Vol-08 Issue 07, July -2024

ISSN: 2456-9348
Impact Factor: 7.936

International Journal of Engineering Technology Research & Management
Published By:
https://www.ijetrm.com/

LUMPED PLASTICITY MODELLING OF RC FRAME USING MULTI MODE PUSHOVER ANALYSIS

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ABSTRACT
This research study's only goal is to advance the field's prior work on multi-mode analysis by creating the Modal Response History Analysis (MRHA) method as an alternate strategy for determining structural seismic demands. The suggested approach involves building single-degree-of-freedom (SDOF) systems for n number of vibrational mode and integrating the answers from the multi-mode system. The research contrasts the reference time history analysis method with a bypass method for computing target displacement. By contrasting the outcomes with those obtained using the time history technique for four representative building with 4, 8, 12, and 20 stories, the accuracy and efficacy of the MRHA method are confirmed. The importance of lumped plasticity modelling in accurately capturing the nonlinear behaviour of the crucial components of the building is also covered in detail in the study, along with the mathematical background. The nonlinear models of the hinges are used in the proposed MRHA method to precisely estimate the plastic rotation requirements of the structure. The study is carried out with the aid of the SAP2000 programme, and the outcomes show the pushover curves of both modes in orthogonal directions which is the behaviour of the building, as well as the force-deformation relationship that is employed in SDOF systems to take into account the nonlinearity of MDOF systems. The research comes to the conclusion that the MRHA technique can be employed as a relatively precise and effective technique for analysing the earthquake resistance of tall buildings constructed of reinforced concrete since it is computationally quicker than the NLRHA method.

Keywords:
Seismic demands, multi-mode system, Seismic evaluation, MRHA, NLRHA, MDOF

INTRODUCTION
There has been a growth in the building of tall structures all over the world as a result of the need to address the expanding population and the limited area for new construction in cities. However, many existing structures might not adhere to current seismic standards despite advances in our understanding of earthquake risk and the behaviour of high-rise buildings. It is vital to evaluate their dynamic performance in respect to both financial loss and personal safety. A higher number of modes of vibration have a crucial influence in the intricate response to earthquakes of tall structures. Currently, the non-linear time history analysis method, which necessitates a 3D nonlinear computer model, is advised for high-rise building seismic evaluation. However, engineers often employ the nonlinear static (pushover) analysis for structural response determination due to its computational efficiency Traditional nonlinear static methods (NSPs) do not take higher mode effects or the loss of structural component stiffness into consideration. A more straightforward nonlinear static technique (NSP) has been suggested and included in numerous seismic evaluation codes as a substitute to the intricate NLRHA method. In this process, lateral pushover displacement of a building is precalculated, and the pushover transverse story forces are dictated by the basic mode shape (Chopra and Goel, 2004). To ensure safety during earthquakes, buildings need to undergo a seismic examination. ATC-40 (1996), FEMA-356 (2000), and ASCE-41-06 (2007) are just a few recommendations among building design standards that advocate using the streamlined NSP technique for seismic evaluation (Goel, Eeri and Chopra, 2005). With this method, the structure is pushed laterally to a predefined "target displacement" while the gravity loads are maintained. The pushover lateral forces are proportional to the fundamental modal gravity forces. This technique, however, is only appropriate for structures whose response is predominantly controlled by the fundamental vibration mode. According to studies by Lopez-Almansa et al. (2014), Poursha et al., Fragiadakis et al. (2014), and Lopez-Almansa et al. (2014), it is not appropriate for tall buildings with strong higher mode effects ((Najam, 2017).
Building design guidelines and standards for seismic evaluation such as ATC-40 [1996], FEMA-356 [2000], and ASCE-41-06 [2007] support the simplified NSP approach (ASCE, 2000). For towering buildings, when higher mode effects are prominent, this approach is ineffective. Therefore, it is crucial to investigate novel seismic evaluation

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techniques for tall structures that can take higher mode effects into consideration and deliver trustworthy results. The Uncoupled Modal Response History analytical (MRHA) approach is studied in this paper as a streamlined analytical method capable of precisely formulating the nonlinear seismic responses of medium-rise to tall concrete reinforced buildings. The reliability of the modal pushover analysis (MPA) needs to be improved in order to correctly identify higher mode impacts in structural responses, according to earlier studies (Chopra and Goel, 2002). The most accurate technique for buildings subjected to intense ground vibrations is time-history analysis. Additionally, the Capacity Spectrum Method (CSM)-based Multi-Mode Pushover (MMP) analysis was created about 20 years ago, but it lacked the capability to statistically evaluate seismic demands. The MMP approach was later improved by weighting the results of the modal pushover study to calculate structural responses. Chopra & Fafjar created the N2 technique to assess the inelastic seismic needs of planar frame structures utilising inelastic response spectra (Mehmood 2016). The MPA technique was created based on structural dynamics theory to include higher mode impacts in structural responses. Analytical proof of the MPA technique's correctness for general and concrete-reinforced moment-resisting frames has been provided. By viewing intricate dynamic responses of linear multi-degree-of-freedom structures as a sum of vibration modes, each of which resembles a single-degree-of-freedom system governed by modal characteristics, the MRHA approach can be thought of as an improved version of the conventional modal analysis (Adrian-Alexandru, 2020).

2. Background and Motivation

According to accepted practise, the Response Spectrum Analysis (RSA) approach suggests using a response modification factor (R) to consistently reduce the overall demand contribution for every vibration mode (Najam & Warnitchai, 2018). However, investigations have shown that when forced to ground motion, various vibration modes exhibit varied degrees of nonlinearity (Reddy, Mothilal & Reddy, 2023). The yield moment at the base of cantilever wall systems limits the shear force demand attributed with each mode, resulting in varying degrees of inelastic behavior (Suwansaya & Warnitchai, 2023). Higher modes typically display lower amounts of nonlinearity, according to Priestley, even if ductility has a substantial impact on the basic mode response of RC cantilever walls (Sullivan, Priestley & Calvi, 2008).

