

Time–frequency and wavelet-based study of an old steel truss bridge before and after retrofitting

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Abstract Signal-processing techniques have been widely used in structural health monitoring and non-destructive evaluation. Wavelet analysis, a relatively new mathematical and signal-processing tool, for damage detection in various civil and mechanical structures. It is a time–frequency analysis that provides more detailed information about nonstationary signals which traditional Fourier analysis miss. This paper aims to provide the damage identification in an existing 100-year-old deck-type steel truss bridge using frequency- and time–frequency-based approaches. The dynamic testing of steel bridge was carried out using accelerometers for the damaged state and after partial retrofitting under similar environmental conditions and instrumental set up. The comparison is carried out using power spectral density, short-time Fourier transform, and wavelet packet transform with respect to both the

upstream and the downstream trusses of the bridge. Higher and uniform dissipation of energy at resonating-frequency of the respective node after retrofitting showed intactness of joints. The variations of power spectral density in the first mode of the upstream and the downstream trusses clearly revealed improvements in the bridge signifying the importance of generating a signature of bridge before and after retrofitting. The status upgradations for the upstream and the downstream trusses obtained were different due to differential levels of damage in the bridge. Also, after retrofitting, the structural elemental behavior obtained was not the same as desired.

Keywords Steel truss bridge · Frequency · Power spectral density · Short-time Fourier transform · Wavelet packet transform

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1 Introduction

Nowadays, transportation plays an important role in various aspects of advanced human life, and bridges are among the most expensive investment asset of any country's civil infrastructure. They have a long service life compared with most commercial products and are rarely replaceable once erected [1]. Most of the Indian bridges had been constructed several decades ago and over the service life of a bridge; its constituent materials are continually subjected to fatigue and wear–tear due to dynamic vehicular loads [2]. Due to continuous increase in traffic volume and axle loading, degradation of bridge components have been observed [3]. In India, mostly the assessment of highway bridges is done by visual inspections which do not detect and notify overall behavioral

defects in the structure [4]. However, these limitations can be overcome by vibration-based methods [5]. A successful condition for the assessment relies heavily on the accuracy of the measured vibration properties, such as natural frequencies and mode shapes of structures [6].

In the present work, a single-span-damaged steel truss bridge is studied through the modal parameter extraction methods, considering vibration signals either stationary or nonstationary. The signal is often transformed to different domains in order to better interpret the physical characteristics inherent in the original signal [7]. The modal parameter identification can be divided into two categories, namely, frequency-domain identification and time domain identification. It transforms measured time domain data to frequency domain with FFT and is identified with power spectral density (PSD) [8]. The Fast Fourier transform (FFT) is a perfect tool for finding the frequency components in a signal [9]. The FFT is being used for the last two decades and is suitable for stationary signals only [10]. The time domain identification uses comparison statistical data of different state of structure. For small and simple structure, both time and frequency domain analysis methods can achieve satisfactory result. However, in large-scale complex structures such as bridges, the modal parameters can access the needed accuracy only after a large number of repeated experiments [11–13]. In case the recorded signal is nonstationary, the extracted modal parameters and their derivatives are always in question if FFT is used for modal parameter extraction [14–16]. Another disadvantage of FFT is that the frequency components can only be extracted from the complete duration of the signal [17]. Thus, any variation in the frequency content during period of vibration cannot be worked out by FFT. The structural elements also exhibit geometrical non-linearity during vibration [18] due to which the structural frequencies are non uniform during the vibration time span. The time–frequency-based methods such as short-time Fourier transform (STFT) and wavelet-based methods are best suited for such applications [19].

The wavelet transform is used for the determination of modal parameters such as natural frequencies, damping ratios, and mode shapes of a vibrating system [20]. The wavelet packet transform (WPT) based component energies with respect to local changes in the system parameters are derived analytically based on the dynamic response sensitivity [21]. In this paper, Wavelet analysis has been considered for damage detection and structural health monitoring and provides a powerful tool to characterize local features of the signal. It is the rationality of these methods that time–frequency-based methods justifies its adoption.

2 History and description of steel truss bridge model

The bridge considered for experimentation had been constructed on the National Highway 20 on Bander Khad near the village Samloti Himachal Pradesh, India by the British Company named as Dorman Long & Company Limited, Middlesbrough, England in the year 1920 (Fig. 1).

The steel sections used were as per the British Standard specifications of 1903 which were imported at the Karachi Stock yard and assembled at the site [22]. Due to increase in traffic load and intensity on the National Highway and being single lane with carriageway of 3 m only, the State Authorities constructed a new Prestressed Concrete Bridge adjacent to this bridge, and old bridge was dismantled and reassembled by the State Public works Department on Bathu Khad on Chughera Kandi Road, District Kangra, Himachal Pradesh, India in the year 1985. This bridge has been working satisfactorily for more than two decades, but in December 2009, settlements were observed in the Bridge Deck. Due to the passing of heavy vehicle and lack of maintenance, end members have buckled and the bridge was closed for traffic to prevent any further damage.

As-built drawing and design data were not available, an extensive investigative program was chalked out, comprising preparation of the 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 [23].

This bridge is simply supported with a hinge support at one end and roller support at the other end. The bridge is having a span of 47.5 m with 10 equidistant panels of 4.75 m each. The deck of the bridge is made up of corrugated metal sheets of 8 mm thickness with overlay of bituminous premix carpet of 40 mm thickness as wearing coat. The vertical members and diagonal members are made up of built-up section as in Fig. 2. The members of the trusses have been joined with rivet connections. The structural modal parameters were determined through the analytical model of the bridge generated in SAP2000 [24] as per the built-up drawing of the bridge to achieve the maximum possible accuracy in the analytical model (Fig. 4). The analytical frequencies of the bridge obtained are 4.50, 10.86 and 17.79 Hz for the first, the second and the third mode, respectively, in the vertical plane of the bridge.

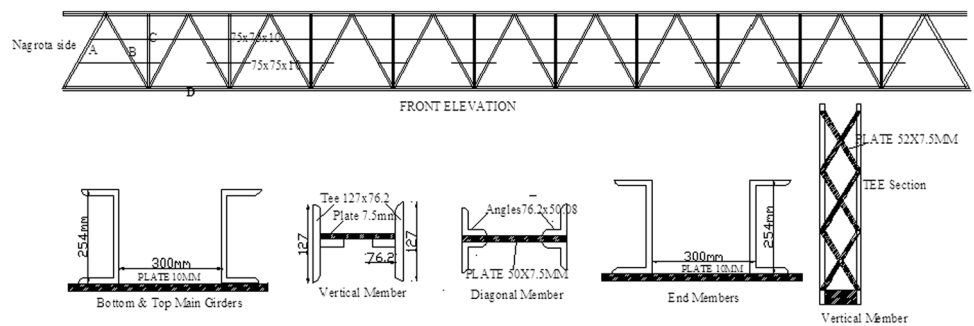
3 Experimentation

The experiment consists of measuring the response generated with the forced vibration and ambient vibrations with single forward and backward movement of the vehicle

Fig. 1 Steel truss bridge of span 47.90 m over Bathu Khad



Fig. 2 As-built drawings of steel truss bridge



over the bridge. The acceleration responses were measured before and after retrofiting. The experiment on the steel bridge was carried out to compare the behaviors of steel truss bridge before and after retrofiting. The basis of the speed selection for the experiments was the possibility of minimum and the maximum speed required for a particular bridge to see whether all the modes of the structure get excited or not. Any variations in the expected behavior of the trusses can be due to a particular steel member being ineffective in taking the stress or the joint has lost its full rigidity and is transferring the load to other members with partial rigidity or the bearing is ineffective in transferring the load to the substructure [25]. In the present study, it has been considered that the various factors such as speed of vehicle, temperature, surface roughness, etc., have similar effect on whole of bridge and not specific to any particular node. Although these factors will have an effect on the individual values of the results, but they get nullified when the relative variations are considered.

The following parameters such as sensor location, input motion, and sampling frequency are considered in order to perform the experiments under the same setup before and after retrofiting. The details of the test model are as follows.

3.1 Sensor and data acquisition system

The sensors used were dual channel MEMS accelerometers having a natural frequency of 150 Hz. Accelerometers provide voltage output, which is proportional to the acceleration of the points of placement. This analog acceleration time history is then fed to a data acquisition system. The data acquisition system, first, conditions the analog signal to its requirement through signal-conditioning amplifier, and then this conditioned analog signal is fed to an AD converter where digitization at a prescribed sampling rate takes place. The digital data are then stored in hard disks of the laptop in ASCII format. The basic

