SEISMIC RISK ASSESSMENT OF REINFORCED CONCRETE BRIDGES IN WASHINGTON STATE

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ABSTRACT

The average age of reinforced concrete bridges in the state of Washington is 48 years, encompassing more than 100 years of design and construction techniques, which reflect evolving views on seismic risk and mitigation. As such, there is great uncertainty as to the actual seismic resistance of existing bridges. Having a simple and accurate process for assessing the seismic vulnerability of a bridge can help identify vulnerable design details and address them, either through retrofit or replacement.

In this paper, a recently proposed performance-based adaptive methodology for the seismic assessment of bridges is used to analyze six reinforced concrete case study structures extracted from the Washington State Department of Transportation database. Using the finite element program RUAUMOKO-2D, models of existing WSDOT bridges were created and pushover analyses were run. By defining damage states of interest and determining the associated displacement profiles, peak ground accelerations (PGAs) of seismic events were then correlated to the damage states. By combining fragility curves encompassing the probabilistic distribution of the PGA at which a damage state might occur with site hazard risks, Risk Indices were calculated for all bridges considered. The bridges were ranked in order to contextualize the Risk Index values. The results of the assessment process were then verified by comparing them to the results of NLTH analysis.

Keywords: Seismic assessment; Reinforced concrete bridges; Adaptive pushover analysis; Capacity spectrum; Non-linear time-history analysis

1. INTRODUCTION

Evolving understanding of both the seismic risk faced in the Puget Sound region and the response of the built environment to earthquakes has highlighted the need to determine the seismic vulnerability of existing structures. Washington has dozens of active faults and fault zones. These include the Seattle fault and southern Whidbey Island fault zone, which cross under major cities, and the Cascadia subduction zone, the largest active fault affecting the whole Pacific Northwest.

In this high seismic hazard context, the age range of bridges in the State spans more than 100 years, a large share of which were designed prior to the 1971 San Fernando earthquake, which precipitated major changes to national seismic design philosophy. Many bridges were thus designed and built in times when the understanding of the seismic behavior of bridges was somewhat limited and the local hazard was poorly understood (Christman, 2017). In response to the high seismic hazard characterizing the area, and in a proactive effort to reducing the seismic vulnerability of the Washington State bridge network, a retrofit program was launched in 1991 by the WSDOT (Babaei and Hawkins, 1991). Bridges with design details known to be vulnerable were prioritized for retrofit in the early phases of work. In order to impose further order on the remaining bridges, WSDOT identified a network of so-called “Seismic Lifeline Routes” essential to recovery of the Puget Sound region in the event of a major earthquake. Current retrofit efforts are concentrated on

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bridges on the Seismic Lifeline Routes, with 85 Lifeline bridges awaiting retrofit as of June 2015 (Washington State Department of Transportation, 2015). However a comprehensive and systematic risk assessment of the entire bridge inventory requires significant effort and resources and has not yet been carried out. A comprehensive assessment of the Washington State network is necessary to move forward with bridge prioritization efforts efficiently and logically.

Using non-linear time history analysis to assess the vulnerability of the State bridges would evidently produce the most accurate results. Unfortunately, using non-linear dynamic analysis represents an impractical solution to perform large scale assessments (i.e. several thousands of bridges) as they take a nontrivial amount of skill to conduct, and are computationally intensive (Federal Highway Administration, 2014). In this context, the ability to perform a simple analysis that yields realistic results is key to simplifying the process of analyzing bridges for seismic vulnerability.

In response to this, a number of simplified assessment methods have been developed over the years. Examples include the Capacity Spectrum Method (CSM) (ATC 1996), the Displacement Coefficient Method (DCM) (ATC 2005) and the N2 method (Fajfar, 1999; CEN 2005). Recently, a performance-based adaptive methodology for the evaluation of the seismic vulnerability and seismic risk of highway bridges was proposed by Cardone et al. (2011). This methodology was here identified as promising and potentially suitable to conduct large scale assessment of the Washington State bridge stock.

