Comparative Assessment of RCC Buildings with Special Moment-Resisting Frame Using Static and Dynamic Procedures of Both Linear and Nonlinear Seismic Analysis

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Abstract

Advances in seismic design concepts have led to the development of sophisticated methods. The goal of this paper is to analyse the structure using different analytical procedures and compare the structural responses obtained. Linear Seismic Analysis (Response Spectrum Analysis) and Nonlinear Seismic Analysis (Pushover Analysis and Nonlinear Time History Analysis) methods were used as seismic analysis procedures. Seismic analysis was performed on the two symmetric Bare Special Reinforced Moment Frame models, having storey numbers of 5 and 10, respectively, using ETABS(v22) software. Provisions of IS 1893:2016 and ATC-40 are considered for performing Response Spectrum and Pushover analysis, respectively. Three ground motion records are provided to structural models as input to perform NTHA. The paper provides global seismic responses like storey drift and storey displacement from all procedures and concludes with a comparison of results among these procedures.

Keywords*—Seismic Analysis, Pushover Analysis, Performance Point, Nonlinear Time History Analysis, ETABS*

1. INTRODUCTION

Earthquakes, being unpredictable but destructive and hazardous natural events, lead to catastrophic infrastructure destruction, human fatalities, and economic collapse (1). Thus, the severity of earthquakes is to be considered while designing structures, especially on earthquake prone zones. With the progression of time, different design philosophies were introduced with the objectives of determining how the structure acts against seismic demands. Strength-based design was the initial seismic design concept, which defined earthquakes as ultimate-strength events and conceptualised the inclusion of inelastic behaviour of the structure (2). Strength-based methods were supplanted by limit state design, which deals with both structural strength and serviceability of the structure (3). The limit states design was then superseded by a more robust approach called Performance-Based Seismic Design (PBSD). PBSD is simply a philosophy of design of structures, based on the predictability of future seismic events by utilising the inelastic response of the structure rather than relying on uneconomical elastic demands, with the belief that "the stronger the structure becomes, the more capable it becomes to resist large earthquakes." (4,5). With the advancement in seismic design concepts, different analytical procedures were also conceptualized (6–10). Generally, seismic analysis can be categorised into two main types: Linear Seismic Analysis (LSA) and Nonlinear Seismic Analysis (NSA).

Some of the notable building codes adopting LSA are ASCE/ SEI 7-22 (US), EN 1998-1 (Europe), IS 1893:2016 (India), NBCC (Canada), and NZS 1170.5 (New Zealand) (11–15). Similarly, Nonlinear Static Procedure (NSP) and Nonlinear Time History Analysis (NTHA) are popular inelastic methods. NSP are still in the developing phase, as different research studies have been published to replace conventional methods (N2, Capacity Spectrum, and Displacement Coefficient) with an improved advanced version that is more capable of determining seismic demands and has more insights into structural behaviour (16). Performance Analysis (PA) following the procedures of the Capacity Spectrum Method, was used by FEMA356 and ATC-40 (CSM), which was then superseded by FEMA 440 following the Improved Capacity Spectrum method (14,17–20). FEMA440 and ASCE 41-17 provided improvised guidelines for Displacement Coefficient methods (DCM) superseding procedures of FEMA356. Meanwhile, the N2-method is being adopted by EN1998-1 for conducting PA. Similarly, NTHA is a sophisticated, powerful approach for solving equations of motion for multi-degree of freedom (MDOF) systems with the help of specific integration methods and proposed iterative algorithms, which can estimate the dynamic response of the structure considering damping effect or cyclic loading and can simulate inelastic behaviour of the structure (21). Since NTHA is a complicated approach, powerful software tools, such as ANSYS, ABAQUS, LUSAS, OpenSees, CSI-based softwares, and Perform 3D, can be used for solving large computational equations (22–26). These advanced software uses sophisticated modelling techniques to model a realistic behaviour of the structures to capture the appropriate responses of the real structure under action forces.

Different studies were conducted to evaluate the structural performance of various types of structure under seismic events with the help of seismic analytical procedures. Krawinkler and Seneviratna (27) conducted a study on a 4-storey steel perimeter frame structure which was severely damaged in the Northridge earthquake. They performed NTHA using nine ground motion records to achieve local and seismic demands, which were compared with the corresponding demands obtained from PA. The conclusion was that carefully performed PA could provide better foresight of seismic demands, especially for low-rise structures where higher modes of effects are not important and the structure has a uniform distribution of inelastic behaviour over the heights. However, when they conducted a similar study on structures from two to forty stories, it showed that for taller structures with higher modal effect, structural responses obtained from PA highly deviate from the results of NTHA. In a similar manner, other research studies have also shown the utility of PA to be favourable for low-rise structures having lower values of fundamental natural period of vibration (28–30). Gupta (31) had proposed an adaptive modal pushover procedure which could account for the effects of higher modes and performed analysis on different structures varying in lateral load resisting systems. The obtained responses from the method are compared with NTHA, and were able to capture important response attributes even for irregular structures. However, the study found the limitations of the method when dealing with shear-critical systems. Worku and Hsiao (32) proposed a simple procedure of modified first-mode pushover analysis (MFPA) for assessing seismic performances of steel buildings with various structural heights, and when results such as storey drift and plastic-hinge distribution over building heights were compared with NTHA, the results from the simple NSP procedure matched those of NTHA. The seismic performance assessment of the structures is challenging process (33), and even with valuable design guidelines, there is a crucial need of comparative assessment of all types of the structures. This study investigates symmetric Bare Special Reinforced Moment Frame (SRMF) for seismic assessment under different analytical procedures (RSA, PA, NTHA) associated with design guidelines of IS 1893-2016. 5 storey and 10 storey models are taken as reference models to evaluate the performance variation among low rise and mid-rise structures. The performance is evaluated on the basis of seismic responses such as drift, displacement, and base shear. Further, the comparison of performance of the structures under each seismic procedures is carried out to investigate the preferability of the given procedures.

2. METHODOLOGY

A. Overview of Seismic Analytical Procedures

Most of the seismic design guidelines around the world include provisions for linear/elastic design of structures (34). Linear Response spectrum analysis (RSA) is the most common method that is used for predicting structural responses under seismic excitations. The response spectrum summarises the peak responses of all possible linear single degree of freedom (SDF) systems to a particular component of ground motion. The linear elastic model of a structure is then analysed as an SDF system, which undergoes modal analysis to determine the modal properties of the system and provides different particular modes that reflect the numerous forms of structural response in the frequency domain. Further, each individual's responses are combined to generate the total structural reaction. Some of the common combination methods of the responses of different modes are Absolute method, Square Root of the Squares (SRSS) and Complete Quadratic Combination (CQC). Since RSA performs as linear analysis, it fails to comprehend the nonlinear behaviour of the structures. Generally, nonlinear procedures are required for performing inelastic analysis of the structures.

Nonlinear static procedures (NSPs), also termed as Pushover analysis (PA), is a static inelastic analysis that can estimate the performance of the structural system by determining deformation and strength demands, and can derive an association between demands and capacities at each performance levels (27). The involvement of various performance parameters such as interstorey drift, global drift, and inelastic element and joint deformation validates the application of PA for precise estimation of performance levels. In conventional first modal inertia PA (19), the structure is subjected to lateral forces which exhibit monotonic increase with respect to targeted displacement. The application of such load patterns in the structure has the intention of representing the distribution of inertial forces developed from the fundamental lower modes of earthquake throughout the structure. In PA, structure is assessed on basis of its performance point, which generally is a point where the demand curve and capacity curve of the structures meet. The performance of the structure can be determined through different levels, in which each level dictates the degree of severity of damage. The discrete structural performance levels are Immediate Occupancy (IO), Life Safety (LS) and Collapse prevention (CS). Each performance level is associated with the applicable acceptance criteria, where force and deformation actions are evaluated for each structural component for the appropriate strength and capabilities required for the entire structure to exist on the given performance levels. The performance levels associated with acceptance criteria for a given structure can be represented from the force-displacement relationship, which could be represented as a capacity curve of the structure required in PA. Various methods are integrated in different seismic codes, which provide an understanding of how structures respond under seismic forces. The Capacity Spectrum Method (CSM) ATC-40 (35–37), Improved Capacity Spectrum Method (ICSM) FEMA-440 (18,38), N2 Method EC-8 (14,39), Displacement Coefficient Method (19,39,40), Improved Coefficient Method (18), Energy Based PA (41,42) are some of the well-known techniques which comes under first modal inertial PA methods (16). It is to be noted that conventional PA neglects other sources of energy dissipation other than material straining, such as kinetic energy, viscous damping and duration effects (43). Such limitations can be countered using more advanced pushover procedures such as Adaptive PA (44), Multimodal PA (9,45), Improved Modal PA (46), Generalised PA (47), Upper-Bound PA (48) and Spectrum-Based PA (49).

