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Reliability evaluation of steel truss bridge due to traffic load based on bridge weigh-in-motion measurement

Widi Nugraha[*1,2], Indra Djati Sidi[2a], Made Suarjana[2b] and Ediansjah Zulkifli[2c]

[1]Directorate General of Highways, Ministry of Public Works and Housing,
Jl AH Nasution No 264 Bandung, Republic of Indonesia
[2]Department of Civil Engineering, Institut Teknologi Bandung,
Jl Ganesha No 10 Bandung, Republic of Indonesia

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Abstract. Steel truss bridge is one of the most widely used bridge types in Indonesia. Out of all Indonesia's national roads, the number of steel truss bridges reaches 12% of the total 17,160 bridges. The application of steel truss bridges is relatively high considering this type of bridge provides advantages in the standardization of design and fabrication of structural elements for typical bridge spans, as well as ease of mobilization. Directorate of Road and Bridge Engineering, Ministry of Works and Housing, has issued a standard design for steel truss bridges commonly used in Indonesia, which is designed against the design load in SNI 1725-2016 Bridge Loading Standards. Along with the development of actual traffic load measurement technology using Bridge Weigh-in-Motion (B-WIM), traffic loading data can be utilized to evaluate the reliability of standard bridges, such as standard steel truss bridges which are commonly used in Indonesia. The result of the B-WIM measurement on the Central Java Pantura National Road, Batang – Kendal undertaken in 2018, which supports the heaviest load and traffic conditions on the national road, is used in this study. In this study, simulation of a sequences of traffic was carried out based on B-WIM data as a moving load on the Australian type Steel Truss Bridge (i.e., Rangka Baja Australia – RBA) structure model with 60 m class A span. The reliability evaluation was then carried out by calculating the reliability index or the probability of structural failure. Based on the analysis conducted in this study, it was found that the reliability index of the 60 m class A span for RBA bridge is 3.04 or the probability of structural failure is 1.18×10^{-3}, which describes the level of reliability of the RBA bridge structure due to the loads from B-WIM measurement in Indonesia. For this RBA Bridge 60 m span class A, it was found that the calibrated nominal live load that met the target reliability is increased by 13% than stated in the code, so the uniform distributed load will be 7.60 kN/m^2 and the axle line equivalent load will be 55.15 kN/m.

Keywords: B-WIM; bridge; moving load; reliability; steel truss

1. Introduction

Indonesia is an archipelagic country at the confluence of the continents of Asia and Australia and varying topographical conditions. To support economic growth and equity in all locations within

*Corresponding author, Ph.D. Student, E-mail: widi.nugraha@pu.go.id
[a]Ph.D.
[b]Ph.D.
[c]Ph.D.

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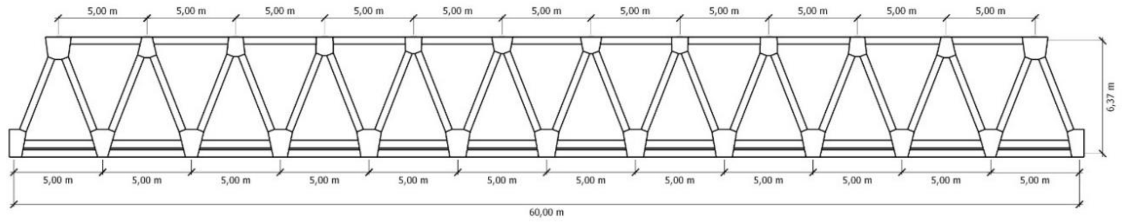

Fig. 1 Long section of RBA class A standard bridge with 60 m span (Kementerian 2005)

Indonesia, feasible infrastructure is required. Due to Indonesia's topography, which has many straits, rivers, valleys, mountains, the road infrastructure also requires bridges. To date, Indonesia has 17,160 bridges on national roads where one of the most widely used bridge types is steel truss bridge, which accounts for 12% of the total number of bridges (Kementerian 2022). The use of steel truss bridges is relatively high considering this type of bridge has the advantages in standardization of design and fabrication of structural elements for typical bridge spans, as well as ease of mobilization (Zhao *et al*. 2018). Steel truss bridges are generally provided in typical design and fabrication with reference to design standards (Tumbeva *et al*. 2021), in this case against the standard loads regulated in SNI 1725-2016 Bridge Loading Standards (Badan Standardisasi Nasional 2016). In terms bridge structure design that refers to SNI, the planning principle used is based on the Load and Resistance Factored Design (LRFD) approach, where the load and strength of the structure are recognized to be variable, hence providing ease of planning for a deterministic standard load value as well as the nominal resistance (Karthik *et al*. 2022). However, a load enlargement factor and a resistance reduction factor are required to estimate the possibility of failure (Nowak and Collins 2007). The reliability of a bridge structure is defined as the probability that failure is not anticipated to occur, hence reliability and probability of failure are interrelated parameters (Iatsko and Nowak 2021). Reliability of different type and span of bridge structures, especially standard bridges that widely used, can be used as an input for developing National LRFD Bridge Loading Standards, i.e., determining new bridge live load model and calibrating live load factors (Kwon *et al*. 2011).