In a preliminary effort to test the potential and applicability of this assessment method, identifying benefits and drawbacks, this paper presents the results of a risk assessment campaign conducted on six real case study structures extracted from the Washington State Department of Transportation database. The outcome of the assessment process is validated via NLTH analysis.

2. DESCRIPTION OF THE RISK ASSESSMENT METHODOLOGY

This section provides a brief overview of the assessment methodology employed and a definition of the bridge damage states considered throughout the project. A more extensive discussion can be found in the work of Cardone et al. (2011) and Cardone (2014).

2.1 Summary of the Assessment Methodology

The assessment process adopted in this study is the performance-based adaptive methodology for the seismic evaluation of reinforced concrete bridges, referred to as IACSM, proposed by Cardone et al. (2011). A detailed description of the fundamentals of the method can be found in the original publication and only a brief review is provided here. The key aspect of the method is the definition of a number of Performance Levels (PLs), for which the seismic vulnerability and risk of the bridge need to be evaluated. Each PL is associated to a number of Damage States (DSs) of the critical members of the bridge. More details on PLs and DSs are provided in Section 2.2.

Once desired PLs and DSs have been selected, a simple numerical model of the bridge is subjected to an adaptive pushover analysis (Figure 1 (a)). For each DS reached during the pushover response, the total base shear and the lateral displacement of all masses are recorded and used to create a single degree of freedom (SDOF) substitute structure, characterized by an effective mass, m_e, effective stiffness, k_e, effective displacement, Δ_e, and effective damping ratio, ξ_e. The effective properties of the equivalent SDOF system are calculated using the equations summarized in Figure 1 (b). More details on the formulation of the substitute structure can be found in the literature (e.g. Cardone, 2014; Cardone et al. 2011; Priestley et al. 2007).

The next step, outlined in Figure 1 (c), consists of calculating the earthquake intensity levels (PGA) corresponding to the attainment of the selected DSs, using over-damped elastic response spectra as demand curves. The overdamped spectrum is obtained through a damping reduction factor η which, in this case, is computed using the Eurocode 8 equation (CEN, 2004). The damping factor is limited to values greater than 0.5, which corresponds to ξ_e values less than 26%.
The seismic vulnerability of the bridge is then expressed by means of fragility curves (Figure 1 (d)), derived based on the PGA values previously obtained for each DS. The seismic risk index (Figure 1 (f)) is finally evaluated as convolution integral of the product between the fragility curves and the seismic hazard curve of the bridge site (Figure 1 (e)).

2.2 Damage State Definition

In order to define coherent Performance Levels (PLs), it is convenient to group DSs of different structural systems into broadly defined categories based on the extent of damage experienced. For this study, four PLs were defined as follows, based on the work of Cardone (2014):

- **PL1** Minor Damage. Easily repaired, requiring little to no downtime before reopening.
- **PL2** Moderate Damage. Widespread damage has occurred, but there is still a comfortable margin against collapse, and repair is both physically and economically feasible.
- **PL3** Severe damage. There is some remaining margin against collapse. The damage that has occurred would technically be repairable, but replacement of the structure would likely be more economical.
- **PL4** Collapse Prevention. Global collapse has not occurred, but aftershocks or other additional loading may trigger global collapse. PL4 represents a level of damage that should be avoided.

In defining the DSs of interest, the flexural and shear behavior of piers were both considered. Damage to wingwalls, shear keys and bearings was also taken into account. A more detailed definition of the DSs and PLs adopted in this study can be found in Chrisman (2017).

