



DYNAMIC BEHAVIOR AND VIBRATION ASSESSMENT OF A MULTISECTOR INTEGRAL BRIDGE: INSIGHTS FROM TRAFFIC AND PEDESTRIAN FIELD TESTS

Mahendra Kumar H M
G P Chandradhar
Goud Mallesh ¹
Vinayak Naik

Received 20.07.2026
Revised 22.08.2026
Accepted 04.09.2026

Keywords:

Integral bridges, Vibration serviceability, Natural frequency, Damping ratio, Dynamic amplification factor, Human structure interaction, Ambient traffic excitation, Finite element modelling

A B S T R A C T

Integral bridges eliminate expansion joints and bearings, which improves durability but produces complex dynamic behavior that must be verified under real operating conditions. This study combines field testing with three-dimensional finite element (FE) modelling in MIDAS Civil to characterize five integral bridges with one to five spans, clear spans of 5 to 18 m, and solid or voided slab decks. Tests were carried out under ambient traffic and synchronized human jumping by 3, 6 and 9 participants. Natural frequencies were identified by Fast Fourier Transform, damping ratios by the logarithmic decrement method, and dynamic amplification factors (DAF) from double-integrated acceleration records. At the single-span Patnem Bridge, the measured fundamental frequency of 10.09 Hz agreed with the FE value of 10.17 Hz (0.8% difference). Its damping ratio was 8.16% and its DAF was 1.21, below the IRC (1.225) and BD 37/01 (1.322) values. Jumping by nine people produced a near-resonant response at 10.50 Hz, within 4% of the traffic-derived frequency. Across all bridges, measured frequencies ranged from 9.67 to 26.2 Hz, damping ratios from 4.59% to 8.16%, and DAF from 0.97 to 1.69. Equivalent stiffness ranged from 5.63 to 65.75 MN/m and agreed within 5–9% with the classical EI/L^3 relationship, generally decreasing with span length. The exception was the four-span, 18 m Thippegonadanahalli Bridge (32.55 MN/m), where continuity, integral abutments and a voided haunched deck raised the stiffness. This bridge also showed the highest DAF (1.69), exceeding both code values. For the multi-span bridges, FE frequencies differed from the measured values by 13 to 30% and FE DAF by 0.04 to 0.25. These results show that bridge-specific testing and experimentally calibrated FE models are needed for reliable vibration serviceability assessment of integral bridges.



© 2026 Published by Faculty of Engineering

¹ Corresponding author: Goud Mallesh
Email: malles@sjce.ac.in

1. INTRODUCTION

Integral bridges have gained wide acceptance in modern transportation infrastructure because of their structural continuity, durability, rider comfort and low maintenance requirements (Vasconez et al., 2022). Unlike conventional bridges, they have no expansion joints or bearings, as the superstructure, abutments and foundation are connected monolithically. This connection improves load transfer and avoids the deterioration usually caused by joint and bearing failures (Arockiasamy et al., 2004). Their lower construction and maintenance costs and longer service life have made them an attractive alternative to conventional jointed bridges for short- to medium-span highway bridges, flyovers and overpasses, where these benefits make them an appealing option (Burke, 2009).

Despite these advantages, the monolithic configuration introduces complex structural interactions involving deck continuity, support restraint, soil structure interaction, thermal effects and traffic induced dynamic loading (Alqarawi et al., 2024). Because the deck, abutments and foundation act as a single structural system, load redistribution and vibration behavior differ considerably from those of conventional bridges. Understanding the dynamic behavior of integral bridges is therefore essential for assessing structural performance, serviceability, durability and long-term safety (Chow, 2022).

Vibration-based dynamic characterization has become an effective tool for evaluating bridge performance, because modal parameters such as natural frequencies, mode shapes, damping ratios and the logarithmic decrement, together with the Dynamic Amplification Factor (DAF), reveal the stiffness, energy dissipation and operational vibration behavior of a structure (Saidin et al., 2022). The DAF expresses the increase in structural response under dynamic loading relative to static loading. It is strongly influenced by vehicle speed, axle configuration, bridge stiffness, damping, resonance and vehicle bridge interaction (Deng et al., 2015). Damping likewise plays a crucial role in vibration decay and energy dissipation, and it is essential for modal identification, finite-element model validation, vibration serviceability evaluation and long term Structural Health Monitoring (SHM) (Meixedo et al., 2021).

The dynamic behavior of bridge systems has been studied from analytical, numerical and monitoring perspectives. On the numerical side, (González et al., 2008) used finite element analysis to examine dynamic amplification under critical loading scenarios and found that it is significantly influenced by bridge stiffness and vehicle loading. Numerical models are only as reliable as their assumptions, so more recent studies have combined them with measured data. Using measurements recorded under random traffic, a digital-twin framework was developed to characterize the in-service behavior of a bridge (Zhao

et al., 2022). Similarly, a finite-element model updating method was proposed that uses vehicle-induced monitoring data and improves the accuracy of the predicted response (Guan et al., 2024). Collectively, these studies indicate that dynamic assessment is most reliable when numerical models are calibrated against field measurements, an approach adopted in the present study.

While Despite these contributions, most previous studies have relied on numerical simulation, a single bridge configuration or ambient traffic monitoring alone. Field studies that combine ambient traffic with controlled human-induced excitation and finite-element modelling on multisector integral bridges remain limited (Gharad & Sonparote., 2023). In addition, the influence of span continuity, slab arrangement, support geometry and haunched sections on dynamic response under actual operating conditions is not well understood.

To address these gaps, this study experimentally and numerically investigates the vibration characteristics and structural performance of multisector integral bridges along National Highways, State Highways and the Expressway. The selected bridges cover a wide range of configurations, with one to five spans, clear spans of 5 to 18 m, solid and voided slab decks, and uniform and varying depth sections. The influence of stiffness variation and support geometry on damping and vibration response is examined in particular for the Patnem and Tippegondanahalli bridges, which have haunched support regions.

