2019 SECED Conference
Earthquake risk and engineering towards a resilient world
9-10 September 2019, Greenwich, London

SEISMIC RISK ASSESSMENT FOR STRUCTURAL UPGRADING OF RC BRIDGES IN INDONESIA

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Abstract: *The development of critical infrastructure, such as bridges, in Indonesia has seen a significant increase in the past few decades to meet the growing infrastructure demand of the country. Such a notorious construction pace has not necessarily resulted in bridge structures that meet the standards of the recently updated seismic design provisions. As such, and considering the latest Indonesian Seismic Design code, released in late 2017, the safety verification and eventual retrofitting of existing bridges has become an aspect of particular relevance. The updating of the hazards models for the country has meant that for bridges designed before 2002 the prior seismic load characterization is often significantly different with respect to the provisions contained in the most recent version of the design code for the same geographical location. In light of such considerations, this study addresses the employment of a probabilistic framework to assess the seismic risk of a recently built RC bridge in Indonesia, prior to the release of the most recent seismic design provisions. A specific seismic hazard characterization is carried out for the location of the bridge. Ground motion records compatible with the estimated hazard were selected to carry out Incremental Dynamic Analysis (IDA). Fragility curves for different damage states were developed and mean annual rates of exceedance and unconditional probabilities of the corresponding damage states were calculated. The results of the proposed methodology applied to the case study bridge shows its suitability for seismic risk and safety assessment.*

Introduction

Indonesia is located on the boundaries of three major plates; India-Australia, and Pacific plates, which means that the country is one of the most seismically active in the world. After the Great Sumatera Earthquake with Mw 9.4 in 2004 and other several high magnitude earthquakes, including the recent Sulawesi Earthquake with Mw 7.5 in 2018, the awareness of the public and the government regarding the seismic hazard and their impacts has increased, thus, the necessity for risk assessment and mitigation approaches as the essential keys to counterbalance the impacts of the seismic hazard present in the area has been properly recognized. Recent efforts have been carried out to mitigate these impacts on regular structures and critical facilities, including the updating of the Indonesian Seismic Design Provisions and Hazard Map, revision of building and infrastructure design and construction codes, the development of the National Earthquake Master Plan and the establishment of the National Centre for Earthquake Studies (Hendriyawan *et al*. 2016).

On the other hand, the number of structures constructed in the past, either without seismic provisions or with outdated specifications, which span from the colonial era to a few decades ago is quite large. Their exposure to future seismic events situates them in a condition of high vulnerability, therefore, the need to assess the seismic risk of a considerable portion of the existing stock has become a concerning matter; this task becomes even more challenging considering that the seismic provisions in Indonesia are constantly under revision and development due to advancement of the state-of-the-art in the field of earthquake engineering.

Furthermore, when transportation networks are considered, these features become even more relevant, given that they provide a continuous flow of goods and services and are especially important for the successful development of both emergency and recovery plans after a disaster such as an earthquake takes place. The seismic vulnerability of transportation networks has been acknowledged in the past, and bridges play a critical role since these structures have been recognized as one of the most vulnerable elements within them. In order to understand seismic

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risk in transportation networks in terms of loss of life, economic losses, disruption time, and social consequences, attention nowadays is focused on resiliency to prevent hazard-related damage and minimize disaster impacts (Miller 2005). The aim of this research is to show the applicability of a comprehensive framework, constructed based on the findings and recommendations of previous research, to assess the seismic risk on a real existing bridge structure taken as case study using several decision variables.

Seismic Hazard and its Implications to Design Codes in Indonesia

The first seismic provision of Indonesia was PPTI-UG (Peraturan Perencanaan Tahan Gempa Indonesia untuk Gedung) – 1983 which provided a maximum peak ground acceleration map for six seismic zones. The code provisions were a result from the Beca Carter Hollings and Ferner study in 1978 under the direction of New Zealand Bilateral Aid Program to Indonesia (Ngeljaratan *et al*. 2011). Each seismic zone was characterized with an acceleration spectrum which included soil-site effects reflecting the local conditions for hard soil and soft soil. The provisions were revised in 2002, from which the SNI 03-1726-2002 document was produced. These new provisions were similar to those contained in UBC 1997, the seismic hazard was characterized via a peak ground acceleration (PGA) map corresponding to events with a probability of exceedance of 10% in 50 years. The map was the result of probabilistic seismic hazard analyses (PSHA) performed by four teams conformed by academics, government institutions and practitioners, using the latest seismo-tectonic data including all the recorded earthquakes in historical seismicity for the entire territory of Indonesia available at the time. Three type of soil were introduced which were hard soil, medium soil and soft soil, in accordance with UBC 1997 guidelines. The PGA for the seismic zones 1 to 6 at bedrock level was set respectively as 0.03 g, 0.10 g, 0.15 g, 0.20 g, 0.25 g and 0.30 g. Zone 6 represents the highest level of seismicity whereas Zone 1 is associated with the lowest level of seismicity.

