CHAPTER 3

FAILURE ANALYSIS OF CHAURAS BRIDGE AND SUCCESS OF GARUDCHATTI BRIDGE: A CASE STUDY

3.1 GENERAL

In the past a number of steel truss bridges have failed during various stages of construction or service. The failures have been partial, or total collapses have taken place. The most common causes of bridge failure include: overstress of structural elements due to section loss, design defects and deficiencies, long-term fatigue and fracture, failures during construction, accidental impacts from ships, trains and aberrant vehicles, lack of inspection and unforeseen events. In case of truss bridges, failure of gusset plates connecting members of truss, and buckling failure of compression members are the most happening failures. Failure during construction is due to unexpected increased load on the bridge which many times might be beyond the scope of structural designer's knowledge.

Lessons from failures in past may be treated as learning experiences, because when a bridge collapses it might have certainly been pushed the existing knowledge to the limit in some way. Therefore, structural collapses in general, and particularly bridge collapses, which are often most spectacular, have a significant effect on the development of the knowledge of structural action and material behavior and have spurred research into particular fields.

Bridge collapse or collapse of any structure is either progressive or sudden. In progressive collapse one can judge probable failure of structure by inspection of various critical parts of the structure and can take preventive measures to fix the problems in the structure. But sudden collapse takes place without any warning and the collapse may occur within few seconds taking many lives and property loss with it.

3.1.1 Brief history of Failure of I-35W Bridge: A Steel truss bridge

One notable example of sudden collapse is collapse of I-35W bridge over the Mississippi River in Minneapolis, Minnesota on August 1, 2007 resulting in deaths of 13 people and injury to more than 100 others (M. Liao, *et. al.*, 2011). Figure 3.1 shows collapsed I-35W bridge (A. Astaneh-Asl, 2008).

Figure 3.1 View of the collapsed I-35W bridge (A. Astaneh-Asl, 2008)

The superstructure of the bridge consisted of two main longitudinal trusses continuous over three spans of 81m, 139m and 81m. The two longitudinal trusses were connected to each other with transverse trusses at each panel point. There were eight lanes of traffic on the bridge (A. Astaneh-Asl, 2008). The bridge underwent a number of repair and modifications during its service life. The reconstructions most significant to the collapse were conducted in 1977 and 1998, and involved increasing the thickness of the concrete deck from 6.5 to 8.5 inches, and the addition of new concrete parapets and guard rails. When the bridge was first opened for traffic, the concrete deck comprised 70% of the total bridge weight and the concrete added in later years increased the weight of the bridge by more than 30% and thus, represented a significant increase in demand on the structural components. In terms of a mental image, the addition of 2.0 inches of concrete to the deck was equivalent to doubling the weight of the steel (A. Astaneh-Asl, 2008).

Structural analysis of the I-35W Bridge determined that the members of the main truss had acceptable safety factors when they were designed. The capacity of the truss members was larger than the demands placed on them throughout the life of the bridge, including those on the day of the collapse. While many truss members fractured when they fell to the ground no evidence indicated failure of a truss member initiated collapse.

All joints of the bridge were connected by 1 inch thick gusset plates, except top chord joint U10, where $\frac{1}{2}$ inch thick gusset plates were used (Figure 3.2). Investigation and finite element analysis by many researchers concluded that, the undersized gusset plate at joint U10 was the cause of catastrophic and sudden failure (M. Liao, *et. al.,* 2011).

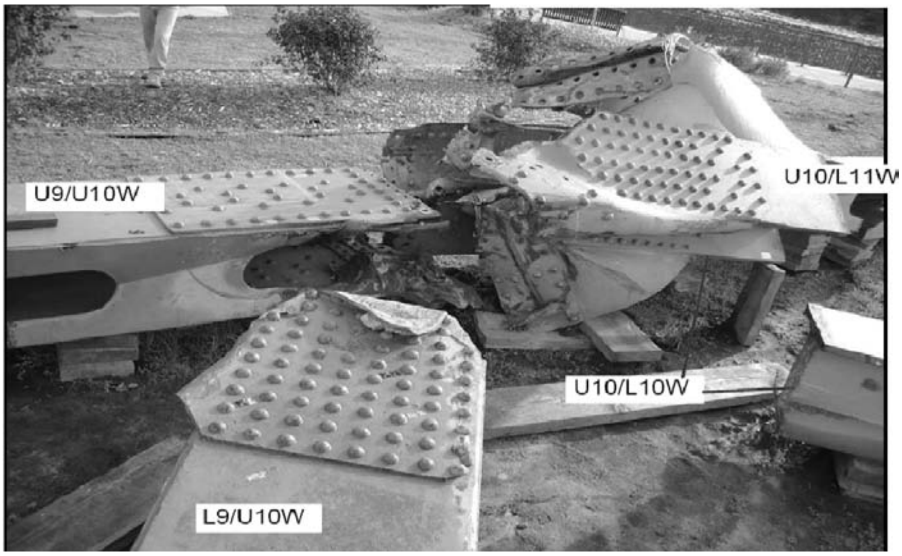


Figure 3.2 Failure of gusset plate at U10 (M. Liao, *et. al.,* 2011)

In the present chapter, the detailed analysis is carried out for two bridges: Chauras Bridge and Garudchatti Bridge at collapse and service conditions. Two continuous steel truss bridges of span 190m were constructed on the same design across river Alaknanda in Srinagar and Dugadda, respectively, in Uttarakhand state of India. Chaurasbridge collapsed during deck slab casting, whereas Garudchatti bridge was successfully constructed and opened to traffic. However, during traffic on the Garudchattibridge, the bridge experienced unacceptable vibrations and it was closed to traffic. Subsequently, strengthening and load testing of the bridge were taken up. Case studies of these two bridges are presented in detail in this chapter, and conclusions are drawn which are helpful in future design and construction of open web steel girder bridges.

3.2 FAILURE ANSLYSIS OF CHAURAS BRIDGE

Failure of 190m long Chauras Bridge in Uttarakhand, India, which was a three span (40m+110m+40m) continuous deck type truss bridge, took place during casting of the deck slab (Figure 3.3). The bridge was proposed to connect two cities namely, Srinagar on left bank and Chauras on right bank of the river Alakhnanda. After launching of the steel truss on two piers and two abutments, casting of deck slab was initiated on 24.03.2012 at 11.00 AM from midpoint of the 110m span of the bridge towards right pier. During deck slab concreting, when concrete was placed in 52.5m length from middle of the 110m span towards right pier, bridge suddenly collapsed calming six lives with it.

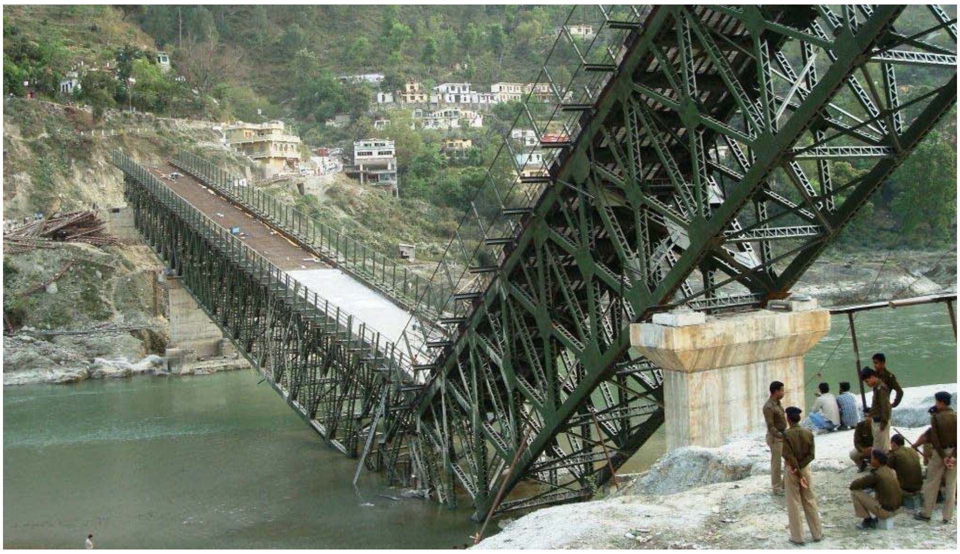


Figure 3.3 View of Chauras Bridge after failure

3.2.1 Geometry of the Chauras bridge

The 190m span lattice truss girder bridge was designed for 2-lanes having 7.5m wide carriageway and 1.5m wide footpaths on either side. Distance between top and bottom chord members was 8.66m, and centre to centre distance between the two trusses was 7.5m. It was divided into 38 panels of 5.0m length each. The bridge consisted of one central span of 110m and two end spans of 40m. Top and bottom chords of the bridge consisted of built up box sections, 500mm wide and 600mm deep, comprising four angles at four corners, and 2x575mm and 2x390mm wide four vertical plates. Diagonal and vertical members of the bridge consisted of channel sections and plates. Figure 3.4 shows elevation details of superstructure and substructure details of Chauras bridge.

