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USER INTERFACE FOR PERFORMANCE-BASED EARTHQUAKE ENGINEERING: A SINGLE BENT BRIDGE PILOT INVESTIGATION

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ABSTRACT

A graphical user interface is under development to combine nonlinear dynamic time history analysis of coupled soil-structure systems with an implementation of performance-based earthquake engineering (PBEE). The user interface builds upon previous code that allowed for analysis of piles in a soil domain under nonlinear static and nonlinear dynamic loads (OpenSeesPL). Functionality was extended for analysis of multiple suites of ground motions and combination of results probabilistically using the performance-based earthquake engineering framework developed by the Pacific Earthquake Engineering Research Center. Definition of the bridge and underlying ground configuration and material properties is greatly facilitated using this new interface. In addition, all stages of the involved analyses are conveniently executed in a systematic fashion, allowing the end user to investigate parametric or what-if scenarios on typical bridge configurations. In this paper, the main elements of this numerical framework are presented including graphical user interface elements and underlying theoretical framework. A simple bridge-ground model was developed and studied at three different levels of mesh refinement. PBEE results are computed and presented for a suite of selected ground motion records. Results show the PBEE results to be relatively insensitive to the chosen set of mesh discretization schemes.

Introduction

Performance-based earthquake engineering (PBEE) aims to quantify the seismic performance and risk of engineered facilities using metrics that are of immediate use to both engineers and stakeholders. The Pacific Earthquake Engineering Research (PEER) Center has been a major proponent for both the theoretical development of a PBEE methodology as well as deployment of the concepts into academia and industry, including incorporation of precepts into the next generation of building/design codes. PBEE as applied to buildings has seen rapid development and adoption recently (e.g., ATC-58 and ATC-63); however, in the bridge and infrastructure arena, there have been relatively few attempts at rigorous development of the data necessary for PBEE or packaging the tools in a form that allows rapid PBEE-based evaluation

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and assessment such as PACT. This is the motivation behind development of OpenSeesPL PBEE, a graphical environment for finite element modeling of coupled soil-structure systems as well as complete PBEE assessment.

Mackie and co-workers have pioneered the development of a bridge performance-based analysis framework (Mackie et al., 2009; Mackie et al., 2007). Bridges with fixed or spring foundation constraints were studied within this framework using the PEER computational platform OpenSees (http://opensees.berkeley.edu/). Based on the response of a series of typical pre-stressed, single-column bent, multi-span, box girder bridges in California, the data flows and requisite information were derived to relate response to damage of individual components within the structure, denoted as performance groups (PGs). Damage to these PGs were tied to explicit repair procedures and repair quantities that could then be used for cost estimation and repair effort necessary to return the bridge to its original level of functionality (direct costs). In addition, other PEER researchers used the same bridge configuration and model, but considered the pile-pinning effect at the abutments (Ledezma and Bray, 2008) and the increase in repair costs due to the presence of a liquefaction-susceptible soil profile (Kramer et al., 2008).

Simultaneously, Elgamal and co-workers (Lu, 2006; Lu et al., 2006) had embarked on development of a three-dimensional (3D) ground-foundation graphical user interface OpenSeesPL that employs OpenSees as the finite element (FE) analysis engine. This interface allows for the execution of pushover and seismic single-pile or pile-group ground simulations. OpenSeesPL includes pre- and post-processing capabilities to generate the mesh, define material properties and boundary conditions, activate OpenSees to conduct the computations, and display/animate the output responses (http://cyclic.ucsd.edu/openseespl). Various ground modification scenarios may be also studied by appropriate specification of the material within the pile zone. The menu of soil materials in OpenSeesPL includes a complementary set of soil modeling parameters representing loose, medium and dense cohesionless soils (with silt, sand or gravel permeability), and soft, medium and stiff clay (J2 plasticity cyclic model).

Recently, an effort was initiated to: i) build on the above PEER PBEE framework by implementing all details within the graphical user-interface, ii) build a module for handling the needed input ground motion ensemble and to compute all salient characteristics, denoted as intensity measures (IMs), iii) modify the graphical interface to automatically generate user-defined bridge-ground FE models, and iv) build the post-processing capability to display the seismic response ensembles, and to display the PBEE outcomes. Elements of this new framework are presented in this paper. A pilot investigation of a single-bent bridge using three different levels of mesh refinement that makes use of the new graphical user-interface environment is presented.

