IMPLEMENTATION OF A MULTI-SPAN BRIDGE-GROUND PBEE FRAMEWORK FOR SEISMIC AND LIQUEFACTION SCENARIOS

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Abstract

In this paper, a performance-based earthquake engineering (PBEE) framework is presented. Efforts to extend the framework to account for multi-span bridge configurations and soil-structure interaction are shown. To facilitate this undertaking, the soil *p-y*, *t-z* curves approach is employed to represent the ground below grade. This approach provides a less complex environment to run Finite Element (FE) simulations compared to the 3D soil mesh approach. A user-friendly graphical user interface (MSBridge) is further adapted to conduct the analysis (using OpenSees) and to display the outcomes. For illustration, a multi-span reinforced concrete highway bridge is considered. On this basis, the framework is utilized to estimate post-earthquake repair cost and repair time. Intensity-dependent repair cost and repair time are disaggregated by performance groups (PGs) to evaluate the contribution of each bridge component to the overall system at different hazard levels. As such, analyses of the deck, columns, abutments, and foundation response mechanisms are integrated within a unified framework using MSBridge. Systematic evaluation of the global system response can be conducted under earthquake input shaking, or liquefaction-induced lateral spreading scenarios. Based on the simulation results, performance evaluation of the bridge is presented in terms of repair cost and repair time. In general, damage states and repair quantities related to the abutments can be among the most significant parameters.

Keywords: PBEE; MSBridge; Bridge; Seismic; Earthquake

1. Introduction

Bridge response during earthquakes has been receiving significant attention in recent decades [1-11]. The failure potential of highway bridges and their susceptibility to damage during extreme events necessitate further understanding of the response characteristics, in order to mitigate the consequences of seismic excitation. For that purpose, advanced numerical tools have been developed and used to simulate the salient bridge response mechanisms [12].

A probabilistic approach is preferred in seismic assessment to account for uncertainties in loading and structural modeling. For that purpose, a well-established methodology is utilized. In this regard, the employed Performance-Based Earthquake Engineering (PBEE) framework was originally proposed by Cornell and Krawinkler [13], and this methodology has been promoted and further developed by the Pacific Earthquake Engineering Research (PEER) Center [4, 8, 9, 14].

On this basis, an integrated computational framework is developed to combine nonlinear Time History Analysis (THA) of multi-span bridge systems with an implementation of the PEER PBEE methodology which quantifies the probabilistic bridge response in terms of repair cost and repair time. All stages of the involved analyses including the PBEE assessment framework are executed with the aid of a developed Graphical use interface (GUI), allowing the end-user to conveniently conduct extensive parametric investigations.

In this GUI (MSBridge, **Error! Reference source not found.**), bridge structures, abutments, and foundation response mechanisms are integrated within a unified framework. MSBridge also allows for addressing possible variability in the bridge deck, bentcap, column, foundation, or soil configuration/ properties (on a bent-by-bent basis). In addition, MSBridge permits the simulation of key scenarios of significance for bridge upgrades, widening, extensions, and retrofits.

