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Evaluation of Safety Performance of Fractured Joints in Steel Truss Bridge

Yi Yi Mon[a]*, San Yu Khaing[b]

[a,b]*Department of Civil Engineering, Mandalay Technological University, The Republic of the Union of Myanmar*

[a]*Email: yiyimon.civil@gmail.com*

[b]*Email: sykpku@gmail.com*

Abstract

This study aims to evaluate safety performance of fractured joints and failure mechanisms of high-strength friction-grip (HSFG) bolted connections in steel bridge. As a case study, the bolted connections on the Ayeyarwady Bridge (Yadanarbon) in Myanmar have been investigated. Initial inspection of the bridge found that a number of bolts were loosened, missing or had already fractured and most were heavily corroded. The original fractured bolts have been replaced with the new ones as remedial work. The safety performance of fractured joints were evaluated based on simulation results of existing bridge by Midas Civil in accordance with AASHTO standards. The failure mechanisms of the fractured bolts were investigated by analyzing through optical and scanned electron microscopy. The main factors contributed to the failure of the bolted connections at the Ayeyarwady Bridge (Yadanarbon) include delayed fracture, stress corrosion cracking, the fluctuating loads, and the corrosive environment it was subjected to. The failure mechanisms of the fractured bolts were mainly due to pitting corrosion and fretting fatigue.

Keywords: highway bridge; HSFG bolts; modes of failure.

1. Introduction

Bridges are one of the most important infrastructure in land transportation. There are many types of bridges according to their advantages in application, construction, economy and maintenance. Among them, steel truss bridges are popular for long span river crossing bridges although it may call for regular maintenance program.

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* Corresponding author.

Connecting members to erect a steel bridge is usually done by welding, or using high strength bolted friction joints. High-strength bolts have many advantages for steel construction, including high joint integrity, easy installation and replacement, and good fatigue strength [1]. Friction-grip high-strength bolts became very popular for construction of steel truss bridges in Myanmar. But in steel truss bridges in particular, the loads on high-strength bolts can be complex and likely to result in premature failure if the bolts are not properly designed, manufactured and installed [2]. In Myanmar, there are large numbers of steel truss bridges crossing the rivers and major creeks. Among them, at Ayeyarwady Bridge (Yadanabon) and Sinphyushin Bridge, early fracture of High strength Bolts were found as an issue [3]. Although Public Works settled down the issue by replacing those with suitable ones after intensive investigation, it still remains to answer some questions not to repeat the same issue for the new ones and for future rational maintenance strategy [4]. It is important to understand the structural behavior of a steel truss bridge and its specific nature for proper maintenance program for it. Thus the focus on the maintenance issues of the steel truss bridge is set up as the study area for the development of a rational maintenance strategy for that type of bridge. In this study, Ayeyarwady Bridge (Yadanarbon) is selected as the case study.

2. Case Study

The Ayeyarwady bridge (Yadanarbon) spans the Ayeyarwady river linking Mandalay suburb and Sagaing city, approximately 2000 feet to the north of the old Ava bridge. It is the first longest span steel Arch bridge in Myanmar and also the gateway to Yangon, Mandalay and any other regions in the country. It comprises two units of 2*112 m continuous steel truss and one unit of 3* 224 m continuous arch truss. The main bridge is 1125.8 m long, its approach on the Mandalay side is 347 m long and on the Sagaing side is 780 m. The carriageway consists of four lanes for motorized traffic and two footpaths for pedestrians on each side of the truss. The truss is 16 m wide, and the free headroom of the carriageway is 5.5 m. The pedestrian footpaths externally on both sides of the truss are 1.8 m. It is designed by China CAMC Engineering co.ltd (CAMCE) and constructed by Public Works of Ministry of Construction in 2001. Some DG suspenders were cracked at wind velocity of 35 miles per hour during erection and its suspender design have been changed after wind tunnel test in China. The construction work was completed with the official opening in 2008. The bridge is maintained by Public Works of Ministry of Construction.

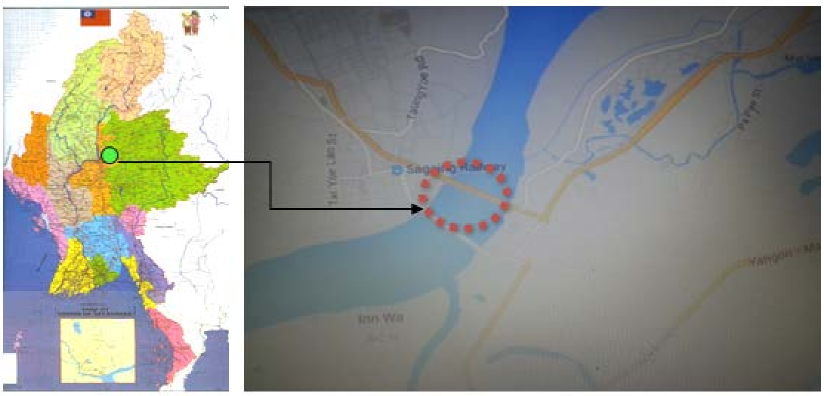


Figure 1: Location Map of Ayeyarwady Bridge (Yadanarbon)

Figure 2: Ayeyarwady Bridge (Yadanarbon)

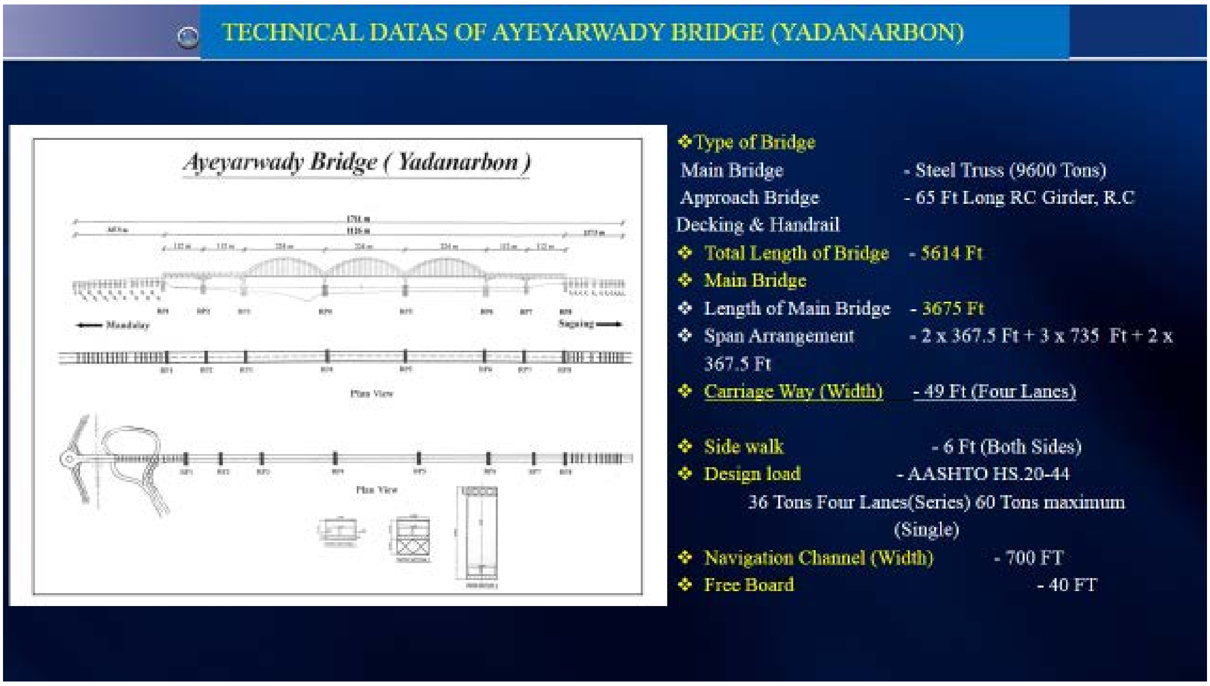


Figure 3: Technical data of Ayeyarwady Bridge (Yadanarbon)

2.1. Issues after one year completion

- After one year later that the new bridge has been constructed, frequent damage of high strength bolts were observed during maintenance.
- Horizontal fatigue crack damages were found in some floor beams five years after construction completion and now can be solved by Stop Hole Method.
- There are 28000 bolts and nuts replaced with S10T bolts and nuts of Japanese product and 5639 number of bolts and nuts that have been replaced with same type of high strength bolts of China until now.

