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Analysis of bridge structure with non-linear time history method

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Abstract. Bridges in Indonesia, which built before the last standard SNI 2833:2016 was issued, need to be reviewed to ensure the safety against earthquakes. This study employs non-linear time history analysis as method to analyze and evaluate the bridge structure due to earthquake loads. The structure undergoes a test with earthquake loads from seven records of scaled acceleration ground motion. From the analysis result, scale adjustment of seven earthquakes records towards the target spectrum response demonstrates various effects on plastic joints scheme and earthquake damage values. The evaluation result based on Cisomang highway/toll-bridge analysis denotes that Response modification factor value (R) is within the limit of permitted value of SNI 2833:2016 standard.

1. Introduction

Indonesia is within the area with medium to high earthquake intensity that requires bridges establishment to meet the current standard bridges planning towards earthquake load of SNI 2833:2016. Bridges in Indonesia, especially those that built before SNI 2833: 2016 authorized in 2016, need to be reviewed so that safety and management of its structure can be ensured. Changes in the regulation occur to correct flaws in the previous regulation, including updates to the earthquake region map based on the facts of large-scale earthquakes in Indonesia.

2. Literature review

SNI 2833: 2016 states that the planned earthquake force in a lower structure, and the relationship between structural elements is determined by dividing the elastic earthquake force with modification factor (R). As an alternative of the R factor utilization for structural relationships, monolithic connections between structural elements, such as the columns relationship to the palm foundation, can be planned to receive the maximum force due to the columns plasticity or the connected columns. In dynamic time history analysis, the response modification factor (R) is taken at 1 for all types of sub-structures and the relationship between structural elements [1,2].

The maximum load due to the effect of the planned earthquake which can be absorbed by the elastic building structure (V_e) and the load that caused by the first melting process in the ductile building structure and elastic building structure due to the effect of the planned earthquake (V_y), then the following relationship applies:

$$V_y = \frac{V_e}{\mu} \tag{1}$$

Where: μ is the ductility factor of the building structure.

Nominal earthquake load (V_n) due to earthquake planning effect must be reviewed in the building structures planning.

$$V_n = \frac{V_y}{f_1} = \frac{V_e}{R} \tag{2}$$

The overloading factor and the material contained in the building structure (f_1) set at 1.6 value.

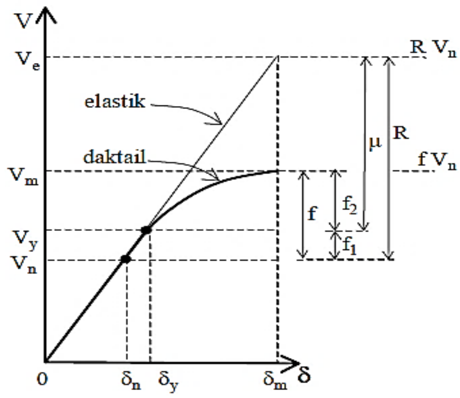


Figure 1. Load – displacement diagram the building structure.

The nominal earthquake load V_n should be reviewed in the planning of building structures. The value of V_n is lower than V_y, thus the ratio of V_y / V_n represents the over-load factor and material (f_1) contained in the building structure. The earthquake reduction factor (R) value varies according to the ductility value (μ), as shown in figure 1 [3].

3. Methodology

This research study will discuss the analysis and evaluation of Cisomang toll-bridge performance towards earthquake loads based on SNI 2833: 2016 standard. The analysis and evaluation will be conducted utilizing the non-linear time history analysis method.

The bridge structure will be tested with seven earthquake records which have convergent characteristics to the spectrum response area and soil type around Cisomang toll-bridge. This earthquake record has occurred elsewhere. Before adjusting the records, the scale will be customized to the spectrum response of Cisomang toll-bridge location.

The object of the bridge structure that is being analyzed and evaluated for its performance is P4 Cisomang toll bridge frame, which is the highest frame among the other frames. This frame also bears the longest span of inter-pillar loads. The aim of this analysis is to obtain the shear force, deformation, yielding point, plastic hinge scheme, and response modification factors. The object of this shown in Figure 2.