3-D finite element models were developed in MIDAS Civil to compute natural frequencies, mode shapes and DAFs (Guebailia et al., 2013; Ho & Nishio, 2024). Field measurements were then carried out under ambient traffic and under controlled synchronized jumping. High-sensitivity accelerometers connected to a DEWE-43A data acquisition system recorded the bridge accelerations. Dominant frequencies were identified by spectral analysis, damping ratios were estimated from free-vibration decay using the logarithmic decrement method, and experimental DAFs were determined to quantify vehicle-bridge interaction under operational loading. The measured and numerical results were also compared with the IRC and BD 37/01 provisions (British Standards Institution, 2001; Indian Roads Congress, 2000). Combining numerical modelling with field testing is a powerful approach to understanding bridge behavior under real service conditions. Field tests capture the actual variability of traffic loading, boundary conditions, soil-structure interaction and material behavior, while finite-element modelling allows the response to be evaluated under controlled loading cases. Agreement between the two supports modal identification, validates numerical predictions and improves the reliability of dynamic performance assessment. Accordingly, the objective of this study is to characterize multisector integral bridges through an integrated numerical and

experimental approach under ambient traffic and controlled human-induced excitation,

2. METHODOLOGY

2.1 Experimental evaluation of damping ratio and dynamic amplification factor

Damping and dynamic amplification are two of the main parameters that describe how a bridge responds to moving traffic. The damping ratio governs how quickly vibrations decay and how much energy the structure dissipates (Chopra, 2017), while the dynamic amplification factor (DAF) measures the increase in response caused by a moving vehicle relative to its quasi-static response (Paultre et al., 1992). Both parameters are sensitive to the boundary conditions, continuity and soil structure interaction of a bridges. Accordingly, five integral bridges, including the Patnem Bridge in Goa, were selected for this study to represent varying regional and traffic conditions, providing a representative basis for evaluating integral bridge performance.

Table 1. Details of integral bridges selected

Sl. No.	Name of the bridge & Location	Number of span / Span Length	Deck type	Region/ Description
1.	Patnem Bridge, Goa	1/15 m	Solid slab with haunched support	Coastal environmental conditions.
2.	Mysuru-Bangalore Expressway, Mysuru	2/12 m	Solid slab	Semi-urban, mixed traffic.
3.	Kadakola Bridge, Nanjangud	3/7 m	Solid slab	Plain terrain
4.	Thippegonadanahalli Bridge, Magadi Road, Bengaluru	4/18 m	voided slab with haunched support	Urban, high traffic intensity
5.	Aversa bridge, Karwar	5/5 m	Solid slab	Mountainous region

To evaluate the dynamic behavior of the selected bridges (Table 1), the damping coefficient and dynamic amplification factor were determined from vibration response characteristics measured using piezoelectric sensors positioned at the quarter-span, mid-span, and support regions of each bridge as shown in Figure.1.

Accelerations were recorded under ordinary traffic conditions, without closing the bridge to traffic, using accelerometers connected to an 8-channel DEWE-43 data acquisition system shown in Figure 2. The sensors captured road-induced vibrations, and the resulting acceleration data were transmitted to a computer for recording and subsequent analysis.

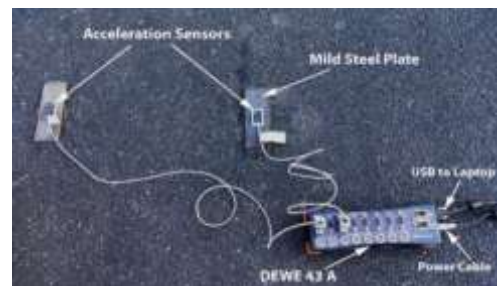


Figure 1. Position of sensors on the bridge



Figure 2. DEWE-43 data acquisition system

Figure 3 shows the raw acceleration time-history, which was processed through baseline correction, noise filtering, and drift removal to obtain a reliable vibration signal, from which individual vehicle-induced vibration events were extracted for further analysis.



Figure 3. Raw acceleration signals recorded from the bridge using the DEWE-43 data acquisition system

In the resulting signal, the higher peaks correspond to the instant when the vehicle was positioned on the bridge deck, marking the point of maximum dynamic response, while the lower, decaying peaks represent the residual free vibration of the bridge after the vehicle left the span. Each extracted vibration event was analysed using three approaches to obtain the natural frequency, damping ratio, and dynamic amplification factor of the bridge. The natural frequency was determined by applying Fast Fourier Transform (FFT) analysis to the acceleration signal, converting it from the time domain to the frequency domain; this transformation identifies the dominant natural vibration frequency of the bridge, corresponding to the peak in the frequency spectrum, as shown in Figure 4.

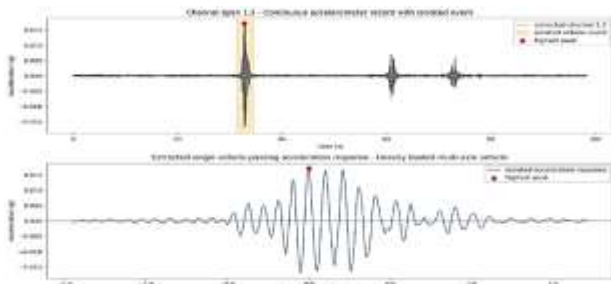


Figure 4. Acceleration response and dominant frequency of the bridge under moving-vehicle loading

The damping ratio was determined using the logarithmic decrement method. Once the vehicle had left the span, the recorded signal transitioned into a free-vibration decay curve, representing the natural, unforced oscillation of the bridge as it dissipated energy over successive cycles, as shown in Figure 5. As the structure vibrated freely, the amplitude of each successive peak decreased due to internal damping. The logarithmic decrement, calculated from the ratio of successive decreasing vibration peaks, quantifies this rate of amplitude decay and was subsequently used to determine the structural damping ratio of the bridge, where a higher logarithmic decrement indicates faster energy dissipation and, correspondingly, a higher damping ratio.

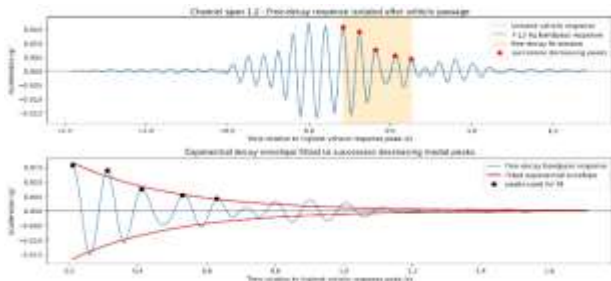


Figure 5. Free-decay response of the bridge

Finally, the Dynamic Amplification Factor (DAF) was determined by integrating the acceleration record twice first to obtain velocity, and subsequently displacement in order to calculate the maximum dynamic displacement of the bridge. This dynamic displacement was then compared with the corresponding quasi-static

displacement, obtained through the ambient vibration approach, to compute the DAF, which represents the amplification of the bridge's response caused by the moving vehicle relative to its static behavior.