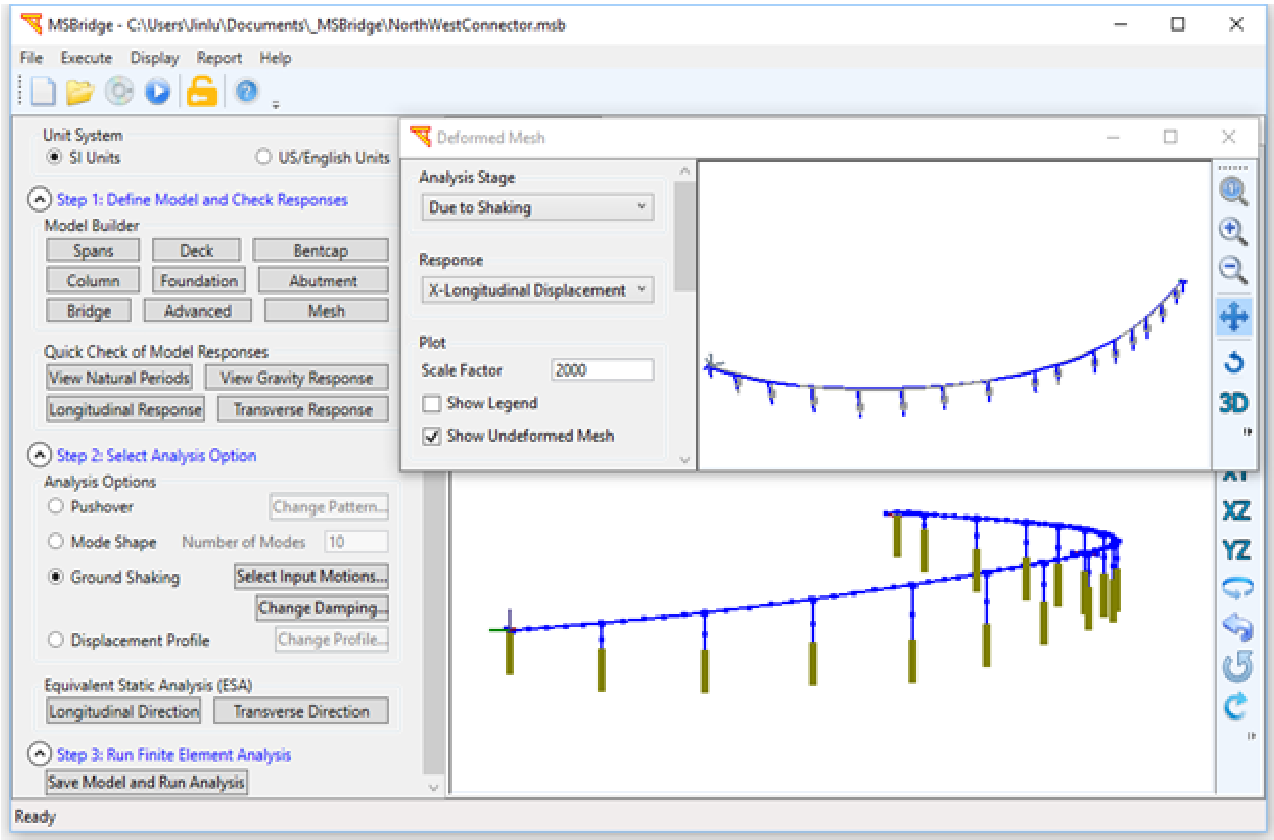


Fig. 1 – MSBridge (http://soilquake.net/msbridge) GUI with mesh showing a highway connector curved bridge with pile foundations (top-right window shows a sample deformed mesh due to earthquake shaking)

The following sections of this paper outline the: 1) computational framework, 2) FE modeling details, 3) employed ground motions to cover a wide range of intensity measures (IMs), and 4) representative PBEE results in terms of loss model and hazard model. Finally, a number of conclusions are presented and discussed.

2. Computational Framework

2.1 Performance-based earthquake engineering framework

Using the total probability theorem, the desired probability distributions are calculated by dividing the task into four probabilistic models, each with its uncertainties and outcomes [15]: (i) Hazard model that uses the input ground motions to determine the Intensity Measures (IMs), (ii) Demand model that uses the FE simulations results to determine the Engineering Demand Parameters (EDPs), (iii) Damage model that connects the EDPs to pre-defined Damage States (DSs), and (iv) Loss model that quantifies the DSs into repair Quantities (Qs) and then correlates Qs to loss outcomes in terms of repair cost and repair time. This framework is based on linearization of the damage model, described as local linearization repair cost and time methodology (LLRCAT). More details about this methodology can be found in [15].

To facilitate the disaggregation when applying the total probability theorem, the bridge was divided [8] into a collection of structural components known as performance groups (PGs). Each PG is characterized by the DSs that are triggered when critical values of the EDPs are reached. As such, the higher DS corresponds to more severe consequences. For example, DS1 in the column is cracking, while DS4 is a complete failure.

The estimated Repair costs (RC) are obtained by multiplying each Q by a corresponding prescribed Unit Cost (UC). Similarly, the estimated Repair times (RT) are obtained using a corresponding prescribed Production Rate (PR). Finally, the expected total RC is estimated from the assembly of loss in each discrete DSs for all PGs. Table 1 shows the PGs (and associated EDPs and DSs) used in this study.

2.2 Finite element computational framework

The Open System for Earthquake Engineering Simulation platform (OpenSees [12], http://opensees.berkeley.edu) was used to conduct the nonlinear bridge-ground system analysis subjected to seismic excitation. OpenSees was developed by PEER, and is widely used for simulation of structural and geotechnical systems including SSI applications [e.g., 5, 17, 18].

2.3 User-interface for nonlinear bridge-foundation analysis

Under earlier PEER support, a tool for conducting 3-Dimensional (3D) nonlinear THA has been developed (BridgePBEE, http://peer.berkeley.edu/bridgepbee/), that also incorporates the PEER PBEE framework. However, this tool was limited to the scenario of single circular column, 2-span bridges. To overcome this limitation, an effort was made to extend the PBEE assessment implementation to multi-span bridge scenarios via MSBridge [19-22]. As such, MSBridge is a unique tool that allows rapid PBEE assessment of multi-span bridge systems. It systematically builds on the earlier PEER research that resulted in the development of BridgePBEE.

MSBridge is a PC-based graphical pre- and post-processor (user-interface) for conducting nonlinear Finite Element (FE) studies for a wide range of multi-span bridge systems [19], that also incorporates the PEER PBEE framework [20-22]. It allows engineers to efficiently conduct nonlinear Time History Analysis (THA) with PBEE assessments for a wide range of multi-span bridge configurations within a seamless integrated simulation environment.

