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Modal parametric changes in a steel bridge with retrofitting

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Abstract. This paper presents the status improvement of an old damaged deck type rural road steel truss bridge through the modal parametric study after partial retrofitting. The dynamic and static tests on bridge were carried out as in damaged state and after partial retrofitting. The dynamic testing on the steel bridge was carried out using accelerometers under similar environmental conditions with same speed of the moving vehicle. The comparison of the modal parameters i.e., frequency, mode shape mode shape curvature, modal strain energy, along with the deflection parameter are studied with respect to structural analytical model parameters. The status up gradation for the upper and downstream truss obtained was different due to differential level of damage in the bridge. Also after retrofitting the structural elemental behavior obtained was not same as desired. The damage level obtained through static tests carried out using total station indicated further retrofitting requirement.

Keywords: steel truss bridge; retrofitting; frequency; mode shape; modal strain energy

1. Introduction

The bridges being integral part of the transportation system, their failure lead to isolation of affected area. Most of the Indian bridges have been constructed several decades ago and are decaying due to aging, deterioration, fatigue and environmental influences. The bridges are increasingly subjected to heavier and fast loads compared to the designed loads. Hence continuous monitoring assists in early identification of distress. Retrofitting of the distressed bridge at early stage prevents unwarned collapses. In the state of Himachal Pradesh, India which is situated at the foothills of Himalayas mostly the assessment of highway bridges is done by visual inspection. These inspections do not detect and notify overall behavioural defects in the structure (Graybeal *et al*. 2002). However these limitations can be overcome by vibration based methods (Fryba and Pirner 2001). Due to continuous increase in traffic volume and axle loading, degradation of bridge components has been observed (Dong and Song 2010) leading to revision of the codal provisions for load and stresses over the decades. The detected defects may be then be strengthened and

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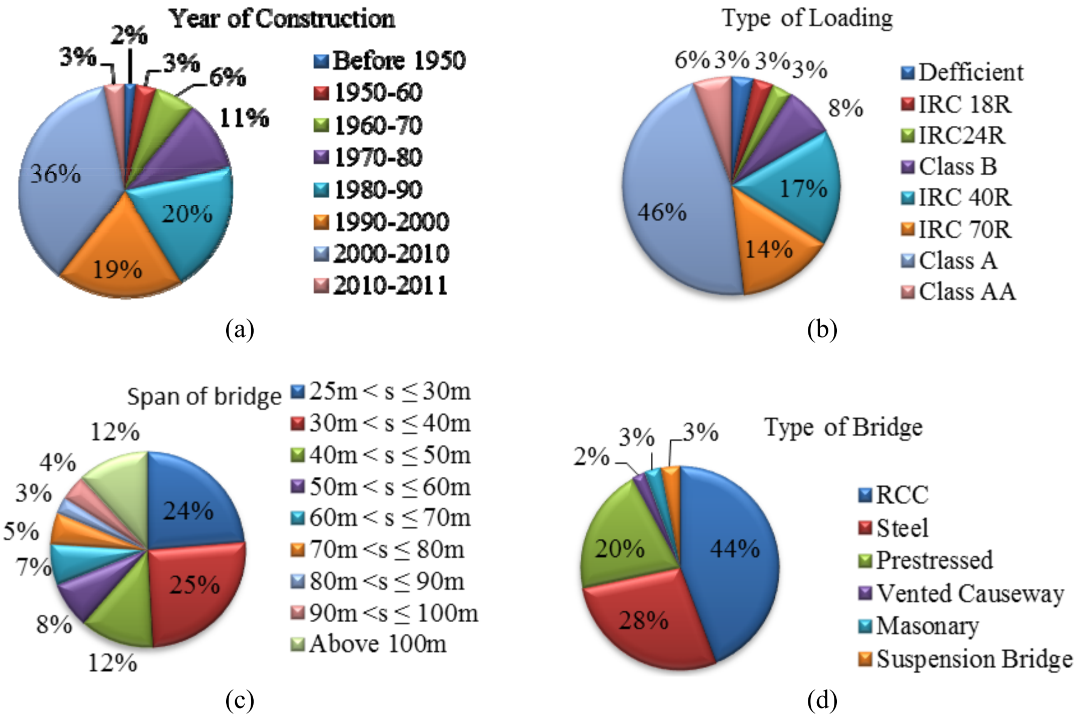


Fig. 1 Status of bridges in state of Himachal Pradesh India w.r.t: (a) Year of construction; (b) Types of loading; (c) Span of bridge; (d) Type of bridge

further retrofitted as per the respective codal provisions of load and stresses (IRC: 6 2000).

In order to have the present status of bridges in the state of Himachal Pradesh India, the data was collected from the different parts of state for about 650 bridges having span more than 25 m. As per the data collected about 61% bridges have been constructed before the year 2000, the year in which the code has been revised (Fig. 1(a)), 3% bridges have been declared as deficient, about 34% bridges have been designed based upon IRC Class 18R,Class 24R, Class 40R and Class B loading (Fig. 1(b)). These are those bridges which are in service with speed and/or load restriction. Most of the bridges are having a span of 25 m to 40 m (Fig. 1(c)). Due to increase in traffic volume and loading these Bridges require immediate rehabilitation as replacement of these bridges after damage would require lot of revenue (Rashidi and Gibson 2011).

About 44% bridges are RCC bridges, 20% bridges are Prestressed bridges and 28% bridges are made of steel structure (Fig. 1(d)). It is necessary to monitor these bridges so that the appropriate precautions can be made before the bridge gets damaged. The present work is the study of improvement of a single span steel truss bridge through static deflections and modal parameters changes from dynamic measurements at different stages of retrofitting.

2. Background

The bridge under consideration was constructed in the year around 1920, as enquired from the local people of the area (Fig. 2). The bridge was built by the British Company named as Dorman

Fig. 2 Steel truss Bridge of span 47.90 m over BathuKhad

Long & Company Limited, Middlesbrough, England on National Highway 20 on Bander Khad near the village Samloti Himachal Pradesh, India. The company had their stockyard at Karachi (now in Pakistan) and their head office was at Clive Street, Calcutta, India. The steel sections used were as per British Standard specifications of 1903. These were imported at the Karachi Stock yard and could have assembled at the site (Constructional Department 1906).