The structure can be seismically analysed by Nonlinear Time History Analysis (NTHA) for simulating the dynamic response of structures under seismic motion, and for accounting nonlinear behaviour of structural elements including yielding, cracking, failure, and SSI effects. (21,50–53). NTHA is applicable to irregular as well as complex structures, and is more dynamic and accurate when compared with PA. The geometric and material nonlinearity are considered when simulating the dynamic response of the structure under seismic loads. The damping models and hysteresis models are also integrated for NTHA analysis to determine the possible energy dissipation of the structure under given seismic forces. This study considered RSA, PA, and NTHA as seismic procedures for assessing the performance of the given structural models to understand both the elastic and inelastic nature of the structure, along with static and dynamic responses of the structures to the seismic demands.

B. Structural Modelling

Table 1: Building Features

Software	ETABS (v22)
Lateral Load Resisting System	RC Buildings with Special Moment Resisting Frame (SMRF)
Support Condition	Fully Restrained (Fixed Base)
Number of bays	3 (Both X and Y direction)
Bay Dimension	4m × 4m
Column size	350mm × 450mm
Beam size	300mm × 400mm
Slab thickness	125mm
Storey height	3m
Concrete Grade	M25 (Material properties are in accordance with IS 456:2000)
Rebar Grade	Fe415 (Constitutive relationship for reinforcing steel is in accordance with IS 456:2000)
Wall Load	Not Considered
Slab Load	3 kN/m^2 + 2 kN/m^2 (Live Load + Floor finishing Load)

Two symmetrical structures were modelled by following the procedures of Table 1. The sectional and material properties are preferred to suit the Indian guidelines and construction practices. The models are analysed as bare frame structures to represent the conservative nature of frame models without considering the significant influence of the infills on stiffness variability and response of the structure. One of the models is built as a 5-storey (G+4) low-rise building, and the other as a 10-storey (G+9) mid-rise building, and can be named as M1L5 and M2L10, respectively. The floor plan and 3D demonstration of both models M1L5 and M2L10 can be visualised from Figures 1 and 2, respectively. Diaphragm action is taken into account for floor slabs to ensure continuous load paths between lateral elements and vertical load-bearing elements.

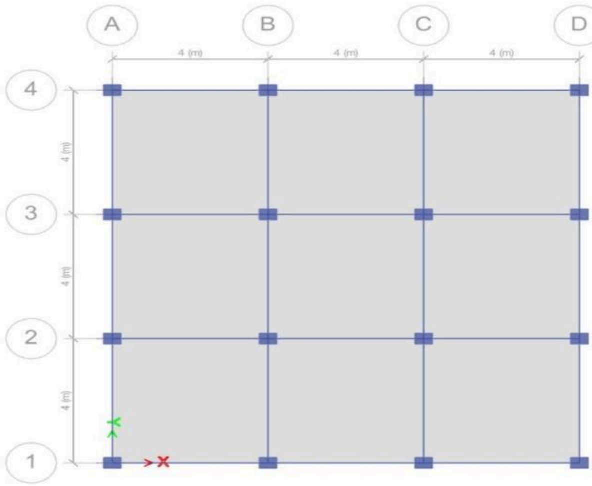
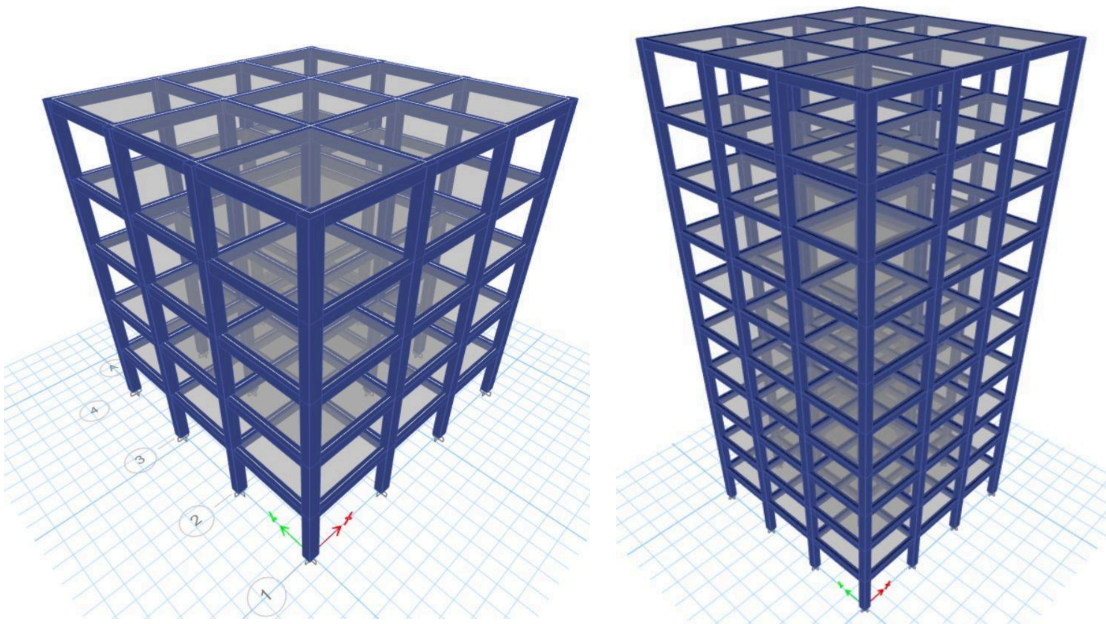


Figure 1: Typical Floor Plan for both M1L5 and M2L10 models

The structural elements of the models, considering the nonlinear analysis, did incorporate the nonlinear behaviour. The nonlinearity is included in beam-column members through FE simulation for the nonlinear material response of such members. (54). In this study, the lumped plasticity model approach is considered, where plastic hinges were provided with a linear-elastic element based on the moment-rotation relationship at end sections to give axial force. Flexural hinges were provided at probable plastic zones, which thus represent column and beam elements under lateral load. Plastic hinges provide nonlinearity at the cross-section level of the structure and represent the nonlinear flexural behaviour, such as cracking, yielding, and post-yielding, which are defined by the moment-rotation relationship of the structural element. The modal parameters of plastic hinges are calculated by the ETABS software. Furthermore, ETABS comprises auto hinge features that automatically define and employ hinges for each and every element. The features of auto hinges in ETABS(v22) are generated following the design guidelines of ASCE 41-17. The flexural hinges in column components are modelled following coupled (P-M2-M3) hinge properties with consideration of the interaction of axial force and bi-axial bending moment at the hinge location, where rotational values are incorporated only for axial forces associated with gravity loads. In a similar manner, the uncoupled (M3) moment hinge is used to model flexural hinges in beam components. Michael R. Willford et al. (55) state that lumped zero-length plastic hinges concentrate the moment-rotation model parameters such that the numerical formulations of the element are efficient and condensed.



a) b)

Figure 2: 3D view of model a) M1L5 b) M2L10

M1L5 and M2L10 models of the present study perform seismic linear analysis (ESF and RSA) by following the provisions of IS 1893:2016. The acceleration coefficient value (A_h) is provided from the design spectra corresponding to the given fundamental time period. A_h values are modified contemplating seismic parameters such as soil type (Medium), Importance Factor (1.2), Response reduction factor (5), Zone factor (0.24), and damping ratio (5%). Modal PA was carried out in ETABS(v22) for both models (M1L5 and M2L10) under load combinations provided by IS 1893-2016 using CSM (20) procedures. CSM estimates the peak inelastic deformation of nonlinear SDOF systems with an analogous period and damping. The values of the effective period and damping are based on ductility estimates. The obtained pushover curve (function of base shear and roof displacement) is converted to a capacity curve (function of spectral acceleration and spectral displacement) by using modal properties of the structure. The capacity curve is plotted in the format of acceleration-response spectrum (ADRS). The conversion of seismic demands in the form of elastic response spectrum into ADRS format is also done to enable the same coordinate system between both demand and capacity curves. ETABS(v22) implements FEMA 440 Equivalent Linearization, which enhances the CSM procedure without changing its core methodology. The structures are pushed only in the X-direction in this study as a referred direction. Three scaled Ground acceleration time series (El-Centro, Gorkha, and Northridge) (56,57) were given as input at base points of both structures with a Load Case of Nonlinear Time history, which follows the procedures of Direct Integration with time steps $\Delta t = 0.005s$. The time step chosen is less than $0.1T_1$ to satisfy the computation efficiency and stability requirements (6). The plot of the acceleration time series of the selected ground motion is presented in Figure 3. IS 1893:2016 allows a minimum of three ground motions for time history analysis. However, other design codes, such as Eurocode and ASCE, require more than seven time histories of different seismic events. Since the study is limited to IS 1893:2016, three types of ground motion were selected. The selected ground motions were scaled between periods $0.2T_1$ to $2T_1$ with the target design response spectrum of IS 1893:2016 provided to the models (58–60), where T_1 is the fundamental period of vibration of the structure. The scaling was performed in Seismomatch software following the algorithm of the improved cosine wavelet function (59). The response spectrum of scaled ground motions can be visualised in Figure 4.

