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The Evolution of Seismic Design Provisions in Indonesia's National Bridge Code

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Highlights:

- It was found that inadequacy of confinement reinforcement of columns and limited ductility are the main issues in pre-and post-seismic design considerations.
- From the pushover analysis results, the performance level of bridges designed in the era before SNI 2833:2016/Seismic Map 2017 will be Operational-Life Safety (LS) (drift = 1.3 % and 1.1%), whereas the performance level of bridges designed according to SNI 2833:2016 will be Elastic-Operational (drift = 0.85% < 1%).
- The performance level of the bridges still satisfies the requirements of NCHRP 949 which is Life Safety under upper-level earthquakes (7% probability of exceedance in 75 years (return period (RP) = 1000 years)).

Abstract. To accommodate increased seismic hazard in Indonesia, provisions regarding structural details in seismic regulations have been tightened. In this paper, variations in seismic hazard and detailing requirements in bridge codes from before 1990 to the present are provided. To examine bridge performance, pushover analysis was carried out based on the latest Indonesian bridge code, SNI 2833:2016/Seismic Map 2017. From the analysis results, the performance of older bridges would typically be less than that of more recently designed structures. The performance level of bridges designed in the era before SNI 2833:2016/Seismic Map 2017 will be Operational-Life Safety (LS), whereas the performance level of bridges designed according to SNI 2833:2016 will be Elastic-Operational. Referring to NCHRP 949 for bridge performance level evaluation, the results of this study showed that the performance level of the bridges still satisfies the requirements of NCHRP 949 which is Life Safety under upper-level earthquakes. Therefore, existing bridges still have adequate capacity under the current seismic load in Seismic Map 2017 (7% probability of exceedance in 75 years (RP = 1000 years)). Evaluation of seismic vulnerability needs to be done to ensure the safety of existing bridges in Indonesia, most of which are located in earthquake-prone areas, especially those that were designed under older version regulations.

Keywords: *bridge design code; ductility; existing bridges; moment-rotation, performance level; seismic hazard.*

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1 Introduction

The territory of Indonesia is located in a very active tectonic zone where the world's three major tectonic plates meet. The presence of interaction between these plates makes Indonesia into a very large area prone to earthquakes, which threatens to disrupt life and damage infrastructure. In conjunction with the occurrence of destructive earthquakes in Indonesia, the majority of casualties and losses are caused by the destruction of infrastructure. It is interesting to note that several earthquakes with intensity less than V MMI have also caused structural damages in recent years. This illustrates that many structures in Indonesia do not meet the provisions for earthquake-resistant structures.

Evaluated from their structural detailing, bridges in Indonesia that have been constructed before 1990 may not have sufficient seismic design detailing. Studies have found that inadequacy of confinement reinforcement of columns and limited ductility are the main issues in pre-and post-seismic design considerations. These results have also been shown in other countries, even in moderate seismic regions, as stated in the studies by Choi, *et al.* in [1], Mitchell, *et al.* in [2], Ramanatan K., *et al.* in [3], Simon and Vigh in [4].

Lessons learned from previous earthquakes have forced the improvement of seismic regulations from time to time to accommodate the increasing hazard. For example, for the DKI Jakarta area, the PGA value is set at 0.15 g on the 2002 Seismic Map [5] (10% probability of exceedance in 50 years, or return period (RP) = 500 years) and at 0.25-0.30 g on the 2010 Seismic Map [6] and the 2017 Seismic Map [7] (7% probability of exceedance in 75 years (RP = 1000 years). The increase in seismic demand certainly has implications for the design provisions in bridge seismic detailing (Table 1). This raises questions regarding the seismic performance of existing bridges that were designed and built many years ago but are still operating until now. Evaluation needs to be done to ensure the safety of these bridges during and after an earthquake and will also be the basis for retrofitting to maintain performance (control and decision support system). The primary goal of this research was to identify the seismic detailing and seismic design force from different versions of Indonesian bridge design codes prior to 1990 until now, as well as to present substantial changes in the seismic performance level of existing bridges that were built in different design/construction eras.

In this paper, to illustrate the changes in seismic performance of older version codes compared to the latest code, the focus of this research was a multi-span girder concrete bridge, which is the bridge type with the largest population (>50%) in Indonesia based on data from the Ministry of Public Works and Housing (Figure 1). The bridge structure analyzed was located in the DKI Jakarta area, which is a strong earthquake zone in Indonesia. Jakarta as center of the economy and the government has various vital infrastructure elements. It needs to be underlined that the results should be applicable to estimate the seismic performance of the bridge stock under conditions of the same typological class (type of pier, type of deck, and type of pier-to-deck connection).

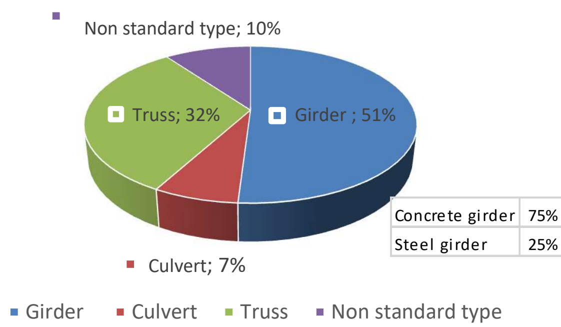


Figure 1 Percentage of bridges on national roads in Indonesia [8].

2 Seismic Hazard and Seismic Design Provisions in Indonesia

All efforts to reduce the risk of earthquake hazard are carried out with preventive disaster management. One of the efforts made is updating seismic regulations. Continuous updating of earthquake maps and bridge codes is an important requirement. Currently, the latest regulations for bridge design used are SNI 2833:2016 and Seismic Map 2017. Earthquake maps that have been used in Indonesia bridge codes can be seen in Figures 2 to 6.

The history of fundamental changes to these regulations, especially related to seismic provisions, is as follows:

1. Before 1990
   Earthquakes were considered in the Indonesian Concrete Code (PBI 1966), Indonesian Loading Regulation PMI 1970 [9] and PPI 1981 based on New Zealand Code 1980. The seismic map used in this code considered only three zones, with an acceleration of 25, 50, 100 gal, respectively. The static equivalent procedure was used in this era to calculate the equivalent horizontal acceleration on structures according to their height.

