Seismic vulnerability assessment of three spans girder bridge in Kuranji – Padang by developing fragility curve

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Abstract. Bridges play an essential role as a connecting road network for emergency response activities and evacuation routes at the moment after earthquakes. The bridge also performs as the main route for transporting commodity goods. In this study, a prestress concrete I girder (PCI) multi-span bridge is located at high risk of earthquake, Bypass Road, Kuranji Village, West Sumatra Province-Indonesia, is analyzed to obtain its performance under earthquake. The bridge is located on the main route from Teluk Bayur Port to Minangkabau International Airport. Besides, this bridge has been planned before 2016, where the earthquake load regulation was smaller than the earthquake required by the latest one. Therefore, to find out the level of damage stage that might occur due to an earthquake on the bridge, it is necessary to have a tool, namely the fragility curve developed by using lognormal distribution function based on the structural response from pushover analysis and non-linear time history analysis. The study results showed that when the response spectrum target occurred, the bridge damage category based on HAZUS grouping was a Moderate category. Then, from the fragility curve developed, the probability of exceeded damage level for the HAZUS category of Slight, Extensive and Complete damage can also be known.

1. Introduction
Fragility curves show a relation between the probability value of the damage level to a building structure due to earthquake intensity. Assessment of building fragility due to earthquake aims to identify the vulnerability (fragility) of a building caused by an earthquake disaster. Fragility curves are used to assess the seismic vulnerability on various structures such as building systems [1] [2], bridge structures [3] [4] [5] [6], mountain tunnels [7], etc. Prestress Concrete Bridge I (PCI) girder on Bypass Road, Kuranji Village, West Sumatra Province, Indonesia, with 40m + 50m + 40m span configuration, is located in a high-risk seismic zone. For decades, the worst earthquake recorded to hit this province was the earthquake in September 2009 with Mw = 7.6 [8]. Since the bridge plays a vital role as a connecting road network for economic growth and other structural developments so that after an earthquake hit a city, the bridge must continue to function to carry out its role in emergency response activities, evacuation routes, and the sustainability of transportation facilities [3]. The PCI girder bridge at Kuranji Bypass is very significant for transportation activities, especially transporting commodities to and from the port of Teluk Bayur. This path is the only route that allows vehicles to carry heavy loads without entering urban areas.

To anticipate infrastructure damage caused by the earthquake, Indonesia has made a minimum regulatory requirement for design. Specifically for seismic bridge design, the latest regulation issued...
by the government is SNI 2833: 2016. The renewal of the earthquake hazard map is also being carried out to deal with the increasing intensity and magnitude [9]. Therefore, to find out the level of damage that might occur due to the earthquake on the Kuranji PCI girder bridge planned before 2016 and given that this bridge plays an important role for commodity transportation, it is necessary to have a tool, namely a fragility curve.

This study will develop the fragility curve by conducting pushover analysis and non-linear time history analysis (NLTHA). The research uses the bridge's three-dimensional (3D) analysis model. It applies the ground motion history due to the Padang, El Centro, Northridge, and Kobe, which have been matched to the Padang Spectrum Response Target with medium site classes (SD). The plastic hinge patterns that occur on the bridge are also observed through pushover analysis. The fragility curve developed is used to identify the potential damage that occurs due to various levels of earthquake intensity.

2. Theoretical

2.1. Fragility curve

Fragility curves are curves that relate the probability value of structure damage level due to earthquake intensity. Assessment of structure fragility due to earthquake aims to identify the vulnerability (fragility) of a building caused by earthquake disasters [3]. Structural vulnerability to earthquakes can be determined through four stages: hazard analysis, structure analysis, damage analysis, and loss analysis. For damage analysis, the input is needed in the form of an Engineering Demand Parameter (EDP), which can be calculated through structural analysis. Damage analysis results in damage level or Damage Measure (DM), which value is stated in the damage state. Thus, the fragility curve can be used to determine the probabilities of exceeding a level of damage or damage state [10].

Previously, many studies are using different equations to produce their version of the fragility curve. Some researchers such as Yamaguchi and Yamazaki (2000), Kirçil and Polat (2006), and Ibrahim and El-Shami (2011) use Equation 1 in their studies. The formula is the simple version used in fragility analysis. Yamaguchi and Yamazaki (2000) tested it for various structures and found it suitable for all structural types [11]. The fragility equation is given in the form of the main parameters as follows:

\[ \text{Fragility} = P[\text{LS}|\text{IM}] = y \]  

Where LS is boundary conditions or damage conditions (DS), IM is a measure of ground motion intensity, and y is a state achieved due to ground motion intensity (IM).

