

A State-Dependent Fragility Framework for Seismically Isolated Reinforced Concrete Bridges under Sequential Earthquake Loading

Dheeraj Kumar Kaurav¹

dheerajkaurav94@gmail.com

Maulana Azad National Institute of Technology

Juned Raheem

Maulana Azad National Institute of Technology

Neeraj Tiwari

Maulana Azad National Institute of Technology

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A State-Dependent Fragility Framework for Seismically Isolated Reinforced Concrete Bridges under Sequential Earthquake Loading

Dheeraj Kumar Kaurav¹, Juned Raheem², Neeraj Tiwari²

^{1,2} Department of Civil Engineering, Maulana Azad National Institute of Technology (MANIT), Bhopal, India

Corresponding Author: Dheeraj Kumar Kaurav (dheerajkaurav94@gmail.com)

Abstract

Sequential earthquake events significantly influence the seismic performance of reinforced concrete (RC) bridges through cumulative damage accumulation, residual deformation, and progressive stiffness degradation. Conventional seismic assessment methodologies generally assume an undamaged structural state for each earthquake event, thereby neglecting state-dependent damage evolution and overestimating the residual seismic capacity of bridge systems. This study presents a state-dependent performance-based framework for the nonlinear seismic assessment and fragility evaluation of seismically isolated RC bridges incorporating nonlinear soil–structure interaction (SSI). The proposed methodology preserves the committed nonlinear material state throughout successive earthquake events, enabling continuous simulation of concrete cracking, reinforcement yielding, lead-rubber bearing (LRB) hysteresis, foundation flexibility, and residual deformation without reinitializing the finite element modeling. A 3-D nonlinear finite element model of a four-span RC bridge was developed in OpenSeesPy using fibre-based force-beam-column elements, bilinear hysteretic LRB bearings, distributed nonlinear API p–y springs, and Rayleigh damping. Sequential nonlinear time-history analyses were performed using the 2001 Bhuj, 1940 El Centro, and 1989 Loma Prieta earthquake records, followed by Incremental Dynamic Analysis (IDA) and state-dependent lognormal fragility assessment. The computational framework integrates nonlinear pushover analysis, adaptive transient solution algorithms, residual-state transfer, and probabilistic fragility modelling within a unified numerical platform. The isolated bridge exhibited a fundamental period of approximately 3.0 s, a maximum pushover capacity of 889.9 kN, and an ultimate deck displacement of 0.50 m. Under sequential earthquake loading, peak pier drift remained below 0.04%, whereas peak LRB displacement progressively increased from 134.32 mm to 194.09 mm, reaching approximately 97% of its design displacement capacity. Residual isolator deformation increased to 22.14 mm, while maximum deck acceleration reached 0.432 g with a maximum base shear below 457 kN, indicating effective force reduction accompanied by progressive damage accumulation within the isolation system. IDA results indicated collapse initiation at $Sa(T_1)=1.81$ g and complete collapse near 2.26 g. State-dependent fragility analysis confirmed that the median spectral acceleration corresponding to whole pier damage decreased by 26.7% after three sequential earthquakes, while the median capacity associated with complete isolator damage decreased from 0.997 g to 0.813 g. The proposed framework provides a computationally efficient and physically consistent methodology for state-dependent seismic performance assessment, fragility evaluation, and resilience-based design of critical bridge infrastructure subjected to repeated earthquake loading.

Keywords: Sequential seismic hazard; Reinforced concrete bridge; Nonlinear finite element analysis; Soil–structure interaction; Lead-rubber bearing isolation; Damage accumulation; Incremental dynamic analysis; Seismic fragility; Residual deformation; OpenSeesPy.

1. Introduction

1.1 Background

Bridges constitute indispensable components of transportation networks, ensuring connectivity, emergency response, and economic continuity following natural disasters. Their uninterrupted functionality is particularly critical in seismically active regions, where damage to bridge infrastructure can severely disrupt rescue operations, supply chains, and post-disaster recovery. Consequently, the seismic performance assessment of reinforced concrete (RC) bridges has become a major research focus within the framework of Performance-Based Earthquake Engineering (PBEE)[1], [2] which seeks to quantify structural reliability, expected damage, and post-earthquake functionality under multiple hazard levels.[2]

The vulnerability of bridge systems to seismic loading has been repeatedly demonstrated by major earthquakes worldwide. The 1995 Kobe earthquake caused severe bridge damage because of weak column shear, sliding off supports, and large deck movements. Similar damage mechanisms were reported during the 1999 Chi-Chi earthquake (Taiwan), the 2008 Wenchuan earthquake (China), the 2010 Maule earthquake (Chile), the 2011

Tohoku earthquake (Japan), and the 2023 Kahramanmaraş earthquake (Türkiye), where numerous highway bridges experienced excessive pier deformation, bearing failures, foundation instability, approach settlement, and permanent residual displacement.[3] These reconnaissance observations confirmed that bridge performance is governed not only by peak seismic demand but also by the accumulation of irreversible damage and the residual structural condition following strong ground motions.

India is particularly vulnerable to seismic hazards because approximately 59% of its land area lies within Seismic Zones III, IV, and V as defined by IS 1893 (Part 1):2016. Several strategically important transportation corridors traverse the Himalayan region, Northeast India, the Indo-Gangetic plains, and western India, where moderate-to-large earthquakes have occurred repeatedly. The 2001 Mw 7.9 Bhuj earthquake remains one of the most destructive seismic events in India's recorded history and caused widespread damage to transportation infrastructure, including reinforced concrete and masonry bridges, highway embankments, and approach roads. Extensive liquefaction-induced ground deformation, settlement of bridge approaches, and damage to bridge substructures highlighted the importance of considering soil–foundation interaction in seismic bridge design.[4]

Post-earthquake reconnaissance following the Bhuj earthquake also revealed damage to reinforced concrete bridge components arising from foundation movements, bearing distress, and embankment failures, despite many bridge superstructures remaining operational. These observations demonstrated that bridge functionality can be significantly compromised even when complete structural collapse does not occur, emphasizing the importance of evaluating serviceability, residual deformation, and post-earthquake operability in addition to ultimate strength.

Modern bridge design has increasingly adopted seismic isolation systems, particularly lead-rubber bearings (LRBs), to mitigate earthquake-induced force demands by lengthening the structural vibration period and dissipating seismic energy through stable hysteretic behaviour. Numerous experimental and analytical investigations have confirmed that seismic isolation substantially reduces pier forces, deck accelerations, and damage concentrations under design-level earthquakes. Likewise, nonlinear soil–structure interaction (SSI)[5] has been recognized as a critical factor influencing bridge response because foundation flexibility modifies the effective stiffness, damping characteristics, and dynamic properties of the entire bridge system. Consequently, realistic seismic assessment requires simultaneous consideration of structural nonlinearity, foundation flexibility, and isolation system behaviour.[6]

Despite these developments, most current seismic assessment methodologies continue to analyse earthquake records independently by assuming that the bridge is initially undamaged before each event. Such an assumption neglects the cumulative evolution of concrete cracking, reinforcement yielding, isolator hysteresis, stiffness degradation, residual deformation, and foundation nonlinearity generated during preceding earthquakes. However, observations from recent earthquake sequences—including the Canterbury (2010–2011), Central Italy (2016–2017), and Kahramanmaraş (2023) events—have demonstrated that bridges often experience multiple strong ground motions before repair or rehabilitation can be undertaken. Under these circumstances, subsequent earthquakes act on an already damaged structural system with altered stiffness, damping, and load-resisting capacity, resulting in significantly different seismic responses compared with those predicted by conventional independent-event analyses.[7]

Although considerable research has been devoted to nonlinear bridge analysis, seismic isolation, and SSI, the coupled effects of sequential earthquake loading, nonlinear foundation flexibility, cumulative damage accumulation, and state-dependent fragility evolution remain inadequately investigated. Existing PBEE frameworks generally neglect the transfer of committed material states between successive earthquake events, thereby potentially overestimating residual seismic capacity and underestimating collapse probability.[8] These limitations are particularly significant for isolated bridges, where repeated cyclic loading can progressively degrade the deformation capacity of isolation bearings while maintaining relatively low levels of pier damage.

Motivated by these research gaps, the present study develops an integrated nonlinear finite element framework for the seismic assessment of a seismically isolated RC bridge subjected to sequential earthquake loading. The proposed methodology combines fibre-based nonlinear modelling, nonlinear pile–soil interaction, lead-rubber bearing hysteresis, adaptive nonlinear time-history (NTHA) analysis, incremental dynamic analysis (IDA), and state-dependent fragility assessment within a unified OpenSeesPy platform.[9] By preserving the committed material state throughout successive earthquake excitations, the framework enables realistic simulation of cumulative damage evolution, residual deformation, degradation of seismic capacity, and progressive fragility,

thereby providing a physically consistent basis for resilience assessment and performance-based design of critical bridge infrastructure.

1.2 Literature Review

The seismic behaviour of reinforced concrete (RC) bridges has been extensively investigated over the past three decades, resulting in significant advances in nonlinear modelling, seismic isolation, soil–structure interaction (SSI), and performance-based earthquake engineering (PBEE). Early studies primarily focused on bridge response under single earthquake events, emphasizing the influence of material nonlinearity, ductility, and energy dissipation on structural performance. With the development of fibre-based finite element modelling and nonlinear dynamic analysis, researchers demonstrated that bridge response is governed by the interaction between superstructure flexibility, pier behaviour, bearing characteristics, and foundation compliance. Consequently, nonlinear time-history analysis has become the preferred analytical approach for evaluating bridge performance under strong ground motions.

Numerous studies have evaluated the performance of seismic isolation systems in mitigating earthquake-induced damage. Lead-rubber bearings (LRBs), friction pendulum systems, and high-damping rubber bearings have been shown to substantially increase the fundamental vibration period, reduce force transmission to bridge piers, and improve energy dissipation. Experimental investigations and numerical simulations consistently indicate that seismic isolation effectively decreases pier curvature demand, deck acceleration, and base shear while enhancing structural resilience.[10] Nevertheless, most available studies evaluate isolated bridges under a single design-level earthquake and generally assume that the mechanical properties of isolation bearings remain unchanged throughout the analysis. The progressive degradation of isolation systems under repeated earthquake loading has received comparatively limited attention.

Soil–structure interaction (SSI) is another critical factor influencing bridge seismic performance. Foundation flexibility modifies the dynamic characteristics of bridge systems by increasing the effective vibration period and introducing additional energy dissipation through soil deformation. Analytical formulations proposed by Gazetas, Novak, and subsequent researchers, together with nonlinear Winkler foundation models based on API p–y curves, have enabled realistic representation of pile-supported bridge foundations. Recent studies have demonstrated that SSI can significantly alter displacement demand, pier force distribution, and bearing deformation, particularly for bridges founded on soft or medium-density soils.[5], [6] However, many bridge fragility investigations continue to neglect nonlinear SSI or adopt simplified spring models that do not adequately represent foundation degradation during severe seismic loading.

The increasing adoption of Performance-Based Earthquake Engineering (PBEE) has shifted bridge assessment from deterministic safety verification towards probabilistic evaluation of damage, functionality, and collapse risk. Within the PBEE framework, nonlinear time-history analysis is combined with Incremental Dynamic Analysis (IDA) to establish relationships between seismic intensity measures and engineering demand parameters. Fragility functions are subsequently developed to estimate the probability of exceeding predefined damage states. These methodologies have become standard tools for seismic risk assessment of highway bridges and have been incorporated into numerous resilience and life-cycle assessment studies.[11] Nevertheless, the majority of available fragility models assume that each earthquake event acts on an initially undamaged bridge, thereby neglecting the influence of accumulated damage and residual structural deformation on subsequent seismic response.