PBEE Analysis Framework

PBEE considers seismic hazard, structural response, resulting damage, and repair costs associated with restoring a structure to its original function, using a fully consistent, probabilistic analysis of the associated parts of the problem (Cornell and Krawinkler, 2000). The uncertainty surrounding the PBEE framework components necessitates a probabilistic approach and acceptance criteria based on levels of confidence that probabilities of failure are acceptably small. Adoption of such PBEE methodology in practice requires abandonment of prescriptive seismic safety specifications and acceptance of performance objectives defined in terms of quantities familiar to engineers, owners, managers, and stakeholders alike. This approach to earthquake engineering is, above all, sustainable because the underlying framework is independent of the performance objectives selected for the particular evaluation or design project, thus allowing for seamless adaptation to specific project needs, new design methods, and innovative structural systems.

A rigorous yet practical implementation of the PEER PBEE methodology was adapted for use in the user interface. The methodology is subdivided to achieve performance objectives stated in terms of the probability of exceeding threshold values of socio-economic decision variables (DVs) in the seismic hazard environment under consideration. The PEER PBEE framework utilizes the total probability theorem to disaggregate the problem into several intermediate probabilistic models. This disaggregation of the decision-making framework outcome involves the following intermediate variables: repair quantities (Q), damage measures (DMs), engineering demand parameters (EDPs), and seismic hazard intensity measures (IMs). Consequently, engineers may choose to scrutinize probabilities of exceeding an EDP, such as strain, while an owner may choose to scrutinize probabilities of exceeding a DV, such as repair cost. An important step enabling effective aggregation of decision data is the association of structural elements and assemblies into performance groups (PGs) based on commonly used repair methods. The numerical implementation of the methodology is described in Mackie et al. (2009).

The EDPs are computed directly from the ensemble of time history analyses performed. These are automatically associated with the PGs and the DSs for each. For example, additional bridge bents will automatically generate additional drift recorders and the distribution of maxima from multiple ground motion records will be compared to a set of damage fragility curves computed for each column PG. The data used to populate the relationships that associate EDPs to DMs and DMs to Qs were previously described in Mackie et al. (2007). There exist default values for all of the built-in repair quantities, including the unit costs and production rates for each one of these items. However, the user has the ability to modify these if more state-specific or site-specific information is available.

Elements of the User Interface

An important consideration for the use of the interface is runtime. Depending on the complexity of the FE model, the number of degrees-of-freedom, and potential for nonlinear response, the ensemble of motions may require several hours to complete the analysis. To facilitate computation time, individual transient analyses can be run in parallel on a multi-processor machine. However, once the response quantities have been computed, PBEE scenarios can be independently investigated without re-running the model. Computation time for the PBEE analysis is negligible. The major components of a PBEE analysis are: specification of ground motions, mesh and soil constitutive model determination, bridge superstructure model and constitutive model determination, specification of PGs and the associated PBEE quantities, and the myriad of post-processing capabilities.

Specification of Ground Motion Input

The framework allows selection of individual ground motions, suites of ground motions, and bins of ground motions. At the current time, all motions are obtained from the PEER NGA database (http://peer.berkeley.edu/nga/) with a future option of importing arbitrary delimited text files or to search the NGA database interactively through a socket. An ensemble of 100 selected ground motions is employed in the PBEE analysis illustrated in this paper. Each motion is composed of 3 perpendicular acceleration time history components (2 lateral and one vertical). These motions were selected through earlier efforts (Gupta and Krawinkler, 2000; Mackie et al., 2007) and were selected 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. The user selects this motion ensemble by specifying the folder where the motion time histories have been stored in text files (Figure 1).

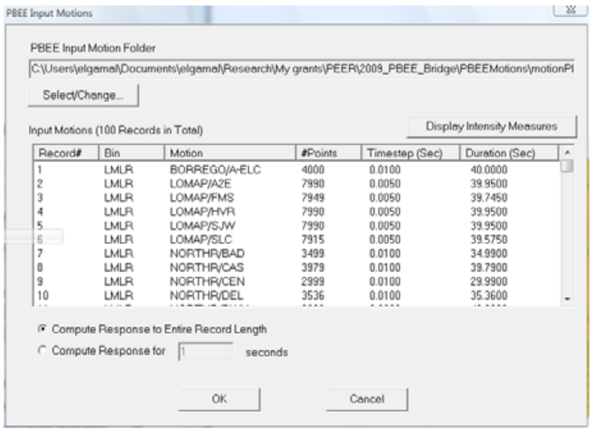


Figure 1. Ground motion selection screen.