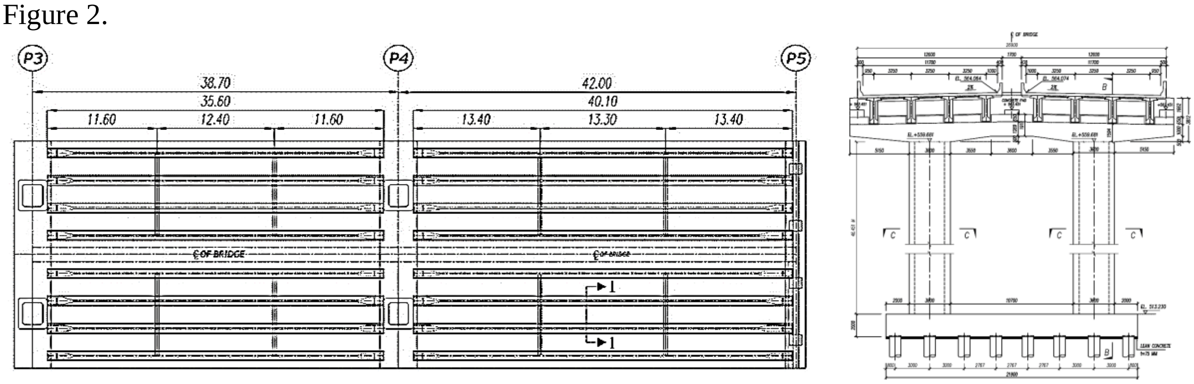


Figure 2. Frame P4 of Cisomang toll-bridge (source: *PT. Jasa Marga Cabang Purbaleunyi*).

4. Results and discussion

The analysis conducted in this study aims to analyze the structure model of Cisomang toll bridge pillar with the help of SAP2000 structural analysis software. The structure dimension and size of the bridge pillar are adjusted to the top structure dimension and size of the Cisomang toll bridge.

Furthermore, the bridge pillar structure will be analyzed under earthquake loads in transversal direction. The analysis exerts non-linear time history analysis method. In this analysis, dynamic loading towards the structure will be executed in the form of earthquake acceleration ground motion records which has been scaled beforehand concerning the design spectrum response of Cisomang toll bridge area based on SNI 2833: 2016 earthquake standard.

4.1. Soil type and soil carrying capacity

Data for the earthquake area and soil type must be determined before calculating the earthquake load. To determine the type of soil, soil investigations are carried out to a depth of 30 m under the bridge structure. Soil investigation had been carried out using the standard penetration test (NSPT) method that represent soil conditions. The results of calculations with an average NSPT value according to SNI 2833: 2016 are 41.27 which is identified into medium soil category.

4.2. The structural model of P4 Cisomang toll bridge pillar

The dimensions of structural elements are modeled according to P4 Cisomang toll-bridge pillar as-built drawing. The material specifications exerted for the pillar are 45 MPa of concrete quality (f'_c) and 400 MPa of steel reinforcement quality.

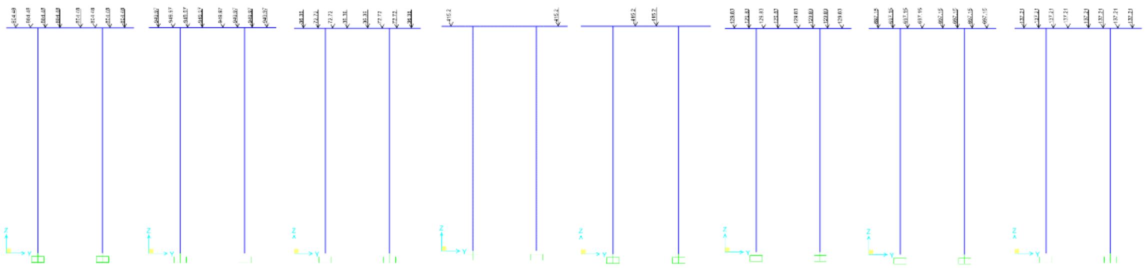
4.3. Loading

The gravity loads included in the non-linear time history analysis are dead load and life load [2]. Cisomang toll bridge is considered as a critical type of bridge, so the living load that runs during an earthquake is multiplied by a reduction factor of 0.50. The pillar and pier head weight is calculated automatically by SAP2000 software.

- PCI girder self-weight is distributed to eight points on the bridge pier head, where each point holds 804.48 kN. The loading scheme shown in Figure 3 (a).
- The self-weight of the floor slab and RC plank is distributed to eight points on the bridge pier head, where each point holds 949.97 kN. The loading scheme shown in Figure 3 (b).
- The self-weight of diaphragm is distributed to each edge of the lane which holds 36.36 kN and at each midpoint of the lane which holds 72.72 kN. The loading scheme shown in Figure 3 (c).
- The self-weight barrier which distributed at each end of the pier head is 415.20 kN. The median weight distributed at the center of the pier head is 830.40 kN. The loading scheme shown in Figures 3 (d) and 3 (e).
- Additional dead load such as the weight of asphalt layer distributed to each point which holds 129.83 kN. The loading schematic can be seen in Figure 3 (f).

