sciendo

Civil and Environmental Engineering
Vol. 17, Issue 1, 209-218, DOI: 10.2478/cee-2021-0022

FRAGILITY CURVE ANALYSIS OF MEDIUM CABLE STAYED BRIDGE

Masrilayanti MASRILAYANTI[1,*], Ade Prayoga NASUTION[2], Ruddy KURNIAWAN[1], Jafril TANJUNG[2], Sarmayenti SARMAYENTI[3]

[1] Department of Civil Engineering, Andalas University, 25163, Padang, Indonesia.
[2] Department of Civil Engineering, Institute of Technology of Sumatera, 35365, Lampung, Indonesia.
[3] Virama Karya Engineering Consultant, 12120, Jakarta, Indonesia.
[*] corresponding author: masrilayanti@eng.unand.ac.id

Abstract

The seismic performance of a bridge can be shown by analyzing the vulnerability of the structure in resisting an earthquake motion and then developing into fragility curves. This study presents a convenient method to establish the fragility curve for the cable-stayed bridge. For this purpose, three spans cable-stayed bridge is assessed using a series of seismic loads in different intensities to ensure that the structure was experiencing damage in several conditions. The fragility curve was obtained by analyzing the structure using Nonlinear Time History (NTHA) and Pushover Analysis. The ground motions of the earthquake were subjected to the bridge in different intensities, which were scaled from the initial ground motion. Hereafter, the structure's ductilities were developed into the fragility curves as the responses of the bridge. HAZUS standard is used for classifying the damages of the bridge, which are grouped into; slight, moderate, extensive, and complete due to the seismic load. The values of the damage states were generated to the fragility curves using the probabilistic values of the damage states. To ensure the validity of the data statistically, Kolmogorov-Smirnov test was conducted to the fragility function. The result revealed that the fragility curve was qualified as the lognormal distribution.

Keywords:

Fragility curves;
Cable stayed bridge;
Nonlinear time history;
HAZUS.

1 Introduction

A bridge is an essential tool in connecting the social and economic needs of society. It has so many functions to ensure the comfort and safety of the community. The cable-stayed bridge is known as one of the most beautiful bridges in the world. The bridge elegance and structural efficiency system are available to extend the spans which are ineligible in the past with the excellent efficiency, and the typology of this bridge can be used in the large span, start from 200 m to 1000 m, and seems like develop in a short time [1]. The bridge has a vital role as the connectivity for economic and social activity; therefore, it should be appropriately designed, especially in earthquake-prone areas [2]. Significantly, the span length of the bridge will cause the complicated level of the design. The cable-stayed bridges have a good principle in resisting the seismic motion. First, the bridge is flexible and induces a long period of motion to reduce the spectral acceleration level. Second, it can decrease the reactions of the supports and the vulnerability of the structure towards the seismic load and the significant displacement in the deck [3]. In Indonesia, the bridge's earthquake design code, SNI 2833: 2016, is used as a reference design in the earthquake zones [4]. However, many bridges have not implemented the regulations in their system; therefore, many bridges are experiencing damage in the post-earthquake and caused the loss of life. Thus, it is crucial to determine the behavior of the structures due to the seismic motion [5].

To define the bridge relating to its vulnerability due to earthquake motion, using the fragility curves is reliable and applicable to manage the hazard [6]. The fragility curve is a value of exceedance probability and earthquake intensity of the structures. Fragility curves can be developed using

empirical or analytical analysis. For developing using the analytical method, data of the structures' responses are needed. Fragility curves using the analytical method are widely used because of general and convenient use on the bridge and geographic area while the lack of bridge damage data. In predicting the extent of probable damages of bridge structures, fragility curves are recommended to be a useful tool. The vulnerability assessment of bridges helps seismic retrofitting decisions, disaster response planning, estimation of direct monetary loss, and evaluation of loss of functionality of highway transportation systems. To guarantee the safety and serviceability of present bridges maintenances are essential all around the Globe and most governments' budget effect by overhaul such infrastructures yearly without any rate of interest in these maintenances. A large proportion of costs are allocated for maintenances costs due to the increasing number of present bridges. Maintenances also have a substantial effect on economic and sustainable development [7]. Based on this background, the analytical study is conducted to develop the fragility curves of the cable-stayed bridge using the seismic response of the bridge in structural analysis software. The existing records of the earthquake were subjected to the structure. The damage states are classified based on the HAZUS standard using the dynamic analysis Nonlinear Time History (NTHA) and then derived from acquiring the cable-stayed bridge's fragility curves [8].