2. 1990-2004
   Subsequently, the Indonesian Earthquake Resistant Planning Regulation for Buildings 1983 (PPTGIUG-1983) was published as a revision of PMI 1970 and PPI 1981. It was basically based on New Zealand Code 1980. The map

used was the 1983 Seismic Map, which distinguishes six earthquake zones, with Region 1 for a high level of seismicity up to Region 6 for a low level of seismicity, with a return period of 200 years (10% probability of occurring within a 20-year period). This map was used in BMS 1992 [10] and the basic shear coefficient, C, was determined depending on the earthquake area, the period of the structure, and the soil conditions where the bridge will be built, which was determined directly using the inelastic response spectra after dividing by a structural ductility of 4. BMS 1992 was the first Indonesian Bridge Code in which the structural response factor was made a function of the period of the structure.

3. 2004-2016
   Pd T-04-2004-B Earthquake Load Planning Guidelines for Bridges [11] was issued to complement the Bridge Load Regulations BMS 1992, which contained dynamic seismic load planning. The earthquake map used was from Pusair (2004), with a return period of 500 years. Two base shear coefficients were introduced, the elastic and the plastic base shear coefficient. In 2008, the earthquake load regulation SNI 2833:2008 [12] was issued, which basically applied the same seismic map and the concept of calculating earthquake loads using Pd T-04-2004-B. The main change was the variation of PGA values in the six earthquake regions based on varying return periods (50 years, 100 years, 200 years, 500 years, and 1000 years).

4. 2016-now
   In 2010, the new Indonesia Seismic Hazard Map 2010 was issued, followed by publication of SNI 2833:2016 [13], which refers to the AASHTO LRFD Bridge Design Specification, 5th Edition, 2012. The revision included updating of the seismic map used, classification of seismic performance categories (Categories 1, 2, 3, 4), earthquake load planning, and details of structural elements. The bridge structure must be designed to withstand earthquake forces with a return period of 1000 years, with a minimum design life of 75 years, and a probability of earthquake forces exceeding 7%. Furthermore, Seismic Map 2017 was issued with the identification of new seismic sources by using the most current methodology and up-to-date data. Previous studies on the development of a new risk coefficient for infrastructure codes were performed by Irsyam, *et al.* [14] and Sengara, *et al.* [15]. In the new seismic map from 2017, earthquake zones are expressed based on acceleration contours in color gradients and PGA, S_s, and S_1 values with more complete return periods for specific structures.

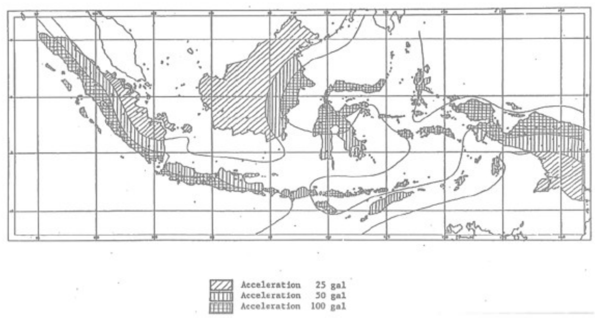


Figure 2 Seismic map 1970 [9].

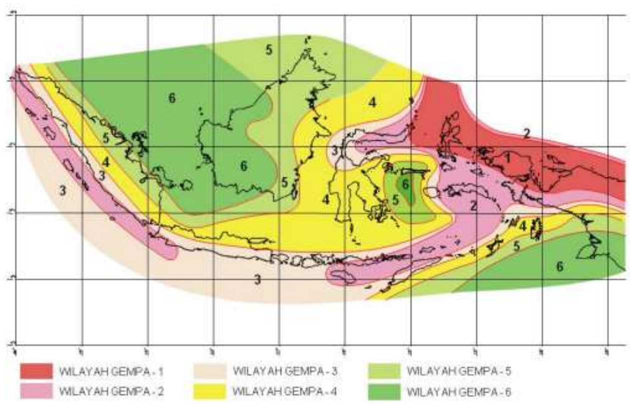


Figure 3 Seismic map 1983 [10].

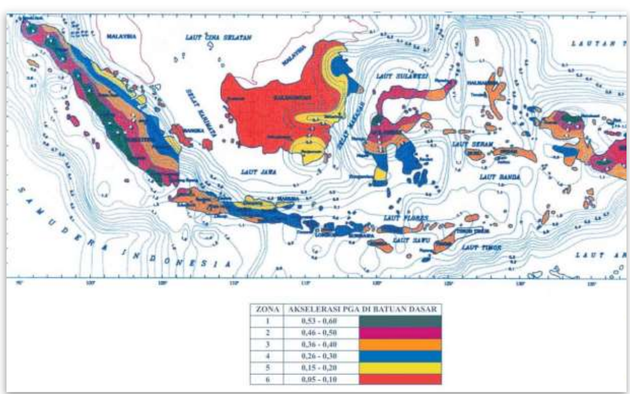


Figure 4 Seismic map in Pd T-04-2004-B/SNI 2833:2008 [11,12].

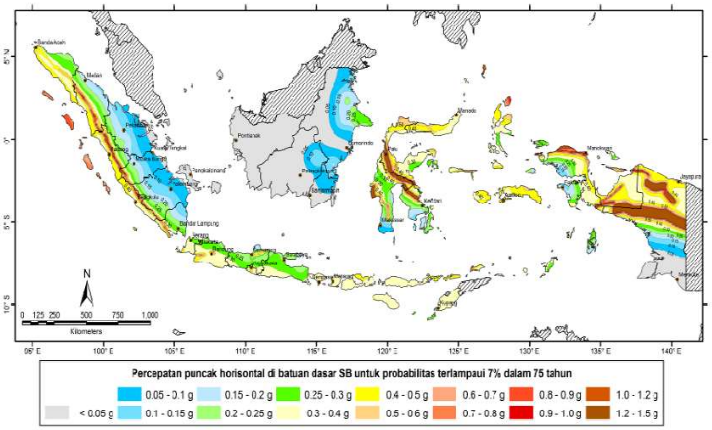


Figure 5 Seismic map 2010 [13].