Following recent earthquake sequences, researchers have increasingly recognised that the assumption of independent earthquake events is inconsistent with observed field performance. Reconnaissance investigations after the Canterbury (2010–2011), Central Italy (2016–2017), Ridgecrest (2019), and Kahramanmaraş (2023) earthquake sequences reported progressive deterioration of bridge components due to repeated seismic excitation before repair could be undertaken. Consequently, several numerical studies have introduced sequential earthquake analysis methodologies in which multiple ground motions are applied consecutively to the same finite element model. These investigations consistently demonstrated that cumulative damage increases residual displacement, reduces stiffness, accelerates reinforcement yielding, and lowers the collapse capacity of bridge structures compared with conventional single-event analyses.[10]

Although significant progress has been achieved in sequential earthquake analysis, several important limitations remain unresolved. First, many studies represent structural damage using simplified stiffness reduction factors

rather than explicitly preserving the nonlinear constitutive history of concrete, reinforcing steel, and isolation bearings. Second, nonlinear soil–structure interaction is frequently neglected or idealised using linear foundation springs, thereby underestimating the influence of foundation flexibility on cumulative structural response. Third, existing investigations generally focus either on conventional fixed-base bridges or isolated bridges without considering the combined interaction between nonlinear SSI and seismic isolation under repeated earthquake loading. Moreover, limited research has investigated the evolution of fragility parameters throughout an earthquake sequence using state-dependent probabilistic frameworks.

Another important limitation is that many available computational frameworks rebuild the finite element model after each earthquake or manually redefine the initial structural state before applying subsequent ground motions. Such procedures interrupt the natural evolution of material damage and residual deformation, resulting in unrealistic predictions of structural performance during aftershock sequences. Consequently, the degradation of seismic capacity and the progressive increase in collapse probability may not be accurately captured. This limitation becomes particularly significant for isolated bridge systems, where cumulative hysteretic deformation of lead-rubber bearings may govern post-mainshock performance even when damage to reinforced concrete piers remains relatively small.

Based on the above review, it is evident that an integrated numerical framework capable of simultaneously considering nonlinear material behaviour, nonlinear soil–structure interaction, seismic isolation, sequential earthquake loading, residual-state transfer, incremental dynamic analysis, and state-dependent fragility assessment is still lacking. The present study addresses these limitations by developing a unified OpenSeesPy-based computational framework in which the committed material state is preserved throughout the entire earthquake sequence.[12], [13] The proposed methodology enables realistic simulation of cumulative damage evolution, residual deformation, degradation of collapse capacity, and progressive fragility of seismically isolated RC bridges, thereby providing a more reliable basis for performance-based seismic assessment and resilience evaluation than conventional independent-event approaches.

1.3 Research Gap, Objectives and Novel Contributions

Despite significant advancements in nonlinear seismic analysis, seismic isolation, soil–structure interaction (SSI), and performance-based earthquake engineering (PBEE), several critical challenges remain unresolved in the seismic assessment of reinforced concrete (RC) bridges subjected to repeated earthquake events. Most existing studies evaluate bridge performance under a single earthquake or treat successive ground motions as independent events by reconstructing the numerical model before each analysis. Such an approach neglects the irreversible evolution of concrete cracking, reinforcement yielding, stiffness degradation, hysteretic energy dissipation, and residual deformation accumulated during previous earthquakes. Consequently, the residual seismic capacity and collapse vulnerability of bridges are often overestimated, leading to unconservative predictions of post-earthquake performance.[14], [15], [16]

Previous investigations on sequential earthquake effects have primarily focused on fixed-base bridge systems, whereas the response of seismically isolated bridges under repeated earthquake loading remains comparatively underexplored. Although lead-rubber bearings (LRBs) have been widely demonstrated to reduce seismic force demand and enhance structural resilience, their cumulative hysteretic degradation and interaction with nonlinear bridge components during earthquake sequences have not been comprehensively quantified. Furthermore, many available numerical models either neglect soil–structure interaction (SSI) or represent foundation flexibility using simplified linear springs, thereby failing to capture the nonlinear degradation of pile-supported foundations under strong cyclic loading.

Another important limitation concerns the probabilistic evaluation of bridge performance. Conventional fragility analyses generally assume that every earthquake acts on an undamaged bridge and therefore ignore the influence of residual damage on subsequent structural reliability. As a result, the progressive evolution of fragility and collapse capacity during earthquake sequences is inadequately represented within existing performance-based assessment frameworks. Moreover, relatively few studies integrate nonlinear finite element modelling, adaptive nonlinear transient analysis, incremental dynamic analysis (IDA), and state-dependent fragility assessment into a single computational framework capable of preserving the committed material state throughout successive earthquake excitations.[15], [17]

To overcome these limitations, the present study develops an integrated nonlinear finite element framework for the state-dependent seismic performance assessment of a seismically isolated RC bridge subjected to sequential earthquake loading. The methodology is implemented entirely within the OpenSeesPy environment and combines fibre-based nonlinear modelling, nonlinear pile–soil interaction, hysteretic lead-rubber bearing behaviour, adaptive nonlinear transient analysis, incremental dynamic analysis, and probabilistic fragility assessment within a unified computational platform. Unlike conventional approaches, the proposed framework preserves the complete nonlinear material history between successive earthquake events, enabling realistic simulation of cumulative damage accumulation, stiffness degradation, residual deformation, and deterioration of seismic capacity throughout the earthquake sequence.[13], [15]

Accordingly, The specific objectives of this study are as follows:

1. To develop a 3-D nonlinear finite element model of a seismically isolated RC bridge incorporating fibre-based RC piers, nonlinear lead-rubber bearings, and nonlinear pile–soil interaction using distributed API p–y springs.
2. To establish a sequential earthquake analysis methodology in which the committed nonlinear material state is preserved between consecutive earthquake excitations without rebuilding the structural model.
3. To evaluate the influence of cumulative earthquake loading on bridge dynamic characteristics, pier drift, base shear, isolator deformation, residual displacement, and other engineering demand parameters.
4. To investigate the degradation of seismic capacity through nonlinear pushover analysis and Incremental Dynamic Analysis (IDA) under sequential earthquake loading.
5. To develop state-dependent seismic fragility functions that explicitly account for cumulative structural damage and residual deformation during repeated earthquake events.
6. To demonstrate the effectiveness of the proposed framework in supporting performance-based seismic assessment and resilience evaluation of critical bridge infrastructure.

The principal novelties of the present study can be summarized as follows:

- Development of a unified OpenSeesPy framework integrating nonlinear structural modelling, nonlinear SSI, seismic isolation, sequential nonlinear time-history analysis, IDA, and probabilistic fragility assessment within a single computational environment.
- Implementation of a state-preserving sequential earthquake algorithm that maintains the committed constitutive history of concrete, reinforcing steel, isolation bearings, and soil springs throughout the complete earthquake sequence without numerical model reconstruction.
- Explicit quantification of cumulative damage evolution through continuous tracking of stiffness degradation, residual deformation, engineering demand parameters, and collapse capacity under successive earthquake excitations.
- Development of state-dependent fragility functions that incorporate residual damage accumulated during previous earthquakes, providing a more realistic estimate of post-mainshock bridge vulnerability than conventional independent-event fragility models.
- Comprehensive assessment of the interaction between nonlinear SSI and seismic isolation under repeated seismic loading, thereby improving the physical realism of bridge performance evaluation.

The proposed framework provides a physically consistent methodology for assessing the seismic resilience of isolated bridge systems subjected to earthquake sequences. The developed computational approach can be readily extended to other bridge configurations and incorporated into performance-based design, post-earthquake safety assessment, retrofit prioritization, and life-cycle seismic risk evaluation of transportation infrastructure.

1.4 Organization of the Paper

The remainder of this paper is organized as follows. Section 2 describes the bridge configuration, material properties, nonlinear finite element formulation, seismic isolation system, and soil–structure interaction model adopted in the numerical analysis. Section 3 presents the analytical methodology, including modal analysis, pushover analysis, sequential NTHA, IDA, and state-dependent fragility assessment. Section 4 discusses the numerical results, including bridge dynamic characteristics, seismic response, cumulative damage evolution, residual deformation, collapse capacity, and fragility relationships. Finally, Section 5 summarizes the principal findings, engineering implications, limitations of the present study, and recommendations for future research.

2. Numerical Modelling

2.1 Prototype Bridge Description

Seismically-Isolated 4-Span RCC Bridge — 3-D FE Frame Model

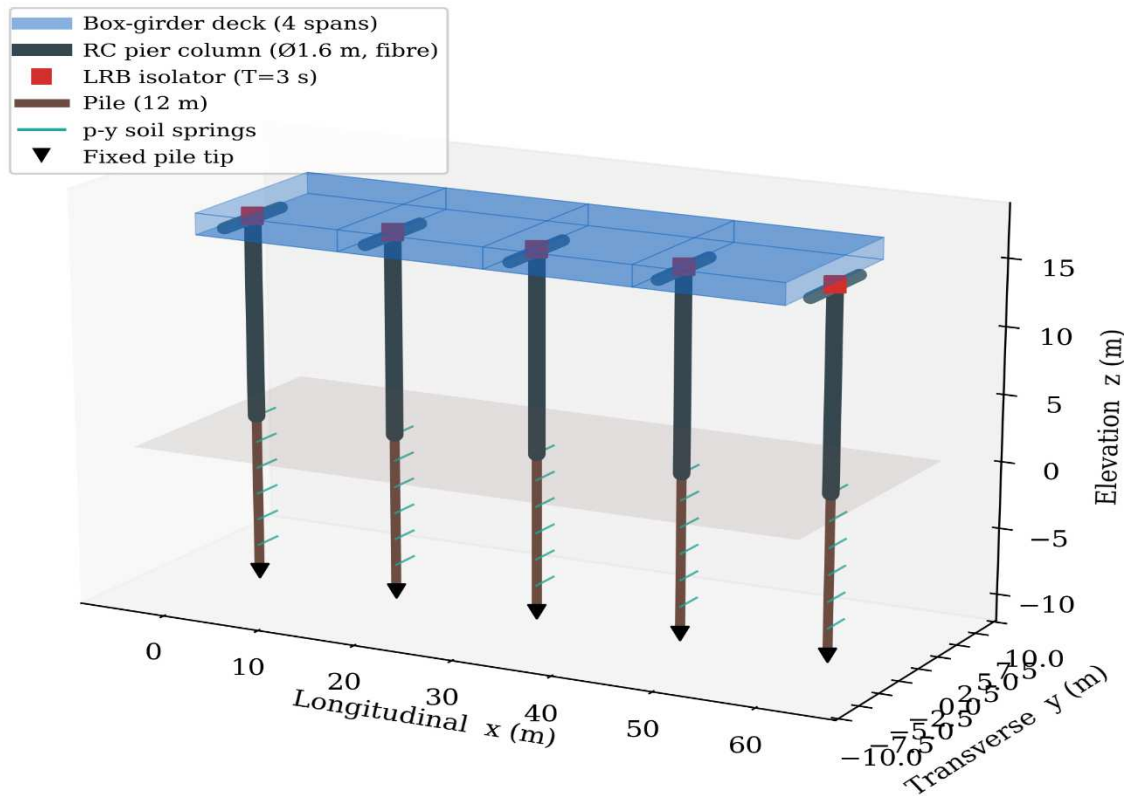


Fig. 1 Finite Element Model For 4 Span Seismically Isolated RCC Bridge

A three-dimensional nonlinear finite element model of a reinforced concrete (RC) highway bridge was developed in OpenSeesPy to evaluate its seismic performance under sequential earthquake loading. The prototype bridge represents a typical medium-span bridge equipped with seismic isolation and pile-supported foundations, enabling the investigation of cumulative damage evolution considering both nonlinear structural behaviour and soil–structure interaction (SSI).[12], [15] The overall finite element idealization of the bridge, comprising the deck, piers, isolation bearings, and pile foundation, is illustrated in Figure 1.

The bridge comprises four equal spans with a total length of 58.8 m. The superstructure is supported by three circular reinforced concrete piers having a diameter of 1.60 m and a height of 15.0 m. The bridge deck is modelled using elastic beam-column elements, whereas the bridge piers are represented by nonlinear fibre-based force-beam-column elements capable of simulating distributed plasticity, concrete cracking, reinforcement yielding, and stiffness degradation under cyclic loading.

