Proceedings of the 10th International Conference on Civil Structural and Transportation Engineering (ICCSTE 2025)
July, 2025 | Imperial College London Conference Center, London, United Kingdom
Paper No. 187
DOI: 10.11159/iccste25.187

A Novel Performance-Based Methodology for Seismic Evaluation of Bridges

Kianosh Ashkani Zadeh[1], Carlos Estuardo Ventura[2]
[1]Kia Engineering Inc.
5955 Oakdale Rd., Burnaby, BC, Canada, V5H 4S5
kashkani@kia-engineering.com; ventura@civil.ubc.ca
[2]The University of British Columbia
6250 Applied Science Ln #2002, Vancouver, BC, Canada, V6T 1Z4

Abstract - Seismic hazards pose critical challenges to bridge safety, amplified by inadequate consideration of nonlinear responses and soil-structure interaction (SSI). Existing assessment methodologies often fail to address these complexities, limiting their applicability in ensuring structural resilience. This paper introduces a novel performance-based seismic evaluation (PBSE) framework tailored for bridges, incorporating advanced modeling and probabilistic techniques to quantify collapse risks. The framework emphasizes nonlinear dynamics, detailed SSI effects, and ground motion variability to improve predictive accuracy.

The proposed methodology is applied to the Meloland Road Overcrossing (MRO) in Southern California as a case study. Four archetype models with varying degrees of SSI representation were developed and analyzed using incremental dynamic analysis (IDA) with 22 ground motions. These ranged from simplified SSI assumptions to a detailed model (D_4) that explicitly captures abutment-soil and pile-soil interactions.

Findings demonstrate that SSI representation critically impacts collapse predictions. The detailed D_4 model exhibited increased collapse probabilities and reduced collapse margin ratios compared to simplified models, highlighting the essential role of accurate SSI characterization. Fragility curves illustrated the interplay between ground motion characteristics and SSI effects, revealing their influence on failure sequences and overall structural performance.

This research redefines seismic evaluation by addressing key uncertainties in collapse risk assessments and advancing SSI modeling techniques. The PBSE framework provides engineers with a robust tool for designing resilient bridge structures and optimizing retrofitting strategies. The proposed methodology contributes to the development of safer infrastructure capable of withstanding future seismic events by accounting for ground motion characteristics and addressing various aspects of data and modeling uncertainties.

Keywords**:** performance-based seismic evaluation (PBSE), soil-structure interaction (SSI), collapse fragility curves (CFC), incremental dynamic analysis (IDA), collapse margin ratio (CMR)

1. Introduction

The insufficient consideration of nonlinear structural behavior and soil-structure interaction (SSI) effects has contributed to the unsafe design of bridges, leading to numerous collapses worldwide. Despite these failures over the past decades, existing code provisions lack practical, comprehensive guidelines to assist engineers in conducting reliable collapse assessments for bridge structures.

This paper aims to address this gap by proposing a simplified methodology, inspired by the FEMA P695 framework [1], for evaluating the seismic performance of bridge structures. The proposed approach integrates a performance-based earthquake engineering framework encompassing seismic hazard analysis, advanced structural modeling, and damage assessment. Key factors such as ground motion spectral shape effects and total system collapse uncertainty are rigorously incorporated into the analysis. Additionally, both simulated and non-simulated failure modes are accounted for in the nonlinear modeling to enhance the accuracy of collapse predictions.

The Meloland Road Overcrossing (MRO) in Southern California is used as a case study to demonstrate the application of this methodology, highlighting its potential to provide practical insights for the seismic evaluation and design of resilient bridge structures.

2. Framework for the Proposed Bridge Seismic Performance Evaluation Methodology

This paper presents a methodology for the seismic assessment of bridges, integrating analytical and statistical approaches within a performance-based seismic design framework. The proposed approach is versatile, applicable to both the design of new structures and the retrofitting of existing bridges. It accommodates both discrete and continuum modeling techniques and incorporates the concept of an acceptable probability of collapse ($P_{acceptable}$). Additionally, it considers ground motion characteristics and systematically addresses data and modeling uncertainties. When required, the methodology can also account for Soil-Structure Interaction (SSI) effects.

As illustrated in Fig. 1, the methodology comprises four key steps: development of nonlinear models for collapse assessment, nonlinear time history analyses, seismic performance evaluation, and thorough documentation with peer review [2].

