



Seismic evaluation of RC bridge pier using analytical fragility curves

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Abstract

This paper aims to evaluate the seismic vulnerability of a RC bridge pier using analytical approach that involves numerical modelling of structure, nonlinear analyses on the model and preparation of damage ranks for different damage states. In addition, simplified method to develop fragility curves for a typical highway bridge pier using nonlinear modelling at element and material levels has been discussed in this study. An existing two-span PSC box girder bridge has been chosen to carry out the analysis. Beam with hinges model for element modelling, reinforcing steel and concrete 01 models have been adopted for steel and concrete materials, respectively. Nonlinear static analysis and time history analyses were carried out to evaluate the capacity of pier, and corresponding responses of pier were studied under different ground motion intensities. By assuming log-normal distribution, fragility curves were constructed in longitudinal and transverse directions. In longitudinal direction, the probability of exceeding slight, moderate and extensive damage states is 73.9%, 65.2% and 58.5%, respectively, at 2.5 g ($g = 9.81 \text{ m/s}^2$) peak ground acceleration (PGA) and the probability of collapse at 2.5 g is 50%. In transverse direction, the probability of exceeding slight, moderate and extensive damage states is 91.7%, 98.2% and 80.75%, respectively, at PGA 3 g, and the probability of collapse is 59.8% in this direction. This simplified method discussed in the present study is useful to construct fragility curves for bridges in India which fall in the same group and similar characteristics. Fragility curves are particularly useful in assessing the seismic vulnerability of bridge piers in highly seismic-prone areas of India where seismic retrofit of bridges and pre-earthquake planning are becoming more prevalent.

Keywords Beam with hinges · Concrete 01 · Fragility curves · Nonlinear static analysis · Nonlinear time history analysis · Reinforcing steel

Introduction

Fragility curves

Structures are vulnerable to earthquakes, especially for high seismic intensity. So, it is essential to know the probable damages that happen to structures for different seismic intensities. In recent years, fragility curves evolved as the widely accepted tool to assess the seismic vulnerability of existing structures where the probability of exceeding a damage state is expressed as a function of ground motion indices such as

peak ground acceleration (PGA) and peak ground velocities (PGV). The fragility function can be represented as [1].

$$\text{Fragility} = P [\text{LS}/\text{IM} = y] \quad (1)$$

where 'LS' represents limit state of the structure at local or global level, 'IM' is the ground motion intensity in terms of PGA or PGV, etc., and 'y' is the realized ground motion intensity (IM).

Necessity of fragility curves to Indian scenario

India has witnessed several destructive earthquakes over the last few decades [2–4]. Some of the major earthquake details are shown in Table 1, which indicates occurrence of different magnitude earthquakes is frequent in India. Damages due to these earthquakes have emphasized the need of seismic vulnerability assessment of existing structures, because most of the structures, especially bridges, were designed and built before the current seismic guidelines were included in codes

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Table 1 Details of few earthquakes occurred in India

SI. No	Earthquake	Station	Year	Magnitude	Site class	*V _s (m/s)
1	Chamoli	Chamoli	2005	5.2	A	700–1400
2	Mandi	Mandi	2006	3.5	B	375–700
3	Uttarkashi	Roorkee	2007	5.0	C	200–375
4	Bangladesh–India	Bongaigaon	2008	4.8	C	200–375
5	Hindukush	Bilaspur	2008	5.8	B	375–700
6	China–India border	Rampur	2009	5.3	A	700–1400
7	Myanmar–India border	Guwahati	2009	5.5	C	200–375
8	Nepal–India border	Roorkee	2011	5.7	C	200–375
9	Jammu–Kashmir Himachal border	Chamba	2013	5.4	A	700–1400
10	Nepal–India	Ghaziabad	2016	5.2	A	700–1620

*V_s = Shear wave velocity

[5, 6]. Hence, it is important to assess the seismic vulnerability of existing bridges, so that by suggesting suitable retrofitting strategies, life span of those structures can be enhanced and failure of structures can be avoided. Pre-earthquake planning and estimation of monetary losses can be done if the current condition of existing structures is known.

In India, practicing engineers still follow the force-based design philosophy for their analyses and design calculations [5, 6]. Force-based design method which controls the strength and indirect displacements is not capable of understanding the performance of structures under dynamic forces. Whereas in performance-based earthquake engineering methodology (PBEE), these issues were addressed and it has become popular because of its simplicity in its understanding and practical application.

After Bhuj earthquake (2001), researchers in India identified the need of revising seismic guidelines in code of practices. Earlier seismic zone map was also revised during this time. The current seismic zone of India is shown in Fig. 1. Entire country was divided into four zones depending upon the seismic intensity and surrounded active faults, etc. Seismic force is estimated depending upon the type of the soil, zone factor, importance and reduction factors provided in the code and design is carried out further. These existing methods to estimate the seismic force on structures are capable of designing earthquake-resistant structures, but there are no strong guidelines provided in these codes to evaluate the seismic vulnerability of existing structures. Hence, it is essential to adopt a widely accepted tool and methodology to address this issue.

Various methods available to develop fragility curves

Fragility curves have been developed by various researchers using various methods. Though the initiation to develop fragility curves was taken in 1975 by Whitman et al. [7], it has got the attention from research fraternity when ATC 25

[8] report introduced continuous fragility curves based on continuous damage functions. A detailed explanation of each method to generate fragility curves is as follows.

Expert-based fragility curves

Expert-based fragility curves were one of the oldest and simplest method to develop fragility curves in which an expert panel was questioned about the probable damage of a structure under various seismic intensities. Based on the panel opinion, probability distribution functions for a particular damage state under various ground motions intensities will be updated. One popular and practical example for expert-based fragility curves for was reported in ATC 13 report for typical California infrastructure. Expert-based fragility curves were not widely accepted because they are extremely subjective, biased and lack of reliability as they are completely based on judgement of panel.

Empirical fragility curves

In this method, fragility curves were developed based on damage distributions after the post-earthquake survey [9, 10]. Different approaches have been adopted by various researchers to develop empirical fragility curves. Basoz and Kiremidjian [9] developed empirical fragility curves by performing logistic regression analysis, whereas Der Kiureghian [10] adopted a Bayesian approach. After the Kobe earthquake, Shinozuka et al. [11] using the damage data from Kobe earthquake applied maximum likelihood method to estimate the parameters of log-normal distribution describing the fragility curves. However, this method also suffers from few drawbacks like large degree of uncertainty and lack of generality. The derivation of robust empirical fragility functions should reliably encompass the selection of damage data sources, characterization of ground motion severity, selection of damage measures, formulation of fragility functions and finally verification of the study results