Fig. 3 Typical sensor location of one of setup on the bridge

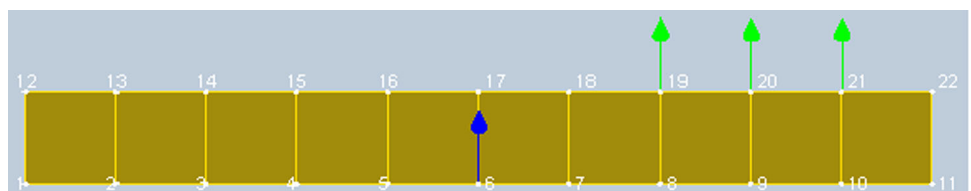


Fig. 4 As-built analytical model of 47.50 m deck-type steel truss bridge

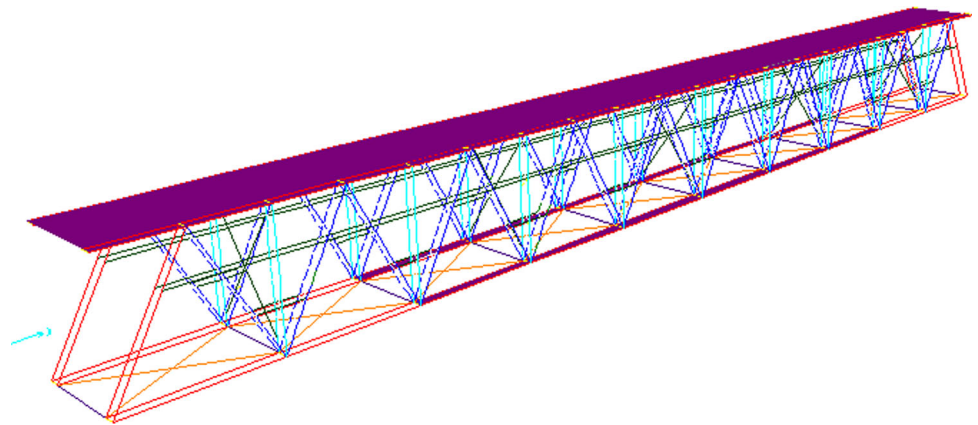


Fig. 5 Bottom and end members **a** before retrofitting **b** after retrofitting

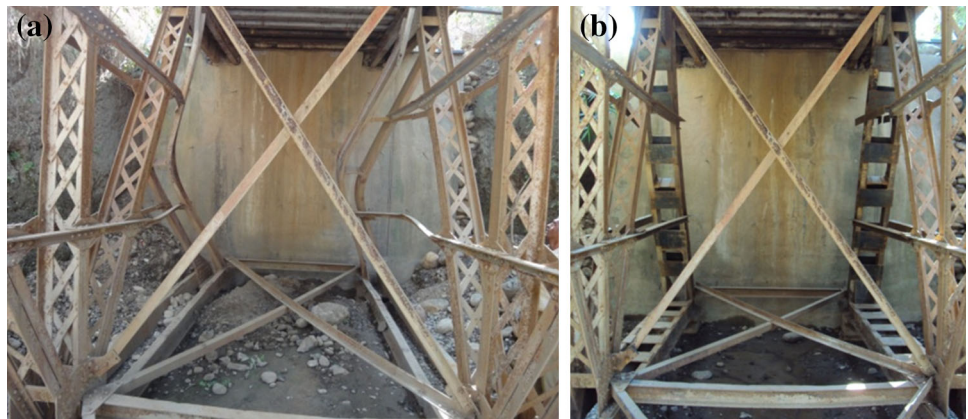


Fig. 6 Retrofitted sections

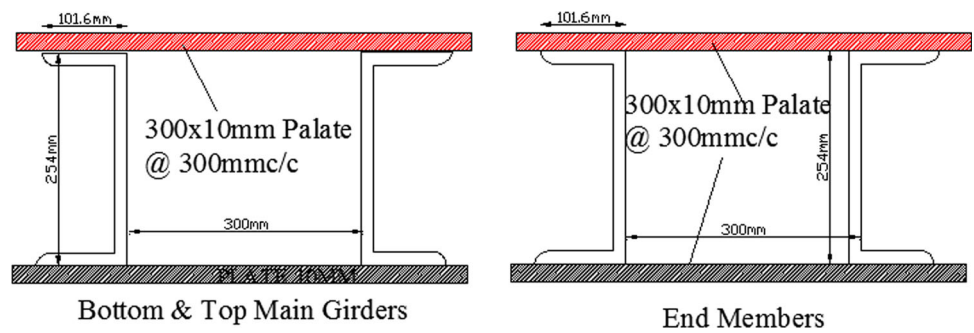


Fig. 7 Bearings before retrofitting and after retrofitting



Table 1 Peak frequencies along the span on the downstream truss and the upstream truss of bridge

Span condition	Node 1/12	Node 2/13	Node 3/14	Node 4/15	Node 5/16	Node 6/17	Node 7/18	Node 8/19	Node 9/20	Node 10/21	Node 11/22
First mode											
B.R.	5.15/5.34	4.52/5.32	5.18/5.08	NP/5.24	5.20/5.24	5.18/6.93	5.23/6.27	5.2/6.37	5.15/6.12	5.10/NP	5.02/NP
A.R.	NP/NP	4.59/5.44	4.59/5.07	4.37/5.15	4.47/5.10	4.36/5.10	4.38/4.90	4.48/5.32	4.47/5.15	4.48/5.68	NP/NP
Second mode											
B.R.	NP/9.94	12.09/9.1	10.1/12.48	NP/9.74	NP/10.30	10.13/12.80	10.43/11.86	NP/10.62	10.2/11.86	10.3/NP	9.78/NP
A.R.	NP/NP	9.50/9.91	10.18/11.03	10.18/9.86	10.48/12.5	11.46/12.03	11.69/11.10	9.32/10.52	10.51/11.67	10.46/12.76	NP/NP
Third mode											
B.R.	NP/15.38	13.08/14.36	12.85/19.84	NP/15.33	13.23/15.33	NP/14.56	13.75/16.74	12.30/14.67	13.23/14.52	13.33/NP	13.95/NP
A.R.	NP/NP	13.1/16.33	13.31/16.47	12.37/17.09	13.38/16.80	12.46/15.03	13.35/16.87	14.44/16.43	14.37/14.47	14.45/15.91	NP/NP

B.R. before retrofitting, A.R. after retrofitting, NP not present

functions of data acquisition system are achieved by using vendor-supplied interactive software and hardware systems. These systems can be standalone or coupled to a computer and have the facility of acquiring simultaneous multiple channels of data from various sensors. The data acquisition system acquired for this particular case study for generating output of the response of the structure had the desired specification to generate the output which has sufficient information to extract the modal parameters after processing accurately. Data acquisition system used for this work was KI4100 Model of Kaptl Instrumentation make. Further specifications related to this model and make of the data acquisition system used can be obtained from the Kaptl instrumentation website—www.kaptl.deindia.com.

3.2 Sensor location

To obtain the distributed response of the structure, sensors were placed on bottom nodes at the intersection point of the vertical and diagonal members in the vertical direction of motion on both the trusses of the bridge. In a single set, four sensors were used for the measurement. Thus, vertical acceleration time histories at 22 locations were measured in six sets. Each sensor location has been identified with unique nodal coordinate number. Figure 3 shows the detail of the placement of the sensors position on the bridge with the nodes named as $n1, n2, \dots, n11$ on the downstream, and $n12, n13, \dots, n22$ on the upstream. One accelerometer has been used as reference sensor. The measurement of the node in the bridge was carried out along the vertical direction. The direction of the installed acceleration pick up was accounted for during the analysis of the data.

3.3 Processing of recorded data

Power spectral density by Welch’s method, STFT, and WPT were determined using MATLAB environment [26] from the recorded data before and after retrofitting. The digitized output signal recorded by the data acquisition system was used for the conditional diagnosis of steel bridge. The sampling rate for acquiring the signal was kept as 200 samples per second (sps) during the whole of the experiment. The vibration was recorded at high sampling rate, and DC offset was carried out before analyzing the signal. The vibration signal was down sampled to 50 sps to concentrate on modal frequencies because our model frequencies are in lower range. This down-sampled signal is sufficient to provide information regarding modal frequencies up to 25 Hz, and three modes can be covered in this range.

Fig. 8 Power spectral densities of Nagrota Bridge **a** before retrofitting **b** after retrofitting for the downstream truss

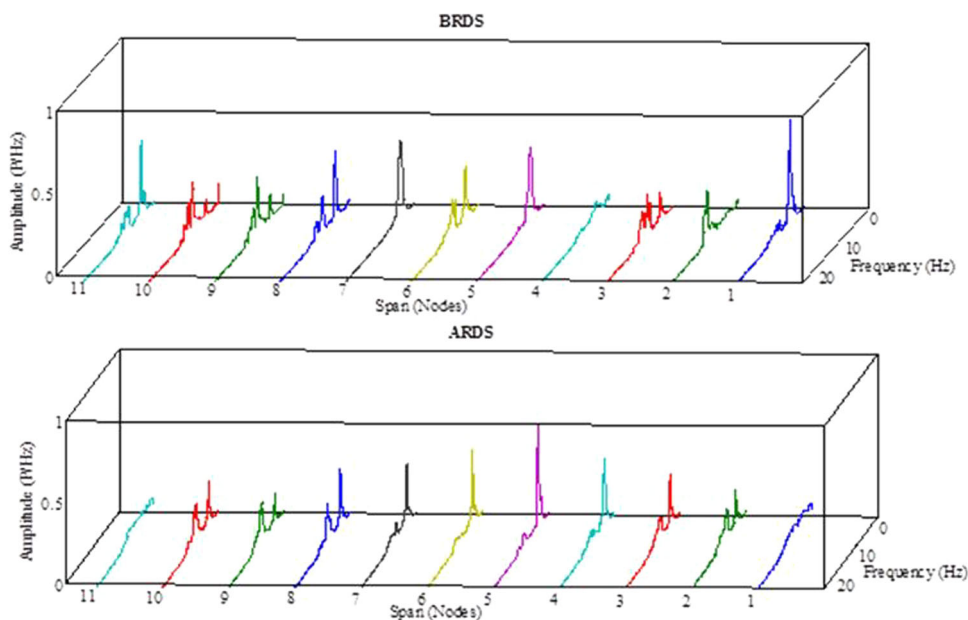
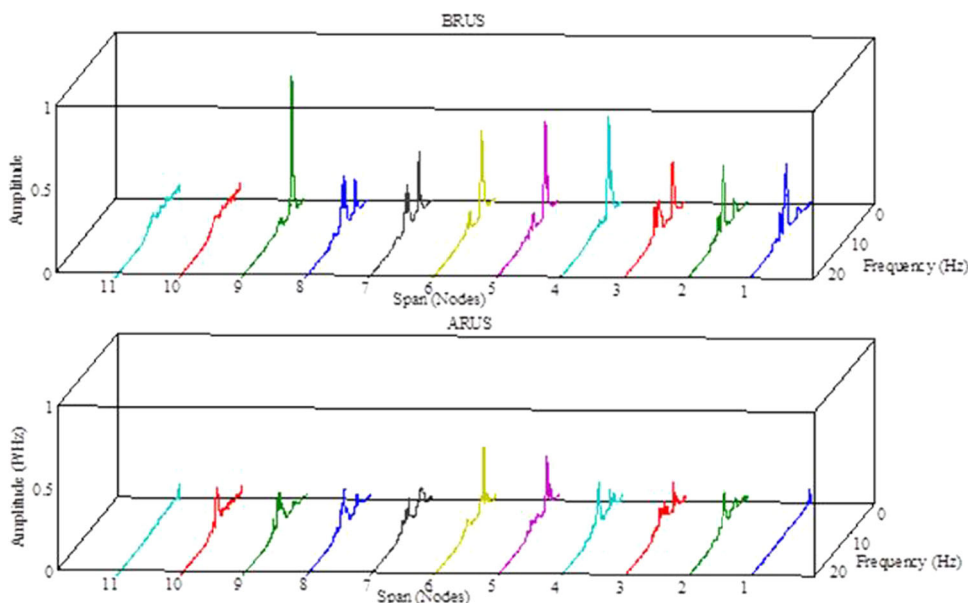