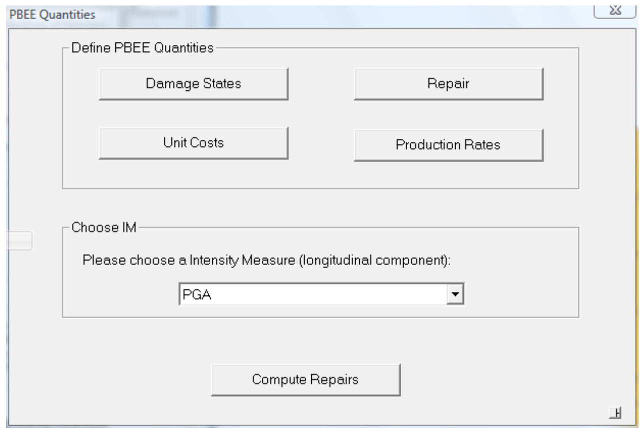


Figure 2. PBEE quantity user interface.

For each set of 3 orthogonal acceleration time histories, a large number of IMs are calculated, including peak ground acceleration (PGA), peak ground velocity (PGV), peak ground displacement (PGD), Arias intensity, strong motion duration ($D_{5\text{-}95}$), and cumulative absolute velocity (CAV). The IMs are calculated and displayed as a vector (one value for each shaking direction), and also in the form of the square root-sum-of-squares (SRSS) in the two horizontal directions (). In addition, for each ground motion component, time histories and frequency domain (spectral) displays are provided () for acceleration, velocity, and displacement. The user can obtain this information by selecting an individual motion (Figure 1).

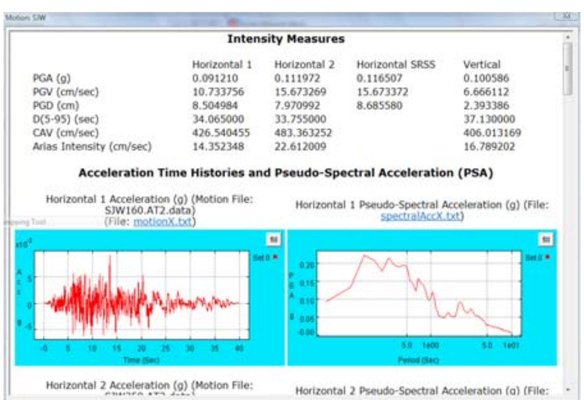


Figure 3. IMs computed for SJW160 record (Loma Prieta 1989 Salinas J&W).

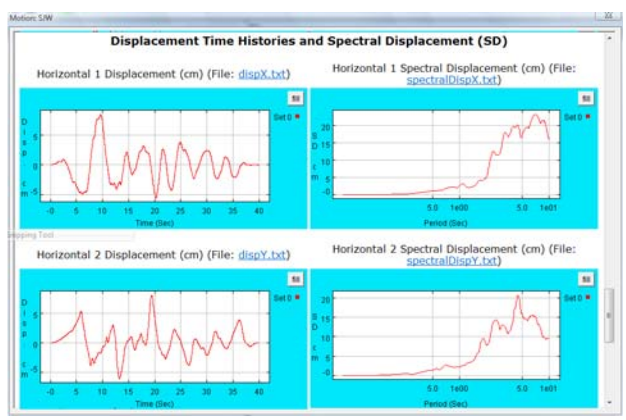


Figure 4. Displacement time history and spectral displacement for SJW.

While the ability to scrutinize individual records has numerous benefits, the use of PBEE necessitates the inclusion of multiple ground motions. Once these motions have been selected and/or binned, it is of interest what the salient characteristics (IMs) are of the group or bin of ground motions. These characteristics of the entire ground motion ensemble are automatically generated and displayed in the form of histograms and cumulative distribution functions (CDF) for each of the IMs calculated. For example, the distribution of PGA values (Figure 5) shows the majority of records utilized have less than or equal 0.25 g PGA; however, the suite contains motions with PGAs as large as 1g. Similarly, the histogram and CDF of PGV are shown in Figure 6.

