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https://doi.org/10.71170/tecoj.2025.1.3.36-48 **Research Paper**

Computational Seismic Evaluation of Existing RC-Building Structures in Karachi Subjected to Multiple Earthquakes

Shifa Moiz[1], Aslam Faqeer Muhammad[1*], Muhmmad Tayyab[1], Saddam Hussain[2] and Hassan Javed[3]

[1] *Dept of Civil Engineering, NED University of Engineering & Technology, Karachi, Pakistan.*
[2] *Department of Civil Engineering and Architecture, Kyushu Institute of Technology, Kitakyushu 804-0015, Japan*
[3] *Department of Civil Engineering, Lahore Leads University, Lahore, Pakistan*
**Corresponding Author Email:* maslam@neduet.edu.pk

Abstract

This study addresses the vulnerability of reinforced concrete structures to multiple seismic events, considering the often-neglected effects of prior and subsequent shaking. The research emphasizes the need for a comprehensive evaluation of various structural systems to gauge the cumulative impact of multiple earthquakes, aiming to inform design processes for enhanced structural safety. The study specifically examines a mid-rise residential reinforced concrete building subjected to seven earthquake sequences, combining actual and artificial repeating events. The objective of the study is to investigate the response of an existing RC building structure located in Karachi, subjected to multiple shocks of earthquake. The scope of the current research covers the response evaluation of low to mid-rise RC building structures (engineered and non-engineered) located in Karachi. To this end, an FEM model has been developed with different sources of nonlinearities (Geometric and Material) and analyzed for recorded and simulated multiple sequences of earthquake. The research assesses the structure's response in terms of drifts, maximum displacements, and damage patterns. The results contribute valuable insights for understanding structural behavior under seismic conditions, with a specific emphasis on the percentage difference in displacement between the main shock and combined main shock and aftershock scenarios. The observed trends, including pronounced variations in displacement across different story levels, inform seismic vulnerability mitigation and retrofitting strategies, ultimately enhancing structural safety in earthquake-prone regions.

Keywords: Multiple Seismic Events, Residual Capacity, Plastic Hinges, Damage Pattern, Maximum Drift, Maximum Displacement

1. Introduction

Globally, multiple earthquakes happen in those regions where there exists complexity in fault systems [1]. A structure damaged by a seismic event is exposed to the risk of aftershocks or another event within a certain time period. In many regions of the world, a repetition of ground motions has been experienced at a short interval of time, which has accumulated damage to the structure, affecting its stiffness, strength, and ductility. Generally, these structures were able to withstand the main shock but collapsed under successive earthquakes. Figure 1. shows the recent damages observed due to the multiple shocks in Turkey [2]. A magnitude 7.8 earthquake struck southern and central Turkey as well as northern and western Syria on February 6, 2023, at 04:17 TRT (01:17 UTC). 37 km to the northwest of Gaziantep was the epicenter. In some areas of Antakya in Hatay Province, the earthquake's Mercalli intensity peaked at XII (Extreme). At 13:24, a Mw 7.7 earthquake occurred. The center of this earthquake was located 95 km northeast of the previous one. Tens of thousands were killed and there was extensive damage. In the next three weeks, there were more than 10,000 aftershocks reported. The earthquake sequence was set off by the shallow strike-slip faults [3]. Reinforced Concrete Structures are vulnerable when

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subjected to shocks. Reported literature shows the vulnerability assessment of existing structures under the most detrimental earthquake, but the effects due to prior and post-shaking were neglected. Therefore, a concise review of building typology, seismic hazard, non-linear response history analysis, evaluation, and retrofit of damaged buildings is needed for the case of multiple sequences of earthquakes, as observed in many regions of the world When the first rupture occurs, the fault systems typically do not release all the built-up strains simultaneously. Instead, a series of earthquakes are caused by successive ruptures along the fault segment or segments, which are typically difficult to identify as fore-, main-, and aftershocks or as a series of earthquakes from nearby fault segments. Generally, a moderate-sized quake is 2 followed by aftershocks of a similar or even greater magnitude, according to surveys conducted in the wake of any seismic event. Some examples of such events are the Northridge (1994), L'Aquila (2009), Tohoku (2011), Darfield (2010), Christchurch (2011), and Kathmandu (2015) Earthquakes. As a result, a great deal of research is ongoing aimed at investigating the response of structures following seismic sequences [4].

Limited work has been done to study the MDOF system under repeated earthquakes. Some researchers such as [5], [6], [7], [8] performed extensive parametric studies to determine the inelastic response of reinforced concrete and Steel Frames. The frame system mentioned in the literature is component-level-based degrading models for simplicity since degrading models for reinforced concrete and steel frames are difficult to incorporate in analyses. [5] evaluates the Sisma-bonus guidelines in seismic assessment, focusing on identifying limitations and proposing practical refinements for future applications. It underscores the need to address shortcomings in current collapse safety codes and emphasizes the importance of user-friendly tools for effective guideline revisions. The discussion also explores recent advancements in seismic assessment tools and their potential integration into risk assessment 8 guidelines. Through practical examples and comparisons with contemporary methods, the paragraph advocates for a proactive and precise approach to seismic safety, aiming to enhance structural integrity during seismic events. The main objective of this was that a building should resist ground motion without collapse, but some damage is acceptable. However, a study discovered that when these buildings face multiple earthquakes, they can fail because the damage adds up over time. This means that the wear and tear from each earthquake can accumulate, eventually causing the building to give in under repeated shaking. Understanding this helps us improve how we design buildings to better handle the challenges of facing multiple earthquakes, making them more resilient and safer [6][7] utilizes the SPO2IDA approach to understand and address the distinct structural behavior of reinforced concrete (RC) frames with masonry infill panels, which differ from typical structures. The focus is on refining the SPO2IDA tool to accurately assess seismic risks in these specific frames by addressing modeling uncertainties. It is suggested that further research to enhance predictive accuracy, aiming to broaden the tool's application in various engineering scenarios. Ultimately, the goal is to contribute to improved seismic assessment methods, promoting increased safety and resilience for buildings with RC frames and masonry infill panels.