Fig. 9 Power spectral densities of Nagrota Bridge **a** before retrofitting **b** after retrofitting for the upstream truss



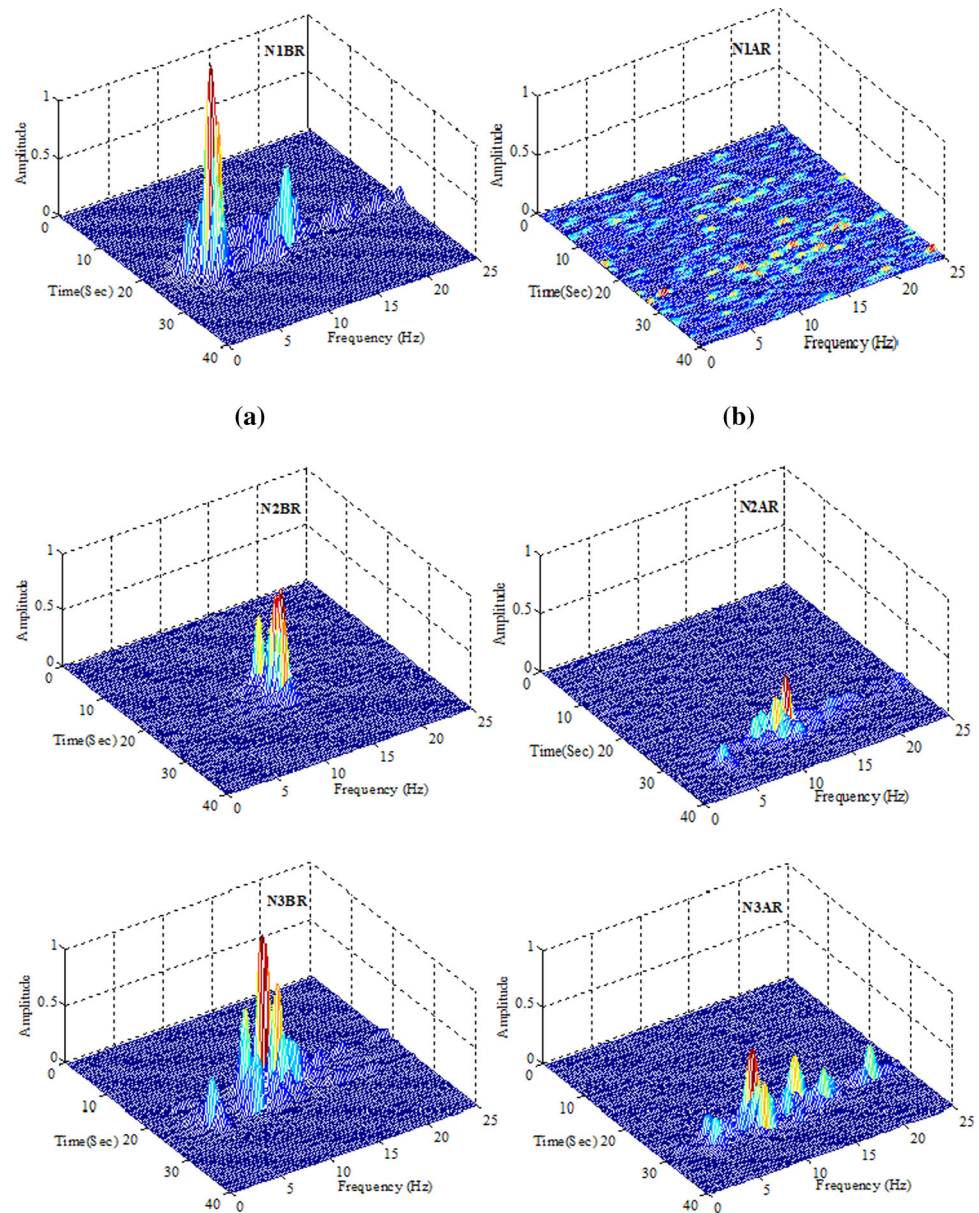
4 Retrofitting

Damage identification and retrofitting of the existing bridges is important as the cost of the monitoring and condition assessment exercise is nominal when compared with the cost, in terms of resources used and revenue lost for replacing such a bridge based on its age [27]. Structural Analysis Program (SAP2000) was used for generating the analytical model of the bridge (Fig. 4) with the structural element as the truss members. The members are assigned specifically as-built cross section to achieve analytical model as accurately as possible. The bridge was analyzed

for Class A and Class B Loadings (IRC: 6 2000) [28]. The bridge was safe for Class B loading but required retrofitting for Class A loading.

The detected defects can be strengthened and further retrofitted as per the respective codal provisions of load and stresses [29]. The bridge was retrofitted for Class A loading. The end members have buckled due to overstress and were replaced with the same section (Fig. 5). In order to hold the main component of the built-up section in their relative position and to equalize stress distribution [30], the end members were stiffened with a plate of dimension 300 x 300 x 10 mm at a spacing of 30 cm c/c. A horizontal

Fig. 10 STFTs of node numbers from 1 to 11 before and after retrofitting for the downstream truss



stiffener of angular section $75 \times 75 \times 10$ mm at a height of 1.35 m from the bottom longitudinal member has been provided.

The bottom longitudinal members were straightened and stiffened with steel plate of $300 \times 300 \times 10$ mm size as shown in Fig. 6. 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 acted as fixed support since the soil has been filled in the rollers. The load was relieved from the bearings by jacking the superstructure, and a temporary support system was introduced. Since Rocker and rollers were in good condition, the 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

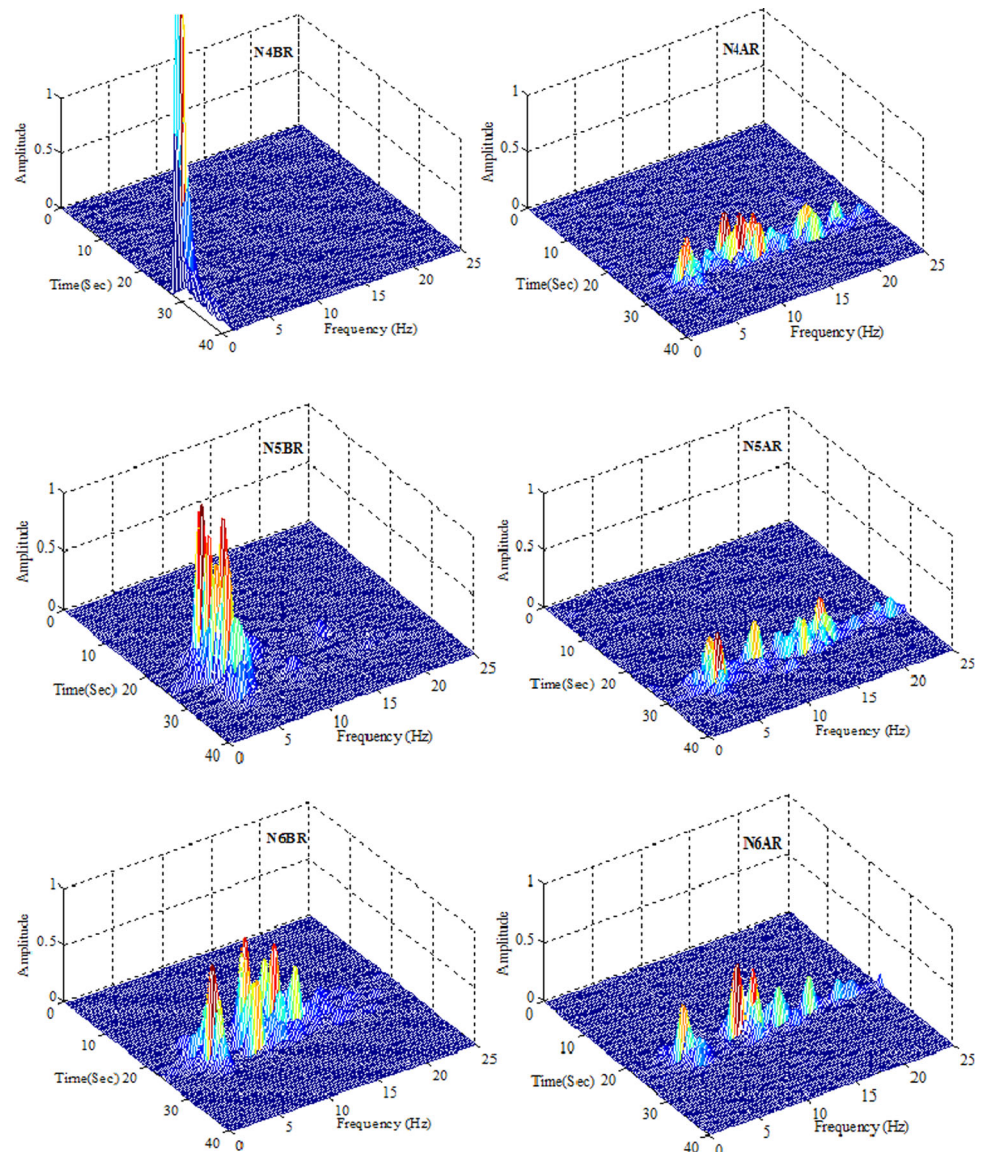
one. Figure 7 shows the status of bearings before and after retrofitting.