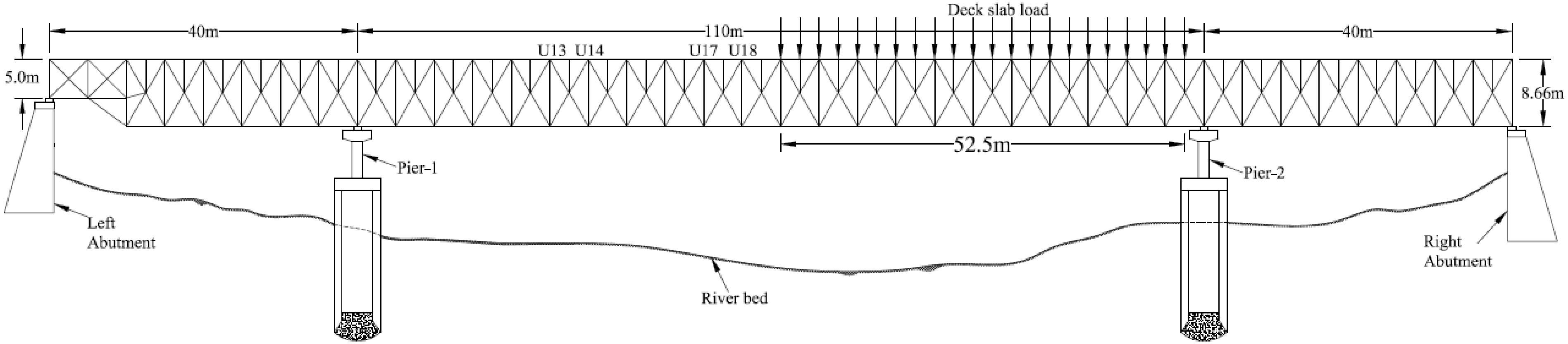


Figure 3.4 Arrangement of Chauras Bridge

3.2.2 Analysis of Chauras Bridge at collapse

A 3D space frame analysis of the Chauras bridge truss using STAAD Pro V8i software was carried out under the loadings existing at the time of collapse. Under self weight of the bridge there were no lifting reactions at abutment supports. During casting of the deck slab lifting of 40m end spans took place. Hence to analyze the bridge at the collapse stage, at abutment locations compression only spring supports were used. At the time of collapse, the bridge was subjected to following loadings.

Weight of steel truss	= 10000 kN
Weight of deck slab (52.5m)	= 2166 kN
Weight of formwork and equipments	= 2 kN/m^2 (assumed)

Under the above loading condition, member forces obtained from STAAD analysis are given in Table 3.1, and variation of axial stress is shown in Figure 3.5,

Table 3.1 Member forces as per STAAD analysis

Member	Cross sectional area (mm^2)	Axial force (kN)	Member stress (N/mm^2)	Permissible stress (N/mm^2)
U13U14	21596	3754.2	173.8	149.8
U14U15	30068	4619.7	153.6	149.7
U15U16	36740	5258.5	143.1	149.7
U16U17	43640	5707.9	139.7	149.7
U17U18	46508	5954.1	128.0	149.7
U18U19	51008	6000.1	117.6	149.7

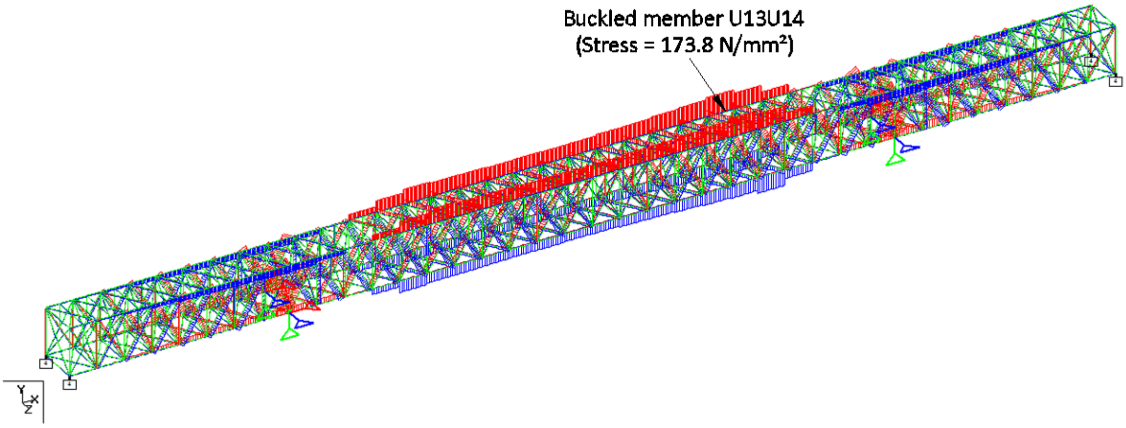


Figure 3.5 Stress diagram under loading at the time of collapse

From the analysis results using STAAD, it is found that the compressive stress in member U13U14 at the time of collapse was 173.8 N/mm^2, and maximum force in the upper chord members was 6000.1 kN in members U18U19.

Buckled view of member U13U14 that was taken place at collapse of Chauras bridge is shown in Figure 3.6. The overstressing of member U13U14 is also observed in the analysis.

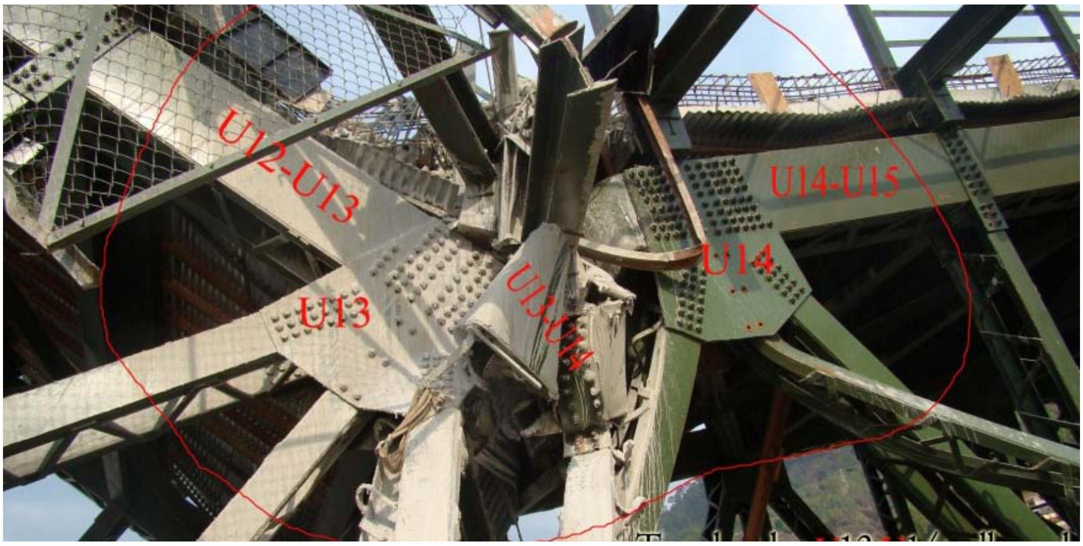
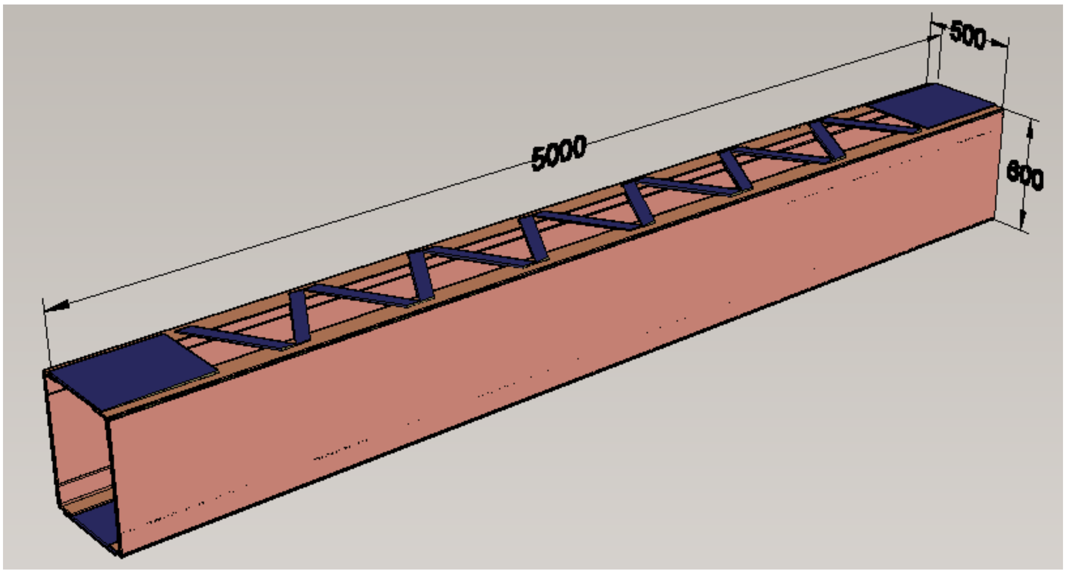


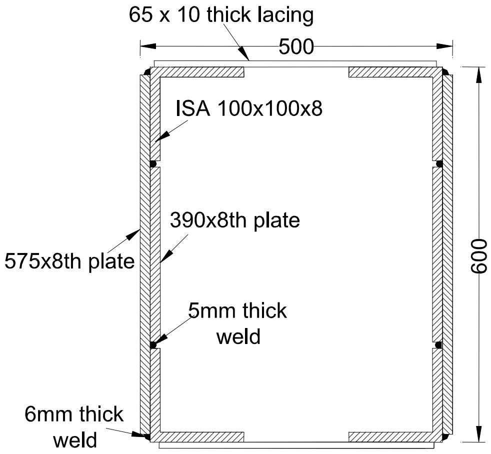
Figure 3.6 Joints U13, U14 and buckled member U13U14