Priestley and Amaris suggested a modified multimodal superposition (MMS) technique that recommends tailoring the response modification factor only to the requirements of the first vibration mode in order to resolve this discrepancy. A somewhat good prediction of the nonlinear force requirements can be obtained by integrating the inelastic shear from the first mode with significant response from higher modes, assuming that they are elastic. The MMS approach may lead to overestimations of narrative shears and other force demands in the case of frame structures since higher modes can also experience inelasticity, according to subsequent findings.

Using the uncoupled modal response history analysis (MRHA) method that was first created by Chopra and Goel (8), Ahmed and Warnitchai conducted a study to examine the nonlinear response contributions of specific vibration modes (Suwansaya & Warnitchai, 2023). The results showed that a small number of nonlinear single-degree-of-freedom (SDF) systems can contribute substantially to the complicated nonlinear responses of high-rise RC buildings. It is possible to think of the MRHA method as a logical extension of traditional modal analysis to inelastic systems.

The assumption is that the nonlinear dynamic response can still be estimated by using the vibration mode forms of the corresponding elastic system, even though the theoretical underpinnings of modal analysis become invalid in the inelastic region. With respect to modal analysis, this supposition enables the derivation of an uncoupled dynamic equation of motion. In this instance, a nonlinear SDF system is used to represent each vibration mode's response.

3. Theoretical Formulation and Basic Concept of Uncoupled Modal Response History Analysis

3.1 Mathematical Background

In this section, a concise review is provided on the theoretical concepts of the Modal Response History Analysis (MRHA) procedure, with a demonstration of these concepts using a 2D multi-story building. The mathematical formulation of the procedure is adapted from Pennung and Tahir seismic evaluation (Jalilkhani, Ghasemi & Danesh, 2020).

The governing equations for a multi-story shear building subjected to horizontal ground motion $\ddot{x}g(t)$ are expressed as:

$$\boldsymbol{M}\ddot{\boldsymbol{x}} + \boldsymbol{C}\dot{\boldsymbol{x}} + \boldsymbol{f}s\,(\boldsymbol{x}, \dot{\boldsymbol{x}}) = -\boldsymbol{M}\boldsymbol{l}\ddot{x}g(t) \qquad \textbf{(3.1)}$$

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The theoretical foundations underlying the Modal History Non-linear analysis technique are briefly reviewed in this section, and these theoretical underpinnings are demonstrated using a 2D multi-story architecture. The mathematical design of the process is based on a seismic evaluation by Pennung and Tahir.When the structural reactions stay within the elastic limits of the building system, fs = Kx is applicable in the context of modal response history analysis (MRHA). In this equation, M and C stand in for the building's mass and damping matrices, respectively, and x stands for the vector of N lateral floor displacements with respect to the ground. The lateral resistive force vector of the building system is represented by fs, while the influence vector, l, comprises unity elements.

By expanding the floor displacements as a sum of modal contributions, we obtain:

$$x(t) = \sum_{i=1}^{N} \phi i q i(t) \qquad (3.2)$$

In the equation above, qi(t) stands for the ith modal coordinate, and i represents the vector carrying the ordinates of the ith fundamental vibration mode shape of the building within its linear range. Link is used to model the SDOF system with lateral loading in SAP 2000 in order to precisely forecast the response of ground motion with varying intensities. The results of the analysis will reveal the absolute maximum displacement (Nonlinear Modeling using PERFORM 3D).

The governing equation of SDof system is represented by $D_{n.}$

$$\ddot{D}i + 2\xi i\omega i\, \dot{D}i + Fsi\,(Di, \dot{D}i)/Li = -\ddot{x}g(t) \qquad (3.3)$$

A system's mass, damping, and stiffness matrices can be calculated using modal analysis. To lessen a structure's dynamic reactivity, tunable mass damper systems can also be used. The MRHA approach delivers enhanced efficiency and precise seismic demand estimation by building on prior studies. To achieve this, the first equation of motion is transformed into a nonlinear form by introducing a nonlinear stiffness matrix, enabling the derivation of uncoupled equations that incorporate modal participation factors and the spatial distribution of summation of every mode forces.

3.2 Proposed Pushover Analysis Modification

The MRHA process employs a multiple stages modal pushover analysis technique, in which the starting structural state at the start of each stage is the same as the conclusion of the stage before it. Up until the roof story displacement reaches the target displacement, the lateral load pattern in every step is in proportion to the elastic mode-shape vector and is displacement controlled. The nonlinear Response History Analysis (RHA) of the associated inelastic modal Single Degree of Freedom (SDOF) system is used to estimate the goal displacement for each mode. By combining the modal responses obtained from the multi-stage pushover studies with the help of the proper modal combination rule, the final structural responses are established..

The step-by-step process of modeling, analysis, and findings using the MRHA technique is as follows:

1. Create an equation-based representation of the structure that includes the deterioration characteristics and monotonic nonlinear behaviour of the structural elements. The base shear-roof displacement (Vbn-Xr) or pushover curve for the nth mode of the structure should be determined using a typical nonlinear static analysis employing the lateral load pattern in the shape of the first two modes.
2. Through two further procedures, idealise the resulting pushover curve as a bilinear curve.
   - 2.1 To match the elastic linear segment of the pushover curve, draw a straight line (Line 1) through the origin.
   - 2.2 At the maximum base-shear capacity, Vbn,max, draw a straight line (Line 2) that cuts through the pushover curve. Consider the point on the pushover curve where a 20% reduction in base-shear capacity is seen as the ultimate roof displacement, Urnt. Construct the inelastic SDOF system for the nth mode of the structure. This system has mass corresponding to the time-period and frequency of the MDOF system and follows the force-deformation relationship determined in the previous step. Additionally, consider a damping ratio equal to the value assigned to the nth mode

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of the original Multiple Degree of Freedom (MDOF) structure (ζn) for this SDOF system.Compute the maximum displacement of all eight SDOF models using the governing equations (Dn). Calculate the target displacement using the above formula.

