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Performance evaluation of in-service open web girder steel railway bridge through full scale experimental investigations

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Abstract. Civil infrastructures, such as bridges and tunnels are most important assets and their failure during service will have significant economic and social impact in any country. Behavior of a bridge can be evaluated only through actual monitoring/measurements of bridge members under the loads of interest. Theoretical analysis alone is not a good predictor of the ability of a bridge. In some cases, theoretical analyses can give less effect than actual since theoretical analyses do not consider the actual condition of the bridge, support conditions, level of corrosion and damage in members and connections etc. Hence actual measurements of bridge response should be considered in making decisions on structural integrity, especially in cases of high value bridges (large spans and major crossings). This paper describes in detail the experimental investigations carried out on an open web type steel railway bridge. Strain gages and displacement transducers were installed at critical locations and responses were measured during passage of locomotives. Stresses were evaluated and extrapolated to maximum design loading. The responses measured from the bridge were within the permissible limits. The methodology adopted shall be used for assessing the structural integrity of the bridge for the design loads.

Keywords: open web girders; railway bridges; stress; strain; performance evaluation

1. Introduction

Civil infrastructures, such as bridges and tunnels are most important assets and their failure during service will have significant economic and social impact in any country. In general, bridge monitoring is aimed at providing the state of the structure so that any damage or deterioration can be detected at an early stage and remedial measures can be suitably taken up. Full scale experimental investigations provide more important data for research and development in the field of bridge engineering (Zhang *et al.* 2007). Through recent years performance evaluation of bridges were carried out using various types of sensors such as strain gauges, accelerometers, global positioning systems (GPS), fiber optic sensors, total stations, etc. (Chang 1997, 1999, Ansari 2005, Balageas 2002, Mufti and Ansari 2004). Behavior of a bridge can be evaluated only through actual monitoring/measurements of bridge members under the loads of interest. Theoretical analysis alone is not a good predictor of the ability of a bridge. In some cases, theoretical analyses can give less effect than actual since theoretical analyses do not consider the actual condition of the bridge,

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support conditions, level of corrosion and damage in members and connections etc. Hence actual measurements of bridge response should be considered in making decisions on ability to carry increased axle loads, especially in cases of high value bridges (large spans and major crossings). Ashebo *et al.* (2007) studied the effect of the skewness on a threespan box girder bridge in Hong Kong having a total length of 73 m. Natural frequencies were determined using SAP2000 software and field measurement. Caglayan *et al.* (2011) performed a dynamic structural assessment of a four-span riveted steel plate girder bridge in Turkey with a total length of 54 m. Natural frequencies were determined using COSMOS software and field measurements. Liu *et al.* (2009) studied a seven-span composite bridge on a high-speed railway line between Turin and Milan in Italy. The total length of the bridge was 322 m. Rodrigues, (2002) used field measurement to determine natural frequencies of a single-span 31.4 m steel truss railway bridge in Portugal for an active tilting train at speeds of up to 200 km/h. Xia *et al.* (2005) presented the experimental results of a 28-span pre-stressed concrete bridge for a high-speed train in China. Deflections, accelerations, strains and forces were measured. Shibeshi (2016) carried out dynamic analysis of a 77-year-old single-span steel truss railway bridge through field measurement, modal analysis using a three-dimensional finite element model of the bridge, and a simple generalised single degree of freedom (SDOF) analysis. Field measurement was conducted using accelerometers and displacement transducers, which were mounted on special sections fixed to an adjacent bridge. Bacinskas *et al.* (2013) presented the investigation results of a historic narrow-gage railway steel truss bridge built in 1936. The aim of this study is to investigate the structural condition and the behaviour of the riveted steel truss bridge with the aid of full-scale static and dynamic testing using two original locomotives. The responses (stresses, static and dynamic displacements, accelerations, mode shapes, corresponding resonant frequencies and modal damping values) of bridge superstructure were determined. A series of dynamic tests, acceleration measurements, evaluation, finite element model simulations and safety index calculations were performed on existing steel railway bridges giving service on railway network by Caglayan *et al.* (2012). Dynamic tests were fulfilled by using a special test train on these bridges to obtain the dynamic parameters and these parameters were then used to refine the finite element models of the bridges. Plachy *et al*. (2017) carried out experimental analysis of more than one hundred years old railway bridge in the Czech Republic for assessment of load capacity. It was mainly focused on measurement of strains and acceleration in critical locations on the steel structure. Thus detailed experimental investigations are required for performance evaluaton of railway bridges in addition to the numerical studies. The experimental investigations gives the exact behavior of the bridge under various loading conditions. This paper describes in detail the experimental investigations carried out on an open web type steel railway bridge. Strain gages and displacement transducers were installed at critical locations and responses were measured during passage of locomotives. Stresses were evaluated and extrapolated to maximum design loading. The methodologies adopted are described in detail in the following sections.

