Performance-based earthquake assessment of multi-span bridge systems including soil-pile-structure interaction

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ABSTRACT
From the earthquake engineering point of view, highway bridges are an integral part of critical lifelines, and as such have received much research attention. Smooth operation of the highway system after a major earthquake facilitates critical rescue and recovery efforts. Disruption of function can also result in substantial, negative wide-scale economic consequences. In this paper, a performance-based earthquake engineering (PBEE) framework is coupled with nonlinear time history analysis to generate probabilistic estimates of repair costs and repair times for multi-span highway bridges. Previous PBEE evaluations of bridge systems were limited to superstructure response. However, for more reliable estimation of bridge performance, consideration of the coupled bridge-foundation-ground response is essential. Therefore, the p-y approach is used to account for lateral stiffness of the underlying pile foundation and the resulting soil-foundation-structure interaction using nonlinear springs. Reinforced concrete highway bridges are considered, with multi-column bents founded on different sites of varying stiffness and strength profiles. The deck, columns, abutments, and foundation response mechanisms are integrated within a unified framework. Systematic evaluation of the global system response is conducted under a wide range of expected earthquake input shaking scenarios This study is related to decision-making strategies for evaluating the contribution of different bridge components to overall performance of the bridge system. The presented results show that the damage state and repair quantities related to the foundation and abutments are among the most significant parameters.

1 INTRODUCTION

As critical lifelines in transportation network, highway bridges response assessments have received much research attention. After an earthquake, there are two main elements necessary for quantifying and minimizing the economical loss, the cost associated with the damage of the structure (i.e. bridge) and the consequences due to the loss of functionality. However, the uncertainties in the structures are unavoidable, not only the seismic hazard in a geographic location, but also the expected loss are random. Therefore, the probabilistic approach is preferred in order to quantify the damage and loss.

Previous performance evaluations of bridge systems were performed solely to the superstructure response (Solberg et al. 2008). However, after major earthquakes the seismic response of bridges is influenced by many factors including the soil-structure interaction effects. In addition, the unrealistic modeling of the foundation not only will affect the response results needed for a proper design option, but it will also affect the repair cost and time estimations. Therefore, and in order to fully address the post-earthquake repair cost and downtime, considering the response of the bridge-foundation-ground is essential.

The performance-based earthquake engineering (PBEE) assessment can be used to compare the effectiveness of different bridge design options and evaluate the performance of existing bridges for different hazard levels. Therefore, it requires the complete structural response from the finite element (FE) run before performing the probabilistic analysis to estimate the repair costs and times required to restore the structure to its original function.

The focus of this paper is the probabilistic seismic performance and loss assessment of a three-span bridge structures supported on pile foundations which are founded in two sites of varying stiffness and strength profiles, rigid rock case and cohesive soil strata with gradually increasing shear moduli and undrained shear strengths. The objectives are to perform an analysis to the bridge using an integrated bridge-foundation-ground model and a performance-based assessment framework for both bridge-ground cases. In addition, to compare the results obtained for each case in terms of structural response, resulting damage and repair costs and times.

2 PBEE ENABLING TECHNOLOGIES

The study was implemented using MSBridge, a PC-based graphical pre- and post-processor (user-interface) for conducting nonlinear FE simulations for multi-span bridge systems as well as complete PBEE assessment. FE computations in MSBridge are performed using OpenSees (http://opensees.berkeley.edu).

In the user-interface, an implementation of the Pacific Earthquake Engineering Research (PEER) Center's performance-based earthquake engineering framework is used to quantify the loss by generating probabilities of exceeding different levels of repair cost and repair time.

3 PERFORMANCE-BASED EARTHQUAKE ENGINEERING FRAMEWORK

By using the total probability theorem, the desired probability distributions are computed by dividing the task into several intermediate probabilistic models with different sources of randomness. The hazard model that uses the input ground motions to determine the intensity measures (IMs), the demand model that uses the response after the FE run to determine the engineering demand parameters (EDPs), and the damage model that connects the EDPs to damage measure (DM) and then to repair quantities (Qs). This methodology is based on linearization of the damage model and called local linearization repair cost and time methodology (LLRCAT), more details about this methodology can be found in (Mackie et al., 2010).

To facilitate the disaggregation when applying the total probability theorem, the bridge system is broken down into a collection of structural components that act as a global-level indicator of structural performance and contribute to repair level decisions. Structural components are then classified into performance groups (PGs) and the damage in each of the PGs is characterized the damage states (DSs) that are defined by critical values of the EDPs so that higher DS corresponds to higher consequences. For example, the DS1 in the column is the cracking, while the DS4 is the complete failure.

