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COLLAPSE ASSESSMENT OF RC BRIDGES CONSIDERING SOIL STRUCTURE INTERACTION (SSI)

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Abstract

In a performance-based analysis and design framework, it is critical to evaluate the performance of the primary structural components and assess failure and collapse of the structure. Studies have shown that Soil-Structure Interaction (SSI) has a significant effect on response of bridge structures, especially if founded on soft soil.

In this paper, effect of soil-structure interaction on the performance of RC bridges is studied using a performance-based approach. In addition, a procedure is proposed for evaluation of seismic performance of the RC bridges.

To benefit from the existing data and past records, Meloland Road Overcrossing (MRO), located in Southern California, is chosen as a case study bridge to develop the index archetype models. MRO was constructed in 1969 and has experienced multiple earthquake events. This bridge has been heavily instrumented and has been the subject of many studies.

Various 3D discrete models simulating dynamic characteristics of MRO are developed using SeismoStruct FE software. In these archetype models, both geometrical and material nonlinearity are simulated while SSI features are addressed to investigate SSI impacts in their responses. Dynamic characteristics of all developed numerical models are verified by comparison with field measurement data from previous studies.

Incremental Dynamic Analysis (IDA) is performed using a selected set of ground motions with probability of exceedance of 2% in 50 years hazard level (return period of 2475 years). The NGA-West2 ground motion database is used in the development of fragility curves.

Strength and displacement capacities of bridge members are defined in the index archetype models as performance criteria using the guidelines provided in TRB's Seismic Retrofitting manual for the highway structures part 1-Bridges.

Collapse capacity of the archetype models are compared at the onset of collapse and collapse levels. It is shown that SSI has a significant impact on response of bridge structures subjected to strong earthquakes. SSI features lead to changes in structural response which subsequently results in changes in failure modes.

Ultimately, an approach inspired by FEMA-P695 is proposed to quantify seismic performance and collapse capacity of the RC bridge structures using Adjusted Collapse Margin Ratio (ACMR) and acceptable Adjusted Collapse Margin Ratio ($ACMR_{acceptable}$) as the collapse safety measure. The proposed procedure can be employed in seismic performance evaluation of both new and existing bridge structures. The proposed procedure is applied to assess the performance of MRO.

Keywords: Performance-based Approach, Soil-Structure Interaction (SSI), Incremental Dynamic Analysis (IDA), Fragility Curves, Bridge Seismic Performance Evaluation and Collapse Capacity.

1. Introduction and Methodology

Strong earthquake shaking has resulted in collapse of several pile-supported bridges worldwide. Lack of understanding and consideration of the effect of SSI is among the major reasons behind these devastating collapses [1]. One of the famous examples where SSI had a major contribution to bridge failure is the collapse of the Cypress Structure in Oakland during the Loma Prieta earthquake in 1989. The loose sand that the structure was built on, contributed to a more severe response of the structure which ultimately resulted in structural collapse of many sections of this bridge [2]. During the Northridge earthquake in 1994, several bridge piers were damaged due to soil-pile-bridge seismic interaction [1, 3].

Despite of the collapse of the bridge structures in the past few decades, there is not a comprehensive study on effect of SSI in changing failure mode of the main structural components of bridges. In addition, there is lack of a practical procedure and guideline in the code provisions that can assist engineers performing collapse assessment of the bridge structures considering soil-structure interaction. In this research, four 3-D finite element models of Meloland Overcrossing Road (MRO) are developed in SeismoStruct software to study soil-structure effect in response of the structure. These numerical models are developed considering different discrete type soil-structure interaction features and indexed as D_1, D_2, D_3, and D_4 archetype model. IDA was performed on all the archetype models using 22 ground motions selected from PEER NGA West 2 database. Outcome of the IDA simulations were studied and compared to investigate the effects of structural system details as well as ground motion characteristics. A sequence of failure modes in different models is also studied. Collapse fragility curves are calculated based on the results of the performed IDA analyses.

Although a simplified approach for performance evaluation of building structures was commissioned by the Federal Emergency Management Agency (FEMA) under the ATC-63 Project, there is a lack of similar guidelines for evaluation of seismic performance of bridges. To address this issue a simplified process, similar to FEMA P695 methodology, for performance evaluation of the bridge structures is required. A FEMA based methodology is introduced for performance evaluation of bridge structures considering SSI where MRO is used as a case study. The purpose of this methodology is to provide a rational basis for determining bridge system performance. When properly implemented in the seismic design or considered in retrofit of seismic-force-resisting systems in existing bridges, this guideline can result in safety against collapse demanded by current seismic codes for bridges with different seismic-force-resisting systems. The proposed procedure also can be used for performance evaluation in the design stage of new bridge structures. In the proposed methodology, the concepts of Collapse Margin Ratio (CMR) and Adjusted Collapse Margin Ratio (ACMR) are employed for development of performance assessment procedure. A workflow chart is developed to assist engineers in implementing the proposed procedure. Finally, the MRO simulations are used as a case study to demonstrate the proposed procedure.