Lead-rubber bearings (LRBs) are employed as seismic isolation devices between the superstructure and substructure to reduce earthquake-induced force transmission by lengthening the effective fundamental period to approximately 3 s. Their nonlinear hysteretic response is explicitly modelled to capture lead-core yielding, post-yield stiffness characteristics, and energy dissipation during sequential earthquake excitations, thereby enabling a realistic representation of isolation system behaviour under repeated seismic loading.[18]

The bridge foundation is represented by pile-supported foundations connected to distributed nonlinear API p–y soil springs. Foundation flexibility and pile–soil interaction are incorporated using the Focht–Koch pile group reduction approach, allowing realistic simulation of nonlinear SSI during strong ground motion.[15]

The complete finite element model consists of elastic beam-column elements for the deck, fibre-based nonlinear beam-column elements for the piers, nonlinear zero-length elements for the LRB isolation system, and distributed nonlinear soil springs representing the surrounding foundation soil. Appropriate boundary conditions, nodal masses, and rigid links are incorporated to ensure realistic force transfer and dynamic characteristics.

Unlike conventional seismic analyses, the developed model preserves the committed material state throughout the earthquake sequence without reconstructing the numerical model. Consequently, concrete damage, reinforcement yielding, isolator hysteresis, soil spring degradation, and residual deformation accumulated during one earthquake are directly transferred to the subsequent excitation, enabling realistic simulation of cumulative damage evolution and state-dependent seismic performance.

The principal geometric characteristics of the prototype bridge are summarized in Table 1.

Table 1. Geometric characteristics of the prototype bridge

Parameter	Value
Bridge type	RC highway bridge
Total bridge length	58.8 m
Number of spans	4
Span length	14.7 m
Number of piers	3
Pier diameter	1.60 m
Pier height	15.0 m
Superstructure	Elastic beam-column elements
Pier model	Fibre-based nonlinear beam-column
Isolation system	Lead-rubber bearing (LRB)
Target isolation period	3.0 s
Foundation	Pile foundation
Soil model	Nonlinear API p–y springs
Analysis platform	OpenSeesPy
Analysis type	Sequential nonlinear time-history analysis

2.2 Material Constitutive Models

The nonlinear seismic response of the bridge was simulated using constitutive models capable of representing the inelastic behaviour of reinforced concrete, reinforcing steel, seismic isolation bearings, and foundation soil. The selected material models have been extensively validated for nonlinear seismic analysis of bridge structures and are implemented within the OpenSeesPy framework.[19] The material properties and modelling parameters adopted for the bridge components are summarized in Table 2.

Table 2. Material properties and modelling parameters of the bridge components

Bridge Component	Material Model	Property	Value	Unit
RC Pier (Core Concrete)	Concrete01	Compressive strength (f_c)	30	MPa
		Strain at peak stress, (ϵ_{c0})	0.002	—
		Crushing strength, (f_{cu})	6	MPa
		Ultimate crushing strain, (ϵ_{cu})	0.010	—
RC Pier (Cover Concrete)	Concrete01	Compressive strength	30	MPa
		Spalling strain	0.006	—
Reinforcement	Steel01	Yield strength, (f_y)	415	MPa
		Elastic modulus, (E_s)	200	GPa
		Strain-hardening ratio(b)	0.01	—
		Number of longitudinal bars	16	—
		Area of each reinforcing bar	490	mm ²
		Concrete cover	50	mm
Bridge Deck	Elastic Beam	Cross-sectional area	10.0	m ²
		Young's modulus	40	GPa
		Shear modulus	20	GPa
		Torsional constant, (J)	100	m ⁴
		Moment of inertia, (I_y)	100	m ⁴
		Moment of inertia, (I_z)	100	m ⁴
Lead–Rubber Bearing (LRB)	Bilinear Steel01	Target isolation period	3.0	s
		Design displacement	200	mm
		Yield force ratio	5% of tributary weight	—
		Effective stiffness	Computed automatically	kN/m
		Post-yield stiffness	Computed automatically	kN/m
Pile Foundation	Fibre Beam-Column	Pile diameter	0.60	m
		Pile depth	12.0	m
		Vertical spacing of p–y springs	2.0	m

Soil Springs	API p–y	Undrained shear strength, (cu)	80		kPa
		Reference strain, (ε_{50})	0.010		–
		Focht–Koch group efficiency factor	0.675		–
		Ultimate soil resistance	Computed from API formulation		kN
		Initial spring stiffness	Computed from API formulation		kN/m
Structural Damping	Rayleigh Damping	Critical damping ratio	5		%
		Modes used	First two vibration modes		–

2.2.1 Concrete

The nonlinear behaviour of reinforced concrete piers was represented using the Concrete01 constitutive model, which follows the monotonic Kent–Scott–Park stress–strain relationship with linear unloading stiffness. Separate material properties were assigned to confined and unconfined concrete to account for the enhanced compressive strength and ductility provided by transverse reinforcement within the pier core.

The compressive stress–strain relationship is expressed as

$$\sigma_c = f_c \left[2 \left(\frac{\varepsilon_c}{\varepsilon_0} \right) - \left(\frac{\varepsilon_c}{\varepsilon_0} \right)^2 \right], \quad 0 \leq \varepsilon_c \leq \varepsilon_0 \quad (1)$$

where (f_c) is the peak compressive strength, ε_0 is the corresponding peak compressive strain, and ε_c is the concrete compressive strain. Beyond the peak stress, a descending branch is adopted until the ultimate crushing strain is reached.

2.2.2 Reinforcing Steel

Longitudinal reinforcement was modelled using the Steel01 constitutive model based on bilinear elastoplastic behaviour with isotropic strain hardening. The model accurately captures yielding and post-yield stiffness under cyclic loading.

The constitutive relationship is defined as

$$\sigma_s = \begin{cases} E_s \varepsilon_s, & |\varepsilon_s| \leq \varepsilon_y \\ f_y + b E_s (\varepsilon_s - \varepsilon_y), & |\varepsilon_s| > \varepsilon_y \end{cases} \quad (2)$$

where (E_s) is the elastic modulus, (f_y) is the yield strength, and (b) is the strain-hardening ratio.

2.2.3 Lead–Rubber Bearings

The bridge is equipped with lead-rubber bearings (LRBs) to reduce seismic force transmission from the superstructure to the substructure. The bearings were represented using a bilinear hysteretic model characterised by an initial elastic stiffness (K_1), post-yield stiffness (K_2), and characteristic yield force (Q_d). The restoring force is expressed as

$$F_b = Q_d + K_2 u_b \quad (3)$$

where (u_b) is the bearing displacement. The hysteretic behaviour of the lead core provides stable energy dissipation during cyclic loading while maintaining sufficient post-yield stiffness to ensure recentering of the superstructure.

2.2.4 Soil–Structure Interaction

Nonlinear soil–structure interaction was incorporated using distributed API p–y springs along the embedded pile length. Each spring represents the nonlinear lateral resistance of the surrounding soil and follows the API-recommended p–y formulation,

$$p = A, p_u, \tanh\left(\frac{kx}{A p_u} y\right) \quad (4)$$

where (p) is the lateral soil resistance per unit length, (p_u) is the ultimate soil resistance, (k) is the initial modulus of subgrade reaction, (y) is the lateral pile displacement, and (A) is an empirical loading coefficient. Pile group interaction was incorporated through the Focht–Koch reduction factor to account for overlapping stress zones between adjacent piles.[15]

The adopted constitutive models enable realistic simulation of nonlinear material behaviour, hysteretic energy dissipation, and foundation flexibility, thereby providing a reliable basis for evaluating cumulative damage accumulation and seismic performance under sequential earthquake loading. The longitudinal elevation of the finite element model, showing the arrangement of the piers, deck, and isolation bearings, is presented in Figure 2.

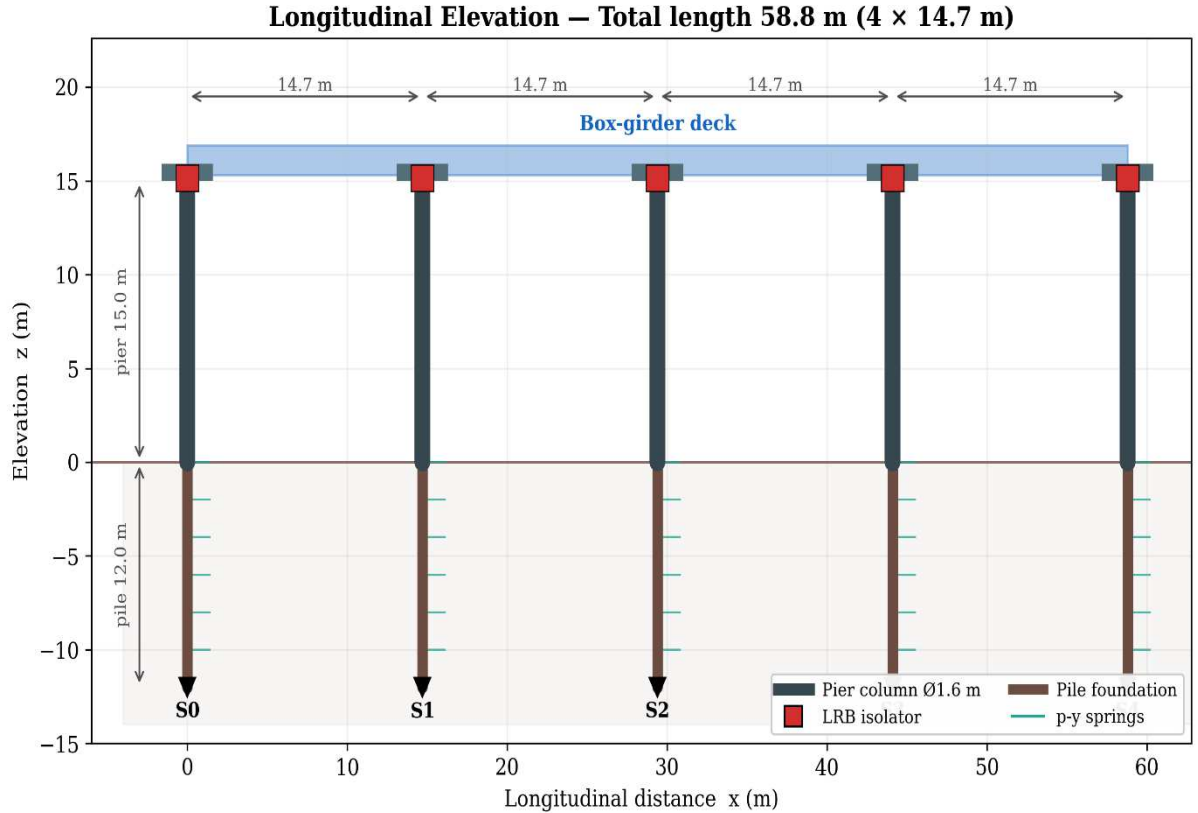


Fig 2 Finite Element Model (Longitudinal Elevation)

2.3 Fibre-Based RC Pier Modelling

The reinforced concrete (RC) bridge piers were modelled in OpenSeesPy using the force-based fibre beam-column element (forceBeamColumn), which captures distributed plasticity by integrating the nonlinear sectional response along the member length using five-point Gauss–Lobatto integration. This formulation enables the gradual spread of yielding and damage without assuming predefined plastic hinge locations.[12], [13]

Each circular pier cross-section was represented by a fibre section (section Fiber), where the confined concrete core, unconfined concrete cover, and longitudinal reinforcement were discretized into individual fibres. The confined core and cover concrete were modelled using concentric circular patches (patch circ), while sixteen 25-mm-diameter reinforcing bars were defined as circular reinforcement layers (layer circ). The confined concrete was assigned a compressive strength of 30 MPa, peak strain of 0.002, crushing strength of 6 MPa, and ultimate strain of 0.010, whereas the cover concrete was assigned a spalling strain of 0.006. The reinforcing steel was modelled using the Steel01 constitutive model with a yield strength of 415 MPa, elastic modulus of 200 GPa, and strain-hardening ratio of 0.01.

The sectional resisting forces were computed by integrating the stress contribution of each fibre over the cross-section,

$$\text{Axial Force} \quad N = \sum_{i=1}^n \sigma_i A_i, \quad (5)$$

$$\text{Moment} \quad M = \sum_{i=1}^n \sigma_i A_i y_i, \quad (6)$$

where (A_i) , (σ_i) , and (y_i) denote the area, stress, and centroidal distance of the i_{th} fibre, respectively. This fibre-based modelling approach accurately captures concrete cracking, concrete crushing, reinforcement yielding, stiffness degradation, and residual deformation, providing a reliable representation of cumulative damage in the bridge piers during sequential earthquake loading.