2. Details of bridge

Open web girders are used for track bridges over valleys and large rivers. Standard length of open web girders in railways are 30.5, 45.7, 61.0 and 76.2 metres. Girna bridge (no. 374) is an open web girder through type bridge near Jalgaon station in Maharashtra on Jalgaon – Surat railway line of Western Railway (WR). The bridge was constructed across river Girna between

Jalgaon (JL) and Paldhi (PLD) railway stations. The bridge has 15 spans each of 30.5 m length. The primary members of the girder are Bottom chord, Top chord, End Rakers, Diagonals, Vertical members and Floor system comprising of Cross girders and Rail bearers or stringers. General view of the bridge is shown in Fig. 1.

3. Instrumentation of bridge

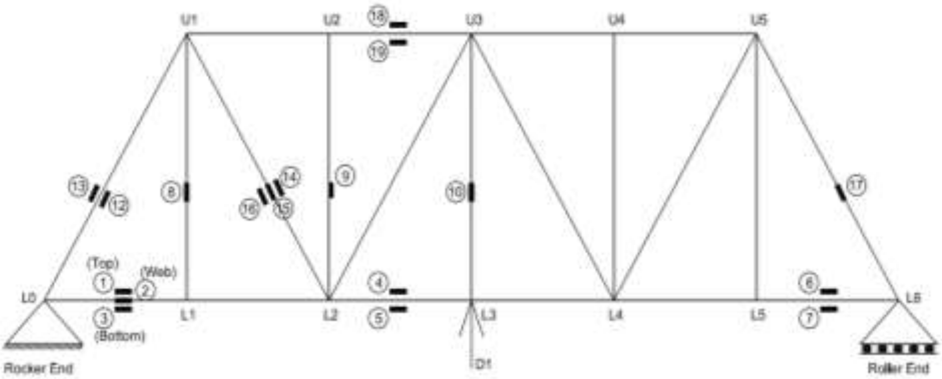
The bridge girder was instrumented with strain gages and displacement transducers at critical locations to measure the response of the bridge during loading. Span 3 from Paldhi end as shown in Fig. 2 was instrumented for response measurement.

In order to obtain the stress developed in the structural members, strain gages were instrumented on the girder at critical locations. Totally 32 locations were instrumented on the various members

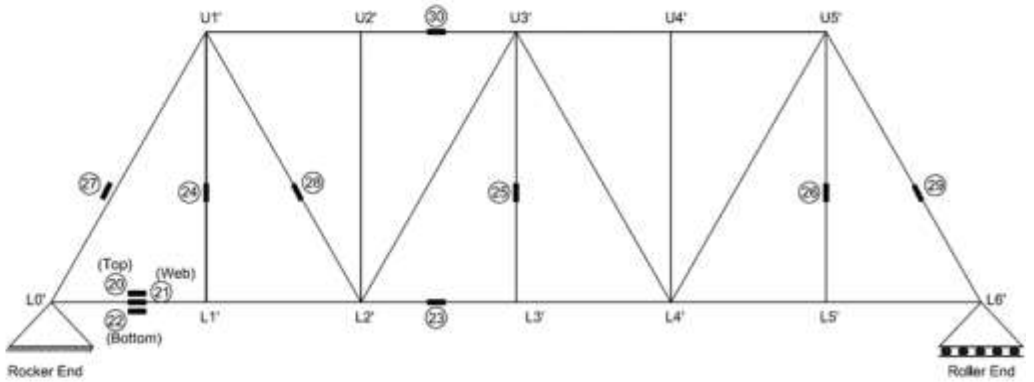
(a) (b)