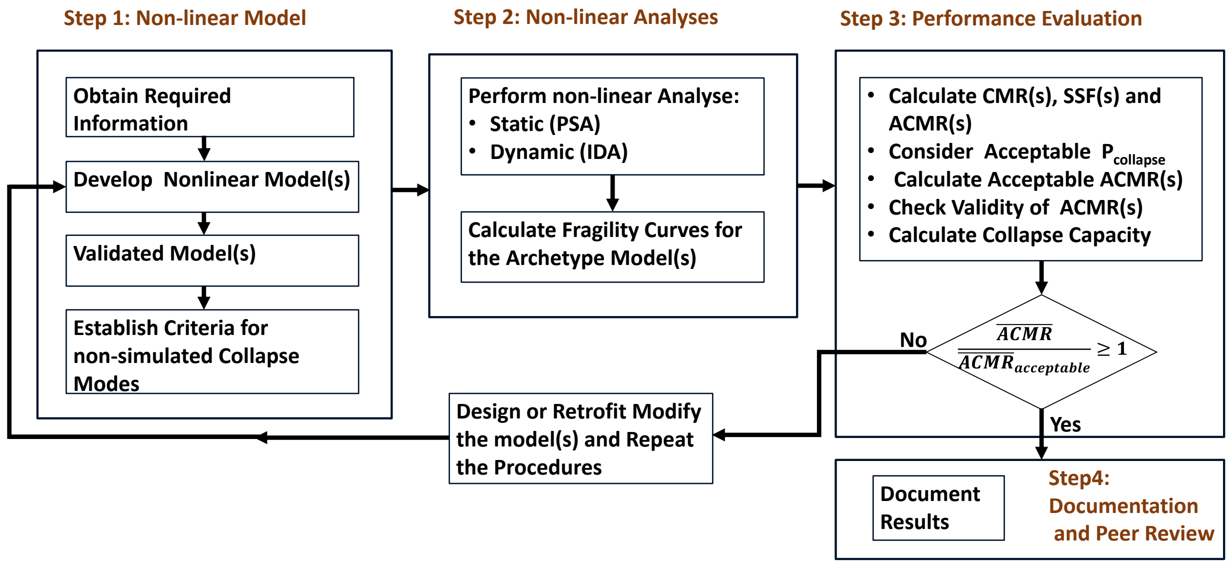
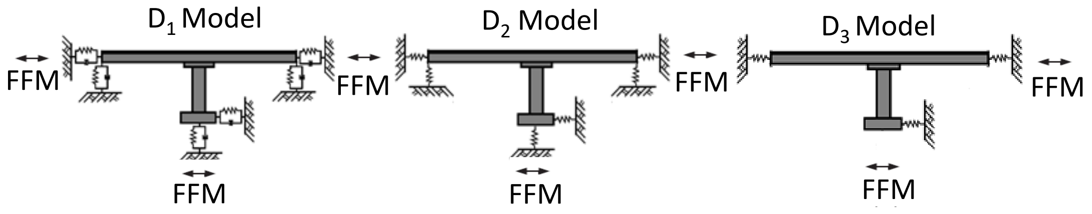


Fig. 1: Proposed bridge seismic Performance evaluation methodology [2].

3. MRO Bridge Nonlinear Models

Four discrete archetype models of the bridge were developed and validated using ambient vibration test results [3]. The strength and displacement capacities of the primary structural components were defined as performance criteria, following the guidelines outlined in the Seismic Retrofitting Manual for Highway Structures, Part 1: Bridges [4]. These models were utilized to conduct Incremental Dynamic Analysis (IDA).

Each archetype model incorporates a distinct level of Soil-Structure Interaction (SSI) representation. Among these, three models—D_1, D_2, and D_3—were adapted from previous studies [5, 6, 7]. A schematic representation of the models and the applied Free Field Motion (FFM) are provided in Fig. 2.