3. RISK ASSESSMENT OF SELECTED CASE STUDY STRUCTURES

The method discussed in Section 2 is used in this section to assess the seismic risk associated to a set of six reinforced concrete case study bridges. This section is divided in three parts: firstly, the case study structures are briefly introduced; secondly, details of the numerical modelling of the structures are provided and finally, the results of the pushover analysis and of the risk assessment process are discussed.
3.1 Bridge Selection

The case study structure selection was limited to reinforced concrete bridges owned by WSDOT. The Washington DOT owns approximately 3,000 bridges, of which 38% are reinforced concrete. The age of these bridges ranges from brand new to over 100 years old. A wide range of seismic design philosophies were adhered to over the years, so while age is not the ultimate indicator of seismic resilience of a structure, it is a useful metric. Bridges built prior to the 1971 San Fernando earthquake are of particular interest, due to the significant shift in design after the earthquake.

Study bridges were selected to be representative of WSDOT’s overall bridge stock. All are located in Western Washington, which is the region of the state with the highest seismic risk. A variety of design years are represented, in order to capture the variety of design philosophies employed over time. Straight, non-skewed, symmetric geometries were favored. Additionally, since current condition of structure was not specifically accounted for in this analysis, study bridges were screened to ensure that none had received a rating of “Poor” in any structural category during the most recent inspection. Basic information about the bridges used for this study is shown in Table 1, while more details can be found in Christman (2017).

Table 1. Study bridge information.

<table>
<thead>
<tr>
<th>Name</th>
<th>Elevation view (WSDOT original drawings)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bridge 1</td>
<td>I-405 over May Cr (built in 1958)</td>
</tr>
<tr>
<td>Bridge 2</td>
<td>I-5 Express over James and Cherry St (built in 1975)</td>
</tr>
<tr>
<td>Bridge 3</td>
<td>SR 522 over Elliot Rd (built in 1961)</td>
</tr>
<tr>
<td>Bridge 4</td>
<td>I-405 over SR 524 (built in 1968)</td>
</tr>
<tr>
<td>Bridge 5</td>
<td>SR 202 over Kimball Cr (built in 1924)</td>
</tr>
<tr>
<td>Bridge 6</td>
<td>I-90 over SR 202 (built in 1975)</td>
</tr>
</tbody>
</table>
3.2 Modeling of Case Study Structures

The bridge models were obtained following the Structural Components Modelling (SCM) approach (Priestley et al. 1996). Accordingly to this approach, each bridge is idealized as an assembly of structural subsystems whose phenomenological response is provided in the form of the members’ force-deformation relationships. For modeling purposes, the finite element software adopted in this study was RUAUMOKO-2D (Carr, 2004).

Superstructure elements such as deck and cap-beams, were assumed to behave rigidly and were modeled accordingly, using rigid elements. Thus, each bridge was modeled as one (or more, depending on the bridge configuration) rigid diaphragm representing the deck, connected to the foundations and the abutments through a series of non-linear elements representing the bearings, the piers, the shear keys and the wingwalls (as sketched in Figure 2).

Acknowledging that the rigid deck assumption is not always appropriate, in this context it was deemed acceptable for two reasons: on one hand, there is the preliminary nature of this work, mostly focused on evaluating the applicability and reliability of the assessment method proposed by Cardone et al. (2011). On the other hand, deck and foundation elements are normally characterized by low seismic vulnerability, compared to piers, abutments and bearing devices (Priestley et al. 1996). It should be noted that the rigid deck hypothesis is substantiated by current seismic codes (e.g. see EC8-part 2, CEN, 2004), which gives the possibility to assume a rigid deck model when the deformation of the deck in a horizontal plane is negligible compared to the displacements of the pier tops.

The bridges were analyzed considering static or dynamic loads acting only in the transverse direction. The mass of each deck was calculated manually as a function of geometry and material, and lumped in the center of mass of each span. In case the total mass of the pier was more than 1/5 of the fraction of the mass of the deck assigned to that pier, a tributary mass of the pier (equal to the sum of the mass of the pier cap and one third of the mass of the pier shaft) was taken into account. The dead load of the superstructure was also calculated manually, and appropriately proportioned and applied as gravity loads to the pier elements.