- Thus, frequent damage of high strength bolt came out as maintenance issue.

2.2. Objectives of the research

- To analyze the possible causes of High-strength bolts fracture failure in steel truss bridge
- To predict potential deterioration in future by evaluating safety performance of existing bridge
- To enhance a preventive rational steel bridge maintenance strategy of similar bridges

3. Research Methodology

- Field Survey and Data Collection on bridge site and respective offices
- Failure analysis of High-strength bolts taken from the bridge
- Review and Analysis of Damage Pattern of HSB
- Modelling and Simulation of existing bridge by Midas Civil 2015 v 1.1
- Evaluation of Safety Performance of fractured joints based on analysis results

3.1. Field Survey and Data Collection on Bridge Site

Figure 4: Sketch of Ayeyarwady Bridge (Yadanarbon)

Axial strain, thickness of paint and members, vibration of the bridges and weather conditions (wind speed and direction, temperature, relative humidity, pressure, etc.) on the bridges were measured. The data measurement were summarized as follows:

- The strain on diagonal member D3 between P6 and P7 varied within +19 με to -22 με which corresponds to the axial stress range between 3.8 MPa and -4.4 MPa, modulus of elasticity of steel as 200 GPa.
- Thickness of paint on members V1, D2, V2 were measured on both web and flange and the average thickness is 181 μm but the minimum thickness of 130 μm was found on D2 (Table 3).
- Classification of cross-cut test (Table 4) for the paint on vertical member is "1" for V1 with original paint system. However classification for diagonal member D2 is "4". D2 is also the original paint but shows remarkable wearing on top coat from the appearance investigation. Classification for V2 is poor value "5" where there is the repair painted part.
- The original paint system consists of two layers, however, the paint type was not identified clearly by the micro-area infra-red spectrometer.
- The average thickness of member V1 was measured as 13.54 mm for the flange and thickness of

member D2 is 19.40 mm for the web and 29.29 mm for the flange (Table 1).

- The fundamental frequencies of some modes of the bridge were found to be around 1.4 Hz, 2.6 Hz and 3.2 Hz by converting the vibration data to FFT graphs (see Figure 5).
- The average values for the wind speed, temperature, relative humidity and atmospheric pressure were measured as 1.9 m/s, 32.6°C, 62.8%, 996 hPa respectively.
- Bolt axial forces are shown in Table 5. According to the Japanese standard and AASHTO, the introduced bolt axial force for bolt of which diameter is 24mm and strength class is F10T or 10.9 should be 238kN and 257kN respectively. Bolt axial force of Bolt 3 and 4 are less than the other bolts due to corrosion (see Figure 6).

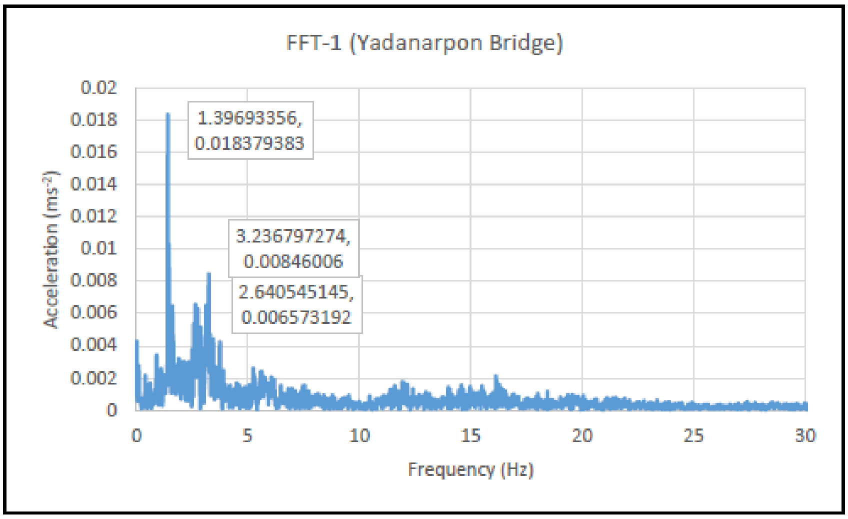


Figure 5: Example of FFT graph for Yadanarpon Bridge

Table 1: Thickness of members

Member	V1	D2 (Truss)	
Position	Flange	Web (Upper Side)	Flange (Outside)
Unit	mm	mm	mm
1st	13.57	19.40	29.28
2nd	13.56	19.39	29.29
3rd	13.57	19.42	29.29
4th	13.44	19.39	29.3
5th	13.55	-	29.29
6th	13.57	-	-
Average	13.54	19.40	29.29

Table 2: Surface salinity

V1	26.6 mg/m^2
D2	36.4 mg/m^2

Table 3: Paint Thickness

Member	V1	D2 (Truss)			V2 (Truss)	
Position	Flange (outside)	Web (upside)	Flange (outside)	Web (lower side)	Flange (outside)	Web
Unit	µm	µm	µm	µm	µm	µm
Average	214	185	163	153	200	171
1st	209	156	157	182	172	156
2nd	244	197	155	154	243	161
3rd	181	181	160	160	202	175
4th	186	160	130	130	192	234
5th	187	193	167	146	200	172
6th	211	245	193	148	222	159
7th	235	162	182	149	171	191
8th	318	-	-	-	197	168
9th	183	-	-	-	-	205
10th	182	-	-	-	-	159
11th	134	-	-	-	-	156
12th	131	-	-	-	-	144
13th	285	-	-	-	-	151
14th	183	-	-	-	-	168
15th	211	-	-	-	-	-
16th	231	-	-	-	-	-
17th	208	-	-	-	-	-
18th	300	-	-	-	-	-
19th	242	-	-	-	-	-

*Data in gray are measured at re-painted area.