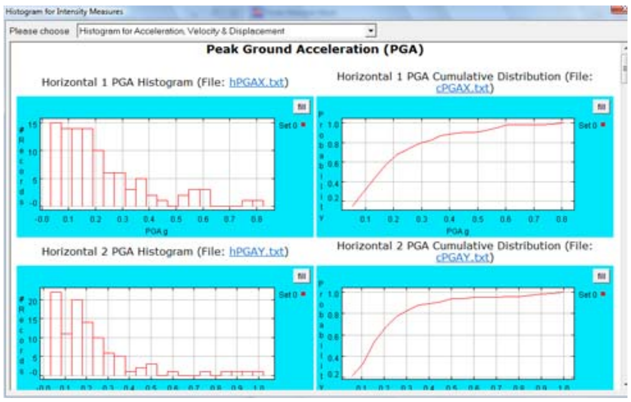


Figure 5. 100 motion PGA distribution.

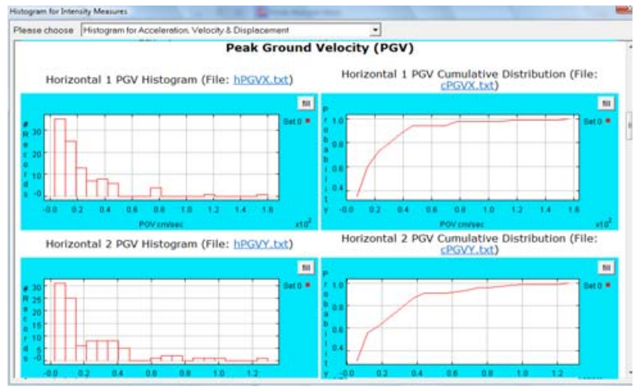


Figure 6. 100 motion PGV distribution.

Bridge-Ground Finite Element Model

The bridge-ground configurations available for construction in the user interface are currently based on single column bents extending into integral Type 1 pile shafts below grade. Mesh refinement is performed automatically surrounding each pile shaft in the ground. The columns are modeled as nonlinear beam-column elements with fiber cross sections. The user has the ability to configure the cross-sectional properties, shape, and materials. The current user interface supports reinforced concrete columns only. The deck is also modeled using two-noded beam-column elements discretized into five separate elements along each clear span. The deck is assumed to be capacity designed so that it responds in the elastic range. The gross or cracked section properties can be specified by the user. At the current stage of development, the approach ramp model connects the bridge longitudinal boundaries to the ground motion as specified by motion of the soil domain below the abutments (Figure 7). Several abutment models are currently available and provide the interface between the approach ramps and the bridge ends. These abutment options include a roller, elastic, simplified, and spring.

The roller abutment model consists of a simple boundary condition module that applies single-point constraints for displacement in the vertical direction and rotation about the bridge longitudinal axis. The elastic abutment model is similar, except it has explicit user-configurable elastic stiffness values in all six degrees-of-freedom between the approach ramp and deck. The simplified abutment model consists of a rigid element of the same length as the superstructure width, connected through a rigid joint to the superstructure centerline, with defined longitudinal, transverse, and vertical nonlinear response at each end. The longitudinal response accounts only for the gap and the embankment fill response, where passive pressures are produced by the abutment back wall pushing into the fill. The elastic-perfectly-plastic (EPP) backbone curve is assigned properties obtained from the Caltrans SDC (2004). The shear resistance of the bearing pads is ignored. In the transverse direction, a zero-length element is defined at each end of the rigid link with an assigned EPP backbone curve representing the backfill, wing wall, and pile system response. The resistance of the brittle shear keys and distributed bearing pads is ignored in this model for simplicity. In the vertical direction, an elastic spring is defined at each end of the rigid link, with a stiffness corresponding to the bearing pads stiffness. The stiffness of the pads in compression and tension is assumed to be identical.

