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Analysis of Failure Probability and Reliability Index of MAOS Steel-Truss Railway Bridge

[1]Suharjanto, [1]Aris Priyadi and [2]Emil Adly
[1]Department of Civil Engineering, Faculty of Engineering, Janabadra University, Yogyakorna, Indonesia
[2]Department of Civil Engineering, Faculty of Engineering, Universitas Muhammadiyah Yogyakarta, Indonesia

Abstract: This study is intended to analyse the condition of Maos steel-truss railway bridge, that has been known as 'Bangunan Hikmah' (BH) 1549 railway bridge, in order to obtain clarity of its status based on logical basic rules inspection and new criteria in the 90's era. Technically, the goal of this study is to inspect feasibility or reliability of the railway bridge after >120 years old. This study was conducted by analysing the BH 1549 railway bridge that has been developed in year 1894 based on Axles Load Scheme (Skema Beban Gandar/SBG) loading specification that has been proposed in year 1988 and by utilising Load and Resistance Factor Design (LRFD) method that has been proposed in year 2005. By feeding the dynamic loading from SBG 1998, then the member force could be calculated. The calculation was carried out by using SAP2000 V14 Software. Besides that, Failure Index (FI) and Reliability Index (RI) were also could be determined. Result of this study shows that the rail bridge has RI value in a range of 2.0328-11.1249. According to Indonesian National Standard (Standar Nasional Indonesia/SNI) 03-1729-2002 for bridge element, minimum standard value for RI is 3 while minimum standard value for joint is 4.5. Hence, it could be concluded that the investigated rail bridge has several elements that are under standard but for the entire rail bridge, the RI value is still quite good.

Key words: Steel-truss, standard, joint, rail bridge, Indonesia

INTRODUCTION

Steel-Truss Railway bridge MAOS (Jembatan Kereta Api MAOS/JKA-M) was built at 1894 and it was an inheritance of Dutch East Indies government. The rail bridge has several renovations until now. Such railway bridge, that is known as Bangunan Hikmat rail bridge (BH, 1549) has total 6 stretches with total area is 290 m^2; 4 stretches with area 42.4 m^2 and 2 stretches with area 60 m^2.

Based on preliminary study, it shows that currently, such rail bridge has truss position shifting. It would affect to the structure of the railway bridge and further, it would affect to the reliability index of the railway bridge. Objective of this study is to analyse steel-truss railway bridge BH 1549 to obtain its feasibility and reliability status after >100 years old based on scientific rules and new criteria in 1990 era.

MATERIALS AND METHODS

Basic theory and related works

Allowable stress design: Alowable Stress Design (ASD) is a traditional method American Instute of Steel Construction (AISC). Specification. Core analysis in ASD method is the analysis of service load that is unit stresses representing elastic structure that must fulfil safety standard or strength standard for such structure.

Previous related works: SBG is a method that uses axles factor for fatigue analysis. It has been widely used to analyse failure and reliability index of a railway or a rail bridge. Several previous studies about the use of axles based analysis in transportation infrastructure analysis are reviewed in the following sections. Pokorny *et al.* (2016) have investigated the use of axles based analysis for predicting the safe operation of train. The analysis was conducted based on residual fatigue lifetime. Fatigue crack initiation is identified based on cracks, scratches or inhomogeneities. In such study, the residual fatigue lifetime is estimated based on number of load cycle or fatigue crack growth from the initial size of the crack up to the critical size. Such study was carried out for different types of trains which are used under different conditions. Besides fatigue lifetime estimation, the study is also shows the influence of the discretisation level of the

Corresponding Author: Suharjanto, Department of Civil Engineering, Faculty of Engineering, Janabadra University, Yogyakorna, Indonesia

continuous load spectrum and effect of magnification or diminution of load spectrum on the calculated residual fatigue lifetime of the railway axle.

Beretta *et al*. (2016) have studied about a railway axles analysis based on effect of load interaction in propagation lifetime. In such study, the fatigue crack growth rate is analysed based on amount of retardation or acceleration subject to the load sequence. For the load interaction analysis, an experimental analysis has been carried out by companion and full-scale specimens. Outcomes of the study show that strip-yield approach performs good in estimating the cracks.

Because railway axles are safety-critical components, then continuous condition monitoring of a railway axles was proposed by Rolek *et al*. (2016). The continuous monitoring is conducted by measuring the axle in-service bending vibration and diagnosing the presence of a fatigue crack. To test performance of the proposed continuous monitoring system, a finite element based full-scale measurement was conducted and result of the study shows the proposed method can be considered as an additional safety measure detecting cracked axles in an advanced stage of the damage process.