Due to increase in traffic on National Highway and having single lane with carriageway of 3m only, the State Authorities were forced to construct a new Prestressed Concrete Bridge adjacent to this bridge. After the construction of new bridge, old bridge was reassembled by State Public works Department Authoritieson Bathu Khad on Chughera kandi Road, District Kangra, Himachal Pradesh, India in the year 1985. The traffic volume over this bridge is 50 commercial vehicles/day (CPVD). This bridge was working satisfactorily for more than two decades, but in December 2009 settlements were observed in Bridge Deck Due to the crossing of heavy vehicle and lack of maintenance, end members have buckled as shown in Fig. 3. The bridge was closed for traffic to prevent any further damage. The owner of bridge desired to evaluate the condition and prescribe necessary retrofitting mechanism to make the bridge serviceable, while respecting the historic fabric of the bridges.

(a) (b)

Fig. 3 Damaged bridge members and deck of steel truss bridge

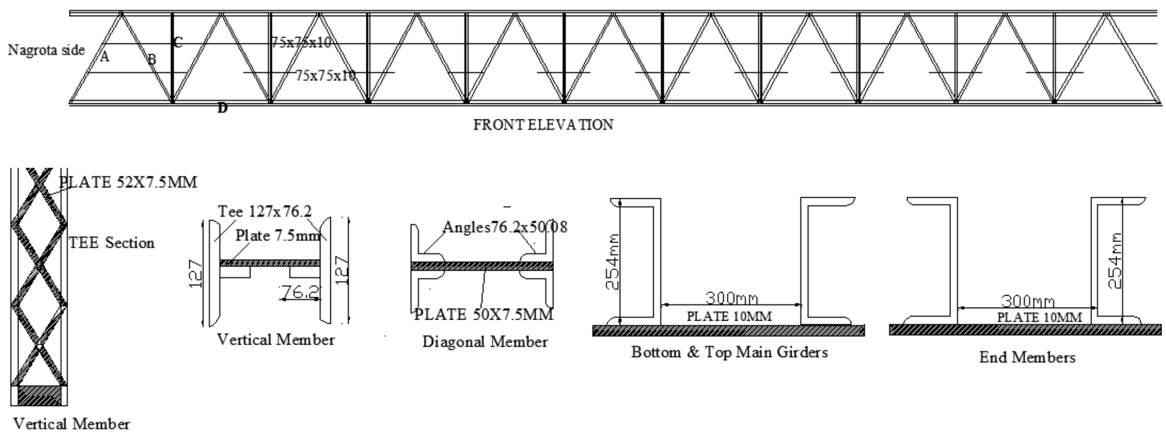


Fig. 4 Detail of bridge components

As-built drawing and design data was not available. Hence an extensive investigative program was chalked out, comprising of preparation of as-built drawings at the site, development of analytical model to obtain the baseline structural parameter and conduct of field tests to obtain the present status of the bridge (Spyrakos *et al*. 2004).

3. Description of steel truss bridge model

This bridge is a simply supported with a hinge support at one end and roller support at the other end. The bridge span is divided into 10 equidistant panels of 4.75 m. The deck of the bridge is made up of corrugated metal sheets of 8 mm thickness with overlay of Bituminous premix carpet of about 40 mm thickness as wearing coat. The vertical members are made up of two T-sections 128 mm × 65 mm × 10 mm joined with double cross bracing of 7.5 mm thick plate section, diagonal members are made up of four angle of 77 mm × 87 mm × 10 mm with double cross lacing of plate section (Fig. 4). The top and bottom horizontal members are made of two channel members 254 mm × 101.6 mm fabricated back to back and riveted with 10 mm thick plate. The members of the trusses have been joined with rivet connections.

4. Experimentation

The bridge was tested for obtaining the vibrational signatures and defections at nodal points before and after retrofitting. Initially (State 1) the testing was done in month of Feb 2010 when the end members had buckled. The testing was carried out again in the month of October 2010 (State II) after the damaged bridge members were replaced; bearing rectified and partial retrofitting was done. The structures are evaluated considering free (ambient) or forced vibrations. Ambient vibration test method for modal parameter determination, does not involve any external continuous source of excitation (Salawu and Williams 1995). But short and medium span bridges require an initial impetuous so that the bridge is set into vibration for recording. Various researchers have carried out vehicle initiated vibration test (Wiberg 2006). Further for an accurate and complete

Fig. 5 Truck load of 9.56 tones at centre

extraction of modal parameters the excitation of the structure to appreciable amplitude becomes an important parameter (Le *et al*. 2010). The experimentation consisted of measuring forced and ambient vibration generated with to and fro movement of the vehicle (Fig. 5). The nodal defections were measured under static load of 9.56 ton using total station and dial gauges for all the above stages to know the damage level from deflection criteria, as serviceability, or deflection (Merkle and Myers 2004). Following are the parameters for the sensor location, input motion, sampling frequency, which are considered to perform the experiment under the same setup so that the records obtained are not affected by the various other parameters.

4.1 Sensor and data acquisition details

The dual channel MEMS accelerometers with 150 hz natural frequency were used. The data acquisition system and laptop were installed at the non-perennial river bed. The simultaneous analog voltage outputs from accelerometers are fed to data acquisition system which is proportional to the acceleration of the points. The conditioned signal from signal conditioning amplifier is further fed to AD Converter where digitization at prescribed sampling rate takes place. This digital data is then stored in ASCII format. Data Acquisition System used for present work was KI4100 Model of Kaptl Instrumentation make.

4.2 Sensor location

The damages in the bridge were in the inclined member and at supports due to which the deck got misaligned and damaged at various locations along the span. Hence actual truss behavior could only be obtained with the sensor placed at the bottom line of the truss. Hence the sensors were placed on bottom longitudinal girder at nodes joining the vertical and diagonal member on both the trusses of the bridge to get the distributed response of the structure (Panigrahi *et al*. 2009). The sensors used were of Freescale Semiconductor make and model MMA7260Q with motion sensing capability. Vertical acceleration time histories at 22 locations were measured. The sensors got excited when the vehicle passed over the bridge. Each sensor location has been identified with unique nodal coordinate number. Fig. 6 shows the detail of the placement of the sensors position on bridge. One accelerometer has been used as reference sensor at node 5. The response measurement was carried out along the vertical direction. The direction of the acceleration pick up was accounted for during the analysis of the data, since the sensor on both the trusses were installed in out of phase. The Reference sensor placed on downstream (D/S) truss and moving sensor on upstream (U/S) and downstream nodes to find the disparity between nodes if any.