Specifically, MSBridge allows users to rapidly build a bridge model, run the FE analysis, and evaluate the performance of the bridge-ground system. Main capabilities of MSBridge include: i) horizontal and vertical alignments, with different skew angles for bents/abutments; ii) nonlinear beam-column elements with fiber section for bridge columns and/or piles; iii) deck hinges, isolation bearings, steel jackets, and abutment models;

and iv) foundation represented by foundation matrix (6x6) or soil springs (p-y, t-z, and q-z). The analysis options available in MSBridge include: i) pushover analysis; ii) mode shape analysis; iii) 3D base input acceleration analysis (for suites of ground motions, built-in and/or user-defined); iv) equivalent static analysis (ESA); and v) PBEE analysis (with PBEE outcomes in terms of repair cost, and repair time).

Table 1 – Performance groups and associated engineering demand parameters after [16]

PG	EDP	DS	Description
PG1	Maximum column tangential drift ratio	DS0	Concrete cracking
		DS1	Onset of spalling
		DS2	Buckling of bars
		DS3	Failure
PG2	Residual column tangential drift ratio	DS0	Tolerable
		DS1	Enlarge pier and jacket
		DS2	Failure
PG3 & PG4	Max. longitudinal relative deck-end/ abutment displacement for left & right abutments	DS0	Joint cleaning
		DS1	Repair joint seal
		DS2	Back wall repair
		DS3	Back wall failure
PG5 & PG6	Max. bearing displacement for left and right abutments	DS0	Yielding
		DS1	Failure
PG7 & PG8	Approach residual vertical displacement for left & right abutments	DS0	Pavement
		DS1	Regrade
		DS2	Rebuild
PG9 & PG10	Residual pile cap-location displacement for left & right abutments	DS0	Add pile threshold
		DS1	Enlarge foundation
PG11	Residual pile cap-location displacement for columns	DS0	Add pile threshold
		DS1	Enlarge foundation

3. Finite Element Model

3.1 Bridge Model

The studied structure is a 3-span reinforced concrete single column bent box girder bridge (**Error! Reference source not found.**). Derived after [23], the dimensions of the bridge considered in this paper are 45.7 m spans, 6.71 m clear column heights, 1.22 m circular column diameter, and 11.9 m wide two-cell box girder. Table 2 shows the bridge deck and bridge pier column material and section properties.

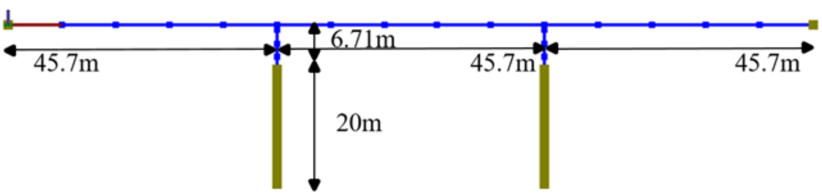


Fig. 2 – Finite element mesh: (a) 3D view, (b) elevation view

Relevant parameters for the concrete model are listed in Table 3. In addition, Fig. 3 illustrates the stress-strain parameters used in the analysis. The longitudinal reinforcing steel used in all the columns is modeled as a bilinear material with an assumed yield strength of 303 MPa (44 ksi), an effective elastic stiffness of 200 GPa, and 0.8% post-yield stiffness (strain hardening ratio b = 0.008). In addition, a seat-type abutment model was specified in this bridge configuration [20-22].

Table 2 – Material and section properties

Parameter	Bridge deck	Bridge pier column
Young's modulus (kPa)	2.53×10^7	2.53×10^7
Shear modulus (kPa)	1.05×10^7	1.05×10^7
Unit weight (kPa)	25.11	25.11
Area of cross-section (m^2)	5.05	5.66
Moment of inertia @ horizontal axis (m^4)	6.78	5.11
Moment of inertia @ vertical axis (m^4)	41.89	1.11
Torsion constant (m^4)	0.98	3.53

Table 3 – Constitutive model parameters for concrete material used in fiber beam-column element

Parameter	Confined concrete	Unconfined concrete
Concrete compressive strength at 28 days, f_c' (kPa)	-3.47×10^4	3.40×10^4
Concrete strain at maximum strength, ε_{c0}	-0.0025	-0.002
Concrete crushing strength, f_{cu}' (kPa)	-2.07×10^4	0
Concrete strain at crushing strength, ε_{cu}	-0.014	-0.005

