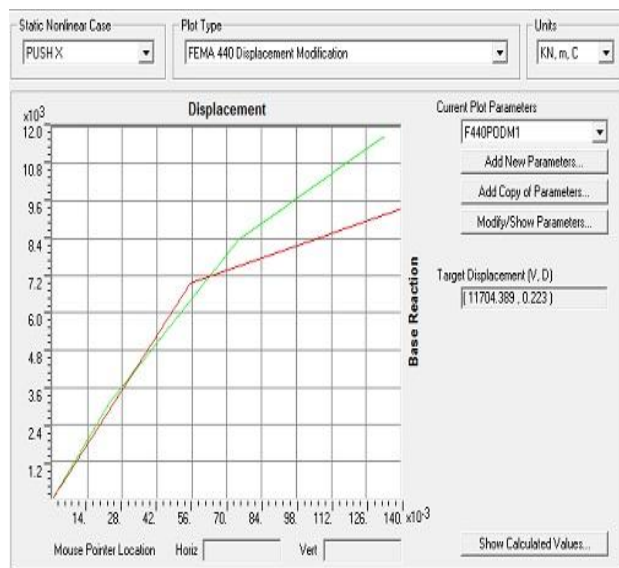


Fig.5: Pushover Curve (FEMA 440)

### **Modal Pushover Analysis**

The pushover analysis has been widely used for analysing the seismic behaviour of any structures. However, this method has limitation, the assumption that the response of the structure is control by its fundamental mode. A multi-modal pushover procedure in which the higher mode effects consider. The procedure comprises several pushover analyses under forcing vectors representing the various modes deemed to be excite in the dynamic response. As higher modes may reveal the failure mechanism not detected by the first mode, a better understanding of the structural performance considering the effect of higher modes becomes mandatory. The Modal pushover curves then plotted and converted to SDOF capacity diagrams, using modal conversion parameters based on the same shapes. Then, the response quantities are separately estimating for each individual mode, and then superimposed using an appropriate modal combination rule.

### **Linear Dynamic Analysis (Response spectrum method)**

In this method, modal analysis is use to get the dynamic characteristics of the structure(natural period and mode shape). To get design forces absolute sum method, CQC, SRSS methods are used. Response spectrum analysis is a procedure for computing the statistical maximum response of a base excitation. Each of vibration modes considered to respond independently as a single-degree-of-freedom system. Design codes specify response spectra, which determine the base acceleration applied to each mode according to its period(To determine peak displacements and member forces due to support accelerations).

### **Non-linear Dynamic Analysis (Time History Analysis)**

In this method, to evaluate response of the structure,ground motion data of previously occurred earthquake are used. This method is more accurate as compare to other methods explained above, but this method requires more time to analyse structure. For overcome the time problem we can use integrated software packages, which is available in market. Modal mass is oscillates back & forth in accordance to a recorded ground motion of an actually occurredearthquake. This ground motion is available in scaled or unscaled in tabular format of acceleration vs. time. In this method, the structure has non-linear hinges assigned the members can undergo stiffness degradation and strength deterioration. Time step should be less than 0.1 in time history analysis.The time history analysis has two methods, direct-integration time history and modal superposition method. In direct-integration time history, the equilibrium equations of motion are fully integrates as a structure is subject to dynamic loading. Analysis involves the integration of structural properties and behaviours at a series of time steps, which are small relative to loading duration. The equation of motion under evaluation is:

$$m\ddot{x} + c\dot{x} + kx = F(t)$$

The direct time integration method has also two methods, one is central difference method, whichis known as explicit methods. In explicit method, calculation at the current time step ( $t + \Delta t$ ) depends only on the information from previous time steps. Another method, Newmark method also called implicit method. Implicit method is more accurate as compare to above method. In implicit method, the unknown values at the end of time step ( $t + \Delta t$ ) are calculated based on the both previous time step  $t$  and current time step ( $t + \Delta t$ ) information.

## **III. LITERATURE REVIEW**

### **INTRODUCTION**

This chapter deals with a brief review of the past and recent study performed by researchers on seismic analysis of different steel frames with and without bracing system. A detailed review of each literature would be difficult to address in this chapter. The literature review focuses on find out most efficient bracing system and carry out the performance based design of steel frame building using optimized bracing system.