The state’s 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.

5 Parametric studies

The variation under the applied dynamic forces leads to different amplitudes of vibration in the structure. In the present study, these different applied forces are generated through the moving vehicle. The results obtained are compared in terms of PSD using Welch method, STFT, and WPT.

Fig. 10 continued



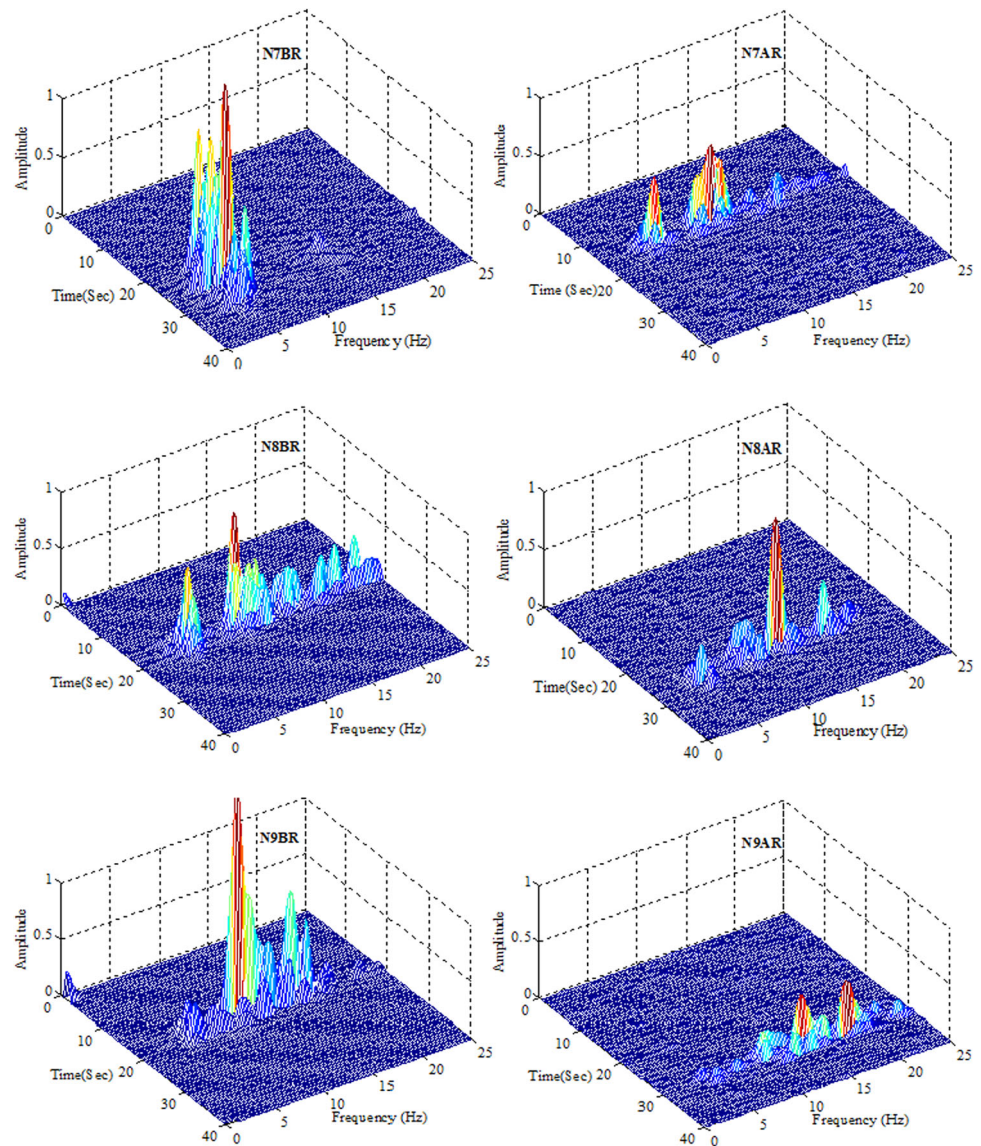
5.1 Power spectral density using Welch method

Considering mass and stiffness of the member as constants, the behavior of the structure changes with the variation in the joint flexibility. The PSD describes distribution of power of signal with frequency, which changes with joint flexibility [31]. Although, in analytical model, the modal parameters are independent of applied dynamic force as a computational parameter, they depend on stiffness, mass, and damping. Taking these features into consideration, joint flexibility variations have been determined at damage and intact conditions (after retrofitting). In the present study, the PSDs of vibration signals recorded from the moving vehicle on the bridge corresponding to damage and intact conditions are extracted using Welch method. Welch method is nonparametric method that includes the

periodograms that have the advantage of possible implementation using the FFT [32]. The well-known form of power density spectrum estimate called as periodogram is not a consistent estimate of true power density spectrum and hence does not converge to the true power density spectrum. Hence, the emphasis of classical Nonparametric Methods is on obtaining a consistent estimate of power spectrum through some averaging and smoothing operations performed directly on the periodogram or directly on the autocorrelation. The frequency peaks of PSD at different nodes before and after retrofitting are as shown in Table 1.

The frequencies of the damaged bridge vary from 4.52 to 5.23 Hz at different nodes for the downstream truss and have higher values than the frequencies of retrofitted bridges which vary from 4.36 to 4.59 Hz at different nodes.

Fig. 10 continued



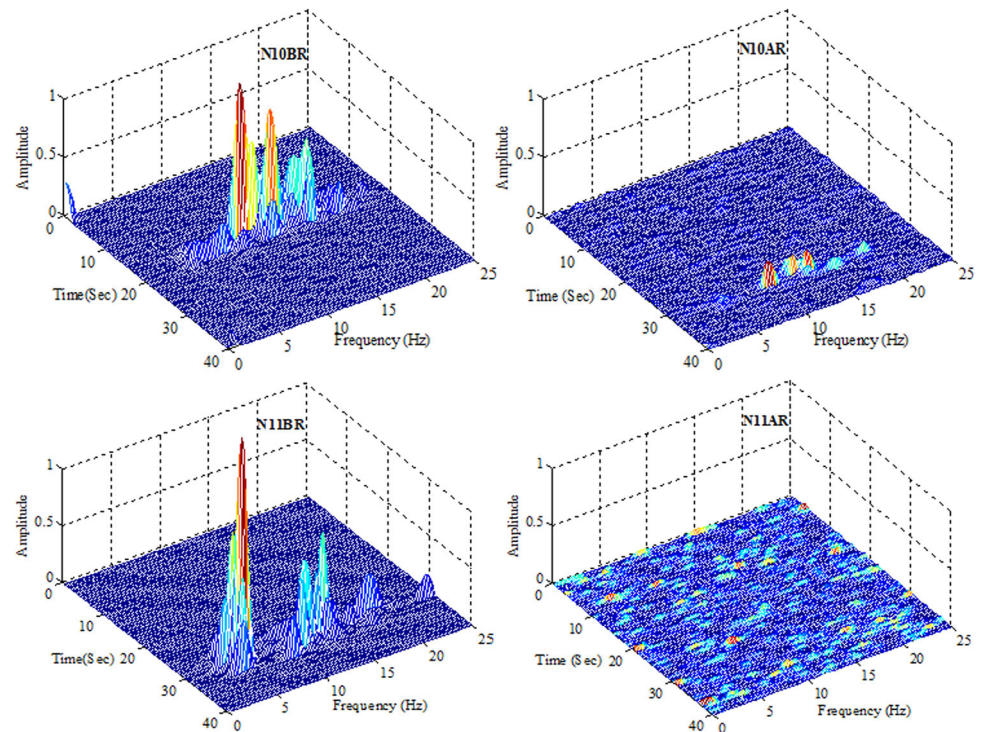
There are large variations in the frequencies of the damaged bridge at different nodes as the bridge is not intact, and different nodes are not behaving structurally perfect as expected. After retrofitting, the nodal frequencies vary from 4.9 to 5.68 Hz at different nodes for the downstream truss indicating improvement in the upstream truss. The higher values of the frequencies of the damaged bridge are due to the fact that effective span of damaged bridge is 42.75 m which is lower than that of a retrofitted bridge span of 47.5 m. This is attributed to the settlement of nodes 10 and 21 in the damaged state of the bridge. Similarly, in the upstream truss, the frequencies varied from 5.08 to 6.93 Hz, and the variation is more toward node 21 in the upstream truss of the bridge.

The PSDs of different nodes for the downstream and the upstream trusses before and after retrofitting are shown in

Figs. 8 and 9, respectively. It is observed that before retrofitting, node 4 did not vibrate as desired since this node did not generate the expected the first and the second modal frequencies. Similarly, node 21 is damaged because the node is not able to generate the modal frequency with desired amplitude. Such nonparticipation of the node in vibration of the structure is due to the flexibility introduced in the joint, thus, making the joint to be designated as damage joints. The modal frequencies were observed at the bearings indicating that the supports were not working as anticipated.

