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Seismic Damage Assessment of Bridges Using Response Surface Model

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ABSTRACT

In the conventional approach for analytical fragility analysis of structures, a single set of seismic fragility curves is developed and utilized for risk assessment of structures having the same classification. While this approach is appropriate for assessment of the average seismic risk to a large population of structures, seismic upgrading of arbitrarily selected structures for risk reduction should not be based on the average structure risk because the physical configuration details differ among individual structures having the same classification. This paper proposes a new method for rapid estimation of the seismic damage to track-on steel-plate-girder (TOSPG) bridges so that a seismic risk analysis of a TOSPG bridge with an arbitrary physical configuration can be effectively performed without significant loss of time and effort. The response surface modeling (RSM) technique is utilized for probabilistic estimation of seismic damage to a TOSPG bridge without the need to repeat a large number of time-history analyses. First, the variables that describe the physical configuration of the bridge are identified. Among the variables, the ones that significantly affect the seismic damage of the bridges are selected as the input variables for the response surface model. The response surface model is then developed to create second-degree polynomial equations for estimation of the anticipated values for the median and variation of the seismic damage due to a specified level of earthquake loading. The accuracy of the established RSM model was statistically validated. The approach developed in this study allows for flexible estimation of the seismic damage and fragility of arbitrarily selected structures in a given class because the simulation is performed not with a number of time-history nonlinear dynamic analyses but with simple numerical equations.

1. INTRODUCTION

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Track-on Steel-Plate-Girder bridges, or TOSPG bridges, consist of more than 40% of the bridge inventory in the conventional railway lines of Korea [1]. Because many of the TOSPG bridges could not satisfy the seismic performance level required by the current seismic code, these bridges need to be seismically upgraded or replaced in order to achieve an acceptable level of seismic risk. In order to seismically retrofit the bridges in an effective way, the current seismic risk level should be estimated first and retrofit should be performed such that the target risk level could be achieved.

Seismic fragility curves are effective tools for seismic risk analysis of structural systems [2,3]. Expected damage of structures within a specified time period could be estimated through seismic risk analysis where fragility analysis of the structures is coupled with seismic hazard analysis. Seismic risk analysis of structures provides engineering basis for making decisions on the seismic design or retrofit. Decision makers, such as central or local government agencies, could make plans and allocate budget in order to reduce the seismic risk considering both the benefits and the costs based on the seismic risk analysis.

The seismic vulnerability of TOSPG bridges have been analytically estimated in a past study [4] where the fragility curves have been developed for a TOSPG bridge that is representative of the TOSPG bridges in Korea. This approach is consistent with the conventional approach for fragility analysis where a single set of seismic fragility curves are developed and utilized for structures in a same classification (e.g., multi-span simply supported concrete girder bridges or multi-span continuous steel girder bridges [5], low-rise unreinforced masonry buildings [3], special moment frame steel structures [6], etc.). In this approach, it is implicitly assumed that all the structures corresponding to the same classification undergo same level of damage under a seismic excitation. While this approach would be appropriate for assessment of an average seismic risk of a large population of structures, seismic upgrading of individual structures for risk reduction should not be based on the average risk of the structures because the detailed physical configurations of structures would be different among the individual structures in a same classification. For example, a wide variety of the number of spans and/or the pier height exists among the multi-span simply supported concrete bridges. The width, the height, or the number of stories of low-rise unreinforced masonry buildings is not constant from building to building. The differences in the configurations would result in the differences in the structural responses because the stiffness, strength, mass and failure mode of the structures are dependent upon the physical configurations. As a result, the seismic behaviors and damage levels of the structures within a classification are not always the same. Therefore, risk reduction activities such as seismic retrofit of structures should be carried out based on the seismic fragility and risk of individual structures. For a structure portfolio that consists of a large number of structures, however, performing a seismic fragility analysis for each of all the individual structures is highly impractical, because in general a fragility analysis is computationally expensive as it requires a numerical simulation with a large number of time-history analyses.

