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Nonlinear Analysis of Reinforced Concrete Walls: Static Vs Dynamic Analysis.

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ABSTRACT

This paper compares static and dynamic approaches to the nonlinear analysis of reinforced concrete (RC) walls. Depending on the mode of the representative seismic load applied to a structure, structural analysis can be categorised as static or dynamic. Despite nonlinear response history analysis (NRHA) being the most accurate method to predict the seismic response of a structure, most of the design codes and guidelines recommend using only the simplified static pushover analysis (POA) for regular buildings. POA is therefore the most common approach of nonlinear analysis of structures, while NRHA is used mainly for torsionally sensitive or very tall structures. Therefore, this paper compares the static and dynamic approaches to nonlinear analysis of RC walls. Previous studies conducted to compare static and dynamic analysis have revealed differences in the results obtained from the two procedures. However, most of the comparative studies are limited to frame systems and far too little attention has been paid to RC walls. Thus, the purpose of this study is to compare the results obtained from static and dynamic nonlinear analyses for RC walls. Incremental dynamic analysis (IDA) and traditional POA are performed on a 10-storey RC wall building designed according to the current New Zealand standards. For this purpose, current state-of-the-art wall model (SFI-MVLEM) in OpenSees is adopted for nonlinear analyses. The static and dynamic analyses are compared in terms of capacity curves and inter-storey drift profiles. The POA is found to significantly underestimate the shear capacity of the RC wall. The findings of this study highlight the differences in the results obtained from the static and dynamic analyses for RC wall buildings and also offer insights into the influence of the number of storeys and the shear span ratio of RC walls on the predicted response.

Keywords: reinforced concrete wall, SFI-MVLEM, pushover analysis, incremental dynamic analysis, inter-story drift profiles.

INTRODUCTION

Nonlinear seismic analysis is an indispensable tool for designing and evaluating structures for seismic performance. It enables the estimation of seismic demand and capacity and is a critical component of Performance-Based Earthquake Engineering (PBEE). Seismic analysis can be classified as either static or dynamic, depending on how the earthquake load is applied to a structure. Furthermore, based on the analytical model used, it can be categorized as linear or nonlinear. The four most commonly used methods for predicting the seismic response of structures are linear static, linear dynamic, nonlinear static, and nonlinear dynamic analyses, as specified in seismic codes and guidelines for designing and assessing structures.

The current seismic codes and guidelines [1,2,3,4] prescribe different analysis approaches based on structural characteristics and expected performance states [5]. Height and irregularity are usually the governing factors that determine the type of analysis to be conducted. Ordinary and regular buildings with fewer complexities are analyzed using linear methods such as equivalent linear static analysis or response-spectrum analysis. However, for tall and torsionally sensitive buildings, most seismic codes require nonlinear procedures such as pushover or nonlinear response history analysis (NRHA).

Nonlinear analyses are more realistic than linear analyses in predicting the seismic response of structures since most structures experience significant inelastic deformations during strong earthquakes. Pushover analysis (POA) is the most commonly used method for evaluating the nonlinear response of structures due to its simplicity. This method is also typically recommended in most of the seismic codes as a tool for nonlinear analyses of structures, whereas NRHA isn't adequately regulated in the codes.

In POA, a nonlinear structural model is pushed laterally using a predefined load pattern distributed along the structure's height. The lateral load is monotonically increased until it reaches a target level of deformation or force. The objective in this approach is to replicate the distribution of horizontal forces generated in a building during an earthquake. This approach is widely used and well-understood by the engineering community, making it a popular method for seismic design and vulnerability assessment of buildings.

While NRHA is the most cutting-edge method for seismic response analysis of structures, it is rarely employed in normal design practices, except for certain complex or critical structures. This is due to the significant amount of time and effort required to select and apply suitable ground motions to the nonlinear structural model, making it a relatively complex process [5]. As a more practical alternative, Pushover Analysis (POA) is often used for seismic design and assessment of typical buildings [6]. While POA is convenient, it has some inherent limitations that make NRHA essential for the study, design, and assessment of irregular, tall, or important structures.

Originally proposed as a tool for understanding the progression of nonlinear structural behavior, rather than quantifying it, the use of Pushover Analysis (POA) has extended to making quantitative conclusions for structural design and assessment [7]. A conventional POA with an invariant lateral load pattern assumes that the response of the structure is controlled by a single mode and that the mode shape remains constant throughout its response. While this assumption is acceptable for structures dominated by a single mode, it fails to capture phenomena dependent on higher modes or modes undergoing significant changes due to inelastic redistribution [8]. Even for buildings of moderate height, higher modes can have a significant impact on estimations of story shear forces and overturning moments. Moreover, POA cannot account for cumulative damage caused by cyclic loading, which is a critical factor in a structure's response to earthquakes. Despite these limitations, POA remains the preferred method for nonlinear analysis in engineering practice and seismic codes, leading to the majority of structures being designed and assessed using this method. Thus, a comparative study between POA and NRHA is crucial for a more informed judgment about the reliability of POA results.

Numerous studies have compared nonlinear static and dynamic analyses for various structural systems, as documented by researchers [9-14]. These studies have suggested that static and dynamic analyses may provide different results, particularly for structures that are more likely to undergo nonlinear responses. While comparative studies have focused mostly on buildings with a frame system, there has been little research on reinforced concrete (RC) wall buildings. According to [14], differences between static and dynamic results are more significant for buildings with a frame-wall system than those with a frame system. Therefore, this study aims to analyze the discrepancies between the static and dynamic responses of RC walls.

