The requirement for performance-based seismic design and the usage of distinct models in order to accomplish this.

Pradyut Anand*

**PhD Scholar, Birla Institute of Technology Mesra, Ranchi, India.*
Email: pradyut.bitmesra@gmail.com; Mob: +91-9008312904

I. Abstract

The performance-based design process (PBSD) begins with the identification of one or more performance objectives. A performance-based seismic design accurately predicts how the structure will perform under the assumed earthquake hazard. Recognizing and evaluating the structure's performance capacity is critical during the design process of performance-based design. This article examines performance-based seismic design concepts and nonlinear analysis of structural models. PBSD is a repeated process that begins with the selection of seismic performance objectives. Subsequently, a variety of pushover analysis (POA) approaches are covered, and the merits and drawbacks of each are highlighted.

Keywords: Performance-Based Design, Performance Capacity, Performance Objective, Pushover Analysis.

II. Introduction

a. General

Earthquakes are a natural occurrence that is capable of wreaking havoc on structures and their inhabitants. As earthquake forces are also catastrophic and unpredictable, it is necessary to improve the engineering technique in order to assess and design the various systems for earthquake loads. The technique of performance-based seismic design (PBSD) assesses how a structure performs primarily during an earthquake and how the system will behave. Nevertheless, the future risk and its consequences cannot often be evaluated using a traditional design technique. The performance-based design begins with selecting design criteria and selecting one or more performance targets [1]–[4]. Each performance target presupposes the acceptable risk of experiencing a particular degree of damage and the consequent losses arising from this risk at a particular level of the seismic phenomena. Thus, the concept of performance-based design is applicable to all structures and their supporting non-structural components and members and is not limited to certain types of buildings.

The performance-based seismic design (PBSD) method promises to create a structure that is anticipated to meet the specified seismic performance target. Performance-based seismic design reliably predicts how a structure will behave under a presumptive earthquake threat. Recognizing and analysing a structure's performance capability is a key aspect of the performance-based design process. PBSD is a cyclical procedure that begins with the selection of a seismic performance objective and is followed by the first design of a structure; if the structure does not achieve the performance objective, it is redesigned and reevaluated until the desired performance level reaches the objective [1]–[4].

When structural designers realise that the conventional code design process is not optimal, the concept of performance-based design is conceived. Varied structures have different performance goals, and developing all structures using the same strategy is not the ideal approach. According to the Indian Standard (IS) code guideline, the base shear is computed using the Average response acceleration coefficient (S_a/g), the Importance factor ("I"), and the Zone factor ("Z"). The computed base shear is spread to all story levels in proportion to the mass calculated for each story level based on its height. The design forces and moments are determined after analysing the lateral force. According to the seismic IS code1983 (part -1):2016 load combination, the dead and live loads as well as the combined forces and moments are computed[2], [5]–[7]. In the past two decades, global earthquake disasters have demonstrated that significant damage occurred even when buildings were designed according to the conventional earthquake-resistant design philosophy (force-based approach), demonstrating the inability of the codes to guarantee the minimum performance of structures under design earthquake. The performance-based seismic design (PBSD) assesses how structures are expected to fare during a simulated earthquake. Compared to the force-based approach, PBSD provides a way for evaluating the seismic performance of a structure, therefore assuring the protection of human life and minimising economic damages. Analyzing the performance of a structure under lateral stresses employs the nonlinear static techniques commonly known as pushover analysis. Pushover analysis provides a pattern of plastic hinge formations in structural members as well as other structural metrics that directly reveal the performance of the member following an earthquake. In this article, a four-story RC structure is modelled and developed in accordance with IS 456:2000, and its life safety performance level is examined using SAP2000 v17. ATC 40 analysis is performed to determine storey drift, pushover curve, capacity spectrum curve, performance point, and plastic hinges in accordance with FEMA 273.From the analysis, it is checked that the performance level of the building is as per the assumption. In most cases, the design of steel structures is heavily influenced by the level of wind loads prescribed by codes and regulations and used in the structural analysis. This is due to the fact that steel structures, being light and ductile systems, are significantly affected by even a small difference in wind loading values. In recent decades, disproportional collapse analysis has attracted a great deal of attention due to the rising frequency of failures exhibiting this pattern. Commonly acknowledged guidelines and techniques of analysis have been developed, with the Department of Defense Facilities criterion or DoD being the most influential. In the DoD and other standards, the loss of a column is indicated as the modelling scenario that a structural system must withstand in order to be resilient. To far, however, all recommendations have dissociated column loss analysis from wind loads and solely done it for gravity loading.. This paper presents the dynamic time history disproportionate collapse analysis of steel frames, including various levels of wind loading. Interesting aspects are discussed through the parametric analysis of five different numerical examples of moment resisting frames. Non-linear static procedure (NSP) has been considered as a popular method to predict seismic force and deformation demands for performance evaluation of the structures, in recent years. However, this evaluation tool is restricted to low-rise and regular buildings in which the fundamental vibration mode dominates the structural behavior. Recently, some advanced procedures have been presented to oversee these conventional procedure deficiencies. In the current study, a new nonlinear static procedure considering the effects of higher modes in structural responses is presented. This approach assigns a contribution factor for each mode based on modal shear distribution. The offered contribution factor can be applied for determining the importance of each mode in lateral load pattern formation. In order to verify the results, some other types of pushover-based analysis are also performed and the responses obtained from each NSP are compared with those of rigorous non-linear response history analysis (NL-RHA). Results demonstrated the efficiency of the proposed method in accurate prediction of the seismic demands of high-rise buildings. This research examines the seismic P– effect in thin reinforced concrete (RC) columns using the layered section approach. For an effective analysis of the cyclic behaviour of RC columns, which exhibit changes in the load–displacement relationship based on the magnitude of the applied axial force, the layered section approach and a procedure for the indirect incorporation of the bond–slip effect into the stress–strain relationship of the reinforcing steel are utilised [5, 8]. Implementation of the bond–slip effect in layered section technique studies of RC frames subjected to cyclic stresses. Nonlinear dynamic analyses are conducted for 60 sets of horizontal and vertical earthquakes with practical ranges of slenderness and stability coefficients to verify the validity of the method and determine the significance of various effects in terms of the global response of the slender RC columns. On the basis of acquired numerical findings, the effects of axial force, the P– effect, and a vertical earthquake are analysed. In addition, their relative contribution is evaluated. In addition, the applicability of the capacity-demand diagram technique to the seismic design of RC structures is evaluated by comparing the findings produced from the new method to those obtained through rigorous nonlinear dynamic studies. [2], [5]–[7]. The structure is developed using the concrete code IS456:2000 based on the force and moment calculations, and then pushover analysis is performed on the structure. The PBSD depicts how a structure will behave in the event of an assumed seismic hazard.