### **LITERATURE REVIEWS**

#### **Soon-Sik LEE, Subhash C. GOEL, and Shih-HoCHAO[2004]**

In this paper, a performance based seismic design procedure for steel moment frames based on pre-selected yield mechanism and target drift. The design base shear is deriving from a modified energy balance equation incorporating the concept of seismic force reduction factor and the displacement amplification factor. A nonlinear dynamic analysis is also presents a new seismic lateral force distribution. Three steel moment frames were design for the proposed lateral



force distribution and the performance plastic design methodology with specified pre-selected target drifts. The inelastic seismic behaviours of the three frames analyse through nonlinear static and dynamic analyses. That is, the structural performance can be well predicts and “controlled” by employing the proposed performance based design procedure.

**Farzad Naeim, Ph.D., S.E, Hussain Bhatia, Ph.D., P.E, Roy M. Lobo, Ph.D., P.E.[2007]**

This paper provides a basic understanding of the promises and limitations of performance based seismic engineering. In ATC-40 and FEMA 273/274, introduced and discussed the performance based design methodologies and techniques embodied in the two leading guidelines. The practical applications of the methods discussed with help of numerical examples.

**Manish S. Takey Prof.S.S.Vidhale[2007]**

In this paper, the analysis of high-rise steel building frame with different bracing section. For present work, the steel moment resisting building frame having (G+9) storey situated in zone III analysed using equivalent static analysis method. The steel moment resisting building frame is analysing by with and without steel bracing system using Software. The braced building of the storey drift decreases as compared to the unbraced building, which indicates that the overall response of the building decreases. The displacement of the building decreases depending upon the different bracing system employed and the bracing sizes.

**L. Di Sarno, A.S. Elnashai[2008]**

In this paper, the seismic performance of steel moment resisting frames (MRFs) retrofitted with different bracing systems like; special concentrically braces (SCBFs), buckling-restrained braces (BRBFs) and mega-braces (MBFs) are studied. A 9-storey steel perimeter MRF designed with lateral stiffness insufficient to satisfy code drift limitations in zones with high seismic hazard. The frame then retrofitted with SCBFs, BRBFs and MBFs. Inelastic time-history analysis carried out to assess the structural performance for earthquake ground motions. Maximum storey drifts of MBFs are 70% lower than MRFs and about 50% lower than SCBFs. The amount of steel for structural elements and their connections in configurations with mega-braces is 20% lower than in SCBFs. This reduces the cost of construction and renders MBFs attractive for seismic retrofitting applications.

**Huu-Tai Thai, Seung-Eock Kim[2011]**

This paper presents, the nonlinear inelastic time-history analysis of truss structures including both geometric and material nonlinearities. The geometric nonlinearity considered based on an updated Lagrangian formulation, while the material nonlinearity captured by tracing an empirical stress–strain relationship in the elasto-plastic range. The presented truss model is capable of capturing several failure modes of member such as buckling, yielding, inelastic post-buckling, unloading, and reloading. An incremental-iterative solution scheme based on the Newmark method and the Newton–Raphson method adopted for solving the nonlinear equations of motion. A computer program is developing to predict the nonlinear inelastic responses of truss structures. Numerical examples presents to demonstrate the capabilities of the proposed program in capturing the nonlinear responses of space truss structures under earthquake loadings.

**Sanda Koboevic, Jonathan Rozon and Robert Tremblay [2012]**

This paper is gives the information related to eccentrically bracing system. Analytical models built in three computer programs. Similar maximum forces but the inelastic deformation predictions at the element and global structural levels showed sensitivity to the modelling employed. These features considered in seismic performance assessment or design. Current design methods failed to predict interstorey drifts and plastic link rotations, but the study confirmed the strong correlation between the two parameters associated with rigid-plastic behaviour. The influence of yielding and flexural buckling of frame members, other than the links on the global frame performance evaluated using the OpenSees model. Inelastic response of braces and columns in such frames did not negatively affect the overall frame behaviour.