The research data in this study is drawn from the POA and NRHA carried out on a 10-storey RC wall building designed according to the current New Zealand Standards [4,15]. Additionally, seven RC wall models are derived from the original 10-storey RC wall model to represent structures with varying numbers of storeys and shear span ratios, and nonlinear static and dynamic analyses are conducted. This study utilizes the SFI-MVLEM model [16] in OpenSees for all the structural analyses. The POA is performed with a triangular lateral load profile, while the NRHA is conducted using an incremental dynamic analysis (IDA) [17]. The IDA uses 44 ground motions from the far-field set of FEMA P695 [18]. The study presents the primary results in terms of global response measures, such as capacity curves and inter-storey drift profiles and conducts a comparative evaluation of static and dynamic responses and highlights differences between POA and NRHA.

DESCRIPTION OF THE CASE STUDY REINFORCED CONCRETE WALL BUILDING

To compare static and dynamic analyses, a case-study RC wall building archetype is designed according to the current New Zealand Standards [4,15]. The ten-storey building features a symmetrical plan with four perimeter RC walls primarily designed to resist lateral loads, while the interior and corner columns are designed to carry only gravity loads. The building's plan layout is partly adopted with modifications from a design example in the New Zealand Red Book [19].

The building's geometry is shown in Figure 1 - it has a square plan with a side length of 30 m and is divided into 3 bays. The RC walls, which are 400 mm thick and 10 m long, are located midway around the building's perimeter. The first storey is 4 m tall, while the remaining storeys have an inter-storey height of 3.6 m, resulting in a total building height of 36.4 m. Consequently, the RC wall's shear span ratio is approximately 2.5.

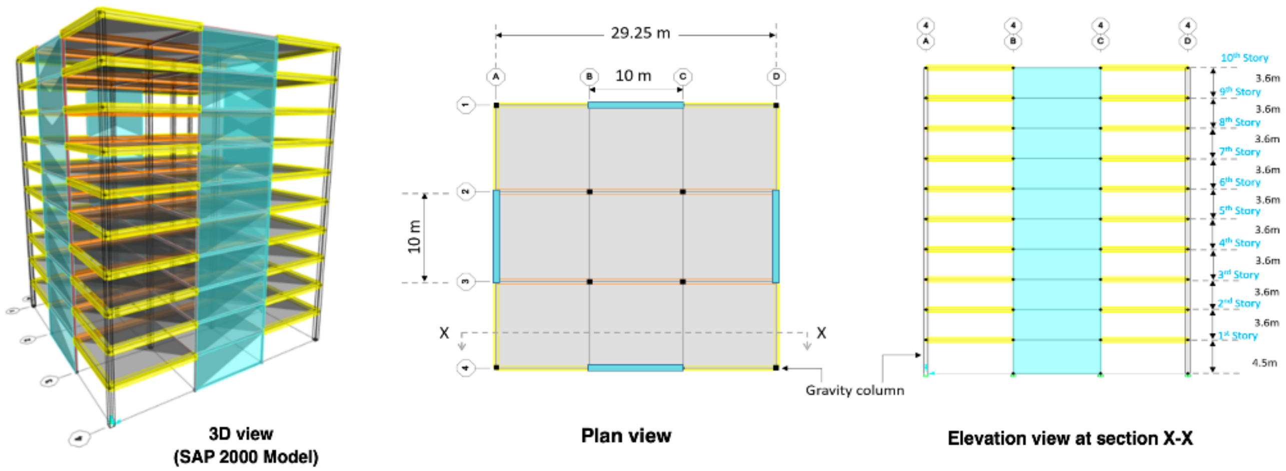


Figure 1. Geometry of the 10-storey RC wall building

The structure is assumed to be located in Christchurch and designed for soil class `D' based on [4] site classification. The structure is initially modeled in SAP2000 [20], and seismic design forces and displacements are estimated using the equivalent lateral force (ELF) method specified in [4].

Due to symmetry of the building's layout and the use of RC walls as the primary lateral load resisting system, the analysis is conducted by including only one RC wall with half of the total seismic weight. As a result, using the SFI-MVLEM model developed by [21], a 2D structural wall model is developed in Open System for Earthquake Engineering Simulation platform (OpenSEES) 3.0 [22], as depicted in Figure 2. The base of the wall model is fixed, assuming the foundation to be fully restrained, disregarding any flexibility resulting from soil-structure interaction.

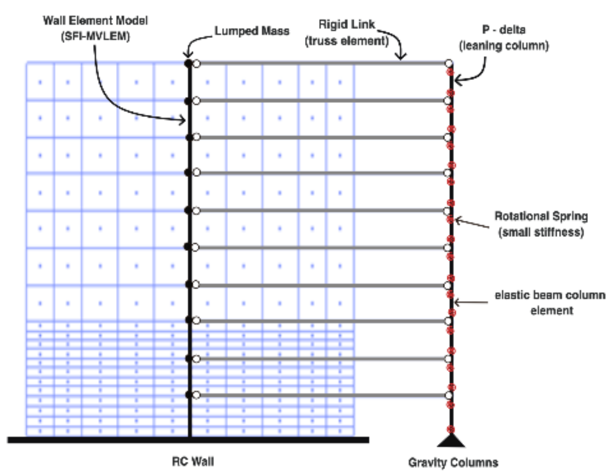


Figure 2. 2D analytical model of the 10-Storey RC Wall building.

To develop the structural model, the mass of each storey was divided in half and concentrated at the floor levels. An eigenvalue analysis was performed, yielding the fundamental time period of the model, T_1 = 0.922s. To account for P-delta effects, a leaning column was incorporated into the model. The column was pin-connected at the base and connected to the RC wall model at each floor level via axially rigid beams (see in Figure 2).

To simulate the behavior of reinforcing bars, SteelMPF model [21] based on the Menegotto-Pinto material model [23], with isotropic strain hardening of 2%, is used. For concrete, ConcreteCM model [16] is used which is based on a hysteretic concrete constitutive model developed by [24].