Based on the free-decay response isolated from the recorded signals the logarithmic decrement was computed using the expression:

$$\delta = \frac{1}{n} \ln \left(\frac{x_1}{x_{n+1}} \right) \quad (1)$$

Where, x_1 -First peak amplitude, x_{n+1} - Last peak amplitude and n = number of vibration cycles

Using the logarithmic decrement obtained above, the structural damping ratio and the Dynamic Amplification Factor were subsequently estimated using Equations (2) and (3), respectively, as discussed below.

The structural damping ratio was estimated using:

$$\zeta = \frac{\delta}{\sqrt{4\pi^2 + \delta^2}} \quad (2)$$

The Dynamic Amplification Factor was computed as:

$$DAF = \frac{R_{dynamic}}{R_{static}} \quad (3)$$

Where,

$R_{dynamic}$ - maximum dynamic displacement,
 R_{static} -maximum quasi-static displacement

2.2 Human Excitation Testing

In addition to operational traffic excitation, controlled human excitation tests were performed to examine the dynamic behavior of the integral bridges under repeatable loading. This method is widely used in structural dynamics and experimental modal analysis because it is simple, economical, and controllable. It allows identification of vibration characteristics such as natural frequency, modal response, damping ratio, and DAF. Compared with traffic excitation, synchronized human excitation produces more consistent harmonic loading, which allows the dominant vibration modes to be identified more clearly under field conditions.

In this study, synchronized jumping tests were carried out at predetermined locations on the bridge deck using three excitation levels: 3, 6, and 9 persons as shown in Figure 6. Participants jumped at an approximately uniform frequency of 2-3 Hz, which is sufficient to excite the dominant low-frequency modes of integral bridges. The periodic force generated by synchronized jumping is represented as:

$$F(t) = F_0 \sin(\omega t) \quad (4)$$

where $F(t)$ is the excitation force at time t , F_0 is the peak excitation force, and $\omega = 2\pi f$ is the circular frequency corresponding to the excitation frequency f . The total excitation force is estimated as:

$$F = n\alpha W \quad (5)$$

where n is the number of participants, W is the average body weight, and α is the dynamic load factor.



Figure 6. Synchronized jumping test

The bridge response to human excitation is idealized as a single-degree-of-freedom system:

$$m\ddot{x} + c\dot{x} + kx = F_0 \sin(\omega t) \quad (6)$$

where m is the generalized mass, c is the damping coefficient, k is the structural stiffness, and x , $\dot{x}$, and $\ddot{x}$ are the displacement, velocity, and acceleration, respectively. The natural frequency is given by:

$$f_n = \left(\frac{1}{2\pi}\right) \sqrt{\frac{k}{m}} \quad (7)$$

During testing, the response to synchronized jumping was recorded using high-sensitivity accelerometers connected to the DEWE-43A data acquisition system.

2.3 Finite Element Analysis for Logarithmic Decrement, Damping Ratio, and DAF

The Dynamic Amplification Factor (DAF) of the integral bridge was evaluated through a detailed finite element analysis carried out using the MIDAS software platform. A three-dimensional finite element model of the Patnem integral bridge was developed using the software, following the step-by-step procedure outlined below.

Step 1: Geometric Modelling

A 3D model of the Patnem integral bridge was created in MIDAS/Civil using the dimensions shown in Figure 7. For the multi-span bridges, a single representative span was modelled, since the geometry and boundary conditions of every span in such a structure are identical. This approach captures the true structural behaviour while significantly reducing computational effort and

analysis time without compromising accuracy, as the repetitive spans do not introduce additional variability in stiffness, mass, or support conditions. The Patnem bridge itself is a single-span bridge, as shown in Figure 6, and was therefore modelled in full.

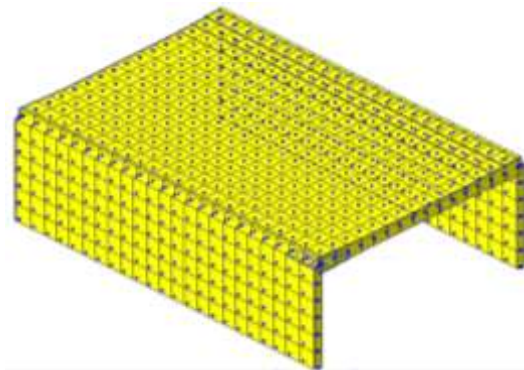


Figure 7. 3D model of the Patnem integral bridge

Step 2: Meshing of Patnem Integral Bridge

The Patnem Bridge was discretized into a three-dimensional finite element model (FEM) in MIDAS Civil using 4-node isoparametric plate/shell elements, which capture both in-plane and out-of-plane bending behaviour. The geometry forms a monolithic, single-cell open portal frame in which the horizontal top deck slab is rigidly connected to two parallel vertical abutment walls, ensuring continuous moment-resisting joints at the corners. To achieve high accuracy and a uniform stress distribution, a structured, uniform rectangular mesh with a 1.0 m element size was applied across the entire structural assembly, as shown in Figure 8. The elements were defined as Plate type, with M30 concrete and a thickness of 1.0548 m. The "Subdivide Boundary Line Elem." option was activated so that the boundary line elements were subdivided consistently with the mesh nodes, maintaining connectivity along the boundaries of the planar areas.

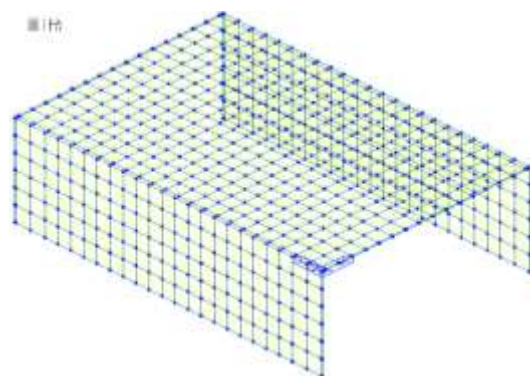


Figure 8. Finite Element Model of Patnem Integral Bridge

The Elements Table confirms that the model consists of 725 "PLATE Thick (w/o Drill)" elements, with connectivity defined through the corresponding nodal coordinates. This mesh was adopted for the subsequent modal, static, and dynamic analyses, providing the

required connectivity and deck geometry representation for evaluating the natural frequencies, damping characteristics, and DAF.