3. Determine the force-deformation relationship using the Capacity Spectrum Method (CSM).
4. Create the inelastic SDOF system for the structure's nth mode. This system follows the force-deformation connection established in the preceding stage and has mass corresponding to the time period and frequency of the MDOF system.
5. Consider a damping ratio for this SDOF system that is equal to the number given to the nth mode of the original Multiple Degree of Freedom (MDOF) structure (n).
6. Utilise the governing equations (Dn) to calculate the maximum displacement for each of the eight SDOF models.

The Capacity Spectrum Method (CSM) for seismic risk assessment was developed in this study as a result of a pilot project for the US Navy carried out in the early 1970s at the Puget Sound Naval Shipyard (Freeman et al., 1975). Freeman (1998) provides considerable documentation of the evolution of this approach. Although the original technique has been altered, the core idea has not changed. Through an iterative procedure that involves the analysis of numerous analogous linear systems with lateral stiffness equal to the secant stiffness, the maximum inelastic displacement is estimated.

The base shear (Vb) and roof displacement (xr) are converted point-by-point into the equivalent spectral acceleration (SA) and spectral displacement (SD) in order to transform the capacity (or pushover) curve into a capacity spectrum. The conversion is achieved using the following expressions:

$$\mathbf{S_a = Vb * W / Cm} \qquad \textbf{(2.14)}$$
$$\mathbf{S_d = xr * \Gamma 1 * \varphi 1 * r} \qquad \textbf{(2.15)}$$

Here, W stands for the building's seismic weight, which is made up of the total of its expected living and dead loads. The first vibration mode's modal mass coefficient and modal participation factor are denoted by the letters Cm and Γ1, respectively, while φ1, the roof level amplitude of the first vibration mode shape is denoted by the letter r. A distinct set of spectral acceleration (Sa), spectral velocity (Sv), and spectral displacement (Sd) values are associated with each point on the capacity spectrum curve.

3.3. Time history analysis

The dynamic behaviour of structures under dynamic loading situations, such as earthquake, wind, or other types of excitations, can be studied using the time history analysis approach. The process entails replicating the loading conditions on a structure for a predetermined amount of time and examining the structure's response at different points in time. Time history analysis' fundamental tenet is to express the dynamic load on a structure as a function of time, commonly referred to as a load time-history, and then utilise this function to examine the structure's reaction to that stress. Different techniques, such as movement of the ground records or the use of wind tunnel tests, can be used to measure or determine the load time history.Numerical techniques like the boundary element method (BEM) or the finite element method (FEM) are used to simulate the structure in the time history analysis. The reaction of the structure is assessed at various points in time after the model has been subjected to the load time-history. Depending on the structure and the type of load, the analysis can be done in the time or frequency domain. One of the key benefits of the analysis of time history is that it enables engineers to examine how a structure behaves under actual loading circumstances while accounting for load fluctuations over time. The structure's ductility, seismic resistance, and damage resistance can all be assessed using the analysis's findings. To effectively capture the reaction of the structure, time history analysis needs a lot of time steps, which might be computationally taxing. The analysis might become more challenging because it also needs a clearly defined input motion and an accurate model of the structure.

4. Selected Building and Ground Motions

4.1 Modelling parameters for Selected buildings

For the detailed analysis utilizing the MRHA and NLRHA procedures, we have selected four moment-resisting frames without infills, namely B1, B2, B3, and B4. These frames serve as representations of high-rise buildings

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Impact Factor: 7.936

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with varying heights and fundamental periods. The buildings range from 4 to 20 stories, with fundamental periods ranging from 1.29 to 3.71 seconds. The study focuses on an industrial district comprising multiple buildings, each featuring distinct frame structures. Among them, one building stands out with a 4-story frame and 9.14-meter bays, while the others have 6.10-meter bays. The typical story height is 3.96 meters, but the highlighted building possesses a taller first story measuring 4.57 meters. Isometric and elevation view of the model displays their distinctive architectural styles.
Each building takes on the dead load and 25% of the live load in terms of seismic masses. Concrete's compressive capacity of 30 MPa is assumed for the beam members and 35 MPa for the column members. The reinforcement's minimum yield strength is specified at 450 MPa. The reinforced concrete special moment-resisting frames (SMRF) is the chosen structural solution in all scenarios. According to IS CODE 1893-2016, the example structures are situated in Zone IV with a 1.5 significance factor, medium soil conditions, and a response reduction value R equals to 5. This strong-column weak-beam (SCWB) concept and the IS building rules are used to design every building, which also satisfies the drift requirement. For each beam-column junction, the beam and column members are created to produce a column-to-beam strength factor greater than 1.2. The specifications of RC SMRFs as stated in IS 456 are followed in the reinforcing details of the beam and column elements. We make use of SAP 2000 functionalities to create a two-dimensional inelastic system of SDoF system model of each example frame. A SDoF system whose mass is lumped at the top and plastic hinge elements are used to simulate the beam and column members; these elements consist of an elastic section with two lumped plastic hinges at each end.