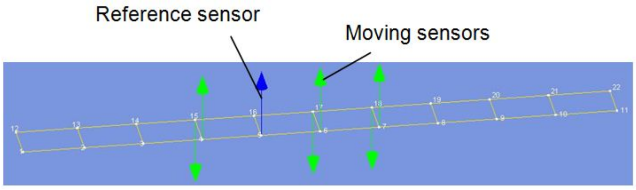


Fig. 6 Sensor location on the bridge

4.3 Input motion

The acceleration responses at different nodes were measured with passing the same vehicle with same speed. The vehicle used for the testing on the bridge was of Tata make with model Truck 1210 SE. The weight of the bridge is 188 ton. Although in order to obtain structural identification it is better to measure the bridge vibration after vehicle passed the bridge as free vibration. But in the present case since the span of bridge was not too large leading to be a bit stiff bridge in terms of duration of free vibration that would obtained after vehicle has passed. This duration would not be sufficient enough for extracting the modal parameters. Hence the data in which the vehicle is on the bridge has also been used. Although the coupling effect between vehicle and bridge are generated but in the present case such effect can be neglected, since very stiff bridges with fundamental frequencies in 10-15 Hz range are more influenced by vehicle mechanical properties of range 2-5 Hz (Paultre *et al*. 1992). The dynamic loading applied by most heavy vehicles are in the range of 1.5-4.5 Hz (Green *et al*. 1995). In the present study, the fundamental frequency of the bridge varies from 5.17 to 4.49 Hz for different state of bridge. However, even if the natural frequency of the vehicle and bridge are identical that could lead to frequency shift of the coupled system. This coupling is noteworthy in the cases where modal bridge mass is 20 times higher than the vehicle mass (Cantieni *et al*. 2000, Broquet *et al*. 2004). In the present study the vehicle mass is 9.6 ton while the bridge modal mass for the vertical vibration is 22 ton. Hence, the bridge vehicle interaction for the present case has not been considered.

4.4 Sampling frequency

The sampling rate for acquiring the signal was fixed at 200 samples per second for the different cases. This sampling rate is sufficient to provide information regarding modal frequencies up to 100 Hz (Nyquist frequency) and three modes can be covered in this range. A typical record of the response of one of the setup obtained on the bridge is given in Fig. 7.

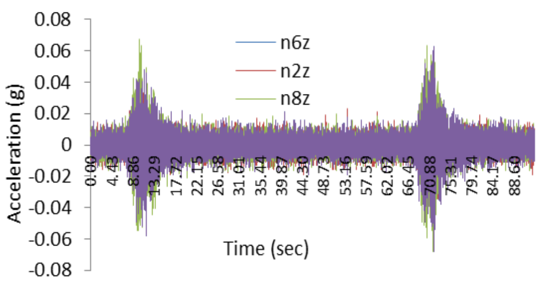


Fig. 7 Typical time history response of accelerometer setup

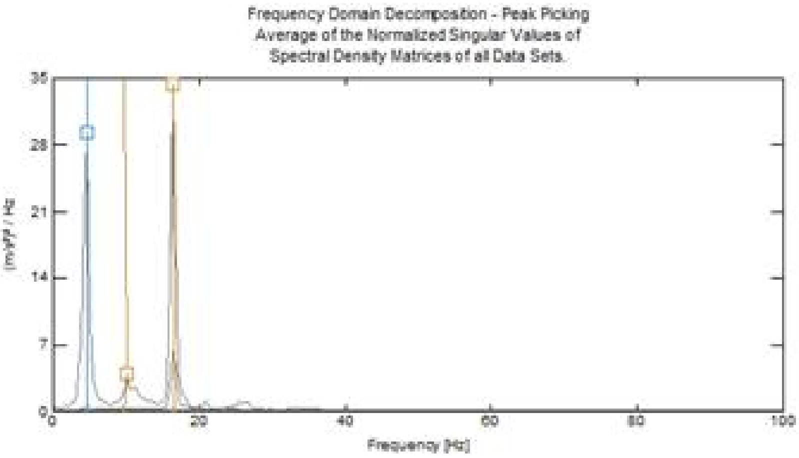


Fig. 8 Typical output from frequency domain decomposition algorithm

4.5 Processing of recorded data

The modal parameters can be extracted by various methods (Doebling *et al*. 2003, Avitabile 2003). The digitized output signals generated by the data acquisition system are given as input. But, the accuracy of extraction of frequency and modal shape depends on the signal to noise ratio (Alampalli *et al*. 1997). Alongwith with various other limitations the extraction of modal parameter are also based on the availability of resources. In the present study, in order to determine the modal properties of the structure from the recorded data ARTeMIS (2004) was used. The natural frequencies of vibration and associated mode shapes were determined from measured responses using Frequency Domain Decomposition algorithm of this software (Altuni *et al*. 2010). The Frequency domain decomposition algorithm provides the normalized mode shapes for each set of the time history. Typical output of the structural frequencies obtained from the Frequency Domain Decomposition algorithm of ARTeMIS software has been shown in Fig. 8. The typical input in the ARTeMIS software is to assign nodal coordinates in the structure, member connecting the nodes, details of the files containing recorded data, which in this case is acceleration time history, direction of the installed sensor during testing which give the information to the software that whether record generated by the sensor is in phase or 180 degree out of phase with the direction of the motion of the structure and master slave node condition considered in the model. The master slave condition involves the dependency of the movement of the non-measured node on the measured nodes.

5. Retrofitting

The analytical model of the bridge was generated in Structural Analysis Program (SAP2000 2006) (Fig. 9) with the structural element as the truss members. The members are assigned specifically as built cross-section to achieve analytical model as accurate as possible. The Bridge was analyzed for Class A and Class B Loading (IRC: 6 2000). The bridge was safe for Class B loading but required retrofitting for Class A loading.