Step 3: Material Definition of the Patnem Integral Bridge

Based on the design specifications, material properties were assigned to the respective structural elements of the model. The primary material is isotropic M30 concrete, defined under the IRC standard, with a modulus of elasticity of 3.1186×10^7 kN/m², a Poisson's ratio of 0.2, and a weight density of 25 kN/m³. A thermal expansion coefficient of 1.2×10^{-5} /°C and a damping ratio of 0.05 were specified for the thermal and dynamic analyses.

Since integral bridges have a monolithic connection between the superstructure and abutments, with no expansion joints or bearings, the abutments were modelled as rigidly connected to the superstructure. The backfill soil was represented by equivalent linear or non-linear elastic springs to simulate soil-structure interaction, capturing the lateral soil resistance mobilised during thermal expansion, contraction, and dynamic loading.

Step 4: Boundary Conditions

Boundary conditions were applied to the plate model of the Patnem Bridge to represent its actual support behaviour. Because the deck slab and the two abutment walls are continuous, the integral connection requires no bearing or expansion-joint supports, so restraints were assigned at two locations instead. Foundation supports were assigned along the base of both abutment walls at the pile foundation level, restraining the translational and rotational degrees of freedom and idealising the pile cap as a fixed base. At the backfill-abutment interface, the walls were not restrained rigidly. Elastic (spring) supports were assigned to the wall nodes so that the lateral soil resistance mobilised during thermal expansion, contraction, and dynamic loading is governed by the assigned spring stiffness. This keeps the abutment-soil interaction compliant, as expected for an integral bridge, while the structure behaves as a monolithic portal frame, as shown in Figure 9.

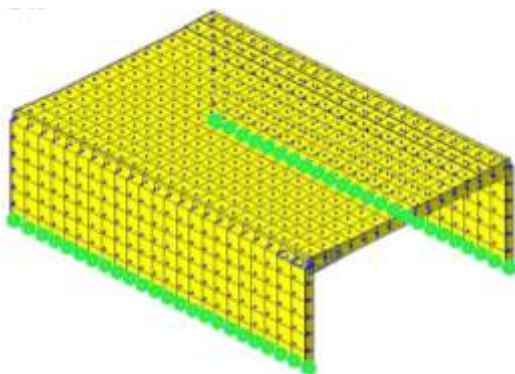


Figure 9. Displacement and rotational restraints at the foundation

Step 5: Load Application, Analysis and DAF Evaluation

Self-weight was applied automatically from the assigned material density. Vehicular live loads, defined as per the applicable bridge loading code (IRC:6), were applied as moving loads across the span to simulate realistic traffic conditions. The IRC Class A/70R vehicle was moved along the defined traffic lanes at 40 km/h, as shown in Figure 10. Static and dynamic analyses were then performed on the finite element model to obtain the peak structural responses (displacement, bending moment, and stress) under the moving load. Further, the DAF was evaluated against the code-specified limits. A similar procedure was used for the other bridges, and finally, the FEA results were compared with the experimental results.

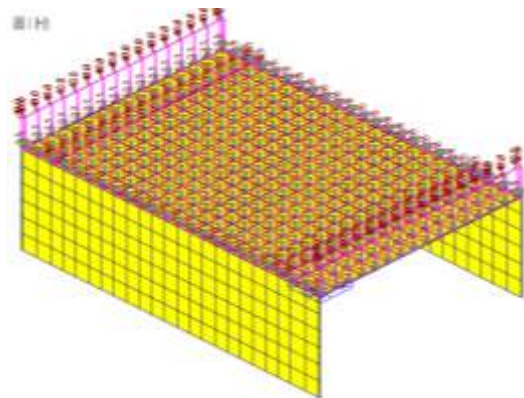


Figure 10. Load and displacement boundary conditions

3. RESULTS AND DISCUSSION

This section presents the results of the experimental investigation and the finite element analysis (FEA) of the integral bridges, followed by a discussion of their dynamic behaviour. The experimental results were obtained from operational traffic excitation and controlled human excitation tests, and include the natural frequencies, damping ratios, and Dynamic Amplification Factors (DAF) of each bridge. The FEA results, comprising peak displacement, bending moment, and stress under IRC Class A/70R moving loads, are then compared with the measured responses to validate the numerical models. Finally, the influence of span length, deck type, and site conditions on the dynamic performance of the bridges is discussed, and the DAF values are evaluated against the code-specified limits.

3.1 Dynamic response of the Patnem bridge

The dynamic response of the single-span integral bridge at Patnem–Canacona on NH-66, near Goa, was recorded using a DEWE-43 data acquisition system while a truck crossed the span at about 40 km/h. The acceleration record, about 98 s long, shows three distinct high-amplitude bursts at approximately 33 s, 61 s and 73 s, each corresponding to a vehicle passage. Between the bursts, the response drops to low-level ambient vibration. After zero-centering to remove the gravitational offset of

about 1 g, the largest burst reached about ± 0.017 g, while the other two reached about ± 0.006 g and ± 0.005 g. This variation suggests that the response depends strongly on the vehicle's mass, axle configuration and position on the span at the instant of excitation. Each burst decayed quickly after the vehicle left the span, indicating a lightly damped structural system. These free-decay segments were used to estimate the logarithmic decrement, damping ratio and dynamic amplification factor (DAF). The sequence of events is shown in Figure 11.



Figure 11. Acceleration response of the Patnem bridge

3.2 Frequency-domain characteristics of the Patnem bridge

Fast Fourier Transform (FFT) analysis of the recorded signals identified the dominant natural frequency of the bridge (Figure 12). The spectrum of the span channel shows a single sharp peak at about 10.09 Hz for the moving vehicle, with most of the energy concentrated between 8 and 12 Hz. Smaller secondary peaks appear near 2.8, 12.3 and 14.8 Hz. The 2.8 Hz peak may reflect vehicle related low frequency content, whereas the higher peaks may indicate higher order modes or vehicle bridge interaction. The small difference between 9.97 and 10.09 Hz lies within the frequency resolution and the event-to-event variation, confirming that the fundamental frequency is stable at about 10 Hz regardless of the excitation. Such a frequency is typical of a short, stiff integral bridge, where the monolithic deck abutment connection increases the stiffness compared with a simply supported span of equal length.