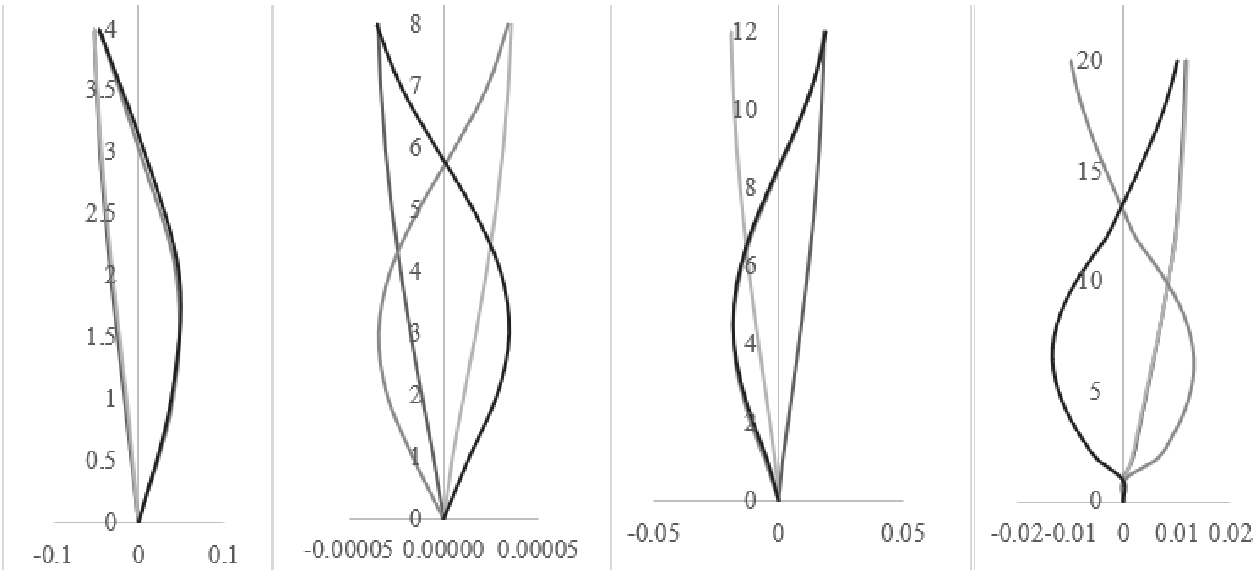

Fig.1. The elastic mode-shapes of the example frames for the first two modes in both the orthogonal directions.

The elastic section of the beam-column members is modelled using the elastic Beam-Column element as defined by SAP software. Nonlinear zero-length rotating springs that exhibit stiffness and strength deterioration characteristics are used to mimic the plastic hinges. The Mander's degradation model is used to define the force-deformation (backbone) curve of the plastic hinges.
The mode shapes of assumed four buildings are shown in Fig.1. The figures describe the two modes in orthogonal directions X and Y and we can see the variation pattern of shape of building when analysed as free vibration not forced vibration. The elevated view of the B1, B2, B3 and B4 are shown in Fig.2 with bay length and storey heights.

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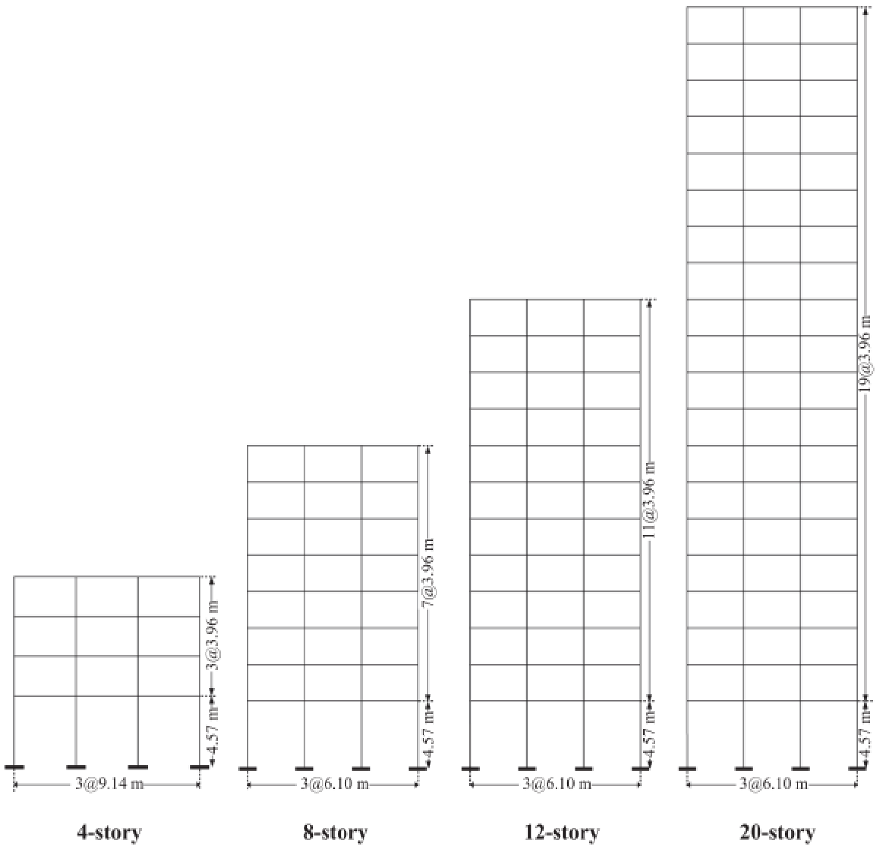

Fig.2. Example structural models with story height and bay measurements.

4.2 Selection of Ground Motion Records

The matching process is of paramount importance as it ensures that the ground motion records accurately represent the seismic characteristics expected in the region under consideration. By adjusting the damping ratio and adhering to the Indian Standard Code, the matched ground motion data provides a reliable and realistic representation of the seismic loading that the structures would experience in the study area. The use of seismomatch software enables the creation of a wide variety of ground motion records with different properties. This adaptability enables researchers to examine how structures respond to various earthquake conditions, advancing our understanding of their behaviour and effectiveness.

In summary, the analysis part of the research paper highlights the ground motion data used in the study. The downloaded peer ground motion data, generated using seismomatch software, is carefully matched to comply with a damping ratio of 5% and the seismic provisions specified in the IS 1893-2016 Indian Standard Code.