The traffic load utilized in this loading calculation are:

- The weight of the evenly distributed load (BTR) at each point by 697.15 kN. The loading schematic can be seen in Figure 3 (g).
- The weight of the centralized line load (BGT) at each point by 137.21 kN. The loading schematic can be seen in Figure 3 (h).



(a) (b) (c) (d) (e) (f) (g) (h)

Figure 3. Loading scheme.

4.4. Dynamic earthquake load

The earthquake data utilized in this study was obtained from the PEER (Pacific Earthquake Engineering Research Center) site. In this analysis, the earthquake data were scaled to the proper design response spectra of Cisomang Bridge location.

The soil type of Cisomang toll bridge is a moderate soil as defined using average NSPT. This soil type will be used further to determine the design response Spectra of Cisomang toll bridge location. The spectrum response of Cisomang can be seen in Figure 4.

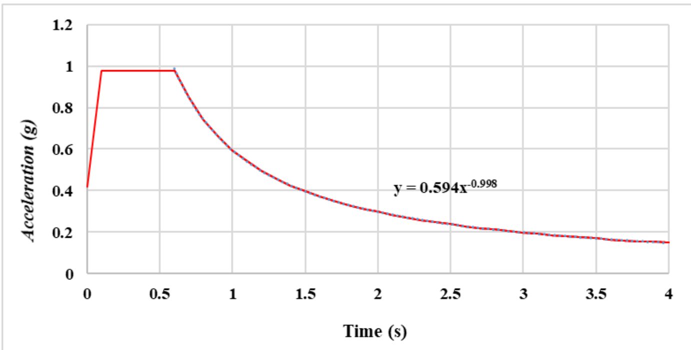


Figure 4. Cisomang medium soil spectrum response (source: *petagempa.pusjatan.pu.go.id*).

4.5. Selection of earthquake ground motion accelerogram

According to Kalkan and Chopra, selected ground motion data could be used in the time history analysis based on several certain criteria [4,5]. Seven earthquake record data are selected, including:

- Imperial Valley Earthquake 1979
- Superstition Earthquake 1987
- Loma Prieta Earthquake 1989
- Erzincan Earthquake, Turkey 1992
- Northridge Earthquake 1994
- Kobe Earthquake, Japan 1995
- Duzce Earthquake, Turkey 1999

4.6. Scale of earthquake ground motion

To input the ground motion acceleration in the non-linear time history analysis, a scaling process is needed due to the differense location and magnitude of the bridge location and the earthquake sources [4-10]. By scaling method, earthquake ground motion can be adjusted to the target spectrum in the reviewed area. In this study, the scaling process is conducted by using the SeismoMatch2018 software.

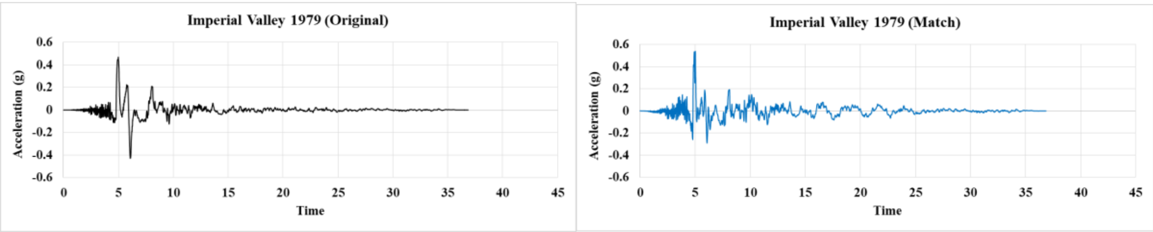
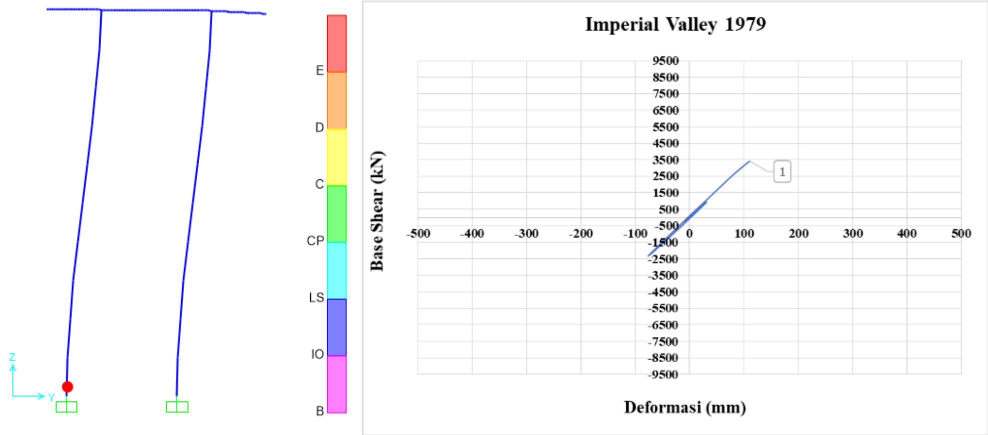


Figure 5. Acceleration ground motion original and acceleration ground motion match.