The SNI 03-1726-2002 document was further revised in 2012, as a result, the SNI 03-1726-2012 document was produced. This new revised version contained a set of new Indonesian Seismic Hazard Maps, which had been published in 2010. Three seismic hazard maps, namely, PGA and spectral acceleration (Sa(T)) at short (0.2s) and long period (1.0s) were plotted for bed rock soil type conditions for two different probabilities of exceedance; 10% in 50 years and 7% in 75 years (Bureau of Indonesian Standard 2012).

Seismic Risk Assessment Framework for Bridges

The lack of post-earthquake data related with bridge damage in Indonesia pushes the framework for seismic risk assessment to be developed based on a comprehensive numerical procedure. Such approaches for seismic risk and vulnerability assessment of highway bridges have been proposed in the past. For instance, Cardone *et al*. (2007) proposed a procedure for the assessment of Italian highway networks. This approach is based on adaptive pushover analysis for the characterization of the seismic resistance of the structure.

Accurate estimation of the structural response for different Engineering Design Parameters (EDPs) under seismic loads is the basis for Performance-Based Earthquake Engineering, procedures such Incremental Dynamic Analysis (IDA) offer a practical manner of determining the relationships among seismic demand, structural response and structural capacity by means of intensive nonlinear dynamic analyses (Vamvatsikos and Cornell 2002). IDA involves performing nonlinear dynamic analyses of the structural model under a suite of ground records. A recent study of seismic fragility assessment for concrete gravity dams in Eastern Canada, developed by Segura *et al*. (2019), discussed the safety assessment approach. The proposed method is constructed based on the recommendations of the research mentioned above as shown in Figure 1, and it is implemented on the case study bridge.

Essentially, the procedure relied on the principles of nonlinear dynamic analysis and seismic fragility assessment. The most important features of the procedure are: (i) characterization of the case study structures and the modeling approach to develop model that reflects nonlinear behaviour of the piers which is obtained based on moment-curvature analyses of their critical cross sections and finally derive the lateral force-displacement relationship, (ii) Incremental Dynamic Analysis (IDA), used to compute IDA curves using selected site-specific earthquake ground motions, (iii) damage levels determined via post-processing of dynamic analysis results, (iv) fragility curves developed using IDA curves and damage thresholds computed using a system level approach, (v) structural vulnerability and natural hazard aspects were integrated together to determine performance measures and to perform a safety assessment via comparison with current code recommendations to determine retrofit measure.