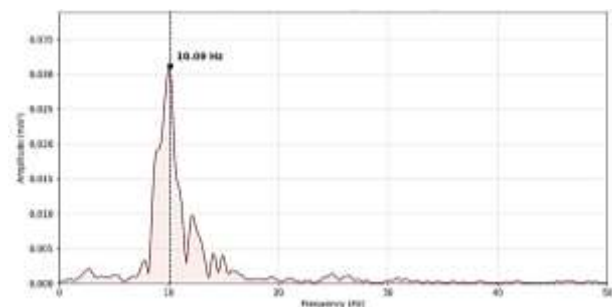


Figure 12. FFT amplitude spectrum of the Patnem bridge

Based on the experimental data, the logarithmic decrement (δ), damping ratio (ζ) and dynamic amplification factor (DAF) were determined using Eqs. (1) – (3), respectively. The logarithmic decrement was calculated from the decay of successive acceleration peaks in the free-vibration portion of the response after the vehicle left the span (Eq. 1). The damping ratio was then obtained from the logarithmic decrement (Eq. 2). The DAF was calculated as the ratio of the maximum dynamic response to the maximum static response (Eq. 3). The resulting values for a single heavy-vehicle passage are listed in Table 2.

Table 2. Logarithmic decrement, damping ratio and DAF for Patnem bridge

Logarithmic decrement	0.5143
Damping ratio	8.16%
DAF	1.21

It is evident from the table that the logarithmic decrement was 0.5143, which corresponds to a damping ratio of 8.16 %. This exceeds the minimum of 5 % recommended in the IRC (Indian Roads Congress [IRC] 2000a, 2000b, 2011, 2014) and indicates adequate energy dissipation. Under these loading conditions, the bridge was therefore in good health and safe to use. The high damping can be

attributed to the monolithic deck–abutment connection and soil structure interaction in the integral bridge.

The measured DAF was 1.21, which is slightly lower than the span-based IRC value of 1.225 and the frequency-based BD 37/01 value of 1.322. The calculated DAF therefore agrees closely with the IRC value. Since code impact factors are conservative upper-bound design values, a measured DAF below them is favorable, showing that the code allowances cover the dynamic amplification observed in this event. Together, the high damping ratio and the low DAF indicate satisfactory dynamic behavior of the bridge under the applied loading.

3.3 Frequency response under Human Excitation

Controlled human excitation tests were performed on the Patnem bridge to examine its dynamic frequency response. Groups of 3, 6 and 9 people, each person weighing approximately 72 kg, jumped on the span, so the total excitation mass was approximately 216 kg, 432 kg and 648 kg, respectively. The jumping was carried out at the mid-span and the resulting vibrations were recorded using the DEWE-43 data acquisition system with accelerometers placed at the bottom of the bridge slab. The recorded acceleration signals were transformed into the frequency domain using the FFT to identify the dominant frequency of the bridge and to examine how the response changes with the number of participants.

Table 3. Dominant frequency under controlled human jumping

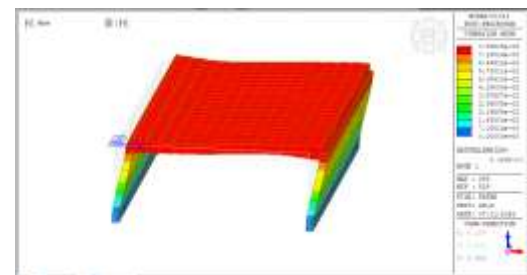
Dominant frequency (Hz)		
3 persons	6 persons	9 persons
6.16	8.12	10.50

Table 3 presents the dominant frequencies under controlled human jumping excitation. The dominant frequency increased with the number of participants, from 6.16 Hz for 3 people to 8.12 Hz for 6 people and 10.50 Hz for 9 people. Since added mass normally lowers a structure's natural frequency, this trend suggests that the measured values mainly reflect the forcing frequency of the jumping group. With 9 people jumping, the dominant frequency (10.50 Hz) was within about 4 % of the 10.09 Hz identified from the vehicle-induced response, indicating a near-resonant condition. Vehicle loading acts as a load moving at a steady speed, whereas human jumping applies repeated impact loads at a fixed location. Near-resonant human-induced vibration may therefore be more critical for serviceability and should be considered in the dynamic assessment of the bridge.

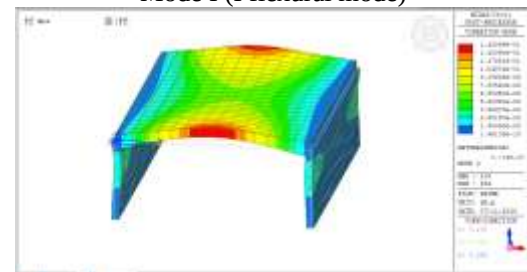
3.4 Numerical Analysis of the Bridge Vibration Response

As described in Section 2.3, a finite element analysis was carried out in MIDAS Civil to determine the natural

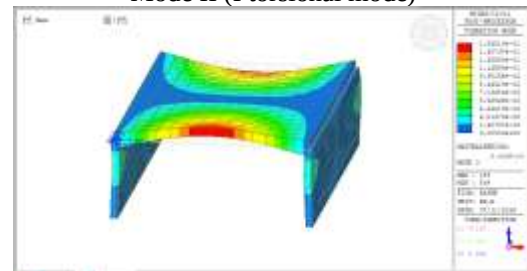
frequency and period of vibration of the bridge through modal analysis. This section presents the numerical results and compares them with the frequencies measured in the field under vehicle and human-induced excitation. The comparison is used to validate the numerical model and to assess the global dynamic behavior of the bridge. The first mode, which corresponds to the predominant vertical bending of the deck, was used to estimate the fundamental frequency, and the mode shapes obtained are presented in Figure 13.



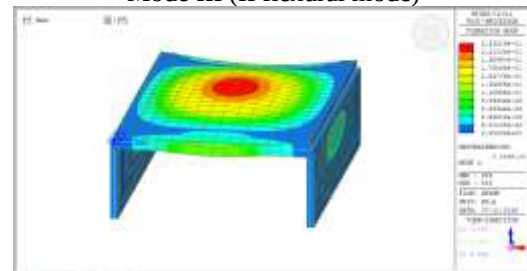
Mode I (I flexural mode)



Mode II (I torsional mode)



Mode III (II flexural mode)



Mode II (II torsional mode)

Figure 13. Mode Shapes of Patnem Bridge

It is evident from the modal analysis MIDAS Civil gave a fundamental period of 0.09836 s, which corresponds to a natural frequency of 10.17 Hz. This agrees closely with the field-measured frequency of 10.09 Hz and with the dominant frequency of 10.50 Hz recorded under 9-person jumping, which validates the numerical model. The first mode is the first flexural mode with predominant vertical

bending of the deck, followed by the first torsional mode (14.04 Hz), the second flexural mode (15.08 Hz) and the second torsional mode (34.63 Hz). The higher modes are well separated from the fundamental mode, so the dynamic behavior of the bridge under the applied loading is governed mainly by the first flexural mode. It is evident from the results that the numerical values differ from the experimental values by only about 2-3 %.