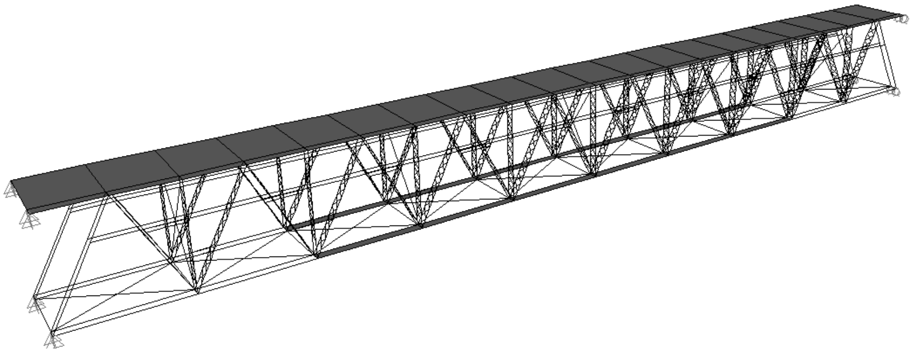


Fig. 9 As built Analytical model of 47.50 m deck type steel truss bridge

(a)

(b)

Fig. 10 End members before and after retrofitting

Although the road is a village road and traffic intensity is low and still the bridge was strengthened for Class A loading. As the end members have buckled due to overstress. Hence members were replaced with same section to strengthen the structure (Fig. 10). In order to hold the main component of the built-up section in their relative position and to equalize stress distribution (Subramanian 2008), the end members were further strengthened by stiffening with 300 mm × 10 mm thick plate at a spacing of 30cm c/c. The horizontal stiffener of angle section 75 mm × 75 mm × 10 mm at a height of 1.35 m from the bottom longitudinal member has been provided as additional member.

The bottom longitudinal members were straightened and stiffened with steel plate of 300 mm × 10 mm size as shown in Figs. 11-12. However the replacement of damaged Deck plates and retrofitting of top longitudinal member is still pending and owner agency is on the Job.

The roller bearing provided towards the Chughera side have become fixed as the earthsoil has been filled up in the rollers. The load was relieved from the bearings by jacking the superstructure and temporary support system was introduced. Since Rocker and rollers were in good condition hence same bearings were used after clearing the soil deposits. The rollers of bearings were reset at correct alignment. The base plate was replaced with new one. Fig. 13 shows the status of bearings before and after retrofitting. The state Public Works Department carried out the retrofitting of the bridge and the bridge was restored for traffic after partial retrofitting to avoid inconvenience to local residents.

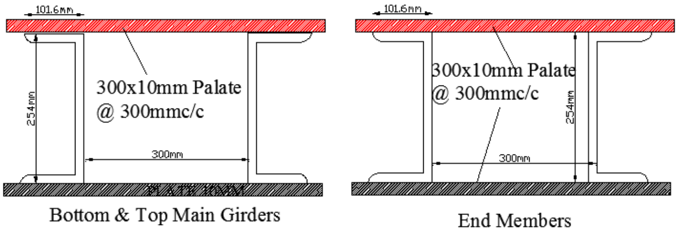


Fig. 11 Retrofitted sections

(a) (b)

Fig. 12 Bottom members before and after retrofitting

(a) (b)

Fig. 13 Bearings before Retrofitting and after retrofitting

6. Parametric studies

Any variation in the expected behavior of the trusses can be due to the fact that a particular steel member is ineffective in taking the stress or the joint has lost its rigidity and is transferring the load to other members with partial rigidity or the bearing has become ineffective in transferring the load to the ground. Such defects can carried out using with vibration based methods on the identification of changes in dynamic properties that is natural frequencies, damping, mode shapes and their derivative to identify the existence, location and quantification of damage (Dutta and Talukdar 2004). The modal parameter sensitivity analysis can further be utilized to

(a)

(b)

Fig. 14 Deflection measurement using: (a) Total station; (b) Dial gauge

understand structural elemental behavior (Parloo *et al*. 2003). However since these parameters depend on various factors of experimentation the accuracy of modal parameter is always in question leading to further computation of their derivatives (Gandomi *et al*. 2008). The accuracy of the modal parameter derivatives justifies their adoption, improper measurement lead to incomplete modal extraction even with the algorithm of established reliability (Yuen *et al*. 2006, Yuen 2011). In the present study the deflection and modal parameters that is the frequency, mode shape and derived modal parameters of the different states has been compared.

6.1 Deflections

The variation in the applied forces leads to different amplitude of displacement in the structure. In the present study displacement has been measured at different nodes by moving the truck along the span. The vertical deflections are significant and can be measured with Total station while vehicle is moving on bridge (Jauregui *et al*. 2003). The displacements were measured at different nodes on both upstream and downstream side when the truck was placed at centre of span (Figs. 14(a)-(b)).

The level at different nodes obtained in the State I with the truck load 9.56 tons placed at the centre is shown in Fig. 15. The deflections in the downstream truss were more than those in upstream truss showing that bridge has been tilted towards downstream side.