Evaluation of reliability can be performed by considering the variability of magnitude and uncertainty of the parameters that affect the resistance and the load acting on the structure (Stawska *et al*. 2022). In this study, a reliability evaluation was carried out on the Australian type steel truss standard bridge (i.e., Rangka Baja Australia – RBA) structure model with 60 m class A span (Kementerian PUPR, 2005) with regards to the actual traffic load measured using Bridge Weigh-in-Motion (B-WIM). B-WIM technology utilizes an instrumented bridge structure to measure the load of the passing traffic without the need to stop the vehicle. B-WIM technology itself has been applied on several national roads and toll roads in Indonesia, one of which is the Pantura Central Java Batang-Semarang national road since 2016 (Nugraha and Sukmara 2016). The results of the B-WIM measurement on the Pantura National Road, Central Java, Batang – Kendal, which supports the heaviest load and traffic conditions on the national road, within the timeframe between January 1, 2018 and January 31, 2019, were used in this study. A sequence of the actual traffic vehicles from the B-WIM data were simulated as a moving vehicle load on the RBA bridge structure model with 60 m span class A using structural analysis software aid. For example, 60 m span can be fitted to the first seven vehicle that was measured using B-WIM, then these 60 m long series of vehicles inputted to the software as moving load series applied to the structure. In this study, daily maximum of 60 m vehicle loading was used for one-year measurement. Subsequently, this process provided

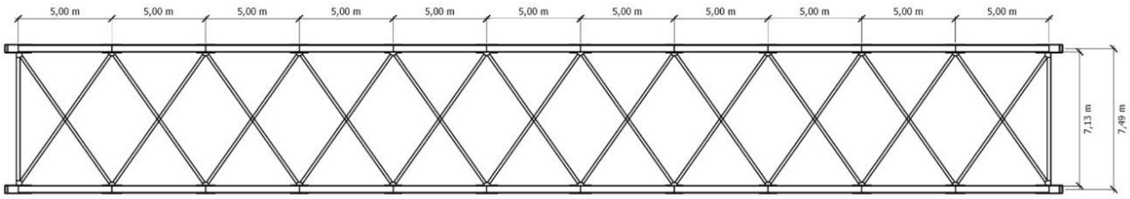

Fig. 2 Plan of wind top chord elements of RBA class A standard bridge with 60 m span (Kementerian 2005)

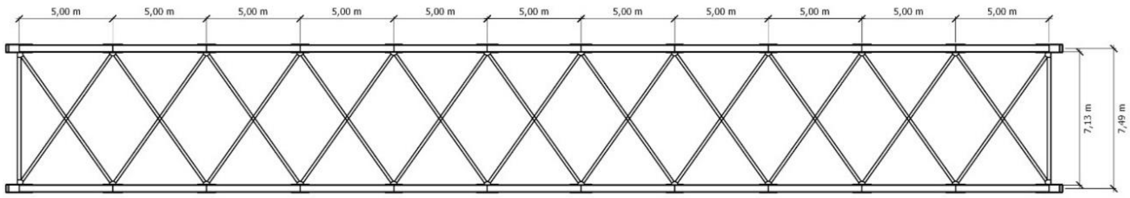

Fig. 3 Plan of bridge slab of RBA class A standard bridge with 60m span (Kementerian 2005)

the distribution of structural response data on the truss elements of the RBA bridge due to the sequences of traffic load simulation from B-WIM, which then used as a load effect variable to calculate the reliability index or the probability of failure of the RBA Standard Bridge structure. The results of the study are expected to provide an overview of the level of reliability of standard bridges in Indonesia due to the actual load. Then, the calibration of bridge nominal live load that met the target reliability of 3.72 will be carried out in this research by using the reliability index result.