Table 4: Classification of cross-cut test results (ISO 2409-2007)

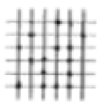
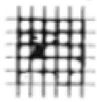
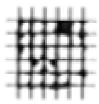
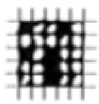
Classification	Description	Appearance of surface of cross-cut area from which flaking has occurred (Example for six parallel cuts)
0	The edges of the cuts are completely smooth; none of the squares of the lattice is detached.	—
1	Detachment of small flakes of the coating at the intersections of the cuts. A cross-cut area not greater than 5 % is affected.	
2	The coating has flaked along the edges and/or at the intersections of the cuts. A cross-cut area greater than 5 %, but not greater than 15 %, is affected.	
3	The coating has flaked along the edges of the cuts partly or wholly in large ribbons, and/or it has flaked partly or wholly on different parts of the squares. A cross-cut area greater than 15 %, but not greater than 35 %, is affected.	
4	The coating has flaked along the edges of the cuts in large ribbons and/or some squares have detached partly or wholly. A cross-cut area greater than 35 %, but not greater than 65 %, is affected.	
5	Any degree of flaking that cannot even be classified by classification 4.	—

Table 5: Bolts axial forces

Specimen		Strain [$*10^{-6}$]			Bolt Axial Force [kN]
		Before losen	After losen	Differential	
Bolt 1	ch 0	-654	893	1547	294.6
	ch1*	(-688)	(10927)	(11615)	(2010.5)
	ch 2	425	2110	1685	176.0
	Avg.				**235.3**
Bolt 2	ch 0	-687	1222	1909	299.2
	ch1*	(15999)	(32900)	(16901)	(3387.1)
	ch 2	102	1188	1086	210.4
	Avg.				**254.8**
Bolt 3	ch 0	1454	2081	627	108.1
	ch1*	(5183)	(13136)	(7953)	(1105.3)
	ch 2	239	1138	899	120.2
	Avg.				**114.2**
Bolt 4	ch 0	-790	658	1448	242.7
	ch1	-94	1147	1241	249.4
	ch 2	373	1207	834	133.4
	Avg.				**208.5**
*Note: Ch1 of Bolt1, 2, and 3 may not be correct due to the trouble in data logger.					

Corrosion of screw
(Bolt 3)
Corrosion of screw (Bolt 4)

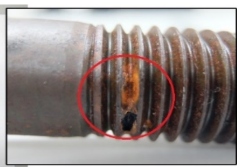


Figure 6: Corrosion of high strength bolts

3.2. Failure analysis of High-strength bolts taken from the bridge

The following tests were conducted to analyse the failure mechanisms of the bolts taken from the bridge.

- Observation of Appearance
- Sectional Tissue

30

- Observation of Fracture Surface Geometry
- Hardness Testing
- Chemical Component Analysis

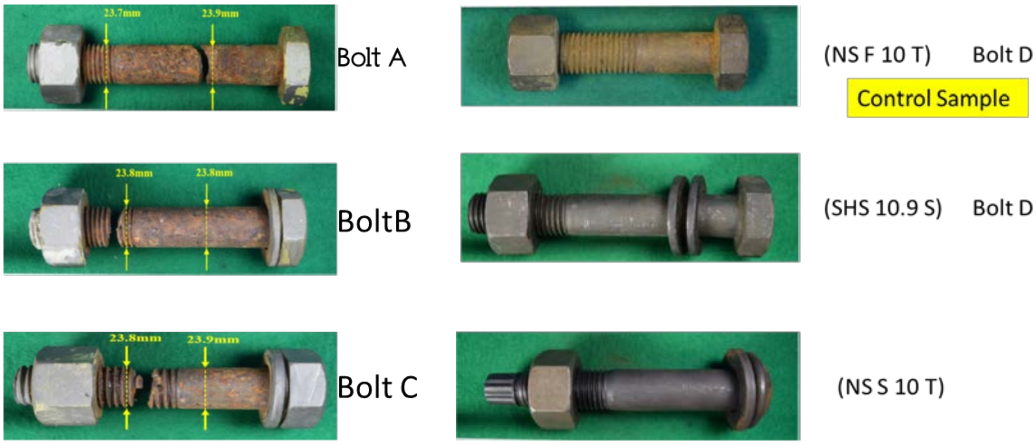


Figure 7: Samples of high strength bolts

3.2.1. Observation of Appearance

Radial fractures occurred in the cylindrical part of Bolt A and in the thread root of Bolts B and C.

These bolts show no conspicuous deformation, such as bending or a change in diameter.

Figure 8: Observation of appearance of high strength bolts

3.2.2. Sectional Tissue

Although the fracture surface geometry is obscured by corrosion, intergranular fractures are present. A number of corrosion pits and microcracks can be seen.

3.2.3. Observation of Fracture Surface Geometry

Fracture surface is completely rusted and a number of microcracks and corrosion pits are visible. Although some parts are obscured by corrosion, the fracture surface is chiefly that of intergranular fracture. This resembles the characteristics of a delayed fracture.

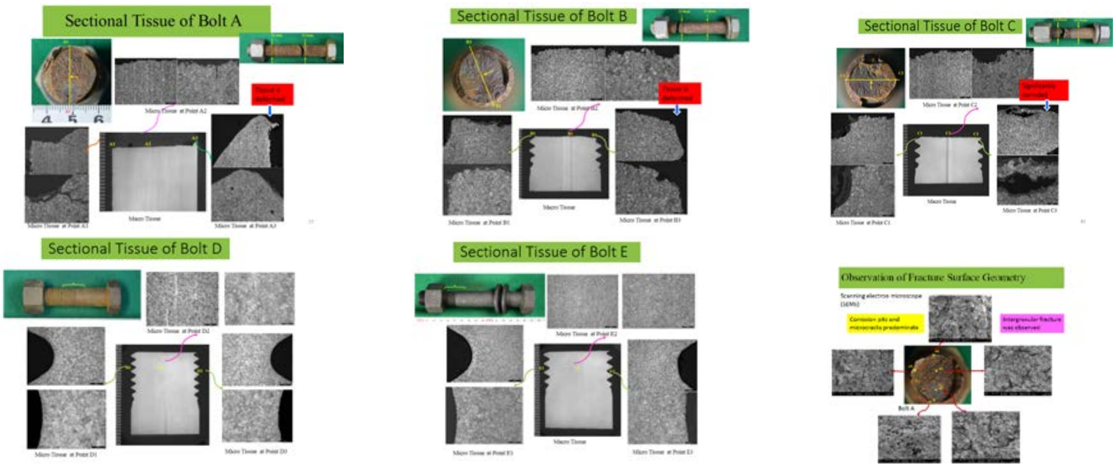


Figure 9: Sectional tissue of high strength bolts

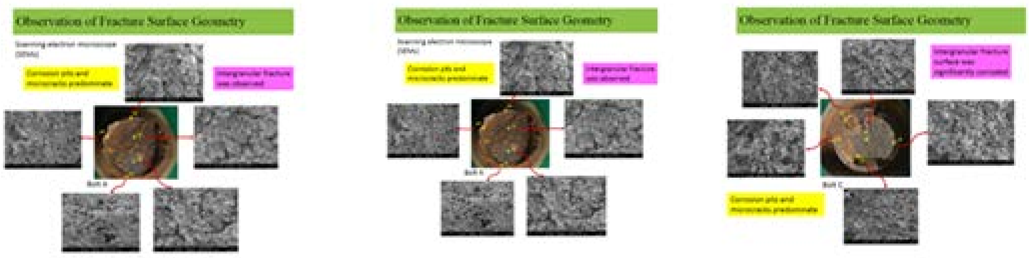
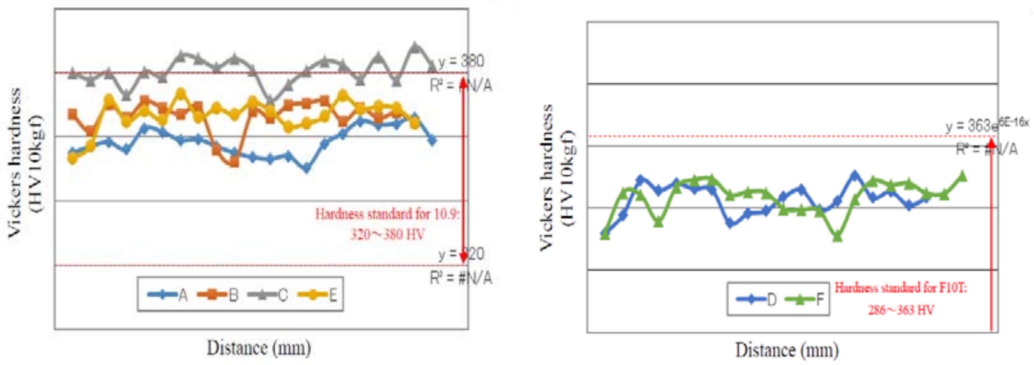