It has been observed from Fig. 8b that after retrofitting, nodes 4, 9, and 10 showed significant improvement in PSD curve. It means that the joints are intact and are able to generate the first mode. After retrofitting, nodes 1, 11, 12, and 22 do not show any nodal frequency because these

Fig. 10 continued



joints are now acting as supports indicating that bearings has been perfectly aligned. The first modal frequencies at different nodes for the downstream truss vary from 4.36 to 4.59 Hz, which are nearer to the analytical frequency of 4.50 Hz. Similarly, the second modal frequencies before retrofitting varied from 9.78 to 12.03 with an average value of 11.88 Hz and after retrofitting, it varied from 9.32 to 11.46 Hz with an average value of 10.42 indicating improvement in the variation in the frequencies and is nearer to an analytical frequency of 10.86 Hz. After retrofitting, the third modal frequencies were observed at all the nodes except at the supports indicating improvement in the bridge. After retrofitting, the first modal frequency for the upstream truss varies from 4.9 to 5.68 Hz with an average value of 4.69 Hz, which is closer to an analytical value of 4.50 Hz. The damage in the bridge resulted in the nonexcitation of some of the structural frequencies in the second and the third mode which got extracted after retrofitting showing improvement in the bridge.

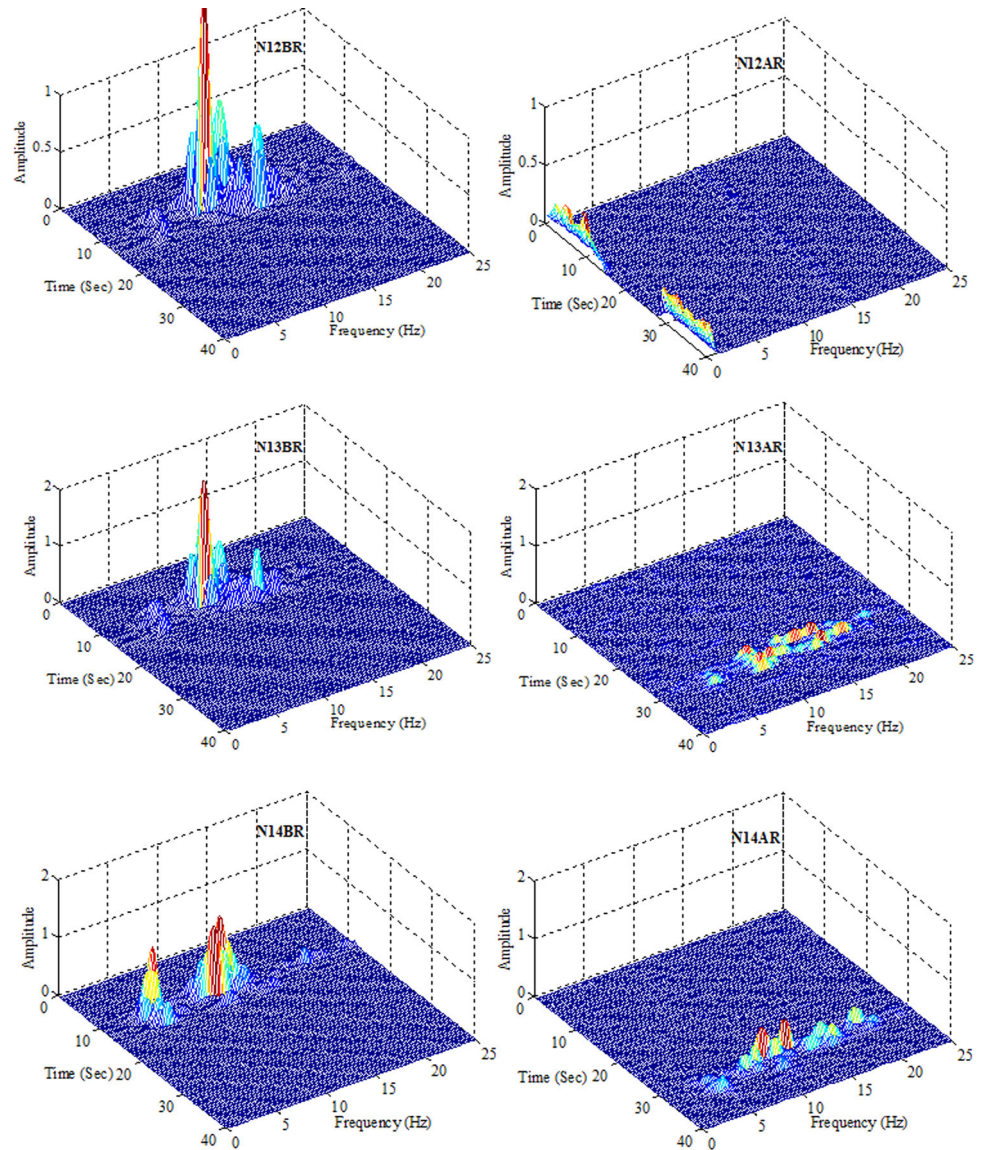
5.2 Short-time Fourier transform

The Fourier analysis performed on the signals to determine the content of frequency spectrum has some serious drawbacks. In the transformation of the signal-to-the frequency domain, information related to time is lost. If the signal is a stationary signal—properties do not change much over time—this drawback is not significant. However, for the present study of movement of a vehicle over

the bridge, the signals are expected to contain numerous nonstationary or transitory characteristics: drift, trends, abrupt changes, etc. Such characteristics are important from the sensitivity analysis aspect of the signal, and Fourier analysis is not sufficiently capable to detect such irregularities. STFT is an effort to correct this deficiency, in which only a small section of the signal is Fourier transformed at a time with windowing technique. STFT maps the signal in a two-dimensional function of time and frequency.

Therefore, the deficiencies in the nodal joints of the trusses along the span were further substantiated with the STFT. Time-dependent variation of joints can further be studied through the determination of frequency content of the random process [33]. The variation in the frequency content with time at different nodes of the structural member reflects the undesired flexibility at the nodes due to nonexcitation at particular structural frequency. The STFTs of the nodal responses before retrofitting were compared to the responses after retrofitting. The reduced excitation of the structural frequency bridge response after retrofitting of the nodal joints revealed the intactness of the bridge. Further, the distributed extraction of the bridge response at the target nodes revealed the nodal sensitivity after retrofitting. In the downstream truss (Fig. 10), nodes 1 and 11 before retrofitting generated nodal frequencies, but after retrofitting, both the nodes 1 and 11 did not generate modal response of a flexible joint, which showed that before retrofitting, supports were defective.

Fig. 11 STFTs of node numbers from 1 to 11 before and after retrofitting for the upstream truss

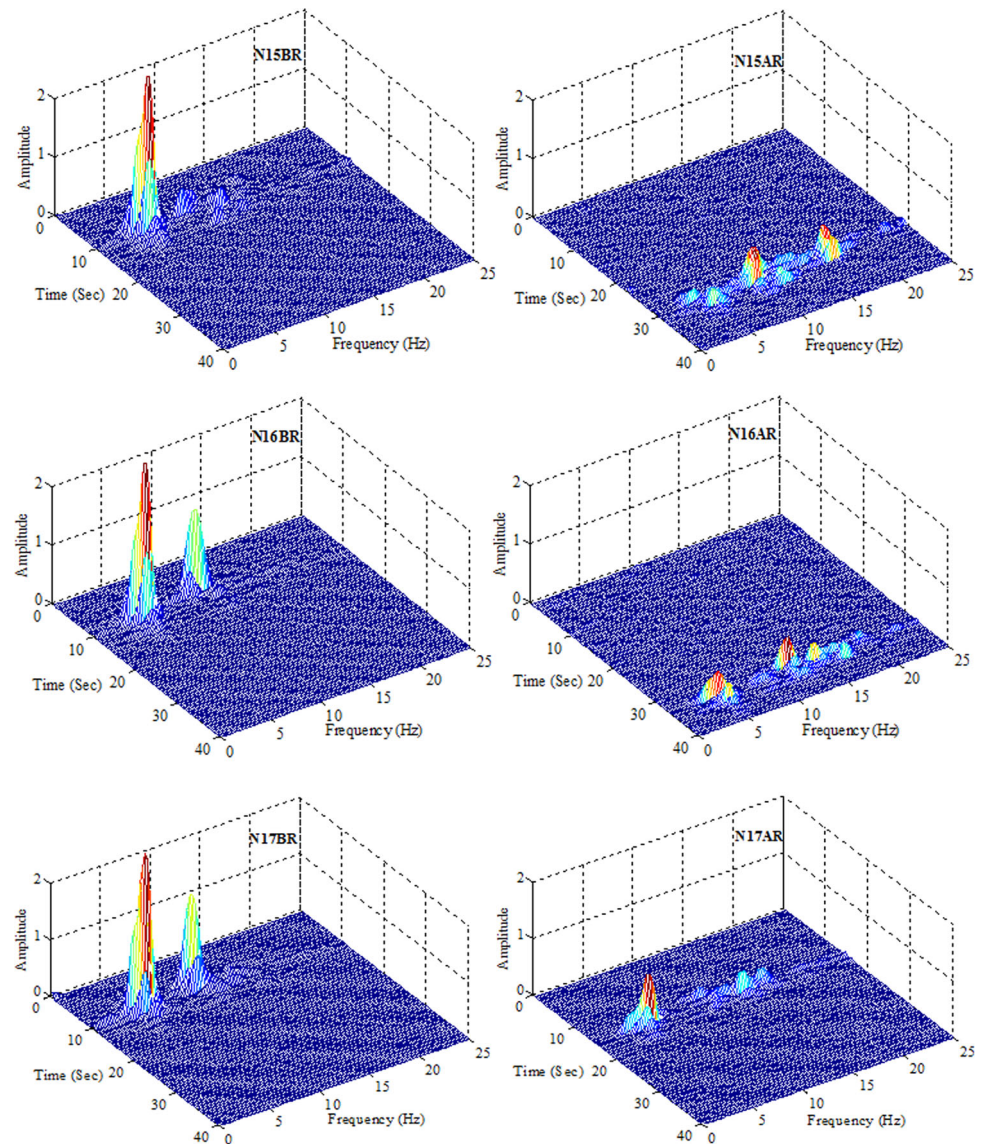


In the downstream truss, node 4 is damaged because it does not generate mode shapes before retrofitting, and after retrofitting, all the three mode shapes have been generated. Before retrofitting, node 5 and node 7 generated the first mode shape and did not generate the second and third mode shapes. After retrofitting, node 5 and node 7 generated all the mode shapes which are also evident in PSD. The Figs. 10 and 11 also show the degree of inappropriateness in the behavior of nodes, especially node 5 and node 7 as for longer time duration, the nodes vibrated at the fundamental structural frequency. The joints that are referred to as damage joint due to the induced nodal flexibility does not offer any stiffness and subsequently does not participate in generating the nodal frequency. The mode shape coefficients of the flexible nodes are amplified and shows undesired higher mode shape of the structure. In general,

the proper mode shapes have not been generated before retrofitting, and after retrofitting, the joints are intact and generated proper mode shapes.