2. Background

2.1 Bina Marga standard bridge type RBA

The bridge structure discussed in this study, as described previously, is the standard bridge of Bina Marga type RBA class A with 60 m span, which refers to the Guidelines document No: 07/BM/2005 Standard drawing of steel truss for class A and B bridges (Kementerian 2005). This type of bridge uses a truss element consisting of a bar with an H profile arranged and connected by bolt connections and gusset plates at the junction between the truss elements. The forces acting on the main elements of the truss bridge structure due to traffic loads are axial forces, especially for the top chord elements, which are compressive axial forces, and for the bottom chord elements, the tensile axial forces. Based on the design drawing shown in Figs. 1-3, the modelling of this type of bridge, can be performed with the aid of structural analysis software to analyze the simulation of a sequences of traffic due to the vehicle load as a result of the B-WIM measurement.

2.2 Vehicle load data from B-WIM measurements

The results of the B-WIM measurement on the Pantura National Road section of Central Java, Batang – Kendal, between the time range of January 1, 2018 to January 31, 2019, were used in this study. The location was selected to measure the B-WIM load considering the section is located on

Fig. 4 The results of measuring the load of a 3-axle truck vehicle with B-WIM

the main road on the Java Island with heavy vehicle traffic and relatively high intensity of vehicle passing, as such it is anticipated to provide a load effect in the form of a high structural response and represent the ultimate condition in traffic loading on the RBA standard bridge structure to evaluate the reliability level. In general, the data obtained from the results of measuring vehicle loads with B-WIM include the time the vehicle passes (time stamp), vehicle speed, Gross Vehicle Weight (GVW), the weight of each vehicle axle (axle load), distance between vehicle axles, and vehicle axle configuration or vehicle classification (Macleod *et al*. 2022).

For example, Fig. 4 shows the measurement results for one of the vehicles that passed above B-WIM, namely a 3-axle truck with a total weight of 17.63 tonf and traveling at a speed of 18.48 km/hour. The information from the vehicle measurement data was then used to construct a vehicle sequences that fitted to 60 m bridge span, that describes the actual traffic loading experienced by the bridge, which then used to simulate the moving load on the bridge structure model in structural analysis software. The vehicle sequences used for the analysis is only the daily maximum for one-year measurement data.

2.3 Reliability evaluation

Reliability, in this case reliability of the structure, is also the inverse of the probability of structural failure based on the probability principle, which is the theoretical basis used in LRFD-based planning (van der Spuy and Lenner 2019). To calculate the reliability index of the bridge structure, it is necessary to know the resistance variable (R), dead load (D), and live load (L), all with the same magnitude of internal forces, such as bending moment. The general equation used to calculate the reliability index (β) with the performance function R-D-L > 0 is as follows, provided that the distribution of the three variables is a normal distribution (Nowak and Collins 2012), otherwise Rosenblatt transformation was performed to transform the statistical parameters of each variable into equivalent normal parameters. (Rosenblatt 1952).

$$\beta = \frac{\mu_R - \mu_D - \mu_L}{\sqrt{(\sigma_R)^2 + (\sigma_D)^2 + (\sigma_L)^2}} \tag{1}$$

Where:

μ_R is the average of R variables
μ_D is the average of D variables
μ_L is the average of L variables
σ_R standard deviation of R variables
σ_D standard deviation of D variables
σ_L standard deviation of L variables

Standards or guidelines for planning and loading for bridges have a target reliability index value that must be achieved and binding factors used as part of design, such as load factors and reduction factors. AASHTO set a target value of β as 3.50 or probability of failure, pf = 2.32 x 10^{-4} (Kulicki 2017). While Nowak (2000) recommended to target 3.75 for bridges with a design life of 50 years, or probability of failure, pf = 10^{-4}. In the previous study, reliability evaluations of the Bina Marga class A standard bridge with a span length of 25 m for reinforced concrete girder type and composite girder type were carried out based on the 3 day WIM load data measurements on the Cikampek - Pamanukan national road in 2011 out and resulted in a reliability index of 5.01 (Nugraha and Sidi 2016) and 7.16 (Nugraha and Hardono 2015), subsequently. On the other hand, an evaluation of the prestressed box girder type bridge resulted in a reliability index of 4.30 (Pribadi and Sidi 2017). The improvement in this study, however is the quality and quantity of WIM load data itself, which is using one year B-WIM load data measurement with class A accuracy according to European WIM guidelines (Laboratoire Central des Ponts et Chaussées 2001) as such it is expected to achieve higher accuracy and the results are representative for the actual conditions of bridge loading due to traffic loads in Indonesia. Indonesia is developing new National LRFD Bridge Loading Standards, i.e., determining new bridge live load model and calibrating live load factors, and reliability evaluation of different type and span of bridge structures in Indonesia is the first step to take. In this research, calibration of bridge nominal live load that met the target reliability will be carried out by using the results of the reliability evaluation.