2 Fragility curves methodology

Fragility curves are very useful tools to determine the damage risk of bridge structures due to seismic load [9, 10, 11].These curves present the probability of exceedance in several damage states of ground motion. There are many types of distribution functions in mathematics; however, only three are widely used in fragility: normal distribution, lognormal distribution, and uniform distribution. The engineers commonly use normal distribution function and cumulative distribution function. The functions are then developed to be a lognormal distribution because reliable to analyze the exceedance of seismic hazard and risk hazard. In other words, the ordinary equation to develop the fragility curves or conditional exceedance is in equation (1):

$$Fragility = P[\, LS|IM \, = \, y] \,, \tag{1}$$

where *LS* is a limit states or damage states *DS*, *IM* is the intensity of ground motion, and *y* is a reached state due to ground motion intensity *IM*.

The fragility curves were developed to represent a prediction of a hazard risk during and after seismic motion. The curves are commonly used to reduce the damage and energy loss during earthquake motion so that they can help to provide the provision code in certain areas. The process of developing fragility curves can be started by conducting nonlinear static analysis or pushover analysis. Since Kobe and Northridge earthquakes, nonlinear static analysis became an accepted method for the seismic evaluation of structures [10].

The fragility curves can be developed in several methods. The first empirical approach depends on the available data post-earthquake and surveys result. However, the lack of data has become a serious problem to develop this method. The second analytical method can be generated using an adequate damage state such as elastic spectral analysis, probabilistic seismic demand model, static analysis, linear or non-linear time history analysis [9, 11, 12]. In this study, the non-linear time history analysis was conducted to develop the cable-stayed bridge's probabilistic seismic demand model.

There are two approaches to develop the PSDM; the scaling approach and the cloud approach [89]. In this research, the cloud approach was utilized to evaluate the fragility function of the cable-stayed bridge. Regression analysis is carried out to calculate the mean and standard deviation by assuming the power-law function [9]. The logarithmic relation between median Engineering demand parameters *EDP* and selected motion intensity *IM* as shown in equation (2):

$$\ln(EDP) = \ln(a) + b\ln(IM), \tag{2}$$

where *a* and *b* are unknown coefficients obtained from the regression analysis of the response of the non-linear time history analysis. Then, the effectiveness demand model is determined using the dispersion of demand in equation (3).

$$\beta_{EDP}|IM = \sqrt{\frac{\sqrt{\sum_{i=1}^{N}((\ln(EDP)-\ln(a(IM)^b)^2}}{N-2}}, \quad (3)$$

where *N* is the number of total simulation of the earthquake. ln(*EDP*) can be obtained by using equation (2). Furthermore, the fragility can be generated using the probabilistic seismic demand and limit states corresponding to various damage states using equation (4).

$$P[\mathrm{LS}|\mathrm{IM}] = \Phi(\frac{Ln\,(IM)-\lambda_c}{\xi_c}), \quad (4)$$

where Φ [.] is the distribution cumulative function.

$$\lambda_c = \frac{\ln(\mu_c)-\ln(a)}{b}, \quad (5)$$

where λc is defined as the median values of the fragility function, noted as equation 4, μ_c is the median value for the damage states, then *a* and *b* are unknown coefficients obtained by regression analysis. And the dispersion component is presented in equation (6).

$$\xi_c = \frac{\sqrt{\beta_{EDP}{}^2+\beta_c{}^2}}{b}, \quad (6)$$

where β_c is the dispersion value for the damage states. Using this equation, the fragility function can be generated for various damage states.

3 Cable-stayed bridge description and modeling

A model of cable-stayed bridge used in this study is a medium-span bridge, located in Batam island, Indonesia, consist of 3 spans (2 pylons) with a total length of 642 m (146 m + 350 m + 146 m). The height of the towers and piers are 84 m and 39 m, respectively. The pylons are rectangular sections with a dimension of 6.0 x 3.0 m. This bridge has a composite deck with longitudinal and transverse girders are connected. The slab is modeled as the gravity load on the beam, with the value of 10 kN/m. The composite deck in each bridge is supported by 14 cables per half span (2).