STRUCTURAL ANALYSIS

After developing the SFI-MVLEM model for a 10-storey RC wall, it is then analyzed using conventional POA and IDA in OpenSees. This section discusses and elaborates on the process and results obtained from each analytical method.

Pushover Analysis (POA)

A conventional approach to POA is chosen which is performed in this study using a monotonously increasing inverted-triangular lateral load pattern (proportional to the first mode) as prescribed in [25] for regular structures.

There are various ways to perform POA, which is generally based on lateral load patterns. One of the most cutting-edge methods is adaptive POA, which changes the load pattern according to the dynamic characteristics of the structures at each stage of the analysis, as opposed to the conventional approach [26]. While some studies have demonstrated the superiority of the adaptive pushover over the conventional approach [27, 28, 29], the additional complexities involved in implementing adaptive load patterns make the conventional approach more prevalent in practice. Moreover, some studies have reported that the application of adaptive POA does not offer significant advantages over conventional POA [30,31].

Despite its simplicity, the invariant load pattern has been shown in various studies to produce a resonably accurate estimate of seismic capacity in the elastic range [14, 30, 32]. According to studies reported in the literature [14, 33], the shear capacity predicted using different loading patterns differs only slightly. A study carried out by [32] has also demonstrated that the invariant inverted triangular load pattern can provide reasonably accurate approximation of the seismic capacity curve of low-to-mid-rise shear type buildings.

The inverted-triangular lateral load profile was applied to the SFI-MVLEM wall model at 0.25mm increments in this study. The load profile was applied until the target roof drift of 2.5% was achieved at the control node, which is the maximum drift limit recommended in the current seismic loading standard [4]. The system of nonlinear equations was solved using the Newton-Raphson nonlinear solution algorithm. In cases where the algorithm failed to solve the system of equations, successive iterations were made using other solution algorithms, load increments, and tolerance limits until the solution converged.

Base shear versus roof drift

The base shear force and storey drifts are recorded at each step of the analysis. The base shear versus roof drift, as illustrated in Figure 3, is the most typical way of characterizing the seismic performance of RC walls. This type of curve is commonly referred to as a pushover curve or capacity curve, and it demonstrates the structure's resistance as it deforms into the inelastic range. Additionally, the pushover curve provides information on the structure's ductility and stiffness.

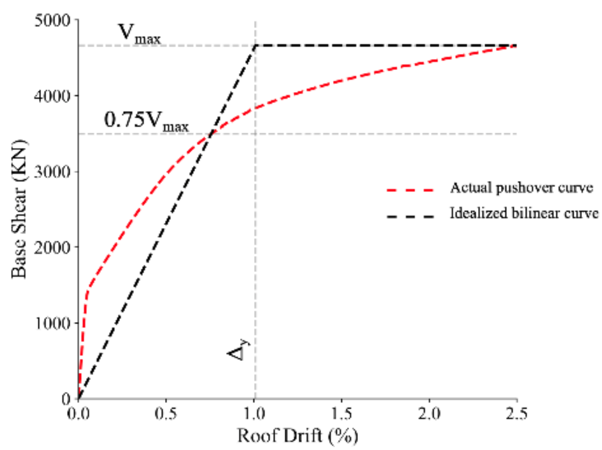


Figure 3. Pushover curve of the RC wall building

The pushover curve in Figure 3 corresponds to the 10-storey RC wall as described earlier. The graph indicates that as the base shear increases, the slope of the curve decreases, indicating a gradual reduction in the structure's stiffness as the lateral deformation increases. An idealized model is constructed with a straight line passing through the origin and the point corresponding to $0.75V_{max}$ (where, V_{max} represents the maximum base shear until 2.5% roof drift is reached) on the pushover curve.

Inter-storey drift ratio profiles

Inter-storey drift ratio (IDR) is a crucial structural response parameter since it has a direct correlation to a building's seismic damage. It measures the relative displacement between two consecutive floors divided by the inter-storey height.

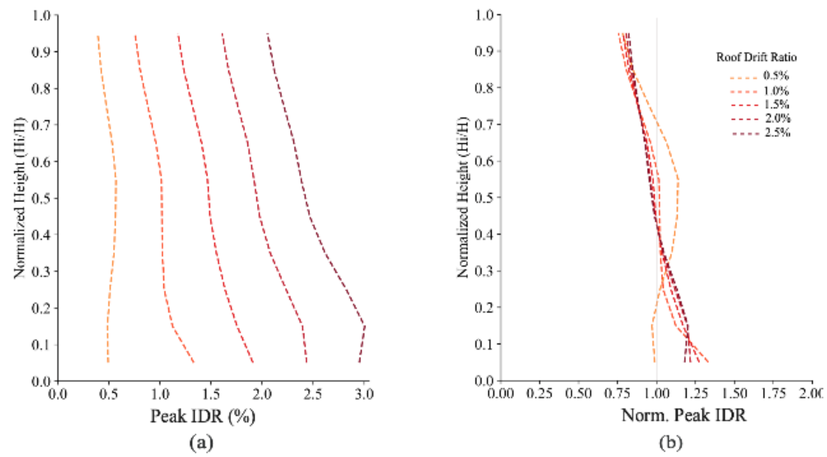


Figure 4. Distribution of (a) IDR ratio (%) and (b) Normalized IDR at varying roof drift levels obtained from POA of the 10-storey RC wall building.

In Figure 4, the distribution of IDR and the normalized IDR are plotted against the normalized height of the RC wall building at various roof drift demands. The IDR profiles are normalized by the corresponding total roof drift ratio. To calculate the normalized height, the height of a storey (H_i) is measured from the base and divided by the overall height (H) of the RC wall building. As the roof drift increases, a shift in the drift profiles occurs, capturing the transition of the wall response from elastic to yielding and finally to inelastic behavior.