All piers were modeled using one-component Giberson beam elements, which were imbued with characteristics reflecting the behavior of all columns making up the pier. The Giberson beam consists of a central elastic section, with rotational springs at one or both ends. This element type has been shown to be both accurate and computationally efficient to model concrete beam and column elements (Mashaly et al. 2011).
The monotonic moment-curvature behavior of the rotational springs was determined using the Matlab-based sectional analysis tool CUMBIA (Montejo and Kowalsky, 2007) and simplified as bilinear. The material models used were the Mander model (Mander et al. 1988) for confined and unconfined concrete, and the Raynor model (Raynor et al. 2002) for steel. Cyclic behavior of the rotational springs was modeled using the modified Takeda hysteresis curve, with parameters of α=0.5, β=0.0, and a reloading stiffness of 1.0, with unloading as described by Emori and Schnobrich (Emori and Schnobrich, 1978), as shown in Figure 2. Plastic hinge lengths, used to determine the position of the plastic hinge springs, were calculated using the equations recommended by Priestley et al. (2007).

Shear behavior of beam-column elements is not captured by RUAUMOKO-2D, so shear strength values were calculated independent of model creation. Total shear capacity of piers was found using the model proposed by Priestley et al. (2007), by summing the various components of shear strength in columns. Equations to calculate the strength contributions from the concrete, reinforcing steel, and axial load respectively can be found in Priestley et al. (2007). Steel jackets, which are retrofitted to columns in order to increase their shear capacity, were added to the columns of Bridge 2. Thus, the added strength from the jackets was calculated as recommended by Priestley et al. (1995).

Laminated elastomeric bearings and steel roller bearings are each present in some study bridges. Bolted elastomeric bearings were modeled as linear elastic springs, with failure achieved at a shear strain of γ=300%. Unbolted elastomeric bearings and steel roller bearings were modeled as non-linear displacement springs with elastic perfectly plastic behavior, as illustrated in Figure 2. Their horizontal strength was computed as the lowest between the shear resistance of neoprene pads and the friction resistance between neoprene and concrete sliding surfaces. For this purpose, the friction coefficient between neoprene and concrete has been taken equal to 0.7. The shear stiffness (K_s) of all elastomeric bearings was evaluated based on the dimensions (cross section area and thickness) of the pads and shear modulus (G) of neoprene. A shear modulus of 1 MPa was assumed

Steel roller bearings are designed to allow differential movement between elements in the longitudinal direction, but are not explicitly intended to contribute to transverse behavior of the structure. Therefore, it was assumed that the behavior of the roller bearings depended solely on the pintles that anchor them to the structural components they connect, and in the event the anchorage fails, the friction between the roller and bearing surfaces. The initial stiffness of the bearings was calibrated to account for both the strength of the pintles and the resistance to friction. The point at which the pintle shear is calculated as a function of the area of the pintle and the material ultimate strength.

Wingwalls at seat-type abutments and shear keys were modeled as nonlinear springs. The damage mode was assumed to be shear at the interface between the wall or key and the structural element to which it is attached. This was modeled using a nonlinear spring with essentially rigid-perfectly plastic behavior. The shear force at which “yield” occurs was calculated using the AASHTO shear-friction interface strength calculation (AASHTO, 2012).

Soil-structure interaction is important to the performance of bridges during earthquakes. However, the focus of this project was on purely structural behavior of bridges, so only basic soil modeling was undertaken. It was assumed that pile- and mat-founded piers and pile-founded abutments were fully fixed at the elevation of the pile cap or mat, regardless of whether the pile cap or mat was buried in soil. This assumption accounts for loss of support if soil subsides, but does not account for possible additional loading that may occur due to soil shifting. Spill-through abutments, which essentially consist of a pier buried in embankments (WisDOT, 2017), were treated like pile-founded piers, assuming that the soil in which the columns were buried was inadequately compacted.