***Figure 1**: Damaged Observed in Turkey-Syria Earthquake, 2023(ref.)*

Table 1. *Summary of selected studies on the effects of mainshock–aftershock (repeated/sequence) ground motions on reinforced concrete (RC) frame structures and building damage performance.*

Reference	Key Context / Highlights	Study Objective / Main Finding
[8]	Highlights the common occurrence of aftershocks in such events, especially in regions near fault lines. These aftershocks can cause additional damage to structures, emphasizing the importance of considering them in structural design and assessment.	Evaluating the effect of near-fault seismic sequences on the accumulated damage of reinforced concrete (RC) frame structure, in which different initial damage levels of the structure after the mainshock are considered.
[9]	This study was an extension of Abdelnaby's work where he compared regular and irregular buildings under Tohoko ground motion sequence.	This study concluded that irregular buildings induced greater damage under repeated ground motion sequences when compared with it's regular counterpart.
[4]	To determine the effect of stiffness and strength degradation on reinforced concrete structures under repeated earthquakes using two real ground motion sequences and introducing a degrading model.	Damaged frames attracted less seismic forces and showed better performance when compared with initially undamaged structures, emphasizing the need to further investigate the effect of multiple ground motion shaking on a structure.

2. Seismic Input

There are no ground motion records available for Karachi as there is still uncertainty about the seismic risk of this region therefore for performing multiple earthquake analyses on the building frame systems a suit of ground motions is selected randomly. The return period of 475 and 2500 years corresponds to life safety and collapse prevention performance indicator respectively. According to the building code of Pakistan [10] PGA for a return period of 2500 years is used for designing Dams but due to uncertainty associated with seismic hazard in Karachi reported by different authors [11]. In the current study, the infilled and their bare counterparts are analyzed through the suite of ground motions scaled on

3.1. Real seismic sequence

Three actual seismic sequences that were recorded over a brief period (up to three days), at the same station, in the same direction, and at the same fault distance make up the first strong ground motion database. These real sequences are the Coalinga, Whittier Narrows, and Imperial Valley earthquakes. These earthquakes were plotted for main and main and after multiple aftershocks subjected to the R.C building to evaluate the response of the building. Table 2 shows the parameters for these real sequences which were downloaded from the strong motion database of the Pacific Earthquake Engineering Research (PEER) Center (Pacific Earthquake Engineering Research Center.)

Table 2. *Seismic parameters of real sequential ground motion (Pacific Earthquake Engineering Research Center).*

Real Seismic Sequence				
Ground Motion	**Station**	**Date and time**	**PGA(g)**	**PGA(g) Matched**
		1983/7/22		
Coalinga	14th & Elm (Old Chap)	1983/7/25	0.42	0.42
		1987/10/01	0.47	0.46

Whittier Narrows	San Marino- SW Academy	1983/10/04	0.13	0.14
		1979/10/15 (23:16)	0.15	0.13
Imperial Valley	5055 Holtville P.O.	1979/10/15 (23:19)	0.26	0.4
		1983/7/22	0.12	0.5

In both the real and repeated artificial seismic sequence cases a time buffer of 100 seconds is applied between the successive ground motion records as shown in Figure 2 having zero acceleration. This is to make sure that the structure is brought to rest before the second event to have no remaining dynamic influence of the first event.

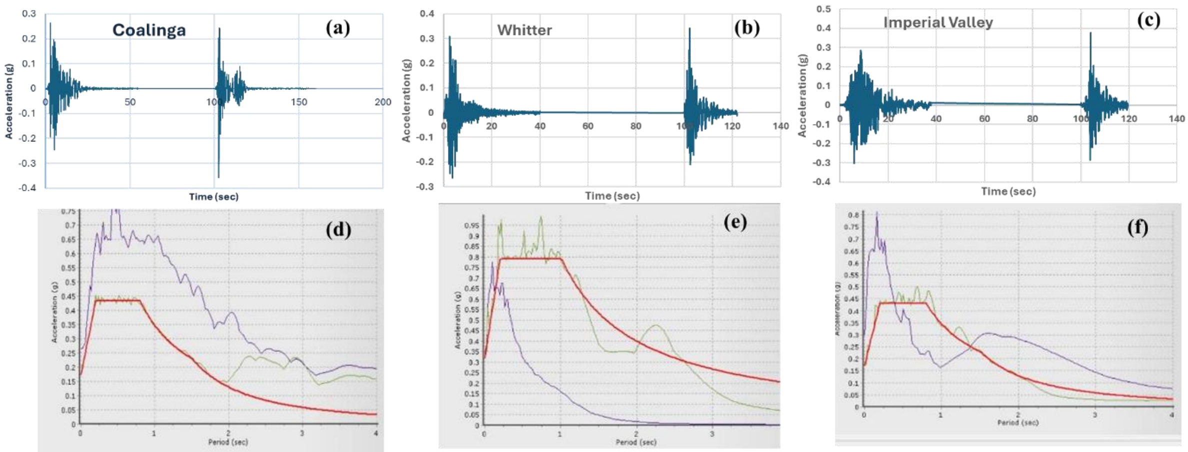

Figure 2*: (a) Acceleration time history record of Whitter Narrows seismic sequence.* ***(b)*** *Acceleration time history record of Imperial Valley seismic sequence. (c) Response spectra of Coalinga ground motions. (d) (Target Spectra ASCE41 OR ASCE7-00) (e): Response spectra of Whittier Narrows ground motions. (f) Response spectra of Imperial Valley ground motions.*

3.2. Artificial seismic sequence

The second strong ground motion database consists of four artificial seismic sequences, more specifically two identical ground motions are applied in series creating a synthetic sequence. Table 3 shows the characteristics of single ground motion which were used to generate artificial sequences (Figure 3).