Incremental Dynamic Analysis (IDA)

The 10-storey RC wall building is subjected to NRHA using an incremental dynamic analysis (IDA) [17]. The IDA is a dynamic analysis procedure for calculating the seismic demand of buildings at various seismic intensity levels. In this study, the analysis is conducted using a suite of FEMA P695 far-field ground motions [18]. Figure 5 displays the elastic acceleration response spectra (with 5% damping) of the 44 ground motions and their mean spectrum.

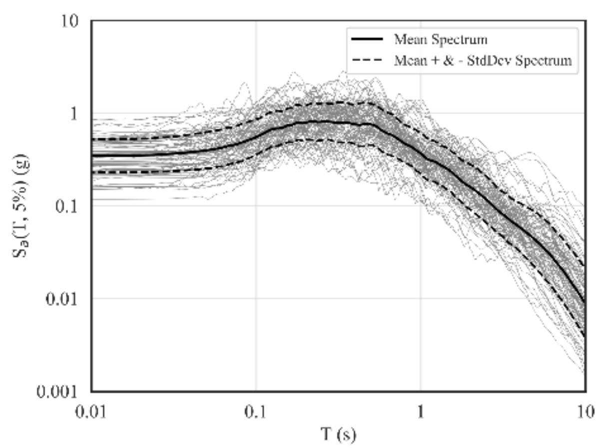


Figure 5. Acceleration response spectrum of the FEMA P695 far-field record set [18].

The IDA was performed on the RC wall model until the maximum IDR reached 2.5%, which is the target roof drift selected for the POA in the preceding section.

The Newmark average acceleration scheme is used for performing the response history analyses using the value of β= 0.25 and γ= 0.5. Damping is defined by the Rayleigh damping model by assigning 5%.

Performing an IDA is computation-intensive, as it entails a series of NRHA in which the intensity of each ground motion is progressively scaled until the target drift or intensity level is reached. As a result, the use of high-performance computing resources is imperative for conducting IDA on multiple RC walls with 44 ground motions. To address this computational challenge, computing resources provided by the Texas Advanced Computing Center (TACC) [34] is used along with an efficient parallel hunt-and-fill algorithm [35] which enables the use of multiple processors in parallel.

An IDA curve relates the seismic intensity of an earthquake and the engineering demand parameter (EDP) of the structure [36]. While conducting the IDA of the 10-storey RC wall, the ground motions are scaled at specific increments of first-mode spectral acceleration ($S_a(T_1)$), therefore, 44 IDA curves, including their median, 15th, and 85th percentiles are plotted as $S_a(T_1)$ versus the peak roof drift ratio as shown in Figure 6 (left). To enable a comparison of the IDA and POA results within the same domain, the equivalent peak base shear (V) recorded during IDA is plotted, as illustrated in Figure 6 (right).

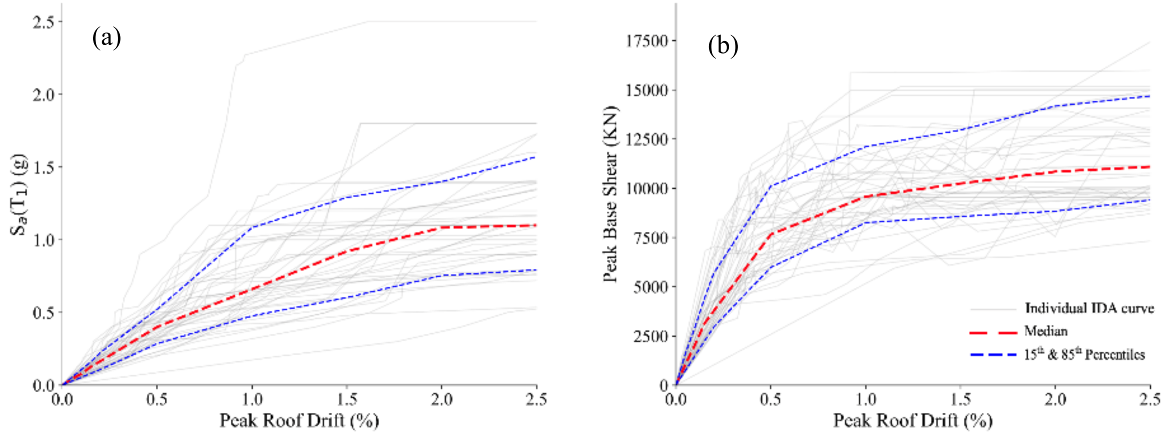


Figure 6 IDA curves and their median along with the 15% and 85% percentiles for the 10-storey RC wall expressed in terms of (a) $S_a(T_1)$ and (b) Base Shear

Inter-storey drift ratio profiles

The peak IDR at each storey of the 10-storey RC wall are recorded for each ground motion during IDA, which are plotted as individual grey lines in Figure 7 (a), alongside their corresponding mean values for different roof drift demands. The plotted IDR profiles represent the maximum IDR value at each floor along the wall height, which can occur at varying times during the response history analysis. As expected, the maximum peak IDR increases with the roof drift ratio.