This study proposes a new method for rapid estimation of seismic damage of TOSPG bridges, such that a fragility analysis of a TOSPG bridge with an arbitrary physical configuration could be effectively performed without significant loss of time and effort. The response surface modeling (RSM) technique is utilized for probabilistic estimation of seismic damage of a TOSPG bridge without repeating a large number of time-history analyses. First, the variables that describe the physical configuration of the bridge are identified. Among the variables, the ones that affect the seismic damage of the bridges are selected as the input variables for the response surface model. The response surface model is then developed to create second-degree polynomial equations for estimation of the anticipated values of the median and the variation of the seismic damage due to a specific level of earthquake loadings. This method allows the seismic fragility curves to be effectively generated because the simulation is performed not with a number of time-history nonlinear dynamic analyses, but with simple numerical equations. The approach developed in this study allows flexible estimation of the seismic damage and fragility of arbitrarily selected structures of a class.

2. TOSPG BRIDGES IN KOREA

In Korea, there are more than 1,200 TOSPG bridges on the conventional railway lines. Most of the bridges have been constructed before 1970 without consideration of the seismic loadings. Typical characteristics of TOSPG bridges are, (1) tracks are installed directly on top of the steel plate girders, (2) linear steel bearings are typically used, (3) the bridges are generally for single track railway lines, and (4) the piers are gravity type piers made of plain concrete (see Figure 1 for typical TOSPG bridges). As mentioned before, seismic fragility curves of a typical TOSPG railway bridge in Korea were analytically developed in a past study [4] through a simulation with a series of nonlinear dynamic time-history analyses. The target structure was a 5-span simply-supported TOSPG bridge with the pier height of 12m and with the span length of 12.83m-13.56m. The fragility curves developed for this particular bridge is shown in Figure 2.

The physical configuration of TOSPG bridges, however, varies much among the individual bridges. Figure 3 shows the frequency plots of the number of spans, pier height, and the span length developed for 211 TOSPG bridges that are selected as relatively important for the Korean railway network. The number of spans of the TOSPG bridges ranges from 1 to more than 20, all of which are simply-supported, single-track bridges. The frequency plot is left-skewed indicating that 5-span bridges are most prevalent. The span length ranges from 5m to more than 25m, but more than 75% of the bridges' span lengths are in the range of 11m-12m, 13m-14m, or 20m-21m. This might be due to a standardized manufacturing process of the girders. The pier height ranges from 4m to more than 15m.

Figure 1 Typical TOSPG Bridges

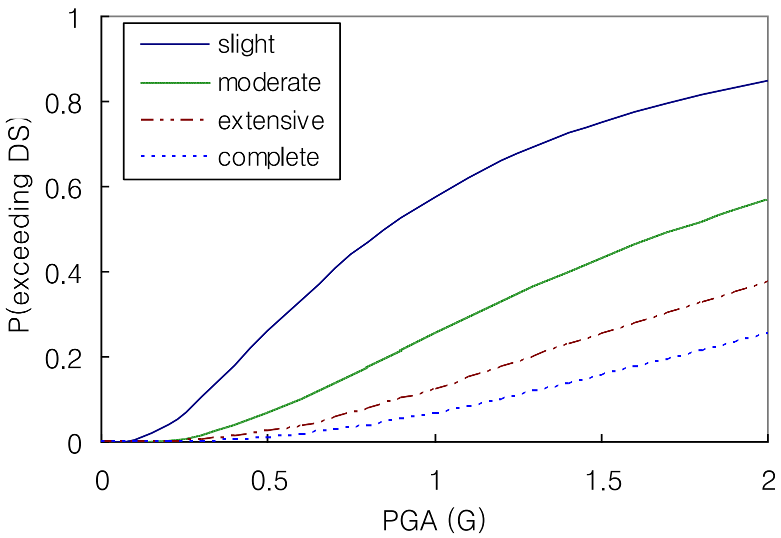
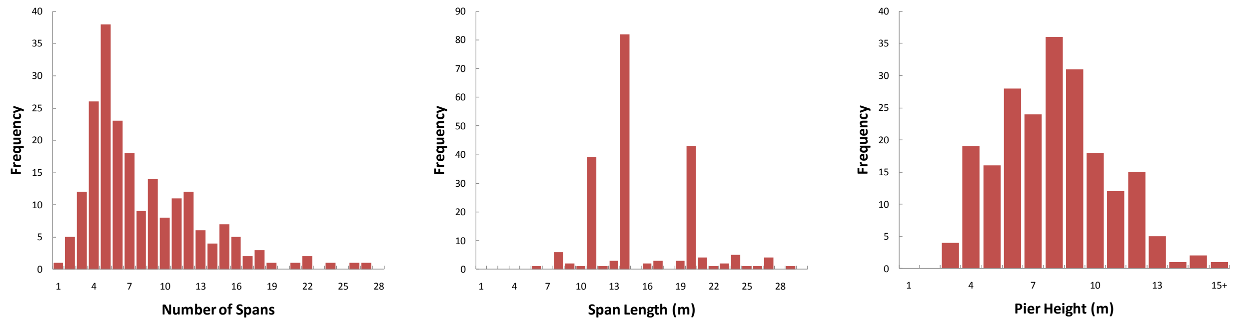