For monolithic abutments without piles, resistance and strength of the soil mass was calculated. Elastic-perfectly plastic behavior was modeled, with sliding occurring after the failure of the soil mass. For this purpose, the recommendations of Karantzakis and Spyroukos (2000) and Priestley et al. (1996) were followed.

3.2 Assessment Process Results

The results of the assessment process described in Section 2 are summarized in Figure 3, where the fragility curves associated to the damage states experienced by each bridge are outlined, along with the type of damage achieved, the damaged structural elements and the PGA level responsible for inducing damage.
It can be seen that the PGA values corresponding to the attainment of the various PLs vary a lot for the different bridges. For instance, the PGAs that induce the first slight damage state (PL1) range between approximately 0.126g and 2.437g, while those corresponding to the first severe damage state (PL3) between approximately 0.111g and 1.873g. One exception is Bridge 4 (I-405 over SR 524) which reaches PL3 at the very low PGA of 0.076g. The reason for this outcome is that, based on the design drawings, no gap between the deck and the wingwalls was provided at the abutments. For this reason, and because of the way the bridge was modeled, the wingwall was the first element engaged in resisting the applied lateral loads and was pushed to failure before the other structural elements could start contributing. It should be noted that although for this bridge the wingwalls appear to give up relatively early in the life of the structure, suggesting high vulnerability of the bridge, much more intense earthquakes seem to be required before other structural elements experience any damage. For example, the first damage following the failure of the wingwalls consists of flexural yielding at the top of one of the piers (which corresponds to PL1), which occurs at a PGA of 0.534g.

This, and the fact that in some instances PLs classified as “more severe” were reached before milder PLs, suggest that a reassessment and a reclassification of the DSs associated to the various PLs may be necessary to avoid misleading outcomes. Perhaps, a case by case definition of the DSs responsible for causing the attainment of a selected PL would be more appropriate (yet more demanding).

The examination of the damage states listed in Figure 3 points out that the behavior of bearings, shear keys and wingwalls can considerably affect the seismic response of rigid deck bridges. It is important that these non-linear elements are correctly modelled, in order to properly estimate the seismic vulnerability of existing bridges, especially when the seismic response of the bridge is governed by the displacement capacity of the bearing devices.

In the last column of the tables in Figure 3, the PGA values on stiff soil, having 2% probability of exceedance in 50 years are reported. A constant PGA value, equal to 0.45g was adopted, on account of all bridges being located in sites characterized by PGA values ranging from 0.4g to 0.5g. The ratio between the assessed PGA values and “PGA 2%/50y” provides a first deterministic measure of the seismic vulnerability of the bridge. It can be noted that in most cases, the PGA corresponding to PL3 for the various bridges is higher than 0.45g. Besides Bridge 4 (already discussed), the only exception is represented by Bridge 3 (SR 522 over Elliot Rd). To some extent, this structure represents a special case in the sample of bridges considered. More specifically, it has piers of different lengths, some being much shorter than others. Calculations showed that the short piers were shear critical and that they would reach PL3 at a PGA of 0.228g.

The fragility curves shown in Figure 3 for all DSs reached, provide a more complete overview of the assessed bridge vulnerabilities. It can be seen that, depending on the bridge configurations and the type and characteristics of the structural elements involved, different DSs and PLs were reached for the different bridges.

In general, the response of bridges without bearings (i.e. with a continuous connection between piers and deck) was governed by the response of the piers. In contrast, the response bridges with deck sitting on bearings was strongly influenced by the characteristics of the bearings with little involvement of the piers, which tended to reach only minor DSs.