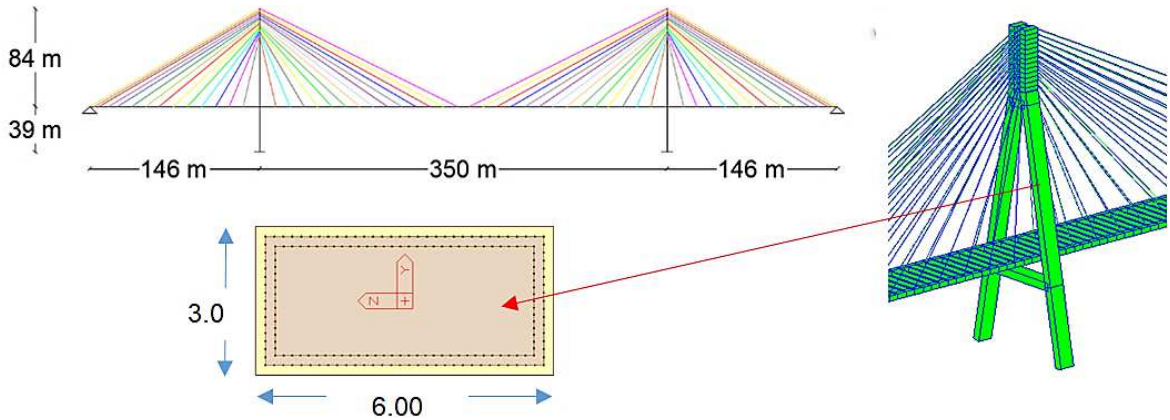


Fig. 1: The modeling of cable-stayed bridge.

The arrangement of cables is a fan type in longitudinal and two-plane systems in the transversal direction, as can be seen from Fig. 1. These cables have various areas from 383.1 mm^2 to 1149.3 mm^2. Also, there are 3 various spaces of the cables arrangement; 4 meter for 2 back stay cables, 8 meter for the next two cables, and then 12 meters for the rest. Meanwhile, the space of cables on the pylon is 2 m. The connection between the pylon and composite deck is suspended on it. The end supports of this bridge are considered as hinge, and because soil conditions are not considered to this study, so that the supports are assumed to be rigid to the ground. The 3D modeling of the cable-stayed bridge was performed using Midas Civil structural analysis program. Girder and pylon were modeled as frame so that they resist all internal forces due to the loading, whereas the cables are

modeled to resist the axial force only and consider the pretension load of each cable. The beam is modeled using the elastic element, and the pylon was modeled using an inelastic element with mass lumped along the centerline. The beam-column joint is highly rigid to connect the elements in the model. The bilinear model of steel was used in the reinforcing material.

The unconfined and confined concrete behavior was modeled using the non-linear confinement model as the confinement effect of the concrete section was considered on reinforcing detailing. The kinematic hardening was defined using the yield forces, initial stiffness of the member, and post-yield hardening ratio, which was calculated following the AASHTO guidelines. The supports of the bridge are nonlinearly modeled through the general link of force type, and the rest of the structure is modeled linear elastically. The boundary is analyzed by converting the member force of the non-linear system into load acting in the linear system because a linear system is analyzed through modal superposition. The support of the pylon is modeled as fixed by considering three rotational and three translational deformation.

4 Ground motion

In this study, several representation ground motions were used to generate the fragility curves for a cable-stayed bridge's critical response. A total of 32 simulated ground motions have been selected to represent a broad range of possible earthquake magnitude and distance. The ground motions chosen have various time history types applied to the structure; El-Centro, Northridge, Tohoku, and Padang, as shown in Table 1. These ground motions are then scaled into 0.25 to 2.00 for each type. This method is presented to obtain the maximum response of the structure during and post-earthquake when the ground motions have great energy released. The suite of ground motions is intended to represent the epistemic and aleatoric variability characterized in the soil attenuation relationships developed for the region.

Table 1: Selected ground motion as excitation of the bridge.

No.	Earthquake	Year	PGA [g]	Time [s]
1	El-Centro	1940	0.36	53.72
2	Northridge	1994	0.88	59.98
3	Tohoku	2011	0.61	240.00
4	Padang	2009	0.71	60.00

5 Pushover analysis

Pushover analysis procedures were initially developed for the seismic performance assessment of structures and later on usd for fragility analysis [10, 11, 13]. This is a static non-linear analysis where the structure receives gravity loads and lateral force, which is assumed to simplify earthquake loads. It comes from the need for a simple method to estimate the non-linear behavior resulting from seismic loads. For this analysis, pushover is conducted till the structures are assumed to collapse. In this research, the lateral forces are given into the top of the pylon that is interpreted as the acceleration with the target displacement 1.00 meter in the longitudinal direction till the structure collapse. Moment curvature is obtained by conducting the section analysis following the AASHTO [11]. It shows the characteristics of the section or the structure element's capacity because this moment-curvature is assigned to each element of the pylon. Fig. 2 explains that the material is experiencing yielding at the moment value as 417996.503 kNm. After conducting the pushover analysis, the capacity curves of the structure can be obtained both in longitudinal and transversal, which is showing the maximum ductility of the structure. The response of the structure used in this study is ductility. This ductility can show the limit states of the bridge to withstand the seismic intensity. Pushover analysis is conducted in the longitudinal and transversal direction by pushing the top of the pylon that is assumed as the lump mass system of the bridge. The pushover curves can be seen in Fig. 2.