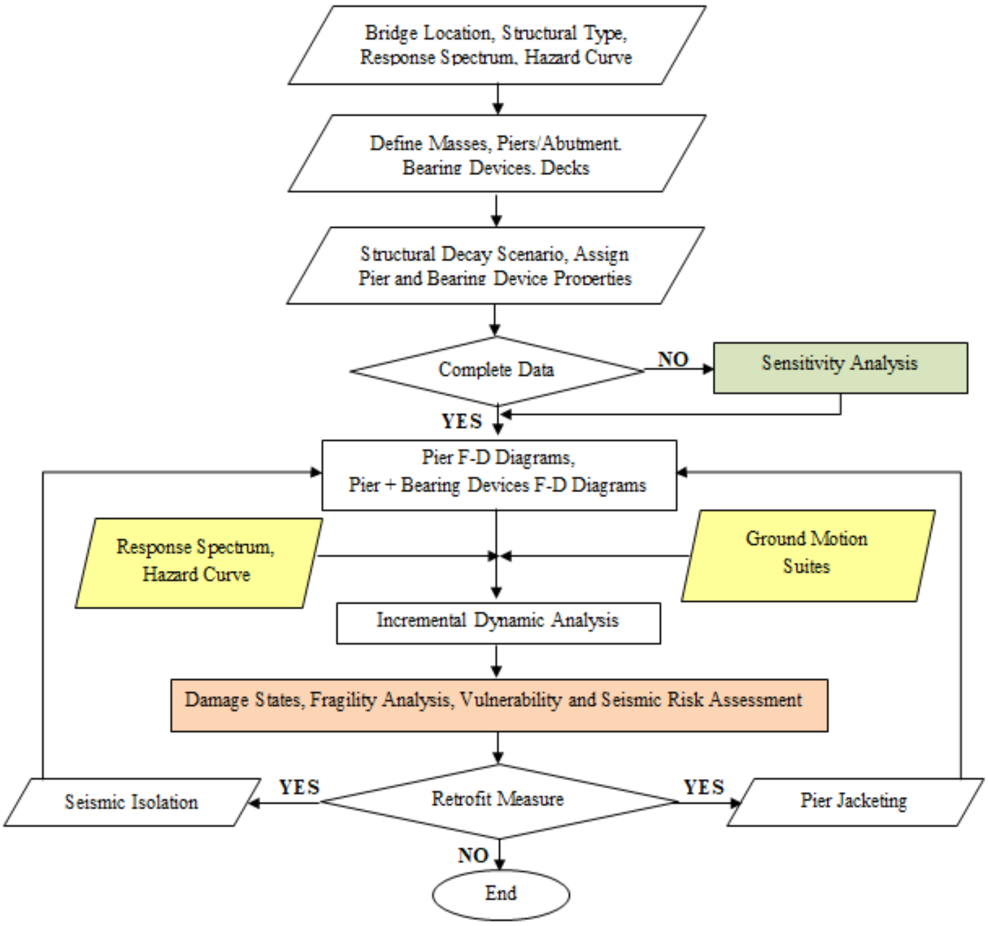


Figure 1. The framework of seismic risk assessment of bridge structure.

Analysis of Existing Bridge: Case Study

General Description and Modeling Approach

The present study is focused on the specific case of the Brantas Bridge in the Surabaya-Mojokerto Toll Road in Kertosono. It was built in 2014 and designed following the 2010 seismic design provisions. Figures 2 and Table 1 present the main characteristics of the case study bridge. In general, the structural system of the bridge is balanced cantilever box girder system with monolithic connection between single column pier to deck and pot bearings with fixed and free articulation at both abutments. The heights of piers are 7.46m and 7.44m for Pier1 and Pier2 respectively. The bridge stands in soft soil type D according to NHERP soil classification system with average Vs30 derived from NSPT correlation from laboratory test as shown in Table 1.

The bridge deck is modeled with elastic beam elements following the center of gravity of the cross section along the length of the bridge with equivalent member properties to represent the overall effective superstructure stiffness. The nonlinear response characteristics of single-column bents is based on moment-curvature analyses at the critical section, taking into account axial load levels as well as transverse reinforcement confinement effects. Columns are modeled using lumped plasticity and distributed plasticity models and their results are compared. P-Delta effects are neglected based on assumption that lateral displacements of bridge piers during a seismic event are typically small compared to the column dimensions. Figure 3(a) presents the nonlinear characteristic of the pier section and Figure 3(b) presents the capacity curve and indicated damage thresholds of the structure. The distributed plasticity model with fixed pier support and spring element at the abutments was selected as the modeling approach for conducting nonlinear dynamic analysis.

Probabilistic Seismic Hazard Analysis and Disaggregation

A classic PSHA approach was used for the characterization of the seismic hazard at the bridge location using the open-source software OpenQuake (GEM, 2015) along with the South East Asia 2007 hazard OpenQuake-compatible model based on original USGS model previously developed by Petersen *et al*. (2007), which was used for developing the 2010 Indonesian Seismic Hazard Map. The PSHA was conducted for the bridge site by employing the local Vs30 based on the soil investigation at the design stage. The hazard curve for spectral acceleration at the fundamental period, T=1.0s, in 50 years of time investigation is shown Figure 4(a) together with its Annual Probability of Exceedance in Figure 4(b).