3. Research method

3.1 Analysis of vehicle load data

Daily vehicle load data from the B-WIM measurement contains data on vehicle speed, Gross Vehicle Weight (GVW), weight of each vehicle axle (axle load), distance between vehicle axles, and vehicle axle configuration or vehicle classification arranged sequentially according to the time when the vehicle passed (time stamp), hence the actual vehicle sequences can be determined based on the time the vehicle passed. The simulation of vehicle loading acted on the bridge is using moving load analysis, which need axle loading and axle distance information. A simplification was used in this study to make a moving load based on several vehicle that fitted into 60 m bridge span that gives maximum probable loading effect on the bridge structure element. The information of that several vehicles on 60 m length is defined as vehicle series/sequences, including axle loading and axle distance information, with distance between vehicle is calculated using speed of the vehicle and the timestamp between two vehicles. Subsequently, considering that the total weight of the vehicle sequences correlates positively with the generated internal force response, only three vehicle sequences with the largest total vehicle weight were analyzed for the daily data. These three daily

Table 1 Data from the 297th sequences from B-WIM measurement on 28 July 2018

No	Vehicle Sub Class	Axle Load						GVW (tonf)	Distance between Vehicle Axles					Total Length (m)
		W1 (tonf)	W2 (tonf)	W3 (tonf)	W4 (tonf)	W5 (tonf)	W6 (tonf)		A1 (m)	A2 (m)	A3 (m)	A4 (m)	A5 (m)	
2215	51	3.81	3.22	3.22				10.25	5.76	1.32				7.08
2216	51	12.39	21.22	21.22				54.82	4.28	1.54				5.82
2217	51	9.47	21.50	21.50				52.46	3.98	1.43				5.40
2218	51	11.57	21.00	21.00				53.57	5.50	1.52				7.02
2219	51	11.79	21.64	21.64				55.07	5.47	1.54				7.01
2220	51	13.12	18.62	18.62				50.36	3.96	1.44				5.41
2221	30	2.84	2.19					5.03	3.28					3.28
2222	30	4.75	13.01					17.77	3.33					3.33
2223	30	3.81	9.49					13.30	3.32					3.32

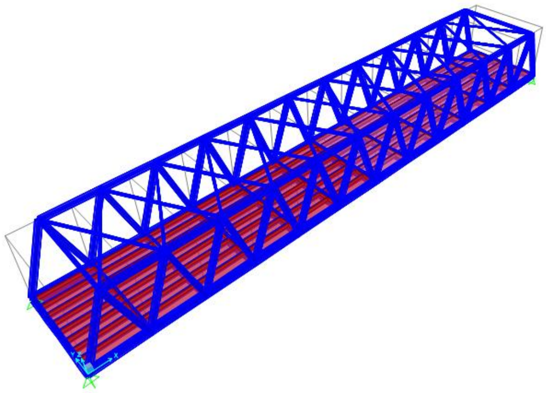

Fig. 5 Structural model

maximum probable vehicle sequences loading is adopted in the next step, namely the analysis of the bridge model structure by simulating the vehicle sequences as a moving load acting on the bridge structure.

3.2 Structural modelling and analysis

A three-dimensional model of the standard bridge structure of Bina Marga class A type RBA with a span length of 60 m with roller-joint supports, 9 m wide, modelled using the CSI Bridge software, is presented in Fig. 5. In order to evaluate the bridge loading code used to designing standard bridge, the bridges structure used in this study is assumed in ideal condition and simple support of joints and roller. Based on the specifications contained in the Guidelines document No: 07/BM/2005, the nominal value of the structural materials used for this bridge model are, among others, standard BJ55 steel truss profiles with yield stress f_y of 460 MPa and modulus of elasticity (E) of 200,000 MPa, concrete slab of the bridge with a compressive strength f_c' of 30 MPa (Kementerian 2005).

Analysis of the structural model was carried out after the vehicle sequences load was defined as a moving load. For example, one of the vehicle sequences measured by B-WIM with the largest total

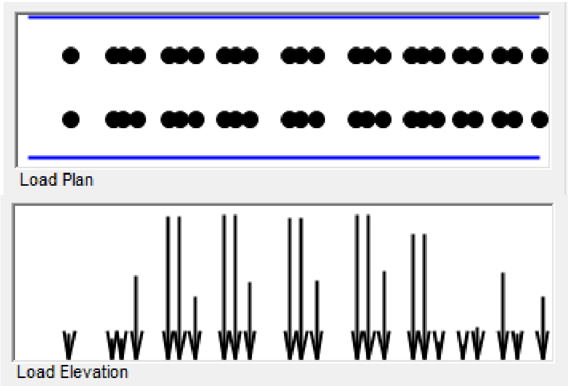

Fig. 6 Moving load simulation from the 297th sequences of BWIM data on 8 July 2018, plan (up) and section (down)