Fig. 1 (a) General side view and (b) General view of the Girna Bridge

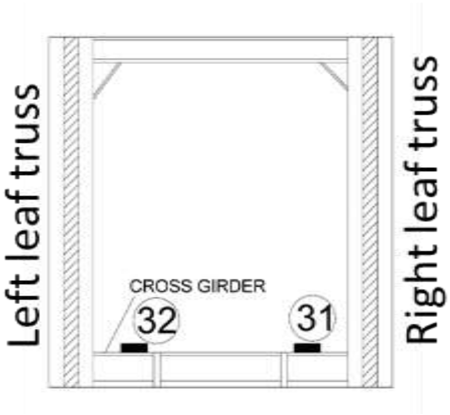
Fig. 2 Typical view of the instrumented span of the bridge



(a) Instrumentation location on left leaf of the bridge



(b) Instrumentation location on right leaf of the bridge



(c) Instrumentation location on cross girder of the bridge

Fig. 3 Schematic representation of Instrumentation locations in the bridge

of the girder. The locations of instrumentation was chosen based on the Research Design and Standards Organization (RDSO) document on "Broad Guidelines for Instrumentation of Bridges for Running Higher Axle Loads" (RDSO, 2010). On the rocker end raker top flange and centroidal axis on left side truss (L0-U1) numbered as {12, 13} and at the centroidal axis on the right side truss (L0’-U1’) numbered as {27} was instrumented.

On the roller end raker, strain gage was instrumented on the centroidal axis on both trusses (U5-L6) and (U5’-L6’) numbered as {17, 29}. From these strain gages the axial stresses in these members are evaluated during load test. On the intermediate diagonal, at the rocker end, strain gage was instrumented on the top flange and bottom flange on left side truss (U1-L2) numbered as {14, 15, 16}, and on the centroidal axis on the right side truss (U1’-L2’) numbered as {28}for evaluating the axial stresses in these members. On the bottom chord at mid-span both top and bottom of left side truss (L2- L3) numbered as {4, 5} and centroidal axis of the right side truss (L2’-L3’) numbered as {23} was instrumented with strain gages. On the top chord of the mid span member, top flange of left side truss (U2-U3) numbered as {18, 19} and on the centroidal axis of the right side truss (U2’-U3’) numbered as {30} was instrumented with strain gages. From the strain gages at this location, the axial stress in the member is obtained. From the difference in the strain values of the top and bottom strain gauges the amount of bending if any can be identified. Two strain gages was instrumented on the centroidal axis of the vertical post, one on either truss (U1-L1, U1’-L1’) numbered as {8, 24} on the rocker side for evaluating the stresses in the vertical posts. Additionally alternate vertical members were also instrumented to understand the stress variation in the vertical members. On the rocker side, first bottom chord was instrumented with strain gages on top, bottom and centroidal axis (L0- L1) & (L0’-L1’) numbered as {1, 2, 3, 20, 21, 22}. Two strain gages were instrumented on the top flange of the cross girders. All the locations were instrumented with 5mm long self-temperature compensated (for steel) surface mounting type linear electrical resistance strain gages of 120 ohm resistance. Standard procedures were followed for installation of strain gages. The strain gages were protected for moisture ingress using polyurethane coating. In order to measure the deflection response of the bridge girder displacement transducers were used. Each span was instrumented at midspan to measure the displacement response during the experimental investigations. The height of the bridge was so high that deflection response could not be obtained using conventional sensors. Special arrangements were made using draw wire type displacement transducer of 500 mm range with 0.01 mm accuracy was used to measure the displacement responses. The instrumentation scheme for span 3 is as shown in Fig. 3. Typical locations instrumented with strain gages and displacement transducer is shown in Figs. 4 to 7.