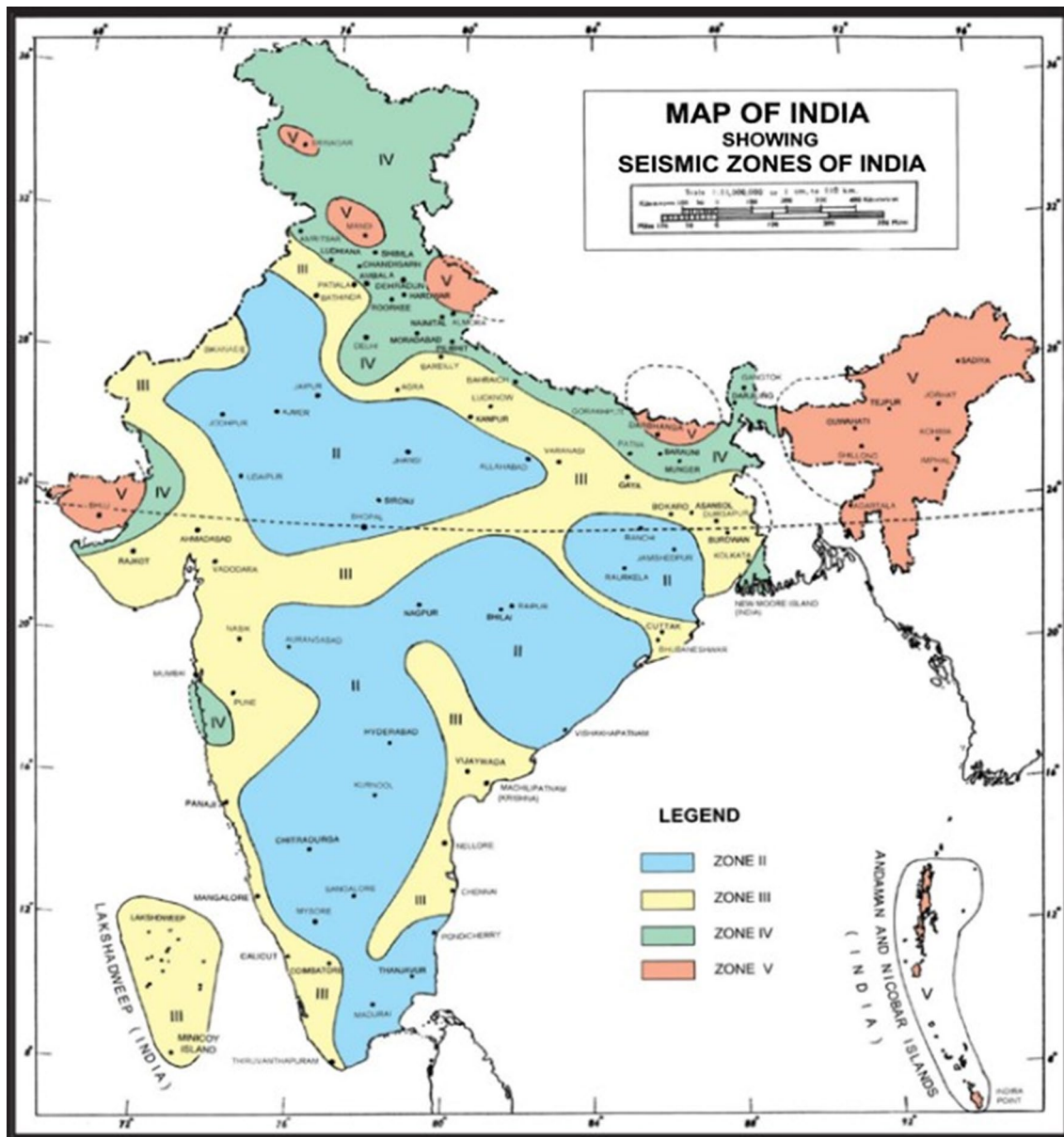


Fig. 1 Seismic zones of India [6]

with observed damage data. Ultimately, loss predictions with a quantifiable reliability would be realized only through the collection and archiving the post-earthquake field damage data coupled with a relevant hazard definition from earthquakes. However, this goal is not even on the horizon of the earthquake engineering community [12]. Yamazaki et al. [13] have also developed empirical fragility curves using damage data from 1995 Kobe earthquake.

Experimental fragility curves

Though experimental results provide a basis for defining various damage measures, it is not economical and laborious

because it requires setting up the equipment and modelling of different structural components and this method requires lot of time. However, few researchers [14, 15] adopted this method.

Analytical fragility curves

Development of fragility curves by analytical methods has been considered as an effective method compared with other methods (mentioned above), because of its wide acceptance and reliability. With the advancement in development of powerful software packages which can accurately assess the structural behaviour under strong ground motions,

this method has successfully overcome the limitations of remaining methods. Analytical fragility functions adopt damage distribution functions simulated from the nonlinear analyses on structural models under various ground motion excitations. This results in a reduced bias and increased in reliability of the fragility estimate. However, ample care in modelling of structural components is prerequisite in this method as the limitations in the structural modelling capacities adversely affect the accuracy of results. Karim and Yamazaki developed fragility curves for highway bridge piers in Japan using fragility curves and compared with empirical fragility curves [16] and studied the effect of ground motions and effect of structural parameters on fragility curves of bridge piers using numerical simulation. Finally, developed a simplified procedure to develop fragility curves for highway bridge piers [17–19].

In this context of evaluating seismic vulnerability of existing structures, analytical fragility curves have been found to be more rational and consistent. Structural parameters and material properties can be incorporated in the modelling and nonlinear analyses, i.e. nonlinear static analysis and nonlinear time history analysis are capable in estimating the damage data efficiently. The rapid development of performance-based earthquake engineering (PBEE) software tools made the job of generating analytical fragility curves easier. In fact, this is the main behind the successful adoption and acceptancy of this method.

Hybrid fragility curves

Development of hybrid fragility curves involves incorporation of at least two of any methods (mentioned in above sections), most likely one is analytical and second one is either experimental or expert-based data [20, 21]. However, this method also has several drawbacks such as extrapolation of damage data and relationship between damage data and level of structural damage. Moreover, this method involves large aleatory and epistemic uncertainty which results in significant dispersion in the probabilistic model [1].

Literature review

Karim and Yamazaki [16] have presented a method to construct analytical fragility curves. A typical bridge pier has been taken for this study and design according to seismic design guidelines of Japan and ground motion record has been taken from 1995 Kobe earthquake. Nonlinear static analysis has been performed to obtain yield and ultimate displacements, and dynamic analysis has been performed to obtain hysteretic energy (E_c). Fragility curves have been calculated with results and obtained and compared with empirical fragility curves developed by Mander and Basoz

[22]. Karim and Yamazaki [18] studied the effects of structural parameters in constructing the fragility curves. In this study, they have considered few bridge piers and designed them according to the seismic design criteria of Japan 1964 and 1998. 250 ground motions have been chosen from the recorded ground motions of the USA, Japan and Taiwan to perform nonlinear time history analyses. Choi and Jeon [23] studied development of fragility curves for bridges commonly found in central and south-eastern United States. Various structural components, i.e. deck, columns, bearings, foundation were modelled as linear and nonlinear elements in this study. Around 100 ground motion data were taken for performing the analysis on various steel and concrete bridges. This study also assessed the effectiveness of the several retrofit measures for the bridges commonly found in this region using fragility curves. Karim and Yamazaki [19] developed a simplified method to construct fragility curves and established a relation between fragility curves and over-strength ratios of structures by performing linear regression analysis. This method was found to be very useful in developing fragility curves for non-isolated bridges in Japan.

Mackie and Stojadinovic [24] developed seismic fragility curves for a reinforced concrete overpass bridge. They have adopted PBEE methodology for fragility formulation for a reinforced concrete highway bridge at component level (column) as well as at global level (whole bridge). This fragility formulation involves the demand model damage model and loss model. Neilson and DesRoches [25] proposed a methodology for generation of analytical fragility curves, considering the contribution of major components like column, bearings and abutments, etc. Taking the individual component fragility, the overall bridge system fragility is estimated using probability tools. The typical bridges present in central and south-eastern United States were taken in this study. The seismic evaluation of the bridges was done using the synthetic ground motions generated by Mid America Earthquake centre. These ground motions suits were developed for a soil profile in Memphis for a moment magnitude M_w (5.5, 6.5 and 7.5) and four different hypo-central distances (10, 20, 50 and 100 kms). Out of the 220 ground motions, 48 ground motions were used by scaling in this study.