A procedure to determine railway axle risk of fatigue failure under service loading was proposed by Beretta and Regazzi (2016). The procedure is for a simple fatigue assessment compliant to modern structural recommendations. In the analysis, a series Monte-Carlo simulation have been utilised in order to determine the maximum allowable stress for a given axle. Result of the proposed procedure is safety factor based on damage calculation for a target reliability against fatigue. Other studies on railway fatigue analysis based on axles analysis have been conducted by previous researchers, Pokorny *et al*. (2015), Hassani-Gangaraj *et al*. (2015) and Regazzi *et al*. (2014) for detail explanations.

The use of LRFD for safety analysis has also being investigated by previous researchers. Cai *et al*. (2011) have proposed a procedure for a reliability-based LRFD. The procedure is used for subsea composite pressure vessel subjected to external hydrostatic pressure. In such study, a sensitivity analysis has also been carried out to investigate the impact of every variable to the output. To test performance of the proposed procedure, an experiment has been carried out and it confirms that analytical output of the procedure is not similar with the experimental output.

Ng and Sritharan (2014) have tried to improve performance of LRFD method by integrating LRFD with construction control and pile setup. In such study, the resistance factors were developed using a reliability theory for a locally calibrated static analysis method and two dynamic analysis methods namely the Wave Equation Analysis Program (WEAP) and the CAse Pile Wave Analysis Program (CAPWAP). The pile design efficiency was improved by minimizing the discrepancy between design and field pile resistances through a proposed probability-based construction control method. Based on several experiments, the proposed LRFD method performs better compared to several methods proposed by previous researchers.

Review of the previous studies above shows that railway axles analysis for reliability investigation of a railway has received major attentions from previous researchers. LRFD is also an established method that widely used by researchers to analyse reliability of a material based on load and resistance factor. Hence, this study also used derivation of axles analysis called as axles load scheme analysis and LRFD method in order to achieve study objective.

RESULTS AND DISCUSSION

In this study, analysis is started by collecting data regarding the current condition of the railway bridge BH 1549. Several data collecting method, such as visual observation, physical measurement, literatures survey and interview with experts of railway bridge have been conducted. LRFD would be used as the load analysis while log normal distribution concept would be used to analyse reliability index of the investigated railway bridge.

Physically, the BH 1549 railway bridge has 42.4 m of length, 4.8 m of width and 8 m of height. Weight estimation of the BH 1549 railway bridge is 1.5 ton/m'/master steel-truss. Free body sketch of the BH 1549 is shown in Fig. 1 and Table 1.

For modelling purpose, joints and members (member) of the BH 1549 steel-truss railway bridge would then be numbered, as shown in Fig. 2. Coordinate of every joint in 3 dimensional space is shown in Table 1 while member definition is shown in Table 2. Computation is conducted using SAP2000 V14 and result about members, influencing lines and ordinate is shown in Table 3. With refer to Fig. 3, p = 18 ton, q = 6 t/m', self-load (bm) = 1.5 t/m', k = 1.5, V = 120 km h^{-1}, L = 42.4 m, U = $8\times M/L^2$ t/m, D = 900 mm. Train Impact Factor (TIF) is shown in Eq. 1:

$$f_s = \frac{538\times k\times v}{(L+6)U\times D} + 0.25 \quad (1)$$

where: fs: TIF (live load factor due to impact between railway carriage).

Force because of dead load (dl) and live load (ll): Maximum load of the railway bridge BH 1459 is shown in Fig. 4. Reaction at joint B_1 and B_2 could be computed.

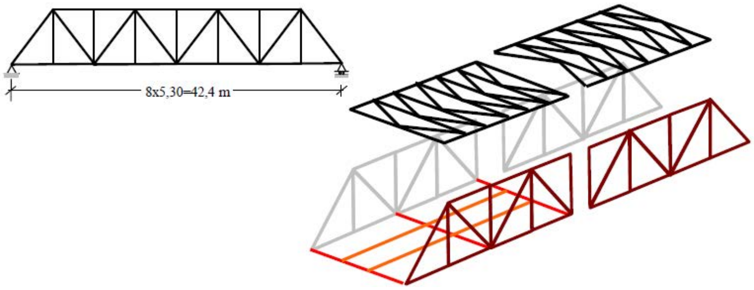


Fig. 1: Free body sketch of the BH 1549

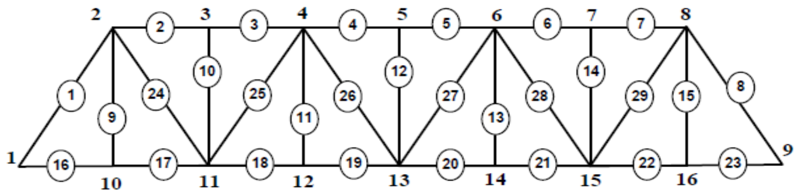


Fig. 2: Numbered joints and members of the BH 1549 steel-truss railway bridge model