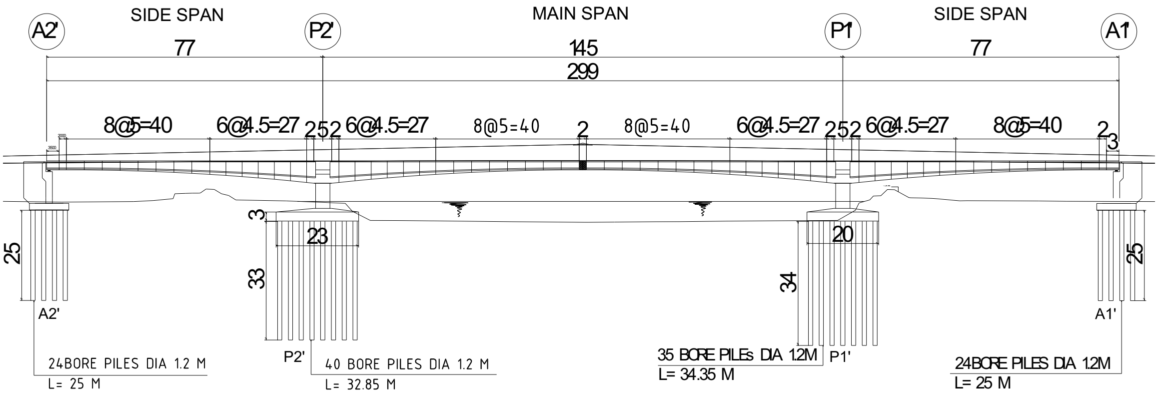


Figure 2. Longitudinal Profile of the bridge structure used in the modelling.

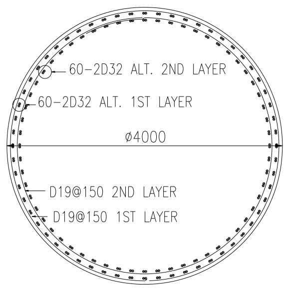
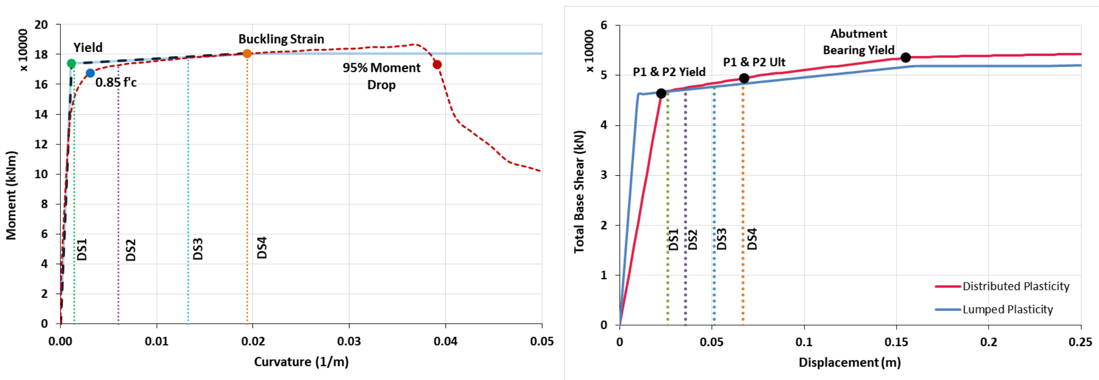
Location	Kertosono, Indonesia		Pier reinforcement
Transportation Network	Surabaya-Mojokerto Toll Road		
Latitude	-7.576339		
Longitude	112.114483		
Soil Test at Location	NSPT average	Vs30 (m/s)	
Abutment 1	22.1	214.1	
Pier 1	18.8	223.8	
Pier 2	29.5	246.7	
Abutment 2	35.4	238.6	

Table 1. Important information of soil condition at bridge location and pier reinforcement detail.



(a) **(b)**

Figure 3. Nonlinear characteristics of the case study bridge (a) Pier Moment-Curvature curve and (b) Capacity Curves for different plasticity model.