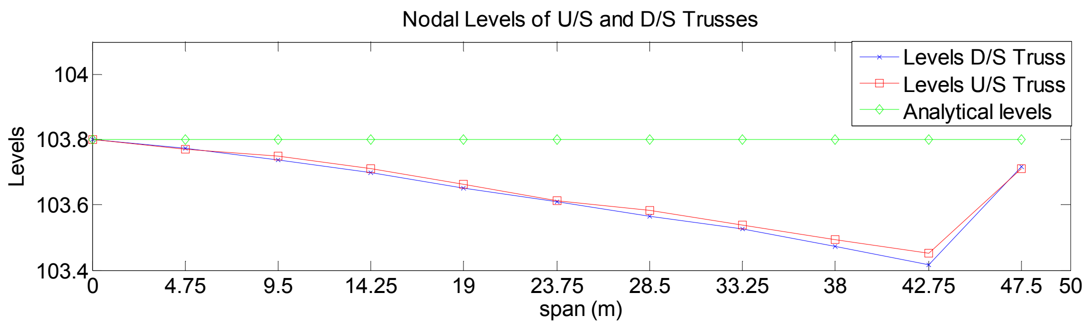


Fig. 15 Level difference between the support points

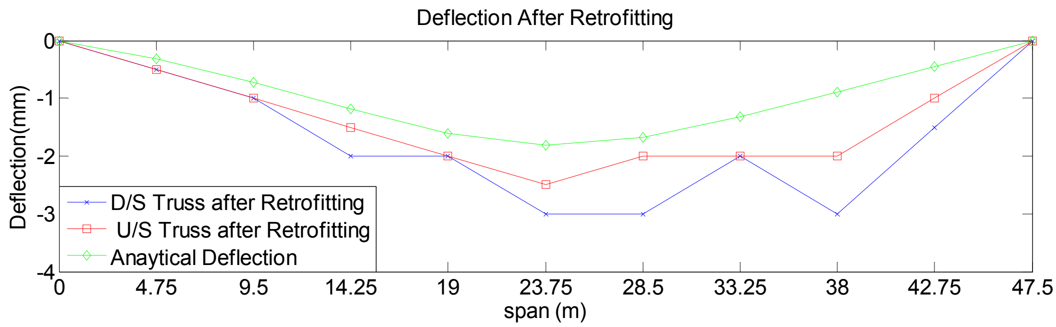


Fig. 16 Nodal deflections after partial retrofitting (state II)

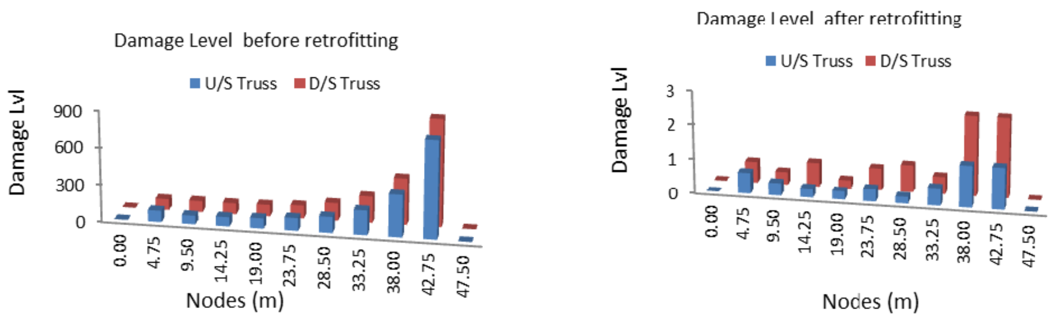
In State II test, the settled nodal point was cleared off debris and load was transferred to roller support. The defection in State II i.e., after retrofitting was obtained to determine the improvement in the structure (Fig. 16). The kink at 38 m indicates that the node was not structurally repair upto the desired requirement. However, after retrofitting general improvement in the bridge is observed.

In order to determine the improvement in level of damages at different nodes, the damage level (D_L) with respect to analytical value has been determined as

$$\text{Damage Level } D_L = (D_E - D_A) / D_A$$

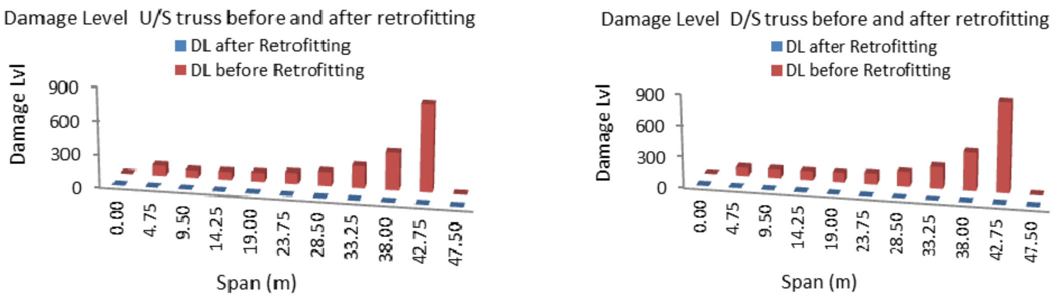
Where, D_E and D_A are the experimental and analytical deflections. Although the damage level can also be defined in terms of the retrofitted structure since the retrofitted structure can be substantially different from the desired analytical model, However this damage level will only depict the improvement in respect of the structure that has been obtained after retrofitting and not the variation of both the retrofitted structure and the earlier damaged structure with respect to a reference analytical model.

The damage level at span 38.0 m and 42.75 m was more than other nodes even after retrofitting and shows flexibility at these nodal points which is not evident from visual inspection. The damage level of downstream truss is higher than that of upstream truss both before and after retrofitting (Figs. 17(a)-(b)). However for both trusses of the bridge the damage level decreased drastically after retrofitting irrespectively of the damage level (Figs. 18(a)-(b)).



(a) (b)

Fig. 17 (a) Damage level before retrofitting; (b) Damage Level after retrofitting



(a) (b)

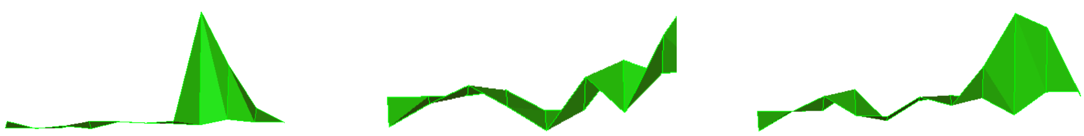
Fig. 18 (a) Damage level U/S truss before and after retrofitting; (b) Damage level D/S truss before and after retrofitting

Table 1 Comparative experimental and analytical frequencies for different state of the bridge

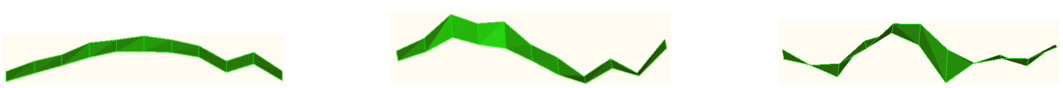
Mode	S-I	S-II	Anal	(S-I Exp)/Anal	(S-II Exp.)/Anal.
1	5.17	4.49	4.50	14.88	0.22
2	10.16	10.55	10.86	6.45	2.94
3	16.99	14.55	17.79	4.49	22.27