Fig. 4 Strain gage instrumented in diagonal member

Fig. 5 Strain gage instrumented in bottom chord

Fig. 6 Strain gage instrumented in cross girder

Fig. 7 Instrumentation of displacement transducer at midspan

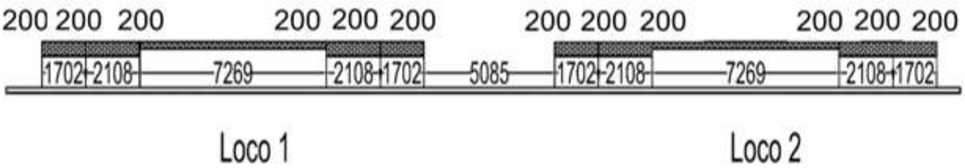
3. Loading and measurement

The instrumented strain gages and displacement transducer was connected to the high speed data acquisition system as shown in Fig. 8. Since the bridge across river Girna is under operation, the testing of the bridge for design load is not feasible. The experimental investigations were conducted for a shorter duration only and hence effects due to temperature are not considered. In order to get the maximum response of the bridge, it was tested with coupled loco (WAG 5) during the experimental investigations as shown in Fig. 9. The axle load of the loco is around 20 tons. The axle spacing and load on each axle is shown in Fig. 10. The responses measured from the experimental investigations will be extrapolated for the maximum design load. Both static and dynamic tests were performed on the bridge. Two static tests were conducted on the bridge for creating maximum bending moment and shear force on the bridge girder. The axles of the loco were placed at predetermined positions on the girder to create maximum bending and shear. For span 3, the fourth axle of second loco was placed at 350 mm from roller end bearing for creating maximum bending moment as shown in Fig. 11. Sixth axle of second loco was placed at 6610 mm from the roller end bearing for creating maximum shear in span 3 as shown in Fig. 12. The axle position were marked in the rail prior to the testing. The loco positioned for maximum bending moment and shear in span 3 is shown in Figs. 13 and 14.

Followed by the static tests, dynamic tests were performed on the bridge. The dynamic tests were performed to study the tractive effect, braking effect and uniform speed effect on the instrumented span. The coupled loco was placed on the span and started with full traction to study the tractive effect on the bridge. In the braking effect case, the coupled loco was made to run from a distance away from the bridge and apply brake on the span to generate braking effect on the span. In the uniform speed case the dual loco was made to cross the span at a uniform speed of 100 kmph. In all the test cases, the instrumented sensors were initialized prior to the loading. After completion of each test the coupled loco was moved away from the bridge span and the test was repeated after initializing the sensors.

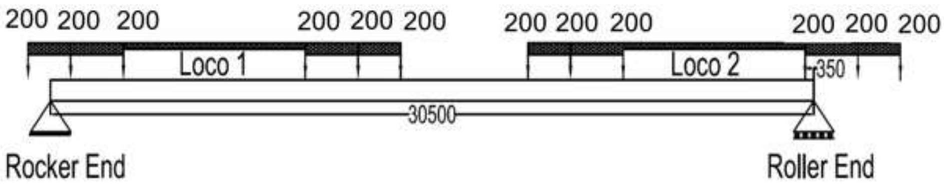
Fig. 8 High speed data acquisition system for measuring the responses

Fig. 9 Coupled loco (WAG 5) used for the experimental investigations



All loads in kN and All dimensions are in mm

Fig. 10 Axle loads and spacing of dual loco used during experimental investigations