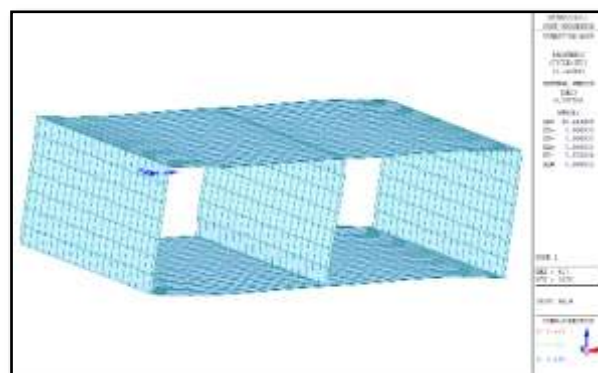
3.4.1 Estimation of Natural Frequency, Damping, and DAF for Different Bridges

MIDAS Civil can be used to estimate the natural frequency of the bridge through modal analysis, and the logarithmic decrement and Dynamic Amplification Factor (DAF) through time-history analysis. The same procedure can be applied to the other bridges in Table 1, which vary in number of spans, span length, deck type, and region. This allows a systematic comparison of their natural frequency (Figure 14), damping, and DAF with the experimental results and with the human excitation produced by nine people jumping on the bridge, as discussed in Section 3.3.

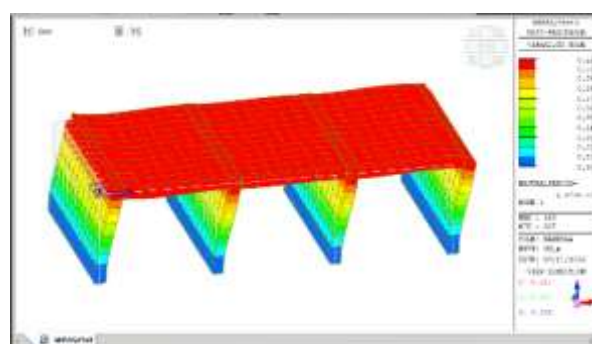
The fundamental natural frequency and natural period were determined for four bridges of differing span configuration. The frequencies obtained from finite element modal analysis in MIDAS Civil were compared with those measured experimentally and under human excitation, as summarized in Table 4. The numerical frequencies ranged from 7.52 to 22.8 Hz (periods of 0.133 to 0.044 s), against experimental values of 9.67 to 26.2 Hz (0.103 to 0.038 s). Aversa Bridge was the stiffest structure and Thippegonadanahalli Bridge the most flexible in the numerical model. Human excitation gave frequencies close to the experimental values. Overall, the modal analysis captured the trend in dynamic stiffness across the bridges.

Following the determination of natural frequencies and periods, the damping and dynamic amplification of the bridges were evaluated, as summarized in Table 3. The logarithmic decrement ranged from 0.288 to 0.513, corresponding to damping ratios of 4.59% to 8.14%, with the highest values for Thippegonadanahalli Bridge and the lowest for Kadakola Bridge. The experimental DAF ranged from 0.97 to 1.69. As discussed in the previous section, the code-based DAF for the Mysuru-Bangalore Expressway bridge was 1.225 (IRC, span-based) and 1.322 (BD 37/01, frequency-based). Compared with these values, the Mysuru-Bangalore Expressway bridge (1.19), Kadakola Bridge (1.05) and Aversa Bridge (0.97) are within the code allowances and can be considered safe for the applied loads. Thippegonadanahalli Bridge (1.69) exceeds both code values and is therefore not safe for the applied loads under the code allowances. This bridge also had the lowest numerical natural frequency, which explains its larger dynamic response. Similarly, numerical analysis was used to estimate the DAF in MIDAS Civil. The same trends were observed, and the

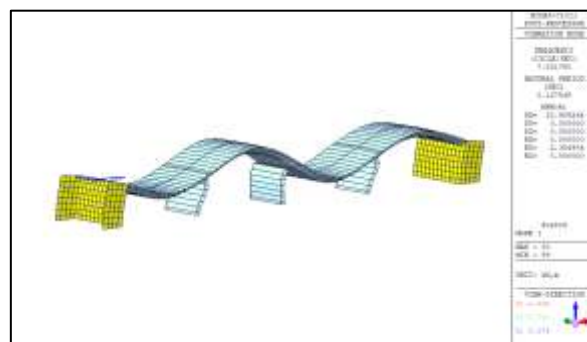
variation between the experimental and numerical values is within the range reported in the literature.



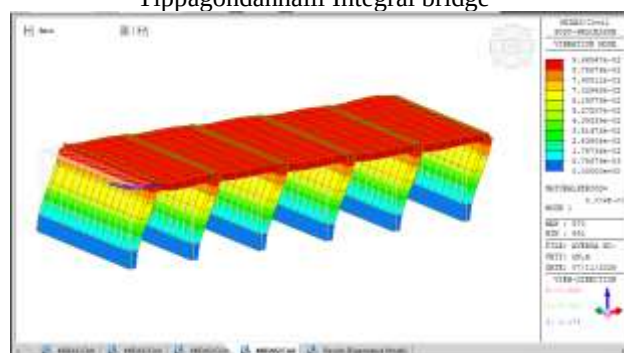
Bangalore Mysuru Expressway



Kadakola Bridge



Tippagondanahalli Integral bridge



Aversa Bridge

Figure 14. Natural frequency of different bridges with varying spans and span Lengths

Table 4. Comparison of numerical and measured natural frequencies of the bridges

Sl. No.	Name of the bridge & Location	Natural Frequency (Hz)		
		MIDAS Civil	Experimental	Human Excitation
1.	Mysuru-Bangalore Expressway, Mysuru	12.55	9.67	10.75
2.	Kadakola Bridge, Nanjangud	15.81	18.1	20.07
3.	Thippegonadanahalli Bridge, Magadi Road, Bengaluru	7.52	10.12	12.01
4.	Aversa bridge, Karwar	22.8	26.2	26.19