**Meng-Haotsai[2012]**

In this study, performance based design approach studied for retrofitting regular building frames with steel braces. The pseudo-static response analysis of an idealised elastic-plastic, single degree-of-freedom system for retrofit design approach is used. Three model frames used to study with incremental dynamic analysis. Analytical relationship between the increment of collapse resistance and structural characteristics is derive to determine the design strength & stiffness of the braces,. Non-linear dynamic analysis results indicate that the column loss response of the braced frames is approximate to the performance target and thus the proposed is feasible for practical applications.

**ZasiahTafheem, ShovonaKhusru[2013]**

A six story steel buildings modelled and then analysed due to lateral earthquake and wind loading, dead and live load. The performance of the same steel building has been investigated for different types of bracing system such as concentric (crossed X) bracing and eccentric (V-type) bracing using HSS sections. For different types of bracing system in comparison to building with no bracing has reduction in lateral displacement. From the present study, the concentric (X) bracing reduced more lateral displacement and thus significantly contributes to greater structural stiffness to the structure. The inter-storey drift is greatly reduced using of bracing system. From result, the bracing system has more influence on the restriction to relative floor-to-floor lateral displacement.

**Jagadish J. S, Tejas D. Dosh[2013]**

In this paper the G+15 stories steel building models is used with same configuration and different bracing systems such as Single-Diagonal, X bracing, Double X bracing, K bracing, V bracing is used for study purpose. The analysis of steel buildings and different parameters are compare using a commercial software package STAAD.ProV8i. Bracings are good to reduce the displacement and in case of K and V-bracing, the displacement is higher than without bracing because of irregularity in shape of the structure. The braced buildings of the storey drift either increases or decreases, as compared to unbraced building with the same configuration for the different bracing system.

**Hendramawat A Safarizki, S.A. Kristiawan, and A. Basukib[2013]**

To evaluate the possible improvement of seismic performance of existing reinforced concrete building (the 5th Building of UNS Engineering Faculty) by the use of steel bracing studied in this paper. Three methods of seismic evaluation are employed for the purpose of the study i.e. Nonlinear Static Pushover Displacement Coefficient Method as described in FEMA 356, Improvement of Nonlinear Static Pushover Displacement Coefficient Method as described in FEMA 440 and dynamic time history analysis following the Indonesian Code of Seismic Resistance Building (SNI 03-1726-2002) criteria. The results show that the target displacement determined from nonlinear pushover analysis of the existing building in X direction is more as compare to in Y direction. The performance of this building in between Life Safety (LS) - Collapse Prevention (CP) and plastic hinges occur in columns. The story drifts in Y direction exceed the serviceability limit criterion when the recorded El Centro ground motion used for dynamic time history analysis. The performance of the existing building improved if steel bracings used for seismic retrofitting. The nonlinear pushover analysis that target displacements in both directions reduces by 16%-55% if the proposed steel bracings are used. From dynamic time history analysis, we can say that the story drifts of the retrofitted building are within the limit criteria. The size of steel bracing elements does not significantly affect the seismic performance of retrofitted building.

**Phu-Cuong Nguyen, Seung-Eock Kim[2014]**

This paper presents a simple, effective numerical procedure based on the beam-column method by using the displacement finite element method for nonlinear inelastic time-history analysis of three-dimensional semi-rigid steel frames. The nonlinear geometry effects analyse by using stability functions and the geometric stiffness matrix. The inelasticity of material is considered by the gradual yielding of plastic hinges. A space zero-length multi-spring element is proposed to simulate the nonlinear cyclic behaviour of steel connections through the independent hardening model. The Hilber-Hughes-Taylor method combined with the Newton-Raphson balance iterative algorithm used to solve the nonlinear equations of motion. The results of nonlinear responses for steel frames with fully rigid and linear semi-rigid connections compare well with those of previous studies and commercial SAP2000 software.

**Phu-Cuong Nguyen, NgocTinhNghiemDoan, CuongNgo-Huu, Seung-EockKim[2014]**

In this paper, a beam-column element formulation and solution procedure for non-linear inelastic analysis of planar steel frame structures under dynamic loadings is studied. The spread of plasticity is trace by the uniaxial stress-strain relationship of each fiber on the cross section of sub-elements. For deriving a nonlinear elemental stiffness matrix, which directly takes into account geometric non-linearity and gradual yielding an elastic-plastic material model with linear strain hardening is used. For solving non-linear equations of motion, a solution procedure based on the combination of the Hilber-Hughes-Taylor method and the Newton-Raphson method is proposed. The non-linear inelastic time-history responses predicted by the proposed program ABAQUS.