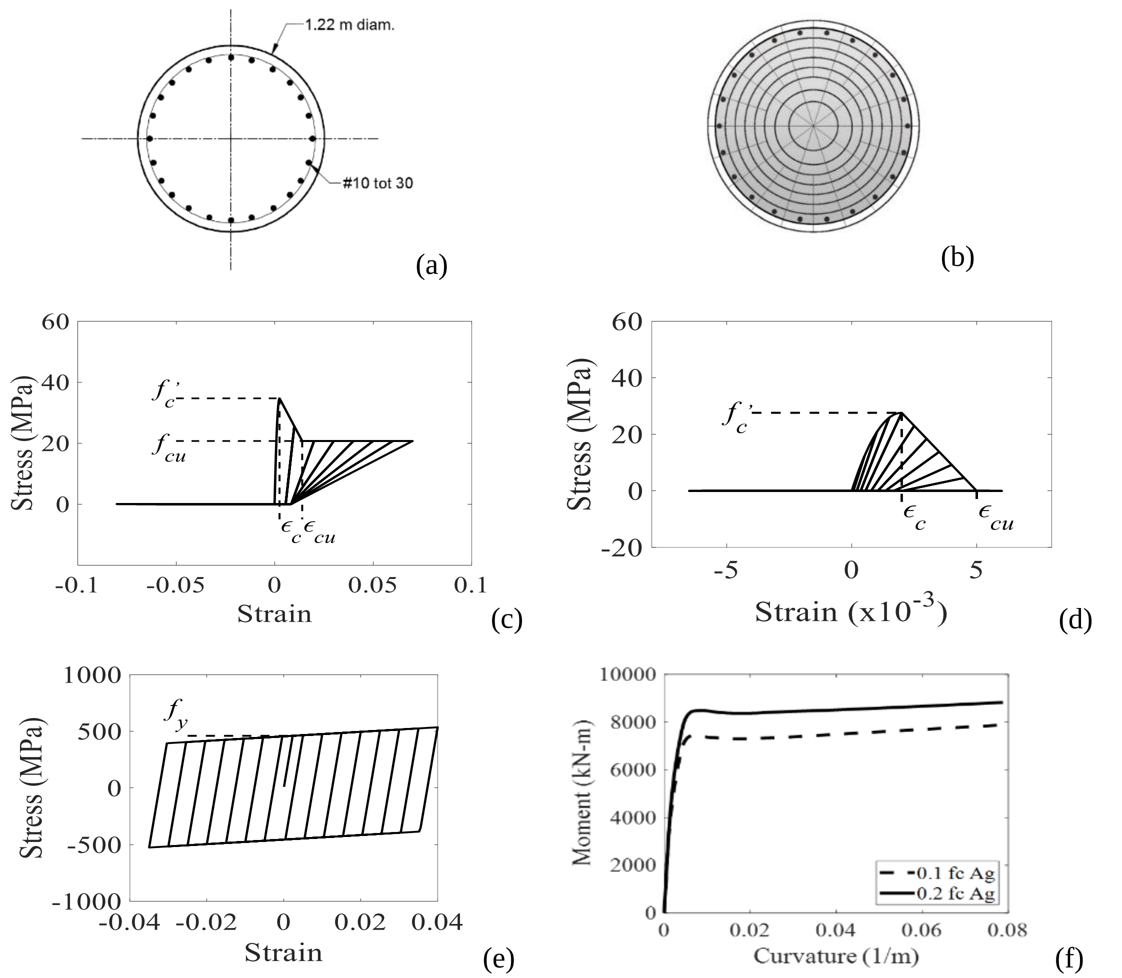


Fig. 3 – Modeling of column-pile: (a) cross-section; (b) fiber discretization; (c) core concrete; (d) cover concrete; (e) reinforcing steel; and (f) moment-curvature response

3.2 P-y curves

The *p-y* curves approach was employed to define the strength of the soil below grade. This approach greatly simplifies the FE simulations. The selected *p-y* curves were based on Clay soil properties [24] as shown in Table 4. **Error! Reference source not found.** shows the *p-y* curves at selected depths. Using this approach, the FE and PBEE analyses were performed.

Table 4 – Soil types and properties

Layer depth (m)	Soil type	Mass density ρ (kg/m3)	Shear Strength S_u (kPa)
0-5	Soft clay	1500	41.5
5-10			74.5
10-15			108
15-20	Stiff clay	2000	142

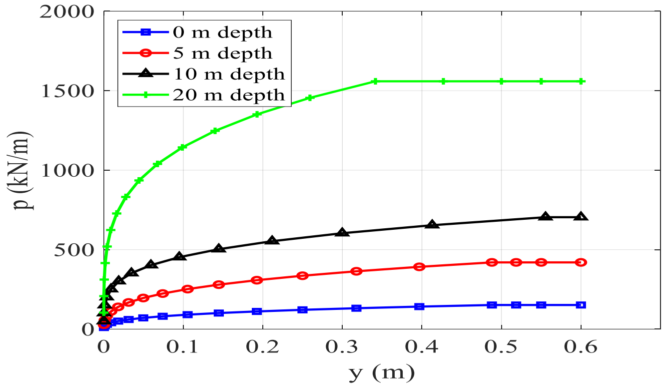


Fig. 4 – *P-y* curves at depths: (a) 0 m; (b) 5 m; (c) 10 m; and (d) 20 m

4. Ground Motions

An ensemble of 100 ground motions was obtained from the PEER NGA database (http://peer.berkeley.edu/nga/). Each motion is composed of 3 perpendicular acceleration time history components (2 lateral and one vertical). These motions were selected through earlier efforts [14. 25] to be representative of seismicity in typical regions of California. The motions are divided into 5 bins of 20 motions each with characteristics: i) moment magnitude (M_w) 6.5-7.2 and closest distance (R) 15-30 km, ii) M_w 6.5-7.2 and R 30-60 km, iii) M_w 5.8-6.5 and R 15-30 km, iv) M_w 5.8-6.5 and R 30-60 km, and v) M_w 5.8-7.2 and R 0-15 km (Fig. 5).