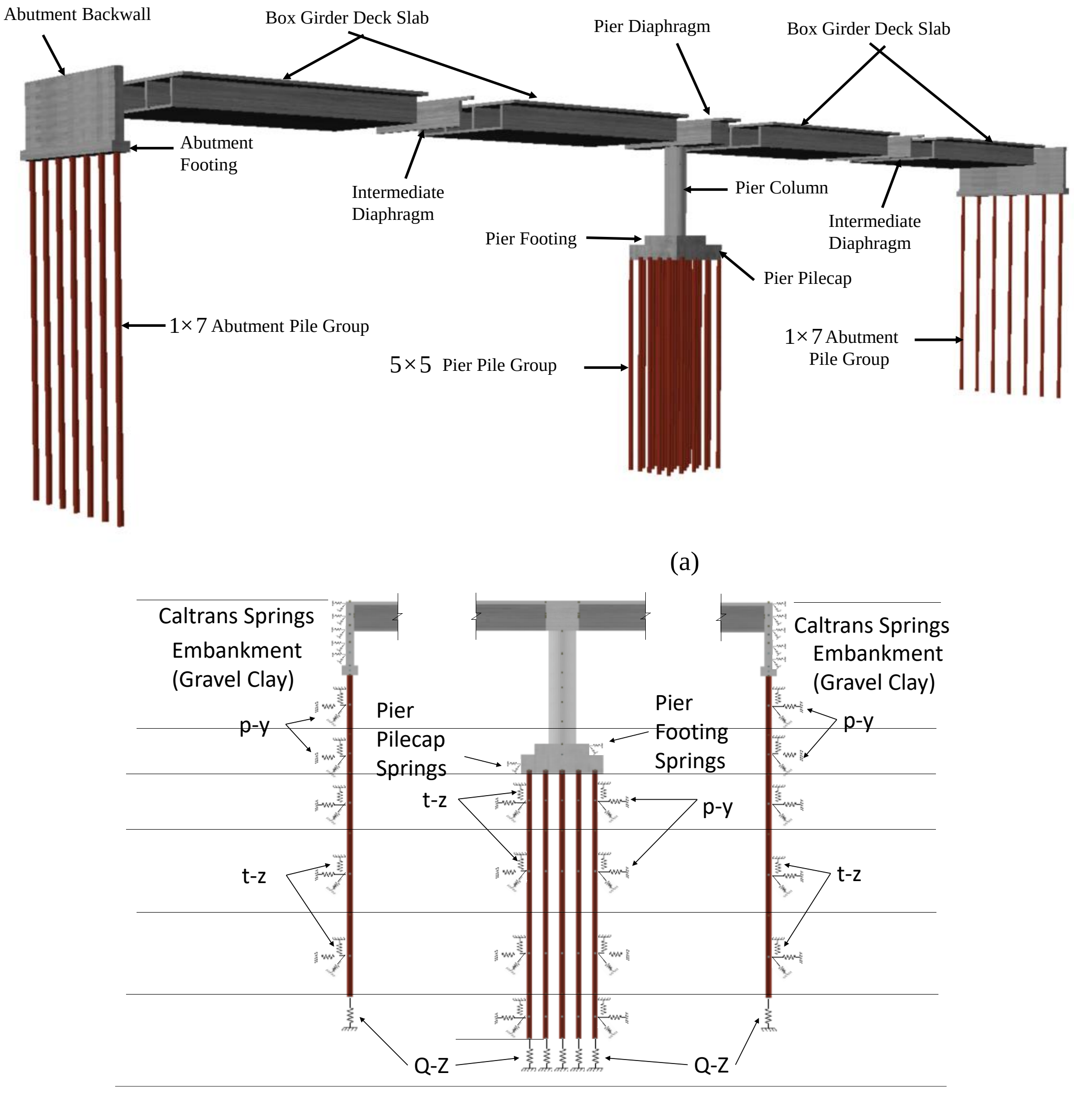


(a) (b) (c)

Fig. 2: MRO models and applied Free Field Motion (FEM): (a) Viscoelastic embankments and center bent [5], (b) and (c) Elastic support at embankments and center bent [6][7].

The archetype model D_4, developed specifically in this study, incorporates a detailed representation of Soil-Structure Interaction (SSI) features within the discrete model framework. This advanced model explicitly includes abutment and pier piles, along with their interactions. The D_4 model accounts for abutment wall-backfill soil interactions, pier foundation-backfill soil interactions, and the lateral and vertical resistance of piles, which are calculated and incorporated into the model [2].

All models, including D_4, were subjected to a suite of 22 ground motions selected from the PEER NGA-West 2 ground motion database [8]. These ground motions, encompassing a range of dynamic characteristics, were chosen to capture hazard uncertainty. Incremental Dynamic Analysis (IDA) was performed for each model to assess their seismic performance under varying levels of intensity.



(a)

(b)

Fig. 3: MRO models: (a) 3D view of the index archetype model D_4 constructed using SeismoStruct software, (b) Soil springs arrangement at pier and abutment piles and embankments in D_4 Model [2].

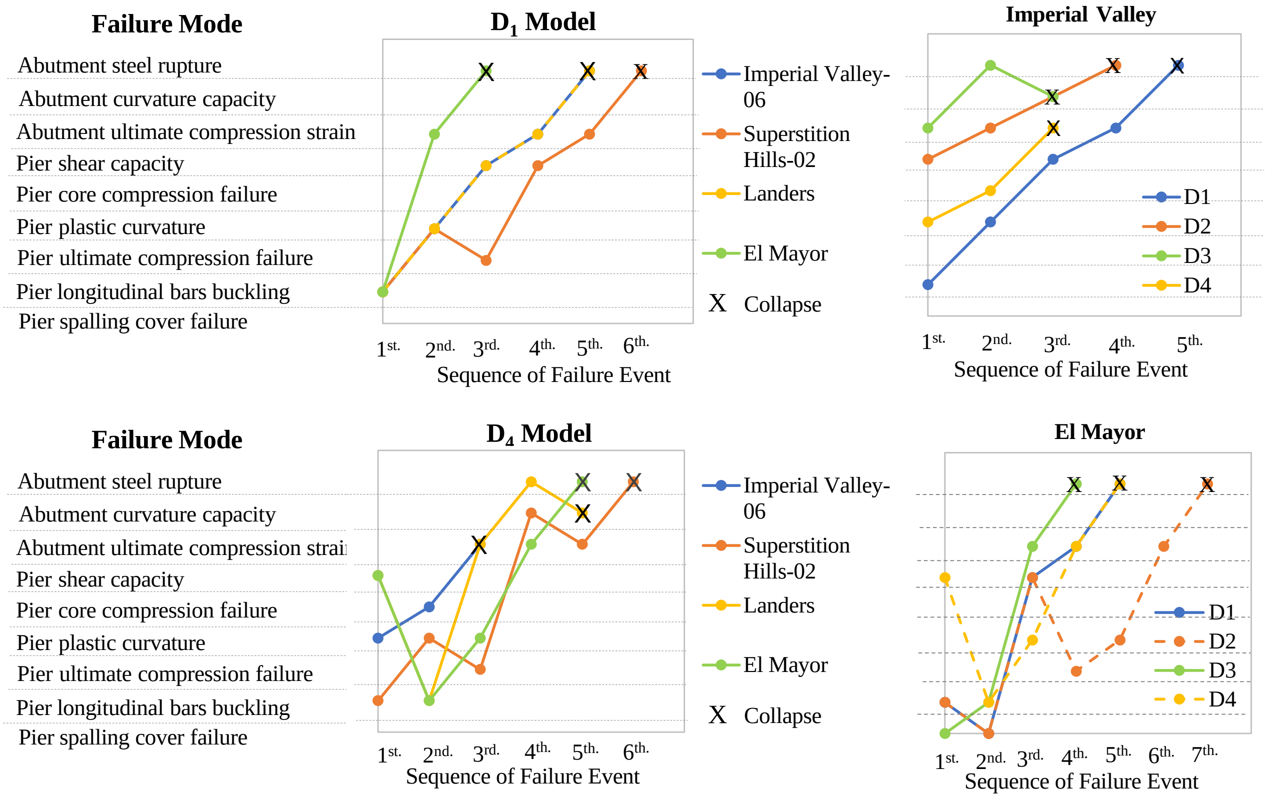
4. Identification and comparison of Structural Failure Modes in Archetype Models