4.7. Plastic hinge distribution scheme
Plastic hinge model is used to define the non-linear force-displacement or moment-curvature behaviors. In this study, it is assumed that plastic hinges may occur at each end of beam and columns.



T = 4,875 s; V = 3411,57 kN; Δ = 110,86 mm

Figure 6. Acceleration ground motion original and acceleration ground motion match.

The yield forces and the related displacements as results of the analysis using seven ground motion data are shown in the following table 1.

Table 1. The results of plate load test.

No.	Earthquake	V_y (kN)	Δ_y (mm)
1	Imperial Valley	3411.57	110.86
2	Superstition Hills	3387.76	111.10
3	Loma Prieta	3420.51	110.75
4	Erzincan Turkey	3453.94	111.27
5	Northridge	3447.71	112.43
6	Kobe Japan	3488.39	111.87
7	Duzce Turkey	3425.19	110.95

4.8. Modification response factor (R)
The results of the analysis are following:

- The fundamental vibration period (T) obtained from the analysis result is 3.93 seconds.
- Earthquake response factor (C) defined based on the fundamental period and the design spectra shown in Figure is 0.1516.
- Total weight of bridge (W_t), obtained from the SAP2000 calculation results which consist of self-weight, additional load, and live load 31,719.94 kN.
- The importance factor (Ie) of Cisomang toll-bridge according to SNI 2833: 2016, which is categorized as critical bridge, is 1.05.

- Bridge elastic base shear (V_e), calculated by the static earthquake method using the response modification factor value (R) =1.0 is as follows.

$$V_e = (C . I . W_t) /R$$
$$= (0, 1516 . 1,05 . 31719,94)/ 1$$
$$= 5048,99 \text{ kN}$$

The base shear Ve is distributed into two pier - pier head joints.

- Displacement due to elastic shear force (Δe) is of 166.37 mm.
- Response modification factor (R) is calculated using the equation R = (V_e / V_y). f_1. where the over strength factor (f_1) related to the material and structural system is of 1.6.

Table 2. Comparison of the response modification factor (R) value.

No.	Earthhquake	V_y (kN)	V_e (kN)	f_1	R
1	Imperial Valley	3411.57			2.37
2	Superstition Hills	3387.76			2.38
3	Loma Prieta	3420.51			2.36
4	Erzincan Turkey	3453.94	5048.99	1.6	2.34
5	Northridge	3447.71			2.34
6	Kobe Japan	3488.39			2.32
7	Duzce Turkey	3425.19			2.36

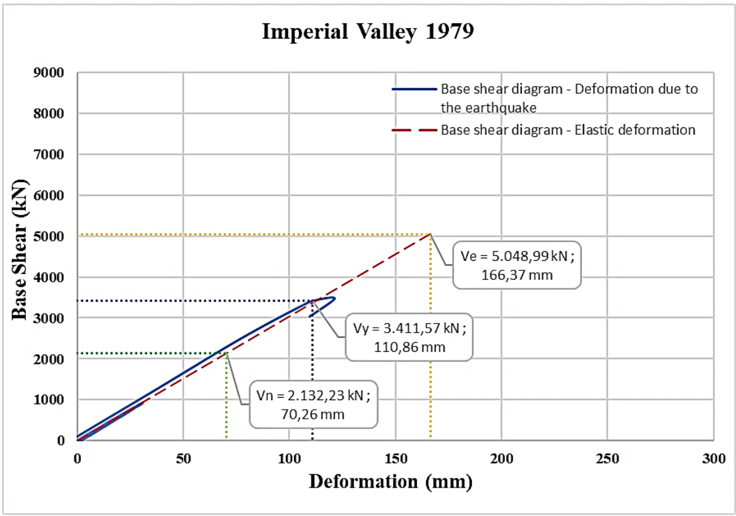


Figure 7. Base shear diagram - deformation due to the imperial valley earthquake.

As an example, elaboration of Figure7 gives the Response Modification Factor R=2.37.

5. Conclusion

The non-linear time history analysis which was conducted using seven ground motion data shows that the Response Modification Factor (R) of Cisomang Bridge ranges from 2.3 up to 2.38. According to SNI 2833:2016, the minimum value of the Response Modification Factor is 1.5. It means that Response Modification Factor of Cisomang Bridge is within the limitation.

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