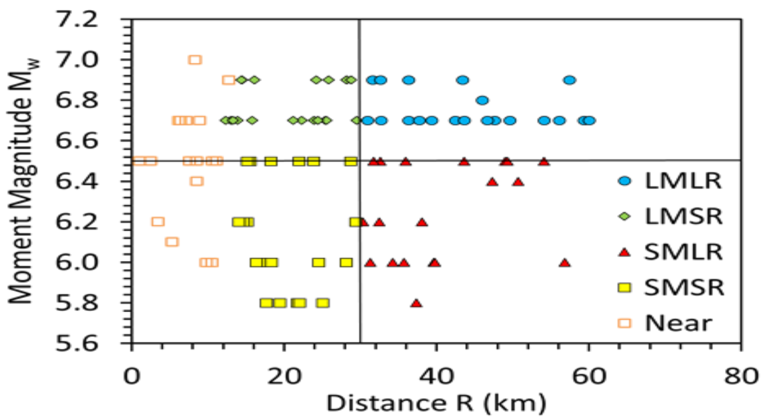


Fig. 5 – Input ground motions in M-R space

MSBridge allows for the specification of numerous Intensity measures, so as to display the outcomes against any of these measures. Herein each earthquake motion will be represented by its PGV as the intensity measure (IM). Fig. 6 shows the distribution of horizontal PGV Square Root Sum of Squares (SRSS) values.

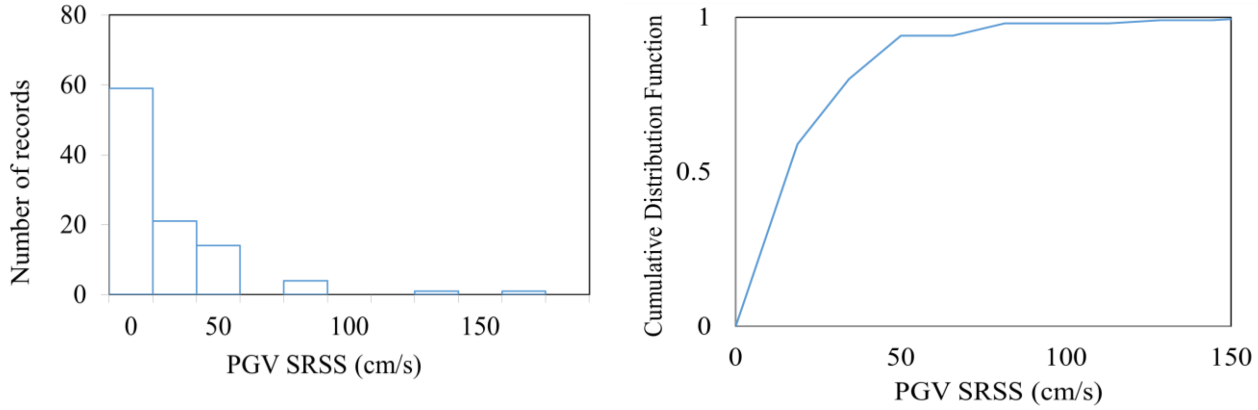


Fig. 6 – PGV distribution for SRSS of the two lateral ground motion components

5. Time History Results

Nonlinear time history analysis (THA) was conducted for the selected 100 input motions using MSBridge. Uniform base excitation was applied using each of these input ground motions. Rayleigh damping was used with a 5% damping ratio (defined at the periods of 1.43 and 0.33 second) in the nonlinear THA. For the time integration scheme, the Newmark average acceleration method ($\gamma = 0.5$ and $\beta = 0.25$) was employed. A variable time-stepping scheme (VariableTransient) was used in the conducted Nonlinear THA. The starting value for each step was 0.02 second.

The time history results were used to create the probabilistic seismic demand models (PSDMs). As such, the EDP values for each PG were used to assess the DSs and the associated repair quantities. Fig. 7a shows the PSDM for the maximum column drift ratio (PG1). In addition, Fig. 7b shows the PSDM for the maximum relative deck-end/abutment displacement (PG3).