The structural integrity of a bridge cannot be maintained when one or more primary structural components of its shear force-resisting system (SFRS) fail. In this study, collapse is defined as the sequential failure of key structural members, ultimately leading to model instability.

The sequence of failure modes observed in Incremental Dynamic Analyses (IDA) depends on various factors, including ground motion characteristics and model specifics. For each archetype model, failure sequences at the collapse level were extracted from the analysis results. Fig. 4(a) illustrates the failure sequences for models D_1 and D_4 under four selected ground motions. The results reveal an overall similarity in failure progression, which generally begins with the pier's failure and propagates to the abutment.

As shown in Fig. 4, both model details and ground motion characteristics significantly influence the failure sequences and ultimate collapse modes. In certain ground motions, characteristics such as the predominant period and peak ground acceleration (PGA) dominate the structural response, resulting in similar failure sequences and collapse modes across all models. In contrast, in other cases, model details, particularly the level of SSI representation, play a more significant role in governing the response and failure patterns. For example, this influence is clearly observed under the Imperial Valley-06 ground motion, as shown in Fig. 4(b).

Additionally, some earthquake events induce multiple failure modes before the final collapse. This behavior highlights the structure's ability to utilize its ductility and energy absorption capacity, delaying complete collapse and enhancing its resilience. The interplay between ground motion properties and structural details underscores the importance of considering both factors in seismic assessments of bridge structures.



(a) (b)

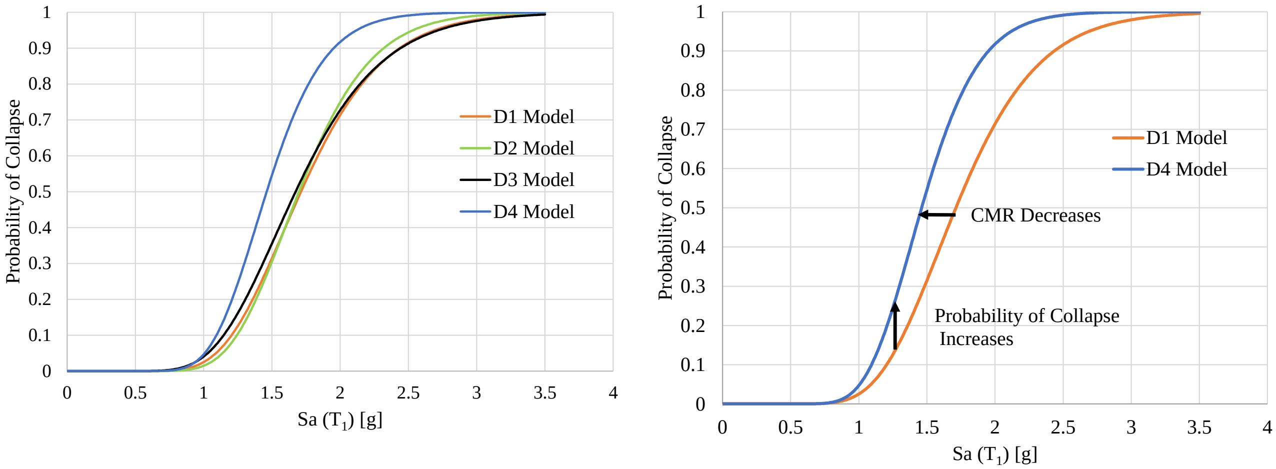
Fig. 4: (a) Failure mode and their sequence of occurrence for the models when subjected to the four earthquake ground motions at the collapse levels corresponding to each model, (b) Failure mode sequence of the models D_1, D_2, D_3, and D_4 due to the collapse level ground motions Imperial Valley and El Mayor [2].