All loads in kN and All dimensions are in mm

Fig. 11 Axle load position for maximum bending moment case

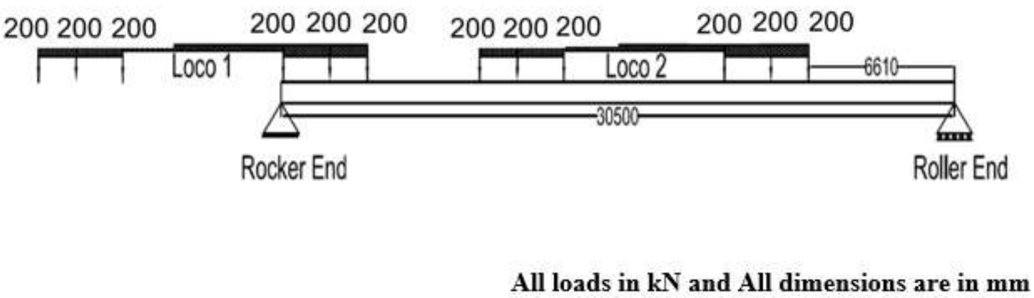


Fig. 12 Axle load position for maximum shear case

Fig. 13 Dual loco placed for maximum bending moment case

Fig. 14 Dual loco placed for maximum shear case

5. Results and discussions

5.1 Strains and deflections

A maximum compressive strain of 188 microstrain was measured at cross girder top flange, tensile strain of 179 microstrain was measured at (L2U1 web) and midspan deflection of 1.25 mm was measured for load case 1. In maximum shear case (load case 2), a maximum compressive strain of 196 microstrain was measured at U2U3 top chord flange plate, tensile strain was 195 microstrain at L2U1 bottom flange and midspan deflection was 1.20 mm. For tractive effect case, a maximum of 227 microstrian (tensile) was measured at L2U1 top flange, 196 microstrain (compression) was measured at U2U3 Top chord flange plate and midspan deflection was 1.35 mm. For braking effect case, 194 mcirostrain (compressive) was measured at cross girder top flange, 139 microstrain (tensile) was measured at L2U1 top flange and the midspan deflection was 1.05 mm. For uniform speed case, 211 microstrain (tensile) was measured at L2’U1’ web, 196 microstrain (compression) was measured at cross girder top flange and midpsan deflection was 1.40 mm. The strain variation with respect to time for various test cases is shown in Figs. 15 and 16.

5.2 Extrapolation of stresses for design loading and evaluation of permissible stresses

Since this bridge is under operation, the experimental investigations cannot be carried out for maximum design loading. Hence the bridge was tested using standard locomotives and the measured values are extrapolated for design loads. Maximum bending moment case is considered for evaluating the stresses due to maximum design load. The Equivalent Uniformly Distributed Load (EUDL) for Bending Moment (BM), for spans upto 10 m, is that uniformly distributed load which produces the BM at the centre of the span equal to the absolute maximum BM developed under the standard loads. For spans above 10m, the EUDL for BM, is that uniformly distributed load which produces the BM at one-sixth of the span equal to the BM developed at that section under the standard loads. (as per bridge rules) (RDSO, 2014)