3.2.2.1 Cross section and permissible stress of failed member U13U14

Top chord member U13U14 was designed as compression member. The 5.0m long U13U14 member was sub divided into two segments of each length 2.5m. Member U13U14 was made built up with four angle sections ISA 100 x 100 x 8 at four corners, two main side plates of size 575mmx8mm and two small plates of size 390mmx8mm. These plates and angle sections were connected together with continuous weld of size 6mm and lacing plates of size 65mmx10mm. Cross sectional details of the top chord member U13U14 are given in Figure 3.7. Effective length of the member is 2130mm and minimum radius of gyration is 192mm. Area moment of inertia about major and minor axis is $I_{zz} = 79516.1 \times 10^4$ mm^4 and $I_{yy} = 122057.9 \times 10^4$ mm^4.



a. Isometric view of member U13U14



b. Cross section of member U13U14

Figure 3.7 Member U13U14

Permissible stress calculation:

For member U13U14, permissible compressive stress in service condition as per IS: 800-2007 is given as below

$$\sigma_{cd} = 0.6\left(\frac{f_{cc}\times f_y}{\left((f_{cc})^n+(f_y)^n\right)^{\frac{1}{n}}}\right) = 149.8\ \text{N/mm}^2 \qquad \text{- working stress method}$$

Where, n = 1.4 and f_y = 250 Mpa

At limit state of strength, permissible compressive stress, as per IS: 800-2007 is;

$$\sigma_{cd} = \frac{\frac{f_y}{\gamma_{m0}}}{\phi+\left[\phi^2-\lambda^2\right]^{0.5}} = 224.7 \text{ N/mm}^2 \quad \text{- limit state of strength}$$

Where, $\phi = 0.5\left[1 + \alpha(\lambda - 0.2) + \lambda^2\right]$, $\lambda = \sqrt{\frac{f_y}{f_{cc}}}$, $f_{cc} = \frac{\pi^2 E}{\left(\frac{KL}{r}\right)^2}$

Elastic critical buckling stress for individual 8mm plates, as per equation 9-7 given by Timoshenko (S. P. Timoshenko, *et. al.*, 2010) is;

$$\sigma_{cr} = k\pi^2 \left(\frac{E}{12(1-\mu^2)}\right)\left(\frac{t^2}{b^2}\right) = 140.0 \text{ N/mm}^2$$

Where, t = 8mm and b = 575mm.

Permissible buckling stress for built up section of member U13U14 without fatigue is 224.7 N/mm^2 and it is 140.0 N/mm^2 for individual 8mm thick plates. Buckling stress of individual plates is less than the actual stress developed at failure (173.8 N/mm^2). Steel samples collected from the collapsed bridge were tested in laboratory and were found to be satisfactory. Therefore, it is concluded that buckling of member U13U14 caused the bridge collapse.

3.2.2.2 Buckling class of the member

Welded box sections, when used as a compression member, have limitations from individual component buckling (IS:800-2007). For welded built up box section (Figure 3.8) the limiting height to thickness $\left(\frac{h}{t_w} < 30\right)$ ratio and width to thickness $\left(\frac{b}{t_f} < 30\right)$ ratio are specified to avoid local buckling of components of compression member in IS: 800-2007 and IRC:24-2010. Minimum width to thickness ratio for built up box sections is limited to 30. In the case of member U13U14 the width to thickness ratio of individual 575mm wide, 8mm thick plates was 72, which was in far excess of the 30 limit. Therefore, buckling of individual 8mm plates may have initiated buckling of the member U13U14 leading to failure of the bridge.

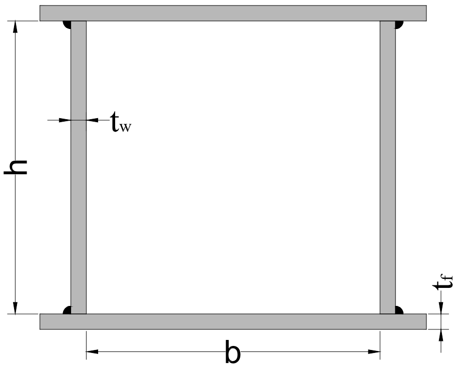


Figure 3.8 Built up box section (IS:800-2007)

3.2.2.3 Casting sequence of deck slab

The bridge was designed for 2-lanes of Class-A loading and on checking of the design it was found to be marginally unsafe. Successful completion of Garudchatti bridge on the same design also affirms it.

Casting of the deck slab of Chauras bridge started from middle of the 110m span, which caused lifting of the 40m end span as seen in Figure 3.3. The ideal casting procedure for the deck slab would have been to start from the end spans and proceed towards the mid span. Proper deck casting procedure might have saved the bridge during deck casting, but the bridge would have remained highly vulnerable under service and overload conditions.

3.3 STRENGTHENING OF GARUDCHATTI BRIDGE

On the same design as 190m span collapsed Chauras bridge, Garudchatti bridge was also constructed at Garudchatti in Uttarakhand state of India (Figure 3.9), and it was successfully completed and opened to traffic. After collapse of Chauras bridge, and due to unacceptable vibrations in the bridge, it was closed to traffic and its strengthening was taken up.

3.3.1 Analysis of Garudchatti bridge

A 3D space frame analysis of Garudchatti bridge truss using STAAD Pro V8i software was carried out under self weight of steel truss, deck slab, superimposed dead load (SIDL), foot path live load (FPLL) and live load (LL) due to four trains of 2-Lane Class-A vehicles. The bridge model adopted for the analyses is based on Figure 3.9. The bridge was analyzed for the following loadings for DL+LL case and 1.2x(DL+LL) case.

Weight of steel truss = 10000 kN (As per estimate)

Weight of deck slab 220 mm thick deck slab = 5.5 kN/m^2

Weight of cantilever 150mm thick foot path slab = 3.75 kN/m^2

Weight of 65mm thick wearing coat = 1.43 kN/m^2

Foot path live load (FPLL) = 1.65 kN/m^2

Live load due to four trains of 2-Lanes Class-A vehicle is taken as per IRC:6-2010.

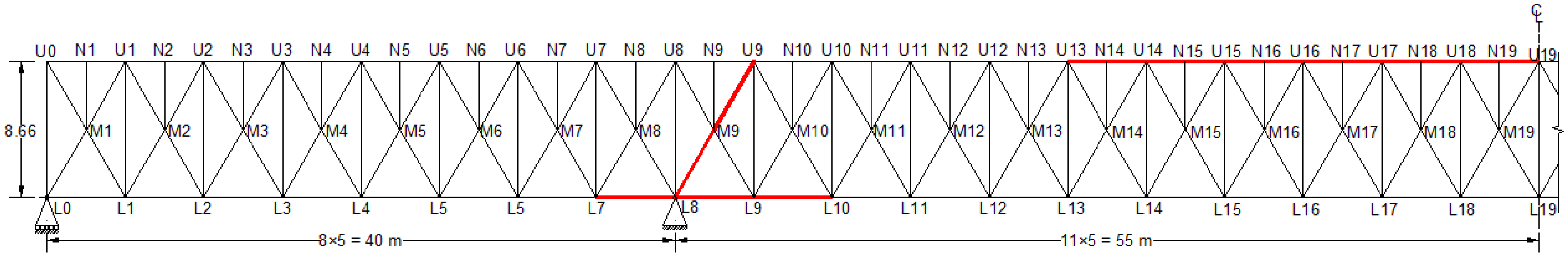


Figure 3.9 Half elevation of Garudchatti bridge

Analysis results in service condition and 1.2x(DL+LL) cases are given in Table 3.2.

Table 3.2 Cross sectional area of critical members and member stresses in the bridge

Member no.	Cross sectional area (cm^2)		Member stresses (N/mm^2)						
			For (DL+LL) case without lifting			(DL+LL) case		1.2x(DL+LL) case without lifting	
	Before strengthening	After strengthening	Self weight + Deck + SIDL	FPLL	Maximum LL condition	With lifting	Without lifting	Before strengthening	After strengthening
U13U14	215.96	471.16	141.2	3.7	42.2	240.7	187.1	224.5	110.8
U14U15	360.68	615.88	117.0	3.0	32.0	183.9	152.0	182.4	110.8
U15U16	367.4	622.60	137.8	3.6	36.2	208.4	177.6	213.1	129.7
U16U17	436.4	691.60	131.5	3.4	33.6	197.1	168.5	202.2	131.5
U17U18	465.08	720.28	132.9	3.5	33.6	198.1	170.0	204.0	135.1
U18U19	510.08	765.28	125.6	3.3	31.5	186.0	160.4	192.5	131.4
L7L8	587.4	842.6	111.2	2.9	29.9	130.6	144.0	172.8	94.8
L8L9	488.0	743.2	109.2	2.8	30.2	123.3	142.2	170.6	94.3
L9L10	389.8	645.0	104.6	2.7	29.6	130.4	136.9	164.3	78.2
L8U9	294.92	402.2	94.8	2.5	33.1	163.9	130.4	156.5	137.2