Table 1: Coordinate of every joint

Joint	X	Y	Z
1	0.00	0	0.0
2	5.30	0	8.0
3	10.6	0	8.0
4	15.9	0	8.0
5	21.2	0	8.0
6	26.5	0	8.0
7	31.8	0	8.0
8	37.1	0	8.0
9	42.4	0	8.0
10	5.30	0	8.0
11	10.6	0	8.0
12	15.9	0	8.0
13	21.2	0	8.0
14	26.5	0	8.0
15	31.8	0	8.0
16	37.1	0	8.0

Table 2: Member definition with refer to its joint

	Joints			Joints			Joints	
Member	Left	Right	Member	Left	Right	Member	Left	Right
1	1(B_1)	2(A_1)	11	4(A_3)	12(B_4)	21	14(B_6)	15(B_7)
2	2(A_1)	3(A_2)	12	5(A_4)	13(B_5)	22	15(B_7)	16(B_8)
3	3(A_2)	4(A_3)	13	6(A_5)	14(B_6)	23	16(B_8)	9(B_9)
4	4(A_3)	5(A_4)	14	7(A_6)	15(B_7)	24	2(A_1)	11(B_3)
5	5(A_4)	6(A_5)	15	8(A_7)	16(B_8)	25	4(A_3)	11(B_3)
6	6(A_5)	7(A_6)	16	1(B_1)	10(B_2)	26	4(A_3)	13(B_5)
7	7(A_7)	8(A_8)	17	10(B_2)	11(B_3)	27	6(A_5)	13(B_5)
8	8(A_8)	9(B_9)	18	11(B_3)	12(B_4)	28	6(A_5)	15(B_7)
9	2(A_1)	10(B_2)	19	12(B_4)	13(B_5)	29	8(A_8)	15
10	3(A_2)	11(B_3)	20	13(B_5)	14(B_6)			

Notation: A_i: Upper joint of truss; B_i= Lower joint of truss

R_{B1} = 1/42.4 ×[8 × (42.4 +40.9+39.4 +33.4+31.9 +30.4 +27.4 +25.9 +24.4+18.4+16.9+15.4) + (1/2×6×13.9)] = 160.9 ton

R_{B9} = 138.5 ton

Midspan bending moment (under axle load of train) at point p, Q and R:

- $M_P = R_{B9}\times(25.9)-18\times(1.5+7.5+9+10.5)-13.9\times6\times(0.5\times13.9+12)$
- M_P = 1493.72 tm
- $M_Q = R_{B9}\times(24.4)-18\times(6+7.5+9)-13.9\times6\times(0.5\times13.9+10.5)$
- M_Q = 1519.07 tm
- $M_R = R_{B9}\,(18.4)-18\,(1.5+3)-13.9\times6\times(0.5\times13.9+4.5)$
- M_R = 1512.47 tm
- $M_{max} = M_Q$ = 1519.07 tm