Figure 10: Observation of Fracture Surface Geometry

3.2.4. Hardness Testing

The hardness values for Bolts A-C and E are 352-388 HV. Only some of the hardness values for bolt C exceed the standard upper limit for the standard hardness values for the strength category of 10.9 (320-380 HV). The hardness values of Bolt D and F are 331-350 HV, which differ from those for Bolts A-C and E. The hardness values for the washers are 400-429 HV. The Standard hardness values for washers in the strength category of 300 HV range from 300 to 370 HV (washers from Bolts A-C ad E). Their values exceed the upper limit of the standard.



Measurement point	Hardness value			
	A	E	D	F
Outer side	428	417	422	400
Center	428	422	429	403
Inner side	423	416	428	408
Average	426	418	426	403
Standard hardness value	JIS B 1256 Hardness category of 300 HV: 300-370 HV		JIS B 1186 Grade F35: 35-45 HRC (=345-446 HV*)	

* Approximated from Hardness conversion table (SAE J 417).

Standard No.	Category or grade	Hardness standard
JIS B 1051	10.9	320 - 380 HV
	12.9	385 - 435 HV
JIS B 1186	F10T	27 - 38HRC (=286 - 363 HV※)
	F11T	30 - 40HRC (= 302 - 392 HV※)

* Approximated from hardness conversion table (SAE J 417).

Figure 11: Hardness testing of high strength bolts

3.2.5. Chemical Component Analysis

Table 6: Chemical component analysis of high strength bolts

		C	Si	Mn	P	S	Ni
Fractured material	A	0.21	0.21	1.38	0.013	0.020	0.06
Unused product	D	0.19	0.12	0.79	0.018	0.008	0.02
	E	0.22	0.24	1.49	0.011	0.004	0.04
	F	0.20	0.15	0.78	0.012	0.005	0.02

		Cr	Mo	Cu	B	Ti	Al
Fractured material	A	0.10	0.02	0.18	0.0020	0.056	0.017
Unused product	D	0.36	0.01	0.01	0.0018	0.021	0.026
	E	0.06	0.01	0.11	0.0021	0.057	0.020
	F	0.36	0.01	0.01	0.0019	0.019	0.031

Analysis method
C and S: burning—infrared absorption method
Others: spark discharge optical emission spectroscopy

Table 7: Standard component of high strength bolts

	C	P	S	B	Mn
JIS B 1051 10.9 Carbon steel	0.25 to 0.55	0.035 or less	0.035 or less	0.003 or less	-
JIS B 1051 10.9 Carbon steel with additives	0.15 to 0.55				0.7* or more
JIS B 1051 10.9 Alloy steel	0.20 to 0.55				-
JIS B 1186 F10T	No component standard				

* Mn: for boron steel containing 0.25% or less of C.

3.3. Review and Analysis of Damage Pattern

According to preliminary analysis of the quantities, location and picture of the fractured HSBs based on maintenance records [5], the fracture occurs in the whole bridge and at random, in disorder and at a wide range.

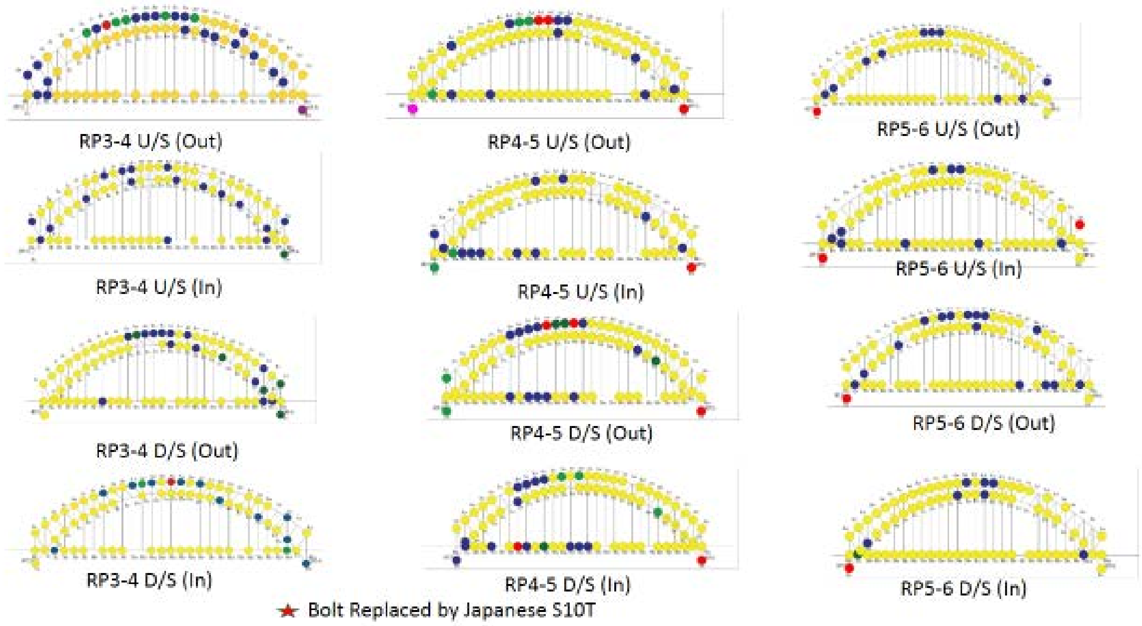


Figure 12: Damage pattern of fractured high strength bolts

3.4. Modelling and Simulation of existing bridge by Midas Civil 2015 v 1.1

In order to study the overall response of the bridge, a 3D bridge model was constructed by separating arch and truss portions and analyzed using Midas Civil 2015 v1.1 [6]. The model geometry was based on the original construction drawings. The support conditions were the same as designed. The finite element model was calibrated to replicate the behavior of the actual bridge by using the data collected during the live load tests.