Performance levels: Generally, performance requirements can be divided into four types: operational (the building can be used after the earthquake), immediate occupancy (the building was lightly damaged, but after minor repairs, it can be used without affecting the structure's purpose), life safety (the building was damaged and required repairs after being emptied), and collapse prevention (the building was damaged and required repairs after being emptied) (the building does not collapse and it got many several damaged requiring demolition).

Performance objective: Seismic performance of the building during the earthquake ground motion defines the maximum allowable non-structural or structural damage for the given seismic risk. The two key components of the performance objective are the seismic hazard and the destruction. During earthquake ground motion, the seismic performance for a specified seismic risk determines the maximum permitted destruction.

b. Performance based seismic design

Earthquake engineering is a branch of civil engineering that focuses on mitigating the risk of earthquake ground motion causing damage to civil structures and contributing to the reduction of fatalities. During the last 40 years, this civil engineering department has seen considerable advancements due to the rapid advancement and launch of computers and software that enhanced investigative capabilities and the emergence of new approaches for

earthquake design and assessment of structures. However, the catastrophic repercussions of an earthquake will be insufficient to overcome this improvement.
However, this enhancement of the design and judging procedures enables the designer to transition from a conventional approach, i.e., force-based methods, to inelastic displacement-based methods for the various structural performance levels. Separating the two methods, however, is a difficult task because forces and displacement are inextricably linked[9], [10].
This section provides an overview of performance-based seismic engineering, as well as its earthquake judging and design. Then, the approach for judging and also for earthquake design purpose is briefly discussed, as well as its shortcomings. The non-linear static pushover analysis method's conceptual framework is then expressed jointly with the various pushover analysis approaches. Performance-based seismic design is a civil engineering method that evolved from the recognition that the difficulty in the seismic action of structures was due to the lack of clear design processes for Operational, Immediate Occupancy, Life Safety, and Collapse Prevention, as mentioned in table 2.
Numerous bodies have advocated alternative ways, including the NEHRP (1997) and SEAOC Vision 2000 (1995). Performance objectives should be developed first, based on the previously described factor. Following that, additional action should be done to minimise structural damage, economic losses, and loss of life.
The structure that is to be developed should be constructed with the structure's intended usage in mind. To accomplish the performance aim outlined in the table, a document titled VISION 2000 was created based on the return period of seismic ground motion. In picture 1, the red dot denotes the performance aim[3].
For instance, the structure erected for the common purpose must be designed in such a way that it is rendered inoperable following an earthquake and does not require extensive structural repairs. While the structure has been constructed for essential purposes, it must be designed in such a way that it will not affect its use following the earthquake, as well as the risky facility must be able to resist damage and withstand wear even during earthquakes with a low probability of occurring and remain completely operational following this. According to the design basis earthquake, just one performance criterion is necessary, namely life safety at a defined level of ground motion in accordance with existing building code provisions.

Table 1: Performance level definition[3]

Performance Level		Description
NEHRP Guidelines	**Vision 2000**	
Operational	Fully Functional	Significant damage has not happened to the structure or its components, and the structure can be used following the earthquake.
Immediate Occupancy	Operational	The structure sustained minimal damage, but following modest repairs, it can be used without impairing the structure's function.
Life Safety	Life Safe	The structure sustained damage, and it required restoration upon its emptying.
Collapse Prevention	Near Collapse	The structure does not collapse, although it has sustained several damages necessitating demolition.