Banerjee and Shinozuka [26] developed fragility curves for a 242 m long, five span Caltran's bridge with one expansion joint by using nonlinear static method. For identification of spectral displacements, this method utilized the capacity spectrum method. The obtained spectral displacements were converted into rotations at bridge ends. The reliability of this method and the developed fragility curves were compared with fragility curves developed by using nonlinear time history analysis and they were in consistent. Lee et al. [27] developed fragility curves for the first time in Korea. The data of 1008 expressway bridges present in Korea were collected. The relationship between PGA and vulnerabilities

was established using logistic curve equations. Finding a logical and rational basis for transforming the structural disruption ratio of a bridge directly to its functional disruption is left for further study. Banerjee and Shinozuka [28] studied the bridge seismic damageability information obtained through empirical, analytical and experimental procedures and threshold damage states were quantified. Experimental damage data were processed to identify and quantify damage states in terms of rotational ductility at bridge ends. Empirical fragility curves were constructed using the data obtained in 1994 Northridge earthquake. A mechanistic model for fragility curves was developed in such a way that model can be calibrated against the empirical curve. Comparison shows an excellent agreement among all methods. Dryden and Fenves [29] studied the response of a two-span RC bridge using shake table experiments. The bridge was subjected to a total of 23 ground motions in the transverse direction ranging from pre-yield and increasing until failure. Nonlinear dynamic analyses of three-dimensional finite element models with different assumptions regarding the reinforced concrete columns were evaluated based on experimental data at both the local and global level. The results of these comparisons lend insight into the implications of modelling decisions for reinforced concrete systems. Wang et al. [30] developed fragility curves for a two span simply supported RC bridge, considering the support of the pier was fixed and abutments as roller supports. The vulnerability of the bridge pier was alone studied in this work. Ground motion data were selected from Berkeley data base. Baylon and Co [31] performed nonlinear static and dynamic analyses to obtain fragility curves for lifeline systems (bridge piers and light rail transit piers) in CAMANAVA (Caloocan-Malabon-Navotas-Valenzuela). Bilinear hysteretic model was assumed and normalized the ground motion data adopted in [17] method.

Nguyen et al. [32] developed seismic fragility curves for a double-curved continuous steel box girder bridge situated in Korea. The analysis was carried out in Opensees platform by modelling pier as a nonlinear beam-column element and superstructure as an elastic beam-column element. A well-scaled set of ground motion data was employed to study the responses of structure under different excitation levels. A series of damage state is defined based on a damage index which is expressed in terms of column displacement ductility ratio. Effect of double curvature shape on the seismic performance was assessed by the comparison with an equivalent straight bridge model. Sharma and Suwal [33] studied the seismic vulnerability evaluation of simply supported multi-span RCC bridge pier using analytical fragility curves. A multi-span simply supported RC T-girder bridge was taken in this study. Nonlinear static and nonlinear time history analyses were performed on this structure to capture the capacity and seismic demand. Finally, analytical

fragility curves were developed using first-order second-order method (FSOM). Firoj et al. [34] performed nonlinear static analysis to evaluate the seismic vulnerability of an existed RC bridge located in Zone IV India. The pushover analysis was performed using displacement coefficient and capacity spectrum method. The supports were assumed as fixed supports in this study. The results indicate that the existing bridge does not meet seismic criteria of spectral demand as per capacity spectrum method, and hence, retrofitting is suggested for bridge component.

Nesrine et al. [35] studied the effect of various parameters such as axial load, section of pile, longitudinal steel ratio of the pile (implanted in different types of sand with varying density) on the seismic fragility curves and performance of Interaction-Soil-Pile-Structure (ISPS). A series of nonlinear static analyses were carried out for better understanding of these phenomena for two different cases such as fixed and ISPS system. By comparing the capacity and fragility curves for all the cases, it was understood that the parameters considered have shown significant impact on the lateral capacity, ductility and seismic fragility. Lallam et al. [36] proposed a numerical method based on fuzzy analytical hierarchy process (FAHP) to assess the condition of masonry arch bridges. The FAHP method proposed in this study has demonstrated the effectiveness in eliminating the uncertainties and ambiguities present in the assessment of masonry bridges. Aviram et al. [37] studied the effect of the abutment modelling on the seismic response of bridge structures. In this study, six existing RC bridges were considered and each bridge was modelled with three different abutment models roller, simplified and spring. Modal, nonlinear static analysis and dynamic analyses were performed on the bridges in Opensees [38] platform to study the responses of the structures. It was found that roller supports are suitable for long-span bridges when responses of abutments are not the study of interest.

Different engineering demand parameters (EDPs) were adopted in previous studies while generating fragility curves. Nonlinear static and dynamic analyses were carried out to obtain the damage data. Among various methods damage index proposed by Park and Ang [39] was found to be more reliable and convincing and it was followed by most of the researchers and the same is adopted in the present study to construct the fragility curves. Most of the studies on seismic evaluation using fragility curves were carried out with linear element modelling of pier and bilinear behaviour assumption of steel [16–19]. Accuracy of any analytical study in structural engineering depends upon the efficiency of the model used in the analysis. When it is essential to study the behaviour of a structural component at micro-level, limiting to linearity is no longer advisable. Stresses, deformations, forces, etc., beyond linear point are essentially to be captured. To address this issue, beam with hinges model was

adopted to model the pier; reinforcing steel and concrete 01 material model were selected to model the steel and concrete, respectively. Most of the studies in India, related to this research area have been carried out using linear elastic models, and to study the dynamic response of the pier, most of the studies have focussed on response spectra only. Studying the nonlinear response of structures under a suite of ground motions having various ground motion intensities is often preferable than adopting response spectra methods. So, in this study, a suite of ground motions was selected to perform the nonlinear time history analyses.

Research objective

This study aims to evaluate the seismic capacity of a RC bridge pier by considering nonlinear element and material modelling. To achieve this objective, PBEE methodology has been adopted and an existing two-span (45.0 m + 45.0 m) PSC box girder with single bent circular RC pier has been chosen. The structural capacity of RC bridge pier using nonlinear static analysis and the dynamic responses of the pier under different ground motion excitations using nonlinear time history analysis are essential to prepare the damage indices of the pier. Using the damage indices data, fragility curves are constructed by assuming log-normal distribution functions for different damage ranks by which probability of exceeding a damage state can be easily understood.

Methodology

An existing bridge has been modelled in Opensees-MS bridge [40], a graphical user interface software tool to perform nonlinear analyses. In Performance-Based Earthquake Engineering (PBEE), performance levels can be defined based on different engineering demand parameters (EDPs). Researchers based on their interest and observations have selected different EDPs in their analyses to define various performance levels or damage states. The threshold values of these EDPs are different for different structural components. For the same EDP, different researchers have suggested different threshold limits based on their experience and observations. EDPs like rotational ductility, curvature ductility, displacement ductility, etc., can be obtained by performing nonlinear analyses on structure. Threshold values for different EDPs for column [1] are shown in Table 2.

As this study is limited to bridge pier, ductility values are taken to find out the damage index proposed by Park and Yang [39]. The adopted methodology is in similar lines with Karim and Yamazaki [17] and HAZUS [56]. The log-normal distribution function for fragility curves is shown in Eq. (2).