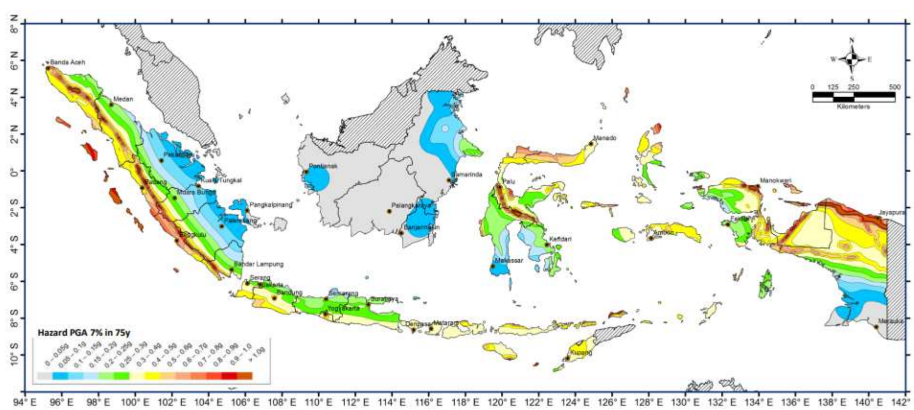
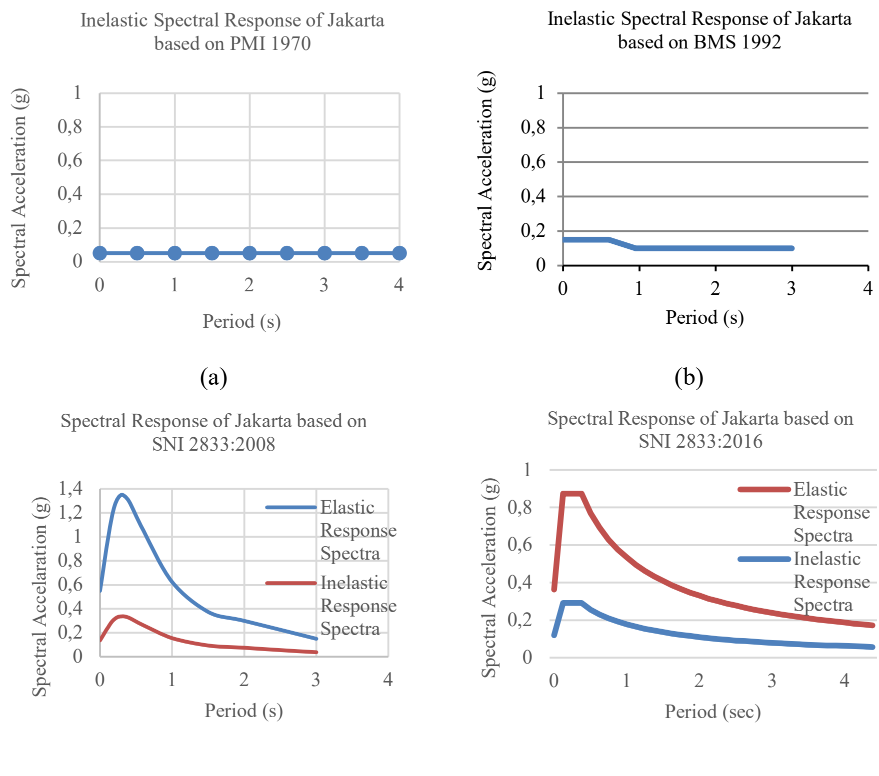


Figure 6 Seismic map 2017 [7].

As an illustrative example, the earthquake design response spectra for the Jakarta region with soft soil condition for various seismic codes can be constructed as shown in Figure 7. It should be underlined that the old seismic codes PMI 1970 and BMS 1992 used inelastic response spectra (plastic base shear coefficient) that were scaled down from the elastic spectrum with a certain ductility value. Older bridge codes did not recognize the concept of inelastic structural behavior or did not implement an understanding of the expected mechanism of bridge collapse. This is in contrast to the 2002 seismic map and subsequent maps, which used elastic response spectra.



(a) (b) (c) (d)

Figure 7 Spectral response of Jakarta based on (a) PMI 1970 (b) BMS 1992 (c) SNI 2833:2008 (d) SNI 2833:2016 (Seismic Map 2017).

Table 1 below provides a summary of the seismic detailing provisions found in various Indonesian bridge code versions.

Table 1 Indonesian seismic codes differences.

Aspects	**PBI 1971/PMI 1970**	**BMS 1992**	**Pd T-04-2004-B**	**SNI 2833:2008**	**SNI 2833:2016**
Seismic Zoning	3 zones	6 zones	6 zones	6 zones	By acceleration contours (in colour gradient)
Seismic Parameter	PGA	PGA	PGA		PGA, S_s, S_1

Table 1 Continued. Indonesian seismic codes differences.