Table 3*. Seismic parameters of artificial sequential ground motion*

Ground Motion	Station	PGA(g)	PGA (g) Matched
Altadena	Eaton Canyon Park	0.44	0.42
Corralit	Eureka Canyon Road	0.63	0.45
Santa Monica	City Hall	0.37	0.45
Hollister	South Street and Pine	0.37	0.43

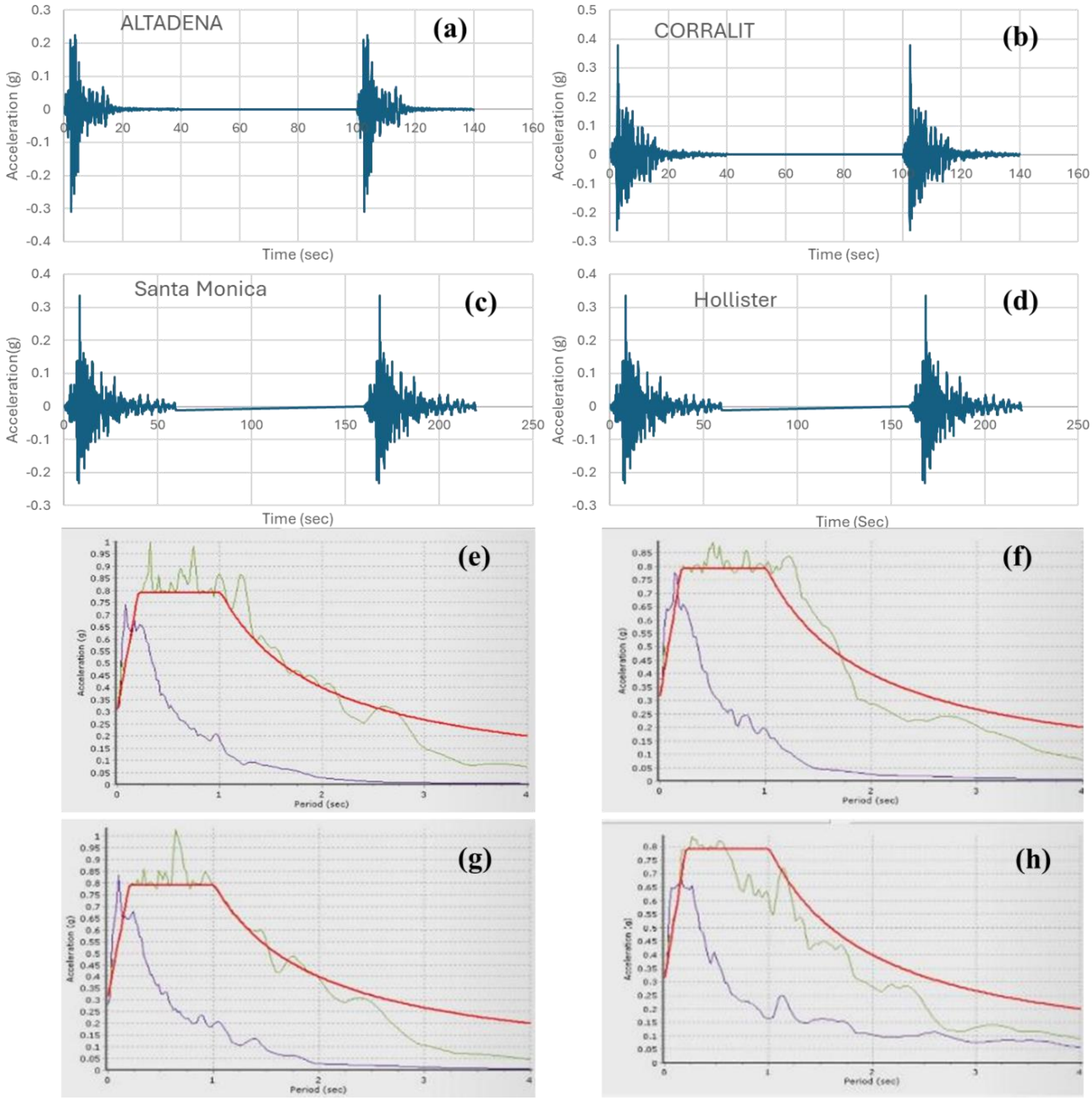

***Figure 3**: (a) Acceleration time history record of repeated Altadena seismic sequence (b) Acceleration time history record of repeated Corralit seismic sequence (c) Acceleration time history record of repeated Santa Monica seismic sequence (d) Acceleration time history record of repeated Hollister seismic sequence **(e)** Response spectra of Altadena ground motion (f) Response spectra of Corralit ground motion **(g)** Response spectra of Santa Monica ground motion **(h)** Response spectra of Hollister ground motion.*

3.2. Results And Discussions

4. 1 Non-Linear Static Pushover Analysis

The process of non-linear static pushover analysis involves calculating the structure's capacity using a straightforward plot between the base shear and the structure's displacement. Inelastic static procedures require simple model representations and less number of analyses when compared with dynamic analyses. [12] Conventional pushover analyses are adopted for conducting on the frames to determine the real capacity of the frames and their performance level. The localized failure in beams and columns is monitored by observing the formation of plastic hinges and soft stories in both frames. Before applying the pushover load, the frames are subjected to constant gravity loads using the combination as per code UBC-97

$$1.4\ D.L \tag{1}$$

$$1.2\ D.L + 1.6\ L.L \tag{2}$$

$$1.1(1.2\ D.L + 0.5\ L.L + 1.0E) \tag{3}$$

$$1.1(0.9\ D.L + 1.0E) \tag{4}$$

Where D.L is dead loads of the structure including self-weight of beam, columns, slabs, walls and superimposed and L.L is live load applied on the structure. Then the structure is subjected to incrementally increasing monotonic loads in a triangular pattern in the X-Direction of the frames. Plotting the relationship between the structure's base shear and peak roof displacement in Figure 4 illustrates the response of the structures. A performance point is found with the intersection of the capacity spectrum and single demand spectrum for both frames. The deformation of the structure is indicated by the formation of plastic hinges mostly at ground level for both frames.