Table 3: Members, influencing lines and ordinate

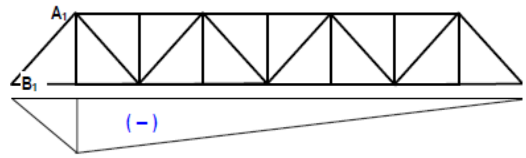
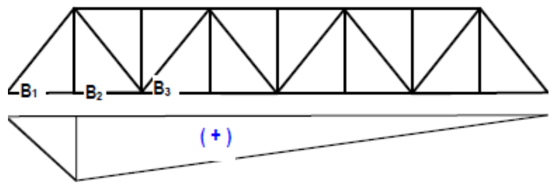
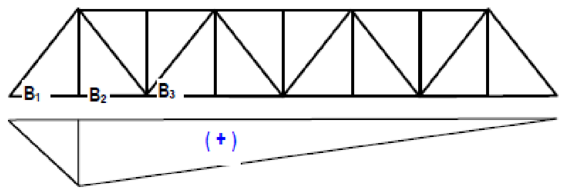
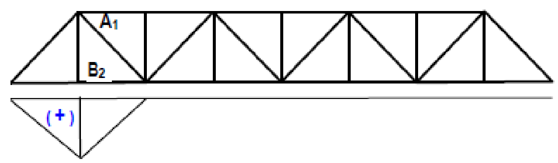
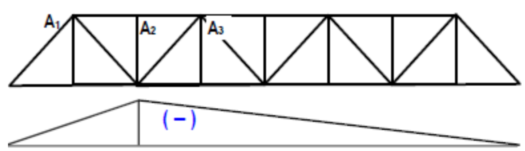
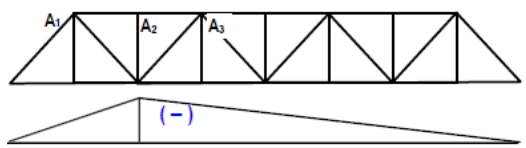
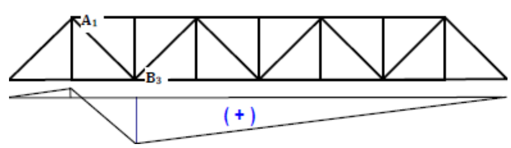
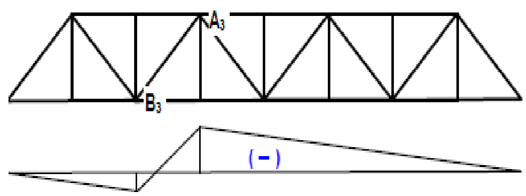
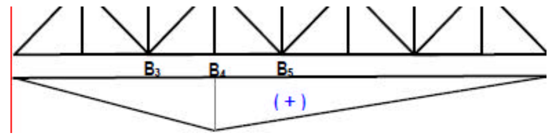
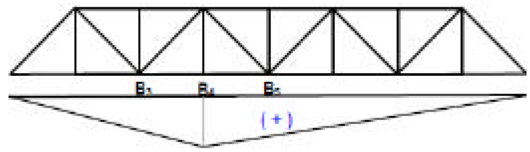
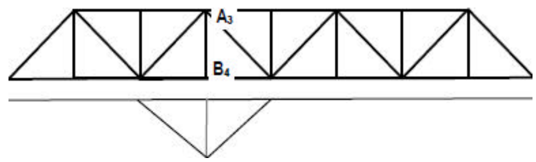
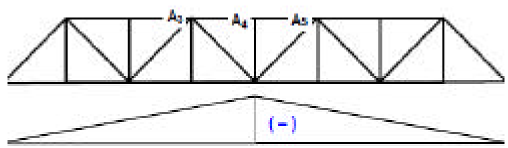
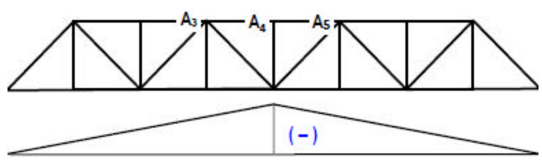
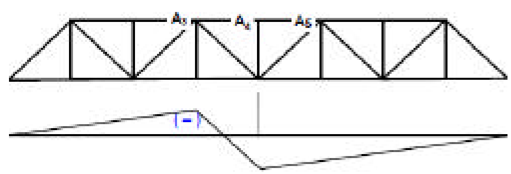
Members	Influencing lines	Ordinate
A_1B_1		-1.05
B_1B_2		+0.58
B_2B_3		+0.58
A_1B_2		+1.0
A_1A_2		-0.9937
A_2A_3		-0.9937
A_1B_3		-0.15; +0.90
A_3B_3		+0.3; -0.75

Table 3: Continue

B_3B_4		+1.2422
B_4B_5		+1.2422
A_3B_4		-1.0
A_3A_4		-1.325
A_4A_5		-1.325
A_3B_5		-0.45; +0.60

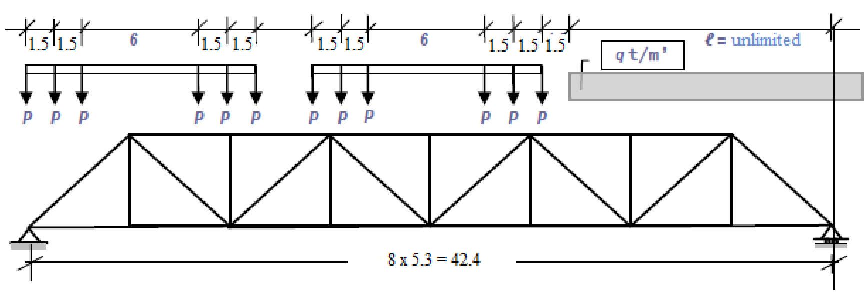


Fig. 3: Identification of members load

$$q_{equivalent} = U = \frac{8.M_Q}{L^2} = \frac{8\times 1519.07}{42.4^2} = 6.76\,t\,m^{-1}$$

$$TIF(f_s) = \frac{538\times 1.5\times 120}{(42.4+6)\times 6.67\times 900} = 0.329$$

Members analysis: Based on analysis on railway bridge BH 1459, influencing lines in every members is shown in Table 4.