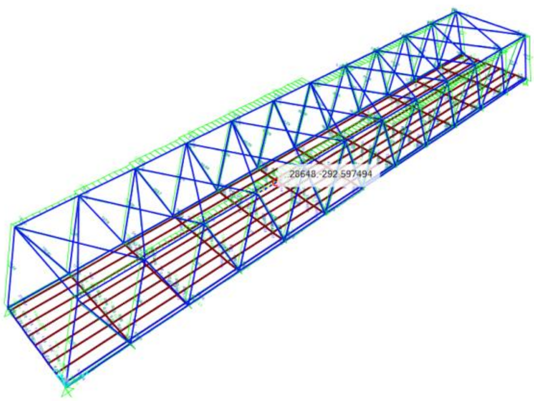

Fig. 7 Internal forces result for the structural analysis due to moving loads from the sequences of vehicles

weight for the 60 m length limit shown in Table 1, namely the 297th sequences on 28 July 2018, was used to simulate a moving vehicle load as defined in Fig. 6. The result of the analysis in the form of structural element forces is demonstrated in Fig. 7, with the top chord in the middle of the span experiencing the greatest internal forces, i.e., the axial compression force of 322.15 tonf and the bending force of 1.26 tonf-m. In this study, this procedure was carried out for three vehicle sequences with the largest total vehicle weight from each daily data in the measurement time range from January 1, 2018 to January 31, 2019. The aim was to obtain the maximum daily force due to the simulation of the vehicle sequences load as measured by B- WIM, which then projected into the maximum average value for the 75-year return period using the projection method from Gumbel probability paper.

3.3 Reliability evaluation

Evaluation of reliability was carried out by referring to Eq. (1). Variable L was obtained from the model structure analysis stage of the vehicle sequences simulation of B-WIM measurement results.

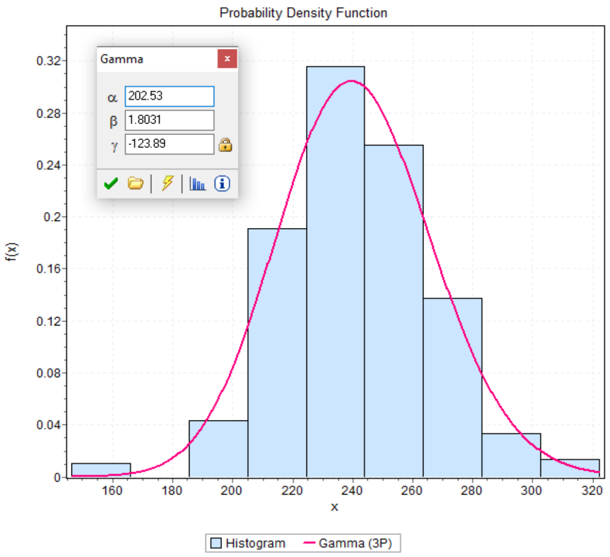

Fig. 8 Force distribution from the structural analysis result based on the moving load from the simulation of vehicle sequences

Table 2 Projection of the average maximum axial force for various return period

Statistics	Value	Percentile	Value
No. of sample	298	Min	146.35
Range	175.8	5%	204.35
Average	241.28	10%	211.74
Variance	646.36	25% (Q1)	224.73
Deviation Standard	25.424	50% (Median)	240.95
Coefficient of Variation	0.10537	75% (Q3)	256.52
Std. Error	1.4728	90%	273.54
Skewness	-0.0449	95%	282.68
Excess Kurtosis	0.92197	Max	322.15

Variable D was obtained from the response of the structural analysis to its own weight and additional dead loads. Variable R was obtained from the cross-sectional capacity of the top chord element in the middle of the span. Distribution fitting was carried out for each variable. If there are variables that do not have the normal distribution type, then the Rosenblatt transformation was carried out to transform the statistical parameters of each variable into equivalent normal parameters (Rosenblatt 1952) and the calculation undertaken based on Eq. (1) was performed iteratively to achieve a convergent reliability index value.

For the purposes of this study, the L variable was projected to be the maximum mean value for the 75-year return period using the projection method from Gumbel probability paper as shown in Fig. 9 below. Based on the calculation results, it was found that the maximum average value for the 75-year return period for internal forces due to vehicle load sequences as a result of the B-WIM measurement is 315.40 tonf with a coefficient of variance of 8.06% as shown in Table 3.