The repair methods for each PG require a combination of several repair quantities (Qs). However, the repair quantities for all PGs are then combined with consideration of the correlation between components, for example not to double count the excavation for different damage states. The estimated Repair costs (RC) are obtained by multiplying the unit cost (UC) by the repair quantity. Similarly, the estimated Repair times (RT) are obtained through a production rate (PR), more details about the DSs, Qs, UCs, and PRs derivation can be found in (Mackie et al. 2011). Finally, approximate repair costs and repair efforts are estimated from the assembly of discrete damage states from all PGS. Table 1 shows the PGs (and associated EDPs and DSs) used in this study.

This table is similar to the table found in (Mackie et al. 2012) but with 14 PGs instead of 11 PGs to account for three spans bridge configuration.

4 SIMULATION MODELS

The bridge-ground configurations are taken from (Mackie et al. 2012) but with two column bents instead of single column bent and the same superstructure properties. However, Figure 1a shows the FE model of the bridge-foundation mesh in 3D view.

Figure 1b shows the model elevation view with dimensions. The nonlinear beam-column elements are used to model the columns with circular cross section of 1.22 m in diameter, 11.9 m wide two-cell box girder for the deck, five elements are used to discretize the deck along each clear span as shown in Figure 1. In addition, Type 1 shaft with continuous column cross section for the column and the pile shaft was used in this paper for simplicity. The clear column heights (above grade) are 6.71 m and 20 m below the grade. Furthermore, the seat-type abutment was used in this study. To simulate the abutment model, the spring abutment model was used. It includes a gap, elastomeric bearing pads, abutment back walls, abutment piles, and soil backfill material. The two column bents bridge is still considered as Ordinary Standard Bridge (OSB) since the span lengths are less than 90 m as per Caltrans Seismic Design Criteria (Caltrans 2006).

5 GROUND PROFILES

Two ground properties are studied in this paper, the rigid foundation and the shallow soft ground stratum. However, the same superstructure geometry and materials are used for both cases

The conventional rigid base response will be similar to the previous PBEE evaluations where only the

Table 1. Performance groups and associated engineering demand parameters and damage states

PG	Engineering Demand Parameters	DS1	DS2	DS3	DS4
PG1 & PG2	Maximum column drift ratio	Concrete cracking	Concrete cover spalling	Longitudinal reinforcing bar buckling	Failure
PG3 & PG4	Residual column drift ratio	Threshold	Thicken pier	Re-center column	Failure
PG5 & PG6	Maximum relative longitudinal deck-end/abutment displacement for left & right abutments	Joint cleaning	Joint seal assembly replacement	Backwall (retaining wall) spalling	Backwall (retaining wall) failure
PG7 & PG8	Maximum lateral bearing displacement for left & right abutments	Yield	Failure	-	-
PG9 & PG10	Approach residual vertical displacement for left & right abutments	Pavement repair	Asphalt concrete regrade	Rebuild	-
PG11 & PG12	Residual lateral pile displacement at ground surface for left & right abutments	Add pile threshold	Enlarge foundation	-	-
PG13 & PG14	Column residual pile displacement at ground surface	Add pile threshold	Enlarge foundation	-	-

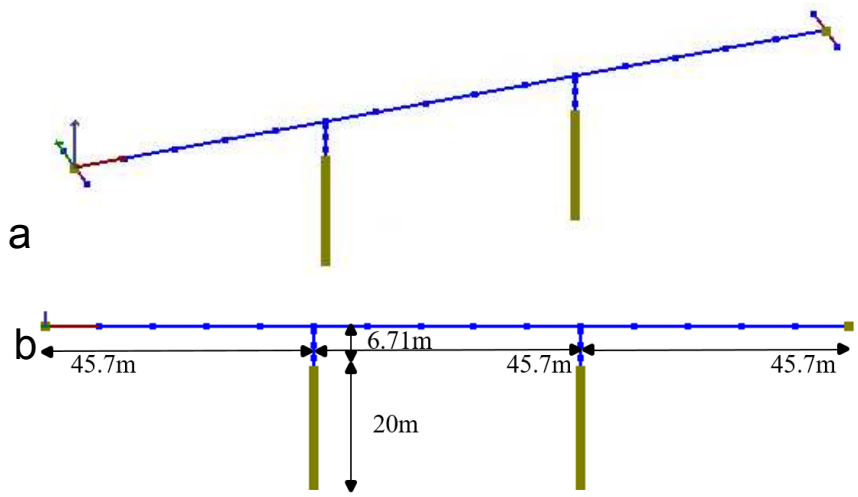

Figure 1. Finite element mesh: (a) 3D view, (b) elevation view

superstructure was studied. On the other hand, the shallow soft soil with gradually increasing strength which will bring the effect of the foundation into the picture. In addition, it is worth noting that the p-y approach is implemented to define the strength of the soil below grade.