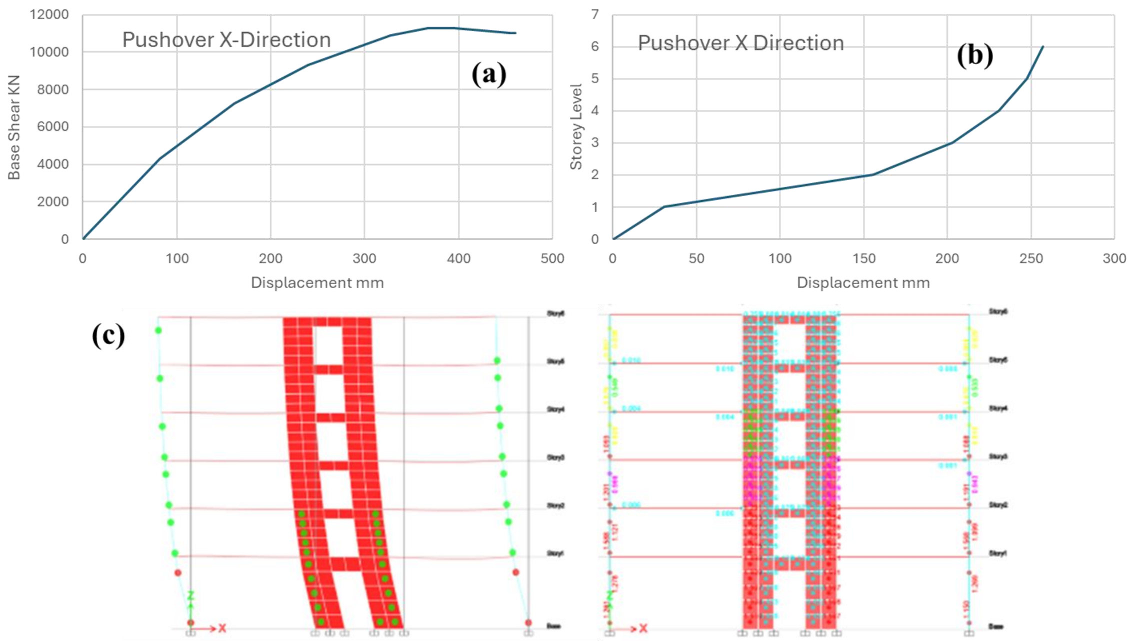

***Figure 4**: (a) Comparison between bare and infill frame in terms of strength (b) Comparison of story displacement between an infill and bare frame at Performance Point (c) Plastic deformation at performance point.*

There are plastic hinges formed in the columns of ground and 1st story reaching CP highest value of D/C ratio is 1.5 where the rotations have reached a collapse prevention level at the performance point of the structure. The roof drift of a structure is a measure of determining the overall damage observed in a structure and is found by performing response spectrum, non-linear static, or dynamic procedures. The drift and displacement of the structure are functions of stiffness, strength, and ductility. Although roof drift is a measure of observed damage to the overall building, they do not reflect the distribution of damage along the height of the structure nor do they identify soft stories. However, the inter-story drift can correlate with the damage at floor level and correspond to design or serviceability checks for beams and columns in the frame. [13] use of drift limits is a way of safeguarding against the loss of human life and property after a seismic event. Various codes have set the drift limits for the three performance levels i.e. Life Safety, Immediate Occupancy, and Collapse Prevention. Table 4 summarizes the code limits set by different codes for concrete frames.

4.2 Nonlinear Dynamic Response History

To examine the behavior of a reinforced concrete frame under multiple earthquake loading, non-linear dynamic response history is performed using a suite of ten ground motions in Section 3 of this report. These motions are split into two categories: real and replicated seismic sequence. Following the development of plastic hinges in the structure, the response of the frames is reported in terms of peak floor accelerations, maximum floor displacements, residual capacity, and interstorey drifts. A comparison of seismic behavior between RC frames under repeated ground motions is also provided. The RC frame was subjected to three real and four synthetic seismic sequence and the results are presented at global level. There are two events in each sequence.

4.2.1 Inter Story Drift

The deviation of a single story from the preceding story is known as story drift. Since the story drift is based on the levels that are close by. Less than 1% drift ratio indicated non-structural damage, however more than 4% drift ratio indicated irreversible damage. The effects of cumulative damage from repetitive inelastic deformation are not taken into account by drift ratios

Table 4*: Inter-story drift limits in various codes for non-linear static analysis.*

Code	**Acceptance Criteria**	**Inter-Story Drift Limits**
		Concrete Frame
FEMA 356/ASCE 41-13	Immediate Occupancy	1%
	Life Safety	2% (transient) 1% (permanent)
	Collapse Prevention	4 %
UBC-97	Allowable Drift	2.5%
EC-8	Immediate Occupancy	0.5%
	Life Safety	0.75%
	Collapse Prevention	1%
ATC-40	Immediate Occupancy	1%
	Life Safety	1-2%
	Collapse Prevention	2%

The results for inter-story drifts were calculated using the displacements obtained from pushover at the highest step size which was step 7 summarized in figures 4 in the form of inter-story drift profiles. ASCE 41-13 code is used in this study to set limiting values in the assessment of the frames so figure 5 interstory drift profile shows ASCE 41-13 code limits. Figure 5 shows the 41 limiting drift values for each performance level. Figure 5 shows that frame exceeds the allowable drift limit for UBC-97 which is 2.5% at story whereas it exceeds 1% of more than one story which crosses the immediate occupancy acceptance criteria for ATC 40, EC-8, and FEMA 356/ASCE 41-13. According to the ASCE 41-13 code the structure is within in Collapse Prevention level, but if we follow the EC 8 and ATC-40 provisions then it has exceeded the collapse prevention. Since the medium of this study is to use ASCE 41-13 the fame is within the collapse prevention performance level which is the acceptance criteria for a mixed-use building