Aspects		**PBI 1971/PMI 1970**	**BMS 1992**	**Pd T-04-2004-B**	**SNI 2833:2008**	**SNI 2833:2016**
Base Shear Coeffi-cient		The static equivalent procedure was used to calculate the equivalent horizontal acceleration according to its height. Fih = ai W ai = ki kd kt	Plastic base shear coefficient based on structure period, seismic zone, and soil condition (6 diagrams)	Semi-dynamic and dynamic analysis Elastic base shear coefficient $C_{elastic} = \frac{1.2AS}{T^{2/3}} \leq 2.5A$; Static analysis Plastic base shear coefficient as mentioned in BMS 1992		Elastic base shear coefficient EQ = Csm/R x Wt For T<To → Csm= (SDS-As) $\frac{T}{To}$+As For To≤T≤Ts → Csm = S_{DS} For T> Ts → Csm $=\frac{S_{D1}}{T}$ $T_0 = 0{,}2\ T_S$, $T_S = \frac{S_{D1}}{S_{DS}}$
Analysis procedure		Static Analysis	Semi-dynamic	Semi dynamic, dynamic, and static analysis		
Seismic performance category		-	Structural types, B, C	Structural design category (SDC) A, B, C, D		Seismic zone 1, 2, 3, 4
Plastic hinge region detailing	Location		$L = max\begin{cases} colun \\ \frac{1}{6} \\ 450 \end{cases}$	$L = max\begin{cases} 1.5\ column\ \varnothing \\ \frac{1}{4}H \\ 600\ mm \end{cases}$		$L = max\begin{cases} column\ \varnothing \\ \frac{1}{6}H \\ 450\ mm \end{cases}$
	Confining steel		$max\begin{cases} \frac{1}{2} * 0.45\ h\ s(\\ \frac{1}{2} *\ 0.12 \end{cases}$	$\rho_s \geq max\begin{cases} 0.45\left(\frac{A_g}{A_c} - 1\right. \\ 0.12\frac{f_c'}{f_{yh}} \end{cases}$ Square stirrups may be used and the area of reinforcement in each major direction of the cross-section greater of:: $A_{sh} = 0{,}3\ s_h\ h_c\ (\frac{Ag}{Ac} - 1)\frac{fc'}{fyh}$ or $A_{sh} = 0{,}12\ s_h\ h_c \frac{fc'}{fyh}$	The volume of a closed spiral or circular is determined from the ratio which is the largest value of: $\rho_s \geq max\begin{cases} 0.45\left(\frac{A_g}{A_c} -\right. \\ 0.12\frac{f}{f} \end{cases}$ $\rho_s \leq 0.018$ Square stirrups may be used and the area of reinforcement in each major direction of the cross-section is the greater of: $A_{sh} = 0{,}3\ s_h\ h_c\ (\frac{Ag}{Ac} - 1)\frac{fc'}{fyh}$ or $A_{sh} = 0{,}12\ s_h\ h_c \frac{fc'}{fyh}$	Shear strength $V_s \leq 0.67\sqrt{f_c'}A_e$ with Ae = 0,8 Ag The area of shear reinforcement for each column core restrained by spiral reinforcement or stirrups must be greater than the given value: $A_v \geq 0.17\frac{D's}{f_{yh}}$
	Confining steel spacing		Min: 15 db; Smallest of section dimension; 100 mm	$s \leq min\begin{cases} 200\ mm \\ 6d_b \end{cases}$		$s \leq min\begin{cases} \frac{1}{4}D \\ 100\ mm \end{cases}$
	Longitudinal reinforcement		-	$0.008Ag \leq \rho \leq 0.06Ag$	$0.01Ag \leq\ \rho\ \leq 0.04Ag$	

It was found that the minimum spacing requirements of transverse reinforcement in the plastic hinge area, plastic hinge length, and longitudinal reinforcement in bridge codes before 1990 meet the requirements, but the confinement reinforcement ratio is less than the minimum required (ρs < ρs min). Furthermore, in BMS 1992 and SNI 2833:2008, the minimum spacing requirement of transverse reinforcement in the plastic hinge area is not as strict as in SNI 2833:2016 (min 1/4D or 100 mm), as shown in Table 2, so in the present study this is categorized as partially confined.

Table 2 Checking seismic detailing in various bridge design codes prior to SNI 2833:2016.

Parameter	Before 1990	1990-2008	2008-2016	2016-now
Longitudinal rebar (SNI 2833:2016 Article 7.4.1.1)	OK	OK	OK	OK
Plastic hinge length, m	OK	OK	OK	OK
Transverse rebar ratio (ρ_s)	NOT OK	OK	OK	OK
Transverse rebar spacing, mm	OK	NOT OK	NOT OK	OK
Category	**Partially confined**			**Fully confined**

3 Methods

To illustrate the changes in seismic performance of older bridge codes compared to the latest SNI 2833:2016 earthquake regulation as well as Seismic Map 2017, the case study used was a reinforced concrete bridge with a pre-stressed girder in Jakarta as the bridge type with the largest population in the DKI Jakarta area. Jakarta has a very high human population and is the center of the economy and the government with various vital infrastructure elements. Particularly Jakarta itself, with its seismic condition as a strong earthquake zone with soft soil condition has been investigated in several studies. A previous study to identify local site conditions in Jakarta was performed by Misliniyati, *et al.* [16]. The bridge structure studied was Cawang-Tanjung Priuk Bridge (P.188), which is located on a toll road in North Jakarta (Jakarta Inner Ring Road-JIRR). The bridge was built before 1990, referring to the PMI 1970 earthquake regulation. The bridge is simply supported by single pier. Based on the data obtained, the technical specifications of the structure are as follows: (a) span length: 35 m (b) number of girders: 10 girders (c) wide span bridge: 25 m (d) pier height: 13.2 m (e) diameter of pier: 3.5 m (f) fc': 29.05 MPa; fy: 400 MPa (g) tendon prestress PC-7-Wire, ASTM A-416, Grade 270. The detailing and configuration of the pier reinforcement: (a) longitudinal rebar diameter: 32 mm, (b) number of longitudinal rebar: 144, (c) plastic hinge length: 3.5 m, (d) transverse rebar ratio: 0,005, (e) transverse rebar spacing: 100 mm. Bridge modeling was carried out in

Midas Civil 2019 as shown in Figure 8. From the modeling, the fundamental period was 1.3 sec in the transversal direction and 0.8 sec in the longitudinal direction.

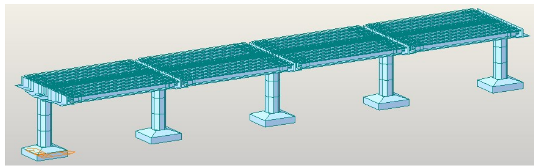


Figure 8 Bridge structure modeling in Midas Civil 2019.

To examine the performance of the bridge, pushover analysis was carried out based on the latest bridge code SNI 2833:2016/Seismic Map 2017. For inelastic modeling, the rotational moment relationship can be obtained by first calculating the cross-sectional moment-curvature relationship. Next, the rotation value is calculated from the curvature value multiplied by the length of the plastic hinge. The concentrated plastic hinge approach was adopted to model the nonlinear behavior of the beams, and the moment-curvature of the sections was obtained from XTRACT [17] by inputting the dimensions of the cross-section, the configuration of the reinforcement, and the axial force on the pier. The inelastic behavior of the concrete material followed the Mander Model [18] and the stress-strain relationship of the steel material followed the Bilinear with Strain Hardening model (Figure 9).