This article is created based on doctoral dissertation of the first author. Readers are encouraged to refer to [4] for further details on modeling approach and results.

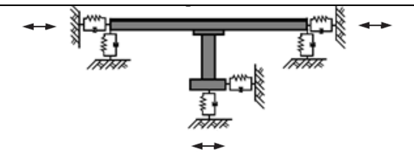
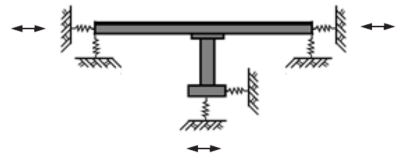
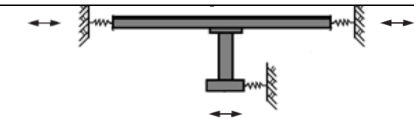
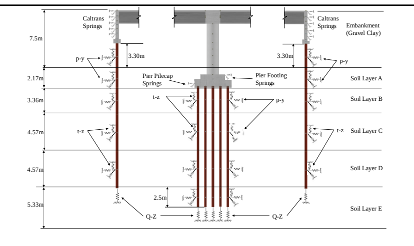
2. Archetype Models

2.1 Model Description

Four archetype models are constructed to simulate the Meloland Road Overcrossing. These 3D archetype models were built and analyzed using SeismoStruct software. Archetype Models D_1 to D_3 are constructed based on previous works reported in the literature. Archetype model D_4 is a more comprehensive model, with a detailed representation of piles and soil supporting the piles. Table 1 provides a brief description of the archetype models.

In the index archetype model D_4, abutment and pier piles are represented in detail. In this archetype model, presence of the abutment embankment (or lateral response of abutment systems) is considered using a numerical simulation model proposed by [5]. In this archetype model, lateral soil resistance around the abutment and pier piles are considered using lateral pile-soil support curves (p-y curves) of the API provision [6] and manual of the computer program Ensoft LPile [7].

Table 1 – Description of the archetype models used in this study

Archetype Model	Schematic	Brief Description
D_1		Developed based on Zhang and Makris, 2002. A viscoelastic soil model consisting of discrete springs and dampers represent the effect of soil supporting embankment and peer foundation in longitudinal and vertical directions [8].
D_2		Developed based on Douglas, Maragakis and Vrontinos, 1991 An elastic model using discrete springs represent effect of soil supporting embankment and peer foundation in longitudinal and vertical directions [8].
D_3		Developed based on Caltrans Method A, 1989. An elastic model using discrete springs represent effect of soil supporting embankment and peer foundation only in longitudinal direction [8].
D_4		This model offers a more detailed SSI representation, including explicit modelling of piles and supporting soil based on MRO bore hole logs [4].

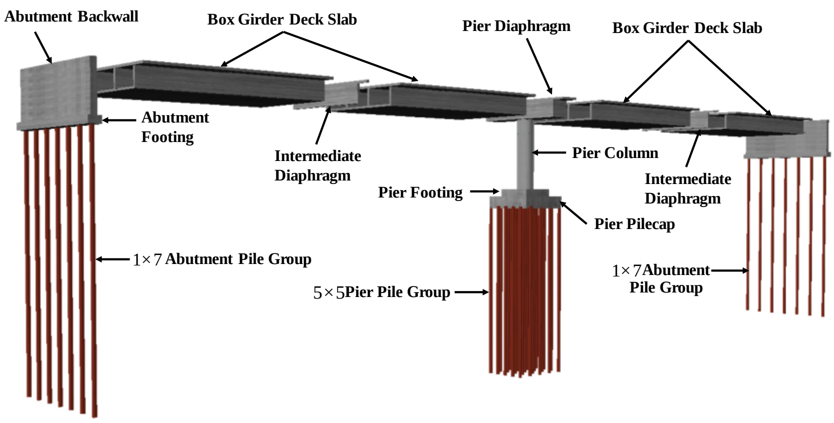


Fig. 1 – 3D view of the index archetype model D_4 constructed using SeismoStruct software [4].

To incorporate the effect of soil surrounding the piles and determine the corresponding vertical and lateral stiffnesses, a detailed study of soil layers is carried out. Idealized soil layers for the bridge and their material properties are adopted identical to the soil layers adopted by Kwon and Elnashai [9] in their works as shown in Fig. 2.