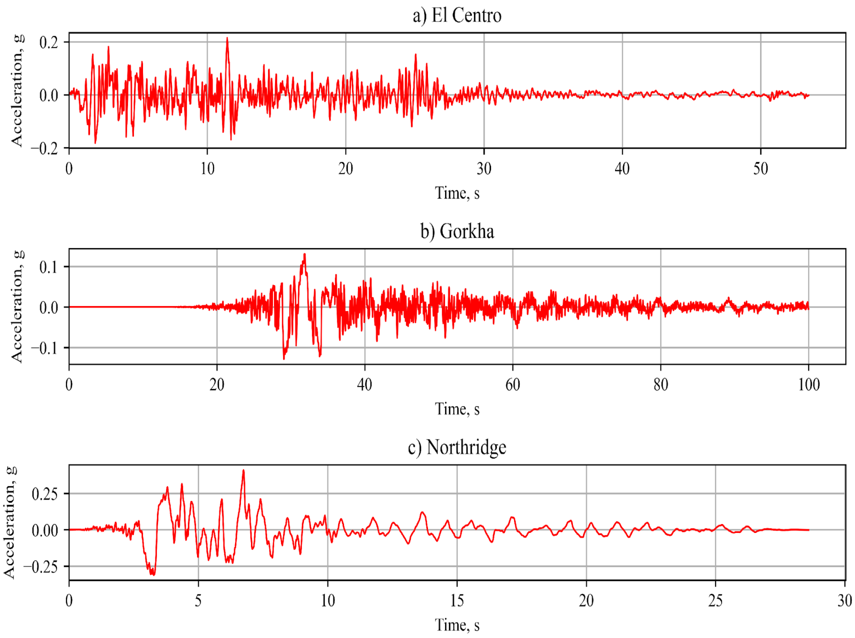


Figure 3: Time History function in H1 Direction of unscaled Ground Motion of a) El-Centro earthquake, 1940 (Station: El Centro Terminal Substation Building), b) Gorkha Earthquake, 2015 (Station: Pulchowk Campus, Institute of Engineering, Tribhuvan University, Patan), c) Northridge earthquake,1994 (Station: Jensen Filter Plant Administrative Building)

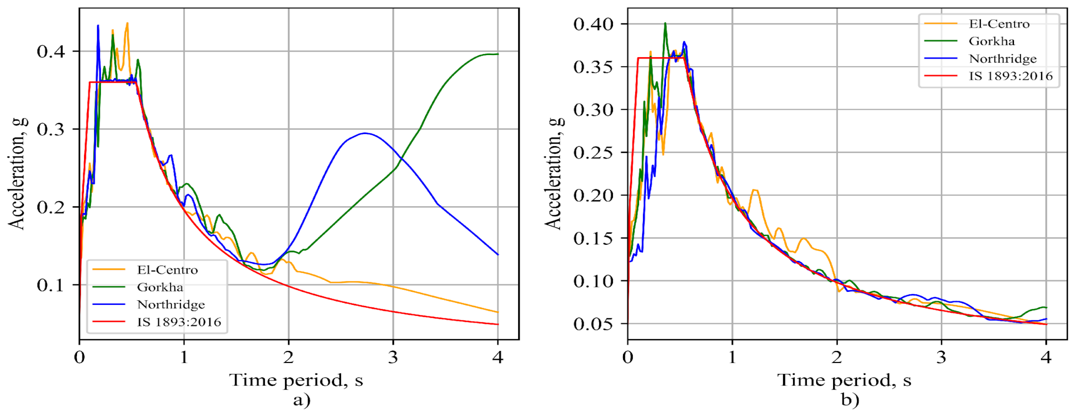


Figure 4: Plot of scaled response spectra of ground motion with Target spectrum (IS 1893:2016, Type-II) for a) M1L5, b) M2L10

3. RESULTS

A. Linear Seismic Analysis

Approximate fundamental translation natural time period T_a of oscillation is calculated for the bare MRF building from IS 1893:2016 for both M1L5 (0.57s) and M2L10 (0.96s) models. The design bse shear ($\overline{V_B}$) obtained by the ESF procedure for M1L5 and M2L10 are 295.04kN and 286.81kN, respectively. Low rise structure with shorter time periods and higher spectral acceleration induce larger base shear than the flexible mid-rise structures with longer time periods. The fundamental time period achieved from ETABS for both models M1L5 (0.93s) and M2L10 (1.9s) is higher than those values achieved from the approximate calculation because of the inclusion of the effective stiffness of building elements (beams and columns) as per IS guidelines. The cumulative modal mass participation for both structures is ≥ 90% from just two translation modes of each direction. The combined responses in both modes sum up the seismic performance of these models. The storey displacement and storey drift of both models can be evaluated under RSA procedures and are presented in Figure 5. The peak roof displacement of models M1L5 and M210 is 12.859mm and 26.727mm, respectively. Likewise, the maximum storey drift of M1L5 and M2L10 are 0.0011 and 0.0013, respectively, and both values do not exceed the limitation (0.004) given by the guidelines. The scaling (M1L5=1.2 and M1L10=1.2) of RSA was done to obtain the given results following IS 1893:2016 guidelines formula ($V_B/\overline{V_B}$) from clause 7.7.3.

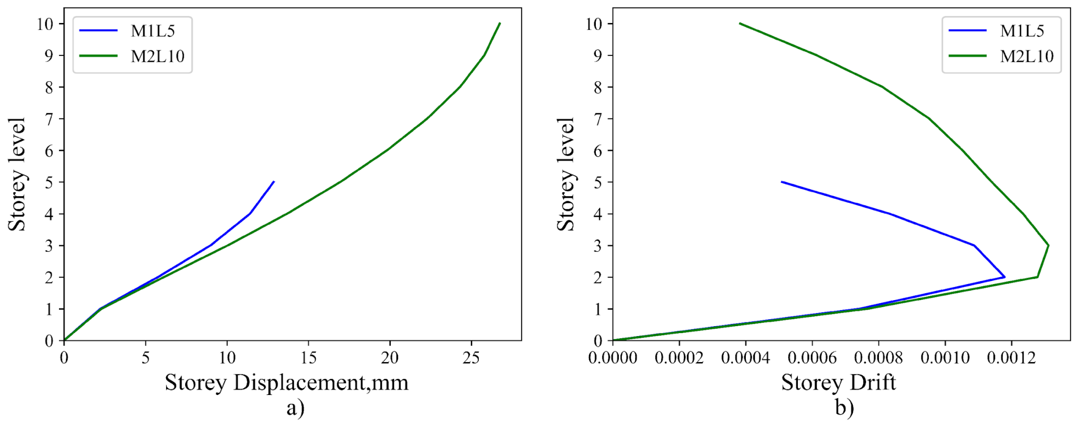


Figure 5: Structural Responses from LSA procedures a) Storey Displacement b) Storey Drift

B. Modal Pushover Analysis

Modal PA was done for the fundamental and second translation mode of the X-direction. Pushover curve (capacity curve) obtained from both structural models after analysis is projected in Figure 6, which determines the values of Base Shear (kN) against roof displacements (mm). M2L10 has higher roof displacements than M1L5 for a particular base shear value, which clearly indicates the flexible nature of tall structures with higher deflection concerns. On the other hand, low-rise structures are stiffer, which attracts larger seismic loads but has lesser deflection risks. Similarly, for M1L5, the yielding point is clearly visible through a change of slope in the first fundamental mode, which is in contradiction to the gentle yielding gradient of M2L10, which suggest flexible and elastic nature of tall-rise structures in the fundamental mode, with the possibility of distribution of stiffness among higher modes leading to a higher energy dissipation. In contradiction, M1L5 has higher stiffness and lower energy dissipating capability. Thus, such a structure induces greater seismic demands, with a quicker possibility of the structure to reach inelastic zone even in low seismic demands.