5.1 Bridge-ground case study 1: Fixed-base/rigid soil

This scenario will not trigger the damage states of the bents and abutments foundation; hence, no repair cost nor repair times will accumulate in the post-earthquake consequences.

It is introduced in this paper to compare with the previous PBEE evaluation and to show the importance of addressing the case with flexible foundation. This scenario is computed consistently with the same interface (i.e. MSBridge). The stiff soil case was implemented using fixed connections at all column bases (in 3 translational and 3 rotational directions and the nodes of the abutment models are fixed as well.

5.2 Bridge-ground case study 2: Benchmark soil profile

By using the same bridge configuration as in the first case study. This scenario will account for the soil-structure interaction, where the bridge-ground configuration is not rigid in all the cases. Thus, the damage states of the bents and abutments foundations will be triggered and contribute in the repair cost and effort estimations. The soil profile was generated that includes cohesive soil strata with gradually increasing shear moduli and undrained shear strengths, this profile is called a benchmark soil profile (Caltrans 2003). The properties of the layers are shown in Figure 2 with the ground water table at 2.4 m.

The p-y springs approach was used to model the column foundation response. This approach provides an easier, less complex, and more applicable way to run the FE model comparing with the approach used previously in (Mackie et al. 2012) where the FE model includes a complex and very fine mesh. In this p-y springs foundation representation, lateral soil resistance is provided as the p-y springs interact with the pile shafts. As such, the values of these curves were converted to proper horizontal soil springs. However, the p-y curves were obtained from LPILE using the soil properties shown in Figure 2. Using this approach the FE and PBEE analyses were performed.

Figure 3 shows the *p-y* curves (at selected depths). Unlike the rigid case, the foundation repair cost and efforts will be included in the repair costs and times.

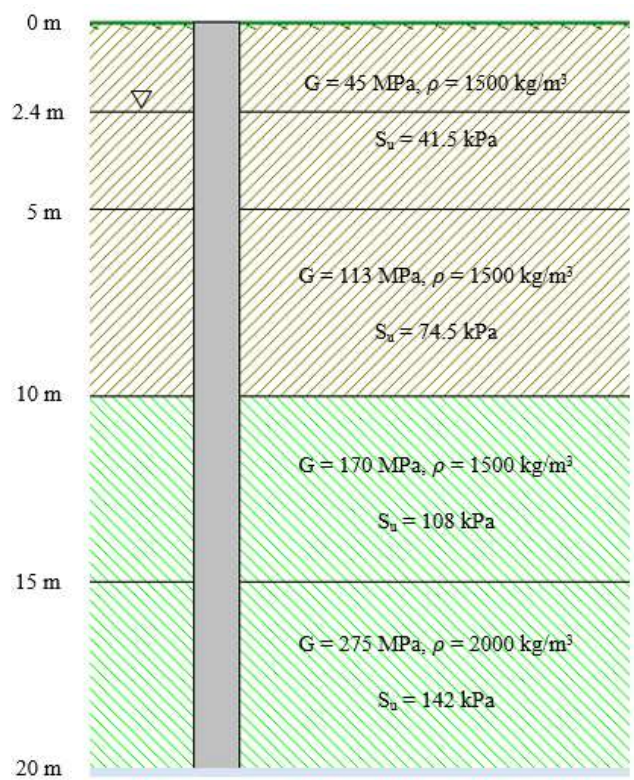

Figure 2. Soil layer properties for case 2 (benchmark)

