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Performance Evaluation of Existing Special Bridges in Indonesia Based on SNI 1725:2016 AND SNI 2833:2016

(Case Study of Dr. Ir. Soekarno Bridge)

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Abstract

Existing special bridges such as cable-stayed bridges which are complex in structure need to be evaluated against SNI 1725:2016 and SNI 2833:2016. Dr. Ir. Soekarno Bridge located in Manado, North Sulawesi, was used as case study. Analysis based on the performance of the bridge was conducted using the Nonlinear Static Pushover Analysis (NSPA) with three different load distributions and Nonlinear Time History Analysis (NLTHA). Due to the standard load of the bridge, the deflection was still below the allowable deflection. Girder has an overstress of 12% in the Service Ic combination. The cable also has overstress in Cable 7 between 0.4% and 6.2%. Lower segment pylon capacity was exceeded by earthquake load combination of 28%. Bridge performance shows that the bridge structure was at fully operational performance level and the element performance was at immediate occupancy level, each method shows different result in base shear and displacement. The result shows that re-evaluation for existing special bridges is required. The bridge performance has different base shear and displacement for each method. Nonlinear Time History Analysis is the better option for special bridge, Nonlinear Static Pushover Analysis method requires a further study to determine the ideal load distribution for special bridge because the load distribution is crucial to determine the result of pushover analysis.

Keywords: Bridge performance, *nonnlinear static pushover analysis, onlinear time history analysis*

Abstrak

Jembatan khusus eksisting seperti jembatan cable stayed yang sangat kompleks perlu dievaluasi kinerjanya terhadap SNI 1725:2016 dan SNI 2833:2016. Studi kasus pada penelitian ini adalah Jembatan DR. Ir. Soekarno di Kota Manado, Sulawesi Utara. Dilakukan pula analisis berdasarkan kinerja dengan Nonlinear Static Pushover Analysis dengan tiga pola beban dan Nonlinear Time History Analysis. Akibat beban standar jembatan, lendutan yang terjadi masih dibawah lendutan ijin. Girder mengalami kelebihan tegangan sebesar 12% pada kombinasi beban Layan Ic. Kabel juga terjadi kelebihan tegangan pada Kabel 7 antara 0,4% sampai 6,2%. Kapasitas pylon segmen bawah terlampaui oleh kombinasi beban gempa sebesar 28%. Analisis kinerja struktur jembatan dengan dua metode didapatkan bahwa tingkat kinerja struktur adalah fully operational dan kinerja elemen adalah immediate occupancy, akan tetapi nilai base shear dan perpindahan berbeda untuk masing-masing metode. Hasil analisis menunjukkan bahwa jembatan khusus eksisting perlu di evaluasi ulang terhadap peraturan terbaru. Kinerja jembatan dengan dua analisis nonlinier yang dilakukan menunjukkan hasil yang berbeda, Nonlinear Time History Analysis tetap menjadi metode yang lebih baik untuk jembatan khusus sedangkan metode Nonlinier Static Pushover Analysis perlu penelitian lanjutan untuk menentukan distribusi beban pushover yang cocok digunakan untuk jembatan cable stayed, mengingat pola distribusi beban sangat menentukan hasil dari analisis pushover.

Kata kunci: Kinerja jembatan, *nonlinier static pushover analysis, nonlinear time history analysis*

1. Introduction

Along with the increasing knowledge about the source, intensity, and strength of earthquakes in Indonesia, where both the intensity and strength of earthquakes are increasing, the current regulations on earthquake need to be updated. The regulatory updates should also let important infrastructures built with the previous

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regulations be evaluated against the latest changes. One of the important infrastructures that needs to be evaluated is the long span bridges or special bridges such as cable stayed. Special bridges such as cable stayed bridges which are very strict in their design criteria need to be evaluated for their performance especially on earthquake loads, since the new earthquake regulations require greater earthquake loads than the previous regulations. The increase in loads that occurred from the Pd T-04-2004-B Guidelines for Planning Earthquake Loads for Bridges to SNI 2833:2016 Bridge Planning for Earthquake Loads is around 28% to 200% (see **Figure 1**) so it is very important to evaluate the performance of the existing special bridges and see whether it can still receive loads with the new regulations. The case study in this study is the Dr. Ir. Soekarno Bridge, located in Manado, North Sulawesi, which was built in 2004 and inaugurated in 2015. An analysis was conducted on the main span which has a span of 2x120 m.