This equation is idealized into the lognormal distribution as follows:

\[ P(x) = \Phi\left(\frac{\ln(x) - \lambda}{\varsigma}\right) \]  

With \( P(x) \) is a probability function, \( X \) is a random variable, \( \varsigma \) is the standard deviation value of \( \ln x \), \( \lambda \) is the average value of \( \ln x \), and \( \Phi \) is the cumulative normalization distribution form. Lognormal is used because it is very suitable for determining various types of damage from the failure of structural components, non-structural components, building collapse, and zero probability at EDP equal to zero and below zero [10].

Hazus developed by FEMA uses Equation 3 to determine the damage probability in the fragility curve. Demand parameters to determine the threshold damage level or the threshold of damage state used is SD (spectral displacement) [12]. An example of a curve of damage probability relationship and demand for spectral displacement parameters can be seen in Figure 1.

\[ P[\text{dslSD}] = \Phi\left[\frac{1}{\beta_{\text{dsl}}} \ln \left(\frac{\text{Sd}}{\text{Sd,ds}}\right)\right] \]  

\( P[\text{dslSD}] \) is a probability function, \( (\text{Sd},)_{\text{dsl}} \) is the median value of spectral displacement, \( \beta_{\text{dsl}} \) is the standard deviation value of \( \ln x \), and \( \Phi \) is cumulative normalization distribution form.
Figure 1. Fragility curve
(Source: Hazus-MH 2.1)

Fragility curves are developed with equations similar to Equation 3, but using PGA (Peak Ground Acceleration) or earthquake acceleration in the bedrock as a demand parameter to determine the threshold level of damage or threshold of damage state as in Equation 4.

$$P[DS_{PGA}] = \Phi \left[ \frac{1}{\beta_{ds}} \ln \left( \frac{PGA}{PGA_{ds}} \right) \right]$$  \hspace{1cm} (4)

where $P[DS_{PGA}]$ is a probability function, $PGA_{ds}$ is the median value of spectral displacement, $\beta_{ds}$ is the standard deviation value of $\ln x$, and $\Phi$ is cumulative normalization distribution form.

2.2. Damage states
Fragility curves are essential for determining the probability of damage (Damage States / DS) in structures with a given earthquake intensity (Intensity Measure / IM) [3]. The level of damage is determined based on the HAZUS method, where it consists of four levels of damage, as shown in the following Table 1:

| Damage     | Description                                                                 |
|------------|-----------------------------------------------------------------------------|
| Slight     | • Cracking and spalling minor on the abutment                               |
|            | • Shear key crack on the abutment                                           |
|            | • Minor cracks and ruptures of plastic joints                              |
|            | • Minor broken pieces (only need minor / non-structural repairs)            |
|            | • Light cracking on decks                                                   |
| Moderate   | • Medium shear and fracture cracks in columns (columns are still structurally strong) |
|            | • There is a moderate displacement in the abutment                           |
|            | • Medium cracks and breaks on the shear key                                 |
|            | • The connection has cracked the shear key                                  |
|            | • Keeper bar collapse, rocker bearing collapse                             |
|            | • There is a moderate settlement on the surface                             |
| Damage      | Description                                                                 |
|-------------|-----------------------------------------------------------------------------|
| Extensive   | There is a decrease in strength without a collapse in the column - failure due to shear - (the column is not structurally safe) |
|             | • There is a large residual displacement at the connection                   |
|             | • There was a large settlement in the approach                              |
|             | • There is a vertical offset in the abutment                               |
|             | • A differential settlement occurred on the connection                      |
|             | • Shear critical collapse on abutments                                     |
| Complete    | • All columns collapse altogether; connection failures in bearing support are immediately followed by deck failures |
|             | • Slanted substructure due to foundation collapse.                         |

Several parameters are used to determine the level of structural damage due to an earthquake. The parameters are displacement, curvature, ductility, drift ratio, etc [14]. The level of structural damage is influenced by the performance of the structure itself. The grouping of performance levels based on the percentage drift parameter refers to 2013 NCHRP 440 as in Table 2. Correlation between Table 1 Damage rate based on HAZUS and Table 2 Bridge Performance / Design Parameters Based on NCHRP 440 (2013).