**D.P. McCruma, B.M. Broderick[2014]**

The paper presents, a seismic response investigation into a code designed concentrically braced frame structure that subjected to in-plan mass eccentricity but not designed for mass eccentricity. The structure has an accidental uneven distribution of mass in plan resulting in an increased torsional component of vibration. The level of inelasticity that key structural elements in plan, mass asymmetric structures are is subjected to is important when analysing their ability to

sustain uneven seismic demands. In-plan mass asymmetry of moment resisting frame and shear wall type structures have received significant investigation, however, the plan asymmetric response of braced frame type structures is less well understood. A three-dimensional non-linear time history analysis model is created to capture the torsional response of the plan mass asymmetric structure to quantify the additional ductility demand, interstory drifts and floor rotations. Results show that the plan mass asymmetric structure performs well in terms of ductility demand, but poorly in terms of interstory drifts and floor rotations when compared to the plan mass symmetric structure. New linear relationships are developed between the normalised ductility demand and normalised slenderness of the bracing on the sides of the plan mass symmetric/asymmetric structures that the mass is distributed towards and away from

**Ali Imanpour, A.M.ASCE, Robert Tremblay, Ali Davaran, Christopher Stoakes and Larry A. Fahnestock, P.E., M.ASCE[2016]**

The seismic response of these frames is studying in this paper. A set of seven special concentrically braced frames (SCBFs), ranging from 9 to 30 m tall with two to six tiers, located in a high seismic area was designed according to the 2010 AISC Seismic Provisions. Fundamental behaviour of the two- and four-tiered frames investigate using three-dimensional (3D) finite element models with shell elements, with particular focus on the buckling response of the columns. The seismic frame response and column stability studied more broadly for all frames using more computationally efficient 3D finite element models with fiber based beam-column elements, which validated against the shell element models.

**F. Ferrario, F. Iori, R. Pucinotti, R. Zandonini[2016]**

In the paper, an analytical and experimental study aimed at supporting new design criteria for the exploitation of steel circular hollow columns made of HSS and subjected to exceptional loads, like earthquakes is presented. In fact, there is an increasing interest in the use of hollow sections of High Strength Steels (HSS). The ambitious targets are to enhance the structural performance of concentrically braced steel frame buildings with tubular columns and to reduce weight and, at the same time, construction costs. The paper initially describes an experimental study of the seismic behaviour of substructures representing a concentric braced frame of a prototype structure: a steel building with concentric bracings for offices, meetings or exhibitions. The brace beam-to-column joints represent the critical component. The objective of the test programme was to characterize the connection behaviour under monotonic, cyclic and random loads. In detail, five tests carried out on specimens with standard braces and two tests on specimens with weakened braces. The interstory drifts used to interpret the results. After the calibration of the numerical models, in order to evaluate the global response under seismic loading, a numerical analysis of the reference building is performing with the OpenSees programme. Pushover and dynamic nonlinear time-history analyses carried out to show results that performance-based design approaches for concentrically braced frames (CBFs) with high strength tubular steel columns.

#### IV. CONCLUSION

The literature review clearly indicated that the performance based seismic design is more beneficial over conventional method. For designing the buildings for performance based seismic design are easy but requires more iteration to obtain performance point. From above literatures, we can say that the pushover analysis is more convenient for designing but less accurate as compare to time history analysis. For non-linear time history analysis, requires at least three ground motions. From literatures, pushover analysis gives us directly performance point but in time history analysis requires more efforts to calculate performance point of the building. Generally, the normal steel buildings are not sufficient to resist earthquake forces. Therefore, the need for a bracing system arises that can increase the lateral strength and stiffness of building and reduces lateral displacement and storey drift. A different research has suggested implementing bracing system in a building, so that the building will be strengthen and can sustain during seismic response. The steel building with or without bracing system gives the different performance in earthquake. In bracing system, concentrically braced system is more efficient than eccentrically braced system for seismic analysis of steel building.



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