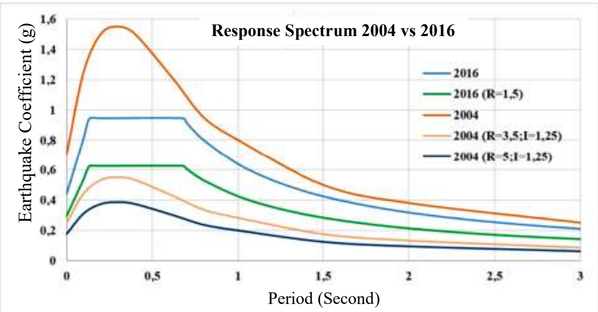

Figure 1. Pd T-04-2004-B and SNI 2833-2016 response spectras

The performance evaluation of this bridge would be divided into two stages. The first stage is the evaluation of the standard load based on SNI 1725: 2016 "Loading for Bridges" and earthquake response spectra based on SNI 2833: 2016 "Bridge Planning for Earthquake Loads" with a force-based approach. This approach is used to assess whether the existing structure can carry forces due to the load that occurs. The second stage is an evaluation with a performance-based approach to find out the performance/behavior of the structure when the loads work. In a performance -based approach, a nonlinear analysis method is used to understand structural behavior when an earthquake occurs. The nonlinear analysis used in this study was the Nonlinear Static Pushover Analysis (NSPA) with 3 loading patterns. The results of using 3 loading patterns would then be compared with the results of Nonlinear Time History Analysis (NLTHA) to find out Nonlinear Static Pushover Analysis (NSPA) with the most reliable loading pattern for special bridges, or in this study, is the cable-stayed bridges.

2. Case Study

Dr. Ir. Soekarno Bridge, located in Manado, North Sulawesi, was chosen as a case study. This bridge was built from 2004 to 2015 with the following technical data:

Bridge Name	:	Dr. Ir. Soekarno
Construction Year	:	2004 – 2015
Upper Building Type	:	Prestressed girder and cable stayed
Lower Building Type	:	Caisson Foundation
Bridge Total Length	:	1.127 m (including the bridge approach)
Bridge Length	:	30 m + 36 m + 36 m + 120 m + 120 m + 30 m (372 m)
Traffic Line Width	:	12 m (undivided 4 lanes with 2 directions)
Sidewalks Width	:	2@2,5 m

At the Dr. Ir. Soekarno Bridge, the quality of concrete used was K-300 (24.9 MPa) used on pillars and K-500 (41.5 MPa) used on the pylon, girder, and floor. The size and details of the cross section used were derived from the As Built Drawing of Dr. Ir. Soekarno Bridge Construction. The pylon has a height of 59.3 meters with a bridge floor elevation as shown in **Figure 2**.

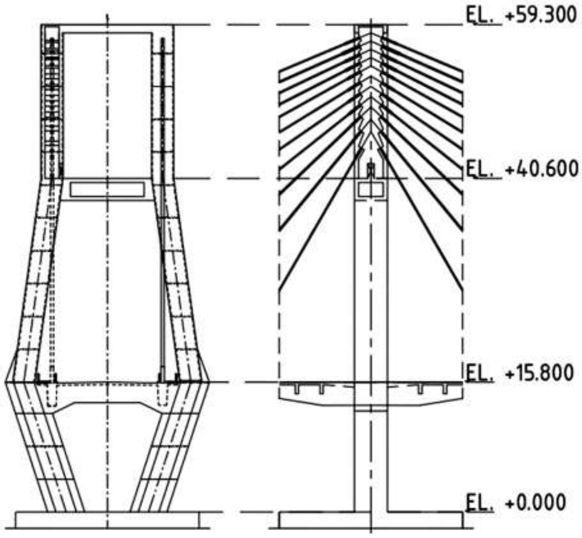

Figure 2. Pylon's longitudinal section and transversal section

The force on the effective prestressed steel was taken at 0.6fpu, while the tensile force on the cable was taken from the Stay Installation data conducted by PT. Freyssinet. The modeling used the Csi Bridge v.20 program and the modeling results can be seen in **Figure 3**.

3. Methodology

The previous modeling was verified with RSNI T-02-2005 Loading Standards for Bridges and Pd T-04-2004 -B Guidelines for Planning Earthquake Loads for Bridges. The analysis shows that all bridge components are safe from loading so they can be used for further analysis.

The bridge condition evaluation starts from the standard bridge load SNI 1725: 2016 which includes dead load, prestressed load, traffic load, pedestrian load, brake load, wind load, temperature influence, and

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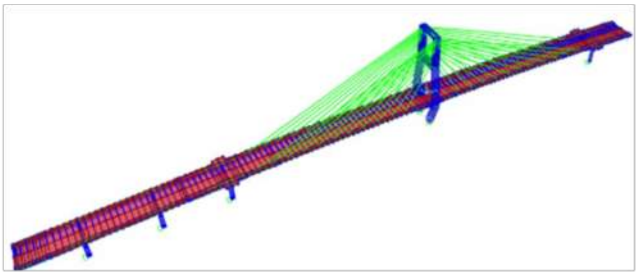