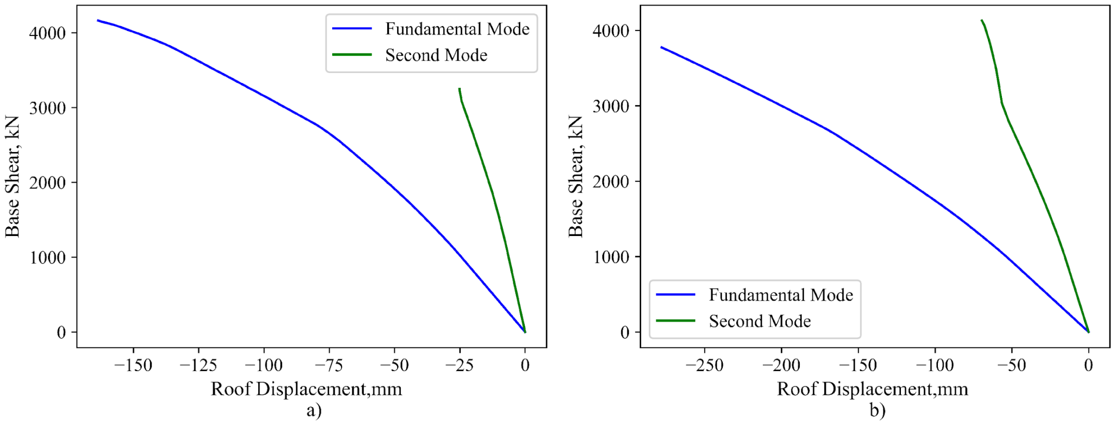
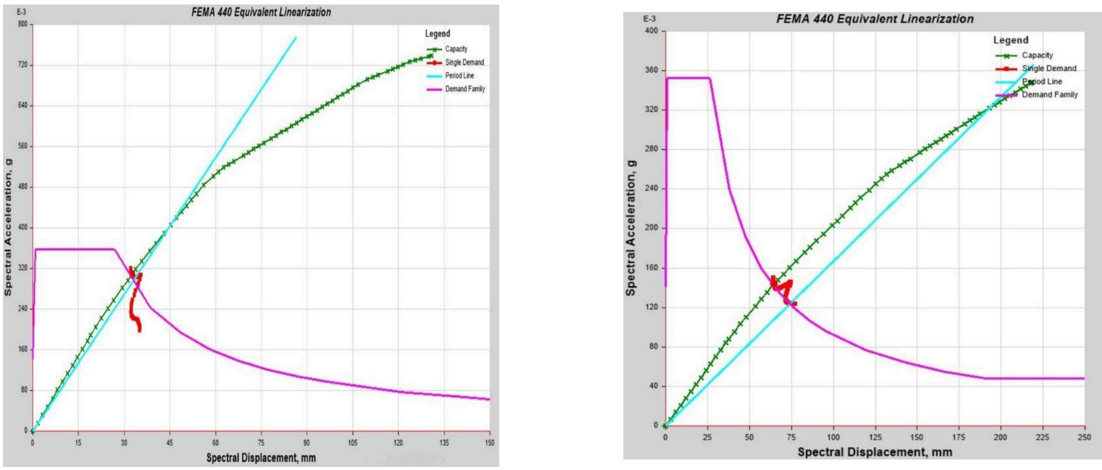


Figure 6: Capacity Curve (Pushover curve) for a) M1L5 b) M2L10



a) b)

Figure 7: Performance point illustration in ETABS using FEMA 440 Equivalent Linearization for a) M1L5, b) M2L10

Table 2: Results for Performance Point

	M1L5		M2L10	
X-direction	Fundamental Mode	Second Mode	Fundamental Mode	Second Mode
Performance Point (mm)	-41.759	-4.93	-83.685	-25.729
Base Shear (kN)	1648.44	763.67	1499.897	1536.487
S_a (g)	0.31	0.33	0.143	0.37

S_d (mm)	32.707	3.758	64.82	17.16
T_{eff} (s)	0.664	0.202	1.367	0.472
Ductility Ratio	1.48	1.01	1.47	1.01
Effective Damping	0.06	0.0505	0.0599	0.0513
Modification Factor	1.03	1.01	1.03	1.01

Following CSM procedures, the capacity curve, which is obtained from modal analysis of the fundamental mode, and the demand curve (IS 1893:2016) are intersected to estimate the performance point. Figure 7 shows the performance point obtained from CSM evaluated in ETABS. Further, Table 2 presents the detailed parametric values obtained after analysis. T_{eff} obtained from both structures is relatively larger than T obtained from RSA. Even though RSA integrates effective stiffness by reducing the gross moment of inertia of members using guidelines, the actual structural time period during a seismic event is obtained more accurately using PA.

For the M1L5 model, displacement related to the performance point is matched at step 19 for the fundamental mode and step 3 for the second mode while generating the capacity curve. Multiple plastic hinges are formed at the beams of the first, second, and third storeys. Likewise, for the M2L10 model, the displacement obtained at the performance point is matched to step 20 for the fundamental mode and step 4 for the second mode, where multiple plastic hinges are formed at the beams of the first to sixth storey levels. Figure 8 shows the hinge status of M1L5 and M2L10 for the fundamental mode. For both M1L5 and M2L10, the performance level of the structure was shown in the BC range at the performance point, and the acceptance criteria lie between the IO and LS state, which indicates significant damage occurred at the beam components where the plastic hinges are formed. Plastic hinges were not formed in columns at the performance point for both models, hinting strong column/ weak beam mechanism implemented in these models.

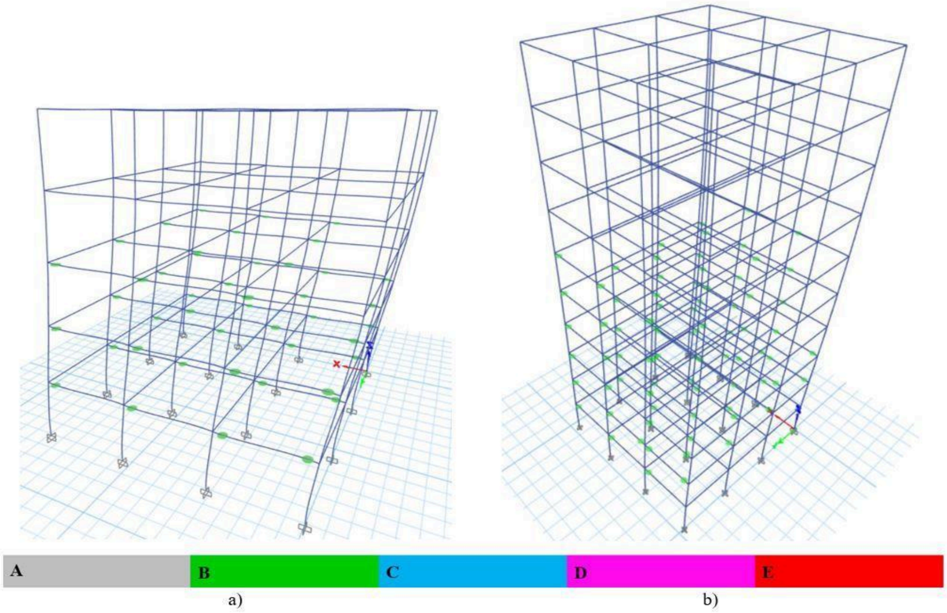


Figure 8: Hinge status at Performance point for a) M1L5 (Step 19), b) M2L10 (Step 20)

The storey displacements and storey drift obtained at Performance points for models M1L5 and M2L10 models in both modes are demonstrated in Figures 10 and 11, respectively. These are the maximum values of the responses that are expected to be faced by the structure for a given design earthquake. The peak displacement obtained for M1L5 and M2L10 was 43.251mm and 85.864mm, respectively. The maximum storey drift of the M1L5 model (0.004) is within the drift limitation as per provisions of IS 1893:2016, but the inter-storey drift of multiple storey levels of M2L10 exceeded this limitation. The exceedance in M2L10 was due to a higher contribution of drift in the second mode, which further corroborates the contribution of higher modes in generating severe responses in high-rise structures during seismic events. The responses obtained from modal PA further corroborate that the contribution of higher modes is also to be considered for seismic assessment of high-rise structures.