The hazard curve was obtained in a discrete manner by calculating the annual rate of exceedance at previously specified intensity measure levels. For risk assessment, a continuous hazard curve needs to be derived. The closed-form function proposed by Bradley *et al*. (2007) is used in this study for fitting a continuous curve to that obtained from the PSHA, it has been shown that it is a good approximation for the range of adopted intensity measure levels investigated in the PSHA, whereas extrapolated values might be inaccurate. The functional form of the fitted hazard curve is given in Equation (1).

$$\hat{\upsilon} = \alpha \exp\left\{\beta\left[\ln\left(\frac{S_s(T_1)}{\gamma}\right)\right]^{-1}\right\} \tag{1}$$

In Equation (1), $\hat{\upsilon}$ is the estimated mean annual frequency of exceedance corresponding to a given intensity measure level (Sa(T1)), α, β and γ are constants characterizing the fitting of the hazard curve data. The fitted curve is also shown in Figure 4(b) with the red dotted line. It is seen that the fitted curve has a close agreement with the result from PSHA, the computed values for the parameters α, β and γ are respectively, 1.1 x 10^4, 9.9 x 10^1 and 1.17 x 10^2. Disaggregation of the seismic hazard is also performed and it is seen that for spectral acceleration at T=1.0s, earthquake events with magnitudes between 6.5 and 7.0, and distances between 55 and 60 km are predominant at the bridge site with an epsilon value equal to 1.42.

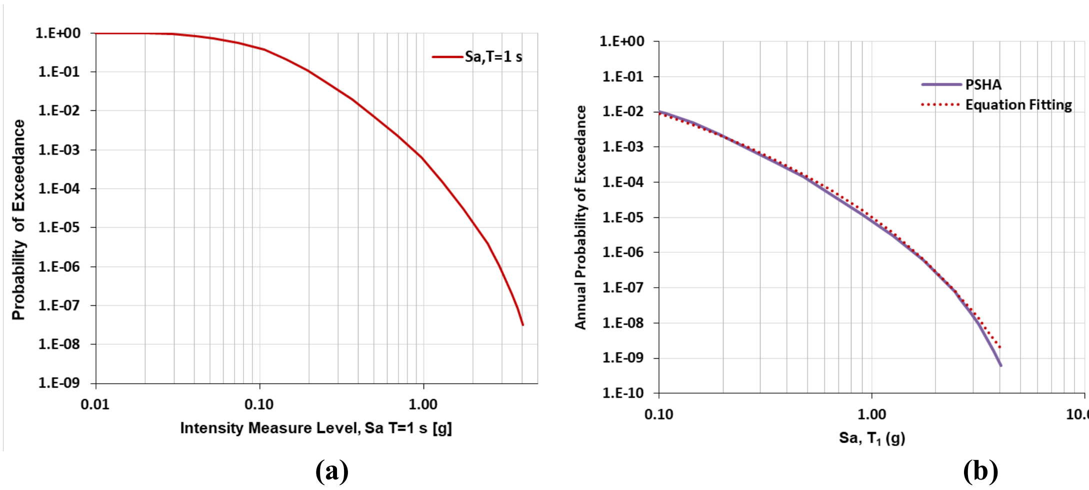


Figure 4. (a) Hazard curve of Sa(T_1=1.0s) in 50 years and (b) Fitted hazard curve of Sa(T_1=1.0s) in 50 years, at the location of the Brantas Bridge.

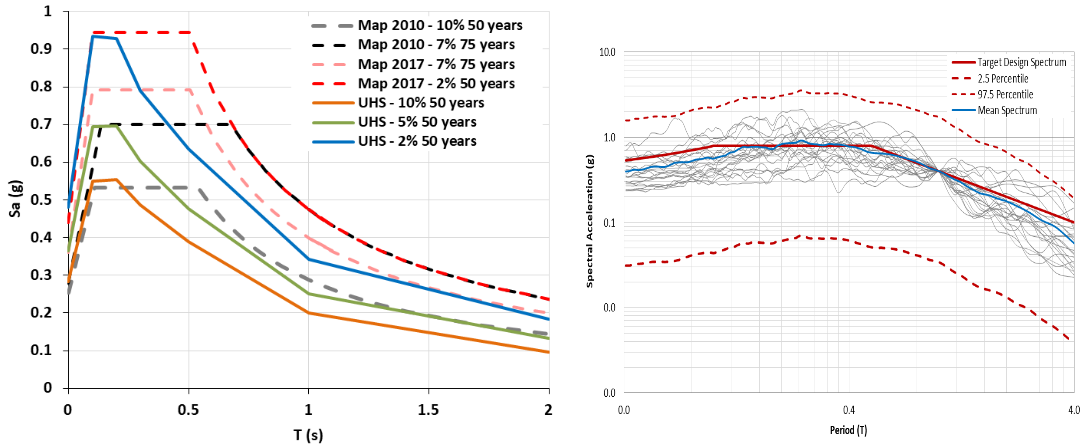
Seismic Demand and Selection of Ground Motions