Figure 3. Dr. Ir. Soekarno bridge modeling

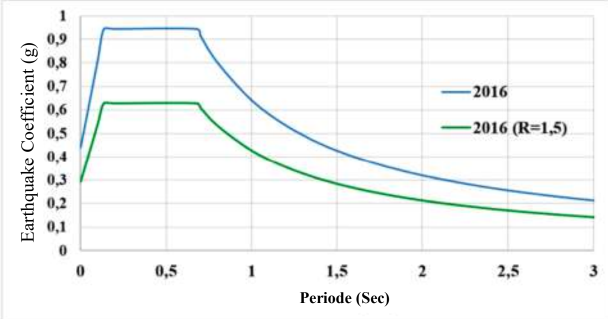

Figure 4. Response spectra of Manado with medium ground

earthquake load. The combination of loading used in accordance with SNI 1725: 2016 is shown in **Table 1**.

The earthquake response spectra was calculated using the SNI 2833:2016. Response spectra is a value that describes the maximum response of a single degree of freedom (SDOF) system at various natural frequencies (natural periods) damped due to an earthquake. In the response spectra analysis, graphs are used between the structural period (T) with elastic earthquake coefficient (Csm). The response spectra prevailing in Manado, North Sulawesi, with medium ground is shown in **Figure 4**. The response modification factor (R) was taken in accordance with the shape of the pylon and the bridge pillar which is 1.5 for the longitudinal direction and transversal direction.

The level of bridge performance would be determined through Nonlinear Static Pushover Analysis (NSPA) and Nonlinear Time History Analysis (NLTHA). In Nonlinear Static Pushover Analysis, there are several methods that can be used. In this study, the Capacity Spectrum Method was used. The Capacity Spectrum Method can represent inelastic spectra. The spectra demand used is simple and accurate enough for the various design assumptions used (Fajfar, 1999). The most important part of pushover analysis is the load distribution pattern used. Reference to the load distribution that is most suitable for cable-stayed bridges has not been found. Therefore, three load distribution patterns in this pushover analysis were used, namely the uniform distribution, static equivalent distribution, and modal analysis distribution.

The uniform distribution load uses the ratio of the mass representing each point reviewed to the total mass of the whole point. The lateral load that occurred at each point was taken with the following formulation:

$$F_i = \frac{m_i}{\sum m_t} \qquad (1)$$

Where:

F_i = lateral load at point i

m_i = mass at point i

m_t = total mass

Static equivalent distribution is a load distribution pattern used in FEMA 356. This load pattern considers the structure's own weight at the point reviewed, the

Table 1. Loading combination based on SNI 1725:2016

Loading Combination	Dead Load	Additional Dead Load	Prestress	Traffic Load	Brake Force	Structure Wind	Vehicle Wind	Uniform Temperature	Gradient Temperature	X-direction Earthquake	Y-direction Earthquake
Strong Ia	1,3	2	1	1,8	1,8	-	-	1,2	-	-	-
Strong Ib	1,3	2	1	1,8	1,8	-	-	1,2	-	-	-
Strong Ic	1,3	2	1	1,8	1,8	-	-	1,2	-	-	-
Strong II	1,3	2	1	1,4	1,4	-	-	1,2	-	-	-
Strong III	1,3	2	1	-	-	1,4	-	1,2	-	-	-
Strong IV	1,3	2	1	-	-	-	-	1,2	-	-	-
Strong V	1,3	2	1	-	-	0,4	1	1,2	-	-	-
Extreme Ixa	1,3	2	1	0,5	0,5	-	-	-	-	1	0,3
Extreme Ixb	1,3	2	1	0,5	0,5	-	-	-	-	-1	0,3
Extreme Ixc	1,3	2	1	0,5	0,5	-	-	-	-	1	-0.3
Extreme Ixd	1,3	2	1	0,5	0,5	-	-	-	-	-1	-0.3
Extreme Iya	1,3	2	1	0,5	0,5	-	-	-	-	1	0,3
Extreme Iyb	1,3	2	1	0,5	0,5	-	-	-	-	-1	0,3
Extreme Iyc	1,3	2	1	0,5	0,5	-	-	-	-	1	-0.3
Extreme Iyd	1,3	2	1	0,5	0,5	-	-	-	-	-1	-0.3
Serve Ia	1	1	1	1	1	0,3	1	1,2	0,5	-	-
Serve Ib	1	1	1	1	1	0,3	1	1,2	0,5	-	-
Serve Ic	1	1	1	1	1	0,3	1	1,2	0,5	-	-
Serve II	1	1	1	1,3	1,3	-	-	1,2	-	-	-
Serve III	1	1	1	0,8	0,8	-	-	1,2	0,5	-	-
Serve IV	1	1	1	-	-	0,7	-	1,2	-	-	-

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height of that point from the base of the structure, and the period factor of the structure. The lateral load of each point was calculated using the following formula:

$$F_x = C_{VX}V \quad (1)$$

$$C_{VX} = \frac{w_x h_x^k}{\sum_{i=1}^{n} w_i h_i^k} \quad (2)$$

Where:

F_x = lateral load at point x

V = total base shear

C_{VX} = distribution factor at point x

w_x = structure weight at point x

h_x = height of point x from the base of the structure

k = 2.0 for T ≥ 2.5 seconds

= 1.0 for T ≤ 0.5 seconds

Modal analysis distribution is also used in FEMA 356, calculating lateral forces using the shear force magnitude calculated from response spectra analysis that is analyzed if the mass participation rate is more than 90%.