2.4 Lead-Rubber Bearing (LRB) Model

Lead-rubber bearings (LRBs) were employed between the bridge deck and pier caps to reduce seismic demand by increasing the fundamental period and dissipating energy through hysteretic deformation. In the OpenSeesPy model, each isolator was represented using a zeroLength element with independent horizontal and vertical material behaviour. The horizontal response was modelled using the bilinear Steel01 constitutive model, while the vertical response was assumed to remain elastic using a high-stiffness material. The isolator properties were automatically determined from the bridge seismic mass to achieve a target isolation period of 3.0 s and a design displacement capacity of 200 mm. The effective stiffness was calculated as

$$K_{eff} = m \left(\frac{2\pi}{T_{iso}} \right)^2 \quad (7)$$

where (m) is the tributary seismic mass supported by each bearing and T_{iso} is the target isolation period.

$$Q_d = 0.05W \quad (8)$$

The characteristic yield force was taken as 5% of the tributary weight, while the post-yield stiffness was determined to satisfy the required effective stiffness. A minimum post-yield stiffness equal to 5% of the effective stiffness was enforced to maintain numerical stability during nonlinear analysis.

$$K_2 = K_{eff} - \frac{Q_d}{D_d} \quad (9)$$

The bilinear hysteretic model accurately represents the pre-yield stiffness, yielding behaviour, post-yield hardening, and energy dissipation characteristics of LRBs. The permanent deformation accumulated in the isolators after each earthquake was preserved and transferred to subsequent earthquake phases, enabling realistic simulation of cumulative damage during sequential earthquake loading.

2.5 Soil–Structure Interaction (SSI) Model

Soil–structure interaction was incorporated using the Winkler foundation approach, in which the surrounding soil was represented by discrete lateral p–y springs connected to the pile foundation. The nonlinear soil springs were developed according to the API RP 2GEO recommendations for medium-stiff clay and implemented as zero-length elements attached to the pile nodes. The pile foundation was discretized using force-based beam-column elements, while the soil springs were distributed at 2.0 m intervals along the embedded pile depth of 12.0 m to capture the variation of lateral soil resistance with depth. The ultimate lateral soil resistance for each spring was evaluated as

$$p_u = 9c_u D \quad (10)$$

where (c_u) is the undrained shear strength of the soil and (D) is the pile diameter. The initial spring stiffness was obtained from the API p–y relationship using the characteristic displacement (y_{50}),

$$k = \frac{p_u}{y_{50}}$$

where

$$y_{50} = 2.5\varepsilon_{50}D$$

To account for pile-group interaction, the spring stiffness was reduced using the Focht–Koch group efficiency factor of 0.675, corresponding to a 2×2 pile group with a spacing-to-diameter ratio of 4.0. The adopted soil properties included an undrained shear strength of 80 kPa, pile diameter of 0.6 m, and ($\varepsilon_{50} = 0.01$). The distributed p–y spring model provides realistic simulation of foundation flexibility, allowing soil deformation, pile–soil interaction, and energy dissipation to influence the dynamic response of the isolated bridge during sequential earthquake loading.[15]

2.6 Boundary Conditions and Meshing

The boundary conditions and modelling details adopted for each structural component of the finite element model are summarized in Table 3.

Table 3. Boundary conditions for Finite element bridge model

Component	Finite Element / Constraint	Boundary Condition / Meshing Details	OpenSeesPy Implementation
Superstructure deck	Elastic beam-column element	Four deck spans connected by elastic beam-column elements	elasticBeamColumn
Reinforced concrete piers	Nonlinear fibre beam-column element	One force-based beam-column element per pier with Lobatto integration (5 integration points)	forceBeamColumn
Pier section	Fibre section	Circular fibre section comprising confined concrete, cover concrete and reinforcing steel fibres	section Fiber
Beam integration	Lobatto integration	Five Gauss–Lobatto integration points along each pier element	beamIntegration('Lobatto',5)
Lead–Rubber Bearings (LRBs)	Zero-length element	Horizontal nonlinear bilinear hysteresis and vertical elastic stiffness	zeroLength
Deck–pier connection	Multi-point constraint	Translational Y and all rotational degrees of freedom constrained between pier and deck nodes; longitudinal movement allowed through LRB	equalDOF (2,4,5,6)
Pile foundation	Nonlinear beam-column element	Pile discretized at 2.0 m intervals over 12 m embedment depth	forceBeamColumn
Soil–pile interaction	Distributed zero-length springs	Nonlinear API p–y springs attached at every pile node except pile tip	zeroLength
Soil spring spacing	Distributed springs	2.0 m vertical spacing	Automatically generated

Ground nodes	Fixed support	Soil spring reference nodes fully restrained in all six degrees of freedom	fix(1,1,1,1,1,1)
Pile tip	Fixed support	Bottom pile node fully restrained	fix(1,1,1,1,1,1)
Deck mass	Lumped mass	Translational seismic mass assigned at each deck node	mass()
Earthquake loading	Uniform base excitation	Ground motion applied in global longitudinal (X) direction	UniformExcitation
Gravity loading	Static nodal loading	Self-weight applied at deck nodes before dynamic analysis	Plain Pattern
Dynamic integration	Implicit Newmark method	Average acceleration scheme ($\gamma=0.5$, $\beta=0.25$)	Newmark(0.5,0.25)
Damping	Rayleigh damping	5% critical damping based on first two vibration modes	Rayleigh ()
Geometric transformation	Corotational formulation	Large-displacement geometric nonlinearity for deck and pier members	geomTransf ('Corotational')

The bridge was modelled as a three-dimensional finite element system in OpenSeesPy using force-based beam-column, elastic beam-column, and zero-length elements. The pile foundations were discretized into beam-column elements with 2.0 m spacing over a 12.0 m embedment depth, while the RC piers were represented using force-based fibre beam-column elements. The bridge deck was modelled as elastic, and lead-rubber bearings (LRBs) were incorporated as zero-length elements between the deck and pier caps to simulate their nonlinear hysteretic behaviour.[20]

The pile-tip nodes were assumed to be fully fixed in all six degrees of freedom, whereas lateral soil resistance was provided through distributed zero-length p-y springs connected between the pile nodes and restrained ground nodes. The bridge deck and pier tops were connected through lead-rubber bearings, with the transverse translation and rotational degrees of freedom constrained using the equal DOF command to ensure compatible deformation while allowing horizontal isolation in the longitudinal direction. Corotational geometric transformations were adopted for both the bridge piers and deck elements to account for geometric nonlinearity during large seismic displacements. The node and element connectivity of the developed finite element model is illustrated in Figure 3.

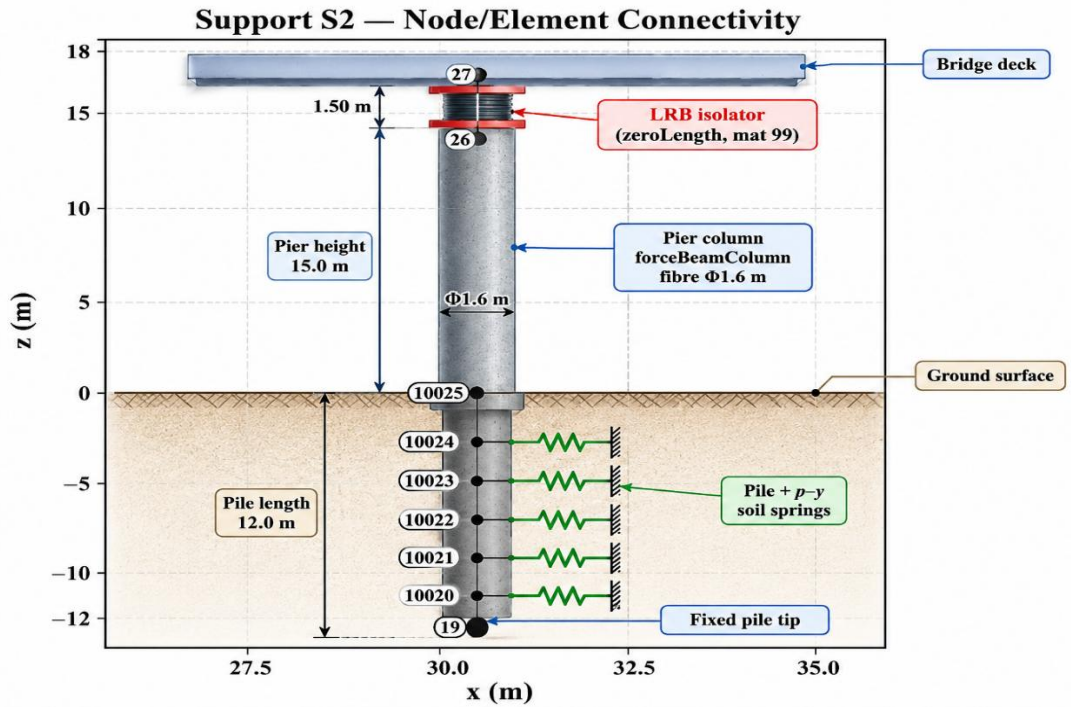


Fig 3 Node / Element Connectivity (Finite Element Model)

The adopted mesh discretization provides an efficient balance between computational accuracy and numerical stability, enabling realistic simulation of nonlinear pier behaviour, soil–foundation interaction, and seismic isolation under sequential earthquake loading. Table 4 summarizes the finite element discretization adopted for each structural component of the numerical bridge model.

Table 4. Finite element discretization of the numerical bridge model

Structural Component	Mesh Description
Bridge spans	4 elastic beam-column elements
Bridge piers	5 nonlinear fibre beam-column elements (one per bent)
Pile foundation	6 beam-column elements per pile (12 m depth with 2 m spacing)
Soil springs	One nonlinear p–y spring at each pile node except the pile tip (30 springs in total)
Lead–Rubber Bearings	5 zero-length bearing elements
Deck nodes	5 deck nodes carrying lumped seismic mass
Pier nodes	One pier-top node and one ground node per bent
Foundation nodes	Fixed pile-tip node plus fixed soil spring nodes along each pile

2.7 Ground Motion Selection, Dynamic Formulation and Scaling

The acceleration time histories of the input ground motions used for the sequential nonlinear dynamic analysis are presented in Figure 4.

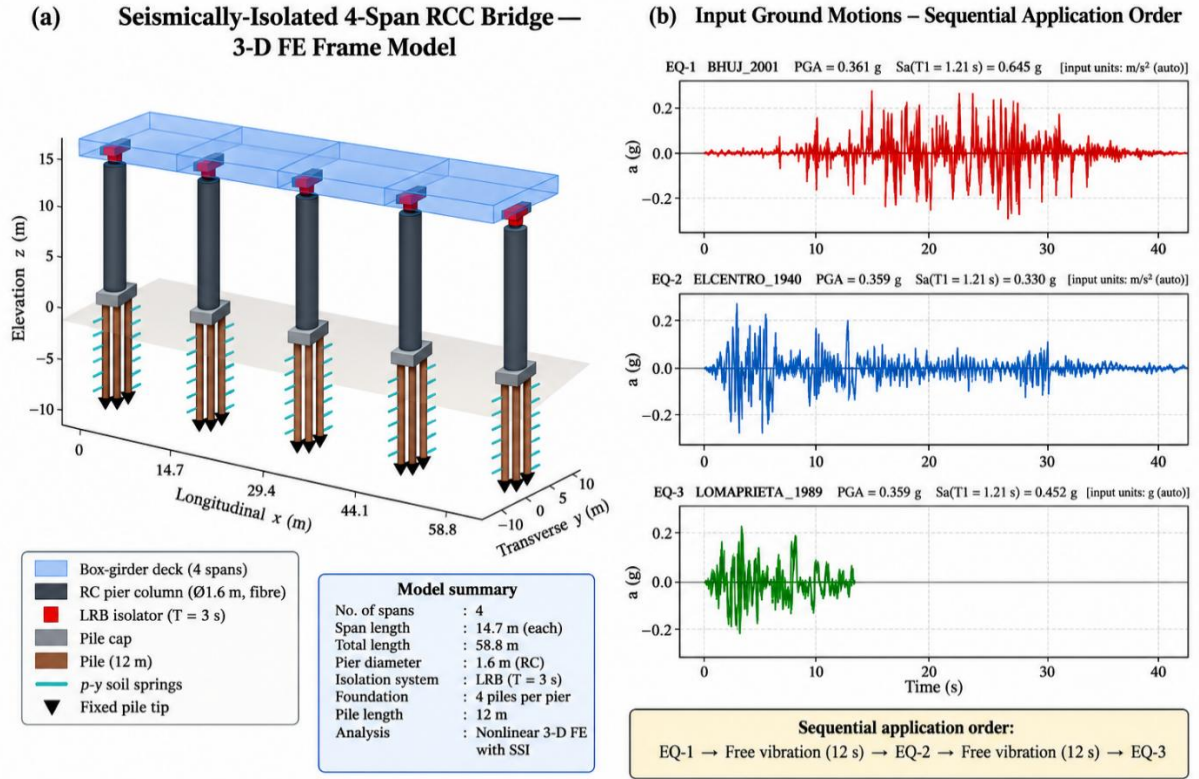


Fig 4 Input Ground Motion for Dynamic Analysis of Bridge System

The nonlinear dynamic response of the seismically isolated reinforced concrete bridge subjected to earthquake excitation was governed by the equation of motion for a nonlinear multi-degree-of-freedom (MDOF) structural system,