6.2 Frequency change

The vehicle based forced vibration test record affects the frequency of the structure (Kim *et al.* 2003). The frequency parameter corresponding to different state of the bridge is extracted. While for analytical model the determination of the modal parameter is independent of the applied dynamic force as a computational parameter since the modal parameters depends on stiffness, mass and damping. Taking these aspects into consideration work was directed in the direction such that the modal parameters at different state of bridge can be studied. Table 1 below shows the comparative frequency of the bridge for State I (S-I) and State 2 (S-II) obtained from the initial stage of damage, and partial retrofitting of bridge except deck and analytical model. The first modal frequency in the damaged stage was more than the retrofitted bridge because in damaged case node 10 acted as partial support and stiffness has increased due to decrease in span causing



(a) 1st Mode Shape (S-I) (b) 2nd Mode Shape (S-1) (c) 3rd Mode Shape (S-I)



(d) 1st Mode Shape (S-II) (e) 2nd Mode Shape (S-1I) (f) 3rd Mode Shape (S-II)

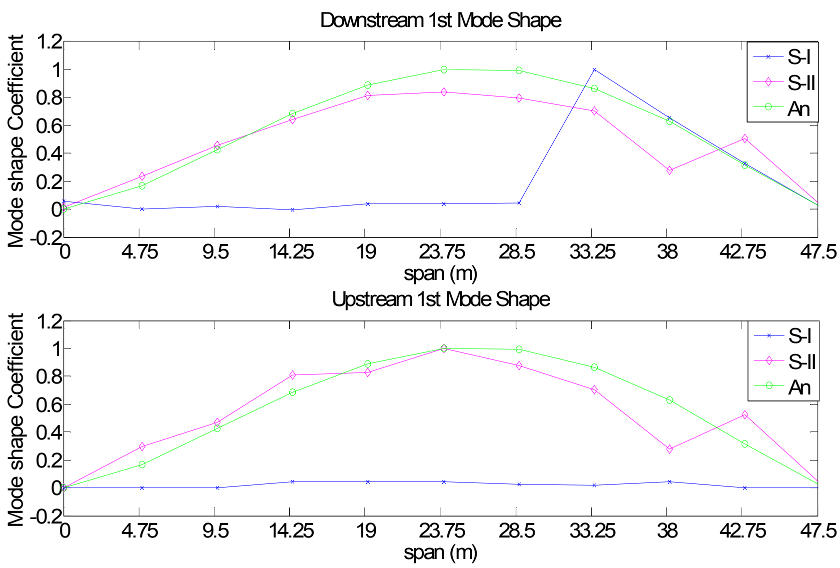
Fig. 19 Experimental mode shapes for different states increase in the frequency. The 1st modal frequency is close to that obtained through analytical modal. The results in 2nd and third mode were not in line with expected results.

6.3 Mode shapes

The mode shapes of the bridge are determined with the combined signal from the nodes of the both with upstream and downstream truss. Thus normalization of the mode shape obtained is either with respect to upstream or downstream truss. The difference between mode shapes of downstream and upstream trusses of the bridge is determined through comparison of graphical output of different states of bridge (Fig. 19), which is only a mere indicator that present an general improvement in the state of the bridge after retrofitting.

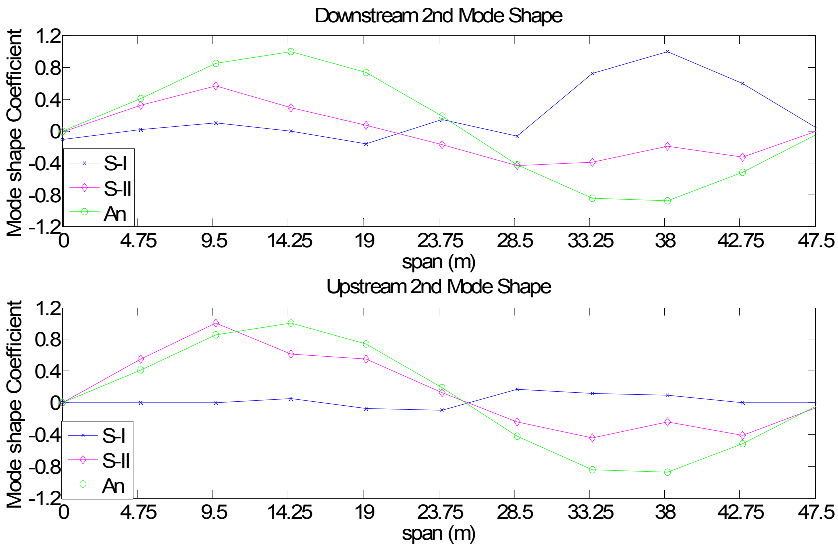
The comparison of the variation of the node for the state 1 and state II of the bridge is as follows:

- The first mode in State I of the bridge shows that the amplification in downstream truss was much more than upstream truss. There is a sharp kink in the mode shape of State I at nodes span 33.25 m due to settlement at 42.75 m. The amplification in the upstream truss and supports to 28.5 m in downstream truss was not observed since nodes at 32.5 m, 38 m and 42.75 m of downstream truss were observed to be highly flexible (Fig. 20(a)).
- Undesired 2nd mode shapes have been obtained for state I due to buckling of vertical member and bending of horizontal members of the bridge (Fig. 20(b)).
- In state II the first mode shapes are in line with the analytical mode shape except at 38 m and 42.75 m span of downstream and upstream truss (Fig. 20(a)).
- The 2nd mode shape of state II showed unexpected behavior at joints 14.25 m and 38 m in upstream truss.