Figure 2 Fragility Curves for a Representative TOSPG bridge [4]



(a) Number of Spans (b) Span Length (c) Pier Height

Figure 3 Frequency Plots of Number of Spans, Span Length, and Pier Height

3. SEISMIC DAMAGE ANALYSIS OF TOSPG BRIDGES

A typical seismic fragility analysis involves probabilistic estimation of the structural responses. The uncertainties inherent in the system capacity and earthquake demand are represented by random variables and a suite of earthquake inputs. The structural responses are then probabilistically estimated through a statistical analysis. The most common approach for the statistical analysis is through Monte Carlo simulation [7]. Typically, the Monte Carlo simulation technique requires a large number of random samples to be evaluated. This means that nonlinear time-history analyses should be run as many times as the number of random samples. In practice, this approach is computationally expensive although it theoretically allows accurate estimation of the probability of damage of structures. Especially when the fragilities of a number of individual structures in a region should be individually estimated and a representative fragility curve could not be used, this approach becomes even more impractical.

The probabilistic damage analysis method proposed in this study is to replace the time-consuming nonlinear time-history analysis step with a much simpler meta-model, such that the computational load could be significantly reduced and eventually the analysis could become much faster. A response surface modeling method is utilized as the meta-modeling technique in this study. In this section, the seismic damage assessment approach for TOSPG bridges is introduced followed by the procedure for utilization of the response surface modeling technique for rapid assessment of the seismic damage.

3.1 Seismic damage assessment of TOSPG bridges

Structural modeling of TOSPG bridges for computational time-history analysis is developed based on the work by Park et al. [4]. Bearings, piers, and abutments are defined as the critical components of TOSPG bridges in terms of the seismic behavior, and their nonlinear behaviors are incorporated. OpenSees [8] is utilized as a software framework for the modeling and computational simulation in a 3-D manner.

The seismic damage of a TOSPG bridge due to an earthquake excitation is defined based on the damages of the critical components. Seismic damage of each component is defined in terms of its maximum value of a response such as displacement or curvature. The level of damage each component undergoes is then determined based on pre-defined damage states or limit states. For development of a response surface model for damage estimation, using a quantitative damage measure defined in a continuous real number scale is more advantageous than using categorized damage states for description of the seismic damage. A generic damage measure proposed by Park et al. [4] is utilized here for measuring the system level damage of TOSPG bridges in a continuous real number scale, as defined in Table 1. Assuming that the most damaged component defines the system level damage, the system damage due to an earthquake can be estimated by identifying the maximum value among the components' damage values. More details about the generic damage measure could be found in Ref.