Uniform hazard spectra (UHS) at the bridge location were also derived using the hazard model mentioned previously for three different probabilities of exceedance, namely, 10% in 50 years, 5% in 50 years and 2% in 50 years. These UHS are then compared with the design spectra constructed from the 2010 and the latest (2017) seismic hazard maps. Figure 5(a) indicates there is a good agreement between the UHS from the hazard model and the design spectra at the relatively short period range. The values of peak ground acceleration and the maximum spectral acceleration are similar for certain levels of exceeding probability; the opposite is observed for the relatively long period range, where the spectral accelerations values are seen to largely differ. For instance, reading from the 2017 design spectrum for a probability of exceedance of 7% in 75 years the Sa(T=1.0s) value corresponds to 0.4g, while from the UHS for probability of exceedance of 5% in 50 years the Sa(T=1.0s) value corresponds to 0.26g, the discrepancy is large since these two spectra roughly correspond to the same return period of 475 years. This discrepancy can be explained by the fact that the 2017 seismic hazard map was developed considering more and likely more recurrent seismic sources in the region of investigation.

In order to consider the most recent hazard characterization and to work with a commonly used level of hazard, the 2017 design spectrum with probability of exceedance 7% in 75 years was used as the benchmark seismic demand criteria for selecting ground motion records. Based on the disaggregation parameters previously discussed, 20 ground motions time histories were selected and scaled using the software REXEL developed by Iervolino *et al.* (2010) to obtain spectrum compatible ground motions within the 2.5% confidence interval as shown in Figure 5(b).

Fragility Analysis

Seismic assessment with fragility analysis provides a framework that ensures that uncertainties in the available information are treated consistently (Segura *et al*. 2017). The structural response from the incremental dynamic analysis, known as IDA curves, are put together with pre-defined thresholds of the EDP that represent specific limit states, these correspond to relevant damage measures and are useful to define the system performance.



(a) **(b)**

Figure 5. (a) Comparison of UHS from hazard model with design spectrum derived from the Indonesian seismic provision and (b) Acceleration spectra of selected records and target spectrum.

Incremental Dynamic Analysis

The 20 selected ground motions were incrementally scaled from a low level where basically elastic response is predicted up to the maximum level of intensity that structure can carry, identified as the point where lateral instability is found in the numerical results. As much as 20 intensity levels were determined for this purpose. Following the general framework presented by Vamvatsikos and Cornell (2002), IDA curves were generated for each one of the selected records. This analysis approach offers a thorough seismic demand and capacity prediction for the structure under investigation. The IDA curves generated from implementing the procedure on the case study bridge are shown in Figure 6(a).

Damage States

When subjected to strong ground motion, the response of bridge structure depends on their seismic energy dissipation mechanism, some of these mechanisms can be described as bridges with monolithic deck column connection, and bridges with bearings connection and non-yielding piers of the wall type (Moschonas, *et al*. 2009).

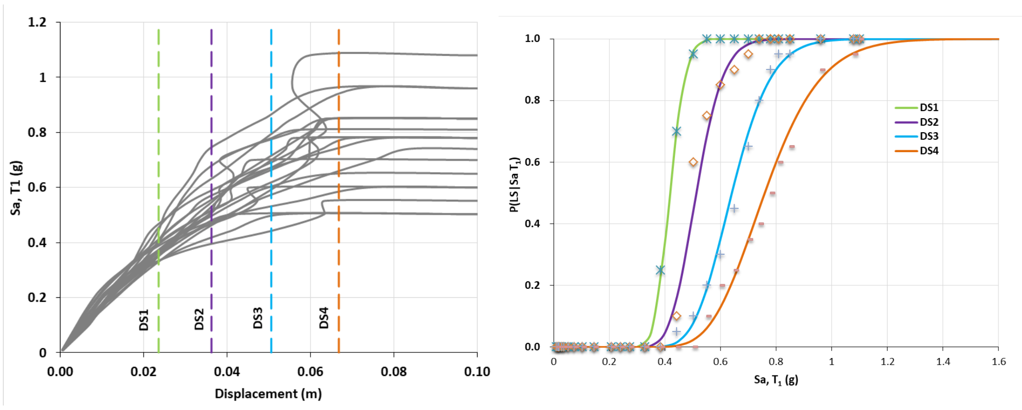
Four damage states were considered in this study: minor/slight (DS1), moderate (DS2), major/extensive (DS3) damage, and failure/collapse (DS4) in line with most of the fragility assessment approaches performed on bridges in previous research, wherein usually analytical fragility curves were calibrated against empirical ones obtained from actual bridge damage data. In the case of bridges with monolithic connection of the column type, which is the case for the case study, the definition of damage states is based on Moschonas *et al.* (2009), in which damage experience from buildings were combined with observations for bridges following Choi *et al.* (2004) and Erduran and Yakut (2004). The damage state is defined directly on the corresponding capacity curve as a function of the yield and the ultimate displacement. The capacity curve of the case study bridge is presented in Figure 3(b) for the transverse direction with the associated damage states as defined in Table 2, notice that the identified thresholds for the damage limit states determined in Figure 3(b) are also shown with the IDA curves as vertical lines in Figure 6(a).