$$M\ddot{u}(t) + C\dot{u}(t) + f_r(u(t)) = -M\Gamma\ddot{u}_g(t) \quad (11)$$

where M is the global mass matrix, C is the damping matrix, f_r is the nonlinear restoring force vector obtained from the fibre-based RC piers, lead-rubber bearings, and nonlinear soil springs, Γ is the influence vector, $\ddot{u}_g(t)$ is the prescribed ground acceleration, and $u(t)$, $\dot{u}(t)$, and $\ddot{u}(t)$ denote the displacement, velocity, and acceleration vectors, respectively. The nonlinear restoring force was continuously updated throughout the analysis to account for concrete cracking, reinforcement yielding, lead-rubber bearing hysteresis, and nonlinear soil–structure interaction.[21]

The governing equation was solved using the implicit Newmark average acceleration method with parameters ($\gamma = 0.5$) and ($\beta = 0.25$), providing unconditional stability for nonlinear transient dynamic analysis. Rayleigh damping corresponding to 5% critical damping was assigned using the first two vibration modes according to

$$C = \alpha_M M + \beta_K K \quad (12)$$

where (α_M) and (β_K) are the mass- and stiffness-proportional damping coefficients determined from the modal frequencies of the bridge.

Three recorded earthquake ground motions—Bhuj (2001), El Centro (1940), and Loma Prieta (1989)—were selected to represent diverse seismic characteristics for evaluating the sequential seismic performance of the isolated RC bridge. The records were imported into OpenSeesPy, converted to consistent SI units (m/s²), baseline corrected, and verified for sampling interval consistency before dynamic analysis..[20] The characteristics of the

three earthquake ground motions selected for the sequential nonlinear time-history analysis are summarized in Table 5.

Table 5. Characteristics of the earthquake ground motions used for sequential nonlinear time-history analysis

Record ID	Earthquake	Year	Magnitude (Mw)	Original PGA (g)	Sampling Interval, Δt (s)	Duration (s)	Role in Analysis
GM-1	Bhuj	2001	7.7	0.36	0.01	7.5	Mainshock (EQ-1)
GM-2	El Centro (Imperial Valley)	1940	6.9	0.359	0.02	As used in record	Second earthquake (EQ-2)
GM-3	Loma Prieta	1989	6.9	0.360	0.02	As used in record	Third earthquake (EQ-3)

To evaluate the bridge response over a wide range of seismic intensities, Incremental Dynamic Analysis (IDA) was performed by scaling each ground motion using intensity factors of 0.5, 1.0, 1.5, 2.0, 2.5, 3.0, 4.0, and 5.0. In addition, three earthquake sequence combinations were considered to account for record-to-record variability in cumulative damage assessment. Table 6 summarizes the intensity scaling scheme adopted for the Incremental Dynamic Analysis.

Table 6. IDA scaling used in this study

Parameter	Value
Intensity Measure (IM)	Peak Ground Acceleration (PGA) / Spectral Acceleration, (Sa(T1))
Scale factors	0.5, 1.0, 1.5, 2.0, 2.5, 3.0, 4.0, 5.0
Analysis method	Sequential nonlinear time-history analysis
Ground motion input	Path Time Series + Uniform Excitation
Structural state	Preserved between successive earthquakes (state-dependent analysis)

The processed acceleration records were applied as longitudinal base excitations using the Uniform Excitation load pattern and Path time series in OpenSeesPy. Unlike conventional independent-event analyses, the finite element model was not reconstructed between successive earthquake records. Instead, the committed nonlinear material state, including concrete cracking, reinforcement yielding, hysteretic bearing deformation, soil spring response, and residual displacements, was preserved throughout the analysis. Consequently, each successive earthquake acted on the damaged structural state produced by the preceding event, enabling realistic simulation of cumulative damage evolution and state-dependent seismic performance.[22]

2.8 Sequential Earthquake Analysis Framework

A sequential nonlinear time-history analysis framework was developed in OpenSeesPy to evaluate the cumulative seismic response of the seismically isolated reinforced concrete bridge subjected to multiple earthquake events. The analysis consisted of four stages: (i) gravity loading, (ii) application of the first earthquake, (iii) a free-vibration rest period to allow the structure to stabilize, and (iv) subsequent application of the second and third earthquakes to the previously damaged structural model. Unlike conventional independent-event approaches, the finite element model was created only once and was not reinitialized between successive earthquake records.

Consequently, all material and geometric nonlinearities accumulated continuously throughout the entire earthquake sequence.[16], [23]

The sequential earthquake analysis can be expressed as the state-transfer process

$$S_{i+1} = \mathcal{F}(S_i, \ddot{u}_{g,i+1}) \quad (13)$$

where S_i denotes the committed structural state after the i^{th} earthquake, $\ddot{u}_{g,i+1}$ is the ground acceleration record of the subsequent earthquake, and $\mathcal{F}(\cdot)$ represents the nonlinear finite element solution operator. Unlike conventional seismic assessment, where the structural state is reset before each analysis, the proposed framework transfers the complete nonlinear response history directly to the next earthquake event.

The committed state vector is defined as

$$S = [u, \dot{u}, \ddot{u}, \varepsilon_p, d, r] \quad (14)$$

where u , $\dot{u}$ and $\ddot{u}$ denote the nodal displacement, velocity, and acceleration vectors, respectively; ε_p represents accumulated plastic strains in the reinforced concrete and lead-rubber bearings; d denotes material damage variables associated with concrete cracking, crushing, and steel yielding; and r represents the residual structural deformation after each earthquake.

The earthquake records were applied using the Path time series and UniformExcitation load pattern. The nonlinear dynamic response was obtained using the implicit Newmark average acceleration method with

$$\gamma=0.5, \beta=0.25,$$

which updates the structural response according to

$$u_{t+\Delta t} = u_t + \Delta t \dot{u}_t + \left(\frac{1}{2} - \beta\right) \Delta t^2 \ddot{u}_t + \beta \Delta t^2 \ddot{u}_{t+\Delta t} \quad (15)$$

$$\dot{u}_{t+\Delta t} = \dot{u}_t + (1 - \gamma) \Delta t \ddot{u}_t + \gamma \Delta t \ddot{u}_{t+\Delta t} \quad (16)$$

A Rayleigh damping model corresponding to 5% critical damping was employed using the first two natural vibration modes,

$$C = \alpha_M M + \beta_K K$$

where (α_M) and (β_K) are the mass- and stiffness-proportional damping coefficients, respectively.

To improve numerical robustness during severe nonlinear response, an adaptive convergence strategy employing Newton, Krylov–Newton, Newton with Line Search, and Modified Newton algorithms was adopted. After each earthquake, the excitation was removed and the bridge was allowed to undergo free vibration for 12 s before the subsequent earthquake was applied. This procedure preserved the committed nonlinear state of the model, including concrete cracking and crushing, reinforcement yielding, lead-rubber bearing hysteretic deformation, nonlinear soil spring response, and residual deck and pier displacements. Consequently, each successive earthquake acted on an already damaged structural system, enabling realistic simulation of cumulative damage evolution, degradation of structural capacity, and state-dependent seismic performance under repeated earthquake loading.

3. Results and Discussion

3.1 Model Characteristics and Dynamic Behaviour

The developed finite element model represents a 4-span reinforced concrete bridge with a total bridge length of 58.8 m. Each span is 14.7 m long and supports a 12.0 m wide bridge deck. The bridge substructure consists of circular reinforced concrete piers having a height of 15.0 m and a diameter of 1.6 m, supported on pile foundations embedded 12.0 m into the supporting soil. The total seismic mass of the bridge is 629.82 t, corresponding to a seismic weight of 6176.44 kN, while each span contributes approximately 157.46 t to the overall structural mass.

Following gravity analysis, an eigenvalue analysis was performed to verify the dynamic characteristics of the bridge model and to determine the modal properties required for Rayleigh damping in the nonlinear time-history analyses.

$$(K - \omega^2 M)\phi = 0$$

$$T = \frac{2\pi}{\omega}$$

Where:

K: The global structural stiffness matrix.

M: The global structural mass matrix.

ω : The natural angular frequency in radians per second (where ω^2 represents the eigenvalue).

ϕ : The mode shape vector (or eigenvector) corresponding to the specific frequency ω .

T: The natural period of the structure in seconds

The introduction of lead-rubber bearings together with soil–structure interaction increased the overall flexibility of the bridge, resulting in a longer fundamental vibration period compared with an equivalent fixed-base configuration. Consequently, the isolation system shifted the dominant structural period away from the high-energy portion of the earthquake response spectrum, thereby reducing the transmission of seismic forces to the bridge piers and foundations.

The verified finite element model was used for pushover, sequential nonlinear time-history, incremental dynamic, and fragility analyses, ensuring consistent assessment of cumulative damage under successive earthquake excitations. The evolution of the LRB isolator hysteretic response under sequential earthquake loading is illustrated in Figure 5.

LRB Isolator Hysteresis – Central Bent (Phase by Phase)

Loops migrate as plastic offset accumulates

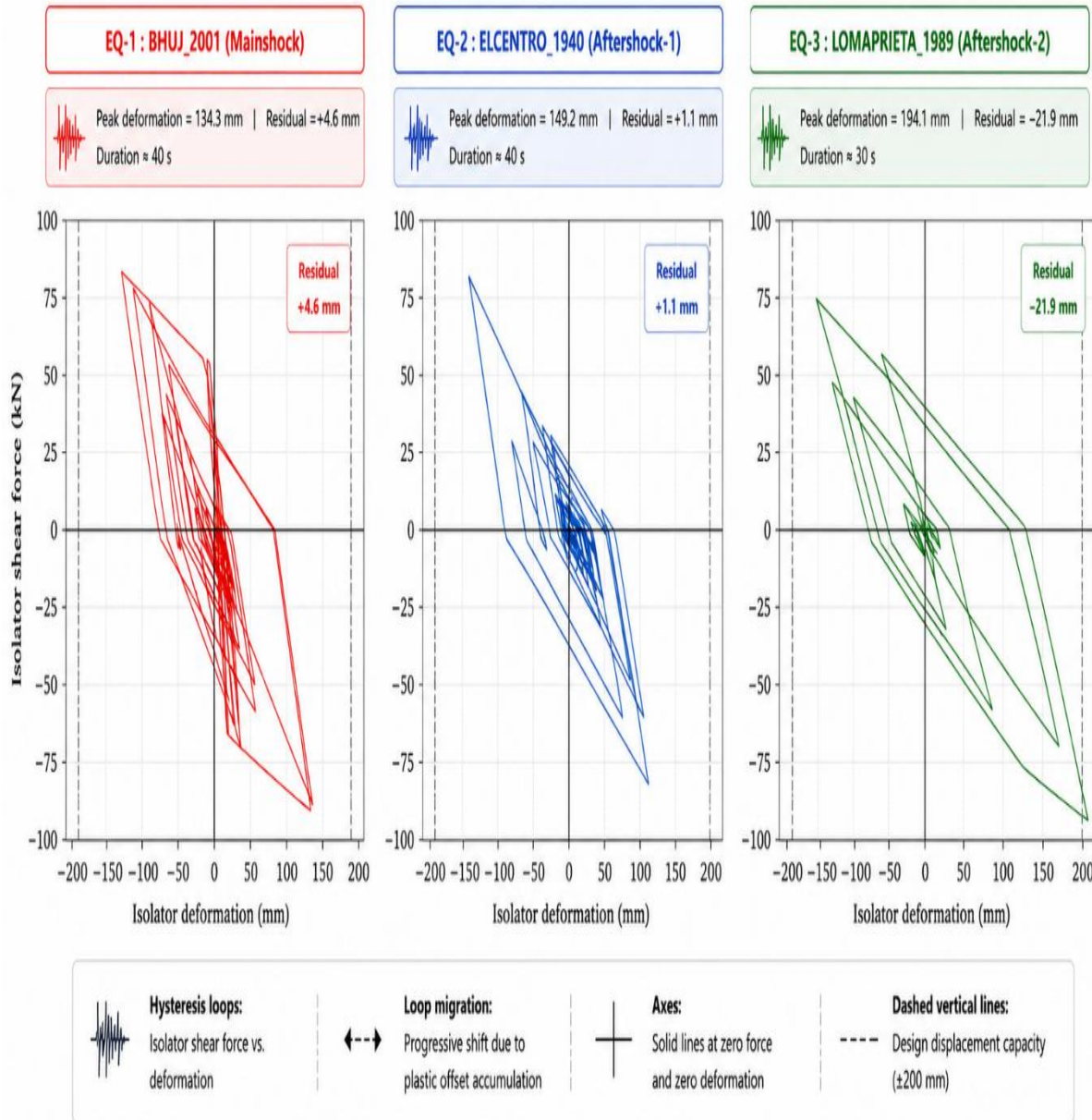


Fig 5 Evolution of LRB isolator hysteretic response under sequential earthquake loading.