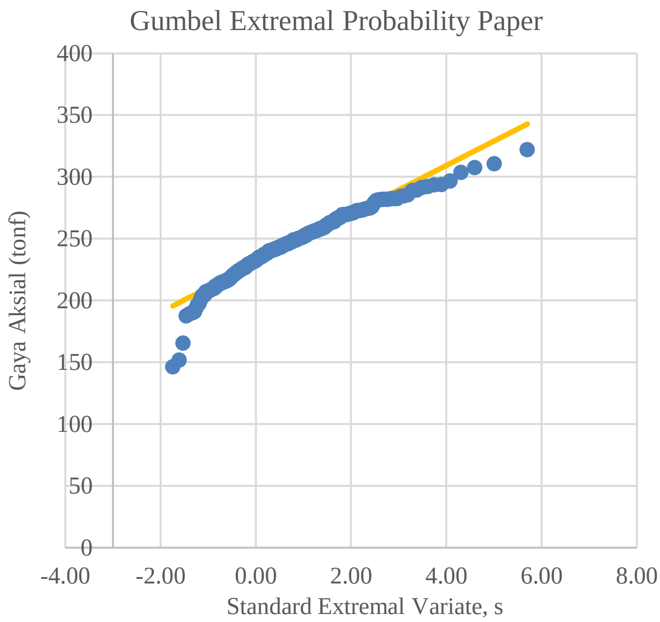

Fig. 9 Projection of internal force (axial) with Gumbel Probability Paper

Table 1 Projection of the average maximum axial force for various return period

Return period (year)	$\overline{P_{LL}}$ (tonf)	Percentile	Ωi
2	237.26	45.08%	10.72%
5	259.69	77.08%	9.79%
25	293.30	97.44%	8.67%
50	307.21	99.23%	8.28%
75	315.30	99.65%	8.06%

4. Results and discussion

4.1 Variable L

Upon completion of the structural analysis procedure for three vehicle sequences with the largest total vehicle weight from each daily data in the measurement time span 1 January 2018 to 31 January 2019, the maximum daily force was obtained from the simulation of the vehicle sequences load from the B-WIM measurement. The distribution of the force data in daily maximums was presented in a histogram, which was used to define the type of distribution that is most suitable for the distribution of the data (i.e., distribution fitting) as shown in Fig. 8, with statistical parameters summarized in Table 2. The distribution obtained was the Gamma distribution, hence it was necessary to carry out the Rosenblatt transformation for further calculations.

4.2 Variable D

The dead load variable or variable D was determined from the structural analysis, which assumed to have a normal distribution (Ellingwood and Galambos 1982). The D variable data was obtained by entering the nominal concrete density of 2400 kg/m^3, the nominal steel density of 7850 kg/m^3, as well as the super imposed dead load (SIDL) in the form of a 50 mm thick asphalt layer with a nominal asphalt density of 2400 kg/m^3 and the weight of a parapet or fence made of two steel pipes with a diameter of 75 mm along the bridge on both sides. Based on the structural analysis, it was found that the nominal axial force due to dead weight and SIDL was 431.93 tonf. These nominal data need to be converted into an average value for use in the reliability calculation. Based on research on statistical properties of loading components in the 1982 LRFD (Ellingwood and Galambos 1982), the ratio of the average value to the nominal value of the variable D is $\frac{\bar{D}}{D_n} = 1.05$. While the appropriate c.o.v. values for variable D is $\Omega_D = 0.10$ (Ellingwood and Galambos 1982), therefore, the average value and standard deviation of the bending moment due to dead load are $\bar{D} = 1.05\ \ D_n = 453.53\ \ tonf$ and $\sigma_D = \Omega_D \bar{D} = 45.35\ \ tonf$.

4.3 Variable R

The nominal axial resistance, Ag fy, of Variable R has a lognormal distribution (Naus *et al*. 1994). The average axial resistance was determined using Eq. (2) below.

$$P = A_g f_y \tag{2}$$

With

A_g is the steel cross sectional area

f_y is the yield stress of the steel profile

The calculation of the average axial force capacity for the cross section of the steel truss profile of the bridge was carried out based on the equation of the steel axial capacity with the average value of each variable, as such the average resistance was obtained: $\bar{R} = 1377.37\ \ tonf$. Meanwhile, to determine the c.o.v. for this axial force capacity, calculations were carried out based on Eq. (3) as follows.

$$\Omega_R = \sqrt{\left(\Omega_{Ag}\right)^2 + \left(\Omega_{f_y}\right)^2} \tag{3}$$

This resulted in Ω_R = 0.10 hence the standard deviation for variable R is σ_R = 134.09 tonf.