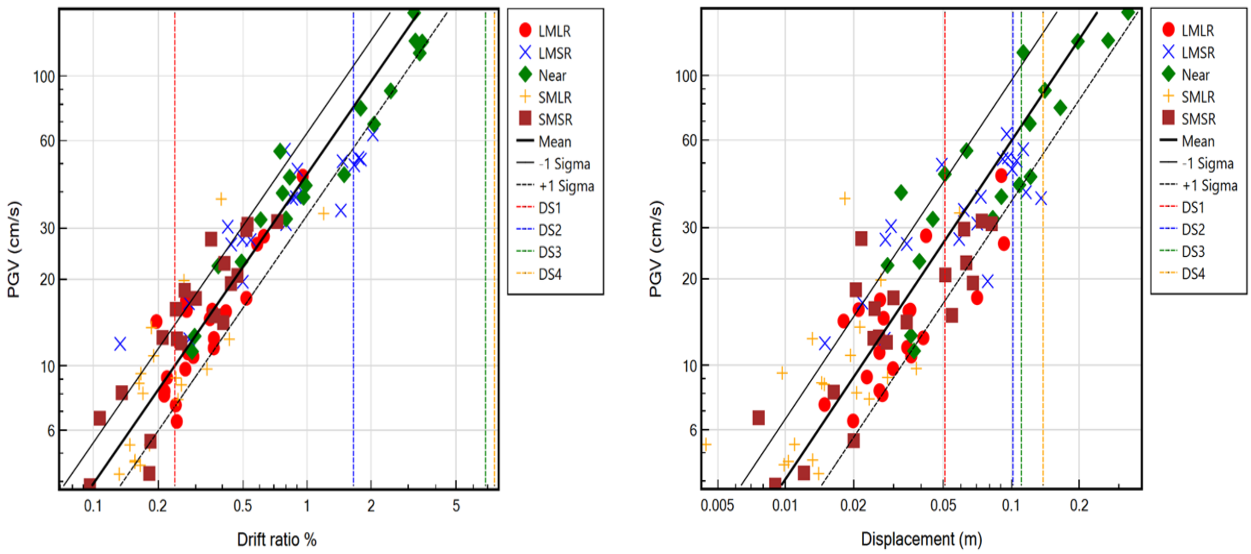


Fig. 7 – PSDMs for the column of bent 2 maximum drift ratio (left), and maximum relative deck end-back wall displacement at left abutment (right)

6. PBEE Results

6.1 Loss Model

Fig. 8 shows the repair cost disaggregated for each PG. In addition, Fig. 9a shows the mean repair cost ratio (RCR), the ratio between cost of repair and cost of replacement (not including demolition), against PGV as the IM. It can be gleaned from the repair cost results (shown in Fig. 9) that the post-earthquake consequences begin to accumulate at a PGV of approximately 20 cm/s. Similarly, Fig. 9b shows the repair time (RT) in crew working day (CWD) against PGV. The jumps in the repair time at a PGV of approximately 20 cm/s is due to triggering of low-event DSs for PG1. In addition, it is worth noting that the repair time reaches a plateau around 70 cm/s, after which no DSs were triggered.

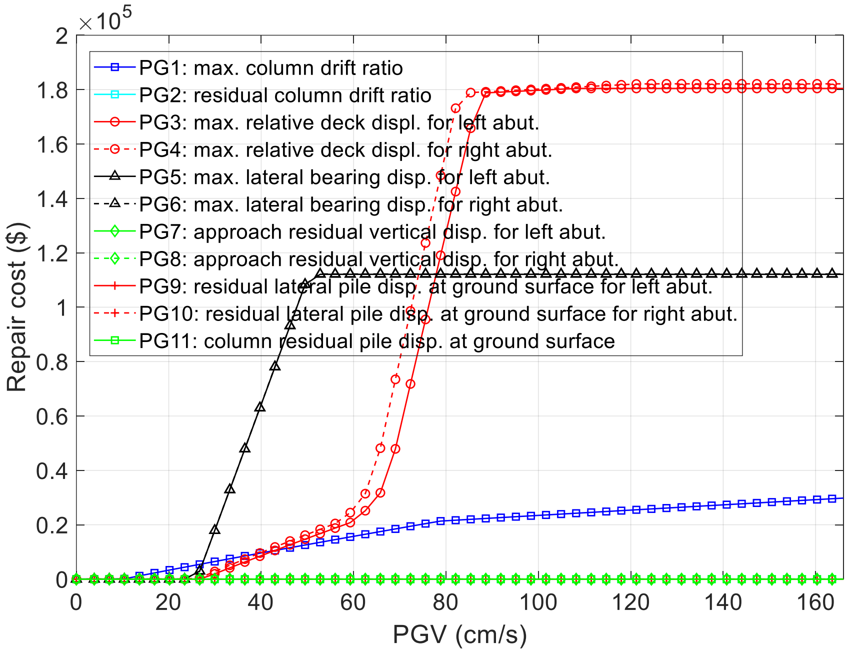


Fig. 8 – Repair cost disaggregated for each PG vs. PGV [22]