Fragility Curve Fitting

Fragility curves represent the conditional probability that a structure will meet or exceed a specified level of damage for a given ground motion intensity measure (IM) level. In this study, the chosen IM is the spectral acceleration at the first mode period, Sa(T_1=1.0s), and the fragility curves were constructed using Baker (2015) approach for analytical fragility curves derived from IDA results using the method of the moments. The fragility curves for all damage states considered are shown in Figure 6(b).

Reference	Damage State			
	DS1	DS2	DS3	DS4
Moschonas et al (2009)	$0.7\delta_y$	$\mathrm{Min}(1.5\ ;1+1/3)*(\delta_u-\delta_y)$	$\mathrm{Min}(3;1+2/3)*(\delta_u-\delta_y)$	δ_u

Table 2. Damage states for bridges with monolithic deck-column connection in the transverse direction.



(a) **(b)**

Figure 6. (a) IDA curves with damage thresholds, and (b) fragility curves fitting.

Safety Assessment

Given that fragility curves relate the probability associated with the bridge exceeding a certain level of damage given an intensity measure level, it is now possible to perform the safety assessment to evaluate the seismic performance of the case-study bridge. For bridge seismic design practice in Indonesia, depending on the important class of the bridge as prescribed in the guidelines, return period of 500 years and 1000 years are specified. For the purpose of safety assessment, the corresponding intensity measure (IM) level was retrieved from the hazard curve obtained from PSHA for a given probability of exceedance for return period range from 475years until 4975years where the hazard level exceed the Maximum Considered Earthquake (MCE) prescribed in the seismic guideline.

Level	Hazard	Return Period	IM	P.o.E of DS4
Significant	10% in 50 years	475 years	0.18g	<<0.01 %
High	5% in 50 years	975 years	0.26g	<<0.01 %
Very high (MCE)	2% in 50 years	2475 years	0.34g	0.01 %
Extreme	1% in 50 years	4975 years	0.45g	0.613 %

Table 3. Probability of exceedance of the extreme limit state of the case-study bridge.

In ASCE 7-16, the number of person at risk for bridge structure is between 100 and 1000 and it implies that bridge structure is included in Risk Category III. For this category, the guideline specifies a value of conditional probability of exceeding extreme limit state as 5% given MCE in seismic hazard return period. The probabilities of exceedance of extreme limit state (DS4) of the bridge for the specified return periods were then extracted from the fragility curve corresponding to associated failure condition prescribed in the code. Then these probabilities of exceedance were compared with the values proposed by the ASCE 7-16. As shown in Table 3, for failure as extreme damage state (DS4), the probability of exceedance is less than 5% for the MCE and higher level of seismic hazard considered.

For completeness, the annual risk of damage, measured as the annual probability of exceedance of each damage state are calculated by convolving the hazard curve resulting from the PSHA with the fragility curves to determine the unconditional annual probability of damage. The probability density of hazard can be approximated by the Equation (2).

$$P_{f,unc} = \int_{Sa(T1)} P[LS \mid IM, Sa(T_1)] \left| \frac{d\hat{\upsilon}}{dSa(T_1)} \right| dSa(T_1) \quad (2)$$

In Equation (2) $P[LS \mid IM, Sa(T_1)$ is given by the fragility curve and $\frac{d\hat{\upsilon}}{dSa(T_1)}$ is the PDF of the median hazard at bridge site. ASCE 7-16 specify values of conditional probability of damage in a range of 4% to 1% given the annual probability of the design earthquake for Risk Category III. Assuming that the term 'design earthquake' in the guideline is determined by the site-specific design spectrum for bridge in this study in which for design practice is associated to a level of Sa(T=1.0s) as 0.4g, the corresponding annual probability of this intensity level equals to 2.26 x 10^{-3}. Threshold values of unconditional annual probability of damage for this assessment approach are taken by convolution of conditional probability of damage with annual probability of design earthquake. Table 4 presents the mean annual probability of exceeding each damage limit state, and the allowable values defined by following the guidelines and risk category proposed in ASCE 7-16.