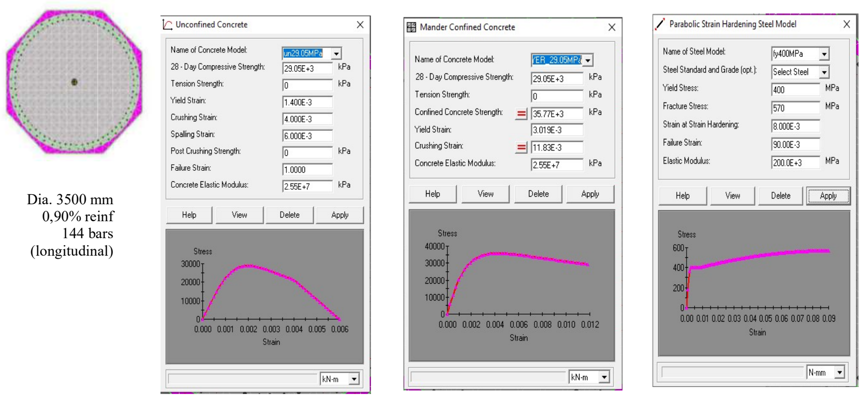


Figure 9 Material properties of the pier section in XTRACT.

Furthermore, calculation of the moment-rotation was carried out by looking at the effect of cyclic loads. Monotonic loads that work continuously will cause a very large degradation of the strength of the specimen and cause a decrease in energy so the effect of cyclic loads, such as when an earthquake occurs, needs to be taken into account. However, monotonic analysis overestimates the strength capacity. The cyclic condition is better for describing inelastic seismic demands. The pre-capping, the post-capping rotation capacities, and the maximum moment were obtained as given by PEER/ATC72-1 [19,20]. Defining the parameters of the first cycle envelope model can be done by modifying the monotonic backbone curve parameters of the XTRACT results (θp* = 0.7 θp; θpc* = 0.5 θpc) (Figure 10).

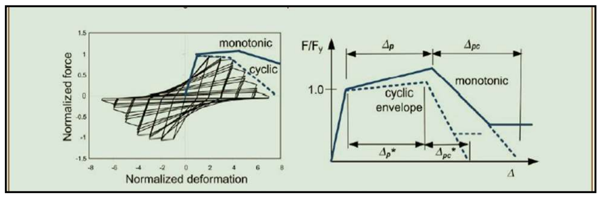


Figure 10 Idealized backbone curves derived from monotonic and cyclic envelope curves (PEER/ATC 2010).

Pushover analysis was carried out by applying a specific static load pattern at the center of mass in the lateral direction and gradually increasing it [21]. In Midas Civil 2019, inelastic hysteresis behavior for pushover analysis is characterized by a FEMA-based skeleton curve with yield and ultimate moment-rotation references (Figure 11). In the analysis, the pier was modeled as a single-pier structure that acts like a structure with a single degree of freedom (SDOF). When performing performance point analysis, the capacity spectrum is formed from the capacity curve so that intersection points with earthquake demand are found.

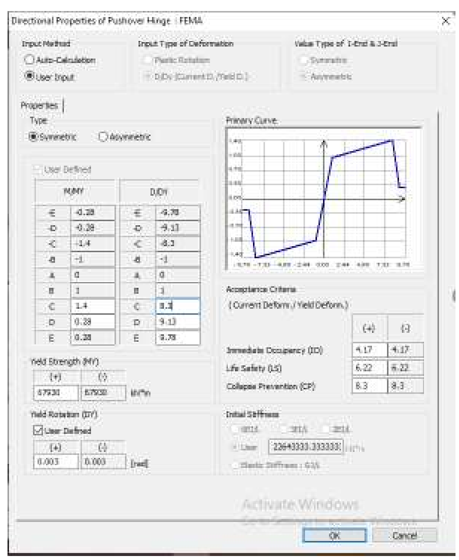


Figure 11 Inelastic hinge properties in Midas Civil 2019.

In this study, seismic response analysis was carried out using a component level approach to the structural pier components as Earthquake Resisting Elements (ERE). Correlation between damage levels and drift (%) as Engineering Demand Parameter (EDP) are given in Table 3, as stated in NCHRP 440 (2013) [22].

Table 3 Bridge performance level (NCHRP 440, 2013).

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	6

According to Zhang and Alam's study, some performance-based requirements for standard bridges are obligated in some references [23], as shown in Table 4.

Table 4 Performance-based requirements for standard bridge.

Reference		Seismic Hazard	Performance Requirements
Japanese Design Specification for Highway Bridges – MLIT	2012	Level 1: Frequent earthquake (Level 1) Level 2: Ground motion from large scale subduction-type (Type I)/major near-field shallow earthquakes that directly strikes the bridges (Type II)	No damage Life safety
California Department of Transportation	2013	Probabilistic spectrum (975-year return period) or deterministic spectrum of near fault effects	Life safety
Federal Highway Administration (FHWA) LRFD Spec	2014	1000 years	Life safety
Washington Department of Transportation	2016	1000 years	Life safety
National Cooperative Highway Research Program (NCHRP) 949	2020	1000 years	Life safety

This study adopted the latest NCHRP 949 Guidelines [24], published in 2020, and applied them in the performance-based evaluation. These guidelines filled a gap in the previous guidelines of the FHWA-LRFD for seismic analysis [25]. There are two levels of earthquake performance acceptance criteria, as also stated in Lim, *et al.* [26]. From Table 5 it can be seen that with the hazard of a 1000-year seismic load, the performance requirements are Life Safety.

Table 5 Performance-based evaluation (NCHRP 949 Guidelines).

Earthquake Ground Motion	Bridge Importance and Service Life Category					
	Standard			Essential		
	ASL 1	ASL 2	ASL 3	ASL 1	ASL 2	ASL 3
Lower-Level Ground Motion 50% probability of exceedance in 75 years; the return period is about 100 years	PL0	PL3	PL3	PL0	PL3	PL3
Upper-Level Ground Motion 7% probability of exceedance in 75 years; return period is about 1000 years	PL0	PL1	PL1	PL0	PL1	PL2

Notes:

1. The Anticipated Service Life categories are ASL 1: 0-15 years; ASL 2: 16-50 years; ASL 3: > 50 years
2. The performance levels are:
   - **PL0:** No minimum level of performance is recommended.
   - **PL1:** Life Safety. Significant damage is sustained and service is significantly disrupted but life safety is preserved. The bridge may need to be replaced after a large earthquake.
   - **PL2:** The operational damage sustained is minimal and service for emergency vehicles should be available after inspection and clearance of debris. The bridge should be reparable with or without restrictions on traffic flow.
   - **PL3:** Fully Operational. No damage is sustained and full service is available for all vehicles immediately after the earthquake. No repairs are required.