| Performance (Level) | Description   | Steel strain | Concrete strain | % Drift | Displacement ductility |
|---------------------|---------------|--------------|-----------------|---------|------------------------|
| II                  | Operational   | 0.005        | 0.0032          | 1       | 1                      |
| III                 | Life Safety   | 0.019        | 0.01            | 3       | 2                      |
| IV                  | Near Collapse | 0.048        | 0.027           | 5       | 6                      |
| V                   | Collapse      | 0.063        | 0.036           | 8.7     | 8                      |

It described as follows [3]:
• Slight - Damage state occurs if the drift value is less than 1% and the structure is categorized as operational performance.
• Moderate - Damage rate occurs if the drift value is between 1% - 3% and the structure is categorized as life safety performance.
• Extensive - Damage level occurs if the drift value is between 3% - 5% and the structure is categorized as near collapse performance.
• Complete - Damage State level occurs when the drift value is between 5% - 8.7%, and the structure is categorized as collapse performance.

2.3. Static-pushover analysis
A static-pushover analysis is a statistical-nonlinear procedure in which the structural system is loaded monotonic, which is increased iteratively, through the final condition, to determine the elastic and inelastic range. As a function of strength and deformation, the resulting non-linear force-deformation...
(F-D) relationship provides an overview of the ductility and limits of understanding. Change parameters can be in the form of translation or combination. Pushover is best suited for systems where the primary mode of interaction. In high contribution rate modes, such as approved higher buildings, dynamic analysis is most effective.

In non-linear static analysis or pushover analysis, structures receive lateral loads and loads whose patterns are determined based on specific lateral deformation patterns. Lateral load associated with the earthquake is considered as a static load applied to the future of structures ranging from small. It continues to grow until the structures that have behaved elastically become inelastic/non-linear, determined by using the first yield (plastic joints) post elastic. Loading continues to increase until it reaches maximum capacity [15].

2.4. Dynamic time history analysis
Dynamic time history analysis is a method of analysis to determine the vibrant response time history of buildings that behave linearly or nonlinearly to earthquake ground motion. It is said dynamic load because earthquake load is a function of time, so the response in the bridge structure also depends on the time of loading.

Dynamic-nonlinear techniques can involve Fast Nonlinear Analysis (FNA) or direct integration methods. FNA is a modal application, whereas, by using direct integration, the equations of motion are integrated into a series of time steps to characterize dynamic responses and inelastic behavior. Loading depends on time, so it is suitable for ground motion recording applications. Time history analysis can explain the non-linear effects of material and P-Delta. Earthquake load plan results in the structure will still behave elastic for linear analysis and act inelastically for non-linear analysis. Usually, time history analysis is more often used for non-linear conditions [16,17].

3. Methodology

3.1. Modeling
The structure of the PCI Girder Kuranji bridge is modeled with the Midas Civil 2019 software based on the information contained in the asbuilt image. Modeling can be seen in Figure 2. Bridge Simple-beam PCI girder is a spanning length 40m + 50m + 40m and width 10.5 m with structure type Reinforced Concrete. Distance between girders is 1.85 m (for spans of 40 m) and 1.4 m (for a span of 50 m). The dimension of pillar cross-section is (1.0 x 7.0 m) and height: 5.4 m. It can see in Figures 3 to 6.
3.2. Loading

3.2.1. Static load The static load applied to the structure is:

- Dead Load (MS)
  Is a burden of the bridge structure itself with a specific gravity of 25 kN/m³ as stipulated in SNI 1725-2016
- Additional Dead Load (MA)
It is asphalt load distributed evenly to girder with a specific gravity of 22 kN / m3 as stipulated in SNI 1725-2016.

This bridge's live load is ignored as stipulated in SNI 2833: 2016 Article 6.1 that the load factor for the live load in the extreme boundary state 1 is zero for the standard bridge category.

3.2.2. Dynamic load. The target spectra response, Figure 7 will be matched the time history acceleration data from the original time history of the Padang, El Centro, Northridge, and Kobe. Matching the spectrum response to time history is conducted by using the SeismoMatch program. SeismoMatch is an application that enables to match the earthquake accelerogram to fit the specific target response spectrum, using the Wavelet algorithm. The results of synchronous to the city of Padang's response spectrum target, as shown in Figures 8 to 10.