Table 1 Definition of generic damage measure

Limit States (j)	D_G
No Damage	$0.0 \le D_G < 0.25$
Slight Damage	$0.25 \le D_G < 0.5$
Moderate Damage	$0.5 \le D_G < 0.75$
Extensive Damage	$0.75 \le D_G < 1.0$
Complete Damage	$1.0 \le D_G$

As mentioned earlier in this paper, same level of seismic damages may not be expected for structures that are in a same classification but with different physical configurations. This section demonstrates this statement by performing stochastic damage analysis of selected TOSPG bridges. Two TOSPG bridges with different configurations, denoted as T1 and T2, are modeled first, and their seismic damages are probabilistically estimated for a demonstrative purpose. They are both simply-supported, single-track TOSPG railway bridges. T1 is a 8-span bridge with the pier height of 7m, whereas T2 is 3-span with the pier height of 3m. Damage probabilities of the structures are obtained through a number of nonlinear dynamic time-history analyses. For input seismic loadings, the ground motions for Korean peninsula developed by Han and Choi [9] are used. Among the ground motions, 20 ground motions corresponding to 2% probability of exceedance in 50 years are selected. Uncertainties inherent in the bridge system are represented by taking the variations of certain material related properties. Following the suggestions by past studies [6] the concrete compressive strength is assumed to be normally distributed with a mean of 31.4MPa and a coefficient of variation of 0.14; the damping ratio is assumed to be uniformly distributed between 0.025 and 0.075. In addition, it is assumed that the direction of the earthquake loading is uniformly distributed between 0 and 180 degrees to the longitudinal axis of the bridge.

Damage probability due to a certain level of input earthquake is obtained using Latin Hypercube sampling (LHS) technique with a sample size of 20 in order to raise efficiency in the simulation. In other words, 20 stratified values of the random variables mentioned above are randomly combined with the 20 input ground motions. Taking each combination as an input, a dynamic time history analysis is performed and the maximum system damage, represented by generic damage measure, is calculated. The values of the maximum system damage from the 20 simulations are used for probabilistic description of the system damage for a given level of ground motion. This step of estimating the damage probability is repeatedly performed over the range of the input earthquake intensity from 0.1g to 1.5g with an increment of 0.3g. Figure 4 shows the damage probabilities of T1 and T2 corresponding to various levels of input earthquake. It also shows the damage trends of the two bridges by plotting the mean damage values. For both cases, the damage apparently increases as the input earthquake level increases. Comparing T1 and T2, T1 undergoes much more damage than T2 does under a same level of input earthquake. The mean damage values of T1 are larger than those of T2 by a factor of 2 to 5. We can also observe that the variance of the damage of T1 is larger than that of T2. This shows that although in the same classification, the seismic damages of bridges with different configurations could be significantly different. This also exemplifies that the fragility curves that are developed to represent the seismic damage of a class of structure may not be directly used for seismic damage estimation of an individual structure.