The probability of exceedance Karim and Yamazaki [16] is

$$P_r = \Phi[(\ln(X) - \lambda)/\zeta] \tag{2}$$

where P_r = Cumulative probability of exceedance, Φ = Cumulative normal distribution function, X = Peak

Table 2 Threshold values of column for different EDPs

Component	Demand parameter	Threshold value				References
		Slight	Moderate	Extensive	Collapse	
Column	Curvature ductility	1.29	2.1	3.52	5.24	Nielson [41]
		1	1.58	3.22	4.18	Ramanathan et al. [42]
		1	5.11	7.5	9	Ramanathan et al. [42]
		4.89	9.15	12.46	13.08	Ramanathan et al. [43]
		1.44	2.7	6.92	4.18	Ramanathan et al. [43]
	Displacement ductility	1	2	4	7	Choi et al. [44]
		1	2.73	4.54	6.5	Jara et al. [45]
		1	1.2	1.76	4.76	Alam et al. [46] and Hwang et al. [47]
		1	2	4	7	Alipour et al. [48]
		2.25	2.9	4.6	5	Banerjee and Prasad [49]
	Drift	1	1.22	1.78	4.8	Billah and Alam [50]
		5	7	11	30	Tavares et al. [51]
		0.7	1.5	2.5	5	Akbari [52]
		1.45	2.6	4.3	6.9	Li et al. [53]
		0.7	1.5	2.5	5	Kim and Shinozuka [54]
Rotational ductility	3.14	3.14–5.9	5.9–9.42	> 9.42	Banerjee and Chi [15]	
	1.58	3.33	6.24	9.16	Banerjee and Shinozuka [55]	
	0.25	0.25–0.75	0.75–1	> 1	Billah and Alam [50]	

ground acceleration in cm/sec^2 , $\lambda = \text{Mean}$, $\zeta = \text{Standard deviation}$.

The adopted statistical formulas in deriving the mean (λ) and standard deviation (ζ) are given in Eqs. (3) and (4), respectively, as follows:

$$\text{Mean } (\lambda) = \frac{\sum_{i=1}^N f_i \cdot \ln(x_i)}{\sum_{i=1}^N f_i} \tag{3}$$

$$\text{Standard deviation } (\zeta) = \sqrt{\frac{\sum_{i=1}^N [f_i (\ln(x_i) - \lambda)^2]}{N - 1}} \tag{4}$$

where f = frequency of damage rank per PGA, x = PGA in cm/s^2 , λ = Mean of the natural logarithm of PGA in cm/s^2 , ζ = Standard deviation of PGA in cm/s^2 .

Damage index proposed by Park and ang [39] is given in Eq. 5

$$\text{Damage Index (DI)} = (\mu_d + \beta \mu_h) / \mu_u \tag{5}$$

where μ_d = displacement ductility $\delta_{\text{max}}(\text{dynamic}) / \delta_{\text{yield}}$, β = constant (0.15), μ_h = Hysteretic energy ductility E_h / E_e , μ_u = Ultimate ductility $\delta_{\text{max}}(\text{static}) / \delta_{\text{yield}}$.

Damage ranks based on damage index (DI) are shown in Table 3.

Table 3 Damage index values range for different damage ranks

Damage index range	Damage rank	Damage level
$0 < \text{DI} \leq 0.14$	D	No damage
$0.14 < \text{DI} \leq 0.40$	C	Slight damage
$0.40 < \text{DI} \leq 0.60$	B	Moderate damage
$0.60 < \text{DI} \leq 1.00$	A	Severe damage
$1.00 \leq \text{DI}$	As	Collapse

The dynamic displacement value will be obtained from nonlinear time history analysis. Ultimate ductility (μ_u) value is the ratio between ultimate displacement and yield displacement. This value will be obtained from nonlinear static analysis. Hence, it is essential to perform both nonlinear static and nonlinear time history analyses to construct fragility curves using ductility values. Using the values obtained from nonlinear analyses damage indices will be calculated, and from damage indices, damage ranks will be decided using Table 3. Ground motions ranging from 0 to 2.5 g ($g = 9.81 \text{ m}/\text{s}^2$) in longitudinal and 0 to 3 g ($g = 9.81 \text{ m}/\text{s}^2$) in transverse direction have been considered in this paper. Assuming log-normal distribution as shown in Eq. 2, fragility curves will be developed for different damage ranks.

Modelling

In this study, 3D modelling of bridge has been done in Open-sees MS bridge, and modelling details of important structural components are given in the following sections.

Superstructure

The existing bridge is a two-span (45.0 m + 45.0 m) PSC box girder bridge. Cross section of superstructure and 3D model of the bridge are presented in Figs. 2 and 3, respectively. In seismic studies, superstructure is less vulnerable than substructure; therefore, it is modelled as linear elastic element.

Pier

The cross-sectional details of the bridge pier are shown in Fig. 4. The height of the pier is 5.5 m. Bridge pier consists of 70 bars of 32 mm diameter as longitudinal reinforcement and 12-mm bars with 150 mm centre to centre distance as

Fig. 2 Cross-sectional details of superstructure

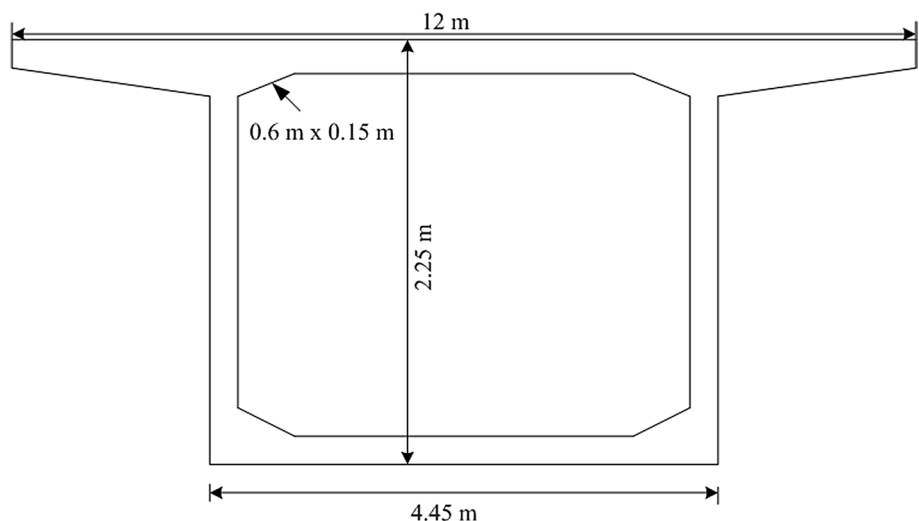


Fig. 3 3D model of bridge

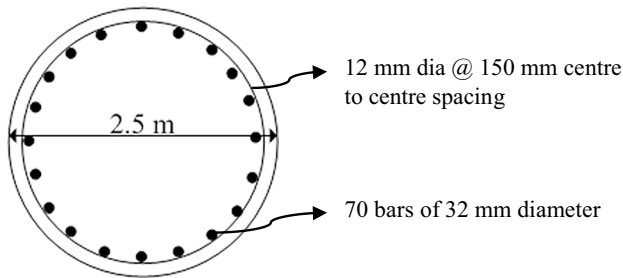
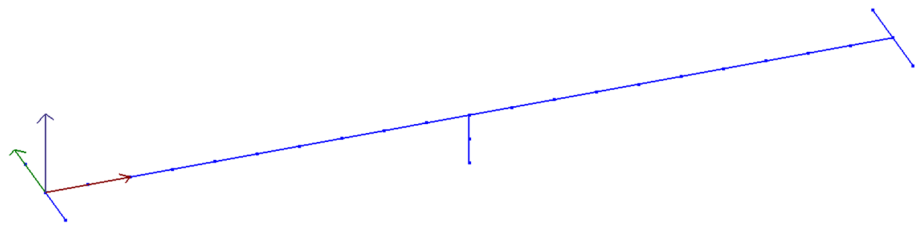


Fig. 4 Cross-sectional details of bridge pier

Table 4 Bridge details of two-span box girder

Item name	Description
Bridge type	box girder
Span configuration	45.0 m + 45.0 m
Superstructure details	Single cell box girder 12.0 m wide and 2.25 m deep 0.275 m deck and 0.3 m soffit
Sub-structure details	Single bent column 5.5 m height and 2.5 m diameter
Column support conditions	Fixed at top and bottom
Foundation details	Rigid
Analysis performed	Modal analysis Pushover analysis Nonlinear time history analysis

transverse reinforcement. Bridge and column details are presented in Tables 4 and 5.