Curvature [1/m]	Moment [kNm]	Curvature [1/m]	Moment [kNm]
0	0	1.765517	428300.6
0.006897	12814.85	1.993103	428733.8
0.027586	51246.68	2.234483	428132.2
0.062069	115183.1	2.489655	425382.5
0.110345	185293.7	2.758621	422452.2
0.172414	214576.1	3.041379	412387.2
0.248276	245304.3	3.337931	385784.7
0.337931	278282.3	3.648276	375011.3
0.441379	314655	3.972414	362117.4
0.689655	378011.5	4.310345	347258.1
0.834483	394856.3	4.662069	306814.2
0.993103	405419.8	5.027586	244100.1
1.165517	414636.1	5.406897	196464.1
1.351724	420477.1	5.8	152970.8
1.551724	425233.2	6.206897	119037.6

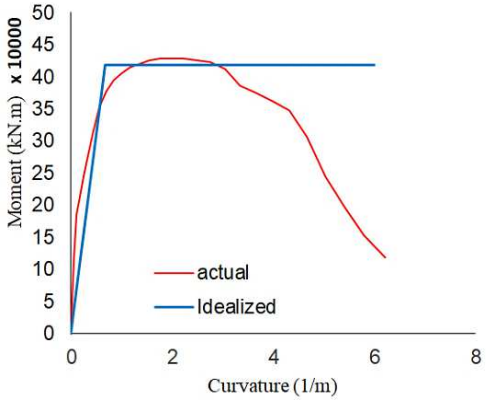
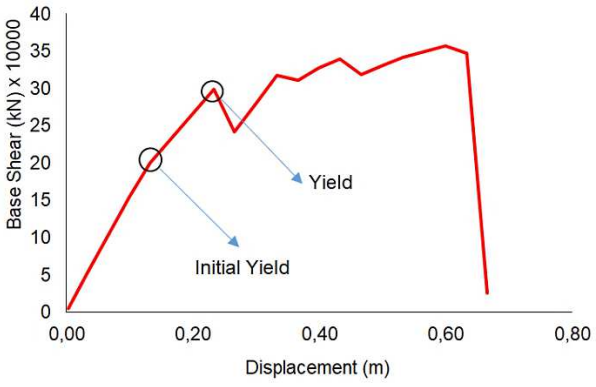


Fig. 2: Pushover curve.

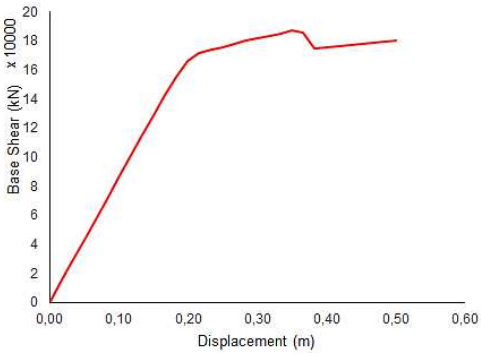
In the longitudinal direction, is noted that the yield displacement δ_y is 0.24 meter with the total base shear of 298.842 kN, and the ultimate displacement δ_u is 0.60 meter so that the ductility μ of the structure can be obtained by δ_y/δ_u; thus the ductility is 2.50.

Step number	Displacement [m]	Base shear [kN]
0	0.00	5160.290
1	0.03	51602.90
2	0.07	103237.00
3	0.10	154903.00
4	0.13	200421.00
5	0.17	233230.00
6	0.20	266035.00
7	0.23	298842.00
8	0.27	241304.00
9	0.30	279336.00
10	0.33	317556.00
11	0.37	311097.00
12	0.40	327372.00
13	0.43	340046.00
14	0.47	318305.00
15	0.50	330439.00
16	0.53	341466.00
17	0.57	349853.00
18	0.60	356778.00
19	0.63	346821.00
20	0.67	25369.100