In the upstream truss (Fig. 11), after retrofitting, both the supports, i.e., nodes 12 and 22, behaved perfectly and showed improvement in the bearing of the bridge. Before retrofitting, most of the nodes showed higher amplitude of the modal frequencies compared to those after retrofitting revealing that nodal joints are more flexible before retrofitting. The amplitude of STFT shows how the frequency evolves over the time and also distribution of power among the frequency content of the STFT spectrum. Higher amplitude of a particular frequency shows higher power in a particular frequency along with the reduced distribution of power across the other frequency content. In the present study, higher STFT amplitude for a particular frequency

Fig. 11 continued

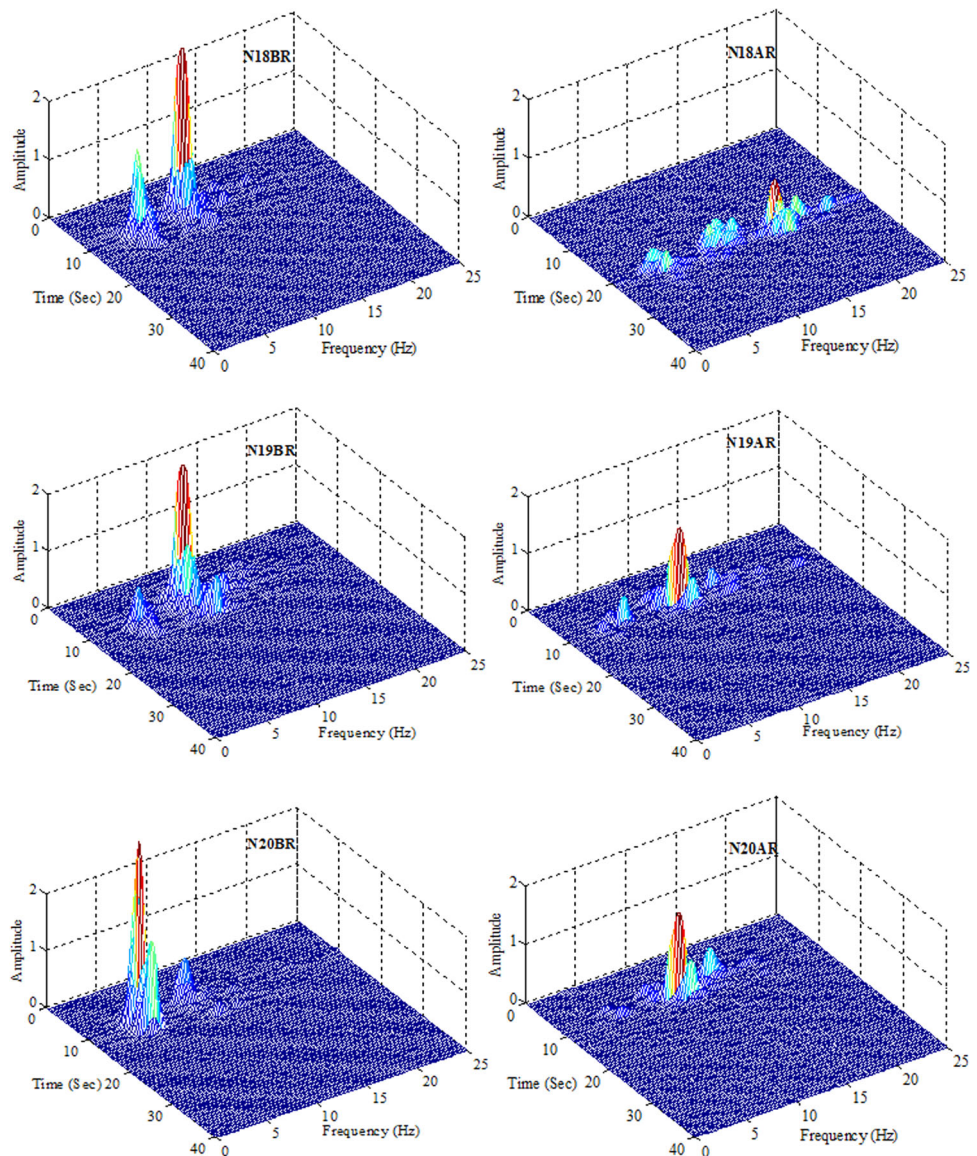


before retrofiting reflected concentration of the power and nonparticipation of other frequency mode shape due to lack of power. However, after retrofiting, the joint behavior changed and the uniform power distribution to the different frequency mode led to the reduced STFT amplitude w.r.t the situation before retrofiting. STFT at nodal point 21 before retrofiting does not generate any mode shape coefficient because this node acted as support as this joint had settled down. After retrofiting, this node generates the second and the third mode shapes.

The nodal frequencies indicated that the node did not behave identically. The variation in the amplitude of extracted nodal frequency indicated the degree of flexibility in the nodes. The nodal frequencies, both for the upstream truss and the downstream truss before retrofiting, although averaged around 5 Hz in the first mode,

12 Hz in the second mode, and 16 Hz in third mode, the flexible intermediate node, however, did not respond to frequency excitation (Fig. 12). The excitation of the support nodes 1, 11, and 12 of the bridge before retrofiting indicated inappropriate hinge and roller supports. The tendency of nonexcitation for the downstream truss before retrofiting in the first mode is more toward node 2 to node 4, than toward node 4 to node 7 in the second mode and further toward node 2 to node 8 in the third mode (Fig. 12a, c, e). However, after retrofiting, except nodes 9 and 10 in the first mode and node 2 in the third mode, all the nodes of the downstream were excited indicating the participation in the modal frequency of the bridge. In the case of the upstream truss before retrofiting, node 21 did not excite in the first and the second modes. However, most of the nodes did not excite in the

Fig. 11 continued



third mode indicating that the upstream truss was less damaged than the downstream truss (Fig. 12b, d, f). After retrofitting, except for the node 21 in the first mode in the upstream truss, all the nodes were excited in all the modes. Thus, the behavior of the upstream truss was much closer to the desired level than that of the downstream truss.

The relative nodal frequency amplitudes of both the downstream and the upstream trusses before retrofitting are higher in magnitude than those after retrofitting irrespective of the mode of vibration. In the downstream truss, except for the node 9 and node 10 and in the upstream truss except for node 21 in the first mode, all the nodes showed stiffness in the joints, Fig. 13. Both the downstream and the upstream trusses showed similar amplification after retrofitting for the first mode with higher-frequency amplitude toward the center of the bridge.

5.3 Wavelet packet transform

The difference in PSD and STFT of both the downstream and the upstream trusses of the bridge after retrofitting indicated improvement in the bridge, but did not present the relative differences quantitatively. STFT analyses the transient signal by assuming stationarity within short interval and thus is employed to extract the localized transient feature in certain cases. STFT provides some information about both when and at what frequencies a signal event occurs with limited precision, depending upon the window size. However, the fixed windowing used in the STFT implies fixed time–frequency resolution. The drawback of this method is that the accuracy of extracting frequency information is limited by the length of the window relative to the duration of the analyzed signal. Many signals require a more flexible approach where we can vary