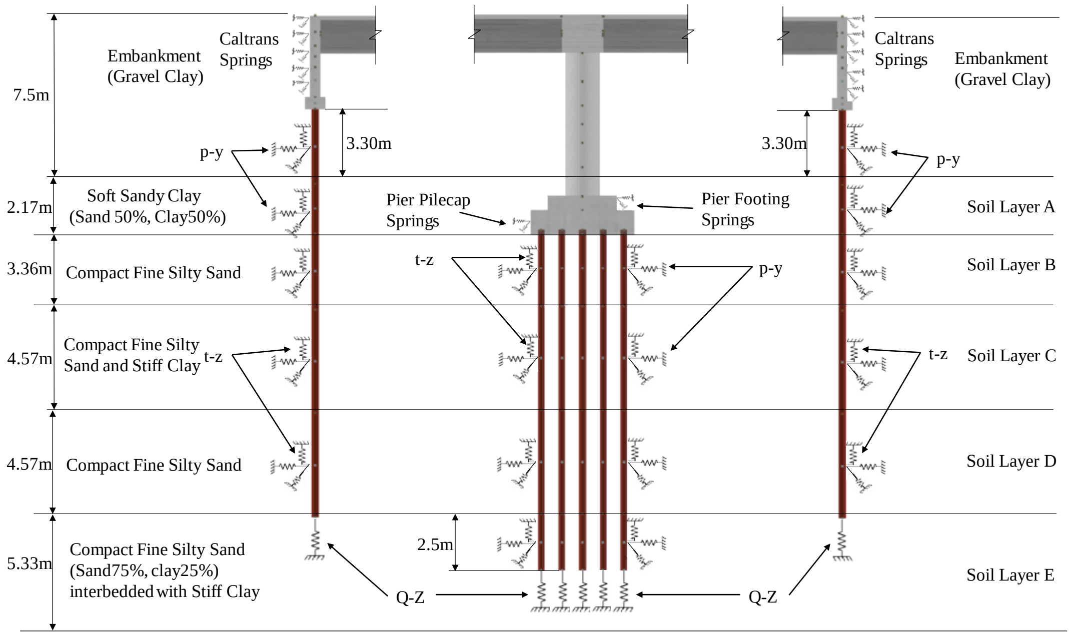


Fig. 2 – Caltrans springs and API springs arrangement are considered in the archetype model D_4 to simulate Abutment-backfill soil interaction and soil-pile interaction, respectively [4].

In all archetype models, the nonlinear concrete model proposed by Mander and co-workers [10] is used as the material model for the pier columns and abutment backwalls in the archetype models. The confinement effects provided by the lateral transverse reinforcement are incorporated through the rules proposed by Mander and co-workers [10] in this constitutive model. In addition, confining pressure is assumed constant throughout the entire stress-strain range. Material properties for the pier column and abutment backwalls are considered identical with the material properties were considered by Werner and co-workers [11] in section property computations. Menegotto-Pinto steel model is considered as a material model for the steel reinforcement. Ten model calibrating parameters are defined in order to fully describe the mechanical characteristics of the material [12].

2.2 Damping

Modal damping ratio for the first transvers mode is identified 18.7% by Zhang and Makris [8] and between 19%-26% by Werner [13]. A Rayleigh damping with a damping ratio of 4% as suggested by Kwon and Elnashai [9] is applied to structural components on modes 1 and 10 as global damping. In addition, a damping ration of 25% is applied to abutment backwalls, foundations and piles to capture the damping effect of embankment and surrounding soil [4].

2.3 Performance Criteria

Using the guidelines provided by the seismic retrofitting manual for the highway structures-bridges part 1 [14], strength and displacement capacity of bridge members for various limit states are calculated [4].

3. Analysis and Discussion

3.1 Eigen Value Analysis

Eigenvalue analysis is performed on the archetype models to compare the fundamental periods of the structure and their corresponding mode shapes with measured ambient test data reported by Ventura and co-workers [15] and validate the model response in elastic regime.

3.2 Hazard Analysis

NGA-West 2 ground motion database provided by the Pacific Earthquake Engineering Research Center (PEER) is employed in this study to select the ground motions. This database includes a comprehensive set of globally recorded shallow crustal earthquakes in active tectonic regimes. Each earthquake in the database contains a set of metadata including various site characterizations, different distance measures, and earthquake source data [16]. In this research, a set of crustal ground motions is chosen as only crustal ground motions affect the site. A set of 22 ground motion is selected using the PEER NGA-West 2 ground motion database [16] considering different magnitude, peak ground acceleration, predominant period and mechanism.

3.3 IDA Analysis

Nonlinear dynamic analysis is performed using earthquake record multiplied by a scale factor. The scale factor is gradually increased until structural collapse occurs. This procedure is repeated for all the ground motions. In this study scale factors started at a value of 0.2 with a 0.2 step increase until collapse. As discussed in Section 3.2, a set of 22 ground motions are employed in the IDA analysis of archetype models. Analysis is performed using a significant duration corresponding to 5% to 95% Arias Intensity (I_A) of each ground motion. For each pair of archetype model and ground motion, the collapse-level spectral acceleration (Sa) values are calculated and IDA curves are graphed. These curves are used to develop the fragility curves discussed in Section 3.4. The IDA analysis also provides important information about failure modes and sequence of failure [4].

3.4 Fragility Curves

Fragility curves are calculated for the four archetype models using the log-normal distribution model proposed by Baker [17]. In this study, fragility curves are calculated based on the results of the incremental dynamic analysis. As Ibarra and Krawinkler [18] suggested, fragility function parameters can be estimated from this data by taking logarithms of each ground motion's Intensity Measure (IM) value associated with onset of collapse and computing their mean and standard deviation as per Eq. (1) and Eq. (2) [17].