To address the variation in the sequence of failure modes, a statistical approach is necessary for analyzing the data. This requires a detailed definition of the structural models, including the representation of Soil-Structure Interaction (SSI). Furthermore, the analysis must encompass a broad range of ground motions with diverse characteristics to ensure a robust and meaningful statistical evaluation of the results. This approach facilitates the identification of critical trends and dependencies, enhancing the reliability of seismic performance assessments.

4.1. Collapse Fragility Curves (CFCs)

Fragility curves are valuable statistical tools for assessing the probability of reaching or exceeding a specified failure state. In this study, the fragility curves represent the estimated probability of collapse. Using a fragility fitting approach and a MATLAB code developed by Baker [9], the fragility curves for all archetype models were computed as a function of spectral acceleration (Sa), as shown in Fig. 5(a). The results indicate that models D_1, D_2, and D_3 produce similar fragility curves. However, model D_4, which incorporates a comprehensive SSI representation, exhibits a significantly different global collapse fragility curve.

Among the simplified SSI models analyzed, D_1, originally developed by Zhang and Makris [5], is the most feature rich. This model includes a detailed representation of soil effects on the embankment and foundations through springs and dashpots. To illustrate the impact of model details on fragility curves and collapse probabilities, a comparison of D_1 and D_4 models is presented in Fig. 5(b). The comparison highlights the influence of SSI representation on the fragility curves, demonstrating the importance of incorporating detailed SSI features for accurate collapse assessments.



(a) (b)

Fig. 5: (a) Fragility curves for the index archetype models D_1 to D_4 for the MRO Bridge, (b) Fragility curves for the index archetype models D_1 and D_4 for the MRO Bridge and CMR comparison [2].

The following conclusions can be drawn from Fig. 5(b):

1) For a given spectral acceleration (Sa), the probability of collapse is higher for model D_4 compared to model D_1.
2) For a specified probability of collapse, the spectral acceleration associated with model D_4 is lower than that of D_1, resulting in a reduced Collapse Margin Ratio (CMR) for D_4.

These findings indicate that model D_4, which incorporates detailed soil-supporting layers, exhibits an increased probability of collapse and consequently poorer performance. This example underscores the sensitivity of model responses to the choice of SSI features, affecting not only component-level failures but also global collapse predictions. Furthermore, the results highlight the uncertainties associated with model characteristics and emphasize the critical need to evaluate multiple models when assessing bridge collapse, particularly when considering SSI effects.

4.2. Collapse Margin Ratio (CMR)

The Collapse Margin Ratio (CMR) provides an objective metric for assessing structural collapse [1]. The process of calculating the CMR involves the following key tasks:

1) Selecting an adequate number of ground motions to ensure a statistically robust analysis.
2) Performing Incremental Dynamic Analysis (IDA) and generating IDA curves for each model.
3) Developing the Collapse Fragility Curve (CFC) based on the IDA results.
4) Identifying the Maximum Considered Earthquake (MCE) for the relevant soil site class and calculating the CMR.

The tasks and their sequence for determining the Collapse Margin Ratio (CMR) are illustrated in Fig. 6, providing a clear workflow for the analysis process.