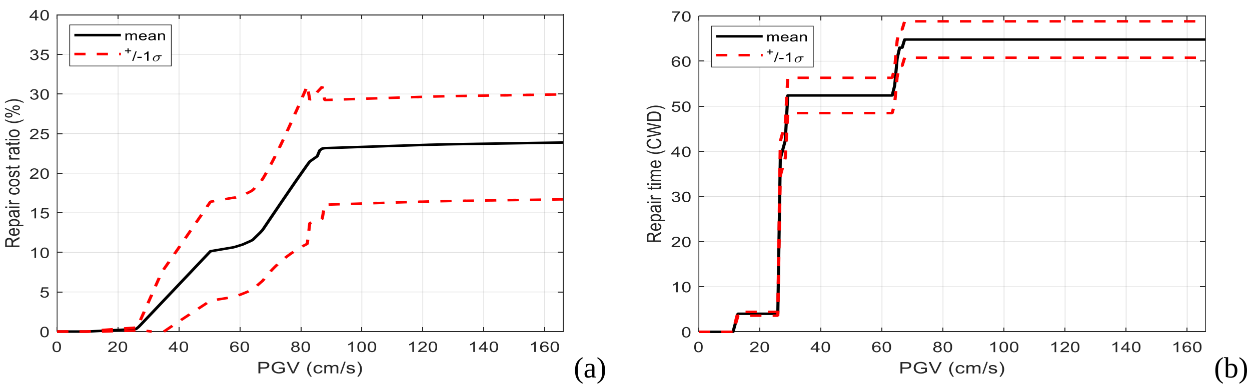
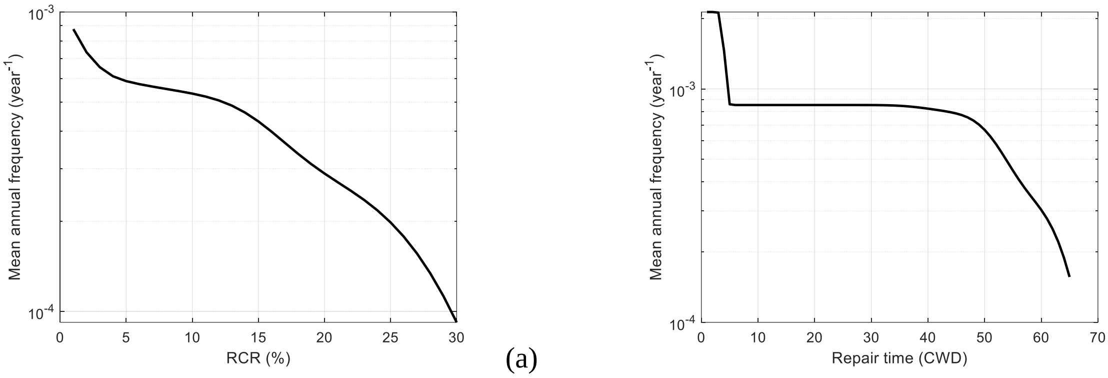


Fig. 9 – Loss models (with probabilistic moments) in terms of: (a) RCR; and (b) RT [22]

6.2 Hazard Model

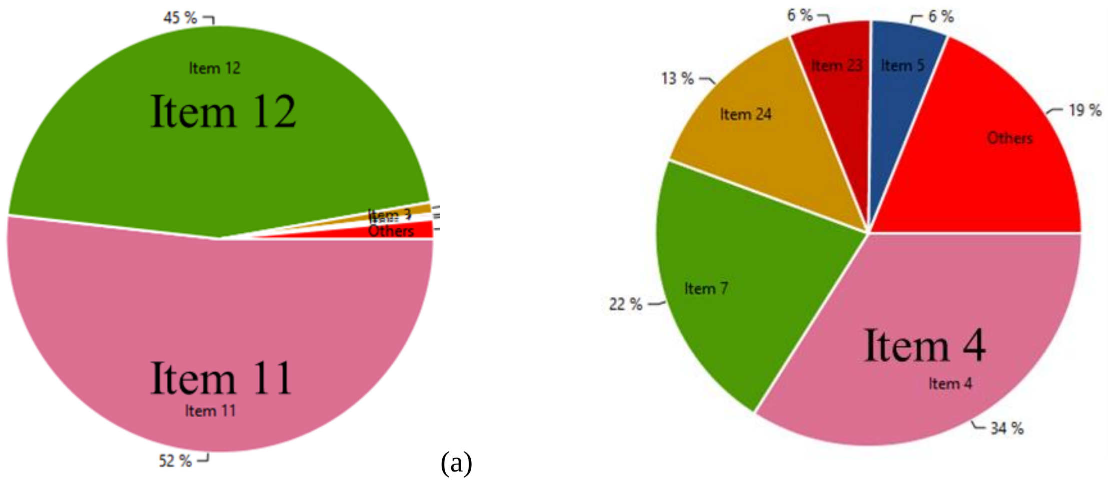
To obtain Hazard Curves for a particular seismicity scenario (based on geographic location), three probabilities of exceedance (2%, 5%, and 10% in 50 years) are needed. The hazard selected (PGV=160, 80, and 20 cm/s) is consistent with infrequent events of larger magnitude such as in the central US. On this basis, Fig. 10a shows the mean annual frequency (MAF) of exceedance vs. RCR. Similarly, Fig. 10b shows the MAF of exceedance vs. Repair Time (RT).