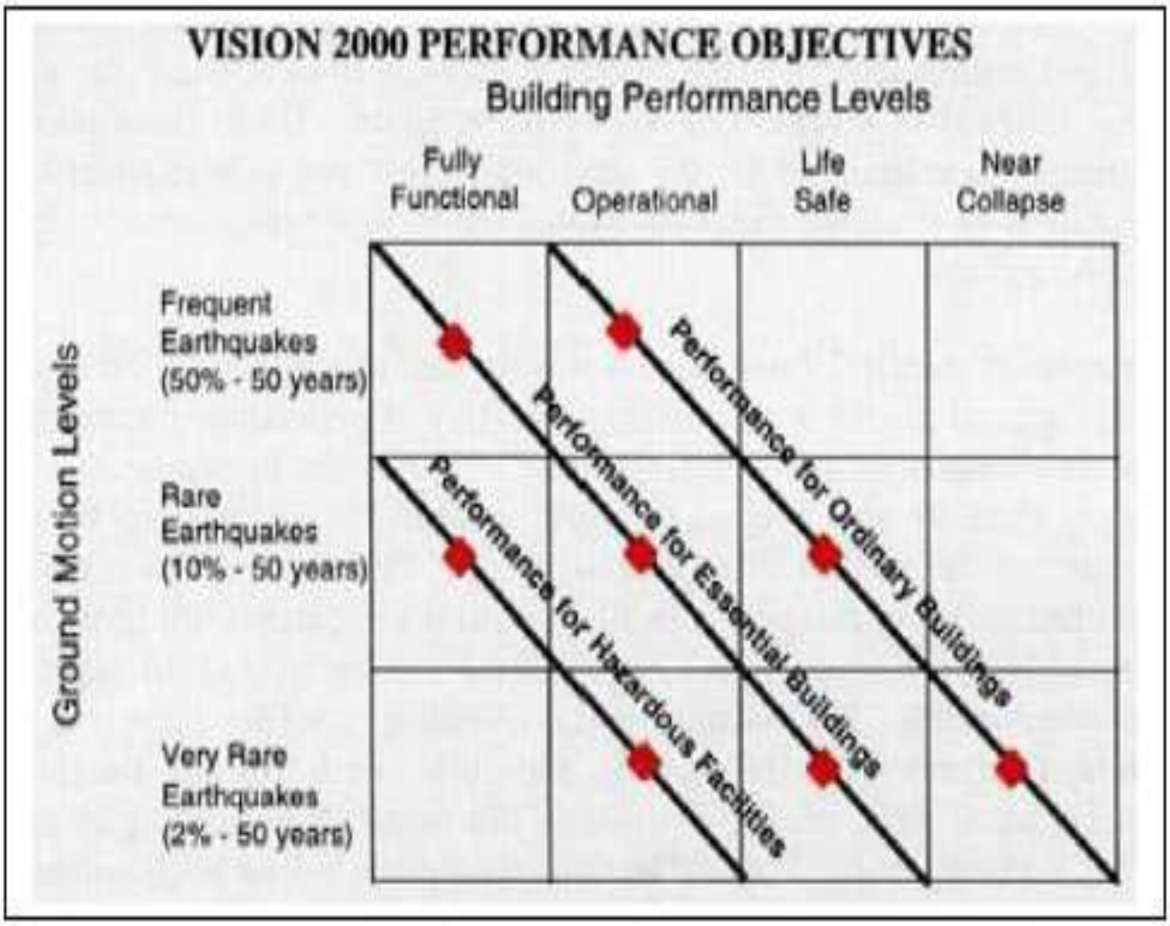

Figure 1 Performance objective from vision 2000[4]

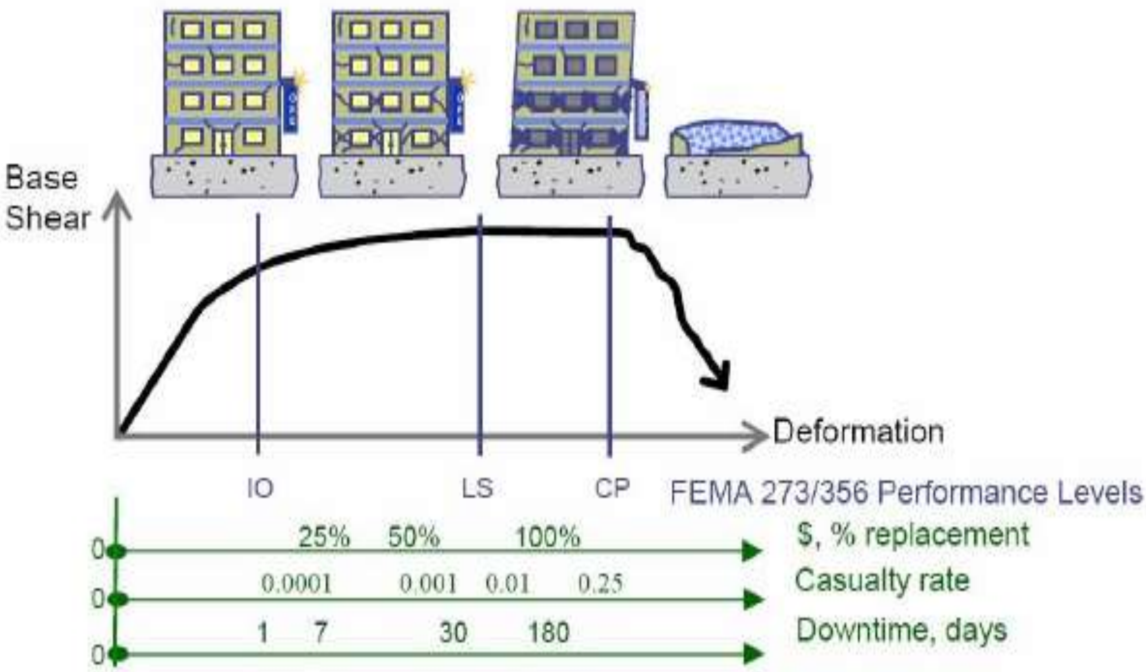

Figure 2 Performance level as per FEMA[4]

c. Performance Based Tools

The performance-based technique necessitated the evaluation and design of two quantities, namely seismic demand and seismic capacity[3], [11]. Seismic demands refer to the seismic effect's ability to withstand the seismic capacity and the earthquake impacts on the building. The performance is evaluated to ensure that capacity exceeds seismic demand in accordance with code ATC-40. All of these results are derived using either time history analysis or nonlinear pushover analysis.