Limit	Annual Probability of Damage	Threshold	Remarks
D>DS1	2.70 x10^{-4}	[4% | Annual Prob DE] = 9.07 x10^{-5}	Exceeded
D>DS2	1.48 x10^{-4}		Exceeded
D>DS3	7.24 x10^{-5}		Not exceeded
D>DS4	4.26 x10^{-5}	[1% | Annual Prob DE] = 2.27 x10^{-5}	Exceeded

Table 4. The annual probability of damage of the bridge in study.

Conclusions

The implication of newly published Indonesian seismic hazard map to the design codes in practice and existing structure is being one of the complex discussions in infrastructure construction point of view. The new provision relatively adds higher seismic demand to structure design compare to the old one and its difference is very significant. Through any quantitative parameter that reflects the existing performance capacity provided by the old structures, the implication may be communicated to the owner and regulatory institution and it is also beneficial as a baseline frame for the development of mitigating action of earthquake risk. This study extended a general methodology proposed in previous research for assessing seismic risk of bridges. A numerical procedure was presented and subsequently applied to an existing bridge, taken as case study. The procedure relied on the principles of nonlinear dynamic analysis and seismic fragility assessment.

In this study one model assumption was chosen, distributed plasticity model with fixed pier support and spring element at the abutments, for conducting the main procedure of seismic fragility analysis which in this methodology use a nonlinear Incremental Dynamic Analysis (IDA). Four damage limit states were determined to be taken from the previous study by Moschonas *et al*. (2009) by firstly investigating its relevance with the structural system in study, the behaviour of structural response and collapse hierarchy that was intended in design approach. This is done by firstly running some nonlinear static analyses, before going through the extensive work in IDA. A set of 20 different records of earthquake ground motion used in IDA with 20 intensity levels for each record. Finally, based on the IDA-generated data set of bridge response under seismic excitation, the analytically derived fragility curves were developed. Later, structural vulnerability and natural hazard aspects were integrated together to determine performance measures and to perform a safety assessment via comparison with current code recommendations. This procedure was used to estimate key parameters, i.e., probabilities of exceedance of extreme limit state and annual probability of damage, that serve to assess the functionality of bridge structures under frequent or rare earthquake events, these parameters should help researchers, owners and regulatory organizations to do prioritization of the bridge stock to plan future retrofitting works. Both approaches give us two different remarks in term of the safety performance of bridge in case study compare to a regulatory seismic design provision that is adopted in Indonesia. Being in Risk Category III according to ASCE 7-16 documentary, bridge needs to meet a criterion of having maximum 5% of probability of exceeding extreme limit state. From the four different hazard levels, the bridge meets the requirement of probability of exceeding extreme limit state below 5% for MCE. The other approach assesses the safety performance by unconditional annual probability of damage or failure as parameter. ASCE 7-16 documentary specifies that structure in Risk Category III needs to meet requirement of maximum 4% probability of damage and 1% probability of failure conditional to the annual probability of design earthquake. According to this specification, the bridge in this study fails to meet the criteria of annual probability of being in damage for limit states slight and moderate while still safe for limit state extensive. For the criteria of annual unconditional probability of failure, the threshold is exceeded by the bridge. From this point of view, it means that according to regulation, any improvement action to increase the safety performance of the bridge should be done by the owner.

The output of this procedure could also be used to estimate direct economic losses or other decision variables. Further investigation on the seismic risk of the transportation system of Indonesia can be developed in the future to integrate the effect of spatial distribution and local site conditions, the results presented in this study should be useful for characterization of transportation networks in which bridges as key components are particularly vulnerable.

Acknowledgements

This work has been supported by data provided from PT Wiratman and extensive research of the development of Indonesian Seismic Hazard Map by the Indonesian Centre of Earthquake Studies.

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