Stiffness governs the natural frequency, dynamic amplification, and vibration serviceability of a bridge, and changes in stiffness indicate structural deterioration. The stiffness of each bridge was calculated from its fundamental natural frequency by treating the bridge as an equivalent single-degree-of-freedom system. For such a system, the natural frequency is related to the stiffness and mass by

$$f = \frac{1}{2\pi} \sqrt{\left(\frac{k}{m}\right)} \quad (8)$$

where f is the fundamental natural frequency (Hz), k is the equivalent stiffness (N/m), and m is the equivalent mass of the bridge (kg). Rearranging Equation (8), the stiffness can be written as

$$k = m(2\pi f)^2 = 4\pi^2 f^2 m \quad (9)$$

The mass m was obtained from the MIDAS Civil model, based on the self-weight of the deck and the superimposed dead loads. The frequency f was taken from the experimental measurements. The stiffness values obtained from Equation (9) are summarized in Table 5 and compared with the classical relationship $K \propto EI/L^3$ to examine the effect of span length and structural configuration.

Table 5. Dynamic response of different bridges

Name of the bridge & Location	Logarithmic Decrement	Damping Coefficient (%)	DAF	
			Expt	MIDAS
Mysuru-Bangalore Expressway, Mysuru	0.436	6.93	1.19	1.15
Kadakola Bridge, Nanjangud	0.288	4.59	1.05	1.25
Thippegonadanahalli Bridge, Magadi Road, Bengaluru	0.513	8.14	1.69	1.44
Aversa bridge, Karwar	0.39179	6.22	0.97	1.05

The stiffness of the bridges calculated from the natural frequency using Equation (9) was compared with the values obtained from the classical relationship $K \propto EI/L^3$,

as presented in Table 6. It is evident from the table that the stiffness ranged from 5.63 MN/m to 72.25 MN/m. The variation between the two sets of results was 5 to 9%, and the difference was generally larger for the stiffer bridges. This may be because the frequency-based method reflects the actual boundary conditions, mass distribution, and in-service condition of each structure, whereas the classical relationship assumes idealized conditions.

Table 6. Comparison of calculated stiffness of the bridges

Name of the bridge & Location	Stiffness	
	Calculated using Equation: (9)	$K \propto EI/L^3$
Patnem Bridge, Goa	5.63	5.93
Mysuru-Bangalore Expressway, Mysuru	9.21	9.8
Kadakola Bridge, Nanjangud	16.58	18.02
Thippegonadanahalli Bridge, Magadi Road, Bengaluru	32.55	35.0
Aversa bridge, Karwar	65.75	72.25

The results follow the trend expected from structural dynamics. Aversa Bridge (5 m span) showed the highest stiffness, and Patnem Bridge, a single 15 m span without intermediate supports, the lowest. Thippegonadanahalli Bridge was the exception: despite its 18 m span, its stiffness (about 33 to 35 MN/m) was high, owing to its four-span continuity, integral abutments, voided slab section, and haunched regions near the supports, which increase the flexural rigidity and restrain the supports.

4. CONCLUSIONS

This study combined field measurements under traffic and human-induced excitation with finite element modelling to characterize the dynamic behavior of five integral bridges. The main findings are:

Model validation (Patnem Bridge). The MIDAS Civil model gave a fundamental period of 0.09836 s (10.17 Hz), against 10.09 Hz measured under traffic (0.8% difference) and 10.50 Hz under nine-person jumping (3.2%). The first torsional (14.04 Hz), second flexural (15.08 Hz) and second torsional (34.63 Hz) modes were well separated from the fundamental mode, so the response is governed by the first flexural mode.

Damping and amplification (Patnem Bridge). A logarithmic decrement of 0.5143 gave a damping ratio of 8.16%, which indicates strong energy dissipation from the monolithic deck–abutment connection and soil–structure interaction. The DAF of 1.21 was below the IRC (1.225) and BD 37/01 (1.322) values, so both codes cover the observed amplification.

Human excitation. The dominant frequency under jumping rose from 6.16 Hz (3 people, 216 kg) to 8.12 Hz (6 people, 432 kg) and 10.50 Hz (9 people, 648 kg). The nine-person response was close to the fundamental frequency, a near-resonant condition that can govern vibration serviceability at a fixed location.

Bridge-to-bridge variation. Across the five bridges, the measured logarithmic decrement ranged from 0.288 to 0.514 and the damping ratio from 4.59% to 8.16%. Four of the five bridges exceeded 5% damping, and Kadakola Bridge (4.59%) was the only one below it. Measured frequencies ranged from 9.67 to 26.2 Hz, with Aversa Bridge the stiffest and Thippegonadanahalli Bridge the most flexible in the FE model.

Dynamic amplification. Experimental DAF ranged from 0.97 to 1.69. Aversa Bridge (0.97), Kadakola Bridge (1.05) and the Mysuru-Bangalore Expressway bridge (1.19) were within the code values. Thippegonadanahalli

Bridge (1.69) exceeded both, which makes it the priority for further serviceability assessment.

Stiffness. The frequency-based stiffness ranged from 5.63 to 65.75 MN/m (an 11.7-fold variation) and agreed within 5–9% with the EI/L^3 trend, with shorter spans stiffer. Thippegonadanahalli Bridge was the exception, with 32.55 MN/m despite the longest span, owing to four-span continuity, integral abutments and a voided haunched section. Span length alone therefore does not determine stiffness.

Limits of the numerical model. The FE model reproduced the measured frequency of the single-span Patnem Bridge closely. For the multi-span bridges, the FE frequencies differed from the measured values by 13 to 30%, and the FE DAF differed by 0.04 to 0.25 (1.15 to 1.44 against 0.97 to 1.69 measured). Reliable assessment therefore requires calibration of boundary conditions and soil–structure interaction against field data.