Result of the members analysis on DL, LL, RIL and Bridgeway impact load (BIL) is shown in Table 5. Force

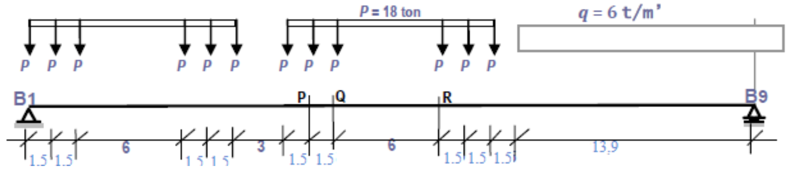


Fig. 4: Identification of members load is shown in

Table 4: In every members

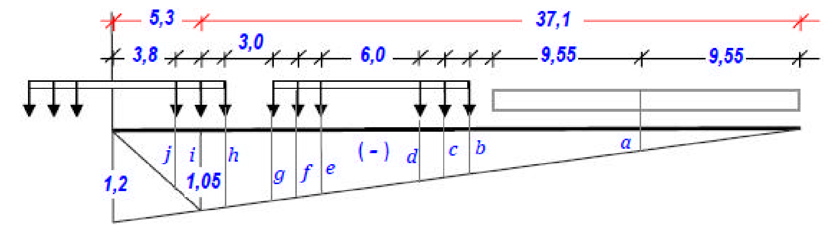
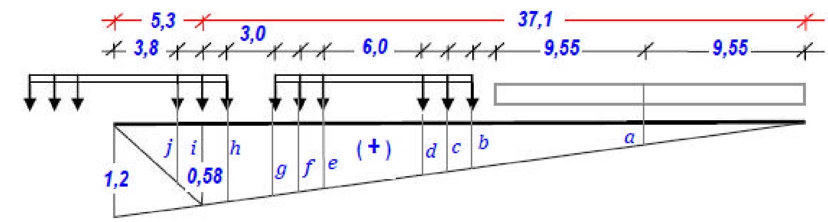
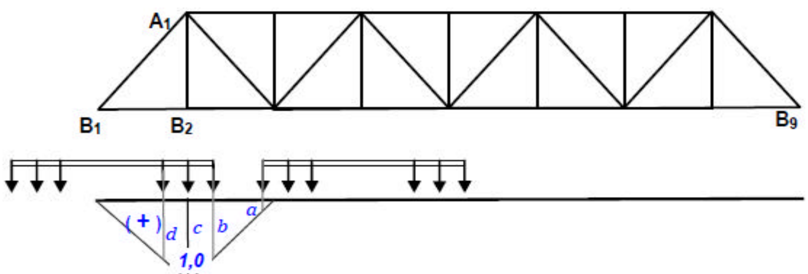
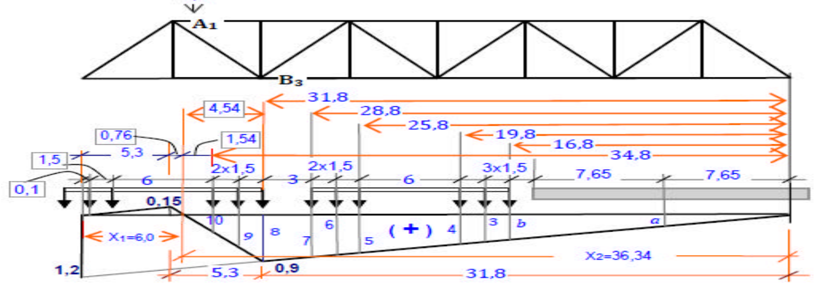
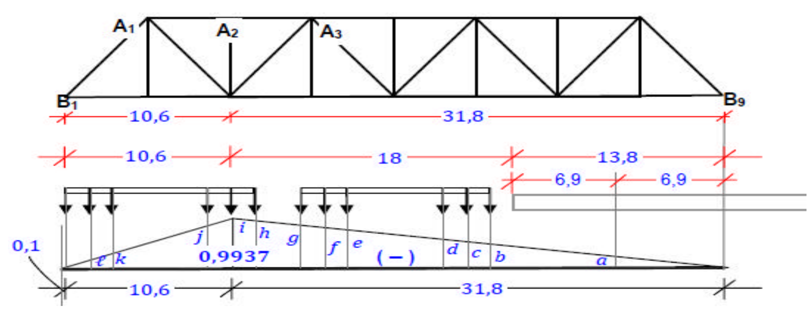
Members	Influencing lines
A_1B_1	
$B_1B_2 = B_2B_3$	
A_1B_2	
$A_1A_2 = A_2A_3$	
A_1B_3	