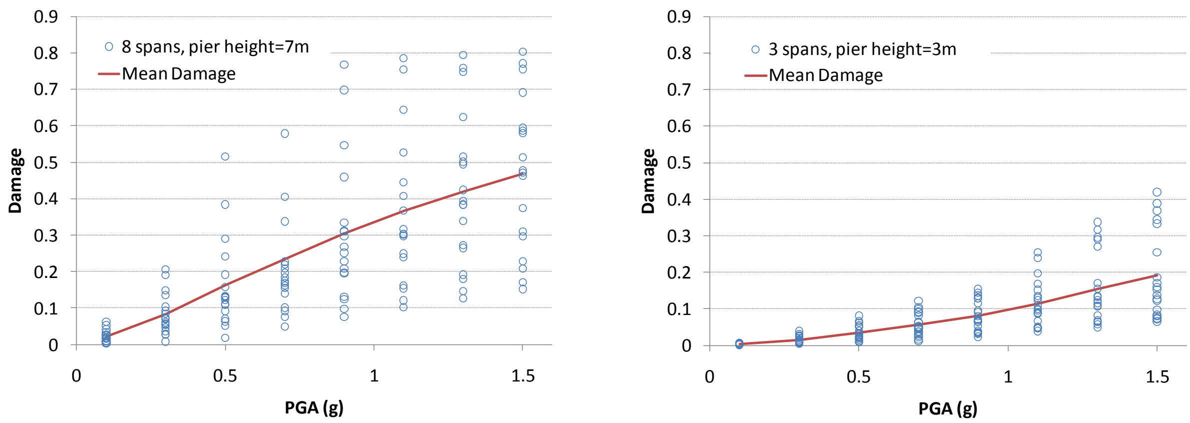


Figure 4 Damage probabilities of T1 and T2

3.2 Input Parameters Screening for RSM

As mentioned earlier, the response surface model in this study is intended for probabilistic estimation of the seismic damage of a structure. That is, the mean and the variation of the seismic damage of a TOSPG bridge with specific physical configurations due to a specified level of input earthquake could be calculated through the response surface model. Therefore, the physical configuration-related parameters and the earthquake intensity (PGA in this study) become the input variables for the response surface model. The variables that describe the physical configuration of a TOSPG bridge are the number of spans, the span length, and the pier height. It should be noted that the width of the superstructure is not considered as an input parameter because most of the TOSPG bridges are for single track railway lines and the variation in the width of the superstructure is minimal.

A screening process is needed to improve the efficiency in constructing the response surface model by identifying some variables that have the most influence on the output among the initial input variables. Reduced number of input variables results in reduced number of analyses that are needed to be run to construct the DOE (design of experiment) table.

The ranges of the input variables are defined such that at least more than 90% of the population could be included in the range. Table 2 shows the ranges of the input variables by specifying their lower and upper bound values along with the median values. Note that the PGA ranges from 0.1g to 1.5g, because the fragility curves are to be developed over the range.

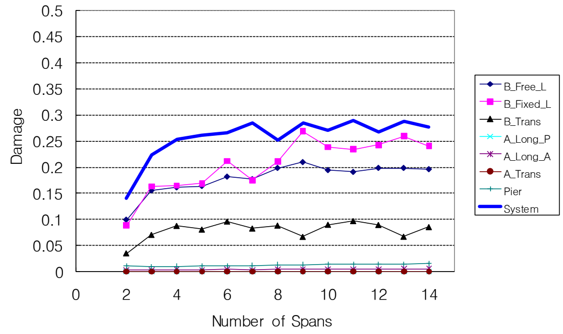
Table 2 Valid range of input variables

	min	med	max
Span length (m)	5	14	23
Number of spans	2	8	14
Pier height (m)	2	7	12
PGA (g)	0.1	0.8	1.5

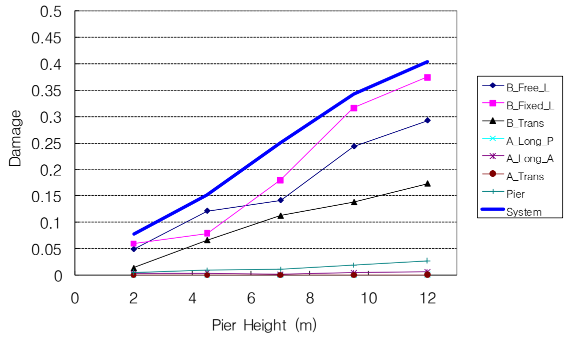
The parameter screening could be conducted by performing sensitivity analyses on the input variables. The output variable is monitored by changing one variable from its minimum to its maximum value, while other variables are fixed at their median values. For each instance of the seismic damage estimation, the same method described in the previous section is used and the median value of the system damage distribution is obtained.

Figure 5 shows the sensitivity plots of the seismic damage of TOSPG bridges upon changes of the input variables. In Figure 5(a), it is observed that for the bridges with the number of spans of 2 to 4, the seismic damage increases as the number of spans increases. The seismic damage level of the bridges with higher numbers of spans, however, is not much sensitive to the number of spans. Figure 5(b) and Figure 5(c) show that the pier height and the PGA are strongly correlated with the seismic damage because the seismic damage apparently shows almost linear increase with the pier height and the PGA. The span length, however, does not affect much to the seismic damage, as the system damage remains almost constant (Figure 5(d)). The damage trends of the components are also shown in the plots. It should be noted that most of the damages occur in the bearings compared to the piers and the abutments, which is consistent with the observation of the past study [4].