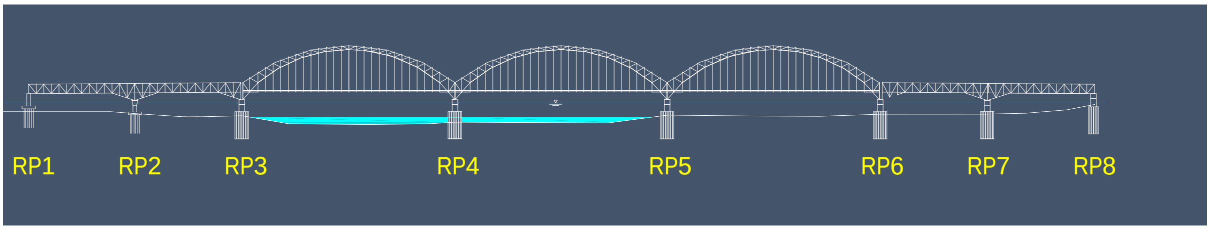


Figure 13: Layout of Ayeyarwady Bridge (Yadanarbon)

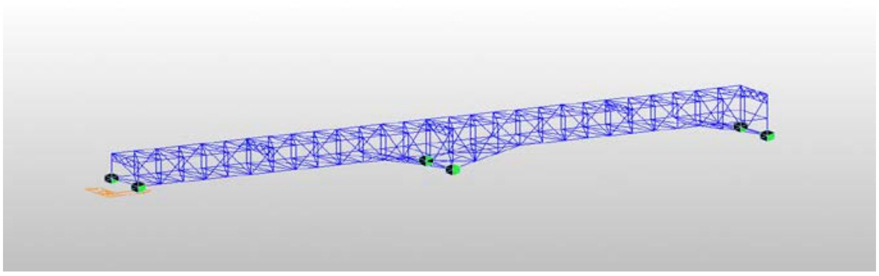


Figure 14: 3-D long span truss bridge model by using MIDAS/Civil

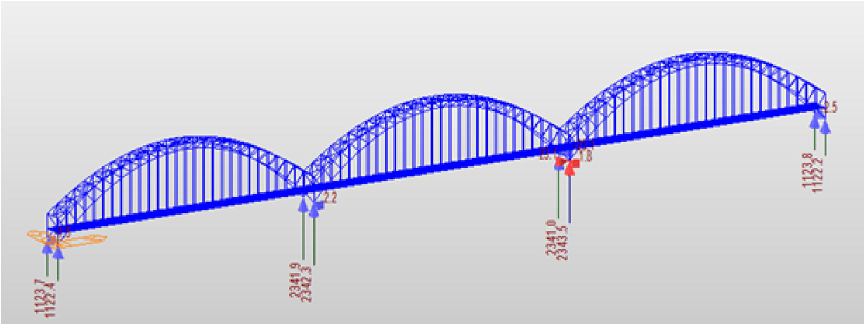


Figure 15: 3-D long span arch bridge model by using MIDAS/Civil

3.4.1. Support Reaction

Supports reaction obtained by MIDAS/Civil in 3 directions (x,y,z) for different loading are listed in Table 8 and 9 .

Table 8: Support reaction of truss bridge

Node	**Load**	**FX (tonf)**	**FY (tonf)**	**FZ (tonf)**
30	DL	0	0	724.327699
44	DL	-6.545124	0	2735.17455
58	DL	0	0	761.558024
88	DL	0	0.840551	724.685918
102	DL	6.545124	-0.711022	2734.765075
116	DL	0	-0.129529	761.248953
30	MVL(all)	0	0	134.949945
44	MVL(all)	-116.37988	0	294.045826
58	MVL(all)	0	0	138.069601
88	MVL(all)	0	-8.809438	134.980625
102	MVL(all)	116.37988	9.112794	295.453136
116	MVL(all)	0	-10.248812	138.019303
30	LCB	0	0	720.817773
44	LCB	-8.771125	0	2714.8085
58	LCB	0	0	754.866264
88	LCB	0	10.021382	727.967271
102	LCB	8.771125	25.798446	2755.588273
116	LCB	0	8.721376	767.712139

Table 9: Support reaction of arch bridge

Node	**Load**	**FX (tonf)**	**FY (tonf)**	**FZ (tonf)**
1	DL	0.00	0.10	1170.51
29	DL	0.00	-0.45	2485.70
170	DL	0.00	0.00	1177.42
196	DL	0.00	0.00	2482.06
329	DL	0.00	0.47	1175.29
355	DL	-10.19	-0.11	2493.76
452	DL	0.00	0.00	1178.47
478	DL	10.19	0.00	2486.38
1	MVL(all)	0.00	0.53	40.92
29	MVL(all)	0.00	-1.59	42.39
170	MVL(all)	0.00	0.00	40.86
196	MVL(all)	0.00	0.00	42.20
329	MVL(all)	0.00	-0.91	41.01
355	MVL(all)	-18.62	1.28	42.32
452	MVL(all)	0.00	0.00	40.91
478	MVL(all)	18.62	0.00	42.18
1	LCB	0.00	68.68	1306.41
29	LCB	0.00	182.67	2886.47
170	LCB	0.00	0.00	1123.36
196	LCB	0.00	0.00	2165.85
329	LCB	0.00	66.95	1299.32
355	LCB	39.11	185.86	2908.61
452	LCB	0.00	0.00	1136.45
478	LCB	-39.11	0.00	2155.90

3.4.2. Displacement and Deflections

The maximum vertical (Z-Direction) deflections at mid span due to dead load and service load of truss bridge (Figure 16) is 0.27 m and that of arch bridge (Figure 17) is 0.3 m. Both are exceeding the allowable limits of L/360.

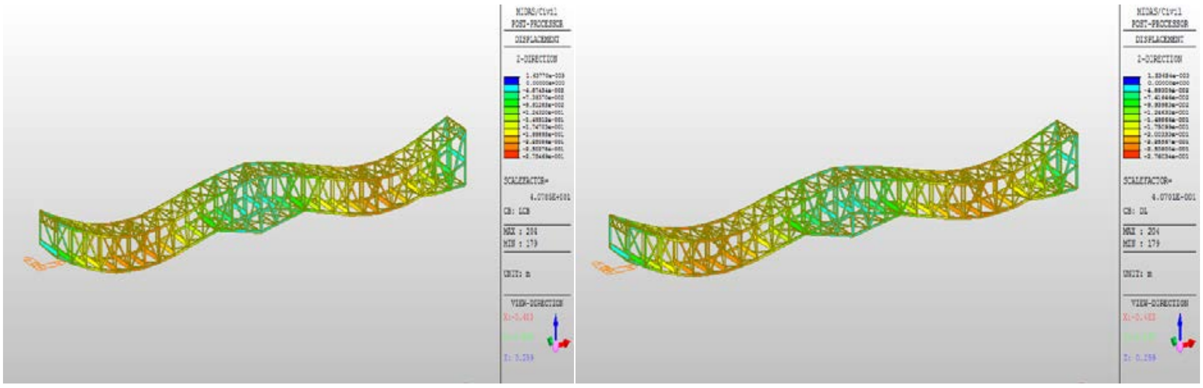


Figure 16: Deflection due to dead and service load of truss bridge

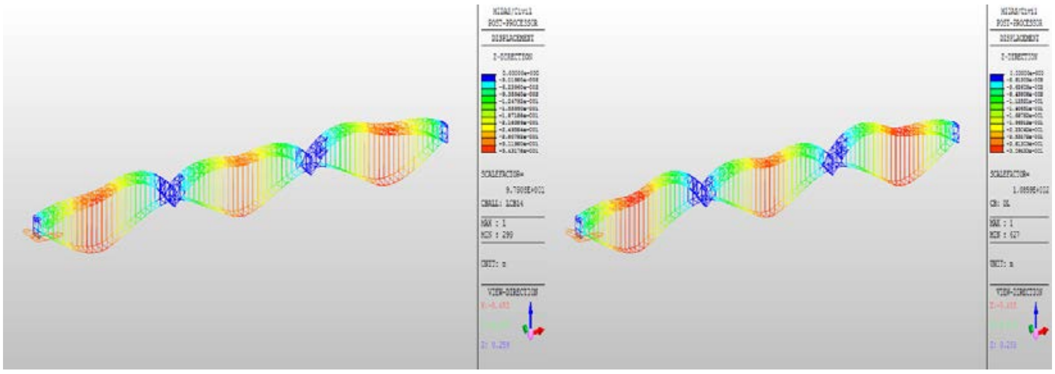


Figure 17: Deflection due to dead and service load of arch bridge

3.4.3. Stress

Maximum stress of truss bridge (Figure 18) for dead load and that of service load is 275 MPa. For arch bridge (Figure 19) maximum stress for dead load is 158 MPa and that of service load is 248 MPa. Allowable stress is 370 MPa and stresses are within allowable limits.