Evaluating the structure's performance is the most pragmatic scientific approach, but it is frequently quite complex and time consuming due to the troublesome nature of complex ground motions. Due to the system's hardness, nonlinear static analysis approaches have become indispensable for assessment and tool design.

Federal Emergency Management Agency (FEMA) and Applied Technology Council (ATC) provisions specify four distinct procedures for design and judging purposes: linear static method, nonlinear static method, linear dynamic method, and nonlinear dynamic method with increasing order of hardness[6]. The tools are as follows:

i. Linear Static Analysis Method

[3], [11]A pseudo lateral static load pattern is used to determine the force and displacement requirement for each structure member due to the ground motion. This workload is compared to the building members' capabilities. This structure cannot be applied for the unique and nonlinear characteristics, including stiffness, distribution of mass, and strength. It does not utilize lateral forces that are orthogonal to the system, or if members have substantial ductility demand, it is not feasible.

ii. Linear Dynamic Analysis Method

[7]The linear dynamic analysis approach utilizes modal analysis, i.e., the response spectrum and time history analysis, to determine the force and displacement demand. Although the modal analysis is preferred over the response spectrum approach, it is recommended over the Single Degree of Freedom System (SDOF) for each mode of vibration due to the lack of time history analysis in the modal analysis. The Response Spectrum Analysis (RSA) of the ground motion is utilized to determine the demand.

iii. Non-Linear Static Analysis Method

The computer model is subjected to a preset lateral load pattern and is referred to as the pushover analysis method. The relative inertia forces are shown at the site of the considerable mass. As load intensity is increased, the structure yields, producing a succession of cracks, plastic hinges development, and the point at which failure will occur. This is a continual operation until the set displacement limit is reached[3], [7].

iv. Non- Linear Dynamic Analysis Method

This method removes the limitation of the static, dynamic, and nonlinear analysis methods, and hence it is the most sophisticated analysis method. The best solution to the design analysis is expected to be found with it. The main tools needed for this approach are models of the structure, ground motion features, and nonlinear material models[3].

d. Mathematical Pushover Analysis

[5]A strong conceptual framework doesn't exist to underpin the pushover analysis method, mostly resting on the assumption. This is supposed to be the case, as it is thought that the modes' form and first mode of vibration were responsible for influencing the structure's behaviour. Additional belief is that it is thought that mode shape will remain constant throughout the structure's response while the structure is under inelastic and elastic conditions. So, it represents the start of the change from dynamic to static problems. The transformation from the Multi Degree of Freedom System (MDOF) system to the Equivalent Single Degree of Freedom System (ESDOF) system is seen in Fig 3.

This governing differential equation may calculate the motion of the elastic or inelastic MDOF system in the presence of an earthquake.

$$[M]\{\ddot{U}\} + [C]\{\dot{U}\} + \{F\} = -[M]\{1\}\ddot{U}_g \quad (1)$$

Where,
M= mass matrix;
C = damping matrix
F = storey forces vector,
$\{1\}$ = the influence vector for the displacement of masses due to ground motion applied

$\ddot{U}_g$ = history of the ground acceleration

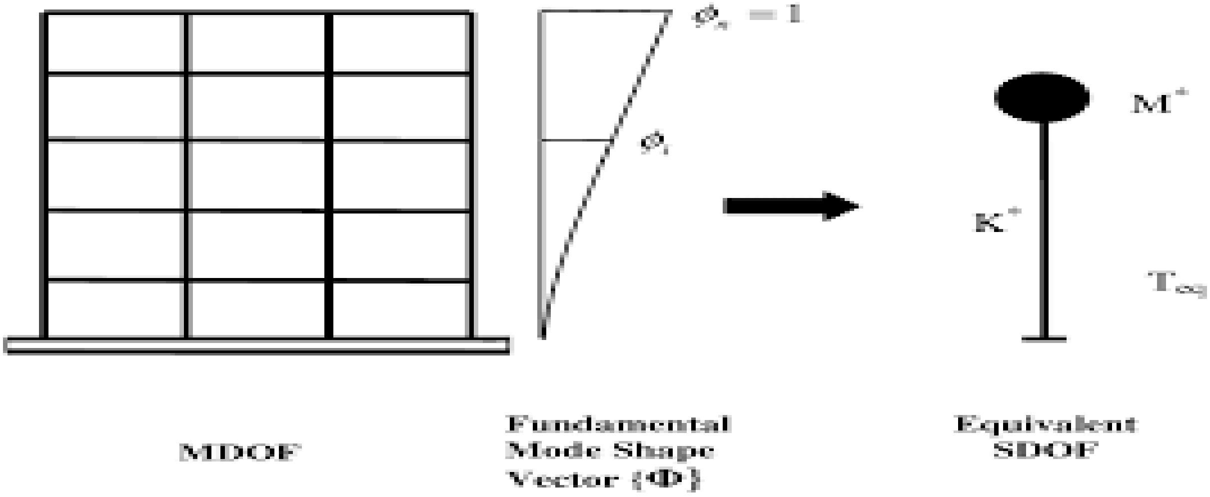

Figure 3 Diagram for the transformation from MDOF to SDOF systems[4]

A single shape vector is assumed, which is not the function of time; u is defined for the MDOF system as the relative displacement vector for the as U = {Φ}u_t.