Nonlinear Time History Analysis uses ground motion taken from the PEER Ground Motion Database. In accordance with SNI 2833: 2016, the number of ground motion used is at least three ground motions that are compatible with time history and they must be used for each earthquake component that represents the planned earthquake. Three orthogonal components (x, y, and z) of the planned earthquake must be included simultaneously when conducting Nonlinear Time History Analysis. The results of the analysis taken are the effect of maximum response from three input earthquakes in each main direction. If there are 7 acceleration records, planning is based on the average response. The most important stages in this analysis are the selection of ground motion and spectral matching. The selection of ground motion is based on three parameters, namely the earthquake magnitude, the distance of the location to the source of the earthquake fault, and soil type conditions. Spectral matching needs to be done for scaling so that the ground motion used is in accordance with the characteristics of the design earthquake.

The bridge performance was measured based on the ratio of roof displacement to parameters taken from Hose and Seible (1999) included in NCHRP Synthesis 440, see **Table 2**.

The nonlinear property of the element must be defined first before conducting a nonlinear analysis. The element's nonlinear property is in the form of a capacity curve and element acceptance criteria. The element capacity curve is described by moment-curvature, moment-rotation or moment-displacement and so are the acceptance criteria. In this study, the element capacity curve used the moment-curvature that occurred due to the actual force that occurred and was calculated by the XTRACT program. Meanwhile, the acceptance criteria used that moment-curvature which was scaled based on FEMA 356.

4. Analysis of Bridge Structure Performance Due to Standard Loads

The bridge structure performance against the standard load is assessed based on the deflection of the bridge floor, the stress on the girder, the stress on the cable, and the pylon capacity. Differences in bridge performance parameters during design and evaluation time are shown in **Table 3**.

Table 2. Performance level of the bridge structure

Level	Description	Steel Strain	Concrete Strain	% Drift	Displacement Ductility
I	Fully operational	<0.005	<0.0032	<1.0	<1.0
II	Operational	0.005	0.0032	1.0	1.0
III	Life safety	0.019	0.01	3.0	2.0
IV	Near collapse	0.048	0.027	5.0	6.0
V	Collapse	0.063	0.036	8.7	8.0

According to **Table 3**, the evaluation of allowable deflection is L/250 for service loads and L/800 for vehicle loads. From the analysis results, the deflection value for service load and vehicle load was still below the allowable deflection as shown in **Table 4**.

The allowable stress requirement on the bridge girders uses the SNI 2847: 2013 Structural concrete requirements for buildings. The allowable compressive stress is 0.45fc = 18.67 MPa and the allowable tensile stress is 0.62fc = 4.011 MPa. The results of the stress analysis on the girder can be seen in **Table 5**. The girder stress due to service load was still below the allowable stress except for the Ic Load where there was an overstress of 112% (4.49 MPa).

The maximum allowable cable stress was taken from the Cable-stayed Bridge Technical Planning Guidelines No. 08 of 2015. The allowable stress is 0.45fpu = 0.45x1860 = 837 MPa for the service load and 0.6fpu = 0.6x1860 = 1116 MPa for the ultimate load. From the results of the analysis, it was found that most of the stress that occurred was still below the allowable voltage, except for several combinations that exceeded the allowable stress on cable 7 (see **Table 6**).

Pylon was analyzed based on its capacity from the interaction diagram. The results of the analysis show that the pylon structure is still capable of carrying ultimate loads and service loads except for the lower pylon. In the lower pylon, the force caused by the combination due to earthquake load (Extreme I) cannot be borne by the pylon. To verify this, an additional analysis was conducted using the SpColumn program (see **Table 7**).