References

- Alqarawi, A. S., Leo, C. J., Liyanapathirana, D. S., & Hu, P. (2024). Soil-structure interaction issues in integral abutment bridges. *Transportation Geotechnics*, 48, 101296. <https://doi.org/10.1016/j.trgeo.2024.101296>
- Arockiasamy, M., Butrieng, N., & Sivakumar, M. (2004). State-of-the-art of integral abutment bridges: design and practice. *Journal of Bridge Engineering*, 9(5), 497-506. [https://doi.org/10.1061/\(ASCE\)1084-0702\(2004\)9:5\(497\)](https://doi.org/10.1061/(ASCE)1084-0702(2004)9:5(497))
- British Standards Institution. (2001). BD 37/01: Loads for highway bridges (Design Manual for Roads and Bridges, Vol. 1, Sec. 3, Part 14). The Stationery Office. Retrieved from <https://pdfcoffee.com/design-manual-for-roads-and-bridges-summary-of-correction-bd-37-01-volume-1-section-3-part-14-loads-for-highways-bridges-3-pdf-free.html>
- Burke Jr, M. P. (2009). *Integral and semi-integral bridges*. John Wiley & Sons
- Chopra, A. K. (2017). *Dynamics of Structures: Theory and Applications to Earthquake Engineering* (5th ed.). Pearson Education. Retrieved from https://api.pageplace.de/preview/DT0400.9781292249209_A39514639/preview-9781292249209_A39514639.pdf
- Chouw, N. (2022). Innovative Bridge Design Handbook (Second Edition) Construction, Rehabilitation and Maintenance (pp. 141-168). Butterworth-Heinemann. <https://doi.org/10.1016/B978-0-12-823550-8.00004-4>
- Deng, L., Yu, Y., Zou, Q., & Cai, C. S. (2015). State-of-the-art review of dynamic impact factors of highway bridges. *Journal of Bridge Engineering*, 20(5), 04014080. [https://doi.org/10.1061/\(ASCE\)BE.1943-5592.0000672](https://doi.org/10.1061/(ASCE)BE.1943-5592.0000672)
- Gharad, A. M., & Sonparote, R. S. (2023). Study of dynamic impact factors of two-track continuous and integral railway bridge subjected to high-speed loads. *Electronic Journal of Structural Engineering*, 23(3), 19-24. <https://doi.org/10.56748/ejse.234203>
- González, A., Rattigan, P., OBrien, E. J., & Caprani, C. (2008). Determination of bridge lifetime dynamic amplification factor using finite element analysis of critical loading scenarios. *Engineering structures*, 30(9), 2330-2337. <https://doi.org/10.1016/j.engstruct.2008.01.017>
- Guan, Z. X., Yang, D. H., Yi, T. H., Li, W. J., & Li, C. (2024). Bridge finite element model updating using stochastic vehicle-induced static response monitoring data. *Engineering Structures*, 301, 117280. <https://doi.org/10.1016/j.engstruct.2023.117280>
- Guebailia, M., Ouelaa, N., & Guyader, J. L. (2013). Solution of the free vibration equation of a multi span bridge deck by local estimation method. *Engineering structures*, 48, 695-703. <https://doi.org/10.1016/j.engstruct.2012.12.004>
- Ho, H., & Nishio, M. (2024). Evaluation of dynamic impact factor of existing bridges with road surface damages based on dynamic response under traffic flow loading. *Structure and Infrastructure Engineering*, 20(9), 1390-1403. <https://doi.org/10.1080/15732479.2022.2145315>
- Indian Roads Congress (IRC). (2000a). IRC:18-2000 – Design Criteria for Prestressed Concrete Road Bridges (Post-Tensioned Concrete). New Delhi, India. Retrieved from <https://law.resource.org/pub/in/bis/irc/irc.gov.in.018.2000.pdf>
- Indian Roads Congress (IRC). (2000b). IRC:21-2000 – Standard Specifications and Code of Practice for Road Bridges. New Delhi, India. Retrieved from <https://law.resource.org/pub/in/bis/irc/irc.gov.in.021.2000.pdf>

- Indian Roads Congress (IRC). (2011). IRC:112-2011 – Code of Practice for Concrete Road Bridges. New Delhi, India. Retrieved from <https://law.resource.org/pub/in/bis/irc/irc.gov.in.112.2011.pdf>
- Indian Roads Congress (IRC). (2014). IRC:6-2014 – Standard Specifications and Code of Practice for Road Bridges. New Delhi, India. Retrieved from <https://law.resource.org/pub/in/bis/irc/irc.gov.in.006.2014.pdf>
- Meixedo, A., Ribeiro, D., Santos, J., Calçada, R., & Todd, M. (2021). Progressive numerical model validation of a bowstring-arch railway bridge based on a structural health monitoring system. *Journal of Civil Structural Health Monitoring*, 11(2), 421-449. <https://doi.org/10.1007/s13349-020-00461-w>
- Paultre, P., Chaallal, O., & Proulx, J. (1992). Bridge dynamics and dynamic amplification factors—a review of analytical and experimental findings. *Canadian Journal of Civil Engineering*, 19(2), 260-278. <https://doi.org/10.1139/192-032>
- Saidin, S. S., Kudus, S. A., Jamadin, A., Anuar, M. A., Amin, N. M., Ibrahim, Z., ... & Sugiura, K. (2022). Operational modal analysis and finite element model updating of ultra-high-performance concrete bridge based on ambient vibration test. *Case Studies in Construction Materials*, 16, e01117. <https://doi.org/10.1016/j.cscm.2022.e01117>
- Vasconez, R., Kustau, A., & Najm, H. (2022). An overview of integral abutments: Current practices, field monitoring and deck replacement measures. *Bridge Structures*, 18(1-2), 27-43. <https://doi.org/10.3233/BRS-220196>
- Zhao, H., Tan, C., OBrien, E. J., Zhang, B., Uddin, N., & Guo, H. (2022). Developing digital twins to characterize bridge behavior using measurements taken under random traffic. *Journal of Bridge Engineering*, 27(1), 04021101. [https://doi.org/10.1061/\(ASCE\)BE.1943-5592.0001814](https://doi.org/10.1061/(ASCE)BE.1943-5592.0001814)

Mahendra Kumar H M

Department of Civil Engineering, JSS
Science and Technology University,
Karnataka,
India
mahendrakumar15@sjce.ac.in
ORCID 0000-0002-2490-769X

G P Chandradhar

Department of Civil Engineering, JSS
Science and Technology University,
Karnataka,
India
chandradhara_gp@sjce.ac.in
ORCID 0009-0004-3612-300X

Goud Mallesh

Department of Mechanical Engineering,
SJCE, JSS STU,
Karnataka,
India.
mallesh@sjce.ac.in
ORCID 0000-0002-4113-107X

Vinayak Naik

Department of Civil Engineering,
Government Polytechnic Karwar,
Karnataka,
India
vinaynayak1987@gmail.com
ORCID 0009-0004-0041-7372