[8]Roof displacement is defined Ut as and this following equation of MDOF system will be transformed to.

$$[M]\{\phi\}\ddot{U}_t + [C]\{\phi\}\dot{U}_t + \{F\} = [M]\{1\}\ddot{U}_g \text{ (2)}$$

u^* the reference displacement for the single degree of freedom system as: [9]

$$U^* = \frac{\{\phi\}^{\Lambda}T[M]\{\phi\}}{\{\phi\}^{\Lambda}T[M]\{1\}} u \text{ (3)}$$

By multiplying the equation 2 by {ϕ}T and placing the value for Ut by using eq 3, the following equation will describe the response for the equivalent single degree of freedom system:

$$M * \ddot{U} + C * \dot{U} + F^* = -M * \ddot{U}_g \text{ (4)}$$

Where,

$M^* = \{\phi\}^T[M]\{1\}$ (5)

$C^* = \{\phi\}^T[C]\{\phi\}\frac{\{\phi\}^T[M]\{1\}}{\{\phi\}^T[M]\{\phi\}}$ (6)

$F^* = \{\phi\}^T\{F\}$ (7)

Nonlinear incremental static analysis of the MDOF system determines the ESDOF system's Force-deformation feature. Valuable information such as the structure's response, as well as the structure's behaviour once it reaches its elastic limit, is contained inside the capacity curve. However, results are dependent on the material model, and so the capacity curve after the post-elastic stage is unknown.

Bilinear curves are good for solving problems that call for simplified solution, as their analysis is made simpler by using bilinear algebra. They can be described as bilinear from which the yield strength V_y, an effective stiffness $K_e = \frac{V_y}{u_Y}$ and a hardening or the softening stiffness $K_s = \alpha K_e$ are defined[3].

Thus, the T_{eq} (initial period) of the ESDOF system will be:

$$T_{eq} = 2\pi\sqrt{\frac{M^*}{K}} \quad (8)$$

Where,

K^* defines the elastic stiffness of the ESDOF system and is calculated by:

$$K^* = \frac{F_Y^*}{U_Y^*} \quad (9)$$

Base shear and roof displacement relationship for the ESDOF system is taken same as the MDOF system, and the strain hardening ratio is taken as α.

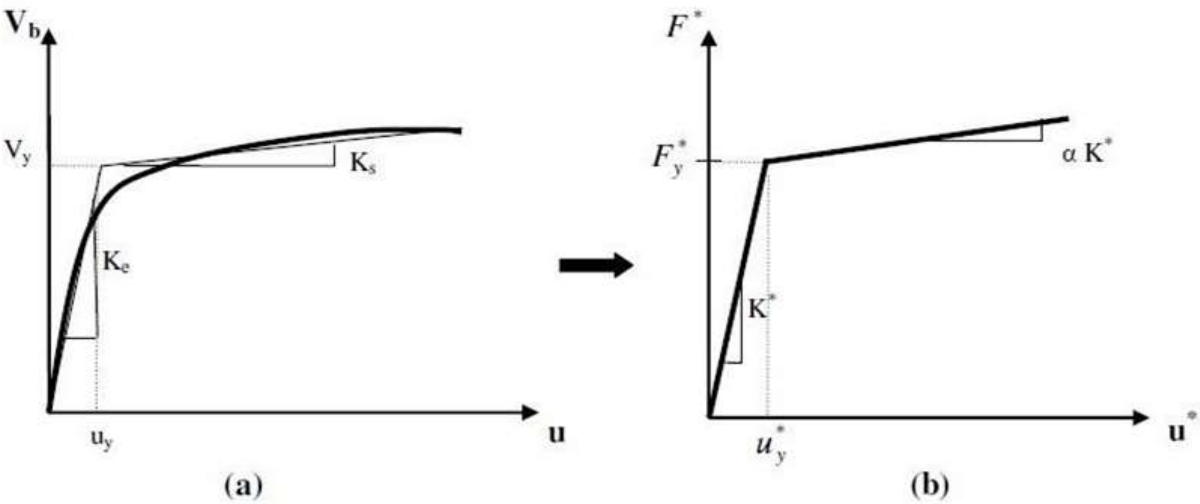

Figure 4 (a) capacity curve for the MDOF system (b) equivalent SDOF system[4]

[12]Using either elastic, inelastic, or time history analysis, the maximum displacement for the SDOF structure is calculated from the given ground motion. The corresponding displacement for the MDOF system can find but rearrange in eq. 3 as follows:

$$U_t = \frac{\{\phi\}^T[M]\{1\}}{\{\emptyset\}^T[M]\{\phi\}} \quad (10)$$

The design of the SDOF equitable system must not present too much responsiveness to outcomes unless the design spectrum is sensitive to short-term variability. Deflected shape for MDOF system can be explained by a single and constant shape vector without considering deformation as it is typical for the pushover analysis.

Based on the selection of the mode shape vector, the target displacement ut is dependent. As shown in a previous paper, the initial mode shape would provide the accurate guessing of target displacement if the primary mode controls the structure behaviour.

Following relationship by the capacity, the curve is obtained by using the following expression by defining the bilinear curves according to Reinhorn.

$$V(u) = V_y \times \left\{\frac{U}{Uy} - (1-a)\left(\frac{U}{U_y} - 1\right) \cup \left({}^{U}/_{U_y} - 1\right)\right\} \quad (11)$$

V_y is the yield strength, and the u_y is the displacement. α is the post-yield stiffens ratio and is given by $\alpha = K_s/K_e$. Step function is given by U (u / u_y -1) and is 0 for value of u / u_y < 1 and for greater than 1 value of it. A simplified expression is mentioned below as equation 12.

$$V(u) = K_{eu} \qquad u < u_y \; (12)$$

$$V(u) = V_y + ak_e(u - u_y) \qquad u > u_y \quad (12)$$

Thus, we see that Reinhorn's approximation appears straightforward when solving for design purposes.