Modelling of element

Basically, the finite element modelling of structural elements has been divided into three categories. One is lumped plasticity, in which plasticity is lumped at the ends of beam-column elements as rotational springs. Second one is distributed plasticity, in which plasticity is spread throughout the element. This model is widely accepted as it overcomes the limitations of lumped plasticity. In this model, force-based approach and displacement-based approach are available. In force-based approach, a force interpolation function is assigned at each section to distribute the nodal concentrated force including moment and axial force. In

Table 5 Reinforced concrete column properties

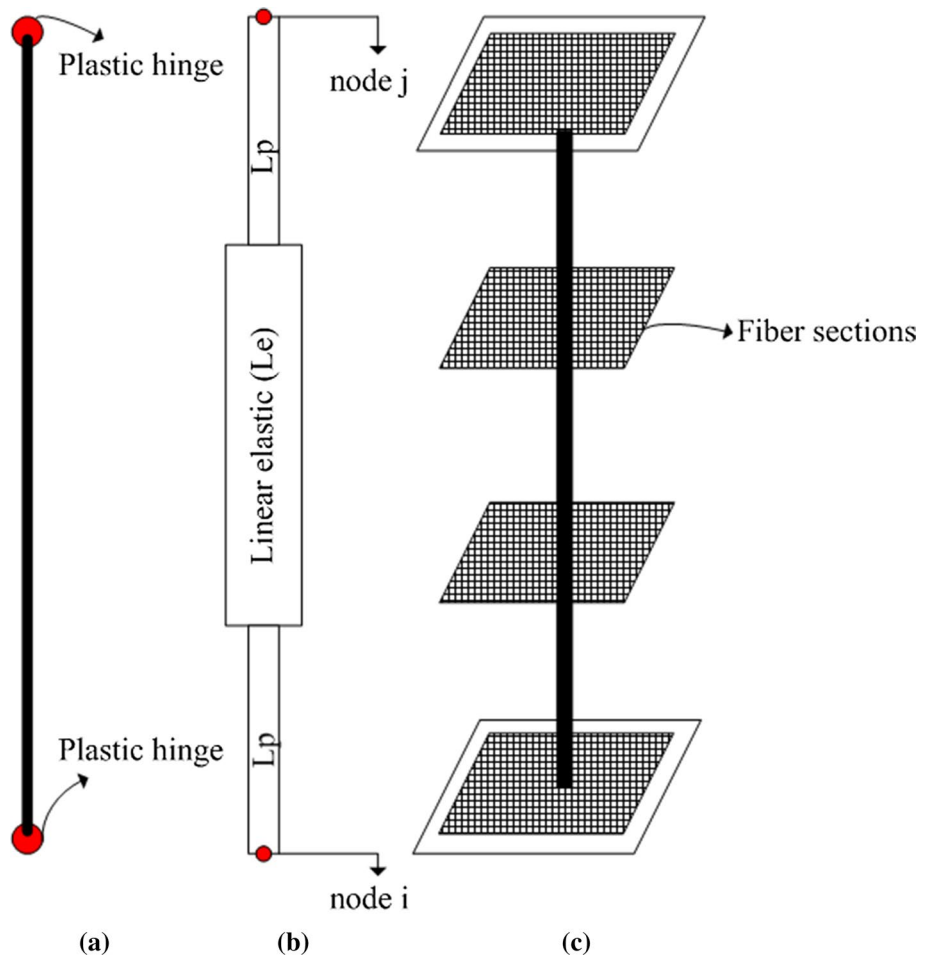
Parameter	Value
Number of longitudinal bars	70
Longitudinal bar size (mm)	32
Longitudinal steel (%)	1.14
Transverse bar size (mm)	12
Transverse bar spacing (mm)	150
Steel unit weight (kN/m ³)	78.5
Steel strain limit	0.09
Concrete unit weight (kN/m ³)	25
Concrete unconfined strength (MPa)	35

displacement-based approach, a displacement shape function is assigned to connect the local and global behaviour. It has been found that force-based distributed plasticity is relatively easier than displacement-based approach in terms of computational difficulty [57]. However, major issue with distributed plasticity is strain-softening behaviour of RC elements due to which localization phenomena occur in beam-column elements. To avoid the problems associated with this model, Scott et al. [58] have suggested an alternative method to model beam-column elements efficiently known as beam with hinges model. In this model, a predetermined plasticity length (L_p) is distributed at the ends and remaining element acts as a linear elastic (L_e) element. Computational efforts are significantly reduced in this modelling technique. Predetermined plastic hinge is calculated based on the formula given by Paulay and Priestley [59], and Gauss-Radau quadrature rule is applied for integration. Details of different element models are shown in Fig. 5. In the present study, bridge pier has been modelled as beam with hinges model. Pier has been divided using three nodes, and each span of superstructure has been divided into ten elements.

Modelling of materials

Material modelling of reinforced concrete bridge pier deals with the modelling of steel and concrete. Various material models were proposed for steel based on its strain hardening behaviour which are available in OpenSees manual [38]. For steel, Steel 01, Steel 02 and reinforcing steel are three widely used material models. Steel

Fig. 5 a Lumped plasticity b Beam with hinges c Distributed plasticity



01 material modelling is ideal where kinematic hardening behaviour is expected and Steel 02 is ideal where isotropic strain hardening behaviour is expected. Reinforcing steel material model is exclusively proposed for fibre modelling. Concrete 01 and Concrete 02 are the two models widely used material models for concrete. Concrete 01 is a uniaxial material which ignores the tensile strength of concrete, whereas concrete 02 is a uniaxial material model with strain-softening behaviour and tensile strength of concrete is not omitted.

As beam with hinges modelling is adopted in element modelling, reinforcing steel model is adopted for steel and Concrete 01 material model is adopted to model the concrete in this study. Lu et al. [60] have performed nonlinear analyses on ordinary standard bridge 2 (OSB 2). To model steel, reinforcing steel material model was adopted as fibre sections are involved and beam with hinges element model was adopted to model column. Material properties for steel and concrete are shown in Tables 6 and 7. The properties of unconfined and confined concrete were calculated based on Mander’s unified stress–strain approach [61]. Reinforcing steel material model properties were calculated as per