(a) (b)

Fig. 10 – Hazard models in terms of: (a) RCR; and (b) RT [22]

Disaggregation of expected repair cost can be presented at different hazard levels (e.g., 50% in 50 years and 2% in 50 years) to highlight the primary contributing repair items (Fig. 11). Inspection of Fig. 11a and Fig. 8 shows for the 50% in 50 years hazard level, repair item 11 and repair item 12 (epoxy inject cracks and repair minor spalls, respectively) are the main drivers when quantifying the repair cost for the columns (PG1) [22]. Furthermore, for the 2% in 50 years hazard level, repair item 4 (Temporary support abutment) is the primary driver [22] to replace the retaining wall (PG3 and PG4).

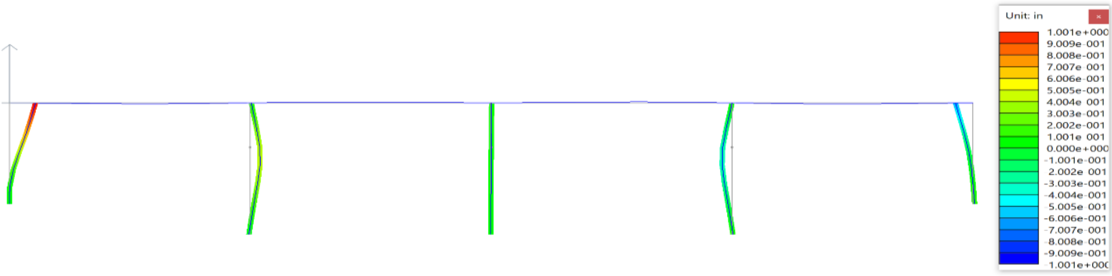


(a) (b)

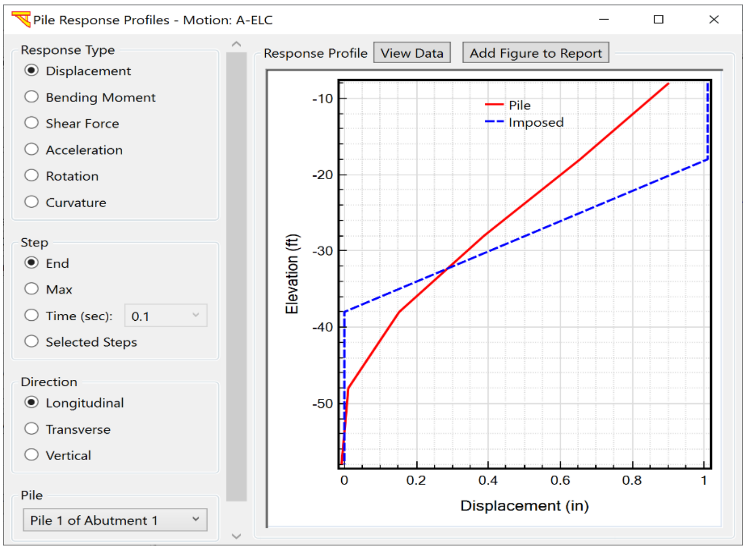
Fig. 11 – Disaggregation of expected repair cost by repair item at two hazard levels

7. Practice-Oriented Lateral Spreading Analysis Approach

Recently, MSBridge was further adapted to study the influence of liquefaction-induced lateral spreading ground deformations (Fig. 12). This is based on a pushover analysis capability, with user prescribed ground displacement profiles applied along the different pile embedment depths. These displacement profiles are applied to the bridge via appropriate *p-y* soil springs. Using this option, the deformation due to liquefaction induced lateral spreading can be assessed, with the ground displacement profiles estimated for instance via guidelines such as those of MTD 20-15 [26].



(a)



(b)

Fig. 12 – MSBridge output of imposed displacement in the bridge longitudinal direction: (a) representative deformed shape; (b) pile response profile

8. Summary and Conclusions

Finite Element (FE) modeling provides an effective mechanism for understanding the integrated structure-foundation-ground system response. OpenSees was utilized to conduct nonlinear FE studies of multi-span bridge systems. To facilitate the OpenSees analyses, the user-interface MSBridge was developed and employed.

Recently, performance-based design in civil engineering has been receiving a great deal of attention, and many design codes are initiating the application of its concepts. As a result, the PEER PBEE framework was employed to estimate the probabilistic seismic system response. The underlying analysis framework is implemented in MSBridge to provide a unique tool that enables nonlinear FE studies as well as performance-based assessment within an integrated simulation environment. This tool systematically provides valuable insights on the demand, damage, and loss models of multi-span bridge-ground systems.

As such, seismic response was addressed by the global modeling of the bridge-ground system as an integral entity. In this regard, nonlinear representation of the bridge deck, columns, abutments, and foundation response are integrated within a unified framework.

A finite element model developed for the bridge-foundation-ground system as an integral entity provides detailed insights and captures the main interaction mechanisms between the ground and the various bridge components. Based on the simulation results, abutment DSs (and corresponding repair quantities) are among the main parameters that contribute to the overall loss estimates.

9. Acknowledgements

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