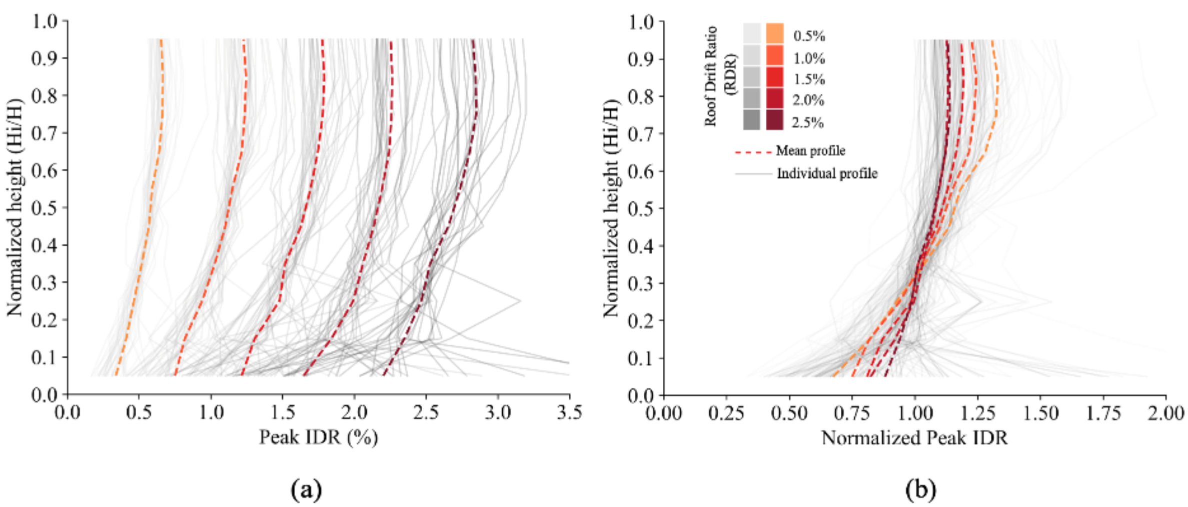


Figure 7. Individual and mean (a) Peak IDR and (b) Normalized Peak IDR profiles at various roof drifts for a 10-storey RC wall building subjected to 44 ground motions

Figure 7 (b) presents the peak Inter-Story Drift Ratios (IDRs) normalized by the overall roof drift ratio of the RC wall. This normalization illustrates how much the peak IDR profile deviates from the average total drift of the building. The IDR profiles reveal that normalized peak IDRs are concentrated around $H_i/H = 0.33$, and that the maximum peak IDR occurs on the 9th floor level (one floor below the roof). At higher roof drift demands, the IDR profile aligns more closely with the average roof drift of a building, while at lower roof drift demands, it tends to deviate from the average. A vertical IDR profile indicates that the structure is rotating about the base or bottom storey, with equal drift ratios in all floors. The deviation observed in the IDR profiles in Figure 7 suggests that the RC wall is responding in a mode such that it rotates more in the upper storeys, which is consistent with response of a cantilever wall.

COMPARISON BETWEEN THE RESULTS FROM STATIC AND DYNAMIC ANALYSIS

In order to compare the two analytical methods, the results from the POA and IDA described in earlier parts are plotted against one another in this section. The peak IDR profiles and capacity curves are used to make the comparisons.

This section aims to compare the results obtained from the two analytical methods described earlier. To achieve this, the peak Inter-Story Drift Ratio (IDR) profiles and capacity curves are compared. These plots serve as the basis for the comparisons between the two analytical methods.

Base shear force vs. lateral roof drift

Figure 8 presents the capacity curves of the analyzed RC wall obtained using both the Pushover Analysis (POA) and Incremental Dynamic Analysis (IDA). In this study, the capacity curves from IDA were plotted using the maximum base shear versus the maximum roof drift, which provides a significant advantage over other methods when compared to the pushover curve [12]. In the elastic range (i.e., at lower seismic demands), the pushover curve closely tracks the median capacity curve obtained from IDA. This result is reasonable as the wall would not have been significantly influenced by higher modes in the elastic region. As inelasticity increases, structures become softer and more vulnerable to higher modes, leading to a deviation between the IDA and POA capacity curves.

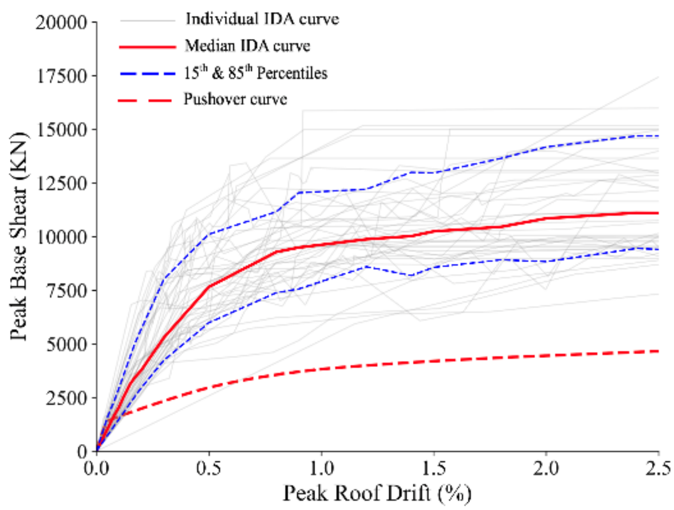


Figure 8. Comparison of the capacity curve from the POA with the median capacity curve from the IDA for the RC wall building.

The capacity curves in the inelastic range were compared in Figure 8, revealing a considerably higher dynamic shear response compared to the static ones. The shear capacity obtained from the dynamic analysis is approximately 2.4 times larger than that obtained from the static analysis. This finding aligns with previous research [5, 8, 37] that has discussed the phenomenon of shear force amplification in wall structures during dynamic analysis in contrast to static analysis.

The dynamic amplification of shear force is attributed to the contribution of higher mode effects, which cannot be accurately captured using the conventional POA. In the conventional POA, the wall is subjected to a lateral force distribution that is approximately proportional to its first mode shape, and therefore, the centroid of the triangular lateral force distribution is located at approximately 2/3 of the total height above the base of the wall.