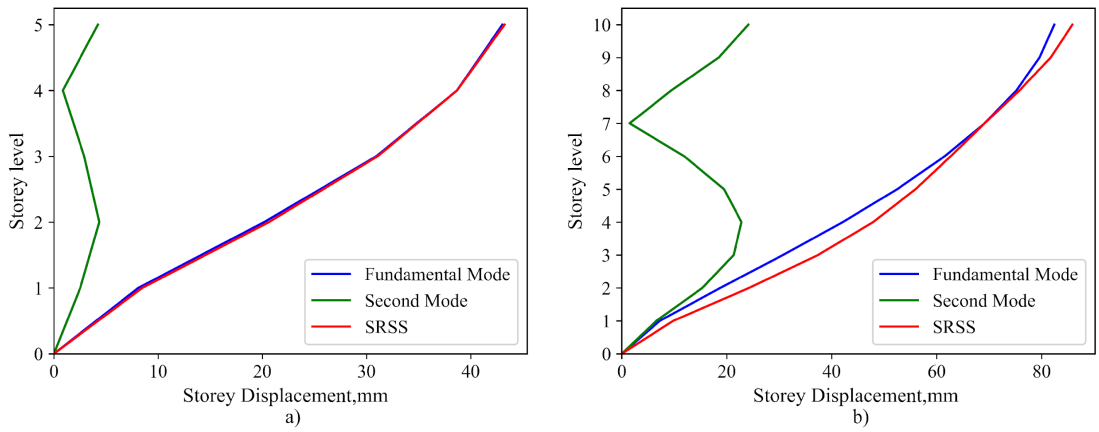


Figure 9: Storey displacement at performance point of a) M1L5 b) M2L10

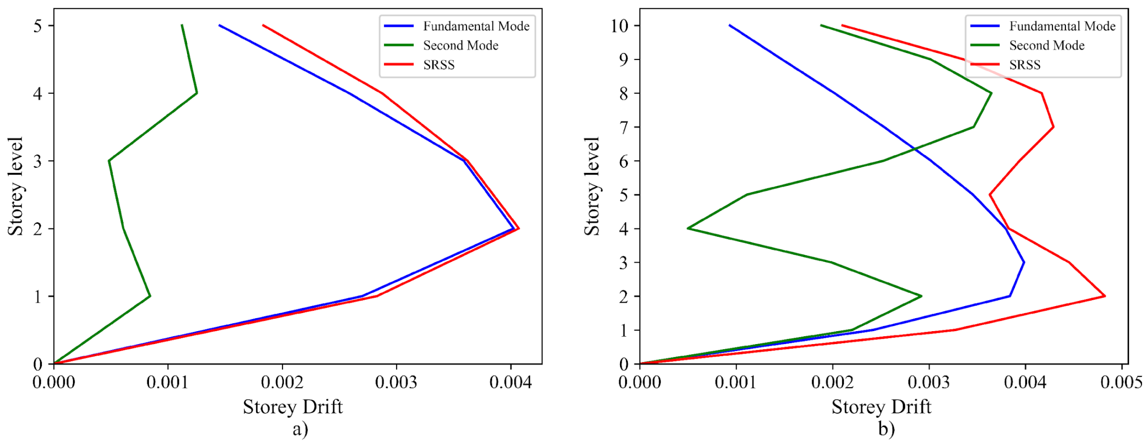


Figure 10: Storey drift at performance point of a) M1L5 b) M2L10

C. Nonlinear Time History Analysis

Peak Base Shear obtained for M1L5 for El-Centro, Gorkha, and Northridge were 1694.297kN, 1819.624kN, and 1440.705kN, respectively, for positive direction, whereas 1769.627kN, 1407.995kN, and 1711.478kN, respectively, for negative direction. Likewise, Peak Base Shear obtained for M2L10 for El-Centro, Gorkha, and Northridge were 1642.481kN, 1717.937kN, and 1676.61kN, respectively, for positive direction, whereas 1662.789kN, 1647.17kN, and 1286.805kN, respectively, for negative direction. Maximum storey drift acquired from ground motions in ETABS software for both M1L5 and M2L10 models is demonstrated in Figure 11. For the M1L5 model, the maximum storey drift of 0.0038 was caused by El-Centro and is followed by Gorkha (0.0037) and Northridge (0.0036) at the second storey. Likewise, for M2L10, the maximum storey drift of 0.0043 was caused by El-Centro at the fifth storey and is followed by Gorkha (0.0042) and Northridge (0.004) at the third storey. Similar to PA, all ground motion in NTHA causes drift exceedance in M2L10 structures. Figure 12 represents the maximum storey displacement in both models M1L5 and M2L10 for both positive and negative directions of the selected ground motion databases. Considering the responses, both structures are more vulnerable to the El-Centro Earthquake, when compared among selected ground motions for performing NTHA. It is to be noted that even when dealing with the same structure, and same scaled spectra is used, the responses (drift, displacement, and base shear) obtained from all three-ground motion varies. These variations are due to characteristics of the ground motion, such as dominant frequency content, duration of earthquake, and acceleration response (6,33,54).

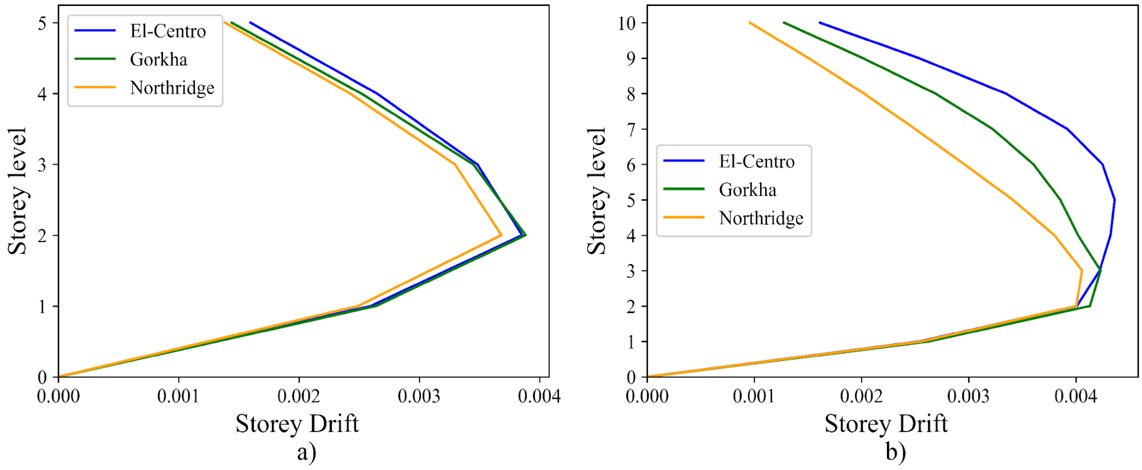


Figure 11: Maximum Storey Drift ratio due to selected ground motions for a) M1L5, b) M2L10

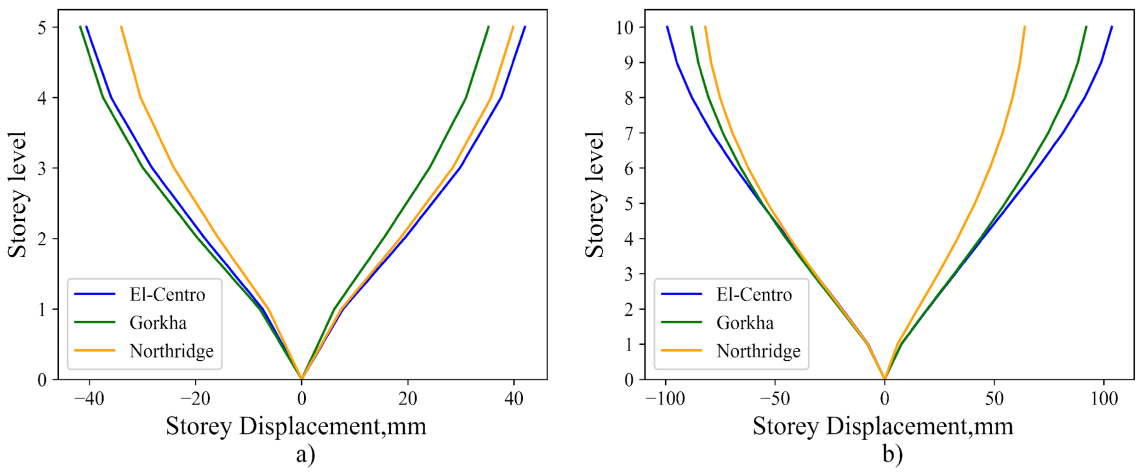


Figure 12: Maximum Storey Displacement due to selected ground motions in both directions for a) M1L5, b) M2L10