Four 'far-field' ground motion recordings, namely 'RSN 71', 'RSN 125', 'RSN 164', and 'RSN 787', are selected from the PEER Ground Motion Database in order to assure a thorough study. In order to effectively represent the seismic loading encountered in the research area, these ground motion records were carefully chosen, allowing for a trustworthy evaluation of the structural reaction to earthquakes. The moment magnitudes of the chosen ground motions are in the range of 6.0 to 7.0, reflecting a variety of seismic occurrences of various intensities. Additionally, these ground motions are relevant to the seismic features of the research area because their closest distance to the fault rupture is between 18.2 and 36.4 km. The chosen ground motion records are scaled using Seismomatch software to assure consistency and homogeneity. The main properties of the ground motions are preserved while allowing for adjustments during the scaling process. The study can effectively depict the effects of various seismic events on the structures under consideration by scaling the ground motions appropriately.

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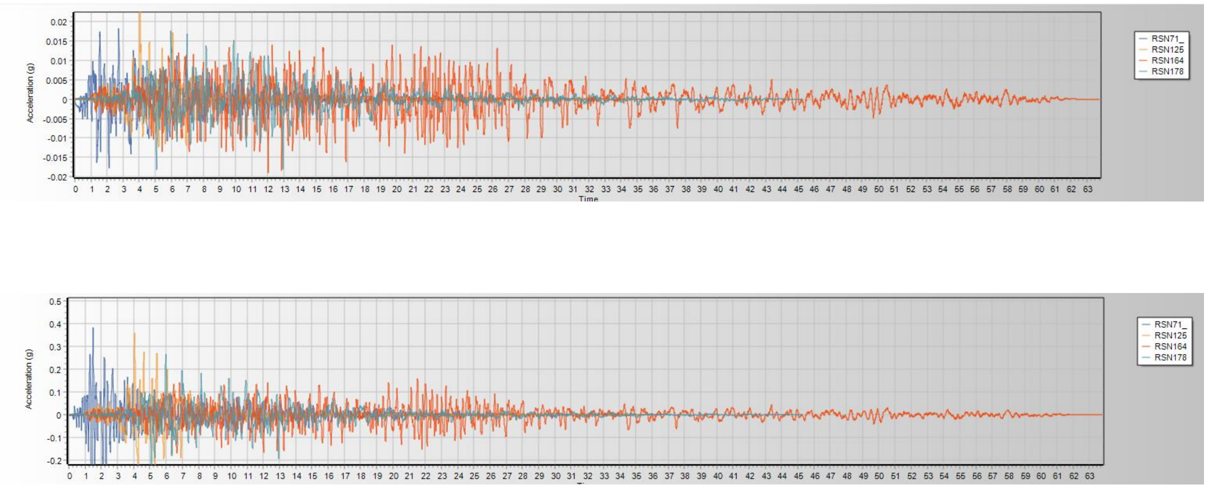

Fig.3. (a) displays the ground motion data from downloaded peers' acceleration over time. In accordance with the IS 1893-2016 Indian Standard Code,
(b) displays the acceleration time history of the matched ground motion while taking a 5% damping ratio into account.

Seismic probabilistic hazard is a technique used to determine the seismic vulnerability efficiently. According to this a set of ground motions are selected which fulfil the hazard characteristics of the site with some probable error. In this study ground motions are peer downloaded and shown in Fig.3.

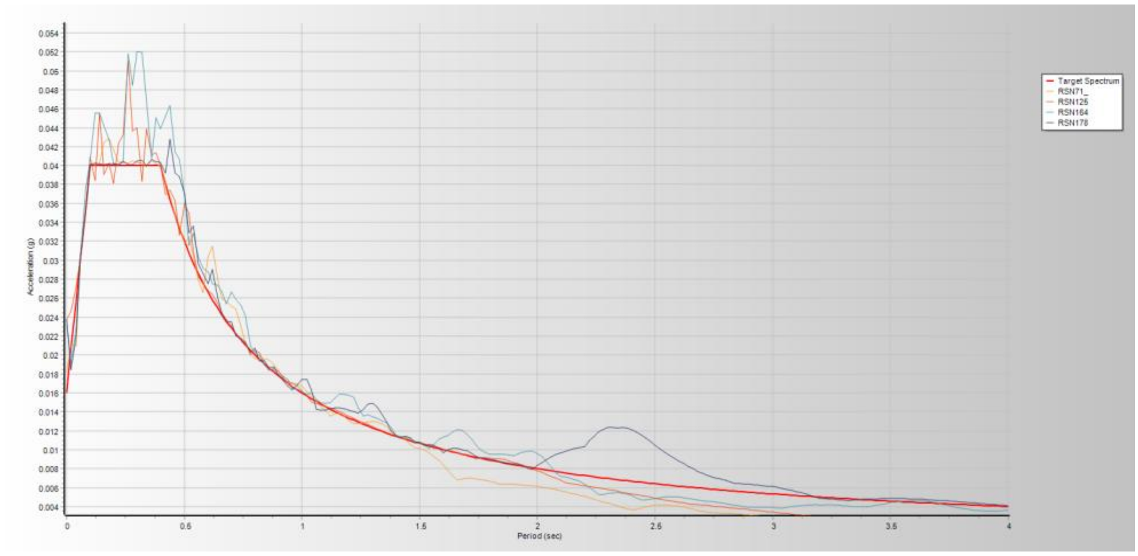

Fig.4. Acceleration response spectrum for each ground motion record, scaled to the target spectrum