From the SpColumn results, there was a value of ΦMn/Mu smaller than one, which was the load combination of Extreme Ix and Extreme Iy, which was a

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Table 3. Bridge performance parameters

Parameters	Design Criteria		Evaluation Criteria	
Deflection	RSNI T-12-2004 Planning of Concrete Structures for Bridges	L/250 & L/800	08_SE_M_2015 Cable-stayed Bridge Technical Planning Guidelines	L/400 & L/800
Girder Stress	Planning of Concrete Structures for Bridges		SNI 2847:2013	
	Compressive	0,45fc'	Compressive	0,45fc'
	Tensile	0,5√fc'	Tensile	0,62√fc'
Pylon Capacity	SNI 2847:2002	Reduction 0,7 - 0,8	SNI 2847:2013	Reduction 0,7 - 0,9
Cable Force	RSNI T-12-2004 Planning of Concrete Structures for Bridges		Cable-stayed Bridge Technical Planning Guidelines (2015)	
	Service Load	0,45fpu	Service Load	0,45fpu
			Ultimate Load	0,6fpu

Tabel 4. Deflection due to standard load SNI 1725:2016

Loading Combination	Deflection Requirements		Deflection from Csi Bridge		
	Requirements	Value (mm)	Max (mm)	Min (mm)	Desc.
Service 1a	L/250	480,00	2,33	-32,97	OK
Service 1b	L/250	480,00	190,05	-151,64	OK
Service 1c	L/250	480,00	184,19	-150,14	OK
Service 2	L/250	480,00	-0,67	-66,41	OK
Service 3	L/250	480,00	7,58	-15,10	OK
Service 4	L/250	480,00	86,63	-5,65	OK
Vehicle Load	L/800	150,00	-1,94	-29,46	OK

Tabel 5. Stress on girder beam due to service SNI 1725:2016

Loading Combination	Girder Position	Top Fiber Stress		Bottom Fiber Stress	
		Max	Min	Max	Min
Service Ia 2016	Left Girder	-2,159	-11,303	-2,436	-13,988
	Right Girder	-2,160	-10,938	-2,796	-13,759
Service Ib 2016	Left Girder	-1,080	-11,999	3,573	-16,712
	Right Girder	-1,198	-11,628	3,414	-16,937
Service Ic 2016	Left Girder	-1,173	-11,973	4,491	-16,998
	Right Girder	-1,131	-11,626	4,156	-17,107
Service II 2016	Left Girder	-1,530	-11,013	-2,532	-15,688
	Right Girder	-1,449	-11,019	-2,608	-15,796
Service III 2016	Left Girder	-2,565	-11,122	-2,007	-12,930
	Right Girder	-2,482	-11,116	-2,203	-13,027
Service IV 2016	Left Girder	-3,462	-11,094	1,789	-13,585
	Right Girder	-3,439	-10,911	1,454	-13,672

combination of earthquake load. The lower pylon structure can be concluded as not being able to bear the load of an earthquake.

5. Performance Based Analysis

5.1 Nonlinear static pushover analysis

The pushover analysis is a static procedure in which a lateral load pattern is applied to a structure and the load is gradually increased until the displacement of the structure reaches its limit state. Prior to pushover the structure will be loaded by the bridge's fixed load in the form of dead load and additional dead load, plus a traffic load of 50%. **Figure 5** shows the capacity curves with three load distribution patterns used, which are the uniform distribution load, static equivalent distribution, and modal analysis distribution.

Using the Csi Bridge program, an analysis with the Capacity Spectrum Method was used to determine the bridge performance. The performance point was obtained by finding a meeting point between the capacity curve and the demand curve in the form of ADRS. The results of the analysis in determining the performance point of the bridge structure can be seen in **Figure 6**.

Bridge element performance was also required, which was obtained from the moment-curvature value that occurred on the bridge element. In this study, the element performance evaluation was focused on the structure of the lower segment pylon because it was the most critical element. **Figure 7** shows the performance point achieved by the lower segment pylon.

From the two results of performance analysis, it can be concluded that from the pushover analysis with the three load patterns used, the structure and elements of the bridge were still safe. The performance level of the

Table 6. Overstress on cable no 7

Cable No.	Load Combination	LNS		SNS		LSS		SSS	
		Cable Stress	%fpu	Cable Stress	%fpu	Cable Stress	%fpu	Cable Stress	%fpu
S7 (2016 Load)	Strength Ia	1162,029	104,1%	1145,796	102,7%	1184,858	106,2%	1150,530	103,1%
	Strength Ib	1003,511	89,9%	987,189	88,5%	1156,338	103,6%	1122,031	100,5%
	Strength Ic	1137,074	101,9%	1121,003	100,4%	1030,421	92,3%	996,086	89,3%
	Strength II	1111,612	99,6%	1095,409	98,2%	1134,584	101,7%	1100,253	98,6%
	Extreme Ixa Max	1105,227	99,0%	1088,305	97,5%	1128,816	101,1%	1094,970	98,1%
	Service II	841,298	100,5%	828,878	99,0%	859,191	102,7%	832,785	99,5%

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Table 7. SpColumn analysis results for lower segment pylon