3.2 Sequential Seismic Response

The nonlinear sequential earthquake analyses were performed using three consecutive earthquake records: Bhuj (2001), El Centro (1940), and Loma Prieta (1989). Between successive earthquakes, the bridge was allowed to undergo free vibration while preserving the complete structural damage state. Consequently, each earthquake acted on the residual condition produced by the preceding event.

During Phase 1 (Bhuj 2001), the bridge experienced a peak pier drift of 0.0384%, a maximum isolator displacement of 134.32 mm, and a peak base shear of 457.0 kN. The corresponding peak deck acceleration was 0.3583 g, while the residual pier drift and residual isolator displacement after the excitation were 0.0018% and 4.65 mm, respectively. The lead-rubber bearings remained within the Slight damage state, and no structural collapse was observed. The corresponding sequential deck displacement time histories are presented in Figure 6.

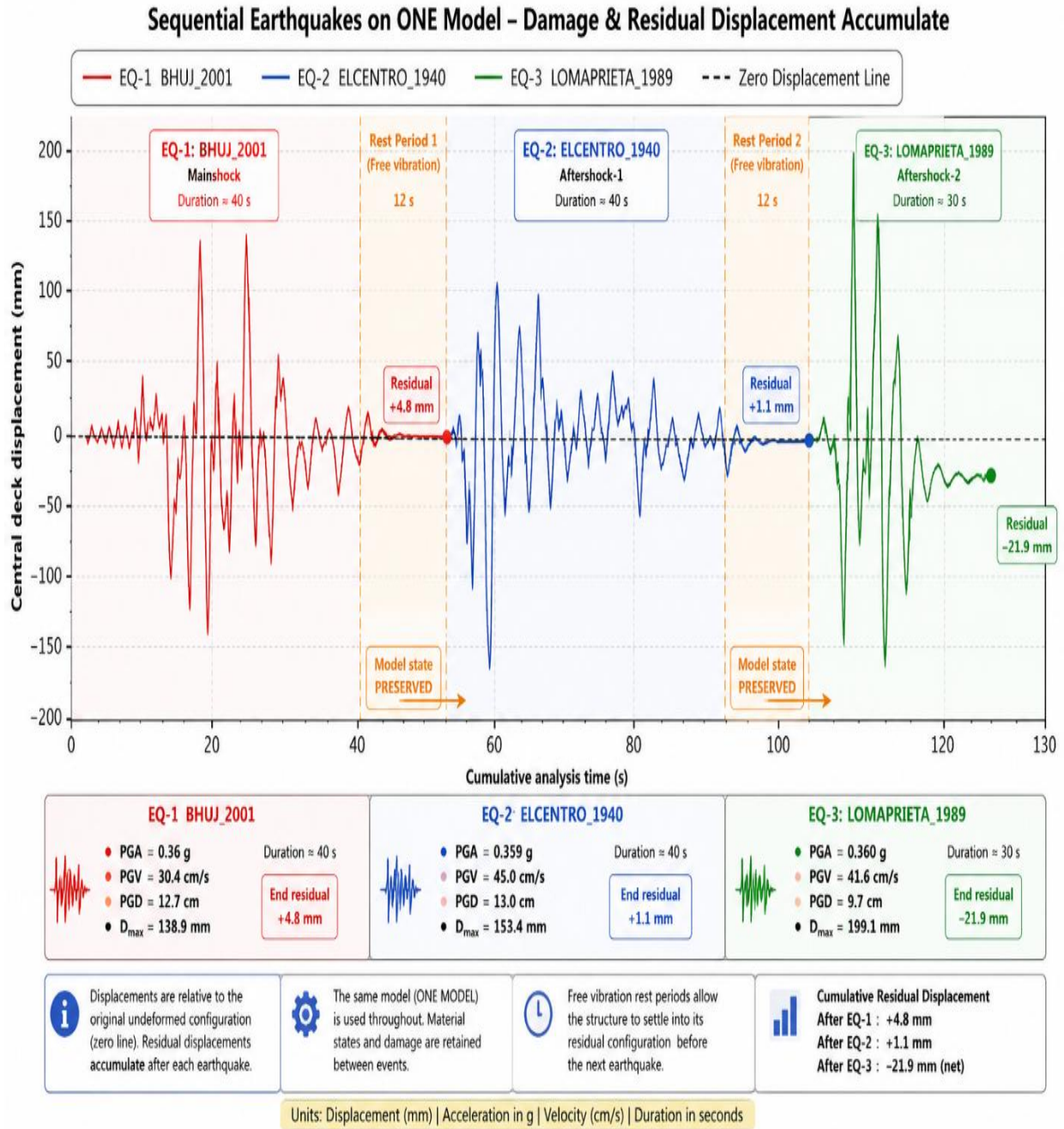


Fig 6 Sequential nonlinear deck displacement time histories of the seismically isolated RC bridge

Following the residual deformation generated during Phase 1, Phase 2 (El Centro 1940) produced a peak pier drift of 0.0358% and an increased maximum isolator displacement of 149.15 mm. The peak base shear decreased slightly to 424.8 kN, with a corresponding peak deck acceleration of 0.3541 g. Residual pier drift remained very small (0.0017%), whereas the residual isolator displacement reduced to 0.64 mm, indicating that the isolation system effectively dissipated seismic energy and limited permanent deformation.

In Phase 3 (Loma Prieta 1989), the bridge exhibited the largest isolator response, with a peak displacement of 194.09 mm, corresponding to a cumulative isolator deformation ratio of 0.97. Although the maximum base shear (443.7 kN) remained comparable to the previous earthquake phases, the peak deck acceleration increased to 0.4325 g because of the higher spectral demand of the ground motion. The residual isolator displacement increased to 22.14 mm, indicating accumulation of permanent deformation within the isolation system. Nevertheless, the peak pier drift remained limited to 0.0372%, and no collapse or severe structural damage was observed.[21], [24]

A comparison of the seismic intensity measures and structural response parameters across the three sequential earthquake phases is presented in Figure 7.

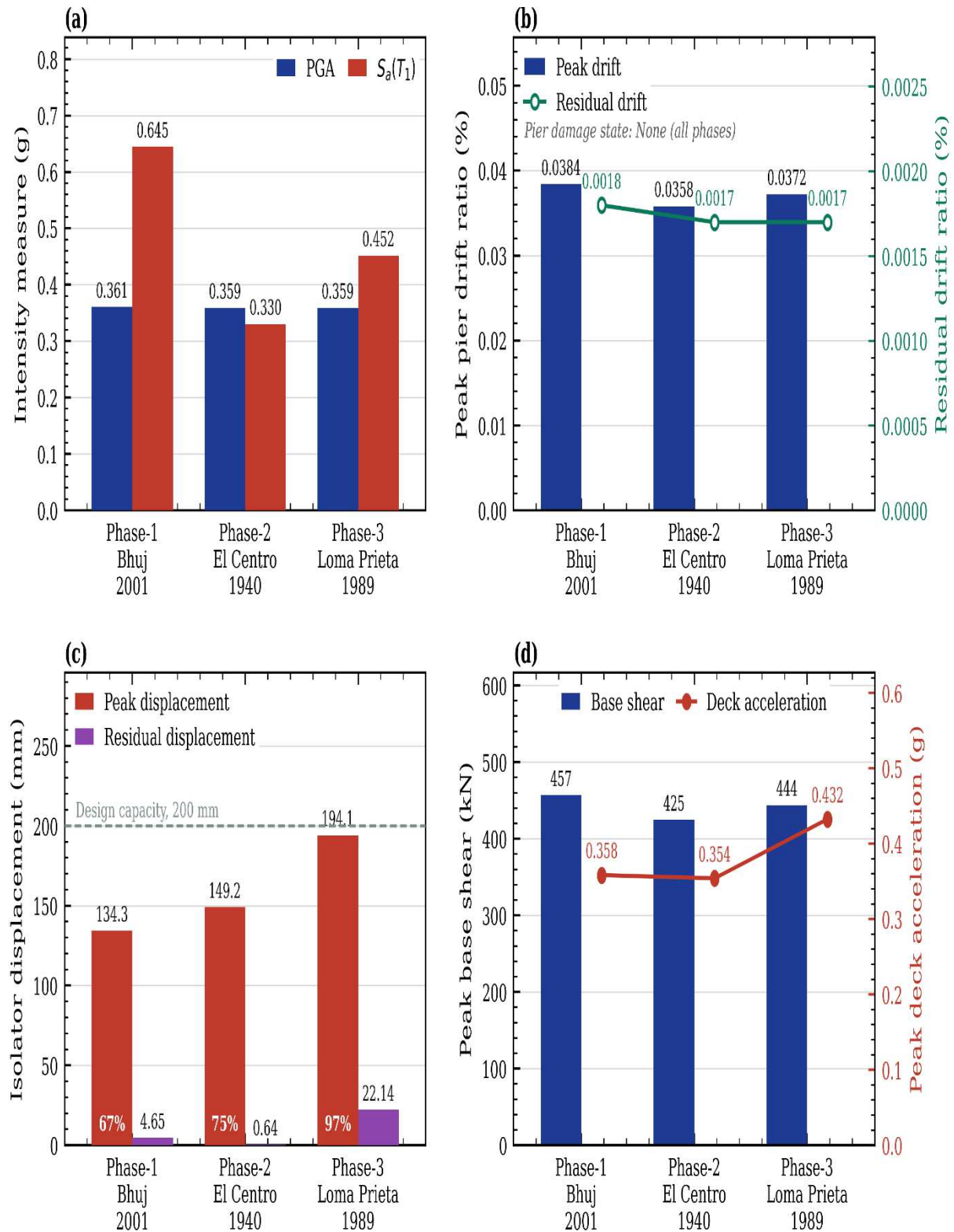


Fig 7 Comparison of seismic intensity measures and structural response parameters under sequential earthquake loading.

Overall, the results demonstrate that the lead-rubber bearing isolation system effectively protected the bridge substructure throughout the sequential earthquake sequence. The bridge maintained consistently low pier drift

ratios (0.0358–0.0384%) and moderate base shears (424.8–457.0 kN), while the primary cumulative effect of repeated seismic loading was observed as an increase in residual isolator displacement rather than deterioration of the reinforced concrete piers. No collapse was predicted during any phase of the sequential analysis, confirming satisfactory seismic resilience of the isolated bridge under the considered earthquake sequence. The evolution of cumulative seismic demand, residual response, and damage state throughout the earthquake sequence is illustrated in Figure 8.