a) Longitudinal direction

Step num	Displ [m]	Base shear [kN]	Step num	Displ [m]	Base shear [kN]
0	0.00	1430.01	16	0.27	178098
1	0.02	14300.10	17	0.28	180272
2	0.03	28600.10	18	0.30	181900
3	0.05	42900.10	19	0.32	183586
4	0.07	57199.90	20	0.33	185273
5	0.08	71499.30	21	0.35	186959
6	0.10	85798.50	22	0.37	185720
7	0.12	100097.0	23	0.38	174467
8	0.13	114395.0	24	0.40	175310
9	0.15	128693.0	25	0.42	176154
10	0.17	142679.0	26	0.43	176997
11	0.18	155279.0	27	0.45	177841
12	0.20	166271.0	28	0.47	178684
13	0.22	171454.0	29	0.48	179528
14	0.23	173751.0	30	0.50	180372
15	0.25	175924.0			



b) Transversel direction

Fig. 3: Pushover curves.

Furthermore, in the transverse direction, the yield displacement δ_y is 0.20 meter with the total base shear of 166.271 kN, and the ultimate displacement δ_u is 0.37 meter; thus the ductility μ of the structure can be obtained by δ_y/δ_u; therefore the ductility is 1.85. It means that the longitudinal ductility of the bridge is larger than the transverse ductility, as shown in Fig. 3.

6 Probabilistic seismic demand model

The probabilistic demand model (PSDM) is used to establish the relationship between component demand and ground motion intensity *IM*. The probabilistic demand model demonstrates the correlation between the engineering demand parameter *EDP* and motion intensity *IM*. In this study, as the motion intensity *IM*, the peak ground acceleration PGA is used to develop the fragility assessment. PGA is an ideal choice to assess the severity of the ground motion of an earthquake because of its sufficiency, practicality, efficiency, and hazard computability [8]. Then, *PGA* is used as the *IM* to develop the PSDM and fragility curves. After performing the non-linear time history analysis, the peak component demand of cable-stayed bridge was recorded. The PSDM for a cable-stayed bridge is developed by plotting the peak component responses against the *IM* for each ground motion. Finally, a regression analysis is carried out to estimate *a, b,* and $\beta_{EDP}|IM$ using equation (2) and equation (3) [12].

Fig. 4 shows the PSDM for cable-stayed bridge component considered in this study. The coefficient correlation R^2 shows the relationship between ductility μ and peak ground acceleration *PGA* in the linear line. The *y* of function *f*(*x*) is ln *EDP*. This coefficient correlation describes the influence of one component over the other and reflects the dynamics of the whole bridge system.

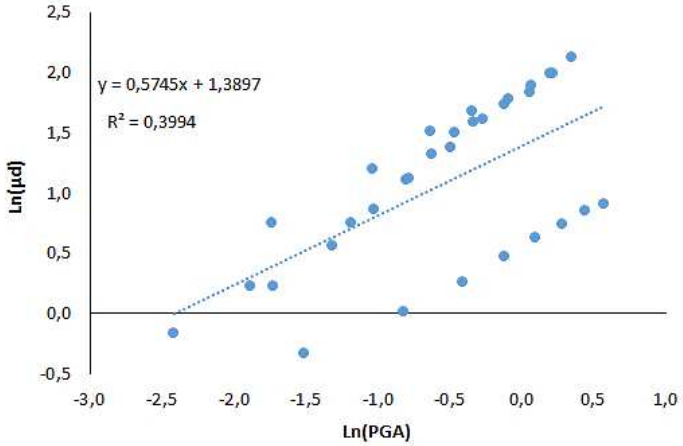


Fig. 4: Regression analysis of cable-stayed bridge.

7 Component damage states

As defined by HAZUS, four damage states are commonly adopted in the vulnerability assessment of engineering structures; slight, moderate, extensive, complete. Table 2 presents the limit state capacities for the different components in terms of the median S_c and lognormal standard deviation β_c required to describe the capacity model. The limit states of the bridge components were assumed to follow a lognormal distribution, which was derived using a perspective approach [12]. There is also uncertainty associated with each median S_c, which must be defined. This uncertainty is given in the form of a lognormal standard deviation or dispersion β_c. The lognormal standard deviation values or dispersion β_c have been obtained following the procedure described in [9, 12]. In this study, the damage state level is obtained by non-linear time history analysis. The ductility of the bridge is obtained by the various PGA artificial. This value is used to develop the fragility curves. Using the relationship between ductility and the PGA determines the exceedance of probability in each model state. The bridge is experiencing ductile, starting from 0.9 to 8.4, which means that the various PGA that excited to the bridge and the pylon deflect and acquire the damage state level.