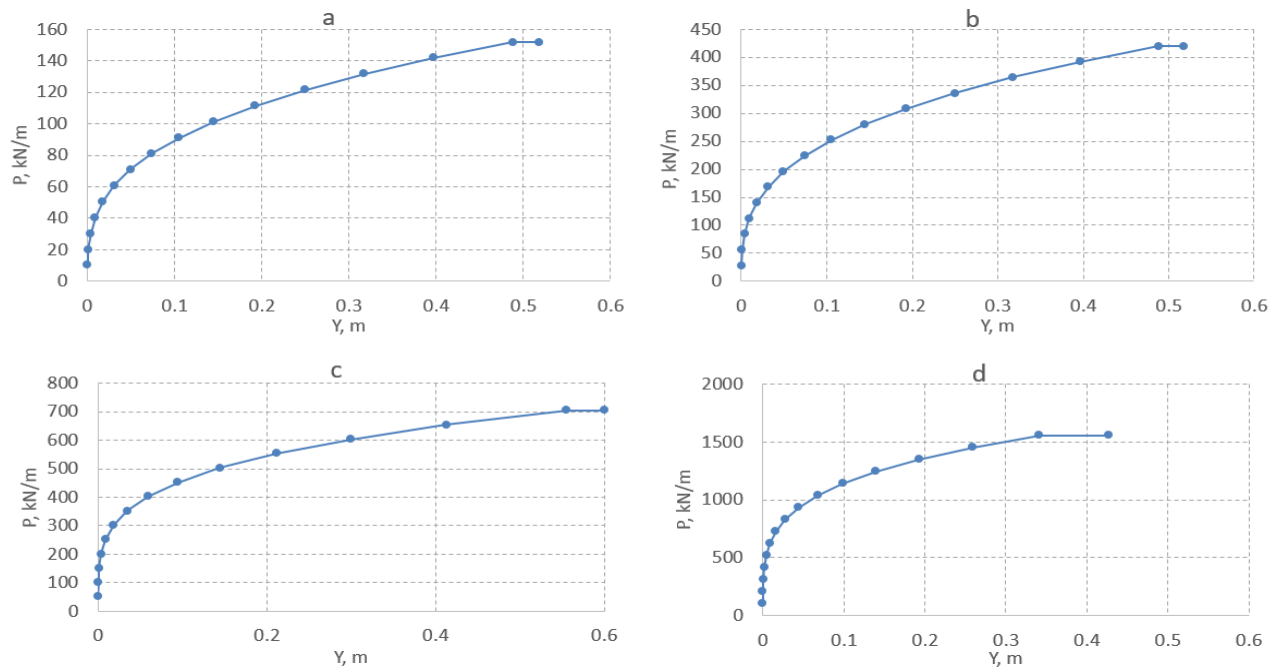

Figure 3. P-Y curves for case 2: (a) at 0 m depth; (b) at 5 m depth; (c) at 10 m depth and (d) at 20 m depth

6 GROUND MOTIONS FOR PBEE ANALYSIS

A set of 100 ground motions that is selected to be representative of seismicity in typical regions in California is employed in the FE and PBEE analyses. Five bins of 20 motions are used to divide the motions based on characteristics: (i) moment magnitude (M_w) 6.5-7.2 and closest distance R 15-30 km (denoted LMSR bin), (ii) M_w 6.5-7.2 and R 30-60 km (LMLR), M_w 5.8-6.5 and R 15-30 km (SMSR), (iv) M_w 5.8-6.5 and R 30-60 km (SMLR), and (v) M_w 5.8-7.2 and R 0-15 km (denoted Near bin.

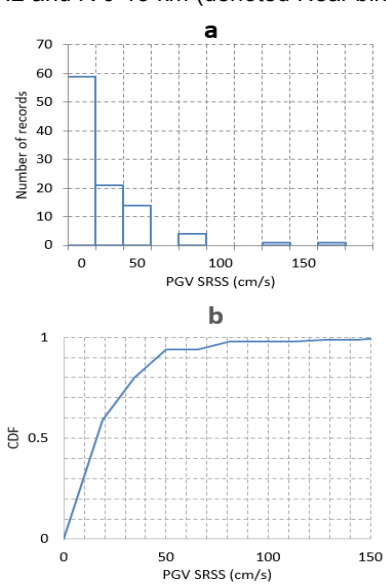

Figure 4. PGV distribution for the SRSS of two lateral ground motion components

In addition, Figure 4 shows the distribution of horizontal PGV SRSS values and its cumulative distribution function (CDF) of the employed ground motions. However, more details about the ground motions data set can be found in (Mackie & Stojadinovic 2005).