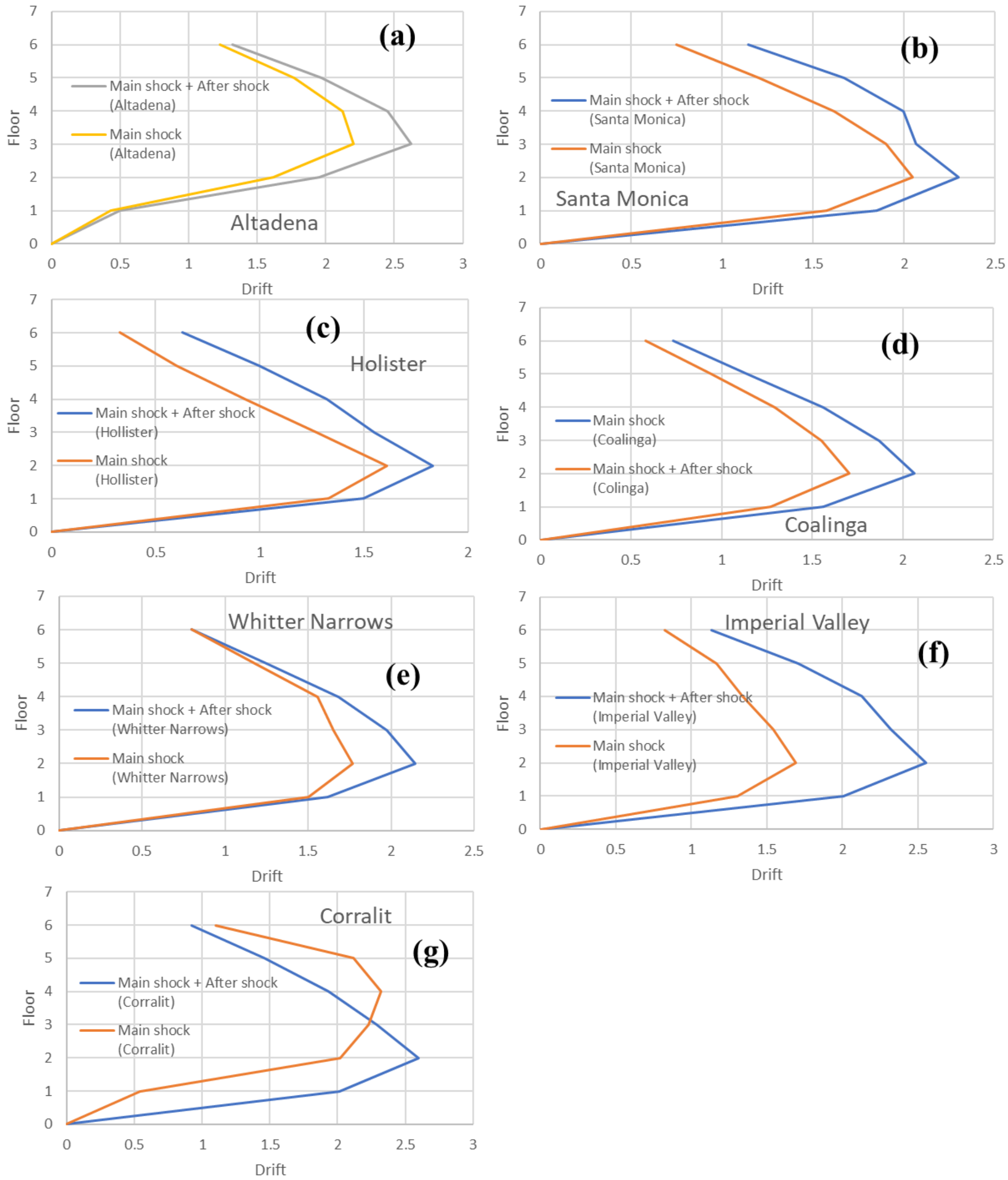

Figure 5: *(a) Graphically representation of both cases of Altadena Ground Motion (b) Graphically representation of both cases of Santa Monica Ground Motion (c) Graphically representation of both cases of Hollister Ground Motion (d) Graphically representation of both cases of Coalinga Ground Motion (e) Graphically representation of both cases of Whittier Narrows Ground Motion (f) Graphically representation of both cases of Imperial Valley Ground Motion (g) Graphically representation of both cases of Corralit Ground Motion.*

4. 2.2 Maximum Floor Displacements

Multiple earthquake events (mainshock–aftershock or repeated sequences) generally produce higher maximum floor displacement demands than a single ground motion because the initial shaking drives inelastic action, leaving the structure with residual deformation, cracked components, and reduced stiffness and strength; as a result, the subsequent event acts on an already weakened system and additional displacement accumulates more easily. The displacement profiles for the different records show that this amplification is most evident at the upper storeys, where lateral deformation typically peaks, indicating period elongation and increased flexibility after the first event. Although the magnitude of increase varies from record to record—reflecting differences in sequence intensity and frequency content—the overall trend confirms that relying on single-event analysis can underestimate deformation demand in regions prone to aftershock activity. These elevated displacement levels imply greater interstorey drift, higher risk of P–Δ effects and instability, and more severe structural and non-structural damage, reinforcing the need to account for damage accumulation and degradation when assessing or designing reinforced concrete frames subjected to earthquake sequences [14] (Figure 6).