Table 2: Bridge performance/design parameters based on NCHRP 440 (2013) [7].

Performance level	Damage level	Description	Steel strain	Concrete strain	% Drift	Displacement ductility S_c
II	Slight	Operational	0.005	0.0032	1	1
III	Moderate	Life safety	0.019	0.01	3	2
IV	Extensive	Near collapse	0.048	0.027	5	6
V	Complete	Collapse	0.063	0.036	8.7	8

8 Fragility assessment

Based on the non-linear time history analysis conducted, it can be seen that the values of PGA can cause the bridge to have failures since the increase of PGA yields the structure to have more internal forces and deformation. The time history used is the existing records of the Indonesia earthquake and other countries, as shown in Table 1. All time history records in Table 1 were then scalled to become total of 32 data of time histories. This data is then developed to interpret the conditions correctly, which offers a correlation between motion intensity and peak ground acceleration. Compared to other types of structures, it is admitted that cable-stayed bridges that have more than three spans are usually more complicated. For anaysing the data, according to [14], the lognormal distribution is a proper function in multiple variables. One of the uses of this function is the fatigue mechanism in the structure that receives varying stresses. Thus the calculation of PGA can be obtained by lognormal distribution in parameters of function; mean μ, standard deviation σ, mean of the lognormal distribution λ, standard deviation of lognormal distribution ξ. In this study, the result of non-linear time history analysis obtains the parameters as follows.

Table 3: Median and dispersion value of fragility curves.

Median and dispersion value of fragility curves								
IM	***LS***	μ_2	***a***	***b***	β_d	β_C	λ_c	ς_c
	DS 1	1.00	4.01	0.57	0.54	0.25	-2.42	1.03
PGA	DS 2	1.20	4.01	0.57	0.54	0.25	-2.10	1.03
	DS 3	1.76	4.01	0.57	0.54	0.47	-1.43	1.24
	DS 4	4.76	4.01	0.57	0.54	0.47	0.30	1.24

From the non-linear time history analysis, the maximum displacement of the pylon was obtained. The displacement is then analyzed to get the ductility of the bridge in each response.

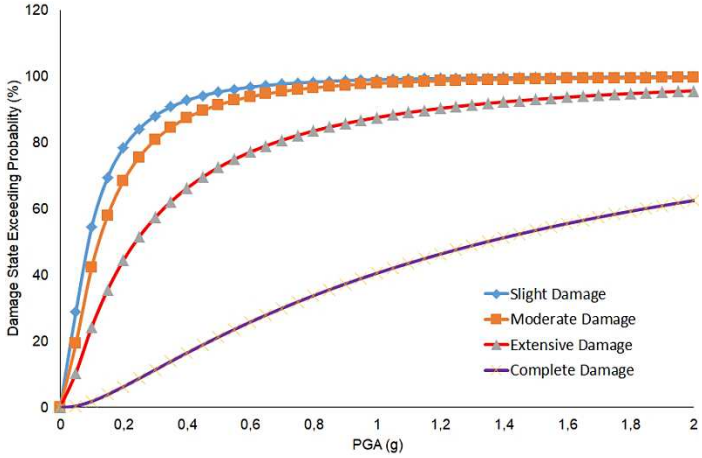


Fig. 5: Fragility curves of the cable-stayed bridge.

In this study, the pylon's maximum displacement due to the seismic load design is used to develop the fragility curves. From Padang earthquake, the maximum displacement is 0.958 meters with the motion intensity 0.61g, and the ductility of the bridge is 4.0, thus based on the design/performance parameters of the bridge [9], the performance of this bridge can be defined as complete/collapse damage. The maximum ductility of the bridge is obtained from the longitudinal excitation of the bridge; it means that the bridge is experiencing the dominant earthquake in a longitudinal direction. Furthermore, from Fig. 5, the probability of exceedance of the designated limit state at a particular motion intensity *IM* can be obtained. For a PGA of 0.61g, the possibility of exceedance for slight, moderate, extensive, and complete are about 8.37 %, 77.2 %, 93.9 %, and 96.8 %, respectively. And the inference of the fragility curve obtained can be explained as, if an earthquake of PGA 0.61g occurs 96.8 %, the ductility of the bridge will exceed the limit 1.00, and if an earthquake of PGA 0.61g occurs 93. 9 % of the ductility of the bridge will exceed the limit of 1.20. If an earthquake of PGA 0.61g occurs 77.2 %, the ductility of the bridge will exceed the limit 1.46, and if an earthquake of PGA 0.61g occurs 25.8 %, the ductility of the bridge will exceed the limit 4.76.