4.4 Reliability index

Variable L, or live load, based on distribution fitting has a Gamma distribution (see Fig. 8), while variable R has a lognormal distribution (Naus *et al.* 1994), and variable D was assumed to have a normal distribution (Ellingwood and Galambos 1982). Based on the general Eq. (1), the following equation was used to calculate the reliability index for the standard bridge of Bina Marga, type RBA, span of 60 m, type A, based on the vehicle load sequences as measured by B-WIM.

$$\beta \cong \frac{\bar{P}_{Resistance} - \bar{P}_{Load\ (total)}}{\sqrt{\sigma^2 P_{Resistance} + \sigma^2 P_{Load(total)}}} \tag{4}$$

Table 2 Iteration process to calculate reliability index β

Iteration no.	Failure point		F_L	f_L	σ_i^N		μ_i^N		β
	$l*$	$r*$			σ_L^N	σ_R^N	μ_L^N	μ_R^N	
1	354.73	1437.50	0.570	0.0061	64.44	170.69	343.31	1427.37	3.354
2	417.39	907.61	0.879	0.0022	103.68	107.77	403.51	1318.57	2.953
3	606.67	1099.07	0.956	0.0005	150.69	130.50	318.79	1386.36	3.003
4	652.41	1136.15	0.968	0.0003	162.06	134.91	286.87	1395.43	3.037
5	656.67	1139.16	0.970	0.0002	163.11	135.26	283.70	1396.11	3.041
6	657.01	1139.40	0.971	0.0002	163.20	135.29	283.44	1396.17	3.041
7	657.04	1139.42	0.971	0.0002	163.21	135.29	283.42	1396.17	3.041
8	657.04	1139.42	0.971	0.0002	163.21	135.29	283.42	1396.17	3.041
9	657.04	1139.42	0.971	0.0002	163.21	135.29	283.42	1396.17	3.041
10	657.04	1139.42	0.971	0.0002	163.21	135.29	283.42	1396.17	3.041

$$P_{Load\,(total)} = P_{DL} + P_{SIDL} + P_{LL} \tag{5}$$

To use the linear performance equation in Eq. (4), all variables need to be calculated in normal distribution. Therefore, the distribution of L and R variables were transformed into normal distribution equivalent using the Rosenblatt transformation (Rosenblatt 1952). The extreme parameter of variable R is $\xi_R = \sqrt{\ln(1+\Omega_R{}^2)} = 0.12$; $\lambda_R = \ln(R) - \frac{1}{2}\xi_R{}^2 = 7.26$; additionally, the average value and standard deviation of the equivalent normal variables are expressed in Eqs. (6) and (7) as follows.

$$\sigma_R^N = \frac{1}{f_R\ (r*)}\phi\left\{\phi^{-1}\ \left[\phi\left(\frac{\ln r-\lambda_R}{\xi_R}\right)\right]\right\} = \frac{1}{f_R\ (r*)}\phi\left(\frac{\ln r-\lambda_R}{\xi_R}\right) = r^*\ \ \xi_R \tag{6}$$

$$\mu_R^N = r^* - \sigma_R^N\phi^{-1}\left[\phi\left(\frac{\ln r-\lambda_R}{\xi_R}\right)\right] = r^* - r^*\ \ \xi_R\left(\frac{\ln r-\lambda_R}{\xi_R}\right) = r^*(1-ln(r^*)+\lambda_R) \tag{7}$$

with $F_R(r) = \phi\left(\frac{\ln r-\lambda_R}{\xi_R}\right)$ and$f_R(r) = \frac{1}{r\ \xi_R}\phi\left(\frac{\ln r-\lambda_R}{\xi_R}\right)$.

Concurrently, the extremal parameter from variable L with Type I asymptotic behavior is as follows: $\alpha = \frac{\pi}{\sqrt{6}}\frac{1}{\sigma_L} = \frac{\pi}{\sqrt{6}}\frac{1}{0.19*354,73} = 0.0019$; $u = \bar{L} - \frac{0,577}{\alpha} = 324.41$; $F_L(l) = \exp\left(-\mathrm{e}^{-\alpha(l-u)}\right)$; $f_L(l) = \alpha\ \ exp\left(-\alpha(l-u) - \mathrm{e}^{-\alpha(l-u)}\right)$.

There were variables with non-normal distribution present as part of determining the reliability index of this bridge structure. Although the function used was linear performance, the required average and standard deviation were not known because the used function was a function of each failure point value. Therefore, the solution in the form of a reliability index value was calculated iteratively as follows (in units of bending moment, namely kNm). It was assumed that for the first iteration, the point of failure is equal to the average value of L (multiplied by the mean FBD) and R, i.e., $l^* = \bar{L} = 354.73$ tonf and $r^* = \bar{R} = 1437.50$ tonf. Therefore, the standard deviation and average values of variable R with normal equivalent were calculated as $\sigma_R^N = r^*\ \ \xi_R = 170.69$ tonf, and $\mu_R^N = r^*(1-\ln(r^*)+\lambda_R) = 1427.37$ tonf.