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**Figure 7.** Padang acceleration and matched to medium soil of the Padang earthquake time history

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**Figure 8.** El Centro Acceleration and matched to medium soil of the Padang earthquake time history
Figure 9. Northridge Acceleration and matched to medium soil of the Padang earthquake time history

Figure 10. Kobe Acceleration and matched to medium soil of the Padang earthquake time history

3.3. Pushover analysis
The analysis is conducted by defining non-linear material and non-linear geometry in the program. This analysis was carried out to obtain structural responses in the form of displacement during the pillar's first yield.

3.4. Dynamic time history analysis
The analysis was carried out to obtain the structural response in the form of maximum displacement due to dynamic time history earthquake loads with various earthquake intensity levels. The investigation is conducted by defining non-linear material, non-linear geometry, and vibrant load time history in the program. The earthquake intensity level is determined by scaling the four dynamic time-history loads. Each earthquake load applied to the bridge structure both in the direction of the bridge
extending and across the bridge must be combined based on the provisions of SNI 2833-2016 article 5.8 that is:
• 100% of the X-direction earthquake force combined with 30% of the Y-direction earthquake force
• 100% earthquake force in Y direction combined with 30% in X direction earthquake force
Each earthquake is scaled from 0 to 2 at intervals of 0.2, so the total seismic load cases are 80.

3.5. Structure Response
The parameters of the response structure used for the development of the fragility curve are the displacement parameters. For this reason, the displacement value of each structural response due to dynamic earthquake scaled must be recapped.

3.6. Development of fragility curve
Fragility curves are illustrated through data structure response processing from pushover analysis and dynamic non-linear time history analysis. Fragility curves are developed using lognormal distribution functions [18].

4. Result
Based on the analysis results, for pushover X direction, the first yield occurs in the tenth step, as shown in Figure 11, and collapse occurs in step 20, as in Figure 12. In contrast, for pushover Y direction, the first yield occurs in the first step, as shown in Figure 13, and collapse occurs in step 2, as shown in Figure 14.

Figure 11. Pier first yield – X direction (long bridge)

Figure 12. Pier collapse – X direction (long bridge)

Figure 13. Pier first yield – Y direction (trans bridge)
Figure 14. Pier collapse – Y direction (long bridge)

From the analysis, the capacity curve is also obtained. It shows the relationship between displacement and base shear acting on the structure. The capacity curve can be seen in Figure 15 for pushover in the bridge's longitudinal direction and Figure 16 for pushover in the transverse direction of the bridge.

Figure 15. The capacity curve of pier minor axis

Figure 16. The capacity curve of pier major axis

The analysis shows that the pillar's significant yield occurs when the displacement is 28 mm due to pushover X direction (longitudinal bridge). For pushover Y direction (transverse bridge), yield occurs when the displacement is shifted 10 mm. The dynamic time history analysis shows that the maximum displacement due to 100% earthquake loading in the towards X direction and 30% in the Y direction is 163.23 mm. Maximum displacement due to loading 100% towards Y and 30% towards X is 17.5 mm.

Based on the non-linear time history analysis results, the maximum displacement that occurred due to the planned earthquake of 0.6 (g) was 32 mm, and the ductility displacement was 1.14. Based on NCHRP, the Kuranji Bypass bridge structure's performance during an earthquake plans to be categorized as Life Safety and based on HAZUS grouping included in the Moderate Damage category. Thus, the probability of the damage state exceeding probability can be determined by referring to Figure 17. Then, the structure response was developed into a fragility curve using lognormal distribution.
Based on the fragility curve figure, the probability of the level of damage exceeded when the earthquake plan PGA 0.6 (g) for the slight damage category is 43%; moderate is 40%, extensive is 33%, Complete is 18%.

5. Conclusion
The result shows that:

- The displacement yield in the elongated bridge (X) is 28 mm. While due to the Y direction pushover (across the bridge), yield occurs when the displacement is 10 mm.
- Maximum displacement due to loading 100% towards X and 30% towards Y is 163.23 mm. Maximum displacement due to loading 100% towards Y and 30% towards X is 17.5 mm.
- When the target seismic design occurred, PGA 0.6 (g), the category of damage to the Kuranji Bypass bridge based on the HAZUS grouping, was life safety with a ductility displacement value 1.31.
- Based on the developed fragility curve, the damage level's probability is exceeded when the earthquake PGA plan is 0.6 (g) for the damage category. Possibility of slight damage rate is 43%, moderate is 40%, extensive is 33%, complete is 18%.

Thus it can be concluded that the Prestress Concrete I (PCI) Bridge girder located on Bypass Road, Kuranji Village has a low seismic fragility.

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