7 NONLINEAR TIME HISTORY ANALYSIS

Nonlinear time history analysis (THA) was conducted for the 100 input motions using MSBridge. Uniform base excitation was studied using each of these input ground motions. Rayleigh damping was used with a 2% damping ratio (defined at the periods of 1 and 0.167 second) in the nonlinear THA. For the time integration scheme, the Newmark average acceleration method (γ = 0.5 and β = 0.25) was employed. Variable time-stepping scheme (`VariableTransient`) was used in the conducted Nonlinear THA. The starting value for each step was 0.02 second (the time step of the input motions)

8 PBEE RESULTS

By applying the ground motion records to the bridge, a corresponding repair cost and repair down time is computed. These values are then correlated with an Intensity measure that represents each particular motion employed. MSBridge allows for 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.

Figure 5 shows the mean repair cost ratio (RCR) against Peak Ground Velocity (PGV) as the intensity measure (IM). (RCR, the normalized repair costs are obtained by using the repair cost ratio between the cost of repair and the cost of replacement that does not include

demolition. It is noted that the unit price of repair cost used in this study is based on $156 per square foot of bridge deck). However, as can be seen, the repair costs do not begin to accumulate until a PGV of approximately 20 cm/s is reached for both cases, since the motions of lower PGV result in Engineering Demand parameters (EDPs) that do not cause the first damage state to occur, with no cost/repair consequences.

In addition, the costs are increasing rapidly in the range of PGV between 40 and 60 cm/s where more repair quantities are triggered, then reach a plateau beyond which no significant additions to the repair quantities are being triggered. Furthermore, a conclusion can be drawn from Figure 5 that the rigid base case accumulates less cost since the repair quantities of the foundations are not contributing in the post-earthquake consequences. For example, at intensity of 100 cm/s around 15% is required for the rigid scenario, while 25% is required for the other scenario.

Furthermore, the RCR loss model shown in Figure 5 alone does not show the cost breakdown in terms of the PGs, therefore, Figure 6 shows the largest contributing PG in the expected repair cost which is the abutment PG. It can be concluded from Figure 6 that as the soil profile is getting more flexible, the repair quantities associated with the abutment are triggered at lower intensities.

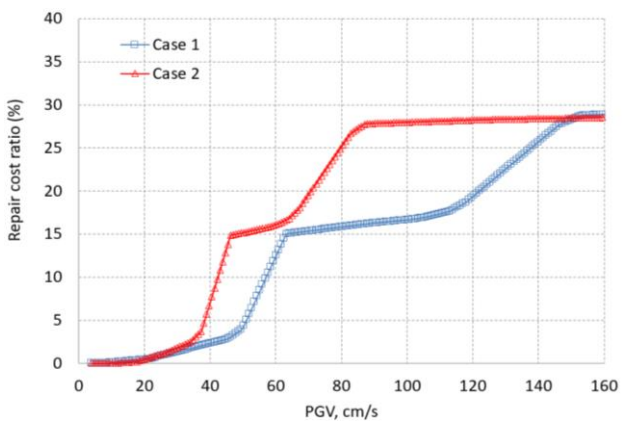

Figure 5. RCR models for each of two scenarios

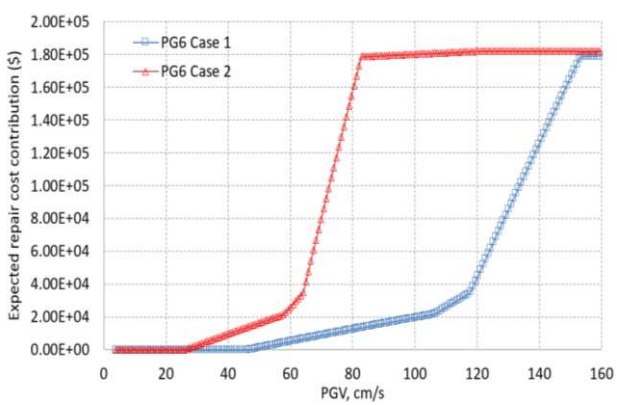

Figure 6. The primary contributing PG (in terms of expected repair cost) compared for both scenarios

Similarly, Figure 7 shows the repair time in crew working day (CWD) against PGV. The jumps in the repair times for both cases are due to different Damage States that require more repair effort. In addition, it is worth noting that the rigid case considered reaches a plateau around 50 cm/s, while the flexible case around 40 cm/s beyond which repair efforts do not increase. In addition, there is no substantial increase at higher intensities which implies that failure DS has not been reached and therefore no column replacement is required.