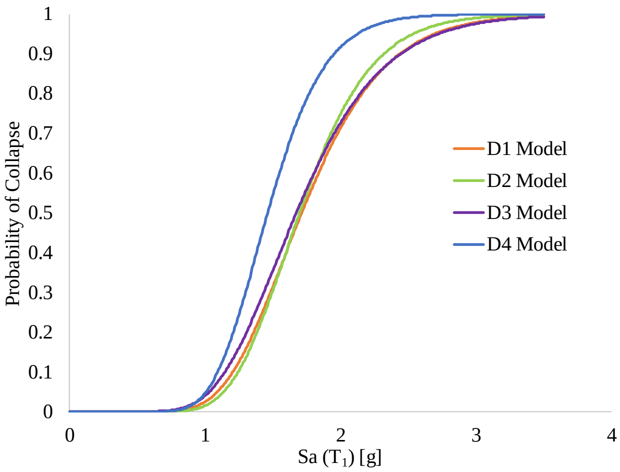
$$\ln \hat{\theta} = \frac{1}{n}\sum_{i=1}^{n} \ln \mathrm{IM}_i \tag{1}$$

$$\hat{\beta} = \sqrt{\frac{1}{n-1}\sum_{i=1}^{n}\left(\ln\left(\mathrm{IM}_i/\hat{\theta}\right)\right)^2} \tag{2}$$

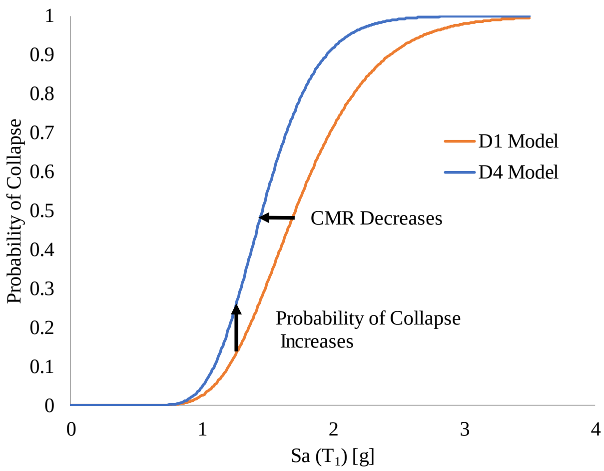
where n is the number of ground motions considered, and IM_i is the IM value (Sa in this case) associated with the onset of collapse for the $i^{th.}$ ground motion. This is a method of moments estimator, as $\ln\hat{\theta}$ and $\hat{\beta}$ are the mean and standard deviation of the normal distribution representing the ln IM values, respectively.

Fig. 3 shows the fragility curves calculated for the four archetype models D_1 to D_4. The simplified models (D_1, D_2, and D_3) show a relatively similar behaviour whereas the more detailed model, D_4, exhibits a higher probability of collapse for a given Sa value. Collapse Margin Ratio (CMR) is defined as ratio of corresponding displacement of the above defined spectral accelerations as per Eq. (3) [19]. where, $\hat{S}_{CT}$ is the median spectral acceleration of the collapse level ground motions and S_{MT} is the 5%-damped spectral acceleration of the MCE ground motions.

$$\mathrm{CMR} = \frac{\hat{S}_{CT}}{S_{MT}} = \frac{SD_{CT}}{SD_{MT}} \tag{3}$$



(a)



(b)

Fig. 3 – (a) Fragility curves for the index archetype models D_1 to D_4 for Meloland Road Overpassing (MRO) (b) Shows reduction of the Collapse Margin Ratio (CMR) and increase of probability of collapse in SSI archetype model D_4 comparing to the archetype model D_1

To incorporate the spectral shape effects discussed by Baker and Cornell [20] and presented in FEMA P695, Adjusted Collapse Margin Ratio (ACMR) is defined as the product of CMR by Spectral Shape Factor (SSF) parameter.

$$\mathrm{ACMR} = \mathrm{SSF} \times \mathrm{CMR} \tag{4}$$

Spectral Shape Factor, SSF, which depends on the fundamental period, T_1, and site class category and hazard level of interest can be obtained as per Eq. (5) [19].

$$\mathrm{SSF} = \exp\left[\beta_1\left(\bar{\varepsilon}_0(T_1) - \bar{\varepsilon}(T_1)_{records}\right)\right] \tag{5}$$

where, $\bar{\varepsilon}_0(T_1)$ is the expected or target value for the site and hazard-level of interest obtained by seismic deaggregation. $\bar{\varepsilon}(T_1)_{records}$ is the mean epsilon value of the Far-Field ground motion set, evaluated at period, T_1. The epsilon value, $\varepsilon(T)$, is the number of standard deviations by which a given $\ln(Sa)$ value differs from its mean predicted value obtained from an attenuation function , $\mu_{\ln Sa}(M,R,T)$. In fact, $\varepsilon(T_1)$ value is a measure of the spectral shape of the records [20]. The epsilon value can be calculated as per Eq. (6) [20, 21, 22].