Table 4: Continue

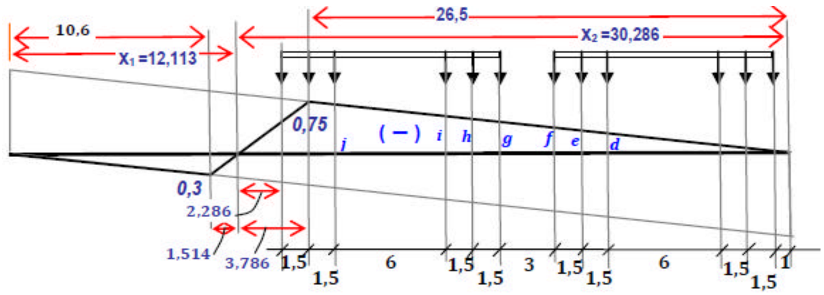
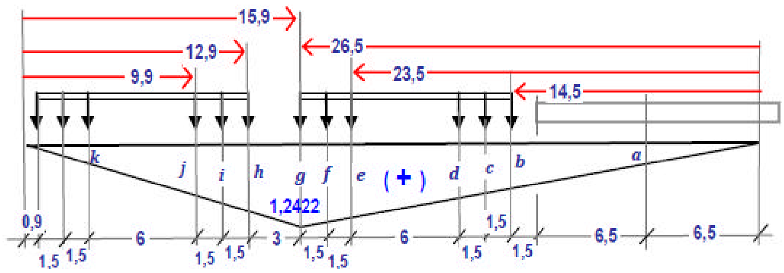
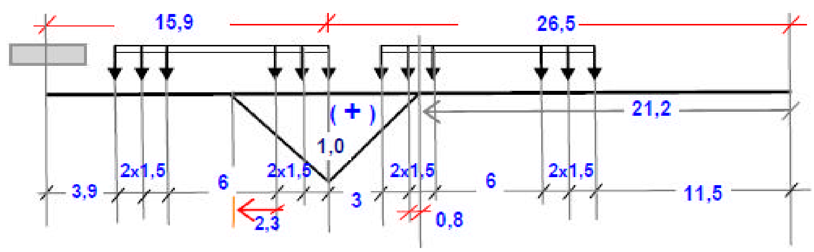
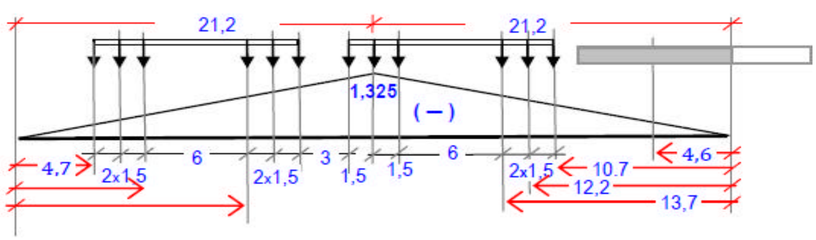
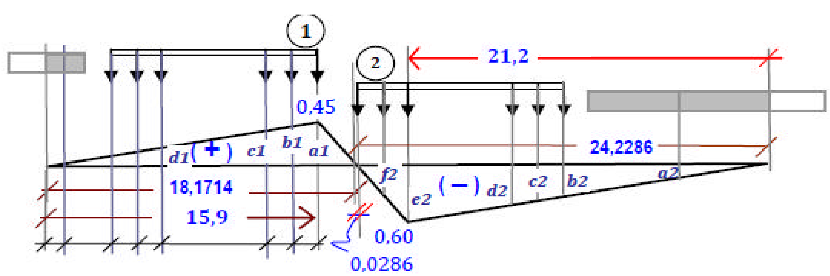
Members	Influencing lines
A_3B_3	
$B_3B_4 = B_4B_5$	
A_3B_4	
$A_3A_4=A_4A_5$	
A_3B_5	

Table 5: Members loads due to DL, LL, TIF and BIL

		Load (ton)			
Members		(DL)	(LL)	TIF	BIL
A1B1	d = 1	-33.390	-81.455	-26.799	-20.363
A1A2	a = 2	-31.601	-72.718	-23.924	-18.179
A2A3	a = 3	-31.601	-72.7179	-23.9242	-18.179
A3A4	a = 4	-42.135	-93.3786	-30.7216	-23.345