No.	Loading Combination	Pu	Mux	Muy	ΦMnx	ΦMny	ΦMn/
		kN	kNm	kNm	kNm	kNm	Mu
1	Strength Ia	85.529,30	35,80	11.261,70	867,38	272.853,47	24,23
4	Strength Ia	82.568,50	- 7,00	10.470,00	- 180,37	269.790,53	25,77
5	Strength Ia	81.008,30	- 228,90	93.543,50	- 655,98	268.067,97	2,87
6	Strength Ia	72.205,60	- 157,10	7.699,00	- 5.259,54	257.752,00	33,48
7	Strength Ia	74.824,70	- 45,60	37.034,40	- 321,77	261.285,03	7,06
8	Extreme Ix Max	67.751,30	345.232,41	94.920,70	220.467,38	60.616,91	0,64
9	Extreme Ix Min	82.650,00	-345.363,91	- 73.350,10	-238.549,27	- 50.664,28	0,69
10	Extreme Iy Max	50.937,50	125.230,80	286.493,69	91.097,49	208.406,08	0,73
11	Extreme Iy Min	99.464,00	-125.362,00	-264.923,00	-113.853,10	-240.601,73	0,91
12	Service Ia	62.700,00	173,80	28.471,20	1.507,86	247.012,94	8,68
13	Service Ib	59.716,20	26.112,80	27.791,20	157.979,41	168.133,56	6,05
14	Service Ic	59.780,30	- 25.699,60	27.620,60	-157.237,45	168.990,64	6,12
15	Service II	62.882,50	73,40	6.008,50	3.018,04	247.057,55	41,12
16	Service III	59.229,20	35,70	4.871,70	1.778,98	242.764,78	49,83
17	Service IV	57.661,10	- 101,80	46.357,70	- 529,16	240.965,97	5,20

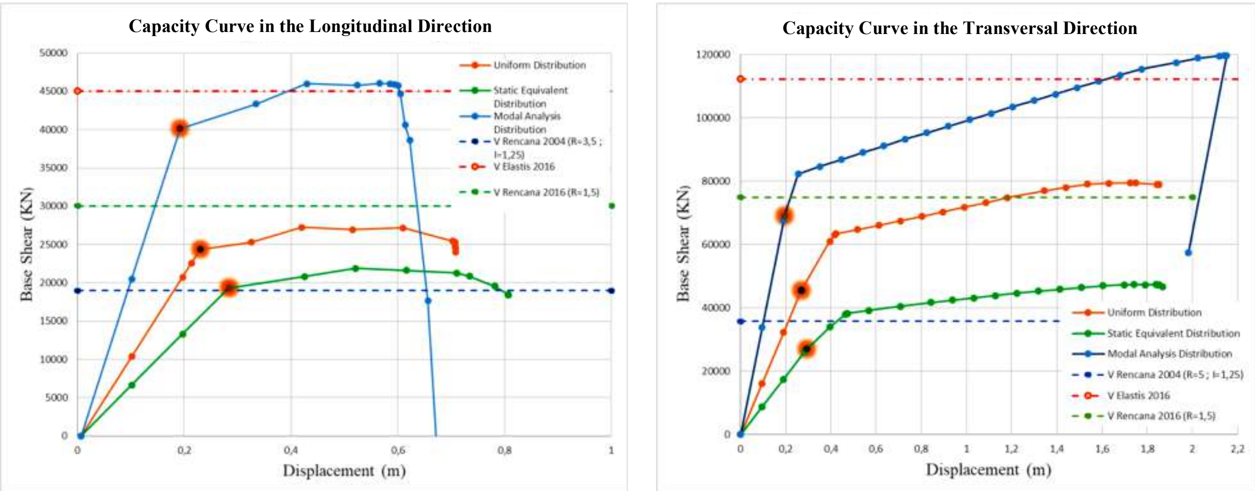

Figure 5. Capacity curves

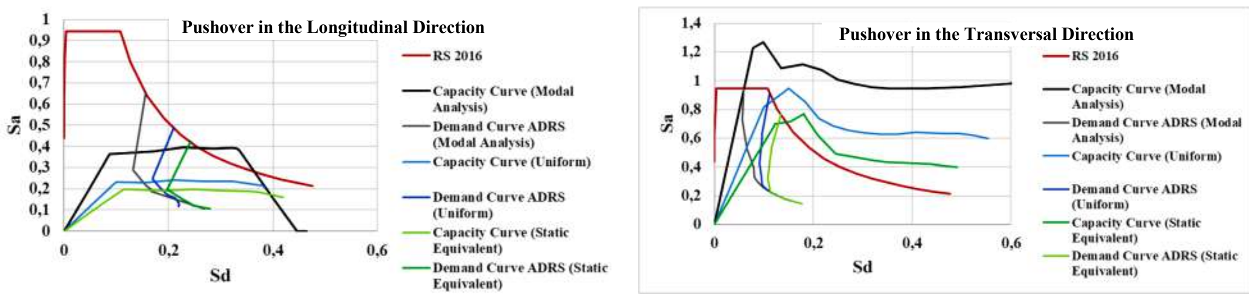

Figure 6. Pushover curves

bridge was at the fully operational level and the performance level of the bridge element was at the immediate occupancy level. Analysis of the girder stress and cable stress shows that overstress occurred on the bottom side of the bridge girder of 148% to 400% of the allowable stress, while the cable stress was far from the cable yield stress.