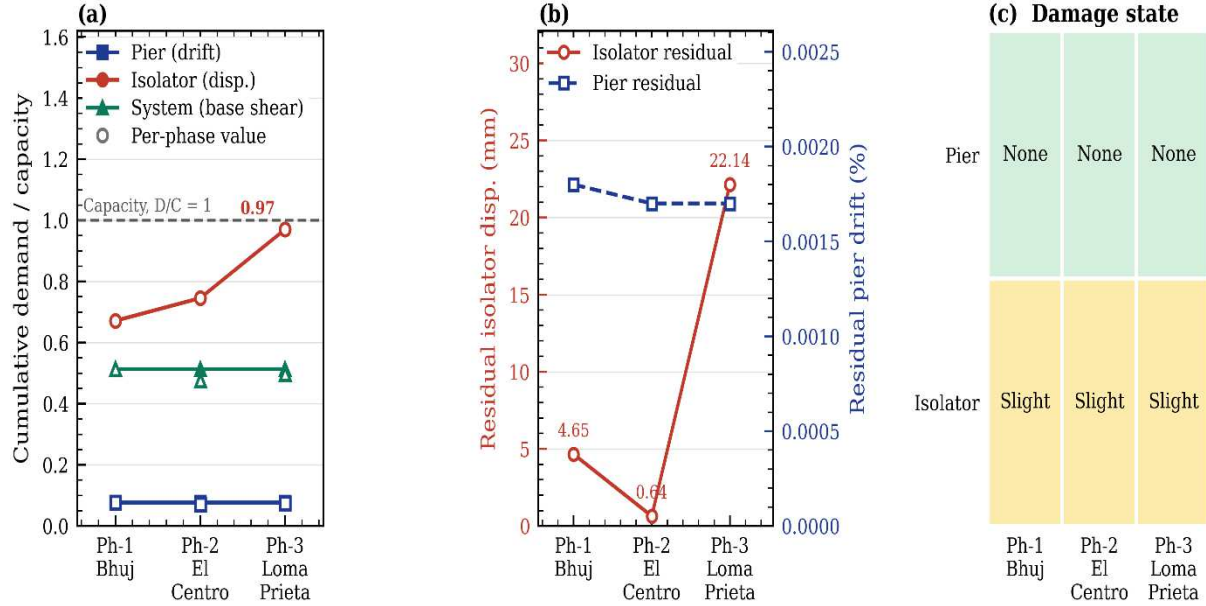


Fig 8. Evolution of cumulative seismic demand, residual response, and damage state under sequential earthquake loading.

3.3 Pushover Analysis and Capacity Assessment

The base shear–displacement pushover capacity curve, together with the displacement-dependent seismic response of the isolated bridge, is presented in Figure 9.

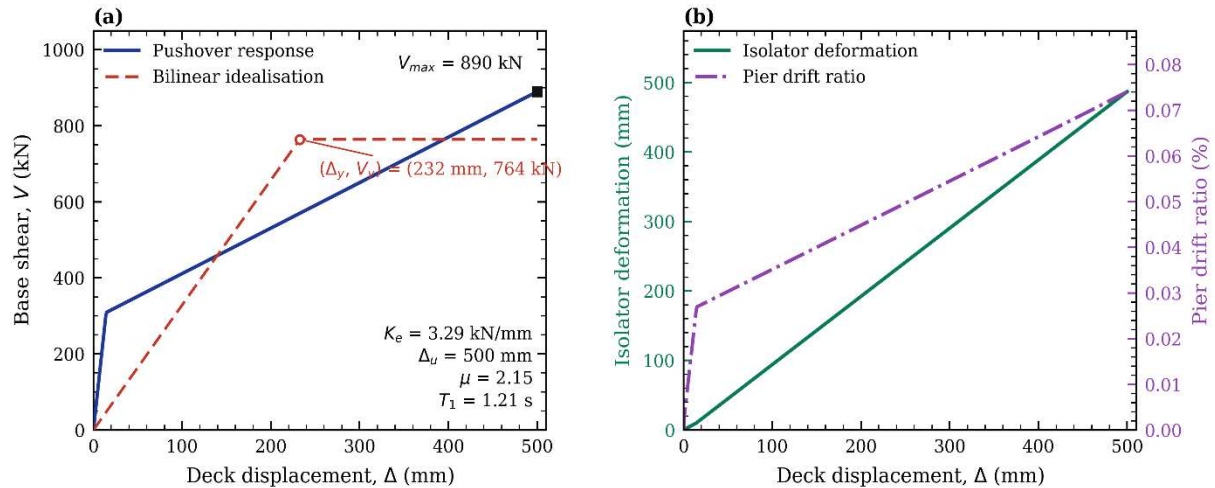


Fig 9 Pushover capacity curve and displacement-dependent seismic response of the isolated RC bridge.

A nonlinear static pushover analysis was performed by applying a monotonically increasing longitudinal lateral load until the control-node displacement reached 0.50 m. The resulting base shear–deck displacement capacity curve was used to evaluate the lateral strength, stiffness, deformation capacity, and overall structural performance of the isolated bridge.[17], [21]

The pushover curve exhibits an almost linear increase in base shear throughout the loading history, indicating that the bridge retains significant lateral stiffness within the investigated displacement range. The maximum lateral resistance achieved was 889.88 kN at a deck displacement of 0.50 m. At this loading stage, the corresponding pier drift ratio was only 0.074%, while the lead-rubber bearings experienced a maximum horizontal deformation of 0.487 m.

The response demonstrates that the majority of the imposed lateral displacement was accommodated by the lead-rubber bearings rather than by the bridge substructure. The isolation system effectively concentrated deformation within the bearings, thereby protecting the reinforced concrete piers from significant flexural demand. Consequently, the bridge maintained a stable load–displacement response without any evidence of abrupt strength degradation or global instability over the analysed displacement range.

These observations are consistent with the sequential earthquake analyses presented in Section 3.2, where the maximum pier drift remained below 0.04% despite repeated seismic loading. The agreement between the static and dynamic analyses confirms that the lead-rubber bearing isolation system successfully limits force transmission to the bridge substructure while providing sufficient displacement capacity to accommodate severe earthquake demands. Overall, the pushover analysis verifies that the bridge possesses adequate lateral strength, high deformation capacity, and stable nonlinear behaviour, supporting its favourable seismic performance under sequential earthquake loading.

3.4 Incremental Dynamic Analysis

Incremental Dynamic Analysis (IDA) was performed by progressively scaling three sequential earthquake records from 0.5 to 5.0 times their original intensity to evaluate the nonlinear response, cumulative damage evolution, and collapse capacity of the seismically isolated bridge. The earthquake intensity was represented by the scaled intensity measure (IM),

$$IM = \lambda IM_0$$

where IM_0 is the original intensity measure of the earthquake record (spectral acceleration or peak ground acceleration), and λ is the scale factor. In the present study, ($\lambda = 0.5, 1.0, 1.5, 2.0, 2.5, 3.0, 4.0,$) and (5.0).

The engineering demand parameters (EDPs) used to evaluate structural performance were the cumulative pier drift ratio and cumulative lead-rubber bearing (LRB) deformation ratio. The pier drift ratio was calculated as

$$\theta = \frac{\delta_{\max}}{H} \times 100\%$$

where $\delta_{\max}$ is the maximum lateral displacement of the pier and (H) is the pier height. The cumulative isolator deformation ratio was evaluated as

$$\mu_{\text{iso}} = \frac{\delta_{\max}}{\delta_d}$$

where $\delta_{\max}$ is the maximum isolator displacement and δ_d is the design displacement capacity of the LRB.

The IDA results demonstrate a gradual increase in structural response with increasing seismic intensity. For Sequence S1, the cumulative pier drift after the third earthquake increased from 0.0330% at a scale factor of 0.5 to 0.1098% at 4.0, while the cumulative isolator deformation ratio increased from 0.4984 to 4.0552. Structural collapse was first observed at the maximum scale factor of 5.0.

For Sequence S2, the bridge exhibited the lowest collapse capacity. The cumulative pier drift increased from 0.0362% at a scale factor of 0.5 to 0.0720% at 2.5, after which collapse occurred during the third earthquake phase at a scale factor of 3.0. This indicates that Sequence S2 produced the most severe cumulative damage among the investigated loading sequences.

In Sequence S3, the cumulative pier drift increased progressively from 0.0340% to 0.0961% as the scale factor increased from 0.5 to 3.0. Collapse initiated during the second and third earthquake phases at a scale factor of 4.0, while complete structural collapse occurred at 5.0.

Overall, the IDA results confirm that the isolated bridge maintains stable nonlinear behaviour under low and moderate earthquake intensities, with deformation primarily concentrated within the lead-rubber bearings. As the seismic intensity increased, cumulative damage progressively reduced the structural collapse capacity. Among the investigated loading histories, Sequence S2 exhibited the lowest collapse resistance, followed by Sequence S3, whereas Sequence S1 showed the highest collapse capacity. These results clearly demonstrate that the seismic vulnerability of isolated bridges depends not only on earthquake intensity but also on the sequence of successive earthquake events, highlighting the importance of state-dependent nonlinear analysis for realistic performance-based seismic assessment.[25] The state-dependent incremental dynamic analysis results for the RC piers and LRB isolation system under sequential earthquake loading are presented in Figure 10.

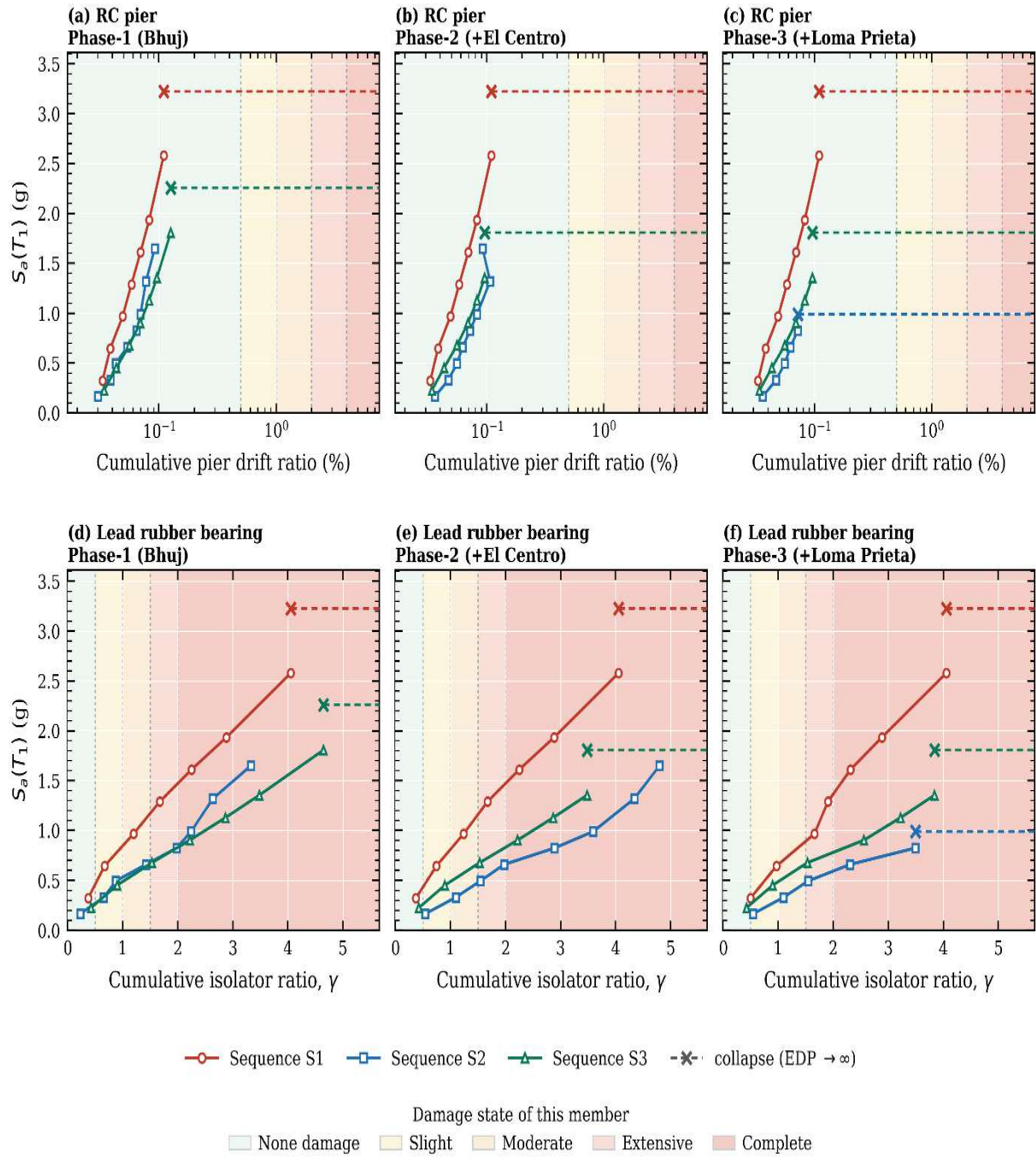


Fig 10. State-dependent incremental dynamic analysis of RC piers and LRB isolation system under sequential earthquake loading.