As previously noted by Cardone et al. (2011), it is interesting to note that, despite an apparent similarity between some of the bridges, the development of the damage states, as well as the PGA values associated to each PL, can differ significantly among bridges of the same “class”, due to differences in either geometric characteristics (pier heights, joint clearance, etc.) or mechanical characteristics (pier reinforcement ratios, bearing device strength, etc.). This suggests that, although more demanding, to adequately assess the vulnerability of a bridge stock it may be necessary to obtain fragility curves for the individual structures rather for classes of similar bridges, as suggested in some previous studies (e.g. Basöz and Mander, 1999; Nielson and DesRoches, 2007).
In order to contextualize the results for individual bridges, they have been ranked according to their likelihood of reaching a given PL, as a function of the computed risk indices. Those bridges that did not reach a DS consistent with a given PL were exempt from ranking for that DS. Bridges are listed from most vulnerable to least vulnerable.

Due to some bridges reaching PLs out of order, it is unsurprising that no one bridge consistently placed as highly vulnerable or invulnerable across all four PLs. In general, if decisions were to be made regarding the need for any of the bridges to undergo interventions or more extensive investigations, it is evident that the ranking obtained cannot be the sole indicator and further some post-processing of the results would clearly be required. This limitation of the method could possibly be overcome through a better definition of DSs and associated PLs that better reflected the expected performance of the bridge.
### Table 2. Case study bridge ranking.

<table>
<thead>
<tr>
<th>Performance Levels</th>
<th>PL1</th>
<th>PL2</th>
<th>PL3</th>
<th>PL4</th>
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<tr>
<td>Bridge #</td>
<td>5</td>
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<td>4</td>
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<td></td>
<td>1</td>
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4. VALIDATION OF THE ASSESSMENT METHODOLOGY

Time-History Analyses (NTHA) have been carried out to evaluate the accuracy of the results discussed in Section 3. The bridge models for the NTHA were those described in Section 3.2. The analyses were performed in RUUMOKO-2D using the set of 20 real ground motions summarized in Section 4.1. The input ground motions were scaled (on average) to the PGA values provided by the assessment process for all the DSs reached. A total of 5,880 NTHA were conducted. The accuracy of the assessment procedure was evaluated by comparing (for each DS) the bridge displacement profile assessed in Section 3 with the envelope of the maximum bridge displacements (averaged on 20 accelerograms) obtained from NTHA. This comparison was performed by means of the error index introduced by Cardone (2014). A perfect PGA prediction would yield an error index value of 1.0, while error values greater than 1.0 are preferred over values lower than 1.0, as it denotes a conservative prediction.

4.1 Ground Motion Selection

In order to gauge the accuracy of predictions based on pushover analysis, a suitable suite of test ground motions was selected for comparison. The target spectrum was based on the ASCE (ASCE, 2010) Design Spectrum for the worst of the bridge locations. A total of 20 ground motions were selected from the Pacific Earthquake Engineering Research Center (PEER) database, based on site parameters gleaned from USGS hazard tools (a summary is provided in Figure 4). Based on the similarity between site parameters for all bridge locations, a single site was used to generate site hazard for all bridges. More details on the ground motions selected can be found in Christman (2017).

![Figure 4. Target spectrum and spectra derived from the selected ground motions](image-url)
4.2 Comparison with Non-Linear Time History Analysis Results

For each earthquake intensity considered, the key results of interest extracted from the NLTH analyses were the peak displacement profiles of the bridges. These displacement profiles were compared with those predicted via pushover analysis over the course of the assessment process. The general trend was that the pushover analyses provided excellent predictions of the pier displacements, but were less good at capturing the displacement of bridge decks sitting on bearings. When bearings (or other elasto-plastic elements) contributed significantly to the overall response of the bridges, the deck displacements resulted more difficult to predict via static analysis, particularly for high earthquake intensities.

The results pertaining to all bridges are summarized in Figure 5, where they are expressed in terms of the “error index” introduced earlier. Consistently with what discussed above, Figure 5 shows that Bridges 4 and 6 were more prone to inaccurate predictions, accounting for the greatest number of “Poor” ratings. This was attributed to the fact that the response of those two bridges was governed by the behavior of the elastomeric bearings.