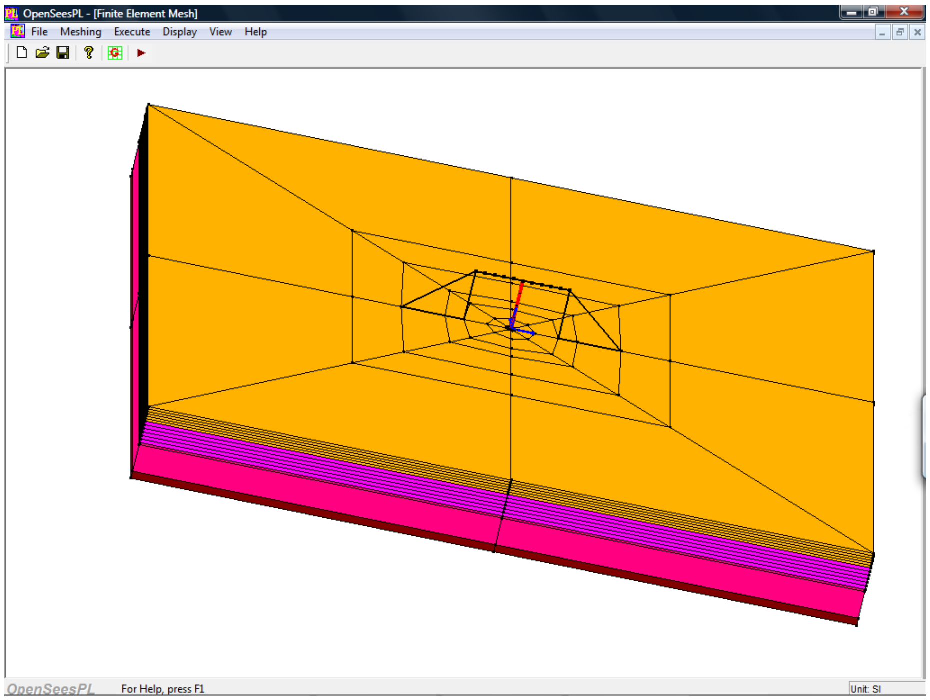


Figure 7. Perspective view of 3D soil-ground domain with different soil layers.

The spring abutment model also includes longitudinal, transverse, and vertical nonlinear response. The longitudinal response is based on the system response of the elastomeric bearing pads, gap, abutment back wall, abutment piles, and soil backfill material. Prior to impact or gap closure, the superstructure forces are transmitted through the elastomeric bearing pads to the stem wall, and subsequently to the piles and backfill, in a series system. After gap closure, the superstructure bears directly on the abutment back wall and mobilizes the full passive backfill pressure. A system of zero-length elements is distributed along two rigid elements oriented in the transverse bridge direction, connected together by bearing pads. The transverse response is based on the system response of the elastomeric bearing pads, exterior concrete shear keys, abutment piles, wing walls, and backfill material. The transverse stiffness and strength of the backfill, wing wall, and pile system is calculated using the same modification of the SDC procedure for the longitudinal direction as defined for the simplified abutment model. The stiffness and strength are distributed equally to the two extreme zero-length elements of the second rigid element. The vertical response of the abutment model includes the vertical stiffness of the bearing pads in series with the vertical stiffness of the trapezoidal embankment. The abutment is assumed to have a nominal mass proportional to the superstructure dead load at the abutment, including a contribution from structural concrete as well as the participating soil mass. More details on the abutment models can be found in Aviram et al. (2008).

The ground domain is specified by: i) definition of the zone occupied by the pile in terms of its diameter, ii) definition of ground below the bridge, iii) definition of the domain to support the approach ramp and abutment zones, iv) definition of outer free-field lateral extent, and v) definition of ground layer depth. Along the depth, the thickness of laterally uniform soil layers may be specified in order to capture the site stratification characteristics. Properties of each layer are defined by selection from an available set of soil models and model properties. A shear-beam type boundary condition is employed for the soil domain, i.e., at any given depth, displacement degrees of freedom of both sides of the longitudinal (and transverse) boundaries are tied together (both horizontally and vertically) to reproduce 1D shear wave propagation mechanism effect.