Table 5: Continue

Members		Load (ton) (DL)	(LL)	TIF	BIL
A4A5	a = 5	-42.135	-93.379	-30.722	-23.345
A5A6	a = 6	-31.601	-72.718	-23.924	-18.179
A6A7	a = 7	-31.601	-72.718	-23.924	-18.179
A7B9	d = 8	-33.390	-81.455	-26.799	-20.363
A1B2	v = 9	+7.950	+23.2638	+7.654	+5.816
A2B3	v = 10	0.000	0.000	0.000	0.000
A3B4	v = 11	+31.8000	+24.6231	+8.1010	+6.1558
A4B5	v = 12	0.000	0.000	0.000	0.000
A5B6	v = 13	+31.800	+24.623	+8.101	+6.156
A6B7	v = 14	0.000	0.000	0.000	0.000
A7B8	v = 15	+7.950	+23.264	+7.654	+5.816
B1B2	b = 16	+18.444	+39.971	+13.150	+9.993
B2B3	b = 17	+18.444	+39.971	+13.150	+9.993
B3B4	b = 18	+39.502	+92.883	+30.559	+23.221
B4B5	b = 19	+39.5020	+92.883	+30.559	+23.221
B5B6	b = 20	+39.5020	+92.883	+30.559	+23.221
B6B7	b = 21	+39.5020	+92.883	+30.559	+23.221
B7B8	b = 22	+18.444	+39.971	+13.150	+9.993
B8B9	b = 23	+18.444	+39.971	+13.150	+9.993
A1B3	d = 24	+23.848	+60.751	+19.987	+15.188
A3B3	d = 25	-14.310	-41.264	-13.576	-10.316
A3B5	d = 26	-10.903	-17.621	-5.797	-4.405
A5B5	d = 27	-10.903	-17.621	-5.797	-4.405
A5B7	d = 28	-14.310	-41.2641	-13.5759	-10.316
A7B7	d = 29	+23.848	+60.751	+19.987	+15.188

a = Upper horizontal member, b = Lower horizontal member; d = Diagonal member, vertical member

Table 6: Result of force member of railway steel-truss

Members	Load (DL)	(LL)	RIL	BIL	Factored load (Q) F_1 = 1.4DL	F_2=1.2DL+1.6LL
A1B1 d = 1	-33.3900	-81.4550	-26.7990	-20.3630	-46.7460	-219.953
A1A2 a = 2	-31.6010	-72.7180	-23.9240	-18.1790	-44.2410	-192.550
A2A3 a = 3	-31.6010	-72.7180	-23.9240	-18.1790	-44.2410	-192.550
A3A4 a = 4	-42.1350	-93.3786	-30.7216	-23.3447	-58.9890	-249.122
A4A5 a = 5	-42.1350	-93.3786	-30.7216	-23.3447	-58.9890	-249.122
A5A6 a = 6	-31.6010	-72.7180	-23.9240	-18.1790	-44.2410	-192.550
A6A7 a = 7	-31.6010	-72.7180	-23.9240	-18.1790	-44.2410	-192.550
A7B9 d = 8	-33.3900	-81.4550	-26.7990	-20.3630	-46.7460	-213.275
A1B2 v = 9	+7.9500	+23.2640	+7.6540	+5.8160	+11.1300	+59.008
A2B3 v = 10	0.0000	0.0000	0.0000	0.0000	0.0000	0.000
A3B4 v = 11	+31.8000	+24.6230	+8.1010	+6.1560	+44.5200	+90.519
A4B5 v = 12	0.0000	0.0000	0.0000	0.0000	0.0000	0.000
A5B6 v = 13	+31.8000	+24.6230	+8.1010	+6.1560	+44.5200	+90.519
A6B7 v = 14	0.0000	0.0000	0.0000	0.0000	0.0000	0.000
A7B8 v = 15	+7.9500	+23.2640	+7.6540	+5.8160	+11.1300	+59.008
B1B2 b = 16	+18.4440	+39.9710	+13.1500	+9.9930	+25.8220	+107.127
B2B3 b = 17	+18.4440	+39.9712	+13.1505	+9.9928	+25.8216	+107.127
B3B4 b = 18	+39.5020	+92.8830	+30.5590	+23.2210	+55.3030	+244.909
B4B5 b = 19	+39.5020	+92.8830	+30.5590	+23.2210	+55.3030	+244.909
B5B6 b = 20	+39.5020	+92.8833	+30.5590	+23.2210	+55.3030	+244.909
B6B7 b = 21	+39.5020	+92.8833	+30.5590	+23.2210	+55.3030	+244.909
B7B8 b = 22	+18.4440	+39.9712	+13.1500	+9.9930	+25.8220	+107.127
B8B9 b = 23	+18.4440	+39.9712	+13.1500	+9.9930	+25.8220	+107.127
A1B3 d = 24	+23.8480	+60.7510	+19.9870	+15.1880	+33.3870	+157.798
A3B3 d = 25	-14.3100	-41.2640	-13.5760	-10.3160	-20.0340	-104.916
A3B5 d = 26	-10.9030	-17.6210	-5.7973	-4.4050	-15.2640	-50.553
A5B5 d = 27	-10.9030	-17.6210	-5.7973	-4.4050	-15.2640	-50.553
A5B7 d = 28	-14.3100	-41.2640	-13.5759	-10.3160	-20.0340	-104.916
A7B7 d = 29	+23.8480	+60.7510	+19.9870	+15.1880	+33.3870	+157.798