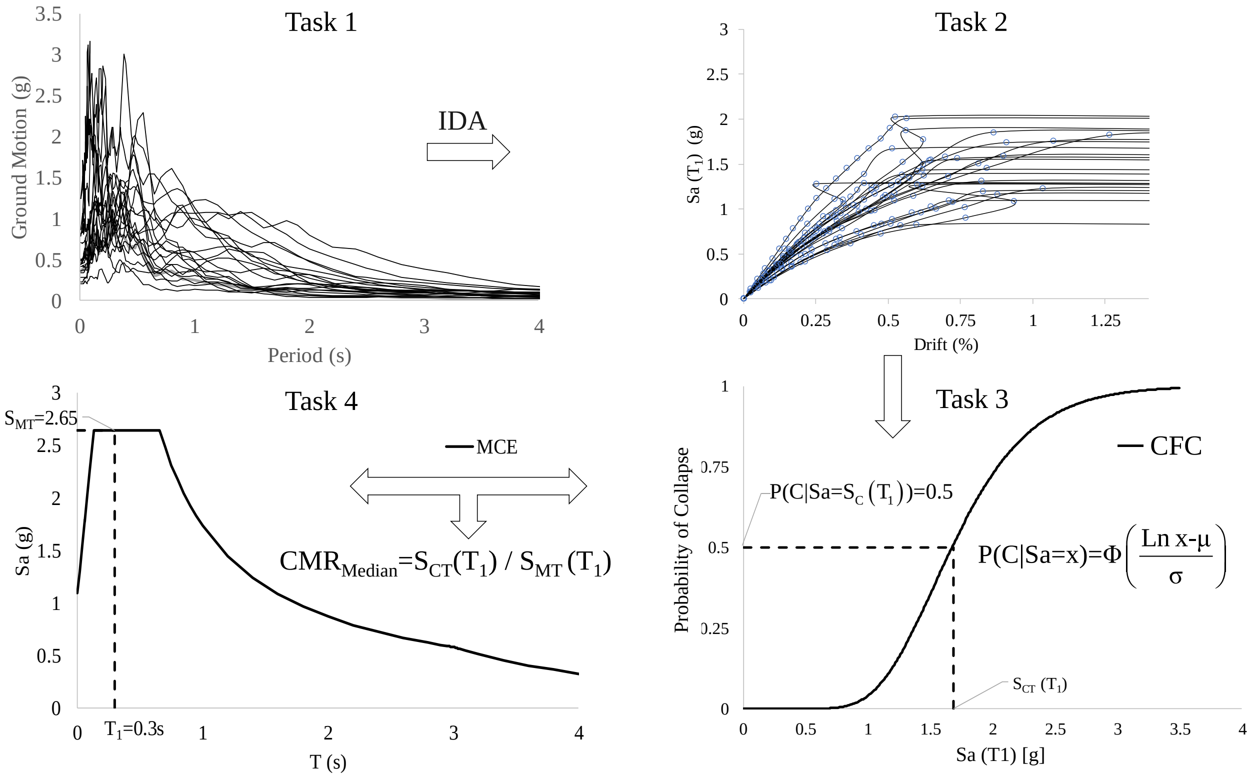


Fig. 6: Procedure for calculating Collapse Margin Ratio (CMR) [2].

4.3. Adjusted Collapse Margin Ratio (ACMR)

The Spectral Shape Factor (SSF) is a parameter used to account for spectral shape effects in seismic assessments. The Adjusted Collapse Margin Ratio (ACMR) is calculated as the product of the SSF parameter and the CMR, as expressed in Eq. (1) [1].

$$ACMR = SSF \times CMR \tag{1}$$

Epsilon (ε) is a measure of the spectral shape of the records. It is defined as the number of standard deviations by which a given ln(Sa) value differs from the mean predicted ln(Sa) value for a given magnitude and distance. This difference is expressed in terms of the number of standard deviations in a logarithmic space as shown in Eq. (2)[10].

Epsilon (ε) is a parameter that quantifies the spectral shape of ground motion records. It is defined as the number of standard deviations by which a given ln(Sa) value differs from the mean predicted ln(Sa) value for a given magnitude and distance. This deviation is measured in logarithmic space and expressed in terms of standard deviations, as shown in Eq. (2) [10].

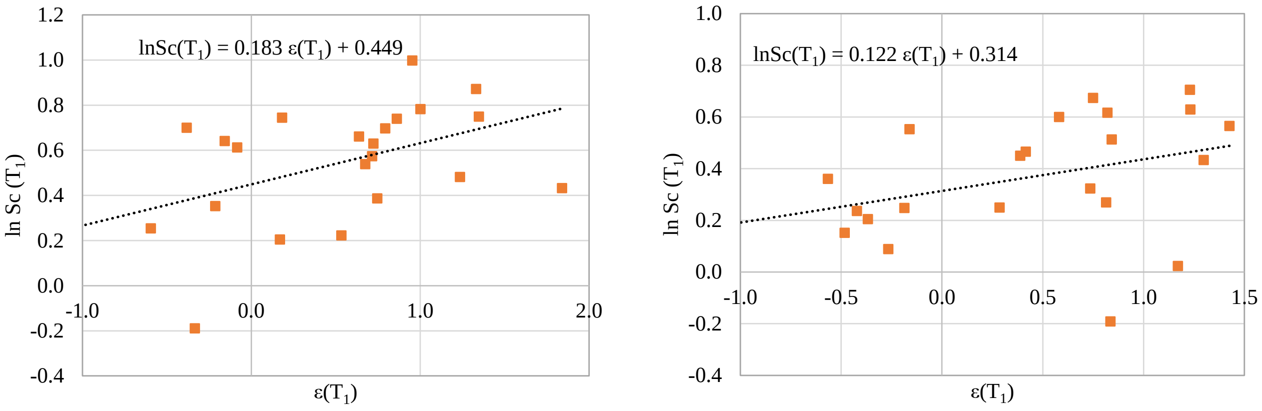
$$\varepsilon(\mathrm{T}) = \frac{\ln(\mathrm{Sa}) - \mu_{\ln\mathrm{Sa}}(\mathrm{M},\mathrm{R},\mathrm{T})}{\sigma_{\ln\mathrm{Sa}}} \tag{2}$$

where, $\mu_{\ln\mathrm{Sa}}$ and $\sigma_{\ln\mathrm{Sa}}$ are mean and standard deviation of ln(Sa) and are calculated using one or more ground motion attenuation equations.

In an IDA analysis, the selected ground motions leading to failure are inherently different from the Maximum Considerable Earthquake (MCE). As a result, the response spectrum of these motions has a different ε parameter compared to the MCE. To account for this discrepancy, the Spectral Shape Factor (SSF) parameter, as defined in Eq. (3), is calculated following the recommendations in FEMA P695. To take this difference into account, the SSF parameter shown in Eq. (3) is calculated as suggested in FEMA P695 [1].

$$\mathrm{SSF} = \exp\left[\beta_1\left(\bar{\varepsilon}_0(\mathrm{T}_1) - \bar{\varepsilon}(\mathrm{T}_1)_{\mathrm{records}}\right)\right] \tag{3}$$

where, $\bar{\varepsilon}_0(\mathrm{T}_1)$ is the expected or target epsilon value for the site and hazard-level of interest obtained from the deaggregation of the seismic hazard of the site. $\bar{\varepsilon}(\mathrm{T}_1)_{\mathrm{records}}$ is the mean epsilon value of the ground motion set, evaluated at period, T_1. The β_1 parameter is the sensitivity of collapse-level spectral acceleration to variation of epsilon of ground motions as shown in Fig. 7 [1][2].