Performance-based Earthquake Engineering Quantities

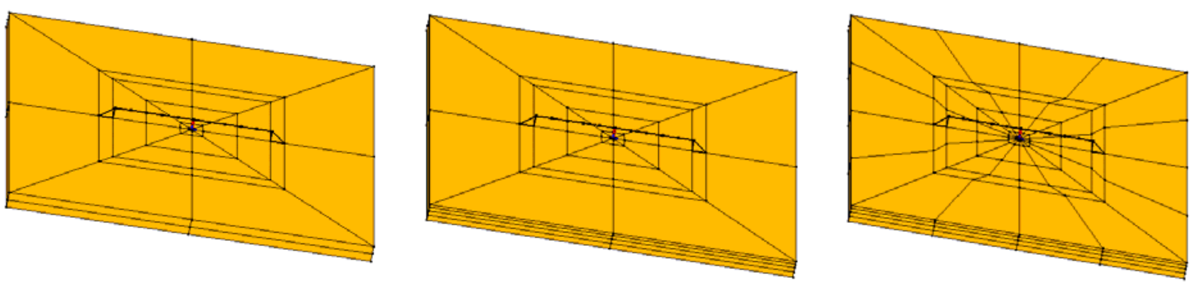
During transient analysis for each ground motion (either as a single ground motion analysis or as part of the ensemble of PBEE motions), response quantities are tracked at each time step. The response quantities of interest are tied directly the PGs that are used in the PBEE analysis for assessing damage and repair. Each major bridge component is grouped into a PG. Each PG contains a collection of components that reflect global-level indicators of structural performance and that contribute significantly to repair-level decisions. The notion of a PG allows grouping several components for related repair work; therefore PGs are not necessarily the same as the individual load-resisting structural components. The PGs (and associated EDP) used in the pilot study are: PG1 - maximum column drift ratio, PG2 - residual column drift ratio, PG3 - maximum relative left deck-end/abutment displacement, PG4 - maximum relative right deck-end/abutment displacement, PG5 - maximum absolute bearing displacement (left abutment), PG6 - maximum absolute bearing displacement (right abutment), PG7 - approach residual vertical displacement (left abutment), PG8 - approach residual vertical displacement (right abutment), PG9 – left abutment residual pile cap displacement, PG10 – right abutment residual pile cap displacement, and PG11 - column residual pile cap displacement.

Discrete damage states (DS) are defined for each performance group. Each damage state has an associated repair method that also has a subset of different repair quantities (Qs). Once the Qs have been established for a given scenario (damage to different PGs), the total repair costs can be generated through a unit cost function. In addition, an estimate of the repair effort can be obtained through a production rate for each Q. The user has the ability to modify the default values specified for all of the repair quantities per damage state, unit costs, and production rates. More information on the derivation of the default DSs, Qs, unit costs, and production rates can be found in Mackie et al. (2007). For the purposes of the user interface, an estimate of the replacement cost of the bridge is automatically generated based on the square footage of the deck and the Caltrans Comparative Bridge Costs (CBC) data, corrected to be consistent with the year 2007 cost data used in the calibration of the unit costs. The CBC includes a 10% mobilization cost but does not include any costs for demolition or removal of existing infrastructure.

Pilot Single-Bent Bridge Case Study

The most prevalent ordinary construction types for new California bridges were selected for a study on the relationship between bridge construction cost and design ground motion level (Ketchum et al., 2004). Of the eleven typical types and configurations, two continuous, five-span, straight, post-tensioned, cast-in-place, box girder bridges on monolithic piers were selected for previous PEER studies (Mackie et al., 2007). Common to both of the bridge types selected are 3 internal spans of 45.7m (150 ft) and 2 end spans of 36.6m (120 ft). The decks are 1.83m (6 ft) deep CIP post-tensioned box girders that accommodate 2 lanes of traffic plus shoulders and barriers on both sides. All bents contain a single monolithic column. The bridge designated as Type 1 has 6.7m (22 ft) clear column heights, and the bridge designated as Type 11 has 15.2m (50 ft) column heights. The abutments at both ends of the bridge are of a seat type. For the pilot investigation using OpenSeesPL PBEE, the same dimensions were selected, but using only a single bent (two spans of 45m).

Three model variants were investigated as part of the pilot study. The finite element meshes for each of the three models is shown in Figure 8. The bridge and ground model geometry and material properties remained consistent throughout the three variants; however, the effect of different levels of mesh refinement were investigated, primarily in the soil domain. The properties varied between the three meshes are detailed in Table 1. The discretization of the soil mesh in the vertical direction required similar subdivision of the pile/column elements (as indicated in the table).



a) Mesh 1 b) Mesh 2 c) Mesh 3

Figure 8. Finite element meshes for three models considered.

Table 1. Characteristics of FE models.