4 Results and Discussion

4.1 Comparisons of Seismic Design Force Levels for Various Codes

From the modeling, the fundamental period is 1.3 sec in the transversal direction and 0.8 sec in the longitudinal direction. By comparing the inelastic response spectra from each era, which are scaled down from the elastic spectrum with a certain ductility value, the inelastic base shear coefficient for various bridge codes can be seen in Table 6. It can be seen that with Seismic Map 2017 there was a change in force due to earthquakes on the structure, where the seismic coefficient was three to four times greater than in PMI 1970. The increase in the value of the seismic coefficient affects the lateral earthquake load.

Table 6 Earthquake loading properties and earthquake loading calculation.

	PMI 1970	**BMS 1992**	**SNI 2833:2008**	**SNI 2833:2016 (with Seismic Map 2017)**
Zone Map	Zone 2	Zone 4	Zone 3	Zone 4
	kd = 50 gal = 0,051 g kt = 1 ai = 0,051	T_{bridge} = 1.3 sec Z = 4 I = 1 Structural type: Type B S = 1 C = 0.1 Kh = 0.1	T_{bridge} = 1.3 sec Rd = 4	T_{bridge} = 1.3 sec PGA (g) = 0.272; F_{PGA} = 1.338 Ss (g) = 0.538; S_1 (g) = 0.223 Fa = 1.625; Fv = 3.108 As = F_{PGA} PGA = 0.364 S_{DS} (g) = Fa Ss = 0.874 S_{D1} (g) = Fv S1 = 0.693 To (sec) = 0.2 Ts = 0.159 Ts (sec) = S_{D1}/S_{DS} = 0.793 R = 3
Inelastic Base Shear Coefficient	0.051	0.1	0.12	0.178

4.2 Evaluation of Seismic Detailing Condition

Referring to the detailing requirements in the latest code, SNI 2833:2016, as shown in Table 1, the pier condition as Earthquake Resisting Element (ERE) for the bridge was checked in Table 7. In line with the statement in Table 2, it was found that for bridges designed in the era before 1990, the minimum spacing requirements of transverse reinforcement, plastic hinge length and longitudinal reinforcement meet the requirements, but the confinement reinforcement ratio is still smaller than the minimum required ($\rho s < \rho s$ min) in the plastic hinge area = 0.005 < 0.01 so in this study, this is categorized as partially confined.

Meanwhile, confinement is important to obtain a ductile structure. The presence of confinement in reinforced concrete elements will increase the ductility and compressive strength of concrete by preventing lateral expansion during an earthquake. Ductility is needed for the performance of earthquake-resistant structures because it is the key to ensuring large deformations without collapsing. A ductile structure is able to maintain its strength when inelastic behavior occurs [27,28].

Table 7 Pier detailing check.

Parameter	**Column parameter as-built (actual condition)**	**Check to SNI 2833:2016**		
Longitudinal rebar (SNI 2833:2016	144 D32 (0.9%) Atot = 115.858,3 mm2	0.01Ag ≤ Atot ≤ 0.04Ag	Ag = 10148750 mm^2 0.01 Ag = 101,487.5 mm^2 0.04 Ag = 405950 mm^2	OK
Plastic hinge length, m	3500 mm	$L = max \begin{cases} column \varnothing \\ \frac{1}{6}H \\ 450\ mm \end{cases}$	D = 3500 mm 1/6 pier height = 2200 mm 450 mm L max = 3500 mm	OK
Transverse rebar ratio (ρ_s)	0.005	$\rho_s \geq max \begin{cases} 0.45\left(\frac{A_g}{A_c} - 1\right)\frac{f_c'}{f_{yh}} \\ 0.12\frac{f_c'}{f_{yh}} \end{cases}$ $\rho_s \leq 0.018$	Ag = 10148750 mm^2 Ac = 7793113 mm^2 Ag/Ac = 1.302 (0.45 (Ag/Ac-1) (fc'/fy)) = 0.01 (0.12*(fc'/fy)) = 0.009 ρ_s req = 0.010	NOT OK
Transverse rebar spacing, mm	100	$min \begin{cases} \frac{1}{4}D \\ 100\ mm \end{cases}$ 100	Min: ¼D = 875 mm 100 mm s_{req} = 100 mm	OK

Figure 12 shows the moment-rotation calculation for this actual condition and also for the condition if the concrete is confined and the condition if it is unconfined. It can be seen that the cross-section of the pier in ideal conditions with confining reinforcement that meets the requirements (ρs = 1%) provides better ductility.

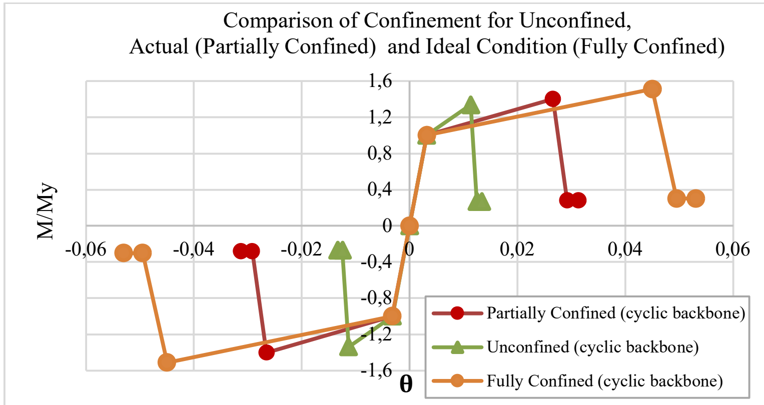


Figure 12 Moment-rotation relationship curves for unconfined, actual condition (partially confined) and ideal condition (confined).