Table 6 Reinforcing steel properties

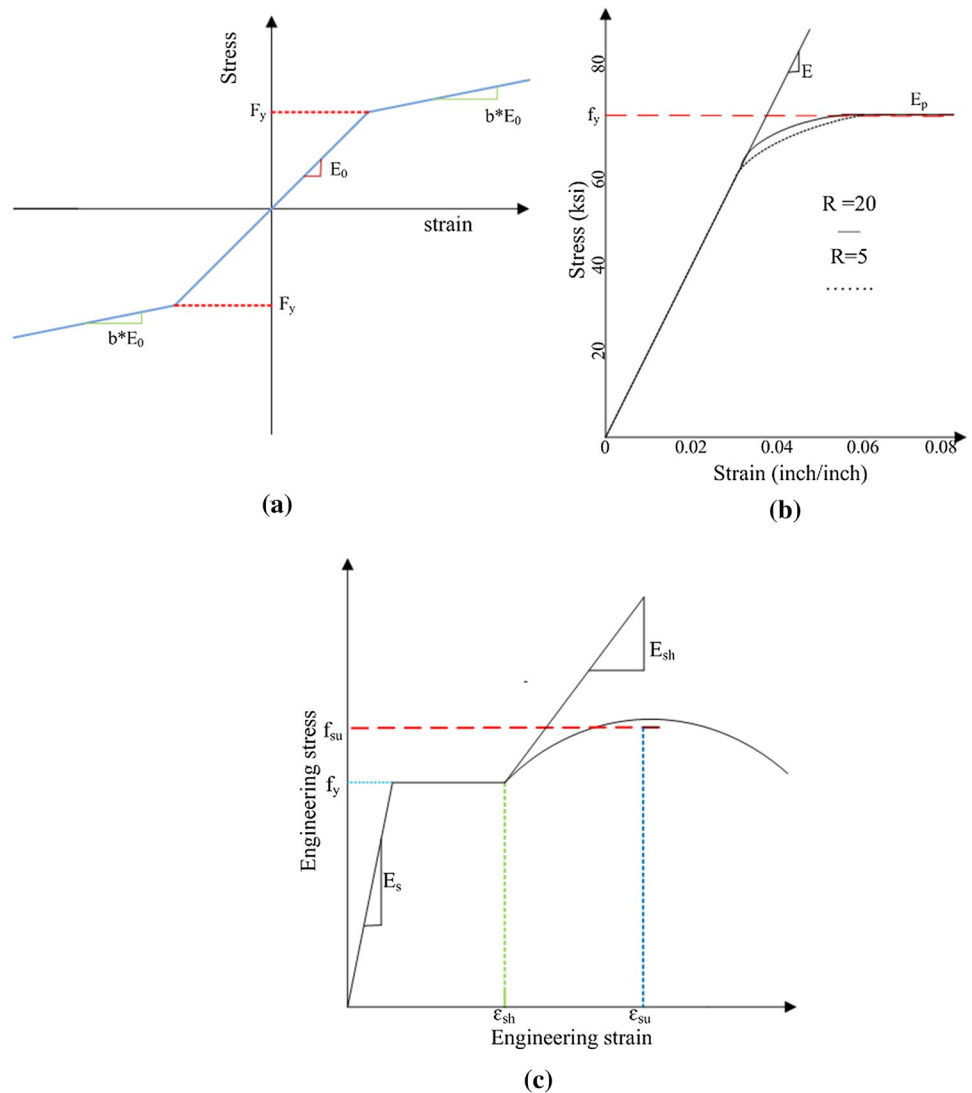
Parameter	Value
Young’s modulus (MPa)	2×10^5
Ultimate stress (MPa)	545
Young’s modulus at initial strain hardening (MPa)	562.5
Strain at initial strain hardening	0.01
Strain at peak stress	0.025

Table 7 Concrete properties

Parameter	Core concrete	Cover concrete
Compressive strength (MPa)	– 37.259	– 35
Strain at maximum strength	– 0.002	– 0.002
Crushing strength (MPa)	– 32.082	0
Strain at crushing strength	– 0.006	– 0.006

the guidelines given in Opensees Manual [38]. Stress–strain curves for steel and concrete models are shown in Figs. 6 and 7.

Fig. 6 a Steel 01 b Steel 02 c Reinforcing steel



Pier end conditions

End conditions of a pier are of three types: fixed–fixed condition, fixed-hinge condition and hinge-fixed condition. Usually, for a single bent column bridge, it is suggested to adopt fixed–fixed condition at top and bottom of pier [40]. The intention is to study the plastic hinge formation and column strength degradation mechanism under nonlinear static and dynamic analyses at pier top, it is necessary to make the end conditions fixed–fixed for bridge pier.

Support conditions for bridge

Different support conditions for bridge have been suggested by Caltrans [62], and Aviram et al. [37] studied the effect of the abutment modelling on the seismic response of bridge structures. Based on this study, roller supports are adopted for this model.

Nonlinear Analysis

Hwang et al. [63] generated fragility curves for Memphis bridge using elastic spectrum method. However, this method is not widely accepted by researchers because of its limitations. This method can be applicable to bridges which will perform within the elastic limit only. So, nonlinear analysis has become essential to study the behaviour of a structural member at micro-level in which the deformations, stresses, and strains exceed their elastic limit and start to behave as nonlinear element. Performing nonlinear analysis used to be time consuming and laborious task earlier, but with rapid development in strong computational tools have made it easier with good accuracy in results. Time consuming and convergence issues have been also resolved with recently developed software packages.

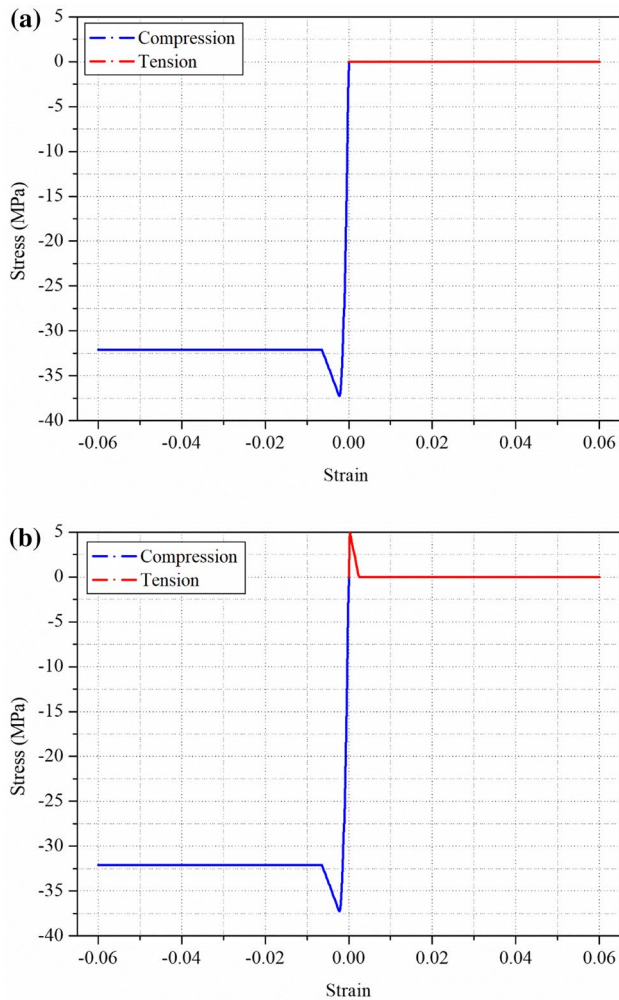


Fig. 7 a Stress–strain curve of Concrete 01 b Stress–strain curve of Concrete 02

Nonlinear Static analysis

Nonlinear static analysis or pushover analysis has been adopted, in which loading is applied monotonically on structure to evaluate the capacity of the structural member. The hinge formation and failure modes of the structure are observed during the application of the load over the structure. The aim of this method is achieved by noticing the yield, ultimate displacements and ultimate ductility values. Pushover analysis can be either load controlled or displacement controlled. In bridges, pushover analysis is displacement control up to a specified a displacement in order to capture the softening behaviour of the structure. Nonlinear static analysis to assess the seismic vulnerability of structures based on ATC 40 [64] capacity spectrum method was limited to buildings earlier. However, [11] and [26] adopted this method to develop fragility curves for bridges. In this method, the capacity of a structural member is determined

using nonlinear static analysis. A scale-down response spectra will be generated which will be super imposed on capacity curve in order to identify the performance levels of the structural member. Currently, pushover analysis on bridges follows the recommendations of Sect. 3 of ATC 32. In this study, nonlinear static analysis on selected bridge pier was performed in longitudinal as well as in transverse direction to obtain yield and ultimate displacements. Yield energy (E_c) and ultimate ductility (μ_u) values can be calculated with yield and ultimate displacement values after performing pushover analysis. These ductility values are needed to calculate damage index (DI).