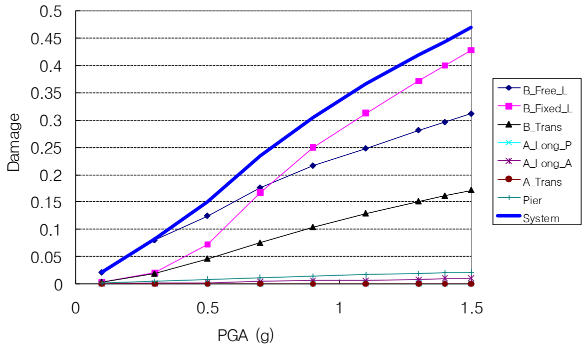
From the parameter screening test conducted, the span length is ruled out from the list and the number of span, pier height, and PGA are selected as the input variables for the response surface model for seismic damage estimation of TOSPG bridges.



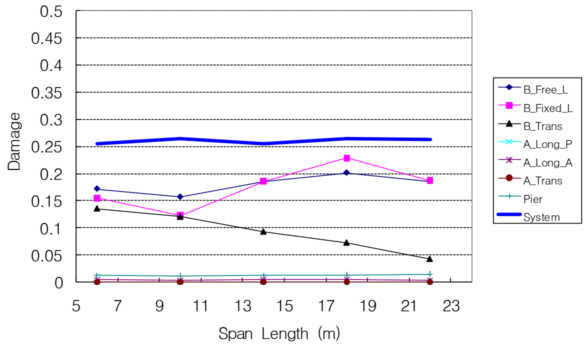
(a) Number of spans



(b) Pier height



(a) PGA



(d) Span length

Figure 5 Damage trend due to change in input variables

4. RESPONSE SURFACE MODEL

For each design of experiment (DOE) case, the DG are assumed to be lognormally distributed with parameters μ and σ (i.e., ln(DG) is normally distributed with the mean μ and standard deviation σ). Two response surface functions are generated from least-square regression analysis: one is for the lognormal distribution parameter both μ' and another is for σ'. Each response surface function is in a second-order polynomial form as follows.

$$Y = \beta_0 + \beta_1 X_1 + \beta_2 X_2 + \beta_3 X_3 + \beta_4 {X_1}^2 + \beta_5 X_1 X_2 + \beta_6 {X_2}^2 + \beta_7 X_1 X_3 + \beta_8 X_2 X_3 + \beta_9 {X_3}^2 \quad (1)$$

where X1, X2 and X3 indicate the number of span, pier height and PGA, respectively, and βis are evaluated as shown in Table 3.

Table 3 Evaluation of β_is

	μ'	σ'
0	5.26053	0.75486
1	0.35168	0.05408
2	0.83805	0.00238
3	1.70762	-0.03681
4	-0.31344	-0.10304
5	0.04524	0.00074
6	-0.29913	0.02418
7	-0.08955	-0.06847
8	-0.10642	-0.11461
9	-0.91328	-0.00577

What this means, for example, is that for a specific bridge that has 8 spans (X1=0), 7-meter pier height (X2=0) and is subjected to a PGA of 0.8g (X3=0), the response surface functions indicate that the bridge's DG (in a unit of per thousand) is Lognomally distributed with parameters μ'(X1=0, X2=0, X3=0) = 5.2605 and σ' (X1=0, X2=0, X3=0) = 0.7549.

The plots for both μ' and σ' do not carry much physical meaning, so surface plots of exp(μ'), which is a median value of the lognormally distributed DG, against different combinations of the number of span, pier height, and the PGA are shown in Figure 6 for illustration purpose instead.

At a given level of the PGA, a Monte Carlo simulation evaluates the response surface functions for the generic damage taking into account both the randomness in the brides' configurations (number of spans and pier height) and the record-to-record dispersion due to earthquakes. After a large number of simulated samples (10,000 samples in this work), a probability distribution of the generic damage can be obtained and probabilities of exceeding the four DG thresholds (the state of 'no damage' was omitted) are calculated. Repeating the simulation for all levels of PGA results in the fragility curves for a population of the TOSPG bridges in Korea as shown in Figure 7.