5.2 Nonlinear time history analysis

The Nonlinear Time History Analysis used the ground motion taken from the PEER Ground Motion Database. The stages in this analysis were the selection of ground motion and spectral matching, structural analysis using the Csi Bridge program and determination of bridge performance level. The choice

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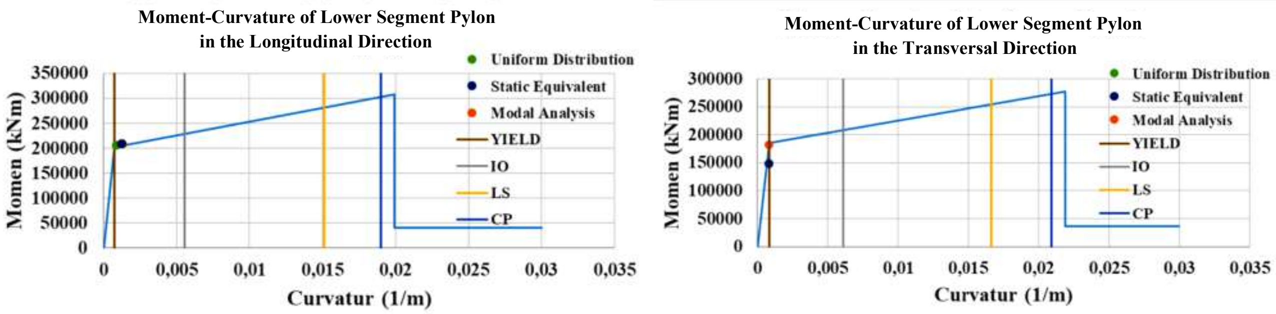

Figure 7. Lower segment pylon element performance due to NSPA

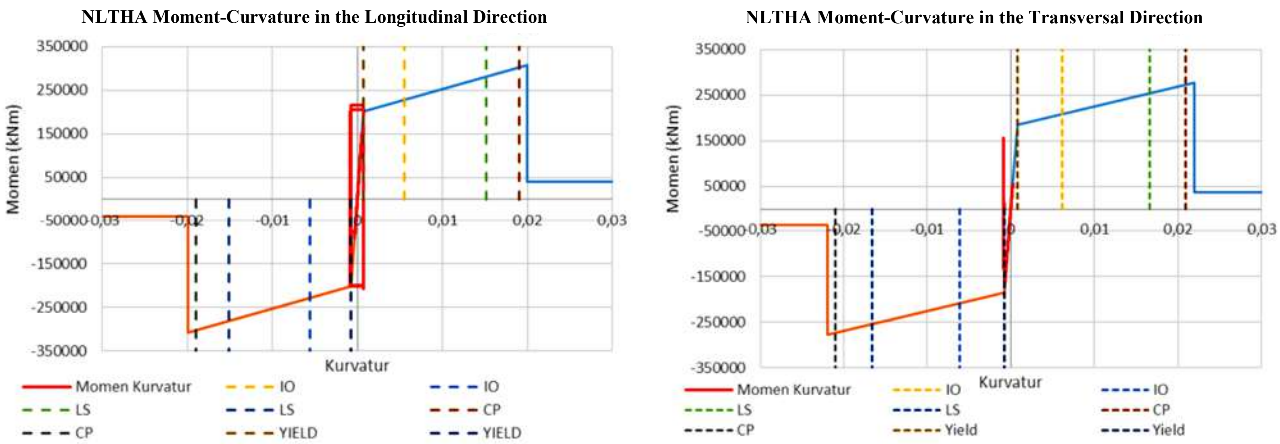

Figure 8. Lower segment pylon element performance due to NLTHA

of the earthquake was based on parameters from the Ground Motion Evaluation Procedure for Performance-Based Design, ie the magnitude of the earthquake is in the range of ± 0.25 target magnitude, the distance to the source of the earthquake fault must be appropriate, and the ground condition must also be appropriate. These parameters were searched by using the earthquake data that occurred around Manado. The data was obtained from BMKG Online. From the data, it is known that the maximum earthquake magnitude that occurred was 6.1 and the maximum closest distance to the earthquake fault was 9.15 km. The ground condition data was obtained from the picture of the plan where the ground condition was in the form of medium ground with shear wave velocity between 175 m/s to 350 m/s. Therefore, four ground motions were selected according to **Table 8**.