5. ANALYSIS

5.1. Nonlinear Modelling of Buildings

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The research paper's analysis section delves into the step-by-step procedure of Modal Response History Analysis (MRHA), elucidating its methodology and significant findings such as the pushover curve and target displacement. To ensure accuracy and reliability, each ground motion record undergoes individual application of the MRHA procedure. The goal displacement for the example frames is established by taking into account the effects of the first two vibration modes and evaluating their suitability using the modal mass requirement of at least 90% provided in ASCE 7-10. The findings of both "single-stage" and "multi-stage" modal pushover analyses, with the latter containing the first two vibration modes for all frames with fundamental periods lower than 3.7 seconds, are combined to provide a thorough evaluation of the structural behaviour. A benchmark solution for each sample frame is also provided by Time History Analysis using the numerical Newton-Raphson approach, using three chosen ground motion recordings from Fig. 4.
When employing the MRHA technique, a building model is created using a programme like SAP2000, with the auto-assignment of hinges modelling the structure's nonlinear behaviour. For the beam and column elements, respectively, plastic hinges at 5% and 95% of the member length capture the nonlinear behaviour of the structure. The pushover curve, representing the lateral force-displacement relationship, is idealized using a 2-point link in SAP2000, with the force deformation relationship provided in Table 4. The target displacement, a vital parameter in the MRHA procedure indicating the expected failure displacement, is investigated for the nonlinear models of four assumed buildings, with beam and column elements modeled as plastic hinges. The incorporation of nonlinear models and the utilization of software tools like SAP2000 enhance the accuracy and reliability of the MRHA process, providing a comprehensive framework for analyzing and retrofitting structures to meet seismic demands. Non linearity of any structures plays a crucial role in this study as its been proved that pushover techniques and decoupling of equation of motion from MDof to SDof leads to a limitations i.e., these procedures can only be applied on symmetrical linear models. Thus, it’s been a paramount goal to achieve accurate results using these techniques for tall high-rise structures.

Table 1
Parameters used in Capacity Spectrum Method for fundamental mode in X direction for idealisation.

Building	Seismic Weight (W) (kN)	Modal Mass coefficient (C_m)	Modal participation factor (Γn)
B1	18113	0.826	25.839
B2	35079	0.814	36.116
B3	53570	0.821	48.6
B4	72790	0.837	47.815

Table 2
Parameters used in Capacity Spectrum Method for second mode in X direction for idealisation.

Building	Seismic Weight (W) (kN)	Modal Mass coefficient (C_m)	Modal participation factor (Γn)
B1	18113	0.11	9.52
B2	35079	0.99	12.69
B3	53570	0.98	17.1
B4	72790	0.97	17.9

Table 1 and Table 2 are the results of modal analysis of all 4 models using SAP 2000 software. Using above table we can evaluate the target displacement in next chapter. Above tables provides the important parameters for manually calculating target displacement.

5.2. Equivalent SDOF Systems

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Modal analysis is done to get the modal properties of the four buildings, such as frequency, time period, modal participation factors, and modal mass participation. Pushover analysis is then done as a load pattern in the shape of the first and second mode in both the orthogonal directions. The analysis results of both modes are evaluated at 0.4% of the building height as monitored displacement. After retrieving 8 pushover curves in X-Y directions respectively, the idealized force and deformation relationship of the 'nth mode' inelastic SDOF system is determined based on the bilinear idealized pushover curve. Six SDOF models are then constructed in SAP 2000 using 2point link elements, and the nonlinear parameters F-D values are assigned, which imparts the nonlinear behavior of the MDOF system in the SDOF system. The Time History Analysis is then run using the matched acceleration time history as ground motions. The optimum roof level target displacement for both the modes (Urnt) is established, and using these displacement value of all 4 models we can get every demand that is required for retrofitting of buildings. The inelastic SDOF model (Dn) displacement will be obtained after the time history analysis under matched ground motion using seismomatch. The absolute maximum values of roof story label point displacement will be our final bypass method target displacement.
Table 3 gives the values used during modelling of SDof systems in SAP 2000 software. The table consists of force and displacement values of non-linear link. F-D values from tables are going to bring non-linearity in models and accumulate the piece-wise linear behaviour of complete building in two modes.

Table 3: Force-Deformation Relationship for Joint Link in SAP

Building	Mode	X Force	X Disp.	Y Force	Y Disp.
B1		0.17	1.216	0.19	0.953
B2	1	0.42	0.67	0.215	0.564
B3		0.58	0.53	0.6	0.48
B4		0.59	0.54	0.6	0.48
B1		0.17	0.133	0.038	0.074
B2	2	0.24	0.065	0.25	0.065
B3		0.55	0.058	0.58	0.51
B4		0.55	0.05	0.58	0.51

Note: The displacements are represented in meters, and the forces are represented in kilonewtons.

7. Evaluation

The evaluation of the developed procedure i.e., MRHA is based on developing the mathematical background and develop a bypass method to get seismic demands of the existing buildings. The pushover curve for two modes in both the orthogonal direction X and Y is shown in the paper, this curve will act as a fundamental behaviour of SDOF system. The non linearity in the buildings and variation in stiffness, degradation of stiffness, peak roof story joint displacements behaviour are various parameters which will define the behaviour of SDOF system.

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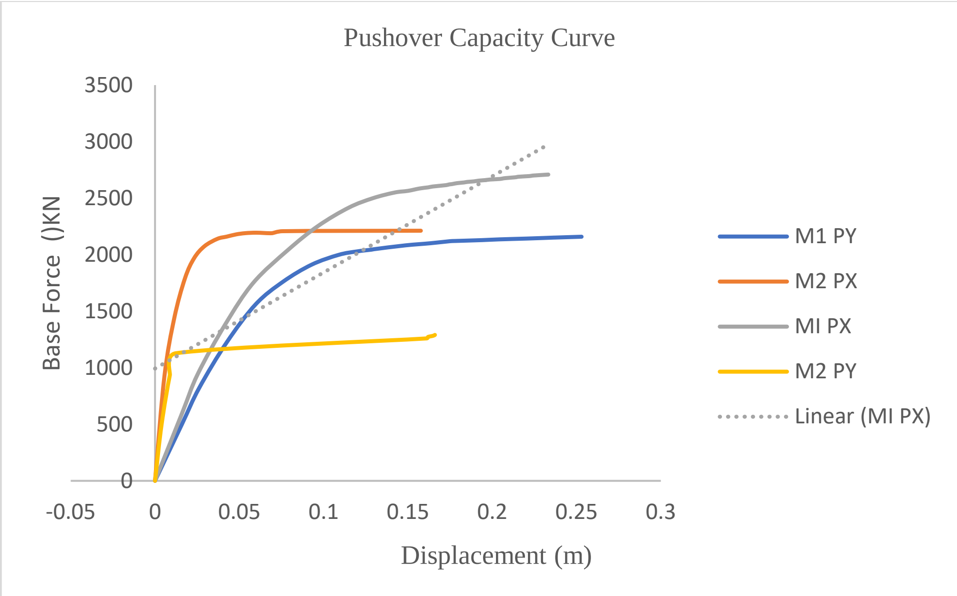