e. Lateral Load Patterns

For performing the pushover analysis of the MDOF structure, a sequence of lateral load patterns must be applied to the mass point of the structure. The main use of this is to show how much stress the structure is going to undergo during an earthquake. Incremental applying of the load pattern reveals progressive yielding of the structural member. The amount of vibration and loss of stiffness that takes place in the system during the inelastic phase can be observed in the deformation of the system's force.
Because of the chosen load pattern, study of the dynamic condition is critical. It needs to respond dynamically to account for changes in conditions. It is stated in the FEMA356 and EC8.Two load patterns are also recommended.

f. Pushover Analysis Methods

The pushover analysis technique is further classified into three categories:

1) The conventional method of determining POA
2) Method of adaptive POA
3) POA technique based on energy

Numerous additional POA techniques are discussed in the publication. The conventional POA approach is referred to in this section:

1) Capacity Spectrum Analysis
2) Improved Capacity Spectrum Analysis
3) N2 Analysis
4) Displacement Coefficient Analysis
5) Modal Pushover Analysis

To compute the maximum displacement in the correct inelastic spectrum, the capacity spectrum approach and the N2 method are compared. The approach for determining the N2 and enhanced capacity spectrums is same. Nonetheless, it is an advancement of the capacity spectrum approach.

1) Capacity Spectrum Method (ATC 40)

[13]When compared to other PBSD techniques, the CSM performs admirably and provides the added benefit of allowing the engineer to visualise the link between demand and capacity. The distinctions between the various strategies are more about unknowns in material behaviour and quantification of energy dissipation than they are about analytical techniques (Freeman, 1998b).

Non-Linear Static POA of the MDOF Model

Vertical distribution of lateral load is applied to the structure based on the fundamental mode of vibration. The capacity curve, roof displacement curve, and base shear are then determined using a nonlinear static analysis.

Definition of Inelastic ESDOF

(V_y, u_y) are the system's global coordinates, while (V_{pi}, u_{pi}) are the system's ultimate displacements. The capacity curve then approximates the relationship as a bilinear one. In fig 5, the area A1 is defined as the yield point (V_y, u_y). It is equal to the area A2 such that each curve has the same amount of energy associated with it.

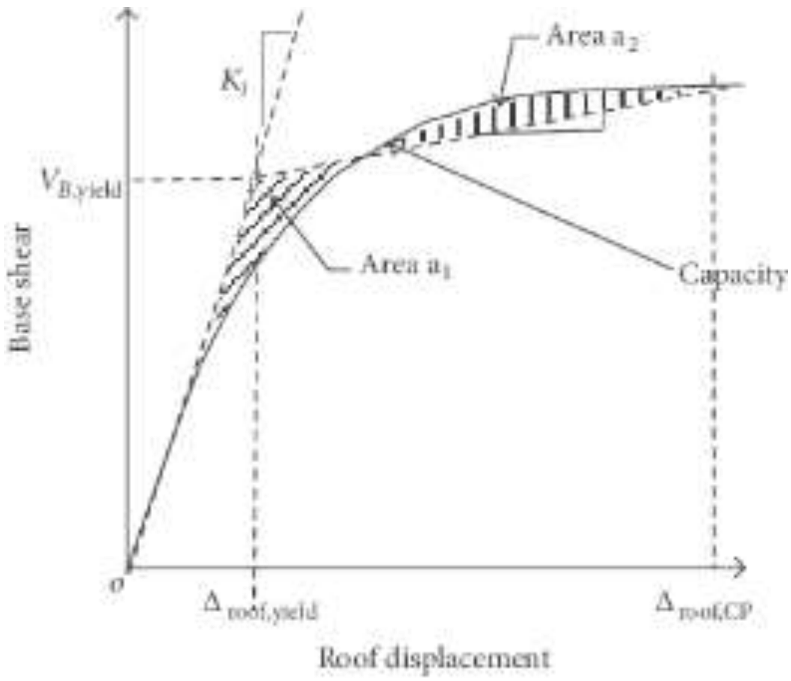

Figure 5 Approximation of the capacity curve [4]

2) Improved Capacity Spectrum Method

Mr. Chopra et al. (2000) proposed the Improved Capacity Spectrum Method in order to demonstrate the continuous ductility for inelastic design using the Capacity Spectrum Method rather than the elastic damped design. Estimating seismic demand for an equivalent single degree of freedom will be different than using the Capacity Spectrum Method. Numerous ductility values intersect the inelastic spectrum of the capacity spectrum curve, as illustrated in Figure 1-5. The point of intersection of the curve will be the location of deformation demand. The ductility factor is calculated from the capacity spectrum and must match the demand curve produced from the intersection. This method, like the Capacity Spectrum Method, needed iteration[10].