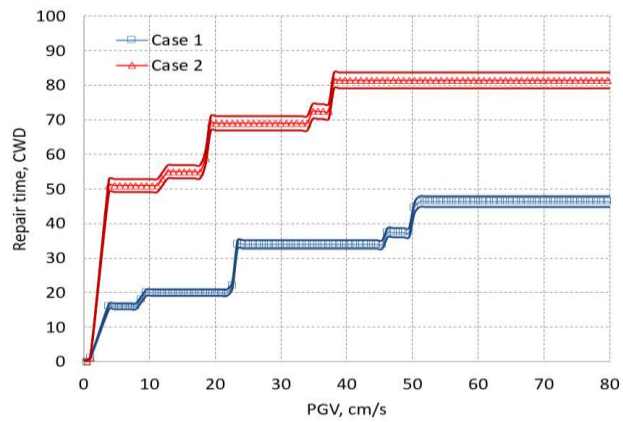

Figure 7. Repair time (CWD) loss model (with probabilistic moments) for both scenarios

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. For the case study, the resulting PGV estimates were 160, 80, 10 cm/s, respectively, which are consistent with infrequent events of larger magnitude such as the central US.

On this basis, Figure 8 shows the mean annual frequency (MAF) of exceedance versus RCR (showing that there is a higher chance to get RCR of at least 10% than 20%). In addition, Figure 8 provides a clear illustration of the increased cost hazard associated with the decreasing strength and stiffness of the soil profiles. Similarly, Figure 9 shows the MAF of exceedance versus Repair Time (RT). The plateau in Figure 9 corresponds to the jump in Figure 7 where the repair time jumps at a specific IM (i.e. PGV).

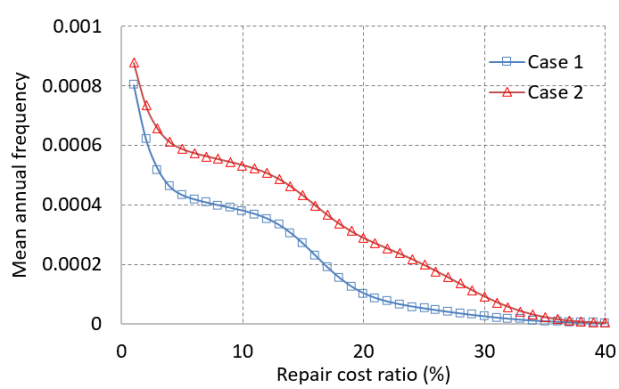

Figure 8. Repair cost ratio hazard curve

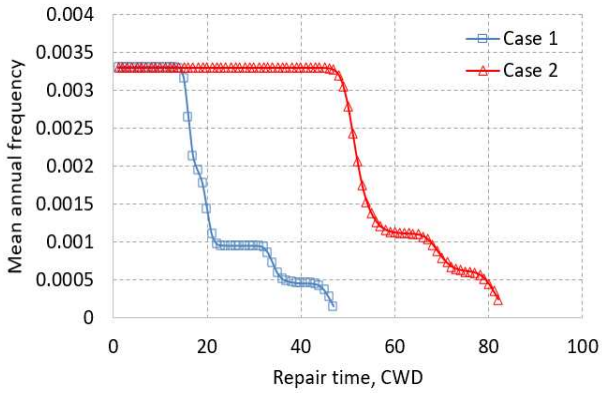

Figure 9. Repair time hazard curve

9 DISCUSSION AND CONCLUSIONS

Performance-based evaluations of bridge systems need to consider bridge-foundation-ground to get more reliable estimation of bridge performance. The OpenSeesMSBridge is utilized in this paper to Combine the finite element analysis and the PBEE framework in a graphical user environment that has enabled such studies to be more easily implemented. The repair costs and repair times for a three-span ordinary standard bridge founded on two different soil scenarios, rigid rock and weak upper soil strata are studied and compared. Furthermore, the seismic hazard for a particular seismicity scenario (geographic location) is studied.

The presented results in this paper show that the repair quantities do not start to accumulate until the first damage states are triggered, then increase as the intensity measure increases. However, the increase in the repairs is not linear, since more damage states are triggered in higher intensity measures. A plateau will be reached at a specific intensity measure value where no more repair costs/times are added, which implies that the seismicity region will not suffer higher consequences. The weaker upper soil strata case results in higher post-earthquake repair. The rigid case does not address the consequences of the foundation repairs that may be dominant in some scenarios.

In addition, the foundation and abutments repairs are among the most significant parameters that contribute in the expected costs and times. Therefore, a special attention should be drawn to these quantities especially when considering the coupled soil-structure systems.

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