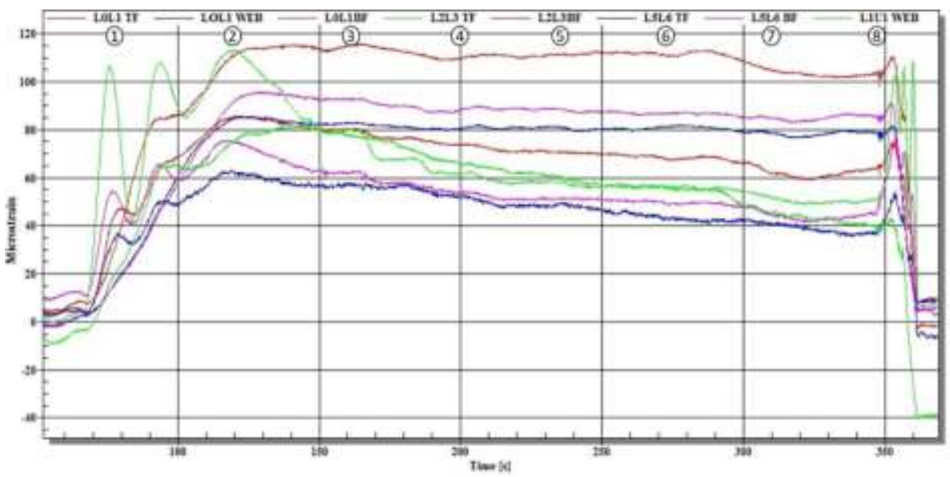


Fig. 15 Variation of strain for Tractive case

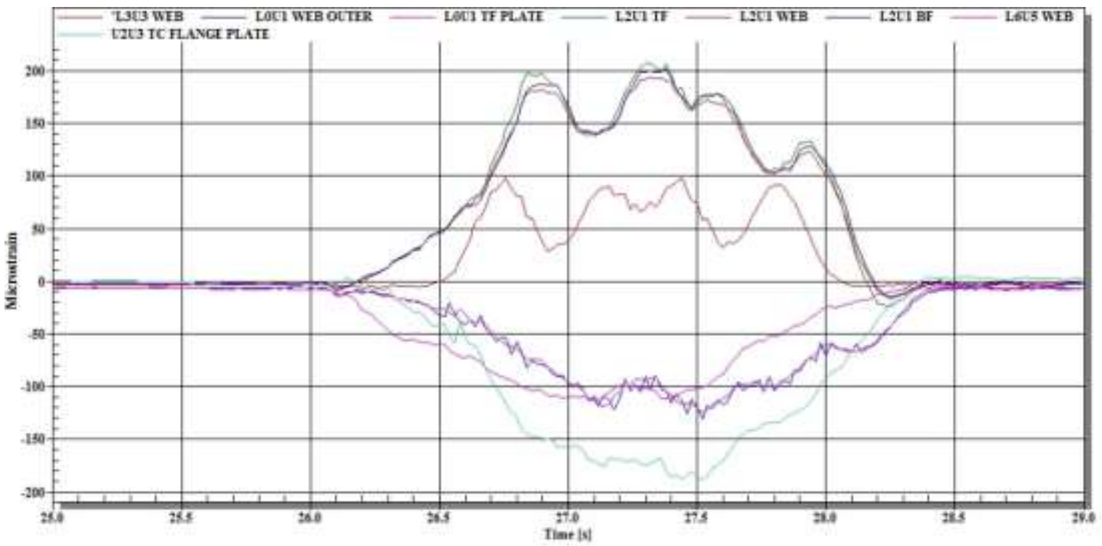


Fig. 16 Variation of strain for Uniform speed case

Total axle load on the span = 1800 kN

Maximum bending moment at midspan = 6926.8 kNm

Equivalent Uniformly Distributed Load (EUDL) for the above bending moment = 1816.8 kN

Maximum design load (EUDL) for 30.5 m span = 2953.5 kN (as per bridge rules) (RDSO, 2014)

Coefficient to be used for extrapolating maximum design load = 2953.5/1816.8 = 1.625

Assuming the Young's modulus of steel as 2x105 N/mm2 and using the above coefficient of 1.625, the maximum stresses in the members of the girder is evaluated from the measured strain responses for the design load of 2953.5 kN (EUDL). The evaluated stresses and deflection for the design load is given in Table 1. From the table it can be seen that a maximum tensile stress of 73.78 N/mm2 was developed at L2U1 and maximum compressive stress of 63.70 N/mm2 was developed at U2U3 in the tractive case for design EUDL. Similarly the maximum deflection evaluated for the maximum design EUDL was 2.275 mm for the uniform speed case which is less than the permissible limit of 22.80 mm. The truss bridge was modelled and analysis was carried out to evaluate the stresses in each member due to dead load. The permissible stresses were evaluated based on the nature of stresses in each member using the Indian standard codal provisions. The total stresses due to dead load and live load along with the permissible stresses (without occasional load) for each member is given in Table 1. From the table it can be seen that the total stresses in each member is within the allowable limits.