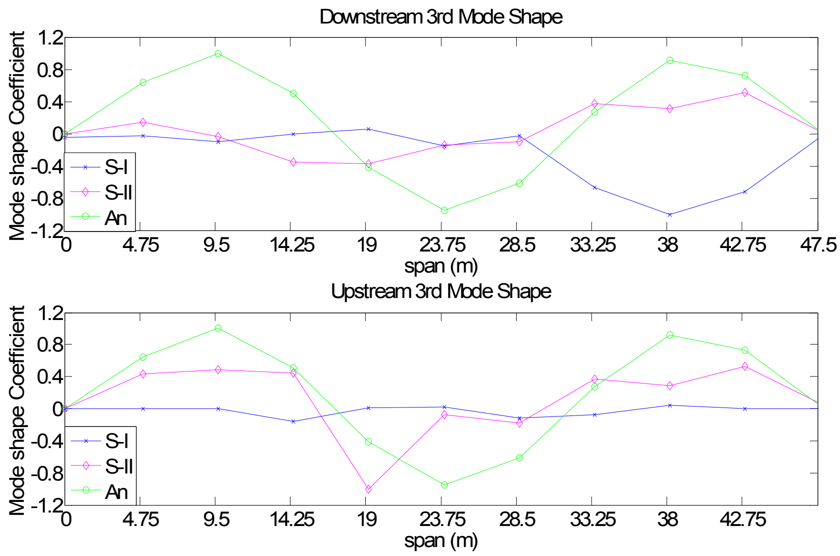


(a)

Fig. 20 (a) First; (b) Second; (c) Third Mode shape at different State and analytical Model of the bridge



(b)



(c)

Fig. 20 Continued

- In 3rd mode shape similar trend has been observed and shows improvement in the bridge at different stages of retrofitting (Fig. 20(c)).

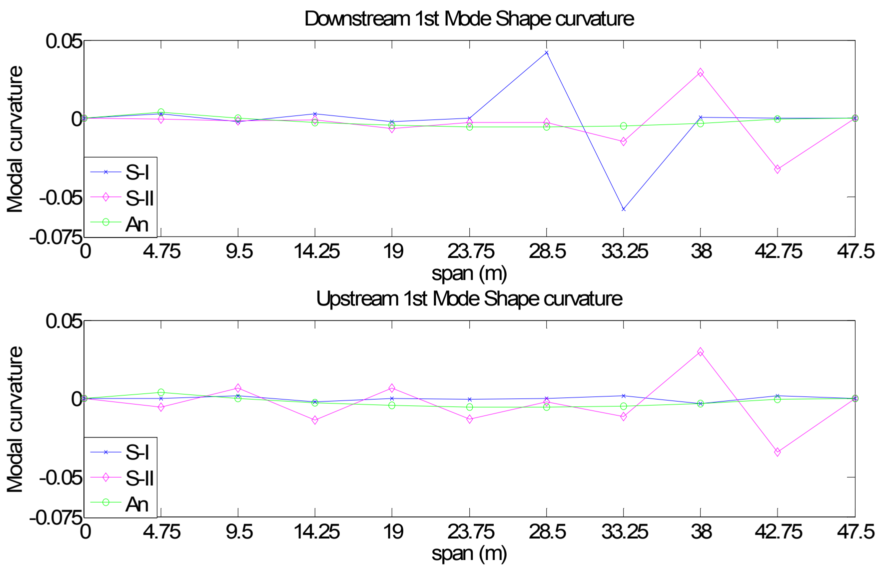
All the mode shapes of state II show a general improvement in the bridge after retrofitting. The difference in the behavior of upstream and downstream truss after retrofitting has reduced considerably.

6.4 Mode shape curvature

The mode shape although are able to reflect the flexible portion of the structure as a whole however the flexibility in the area of interest can still be amplified through modal shape curvature (Wahab *et al*. 1999) The difference in the mode shape of both downstream and upstream trusses for the different stages indicated some variation with analytical model for but did not presented the relative bending at different nodal locations of the bridge (Figs. 21(a)-(c)). The comparison of curvature variations in respect of analytical model of the bridge present uneven bending along the bridge. Although the estimation of mode shape curvature is not reliable, considering spacing and estimation error for the mode shapes. However in present case the study is oriented towards relative rather than absolute variation of the mode shape curvature. In Stage I the sharp change in the mode shape curvature in first mode between nodal point at 28 m and 38 m in downstream truss indicate higher amplification of the portion while the upstream truss does not indicate any variation in mode shape curvature indicating less damage compared to downstream truss. In 2nd and 3rd mode shape curvature the variation in mode shape curvature between points 28.5 m and 42.5 m indicates flexibility of nodal points. In stage II i.e., after retrofitting the nodal points at 33.25 m, 38 m and 42.75 m in both upstream and downstream truss shows different trend than that of the analytical results.

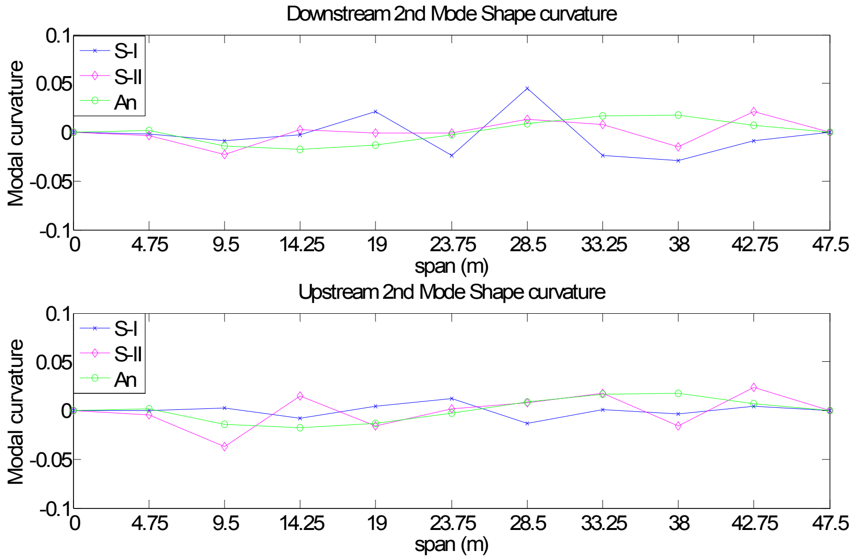
6.5 Modal strain energy difference

The mode shape and mode shape curvature involves the relative variation of the respective nodal parameter. However another modal parameter derivative i.e., the modal strain energy involves independent variation of energy level at different nodes of the structures. In the present study the modal strain energy difference ratio of an element (*MSEDR*) is defined which is the ratio