9 Fragility evaluation

Following the previous study of fragility curves where this function is distributed lognormally, PGA parameters are then calculated to obtain the validity of lognormal distribution. The evaluation of this fragility curve was conducted by using the Kolmogorov-Smirnov test to ensure the goodness of fit data statically [15, 16]. There are two alternatives to develop the fragility curves; normal distribution and lognormal distribution. The Kolmogorov-Smirnov test is appreciable good; it qualifies this requirement as follows:

$$D_n \leq D_n^a \ , \tag{7}$$

and,

$$D_n = \max |F_n(x_i) - S_n(x_i)| \ , \tag{8}$$

where D_n is the maximum error values, $F_n(x)$ is the lognormal distribution function, $S_n(x)$ is the staircase function.

The total PGA of 32 data, and the significant level α of 5 %, thus the values of D_n^a is 0.235. The value obtained from interpolating the table test Kolmogorov-Smirnov can be seen in Table 4. This calculation is a test of the hypothesis in the probabilistic model.

Table 4: Kolmogorov-Smirnov test [16].

n / α	0.2	0.1	0.05	0.01
5	0.45	0.51	0.56	0.67
10	0.32	0.37	0.41	0.49
15	0.27	0.3	0.34	0.4
20	0.23	0.26	0.29	0.36
25	0.21	0.24	0.27	0.32
30	0.19	0.22	0.24	0.29
35	0.18	0.2	0.23	0.27
40	0.17	0.19	0.21	0.25
45	0.16	0.18	0.2	0.24
50	0.15	0.17	0.19	0.23
n > 50	$\frac{1.07}{\sqrt{n}}$	$\frac{1.22}{\sqrt{n}}$	$\frac{1.36}{\sqrt{n}}$	$\frac{1.63}{\sqrt{n}}$

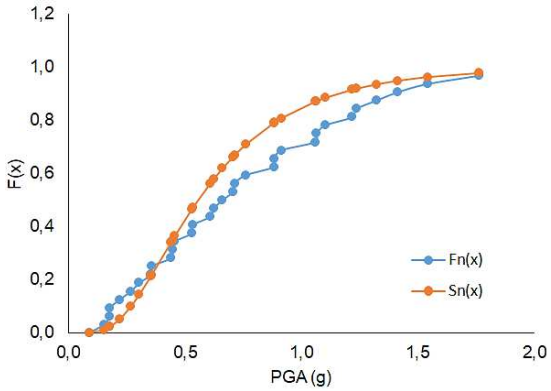


Fig. 6: Kolmogorov-Smirnov test function.

Furthermore, the error value of the fragility curves is evaluated by conducting and plotting to the function, as shown in Fig. 6; the orange line is showing the lognormal distribution, and the blue line showing the staircase function. From Fig. 6, the function's difference is observed, and both of the functions are cumulative function. To obtain the error of the cumulative distribution, the D_n is calculated for every step in the function. In the range of PGA 1g, the difference between the function $F_n(x)$ and $S_n(x)$ is showing the maximum error $D_{n,\max}$. From the calculation, $D_{n,\max}$ obtained is 0.166, which has a significant error for the lognormal distribution function. However, the Kolmogorov-Smirnov test has limitation states for different significant levels. And with data of PGA are 32 data and considerable level and the significant level α 5 % obtained the K-S value is 0.235. Thus, the fragility curves qualify equation (7), and the data showed that the bridge's fragility curves are lognormally distributed. Therefore this fragility curve can be used for seismic evaluation of the bridge.

10 Conclusion

In this study, the cable-stayed bridge with a total length of 642 meters was excited under different load cases, including the pretension load, dead load, superimposed dead load, and earthquake load. For predicting damages of the bridge structures, it is comfortable to use fragility curves. The vulnerability assessment of bridges is useful for seismic retrofitting decisions, disaster response planning, estimation of direct monetary loss, and the evaluation of loss of highway transportation systems functionality. The fragility curves were developed by conducting the non-linear time history analysis. To obtain the yield displacement of the structure, it should perform the pushover

analysis. Thirty-two acceleration time histories are excited to the system in a longitudinal and transversal direction to obtain the response of the structure.

The following conclusions can be derived from the study results.

1) The pushover analysis shows that the yield displacement of the pylon is 0.24 meters in the longitudinal direction, and the yield displacement of the pylon is 0.20 meters in the transversal direction.