4. DISCUSSION

The comparison between the structural responses from different seismic procedures can be visualised from Figures 14, 15, and 16. From the response comparison, Elastic RSA shows a larger deflection when compared to nonlinear PSA and NTHA analysis. RSA=5, though it comprises reduction factors, overstrength factors, and effective stiffness, does not tend to capture nonlinear effects, and has lower responses when compared with Nonlinear analysis. Both M1L5 and M2L10 structures passed on the RSA, whereas M2L10 failed by drift exceedance in both NSA methods, which indicates that inelastic deflection demand, especially in high-rise structures, is not properly estimated by RSA methods. However, RSA offers faster computation and can act as a quicker alternative to complex NSA for preliminary evaluation of the structure. The preliminary RSA analysis must be followed by nonlinear methods to delve further into inelastic seismic response of the structure and its effect on the structural integrity.

NTHA and PA, both as nonlinear seismic analyses, almost show similar behaviour of the responses of the structure. Moreover, NTHA requires higher computing time when compared with other nonlinear procedures, such as PA, but the results shown by NTHA are exact and represent a precise dynamic representation of the structure. (61,62). However, PA, being a static procedure, does not focus on the dynamic behaviour of the structure, as the loading itself is a monotonic push; no reversal effect of loading is considered. Moreover, it focuses on lower fundamental loads and material straining without considering sources of energy dissipation such as kinetic energy, duration effects, and viscous damping. (43). Since the structures modelled in this study are symmetrical, regular, and low-rise structures, the performance of the PA could predict better responses under seismic action. However, the PA tends to overestimate the strength and stiffness, which can lead to underestimating the displacement demand. In contrast to PA, NTHA does account for dynamic effects while analysing structural behaviour, making it more precise in capturing earthquake effects on the structural response of the structure. It is crucial to select the appropriate number of ground motions, as the results are not realistic for lower records to have definite conclusions. (63). In contrast, a higher number of records will provide more accurate and realistic results when compared with responses obtained from other procedures.

Every method applied in this study (RSA, EF, PA, NTHA) has its strengths and weaknesses. RSA, being faster in computation, is preferred in preliminary seismic design of the structure. NTHA are more sophisticated and can provide appropriate responses of the structure against seismic events and thus can simulate the effects of real earthquakes on buildings. On the other hand, if the objectives of the analysis are simplicity and cost efficiency, PA could provide approximate nonlinear structural responses. The newer improvised PA methods are more versatile and can provide similar responses when compared with the realistic responses of NTHA. The preferability of using procedures may be based on varied reasons such as availability, computational cost, precision, importance factor of the structure, seismic zones, and engineering proficiency. The purpose of any individual procedure cannot be discarded or even preferred for a particular reason. It is suitable to state that the prime focus of seismic analysis is to be given to the safety of the structure without any life-threatening damage and risk of collapse under possible future severe earthquakes.

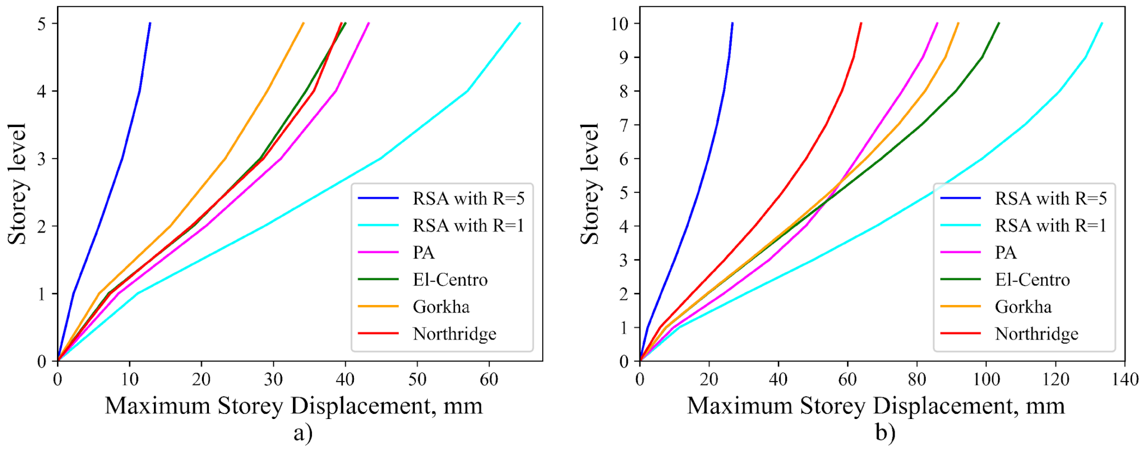


Figure 13: Comparison of Storey Displacement among different seismic analysis methods for a) M1L5, b) M2L10

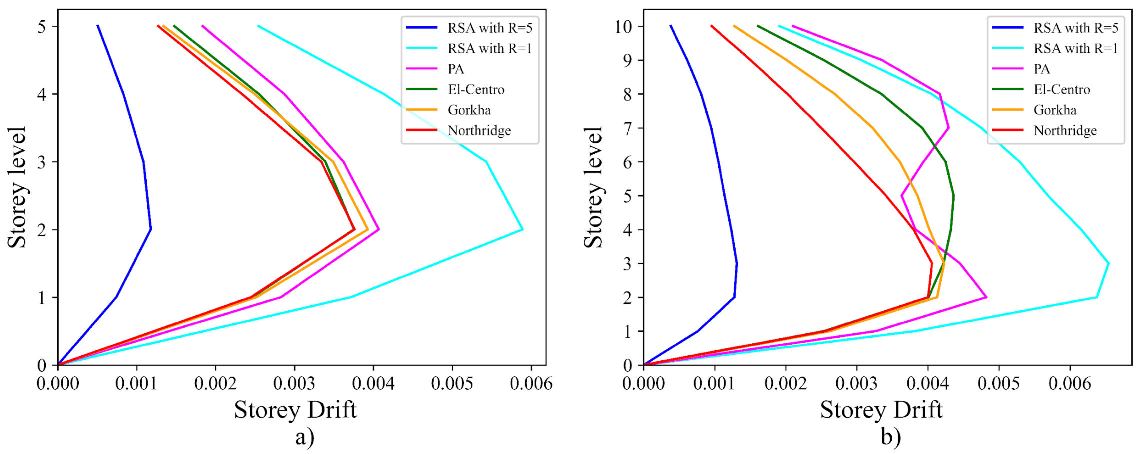


Figure 14: Comparison of Storey Drift ratio among different seismic analysis methods for a) M1L5, b) M2L10

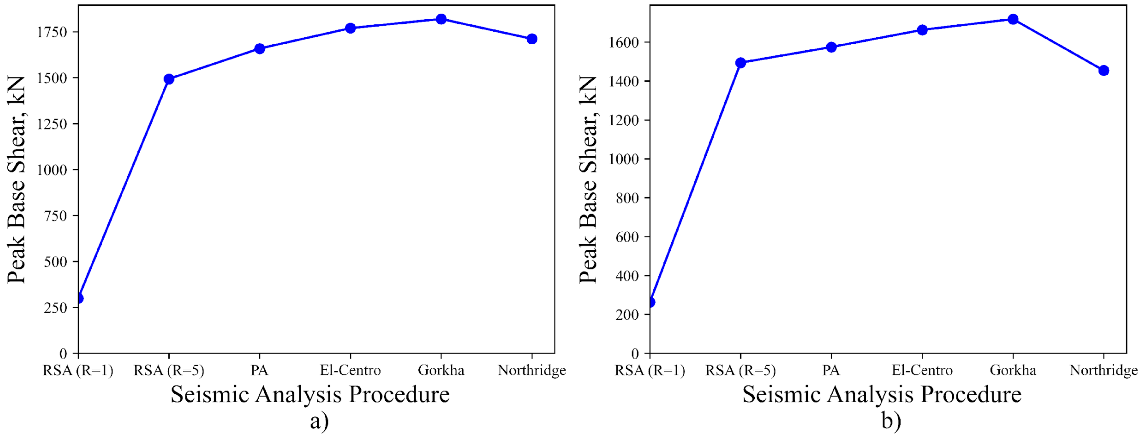


Figure 15: Comparison of base shear among different seismic analysis methods for a) M1L5, b) M2L10