Nonlinear time history analysis

Nonlinear time history analysis has got much significance and attention from researchers because of its reliability in generating the fragility curves. Seismic risk assessment of structures using nonlinear time history analysis was found to be more efficient and accurate among all the methods as it accounts nonlinearities and strength degradation effects [65]. In this method, structure or structural component is subjected to a ground motion suite that consists of various ground motion intensities. Damage indices are calculated based on the results obtained. Still, there is an argument among the researchers that how many ground motions are exactly required to perform nonlinear time history analysis [1]. A set of fragility curves for various damage states were developed using nonlinear time history for central and south-eastern United States [23]. Fragility curves for Greek bridges, Korean bridges [32] and Italian bridges were developed using this method. Incremental dynamic analysis was also used by some researchers in which a suite of ground motions is scaled using a scale factor to generate a greater number of ground motions of different intensities. This method has its own advantages and disadvantages as some overestimates and underestimates can be possible in results due to scaling of ground motions. Though nonlinear time history is computationally taking more time and convergence issues frequently occur while performing the analyses on structures this method has succeeded in generating more reliable fragility curves. Few researchers [25] have utilized synthetic ground motions of different intensities based on the real earthquake data to construct fragility curves. Few researchers [18] have adopted the real-time earthquake record data to generate the fragility curves. In order to calibrate damage ranks based on damage index HAZUS [56], it is essential to perform both nonlinear static and nonlinear time history analysis. In this study, authors have performed both nonlinear analyses on modelled bridge pier. A set of 100 ground motions from PEER NGA database is available in Opensees-MS Bridge website. They have been scaled and 300 ground motions in longitudinal and transverse directions

were generated in order to attain higher intensity of ground motions. The magnitude of above-mentioned 100 ground motion suites is in the range of 5.8–7.2 and distances 0 to 60 km. Nonlinear time history analysis has been performed on 5.5 m bridge pier, and corresponding hysteretic energy (E_e) for each ground motion has been calculated. Maximum dynamic displacement values for all ground motions were also obtained. These values have been taken to calculate damage indices and damage ranks. Mean and standard deviation values have been calculated and log-normal distribution was assumed to construct fragility curves for each damage rank. A set of 300 ground motions in each direction have been considered in this study. Nonlinear time history is essential to calculate the dynamic ductility (μ_d) and hysteric energy (E_h).

Results and discussion

Modal analysis has been performed initially to determine the fundamental time period and frequency values. First ten modes have been considered, and mode shapes and time period values are shown in Fig. 8 and Table 8, respectively.

Nonlinear static analysis has been performed on the model in longitudinal and transverse directions. As the cross section of the pier is circular, the pushover response in both the directions is same. The bridge pier is subjected to monotonic loading, yield and ultimate displacement values and corresponding shear forces have been identified using pushover curve shown in Fig. 9. The pier has started yielding at 20 mm, and corresponding shear force is 8796.6 kN. The ultimate displacement value was found to be 210 mm, and corresponding shear force was identified as 11,728.8 kN. The yield energy (E_e) is the area of the triangle shown in pushover curve and ultimate ductility (μ_u) value was calculated using Eq. (5). Damage ratios and damage ranks estimated in longitudinal as well as transverse directions are shown in Tables 9, 10, 11 and 12). Using the log-normal distribution (Eq. 2), the obtained fragility curves of circular RC bridge pier for various damage ranks in longitudinal and transverse directions are presented in Figs. 10 and 11. The major findings from these figures are as follows. In longitudinal direction, the probability of exceeding slight, moderate and extensive damage states is 73.9%, 65.2 and 58.5%, respectively. The probability of collapse at 2.5 g is almost 50% in this direction. In transverse direction, the probability of exceeding slight, moderate and extensive damage states is 91.7%, 98.2% and 80.75%, respectively, at PGA 3 g. The probability of collapse is 59.8% in this direction.

The regression equations given by Karim and Yamazaki 2002 [18] to evaluate the median values for different damage states based on the height of the pier have been compared with the present study and shown in Table 13. However,

comparing these two models is not realistic because both the studies were carried out with different input parameters and analyses were carried out on two different platforms. But, for research purpose and to have an idea to the readers, the comparison was made. Slight deviation in these values is due to the fact that the literature models are single degree of freedom (SDOF) system models and simple bilinear steel model was assumed.; whereas in the current study, entire bridge was modelled and pier was modelled by considering plasticity at the ends (beam with hinges model). The strength of this study over previous studies is the effective modelling of pier and incorporating reinforcing steel material model. Moreover, modelling steel as reinforcing steel behaviour where the strain hardening behaviour was taken into account and modelling concrete using widely accepted Mander's model strengthens the accuracy of the present study. In the current seismic evaluation study, post-yielding stiffness has been calculated accurately by considering stress–strain relationship and strain hardening behaviour of steel, whereas in most of the previous studies, it was simply assumed as ten per cent of the yield stiffness. This study has overcome the limitations of previous studies in elemental and material modelling, and accurate results were obtained by performing nonlinear analyses effectively. The probability of exceeding various damage states in longitudinal and transverse directions has been demonstrated clearly based on which the retrofitting strategies can be planned and implemented to avoid the failure of structures.

Conclusions

Seismic assessment of a circular RC bridge pier using analytical method was presented in this study. A three-dimensional bridge modelling was done in Opensees MS bridge software, and the methodology adopted is more rational and consistent. Fragility curves developed for RC bridge pier for different damage states give a physical interpretation failure probability. The following conclusions are drawn from the present study.

- In longitudinal direction, the probability of exceeding slight, moderate and extensive damage states is 73.9%, 65.2 and 58.5%, respectively, at 2.5 g, and for collapse damage state, it is 50%.
- In transverse direction, the probability of exceeding slight, moderate and extensive damage states is 91.7%, 98.2% and 80.75%, respectively, at PGA 3 g. The probability of collapse damage state is 59.8% in this direction.
- The adopted beam with hinges model is efficient in capturing the nonlinear responses of structures over linear elements. Reinforcing steel material model for steel is