$$\varepsilon(T) = \frac{\ln(Sa) - \mu_{\ln Sa}(M,R,T)}{\sigma_{\ln Sa}} \tag{6}$$

which, $\mu_{\ln Sa}(M,R,T)$ and $\sigma_{\ln Sa}$ are mean and standard deviation of $\ln(Sa)$ are calculated using the one or more ground motion well-known attenuation equation. β_1 in Eq. (5) indicates how sensitive the collapse capacity (S_C) is to changes in the epsilon value. The value of β_1 can be calculated performing a regression analysis to derive a relationship between the natural logarithm of the collapse capacities and the value of $\varepsilon(T_1)$ for each record as per Eq. (7) [19, 23].

$$\ln(Sc(T_1)) = \beta_1 \varepsilon(T_1) + \beta_0 \tag{7}$$

where, β_0 is the average collapse capacity when $\varepsilon(T_1) = 0$.

4. Bridge Performance Assessment

4.1 Proposed Simplified Bridge Seismic Performance Evaluation Process (SBSPEP)

A simplified approach is proposed here for seismic performance evaluation of bridges. The methodology of the proposed Simplified Bridge Seismic Performance Evaluation Process (SBSPEP) is based on comparison of values of the calculated ACMR with the value of acceptable ACMR as recommended in provisions of FEMA P695 [19].

To determine an acceptable value of ACMR, acceptable risk needs to be determined. To evaluate performance of the archetype models, adjusted collapse margin ratio (ACMR) is compared with an acceptable threshold of adjusted collapse margin ratio ($ACMR_{acceptable}$). The $ACMR_{acceptable}$ is calculated considering a given probability of collapse when the models are subjected to MCE-level ground motions. Ultimately, capacity of the system is calculated considering spectral shape effects. The general steps required for the collapse assessment of a RC bridge structure are shown in Fig. 4. Based on the proposed Simplified Bridge Seismic Performance Evaluation Process (SBSPEP), value of acceptable collapse margin ratio ($CMR_{acceptable}$) is required to evaluate a bridge performance. As Tehrani [23] and Ashkani Zadeh [4] suggested, $CMR_{acceptable}$ can be calculated based on Eq. (8). Consequently, $ACMR_{aceptable}$ can be calculated by multiplying spectral shape factor calculated to the $CMR_{acceptable}$.

$$CMR_{acceptable} = \frac{1}{\exp\left(\beta_{TOT} \times \Phi^{-1}\left(P^{C}_{acceptable}\right)\right)} \tag{8}$$

where, Φ^{-1} is the inverse cumulative normal distribution function, and $P_{acceptable}$ is the acceptable probability of collapse. β_{TOT} is the total system collapse uncertainty in predicting the collapse capacity of the structure ranging from 0.275 to 0.95. FEMA P695 provision provides a simplified assessment method to estimate the total uncertainty in the prediction of the collapse capacity (β_{TOT}). FEMA P695 provides a table of recommended (β_{TOT}) values based on quality of design and available test data for the structures with period-based ductility equal or greater than three ($\mu_T \geq 3$) [19].

Similar to FEMA P695 (FEMA P695, 2009), an acceptable collapse probability of 10% or less is consistent with the collapse performance objectives of this methodology. Based on the proposed methodology, acceptable probability of collapse and the modified collapse capacity ($Sa_m(T_1)$) can be calculated as per calculated based on Eq. (9) and Eq. (10), respectively [4, 23, 24].

$$P_{acceptable} = \Phi\left(\frac{\ln\left(IM/\hat{\theta}\right)}{\beta_{TOT}}\right) \leq 10\% \tag{9}$$

$$Sa_m\left(T_1\right)_{collapse} = ACMR \times Sa\left(T_1\right)_{MCE} = SSF \times S_{CT}\left(T_1\right) \tag{10}$$

where, $Sa\left(T_1\right)_{MCE}$ is the spectral acceleration at the fundamental period of the structure from the Maximum Considered Earthquake (MCE) spectrum. Base on FEMA P695, acceptable performance is accomplished when Adjusted Collapse Margin Ratio (ACMR) of each performance group for each index archetype model meet the following conditions: a) The average value of adjusted collapse margin ratio for each performance group ($\overline{ACMR_i}$) exceeds $ACMR_{10\%}$ ($\overline{ACMR_i} \geq ACMR_{10\%}$). And b) Individual values of adjusted collapse margin ratio for each index archetype model ($ACMR_i$) within a performance group exceeds $ACMR_{20\%}$ ($ACMR_i \geq ACMR_{20\%}$)[19].