During dynamic analysis, once a plastic hinge forms at the base of the wall, dynamic response characteristics of the wall change drastically, and different modes become predominant. The wall is then subjected to a lateral force distribution proportional to the mode shape representing higher modes of response and the centroid of the force distribution tends to move towards the base of the wall. Correspondingly, the dynamic shear force that can be generated become much larger than that indicated by a lateral force distribution in which the resultant lateral force is applied approximately at two-third of the wall's height [38].

Effects of RC wall parameters

The behavior of an RC wall is influenced by several structural parameters. Based on numerous experimental studies conducted on RC walls (e.g., [39, 40, 41]) and the fundamental understanding of their structural behavior, several key parameters are known to significantly impact the seismic performance of RC walls. Among them, number of storeys and shear span ratio

(SSR) are used in this study to investigate their effects on the comparison of the static and the dynamic analyses. Therefore, the previously analyzed 10-storey RC wall is used as a baseline model to generate other RC wall variants only to study the parametric effects. The range of variation for the structural parameters of the RC walls is summarized in Table 1.

Table 1 Structural Parameters Considered for RC Wall Cases.

Parameters	**Range**
Number of Storeys	4, 8, 10, 16
Shear Span Ratio (SSR)	6.0, 4.0, 3.0, 2.5, 2.0

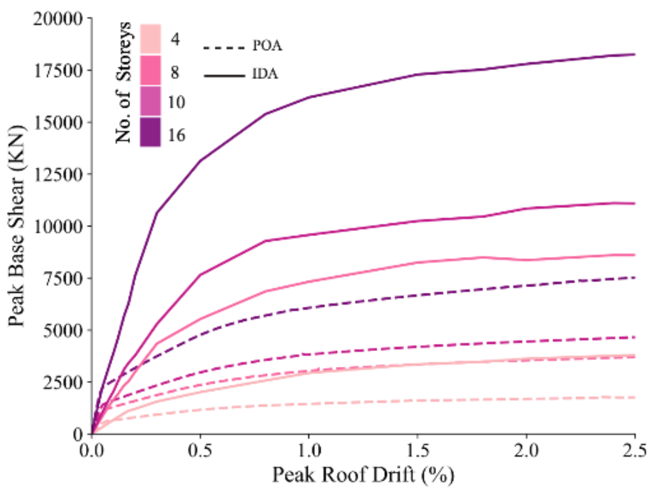
Effect of number of storeys

In order to investigate the effect of number of storeys on the static and the dynamic response of RC walls, three RC walls with 4, 8, and 16 storeys are modeled in OpenSees using SFI-MVLEM with constant axial load ratio, SSR, and reinforcement ratio, which was achieved by changing the wall length and the axial load. The fundamental time periods of the RC walls are obtained from eigenvalue analysis of the analytical model, which are presented in Table 2.

Table 2 Structural parameters for RC walls with varying number of storeys.

Number of Storeys	**SSR**	**ALR**	**Rebar Ratio**	**T1**
4	2.5	0.1	0.13	0.64
8	2.5	0.1	0.13	0.84
10	2.5	0.1	0.13	0.92
16	2.5	0.1	0.13	1.14

Figure 9 presents a comparison of capacity curves obtained using IDA and POA for four different RC walls with varying numbers of storeys. It is evident from the figure that the shear capacity of the walls increases with an increase in the number of storeys. This is because the length of the wall is increased to maintain a constant SSR across all RC wall cases with different numbers of storeys.



RC Walls	DAF
4-Storey	1.8
8-Storey	2.2
10-Storey	2.4
16-Storey	2.8

Figure 9. Comparison of capacity curves obtained from IDA and POA for RC wall buildings with different number of storeys and its effect on dynamic amplification factor (DAF) computed as the ratio of maximum base shear from IDA to maximum base shear from POA

Furthermore, Figure 9 also provides a comparison of the dynamic amplification factor (DAF) for the four RC walls. The DAF is defined in this study as the ratio of the maximum base shear obtained from dynamic analysis to that from static analysis. The values of DAF in the figure illustrate the variation in relation to the number of storeys since the amplification of the base shear depends on the mode shape of the structure as well as the degree of plasticity in the walls. As shown, the DAF is the lowest for the 4-storey RC wall and increases with the number of storeys.

Effect of shear span ratio (SSR)

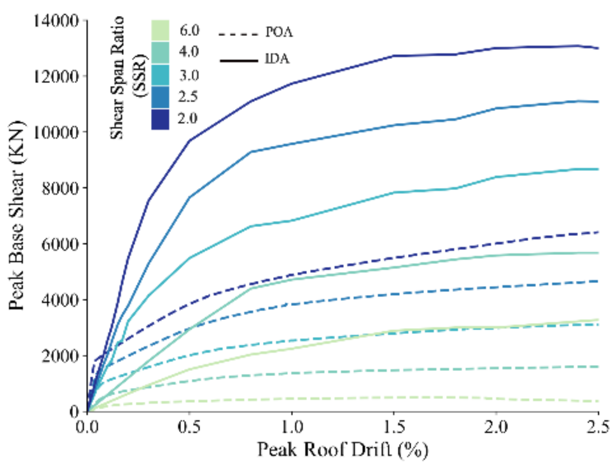
The SSR is defined as the ratio of the moment (M) developed at the base of the wall to the shear (V) developed at the same point, divided by the wall length (L_w). Since the SSR varies depending on the distribution of lateral load, an approximate value can be computed by assuming a triangular lateral load pattern. For cantilever RC walls, the SSR can be estimated as two-thirds of the vertical aspect ratio (H_w/L_w, where H_w is the total height of the wall). To simplify the calculation, we have approximated the SSR for the RC walls in our study as two-thirds of their H_w/L_w ratio. This approximation provides a reasonable estimate of the SSR.