5. STATISTICAL VALIDATION OF THE RSM

There are a number of statistical measures that can be used to verify linear regression models. However, statistical testing is inappropriate in this case where outputs are computed by deterministic computer analyses rather than physical experiment trials because the random error term does not exist. The simplest measure for verifying the model adequacy in the deterministic computer experiments is a coefficient of determination (R2). The value of R2 characterizes the fraction of total variation of the data points that is explained by the fitted model. However, the R2 can be misleading since it always increases as more input variables are added. Alternatively, an adjusted-R2 (R2A), which takes into account the number of parameters in the model, can be used for evaluating the goodness-of-fit of the model. The R2A of 0.98 and 0.44 were calculated for the response surface functions of μ' and σ’, respectively. The high value of R2A for μ’ which is directly related to the median of DG, indicates a good fit.

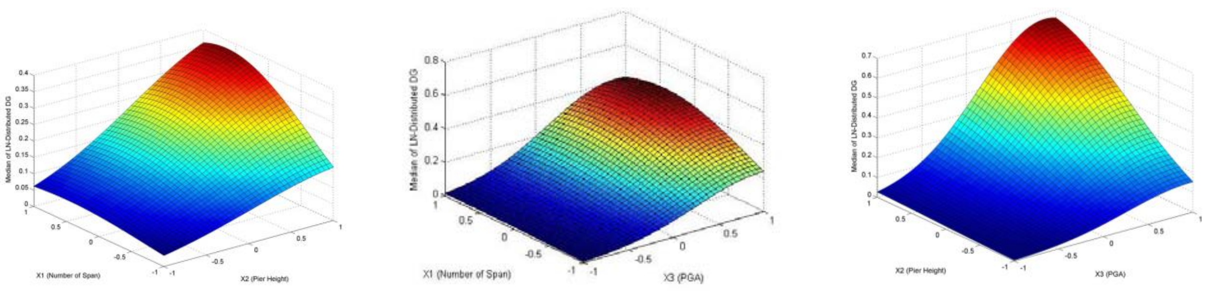
Even though the R2A value explains how well the model fits to the predefined experimental points, the value does not indicate good prediction potential of the model for regions other than at the design points. In order to verify the overall accuracy of the response surface models, statistical tests at additional random data points within the design space must be performed. Those tests include the Average Absolute Error (%AvgErr), the Maximum Absolute Error (%MaxErr), and the Root Mean Square Error (%RMSE). For the purpose of these statistical tests, 60 additional combinations of input variables (N=60) are generated at random and those statistical measures are computed as follows:

$$\%\mathrm{AvgErr} = 100 \cdot \frac{\frac{1}{N} \cdot \sum_{i=1}^{N} \left| y_i - \hat{y}_i \right|}{\frac{1}{N} \cdot \sum_{i=1}^{N} y_i} \tag{2}$$

$$\%\mathrm{MaxErr} = \underset{i}{\mathrm{Max}} \left[100 \cdot \frac{\left| y_i - \hat{y}_i \right|}{\frac{1}{N} \cdot \sum_{j=1}^{N} y_j} \right] \tag{3}$$

$$\%\mathrm{RMSE} = 100 \cdot \frac{\sqrt{\frac{1}{N} \cdot \sum_{i=1}^{N} \left(y_i - \hat{y}_i \right)^2}}{\frac{1}{N} \cdot \sum_{i=1}^{N} y_i} \tag{4}$$

where yi is the actual values of μ’ and σ’ from nonlinear dynamic analyses and ŷi is the response-surface-predicted values of μ’ and σ’.
%AvgErr and %RMSE quantify errors, as percentages, that the values predicted by the response surface models depart from the actual values on an average basis,
while %MaxErr indicates the case where the predicted value deviates most from the actual value. The calculated statistical measures from the 60 random points are shown in Table 4.