Fig. 8 First ten-mode shapes of bridge

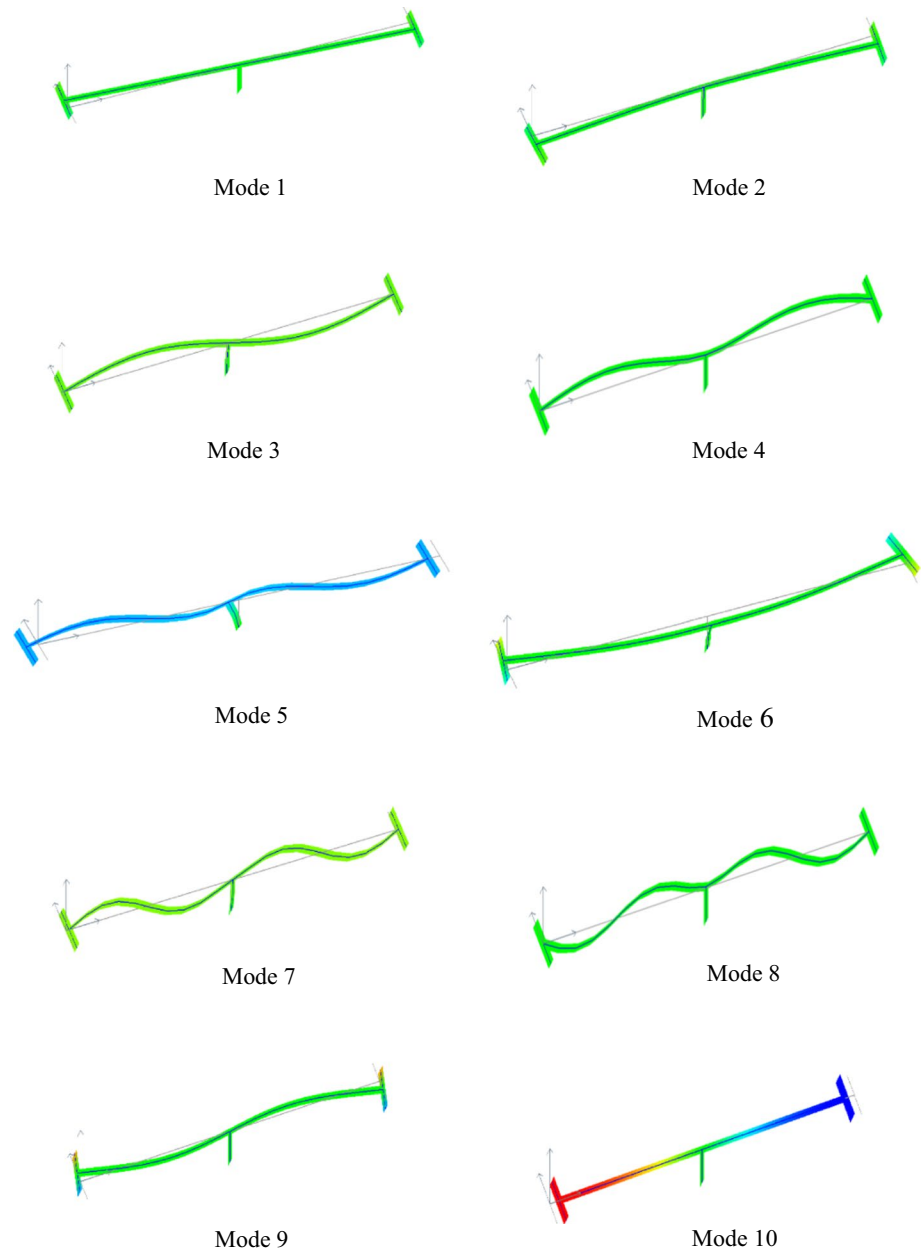


Table 8 Time period frequency of ten modes

Mode	Time period (s)	Frequency (Hz)
1	3.961	0.252
2	0.424	2.353
3	0.416	2.403
4	0.276	3.618
5	0.160	6.243
6	0.145	6.852
7	0.096	10.373
8	0.087	11.404
9	0.086	11.563
10	0.049	20.134

more reliable and convincing over bilinear steel model to predict the nonlinear response of pier.

- The developed fragility curves are useful in determining the potential losses resulting from earthquake shocks and can be used to assign prioritization for retrofitting.

Based on the analytical work carried out in this study, it is recommended to adopt beam with hinges model over linear element models and consider the strain hardening effect of steel while performing nonlinear analyses on structures. Further, it is essential to incorporate analytical fragility curves as seismic evaluation method in existing seismic codes of India. This study can be extended by

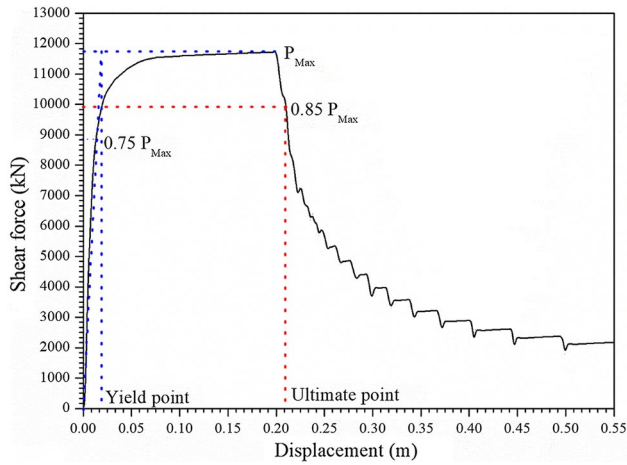


Fig. 9 Pushover curve of 5.5-m bridge pier

Table 9 Damage ranks for different peak ground accelerations (PGAs) in longitudinal direction

PGA in g	D	C	B	A	As
0.5	217	0	0	0	0
1	37	19	0	0	0
1.5	0	16	1	0	0
2	0	1	3	4	0
2.5	0	0	0	1	1

Table 10 Damage ratios for different peak ground accelerations (PGAs) in longitudinal direction

PGA in g	D	C	B	A	As
0.5	1	0	0	0	0
1	0.66	0.34	0	0	0
1.5	0	0.94	0.06	0	0
2	0	0.125	0.375	0.5	0
2.5	0	0	0	0.5	0.5

Table 11 Damage ranks for different peak ground accelerations (PGAs) in transverse direction

PGA in g	D	C	B	A	As
0.5	196	22	0	0	0
1	7	43	10	0	0
1.5	0	3	2	5	1
2	0	0	1	3	2
2.5	0	0	0	0	1
3.0	0	0	0	0	4

Table 12 Damage ratios for different peak ground accelerations (PGAs) in transverse direction

PGA in g	D	C	B	A	As
0.5	0.9	0.1	0	0	0
1	0.117	0.716	0.167	0	0
1.5	0	0.273	0.182	0.455	0.09
2	0	0	0.167	0.5	0.333
2.5	0	0	0	0	1
3.0	0	0	0	0	1

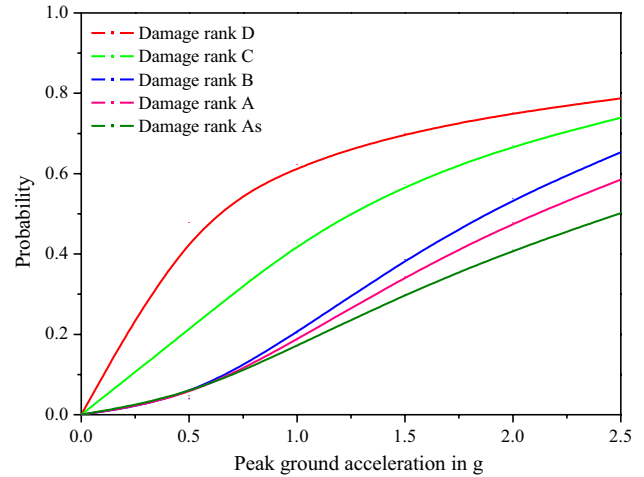


Fig. 10 Fragility curve for 5.5-m bridge pier in longitudinal direction ($g = 9.81 \text{ m/s}^2$)

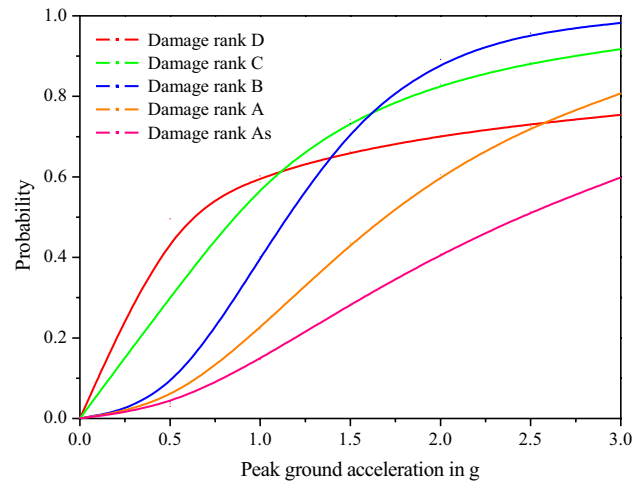


Fig. 11 Fragility curves for 5.5-m bridge pier in transverse direction ($g = 9.81 \text{ m/s}^2$)

Table 13 Mean values comparison

S. No	Damage state	Regression equation	Analytical	Ratio
1	Slight	5.4295	7.11	1.30
2	Moderate	6.1445	7.53	1.22
3	Extensive	6.3445	7.65	1.20
4	Collapse	6.5445	7.82	1.19

considering soil structure interaction effects and multi-support excitation phenomena. Further investigations can be carried out to evaluate the seismic capacity of remaining structural components of bridge like bearings, shear keys etc.

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Declarations

Conflict of interest On behalf of all the authors, the corresponding author states that there is no conflict of interest.

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