Fig.5. Base shear vs Displacement graph of B1 for mode 1 and mode 2 in both the orthogonal directions.

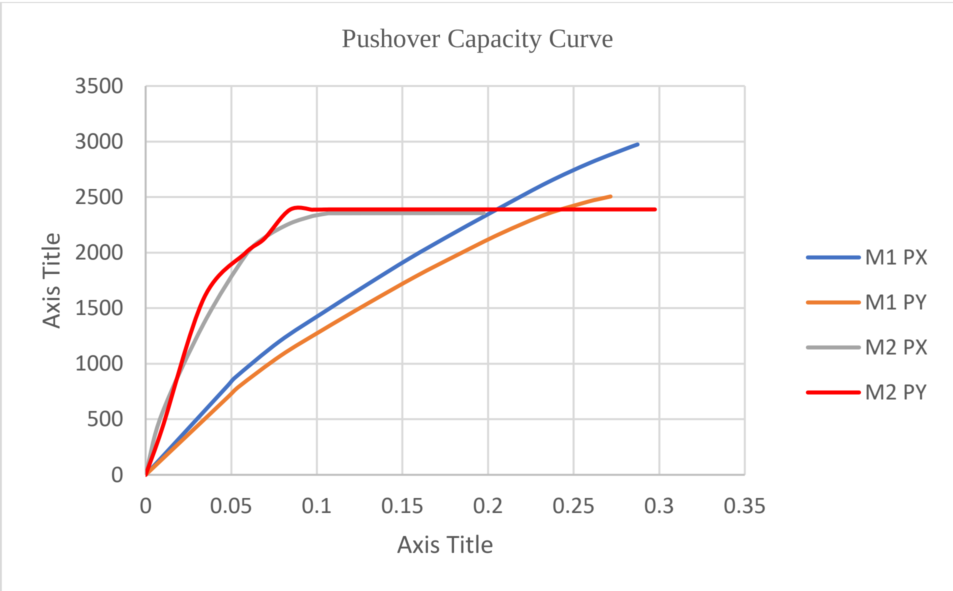

Fig.6. Base shear vs Displacement graph of B2 for mode 1 and mode 2 in both the orthogonal directions.

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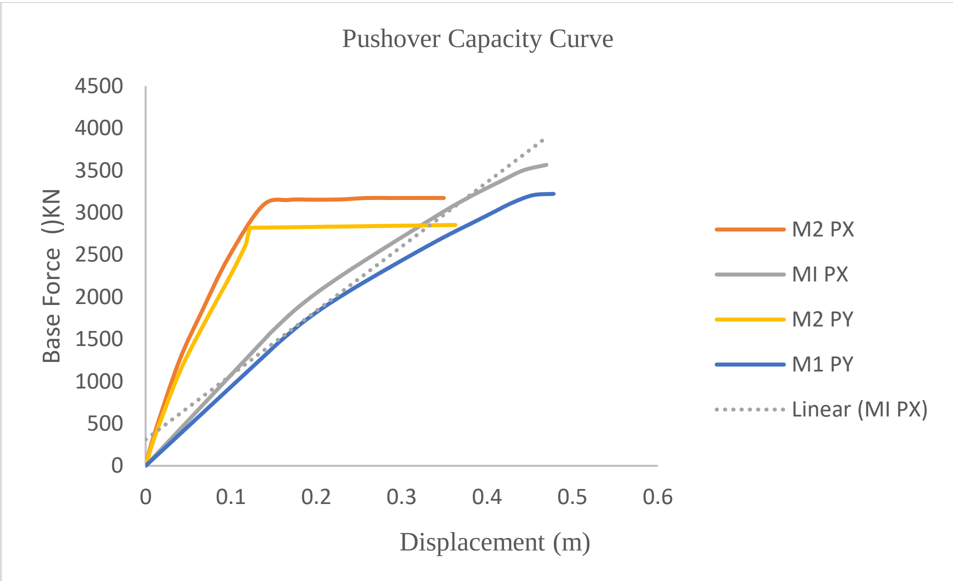

Fig.7. Base shear vs Displacement graph of B_3 for mode 1 and mode 2 in both the orthogonal directions.

Base shear is the maximum lateral shear at the base of the structures due to the applied ground motion. Pushover curve depicts the behavior of complete structures using base shear and displacement of roof storey values. Fig 5 and Fig.6 clearly gives and idea about the failure criteria and when and which part of the structure will fail at first. These curves give a general idea according to our requirements. Assumed models are low to high rise buildings, in Fig. 5 and Fig. 6 contributions from both the modes are different but they are same if we compare the contributions of the individual modes in their respective orthogonal direction X and Y. Curves of Fig 7 and Fig 8 (B3 and B4 model) perform a little different pattern. These curves modal contribution are similar but contributions in their respective orthogonal directions are more for mode number 1.