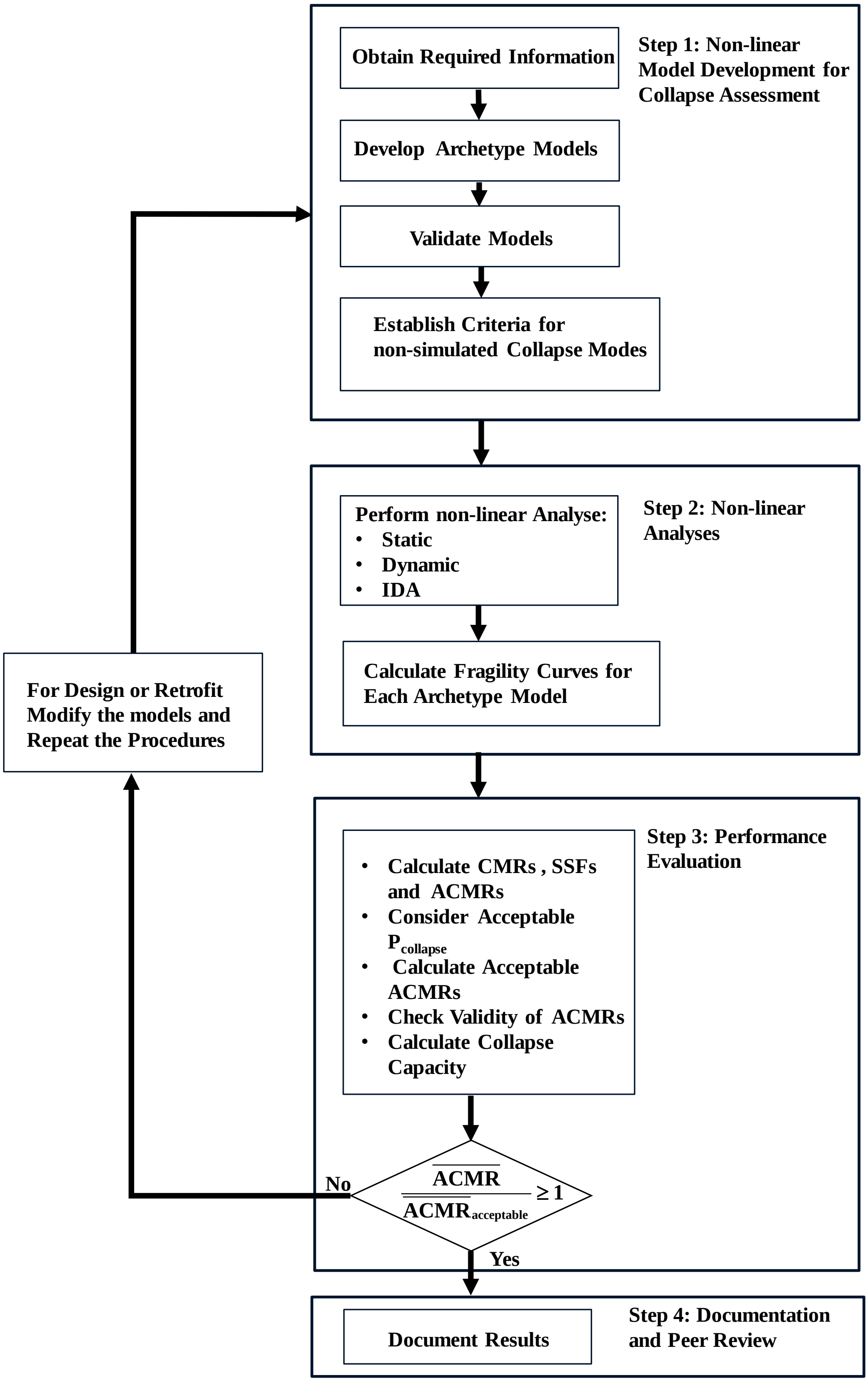


Fig. 4 Proposed Simplified Bridge Seismic Performance Evaluation Process (SBSPEP) [4].

The IDA analysis and performance evaluation step of the proposed Simplified Bridge Seismic Performance Evaluation Process (SBSPEP) are summarized in Fig. 5.