(a) (b)

Fig. 7: β_1 shown as the slope of the fitted line (a) for the model D_1 ,(b) for the model D_4 [2].

4.4. Acceptable Collapse Margin Ratio ($\mathbf{ACMR_{acceptable}}$)

To evaluate the performance of the archetype models, the adjusted collapse margin ratio (ACMR) is compared with an acceptable threshold of adjusted collapse margin ratio ($\mathrm{ACMR_{acceptable}}$). The $\mathrm{ACMR_{acceptable}}$ is calculated considering a given probability of collapse when the model is subjected to MCE-level ground motions. $\mathrm{ACMR_{acceptable}}$ is calculated using Eq. (4) [2][11][12].

To assess the performance of the archetype models, the Adjusted Collapse Margin Ratio (ACMR) is compared against an acceptable threshold, denoted as $\mathrm{ACMR_{acceptable}}$. The $\mathrm{ACMR_{acceptable}}$ is determined based on a specified probability of collapse when the model is subjected to MCE-level ground motions. The calculation of $\mathrm{ACMR_{acceptable}}$ follows the procedure outlined in Eq. (4) [2][11][12].

$$\mathrm{ACMR_{acceptable}} = \frac{\mathrm{SSF}}{\exp\left(\beta_{\mathrm{TOT}} \times \Phi^{-1}\left(\mathrm{P^{C}_{acceptable}}\right)\right)} \tag{4}$$

where, Φ^{-1} is the inverse cumulative normal distribution function, $\mathrm{P^{c}_{acceptable}}$ is the acceptable probability of collapse, SSF is given by Eq. (3), and β_{TOT} represents system uncertainty in predicting the collapse capacity of the structure. Based on FEMA P695, the total system collapse uncertainty can be calculated as per Eq. (5) [1].

$$\beta_{TOT} = \sqrt{\beta_{RTR}{}^{2} + \beta_{DR}{}^{2} + \beta_{TD}{}^{2} + \beta_{MDL}{}^{2}} \tag{5}$$

where, β_{RTR} is the record-to-record collapse uncertainty (0.20 – 0.40), β_{DR} is the design requirements-related collapse uncertainty (0.10 – 0.50), β_{TD} is the test data-related collapse uncertainty (0.10 – 0.50), and β_{MDL} is the modelling-related collapse uncertainty (0.10 – 0.50).

FEMA P695 provides a simplified method for estimating the total uncertainty (β_{TOT}) in predicting collapse capacity. Table 1 lists β_{TOT} values for superior model quality and index archetype models with a period-based ductility (μ_{T}) of $\mu_{\mathrm{T}} \geq 3$. However, the selection of β_{TOT} introduces an additional layer of uncertainty, as it involves judgmental decisions.

The accuracy of a performance evaluation using the proposed method is highly sensitive to the assumed β_{TOT} value. Therefore, careful consideration is required when selecting this parameter during the evaluation process. This can be achieved by investigating uncertainties related to record-to-record variability, test data, and modeling requirements. Additionally, conducting a sensitivity analysis before the performance evaluation is crucial to ensure robust and reliable outcomes.

Table 1: Proposed total system collapse uncertainty (β_{TOT}) based on quality of model and design for the period-based ductility, $\mu_T \geq 3$ [1].

Quality of Test Data	Quality of Design Requirements			
	(A) Superior	**(B) Good**	**(C) Fair**	**(D) Poor**
(A) Superior	0.425	0.475	0.550	0.650
(B) Good	0.475	0.500	0.575	0.675
(C) Fair	0.550	0.575	0.650	0.725
(D) Poor	0.650	0.675	0.725	0.825

In this study a total collapse system uncertainty 0.475 is adopted to calculate $ACMR_{acceptable}$ [2].

5. Evaluating the Seismic Resilience of MRO Bridge

To achieve an acceptable performance, the following two criteria need to be satisfied [1][2]:

1) The average value of adjusted collapse margin ratio for each performance group exceeds $\overline{ACMR}_{10\%}$ ($\overline{ACMR} \geq \overline{ACMR}_{10\%}$)
2) Individual values of adjusted collapse margin ratio for each index archetype model ($ACMR_j$) within a performance group exceeds $ACMR_{20\%}$ ($ACMR_j \geq ACMR_{20\%}$)

Table 2: ACMR, $ACMR_{aceptable}$ and their ratio (ACMR/ $ACMR_{aceptable}$) corresponding to MCE level (2% in 50 years), $Sa(T_1)$ MCE=2.65g [4].