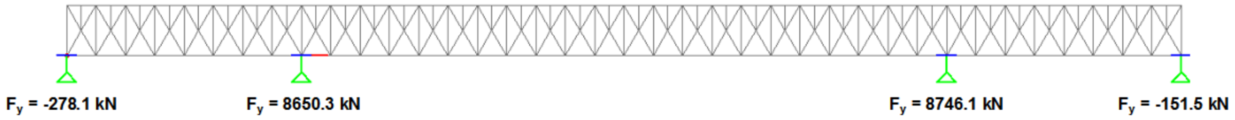


Figure 3.10 Support reactions in service condition

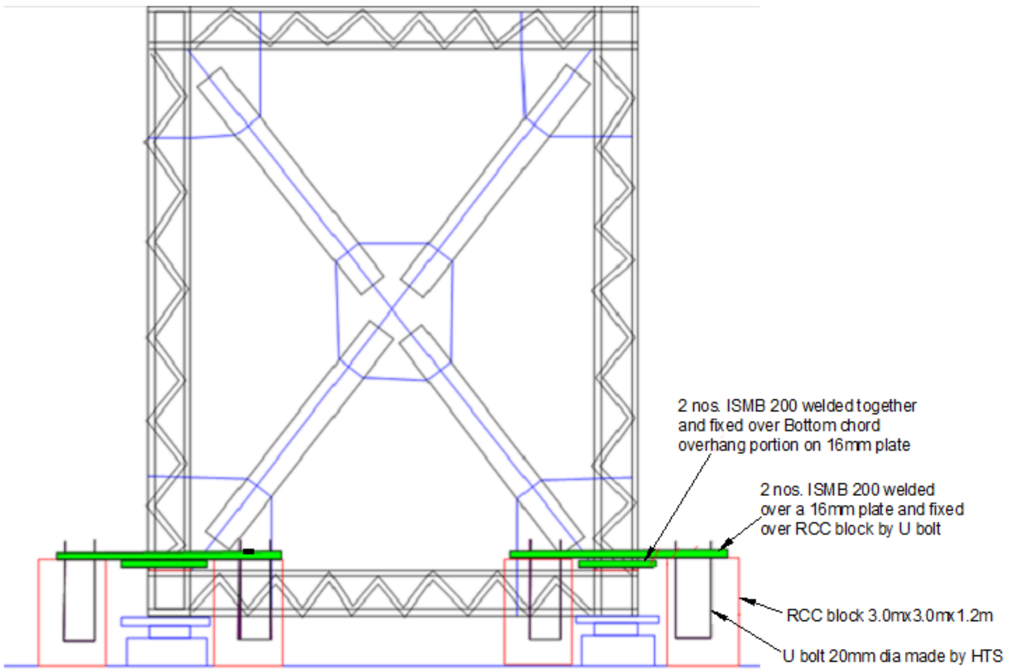


Figure 3.11 Arrangement of anchorage blocks at abutment locations

Figure 3.10 gives support reactions in service condition. There is an uplift of 278.0 kN at the end supports, for which anchor weights of RCC are designed, details of which are shown in Figure 3.11.

3.3.2 Strengthening details of critical members

In order to ensure safety of the bridge, particularly during load testing, strengthening of the critical members (Table 3.2) was carried out for 1.2x(DL+LL) case. Additional area required for strengthening of the compression members was calculated for excess stress beyond the permissible stress in the members, details of which are given in Table 3.3.

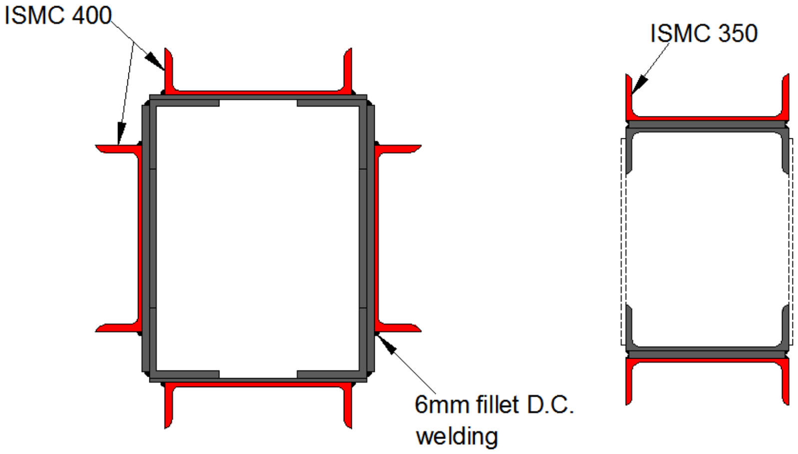
Table 3.3 Strengthening details of critical members

Member	Cross section details					Existing cross sectional area (cm^2)	Stress in member under 1.2x(DL +LL) (N/mm^2)	Permissible stress (N/mm^2)	Required cross sectional area (cm^2)	Extra cross sectional area provided (cm^2)	Total cross sectional area after strengthening (cm^2)
U13 U14	a	Plate 390 x 8th	2	No.	Existing	215.96	224.5	148.8	325.8	255.2	471.16
	b	ISA 100 x 100 x 8	4	No.							
	c	Lacing not considered in X/S area									
	d	Plate 575 x 8th	2	No.							
	e	**ISMC 400**	4	No.	Extra provision to achieve x/s area as required						
U14 U15	a	Plate 290 x 12th	2	No.	Existing	360.68	182.4	148.8	442.1	255.2	615.88
	b	ISA 150 x 150 x 12	4	No.							
	c	Lacing not considered in X/S area									
	d	Plate 575 x 8th	2	No.							
	e	**ISMC 400**	4	No.	Extra provision to achieve x/s area as required						
U15U16	a	Plate 290 x 16th	2	No.	Existing	367.4	213.1	148.8	526.2	255.2	622.6
	b	ISA 150 x 150 x 16	4	No.							
	c	Lacing not considered in X/S area									
	d	Plate 575 x 8th	2	No.							
	e	**ISMC 400**	4	No.	Extra provision to achieve x/s area as required						
U16 U17	a	Plate 290 x 16th	2	No.	Existing	436.4	202.2	148.8	593.0	255.2	691.6
	b	ISA 150 x 150 x 16	4	No.							
	c	Lacing not consider in X/S area									

	d	Plate 575 x 14th	2	No.							
	e	**ISMC 400**	4	No.	Extra provision to achieve x/s area as required						
U17 U18	a	Plate 190 x 16th	2	No.	Existing	465.08	204	148.8	637.6	255.2	720.28
	b	ISA 200 x 200 x 16	4	No.							
	c	Lacing not consider in X/S area									
	d	Plate 575 x 6th(2 No)& 550 x 8th(2 No)	4	No.							
	e	**ISMC 400**	4	No.	Extra provision to achieve x/s area as required						
U18 U19	a	Plate 390 x 8th	2	No.	Existing	510.08	192.5	148.8	659.9	255.2	765.28
	b	ISA 100 x 100 x 8	4	No.							
	c	Lacing not considered in X/S area									
	d	Plate 575 x 8th	2	No.							
	e	**ISMC 400**	4	No.	Extra provision to achieve x/s area as required						
L7L8	a	Plate 190 x 20th	2	No.	Existing	587.4	172.8	148.8	682.1	255.2	842.60
	b	ISA 200 x 200 x 20	4	No.							
	c	Lacing not consider in X/S area									
	d	Plate 575 x 6th + pl.550 x 6th	2	No.							
	e	**ISMC 400**	4	No.	Extra provision to achieve x/s area as required						
L8L9	a	Plate 190 x 16th	2	No.	Existing	488	170.6	148.8	559.5	255.2	743.20
	b	ISA 200 x 200 x 16	4	No.							
	c	Lacing not consider in X/S area									
	d	Plate 575 x 8th + pl.550 x 8th	2	No.							
	e	**ISMC 400**	4	No.	Extra provision to achieve x/s area as required						
L9L10	a	Plate 290 x 12th	2	No.	Existing	389.8	164.3	148.8	430.4	255.2	645.00
	b	ISA 150 x 150 x 12	4	No.							
	c	Lacing not consider in X/S area									
	d	Plate 575 x 10th + pl.550 x 6th	2	No.							
	e	**ISMC 400**	4	No.	Extra provision to achieve x/s area as required						
L8U9	a	Plate 335x 28th	2	No.	Existing	294.90	156.50	148.80	310.2	255.2	550.10
	b	ISMC 350	2	No.							
	c	Lacing not consider in X/S area									
	e	**ISMC 400**	4	No.	Extra provision to achieve x/s area as required						

Top chord compression members U13U14 to U18U19, bottom chord compression members L7L8 to L9L10 and diagonal member L8U9 (Figure 3.9) were found to be over stressed in (DL+LL) case (Table 3.2). In order to prevent collapse of the bridge due to sudden buckling of the compression members during load testing, full strengthening of the compression members was carried out for 1.2x(DL+LL) load case.