4.3 Pushover Analysis and Performance Level of Existing Bridges in Various Eras

To get a more comprehensive analysis of the seismic performance of bridges in other eras, the seismic detailing of piers from the BMS 1992, SNI 2833:2008 and SNI 2833:2016 eras was defined as required by the design code from these eras, as shown in Table 1, so that the bridge pier detailing from each era could be obtained. For bridges from the BMS 1992 and SNI 2833:2008 eras, $\rho_s = 0.01$, s=200 mm and for bridge from the SNI 2833:2016 era, $\rho_s = 0.01$, s = 100 mm (fully confined).

Pushover analysis was then carried out based on the latest bridge code SNI 2833:2016 as well as Seismic Map 2017. Based on Figure 13 below, it can be seen that the pushover due to Seismic Map 2017, for bridges in the era before SNI 2833:2016/Seismic Map 2017, the structural performance point after inelastic deformation exceeds the yield capacity so the bridge reaches an inelastic condition was obtained. However, bridges designed in accordance with SNI 2833:2016 remain elastic. The drift value as Engineering Demand Parameter (EDP) was obtained from the comparison of the deformation values with the length of the pier elements under consideration. The bridge performance level refers to NCHRP Synthesis 440. The performance level of bridges in the era before SNI 2833:2016/Seismic Map 2017 will be Operational-Life Safety (LS) (drift 1.3 % and 1.1%), whereas the performance level of bridge designed with SNI 2833:2016 will be Elastic-Operational (drift = 0.85% < 1%). The results of the calculation are shown in Table 8.

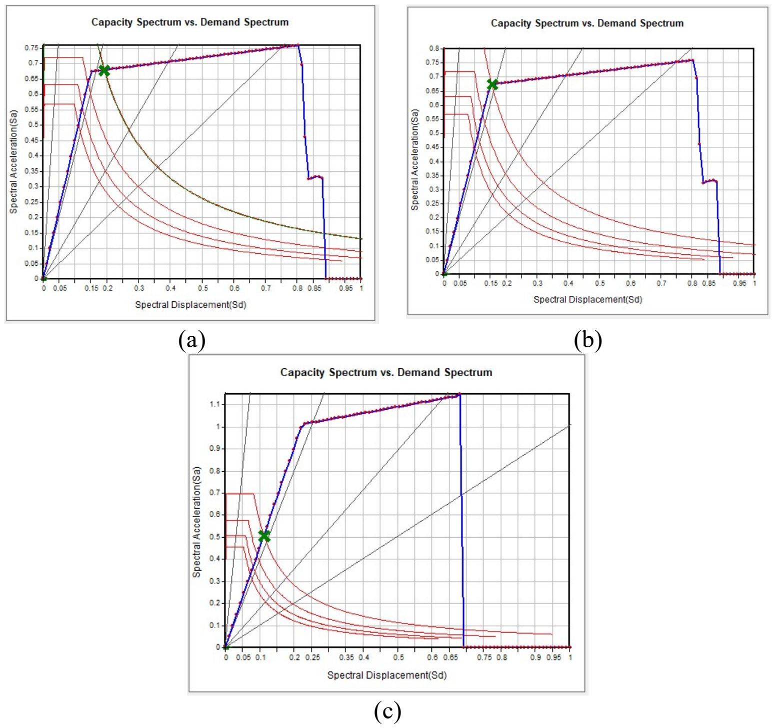


Figure 13 (a) Pier structure performance point due to seismic load in SNI 2833:2016/seismic map 2017 for various bridge design code eras: (a) before 1990, (b) BMS 1992 and SNI 2833:2008, (c) SNI 2833:2016.

Table 8 Comparison of pier drift (%).

Seismic Bridge Design Code	Displacement (m)	Pier Height (m)	% Drift	Performance Level NCHRP Synthesis 440
PMI 1970	0,1658	13,2	1,3	Operational-Life Safety
BMS 1992; SNI 2833:2008	0,1551	13,2	1,18	Operational-Life Safety
SNI 2833:2016	0,1122	13,2	0.85	Elastic-Operational

Referring to the latest NCHRP 949 performance-based evaluation (Table 5), it can be seen that with a hazard of a 1000-year seismic load, the performance requirement is Life Safety. The results show that the performance level of the bridge still satisfies the requirement of NCHRP 949, which is Life Safety under an upper-level earthquake (RP = 1000 years). Therefore, the existing bridge shows adequate capacity under the current seismic load SNI 2833:2016/Seismic Map 2017. It needs to be underlined that these results should be applicable to estimate the seismic performance of the bridge stock under conditions of the same typological class (type of pier, type of deck, and type-of-pier to deck connection).

5 Conclusions

In this paper, a comprehensive summary of seismic hazard and structural detailing provisions from various bridge codes from eras before 1990 to the present is provided, which can be used as the governing parameter to estimate the performance level of existing bridges from each era as the basis for bridge evaluation in Indonesia. The seismic code Indonesian Loading Regulation PMI 1970 and BMS 1992 used inelastic response spectra, in contrast with SNI 2833:2008 and SNI 2833:2016, which used elastic response spectra. Older bridge codes did not recognize the concept of inelastic structural behavior or did not implement an understanding of the expected mechanism of bridge collapse. By comparing the inelastic response spectra from each era, it can be seen that with Seismic Map 2017, there is a change in force due to earthquakes on the structure, where the inelastic seismic coefficient is about three to four times greater than in PMI 1970.

It was found that inadequacy of confinement reinforcement in a column and limited ductility are the main issues in pre-and post-seismic design considerations. In bridge codes before 1990, the minimum spacing requirements of transverse reinforcement, plastic hinge length and longitudinal reinforcement meet the requirements in SNI 2833:2016, but the confinement reinforcement ratio is still smaller than the minimum requirement ($\rho s < \rho s$ min). Furthermore, in BMS 1992 and SNI 2833:2008, the minimum spacing requirement for transverse reinforcement in the plastic hinge area is not as strict as in SNI 2833:2016. From the analysis results, the performance of older bridges would typically be less than more recently designed structures. The performance level of bridges from the era before SNI 2833:2016/Seismic Map 2017 will be Operational-Life Safety (LS) (drift = 1.3 % and 1.1%), whereas the performance level of bridges designed with SNI 2833:2016 will be Elastic-Operational (drift = 0.85% < 1%). Meanwhile, the performance level of these bridges still satisfies the requirement of NCHRP 949, which is Life Safety under upper-level earthquakes (RP = 1000 years). Therefore, existing bridges have adequate capacity under the current seismic load from SNI 2833:2016/Seismic Map 2017 (7% probability of exceedance in 75 years).