Table 1 Total stresses in the members including dead load stresses

Member	Dead load stresses in N/mm2	Total stresses in N/mm2					Permissible stresses in N/mm2
		LC1	LC2	LC3	LC4	LC5	
L0L1	10.98	32.43	35.03	38.93	35.03	34.71	125.70
L2L3	14.46	48.27	45.34	52.17	49.24	48.27	125.00
L5L6	10.98	34.38	34.38	42.18	39.58	39.91	125.70
L1U1	8.58	30.68	39.13	45.31	42.71	43.68	120.20
L3U3	8.58	36.86	25.81	42.71	39.78	40.76	120.20
L0U1	-11.57	-49.27	-64.87	-56.74	-52.84	-54.14	-111.20
L2U1	14.62	72.48	75.40	88.40	84.18	82.23	113.00
L6U5	-11.57	-43.42	-45.04	-45.04	-46.34	-48.29	-111.20
U2U3	-16.16	-72.06	-79.86	-79.86	-77.91	-77.59	-129.90
L0'L1'	10.98	25.28	28.21	35.36	32.11	36.33	125.70
L2'L3'	14.46	37.54	39.49	49.57	43.72	46.97	125.00
L3'U3'	8.58	31.98	23.53	41.41	38.48	42.38	120.20
L0'U1'	-11.57	-46.99	-48.94	-46.67	-44.39	-44.39	-111.20
L2'U1'	14.62	68.90	68.90	12.35	81.25	83.20	113.00
L6'U5'	-11.57	-42.12	-45.69	-41.79	-44.07	-49.27	-111.20
U2'U3'	-16.16	-72.71	-75.96	-73.36	-74.01	-74.99	-129.90
Cross girder (top flange)	-13.94	-75.04	-52.61	-74.39	-76.99	-77.31	-114.60

5. Conclusions

Full scale experimental studies were carried out on Girna bridge (no. 374), a open web girder through type bridge to evaluate the performance of the bridge under train loading. The various members of the bridge truss was instrumented using strain gages. The deflection was measured using a displacement transducer. Since the bridge was under operation, it was tested using a coupled loco. Both static and dynamic tests were carried out on the bridge. The measured responses were extrapolated for the maximum design loads. Dead load stresses were evaluated from the model and used for getting the total stresses. The permissible stresses for each members has been evaluated considering the nature of stress.

From the experimental investigations it can be seen that a maximum tensile stress of 73.78 N/mm2 was developed at L2U1 and maximum compressive stress of 63.70 N/mm2 was developed at U2U3 top chord in the tractive case for design EUDL. The maximum deflection evaluated for the design EUDL was 2.275 mm in the uniform speed case. The total stresses due to dead load and design live load for each member was calculated. From the results it can be seen that the total stresses in the instrumented members are within the allowable limits. The nature of strains obtained from the experimental investigations were also matching with that of the design calculations. The maximum deflection measured during the experimental investigation is less than the permissible limit (22.80 mm). Exhaustive instrumentation of the truss was done and from the experimental investigations the maximum stresses developed due to the design loading was obtained. Thus the structural integrity of the truss was evaluated using the measured responses. There was no abnormal stress variations observed during the experimental investigations. The limitations in the present study is, the responses measured are only stresses and lack of vibration characteristics of the bridge. Further studies are necessary to evaluate fatigue behavior of the bridge in view of the large variation of stress due to passage of trains.

Acknowledgments

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