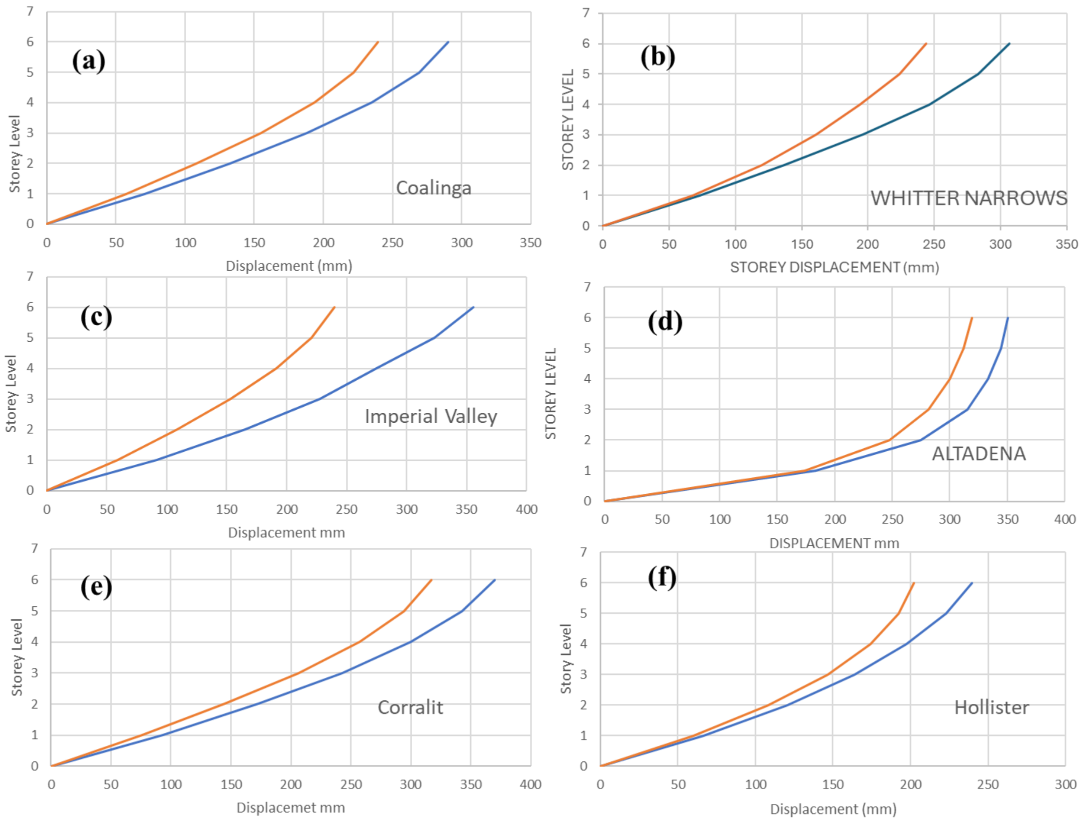

***Figure 6**: Maximum horizontal displacement profile under Coalinga ground motion **(b)** Maximum horizontal displacement profile under Whittier Narrows ground motion (c) Maximum horizontal displacement profile for Imperial Valley (d) Maximum horizontal displacement profile for Altadena ground motion (e) Maximum horizontal displacement profile for Corralit ground motion (f) Maximum horizontal displacement profile for Hollister ground motion.*

4. 2.3 Damage Patterns

Damage patterns in reinforced concrete (RC) frame structures subjected to seismic loading are most clearly interpreted through the formation and progression of plastic hinges, which provide a direct indication of where inelastic demand is concentrated and how the lateral-force-resisting system dissipates energy. In this study, damage is evaluated using a lumped plasticity modelling approach, in which nonlinear behavior is represented by assigning concentrated plastic hinges at critical locations along frame members (typically at beam and column ends) and at designated regions of RC walls. This approach is widely used in nonlinear seismic assessment because it captures the essential inelastic response—yielding, stiffness degradation, and strength deterioration without the computational burden of distributed plasticity models, while still providing meaningful insight into expected failure mechanisms and performance levels under strong shaking. Under a single earthquake excitation, plastic hinges commonly initiate at beam ends in ductile RC moment-resisting frames, reflecting the desirable "strong-column–weak-beam" mechanism that promotes energy dissipation through flexural yielding in beams while limiting brittle column failures. However, when the structure is subjected to multiple earthquakes or mainshock–aftershock sequences, the damage pattern typically evolves beyond the initial hinge formation. The first event may create a set of hinges at highly demanded locations and may also induce residual drifts and cracking that reduce effective stiffness. Consequently, the subsequent event does not begin from an intact state; instead, it drives the system further into the inelastic range, leading to hinge reactivation (additional rotation demand at already yielded sections), hinge spreading (formation of new hinges at adjacent ends or other storeys), and in severe cases, progression from beam-dominated hinging to column hinging, which is more critical for global stability. This transition is particularly important because column hinges can indicate a shift toward an undesirable "soft-storey" or "weak-storey" mechanism, increasing the likelihood of instability under amplified drift demands.

For RC walls, repeated ground motions can cause progressive damage accumulation in regions of high curvature and shear demand, such as the wall base (plastic hinge region) and coupling zones (if present). In a lumped plasticity representation, this is reflected by increasing hinge states at these critical wall locations as shaking sequences continue. Unlike beam yielding, wall damage may include mixed flexure–shear interaction, stiffness loss, and reduced lateral resistance that can substantially alter the building's deformation shape and redistribute demands to the surrounding frame elements. As the wall stiffness degrades, the system may experience greater participation from the frame, causing additional hinge formation in beams and columns that were less engaged during the initial event. This redistribution under sequences is a key mechanism by which multiple earthquakes can transform an initially acceptable damage state into a more critical condition, even if the aftershock is of moderate intensity. Figure 7 illustrates these trends by showing the development and distribution of plastic hinges under repeated artificial and real seismic sequences. In general, the figures are expected to demonstrate that (i) plastic hinges form earlier and in greater number under sequences than under a single motion, (ii) hinge severity increases with repeated cycling, indicating cumulative rotation demand and degradation, and (iii) hinge concentration may become more pronounced at particular storeys, revealing potential weak-storey behavior depending on the building configuration and the characteristics of the seismic sequence. The comparison between artificial and real sequences is also important: artificial sequences can be designed to impose controlled intensity and frequency content, whereas real sequences capture record-to-record variability and the irregular nature of aftershock excitation. Despite these differences, both types consistently emphasize the same fundamental outcome—damage is cumulative, and repeated shaking can escalate localized yielding into a broader mechanism that threatens serviceability, repairability, and ultimately collapse safety if deformation and degradation become excessive.