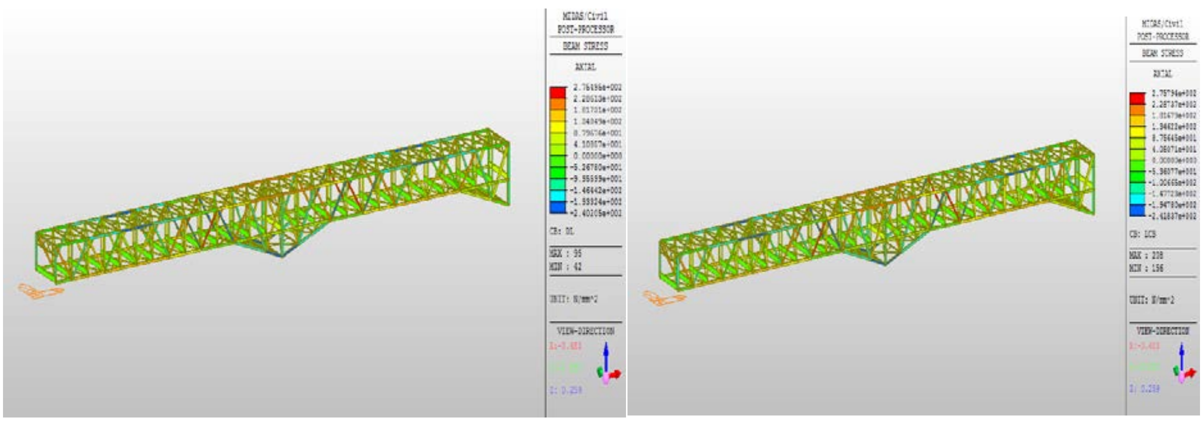


Figure 18: Stress due to dead and service load of truss bridge

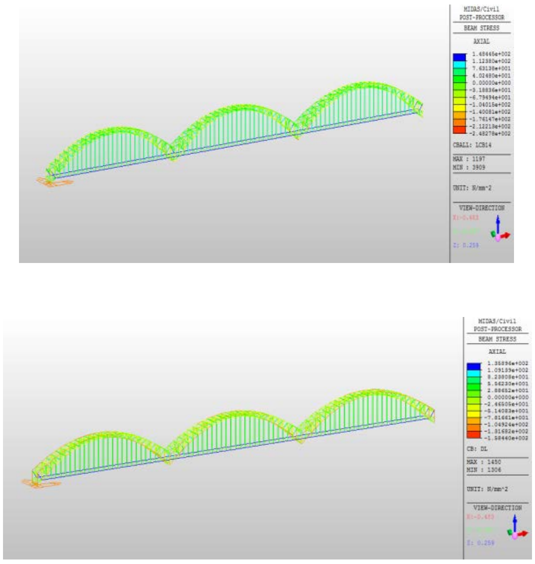


Figure 19: Stress due to dead and service load of arch bridge

3.5. Evaluation of Safety Performance of fractured joints based on analysis results

Safety performance of fractured joints illustrated in Figure 20 are calculated based on analysis results of Midas Civil. A8, A14, A42 and A70 shown in yellow color are top chord joints. E28, E46, E51 and E62 of green color are bottom chord joints and M14, M16, M26, M30, M35, and M82 of blue color are middle chord joints. Their corresponding safety performance are shown in Table 10. Table 11 shows safety factor and percent fracture of considered joints.

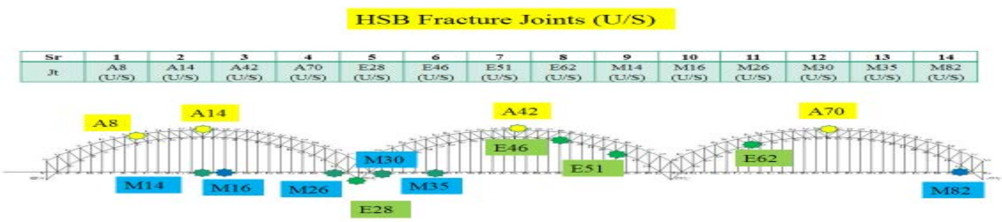


Figure 20: Considered fractured joints of arch bridge

According to AISC manual connection, for slip critical joints designed at service load level is as follows [7];

$\phi R_s = D\mu T_m N_b N_s$ (LRFD 5.1) AISC-LRFD MANUAL CONNECTION

Where,

R_s = nominal slip resistance of a bolt for use at nominal loads, kips

T_m = minimum fastener tension, kips

N_b = number of bolts in the joint

N_s = number of slip planes

D = Slip Probability Factor (0.81 for μ equal to 0.33, 0.86 for μ equal to 0.40, 0.86 for μ equal to 0.50)

μ = mean slip coefficient for Class A, B or C surfaces

= 0.33 for Class A surfaces (unpainted clean mill scale steel surfaces or surfaces with Class A coating on blast-cleaned steel)

= 0.50 for Class B surfaces (unpainted blast-cleaned steel surfaces or surfaces with Class B coatings on blast-cleaned steel)

= 0.40 for Class C surfaces (hot-dip galvanized and roughed surfaces)

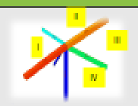
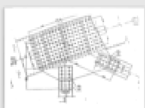
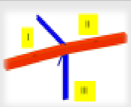
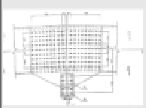
ϕ = 1.0 for standard holes

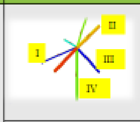
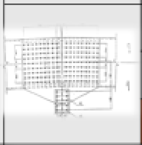
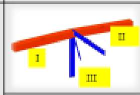
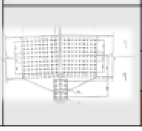
= 0.85 for oversize and short slotted holes

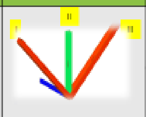
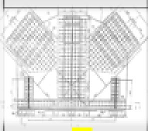
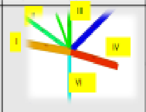
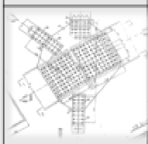
= 0.70 for long slotted holes transverse to the direction of load

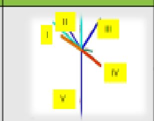
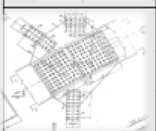
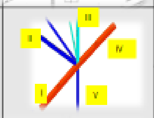
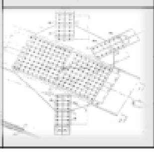
= 0.60 for long slotted holes parallel to the direction of load