![Figure 5. Distribution of calculated error index values](image)

In categorizing the accuracy of predictions, error values between 0.75 and 1.25 were considered “Satisfactory,” values between 0.5 and 0.75 and between 1.25 and 2.0 were considered “Moderate,” and values less than 0.5 or greater than 2.0 were considered “Poor.” The most obvious measure of utility of a scaling method is accuracy of predictions. To this end, 62% of PGA predictions were considered “Satisfactory”, while only 2.7% were classified as “Poor” predictions.

Accuracy is not the only measure of usability of the prediction methods. If predictions are “Satisfactory,” but predict attainment of a DS at a higher PGA than reality, there's risk of underestimating the vulnerability of a bridge. In this regard, conservative predictions were obtained for 60% of the DSs considered.

5. CONCLUSIONS

In this paper, the performance based adaptive methodology referred to as IACSM proposed by Cardone et al. (2011) was used to assess the vulnerability and seismic risk of six real case study structures selected from the WSDOT database. This assessment tool was identified as potentially suitable to conduct large assessment campaigns of existing bridges, something of great need in all active seismic areas.

One objective of this study was thus to gauge whether the IACSM had the characteristics to effectively be used to quickly analyze very large numbers (in the order of thousands) of bridge structures,
providing useful information to guide and prioritize retrofitting intervention efforts to be undertaken. It was found that, constructing the numerical models of the structures is the operation that requires the most effort but that, if the models are kept reasonably simple, the method may still remain suitable for large scale applications. The construction of fragility curves and convolution with the site hazard curve proved to be a straightforward method of quantifying risk, providing a convenient way to rank vulnerability of bridges with respect to one another.

Another important objective of this study was to test the reliability of the predictions obtained using this approach. In particular, the methodology was originally specialized to multi-span bridges, with either simple or continuous deck supported on bearings, and validated considering only a limited number of case study structures. Thus, the predictions provided by IACSM have been compared with the results of NTHA, carried out using a set of twenty real ground motions, compatible with the demand spectrum scaled to the assessed PGA, for all Damage States considered. The comparisons were made in terms of maximum displacements of the deck center of mass, joint displacements and top pier displacements. The results of IACSM and of the NTHA were in good agreement, and overall consistent with the findings of Cardone et al. (2011), Cardone et al. (2011b) and Cardone (2014). Furthermore, the results of this study demonstrated that the methodology can work well for bridge types that differ (sometimes significantly) from those the assessment process was originally developed for. Although the analysis were limited to only six structures, this study provided some additional evidence that the method is promising and should be further explored.

A number of limitations were also identified, that should be the object of future studies. For instance, the definitions of the DSs and PLs do not always appear to be consistent and/or representative of the damage experienced by the structures. While having “independent” definitions contributes to make the process more straightforward, it may be necessary in some occasions to redefine the relationships between DSs and PLs on a case by case basis. Analogously, while ranking bridges as a function of the estimated risk indices is a great starting point, interpreting the results may not be as straightforward as desirable. In fact, it may be difficult to base practical decisions on these rankings as their meaning may be somewhat misinterpreted, or affected by the definitions adopted for DSs and PLs. Post processing of these results would most likely be required and providing some guidance in this sense would be a useful addition to the assessment framework.

Finally, modeling the structures is an operation that may require time, effort and experience. Keeping the models as simple as possible is one of the key aspects of the assessment process. However, the results of the analysis are obviously strongly influenced by the modeling assumptions. Excessive or inaccurate simplifications may result into unreliable vulnerability and risk assessment. This may become an issue, particularly when dealing with bridges characterized by complex configurations such as curved and skewed bridges, or systems in which the soil-structure interaction effects need to be accounted for. Thus, while simplicity of modeling should be advocated, future research should focus on the influence of different modeling assumptions, possibly considering more refined approaches than those suggested in the original version of the assessment method and adopted in this paper.

6. REFERENCES