Repeated-earthquake loading also has an important implication for how "damage" should be interpreted in performance-based assessment. Under a single event, hinge formation is often treated as an endpoint indicator of exceedance of a limit state (e.g., Immediate Occupancy, Life Safety, Collapse Prevention). Under sequences, however, the same hinge state can represent a starting condition for the next excitation, meaning that performance is governed not only by peak response but also by the residual capacity that remains after the mainshock. In practical terms, this requires tracking both peak plastic rotations and residual deformations (residual interstorey drift ratios) as complementary damage measures, because residual drift can control repairability and can predispose the structure to concentration of inelastic demands in the aftershock. Even when peak drift demands during the aftershock are not dramatically higher than those of the mainshock, the presence of pre-existing cracking, reduced stiffness, and accumulated cyclic deterioration may cause a disproportionate increase in hinge severity and a faster transition toward instability-sensitive mechanisms, particularly in lower storeys. In addition, the evolution of damage patterns under sequences is strongly influenced by the characteristics of the ground-motion records and the structural configuration. Near-fault motions with velocity pulses can impose large, rapid drift demands that trigger early yielding and large hinge rotations, while longer-duration motions may accumulate damage through repeated cycles, amplifying degradation even at moderate drift levels.

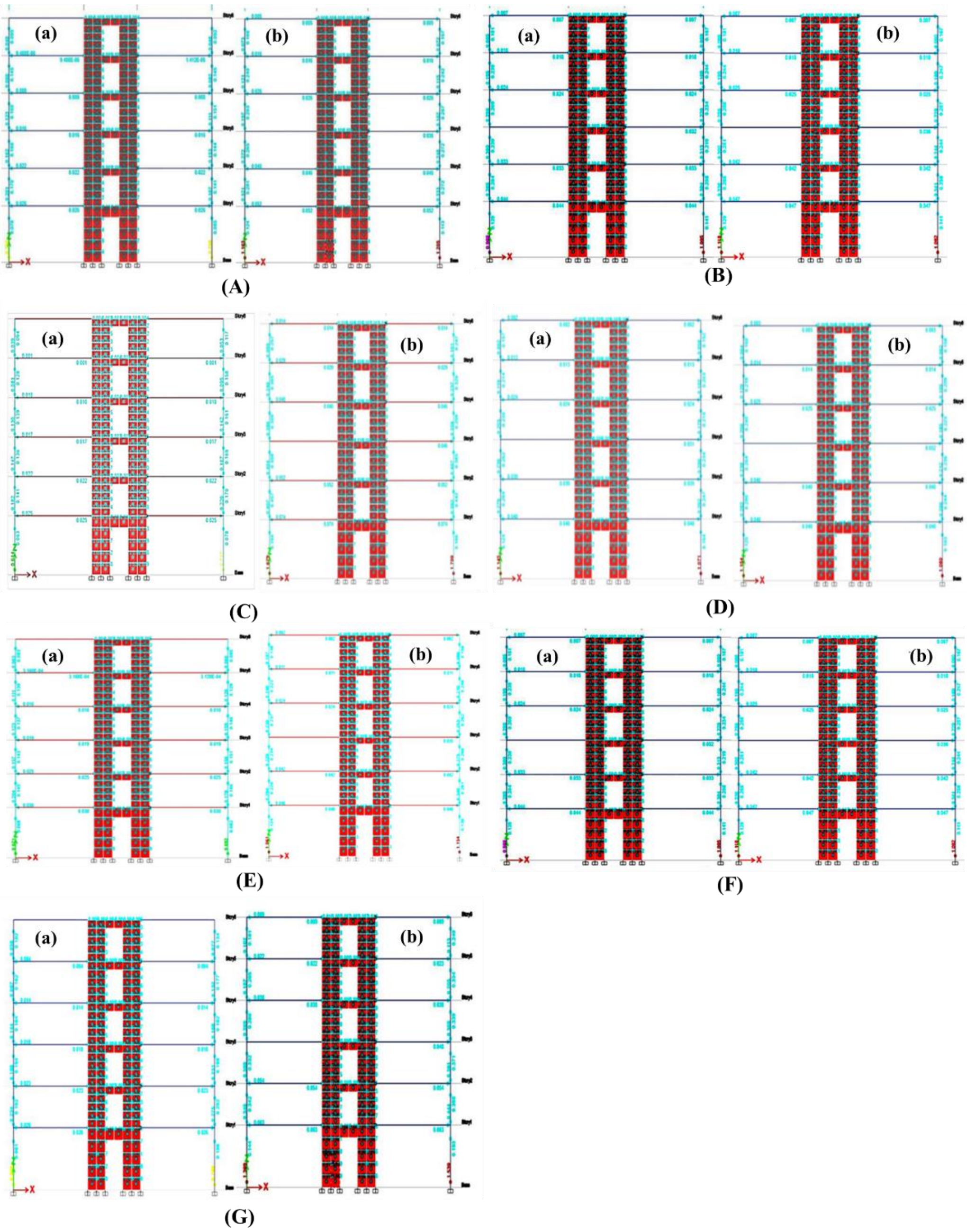

***Figure 7**: (A) Plastic hinge formation at (a) Main shock and (b) Main and After shock of Altadena ground motion for RC frame (B) Plastic hinge formation at (a) Event 1 and (b) Event 2 of Corralit ground motion for RC frame (C) Plastic hinge formation at (a) Event 1 and (b) Event 2 of Santa Monica ground motion for RC frame (D) Plastic hinge formation at (a) Main Shock and (b) Main and Aftershock of Hollister ground motion (E) Plastic hinge formation at (a) Event 1 (b) Event 2 of Coalinga ground motion for RC frame (F) Plastic hinge formation at (a) Event 1 (b) Event 2 of Whittier Narrows ground motion (G)Plastic hinge formation at (a) Event 1 and (b) Event 2 of Imperial Valley ground motion for RC frame*

4.3 Residual Capacity

Residual capacity refers to the remaining strength or load-bearing capability of a structure or component after being subjected to various stresses, damage, or deterioration. It is a crucial parameter in assessing the structural integrity and safety of a system, especially in the context of seismic events or other extreme conditions. Residual capacity is determined by evaluating the structure's ability to withstand loads or forces beyond its original design parameters, considering any damage or degradation it may have experienced over time. During the Santamonica, Coaralit and Coalinga Sequence in the first earthquake event, the base shear exceeds the limit set by the Nonlinear Static Pushover Curve, indicating that the structure faced a stronger seismic force than anticipated in its design. Whereas, during the second seismic event, the Base Shear remains within the predefined limit of the Nonlinear Pushover Curve, specifically at 11298 kN. This signifies that the RC frame performed within expected parameters during the second earthquake, showcasing its resilience and ability to withstand seismic forces without exceeding pushover limits. This concept is essential for engineers and analysts when assessing the performance and potential retrofitting needs of structures to ensure their continued functionality and safety (Figure 8).