Based on the SSR of RC walls, their behaviour can be categorised as slender, moderately slender, or squat. Slender walls with SSR > 2.5 exhibit flexure-dominant behaviour, while squat walls with SSR < 1.5 exhibit shear-dominant behaviour. Moderately slender walls with SSR between 1.5 and 2.5 are generally influenced by both nonlinear flexural and shear deformations.

Similar to how the RC wall variants were generated to investigate the impacts of the number of storeys, further RC walls are generated to study the influence of the SSR on comparison of the static and the dynamic response of RC walls. While maintaining the same number of storeys, axial load ratio, storey mass, and other parameters, four RC wall models with varying SSRs (2.0, 3.0, 4.0, and 6.0) are derived from the 10-storey base model by modifying the wall length and the axial load. The fundamental time periods of the RC walls are obtained from eigenvalue analysis of the analytical model, which are presented in Table 3.

Table 3 Structural parameters for RC walls with varying SSR

SSR	Number of Storeys	ALR	Rebar Ratio	T1
6.0	10	0.1	0.13	3.77
4.0	10	0.1	0.13	1.96
3.0	10	0.1	0.13	1.27
2.5	10	0.1	0.13	0.92
2.0	10	0.1	0.13	0.71



SSR	DAF
2.0	1.9
2.5	2.4
3.0	2.7
4.0	3.6
6.0	6.6

Figure 10. Comparison of capacity curves obtained from IDA and POA for RC wall buildings with different SSR and its effect on dynamic amplification factor (DAF) computed as the ratio of maximum base shear from IDA to maximum base shear from POA.

Figure 10 compares the capacity curves obtained from IDA and POA for five RC walls with different SSR. The figure shows that as the SSR increases, the shear capacity of the RC wall decreases due to the decrease in the wall length, which reduces the wall's section capacity. Furthermore, the influence of SSR on the DAF is illustrated in Figure 10. The wall with SSR of 6.0 exhibits the highest amplification of base shear, while the amplification gradually decreases as the SSR decreases to 2.0. This finding highlights the impact of SSR of RC walls on DAF.

Inter-storey drift ratio (IDR)

Figure 11 presents the absolute and normalized IDR profiles for various levels of roof drifts obtained by POA and IDA. The mean peak IDR values from 44 records are derived for IDA at each storey level. The IDR for different levels of roof drifts obtained from POA and IDA are presented in the form of absolute and normalized profiles in Figure 11 (a) and (b) respectively. For IDA, the mean values of peak IDR from 44 ground motions are plotted at each storey level. The IDR profiles clearly illustrate a progressive deformation that correlates with the increasing roof drift demand.

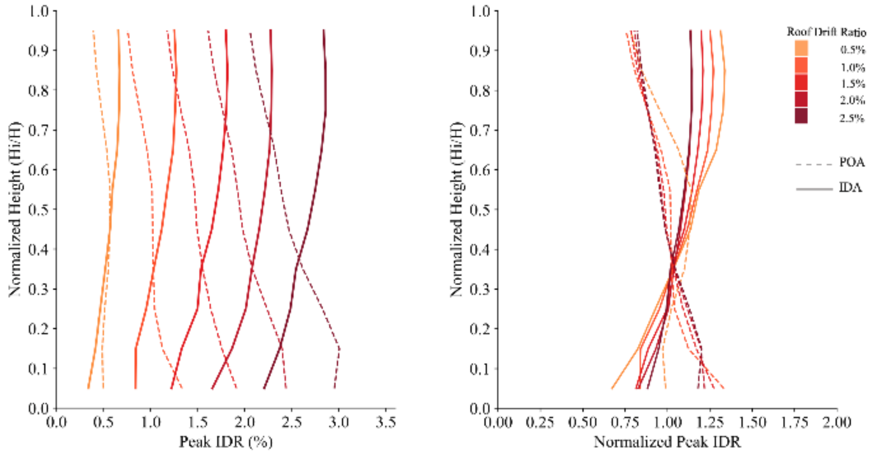


Figure 11. Comparison of IDR profiles at varying roof drift levels obtained from POA and IDA of the 10-storey RC wall building.

Figure 11 shows that the IDR profiles obtained from POA is distinctly different to that from the IDA. Based on dynamic analysis of the 10-storey RC wall, the IDRs reach their maximum value on the ninth floor. In contrast, the IDRs calculated by the static analysis are at their maximum on the first floor of the building.

Effect of number of storeys

Figure 12 compares the mean values of peak IDR profiles obtained from the IDA with the POA results for RC walls with different number of storeys. The narrow bands of the IDR profiles in the figure indicate that the IDR profiles do not differ noticeably with the number of storeys.

Effect of shear span ratio (SSR)

The results of peak IDR of the RC walls with varying SSR derived from the POA and the IDA are compared in Figure 13. According to Figure 13, SSR has a substantial effect on the peak IDR of an RC wall. As the SSR increases, the maximum peak IDR of the RC walls increases, indicating a correlation between RC wall deformation capacity and slenderness. The IDR profiles in the Figure 13 indicates that the IDA predicts the IDR to be greater in upper half, whereas POA predicts IDR to be higher in the lower half, and they cross-over at IDR =1 around the mid-height.