3.5 Fragility Assessment

A probabilistic fragility analysis was conducted to quantify the probability of exceeding predefined damage states under sequential earthquake loading. Fragility curves were developed by assuming a lognormal distribution of structural capacity, consistent with the Performance-Based Earthquake Engineering (PBEE) framework. The probability of exceeding a given damage state for an intensity measure (IM) is expressed as:

$$P[DS \geq ds | IM] \Phi \left(\frac{\ln(IM) - \ln(\theta)}{\beta} \right) \quad (17)$$

where $(P[DS \geq ds | IM])$ is the probability of exceeding damage state (ds), (Φ) is the standard normal cumulative distribution function, (IM) is the spectral acceleration (S_a), θ is the median spectral acceleration corresponding to the damage state, and β is the logarithmic standard deviation (dispersion).

The median capacity was estimated from the nonlinear dynamic analyses as

$$\theta = \exp \left(\frac{1}{n} \sum_{i=1}^n \ln(IM_i) \right)$$

while the logarithmic dispersion was calculated as

$$\beta = \sqrt{\frac{1}{n-1} \sum_{i=1}^n (\ln(IM_i) - \ln(\theta))^2}$$

The fragility results indicate progressive deterioration of the bridge under sequential earthquake loading. For the reinforced concrete piers, the median spectral acceleration decreased from 2.47 g during Phase 1 to 2.26 g in Phase 2, and further to 1.81 g in Phase 3, demonstrating a continuous reduction in seismic capacity due to cumulative damage. Simultaneously, the logarithmic dispersion increased from 0.184 to 0.616, indicating greater variability and uncertainty in structural performance after successive earthquake events.[26], [27]

The lead-rubber bearings exhibited considerably lower median capacities than the RC piers, confirming that they function as the primary energy-dissipation component. The median spectral acceleration for the Complete damage state reduced from 0.997 g in Phase 1 to 0.910 g in Phase 2, and further to 0.813 g in Phase 3. Correspondingly, the probability of bearing damage increased with each successive earthquake, reflecting the accumulation of permanent deformation within the isolation system.[26]

Overall, the fragility assessment demonstrates that sequential earthquake loading significantly reduces the seismic capacity of the bridge. Although the RC piers retain relatively high resistance throughout the loading history, the lead-rubber bearings experience the greatest increase in damage probability. This behaviour confirms that the isolation system effectively concentrates nonlinear deformation within the bearings, thereby protecting the bridge substructure while maintaining the overall seismic resilience of the bridge under repeated earthquake excitation. The evolution of the state-dependent fragility curves for the LRB isolation system and RC piers is presented in Figure 11.

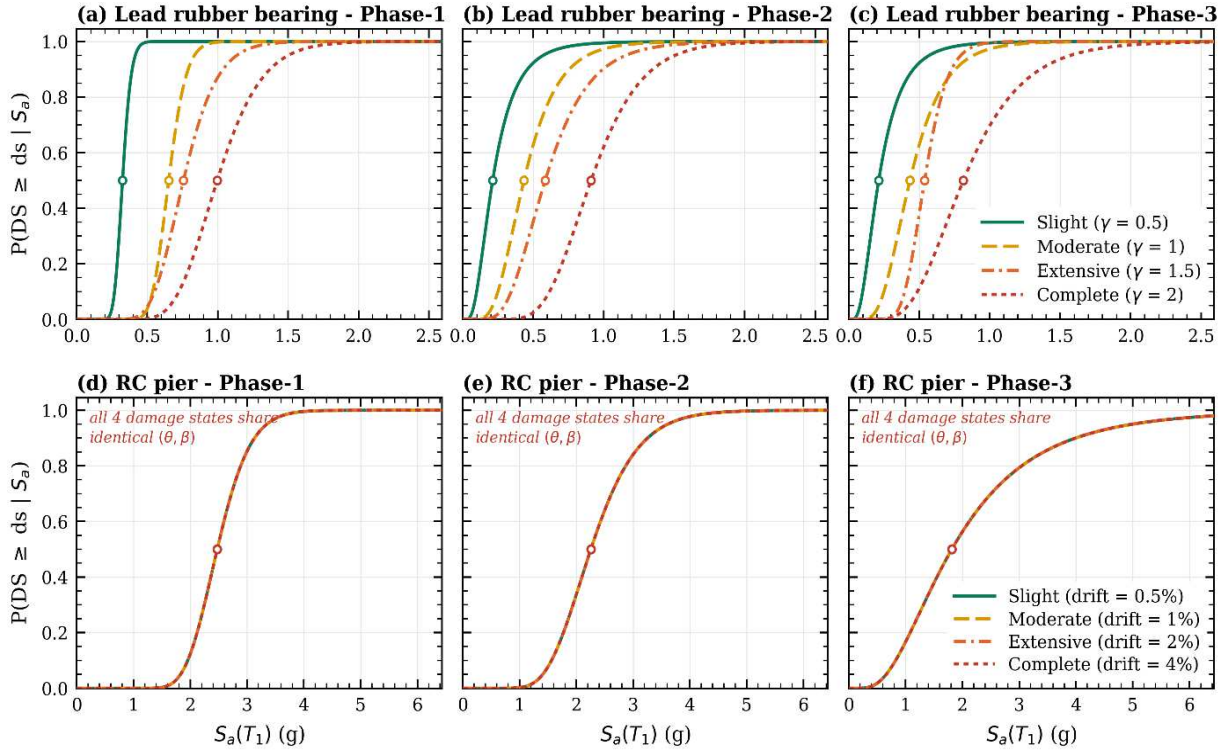


Fig 11. Evolution of state-dependent fragility curves for the LRB isolation system and RC piers under sequential earthquake loading.

3.6 Discussion of Seismic Performance and Engineering Implications

This study addresses an important limitation in conventional seismic assessment of bridges, where structural performance is generally evaluated under a single earthquake event, assuming the bridge remains undamaged before the onset of the ground motion. Such an assumption does not represent realistic seismic scenarios in which bridges may experience multiple strong earthquakes, including mainshock–aftershock sequences, while retaining residual deformation and accumulated damage. The present study bridges this research gap by developing an integrated sequential nonlinear time-history analysis framework that preserves the structural state between successive earthquakes, thereby enabling realistic assessment of cumulative damage and deterioration.[14], [16], [23]

The results demonstrate that bridge performance is strongly influenced by earthquake sequence. Although the isolated bridge maintained low pier drift ratios and stable lateral resistance during sequential loading, the IDA and fragility analyses showed a progressive reduction in collapse capacity and median spectral acceleration with increasing damage accumulation. Among the investigated loading histories, Sequence S2 produced the lowest collapse capacity, highlighting that seismic vulnerability depends not only on earthquake intensity but also on the order in which earthquakes occur. This observation cannot be captured using conventional single-event analyses.

Another significant contribution of this study is the integration of soil–structure interaction, fibre-based pier modelling, lead-rubber bearing isolation, Incremental Dynamic Analysis, and probabilistic fragility assessment within a unified OpenSeesPy framework.[12], [13] This comprehensive methodology enables consistent evaluation of structural response from elastic behaviour through collapse while accounting for residual deformation and cumulative damage. The analyses further confirm that the lead-rubber bearings effectively concentrate nonlinear deformation, protecting the reinforced concrete piers and maintaining structural stability throughout the sequential earthquake sequence.

The developed framework provides a practical tool for the performance-based seismic assessment of isolated bridges and can support the design, evaluation, and post-earthquake management of critical transportation infrastructure. By explicitly considering cumulative damage and state-dependent behaviour, the proposed

methodology offers a more realistic basis for estimating bridge resilience than conventional single-earthquake assessments and contributes to the advancement of multi-event seismic performance evaluation

4. Conclusions

This study presented a comprehensive numerical framework for evaluating the seismic performance of a lead-rubber bearing isolated reinforced concrete bridge under sequential earthquake loading. A three-dimensional fibre-based finite element model incorporating soil–structure interaction, nonlinear material behaviour, and seismic isolation was developed in OpenSeesPy. Sequential nonlinear time-history analysis, pushover analysis, Incremental Dynamic Analysis (IDA), and probabilistic fragility assessment were performed to investigate cumulative damage and structural resilience. The principal conclusions are as follows:

1. A sequential nonlinear analysis framework was successfully developed that preserves the structural damage state between successive earthquakes, enabling realistic simulation of cumulative damage that cannot be captured using conventional single-event seismic analyses.
2. The lead-rubber bearing isolation system effectively reduced seismic demand on the bridge substructure. Throughout the sequential earthquake analyses, the bridge maintained low pier drift ratios, while nonlinear deformation was primarily concentrated within the isolation bearings.
3. The pushover analysis demonstrated stable lateral behaviour with a peak base shear of 889.88 kN at a deck displacement of 0.50 m, indicating adequate lateral strength and deformation capacity without significant strength degradation.
4. The Incremental Dynamic Analysis revealed that structural performance depends strongly on the sequence of earthquake occurrence. Among the investigated loading histories, Sequence S2 exhibited the lowest collapse capacity, confirming that earthquake order significantly influences cumulative damage and seismic vulnerability.
5. Fragility analysis indicated a progressive reduction in seismic capacity with repeated earthquake loading. The median spectral acceleration of the reinforced concrete piers decreased from 2.47 g in Phase 1 to 1.81 g in Phase 3, while the corresponding capacity of the lead-rubber bearings decreased from 0.997 g to 0.813 g, demonstrating progressive deterioration of the isolation system.
6. The integrated modelling framework combining soil–structure interaction, fibre-based pier modelling, lead-rubber bearing isolation, sequential nonlinear analysis, IDA, and fragility assessment provides a robust methodology for the performance-based seismic assessment of isolated bridges. The proposed approach offers a more realistic representation of bridge behaviour under repeated earthquake events and can support the design, assessment, and resilience evaluation of critical bridge infrastructure in seismic regions.

Overall, this study demonstrates that considering sequential earthquake loading is essential for accurately evaluating the seismic resilience of isolated reinforced concrete bridges. By accounting for cumulative damage and residual structural response, the proposed framework provides a more realistic and reliable alternative to conventional single-earthquake assessments, thereby advancing performance-based bridge engineering.

The proposed framework also establishes a strong foundation for future research. Although the present work focuses on earthquake–earthquake sequences, it can be readily extended to multi-hazard scenarios, including earthquake–flood, earthquake–scour, and earthquake–corrosion interactions. Future studies may further enhance the framework through uncertainty quantification, time-dependent deterioration modeling, and experimental validation, supporting its broader application in resilient bridge design and assessment.

Credit Author Contributions

Dheeraj Kumar Kaurav: Conceptualization, Methodology, Software, Formal analysis, Investigation, Data curation, Validation, Visualization, Writing – Original Draft.

Dr. Juned Raheem: Conceptualization, Methodology, Supervision, Validation, Formal analysis, Interpretation of results, Writing – Review & Editing.

Dr. Neeraj Tiwari: Supervision, Resources, Project administration, Funding acquisition, Methodology, Validation, Writing – Review & Editing.

All authors contributed to the conception, design, and critical revision of the manuscript, approved the final version for publication, and agreed to be accountable for all aspects of the work.

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Declaration of Competing Interest

The authors declare no conflicts of interest. They have no known competing financial interests or personal relationships that could have influenced the work reported in this paper.

Data Availability Statement

The data and computational models supporting the findings of this study are available from the corresponding author upon reasonable request. The OpenSeesPy scripts and processed datasets can be provided to facilitate the reproducibility of the reported results.

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