While for parameter L, $F_L(l^*) = \exp\left(-e^{-\alpha(l*-u)}\right) = 0.570$; $f_L(l^*) = \alpha\ exp\left(-\alpha(l-u) - e^{-\alpha(l-u)}\right) = 0.0061$. Subsequently, the standard deviation and average values of variable L normal equivalent are expressed as $\sigma_L^N = \frac{\frac{1}{\sqrt{2\pi}} exp\left[-\frac{1}{2}\{\Phi^{-1}(0,57)\}^2\right]}{0,0061} = 64.44$ tonf and $\mu_L^N = l^* - \sigma_L^N.\Phi^{-1}(0,57) = 343.31$ tonf. Concurrently, for variable D which has a normal distribution, the standard deviation and average values of variable D are $\sigma_D^N = \sigma_D = 45.35$ tonf and $\mu_D^N = \mu_D = 453.53$ tonf.

The reliability index, β, was then calculated using Eq. (4) which results in $\beta = 3.35$. The failure point based on reliability index β becomes $l^* = \mu_L^N - \alpha_L^* \beta \sigma_L^N$ and $r^* = \mu_R^N - \alpha_R^* \beta \sigma_R^N$, with $\alpha_L^* = \frac{\sigma_L^N}{\sqrt{(\sigma_R^N)^2 + (\sigma_D)^2 + (\sigma_L^N)}^2} = -0.343$ and $\alpha_R^* = \frac{\sigma_R^N}{\sqrt{(\sigma_R^N)^2 + (\sigma_D)^2 + (\sigma_L^N)}^2} = 0.908$. Hence, the failure point becomes $l^* = \mu_L^N - \alpha_L^* \beta \sigma_L^N = 417.39$ tonf dan $r^* = \mu_R^N - \alpha_R^* \beta \sigma_R^N = 907.61$ tonf. This procedure of index reliability calculation was repeated from the initial stage when determining the failure point and iteration was carried out until a convergent reliability index β from the previous iteration process was achieved.

To simplify the iteration process, this study used a table with each row representing one iteration stage. From the calculation results, as shown in Table 4, the β value becomes convergent after the seventh iteration process, with value of 3.04 which is smaller compared to the reliability target. Even so, the design of the standard steel truss bridge type RBA with 60 m span class A due to the sequences of vehicle load measured by B-WIM for the 75-year return period is still in the reliable category with an index of 3.04 or a probability of structural failure of 1.18 x 10^{-3}.

The reliability index for this case can be used to calibrate the nominal live load of Bridge Design Loading Code to meet the target reliability of 3.72. Iteration procedure was done to get the reliability index number to meet 3.72 by changing variable R as the unknown variable. By using current LRFD load combinations and respective load factors for Dead Load (1.2) and Design Live Load (1.8), the calibrated nominal live load can be calculated. For this RBA Bridge 60 m span class A, it was found that the calibrated nominal live load that met the target reliability is increased by 13% than stated in the code. It means that the current code nominal live load for 60 m span bridge that consists of 6.75 kN/m^2 uniform distributed load and 49 kN/m axle line equivalent load, needs to be increased by 13% to meet the target reliability of 3.72, so the uniform distributed load will be 7.60 kN/m^2 and the axle line equivalent load will be 55.15 kN/m.

5. Conclusions

Based on the analysis carried out in this study, it was found that the largest internal forces occur in the top chord elements in the middle of the span, i.e., axial compressive force of 322.15 tonf and bending force of 1.26 tonf, which form the basis of the calculation for the reliability index of the RBA Bridge 60 m span class A. The reliability index of the 60 m span class A RBA bridge is 3.04 or the probability of structural failure is 1.18 x 10^{-3}, which indicates that the level of reliability of the standard RBA bridge structure due to the load from the B-WIM measurement in Indonesia is still relatively high. For this RBA Bridge 60 m span class A, it was found that the calibrated nominal live load that met the target reliability is increased by 13% than stated in the code, so the uniform distributed load will be 7.60 kN/m^2 and the axle line equivalent load will be 55.15 kN/m. It is anticipated that the results of this study can be an input for further research addressing reliability to determine standard loading and load factors, as well as resistance factors in a more comprehensive bridge design based on actual load data measured in Indonesia in the future.

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