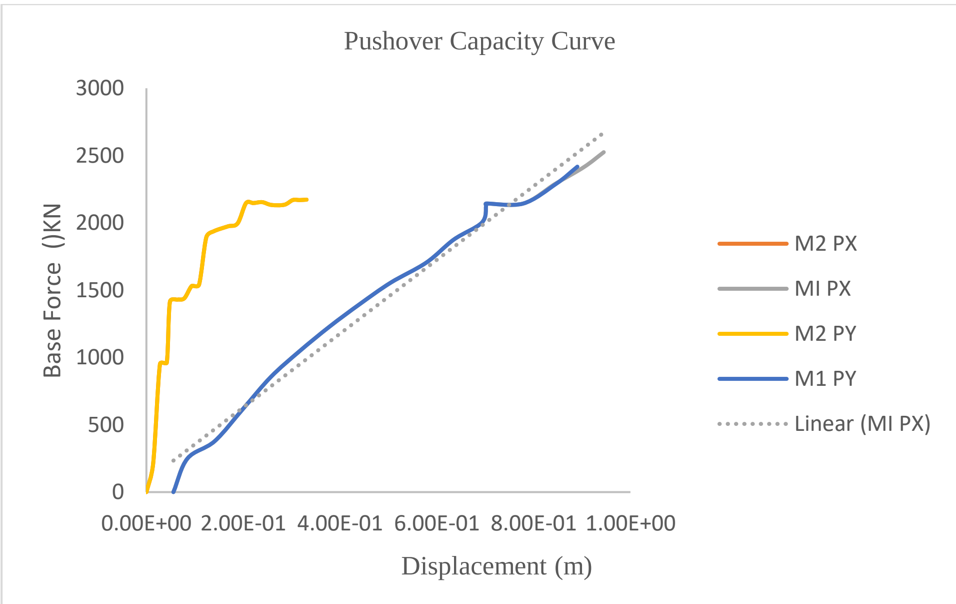

Fig.8. Base shear vs Displacement graph of B_4 for mode 1 and mode 2 in both the orthogonal directions.

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The MRHA approach is able to precisely forecast the target displacement of the structure, as evidenced by the two-way comparison of demand displacement in MRHA and Time History Analysis. The findings demonstrate that the target displacement predicted by the MRHA method and that projected by the NLRHA procedure are extremely close. As a result, it can be concluded that the MRHA process is a trustworthy and accurate way to assess the nonlinear seismic reactions of tall structures made of reinforced concrete.
The computational calculation of the target-displacement using the MRHA technique for the 12-story sample frame is summarised in Table 4.

Table 4 Elastic Modal Parameters and Target Displacement

Elastic Modal Parameters	Target Displacement	
	Mode n =1	Mode n=2
ω_n n (rad/s)	2.07	6.43
Φ_{rn}	0.019	0.019
Γn	48.6	17.07
D_n (mm)	236.95	178.5
$U_{rnt} = \Phi_{nr} \Gamma n\, D_n$ (mm)	218.8	57.9
ut (mm)	276.7	

The target displacement (ut) for this frame is calculated using the elastic modal parameters of the structure, including time period, frequency, and modal mass participation factor (n = 1 and 2).
A comprehensive forecast of hinges at crucial points in the construction is provided by the plastic hinge acceptance criteria. The position and size of non-linear hinges in the construction are decided using the plastic hinge acceptance criteria. Initial findings show that the plastic hinge acceptance criteria can accurately predict where the hinge will be and what will happen to it when it is loaded; the inherited stiffness degradation curves will be the behavioural attribute for how flexible the hinge position will be. This information is necessary for construction planning and retrofitting to withstand seismic forces.
Initial results display that the plastic hinge acceptance criteria may properly anticipate the location and its property when subjected to load, the inherited stiffness degradation curves will be the behavioural property for extent of plasticity in the location of hinges. This data is critical for planning and retrofitting structures to resist seismic loads.
When MRHA and NLRHA analysis times are compared, the MRHA method requires a lot less computational time than the NLRHA process. This is because the MRHA method, which requires less computational effort than the NLRHA method, generates the result of each particular vibration mode. Because of this, evaluating the nonlinear earthquake response of tall buildings made of reinforced concrete may be done more effectively and practically using the MRHA process.
The NLRHA approach requires roughly 8 hours to process a single building with a single ground motion record on a workstation with a 3.4 GHz processor and 16.0 GB of DDR memory. In another two hours, the computed dynamic reactions must be transformed into the final format (Fig. 5 or 6). It takes around an hour to complete three vibration modes' pushover tests, ten minutes to evaluate three equivalents nonlinear SDOF systems, and twenty minutes to convert estimated response into the final MRHA technique format. The MRHA approach takes significantly less computer effort than the NLRHA method, there is no doubt about it. Due to the small computational effort required, we will effectively be able to grasp the nonlinear response of both low and high-altitude buildings to various probable ground motions.

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8. Conclusion

The MRHA process is a trustworthy and accurate way to assess the nonlinear seismic responses of tall buildings made of reinforced concrete, in conclusion. The plastic hinge criteria for acceptance are a crucial tool for determining where and how many plastic hinges will be used in the construction. The MRHA process is more effective and practical for assessing the nonlinear seismic responses of tall concrete- reinforced buildings because it involves substantially less computer work than the NLRHA procedure. Important elements in this process include the creation of SDOF models in SAP 2000, the use of modal analysis and pushover analysis, and more. The structure's peak displacement and final target displacement are then calculated using the NLRHA.
Important takes from the research are discussed in points.

1. This MRHA analysis will be the efficient and much easy way to develop various forces and determine the seismic demands at desired seismic hazards conditions.
2. Hinges which are defined in beams and columns are going to help in understanding the acceptance criteria which we will define on the program. This acceptance criteria and assignment of hinges at ends of sections with a plastic hinge length are going to be the base of my paper.
3. Developing a SDOF system for every mode of a structure which contributes to the higher mode responses for tall RC shear buildings.

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