5. CONCLUSION

This study introduced two Bare RC SMF Buildings and modelled under the design guidelines of IS 1893:2016 using ETABS(v22). RSA, PA, and NTHA are considered for seismic analytical procedures, and further ATC-20 and FEMA 356 are followed for PA analysis. The results from all analytical methods are achieved and are compared thoroughly to reach certain conclusions, which are explained below:

1. LSA procedures are better in the preliminary design of the structure. Element sizes can be approximated using RSA and ESF, and probable limitations in structural responses, such as drift ratio, base shear, and roof displacement, could be checked. LSA assumes the structure to be elastic and compensates for the inelasticity effects, due to which the real structural behaviour cannot be determined under earthquake forces.
2. PA generates results faster even when performing nonlinear analysis procedures. PA provided the performance point of the structure (M1L5 = 42.04mm, M2L10 = 87.55mm), which gives the maximum expected response of the structure under the provided seismic demands. Plastic hinges were formed on the beams only for both models, suggesting a strong column-weak beam mechanism.
3. NTHA under an individual ground motion has shown lower responses when compared with PA for low-rise structures, but are marginally aligned for high-rise structures. PA is computationally easier when compared with NTHA analysis, as the selection of appropriate individual ground motion and the scaling process of NTHA are complex.
4. For both M1L5 and M2L10, deflection and drift demand is shown higher in RSA=1, which is followed by NSA methods (PSA, NTHA). RSA with reduction factor (R=5) is found to show lower deflection among other structures. For, base shear, NSA shows higher values in both M1L5 and M2L10. Additionally, M1L5 with high acceleration demand and lower fundamental time period, acts as stiff structure leading to higher

seismic force demand. In contrary, M2L10 is flexible structure which induce larger deformation and drift demand with larger fundamental time period.

5. All procedures, such as RSA, ESF, PA, and NTHA, have their strengths and weaknesses. The required procedures are selected on the basis of computational cost, efficiency, precision, availability, and complexity of structure.

DECLARATION OF COMPETING INTEREST

The author declares that they have no known competing financial interests or personal relationships that could have appeared to influence the work reported in this paper.

DATA AVAILABILITY

The data that support the findings of this study are available from the corresponding author upon reasonable request.

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ETHICS DECLARATION

Not applicable

CONSENT TO PUBLISH

Not applicable

CONSENT TO PARTICIPATE

Not applicable

CLINICAL TRIAL NUMBER

Not applicable

REFERENCES

[1] Anbarci N, Escaleras M, Register CA. Earthquake fatalities: the interaction of nature and political economy. J Public Econ. 2005;89(9–10):1907–33.

[2] Dhakal RP. Structural design for earthquake resistance: Past, present and future. Canterbury Earthquakes Royal Commission; 2011.

[3] Kotsovos MD, Pavlovic M. Ultimate limit-state design of concrete structures: a new approach. Thomas Telford London; 1999.

[4] Kansagara RK. Performance Based Seismic Design of Reinforced Concrete Building. Institute of Technology, Nirma University; 2017.

[5] Priestley MJN. Performance based seismic design. Bulletin of the New Zealand Society for Earthquake Engineering. 2000 Sep 30;33(3):325–46. doi:10.5459/bnzsee.33.3.325-346

[6] Chopra AK. Dynamics of Structure: Theory and Applications to Earthquake Engineering. 4th ed. Prentice-Hall, New Jersey; 2012.

[7] Clough RW, Penzien J. Dynamics of the Structure. 2nd ed. McGraw-Hill; 1993.

[8] Takeda T, Sozen MA, Nielsen NN. Reinforced Concrete Response to Simulated Earthquakes. Journal of the Structural Division. 1970 Dec;96(12):2557–73. doi:10.1061/JSDEAG.0002765

[9] Chopra AK, Goel RK. A modal pushover analysis procedure for estimating seismic demands for buildings . Earthq Eng Struct Dyn. 2002;31:561–82.

[10] Ibarra LF, Medina RA, Krawinkler H. Hysteretic models that incorporate strength and stiffness deterioration. Earthq Eng Struct Dyn. 2005 Oct 1;34(12):1489–511. doi:https://doi.org/10.1002/eqe.495

[11] Standards New Zealand. NZS 1170.5:2004 Structural Design Actions Part 5: Earthquake Actions. Wellington, New Zealand: Standards New Zealand; 2004.

[12] National Research Council Canada. National Building Code of Canada 2020. Ottawa, Canada: Canadian Commission on Building and Fire Codes; 2020.

[13] Bureau of Indian Standards. IS 1893 (Part 1):2016 – Criteria for Earthquake Resistant Design of Structures – Part 1: General Provisions and Buildings. New Delhi: Bureau of Indian Standards; 2016.

[14] European Committee for Standardization. Eurocode 8: Design of structures for earthquake resistance Part 1: General rules, seismic actions and rules for buildings. Brussels: CEN; 2004.

[15] American Society of Civil Engineers. Minimum Design Loads and Associated Criteria for Buildings and Other Structures (ASCE/SEI 7-22). Reston, VA: American Society of Civil Engineers; 2022.

[16] Kuria KK, Kegyes-Brassai OK. Pushover Analysis in Seismic Engineering: A Detailed Chronology and Review of Techniques for Structural Assessment. Applied Sciences. 2023 Dec 23;14(1):151. doi:10.3390/app14010151

[17] American Society of Civil Engineers. Seismic Evaluation and Retrofit of Existing Buildings (ASCE/SEI 41-17). Reston, VA: American Society of Civil Engineers; 2017.

[18] FEMA 440. Improvement of Nonlinear Static Seismic Analysis Procedures FEMA 440. Redwood City, CA, USA; 2005.

[19] FEMA 356. Prestandard and Commentary for the Seismic Rehabilitation of Buildings, American Society of Civil Engineers, USA. 2000.

[20] ATC-40. Seismic evaluation and retrofit of concrete buildings Volume 1. Twin Dolphin Drive, Suite 550 Redwood City, California 94065; 1996.

[21] Hashemi G, Ramhormozian S, Clifton GC. A Review on Nonlinear Time History Analysis of Structures [Internet]. 2024 [cited 2025 Jul 9]. Available from: https://repo.nzsee.org.nz/handle/nzsee/2696

[22] Mazzoni S, Scott MH, McKenna F, Fenves GL. Open System for Earthquake Engineering Simulation–user manual (version 1.7. 3), Pacific Earthquake Engineering Research Center. University of California, Berkeley, California [Internet]. 2006 [cited 2025 Jul 6]. Available from: https://opensees.berkeley.edu/wiki/index.php/OpenSees_Users_Manual

[23] LUSAS. LUSAS Programmable Interface (LPI) Customisation and Automation Guide. [Internet]. Forge House, 66 High Street, Kingston upon Thames, Surrey, KT1 1HN, United Kingdom; 2024 [cited 2025 Jul 6]. Available from: https://www.lusas.com/user_area/documentation/V21_0/LPI%20Customisation%20and%20Automation%20Guide.pdf

[24] Dassault Systèmes Simulia Corp. ABAQUS 2024 Documentation. Providence, RI: Dassault Systèmes; 2024.

[25] Computers and Structures I. CSI Analysis Reference Manual For SAP2000, ETABS, SAFE and CSiBridge [Internet]. 2016 [cited 2025 Jul 6]. Available from: https://docs.csiamerica.com/manuals/etabs/User’s%20Guide.pdf

[26] ANSYS R. Academic Research Mechanical, Release 19.2, Help System. Coupled Field Analysis Guide. ANSYS, Inc. Canonsburg, PA, USA; 2020.