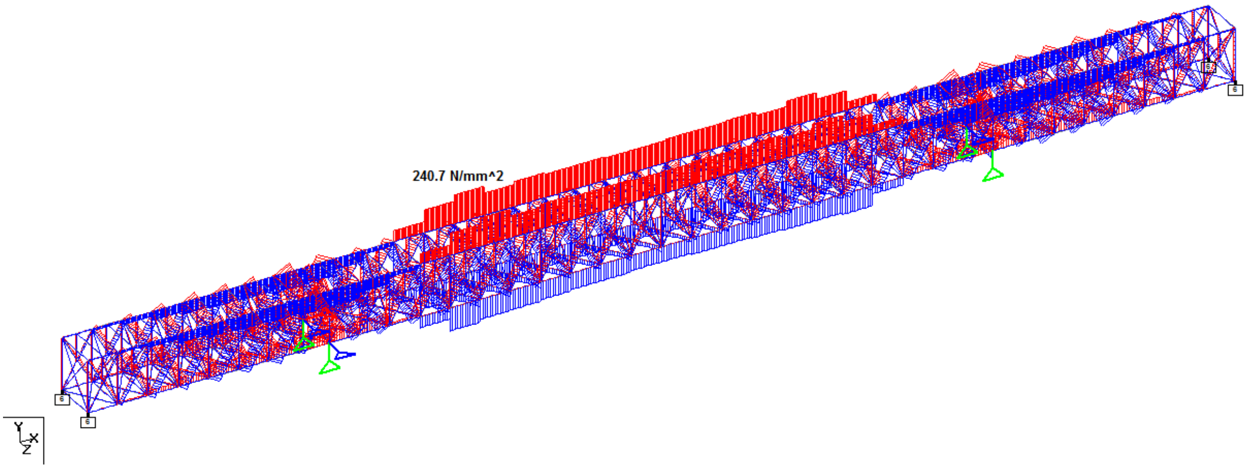
The overstressed top and bottom chord compression members, for 1.2x(DL+LL) case, were strengthened by welding four ISMC 400 channel sections, and diagonal member L8U9 was strengthened by welding two ISMC 350 channel sections to the existing sections as shown in Figure 3.12.



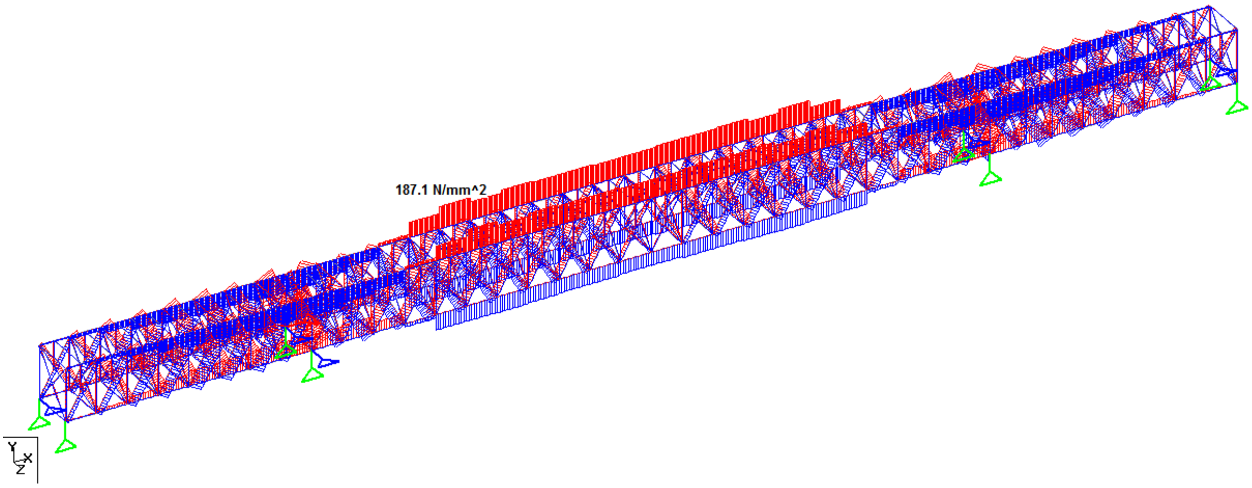
a. Top and bottom chord members b. Member L8U9

Figure 3.12. Strengthening details

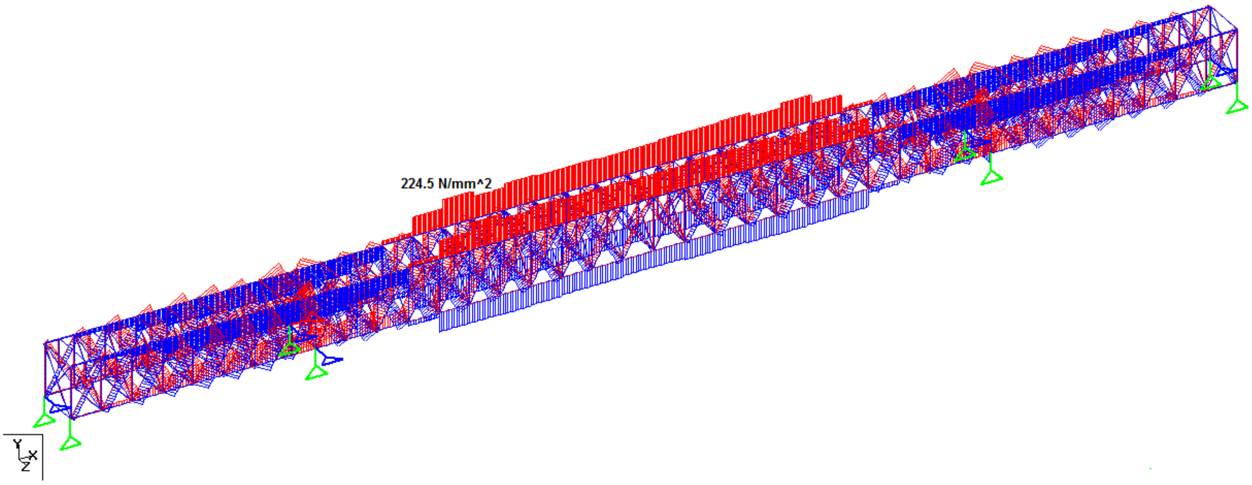
For comprehensive comparison, member stresses for the above different cases, with maximum value of stresses marked, are shown in Figure 3.13. Red contour in stress diagram represents compressive stress and blue contour represents tensile stresses.



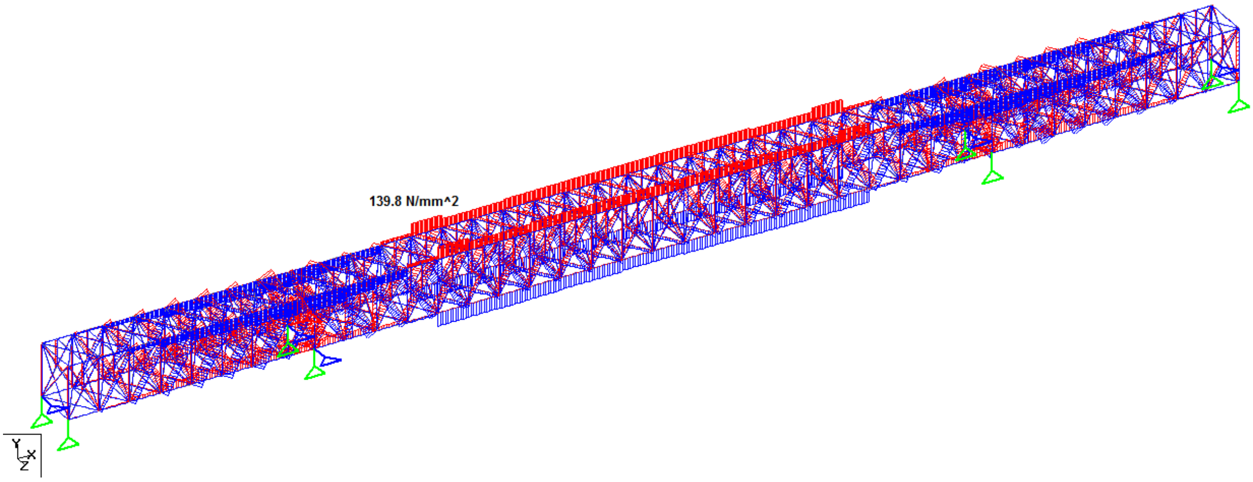
a. (DL+LL) case with lifting at end supports



b. (DL+LL) case without lifting at end support



c. 1.2(DL+LL) case before strengthening without lifting at end supports



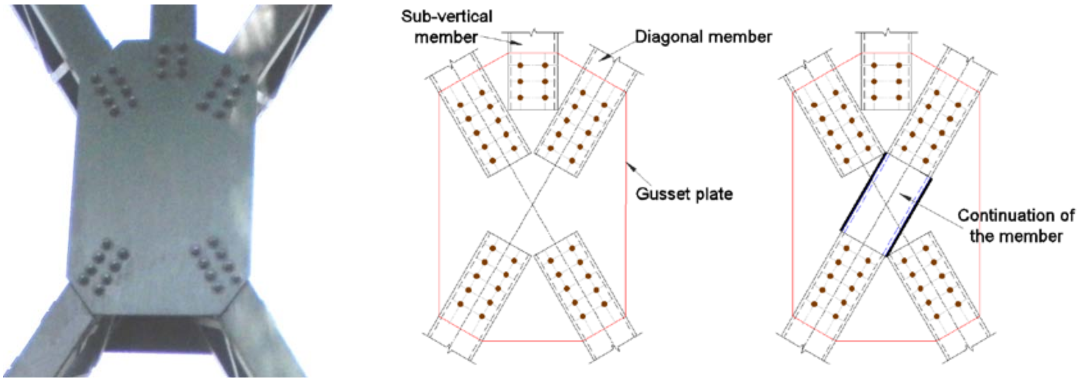
d. 1.2(DL+LL) case after strengthening without lifting at end supports

Figure 3.13. Member Stresses in the bridge

It is seen from Table 3.2 and Figure 3.13.a that compressive stress is maximum (240.8 N/mm^2) in member U13U14 under critical (DL+LL) case with girder lifting at the end support. This stress is in far excess of the permissible stress of 149.8 N/mm^2. Therefore, it was necessary to provide suitable anchorages at the end supports to prevent their lifting.