a = Upper horizontal member, b = Lower horizontal member, d = Diagonal member, vertical member

Table 7: Reliability index of every member

Members	λ_L	ξ^2_L	λ_R	ξ^2_L	β
A1B1 d = 1	9.7004	0.086	10.4940	0.0223	2.4114
A1A2 a = 2	9.5674	0.086	10.4940	0.0223	2.8156
A2A3 a = 3	9.5674	0.086	10.4940	0.0223	2.8156
A3A4 a = 4	9.8250	0.086	10.4940	0.0223	2.0328
A4A5 a = 5	9.8250	0.086	10.4940	0.0223	2.0328
A5A6 a = 6	9.5674	0.086	10.4940	0.0223	2.8156
A6A7 a = 7	9.5674	0.086	10.4940	0.0223	2.8156
A7B9 d = 8	9.6696	0.086	10.4940	0.0223	2.5050
A1B2 v = 9	8.3847	0.086	10.1559	0.0223	6.4093
A2B3 v = 10	6.8328	0.086	10.1559	0.0223	11.1249
A3B4 v = 11	8.8126	0.086	10.1559	0.0223	5.1091
A4B5 v = 12	6.8328	0.086	10.1559	0.0223	11.1249
A5B6 v = 13	8.8126	0.086	10.1559	0.0223	5.1091
A6B7 v = 14	6.8328	0.086	10.1559	0.0223	11.1249
A7B8 v = 15	8.3847	0.086	10.1559	0.0223	6.4093
B1B2 b = 16	8.9810	0.086	10.4940	0.0223	4.5974
B2B3 b = 17	8.9810	0.086	10.4940	0.0223	4.5974
B3B4 b = 18	9.8079	0.086	10.4940	0.0223	2.0848
B4B5 b = 19	9.8079	0.086	10.4940	0.0223	2.0848
B5B6 b = 20	9.8079	0.086	10.4940	0.0223	2.0848
B6B7 b = 21	9.8079	0.086	10.4940	0.0223	2.0848
B7B8 b = 22	8.9810	0.086	10.4940	0.0223	4.5974
B8B9 b = 23	8.9810	0.086	10.4940	0.0223	4.5974
A1B3 d = 24	9.3683	0.086	10.4940	0.0223	3.4205
A3B3 d = 25	8.9602	0.086	10.4940	0.0223	4.6606
A3B5 d = 26	8.2300	0.086	10.4940	0.0223	6.8794
A5B5 d = 27	8.2300	0.086	10.4940	0.0223	6.8794
A5B7 d = 28	8.9602	0.086	10.4940	0.0223	4.6606
A7B7 d = 29	9.3683	0.086	10.4940	0.0223	3.4205

a = Upper horizontal member, b = Lower horizontal member, d = Diagonal member, vertical member

member analysis of Railway Steel-Truss due to factored load is shown in Table 6. Finally, reliability index for every members is shown in Table 7.

Based on Table 7 above, it could be seen that reliability index of the members in railway bridge BH 1459 is in a range of 2.0328 11.1249. Based on SNI 03-1729-2002, the minimum allowable β value for element is 3.0 and for joint is 4.5. However, there are still 17 members which have reliability index above standard value.

CONCLUSION

Even though, there are more than half number members that still have reliability index above standard, however, reliability performance of an engineering system would be determined based on the minimum reliability index of the sub-system. Therefore, from safety point of view, the investigated railway bridge could be said not reliable anymore. For safety reason, redesign of the railway bridge to meet the minimum requirement become very important.

SUGGESTIONS

Suggestion for further research is by considering dynamic load on the railway bridge in order to be closed to the real condition. The A3 dimensional analysis on the railway bridge is also considerable in order to get more accurate result.

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