[27] Krawinkler H, Seneviratna GDPK. Pros and cons of a pushover analysis of seismic performance evaluation. Eng Struct. 1998 Apr;20(4–6):452–64. doi:10.1016/S0141-0296(97)00092-8

[28] Kadid A, Boumrkik A. Pushover analysis of reinforced concrete frame structures. 2008.

[29] Mwafy AM, Elnashai AS. Static pushover versus dynamic collapse analysis of RC buildings. Eng Struct. 2001 May;23(5):407–24. doi:10.1016/S0141-0296(00)00068-7

[30] Inel M, Tjhin T, Aschheim M. The significance of lateral load pattern in pushover analysis. In: Proceedings Fifth National Conference on Earthquake Engineering Istanbul, Turkey. 2003.

[31] Gupta B. Enhanced pushover procedure and inelastic demand estimation for performance-based seismic evaluation of buildings. University of Central Florida; 1998.

[32] Worku AM, Hsiao PC. An improved first-mode-based pushover analytical procedure for assessing seismic performance of special moment resisting frame building structures. Eng Struct. 2022 Feb;252:113587. doi:10.1016/j.engstruct.2021.113587

[33] Bento R, Simões A. Seismic Performance Assessment of Buildings. Buildings. 2021;11(10). doi:10.3390/buildings11100440

[34] Crowley H, Pinho R. Revisiting Eurocode 8 formulae for periods of vibration and their employment in linear seismic analysis. Earthq Eng Struct Dyn. 2010 Feb;39(2):223–35. doi:10.1002/eqe.949

[35] Freeman SA. Evaluations of existing buildings for seismic risk-A case study of Puget Sound Naval Shipyard. In: Proc. 1st US Nat. Conf. on Earthquake Engrg., Bremerton, Washington, 1975. 1975. p. 113–22.

[36] Comartin CD, Niewiarowski RW, Freeman SA, Turner FM. Seismic Evaluation and Retrofit of Concrete Buildings: A Practical Overview of the ATC 40 Document. Earthquake Spectra. 2000 Feb 1;16(1):241–61. doi:10.1193/1.1586093

[37] Aşıkoğlu A, Vasconcelos G, Lourenço PB. Overview on the Nonlinear Static Procedures and Performance-Based Approach on Modern Unreinforced Masonry Buildings with Structural Irregularity. Buildings. 2021 Apr 1;11(4):147. doi:10.3390/buildings11040147

[38] Goel RK. Evaluation of Current Nonlinear Static Procedures Using Strong Motion Records. In: Structural Engineering Research Frontiers [Internet]. Reston, VA: American Society of Civil Engineers; 2007. p. 1–12. Available from: http://ascelibrary.org/doi/abs/10.1061/40944%28249%2976 doi:10.1061/40944(249)76

[39] Fajfar P, Fischinger M. N2-A method for non-linear seismic analysis of regular buildings. In: Proceedings of the ninth world conference in earthquake engineering. 1988. p. 111–6.

[40] (US) BSSC, Council AT. NEHRP guidelines for the seismic rehabilitation of buildings (FEMA 273). Vol. 1. Washington, DC, USA: Federal Emergency Management Agency; 1997.

[41] Hernandez-Montes E, Kwon OS, Aschheim MA. An energy-based formulation for first-and multiple-mode nonlinear static (pushover) analyses. Journal of Earthquake Engineering. 2004;8(01):69–88. doi:10.1142/S1363246904001390

[42] Kotanidis C, Doudoumis IN. Energy-based approach of static pushover analysis. In: The 14th World Conference on Earthquake Engineering October. 2008. p. 12–7.

[43] ANTONIOU S, PINHO R. ADVANTAGES AND LIMITATIONS OF ADAPTIVE AND NON-ADAPTIVE FORCE-BASED PUSHOVER PROCEDURES. Journal of Earthquake Engineering. 2004 Jul 3;8(4):497–522. doi:10.1080/13632460409350498

[44] Gupta B, Kunnath SK. Adaptive Spectra-Based Pushover Procedure for Seismic Evaluation of Structures. Earthquake Spectra. 2000 May 1;16(2):367–91. doi:10.1193/1.1586117

[45] Moghadam AS, Tso WK. A pushover procedure for tall buildings. In: Proc. of the Twelfth European Conference on Earthquake Engineering, London, United Kingdom, Paper. 2002.

[46] Mao J, Zhai C, Xie L. An improved modal pushover analysis procedure for estimating seismic demands of structures. Earthquake Engineering and Engineering Vibration. 2008;7(1):25–31. doi:10.1007/s11803-008-0786-y

[47] Sucuoğlu H, Günay MS. Generalized force vectors for multi‐mode pushover analysis. Earthq Eng Struct Dyn. 2011 Jan 28;40(1):55–74. doi:10.1002/eqe.1020

[48] Jan TS, Liu MW, Kao YC. An upper-bound pushover analysis procedure for estimating the seismic demands of high-rise buildings. Eng Struct. 2004;26(1):117–28. doi:https://doi.org/10.1016/j.engstruct.2003.09.003

[49] Liu Y, Kuang JS. Spectrum-based pushover analysis for estimating seismic demand of tall buildings. Bulletin of Earthquake Engineering. 2017;15(10):4193–214. doi:10.1007/s10518-017-0132-8

[50] Cruz PJS, Marí AR, Roca P. Nonlinear Time-Dependent Analysis of Segmentally Constructed Structures. Journal of Structural Engineering. 1998 Mar 1;124(3):278–87. doi:10.1061/(ASCE)0733-9445(1998)124:3(278)

[51] Åldstedt E, Bergan PG. Nonlinear Time-Dependent Concrete-Frame Analysis. Journal of the Structural Division. 1978 Jul;104(7):1077–92. doi:10.1061/JSDEAG.0004951

[52] Loli M, Anastasopoulos I, Gazetas G. Nonlinear analysis of earthquake fault rupture interaction with historic masonry buildings. Bulletin of Earthquake Engineering. 2015 Jan 25;13(1):83–95. doi:10.1007/s10518-014-9607-z

[53] Nguyen QH, Hjiaj M. Nonlinear Time-Dependent Behavior of Composite Steel-Concrete Beams. Journal of Structural Engineering. 2016 May;142(5):04015175. doi:10.1061/(ASCE)ST.1943-541X.0001432

[54] Fragiadakis M, Vamvatsikos D, Karlaftis MG, Lagaros ND, Papadrakakis M. Seismic assessment of structures and lifelines. J Sound Vib. 2015;334:29–56. doi:https://doi.org/10.1016/j.jsv.2013.12.031

[55] Michael R. Willford, Deierlein GG, Reinborn AM, Willford MM. NIST GCR 10-917-5: NEHRP Seismic Design Technical Brief No. 4, Nonlinear Structural Analysis for Seismic Design. 2010.

[56] Pacific Earthquake Engineering Research Center (PEER). Pacific Earthquake Engineering Research Center (PEER) [Internet]. [cited 2025 May 26]. Available from: https://ngawest2.berkeley.edu

[57] Takai N, Shigefuji M, Rajaure S, Bijukchhen S, Ichiyanagi M, Dhital MR, et al. Strong ground motion in the Kathmandu Valley during the 2015 Gorkha, Nepal, earthquake. Earth, Planets and Space. 2016 Dec 26;68(1):10. doi:10.1186/s40623-016-0383-7

[58] Seismosoft. SEISMOMATCH [Internet]. 2022 [cited 2025 Sep 6]. Available from: https://www.seismosoft.com/seismomatch

[59] Al Atik L, Abrahamson N. An improved method for nonstationary spectral matching. Earthquake spectra. 2010;26(3):601–17.

[60] Abrahamson NA. Non-stationary spectral matching. Seismological research letters. 1992;63(1):30.

[61] Çavdar Ö, Bayraktar A. Pushover and nonlinear time history analysis evaluation of a RC building collapsed during the Van (Turkey) earthquake on October 23, 2011. Natural Hazards. 2014 Jan 5;70(1):657–73. doi:10.1007/s11069-013-0835-3

[62] Rodríguez CA, Rodríguez Pérez ÁM, López R, Caparrós Mancera JJ. Comparative Analysis and Evaluation of Seismic Response in Structures: Perspectives from Non-Linear Dynamic Analysis to Pushover Analysis. Applied Sciences. 2024 Mar 15;14(6):2504. doi:10.3390/app14062504

[63] Merter O, Ucar T. A comparative study on nonlinear static and dynamic analysis of RC frame structures. Journal of civil Engineering and Science. 2013;2(3):155–62.