(a) Pier height vs. number of span (b) PGA vs. number of span (c) Pier height vs. PGA

Figure 6 Response surface for selected combinations of the input parameters

It can be seen that the errors in the model are quite low, especially in the case for μ’, suggesting good prediction accuracy of the response surface models in this study. Finally, in addition to the use of these statistical measures, visual assessment of the residual and the correlation plots are performed to determine the model accuracy as shown in Figure 8. This correlation plot confirms that the simpler response surface functions can provide a good approximation for the much more complex nonlinear dynamic analysis.

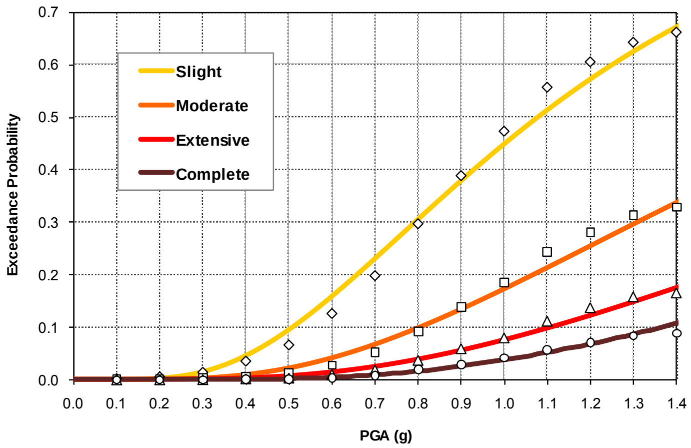


Figure 7 Fragility curves of TOSPG bridges in Korea generated from RSM

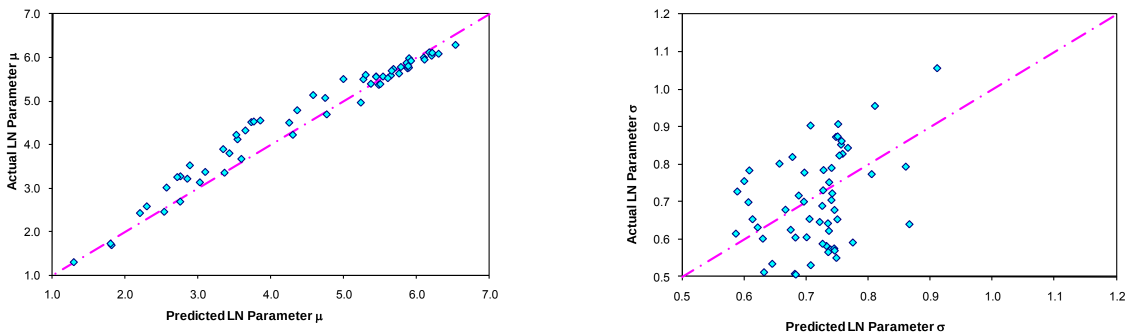


Figure 8 Correlation plots for μ’ and σ

Table 4 Statistical measures for μ’ and σ’

	μ'	σ'
%AvgErr	5.23	15.14
%MaxErr	16.89	33.05
%RMSE	7.09	17.59

6. CONCLUSIONS

This study proposes a new method for rapid estimation of seismic damage of Track-on Steel-Plate-Girder (TOSPG) bridges, such that a fragility analysis of a TOSPG bridge with an arbitrary physical configuration could be effectively performed without significant loss of time and effort. The response surface modeling (RSM) technique is utilized for probabilistic estimation of seismic damage of a TOSPG bridge without repeating a large number of time-history analyses. The number of spans, pier height, and input ground motion magnitude are selected as the input variables, whereas the span length is ruled out as it turned out to have almost no effect on the seismic damage of the structure. Assuming that the seismic damage of TOSPG bridges are lognormally distributed, second-degree polynomial response surface models are developed such that the mean and the standard deviation of $\ln(D_G)$ could be anticipated. The RSM developed is statistically validated through various validation measures. Finally, the fragility curves of TOSPG bridges in Korea are generated using the RSM. The approach developed in this study allows the seismic fragility curves to be effectively generated because the simulation is performed not with a number of time-history nonlinear dynamic analyses, but with simple numerical equations. It also allows flexible estimation of the seismic damage and fragility of arbitrarily selected structures of a class.

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