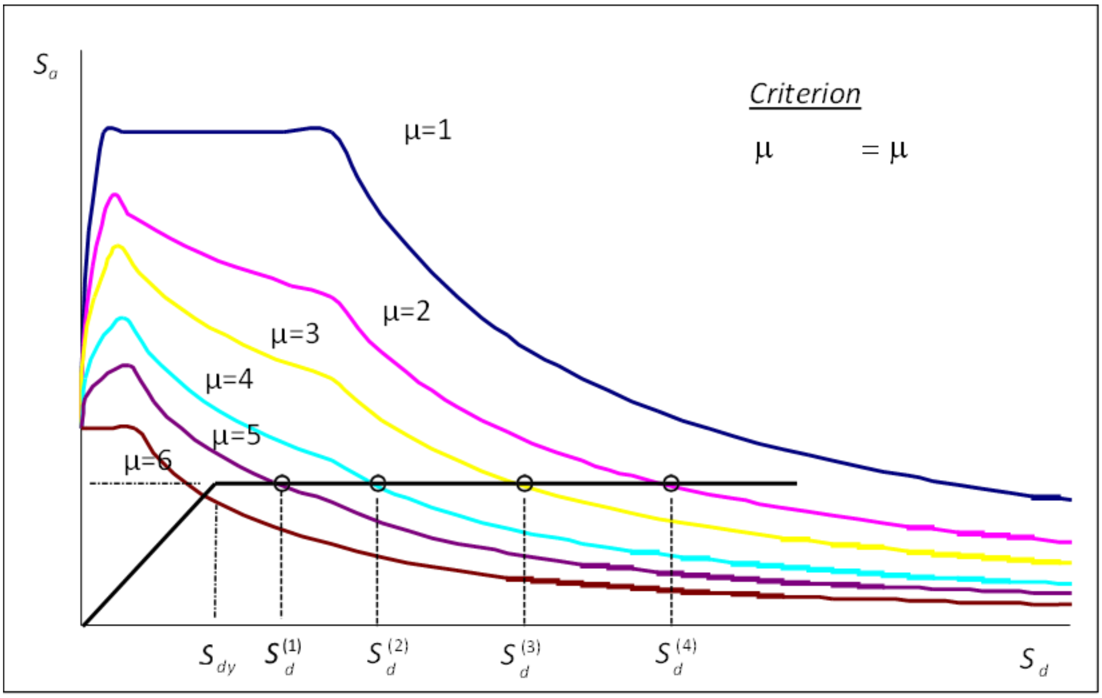

Figure 6 ICSM Method Application[4]

3) N2 Method

Fajfar et al. were the first to explain the N2 method in 1988 as the alternative to the Capacity Spectrum Method. The Saiidi et al. is used as the basic idea for the N2 way further based on Gulkan et al. For calculating target displacement, and the demand spectrum is used. They differ with the capacity spectrum.

Non-Linear Static POA for the MDOF Model

Capacity Spectrum Method steps are similar to this method.

4) Displacement Coefficient Method

The displacement-controlled approach differs from the capacity spectrum and N2 methods in that it measures target migration directly and does not require conversion of the capacity spectrum to the capacity curve. The preceding steps are used to calculate the structure's force deformation, and the mentioned equation is used to calculate the target displacement:

$$u_t = C_0 C_1 C_2 C_3 S_a \frac{T^2 e}{4\pi^2} \qquad (13)$$

Where,
C_0 = this is the modification factor related to the single degree of freedom for the spectral displacement.
C_1 = this is the medication factor for the maximum inelastic single degree of freedom.

$$C_1 = 1 \qquad \text{for} \qquad T_e >= T_c \ (14)$$
$$[1.0+(R-1)\,T_c / T_e]\ R^{-1} \qquad \text{for} \qquad T_e < T_c \ (14)$$

Where,
T_c = characteristic period of the response spectrum means.
T_e = Fundamental period, which is effective.
R = Ratio of inelastic strength demand to calculate yield strength.
Calculated as mention here:

$$\frac{R = S_a/g \cdot 1}{V_y/w \cdot C_a} \ (15)$$

C_2 = hysteresis shape modification factor for maximum displacement.

C_3 = this is the modification factor for the displacement due to 2nd order effects. For building having (+ve post-yield stiffness), the C_3 is equal to 1, and for (-ve post-yield stiffness), C3 is calculated by the given formula:

$$C_3 = 1 + \frac{|\alpha|(R-1)^{3/2}}{T_e} \ (16)$$

Where,
α = Ratio of post-yield stiffness to elastic stiffness relationship is bilinear for the force deformation.
S_a = spectral acceleration
V_y = bilinear representation for the capacity curve for the yield strength.
W= dead load and live load

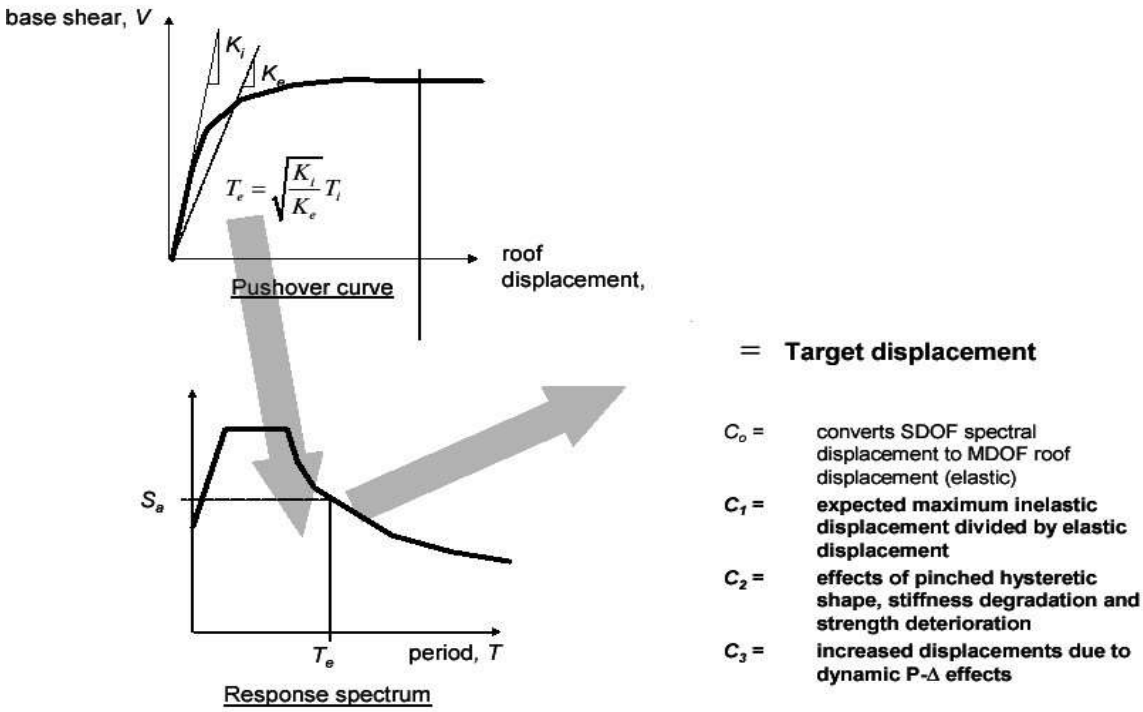

Figure 7 DCM method procedure[4]