Fig. 11 continued

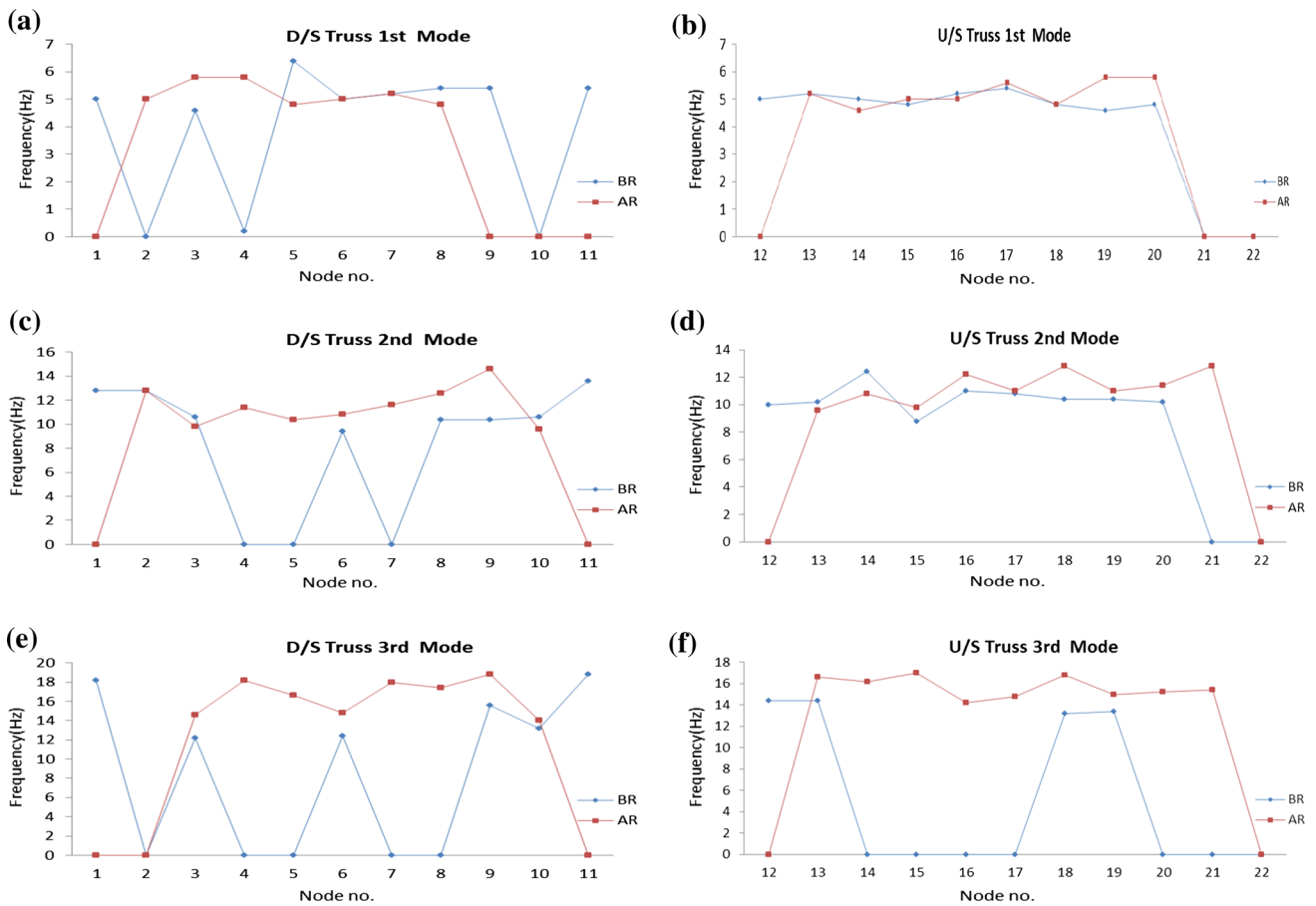
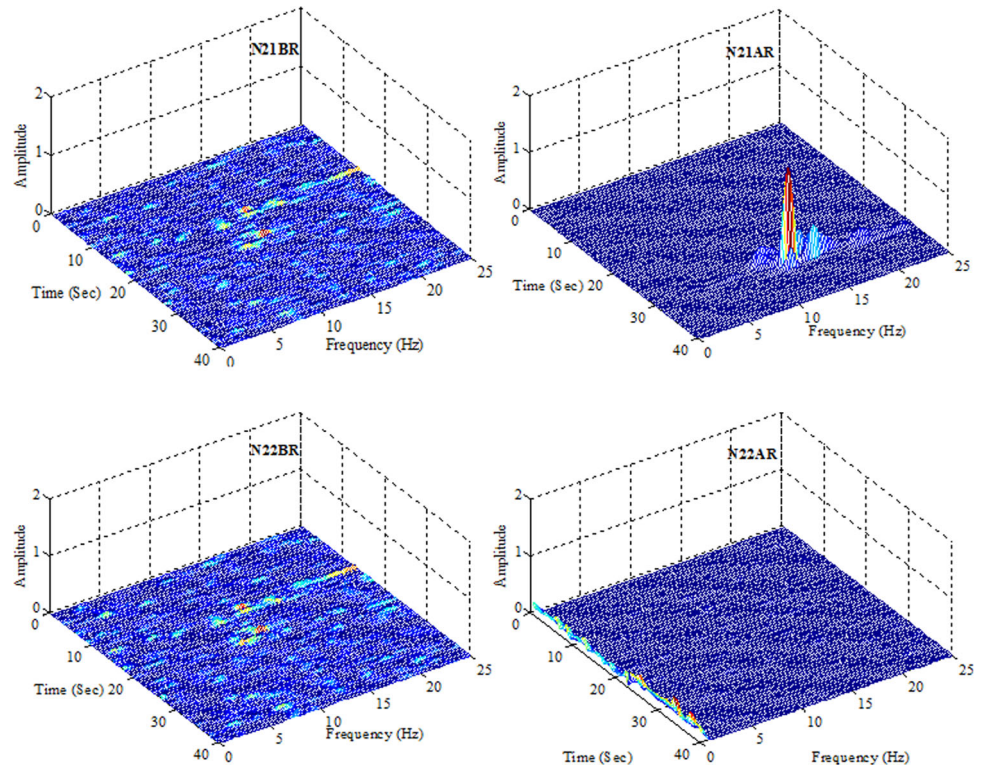


Fig. 12 Nodal peak frequencies of D/S and U/S trusses before and after retrofitting

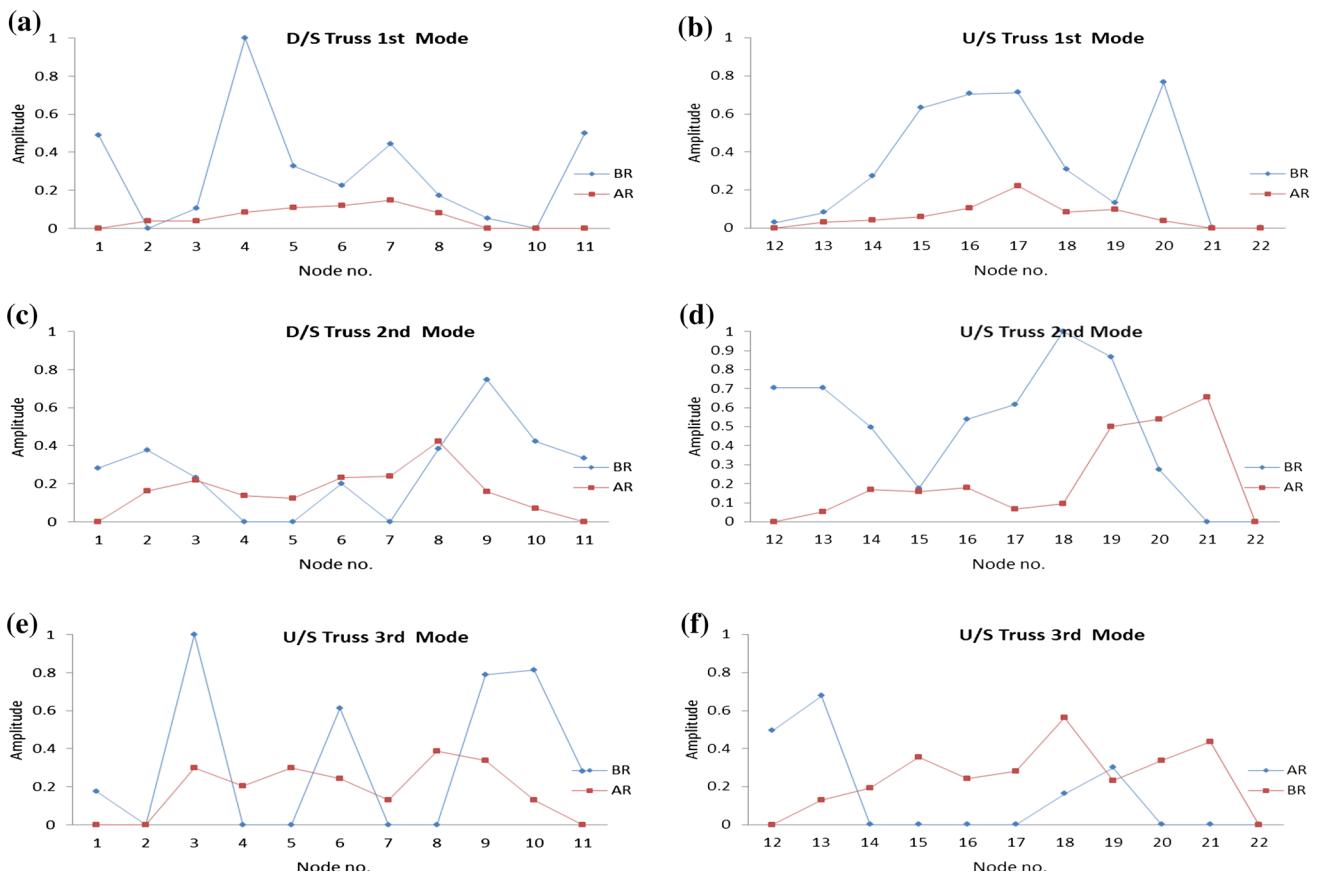


Fig. 13 Relative nodal frequency amplitudes of D/S and U/S trusses before and after retrofitting