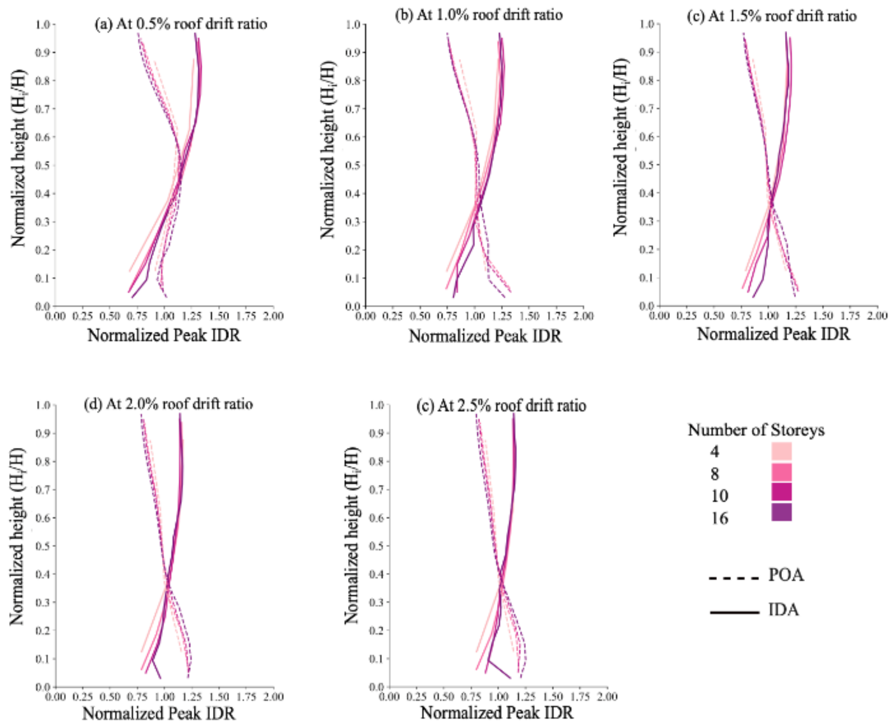


Figure 12. Comparison of IDR profiles at varying roof drift levels obtained from POA and IDA of the RC wall buildings with different number of storeys.

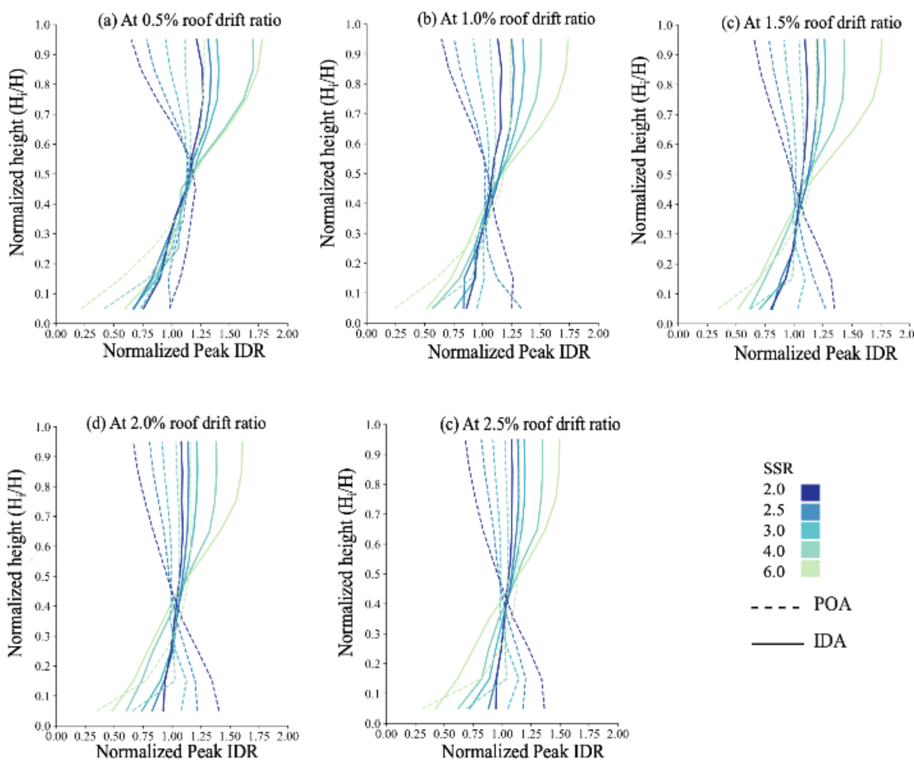


Figure 13. Comparison of IDR profiles at varying roof drift levels obtained from POA and IDA of the RC wall buildings with different SSR.

The parametric analysis results presented above suggest that seismic response of RC walls is mostly influenced by SSR, whereas the number of storeys has a little effect, as shown by plots of the peak IDR profiles in Figure 12 and Figure 13.

Across all the RC wall cases investigated in this study, it is apparent that POA consistently overestimates the IDR in the lower storeys and underestimates it at higher storeys for all variants of the RC walls. These discrepancies remain consistent regardless of the number of storeys or SSR of the RC walls.

CONCLUSIONS

This study has demonstrated the significant deviations in the response of RC walls obtained from conventional pushover analysis (POA) and those obtained using incremental dynamic analysis (IDA). The findings from the study suggest that the conventional POA significantly underestimates the shear capacity of analyzed RC wall buildings, mainly due to its inability to account for higher mode effects, in comparison with the IDA. The differences in shear capacity between POA and incremental dynamic analysis (IDA) have been quantified using the dynamic amplification factor (DAF), which is affected by the number of storeys and the shear span ratio (SSR) of the RC wall. Taller and slender RC walls exhibit higher dynamic amplification of base shear. Moreover, the inter-storey drift ratio (IDR) profiles obtained from POA differ significantly from the mean peak IDR profiles obtained from IDA, with these discrepancies remaining consistent across all RC wall cases analyzed, irrespective of the number of storeys or the SSR of the RC wall.

Overall, despite the popularity of POA in engineering practice for structural analysis, the essential differences between the results obtained from IDA and POA make it challenging to rely on POA to estimate the seismic demand of RC wall buildings accurately. Therefore, it is crucial to consider advanced analysis techniques like IDA to obtain more accurate and reliable results.

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