References

[1] Choi, E., DesRoches, R. & Nileson, B., *Seismic Fragility of Typical Bridges in Moderate Seismic Zone*, Eng. Struct., **26**, pp. 187-199, 2004. DOI: 10.1016/j.engstruct.2003.09.006.

[2] Mitchell, *Evolution of Seismic Design Provisions in the National Building Code of Canada*, Canadian Journal of Civil Enginnering, **37**(9), pp. 1157-1170, 2010. DOI: 10.1139/L10-054.

[3] Ramanatan, K., DesRoches, R. & Padgett, J.E., *A Comparison of Pre-and Post-Seismic Design Considerations in Moderate Seismic Zones Through The Fragility Assessment of Multispan Bridge Classes*, Eng. Struct., **45**, pp. 559-573, 2012. DOI: 10.1016/j.engstruct.2012.07.004.

[4] Simon, J. & Vigh, L.G., *Seismic Fragility Assessment of Integral Precast Multi-Span Bridges in Areas of Moderate Seismicity*, Bull. Earthq. Eng., **14**(7), 2016. DOI: 10.1007/s10518-016-9947-y.

[5] Ministry of Public Works, *Indonesia Earthquake Source and Hazard Map 2002,* 2002.

[6] Ministry of Public Works, *Indonesia Earthquake Source and Hazard Map 2010,* 2010.

[7] Ministry of Public Works and Housing, *Indonesia Earthquake Source and Hazard Map 2017*, National Earthquake Study Center, 2017.

[8] Directorate General of Highways Ministry of Public Works and Housing, Indonesia [https://data.pu.go.id].(26 July 2021).

[9] Building Problem Research Board, *Indonesian Load Regulations 1970, NI 18*, 1969.

[10] Department of Public Works, *Bridge Management System (BMS) 1992*.

[11] Ministry of Public Works, *Guidelines for Bridge Design for Earthquake Loads, PdT-04-2004-B*.

[12] Indonesian National Standard, *Bridge Design for Earthquake Loads, SNI 2833:2008*, 2008.

[13] Indonesian National Standard, *Bridge Design for Earthquake Loads, SNI 2833:2016,* 2016.

[14] Irsyam, M., Asrurifak, M, Mikhail, R., Wahdiny I., Rustiani, S. & Munirwansyah, *Development of Nationwide Vs30 Map and Calibrated Conversion Table for Indonesia using Automated Topographical Classification*, Journal of Engineering and Technological Sciences, **49**(4), pp. 457-471, 2017. DOI: 10.5614/j.eng.technol.sci.2017.49.4.3.

[15] Sengara I. W., Sidhi, I. D., Mulia, A, Asrurifak, M. & Hutabarat, D., *Development of Risk Coefficient for Input to New Indonesian Seismic Building Codes*, Journal of Engineering and Technological Sciences, **48**(1), pp. 49-65, 2016. DOI: 10.5614/j.eng.technol.sci.2016.48.1.5.

[16] Misliniyati, R., Sahadewa, A., Hendriyawan, & Irsyam, M., *Parametric Study of One-Dimensional Seismic Site Response Analyses Based on Local Soil Condition of Jakarta*, Journal of Engineering and Technological Sciences, **51**(3), pp. 392-410, 2019. DOI: 10.5614/j.eng.technol.sci. 2019.51.3.7.

[17] TRC/Imbsen, *XTRACT: Cross Section Analysis Software for Structural and Earthquake Engineering*, 2010.

[18] Mander, J.B. Priestley M.J.N. & Park. R., *Theoretical Stress-Strain Model for Confined Concrete*, J.Struct.Eng, **114**(8), pp. 1804-1824, August 1988.
[19] ATC 72-1, *Modelling and Acceptance Criteria for Seismic Design and Analysis of Tall Buildings*, PEER/ATC, 2010.
[20] Maison, B.F. & Speicher, M.S., *Loading Protocols for ASCE 41 Backbone Curves.* Earthquake Spectra, 2016. DOI: 10.1193/010816EQS007EP
[21] Chopra, A.K. & Goel, R., *A Modal Pushover Analysis Procedure for Estimating Seismic Demands for Buildings*, Earthquake Engineering and Structural Dynamics, **31**(5), pp. 561-582, 2002.
[22] Transportation Research Board of The National Academies, *Performance-Based Seismic Bridge Design NCHRP Synthesis 440*, Washington: Transportation Research Board, 2013.
[23] Zhang, Q & Alam, M.S., *Performance-Based Seismic Design f Bridges: A Global Perspective and Critical Review of Past, Present and Future Directions*, Structure and Infrastructure Engineering, **15**(4), pp. 539-554, 2019. DOI: 10.1080/15732479.2018.1558269
[24] National Academies of Sciences, Engineering & Medicine, *NCHRP 949 Research Report – Proposed AASHTO Guidelines for Performance-Based Seismic Bridge Design*, Washington, 2020.
[25] Marsh, ML.,, *LRFD Seismic Analysis and Design of Bridges Reference Manual* (Publication No. FHWANHI-15-004), Department of Transportation Federal Highway Administration, 2014.
[26] Lim, E., Kusumastuti, D., Rildova, Asneindram, M. & Mulyadi, S.S., *Performance Based Evaluation of an Existing Continuous Reinforced Concrete Bridge-A Case Study*., E3S Web of Conference, **331**, pp. 0500 8, 2021. DOI: 10.1051/e3sconf/202133105008
[27] American Association of State Highway and Transportation Officials, *Guide Specifications for LRFD Seismic Bridge Design,* in Subcommittee T-3 for Seismic Effects on Bridges, USA, 2011.
[28] American Association of State Highway and Transportation Officials, *AASHTO LRFD Bridge Design Specification, 9th Edition*, Washington, USA, 2020.