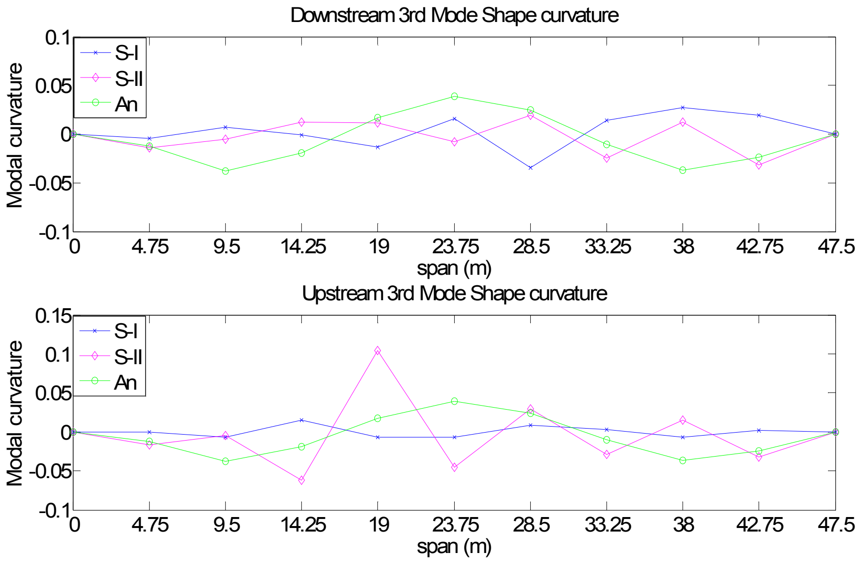


(a)

Fig. 21 Mode shape curvature for (a) 1st; (b) 2nd; (c) 3rd modes for different states of bridge



(b)

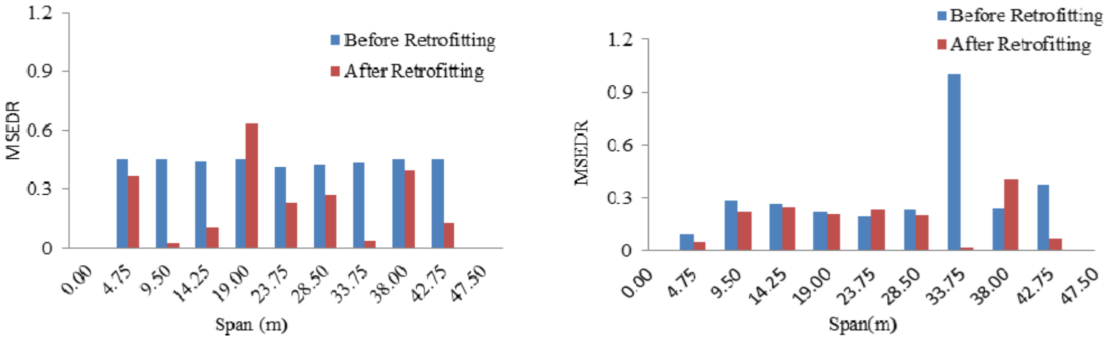


(c)

Fig. 21 Continued

of the difference of the Experimental Modal Strain Energy (MSE_{Exp}) and Analytical Modal Strain Energy (MSE_{Anl}) to the Analytical Modal Strain Energy (MSE_{Anl}).

$$MSEDR_{ij} = \frac{MSC_{ijExp} - MSC_{ijAnl}}{MSC_{ijAnl}} \tag{1}$$



(a) (b)

Fig. 22 (a) Downstream; (b) Upstream modal strain energy difference ratio for State I and State II

The average of the summation of $MSECR_{ij}$ for the first three modes normalized with respect to the largest value $MSECR_{i\max}$ of each mode is calculated and $MSEDR_j$ is the damage localization indicator of jth element determined as

$$MSEDR_j = \frac{1}{3}\sum_{i=1}^{3}\frac{MSECR_{ij}}{MSECR_{i\max}} \tag{2}$$

The *MSEDR* of any damaged element is larger than any other undamaged element. Elements that are linked with the damaged have a smaller *MSEDR* value, and, if an element is far away from the damaged element, the *MSEDR* of this element will be much smaller. In multiple damages in the structure similar results can be obtained (Shi *et al*. 2000b). The damage localization results are shown in Figs. 22(a)-(b) with the *MSEDR* plotted against the span. *MSEDR* in general decreased with retrofitting due to decrease in damage level in both upstream and downstream trusses. When the bridge was retrofitted and open to traffic, flexibility at most of the nodal span in both the truss of the bridge was not observed. However, higher *MSDER* values were obtained at 19 m and 38 m in the downstream and upstream truss respectively.

7. Conclusions

- The total station which is generally available with owner agency of the bridges can be used to determine the deflection of the bridge for damage assessment. However this damage level obtained from the deflection parameter presents a misleading relative damage at all the nodal points with maximum level at the affected location.
- In the present study as the damaged bridge had settled at one of the intermediate nodes, the decrease in the bridge frequency was obtained after retrofitting. The increase in stiffness due to retrofitting was less than the decrease in stiffness due to the increase in the bridge length.
- Although after retrofitting the structural repair might pass visual inspection test but undesired flexibility can still be obtained at the damage location from the deflection measurement.
- All the mode shapes from lower to higher showed higher flexibility at the damage location. The non-similar behaviour of the upstream and downstream truss divergent to the analytical

mode shape of the bridge is attributed to different damage level of the truss. The retrofitted joint of the bridge did not exhibit the requisite behavior through mode shape thus signifying the importance of generating bridge signature after retrofitting.

- The shift in all the modal curvature amplification of downstream truss of the damaged bridge is due to the intermediate node which acted as the support due to settlement. After retrofitting amplified modal curvature for the downstream truss was obtained at the repaired area. However unanticipated increase in the mode shape curvature of the upstream truss at other nodes was also observed.
- The modal strain energy change clearly indicates that the defective portion showed erratic modal strain energy values. The structure with undamaged element distributes the load efficiently through the development of strain whereas the damaged elements developed higher strain values.

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