Mesh #	Number of elements for Bridge Column	Number of elements for Soil Domain
1	6	104
2	8	208
3	8	416

PBEE Study Results

From these 3 simulations, the salient PBEE response outcomes are listed in Table 2. The disaggregation of cost by performance group is performed at a PGV = 167 m/sec. Note, PGs 7 and 8 do not contribute in this study because no nonlinearities are included in the approach ramp. One benefit of the PBEE method utilized in the user interface is that the intensity-dependent repair cost or repair time response can be obtained for each simulation. One convenient method of expressing cost is in terms of the repair cost ratio (RCR), or the repair cost normalized by the replacement cost. The mean and ±1 standard deviation RCR values for each of the three scenarios considered in the pilot study are shown in . The user has the ability to specify three intensity hazard levels at a selected site (e.g., 2% probability of exceedance in 50 years). The derivative of a fitted power-law hazard curve is then integrated with the RCR-IM probabilistic relationship to obtain a loss hazard curve, as shown in for the three scenarios.

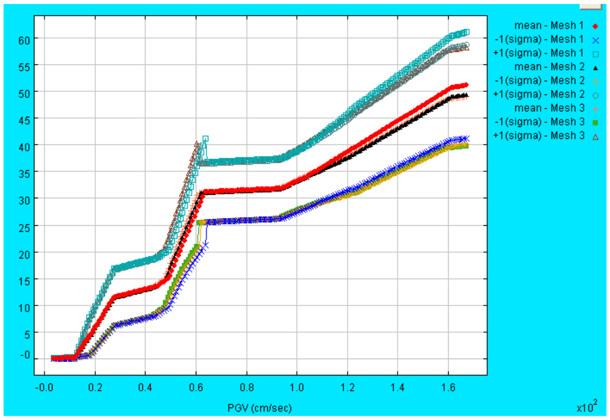


Figure 9. Intensity-dependent RCR (in percent)

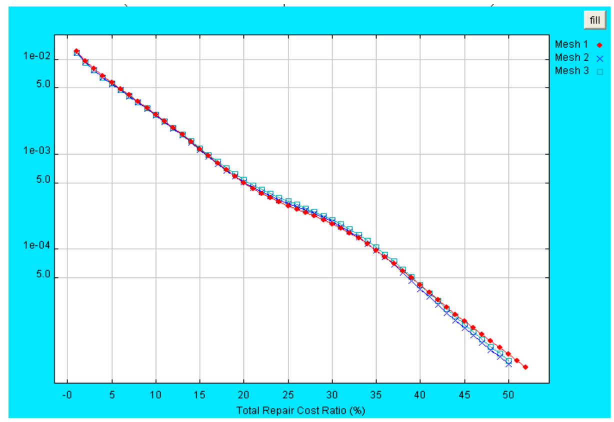


Figure 10. RCR hazard curves (vertical axis: mean annual frequency of exceedance)

Table 2. PBEE outcomes (PGV = 167 cm/sec).

Model	Mesh 1	Mesh 2	Mesh 3
Total repair cost	$914,464	$884,221	$877,933
PG 1	$36,623	$35,618	$35,516
PG 2	$75,494	$46,218	$39,259
PG 3	$182,175	$182,189	$182,223
PG 4	$182,175	$182,189	$182,223
PG 5	$91,147	$90,028	$90,488
PG 6	$91,147	$90,028	$90,488
PG 9	$5	$5	$5
PG 10	$4	$4	$4
PG 11	$255,694	$257,942	$257,727

It may be observed that all of the PBEE results are relatively insensitive to the mesh discretization scheme chosen in this particular study. While each mesh type has an impact on individual ground motion response, when considering the ensemble of responses, the repair consequences are not impacted dramatically. Nevertheless, it appears that the coarse mesh results in a more conservative repair cost estimate. This trend is consistent between all meshes, PBEE outcomes, and intensity levels. The primary difference is an increase in repair due to increased residual displacements of the column. For a very coarse soil mesh, the residual displacements predicted are minimal and therefore do not contribute greatly to the overall response.

Conclusions

By coupling a refined graphical user interface for modeling of bridge-ground FE models with a PBEE framework, OpenSeesPL PBEE has enabled more transparent access to performance-based assessment for typical highway bridges. The elements of this new framework were presented in this paper along with a pilot investigation of a single-bent bridge using three different levels of mesh refinement. The pilot study is one illustration of the parametric studies that are possible with the new interface. Sufficient flexibility is provided in the interface that the user can select their own ground motion suites, bridge geometry and configuration, constitutive models, unit costs, and production rates. The new interface allows the user, be it a researcher or a practitioner, to focus on the PBEE outcomes and decision variable drivers rather than becoming inundated with the details of ground motion selection, FE modeling, constitutive model parameter calibration, and damage and repair data selection.

Acknowledgments

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