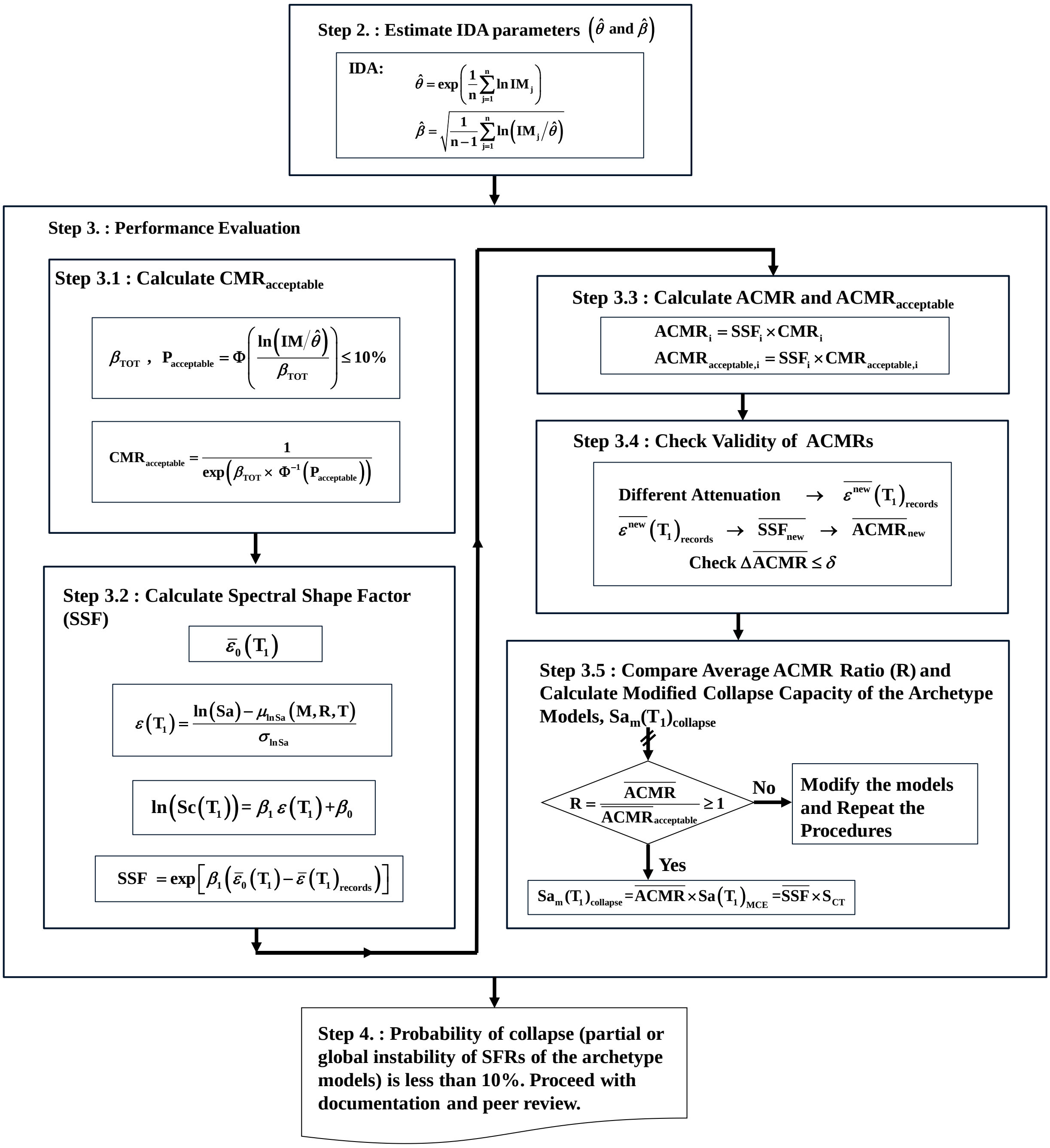


Fig. 5 – Detailed procedures for Step 2 to 4 of the proposed Simplified Bridge Seismic Performance Evaluation Process (SBSPEP) [4].

4.2 MRO Example Application

The USGS Vs30 map viewer, gives an average shear wave velocity of 227.8m in the top 30m for the Meloland site [25]. According to Table 3.10.3.1-1 AASHTO [26], site Class D should be considered for calculating response spectrum in Meloland Road Overcrossing site. The expected $\bar{\varepsilon}_0(T)$ value depends on the site and hazard level of interest. It should be predicted for seismic design category D for Southern California and for recommended 0.5% frequency of exceedance in 50 year [27]. Using a table that mean expected epsilon value is listed for cities in California for different seismic design category and site classes for period of 0.2s and 1.0s for different hazard levels, $\bar{\varepsilon}_0(T)$ value is considered 2.05. This value is confirmed by performing seismic deaggregation for MRO hazard site using Dynamic: Conterminous U.S. 2008 (v3.3.1) data considering Boore Atkinson (2008), Campbell and Bozorgnia (2008), and Chiou and Youngs (2008) attenuation equations [25]. The mean and standard deviation of the logarithmic spectral acceleration ($\mu_{\ln Sa(T_1)}$ and $\sigma_{\ln Sa(T_1)}$) are calculated using MATLAB codes developed by Baker and co-workers [28] based on Boore and Atkinson (2008), Abrahamson and Silva (1997), Boore et al. (1997), and Campbell (1997) attenuation models. Then mean epsilon of the records ($\bar{\varepsilon}(T_1)_{records}$) is calculated using Eq. (6) at the fundamental period of the archetype models [4]. To estimate β_0 and β_1 coefficients, a regression analysis is carried out for each archetype model. Relationship between $\varepsilon(T_1)$ and $\ln(S_c(T_1))$ for archetype model D_1 and D_4 are shown in Fig. 6. For each archetype and for each attenuation equation average epsilon of the records, $\bar{\varepsilon}(T_1)_{records}$, and regression coefficient, β_1 are calculated. Ultimately, Spectral Shape Factors (SSF) of each archetype is calculated. For The developed archetype model of the Meloland road Overcrossing, ACMR and $ACMR_{acceptable}$ are calculated considering the above calculated SSF using Eq. (5) and Eq. (8). Calculated $ACMR_{acceptable}$ and ACMR values for the different archetype model are listed in Table 2. As it can be seen from Table 2, the average value of adjusted collapse margin ratio for the performance group ($\overline{ACMR}$) exceeds $\overline{ACMR}_{10\%}$ by 1%. In addition, individual values of adjusted collapse margin ratio for each index archetype within the performance group ($ACMR_i$) except index archetype model D_4 exceeds $ACMR_{20\%}$. As a result, the archetype model does not have enough collapse resistance and needs model redefinition by the proposed methodology.