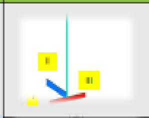
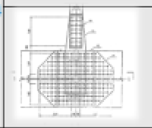
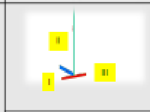
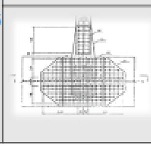
Table 10: Safety performance of fractured joints

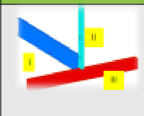
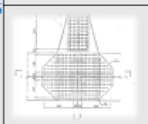
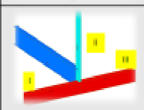
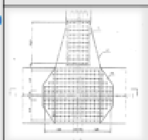
Sr.	Jt No	Joint		Member I	Member II	Member III	Member IV	Member V	
1.	A8		Member Forces (KN)	-11661.7	-12060.3	825.31	208.06	-	
			Slippage Resistance (KN)	15232.32	15046.6	1486.08	1486.08	-	
			No. of Bolts	164	162	28	28	-	382
			Damaged Bolts					-	51 (13.4%)
			S.F	1.3	1.2	1.8	7.1	-	2.8
2.	A14		Member Forces (KN)	-11395.49	-11351.1	689.82	-	-	
			Slippage Resistance (KN)	15046.56	-15046.56	1114.56	-	-	
			No. of Bolts	162	162	20	-	-	344
			Damaged Bolts						53 (15.4%)
			S.F	1.3	1.3	1.6	-	-	1.4

Sr	Jt No	Joint		Member I	Member II	Member III	Member IV	Member V	
			Member Forces (KN)	-11229.58	-11230.02	641.13	-	-	
			Slippage Resistance (KN)	15046.56	15046.56	1114.56	-	-	
3.	A42		No. of Bolts	164	162	28	-	-	354
			Damaged Bolts						53 (15%)
			S.F	1.3	1.3	1.8	-	-	1.5
			Member Forces (KN)	-11167.49	-11208.52	669.42	-	-	
			Slippage Resistance (KN)	15046.56	15046.56	1857.6	-	-	
4.	A70		No. of Bolts	162	162	20	-	-	344
			Damaged Bolts						28 (8.1%)
			S.F	1.4	1.4	2.9	-	-	1.9

Sr.	Jt No	Joint		Member I	Member II	Member III	Member IV	Member V	
			Member Forces (KN)	-12041.53	-5159	-11969.41	-	-	
			Slippage Resistance (KN)	18761.76	24891.84	18761.76	-	-	
5.	E28		No. of Bolts	312	268	312	-	-	892
			Damaged Bolts						82 (9.2%)
			S.F	1.6	4.8	1.6	-	-	2.7
			Member Forces (KN)	-4754.1	-478.28	1568.31	-6107.54	826.7	
			Slippage Resistance (KN)	14860.8	2600.64	1486.08	15046.56	1486.08	
6.	E46		No. of Bolts	170	36	32	162	32	432
			Damaged Bolts						0%
			S.F	4.5	4.8	0.9	3.2	1.9	3.1

Sr.	Jt No	Joint		Member I	Member II	Member III	Member IV	Member V	
			Member Forces (KN)	-11124.04	-1181.16	1265.39	-11815.61	955.52	
			Slippage Resistance (KN)	14860.8	2600.64	1486.08	15046.56	1486.08	-
7.	E51		No. of Bolts	170	36	32	162	32	432
			Damaged Bolts						67(15.5%)
			S.F	1.4	2.2	1.2	1.3	1.6	1.5
			Member Forces (KN)	-12745.54	980.76	-432.02	-11776.74	939	
			Slippage Resistance (KN)	15046.56	1486.08	1486.08	14860.8	1486.08	
8.	E62		No. of Bolts	162	28	28	170	32	420
			Damaged Bolts						25(6%)
			S.F	1.2	1.5	3.4	1.3	1.6	1.8

Sr.	Jt No	Joint		Member I	Member II	Member III	Member IV	Member V	
9.	M14		Member Forces (KN)	11996.96	891.96	11998.16	-	-	
			Slippage Resistance (KN)	14860.8	2972.16	14860.8	-	-	
			No. of Bolts	160	32	160	-	-	352
			Damaged Bolts						3(0.9%)
			S.F	1.2	3.2	1.2			1.9
10.	M16		Member Forces (KN)	12006.06	958.64	12020.49	-	-	
			Slippage Resistance (KN)	14860.8	2972.16	14860.8	-	-	
			No. of Bolts	160	32	160	-	-	352
			Damaged Bolts						0%
			S.F	1.2	3	1.2	-	-	1.8

Sr.	Jt No	Joint		Member I	Member II	Member III	Member IV	Member V	
11.	M26		Member Forces (KN)	12312.22	1017.83	12301.13	-	-	
			Slippage Resistance (KN)	15232.32	8173.44	15603.84	-	-	
			No. of Bolts	164	88	168	-	-	420
			Damaged Bolts						23(5.5%)
			S.F	1.2	7.7	1.3	-	-	3.4
12.	M30		Member Forces (KN)	11904.45	1051.1	11911.27	-	-	
			Slippage Resistance (KN)	13746.24	8173.44	13746.24	-	-	
			No. of Bolts	148	88	148	-	-	384
			Damaged Bolts						49(12.8%)
			S.F	1.2	7.4	1.2	-	-	3.3

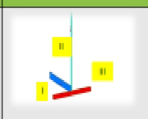
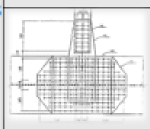
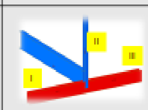
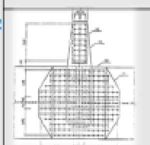
Sr.	Jt No	Joint		Member I	Member II	Member III	Member IV	Member V	
13.	M35		Member Forces (KN)	11722.65	955.28	11675.13	-	-	
			Slippage Resistance (KN)	13746.24	2972.16	14860.8	-	-	
			No. of Bolts	148	32	148	-	-	328
			Damaged Bolts						3(0.9%)
			S.F	1.2	3.1	1.3	-	-	1.9
14.	M82		Member Forces (KN)	12386.01	1078.52	12377.88	-	-	
			Slippage Resistance (KN)	13746.24	2972.16	13746.24	-	-	
			No. of Bolts	148	32	148	-	-	328
			Damaged Bolts						47(14.3%)
			S.F	1.1	2.9	1.1			1.7

Table 11: Safety factor and percent fractured of joints

Sr No.	Jt	Safety Factor	Percent Fractured
1	A14	1.4	15.40%
2	A42	1.5	13.40%
3	E51	1.5	15.00%
4	M82	1.7	14.30%
5	E62	1.8	6.00%
6	M16	1.8	0%
7	M14	1.8	0.90%
8	M35	1.8	0.90%
9	A70	1.9	8.10%
10	E28	2.5	9.20%
11	A8	2.9	13.40%
12	E46	3.1	0%
13	M30	3.3	12.80%
14	M26	3.4	5.50%

It can be seen that percent fracture decrease as safety factor increase (see Figure 21).