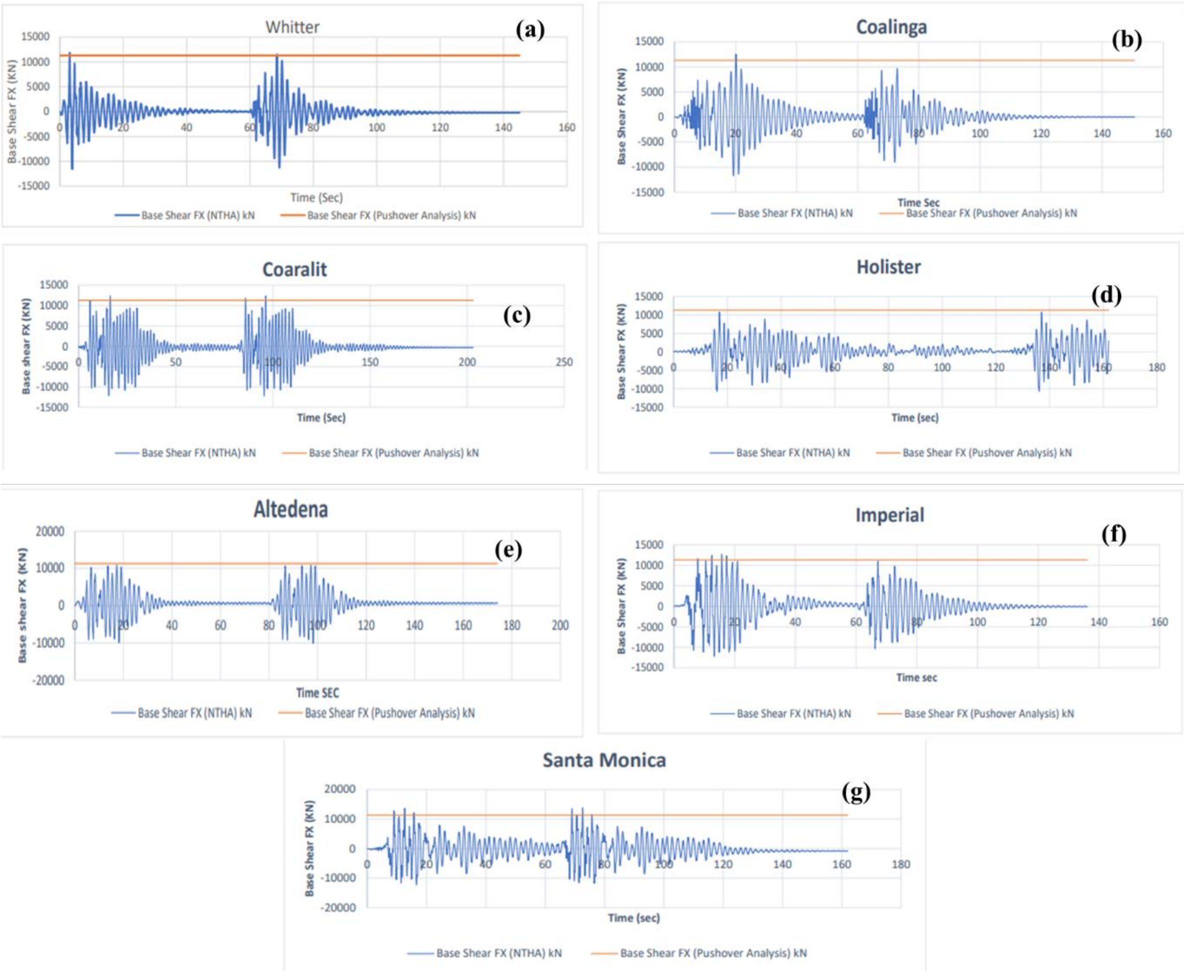

***Figure 8**: (a) Base shear sequence under Whittier Narrows ground motion record (b) Base shear sequence under Coalinga ground motion record (c) Base shear sequence under Coaralit ground motion record (d) Base shear sequence under Hollister ground motion record (e) Base shear sequence under Altadena ground motion record (f) Base shear sequence under Imperial ground motion record (g) Base shear sequence under Santa Monica ground motion record*

5. Conclusion

The reinforced concrete frame is subjected to a total of seven earthquake sequences in the current study, of which three are actual and four are artificial repeating sequences. In the current study, three types of analyses are performed to assess the vulnerability of structures. Eigenvalue analysis results for the fundamental period show that analysis is proceeding in the correct direction. Moreover, the mass participation ratios show that the first mode is governing in all structures. For reinforced concrete frame systems, the response of the structure is studied in terms of drifts, maximum displacements, and damage pattern creation. These measurements lead to the following results. In conclusion, the examination of the percentage difference in displacement between the main shock and the combined effect of the main shock and aftershock reveals significant findings. Notably, there is a pronounced variation in displacement across different story levels, with a particular emphasis on the first floor in both the main shock and main + aftershock scenarios. The Santa Monica Sequence stands out for displaying the maximum disparity in displacement, while the Whittier Narrows Sequence shows the least variation. Importantly, the observed trend suggests a diminishing percentage difference in displacement beyond the first floor, indicating a reduction in comparative displacement as one ascends through the building levels during both main shock and main + aftershock events. These insights offer valuable information for understanding and addressing structural vulnerabilities in seismic events.

Competing Interests: The authors declare that they have no competing interests.

Data Availability Statement: The supported data associated with this researcher is available upon request from the corresponding author.

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