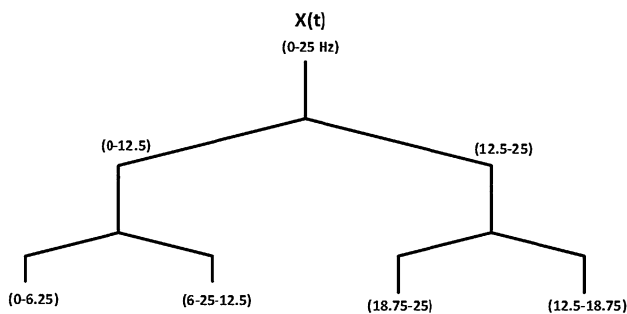


Fig. 14 Two-level wavelet packet decomposition tree

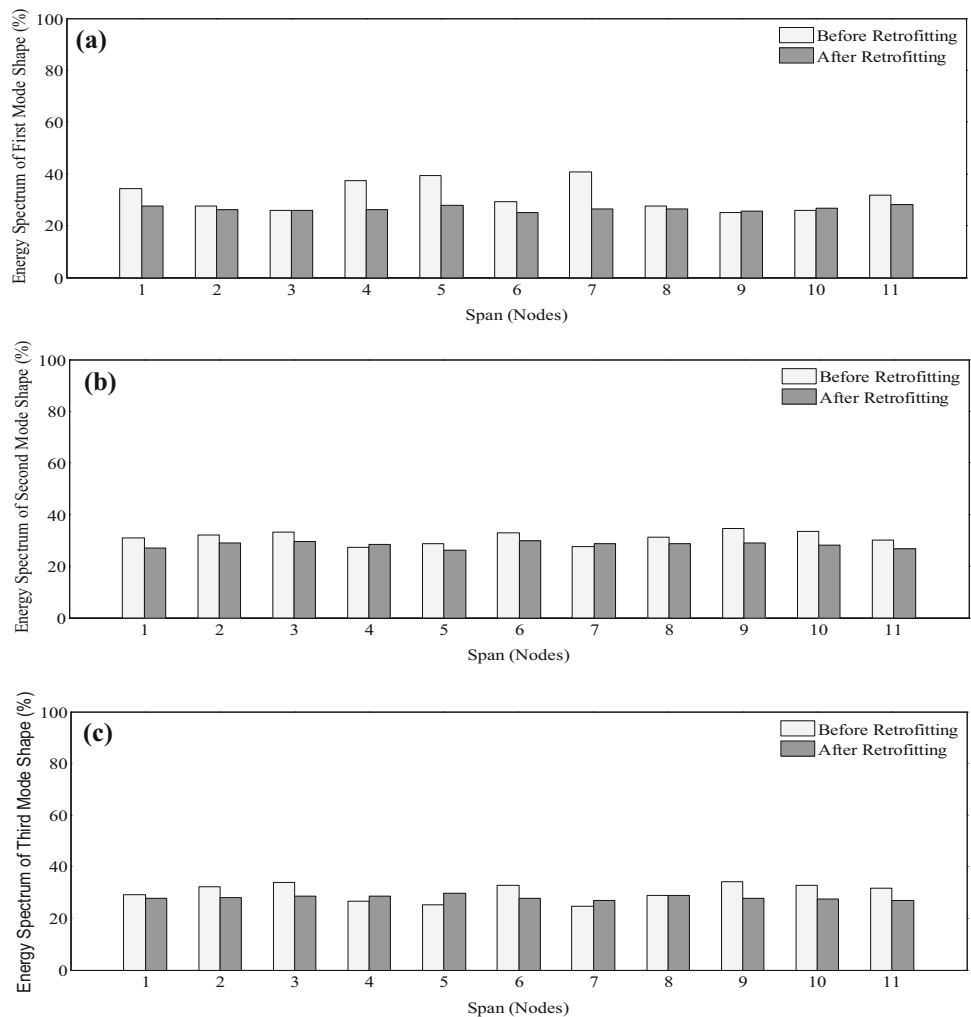
the window size to determine more accurately either time or frequency. For this, wavelet analysis can be used. Thus, Wavelet analysis represents the next logical step: a windowing technique with variable-sized regions. Wavelet analysis allows the use of long time intervals where more precise low-frequency information is required and shorter regions where high-frequency information is required.

Wavelet transforms have advantages when the signal is nonstationary as signal analysis both in frequency and time domains is allowed [34]. The WPT is a technique to

decompose a signal repeatedly into successive low and high-frequency components. Wavelet transforms have advantages when the signal is nonstationary since it allows signal analysis both in frequency and time domains. The only difference between WPT and DWT is that WPT decompose not only the approximation but also the details at a given level of decomposition. Therefore it is more flexible and have wider base for the analysis of signals. WPT divides not only the low-frequency subband, but also the high-frequency subband. In the wavelet packet analysis, the details as well as the approximations can be split, so a signal is split into an approximation coefficient and a detail coefficient. The approximation coefficient then itself split into a second-level approximation coefficients and detail coefficients, and the process is repeated. WPT also enables multi resolution damage detection since it can localize multifrequency bands in time domain. Wavelet transform- and wavelet packets-based damage detection is widely used in many vibration-based structural health monitoring and damage detection [35].

Continuous wavelet transform (CWT) is similar form of STFT with advanced resolution capabilities due to variable

Fig. 15 Energy spectra of **a** first, **b** second, and **c** third mode shapes before and after retrofitting for the downstream truss



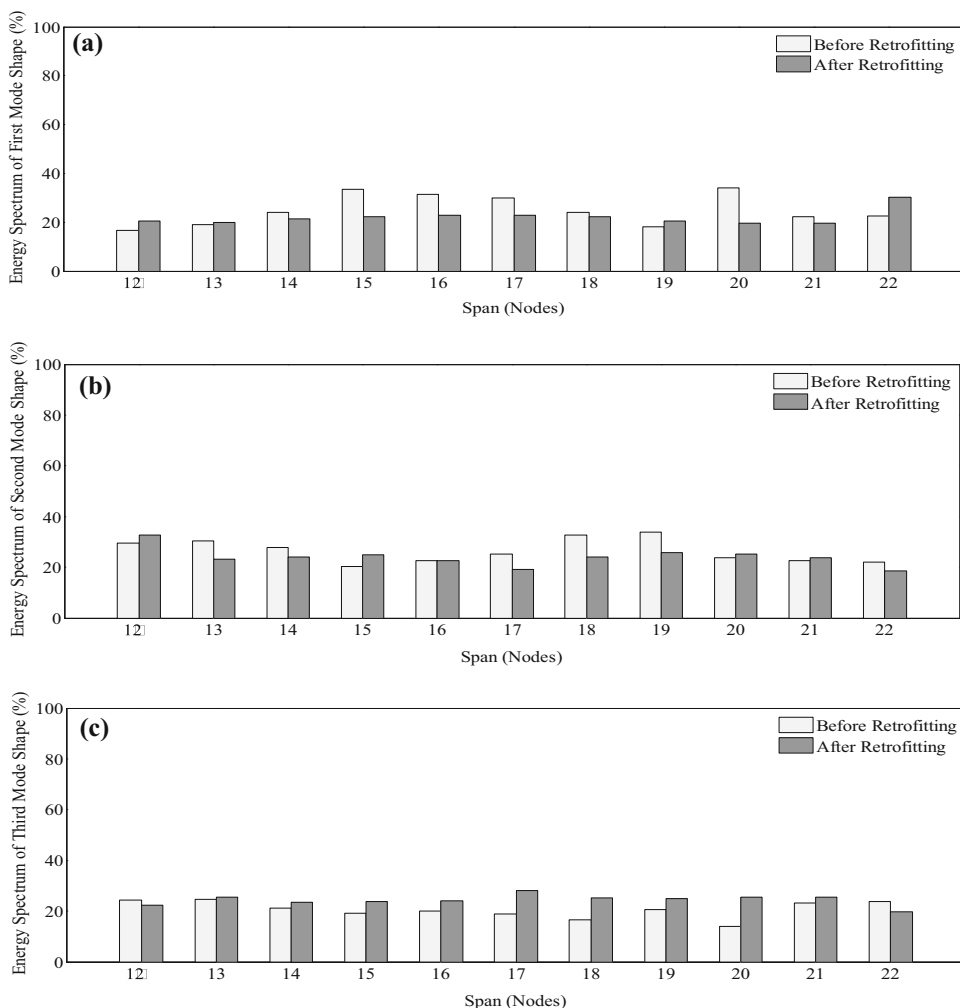
window length hence not adopted for further interpretation [36]. The Daubechies family db10 is used as a mother wavelet for wavelet packet analysis. The level of decomposition should be determined so that fundamental natural frequencies are separated into each wavelet packet. It is suggested to choose the wavelet packet of the frequency band that includes natural frequencies corresponding to fundamental modes. It is postulated that fundamental modes are more influenced by component damage than higher modes. Hence, in WPT the recorded signal is decomposed up to two levels and reconstructed because the frequency of interest was achieved up to this level. At level two, four components of the original signal are obtained with different frequency band (Fig. 14).

Figure 14 schematically illustrates a WPT-based decomposition process, where a two level WPT produces a total four sub-bands. The enhanced signal decomposition capability makes WPT an attractive tool for detecting and differentiating transient elements with high-frequency characteristics.

Further energy of each component is calculated using Parsvel's theorem, in damage state and after retrofitting. Since the comparison of the energy level is being used to determine the structural deficiency hence the accuracy of the obtained values are not the governing factor. The energy spectra before and after repair do not follow the same trend; sometimes it decreases and at other times, it increases. However, the energy diagram of the first mode shape (Fig. 15a) depicts that the downstream truss has significant improvement after retrofitting, as energy levels at various nodes have decreased. Figure 15b, c also exhibited the same result which belongs to the energy spectra of the second and the third mode shapes.

Energy levels at various nodes in the upstream truss show significant improvement in the bridge after retrofitting (Fig. 16). The first mode shape energy spectrum shows maximum decrease in energy level at node 20. Before retrofitting, this portion had maximum deflection and settlement. In the second mode, shape energy spectra at

Fig. 16 Energy spectra of **a** first, **b** second, and **c** third mode shape before and after retrofitting for the upstream truss



nodes 18 and 19 show the maximum difference between energies before and after retrofitting. However, the third mode shape energy spectrum gives contrary results, and energy distribution is uniform in the upstream truss after retrofitting.

6 Conclusions

- Most in-service structures need maintenance and monitoring to insure their integrity. It is observed that the joints (supports not considered) found to be flexible showed lower amplitude of power per frequency before retrofitting in the PSD diagrams, thus reflecting the incapability of the damaged joint to dissipate the energy. The same joints after retrofitting being more intact showed higher dissipation of power at the resonating frequency of the respective node. The supports being flexible before retrofitting dissipated the

contributing power which is greatly reduced after retrofitting in the PSD spectrum.

- The variations of PSD in the first mode of the upstream and the downstream trusses clearly revealed improvements in the bridge signifying the importance of generating a bridge signature before and after retrofitting.
- STFT also showed the more flexibility in the joints of the downstream truss and the upstream truss before retrofitting with respect to the excitation. The damaged nodes did not respond in resonance with structural frequency. The relative nodal-frequency amplitudes of the bridge before retrofitting are higher in magnitude than those after retrofitting irrespective of the mode of vibration.
- The WPT technique reflected nondependence on particular modal response for flexibility identification contrary to PSD technique.
- Although with the WPT the damage location is not identified before retrofitting, the energy distribution is

highly nonuniform which got converted into an almost uniform energy distribution pattern after retrofitting in both the trusses indicating better structural behavior of the bridge.

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