2) In Padang's target ground motion, the ductility of the structure is 4.0, with a peak ground acceleration of 0.61g. And the bridge performance is classified as "collapse."

3) In Padang's target ground motion, the bridge has the probability of exceedance for slight, moderate, extensive, and complete are 96.91 %, 94.09 %, 84.60 %, and 26.20 %, respectively.

4) In evaluating the goodness of fit data of fragility curves using the Kolmogorov-Smirnov test, the maximum error obtained is 0.166. The satisfaction level is 95 %, which means that the fragility curves can evaluate the seismic hazard of this cable-stayed bridge.

Acknowledgments

Authors would like to thank to Engineering Faculty of Andalas University, Indonesia, for supporting this publication by Contract No.028/UN.16.09.D/PL/2020.

References

[1] MASRILAYANTI, M. – ARYANTI, R - KURNIAWAN, R. – SIREGAR, Z.: Behaviour of Cable-Stayed Bridge's Girder to Multi-Support Excitation. MATEC Web Conference, Vol. 276, 2019.
[2] MASRILAYANTI, M. – NASUTION, A. P. – KURNIAWAN, R.: Seismic Response of Cable-Stayed Bridge Subjected to Single-Support Excitation on Various Soil Condition. E3S Web of Conferences, Vol. 156, 2020.
[3] ARNAUD, S. – MATSUNAGA, N. – NAGANO, S. – RAGARU, J. P.: Behavior of A Multiple Spans Cable-Stayed Bridge. Proceedings of the International FIB Symposium 2008 - Tailor Made Concrete Structures: New Solutions for our Society, 2008.
[4] BSNI.: Indonesian Bridge Designing Code of Earthquake Loads (in bahasa), SNI 2833, 2016.
[5] SILITONGA, D.R – IMRAN, I.: Seismic Vulnerability Assessment of Existing Compound Span Continuous Concrete Box Girder Bridges through Development of Analytical Fragility Curves (in bahasa). Jurnal Teknik Sipil, Vol. 26, 2019.
[6] BAKHTIARI P. - BARGI K.: Seismic Vulnerability Assessment of High-Speed Railway Bridges Using Fragility Curves and Considering Soil-Structure Interaction. Civil Environment Engineering, Vol. 16, Iss. 1, 2020, pp. 170-183.
[7] KIA, S. – SHAHHOSSEINI, V. – SEBT, M. H. – MEILICH, O.: Reliability-Based Life Cycle Assessment of the Concrete Slab in Bridges. Civil Environment Engineering, Vol. 15, Iss. 2, 2020, pp. 142-153.
[8] KIRCHER, C. A. - WHITMAN, R. V. - HOLMES W. T.: HAZUS Earthquake Loss Estimation Methods. Natural Hazards Rev, Vol. 7, Iss. 2, 2006.
[9] HOSSEINPOUR, F. – ABDELNABY, A. E.: Fragility Curves for RC Frames under Multiple Earthquakes. Soil Dynamic Earthquake Engineering, Vol. 98, 2017, pp. 222-234.
[10] VARGA, S. – CHIOREAN C. G.: Seismic Assessment of Reinforced Concrete Frameworks Through Advanced Pushover Analysis and Nonlinear Response of A SDOF Oscillator, Procedia Engineering, Vol. 161, 2016, pp. 332-336.
[11] ASTRIANA, L. – SANGADJI, S. – PURWANTO, E. – KRISTIAWAN, S. A.: Assessing Seismic Performance of Moment Resisting Frame and Frame-Shear Wall System using Fragility Curve, Procedia Engineering, Vol. 171, 2017, pp. 1069-1076.
[12] NAZRI, F. M.: Fragility Curves. SpringerBriefs in Applied Sciences and Technology, 2018.
[13] HASAN, L. – XU, L.: Push-over Analysis for Performance-Based Seismic Design, Computer & Structures, Vol. 80, Iss. 31, 2002, pp. 2483-2493.
[14] RANCK, F.: Applying Safety and Operational Effects of Highway Design features to Two-Lane Rural Highways. TAC/ATC 2003 - 2003 Annual Conference and Exhibition of the Transportation Association of Canada, The Transportation Factor, 2003.
[15] GEHL, P. - D'AYALA, D.: Accuracy of Simplified Procedures for the Fragility Assessment of Bridges. SECED 2015 Conference, 2015.
[16] FACCHINETTI, S.: A Procedure to Find Exact Critical Values of Kolmogorov-Smirnov Test, Statistica Applicata – Italian Journal of Applied Statistics, Vol. 21, 2009, pp. 337-359.