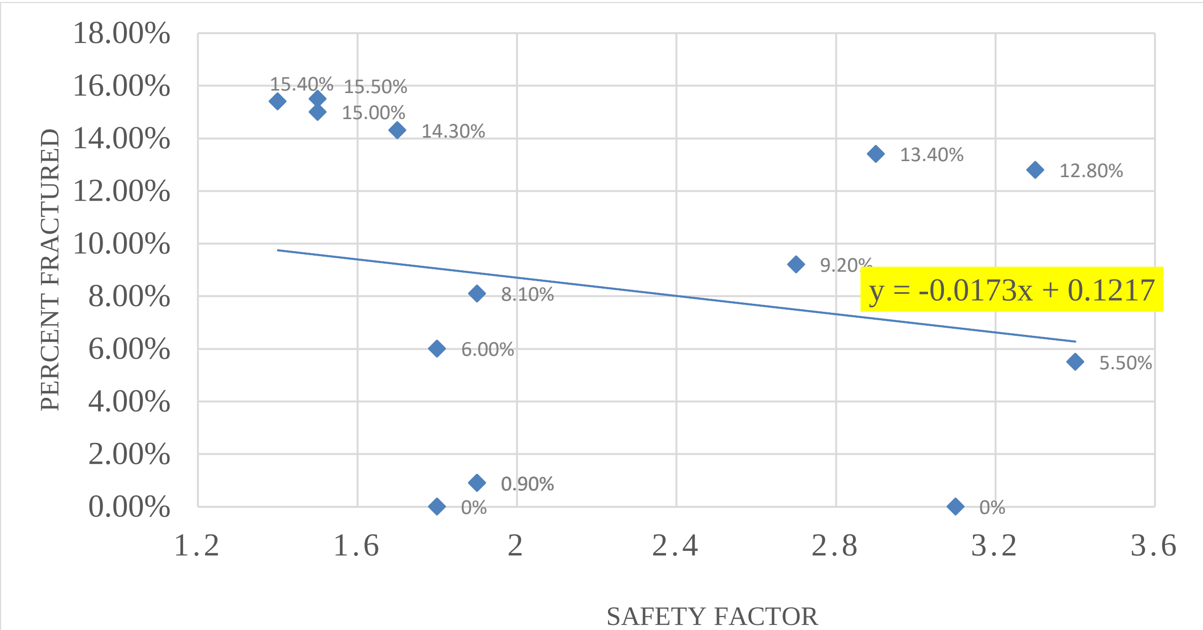


Figure 21: Safety performance of fractured joints

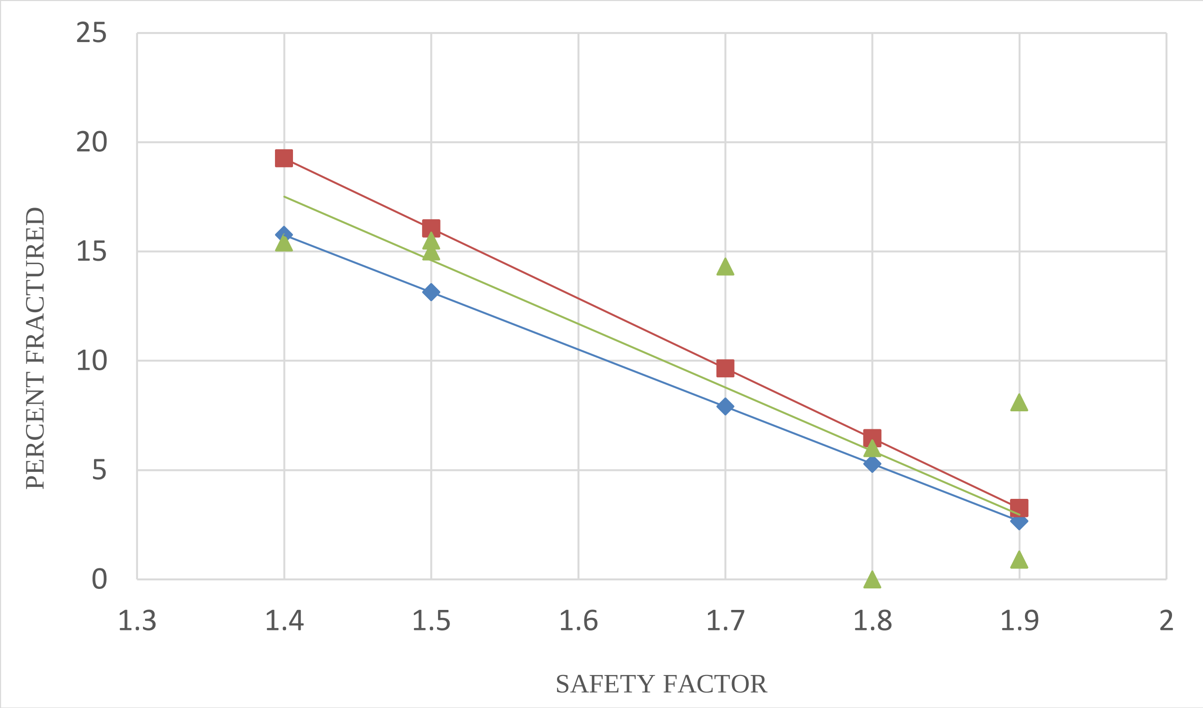


Figure 22: Safety performance of fractured joints with 10% variation

4. Discussions and Conclusions

- Fracture surface is completely rusted and a number of microcracks and corrosion pits are visible. Although some parts are obscured by corrosion, the fracture surface is chiefly that of intergranular fracture. This resembles the characteristics of a delayed fracture.
- Moreover, some of the measured values for Bolt C, one of the fractured bolts, exceed the standard upper limit (equivalent to the value of F11T). The measured values for Bolts A and B also are near the upper limit of the hardness standard for bolts in the strength category of 10.9.
- Furthermore, the tissue is uniform and has no defects, such as large inclusions, that could become the starting point of fracture.
- These findings indicate that the damage to the hexagonal bolts was caused by delayed fractures.
- A delayed fracture is a fracture that occurs without prior plastic deformation through the interaction of materials, stress, and environment. Bolts with higher strength generally have greater delayed fracture sensitivity.
- Although some of the damaged bolts in this investigation fall within the strength category of 10.9, the materials likely were exposed to a severe usage environment or had undergone excessive stress.
- Moreover, the damage did not occur in the regions in which stress tends to be concentrated, such as under the bolt head or at an incomplete thread, but in the cylindrical part of Bolt A and the threads of Bolts B and C. From the installed positions and the observed situations, stress likely was concentrated in the damaged areas.
- Based on analysis to the fractured HSBs there is no defect in raw material and other mechanism may take place and cause early fracture of HSBs in Ayeyarwady Bridge (Yadanarbon) such as corrosion due

to water entering, very corrosive environment, or vibration is so large and a fabrication defects and over-tightening in erection, the bolts are gradually loosened.

- Based on evaluation of safety factor of fractured joints, percent fractured relates to safety factor of joints and more fractured joints are occurred factor of safety less than 2. Safety factor of joints must be careful in connection design of steel bridge and safety factor should be at least 2.
- Fasteners with dry, dirty, and rusty thread surface conditions do not install properly and result in inadequate bolt tension. In addition, high-strength bolts should be stored properly to avoid rust and contamination. The torque wrench for tightening bolts needs to be calibrated accurately before use.
- Pitting corrosion may be found at the locations where water can accumulate thus installing high strength bolts in rainy season should be avoided.
- Inadequate design considerations, material problems, insufficient preloading, loosening and excessive loads can all contribute to fastener fatigue.
- Moreover, some corresponding preventative measures concerning aspects of material selection, construction, assembly, maintenance and management are proposed in this study, including, namely, making strict checks of the material's specification compliance, carefully controlling the quality of the installation and performing managed maintenance measures to ensure the safety of bolted joints.

References

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