Index Archetype Model	SSF	ACMR	Acceptable Probability of Collapse			
			$P^C_{acceptable} = 10\%$		$P^C_{acceptable} = 20\%$	
			$ACMR_{acceptable}$	**Ratio**	**$ACMR_{acceptable}$**	**Ratio**
D_1	1.33	1.04	0.95	1.10	0.86	1.21(Y)
D_2	1.35	1.04	0.96	1.08	0.87	1.19(Y)
D_3	1.38	1.04	1.00	1.04	0.90	1.16(Y)
D_4	1.22	0.81	0.98	0.83	0.89	0.92(N)
Average	1.32	0.98	0.97	1.01(Y)	0.88	1.12(Y)
Note	(Y) : Methodology requirement is fulfilled (N) : Methodology requirement is NOT fulfilled					

As presented in Table 2, the average value of the adjusted collapse margin ratio for the performance group ($\overline{ACMR}$) exceeds $\overline{ACMR}_{10\%}$. However, ACMR for D_4 model was less than its corresponding $ACMR_{20\%}$ acceptable value. As a result, the D_4 model lacks sufficient collapse resistance, highlighting its vulnerability under seismic loading conditions.

6. Conclusions

A FEMA-based methodology has been developed for the seismic assessment of bridges using a performance-based seismic design framework. This methodology relies on comparing the calculated Adjusted Collapse Margin Ratio (ACMR) for each model with its corresponding acceptable threshold value ($ACMR_{acceptable}$).

The proposed methodology applies to both the design of new bridges and the retrofitting of existing structures. It accommodates discrete and continuum modelling approaches and incorporates an acceptable probability of collapse ($P_{acceptable}$), ground motion spectral shape effects, and total collapse system uncertainty, ultimately enhancing the resilience of infrastructure against future seismic events.

To enhance the understanding of a structure's performance, it is recommended to utilize a set of models that account for modeling uncertainties. However, a single model may be employed if it adequately represents the key features of the structure-soil system and potential failure modes.

The Meloland Road Overcrossing (MRO) was used as a case study to demonstrate the workflow of the proposed methodology. The study highlights the significant role of Soil-Structure Interaction (SSI) in both component-level and global structural collapse predictions. It also emphasizes the uncertainties associated with SSI representation and underscores the importance of considering various archetypes in the collapse assessment of bridges. The results of this study indicate that the D_4 model does not meet the performance requirements of the proposed methodology, suggesting that retrofitting the bridge is necessary to improve its seismic resilience.

Acknowledgements

The authors express their heartfelt gratitude to Dr. Liam Finn for his invaluable feedback and guidance, which played a pivotal role in the success of this research.

References

[1] FEMA-P695, *Quantification of Building Seismic Performance Factors*, Washington, DC: Federal Emergency Management Agency, 2009.

[2] K. Ashkani Zadeh, “Effect of soil-structure interaction on performance-based design of integral abutment bridges,” Ph.D. dissertation, Dept. Civil Eng., The Univ. of British Columbia, Vancouver, Canada, 2020.

[3] C. E. Ventura, J. C. Carvajal, L. Finn, and J. Traber, “Ambient vibration testing of the Meloland Road Overpass,” in *Proc. 4th Int. Oper. Modal Anal. Conf. (IOMAC 2011)*, Istanbul, Turkey, 2011.

[4] Federal Highway Administration, “Seismic Retrofitting Manual for Highway Structures: Part 1 - Bridges,” in *Report No. FHWA-HRT-06-032*, U.S. Department of Transportation, McLean, VA, 2006.

[5] J. Zhang and N. Makris, “Kinematic response functions and dynamic stiffness of bridge embankments,” *Earthq. Eng. Struct. Dyn.*, vol. 31, no. 11, pp. 1933–1966, 2002.

[6] B. M. Douglas, E. Maragakis, and S. Vrontinos, “Parameter identification studies of the Meloland Road Overcrossing,” in *Proc. Pacific Conf. Earthquake Eng.*, Wellington, New Zealand, 1991.

[7] Caltrans, *Bridge Design Aids 14-1*, Sacramento, CA: California Department of Transportation, 1989.

[8] Pacific Earthquake Engineering Research Center, “NGA-West2 Database,” Univ. of California, Berkeley, CA, 2019. [Online]. Available: https://peer.berkeley.edu/research/nga-west-2

[9] J. W. Baker, “Efficient analytical fragility function fitting using dynamic structural analysis,” *Earthquake Spectra*, vol. 31, no. 1, pp. 579–599, 2015.

[10] J. W. Baker, “Conditional mean spectrum: Tool for ground-motion selection,” *J. Struct. Eng.*, vol. 137, no. 3, pp. 322–331, 2011.

[11] P. Tehrani and D. Mitchell, “Seismic performance assessment of bridges in Montreal using incremental dynamic analysis,” in *Proc. 15th World Conf. Earthquake Eng. (15WCEE)*, Lisbon, Portugal, 2012.

[12] P. Tehrani, “Seismic behavior and analysis of continuous reinforced concrete bridges,” Ph.D. dissertation, Dept. Civil Eng., McGill Univ., Montreal, Canada, 2012.