3.3.3 Continuation of diagonal compression member at M-Joints

At all M-Joints of the trusses, all meeting members were cut and riveted to the gusset plate (Figure 3.14). Diagonal members of Garudchatti bridge were discontinuous at the M-joints (Figure 3.14.a). Due to this discontinuity major portion of gusset plate connecting the diagonal members was unsupported and susceptible to buckling. Therefore, the diagonal members under compression at the M-Joint were made continuous (Figure 3.14.c).



a. M-Joint b. before strengthening c. after strengthening

Figure 3.14 Discontinuity of diagonal members at M-joints

3.3.4 Load testing of Garudchatti bridge

After strengthening of the bridge its load testing was carried out as per IRC: SP: 37-1991 and IRC:SP: 51-1999 provisions, details for which are given in Figure 3.15.

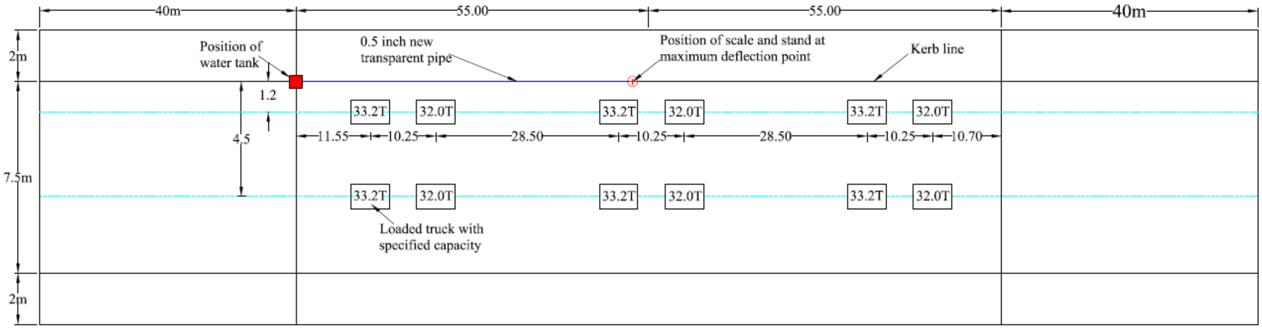
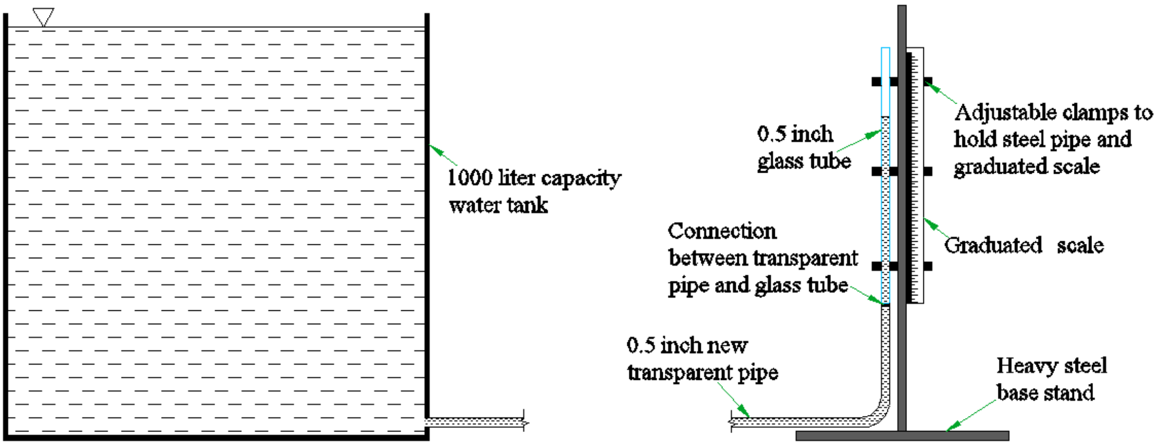


Figure 3.15 Loading arrangement

Deflection of the bridge at the middle of 110m span was taken with the help of water column arrangement shown in Figure 3.16.



a. Tank position over the pier

b. Water column measurement arrangement at the middle of 110m span

Figure 3.16 Arrangement for measurement of deflection

The bridge with loaded trucks as per loading shown in Figure 3.15 is seen in Figure 3.17.

Figure 3.17 Garudchatti bridge over Alaknanda river, India

Fifteen loaded trucks of requisite weights were deployed for gradual loading of the bridge as per detailed procedure given below.

i. Load positions for 2-lanes of IRC Class-A vehicles with impact for maximum deflection at the middle point of 110m span is worked out from STAAD Pro. software and is given in Figure 3.15. Maximum deflection obtained from

STAAD Pro. for live load with impact only was 42.5mm against permissible deflection of 137.5mm (L/800).

ii. Fifteen loaded trucks of the requisite weights (Table 3.4) were gradually placed on the bridge deck as per loading positions given in Figure 3.15.

Table 3.4 Weight of loaded trucks

Sr.No	Truck detail	Weight of trucks	Total weight
1	UA 07C 7795	26.5 T	130.10 T Load at RHS (Bharmpuri side)
2	UA07C 7799	25.3 T	
3	UA07A 6837	26.7 T	
4	UA07C 7953	25.7 T	
5	UK07C 1737	25.9 T	
6	UA07C 7862	25.1 T	127.20 T Load at middle location.
7	UK07C 4708	27.0 T	
8	UK07C 1008	24.6 T	
9	UK07CC 2007	25.0 T	
10	UA07C 8304	25.5 T	
11	UA07C 3051	26.1 T	128.90 T Load at LHS (Garudchatti Side)
12	UA07CA 0766	26.0 T	
13	UK07C 7325	25.6 T	
14	UK07 CC 1520	25.4 T	
15	UA07C 9971	25.8 T	
Total load =			386.20 T

iii. The bridge was loaded on 31/05/2013 at 11.00 AM and deflections were measured as per IRC:SP: 37-1991 and IRC:SP: 51-1999, and the summary is given in Table 3.5.

iv. Recovery in deflections was checked as per acceptance criteria given under cl.5.2 of IRC:SP:51-1999.

Percentage recovery of deflection at 24 hours after removal of loading;

= (37/40x100) = **92.5 %** > **85%**

Since the recovery in deflection was more than 85% stipulated in the code, performance of the bridge after strengthening was satisfactory.

Table 3.5 Summary of deflections

Date	Time	Load at LHS (Garudchatti Side)	Load at middle location.	Load at RHS (Brahmapuri side)	Deflection
31.5.2013	11.00 A.M.	Initial deflection			0 mm
	11.35 A.M.		127.20 T		17 mm
	11.55 A.M.	128.90 T	127.20 T	130.10 T	30 mm
1.6.2013	12.00 A.M.(mid night)	128.90 T	127.20 T	130.10 T	34 mm
	10.00 A.M.	128.90 T	127.20 T	130.10 T	40 mm
	10.30 A.M.	After unloading			8 mm
	10.30 P.M.				6 mm
2.6.2013	12.00 P.M.	After 24 hours of unloading			3 mm

3.3.5 Precautions in load testing

During load testing of a steel truss bridge, the bridge is loaded to full live load with impact as per cl. 5.2 of IRC: SP: 51-1999. The bridge is also designed for this load. Therefore, if there is any shortcoming in the design or overloading during load testing, compression members of the truss may suddenly collapse. Therefore, it is not advisable to load test steel truss bridges, design for which has not been checked at the limit state of strength with adequate Load Factor.

3.4 DISCUSSION ON CHAURAS AND GARUDCHATTI BRIDGES

As per BS: 5400-1978 Part II, load factor for serviceability limit state over nominal load which has a return period of 120 years, is given as 1.05 and for ultimate limit state, it is specified as 1.75. Thus, for design of steel bridges the ultimate limit state is also covered in the code. As per AASHTO LRFD Bridge Design Specifications, separate design calculations are required for Allowable Stress Design (ASD) and Load Factor Design (LFD). In India, limit state of strength design is specified for steel truss bridges in IRC:24-2010. But, design of steel truss bridges in India is still widely based on working stress method.

3.4.1 Critical discussion on failure of Chauras bridge

In the past a number of bridges have failed due to buckling, details of which have been presented under literature review. Notable among these is the Quebec truss bridge where the main cause of failure was increase in dead load and lack of knowledge of behavior of compression members. Failure of Chauras bridge also took place due to buckling of its compression members U13U14 during casting of the deck slab. Had the bridge not collapsed at this stage due to marginal design of the bridge for service condition, it might have collapsed on completion in future during more severe loading condition, claiming more lives with it. Therefore, checking of the bridge design at limit state of strength with appropriate additional load factor for maximum possible loading condition during life time of the bridge is necessary.