On those ground motions, spectral matching was then carried out in the range of 0.2T to 1.5T. The ground motion input was carried out in the x, y and z directions. The parameters used in the Csi Bridge program were as follows:

- Damping used was the proportional damping of 5% with a coefficient calculated by Csi Bridge.
- Time integration used was the Newark method with a value of Gamma = 0,5; Beta = 0,25.

Table 8. Selected ground motions

Record Sequence Number	Earthquake Name	Year	Magnitude	Distance to Fault	V30 (m/s)
232	Mammoth Lakes-01	1980	6,06	4,67	346,82
461	Morgan Hill	1984	6,19	3,48	281,61
4146	Parkfield-02_ CA	2004	6,00	9,14	341,70
8118	Christchurch_ New Zealand	2011	6,20	9,06	263,20

The results of the analysis can be seen in **Table 9**, then the envelope value was selected to determine the structure performance level. The direction of displacement and base shear were reviewed only in the x (longitudinal) and y (transversal) directions. The performance of the elements represented by the lower segment pylon elements is shown through the moment-curvature included in **Figure 8**.

It can be concluded that from Nonlinear Time History Analysis with four ground motions that were used, both the structure and elements of the bridge were still in a safe condition. The performance level of the bridge was at the fully operational level and the performance level of the bridge element was at the immediate occupancy level. Analysis of the girder stress and cable stress shows that overstress occurred on the bottom side of the bridge girder of 110% to 240% of the allowable stress, while the cable stress was far from the yield stress.

5.3 Discussion on the results of NSPA and NLTHA

Based on the results of the nonlinear static pushover analysis and nonlinear time history analysis, the bridge performance was at the fully operational level with the immediate occupancy element performance level. Despite having the same level of performance, the value of the base shear and displacement that occurred

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Table 9. Nonlinear time history analysis results

Record Sequence Number	Earthquake Name	Max/Min	Base Shear (kN)		Displacement (m)	
			X Direction	Y Direction	X Direction	Y Direction
232	Mammoth Lakes-01	Max	62.235,9	90.712,2	0,164	0,215
		Min	- 53.771,3	- 57.566,5	-0,140	-0,226
461	Morgan Hill	Max	49.531,0	84.214,8	0,153	0,195
		Min	- 55.238,7	- 65.649,9	-0,174	-0,230
4146	Parkfield-02_ CA	Max	38.617,3	88.021,3	0,164	0,203
		Min	- 58.741,1	- 71.304,0	-0,185	-0,208
8118	Christchurch_ New Zealand	Max	54.830,8	76.504,6	0,183	0,229
		Min	- 61.081,1	- 67.805,6	-0,130	-0,231

were different. These differences can be seen in **Table 10**.

From the results in **Table 10**, it can be concluded that the pattern of load distribution used would determine or influence the results of the analysis. Of the 3 load patterns used in nonlinear static pushover analysis, the value of the base shear approaching the results of nonlinear time history analysis was the modal analysis distribution, while the displacement value obtained was still quite large.

Table 10. Base shear and displacement values

Lateral Load Patterns	Direction	Base Shear (kN)	Displacement (m)	Desc.
Desain	Longitudinal	36537,57	-	
	Transversal	91022,90	-	
NSPA Uniform Distribution	Longitudinal	25956,78	0,357	
	Transversal	48686,14	0,295	
NSPA Static Equivalent Distribution	Longitudinal	20935,69	0,437	
	Transversal	28656,65	0,319	
NSPA Modal Analysis Distribution	Longitudinal	42212,05	0,283	
	Transversal	52405,67	0,148	
NLTHA	Longitudinal	62235,91	0,185	Max
		38617,34	0,130	Min
	Transversal	90712,24	0,301	Max
		57566,48	0,203	Min

6. Conclusions and Recommendations

- With a standard load, there were several bridge elements that exceeded the allowable stress. The girder has an overstress of 12% and cable 7 has an overstress of 6.2%. The capacity of the lower segment pylon has also been exceeded by 128%.
- The results of the NSPA and NLTHA analysis show that the bridge category was still in the safe category, which was fully operational with the immediate occupancy element performance. Cable stress when the performance point was reached was still far from the yield stress, while the bridge girder has overstressed at the bottom by 110% to 400% of the allowable stress.
- From point a and point b above, it is recommended that any special bridges built before SNI 1725: 2016 and SNI 2833: 2016 apply to be re-analyzed using these regulations. This is to anticipate the possibility of the bridges needing additional strengthening as the impact of the increased loads.
- On special bridges, especially cable-stayed bridges, the results of nonlinear time history analysis are still better than nonlinear static pushover analysis. Further study is required to determine the ideal pushover load distribution for cable-stayed bridges, considering that the load distribution pattern is crucial in determining the result of pushover analysis.

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