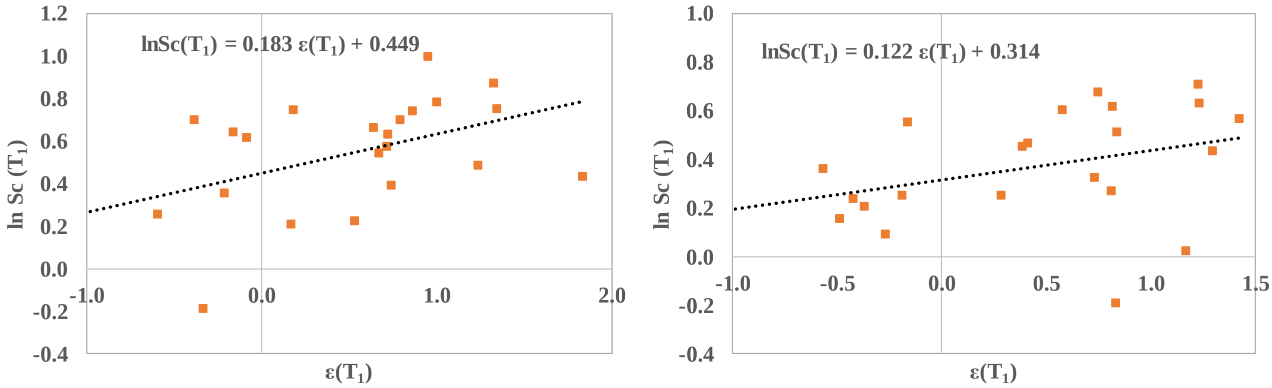


Fig. 6 – Estimated β_0 and β_1 coeficientes for the archetype model D_1 (left) and archetype model D_4 (right)

More regression analysis results can be found in Ashkani Zadeh [4].

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

Index Archetype Model	**SSF**	**ACMR**	Aceptable 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(Y)	0.86	1.21(Y)
D_2	1.35	1.04	0.96	1.08(Y)	0.87	1.19(Y)
D_3	1.38	1.04	1.00	1.04(Y)	0.90	1.16(Y)
D_4	1.22	0.81	0.98	0.83(N)	0.89	0.92(N)
Average	1.32	0.98	0.97	1.01(Y)	0.88	1.12(Y)
Note	Green : Methodology requirement is fulfilled Red : Methodology requirement is NOT fulfilled					

Value of the calculated spectral shape factor for each archetype model listed in Table 2. The modified collapse capacity of the archetype models ($Sa_m(T_1)_{collapse}$) is calculated using Eq. (10). The modified collapse capacity of each archetype model considering different attenuation relationship is calculated as:

$$D_1: Sa_m(T_1)_{collapse} = 2.29g,\ D_2: Sa_m(T_1)_{collapse} = 2.29g,\ D_3: Sa_m(T_1)_{collapse} = 2.30g,\ D_4: Sa_m(T_1)_{collapse} = 1.79g$$

The proposed methodology implies that average adjusted collapse margin ratio ($\overline{ACMR}$) of the archetype models equal or greater than average acceptable adjusted collapse margin ratio ($\overline{ACMR}_{acceptable}$) for the probability of collapse equal or less than 10% for the performance evaluation within 2% in 50 year hazard level. In addition, it implies that adjusted collapse margin ratio of each archetype model ($ACMR_i$) equal or greater than its corresponding acceptable adjusted margin ratio ($ACMR_{i,acceptable}$) for probability of collapse 20% with MCE level ground motions. Overall, the archetype models did not fulfill the requirement of the proposed methodology. Since the archetype models were simulated based on the Meloland bridge information, it can be concluded that the bridge performance does not satisfy the current provisions for risk of collapse less than 10% encountering MCE level ground motion.

5. Concluding Remarks

A methodology for collapse assessment of RC bridges was proposed. This methodology, which is based on provisions of FEMA P695, includes performing IDA analysis on various archetype models, development of fragility curves, calculation of CMR and ACMR parameters, calculation of acceptable ACMR parameters and finally calculating the modified collapse capacity. Meloland Road Overcrossing (MRO) was used as the case study example, where four archetype models were developed and analyzed using 22 ground motions selected from PEER database. It was shown that the archetype models fail to satisfy the acceptable performance requirements.

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