Buckling of compression members

Structural steel of grade E250 used in Chauras bridge had ultimate tensile strength (f_u) of 410N/mm^2 and yield strength (f_y) of 250 N/mm^2. Permissible tensile stress in service condition for mild steel as per Indian standards is 0.6 f_y (=150 N/mm^2). For slenderness ratio less than 10 maximum permissible compressive stress is also 0.6 f_y (=150 N/mm^2), which decreases with increase in the slenderness ratio. Similarly, for E410 grade of steel yield strength is 410 N/mm^2 and ultimate tensile strength is 540 N/mm^2. Permissible tensile stress and maximum permissible compressive stress for this steel is also 0.6 f_y (Figure 3.18).

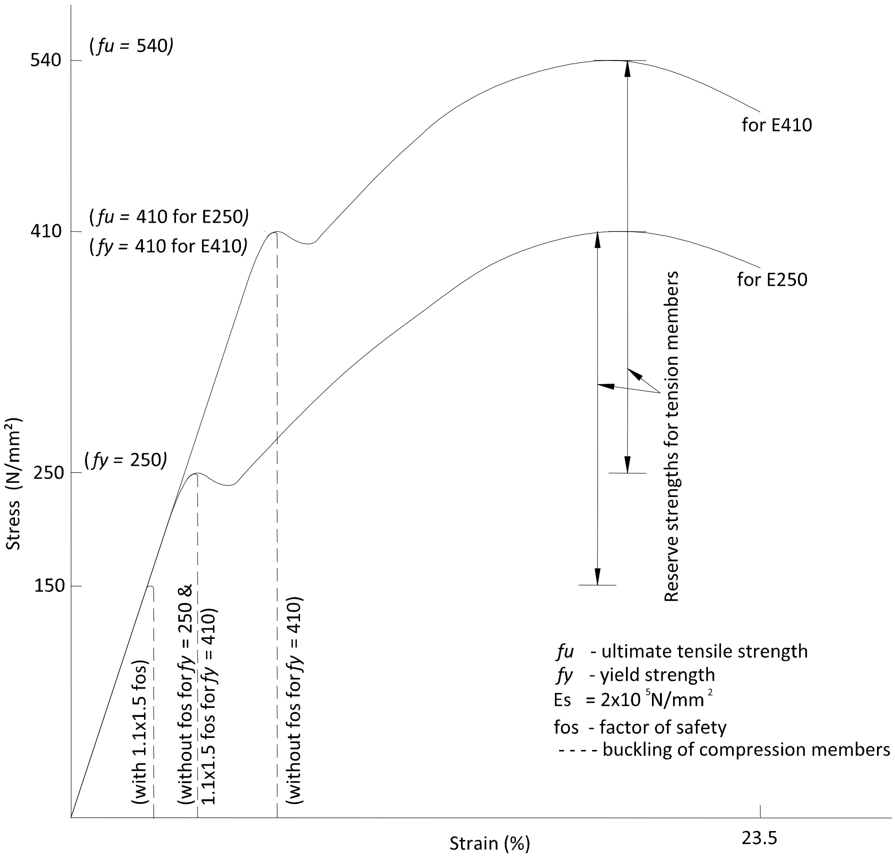


Figure 3.18 Stress-Strain curves of compression or tension member for mild steel of grades E250 and E410

Tension and compression members of a steel truss have entirely different behaviour before failure (Figure 3.18). Compression members suddenly buckle and fail without any reserve strength in them beyond maximum buckling stress, while tension members have reserve strength after yielding up to the ultimate tensile strength.

Behavior of gusset plates

Maximum compressive force in the member U18U19 of Chauras bridge at collapse stage was 6000.1 kN and corresponding stress in the connecting 12mm thick gusset plate was 416.7 N/mm^2. But the gusset plate didn't fail even at such a high compressive stress, as it was prevented against buckling by the rivets.

Compressive force in member U13U14 was 3754.2 kN and corresponding stress in the gusset plate was 260.7 N/mm^2. Gusset plates at joints U13 and U14 remained intact at this high stress, whereas, member U13U14 buckled and failed at a lower stress of 173.8 N/mm^2 as shown in Figure 3.19.

Figure 3.19 Intact gusset plates at joints U13 and U14

Thus, gusset plates if connected properly to the members and prevented from buckling can take compressive or tensile stress up to the ultimate strength of the plate.

3.4.1.1 Design at Limit State of Serviceability

Generally steel bridges are designed for 100 years of service life and six million fatigue load cycles.

Fracture endurance limits for E250 (fy=250 N/mm^2, fu=410 N/mm^2) and E410 (fy=410 N/mm^2, fu=540 N/mm^2) mild steel are shown in Figure 3.20. Endurance limit for yielding is proportionately scaled down from the fracture endurance limit curve. Therefore, whereas fracture endurance limit for six million cycles is taken as $\frac{2}{3}fu$ (= 273 N/mm^2), endurance limit for yielding is $\frac{2}{3}fy$ (= 167 N/mm^2). Adapting material safety factor as 1.1, permissible stress in compression or tension for the limit state of serviceability is adopted as 0.6fy as also specified in IS: 800 -2007.

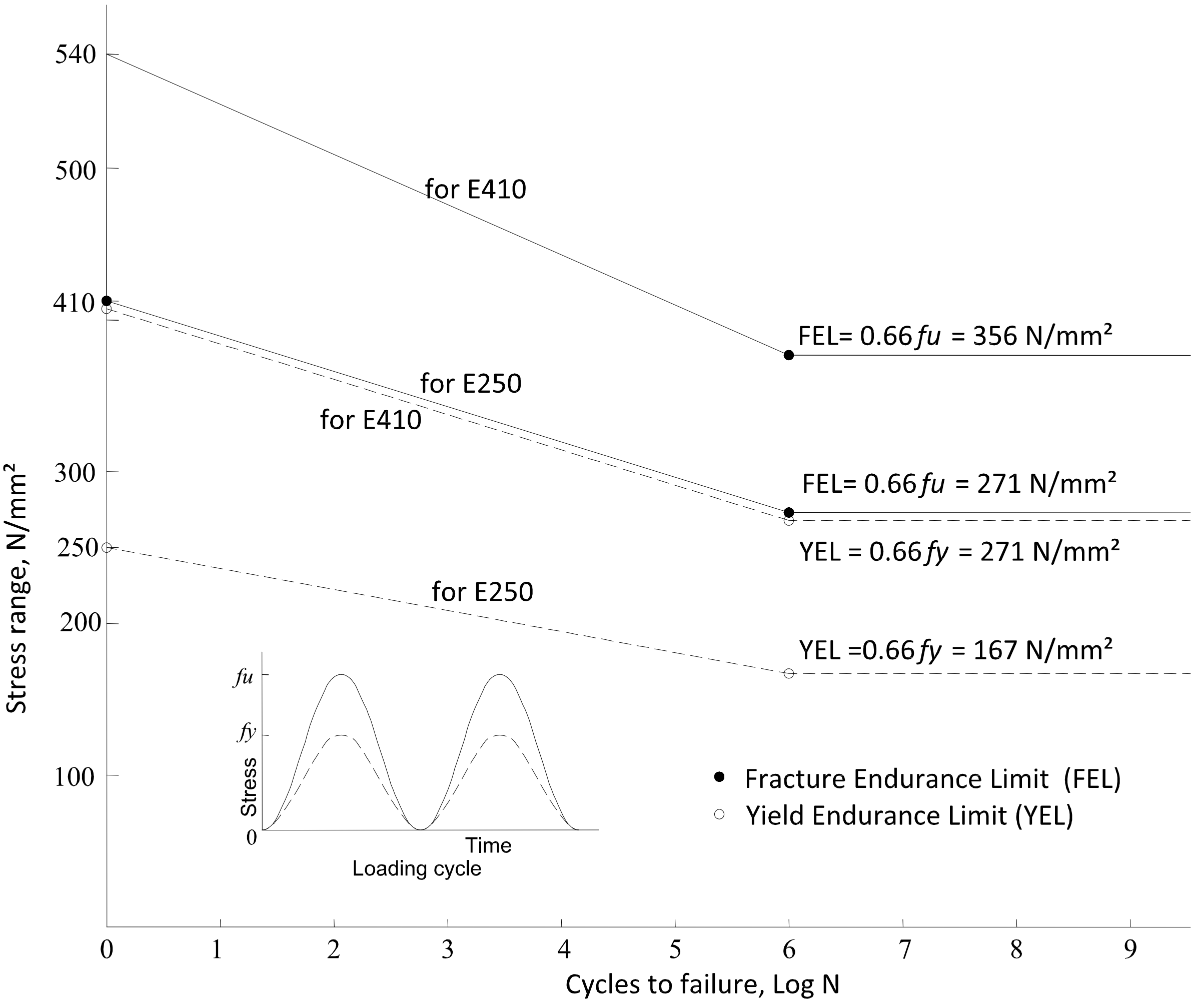


Figure 3.20 Endurance limits for E250 and E410 grade steel

In a steel truss bridge, in service condition, maximum permissible member stresses, from deformation criterion, for E250 and E410 grade steel shall be limited to their yield stresses, after which unacceptable plastic deformation will occur.

Design of truss members

As evidenced in Chauras bridge case, sudden collapse of the bridge took place due to buckling of top chord compression members U13U14 without any warning and therefore, design of tension and compression members of a truss require separate design considerations.

a. Design of tension members

Tension members designed as per ultimate tensile strength will have excessive deformation due to yielding. Therefore, permissible fatigue stress in service condition of 167N/mm^2 for E250, and 271N/mm^2 for E410 as may be adopted for design.