5) Modal Pushover Analysis

Chopra and Goel (2001, 2002) have developed the modal pushover analysis to conclude the Modal Pushover steps by Paret et al. (1996).

Dynamic Property Calculation:
The natural period and modes of vibration of a structure are calculated using elastic vibration. The distribution of horizontal force is expected to be applied as a result of the influence of the fundamental and another mode shape, and this formula calculates the distribution for each mode shape:

$$s_j = [M]\phi_j \quad (17)$$

Where [M] is for the matrix of mass of the structure.

III. Result

The broad introduction to performance-based seismic design (PBSD) as well as the reasons for the requirement of PBSD are explored in this article. After that, a tool for the PBSD is discussed, in which the linear static analysis method, the linear dynamic analysis method, the nonlinear static analysis method, and the nonlinear dynamic analysis method are all discussed [13], and then pushover analysis is discussed, in which the mathematical expressions are discussed. Following that, numerous types of pushover analysis are discussed, along with their advantages and disadvantages, making it simple to select one model for conducting structural checks and designing structures by the model.

References

[1] A. S. Khedkar, R. A. Dubal, and S. A. Vasanwala, "Performance Based Seismic Design Of Reinforced Concrete Moment Resistant Frame With Vertical Setback," *International Journal of Engineering Research*, vol. 3, no. 2, p. 6, 2014.

[2] D. J. Chaudhari and G. O. Dhoot, "Performance Based Seismic Design of Reinforced Concrete Building," *Open Journal of Civil Engineering*, vol. 6, no. 2, Art. no. 2, Mar. 2016, doi: 10.4236/ojce.2016.62017.

[3] M. H. S. Elawady, "Ductility considerations in seismic design of reinforced concrete building," Dec. 2017, Accessed: Sep. 22, 2021. [Online]. Available: https://iconline.ipleiria.pt/handle/10400.8/3031

[4] S. Themelis, "PUSHOVER ANALYSIS FOR SEISMIC ASSESSMENT AND DESIGN OF STRUCTURES," p. 287.

[5] H.-G. Kwak and J.-K. Kim, "P–Δ effect of slender RC columns under seismic load," *Engineering Structures*, vol. 29, no. 11, pp. 3121–3133, Nov. 2007, doi: 10.1016/j.engstruct.2007.02.008.

[6] S. Gerasimidis and C. C. Baniotopoulos, "Evaluation of wind load integration in disproportionate collapse analysis of steel moment frames for column loss," *Journal of Wind Engineering and Industrial Aerodynamics*, vol. 99, no. 11, pp. 1162–1173, Nov. 2011, doi: 10.1016/j.jweia.2011.09.009.

[7] M. H. Vafaee and H. Saffari, "A modal shear-based pushover procedure for estimating the seismic demands of tall building structures," *Soil Dynamics and Earthquake Engineering*, vol. 92, pp. 95–108, Jan. 2017, doi: 10.1016/j.soildyn.2016.09.033.

[8] Y. Fu and S. Cherry, "Simplified seismic code design procedure for friction-damped steel frames," *Can. J. Civ. Eng.*, vol. 26, no. 1, pp. 55–71, Feb. 1999, doi: 10.1139/l98-043.

[9] L. Ferrara, M. Faifer, M. Muhaxheri, and S. Toscani, "A magnetic method for non destructive monitoring of fiber dispersion and orientation in steel fiber reinforced cementitious composites. Part 2: Correlation to tensile fracture toughness," *Mater Struct*, vol. 45, no. 4, pp. 591–598, Apr. 2012, doi: 10.1617/s11527-011-9794-x.

[10] A. K. Chopra and R. K. Goel, "Capacity-Demand-Diagram Methods Based on Inelastic Design Spectrum," *Earthquake Spectra*, vol. 15, no. 4, pp. 637–656, Nov. 1999, doi: 10.1193/1.1586065.

[11] "Performance Based Seismic Design of Reinforced Concrete Building." https://www.scirp.org/journal/paperinformation.aspx?paperid=65131 (accessed Sep. 22, 2021).

[12] D. Cardone, "Nonlinear Static Methods vs. Experimental Shaking Table Test Results," *Journal of Earthquake Engineering*, vol. 11, no. 6, pp. 847–875, Nov. 2007, doi: 10.1080/13632460601173938.

[13] S. A. Freeman, E. Associates, and P. Street, "REVIEW OF THE DEVELOPMENT OF THE CAPACITY SPECTRUM METHOD," p. 13, 2004.

[14] H. G. Fu, Y. P. Lei, J. D. Xing, and L. M. Huang, "Investigations on microstructures and properties of B containing cast steel for wear resistance applications," *Ironmaking & Steelmaking*, vol. 35, no. 5, pp. 371–378, Jul. 2008, doi: 10.1179/174328108X271484.