Thus, in the limit state of serviceability condition, adapting 1.1 material safety factor and 1.5 fatigue factor, tension members should be designed for permissible stress of 0.6fy (= 0.66fy/1.1).

b. Design of compression members

Referring to Figure 3.18 and adapting 1.1 material safety factor and 1.5 fatigue factor, maximum permissible compressive stress in the serviceability condition for compression members of grade E250 and E410 may also be adopted as 0.6fy (IS: 800-2007).

3.4.1.2 Recommended design at Limit State of Strength

Unexpected circumstances may take place during construction and service stages of the bridge. Apart from uncertainty in the material strength for which material safety factor of 1.1 is generally adopted, the following uncertainties affecting the safety of the bridge may be there;

a. Uncertainty about loading
b. Uncertainty about structural dimensions and behavior.

a. Uncertainty about dead load

Dead load (DL) on the bridge may increase even after its construction due to time to time repair works. In the case of I-35W bridge, due to two major deck repairs the dead load of the bridge increased by 30% (NTSB-Dockets, 2007-08). Therefore, safe increased dead load on a bridge during its lifetime may be taken up to 1.5 times its dead load.

b. Uncertainty about live load

Bridge loading standards provide for specified gap between two trains of vehicles. In adverse conditions these gaps may also be occupied by vehicles.

In order to calculate increase in live load (LL) due to full occupancy of the deck, 7.5m wide carriageway with two lanes of Class-A train of vehicles (IRC: 6-2010), having total plan area of 18.8mx5.5m and nose to tail spacing of 20.0m is considered. The remaining 2.0m width of the deck is considered loaded with 5.0 kN/m^2 other live loads (Figure 3.21).

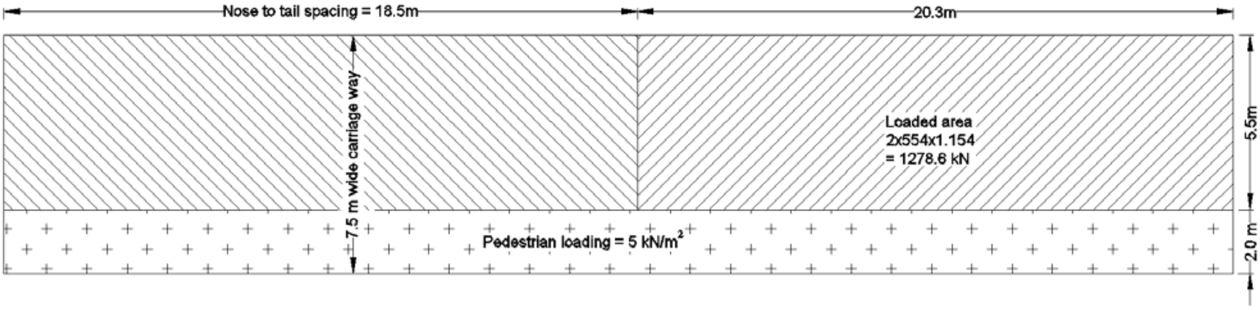


Figure 3.21 Arrangement of vehicles for fully loaded deck condition

Thus, maximum possible live load (without impact) to normal live load (with impact) ratio;

$$= [2\times554(1 + 18.5/20.3) + 5.0\times2.0\times38.8] / (2\times554\times1.154) = 1.96$$

Vehicle loads keep on increasing as the years pass and consequently the loading standards are revised. Corrosion and wear and tear of the bridge also require higher load factor.

Design load at limit state of strength

As per IRC:6-2014, Table 3.2, load factor for DL, snow load and SIDL is 1.35 and for surfacing it is 1.75. For LL a load factor of 1.5 is given.

As presented above, an additional load factor of 1.5 for DL and 1.96 for LL is required for the limit state of strength condition. In addition to this further increased load factor is advisable for ever increasing loading standards, and corrosion and wear and tear of the bridge. In case of composite steel truss bridges, lateral restraint to top chord members is provided and therefore, only web compression members remain laterally unsupported. Thus, in case of overload only these members remain critical and their cross sectional areas have to be suitably increased for the overload condition, which is not significantly high. Therefore, a higher load factor at the limit state of strength does not cause significant increase in the bridge cost. A higher load factor would also cause the bridge to deform before failure and eliminate the chances of sudden failure under overload condition. Thus, an overall load factor at the limit state of strength of 2.25 is recommended both for DL and LL.

Permissible stress as per IRC:24-2010 in the service condition is $= \dfrac{f_y}{\gamma_m \gamma_f}$

And for limit state of strength condition it is $= \dfrac{f_y}{\gamma_m}$

Where, γ_f is the fatigue factor taken as 1.5.

To account for this in the design in service condition, the interaction ratio for laterally unsupported web compression members may be limited to 0.66.

Design of tension members

Design of tension members in service condition for (DL+LL) case, including 1.1 material safety factor and 1.5 fatigue factor in the permissible stress, is adequate as reserve strength for tension members is sufficient (Figure 3.10).

Design of tension members for service condition is adequate at the limit state of strength also, as fatigue factor of 1.5 is absent in this case and in its place a load factor of 1.5 has to be applied.

Design of compression members

Compression members buckle and suddenly fail without warning causing loss to life and property, and consequently an additional load factor of 1.5 is required at the limit state of strength. Therefore, in the case of composite steel truss bridge design at the limit state of strength, laterally unsupported web compression members may be designed for 2.25x(DL+LL) case.

In case lateral buckling of compression members is prevented, as in the case of top chord compression members in composite deck system, no additional load factor is required for the limit state of strength, as the compression members will continue to take load up to their ultimate strength, and load factor of 1.5 is sufficient.

Design of truss joints

Buckling of the gusset plates at the truss joints is effectively prevented by the weld, rivet or bolt, and therefore, in the limit state of strength condition no additional load factor is required, and the design for serviceability condition is adequate.

3.4.2 Discussion on strengthening and load testing of Garudchatti bridge

The design of Chauras and Garudchatti bridges was only marginally unsafe leading to collapse of one bridge and successful completion of the other. Checking of the design of the bridge at the limit state of strength with appropriate load factor, would have resulted in safe design for the two bridges.

3D space frame analyses of the bridge were carried out to access its weaknesses and to find remedial measures. Accordingly, critical members of the bridge were strengthened. After strengthening, load testing was also carried out to ensure safety of bridge in service condition. During load testing maximum recorded deflection under live load alone was 40.0mm, which was lower than 137.5mm permissible deflection. Also, recovery in deflection after 24 hours of load removal was 92.5%, which was greater than the stipulated recovery of 85%.

A number of bridges in the past have failed during load testing (D. Imhof, 2004). If the failure is due to buckling of one of the compression members, the failure would be sudden without any warning. Therefore, before taking up load testing of the bridge under maximum service load condition, it is necessary to ascertain that the bridge has adequate reserve strength at the limit state of strength. In the case of Garudchatti bridge, safety of the bridge under additional load factor of 1.2 was insured by welding additional sections in the compression members. The additional welded sections would not be fully effective during elastic condition, but in the plastic condition both sections would be fully effective. Thus, at the limit state of strength, requisite factor of safety against sudden collapse due to buckling of compression members during load testing was insured.

During the load testing, recovery of 92.5% deflection after 24 hours of removal of the load insured that the bridge remained in elastic stage.

3.5 CONCLUDING REMARKS

Chauras and Garudchatti bridges were constructed across Alaknanda river on the same design. Whereas, Chauras bridge failed during casting of the deck slab, Garudchatti bridge was successfully completed, but it had to be suitably strengthened.

In Chauras bridge, gusset plates connecting member U13U14 at joints U13 and U14 were subjected to high stress of 249.0 N/mm^2, but these plates did not fail due to their riveting with the members meeting at the joint, and thereby preventing buckling of the plates. Thus, gusset plates if connected properly to the members and prevented from buckling, can take compressive or tensile stress up to their ultimate tensile strength. Therefore, joints designed for service load are safe for the limit state of strength condition also.

Failure analysis results of Chauras bridge show that the compressive stress in the top chord compression members U13U14 at the time of collapse was 173.8 N/mm^2 against critical buckling stress for individual buckling of 8mm thick plates of 140.0

N/mm^2. Therefore, failure took place due to buckling of members U13U14, which was due to the compression members having 8mm thick plates. Welded box sections, when used as a compression member, have limitations from individual component buckling. In the case of member U13U14, the width to thickness ratio of individual 575mm wide, 8mm thick plates was 72, which was in far excess of the 30 limit specified by IRC:24-2010. Therefore, buckling of member U13U14 was initiated by buckling of the individual 8mm plates leading to the bridge collapse.

Compression members may buckle and suddenly fail without warning and do not have reserve strength like tension members, which have reserve strength beyond yield stress up to their ultimate strength. Therefore, it is recommended that design of laterally unsupported compression members at the limit state of strength may be checked with higher load factor of 2.25 instead of 1.5 recommended by IRC:24-2010.

After 3D analysis for 3-trains and 2-lanes of IRC Class-A vehicle loading, and strengthening of Garudchatti bridge, its load testing was carried according to IRC:SP: 37-1991 and IRC:SP: 51-1999. A number of bridges in the past have failed during load testing and therefore, before taking up load testing of the Garudchatti bridge its design was checked for 1.2x(DL+LL) by welding additional sections to the critical compression members.